Exhibit #2 for the record BOS 8/6/25

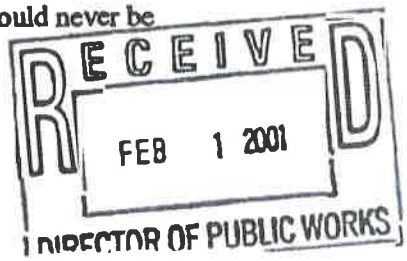
EXHIBIT 1 Russo / Landmark FT. Public Hearing

MINUTES FROM PHONE CONFERENCE WITH ATTY. FULLER
FEBRUARY 1, 2001

Mr. Fraser suggested that a review of the regulations and proposals be made by staff and an agreement should be reached on what can be built in Oswegatchie Hills. He added that he would then have Waterford, Regional Planning, State Highway, and an independent engineer come to a fair and honest proposal about what can be done. He stressed staff should only consider and review the 230 acres, not the entire parcel. He added that this small parcel abuts Route 1 and there is approximately 85 feet on Boston Post Road, so the frontage is very limited. It was his understanding that the applicant owns approximately 80 acres and has options on the remaining 150 acres. He added the frontage is very steep and runs along Latimer Brook and the rest of the property is landlocked.

First Selectman Wayne Fraser, Director of Public Works Fred Thumm, Town Planner Jean Davies, and Zoning Official Bill Mulholland were in attendance for a phone call placed at 9:00 a.m. to Atty. Fuller.

Fred Thumm noted that this property was outside the sewer shed and would never be watered or sewered.



Jean Davies stated that there were three items in question:

- the appraisal
- the zone change AFD
- the appeal – zoning.

Atty. Fuller inquired what the real likelihood of the State coming up with money to purchase this land was.

Mr. Fraser responded the change was very good due to the open space funding that was available and the State was making this a major priority. He has been informed by the State that the money is not the problem, but a fair appraisal is.

Atty. Fuller noted that an evaluation would be made to determine the highest and best use of a property and therefore, the stage of development on this parcel is important. He added that a parcel with an approved subdivision is worth much more than raw land and if DEP condemned it would be our best bet. He added that there is a big difference between filing conceptual plans and an approval.

Staff agreed that there is no approved application or significant one underway.

Mr. Fraser noted that David Leff of DEP was fully supportive of the Town's actions however, there was never any talk of condemnation.

Atty. Fuller stated that even if the 5-acre zoning does not stand up, you still have 3-acre zoning and you cannot get that much out of it.

Fred Thumm noted that the Facilities Plan in 1981 excluded this portion of land from the sewer shed. East Lyme has purchased 1.5 million gallons per day and we are presently at 55% usage. The ultimate build out of the sewer shed will take up all of the capacity.

Atty. Fuller stated that Affordable Housing cannot override sewer and the Water and Sewer Commission does not have to accommodate.

Mr. Fraser inquired about another proposed development in this vicinity of Boston Post Road. He noted that the pipe size would be controlled however, the pipe would pass in front.

Atty. Fuller stated that this would be defensible and there would still be a capacity problem and in a sewer shed area, capacity is taken into account.

Mr. Thumm noted that this area is isolated by I-95 and abuts Waterford. Waterford had been contacted to supply water and they will not allow more than 50,000 per day additional.

Atty. Fuller suggested getting all documentation from Water and Sewer for the public hearing.

TRAFFIC:
Atty. Fuller stated that if a study is submitted that states that this development will not adversely affect present traffic conditions. The Zoning Commission could request a traffic study by the applicant or do one itself. He added that Bill Mulholland could ask them if they are going to do a traffic study. He noted that traffic could be a reason to deny the Zoning application.

Atty. Fuller stated that the Town could adopt their own affordable housing regulations but it does not mean that a developer has to follow them. The developer could submit their own.

Atty. Fuller noted that the Zoning Commission should treat both items together, but notice separately.

It was agreed that the next conference call with staff would occur on February 9 at 9:00 a.m.

The call ended at 10:30 a.m.

Mr. Thumm and Ms. Davies were to check on traffic studies regarding time needed to perform and review.

Ms. Davies would check with DEP to have them inquire if DOT would perform traffic study.

write memo to Bill M. asking about traffic study

POTENTIALS

1. NO AVAILABILITY FOR WATER AND SEWER
   - Not in sewer shed, commitment elsewhere for availability. This plan would consume a lot of sewer and would require an extension.
   - Without water and sewer, cannot get affordable housing project through.
   - WATER AND SEWER COMMISSION HAS NO OBLIGATION TO EXTEND TO PROPERTY – DOES NOT FALL UNDER AFFORDABLE HOUSING ACT.

li. RECOMMENDATION IN PLAN OF DEVELOPMENT AS OPEN SPACE SUPREME Court DECISION – CHRISTIAN ACTIVITIVITES VS. GLASTONBURY RE OPEN SPACE.
   -Parcel always recommended for open space in Plan of Development.
Supreme Court said this was a viable reason to deny.

3. TRAFFIC CONSIDERATIONS
Mr. Fraser noted this proposed development would enter and exit on Boston Post Road within ¼ mile of I-95 and within 300 feet of Route 1. He added there were severe site line issues.

Atty. Fuller inquired about a traffic report. He added that we need to resolve the question if they have submitted enough or if the Zoning Commission can request more. He added that Mr. Fraser should not appear on the record or before the Commission in this matter. He stated that the Planning Commission could take an official position. He suggested that an official booklet be set up utilizing the chronology as an index and passed as an Exhibit to the Zoning Commission, noting it was important to get this information into the record. He added that all evidence should be offered at public hearing, adding that the only exception to this was consultants to the commission could submit date to explain things – reports could be submitted later. He stressed whatever we submit, submit it at the public hearing – this gives the other side the opportunity to comment.

4. ECOLOGICAL:
Atty. Fuller noted that an environmental consultant could be hired to discuss what extent this proposed development might adversely impact. Review archeological and wetlands.

Bill Mulholland noted he would circulate the conceptual plans to other departments, as is custom, to gather comments.

Atty. Fuller stated that the Zoning Commission needs a basis for denial. He suggested including the water and sewer report, addressing traffic and environmental, and the Planning Commission's report in the record.

MINUTES FROM PHONE CONFERENCE WITH ATTY. FULLER
FEBRUARY 9, 2001

A phone call was placed by Jean Davies to Atty. Fuller at 9:10 a.m. She explained that Mr. Fraser was in Hartford testifying before the Legislature, so the following were present: Director of Public Works Fred Thumm, Planner Jean Davies, Zoning Official Bill Mulholland, Sanitarian George Calkins.

It was decided that a set of minutes would be sent to Atty. Fuller.

STATUS UPDATE:
Ms. Davies noted that she had spoken with DEP and they felt the negotiations were not working and were considering notifying Landmark that they would be withdrawing their proposal. She explained that the problem is not the concept, but the price. It is believed that Landmark does not want to build this, he is trying to "jack up" the price with the highest and best use of the property.

Wayne Fraser has been actively talking with Glen Russo of Landmark. Russo's Attorney has contacted Bill Mulholland about potentially withdrawing. We submitted a request to go on to the property to perform an environmental review (free to the Town by the County Soil and Conservation through USDA). They then started talking more aggressively about withdrawing. We have not received a withdrawal notice. As of yesterday, we are still moving ahead with the defense of the application and he is still moving ahead regarding issues of appraisal – what are the potential options for this property. Friends of Oswegatchie Hills contacted the Town yesterday, making a plea on the part of Russo and Landmark noting that there actually might be two points of access. If you want to go in for subdivision, are you willing to sit down with them....

We will be talking to Waterford – the neighboring town within 500 feet of the border. Water and Sewer will be coming across from Waterford.

Atty. Fuller inquired what the Regional Planning agency say about this?

Bill Mulholland noted that all the referrals are out to the appropriate agencies, but we have not had responses. He has talked to all of them and they are very supportive.

Wayne talked with DOT re: getting an STC evaluation on Route 1. Preliminary feedback so far is: definitively a stoplight and most probably with the volume of traffic extra turning lanes needed.

Atty. Fuller inquired of any indication of level of service at the present time.

Ms. Davies responded that they had not gotten back to Wayne on that.

Fred Thumm stated that the sewer capacity is not there – not even a close call. The last time the sewer shed was adjusted (it was extended for another subdivision) which was

two years ago, we did the capacity analysis at that time. They said if we make any further adjustments in the sewer shed, we would have to take out properties from the sewer shed that have been promised sewer availability in the future. These statements were in the record.

Atty. Fuller requested a copy of this information.

Mr. Thumm talked to a traffic-reporting firm about cost and to review someone else's report would cost $5000, to perform one for us would cost $10,000 - $12,000.

Atty. Fuller stated that the Commission should require a traffic study - they should definitely do one. My initial reaction is that they would do the report. At that point it should be reviewed and make sure that they can tell from their review if they need to go further with a full study.

Atty. Fuller noted that if you do not have the requirement for a traffic study in the regulations, you cannot compel them to perform one however, it is not unreasonable to request one. He added that they are trying to sell to the Zoning Commission that this property can handle this development. Even if they do not intend to build, they should agree to do it. He added he assumed that they would automatically perform a traffic study.

Mr. Mulholland inquired if they do not submit a traffic study and the Zoning Commission denies the application, can the Board cite the traffic concerns as a reason.

Atty. Fuller responded in the affirmative. You can get the basic data from the DOT. They can tell you what your level of service is and give you some input. They talked about a traffic light so they're talking about some change here. If they don't provide the information, as long as you document with some more specifics besides just saying they did not provide it, I think you definitely have a point here. I would be surprised if they did not do a traffic study.

Ms. Davies noted we are concerned with getting permission from them to access property. She inquired what fall back we have.

Atty. Fuller responded that you cannot force them. You have to have some basis to say why you really have to do this. There is a difference between knowing there is a real, potential problem and therefore having to have the information and or just going out there to find something wrong with the property. Generally I do not want the opposition or the Town doing studies on my clients property for a number of reasons. They may very well tell you you cannot do it and you cannot force them to do it. If you are going to raise that as a point, you should have something preliminary to suggest that there might be a problem that you want to investigate. If they say no, they say no. You may have something here about environmental constraints on the property – and you may have this somewhat on the record from this 5-acre zone business. I assume the zone thing was done with a purpose.

The Conservation Commission as part of their application for a permit, has a waiver at the bottom that says when you sign this permit you are allowing us to get on the property to do inspections and to look at property. If it gets there....

Atty. Fuller inquired what does the Conservation Commission do – conserve or...

Ms. Davies replied they are our wetlands agency – going on the property to do a site survey.

Atty. Fuller noted that if the Commission members want to go out to the property I would say that's ok, I would assume they would not object to that. That does not mean you go out with backhoes and do test holes. – a non-invasive study. He added that the environmental group could not go in with the wetlands group. You cannot do full blown environmental study.

Bill Mulholland inquired at the public hearing if we have the regulation amendment change first and the commission denies it and they choose to go to a subcommittee to write their own regulations.....

Atty. Fuller responded that they could propose their own regulations irregardless if the Town has their own or not. The applicant does not have to go under those, he can go under his own – the two are really independent of one another. You can do your own regulations anyway.

Ms. Davies noted that we had reviewed density – they can propose their own regulations but if the density is not right for the property, it won't carry it.

Atty. Fuller responded that you want to propose a set of regulations that fit your property and make sense. You have in there that water and sewer has to be available and the lot size is whatever, and so on.

Atty. Fuller continued that you need to look at their regulation in addition to the zone change – you have to act on both. The regulation should tie in with what they are proposing for their land. Your reasons for denial might be the regulation in abstract without dealing with the specifics of the property, the regulations in the abstract are too problematical because of —— and you have to have decent reasons for it.

Ms. Davies noted the Town does not want to appear exclusionary because we are not.

Atty. Fuller stated if it is unreasonable you can turn it down – you need good reasons to deny the regulation change to – the fact that you have your own is not in and of itself an automatic reason to deny, but certainly worth mentioning.

Ms. Davies said she was thinking the opposite – we do not have a regulation to promote affordable housing – we do have some affordable housing in town. When they met with

me they noted that since the Town did not have any affordable housing regulations they said they were coming in to provide us with this benefit and the town has been so exclusionary (we have approximately 4.3% - our quota is 10%) he would complete our quota.

Atty. Fuller stated that if the development of the property is unreasonable you can turn them down – you just need good reasons. He added that it is not a question of whether or not you are exclusionary, it's a question of if you are meeting the goal. Has to be a suitable project.

Ms. Davies noted that Wayne has requested staff not to talk to Russo and Landmark about any other potential development. If they come in and want to talk subdivisions that can be allowed on the property currently.....

Atty. Fuller stated there is no reason not to talk to him. Let them come in like anybody else.

Ms. Davies stated that a future meeting would be at the discretion of Mr. Fraser.

Atty. Fuller responded we will see what happens – see if they withdraw. Wayne can call when he wants and we will go from there.

The call ended at 9:40 a.m.

Nickerson How?

Bellis Because it said, 'it's especially true' of that particular site.

Nickerson Okay, we'll move on. Sir.

Bulmer Mr. Bellis, I have some questions. I'd like to refer to the letter that you just read. No page numbers here, but.....Oh, okay, page 7, third reason, well let me back up a few sentences so I don't lose the gist of this. Starting right in the middle. See that 'Moreover, the Niantic River"? Middle of the first paragraph. "Moreover the Niantic River in the area of the applicant's property has been acknowledged to be polluted by failing septic systems in the Golden Spur area." Now, this is the sentence I want to call your attention to. 'The applicant's willingness to pay for the extension of sewer lines to its property through Golden Spur area would actually provide the means to correct, not add to that pollution'. We had a rather animated discussion last week about sewer lines coming out. You had your expert, Mr. Jason Sarojak, who spoke to sewer lines and water lines last week. And I kept trying to push him on the issue of where to connect the sewer lines and water lines to the point where Mr. Zizka jumped in to save him because he didn't come up with a good answer. But, let me quote you a few lines.

Bellis I don't agree with that characterization.

Bulmer I understand. But this is my point that I'm making. And you can respond to it when I'm finished please. Let me quote to you some items from prior testimony. "Lack of public sanitary sewers: The statutory report of the Planning Commission, Exhibit 6, (this was back in the initial testimony before the appeal) and supporting documents and testimony of the Planning Director and Director of Public Works- Exhibit 13- the testimony of the 1st Selectman and Chairman of the Water & Sewer Commission-Exhibit 11 (these three pieces of testimony) reference report from the State of CT Office of Long Island Sound Programs –Exhibit 10-(there's four different people testifying) has provided sufficient evidence that public sewers are not available and that the subject area is outside the sewer shed boundary". You heard additional testimony tonight to that effect from both the 1st Selectman and from the Office of Long Island Sound Programs. So, I ask you again, where are you going to connect your sewer lines?

Exhibit 3

DOCKET NO: HHD CV-15-6056637-S	: SUPERIOR COURT
LANDMARK DEVELOPMENT GROUP, LLC Et Al	: JUDICIAL DISTRICT OF
V.	: HARTFORD
EAST LYME WATER & SEWER COMMISSION	: JULY 6 2016

MEMORANDUM OF DECISION

Prior to the commencement of the present action, the plaintiff, Landmark Development Group, LLC, brought an appeal against the defendant, East Lyme Water and Sewer Commission, regarding a sewer capacity determination. Before rendering a decision, the court reviewed the record, including the methodology for the grant of capacity. On June 26, 2014, the court ruled that the defendant must reconsider the allocation of sewer capacity in the amount of 13,000 gallons per day to the plaintiff, Landmark Development Group, LLC. See *Landmark Development Group, LLC* v. *East Lyme Water & Sewer Commission*, Superior Court, judicial district of Hartford, Docket No. CV-13-6040390-S (June 26, 2014, *Cohn, J.*). In so ruling, the court indicated that the defendant must consider the *Forest Walk, LLC* v. *Water Pollution Control Authority*, 291 Conn. 271, 968 A.2d 345 (2009) factors. More specifically, in regard to capacity, the defendant must "consider the remaining capacity for the entire town, the land area represented by the property versus the available land area in the town, the safe design standards

Mailed to all counsel & OCR 7/6/16 ab/co

for the public sewer, and the percentage of the allocation versus the total remaining capacity." *Landmark Development Group, LLC* v. *East Lyme Water & Sewer Commission*, supra, Superior Court, Docket No. CV-13-6040390-S. On July 29, 2014, the court denied the defendant's motion to reargue. See *Landmark Development Group, LLC* v. *East Lyme Water & Sewer Commission*, Superior Court, judicial district of Hartford, Docket No. CV-13-6040390-S (June 29, 2014, *Cohn, J.*).

In the present action, which was commenced on November 24, 2014, the plaintiffs, Landmark Development Group, LLC, and Jarvis of Cheshire, LLC, ask the court to review a grant of capacity of 14,434 gallons per day to the plaintiffs by the Board. On February 19, 2015, the plaintiffs filed their appeal brief. On March 16, 2015, the defendant, East Lyme Water and Sewer Commission, filed its appeal brief.[1] On March 30, 2015, the plaintiffs filed a motion for permission to supplement the record in administrative appeal. The court heard oral argument on April 2, 2015. On the same day, the court granted the plaintiffs' request, but only as to exhibit C, a letter from Mark S. Zamarka.

On July 23, 2015, the plaintiffs filed a motion to conduct further discovery/deposition, and to supplement the record. Specifically, the plaintiffs asked the court for permission to take

[1] The two intervening entities, Friends of the Oswegatchie Hills Nature Preserve, Inc., and Save the River-Save the Hills, Inc., have also filed briefs in this action.

the deposition of the Board's administrator, Bradford Kargl, regarding approval of the connection application by Gateway (a similarly-situated apartment complex being developed) where over 160,000 gallons per day capacity was contemplated. The motion was granted by the court on September 8, 2015. The deposition revealed that although Kargl was aware of the Gateway capacity need (Plaintiffs' Exhibit 1, Deposition of Kargl, pp. 39-42/A28-A31, 52/A41, 62/A50), and had the duty to monitor this need (Plaintiffs' Exhibit 1, pp. 15/A9, 17/A10, 61-63/A49-51, 69/A57), he approved the connection application without making a capacity determination (Plaintiffs' Exhibit 1, pp. 33/A23, 66-71/A54-58, 74/A62), and without further reference to the Board (Plaintiffs' Exhibit 21).[2]

The court, as indicated in prior rulings, does not believe that a capacity determining action is ministerial, but is instead a matter of discretion for the Board. See *Forest Walk, LLC* v. *Water Pollution Control Authority*, supra, 291 Conn. 282 ("[A] municipality has wide discretion in connection with the decision to supply sewerage. . . . Although this discretion is not absolute, [t]he date of construction, the nature, capacity, location, number and cost of sewers and drains are matters within the municipal discretion with which the courts will not interfere, unless there appears fraud, oppression or arbitrary action." [Internal quotation marks omitted.]); see also

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[2] The fact that Kargl failed to even review capacity as to Gateway distinguishes this case from the *Forest Walk* factors which have guided the court to this point.

Straw Pond Associates, LLC v. *Water Pollution Control Authority*, Superior Court, judicial district of Waterbury, Docket No. CV-08-4015126-S (March 8, 2011, *Gallagher, J.*) (discretionary standard of review applied to determination of availability of sewer capacity). The defendant's actions are discretionary even where there is a request for a sewer extension permit. See *Landmark Development Group, LLC* v. *East Lyme*, 374 Fed. Appx. 58, 60 (2d Cir. 2010) ("Plaintiffs had no legitimate claim of entitlement to a sewer-extension permit. Defendants plainly have discretion to deny such permits.").

In light of the supplemental evidence, the court concludes that there is at least 200,000 gallons per day capacity (358,000 gallons per day less 160,000 gallons per day to Gateway) for the entire sewer system.[3] The defendant had broad discretion in determining capacity, but the defendant was obligated to consider capacity when it approved the connection application for Gateway. As to the plaintiff, the court finds that with the large amount of capacity remaining,

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[3] In its prior June 26, 2014 decision, this court noted that, as to remaining capacity, "[t]he record before the court shows a range of 130,000 gpd to 225,000 gpd. At the meeting of the commission on February 25, 2014, the figure of 177,000 gpd was used as a compromise. In court on May 27, 2014, the commission's attorney conceded that the commission would not object to a figure of 250,000 gpd. Finally, Landmark points to a reduced usage by the town and state facilities so that the correct figure is between 308,000 gpd and 358,000 gpd." *Landmark Development Group, LLC* v. *East Lyme Water & Sewer Commission*, supra, Superior Court, Docket No. CV-13-6040390-S. More recently, during the commission's October 2014 remand proceeding and resolution, the commission applied the plaintiff's figure of 358,000 gallons per day. (Amended Return of Record, Exhibit D, Postproceeding Exhibits 2, 3).

the capacity figure of 14,434 gallons per day is excessively low. There is an abuse of discretion[4] that the Board must correct. Although the Board is not required to grant the plaintiffs their request for 118,000 gallons per day, the capacity figure of 14,434 gallons per day is insufficient in view of the present remaining capacity of at least 200,000 gallons per day, and in view of the 160,000 gallons per day that was approved for Gateway. In reconsidering the allocation of the sewer capacity, the Board must comply with applicable sewer statutes, regulations and ordinances, and the Board should take into account the demands of the plaintiffs' sewer project and the effect on remaining capacity. Nevertheless, the Board must provide the plaintiffs with sufficient capacity to further the development of their project, and, as such, the Board may not settle on a figure for capacity that would completely foreclose the development of the plaintiffs' project.

This matter is remanded to the Board for a further ruling and is a final decision for purposes of appeal.

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[4] "When a water pollution control authority performs its administrative functions, a reviewing court's standard of review of the [authority's] action is limited to whether it was illegal, arbitrary or in abuse of [its] discretion . . . . Moreover, there is a strong presumption of regularity in the proceedings of a public agency, and we give such agencies broad discretion in the performance of their administrative duties, provided that no statute or regulation is violated." (Citation omitted; internal quotation marks omitted.) *Forest Walk, LLC* v. *Water Pollution Control Authority*, supra, 291 Conn. 285-86.

SO ORDERED,

COHN, JTR

Exhibit 4

RESOLUTION REGARDING INTERIM SEWER CONNECTION PROCEDURE

SEPTEMBER 25, 2018

WHEREAS, on June 1, 2012, Landmark Development Group, LLC and Jarvis of Cheshire ("Applicant") filed with the East Lyme Water and Sewer Commission ("Commission"), acting as the East Lyme Water Pollution Control Authority, an application "pursuant to §7-246a(1) of the General Statutes, seeking confirmation of the availability of 237,090 gallons per day of sewage disposal capacity in the Town's sewer system to serve Landmark Development's proposed residential development adjacent to Caulkins Road"; and

WHEREAS, at the public hearing on the application held on August 24, 2012, Landmark amended its application to request availability of 118,000 gallons per day of sewage disposal capacity in the Town of East Lyme's ("Town") sewer system; and

WHEREAS, the Commission held three public hearings on the application and listened to hours of testimony during those hearings. Numerous exhibits were submitted by Landmark, the Commission, and individuals for consideration during the hearing process. In making its decision the Commission is considering and taking into account all of the testimony and exhibits submitted at the three hearings; and

WHEREAS, the Commission has wide discretion in connection with the decision to supply sewer service to particular properties; and

WHEREAS, the Commission found that as of Landmark's application in 2012, the Town had between 130,000 and 225,000 gallons per day of remaining sewage treatment capacity; and

WHEREAS, Landmark appealed the Commission's capacity allocations to the Connecticut Superior Court; and

WHEREAS, the New Britain Superior Court (Cohn, J.) (the "Trial Court") allowed Landmark to conduct discovery regarding a sewer connection permit for a different development project, known as "Gateway," and allowed Landmark to supplement the record on appeal with documents related to the Gateway connection application; and

WHEREAS, on July 6, 2016, the Trial Court issued a Memorandum of Decision holding in part that:

1. The Commission "... is not required to grant the plaintiffs their request for 118,000 gallons per day ..."
2. The Commission "... must provide the plaintiffs with sufficient capacity to further development of their project, and ... may not settle on a figure that would completely foreclose the development of the plaintiffs' project."

3. The Commission "... was obligated to consider capacity when it approved the connection application for Gateway."

WHEREAS, the Commission appealed the Memorandum of Decision to the Connecticut Appellate Court; and

WHEREAS, on August 21, 2018, the Appellate Court issued its decision ("Decision") on the Commission's appeal, which upheld the Trial Court Memorandum of Decision, and held that the Commission is required to perform a sewer capacity analysis when considering applications to connect to the East Lyme sewer system; and

WHEREAS, the Commission disagrees with the Decision and has filed a petition for certification to the Connecticut Supreme Court, which is currently pending; and

WHEREAS, by a letter dated September 17, 2018, Landmark requested that the Commission approve an allocation for its full 118,000 gpd sewer capacity request, pending final resolution of its appeal; and

WHEREAS, neither the Trial Court nor the Appellate Court held that Landmark was entitled to the full amount of its capacity request, and the proceedings are stayed until the Supreme Court acts on the Commission's petition for certification. While reserving all of its rights set forth during the appeal process, the Commission nevertheless does not want to ignore the Trial Court and Appellate Court holdings that require a sewer capacity analysis be done in conjunction with a sewer connection permit application.

BE IT THEREFORE RESOLVED, that the East Lyme Water and Sewer Commission, acting as the Town's Water Pollution Control Authority, hereby enacts the following interim procedure:

1. An application to connect to the East Lyme sewer system for a project that either (a) requests a connection for more than 20 residential units or (b) requires more than 5K gallons per day of sewage treatment capacity, shall also require an application for determination of sewer capacity pursuant to General Statutes §7-246a;
2. Said application for determination of sewer capacity shall be submitted either prior to or contemporaneously with a sewer connection application;
3. An application to connect to the East Lyme sewer system may not be granted if the Commission determines that there is not adequate sewer capacity for the proposed use of land.

BE IT FURTHER RESOLVED that the above procedure does not reflect official policy or procedure of the Commission or the Town of East Lyme. Rather, it is adopted on an interim basis only in direct response to the Appellate Court Decision, and shall be in place only during the pendency of the Landmark sewer capacity appeal process. In enacting this interim procedure, the Commission does not agree with the holdings of the Trial Court Memorandum of Decision or the Appellate Court Decision. Any findings made pursuant to this interim procedure (i.e. available sewer capacity, etc.) shall be for the purposes of that sewer capacity application only, and shall not be adopted, incorporated or made part of the record in the pending Landmark sewer appeal.

Exhibit 5

APPLICATIONS FOR DETERMINATION OF ADEQUACY OF SEWER CAPACITY PURSUANT TO GENERAL STATUTES §7-246a(a)(1)

Sewage treatment for the Town of East Lyme is limited. Pursuant to an agreement with the City of New London and Town of Waterford, East Lyme is currently entitled to a maximum of 1.5 million gallons per day of sewer treatment capacity at the New London Regional Water Pollution Control Facility. In order to ensure that there is adequate capacity for all customers, the Commission adopts the following regulation for applications for sewer treatment capacity pursuant to General Statutes §7-246a(a)(1).

I. Application. For all development projects that either (a) request a connection for more than 20 residential units or (b) require more than 5,000 gallons per day of sewage treatment capacity, an application, pursuant to General Statutes §7-246a(a)(1), for determination of adequacy of sewer capacity related to a proposed use of land, shall be submitted to the East Lyme Water and Sewer Commission ("Commission") on a form satisfactory to the Commission, and shall include all of the following:

1. A class A-2 survey of the property to be developed, showing the general layout of the proposed use of land;

2. Proof that the applicant owns the property to be developed, or has the right to develop the property, and

3. Documentation supporting the amount of capacity being requested.

   a. Documentation related to a proposed residential development shall include the number of residential units, the numbers of bedrooms per unit, and the methodology used in calculating the amount of capacity being requested.

   b. Documentation related to a proposed non-residential or commercial development shall include the methodology used in calculating the amount of capacity being requested, and any special circumstances (i.e. the type of sewage being treated, design specifications, etc.) that would affect the amount of capacity being requested.

   c. The Commission reserves the right to request from an applicant such other information that it deems necessary.

4. ~~An application fee of $250.00~~ A non-refundable application fee of $500.00 shall be paid when ~~an application is submitted~~ by the applicant submitting the application.

Should a public hearing be required there will be an additional $450.— due to cover the Public Hearing costs —

Final - Was Spec. Mtg 1/14/19

III. Public Hearing. The Commission may, in its sole discretion, hold a public hearing on any application. Any such public hearing shall be in accordance with the provisions of General Statutes 8-7d.

IV. Criteria. In making a decision on an application the Commission may consider, without limitation, the following:

Need for service in the proposed development area

Other pending applications and areas in town designated for sewer service

Pollution abatement and public health

Limitations and policies for sewer service

Local and state Plans of Conservation and Development

Effect of inflow and infiltration on available capacity

Whether the proposed development area can be serviced by other means

Whether the proposed development area is within the East Lyme Sewer Service District

Size of property proposed to be developed

Remaining sewered and unsewered land area of town

Effect of the allocation on remaining capacity

Safe design standards of the East Lyme sewer system

V. Prior Regulation. This Regulation shall supersede the Interim Sewer Connection Procedure adopted by the Commission on September 25, 2018.

Exhibit 6

theday

Appellate court rules against East Lyme in sewage capacity case

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By **Martha Shanahan** (/apps/pbcs.dll/personalia?ID=m.shanahan) Day staff writer

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In the latest step of a **protracted legal battle (http://civilinquiry.jud.ct.gov/CaseDetail/PublicCaseDetail.aspx?DocketNo=HHDCV156056637S)** between East Lyme's Water and Sewer Commission and the developer of a proposed housing development, an appellate court in Hartford **has ruled (http://civilinquiry.jud.ct.gov/DocumentInquiry/DocumentInquiry.aspx?DocumentNo=15246021)** that the commission must grant the developer more access to the town's sewer system than the commission wants to give it.

The town's lawyers plan to petition the state Supreme Court to appeal the Aug. 21 ruling, which affirms a **state Superior Court judge's 2016 order (https://www.theday.com/local/20160724/judge-says-east-lyme-commission-must-reconsider-sewage-capacity-for-oswegatchie-hills-development)** that the commission must reconsider the amount of sewage capacity it is willing to grant for a proposed 840-unit residential development adjacent to the Oswegatchie Hills Nature Preserve along the Niantic River.

Over more than a decade, Landmark Development has sought to develop houses on the 236 acres it owns in the Oswegatchie Hills.

The plan has generated local opposition **(http://www.theday.com/local/20160609/group-seeking-to-prevent-oswegatchie-hills-development-rallies-for-support)**, which in recent years has taken the form of a coalition between Connecticut Fund for the Environment and two local groups arguing that the development would pollute the Niantic River and degrade wetlands on the property.

Landmark Development and its president, Glenn Russo, also have hit speedbumps before the town's Water and Sewer Commission, which regulates new connections to the pipes and pumps that bring sewage from East Lyme buildings through Waterford to a sewage treatment plant in New London.

A deal between East Lyme, Waterford and New London allows each town to send a certain amount of sewage to the New London sewage treatment plant — 15 percent of the plant's capacity, or about 1 million gallons a month in East Lyme's case — and limits the towns' ability to grant permission to build new sewer lines or allow new developments to connect to the existing ones.

In 2014, the Water and Sewer Commission denied Landmark's request for a guaranteed 118,000 gallons of sewage capacity per day for the development.

Landmark **appealed that decision (http://www.theday.com/article/20130107/NWS01/301079951/0/search)** in New London Superior Court in 2014, kicking off the five-year ongoing debate in several courts over the commission's claims that the town's sewage system can't handle the amount of wastewater that a development the size of the Landmark proposal would generate.

The commission's members said that year that it could allow Landmark to generate only 14,434 gallons per day in sewage for the proposed houses, a fraction of the 118,000 gallons per day Landmark asked for in 2014.

Landmark's lawyers **have argued (http://civilinquiry.jud.ct.gov/DocumentInquiry/DocumentInquiry.aspx?DocumentNo=9689953)** that the commission granted the developer of a different housing complex in East Lyme, Gateway Commons, about 70,000 gallons of sewage capacity per day and told Gateway developers that the town had the capacity to handle about 100,000 additional gallons per day from the development. The commission's decision to grant that capacity to the Gateway development shows the town has "ample" sewage capacity for the Oswegatchie Hills proposal, they said.

Hartford Superior Court Judge Henry S. Cohn said in his 2016 ruling that 14,434 gallons per day is "excessively low" in light of the allocation to Gateway, and remanded the issue to the commission.

Town lawyers say the Gateway development's sewer capacity has no bearing on the Landmark case, because Gateway Commons is near one of the town's existing sewer lines and was relatively easy to connect to the system, whereas Landmark's proposal would require the construction of a new line.

The two development projects are "like apples and oranges," said East Lyme First Selectman Mark Nickerson, who is also the chairman of the Water and Sewer Commission as directed by the town's charter. "There's a difference between a connection and an extension," he said.

The appeals court dismissed that argument last month.

"Although the commission concluded that it did not have sufficient capacity to grant the plaintiff's application for up to 118,000 gallons per day, (Gateway) had effectively been granted an allocation of approximately 166,000 gallons per day," the court wrote in its ruling.

"At the end of the day that's not a valid argument," said Timothy Hollister, an attorney with the Hartford law firm Shipman & Goodman representing Landmark in the case. "The Water and Sewer Commission ... determined that the town as a whole has so much capacity that they can grant 166,000 gallons to Gateway ... but they have fought Landmark tooth and nail on every gallon of our request."

Nickerson said he is confident in the town's appeal.

The commission should have the ability to oversee management of its sewage systems without court interference, he said.

"The judges can't force us to put the sewer in there," he said.

He added that the extension of the sewer lines to the Oswegatchie Hills would constitute an unsuitable use of the town's increasingly limited capacity for adding new inputs to the sewer system and would eat up sewage capacity the town is saving for other neighborhoods where the houses still use septic systems.

The Department of Energy and Environmental Protection has put pressure on the town to expand sewer capacity to those neighborhoods to alleviate pressure on aging septic systems, which takes priority over development proposals like the Landmark plan, Nickerson said.

"If we had unlimited capacity and unlimited funds, we would give out all sorts of capacity," he said.

Exh. 37 9/8/04

Exhibit 7

RESOLVED: The East Lyme Board of Selectmen desires to state as a matter of public record that it is worthwhile to remind the Zoning Commission that the Oswegatchie Hills is especially suitable for preservation as open space, and that the public interest is best served by insuring that this property remain in it's present undeveloped state for use by future generations of the public. The Board of Selectmen urges the Zoning Commission to take this into consideration as it makes its decision on Landmark Investment Groups' application.

9/2/04

Rose Ann Hardy
read - requested for
Board of Selectmen

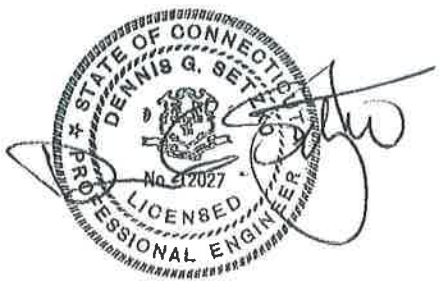
Exhibit 8

Evaluation of Capacity at the Thomas E. Piacenti Wastewater Treatment Plant New London, Connecticut

Prepared for:

PUBLIC UTILITIES
CITY OF NEW LONDON, CONNECTICUT

May, 2012



Prepared by:

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I. Executive Summary

In 2006, an Order was issued to the WWTF in New London and its member communities by Connecticut DEEP because of permit excursions for BOD, TSS, and chlorine residual and also because the flow at the plant exceeded 90% of the arithmetic mean for the previous 180 days. Future flow capacity needs of the member communities were cited in this Order as an additional reason to investigate capacity at the plant. This Order required that an engineering report be developed to address these issues.

AECOM was retained to develop this engineering report. The scope of services was tailored to evaluate the individual unit processes and determine the actual plant capacity in light of future flows and permit needs, and to determine what, if anything, needs to be improved to provide this capacity. To effectively evaluate plant capacity, AECOM:

- determined the amount of growth expected over the next 20 years;
- evaluated the treatment plant's ability to meet that growth from both a hydraulic and treatment process standpoint;
- identified process and hydraulic bottlenecks or limitations;
- identified improvements necessary to meet future capacity needs;
- determined at which time these improvements would be required.

Cost estimates, proposed schedules and potential funding sources are identified.

A. Future Flows and Organic Loads

Future flows and organic loads for the City of New London were developed using current flows and loads as a baseline, water use records and the most current Water Supply Plan, population projections, and available planning documents. Information from the communities of Waterford, East Lyme and Old Lyme were provided by the individual towns or from DEEP. Table I-1 presents the future flows and loads that are projected for the WWTF for a 20 year planning period, ending in the year 2032.

Table I-1: Projected Future Flows and Loads

	Annual Average	Maximum Monthly Average	Hydraulic Peak
Flow, mgd	10.85	15.45	28
BOD, lb/d	21,282	25,600	
TSS, lb/d	18,828	24,500	
TKN, lb/d	2,798	3,450	

The current plant rated capacity is 10 mgd. Most of the additional flow needs are from East Lyme.

B. Treatment Process Evaluation

The Thomas E. Piacenti plant was designed to perform to secondary treatment levels in a mid-1970's upgrade and was again modified in the late 1990's to nitrify and partially denitrify. To determine how much process capacity remains after these modifications, a wastewater characterization program was undertaken as the first step of the process modeling required for this project. The characterization program comprised 6 days' worth of sampling spread over 2 weeks and sampled wastewater through the treatment process. A wastewater process model was created and calibrated, and future performance was able to be predicted with this model. The following process performance limitations were identified and the trigger point at which action is advised is provided:

Table I-2: Process Performance Limitations

Process	Flow at which action is required	Operational Change	Structural Change	Comment
Preliminary Treatment	None			No action required
Primary Clarifiers	9.8 mgd ADF, 19.6 mgd peak hourly	Yes	No	Clarifier #3 configuration needs modifications to provide continuous and effective level of service. Clarifier #3 operates best when operated during flow periods when velocities are higher.
Aeration tanks	None	Yes	No	Increase MLSS as loadings increase up to a maximum of 4000 mg/L. Use both aeration tanks.
Secondary Clarifiers	10 mgd. See comment section	Yes	Yes	Use clarifiers in accordance with a maximum SLR of 15.4 lb/sf/d at an SVI of 200 mL/g and up to 30.8 lb/sf/d for an SVI of 150 mL/g. Add a fourth secondary clarifier when the MLSS required to maintain nitrification reaches 3300 mg/l.

Secondary clarifier capacity is dependent on solids loading, sludge settling characteristics and settled sludge concentration. As such, it is frequently changing. Assuming that the treatment capacity of the aeration basins is maximized in the future (i.e. high MLSS) which is reasonable considering the additional flows expected, and assuming an SVI of 150, temperatures of 13°C,

and a required safety factor of 1.3, the maximum month clarifier capacity is 14.3 MGD. Deducting for the average to max month ratio of 1.42, the plant capacity is 10.0 MGD.

C. Treatment Plant Hydraulic Evaluation

A hydraulic evaluation of the facility using engineering hydraulic spreadsheets and Computational Fluid Dynamic (CFD) modeling was completed. Spreadsheets are commonly used to develop a hydraulic profile for different flow regimes, determine hydraulic losses, and identify elevations of water surfaces. CFD on the other hand, actually simulates the interaction of the wastewater with the structures boundary conditions. This provides a more detailed evaluation of individual process performance and is helpful in determining solutions to complex hydraulic problems.

The results of the desktop and CFD analysis show the following:

- The openings between the anoxic and aerobic tanks should be increased in size to reduce losses between the two zones and provide hydraulic capacity for the peak flow of 28 mgd and recommended internal recycle rate;
- The weirs at the end of the two aerobic tanks are at slightly different elevations (average 0.6 inch difference) and should be adjusted to be equal;
- The flow split to Secondary Clarifiers 1 and 2 is equal within reasonable expectations;
- The flow split to Secondary Clarifiers 1 and/or 2 when Secondary Clarifier 3 is also in operation is unequal causing overloading of Clarifier 3 and underloading of Clarifier 1 and/or 2. This can be corrected with adjustments of the slide gates that isolate the clarifiers when these configurations are in use;
- The operational status of Primary Clarifier #3 is that it is not suitable for continuous use unless a better method of scum removal is provided. Primary Clarifiers #1 and #2 should be used preferentially from an operational perspective. Primary Clarifier #3 works best in combination with Primary Clarifiers #1 or #2 when the velocities are higher;
- When flows exceed 9.8 mgd ADF or 19.6 mgd peak flow, Primary Clarifier #3 should be brought on-line to address hydraulic overload;

D. Additional Findings for Process Improvements

1. pH Control

The pH at the plant averages approximately 6.7 which is not uncommon in plants that nitrify. However, at a pH of 6.7, the nitrification rate, which is the governing factor in aeration tank capacity, is approximately 70% of maximum. The effect of the depressed pH is to lengthen the overall SRT needed for full nitrification. Ultimately, this reduces the capacity of the aeration basins. AECOM does not recommend adding an alkalinity control system at this time, although it will be cost effective to add this capability in the future to increase the capacity of the aeration basins slightly and reduce solids loading

to the secondary clarifiers. This process would require approximately 1,000 gallons of sodium hydroxide per day.

2. Aeration Capacity

The current blowers do not have a great deal of speed and capacity turndown and, as a result, provide more air to the aeration basins than is necessary and desired. This increases operational cost and reduces nutrient removal performance. Increasing the turndown either through blower rehabilitation (vane replacement) or replacement with smaller, more efficient blowers is recommended. This work can be part of a capital improvement plan.

E. Implementation Schedule

The estimated schedule of implementation based on capacity or performance needs is shown below. Note that this is not a time driven schedule, but one based on either future flows or loadings or improved process performance. As such, a time schedule is not provided since NPDES permit performance is currently not an issue.

Near Term:

- Set weir elevations at aeration tanks to be equal;
- Increase size of wall openings in the aeration/anoxic zone walls;
- Add optional jockey blower with higher efficiency and greater turndown;

Intermediate Term:

- Reconfigure Primary Clarifier #3 to enhance scum removal.

Loading Based Improvements:

- Add a fourth secondary clarifier when the MLSS needed to maintain nitrification is 3,300 mg/L.
- Consider pH control system to add alkalinity and reduce SRT;

F. Cost Estimate

Estimated construction costs including construction contingencies, administrative, legal, construction engineering, environmental and regulatory permitting costs were developed for the recommendations from this report:

- Reconfiguring Primary Clarifier #3;
- Cut openings in aeration tank walls;
- Addition of a fourth secondary clarifier;
- Addition of an optional jockey blower.

Operation and maintenance and life cycle costs were also estimated for these recommendations. The table below includes the life cycle costs of these improvements.

Table I-3: Life Cycle Cost Estimates

	Rehabilitate Primary Clarifier #3	Additional Secondary Clarifier	Aeration Basin Wall Openings	Additional Jockey Blower
Civil	$0.00	$188,900.00	$0.00	$0.00
Structural	$60,000.00	$598,500.00	$25,000.00	$5,000.00
Architectural	$0.00	$0.00	$0.00	$0.00
Mechanical	$217,800.00	$325,500.00	$0.00	$180,000.00
HVAC	$0.00	$0.00	$0.00	$0.00
Plumbing	$0.00	$0.00	$0.00	$0.00
Electrical & Instrumentation	$25,000.00	$75,000.00	$0.00	$35,000.00
Subtotal	$302,800.00	$1,187,900.00	$25,000.00	$220,000.00
Contingency (30%)	$90,900.00	$356,400.00	$7,500.00	$66,000.00
Subtotal	$393,700.00	$1,544,300.00	$32,500.00	$286,000.00
Contractor Overhead & Profit (20%)	$78,800.00	$308,900.00	$6,500.00	$57,200.00
Estimated Construction Cost (2011)	$472,500.00	$1,853,200.00	$39,000.00	$343,200.00
Professional Services (20%) *(includes Legal and Administrative)*	$95,000.00	$371,000.00	$8,000.00	$69,000.00
Total Capital Cost (2011)	$567,500.00	$2,224,200.00	$47,000.00	$412,200.00

G. Funding

Capital improvements to wastewater treatment facilities are eligible for the state grant-loan program funding through the DEEP Clean Water Fund Program. Priority points are assigned to the project and funds disbursed to those projects with the most points. This list is updated every other year with the next update scheduled for January 2012.

Connecticut Light and Power (CL&P) offers incentives to encourage the design of energy efficient plant improvements such as blower replacements, high efficiency motors and variable frequency drives.

III. Treatment Plant Flows and Loads

A. Current Flows and Loads

The wastewater flows and loads at the treatment plant are comprised of a number of components each with their own separate characteristics. Sanitary wastewater within the sewer system is broadly characterized as either of residential, commercial or industrial origin. The infiltration and inflow (I/I) component of the wastewater can sometimes generate significant flows but very little loading. Septage is a separate component not found in the collection system itself but is delivered to the treatment plant on a regular basis for treatment. The septage component of wastewater can at times create a significant load at the plant but have a negligible flow component.

Table III-1 presents the current flows and loadings at the New London treatment facility. The flow information provided in this table is based on 2½ years of MOR data from March, 2008 through August, 2010.

Table III-1: Current Wastewater Flows and Loads

Flow and Load	Quantity
Average Daily Flow (GPD)	8,320,000
Maximum Month Flow (GPD)	12,860,000
Annual Average BOD (pounds/day)	15,094
Annual Average TSS (pounds/day)	11,780
Annual Average TKN (pounds/day)	1,673

Figure III-1 through Figure III-4 depict facility influent flow and BOD, TSS and TKN loadings for this period.

II. Introduction

In 2006, an Order (refer to Appendix A) was issued to the WWTF in New London and its member communities by Connecticut DEEP because of permit excursions for BOD, TSS, and chlorine residual and also because the flow at the plant exceeded 90% of the arithmetic mean for the previous 180 days. Future flow capacity needs of the member communities were cited in this Order as an additional reason to investigate capacity at the plant. This Order required that an engineering report be developed to address these issues.

AECOM was retained to develop this engineering report. The scope of services was tailored to evaluate the individual unit processes and determine the actual plant capacity in light of future flows and permit needs, and to determine what, if anything, need to be improved to provide this capacity. To effectively evaluate plant capacity, AECOM:

- determined the amount of growth expected over the next 20 years;
- evaluated the treatment plant's ability to meet that growth from both a hydraulic and treatment process standpoint;
- identified process and hydraulic bottlenecks or limitations;
- identified improvements necessary to meet future capacity needs;
- determined at which time these improvements would be required.

Various improvements are recommended and a timeframe for their implementation is provided. Cost estimates, proposed schedules and potential funding sources are identified.

Previous facilities planning studies include the "September, 1998 Facilities Planning Study for the City of New London" by Camp, Dresser & McKee and the "November, 2005 Facilities Planning Study for the City of New London" by Malcolm Pirnie.

Figure III-1: Influent Flow

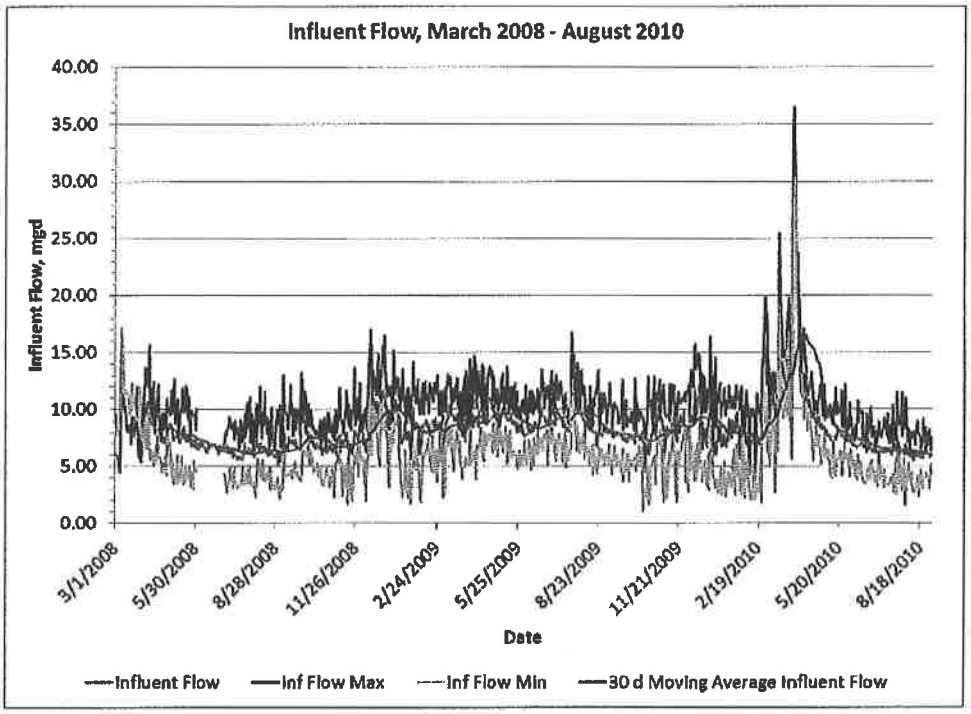


Figure III-2: Influent BOD Loading

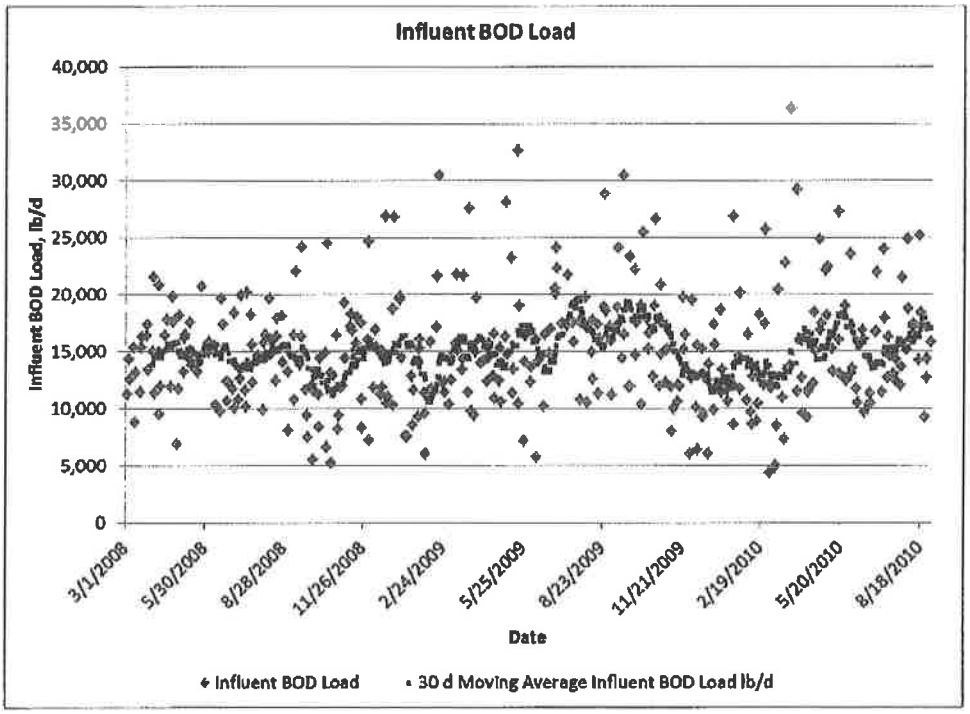


Figure III-3: Influent TSS Loading

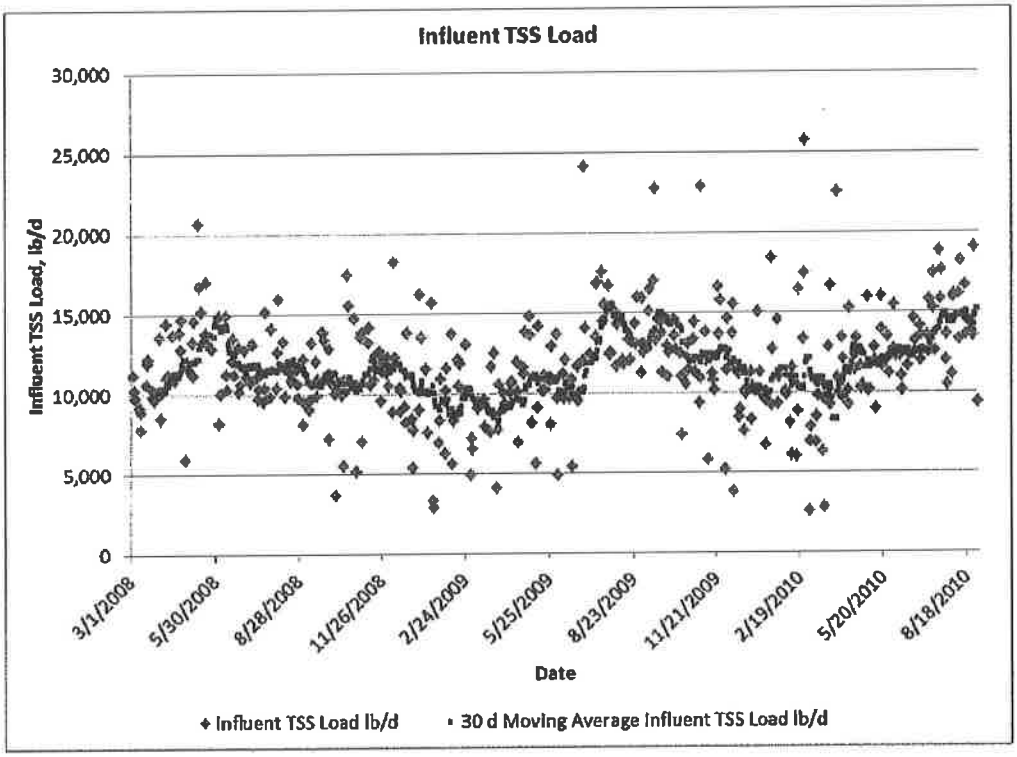
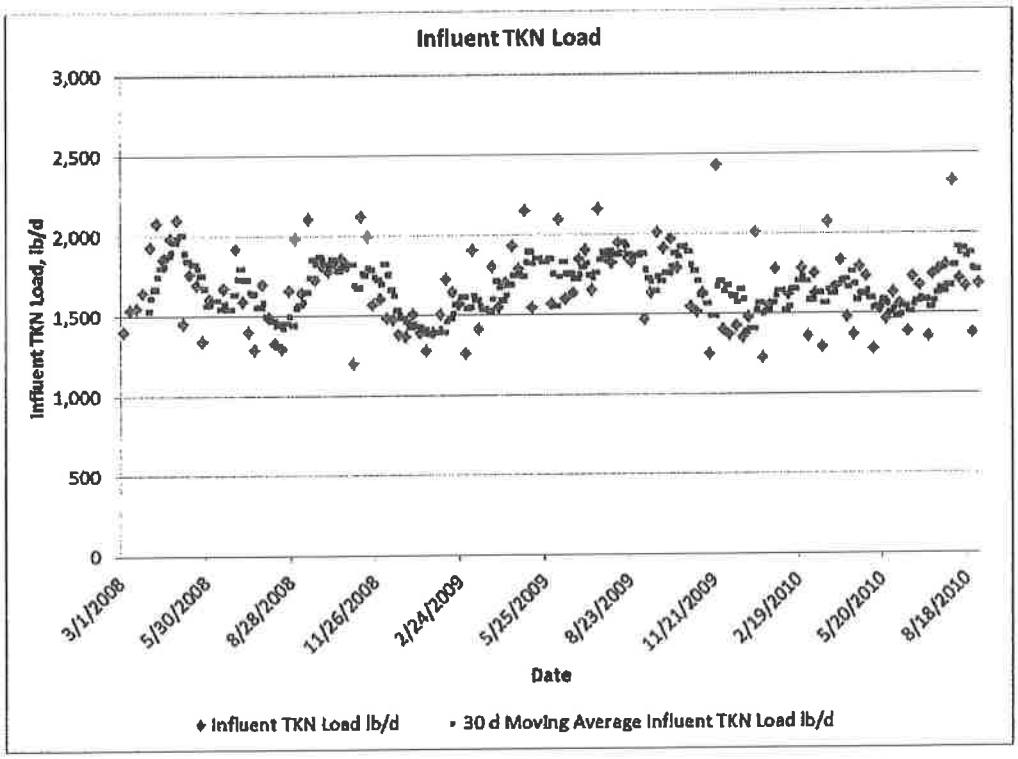


Figure III-4: Influent TKN Loading



At the New London wastewater facility, flows are contributed by New London and sections of Waterford and East Lyme. The Point O' Woods beach association in Old Lyme recently connected to the sewer system, however, for the time period the data was compiled, Old Lyme did not contribute any wastewater. The proportion of flows and loads and their distribution are discussed below.

1. Existing Wastewater Flows from Waterford

Table III-2 presents the wastewater flows for the period from January, 2006 to July, 2008 as provided by the engineering consultant for the Town of Waterford. It is reasonable to assume that the wastewater flow has not changed significantly since this time period. Waterford flows are based on flowmeter data from the Evergreen pump station in Waterford. All of the flow from East Lyme is directed here as well as most of Waterford. There are small areas in New London which flow into Waterford (line 3 below in Table III-2). There is also an area in Waterford which flows to New London (line 4). The flows from East Lyme (line 2) and New London (line 3) are subtracted to obtain the current average flow from Waterford of 2.37 MGD.

Table III-2: Town of Waterford Current Wastewater Flows

	Annual Average Flow (MGD)
(1) Total Flow to Evergreen Pump Station	3.73
(2) East Lyme Flow to Evergreen Pump Station	- 1.18
(3) New London flow into Waterford	- 0.19
(4) Waterford flow to New London	+ 0.013
Total Current Flow – Waterford	2.37

**Based on flow records for the Evergreen Pump Station from January, 2006 to July, 2008*

2. Existing Wastewater Flows from East Lyme

The Town's sewerage system was completed in 1991 and consists of approximately 2,800 sewer connections. East Lyme currently has 1,500,000 gallons per day of allocated treatment capacity at New London's WPCF. According to the inter-municipal agreement between East Lyme and New London, the limit is based on 15% of the current 10 MGD capacity at the New London WPCF.

According to the "September, 2007 East Lyme Wastewater Collection System Capacity Analysis Planning Report", a flowmeter at the Niantic Pump Station measures all of the wastewater flow from East Lyme before being pumped to Waterford. Flow records provided by the Town of East Lyme indicate an average daily flow of 1.1 MGD for 2007 and 2008. The maximum monthly flow for that period was 1.206 MGD. It is reasonable to assume that the wastewater flow has not changed significantly since this time period. According to the report, there is a seasonal increase in the wastewater flow due to an influx of seasonal residents during the summer.

The State of Connecticut has an agreement with East Lyme to allocate a portion of East Lyme's sewer capacity to State facilities. This reserved capacity serves the Governor's State Camp, Rocky Neck State Park and the Gates and York Correctional Facilities. According to the 2007 Report, the existing flow from the State facilities in 2004 was 249,000 GPD.

3. Existing Wastewater Flows from Old Lyme

In the spring, 2010, construction of a low pressure sewer system was completed to serve the Point O' Woods Beach Association. The wastewater from this area connects to the New London regional system in East Lyme. The homes in this area are in the process of connecting to the sewer system, therefore the flows are low at this point in time. For the time period that is used to evaluate current flows and loads (March '08 to August '10), Old Lyme did not contribute any wastewater. There is a flowmeter to monitor the flows and flow recording just began in March, 2011.

4. Existing Septage Quantities

Septage is generated from New London, East Lyme and Waterford residents. East Lyme and Waterford are permitted to discharge septage based on the intermunicipal agreements with New London. On average, the treatment plant receives 12,800 gallons of septage each day with a maximum monthly average of 19,300 gallons per day. Based on treatment plant records, peak day septage delivery over the time period was 34,200 gpd.

5. Existing Wastewater Flows from New London

To determine the existing wastewater flow from New London, the existing flows from the contributing towns described above were subtracted from the total flows at the treatment plant. Table III-3 presents a breakdown of existing flows for each municipality.

Table III-3: Current Flows New London WWTF

	Annual Average (GPD)	Maximum Monthly Average (GPD)	Hydraulic Peak (GPD)
New London	4,837,000	7,809,000	
Waterford	2,370,000	3,826,000	
East Lyme	1,100,000	1,206,000	
Septage	12,800	19,300	
Total Flows	8,320,000	12,860,000	25,500,000

a) Infiltration and Inflow:

To determine the amount of infiltration and inflow (I/I) in the New London system, water consumption data for New London was reviewed from the "New London Water

Supply Plan Update", revised March, 2008. The Plan provides an average consumption from the years 1998 to 2005 as shown in the following table:

Table III-4: City of New London Water Consumption

	Average Annual Consumption (MGD)
Residential	0.88
Commercial	1.04
Public Authority*	0.57
Industrial	0.21
Total Consumption	2.70

**Includes Institutional and Governmental*

For planning purposes, it is assumed all of the population in New London is served by public water and sanitary sewer. Assuming approximately 10% of the 2.70 MGD used is lost to consumption (Metcalf & Eddy, 4th Edition, p.155), the wastewater base flow that enters the sanitary sewer system is approximately 2.43 MGD.

As presented in Table III-3, the total existing wastewater flow from New London is 4,837,000 GPD. The current estimated base wastewater flow rate is 2,430,000 GPD (including residential, commercial, public authority and industrial flows). The average daily I/I at the treatment plant is therefore the average daily flow of 4,837,000 GPD less the base flow of 2,430,000 GPD or 2,407,000 GPD. These values are consistent with the values presented in the November, 2005 New London Facilities Planning Study.

b) Peak Flow:

Peak flows for the period from March, 2008 through August, 2010 were evaluated. On March 14, 2010, the flow at the plant reached 25.5 MGD after 5" of rain. Two weeks later on March 30, 2010, the flow reached a peak flow of 36.5 MGD and possibly higher after a rain event of over 4". This last storm flooded portions of the treatment plant

The peak flow rate of 36.5 MGD equates to a peaking factor at the facility of 4.4 - much higher than typical peaking factors at similar facilities and higher than recommended by TR-16. Since there are no other plant records of flows of this magnitude, this data point was discarded as a one-time event and was not use in calculations of plant capacity. Instead, using a peak flow rate of 25.5 MGD equates to a peaking factor of 3.06, a more reasonable value and in-line with TR-16 guidelines.

6. Treatment Plant Flows and Loads

Table III-6 presents a breakdown of the current flows and loads at the facility. Loads from septage, a highly variable waste stream, were calculated using TR-16 and EPA guidelines. For lack of individual member community data, loads for each town were

considered equivalent and distributed based on the amount of flow contributed. The data indicates BOD averaged 218 mg/l and the TSS at the plant averaged 170 mg/l. The historical data for nitrogen nutrient loadings at the treatment plant show that the average influent TKN load to the plant is 17 mg/l. This is in a typical range for most municipal wastewaters.

To obtain per capita loading rates, the current plant loadings were divided among the connected population. Table III-5 shows the per capita loading rates

Table III-5: Current Per Capita Loading Rates

Constituent	New London	TR-16
BOD, lbs/cap/d	0.31	0.17
TSS, lbs/cap/d	0.21	0.20
Nitrogen, lbs/cap/d	0.034	0.04

The per capita BOD concentration is significantly higher than common guidelines while the others are within a reasonable correlation with guidelines. Therefore, the current loads will be carried forward as-is and future loading calculations will use the standard guidelines for per capita loading calculations.

Table III-6: Current Flows and Loads

		ANNUAL AVERAGE	MAXIMUM MONTHLY AVERAGE	HYDRAULIC PEAK
NEW LONDON: ALL FLOWS *				
	FLOW. gpd	4,837,000	7,809,000	
	BOD, lb/d	8,478	10,278	
	TSS, lb/d	5,927	7,513	
	TKN, lb/d	931	1,092	
WATERFORD: ALL FLOWS*				
	FLOW. gpd	2,370,000	3,826,000	
	BOD, lb/d	4,154	5,036	
	TSS, lb/d	2,904	3,681	
	TKN, lb/d	456	535	
EAST LYME: ALL FLOWS*				
	FLOW. gpd	1,100,000	1,206,000	
	BOD, lb/d	1,928	2,337	
	TSS, lb/d	1,348	1,709	
	TKN, lb/d	212	248	
SEPTAGE***				
	FLOW. gpd	12,800	19,300	
	BOD, lb/d	534	805	
	TSS, lb/d	1,601	2,414	
	TKN, lb/d	75	113	
TOTALS**				
	FLOW. gpd	8,320,000	12,860,000	25,500,000
	BOD, lb/d	15,094	18,457	
	TSS, lb/d	11,780	15,318	
	TKN, lb/d	1,673	1,988	

** All flows include Infiltration and Inflow*
*** For the time period evaluated (March '08 to August '10) Old Lyme had not yet connected to the system*
**** Using 5,000/15,000/700 mg/l as a basis*

B. Future Flows and Loads

This section quantifies future wastewater flows within the tributary area based upon a 20-year planning period ending in the year 2032.

The following section describes the methodology for future flow development and flows were established for the planning period for New London. Flows from Waterford, East Lyme and Old Lyme were provided by the individual towns' consultants or approved plans and are also discussed below.

1. Future Wastewater Flows from New London

To evaluate future flows for the planning period, the available land for residential, commercial and industrial development, population projections and growth must all be established as accurately as possible. The City zoning map, the 2007 Plan of Development, GIS mapping and the sanitary sewer layout were reviewed. The City Planner and the Director of Economic Development provided valuable input on potential future development throughout the City. New London is also the home of the Coast Guard Academy, Mitchell College, Connecticut College, Lawrence and Memorial Hospital and the Williams School. These 'Public Authority' groups were contacted as well. This information provides the basis for future development and population growth which are then used to establish the future waste flows and loads to the treatment plant.

According to the 2007 Plan of Development, over 880 acres is dedicated to residential land use. Commercial land use comprises 251 acres of the City's land area. Private and public institutional land occupies a total of 570 acres. However, only a small amount of the city's land area is vacant and many of these sites are too small to be developed or have serious development limitations such as wetland soils or difficult topography.

a) Population and Water Consumption Data:

To determine future residential, commercial and municipal flows, water consumption data and population data was used. The New London Water Supply Plan Update revised on March 7, 2008 provides historical water consumption in gallons per capita per day (GPCD) for each category presented in Table III-7.

Table III-7: Historical Water Consumption

Population Served (GPCD)	1990	2000	2002	2004	Estimate Used
Residential	33.9	34.1	33.2	34.4	34
Commercial	37.8	39.9	40.9	41.5	40
Public Authority	20.9	21.7	21.4	22.6	22
Combined Residential and Commercial flow rate					74

Population projections for New London are as follows:

Table III-8: Population Projections City of New London

Year	2007 Plan of Development*	CT State Data Center
1990 (actual)	28,540	28,540
2000 (actual)	25,671	25,671
2005	-	26,281
2009	-	-
2010	27,900	26,739
2014	-	-
2020	31,020	27,548
2030	-	28,425
	1% Increase	0.36% Increase

** Population projections taken from 1995 CT Office of Policy and Management estimates*

The data from the 2007 Plan of Development indicates an overall population increase of approximately 1% per year. The State Data Center data presents an overall population increase of approximately 0.36% per year. The OPM projections for the year 2032 need to be projected by extrapolating the trend line from 2000 to 2020, so a 1% increase in population beyond 2020 may be incorrect and unrealistic particularly with the little amount of developable land in New London. Therefore, a population increase of 0.36% was used. By interpolating this information and using a population increase of 0.36% per year, it is estimated that the future population at the end of the planning period in 2032 will be 28,799. This is an increase in population of 1,997 people from the 2012 population estimate of 26,802. Using this increase and the combined flow rate for residential and commercial flow of 74 GPCD from Table III-7, the future flow estimate for residential and commercial flows is approximately 148,000 GPD.

b) Potential Development:

As a check against different sources of population projections, another method to estimate future flows is to review where potential development may occur. The City Planner and the Director of Economic Development provided information on where potential development may occur during the planning period. There is very little vacant developable land available in town. This information is based on where there has been an interest by developers and what has been proposed. Wastewater flow may also increase due to redevelopment of existing properties to the conversion of apartments or condominium units in the City. According to U.S. Census 2000, the average number of people per household is 2.52. Using a combined residential and commercial flow rate of 74 GPCD and 2.52 people per household in New London, Table III-9 presents a summary of potential development that may occur during the planning period. Due to potential residential development, a flow increase of 181,000 GPD is estimated. This is higher than one source and lower than another so is likely reasonable compared with the previous flow estimate when calculated using only population projections.

To account for additional flow if any of these potential projects occur during the planning period, an additional flow of 45,000 GPD was added.

Based on this analysis, the potential future flow during the 20-year planning period is expected to be approximately 226,000 GPD.

c) Infiltration and Inflow:

The City is in the process of reducing infiltration and inflow (I/I) in its collection systems. The City began an aggressive Infiltration/Inflow rehabilitation program in the early 1990's and has continued with the program. Any I/I increases due to normal deterioration of piping shall be offset by the work of the I/I program, therefore I/I is not expected to increase nor decrease during the planning period.

Based on the evaluation above, the future flow from New London is estimated to be 226,000 GPD. Adding this to the existing current flow of 4,837,000 GPD, the future estimate for New London wastewater flow is 5,063,000 GPD.

2. Future Wastewater Flows from Waterford

Future flow estimates for Waterford were provided by the Town through their engineering consultant. The estimated future flow increase is expected to be 131,850 GPD with an increase in peak flow of 527,400 GPD. Therefore, when adding this increase to the current flow of 2.37 MGD, the estimated average future flow from Waterford is 2,502,000 GPD.

3. Future Wastewater Flows from East Lyme

Future flow estimates for East Lyme were provided by the Town in the "September, 2007 Wastewater Collection System Capacity Analysis Planning Report". According to the report, due to new connections to existing sewers and future sewer extension projects, wastewater flows are expected to continue to increase in East Lyme. For the 20-year planning period, the average future wastewater flow is estimated to be 3,050,000 GPD. This includes a reserved wastewater allocation for the State's facilities in East Lyme (Rocky Neck State Park, Governor's Camp and Gates/York Correctional Facilities).

4. Future Wastewater Flows from Old Lyme

The Point O' Woods Association in Old Lyme is currently the only area that is connected to the regional sanitary sewer system. According to the Director of Utilities in East Lyme, the anticipated future flow is 105,000 GPD for this area. According to the consultant for Point O' Woods, because it is a seasonal community, its maximum average daily flow (105,000 GPD) will only occur between June 15 and September 15. The rest of the time, flows will be much less. During winter months (November – April) flows are likely to be 10% to 15% of maximum flows. This number will go up over time as more and more properties convert to year round.

Table III-9: City of New London Potential Development During Planning Period

DESCRIPTION		NUMBER OF UNITS	FLOW (GPD) [2]	COMMENTS
RESIDENTIAL				
	Age 55+ condominiums	20	3,730	conceptual in southern New London - 15 to 20 units
	Georgetown Condos	31	5,781	31 units approved in 2004 - 05
	just north of Georgetown Condos	30	5,594	up to 30 possible but difficult terrain
	Vacant parcel	30	5,594	currently used for a much needed parking lot for marinas, development not likely
	south of Maine Road	0	-	parcel could be developed but likely a parking lot for the hospital
	Condominium units	50	9,324	owner: Sutip kanajar, 50 to 100 units, 50 likely
	Near Jefferson	12	2,238	12 units approved, near high school, Bates Woods
	Chester Street	6	1,119	6 units total approved
	Proposed development south of arboretum	180	33,566	tough property to develop, steep slopes, began plans in the '80s, 180 possible
	Bayonet Street (owner A. Becker)	40	7,459	approved - starting in a few weeks
	Uncas Avenue (north)	10	1,865	10 units possible
	Shaw's Landing	127	23,683	35 units currently built but not all sold, 92 more potential
	Bank Street	52	9,697	9 stories of condos, 52 units approved, just waiting to sell condos
	Fort Trumbull	104	19,394	80 new units definite and 24 units in an extended stay suite, NLD Corporation, MDC Municipal Development Plan
	Town Center revitalization	200	37,296	Costs are prohibitive to renovate town center into residential units. A study at the Town Center - if all was residentially built-out = 400 units but not likely. 200 units used
TOTAL POTENTIAL RESIDENTIAL DEVELOPMENT		892	166,000	
COMMERCIAL				
	Near Fort Trumbull (near Howard St)	0		brownfield grant to test soil, designated as commercial, possible office, assume commerical flow will increase with residential flow
	Tract near Shaw's Cove		-	2.5 acres could have potential development
	Near Route I-95	0	-	Vacant land but not much potential for development, assume if developed it would be commercial and that will be included in the per capita water use rate
INDUSTRIAL				
	Pfizer property [3]	1000	15,000	Electric Boat purchased property. In 3 years there is expected to be 1,000 additional people
	Coca cola	no change		moving to Waterford, not used for bottling, no significant increase in water expected
	TOTAL ADDITIONAL FLOW	181,000		

[1] Based on discussions with the City Planner and Director of Economic Development

[2] 74 GPCD includes residential and commercial, based on US Census 2000, household density for New London = 2.52

[3] Assume office staff = 15 GPCD based on M&E Wastewater Engineering Guidelines

The Coast Guard Academy, Mitchell College, Connecticut College, Lawrence and Memorial Hospital and the Williams School are also located in New London. These 'Public Authority' groups were contacted as well. The Williams School does not expect significant changes in enrollment or staff over the planning period. The 25 year Master Plan for the Coast Guard Academy discusses a possible expansion of approximately 9 acres. However, the overall population of the academy is not expected to change much as many of these functions currently take place within existing buildings on campus. In addition, many of the new construction projects will be replacement buildings for the existing aged facilities with low water use equipment. Connecticut College may construct a new science center. The Mitchell College may construct a new dormitory.

At the present time, three other beach associations in Old Lyme have contacted the DEP and two of them have already procured consultants for the purpose of wastewater facilities planning. The studies will evaluate existing environmental conditions, develop alternatives to address deficiencies and recommend solutions to the existing wastewater disposal problems. It is possible the recommended solutions may include connecting to the sanitary sewer system tributary to the New London wastewater treatment facility. For planning purposes, the DEP estimates there are approximately 650 residences in the three communities (Old Colony Beach, Old Lyme shores and Miami Beach). According to the U.S. Census 2000 data, the average household size in Old Lyme is 2.50. To estimate future wastewater flows from the three additional beach associations, a wastewater flow rate of 70 GPCD was used in accordance with TR-16 guidelines.

The anticipated future flows from Old Lyme are as follows:

Table III-10: Future Additional Flows Town of Old Lyme

Location	Average Future Flows (GPD)
Point O' Woods	105,000
Old Colony Beach, Old Lyme Shores & Miami Beach	114,000
Total Old Lyme	**219,000**

5. Total System Flows

A summary of the total future flows and loads for the 20-year planning period is presented in Table III-11.

Table III-11: Future Wastewater Flow Estimate

Location	Average Future Flows (GPD)
New London	5,063,000
Waterford	2,502,000
East Lyme	3,050,000
Old Lyme	219,000
Septage	13,000
TOTAL FUTURE FLOW	**10,847,000**

The current peak flow shown on Table 5 is 25.5 MGD. The increase in flows for all of the towns were added to this peak flow for a future peak flow estimate of 28 MGD since the assumption is that I/I will not increase nor decrease in the planning period.

6. Projected Wastewater Loads

As stated previously, typical pollutant concentrations from the TR-16 "Guides for the Design of Wastewater Treatment Works" will be used for projecting future plant loadings in lieu of values determine from review of plant records. This is because the current values for per capita BOD (0.31 vs 0.22 lb/cap/d) would skew the loading

projections to a much higher value than would otherwise be predicted causing an over-projection of plant loading. These are presented in Table III-12.

Table III-12: Per Capita Pollutant Loadings for Future Flow Estimates

Parameter	Average Loading lb/cap/d	Monthly Max. Loading lb/cap/d	Ratio
BOD5 (Residential)	0.220	0.253	1.15
BOD5 (Industrial)	0.170	0.196	1.15
Total Suspended Solids (TSS)	0.250	0.325	1.30
Total Kjeldahl Nitrogen (TKN)	0.040	0.052	1.30

7. Future Projected Flows and Loads

Table III-13 presents a summary of the future flows and loads at the New London wastewater treatment facility.

Table III-13: Future Flows and Loads

			ANNUAL AVERAGE	MAXIMUM MONTHLY AVERAGE	HYDRAULIC PEAK
NEW LONDON: RESIDENTIAL, COMMERCIAL, PUBLIC*					
	FLOW. gpd		5,063,000	8,035,000	
	BOD, lb/d		8,972	10,847	
	TSS, lb/d		6,489	8,244	
	TKN, lb/d		1,021	1,209	
WATERFORD: ALL FLOWS*					
	FLOW. gpd		2,502,000	3,958,000	
	BOD, lb/d		4476	5407	
	TSS, lb/d		3270	4157	
	TKN, lb/d		515	611	
EAST LYME: ALL FLOWS*					
	FLOW. gpd		3,050,000	3,156,000	
	BOD, lb/d		6603	7714	
	TSS, lb/d		6660	8615	
	TKN, lb/d		1062	1353	
OLD LYME: ALL FLOWS*					
	FLOW. gpd		219,000	285,000	
	BOD, lb/d		688	792	
	TSS, lb/d		782	1017	
	TKN, lb/d		125	163	
SEPTAGE					
	FLOW. gpd		13,000	20,000	
	BOD, lb/d	(5,000 mg/l)	542	834	
	TSS, lb/d	(15,000 mg/l)	1,626	2,502	
	TKN, lb/d	(700 mg/l)	76	117	
TOTALS					
	FLOW. gpd		10,847,000	15,454,000	28,000,000
	BOD, lb/d		21,282	25,593	
	TSS, lb/d		18,828	24,535	
	TKN, lb/d		2,798	3,453	

** Includes Infiltration and Inflow*

The laboratory test procedures used to complete the wastewater characterization are summarized in the table below. Samples were analyzed by Rhode Island Analytical, Inc. After the samples were analyzed, the data were reviewed and erroneous and unrealistic data points were discarded based on typical ranges or ratios. The final sets of data are included in Appendix A.

Table IV-3: Laboratory Test Procedures

Constituent	Test Procedure
TSS	Standard Method – 2540D
VSS	Standard Method – 2540E
COD	Standard Method – 5220D
sCOD (0.45 micron)	Standard Method – 5220D
dCOD (GF)	Standard Method – 5220D
ffCOD	Standard Method – 5220D
$TBOD_5$	EPA – 405.1
$CBOD_5$	Standard Method – 5210
dCBOD (GF)	Standard Method – 5210
TKN	EPA – 351.2
sTKN (0.45 micron)	EPA – 351.2
dTKN (GF)	EPA – 351.2
NH_3-N	Standard Method 4500
NO_3-N	Standard Method 4500
NO_2-N	Standard Method 4500
TP	Standard Method 4500
dTP (GF)	Standard Method 4500
PO_4-P	EPA – 365.2
Alkalinity	Standard Method – 2320B

Table IV-2: Daily Grab Sample Constituents and Locations

Constituent	Anoxic	Aerobic	RAS	WAS	Primary Sludge	Belt Press Supernatant	Septage[1]
Flow			✓	✓	✓	✓	✓
TSS		✓	✓	✓	✓	✓	✓
VSS		✓	✓		✓	✓	✓
COD		✓				✓	✓
sCOD (0.45 micron)							
dCOD (GF)						✓	✓
ffCOD							
CBOD						✓	✓
dCBOD (GF)	✓					✓	✓
TKN						✓	
NH_3-N	✓						
NO_3-N	✓						✓
NO_2-N			✓		✓		
TP						✓	✓
PO_4-P						✓	✓
Alkalinity							
pH		✓					
Dissolved Oxygen		✓					
Temperature							

Notes:
1. Septage samples taken 3 of the 6 days of sampling.

Influent samples were collected at the existing automatic sampler downstream of screening and grit removal. Primary effluent samples were collected at the existing automatic sampler. Secondary effluent samples were collected at the existing automatic sampler.

Spot samples were collected at the following locations:

- Mixed Liquor was collected from each of the aerobic zones in each of the basins and mixed;
- RAS samples were collected from a sample tap on the discharge side of a RAS pump;
- WAS samples were collected from a sample tap on the discharge side of a WAS pump;
- Primary sludge samples were collected from a sample tap on the discharge side of a sludge pump;
- RAS, WAS, and primary sludge flow was measured via flow meters;
- Belt press supernatant samples were taken with grab samples on the discharge end of the belt press;
- Septage samples were the combination of several grab samples from several different loads of septage throughout the course of a day.

IV. Wastewater Characterization

A wastewater characterization program was undertaken as the first step of the process modeling required for this project. The characterization program comprised 6 days' worth of sampling spread over 2 weeks and sampled wastewater through the treatment process. Samples were taken of raw influent, primary effluent, final effluent, septage, primary sludge, belt press supernatant, mixed liquor, RAS, and WAS. Composite and grab samples were taken between November 30, 2010 and December 10, 2010.

Table IV-1 and Table IV-2 below summarize the sampling locations and which constituents were measured at each location.

Table IV-1: Composite Sample Constituents and Locations

Constituent	Influent Wastewater	Primary Effluent	Secondary Effluent
Flow	✓		
TSS	✓	✓	✓
VSS	✓	✓	
COD	✓	✓	✓
sCOD (0.45 micron)			✓[1]
dCOD (GF)	✓	✓[1]	
ffCOD	✓	✓[1]	✓
$TBOD_5$	✓	✓	✓
$CBOD_5$	✓	✓	✓
dCBOD (GF)	✓		
TKN	✓	✓	✓
sTKN (0.45 micron)			✓[1]
dTKN (GF)	✓	✓[1]	
NH_3-N	✓	✓[1]	✓
NO_3-N	✓[1]		✓
NO_2-N			✓[1]
TP	✓	✓	✓
dTP (GF)	✓	✓	
PO_4-P	✓		
Alkalinity	✓	✓	✓
pH	✓	✓	✓
Temperature	✓		

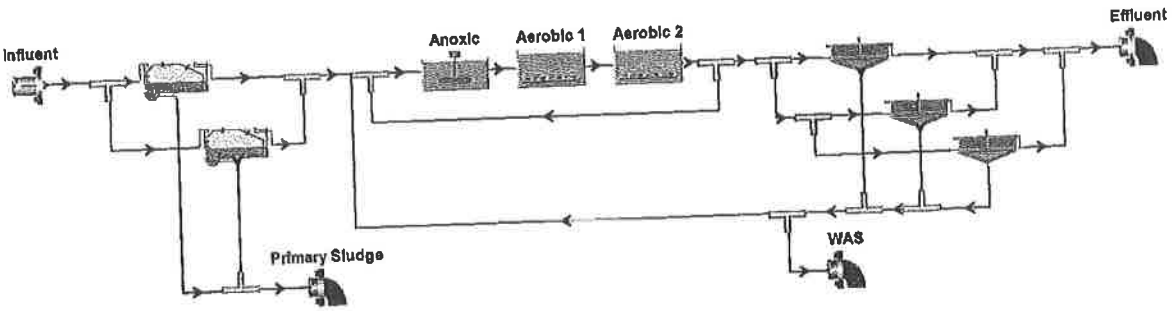
Notes:

1. Composite samples taken 3 of the 6 days of sampling.
2. 0.45 micron means the analyses were conducted on the sample filtrate using a 0.45 micron filter
3. GF means the analyses were conducted on the sample filtrate using a glass filter
4. ffCOD samples were prepared per the guidelines in WEF's "Methods for Wastewater Characterization In Activated Sludge Modeling, page 7-10".

V. Model Calibration

After collecting the wastewater characterization data, the process model was constructed within the Biowin® software using the dimensions of the process units and the characterization data. Figure V-1 displays the process model schematic.

Figure V-1: Process Model Schematic



The tables below present the results of the model calibration. As seen, the model is calibrated to within industry standards for influent, primary effluent, and effluent concentrations. Some interesting observations include:

- The measured value of nitrite in the anoxic zone is higher than the predicted value. This situation suggests that the plant may be operating at an SRT where incomplete nitrification takes place and leaves nitrite as an intermediate product or only partly denitrifying. Since nitrate in the effluent matched the model, the intermediate speciation is not a concern;
- The model under-predicted pH. It is likely that reason for this is because the characterization measurements were measured at the laboratory after transport rather than when taken. Because of the low alkalinity, there is not much buffering and additional carbon dioxide stripping from transport raises the pH further. No additional investigation into this aspect of the model was conducted because the effluent results closely match measured values.
- Since there is no flow measurement capability for internal recycle flows, the internal recycle rate was obtained through trial and error. The calibrated internal recycle rate was 90% of plant influent flow.

Table V-1: Influent Wastewater Calibration

	Measured Values (mg/L)	Calculated Values (mg/L)
TSS	300.00	289.61
VSS	264.00	269.56
COD	592.00	592
$CBOD_5$	254.76	278.98
TKN	35.20	35.20
NH_3-N	20.20	20.20
Total P	6.74	6.74

	Measured Values (mg/L)	Calculated Values (mg/L)
PO_4-P	3.16	3.16
Alkalinity	3.24	3.24
pH	7.07	7.04

Table V-2: Effluent Wastewater Calibration

	Measured Values (mg/L)	Calculated Values (mg/L)
TSS	4.17	4.65
COD	36.40	24.74
$CBOD_5$	<2	2.4
TKN	1.55	1.88
NH_3-N	0.35	0.30
NO_3-N	7.08	7.11
Alkalinity	0.98	1.37
pH	6.87	6.56

VI. Existing Process Capacity

A. Preliminary Treatment

The capacity of the preliminary treatment processes will be discussed in a subsequent section on hydraulics as its capacity is primarily driven by hydraulics.

B. Primary Treatment

The capacity of the primary treatment process will be discussed in a subsequent section on hydraulics as its capacity is primarily driven by hydraulics.

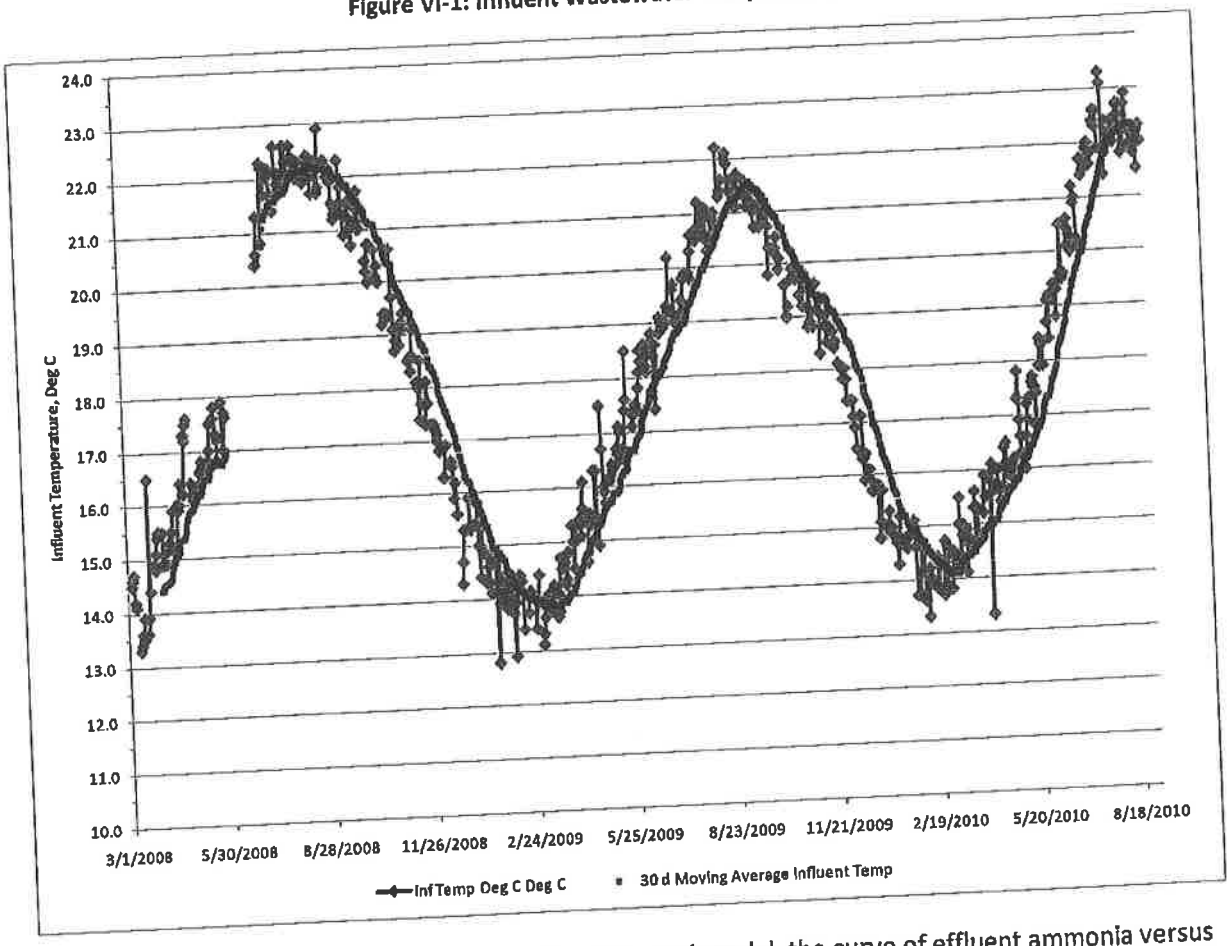
C. Secondary Treatment

1. SRT

The first step in evaluating the capacity of the secondary treatment process is to determine the design Solids Retention Time (SRT) based on model results. In a Modified Ludzak-Ettinger (MLE) process like that at the WWTF, the aerobic SRT is the limiting factor because nitrifying organisms grow slower than denitrifying organisms. Because of this fact, the first step in evaluating the design SRT is to determine the required aerobic SRT.

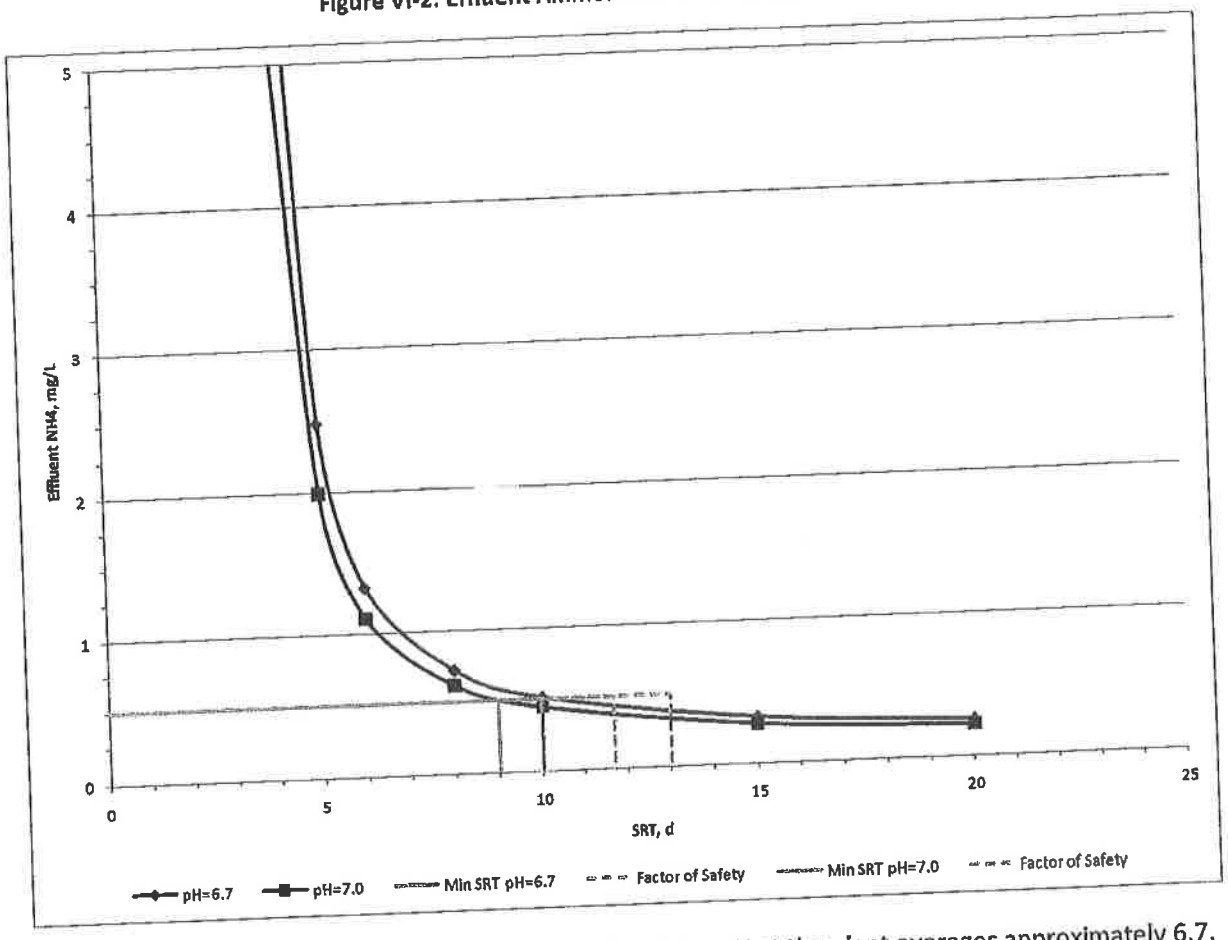
The required SRT is largely dependent on temperature, which is especially true for nitrifying organisms. Figure VI-1 below displays the historical wastewater temperature trends. A minimum design temperature of 13°C has been used for this evaluation based on an evaluation of plant influent temperature data. A maximum ammonia value of 0.5 mg/L was used in this evaluation as the basis for determination of complete nitrification. Note that in this graph that influent temperature for June 2008 was not available.

Figure VI-1: Influent Wastewater Temperature



Using this temperature and the calibrated model, the curve of effluent ammonia versus aerobic SRT shown in Figure VI-2 was generated. Using this curve, the minimum aerobic SRT needed for an effluent ammonia value of 0.5 mg/L at a temperature of 13°C and a pH of 6.7 is determined to be 10 days. This graph was then back-checked with hand calculations and confirmed. Adding a factor of safety of 1.3 to allow for variations in loadings and other process variables, the design aerobic SRT for this temperature, pH, and effluent ammonia would be 13 days.

Figure VI-2: Effluent Ammonia vs. Aerobic SRT Curve



During the evaluation, it was noted that the pH at the plant averages approximately 6.7, which is not uncommon in plants that nitrify. However, at a pH of 6.7, the nitrification rate, which is the governing factor in aeration tank capacity, is approximately 70% of maximum. The effect of the depressed pH is to lengthen the overall SRT needed for full nitrification. Figure VI-2 shows the effect on SRT if pH is increased to 7.0. The minimum aerobic SRT decreases about 1 day to 9 days. When the factor of safety is added, it decreases 1.3 days for a minimum aerobic SRT with safety factor of 11.7 days.

So as this evaluation continues, it is important to note that an increase in pH in the aeration tanks (by the addition of sodium hydroxide or some other basic chemical) will improve aeration tank capacity by reducing the necessary SRT for nitrification. This may become an important concept as the space for adding aeration tank capacity is non-existent without the purchase of additional land. During the last two construction projects, a new blower building and part of Secondary Clarifier 3 have been constructed in the location where future aeration tank space was reserved. Additional secondary treatment capacity will need to be realized by increasing the concentration of solids in the aeration tanks and by adding secondary clarifier capacity.

2. Secondary Treatment Capacity Evaluation

After developing the minimum aerobic SRT, it is possible to evaluate the current secondary treatment process. The objective of this evaluation is to find out if it can treat the future average flow of 10.85 MGD and future maximum month flow of 15.45 MGD.

Table VI-1 below lists the concentrations used in the model for this evaluation and are based on the information provided previously in Table III-13.

Table VI-1: Future Influent Wastewater Concentrations (from Table III-13)

	Average Month	Max Month
COD, mg/L	427	361
BOD, mg/L	235	199
TSS, mg/L	208	190
VSS, mg/L	182	167
ISS, mg/L	26	24
TKN, mg/L	31	27
TP, mg/L	4.8	4.1

To provide a baseline for the evaluation because there are a number of process variables that affect capacity, a maximum MLSS in the aeration tank of 4,000 mg/L was used as a likely upper limit. MLSS operating values above this are uncommon and stress other system components such as clarifiers, mixing and aeration systems. Additionally, they carry special concerns in process operation and require close attention by operators.

a) Aeration Basin

The WWTF has two aeration basins, each configured to operate as a Modified Ludzack-Ettinger (MLE) process with a pre-anoxic zone followed by an aerobic zone with an internal recycle to return a high volume of nitrate rich mixed liquor back to the anoxic zone. Effluent nitrogen from an MLE process can be as low as 5 to 8 mg/L given the right influent characteristics and SRT. The following conditions were used in evaluating the aeration basin:

- Maximum month loadings;
- No pH adjustment (pH of 6.7);
- Temperature of 13°C;
- Desired effluent ammonia concentration of 0.5 mg/L;
- MLSS concentration of 4,000 mg/L;
- Internal recycle rate of 100% of influent flow based on calibration;
- RAS rate of 73% of influent flow based on historical operating data;
- Dissolved Oxygen concentration of 2.0 mg/L in the aerobic zone.

Under this scenario, the aerobic SRT that maintains a mixed liquor concentration of 4,000 mg/L was found to be 9 days. On the surface, this result means that the aeration basins are undersized for the expected future flows and loads as 9 days is clearly less than the 13 day requirement determined earlier. However, the chart in Figure VI-2 developed by the model can be used to help evaluate the impact that operating an aerobic SRT of 9 days will have. Using the "pH=6.7" curve, an aerobic SRT of 9 days is expected to result in an effluent ammonia value of slightly more than 0.5 mg/L. When a factor of safety is added to the 9 days, the resulting SRT is just under 7 days, which corresponds to an effluent ammonia value of 1.0 mg/L. At 1.0 mg/L effluent ammonia, nitrification will still be taking place, but will be rapidly lost if the SRT is decreased much at all. Figure VI-3 shows this graphically. It can therefore be concluded that the future maximum month loadings shown in Table III-13 are the upper limit of treatment capacity in the aeration basins assuming that the influent wastewater characteristics do not change. This method of operation provides little margin of error, will require tight operational control, and is potentially more susceptible to wet weather washout.

Figure VI-3: Impact of an Aerobic SRT of 9 days at 13°C

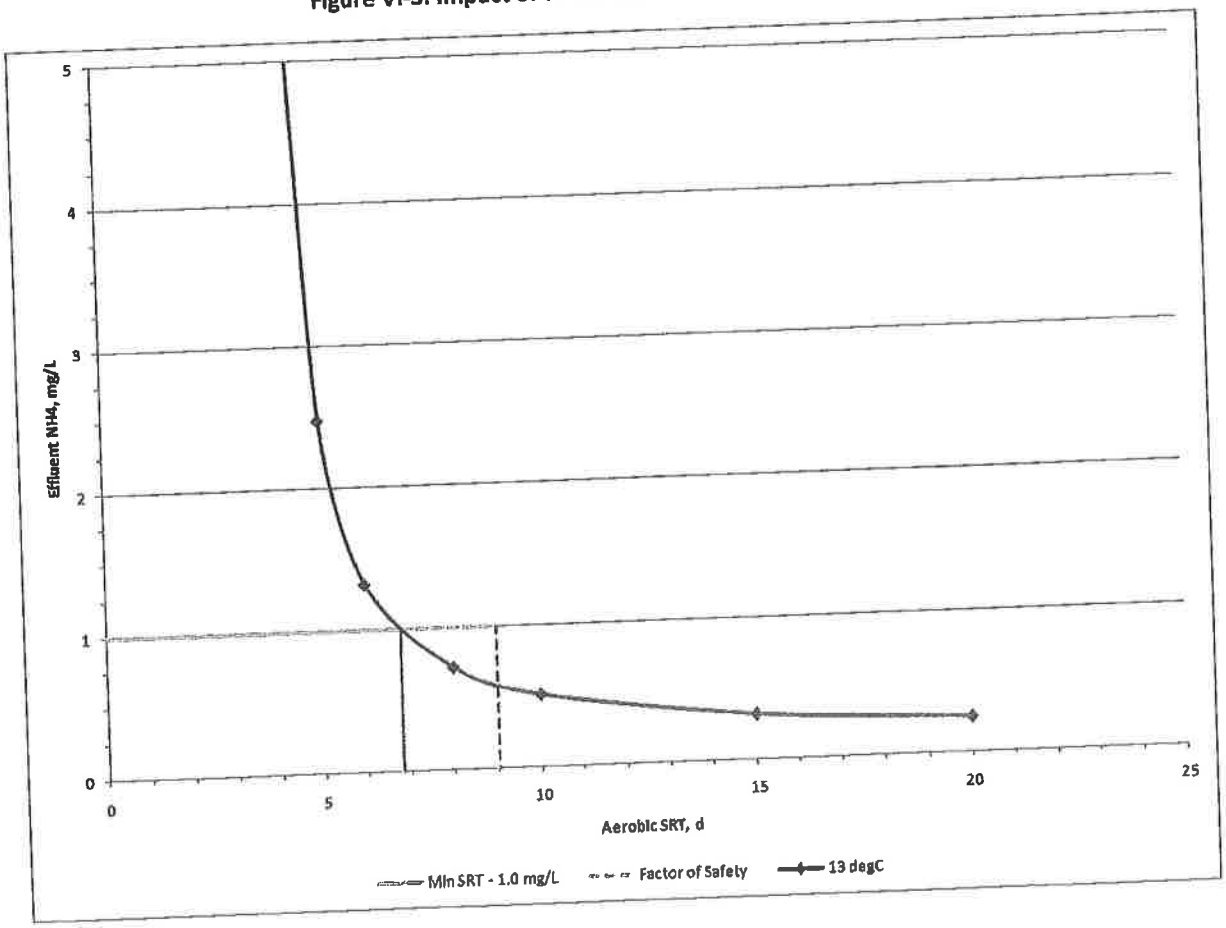


Figure VI-4: Solids Loading Rate at Failure (From TR-16)

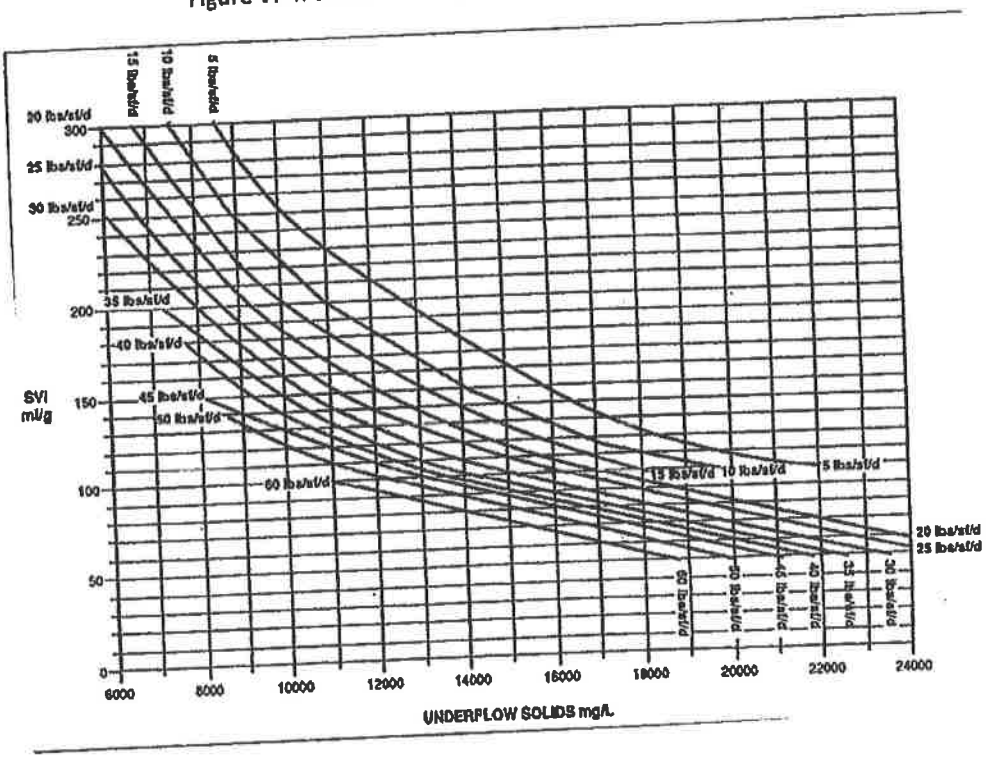
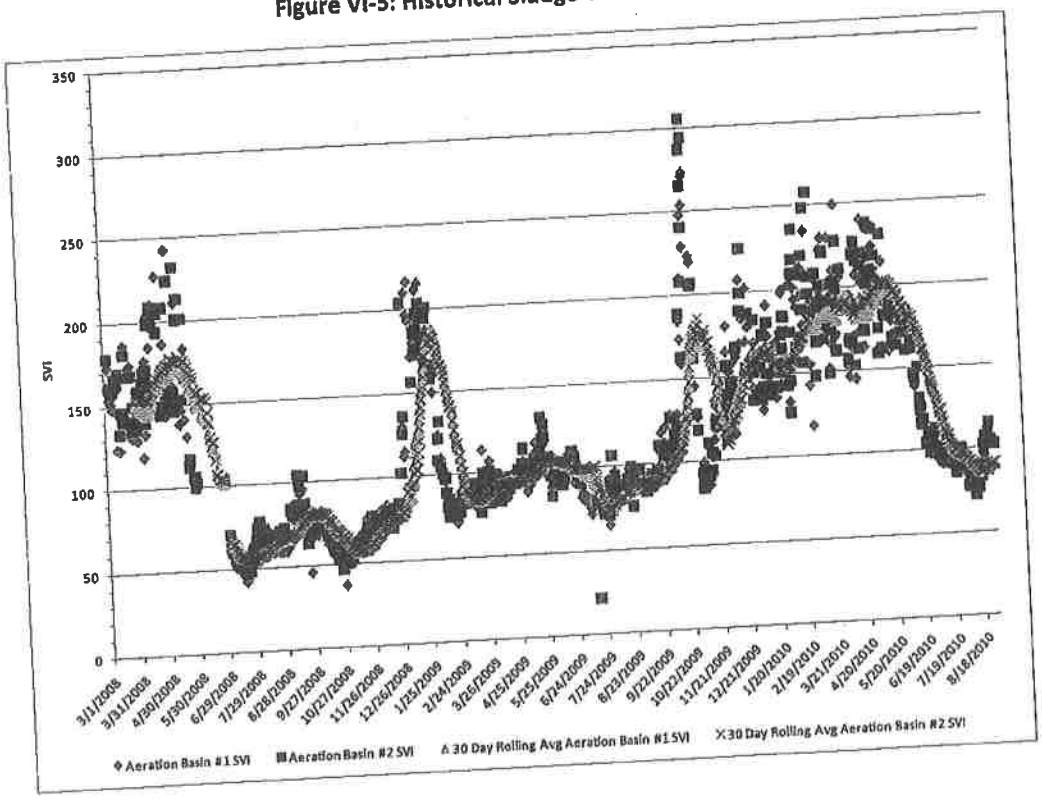


Figure VI-5: Historical Sludge Volume Index



b) Secondary Clarifier

The WWTF has three 100' diameter secondary clarifiers. TR-16 provides clear design guidelines for secondary clarifiers. Clarifiers should be designed based on solids loading rate (SLR), sludge settling characteristics (which were based on tests during the characterization phase), settled sludge concentration and return sludge rates rather than entirely on hydraulic overflow rates.

In the case of New London, maximum day flow was based on a review of existing data as well as the 2005 Facility Plan. Ignoring the large storm in late March 2010, historical data shows a number of days of approximately 17 MGD, with occasional excursions above this value. Similarly, the 2005 Facility Plan identified the maximum day flow as 17 MGD. The Facility Plan includes a ratio of maximum day to average month flow of 1.7. This ratio is based on published curves in TR-16 and was continued for this evaluation. Using this ratio, the projected future maximum day flow is 18.44 MGD.

From TR-16 guidelines, the solids loading rate (SLR) at failure can be determined using the chart in Figure VI-4. In this instance, a RAS concentration of 9,300 mg/L was assumed based on the process model results. A range of 150 to 200 mL/g was used for sludge volume index (SVI) based on historical data shown in Figure VI-5. Using these assumptions a range of SLR at failure from 20 to 40 lb/d/ft^2 is determined. A safety factor of 1.3 to 1.5 is applied to this number in accordance with the guidelines, resulting in a range of maximum SLR of 13.3 to 30.8. lb/d/ft^2. This is the range of maximum acceptable SLR during maximum day flow.

The maximum MLSS concentration for a range of influent flows can then be determined using the equation for solids loading rate:

$$Solids\ Loading\ Rate = \frac{(Q + Q_R) \times X}{A}$$

Q = Max Day Flow

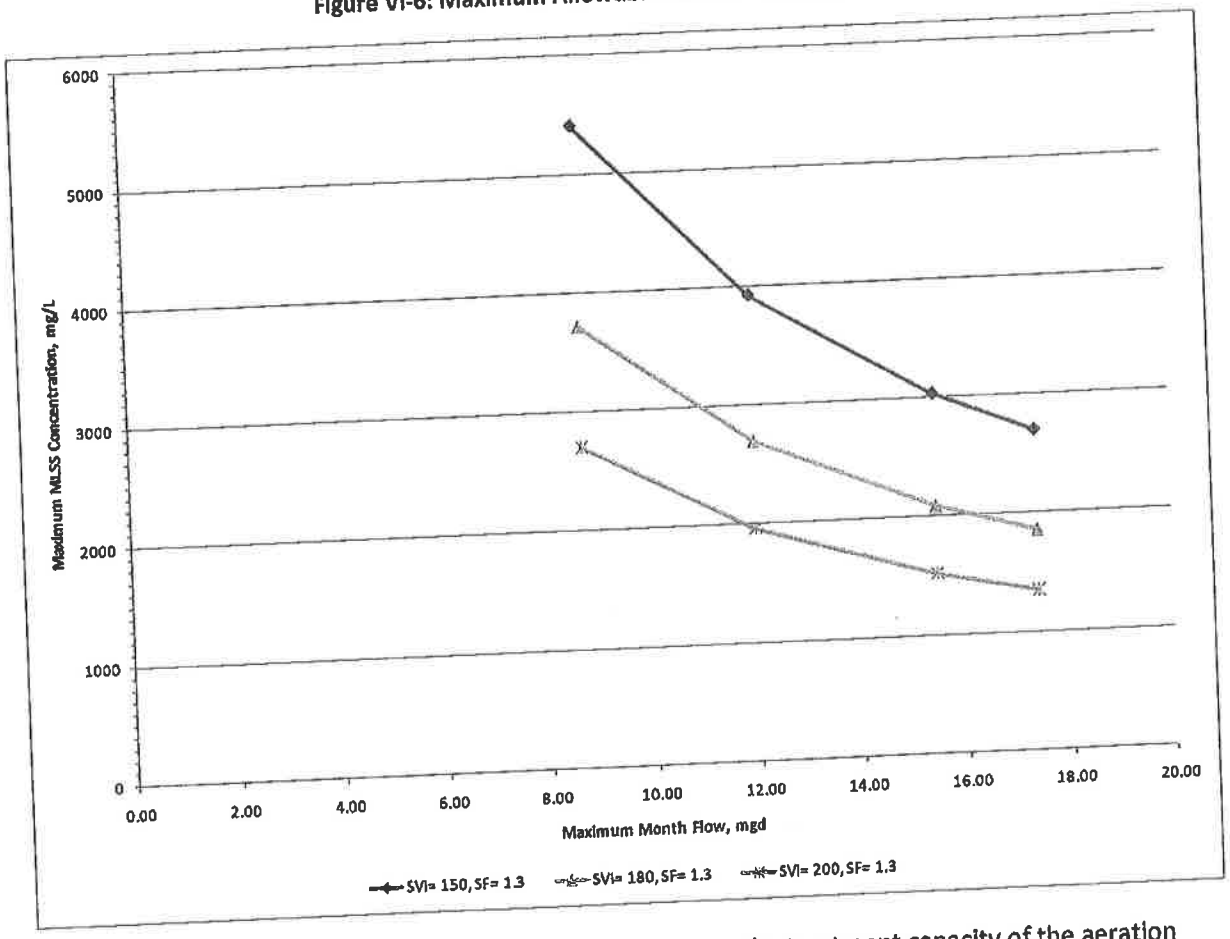
Q_R = RAS Flow

X = MLSS Concentration

A = Clarifier Surface Area

Using all three secondary clarifiers and assuming RAS flow during max day is reduced to 55% of influent, which is in line with historical operating parameters, Figure VI-6 was developed to show the maximum allowable MLSS concentrations over a range of flows.

Figure VI-6: Maximum Allowable MLSS Concentration



As discussed in the prior section, to maximize the treatment capacity of the aeration basins means maximizing the mixed liquor concentration. Based on this analysis, an MLSS concentration of 4,000 mg/L translates to a maximum month flow capacity of approximately 11.7 MGD, assuming an SVI of 150 mL/g (an expected value based on TR-16 guidelines that a designer would use if there were no other data on SVI available) and

three clarifiers in operation. However, Figure VI-5 shows that SVI's above 150 occur frequently, and at a MLSS concentration of 4,000 mg/l, the flow capacity would only be 6.5 MGD. Both 11.7 MGD and 6.5 MGD are below the predicted maximum month flow of 15.45 MGD and the conclusion is that as flows increase at this plant, additional clarifier capacity will be needed. Because SVI and MLSS have such a large effect on plant capacity, it is critical that plant operations keep SVI in check to maintain plant capacity.

Obviously the WWTF has treated significantly more flow than this with no issues. How much flow can be treated is dependent on the operating conditions at the time. Usually, the facility does not operate with the MLSS concentration as high as 4,000 mg/L – which AECOM believes will be required for future loadings. A significantly lower MLSS concentration such as is the current operation will limit the treatment capacity of the aeration basin, but it will increase the capacity of the secondary clarifier by reducing the solids loading rate.

In the course of this evaluation, it was noted that the 2005 Facility Plan also used a design MLSS of 4,000 mg/L and RAS concentration of 9,000 mg/L. The report, however, did not identify capacity as an issue with the secondary clarifiers. The calculations in the report appear to have mistakenly calculated SLR by leaving out Q_R from the equation presented above. As a result, the Facility Plan concluded that the secondary clarifiers were sufficient for a maximum day flow of 17 mgd at an MLSS of 4,000 mg/L. At this flow rate and MLSS concentration, the solids loading criteria in TR-16 are exceeded.

Subsequent sections will discuss modifications to the existing treatment process to be able to handle the predicted future flows.

D. Disinfection

Disinfection is currently achieved through chlorination in the chlorine contact tanks and outfall pipe. Effluent is chlorinated prior to two 64,000 gallon capacity contact tanks and then flows through a lengthy outfall pipe prior to discharge into the river. Current TR-16 guidelines call for 30 minutes of detention time at the peak hydraulic flow. Connecticut DEEP has accepted this 30 minute standard although there is some contact time latitude given to those facilities that were constructed prior to these regulations taking effect and when the allowable contact time was 15 minutes. The contact time provided by the current contact tanks is shown in the table below. Additional contact time is provided in the outfall pipe, although the actual amount of time provided is not known.

Table VII-1: Hydraulic Profile Assumptions – Future Flows

	Annual Average Flow	Maximum Month Flow	Peak Hydraulic Flow
Influent Flow, mgd	10.85	15.45	28
Internal Recycle Flow, mgd	15	27	27
RAS Flow, mgd	7.9	11.3	11.3

Based on these flows, the hydraulic evaluation identified the following:

B. Preliminary Treatment

The hydraulic analysis found that there are no flow capacity issues in the preliminary treatment processes (grit removal, screening, flow measurement) at the peak hydraulic flow of 28 MGD.

C. Primary Treatment

The WWTF has three primary clarifiers. Two are 78' diameter and one is 95' diameter. All have a depth of 9'. Capacity in primary clarifiers is determined through the use of hydraulic loading rate and weir loading rate as follows:

Table VII-2: Typical Primary Clarifier Loading Rates

Flow	Surface loading rate	Weir loading rate
Average day	1200 gpd/sf	10,000 – 20,000 gpd/ft
Peak hourly	3000	40,000

The largest and newest of the primary clarifiers at the treatment plant, Clarifier #3, is not often used because of operational problems related mostly to scum removal. Unlike the other two primary clarifiers which have center feed systems, Clarifier #3 is a peripheral feed style where influent flow is introduced at the perimeter of the clarifier instead of in the center column. Although classified as a peripheral feed clarifier, Clarifier #3 lacks the typical peripheral take-off and peripheral scum removal system, and flow instead exits the clarifier via a suspended effluent weir structure in the center of the clarifier. This center outlet creates operational problems as there can be no effective method of scum removal. Scum therefore collects on the surface, in the weir v-notches and either plugs them, or scum overflows to downstream processes. When loaded under higher flows, the clarifier performs well enough and can be used acceptably on a short-term basis but would not be recommended for a long term operation as scum will negatively affect the performance of downstream processes. Figure VII-1 shows the effluent weirs of the clarifier suspended in the middle of the clarifier.

VII. Hydraulic Evaluation

This section presents the hydraulic evaluation of the facility, which is comprised of two main elements: hydraulic profiles/capacities and CFD modeling. Hydraulic profiles were created to evaluate the plant at the future flow rates to determine if there were any under-capacity processes or choke points in the processes or piping. Computational Fluid Dynamics (CFD) modeling was performed to evaluate in detail the identified or potential choke points or distribution problems in the plant, determine a solution, and to also evaluate secondary clarifier performance.

A. Hydraulic Profile

To understand plant hydraulic capacity and create an accurate hydraulic profile for the New London WWTP, a field survey was performed on November 16, 2010 between 9am and 12:30 pm to collect water surface elevation data. The plant flow at this time ranged from 6 to 8 MGD and average 6.8 mgd. The following processes and operational activity was noted:

- Returned Activated Sludge (RAS) of 5.4 MGD;
- Internal Recirculation (IR) of 14.3 MGD (10,000 gpm) per aeration tank was used and based on an evaluation of the pump curve and likely hydraulic losses;
- Both influent screens were in operation. There is no flow measurement device for the IR pumps;
- Primary Clarifier No.1 and 2 were in operation;
- Both aeration tanks were in operation;
- Secondary Clarifiers No.1 and 3 were in operation;
- Both chlorine tanks were in operation.

Using the survey information a hydraulic profile was created, calculating the flows from the downstream end of the plant at the weir at Chlorine Contact Basin, to the upstream end (Screens). The hydraulic grade was calibrated and loss coefficients modified until calculations matched the survey information. Appendix B includes the plant hydraulic profile for the above mentioned flow of 6.8 mgd.

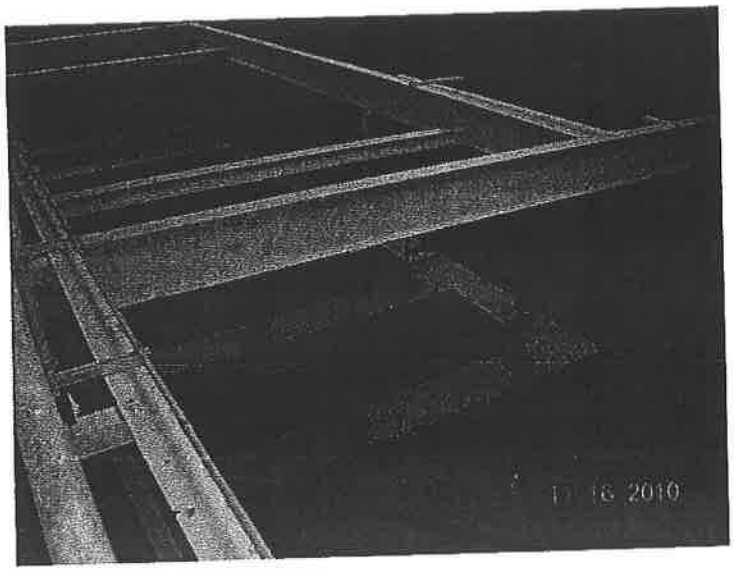
Hydraulic profiles were created for the future average day, future maximum month, and future peak hydraulic flows as presented in earlier sections. The assumptions used in each of these scenarios are listed below in Table VII-1. The assumptions for RAS rate, Internal Recycle flow and the amount of process tankage online duplicate the assumptions made in the process evaluation section and take into account the proposed modifications to the internal recycle flow rate discussed in subsequent sections. Note that the peak hydraulic flow is assumed to occur on the maximum day flow. It is assumed that maximum day flow is equal to 18.44 MGD as discussed earlier.

Table VI-2: Disinfection Capacity

	Flow in MGD	Contact Time (minutes)
Current ADF	8.3	21
Current Max Month	12.86	14.3
Current Peak	25.5	7.2
Future ADF	10.85	17
Future Max Month	15.45	11.9
Future Peak	28	6.6

The existing chlorine contact tanks have historically provided adequate disinfection and have consistently met permit levels for disinfection.

Figure VII-1: Primary Clarifier No.3



The following table summarizes the capacity of the primary clarifiers at the WWTF, with and without the third clarifier. For this evaluation, it is assumed that the two center feed clarifiers are commonly in operation, and that the third clarifier will be brought on-line when the flows increase to the noted values which will reduce the concerns described above.

Table VII-3: Existing Primary Clarifier Capacity

Flow Condition	Number of Clarifiers	Weir Loading Rate, gpd/LF	Hydraulic Loading Rate, gpd/ft^2	Limiting Capacity, MGD
Average	2	20,000	1,200	9.8
Peak Hour		40,000	3,000	19.6
Average	3	20,000	1,200	15.8
Peak Hour		40,000	3,000	31.6

In each case, the weir loading rate is the limiting factor and determines the capacity of the primary clarifiers. The average flow capacity with three clarifiers online is more than the predicted future average flow and maximum month flow, so primary clarifier capacity is sufficient. Based on this evaluation, using two primary clarifiers under most current flow circumstances will be adequate. AECOM suggests bringing the third clarifier on line when average daily flows reach 9.8 mgd or peak flows reach 19.6 mgd or when needed because one of the other primary clarifiers is out of service.

The hydraulic loading rates that correspond to the maximum flow capacities are approximately 1,000 gpd/ft^2 at average flow and 2,000 gpd/ft^2 at peak hour flow. These values are very reasonable and well within normal design standards.

D. Aeration Tank

The following were noted and are suggested to be addressed in the near future to allow for process treatment or hydraulic capacity:

1. Wall Openings between the Anoxic and Aerobic Zones

Currently there are two 2' x 2' and one 1' x 1' opening in the wall separating each anoxic zone from the aerobic zone. At high flows, this creates a head difference of over 12 inches. The large head loss reduces the hydraulic capacity of the facility as well as the ability to remove nitrogen because the internal recycle pumps are not capable of pumping large enough volumes of water at these high head losses.

Therefore, it is recommended that the current window openings either be enlarged to 4' x 4' to bring the head loss down to less than three inches or install new openings that will provide the additional cross-sectional area at the wetted surface.

2. Aeration Basin Effluent Weirs

The two aeration tanks have fixed weirs located at the end of the treatment process. The survey performed showed that the weir elevations in the two tanks were different by 0.05 feet. This can cause upward of a 40/60 split in aeration tank flow split hydraulics. The weirs on these tanks should be set at an equal elevation of 17.50 to equalize flow to both tanks.

E. CFD Modeling

CFD modeling was used to examine several areas of the plant more closely. The FLOW-3D Computational Fluid Dynamics (CFD) model was used for this evaluation. FLOW-3D is an all-purpose solver that solves the Reynolds-averaged Navier-Stokes equations while providing turbulence closure. This type of CFD modeling allows for the 3D evaluation of the flow distribution through complex structures such as distribution boxes. It also allows for the evaluation of density currents and settling effectiveness inside secondary clarifiers.

The sections below present the areas of the plant which were evaluated using CFD modeling.

1. Secondary Clarifier Flow Distribution

The aeration basin effluent channel is common to both aeration basins and has an unusual flow distribution to the three secondary clarifiers, mostly because the third clarifier was added after the first two, there was a desire for process flexibility, and it was installed in a location not previously identified as space for an additional clarifier.

Currently, the aeration tank effluent channel has a single 42" pipe which distributes flow to the distribution box for Secondary Clarifier 1 and 2. There is also a 36" pipe at the northern end of the aeration tank channel which flows directly to Secondary Clarifier 3 via a distribution box attached to the northern aeration tank. There is also a 36" pipe that flows from the southern aeration tank effluent box to the distribution box for

Clarifier #3 so as to be able to use Clarifier #3 when the northern aeration tank is not in operation. Figure VII-2 below shows a schematic of this arrangement. As the geometry of this channel and the size and location of pipes which drain into the clarifiers are unique and there are a number of slide gates that can be used to direct flow, a CFD model was created to determine whether the flow split to the clarifiers was equal and if not, what could be done to provide flow equality. Since the evaluation in the previous section shows that both aeration tanks are needed under most operating conditions, the boundary conditions assumes that both aeration tanks are in operation.

First, the aeration tank effluent channel was created in AutoCAD and imported into the CFD model. The evaluation examined future annual average flow (10.85 MGD plus 7.9 MGD RAS) and future maximum month flow (15.45 MGD plus 11.3 MGD RAS). In situations where either all three clarifiers of a combination of Clarifier 3 and Clarifier 1 or 2 are online, the CFD model results show that 56% of the flow will go through the 42" pipe to Clarifiers 1 and 2 and 44% of the flow will go through the 36" pipe to Clarifier 3. Ideally, the flow split would be 50/50 when two clarifiers are online and 33% to each of the three clarifiers when all three clarifiers are online.

Because flow is sometimes directed only to Secondary Clarifiers 1 and/or 2, the Secondary Clarifier Distribution Box was also modeled to better understand the flow distribution between Secondary Clarifier 1 and 2. The distribution box was created using AutoCAD and imported into the CFD model. The evaluation used an effluent flow of 6.8 MGD and a RAS flow of 5.4 MGD as measured during the field survey.

This evaluation showed that when Clarifiers 1 and 2 are used, 53% of the flow will pass to Secondary Clarifier 1 and 47% will pass to Secondary Clarifier 2. This is considered a relatively equal flow split. The results are shown graphically in Figure VII-3, Figure VII-4m and Figure VII-5 and display the velocity profile and flow direction in the distribution box.

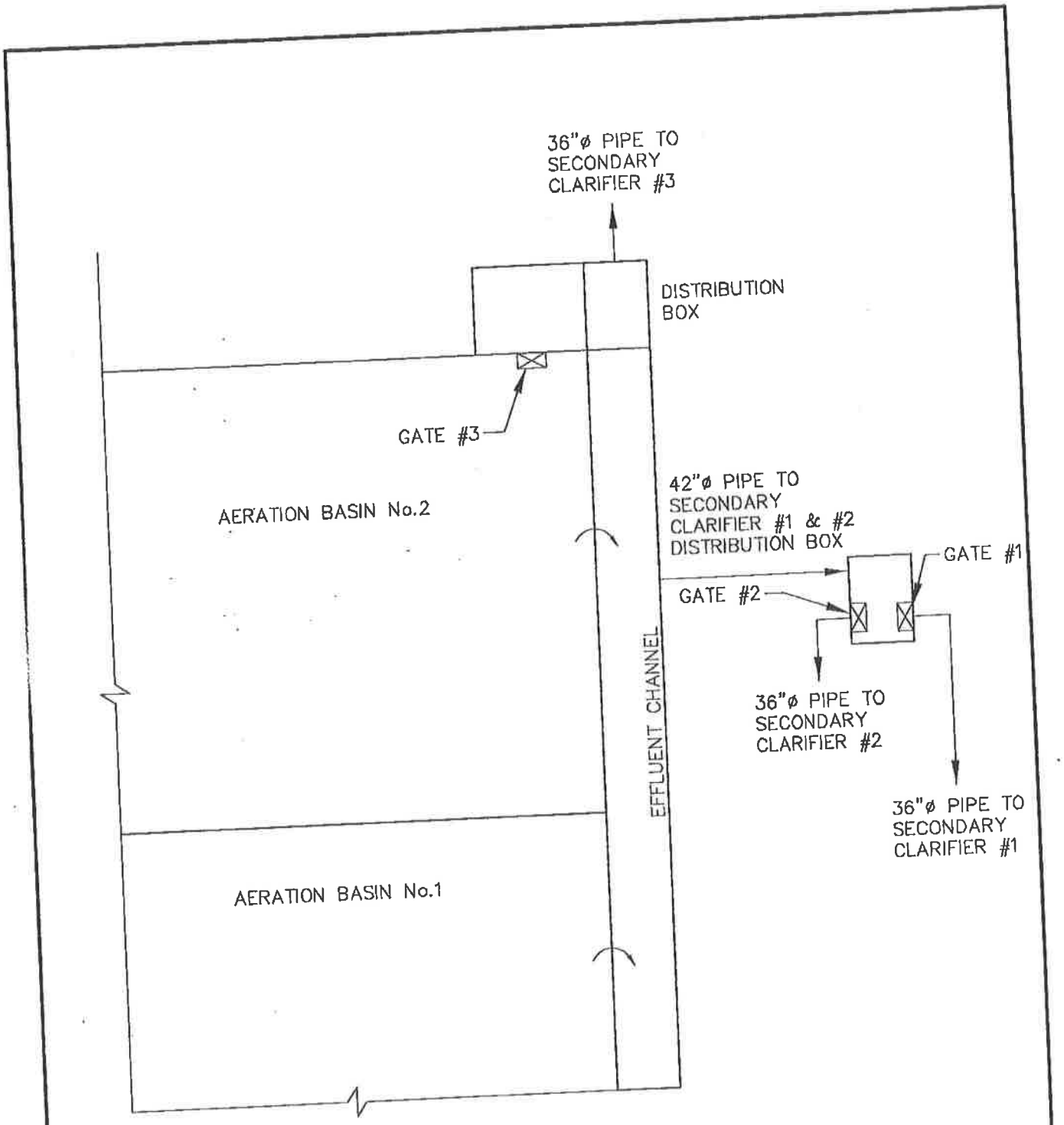


FIGURE VII-2
SECONDARY CLARIFIER DISTRIBUTION FLOW SCHEMATIC

AECOM

SCALE: NTS

DATE: DEC. 2011

SHEET REFERENCE:

ANSI A – 2-Dec-11

Figure VII-3: Secondary Clarifier Distribution Box Flow Split

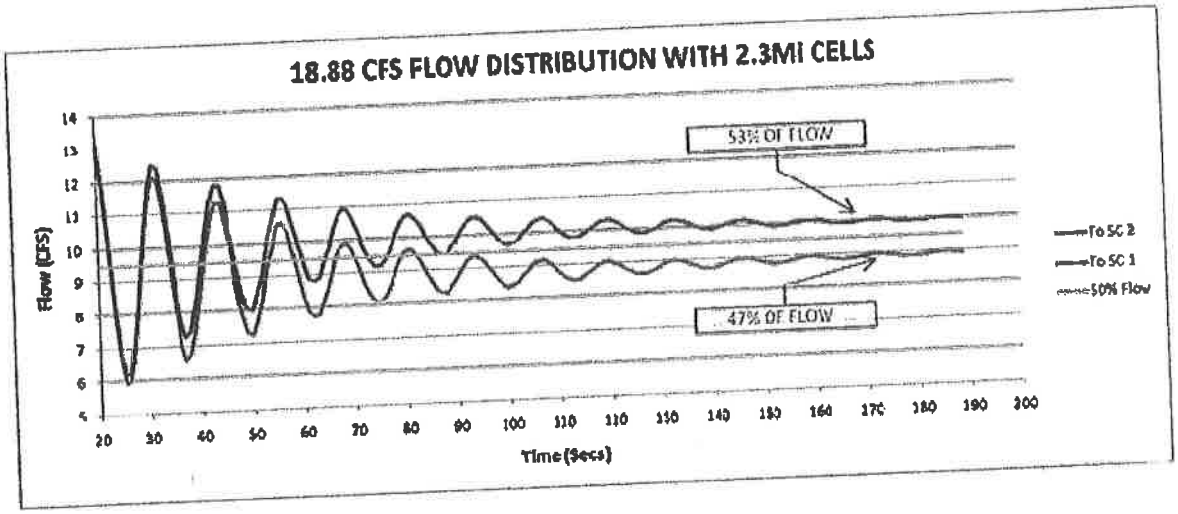


Figure VII-4: Secondary Clarifier Distribution Box Velocity Profile

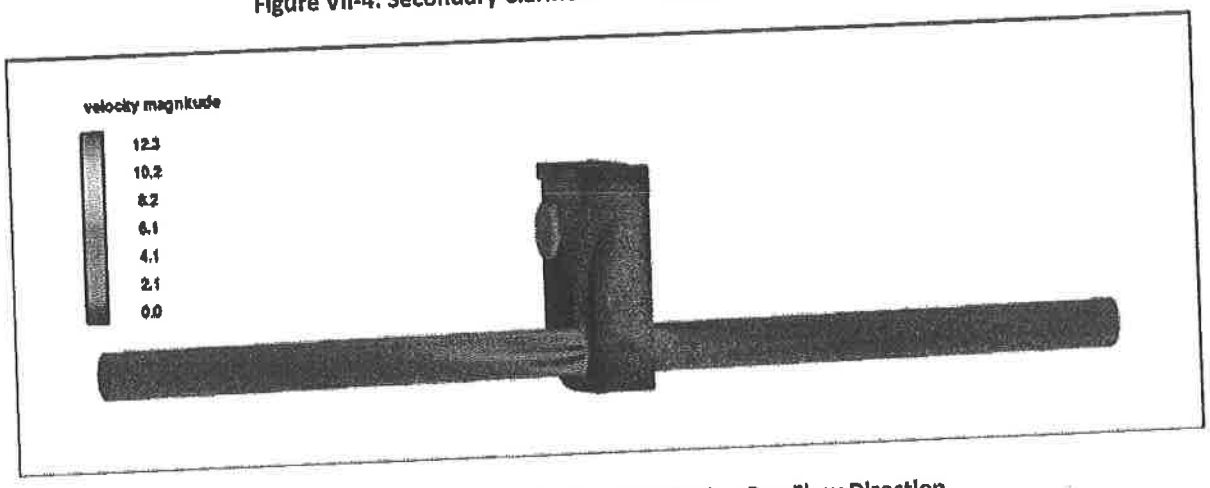
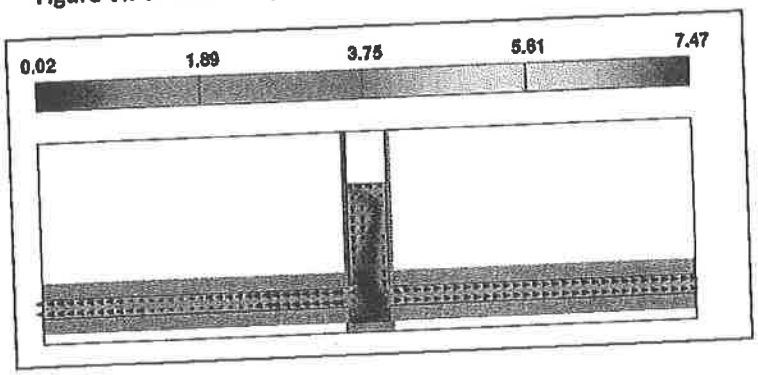


Figure VII-5: Secondary Clarifier Distribution Box Flow Direction



Since the flow split to these two clarifiers is considered equal, and using Clarifier 3 creates an unequal flow split, the suggestion for operation of the secondary clarifiers is as follows:

- Any clarifier can be used during low flows when only one clarifier is needed;
- The preference would be to use Secondary Clarifiers 1 and 2 in combination;
- The preference is not to use Clarifier 3 in combination with 1 or 2;
- When all three clarifiers are needed or if Clarifier 3 is needed with Clarifier 1 or 2, throttle the Gate #3 to Clarifier #3 as described below:
    - Close the gate one foot (24" opening to the 36" pipe).
- When either Clarifier 1 or 2 need to be used in combination with Clarifier #3, throttle Gate #1 or Gate #2 to Clarifier 1 or 2 at the Secondary Clarifier Distribution Box as described below:
    - Close the gate one and a half feet (18" opening to the 36" pipe)

The flow splits expected under the conditions described above are very close to equal and were examined over the range of expected flows.

2. Secondary Clarifier

The CFD model was also used to evaluate the performance of the secondary clarifier as a check against the process model's predicted results of clarifier capacity and point of clarifier failure. The first step in evaluating the secondary clarifier performance was to gain an in-depth knowledge of the plant's unique settling characteristics by performing a number of jar tests. After that, it was possible to calibrate the CFD model and use it to evaluate performance.

a) Jar Tests and Model Calibration

Four settleometer tests were performed at the plant on February 15, 2011. Two 2-L settleometer was used for these tests. The sludge level was observed over the course of two hours to determine the settling characteristics of the sludge. Plant TSS samples, sludge blanket depth, flow rate, RAS flow, and WAS flow data were also collected. The model was calibrated to this data and the calibration was independently confirmed using plant data from January 12, 2010.

The results of the calibration are shown in Figure VII-6. The clear liquid, shown in blue, has the same density as water 62.4 lb/ft^3 (1.940 slug/ft^3)and can be seen on top and thicker sludge, shown in red, with a density of up to 62.56 lb/ft^3 (1.945 slug/ft^3) can be seen at the bottom.

Figure VII-7, Figure VII-8, and Figure VII-9 display the results of these model runs. As mentioned earlier, the density of the dark blue color corresponds with clear water whereas the density of the darker yellows and reds corresponds to sludge. The secondary clarifier density profiles below show that clarifier performance is impacted under future annual average flows with two clarifiers online and under future maximum month flows with three clarifiers online. These results confirm the analysis conducted earlier that the three existing clarifiers will be unable to properly treat the future flows. The results show that the additional clarifier volume solves this problem and results in clear effluent during predicted future maximum day flows. Subsequent sections will review the process calculations associated with the additional clarifier volume.

Figure VII-7: Secondary Clarifier Profile Under Future Annual Average Conditions

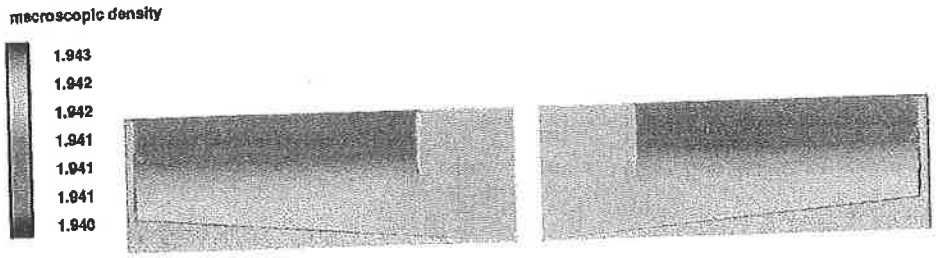


Figure VII-8: Secondary Clarifier Profile Under Future Maximum Month Conditions

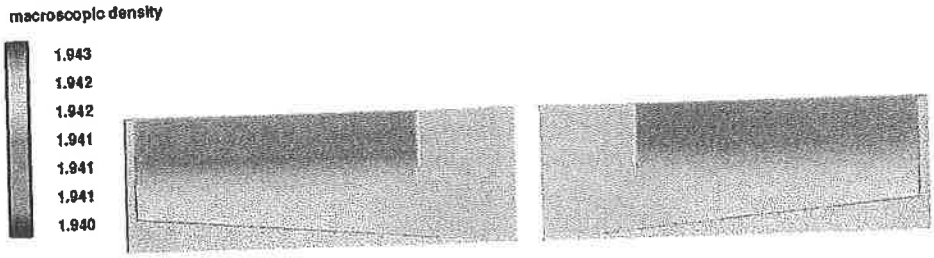


Figure VII-9: Secondary Clarifier Profile Under Future Maximum Day Conditions

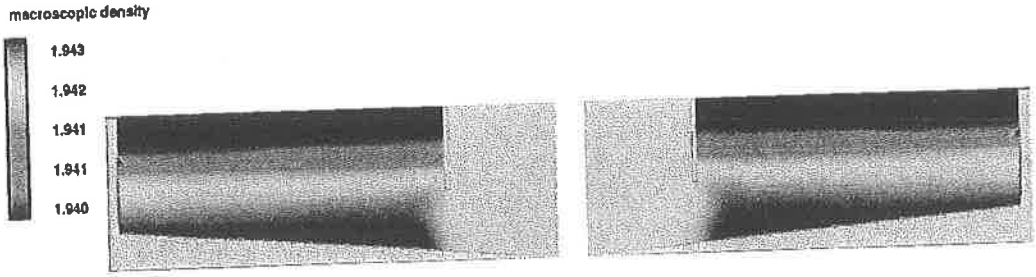
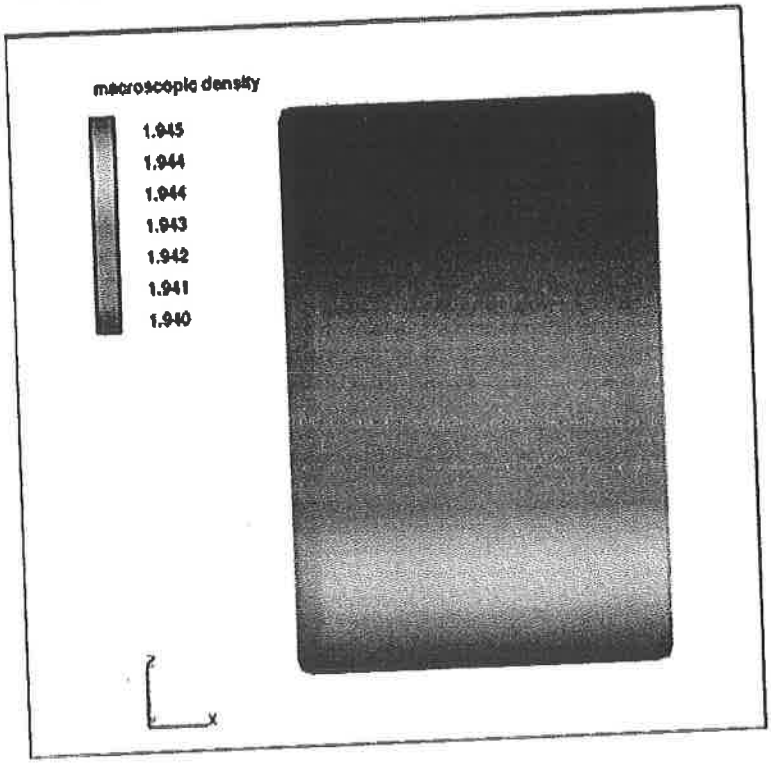


Figure VII-6: CFD Output for the Settleometer Test Showing Macroscopic Density



b) Expected Secondary Clarifier Performance

After calibrating the CFD model to the jar test results and verifying that it matches historical performance, the model was used to study settling in the clarifier under future flow conditions. Density profiles for the scenarios listed in the table below were created to evaluate performance.

Table VII-4: Future Secondary Clarifier Performance Scenarios

	Scenario 1 Annual Average Flow	Scenario 2 Maximum Month Flow	Scenario 3 Maximum Day Flow
Influent Flow, mgd	10.847	15.454	18.44
RAS Flow, mgd	7.9	11.3	11.3
No. of Secondary Clarifiers On-line	2	3	4
Flow to Each Clarifier, mgd	9.37	8.92	7.44

VIII. Proposed Modifications

This section outlines the proposed modifications necessary to address the process and hydraulic limitations raised in the previous sections. It also outlines various steps that can be taken to optimize the existing operation and provide additional capacity.

A. Preliminary Treatment

As mentioned earlier, the screening and grit systems have sufficient capacity to meet future flows and loads.

B. Primary Treatment

In order to treat future design flows and provide a backup to the other two primary clarifiers, the third clarifier will be used more frequently. The current peripheral feed, center take-off configuration of the system does not work well and is not even offered anymore by equipment manufacturers as a method for primary treatment. The clarifier will perform better if the scum removal system is upgraded to be a full diameter, peripheral take-off type. This will eliminate the center take-off and weirs, and allow a full radius skimmer to be installed. A new influent and effluent raceway will be installed and the drive mechanism replaced to allow for sludge removal and full radius skimming.

C. Secondary Treatment

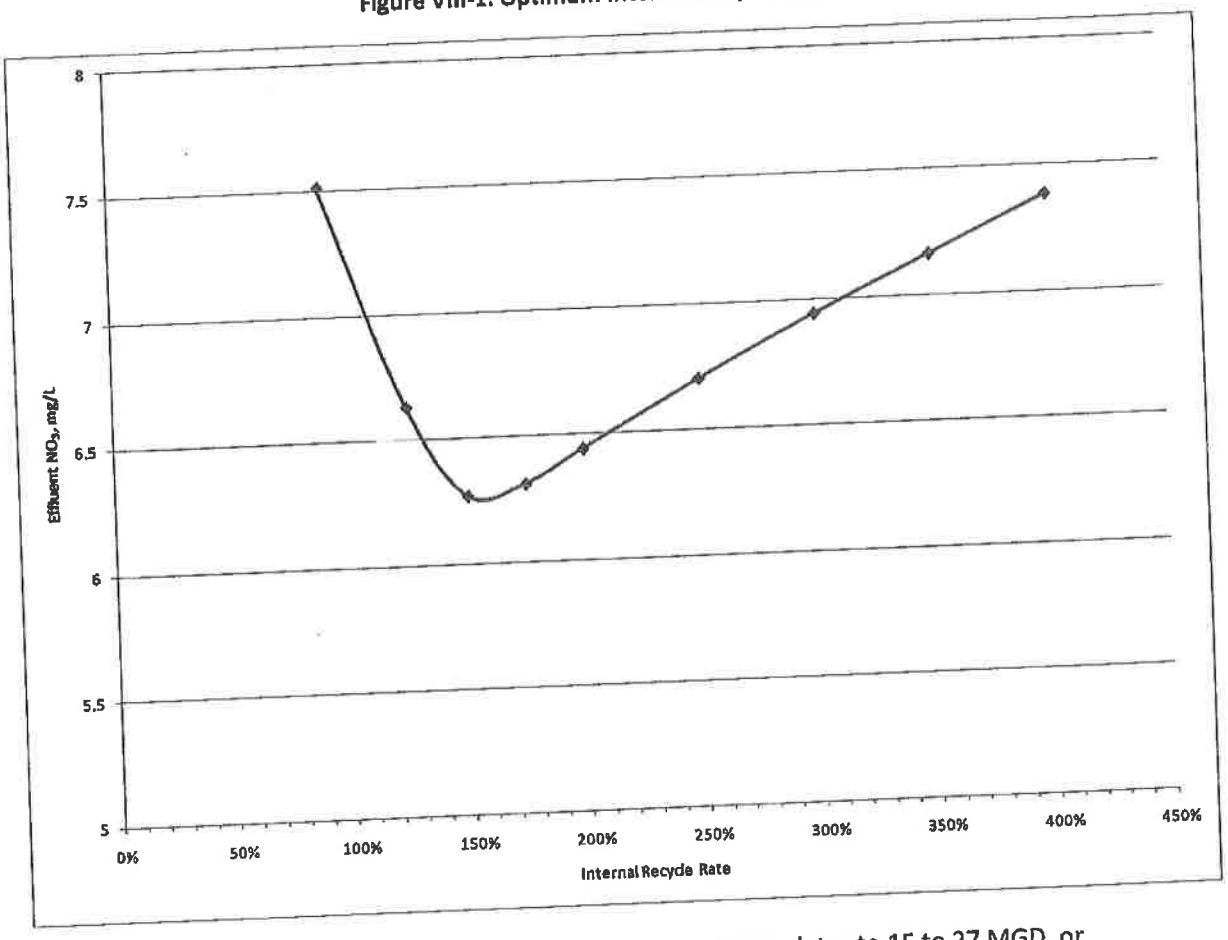
1. Aeration Basin

The aeration basin is split into three zones. The first of these zones is anoxic and denitrifies while the next two are aerobic and performs nitrification. There is a high-volume internal recycle from the aerobic zone to the anoxic zone which returns high concentrations of nitrate to be converted into nitrogen gas. This section divides recommendations between the nitrification and denitrification processes.

a) Denitrification Process

Denitrifying bacteria grow significantly faster than nitrifying bacteria, so anoxic SRT is not a controlling factor in this process. The main factors affecting denitrification at the WWTF will be influent carbon and the internal recycle rate. The denitrifying bacteria require a carbon source from the influent wastewater to convert nitrate to nitrogen gas. We cannot control the carbon source, but we can evaluate the optimum internal recycle rate. The chart below displays the effluent TN as a function of internal recycle rate. As seen in the chart, the optimum IR rate is between 150 and 175% of influent flow. This trend holds true at both average flow and maximum month.

Figure VIII-1: Optimum Internal Recycle Rate



At the future flows, the optimum recycle rate translates to 15 to 27 MGD, or approximately 10,400 to 18,750 gpm. Split between two basins this is 5,200 to 9,375 gpm. The design point of the existing internal recycle pumps is 10,700 gpm. These pumps are run with variable frequency drives and according to the pump curves have sufficient turndown to meet these flows.

Flow meters are sometimes placed on internal recycle lines, although more often than not there is no flow measurement since the flow volumes are large and do not need to be finely controlled to get similar results. In this case, the difference between 150% of influent flow and 175% of influent flow is so small that the installation of a flow meter likely is not worth it, especially since submersible flow meters are difficult to install and very expensive.

Because exact flow measurement of internal recycle is not important, it is suggested that plant operators optimize this system in a systematic manner using speed set points on the variable frequency drives.

b) *Nitrification*

If the City's discharge permit were a monthly average, additional aerobic volume would be required to treat the expected increase in flow and, more importantly, load. However, because the discharge permit is an annual average for nitrogen, there is some flexibility in the analysis. Earlier, the minimum desired aerobic SRT was set at 13 days using an effluent ammonia concentration of 0.5 mg/L. If the effluent ammonia concentration is allowed to increase during cold weather to 1.0 mg/L, then the minimum desired aerobic SRT will decrease. Increasing the effluent ammonia above 1 mg/L is not recommended because nitrification rapidly decreases and the nitrification process is difficult to restart once lost.

The existing aeration basins have sufficient capacity to maintain minimal nitrification during the minimum temperature and maximum month flow and load. Maintaining nitrification will require close operational control during this time frame. To eliminate the risk of losing nitrification during future high flow and high load months and to provide some level of operational flexibility, AECOM recommends installing aeration diffuser grids in the anoxic zones in the future so as to have the ability to use the volume as a swing zone. The use of these grids would eliminate denitrification when in use, but it will maintain nitrification which is a much more difficult process to restore once lost.

c) *pH Control*

As seen in the figure below, the pH at the plant averages approximately 6.7. Nitrification operates most efficiently at a pH of approximately 7.2. Figure VIII-3 shows a graph from the EPA Nitrogen Control Manual that shows the effect pH on nitrification rate. Because of the nature of the graph, even changes of a tenth of a pH unit can have a large effect on the nitrification rate. At a pH of 6.7, the nitrification rate is approximately 70% of maximum. If the average pH of the plant is 6.7, half of the time the nitrification rate is less than 70% of maximum. The effect of the depressed pH is to lengthen the overall SRT needed for full nitrification. Ultimately, this reduces the capacity of the aeration basins.

Figure VIII-2: Historical Effluent pH

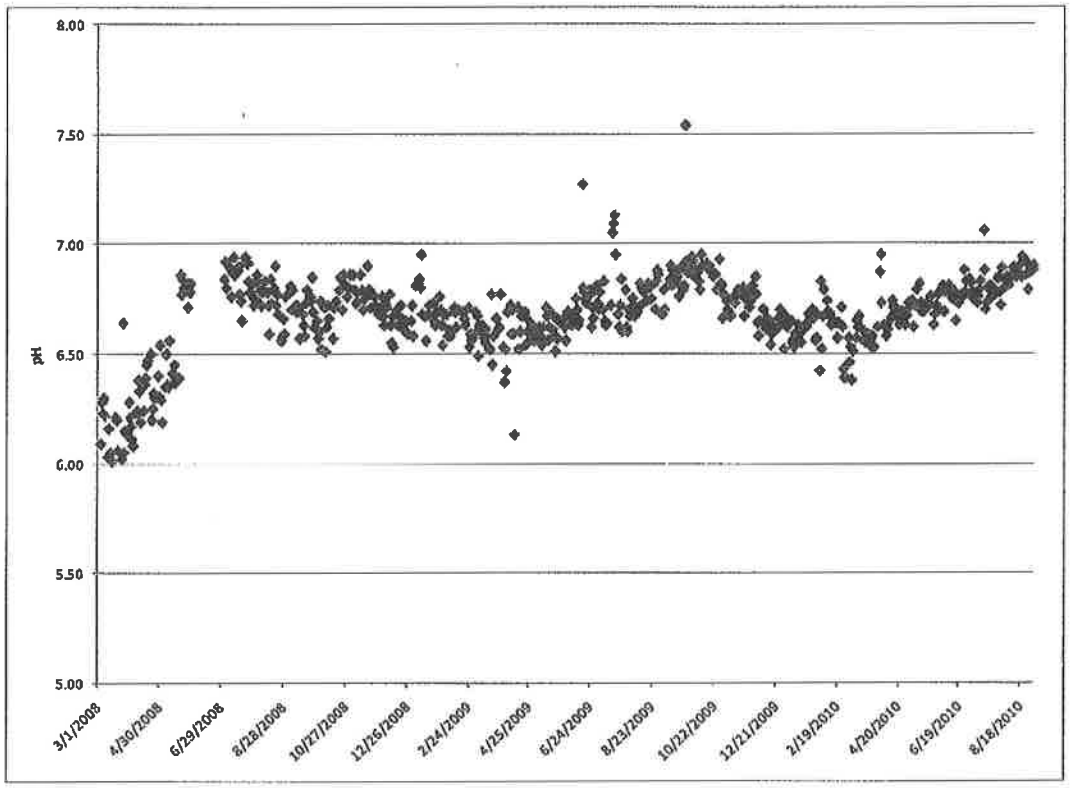
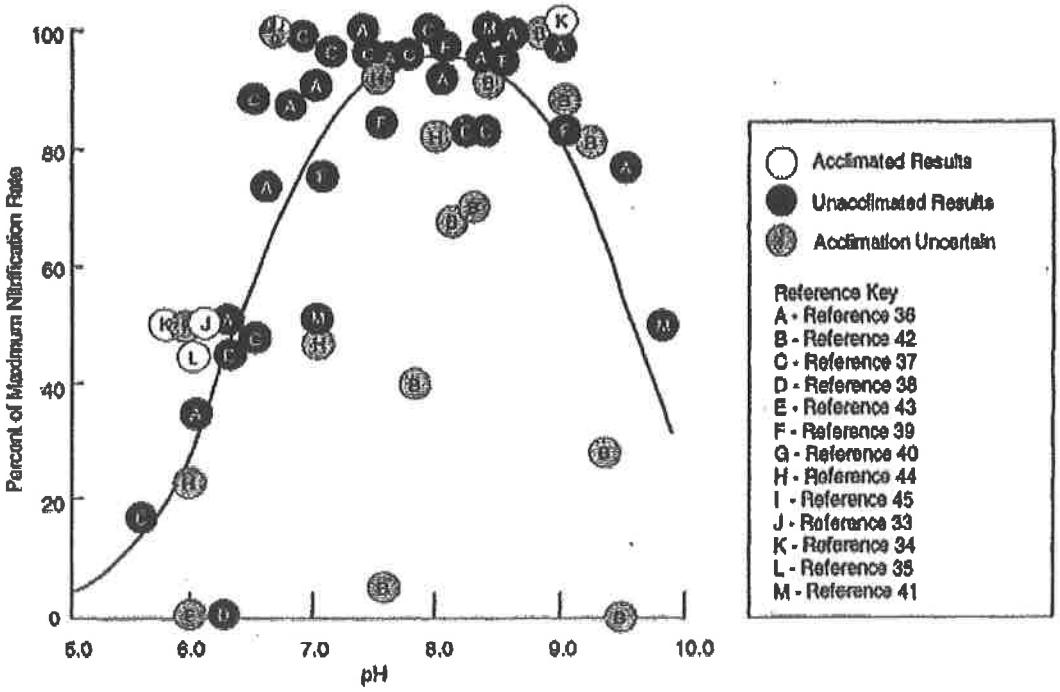


Figure VIII-3: Effect of pH on Nitrification Rate (from EPA Nitrogen Control Manual)



Operating at a pH of 7.2 often becomes economically burdensome, but operating at a pH of 7.0 is recommended. The process model was used to determine how much additional alkalinity would be necessary to raise the pH of the aeration basin to 7.0. Simple titration curves using actual mixed liquor could be used to verify this result. Sodium hydroxide was assumed for this evaluation. There are a number of chemicals that can be used for the control of pH such as sodium hydroxide, lime, sodium bicarbonate or magnesium hydroxide. Table VIII-1 summarizes the amount of chemical needed at average and maximum month flow rates. Sodium hydroxide was added to the process downstream of the anoxic zone.

Table VIII-1: Alkalinity Necessary for pH Control

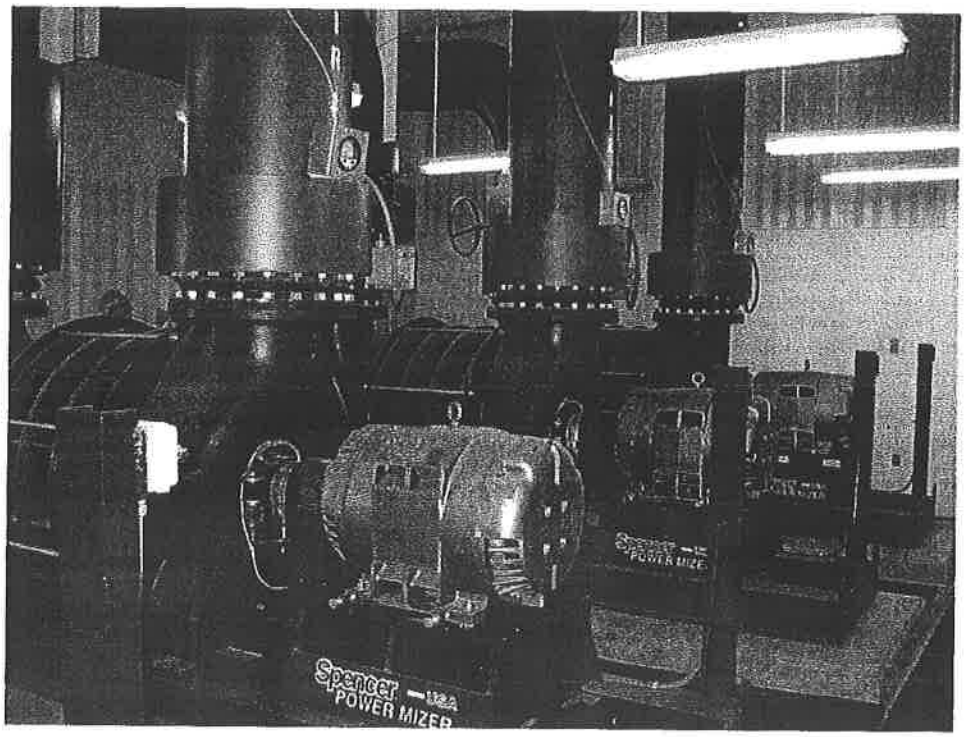
Flow Rate, MGD	Sodium Hydroxide Needed, gpd
Annual Average (10.847 MGD)	1,000
Maximum Month (15.454 MGD)	1,100

An alkalinity addition system is not recommended at this time but is an option for the City to consider should they wish to increase capacity at the plant. At such time as this system is installed, AECOM recommends completing a life cycle cost analysis as the cost of chemicals changes frequently. There are also other factors to be considered in this evaluation such as material handling and operator safety.

d) Aeration

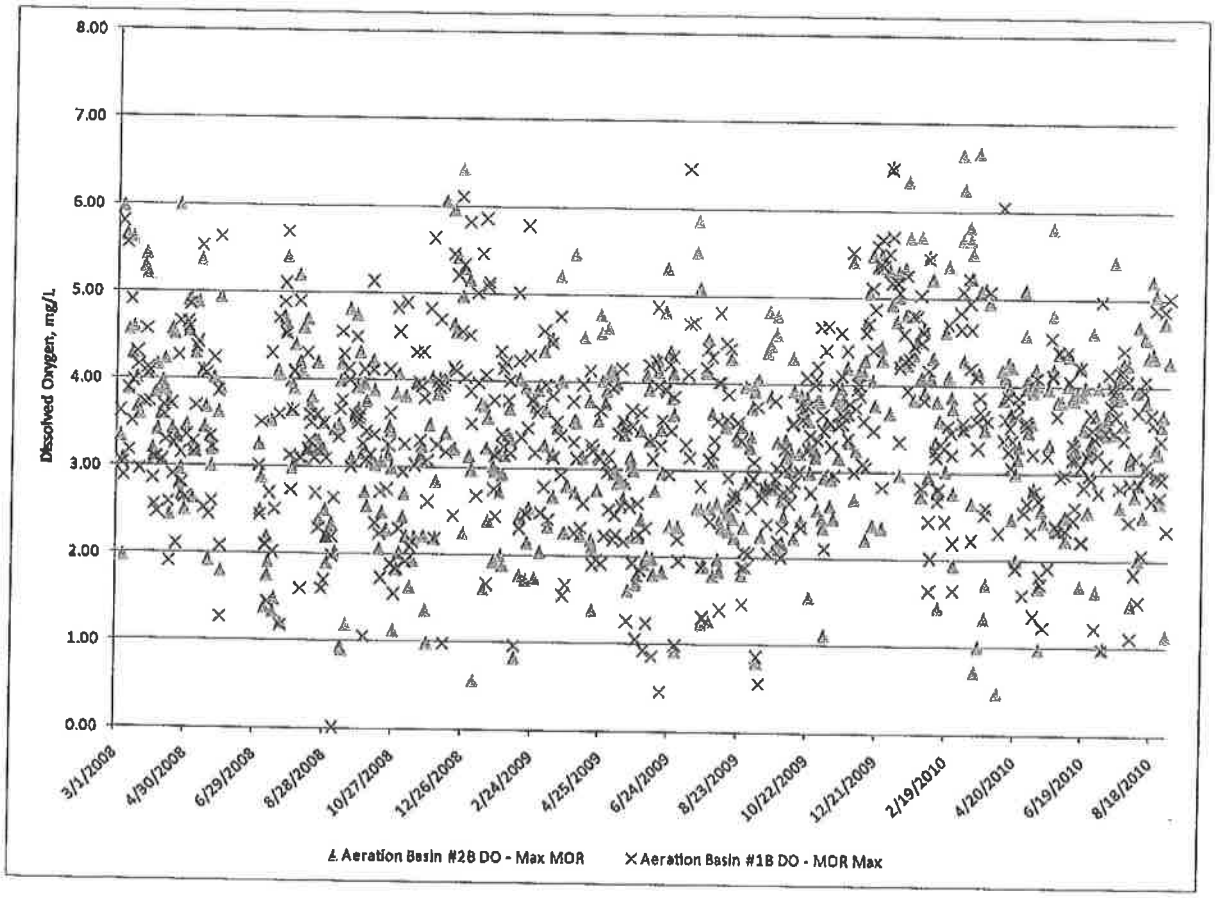
As alluded to earlier, the plant operates with a higher than optimal dissolved oxygen concentration. There are three 250 HP blowers and one 150 HP blower installed. The 250 HP blowers are rated for 6,250 scfm at 5.7 psig. The 150 HP blower is rated for 3,500 scfm at 5.7 psig. Typically, the 150 HP blower only is operated. It is turned down as far as possible overnight and still over-aerates the mixed liquor.

Figure VIII-4: Aeration Blowers



The figure below displays the historical DO concentration, which is typically between 2 and 5 mg/L. The result of the high DO concentration is that oxygen is carried back to the anoxic zone via the internal recycle pump which limits the capacity of the anoxic zone. The oxygen in this zone must be consumed before denitrification begins to take place, which also consumes some of the carbon needed for denitrification. DO concentration is generally recommended to be 2.0 mg/L or less. Tapered aeration concentrations are common if multiple drop pipes exist.

Figure VIII-5: Historical DO Concentration



The model was used to determine the air requirements under current and future conditions which are shown in Table VIII-2 and Table VIII-3. It is important to note that these values do not include diurnal fluctuations or maximum day loading, both of which help drive blower selection. Further refinement will be necessary if improvements are made to the existing blower system. When compared to air flows recorded in June 2011, the current air flow rate requirements appear to correlate well with reality. Therefore, it is believed that the future loads can be used to relatively accurately predict future air flows. It is important to note that the future air flows are at SRTs less than 13 days because that is all the existing aeration basins will support. If additional aerobic volume is constructed or if SRT is increased by other means, these values will change.

Table VIII-2: Current Flow Air Requirements

	13°C	23°C
Average Annual Flow, scfm	4,780	5,070
Maximum Month Flow, scfm	5,950	6,120

Table VIII-3: Future Flow Air Requirements

	13°C	23°C
Average Annual Flow, scfm	7,400	7,800
Maximum Month Flow, scfm	8,800	9,400

The discharge pressure seen by the blowers is largely due to the depth of water in the aeration basin. The water depth in the basin is approximately 12.1 ft or 5.2 psi. After taking into account the diffuser depth, loss through the diffuser, and minor losses in piping system, it is estimated that the discharge pressure seen by the blowers is approximately 6.5 psig. The blower curves for multi-stage centrifugal blowers such as these are frequently very flat, which results in very little turndown to meet minimum needs and in some cases, diurnal fluctuations. Based on the blower curves and operator input, the minimum flow capacity of the 250 HP machines is estimated to be approximately 3,800 scfm. The minimum turndown capacity of the 150 HP machine is estimated to be 2,700 scfm. At these flows, even small changes in discharge pressure will cause the blower to go into surge, so actual minimum turndown capacities may not even reach these levels. Based on these estimates, it is not surprising that the dissolved oxygen concentrations in the basin are frequently much higher than 2 mg/L.

As flow and load increases, the existing blowers appear to be sufficient to satisfy future demand from a flow standpoint. By using the 150 HP blower as a jockey blower, the system appears to have the capacity to run continuously from 2,700 scfm to over 20,000 scfm. In the interim, however, the blowers will continue to provide more oxygen than the process requires. A smaller blower should be considered.

e) Hydraulic Capacity

As mentioned earlier, there are two recommended modifications to ensure that the existing aeration basins have the necessary hydraulic capacity to meet the future demands. The City should enlarge the window openings between the anoxic and aerobic zones to 4' x 4' to bring the head loss down to less than 3 inches. A structural evaluation of the wall should take place prior to enlarging these openings to ensure that the wall will still be stable after large openings are cut.

Additionally, the effluent weirs should be set at an equal elevation of 17.50.

2. Secondary Clarifier

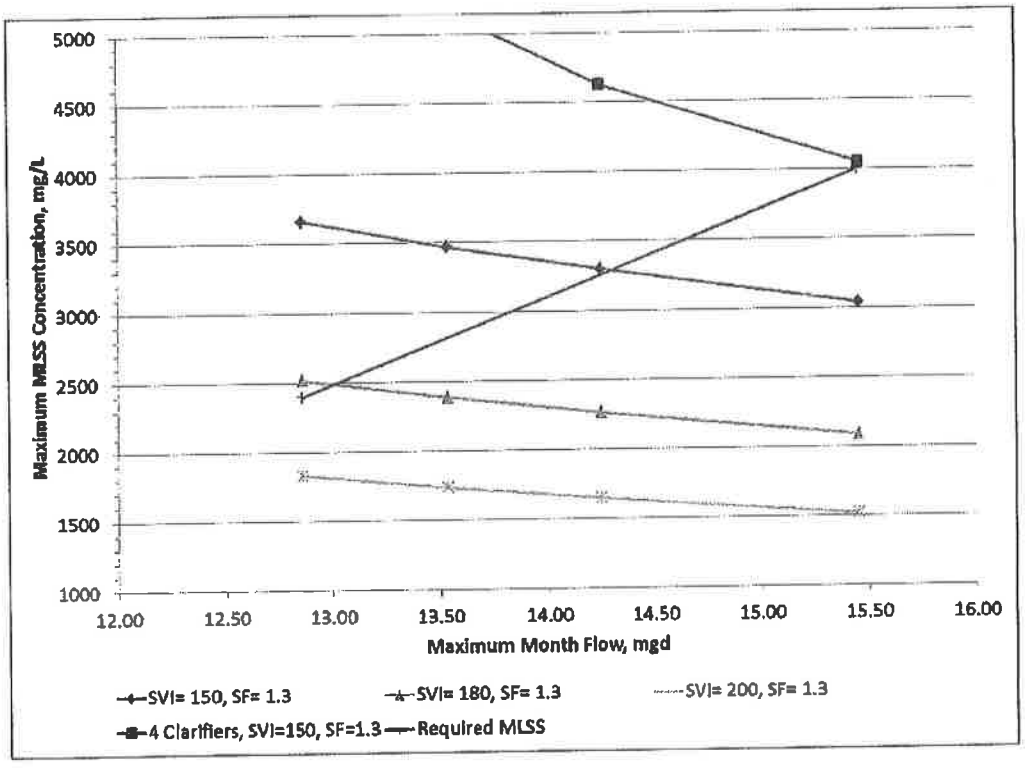
As mentioned earlier, the three existing secondary clarifiers are not sufficient to treat future flows, and their ultimate capacity is tied into the operational characteristics of the treatment plant and the SVI.

TR-16 states that processes with selectors, such as New London, should design using an SVI of 150 mL/g. However, historical data show that the SVI is often above 150 mL/g

and the maximum 30 day rolling average is 200 mL/g so fluctuating SVI's will need to be addressed in this evaluation. Refer to previous Figure VI-5.

While ultimate mixed liquor concentrations are predicted to be 4,000 mg/L, current and interim MLSS concentration will be less than that, which will also impact the timing of the additional capacity. The timing of the additional clarifier capacity can be drawn from Figure VIII-6. As shown earlier, clarifier capacity depends a great deal on SVI. Assuming that an SVI of 150 mL/g is maintained and that the required MLSS increases linearly as flow and loading increases, the existing clarifiers have a maximum month flow capacity of approximately 14.3 mgd. This corresponds to an annual average flow capacity of 10 mgd.

Figure VIII-6: Timing of Additional Secondary Clarifier



Above an annual average flow of 10 MGD, additional clarifier capacity will be required. The following assumptions were used to determine how much clarifier capacity is needed for future flows:

- Maximum day flow of 18.44 MGD
- RAS rate of 55% of influent flow during maximum day
- MLSS concentration of 4,000 mg/L
- RAS concentration of 9,300 mg/L based on the model results

Table VIII-4 displays the number of clarifiers needed over a range of SVI's. The $SLR_{failure}$ term is the SLR when the clarifier is expected to fail, meaning the solids blanket will become unstable and solids will start overflowing the weir with resulting permit violations. The SLR_{design} term includes a factor of safety of 1.3 per TR-16. The SLR_{actual} term is calculated SLR under maximum month conditions with the listed number of additional clarifiers. Of note is that the SLR_{actual} is not much lower than the SLR_{design} term, which means that there is not a great deal of extra clarifier capacity. Provided that the plant is operated with an SVI of 150 mL/g or less, only one additional clarifier will be necessary.

Table VIII-4: New Clarifiers Needed

SVI, mL/g	$SLR_{failure}$, lb/d/ft^2	SLR_{design}, lb/d/ft^2	New Clarifiers Needed	SLR_{actual}, lb/d/ft^2
150	40	30.8	1	28.8
180	27.5	21.2	3	19.8
200	20	15.4	5	14.4

Table IX-2: O&M Costs of Recommended Improvements

	Additional Secondary Clarifier		
	Unit Cost	Number	Cost
Labor	$30.00	208	$6,240
Power	$0.10	215,567	$21,557
Chemical	$0.00		$0
	$0.00		$0
	$0.00		$0
Annual Operating Cost			$27,800

Table IX-3: Life Cycle Costs of Recommended Improvements

	Rehabilitate Primary Clarifier #3	Additional Secondary Clarifier	Aeration Basin Wall Openings	Additional Jockey Blower
Escalated Capital Cost (2011)	$567,500	$2,224,200	$47,000	$412,200
Annual Operating Cost	$0	$27,800	$0	$0
Present Worth Operating Cost	$0	$395,200	$0	$0
Total Life Cycle Cost	$567,500	$2,619,400	$47,000	$412,200

IX. Cost Estimate

This section contains Planning Level Project Costs which are preliminary in nature and contain construction cost, construction contingencies, administrative, legal, construction engineering, environmental and regulatory permitting. Costs have been developed for the four recommendations mentioned earlier:

- Addition of wall openings to the Aeration Basins
- Rehabilitation of Primary Clarifier #3
- Addition of a Secondary Clarifier
- Addition of a jockey blower

The cost estimates are Class 4 opinion of probable construction costs and were developed in accordance with "AACE International Recommended Practice No. 18R-97 - Cost Estimate Classification System – As Applied in Engineering, Procurement, and Construction for the Process Industries" as prepared by the Association for the Advancement of Cost Estimating (AACE) International dated February 2, 2005.

Table IX-1 through Table IX-3 present the capital costs, O&M costs, and life cycle costs of the recommended improvements. The O&M costs for rehabilitating the primary clarifier, increasing the size of the wall openings, and the blower addition are not included because these items will not increase the existing O&M budget. The total life cycle cost of the recommended improvements is $3.6M.

Table IX-1: Capital Costs of Recommended Improvements

	Rehabilitate Primary Clarifier #3	Additional Secondary Clarifier	Aeration Basin Wall Openings	Additional Jockey Blower
Civil	$0.00	$188,900.00	$0.00	$0.00
Structural	$60,000.00	$598,500.00	$25,000.00	$5,000.00
Architectural	$0.00	$0.00	$0.00	$0.00
Mechanical	$217,800.00	$325,500.00	$0.00	$180,000.00
HVAC	$0.00	$0.00	$0.00	$0.00
Plumbing	$0.00	$0.00	$0.00	$0.00
Electrical & Instrumentation	$25,000.00	$75,000.00	$0.00	$35,000.00
Subtotal	$302,800.00	$1,187,900.00	$25,000.00	$220,000.00
Contingency (30%)	$90,900.00	$356,400.00	$7,500.00	$66,000.00
Subtotal	$393,700.00	$1,544,300.00	$32,500.00	$286,000.00
Contractor Overhead & Profit (20%)	$78,800.00	$308,900.00	$6,500.00	$57,200.00
Estimated Construction Cost (2011)	$472,500.00	$1,853,200.00	$39,000.00	$343,200.00
Professional Services (20%) *(includes Legal and Administrative)*	$95,000.00	$371,000.00	$8,000.00	$69,000.00
Total Capital Cost (2011)	$567,500.00	$2,224,200.00	$47,000.00	$412,200.00

X. Funding

In Connecticut, the Clean Water State Revolving Fund (CWSRF) is a program that can assist in financing wastewater treatment plant improvement projects. The fund was established in 1986 to provide financial assistance to municipalities for planning, design and construction of wastewater collection and treatment projects. Financial aid is provided to partially offset the cost of treatment plant improvements through grants and/or low-interest loans. Typically, the State funding availability for treatment plant projects consists of a grant on eligible items and a 2%, 20 year loan on the remaining portion.

Although all capital improvements to wastewater treatment facilities are eligible for the state grant-loan program, the projects in New London are unlikely to have sufficient priority points to receive funding. Points are assigned to each project using a priority rating system and those projects which remove nutrients such as nitrogen receive the highest priority points. A priority rating score for each wastewater infrastructure project is established and becomes the prime consideration in the overall priority list ranking. Given the number of other proposed projects in Connecticut and their likely priority points, the projects included in this report would likely not be very high on the current priority list and would not be fundable.

Connecticut Light and Power (CL&P) offers incentives to encourage the design of energy efficient plant improvements such as blower replacements, high efficiency motors and variable frequency drives. These incentives are applied for early on in the design process so as to maximize the incentive from the power company. Any design projects that move forward as a result of this report should consider this potential funding source.

Appendix A
Wastewater Characterization Data

Values that are excluded by entering a * in front of value

Questionable data based on analysis date

Listed as No Data (ND)

INFLUENT
MEASURED PARAMETERS

Day	Flow	TSS	VSS	COD	sCOD(0.45 micron)	dCOD(GF)	TCOD	TBOD5	NO2 at end of TBOD test	CBOD5	sCBOD (GF)	TKN	sTKN (0.45 micron)	sTKN (GF)	NH3-N	NO3-N	NO2-N	TP	sTP (0.45 micron)	sTP (GF)	PO4-P	Alk	pH	Dissolved Oxygen	Dissolved Sulfide	Temp
Daily																										
Grab										260	100	36			17											17.0
Diurnal										160	50	37			20											17.5
11/30/10	5.80	370	300	680	110	160	84	590		170	<15	35			25						3.9	3.8	160	6.9		17.4
12/2/10	5.82	340	300	680	100	100		310										5.1			3	2.7	170	7.1		16.9
12/3/10	6.09	340	300	680				280										7.0				3.3	160	7		16.8
12/7/10																		6.6								16.1
12/9/10	9.77	250	200	500	110	150	75	270		150	57	20			24	21	<0.01	<0.01	0.3		3.8	3.2	170	7.2		17.10
12/10/10	8.45	200	170	500	88	110	60	480		310	73	34			29	19			5.8		3.3	3.1	160	7.1		
Average	6.00	308.00	264.00	642.00	99.40	142.50	72.33	376.00	#DIV/0!	214.00	70.00	38.33	#DIV/0!		25.50	20.00	#DIV/0!	#DIV/0!	5.74	#DIV/0!	3.45	3.15	160.00	7.04	#DIV/0!	#DIV/0!
Count	5.00	5.00	5.00	5.00	5.00	4.00	3.00	5.00	0.00	5.00	4.00	3.00	0.00		4.00	3.00	0.00	0.00	5.00	0.00	4.00	5.00	5.00	5.00	0.00	6.00

CALCULATED PARAMETERS

Day	SRT	VSS/TSS		TSS/TBOD5	sCOD/COD	sCOD/COD	TCOD/sCOD	COD/TBOD5	Prt COD/VSS	Tot COD/VSS	CBOD5/TBOD5		TSS/COD	TKN/COD	TKN/VSS	NH3/TKN	PO4/TP	sTP/TP	TP/COD	sTKN/TKN
30-Nov	40	0.86		0.83	0.30	0.24	0.55	1.15	1.58	2.06	0.47		0.08	0.08	0.12	0.45	0.43	0.48	03.05	
2-Dec	40	0.86		1.10	0.12			1.80	1.70	1.97	0.62		0.07	0.08	0.12	0.60	0.38	0.42	83.10	0.70
3-Dec	40	0.86		1.31	0.15	0.29		2.54		2.29	0.66		0.06	0.06	0.13	0.66	0.50		100.00	0.81
7-Dec							0.50	0.40	1.58	2.27	0.71		0.06	0.06	0.13	0.72	0.61	0.57	78.07	0.83
8-Dec	30	0.86		1.19	0.22	0.31	0.55	1.10	2.47	3.12	0.65		0.06	0.06	0.20	0.56	0.55	0.69	84.64	0.85
10-Dec	30	0.86		0.42	0.17	0.24	0.50	1.62	1.83	2.52	0.66		0.06	0.06	0.14	0.58	0.47	0.52	86.21	0.75
Average	38	0.86		0.93	0.16															

Values that are excluded by entering a * in front of value
No analysis completed
Questionable data based on analysis date

EQUALIZED PRIMARY EFFLUENT
MEASURED PARAMETERS

PRIMARY EFFLUENT

Day	Flow	TSS	VSS	COD	sCOD(0.45 micron)	dCOD(GF)	tCOD	TBOD5	NO3 at end of TBOD test	CBOD5	dCBOD (GF)	TKN	sTKN (0.45 micron)	dTKN (GF)	NH3-N	NO3-N	NO2-N	TP	sTP (0.45 micron)	dTP (GF)	PO4-P	Alk	pH	Dissolved Oxygen*	Dissolved Sulfide	Temp*
Daily																										
Spot																										
Diurnal																		5.4		3		160	6.9			
30-Nov		100	91	360				370		260		34						5.2		3.1		170	6.9			
2-Dec		100	91	360	84	120	84	250		170		31		25	19			5.0		3.3		150	7			
3-Dec		110	100	330														4.9		2.9		170	7.1			
7-Dec		92	82	310	78	130	78	310		150		27		26	22			4.8		3.6		160	7.2			
8-Dec		98	88	360		120	93	240		160		35		27	24			5.6		2		170	7.1			
10-Dec		90	81	300				200		200		35														
Average	#DIV/0!	98.50	88.50	320.33	70.00	123.33	77.67	260.00	#DIV/0!	182.00	#DIV/0!	32.40	#DIV/0!	26.00	21.67	#DIV/0!	#DIV/0!	5.27	#DIV/0!	3.13	#DIV/0!	163.33	7.03	#DIV/0!	#DIV/0!	#DIV/0!
Count	0.00	6.00	6.00	6.00	2.00	3.00	3.00	5.00	0.00	5.00	0.00	5.00	0.00	3.00	3.00	0.00	0.00	6.00	0.00	6.00	0.00	6.00	6.00	0.00	0.00	0.00

CALCULATED PARAMETERS

Day	ISS	VSS / TSS	TSS / COD	TSS / CBOD5	sCOD / COD	tCOD / dCOD	COD / CBOD5	Prt COD / VSS	Tot COD / VSS	TKN / COD	TKN / VSS	NH3 / TKN	Fnox	Fus*	Fps*	Fpa*
	9	0.91		0.69			1.80		3.96	0.09	0.37					#REF!
	9	0.91		0.69	0.43	0.53	1.85	1.76	3.06	0.11	0.34	0.61				#REF!
	10	0.91							3.90							#REF!
	10	0.89		0.61	0.42	0.58	2.07	2.20	3.76	0.09	0.33	0.61				#REF!
	10	0.90		0.51	0.33	0.78	1.89	2.76	4.19	0.10	0.41	0.69	*	*	*	#REF!
	12	0.87		0.47			1.80		3.78	0.12	0.43					
Average	10	0.90		0.53	0.39	0.63	1.79	2.25	3.67	0.10	0.38	0.70				

MIXED LIQUOR, RAS/WAS, FILTRATE

Values that are excluded by entering a ' in front of value

Anoxic Zone

Day	Flow	TSS	VSS	COD	sCOD(0.45 micron)	dCOD(GF)	fCOD	TBOD5	NO3 at end of TBOD test	CBOD5	dCBOD (GF)	TKN	sTKN (0.45 micron)	dTKN (GF)	NH3-N	NO3-N	NO2-N	TP	sTP (0.45 micron)	dTP (GF)	PO4-P	Alk	pH	Dissolved Oxygen	Dissolved Sulfide	Temp
Daily																										
Spot																										
Diurnal																		<0.01								
30-Nov											<5					3.10	0.44									
2-Dec											<5					2.60	0.25									
3-Dec											13					3.10	0.21									
7-Dec											<5					2.20	0.38									
9-Dec											<5					3.70	0.40									
10-Dec											<5					3.04	0.37	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Ave	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	13.00	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	5.00	5.00	0.00	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Count	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	0.00	1.00	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Aerobic Zone

Day	Flow	TSS	VSS	COD	sCOD(0.45 micron)	dCOD(GF)	fCOD	TBOD5	NO3 at end of TBOD test	CBOD5	dCBOD (GF)	TKN	sTKN (0.45 micron)	dTKN (GF)	NH3-N	NO3-N	NO2-N	TP	sTP (0.45 micron)	dTP (GF)	PO4-P	Alk	pH	Dissolved Oxygen	Dissolved Sulfide	Temp
Daily																										
Spot																										
Diurnal																										
30-Nov		2500	2000	3400																				2.28		16.6111
2-Dec		2800	1700	3400																				3.365		16.6111
3-Dec		1800	1400	3800																				3.28		16.25
7-Dec		2500	1976	3400																				2.495		15.2778
9-Dec		2300	2100																					3.725		15.0556
10-Dec		2200	2000	3400																				3.145		14.7222
Ave	#DIV/0!	21.66.67	1895.00	3440.00	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0.00	0.00	0.00	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	3.22	#DIV/0!	15.70
Count	0.00	6.00	6.00	5.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	6.00

RAS

Day	Flow	TSS	VSS	COD	sCOD(0.45 micron)	dCOD(GF)	fCOD	TBOD5	NO3 at end of TBOD test	CBOD5	dCBOD (GF)	TKN	sTKN (0.45 micron)	dTKN (GF)	NH3-N	NO3-N	NO2-N	TP	sTP (0.45 micron)	dTP (GF)	PO4-P	Alk	pH	Dissolved Oxygen	Dissolved Sulfide	Temp
Daily																										
Spot																		94								
Diurnal																		92								
30-Nov	5.4	5100	4400															98								
2-Dec	5.4	5100	4500															100								
3-Dec	5.4	5200	4500															94								
7-Dec	5.7	4800	4064															72								
9-Dec	6.1	4600	4200																							
10-Dec	6.6	3800	3360															91.33								
Ave	5.60	4760.67	4180.67	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	91.33	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Count	6.00	6.00	6.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	6.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

WAS

Day	Flow	TSS	VSS	COD	sCOD(0.45 micron)	dCOD(GF)	fCOD	TBOD5	NO3 at end of TBOD test	CBOD5	dCBOD (GF)	TKN	sTKN (0.45 micron)	dTKN (GF)	NH3-N	NO3-N	NO2-N	TP	sTP (0.45 micron)	dTP (GF)	PO4-P	Alk	pH	Dissolved Oxygen	Dissolved Sulfide	Temp
Daily																										
Spot																										
Diurnal																										
30-Nov	0.002	5900	5208																							
2-Dec	0.002	7600	6900																							
3-Dec	0.002	6600	5700																							
7-Dec	0.005	5600	4900																							
9-Dec	0.009	5000	5100																							
10-Dec	0.017	12000	4200																							
Ave	0.006	6116.67	5333.33	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Count	6.00	6.00	6.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

SECONDARY EFFLUENT

Values that are excluded by entering a * in front of value

Questionable data based on analysis date
Listed as No Data (ND)

SECONDARY EFFLUENT																												
Day	Flow	TSS	VSS	COD	sCOD(0.45 micron)	dCOD(GF)	ffCOD	TBOD5	NO3 at en	CBOD5	dCBOD (GF)	TKN	sTKN (0.45 micron)	dTKN (GF)	NH3-N	NO3-N	NO2-N	TP	sTP (0.45 micron)	dTP (GF)	PO4-P	Alk	pH	Dissolved	Dissolved	Temp°		
Daily																												
Spot																												
Diurnal																												
30-Nov		4.3		44								1.7			0.22	3.9		3.4				53	6.8				
2-Dec		3.7		45	16	32	16	<3		<2		1.4	1.10		0.19	5.7	<0.01	4.1				45	6.8				
3-Dec		3		30	19		19	<3		<2		1.9			0.80	9.0		3.6				44	6.8				
7-Dec		4.7		40	21	29	21	<3		<2		1.4	0.86		0.26	5.8	<0.01	3.1				53	7				
9-Dec		5				26	23	<3		<2		1.3	1.0		0.29	7.4	<0.01	3.4				47	6.7				
10-Dec		4.3		23			17	<3		<2		1.6			0.32	7.9		4.7				52	6.9				
Ave	0.00	4.17	#DIV/0!	39.40	18.67	29.67	19.20	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	1.55	0.99	#DIV/0!	0.35	7.08	#DIV/0!	3.72	#DIV/0!	#DIV/0!	#DIV/0!	49.00	6.87	#DIV/0!	#DIV/0!	#DIV/0!		
Count	5.00	6.00	0.00	5.00	3.00	3.00	5.00	0.00	0.00	0.00	0.00	6.00	3.00	0.00	6.00	6.00	0.00	6.00	0.00	0.00	0.00	6.00	6.00	0.00	0.00	0.00		

Recycle Streams

[redacted] Values that are excluded by entering a * in front of value

Primary Sludge

Day	Flow	TSS	VSS	cCOD	sCOD(0.45 micron)	dCOD(GF)	fXCOD	TBODS	NO3 at end of TBOD test	CBODS	dCBOD (GF)	TKN	sTKN (0.45 micron)	dTKN (GF)	NH3-N	NO3-N	NO2-N	TP	sTP (0.45 micron)	dTP (GF)	PO4-P	Alk	pH	Dissolved Oxygen*	Dissolved Sulfide	Temp*
Daily	[redacted]																									
Spot																										
Diurnal																										
30-Nov	0.031	20000	19000															150								
3-Dec	0.031	30000	29000															250								
3-Dec	0.031	23000	22000															190								
7-Dec	0.031	28000	27000															180								
8-Dec	0.031	[redacted]																								
10-Dec	0.031	15000	14000															120								
Ave	0.031	23000.00	21400.00	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	178.00	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Count	5.00	5.00	5.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	5.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

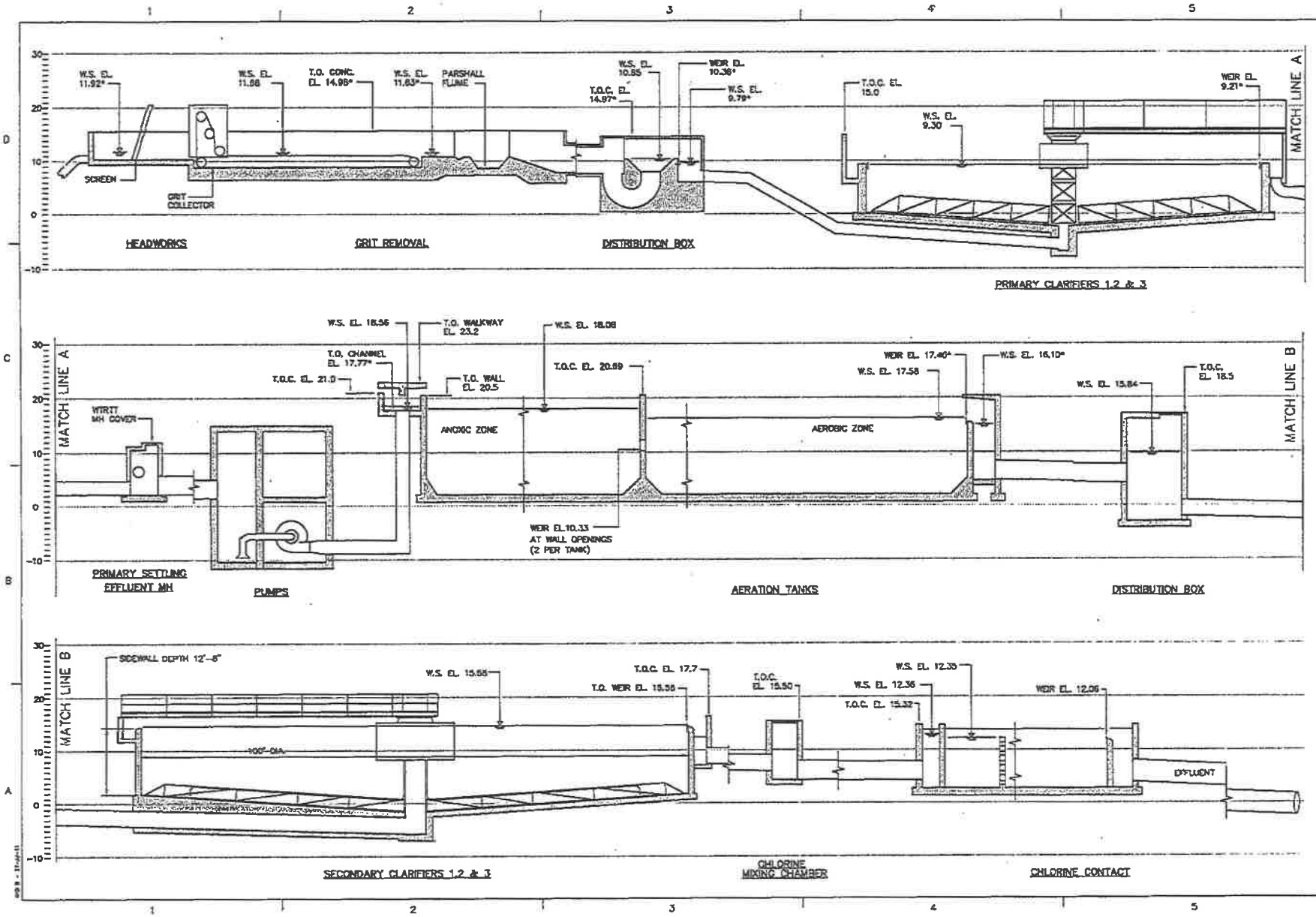
Belt Press Supernatant

Day	Flow	TSS	VSS	cCOD	sCOD(0.45 micron)	dCOD(GF)	fXCOD	TBODS	NO3 at end of TBOD test	CBODS	dCBOD (GF)	TKN	sTKN (0.45 micron)	dTKN (GF)	NH3-N	NO3-N	NO2-N	TP	sTP (0.45 micron)	dTP (GF)	PO4-P	Alk	pH	Dissolved Oxygen*	Dissolved Sulfide	Temp*
Daily	[redacted]																									
Spot																										
Diurnal																										
30-Nov		1200	1000	2100						790		120										160	6.2			
2-Dec		1900	1700	3560		810				840		120			44	0.03						200	6.3			
3-Dec		2400	2100	3200						630		100			46.00							190	6.3			
7-Dec		2000	1764	2700		400				1200		87			32.00	<0.01						180	6.3			
8-Dec		1400	1200	[redacted]						630		110			39.90	0.06						170	6.5			
10-Dec		900	810	1600						510		90										170	6.8			
Ave	#DIV/0!	1633.33	1449.00	2432.50	#DIV/0!	480.00	#DIV/0!	#DIV/0!	#DIV/0!	780.00	#DIV/0!	115.00	#DIV/0!	#DIV/0!	40.30	0.05	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	6.40	#DIV/0!	#DIV/0!	#DIV/0!
Count	0.00	6.00	6.00	5.00	0.00	2.00	0.00	0.00	0.00	6.00	0.00	6.00	0.00	0.00	4.00	2.00	0.00	0.00	0.00	0.00	0.00	6.00	6.00	0.00	0.00	0.00

Settage

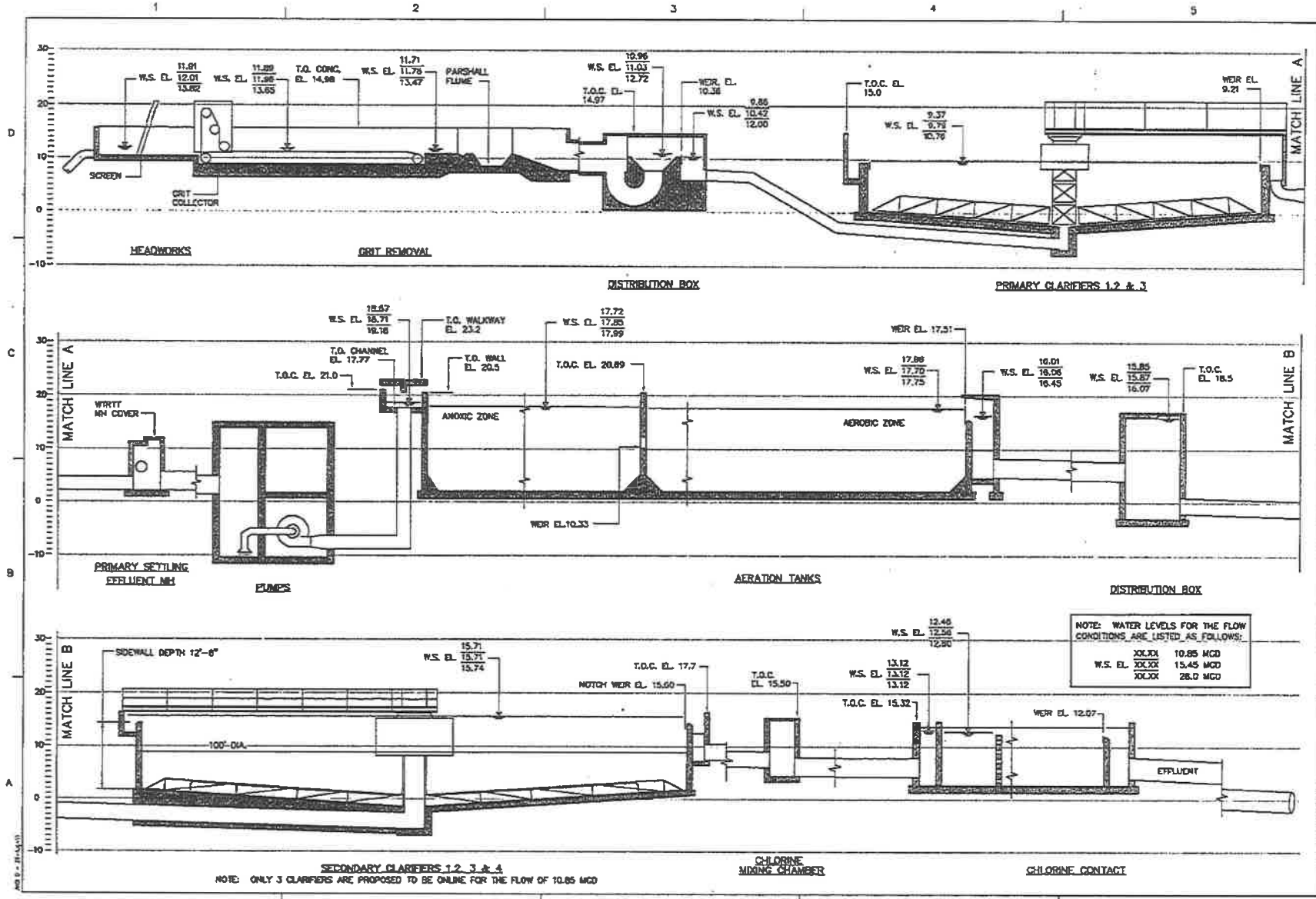
Day	Flow	TSS	VSS	cCOD	sCOD(0.45 micron)	dCOD(GF)	fXCOD	TBODS	NO3 at end of TBOD test	CBODS	dCBOD (GF)	TKN	sTKN (0.45 micron)	dTKN (GF)	NH3-N	NO3-N	NO2-N	TP	sTP (0.45 micron)	dTP (GF)	PO4-P	Alk	pH	Dissolved Oxygen*	Dissolved Sulfide	Temp*
Daily	[redacted]																									
Spot																										
Diurnal																										
3-Dec	0.0153	>400	3200	5700		400				1300		160			46			37			19	310	6.50			
3-Dec	0.008	3400	2700	8800		200				960		150			44.00			31			11	270	6.47			
7-Dec	0.01335	2600	2365	>400		340				1160		92			53.00			37			16	410	6.48*			
Ave	0.01	3000.00	2755.33	6550.33	#DIV/0!	420.00	#DIV/0!	#DIV/0!	#DIV/0!	1140.00	#DIV/0!	122.67	#DIV/0!	#DIV/0!	51.00	#DIV/0!	#DIV/0!	35.00	#DIV/0!	#DIV/0!	14.67	330.00	6.51	#DIV/0!	#DIV/0!	#DIV/0!
Count	3.00	3.00	3.00	2.00	0.00	3.00	0.00	0.00	0.00	3.00	0.00	3.00	0.00	0.00	3.00	0.00	0.00	3.00	0.00	0.00	3.00	3.00	3.00	0.00	0.00	0.00

Appendix B
Hydraulic Profile



CITY OF NEW LONDON, CT
WASTEWATER TREATMENT FACILITY
HYDRAULIC PROFILE
FOR A FORWARD FLOW OF
8.8 MGD

FIG. 1A



AECOM

CITY OF NEW LONDON, CT
WASTEWATER TREATMENT FACILITY
HYDRAULIC PROFILE
FOR FORWARD FLOWS OF
10.85, 15.45, AND 28.0 MGD

FIG. 1

Exhibit 9

SLR International Corporation
99 Realty Drive, Cheshire, Connecticut, 06410

August 6, 2025

Mr. Glenn Russo
Landmark Development Group
P.O. Box 660
Middletown, CT 06457

SLR Project No.: 141.051079.00001

Client Reference No.: 14845

RE: East Lyme Wastewater Flow Allocations
Riverview Heights
East Lyme, Connecticut

Dear Mr. Russo,

Pursuant to your request, we have evaluated the regular meeting minutes of the East Lyme Board of Selectmen dated June 24, 2025. Exhibit A, Sewer Capacity Allocations – May 2025 Update summarizes the sewer flows and current list of sewer allocations. Excluding state properties, East Lyme has an average daily flow capacity allocation of 1,022,000 gallons per day (gpd) and an average daily flow over the past 2 years of 770,000 gpd, which yields an average daily flow available of 252,000 gpd. With the previously approved allocation for Landmark Development Group of 118,400 gpd and five prior approved projects under construction, there is an available capacity of 110,746 gpd.

The total allocation for East Lyme (1,022,000 gpd) and state properties (478,000 gpd) is 1,500,000 gpd. Flow data for the combination of East Lyme and the state properties was provided from the town. The average daily flow for 2023-2024 was 988,507 gpd, so subtracting the East Lyme flow of 770,000 gpd for the same period yields a state properties average daily flow of 218,507 gpd. That leaves an available capacity of 259,493 gpd in the state properties flow allocation. Adding this to the above flow of 110,476 gpd yields an available unused capacity of 369,969 gpd of the total combined 1,500,000 gpd allocation, or 25 percent available capacity.

Let us know if you have any questions or would like to discuss this data further.

Regards,

SLR International Corporation

Thomas A. Knowlton, PE
Principal Water & Wastewater Engineer
tknowlton@slrconsulting.com

Attachments

141.14845.00004.au625.ltr-2

East Lyme Sewer Department
Sewer Capacity Allocations - May 2025 Update

Exhibit A
all figures in gallons

	Average Daily Flow Capacity Allocation					1,022,000
	Average Daily Flow - 2 Year Average					770,000
	Average Daily Flow Remaining - 2 Year Average					**252,000**
A	**Applicant/Development**	Type of Use	Additional Project Description		Sewer Capacity Requested or Need Anticipated	Sewer Capacity Allocated and Anticipated
1	Landmark Dev. Group	Residential	Apartments		118,400	118,400
	Subtotal Gallons Per Day (Group A)					**118,400**
B	**Prior Approved Projects Under Construction (>5,000 gpd and greater)**			Original Capacity Requested	Construction Completion to date %	
1	Village Crossing (4/2025 Update)	Residential	Condominiums	14,400	75%	3,600
2	Orchards Subdivision (4/2025 Update)	Residential	Single Family	42,600	90%	4,260
3	183-185 Main St (ZDM, not started yet)	Residential	Condominiums	3,600	0%	3,600
4	Brookside Apartments (4/2025 Update)	Residential	Apartments	35,400	90%	3,540
5	Parkers Place LLC (not started yet)	Residential	Apartments	8124	0%	8,124
	Subtotal GPD (Group B)					**23,124**
	Total Sewer Capacity Approved to Date (A+B)					**141,524**
C	**Estimated Sewer Capacity Needs Based on Previously Assessed Properties Not Presently Connected (Last Updated 2/2023)**					
1	Existing Buildings Assessed but not Connected	Res/ Comm/ Ind	Sewer		76,300	76,300
2	Vacant Properties in Assessed Areas	Res/ Comm/ Ind	Sewer		60,700	60,700
	Subtotal GPD (Group C)					**137,000**
D	**Total Sewer Capacity Allocated, Anticipated, and Requested (A+B+C+D)**					**278,524**
	Average Daily Flow Remaining 2 Year Average					**252,000**
	Sewer Capacity Remaining					**-26,524**

Exhibit A

Board of Selectmen
JUL 02 '25
Agenda Item #20

EAST LYME SEWER FLOWS - HISTORY

	2015	2016	2017	2018	2019	2020	2021	2022	2023	2024	AVG. Prev. Years	2025	% +/- AVG. Prev. Years	Precip. 2025 (In.)
JAN.	787,646	747,284	784,837	781,519	1,090,311	849,497	938,302	942,646	1,029,157	1,177,819	912,902	832,968	-8.8%	1.45
FEB.	832,681	809,701	765,648	865,263	842,611	859,175	911,422	988,646	997,413	912,457	878,502	836,250	-4.8%	3.88
MAR.	1,017,280	790,851	777,452	927,771	893,805	832,803	886,441	948,873	984,116	1,048,941	910,833	875,581	-3.9%	4.72
APR.	938,861	796,611	897,161	778,780	918,456	885,983	962,591	965,456	1,015,438	1,066,788	922,613	912,157	-1.15%	3.68
MAY	913,816	777,446	872,268	746,049	947,042	900,485	951,501	922,857	1,061,763	989,756	908,298	1,001,494	9.31%	8.74
JUN.	880,190	815,281	849,504	906,535	875,000	882,463	976,981	989,299	984,241	966,607	912,610			
JUL.	1,048,427	879,952	883,851	1,026,307	977,552	853,930	1,047,771	995,433	1,086,674	991,582	979,148			
AUG.	977,543	868,636	873,017	905,718	932,181	911,419	978,158	1,000,871	1,063,381	955,027	946,595			
SEPT.	878,563	762,544	769,493	875,918	833,237	823,590	1,051,008	921,227	1,020,678	851,600	878,786			
OCT.	861,521	738,247	752,273	903,915	806,576	812,506	917,384	905,482	1,053,620	813,935	856,546			
NOV.	803,842	709,481	732,848	871,111	815,129	786,482	937,414	864,223	954,365	787,600	826,250			
DEC.	788,121	728,649	728,437	894,050	927,335	896,694	895,121	950,524	1,057,605	853,600	872,014			
AVG.	894,041	785,390	807,232	873,578	904,936	857,919	954,508	949,628	1,025,704	951,309	900,425	891,690	-1.9%	4.49
													Precip. Total	22.47