

Dr. Robert Bagoff- Chair
West Orange Planning Board
66 Main Street
West Orange, NJ 07052

March 10, 2025

RE: PB-24-01 West Essex Highlands, Inc
Block: 179 Lot: 32 Zone: IH-1 Warner Road
West Orange Township, Essex County, New Jersey

Dear Dr. Bagoff,

At the request of attorney Bruce Afran, counsel for objectors and We Care NJ Corp., Princeton Hydro has performed an independent technical review of the aforementioned development located in West Orange Township, NJ. The project is also the subject of a Planning Board Approval from the West Orange Planning Board.

Documents central to the contents of this letter include but are not limited to the following:

- Subsurface Investigation Report Subsurface Stormwater Management System West Orange, New Jersey for Garden Homes Short Hills, New Jersey Prepared by Sor Consulting Engineers, Inc. 98 Sand Park Road, Cedar Grove, NJ 07009, Report Number 23-C-17 May 23, 2023, Job No. 23-C-16
- Subsurface Investigation Report Subsurface Stormwater Management System West Orange, New Jersey for Garden Homes Short Hills, New Jersey Prepared by Sor Consulting Engineers, Inc. 98 Sand Park Road, Cedar Grove, NJ 07009, Report Number 21-C-21 May April 19, 2021, Job No. 21-C-01
- Subsurface Investigation Report Subsurface Stormwater Management System West Orange, New Jersey for Garden Homes Short Hills, New Jersey Prepared by Sor Consulting Engineers, Inc. 98 Sand Park Road, Cedar Grove, NJ 07009, Report Number 24-C-11 April 4, 2024, Job No. 24-C-02
- Subsurface Investigation Report Subsurface Stormwater Management System West Orange, New Jersey for Garden Homes Short Hills, New Jersey Prepared by Sor Consulting Engineers, Inc. 98 Sand Park Road, Cedar Grove, NJ 07009, Report Number 24-C-02 February 12, 2024, Job No. 24-C-02
- Plan titled, "Preliminary and Final Site Plans West Essex Highlands Block 179 Lot 32, Block 179.17 Lots 8 and 9 Township of West Orange Essex County, New Jersey, Overall Drainage Area Map" prepared by Paul W. Anderson, PE of Anderson Consulting Services LLC., dated September 23, 2022, last revised October 28, 2024.
- Drainage Report, "Stormwater Management Report (formerly Drainage Report) West Essex Highlands Block 179, Lot 32, Township of West Orange, Essex County, New Jersey," prepared for West Essex Highlands Inc., prepared by Anderson Consulting Services LLC., dated March 12, 2023, last revised October 28, 2024.

It is our professional opinion that the application contains critical deficiencies and errors related to stormwater management design. For this reason, the application should not be approved at this time.





1.0 INTRODUCTION

The application entails the proposed construction of multi-family residential development with recreational facilities and a clubhouse for residents of the facility with a limit of disturbance of 1,233,793 sf (28.32 acres). The 120-acre parcel is currently undeveloped. The project has not received any Municipal but has received DEP approval in October 2024.

The purpose of this letter is to briefly summarize deficiencies that we have observed in the revised application materials related to critical aspects of the application's compliance with the New Jersey Stormwater Management Rules at N.J.A.C. 7:8 and the Township's local stormwater ordinance, §25-29 of the general ordinances.

2.0 STORMWATER MANAGEMENT

The site plans and stormwater calculations have been reviewed to determine if the design engineer has demonstrated compliance with N.J.A.C. 7:8, the New Jersey Stormwater BMP Manual and §25-29 of the local general ordinances. We find the application fails to demonstrate it meets the requirements as it relates to the soil testing requirements (Chapter 12 of the New Jersey Department Best Management Practices Manual, BMP Manual) and the two of the three primary stormwater management performance metrics, groundwater recharge (section P of §25-29) and stormwater runoff quantity standards (section R of §25-29). The following subsections outline our specific concerns as they relate to these three areas with the application submission. It is our professional opinion that the applicant has failed to demonstrate compliance with the requirements of N.J.A.C. 7:8.

2.1 SOIL TESTING REQUIREMENTS

Before discussing the issues of compliance with two of the three primary stormwater management performance metrics, this section will provide a breakdown of the soil testing requirements as stated in Chapter 12 of the BMP Manual and compare it to the soil testing provided in this application. The BMP manual was developed to provide examples of ways to meet stormwater design standards set by the state. The BMP Manual was developed by the New Jersey Department of Environmental Protection, in coordination with the New Jersey Department of Agriculture, the New Jersey Department of Community Affairs, the New Jersey Department of Transportation, municipal engineers, county engineers, consulting firms, contractors, and environmental organizations. As stated in Chapter 12 of the BMP Manual, understanding the character and saturated conductivity of surface and subsurface soils at a proposed land development site is crucial to the design of stormwater best management practices (BMPs) that meet the requirements of the New Jersey Stormwater Management rules at N.J.A.C. 7:8 and §25-29 of the local general ordinances. Section 2 of Chapter 12 further clarifies that saturated soil hydraulic conductivity and depth to Seasonal-High Water Table (SHWT) are of high importance for the design of stormwater BMPs. Subsection 1d of Chapter 12 provides guidelines for determining the upper limit of the zone of saturation which is known as the SHWT. The chapter states there are two methods for determining the SHWT:

1. When mottling showing the redoximorphic features is observed at any season of the year, the SHWT shall be taken at the highest level at which mottling is observed except when the water table is observed at a higher elevation.
2. When mottling showing redoximorphic features is not observed the SHWT shall be determined by water level elevation observed in the soil pit or boring when the testing took place from January through April.



If the work was done at other times, the depth to SHWT can be obtained from the NRCS Web Soil Survey.

Mottling is defined as contrasting or “blotchy” color patterns within a dominant soil color. It forms as the SHWT rises into oxygenated soils, changing the conditions in the soils from aerobic to anoxic resulting in a gray “blotches”, see Figure 1 below. The coloring of the soil is caused when the water table rises, creating anaerobic (oxygen-depleted) conditions. Bacteria that thrive in anaerobic conditions use the oxygen in iron and manganese oxides, changing the color of the minerals. When the water table drops, the dissolved minerals are deposited on the soil particles. The deposited minerals appear as coatings on the soil particles in colors such as yellow, red, orange, brown, blue, or black. Areas that have been saturated for long periods can lose all of their iron and manganese oxides, turning gray. The mottling will show the highest elevation that SHWT has occurred historically. This process takes years to occur and is considered more accurate than the water elevation because it is not impacted by recent weather conditions.



Figure 1: Example of Mottling in Soil

While reviewing the multiple subsurface investigation reports for this project, the soil profiles never make any mention of mottling. That is very unusual, as stated above the BMP Manual states that mottling should be used to established using mottling with the observed water level. Importantly, soil logs must also be specific in their identification of various color throughout the profile, as is consistent with the *Field Indicators of Hydric Soils in the United States* referenced by the NJDEP BMP Manual. Color identification is key in correctly identifying mottling by recording both the number of different colors observed, as well as the value and chroma of the observed colors. Figure 2 provides a comparison of a sample soil profile which uses the Munsell method per the *Field Indicators of Hydric Soils* (top) and the soil profiles for this project (bottom).

It appears that the SHWT was only established using the elevation of the water found in the test pit or boring and no effort was made to document mottling. As noted in the NJDEP BMP Manual, there are instances where mottling can be absent from a soil profile, but it is highly unusual that mottling was not found in any of the soil test pits or borings provided by the applicant. Mottling is a natural and common process, thus it is fair to assume mottling should have been observed in some of the soil profiles. The concern is that the groundwater level found in a test pit can fluctuate on a daily basis based on the recent weather. Using the mottling to determine the SHWT is recommended because it estimates the highest (most conservative) elevation that does not vary with time. If one only uses the observed groundwater elevation, it is safe to assume that the SHWT elevation is probably underestimated. It is recommended that the professionals who documented these test pits should provide the Planning Board with testimony to explain why mottling was never documented and discuss the how their reports could be underestimating the elevation of SHWT.



The NJDEP BMP Manual requires that BMPs maintain a minimum vertical difference of 2 feet from the bottom of the BMP to the SHWT elevation for BMPs without underdrains. The BMPs in this project do not have underdrains. BMPs 2, 6 and 7 maintain that minimum separation. If the SHWT elevations are higher than what was recorded in this plan, then there is a risk these designs are not compliant with the NJDEP BMP Manual. The manual requires these depths as a precaution to prevent groundwater from mounding into the storage area of the basins during rain storms. If the groundwater mounding encroaches on the storage area, it will increase the stormwater runoff from the BMP and decrease the amount of groundwater recharge the BMP provides. N.J.A.C. 7:8 5.4 (§25-29.4 P) sets the groundwater recharge requirements and 7:8 5.6 (§25-29.4 R) sets the minimum reductions in the peak flow rates for runoff exiting a site. If the SHWT is higher in BMPs 2, 6 or 7 that would impact the site's compliance with those two standards.

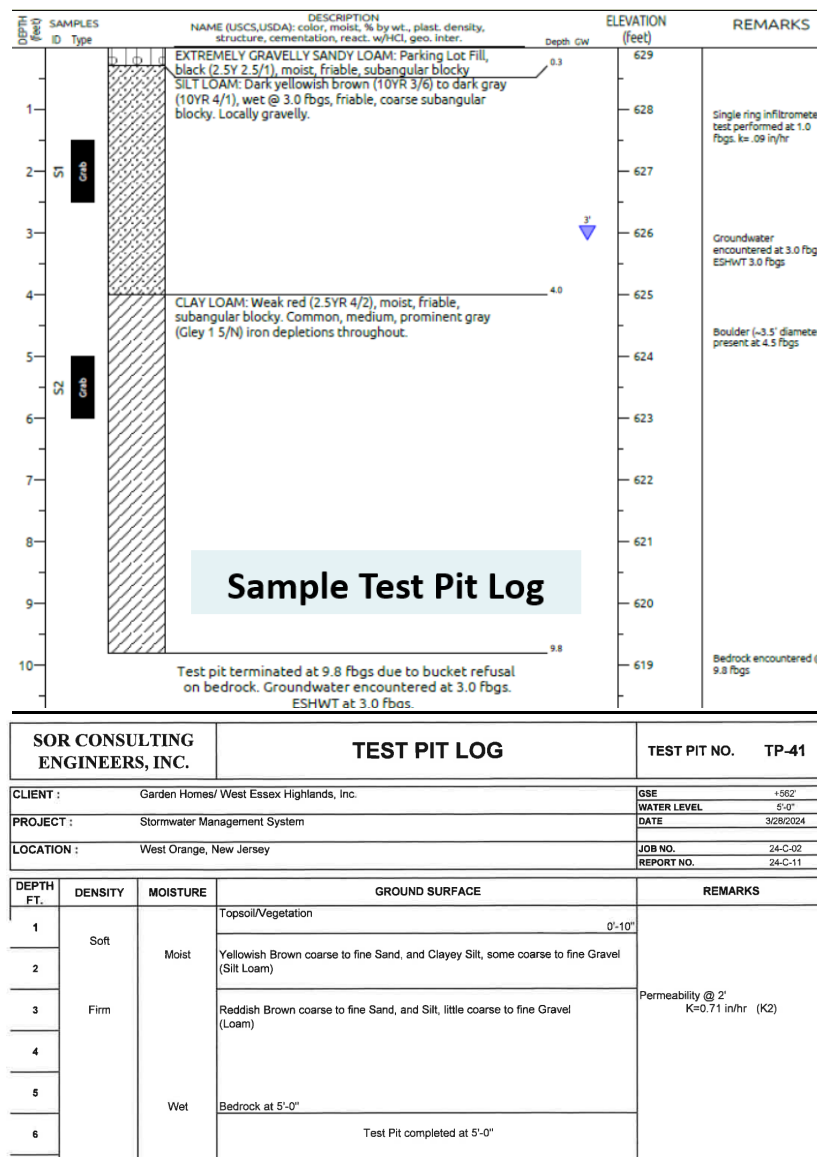


Figure 2: Example of Soil Profiles that use the Munsell Method to document SHWT and the Soil Profiles for this project



2.2 BMP DESIGN STANDARDS AND SHWT

Princeton Hydro reviewed the SHWT elevations documented in the subsurface investigations against the proposed BMP designs to confirm that each of the proposed BMPs met the minimum two vertical feet of separate from the bottom of the basin to the SHWT. Princeton Hydro found that the following BMPs did not meet that minimum standard:

- BMP 6 (Porous Pavement)
- BMP 4 (Porous Pavement)
- BMP 9 (Aboveground basin)

BMP 6 shows a SHWT of 598 ft using the 2024 soil testing data but 2021 shows a different story. TP 4 shows seepage at 608 ft, which is 10 feet higher than what the plans show. Based on this finding, the porous pavement design not compliant. The SHWT intersects the storage volume of the basin as shown in Figure 3.

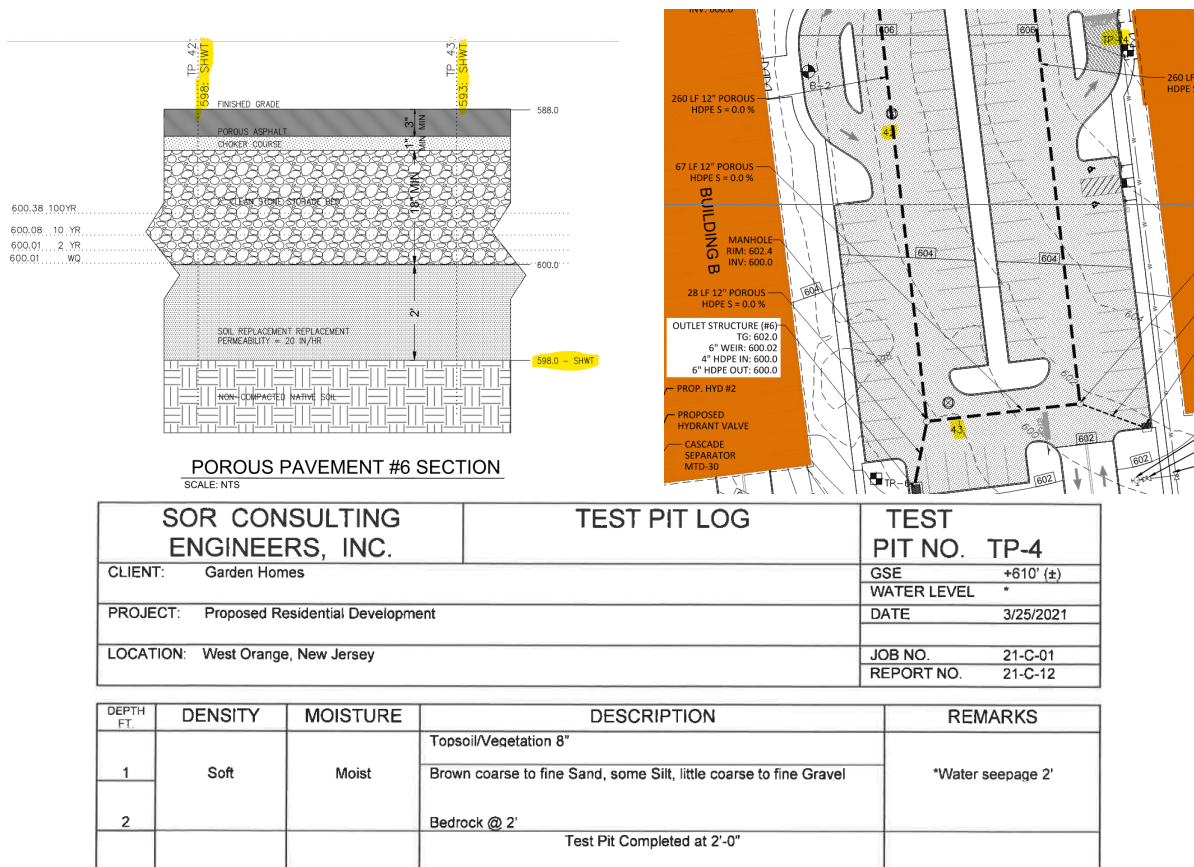


Figure 3: Cross Section of PP6, Proximity of TP-4 to PP6 and TP-4 Soil Profile

Likewise, BMP 4 records the SHWT at 567 ft (Figure 4) on the plans but that is only using data from the soil tests taken in 2024 (TP 28). In 2021, soil tests were also taken and TP 10 (Figure 4) is very close to BMP 4. TP 10 shows a SHWT of 573 ft, 6 feet higher than TP 28 which would have the SHWT intersecting the storage bed of the BMP. Please note that there is an error on the TP-10 soil profile. On the right-hand side of the document, there is an entry for the water level found; it is left blank. Elsewhere on the document it states that seepage was found at 3



feet below grade which is an elevation of 573 ft (576 ft - 3 fbg). Seepage is water leaking through the side walls of the soil profile, the seepage is an indicator of the SHWT and it should have been marked that way.

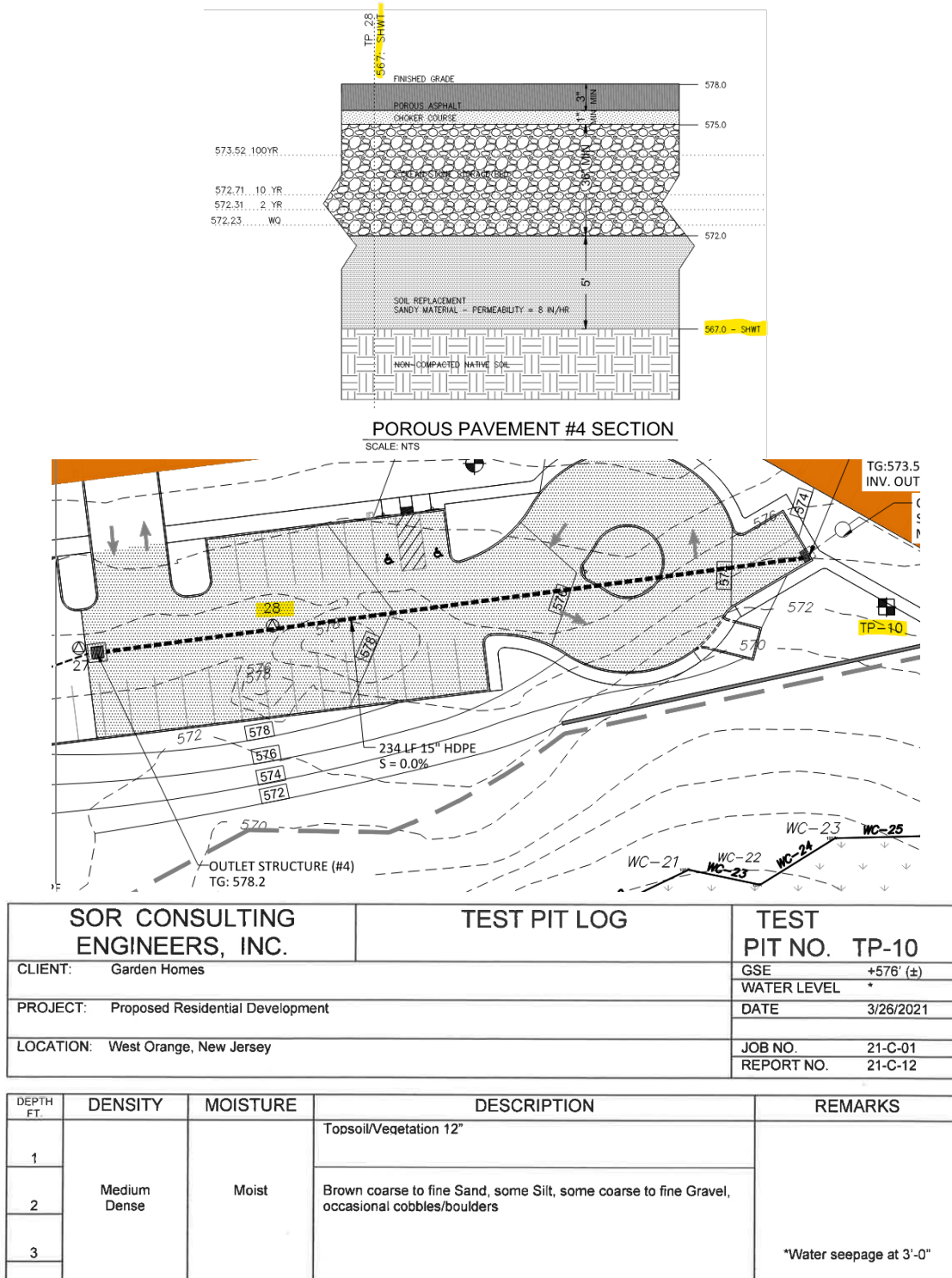
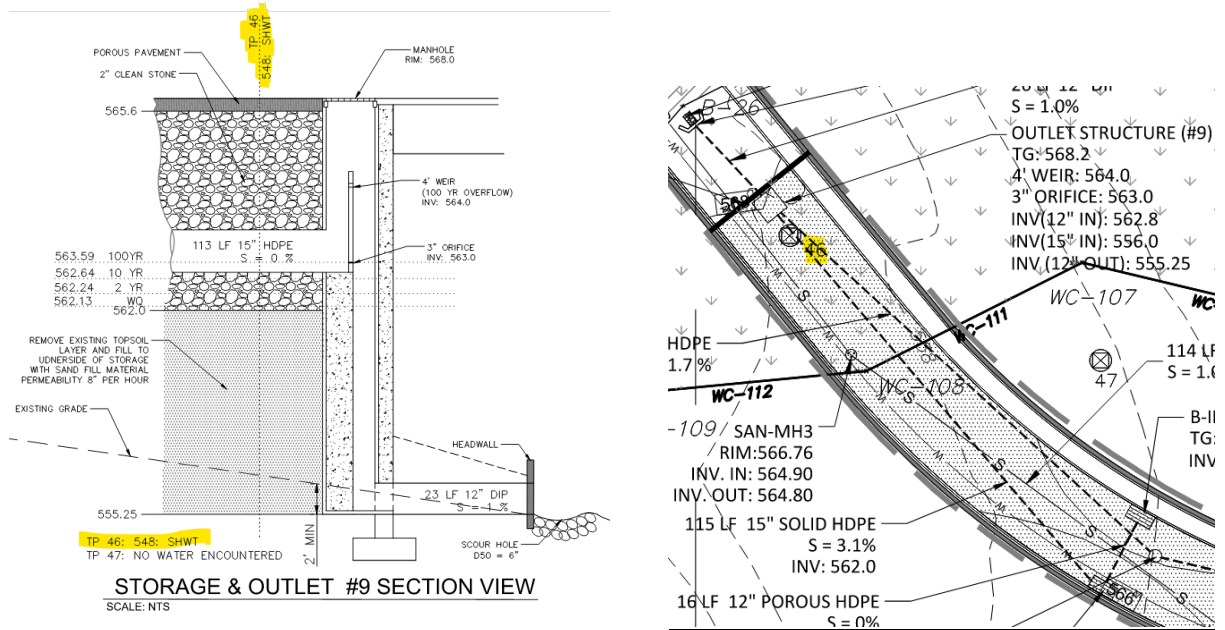


Figure 4: Cross Section of PP4 from Construction Plans, Proximity of TP-10 to PP4, TP-10 Soil Profile



Finally, there is an error with BMP 9. The construction detail states that TP 46 has a SHWT at 548 ft, but the soil profile states the ground surface elevation is 574 ft and the depth to SHWT is 8 ft, which would locate the SHWT at 566 ft. Elevation 566 ft is much higher than 548 and would require the design to be significantly altered. See Figure 5 below for references to the plans and soil profiles.



SOR CONSULTING ENGINEERS, INC.		TEST PIT LOG		TEST PIT NO.	TP-46
CLIENT :		Garden Homes/ West Essex Highlands, Inc.		GSE	+574'
PROJECT :		Stormwater Management System		WATER LEVEL	8'-0"
LOCATION :		West Orange, New Jersey		DATE	3/28/2024
				JOB NO.	24-C-02
				REPORT NO.	24-C-11
DEPTH FT.	DENSITY	MOISTURE	GROUND SURFACE	REMARKS	
1	Soft	Moist	Topsoil/Vegetation	1'-3"	
2			Gray to Reddish Brown Clayey Silt, little coarse to fine Sand, little coarse to fine Gravel (Clay Loam)		
3					
4	Stiff		Light Brown Clayey Silt, little coarse to fine Sand, kluittle coarse to fine Gravel (Clay Loam)	4'-0"	
5					
6			Reddish Brown coarse to fine Sand, some Silt, and medium to fine Gravel (Sandy Loam)		
7				Permeability @ 6' K=0.99 in/hr (K2)	

Figure 5: Cross section at PP9, Location of TP-46, TP-46 Soil Profile

These findings show that BMPs 4, 6 and 9 are not compliant with NJDEP BMP Manual. The SHWT is too high and would occupy a portion of the storage volume of each of the BMPs. This would result in less groundwater recharge than what is calculated in the stormwater report and higher peak flow rates from each BMP, which



may also affect water quantity compliance. As such, the BMPs need to be revised. These revisions could require the BMPs to be widened, shortened, or relocated in general. The bottom of the BMPs need to maintain a minimum separation of two feet to prevent groundwater from mounding into the storage system, reducing the system's storage capacity and increasing the peak flow rate from each BMP.

2.3 SOIL TESTING REQUIREMENTS FOR BMPs

The BMP Manual provides guidance on how many test pits are required based on the size of the basin in Subsection 2a of Chapter 12 of the NJDEP BMP Manual. The soil testing allows a designer to understand how the soil will respond to rainfall, determine the elevation of the SHWT, and measure the infiltration rate of the soil at the elevation where infiltration will occur. It is a crucial part of the design process.

Typically, each BMP is required to have two test pits, but more can be required based on how large the proposed BMP is. The SHWT can vary based on its location and it's important to take measurements at different locations to better understand that variation. If an insufficient number of soil tests are taken, the engineer runs the risk of not understanding the elevation of the SHWT across the BMP and could design a BMP that is below the SHWT. A good example of this is PP 4 and 6. If you look at only some of the test pit data, the BMP seems properly designed. However, when considering the 2021 data, you can see how widely the SHWT varies by location.

Princeton Hydro compared the number of test pits for each BMP to the number required by the NJDEP Manual to confirm the applicant performed the minimum number of soil tests. Princeton Hydro compared the infiltration area of each BMP on the plan to what was entered into the model. Table 1 is a comparison of the infiltration area found in the model calculations and the infiltration area shown on the plan. The infiltration area is the bottom of the basin, not the side slopes.

Table 1: Comparison of Infiltration Areas Shown in the Plan and Model

BMP ID- BMP Type	Infiltration Area, Model (SF)	Infiltration Area, Plan (SF)	Difference, Model – Plan (SF)
1 – Large Scale Infiltration Basin	42,000	45,490	-3,490
2 – Small Scale Infiltration Basin	26,023	25,830	193
3 – Porous Pavement	5,227	5,558	-331
4 – Porous Pavement	12,197	12,369	-172
5 – Porous Pavement	14,810	14,611	199
6 – Porous Pavement	35,719	37,473	-1,754
7 – Small Scale Infiltration Basin	14,375	13,405	970
8 – Porous Pavement	10,019	15,900	-5,881
9 – Porous Pavement	6,534	17,981	-11,447
10 – Small Scale Infiltration Basin	9,583	9,467	116
11 – Porous Pavement	13,504	22,826	-9,322
12 – Small Scale Infiltration Basin	900	1,004	-104

As demonstrated, there are differences between what was modeled and what was shown on the plan. A majority of these differences are not consequential, but the differences at BMP 1, 8, 9 and 11 are much more significant as the infiltration areas as modeled are much larger than the final BMP footprints as proposed on the plans. The modeling must be revised accurately analyze the proposed conditions.

Table 2 provides a comparison of the number of test pits required by the NJDEP BMP Manual against those performed by the applicant. The BMP footprint as shown on the plan was used as the reference to determine the required number of test pits.



Table 2: Number of Soil Tests Performed for each BMP against the Minimum Number Required.

BMP ID	Infiltration Area, Plan (SF)	Number of Tests Provided	Number of Test Required	Compliant (Y/N)
1	45,490	12	6	Y
2	25,830	4	3	Y
3	5,558	2	2	Y
4	12,369	3	3	Y
5	14,611	3	3	Y
6	37,473	4	5	N
7	13,405	1	3	N
8	15,900	2	3	N
9	17,981	3	3	Y
10	9,467	1	2	N
11	22,826	4	4	Y
12	1,004	2	2	Y

If you look at the plan, it will appear that BMP 7 has more soil tests than is shown on the table above. While the applicant did perform more testing than what is recorded above, it was performed at the wrong time of the year to establish SHWT with the encountered groundwater elevation alone. Mottling to establish the SHWT is a reliable method that may be performed at any time of the year, but the encountered groundwater elevation can only be used to determine SHWT if the work was done between January and April per the BMP Manual. The test pits labeled with letters (not numbers) were performed after April and cannot be counted towards compliance.

BMPs 6, 8, and 10 are short one soil test and BMPs 7 is short two test pits. 76,425 sf of the 221,914 sf of infiltration areas are not compliant with local ordinance. Though this may seem minor, but the SHWT can vary greatly as exemplified by BMP 6 and therefore it's important to do enough testing to confirm the design conforms with standards. This is a vulnerability in the design that shows how the project is potentially not conforming to N.J.A.C. 7:8 5.4 (§25-29.4 P) 7:8 5.6 (§25-29.4 R) for groundwater recharge and stormwater quantity control standards.

2.4 GROUNDWATER MOUNDING CALCULATIONS

The soil testing showed very low infiltration rates for most of the site. The applicant proposes to replace the existing soil from the bottom of the BMPs to the SHWT with a sandy soil that has much higher infiltration rate to help meet the requirement that all basins must empty in 72 hours. This is a commonly used practice that is accepted by the NJDEP, which works well if infiltration is not used in the stormwater runoff routing calculations. However, the applicant did use infiltration in the runoff routing calculations which means that the model assumes some of the runoff is infiltrated and does not travel through the stormwater piping network. When one uses infiltration in their runoff routing calculations, then one must provide groundwater mounding calculations for the water quality and 100 year storm. During a rain storm, the groundwater will temporarily rise (mound) below the BMPs that infiltrate, thus calculations are necessary to confirm the mounding does not encroach on the storage volume of the BMP and impact its ability to perform as designed. The applicant provided calculations for both scenarios. Unfortunately, the groundwater mounding analysis for the 100 year storm has errors in it and upon further investigation, the groundwater mounding will reach into the storage volume for the following BMPs:

- BMP 3
- BMP 4
- BMP10



Princeton Hydro shall use PP3 as an example of the analysis that was performed on each of the BMPs to determine the mounding issue under the 100 year storm scenario.

Princeton Hydro observed that the duration of infiltration in the groundwater mounding analysis for the water quality design storm (WQDS) and the 100 year storm were nearly identical for both storms (0.21 hours for water quality and 0.2 for 100 year). The analysis for the water quality storm can be found on page 821 of 946 in the Drainage Report and the analysis for the 100 year storm on page 831 of 946 in the Drainage Report. The duration of these two storms should be very different from one another. The water quality storm is only 1.25 inches whereas the 100-year storm is 8.33 inches of rain. The 100 year storm rainfall total is 6.64 times more than that of the WQDS; the sheer difference in volume would make it nearly impossible to have identical durations.

It was also noted that the recharge rate is also different for both storms, which should be constant across any storm. The design proposes to replace the existing soil from the bottom of the BMP to the SHWT with a sandy soil that has a higher infiltration rate to allow the runoff to infiltrate quicker through the BMP storage area. The soil at the SHWT still has a very low infiltration rate so the "infiltrated runoff" from the BMP will back up in the replacement soil while it infiltrates below the elevation of the SHWT at a much slower rate. Typically, the stormwater report would provide calculations demonstrating that there was enough storage in the replacement soil to retain the "infiltrated runoff" to allow it to infiltrate below the SHWT at a much slower rate than would be acceptable during the storm event. The calculations would show that the "infiltrated runoff" would be held in the replacement soil and not mound into the storage volume of the BMP. If it did mound into the storage volume, it would impact the runoff peak flow rate calculations and the groundwater recharge calculations by reducing the amount of volume available to hold runoff in the BMP.

During the water quality storm, BMP 3 infiltrates 880 cf of runoff. The area of the infiltration basin is 5,558 sf. 880 cf of water is equivalent to about 2 inches of water over the area of the basin. That means that two vertical inches of water will be held above the SHWT and slowly infiltrate. Assuming a void space factor of 0.4 for the sand layer, the 2 inches of vertical water realistically equates to about 5 inches, meaning that there is 5 inches of water in the sand layer between the SHWT and the bottom of the storage for PP3. The system works under this scenario because the soil replacement area is 2 feet deep. Under the 100 year storm, the model assumes that 10,971 cf of water will infiltrate through the system (page 529 of 946 in Drainage Report). That is because the model is assuming the runoff is infiltrating through the soil replacement area at a constant rate of 10 inches per hour. The reality is the at the surface of the basin, the runoff will infiltrate at that rate but once it reaches the SHWT, it will infiltrate at the much slower rate of 0.25 inches/hour (assuming a factor of safety of 2, based on the soil testing results) and will cause water to back up into the sand layer. There needs to be thick enough layer of sand to prevent that runoff from backing up into the storage area of the BMP 3. 10,971 cf of water over the area of the basin is equal to 1.97 ft of water. As noted above when you apply the 1.97 feet of water to the void space ratio of sand (0.4), the height of the sand and the water is 4.93 feet (59.16 inches) from the SHWT. BMP 3 only provides 2 feet of sand storage and an additional 26 inches of stone storage (see Figure 6). The system will become overwhelmed during a 100 year storm and the peak flow rates will not be reduced. Instead of infiltrating, a large portion of the 10,971 cf that the model says will infiltrate will discharge through the stormwater system. The design will fail and potentially create higher peak flow runoff rates than currently exist. Furthermore, this issue will compound elsewhere on the site. When BMP 3 is overwhelmed, the excess flow will be captured by inlets further downstream, potentially exceeding their capacity, thus resulting in uncontrolled discharge from the site and risking flooding downstream. If the excess runoff is captured by the inlets, the runoff would then enter either BMP 1 or 2, and similarly could overwhelm those systems and increase the peak flow rates from those BMPs as well.

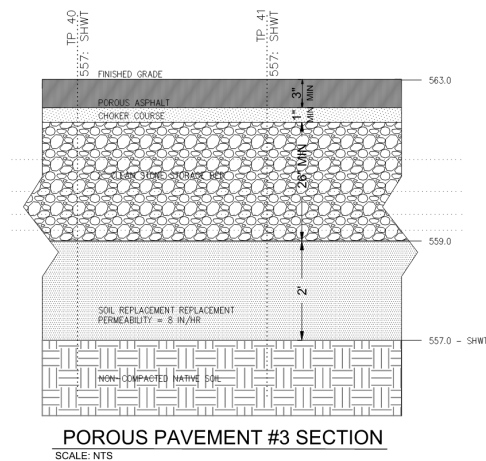


Figure 6: Cross Section of BMP 3.

2.5 IMPACTS TO WETLANDS

In the existing conditions, the stormwater runoff generated from the existing topography travels over the surface and reaches the upgradient wetlands, providing valuable water to the wetland ecosystem. In the proposed condition, the discharge locations for each of the drainage areas are at the bottom of the project area, as illustrated in Figure 7. The blue arrows indicate the existing flow paths and the red arrows indicate proposed. The proposed stormwater system intends to route the stormwater runoff through a pipe network to down-gradient basins where all of the surface runoff will discharge much further downstream. The wetlands uphill of the discharge locations will therefore collect less stormwater runoff due to their reduced contributory drainage areas.

Princeton Hydro included two attachments that show the existing drainage areas for eastern and western wetlands. The western wetlands have a drainage area of approximately 32 acres. The proposed improvements to this area will reduce the drainage area by 9 acres, or 28% of the total existing drainage area. The eastern wetlands have a drainage area of 48.4 acres. The proposed improvements to this area will reduce the drainage area by 16 acres, or 33% of the total existing drainage area.

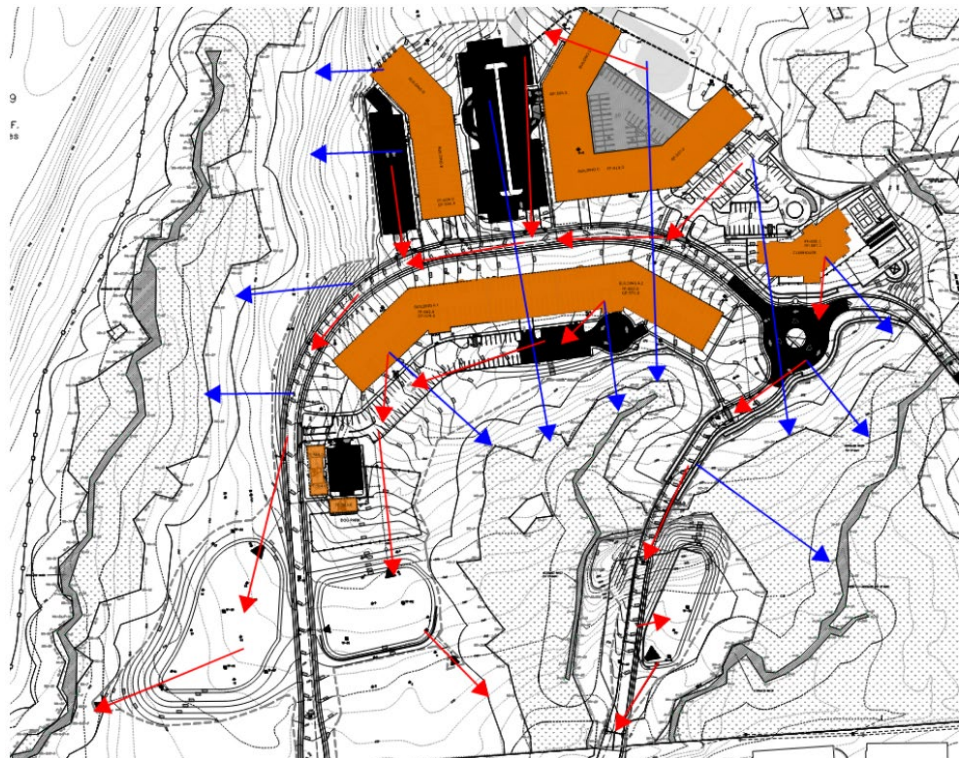


Figure 7: Proposed Grading Plan with Blue Arrows showing the existing drainage path and the red arrows showing the proposed drainage paths and how the runoff is being diverted from the wetlands.



2.6 PEAK FLOW CALCULATIONS

Based on our review of the existing conditions mapping and modeling, we believe the time of concentration computations were done incorrectly for this project. The time of concentration (TC) is the time it takes for runoff to travel through the watershed on the most hydraulically restrictive path. Imagine taking a bucket of water and pouring it out at the highest part of the drainage area. The time it takes for that water to travel to the discharge location at the bottom of the drainage area is the time of concentration. It appears that the designer used an old method to determine the TC for drainage areas POA 1 and 2. Due to the very small size of the drainage area for POA 3, it was not included in this analysis. There is a clue that an older method was used because the plans describe the land use for DA-1 as "unpaved," which is an older land use classification that is no longer used. Princeton Hydro would note that the NJDEP ruling on this project means that the latest (2023) Stormwater Rule update does not apply. However, the TC calculation methodology referenced above was updated with the previous rule change in 2021.

TC for this system was broken down into two different flows: sheet flow and shallow concentrated flow. Sheet flow is how runoff initially travels overland. Sheet flow typically occurs at depths less than or equal to 0.1 ft. Runoff transitions from sheet flow to shallow concentrated flow. Shallow concentrated flow occurs after sheet flow and the depths range from 0.1 to 0.5 ft. To demonstrate the magnitude of how incorrect TC calculations are, Princeton Hydro recalculated the shallow concentrated flow portion of the TC for both POA 1 and 2.

For POA 1, the designer stated that it took 7.2 minutes for the runoff travel 1,625 ft via shallow concentrated flow. The average slope for that length is 5.5%. According to chapter 5 of the BMP manual, the velocity of runoff traveling through a forested area with heavy ground litter is 0.55 ft/s when the slope is 5.5% (Figure 5-13 of NJDEP BMP Manual Chapter 5). 1,625 ft divided by 0.55 f/s is equal to 2954 seconds or 49 minutes. The revised TC is almost 7 times longer than the what is shown on the stormwater report.

For POA 2, the report states that it takes 1.9 minutes to travel 564 ft via shallow concentrated flow. The average slope for that length is 9.2%. That is an average velocity of 5 fps. The NJDEP manual states the velocity of runoff at a slope of 9.2% for wooded area is 0.75 fps (Figure 5-13 of NJDEP BMP Manual Chapter 5). Using that velocity, the TC is 12.5 minutes. Again, the revised TC is almost 7 times longer than what is shown on the stormwater report.

Establishing the existing TC and peak flow rates are crucial to a robust design. The TC has a significant impact on the existing peak flow rate for a proposed site. All development projects are required to have their peak flow rates leaving the site be less than the existing peak flow rate. The shorter the TC, the higher your peak flow rate is. The higher your peak flow rate is, the easier it is to meet your peak flow rate reduction requirements. Keeping the peak flow rate as high as possible allows you to reduce the size of your stormwater basins. The problem is that it doesn't reflect reality and instead of reducing the peak flow rate coming out of the project you could potentially be increasing the flow rate and risking the stormwater runoff from overwhelming the stormwater system downstream and causing a flooding event. That is why the miscalculation of the existing conditions TC is alarming. An easier way to think about this is lowering the height of a basketball net below the regulated height so you can dunk but still telling people you can dunk at regulation height.

The flow path shown on the drainage area map for POA 2 is only 664 feet long and that matches what is shown on the model. The TC is meant to measure the time it takes for the water to travel the entire flow path. The actual flow path for POA 2 is greater than 664 ft. The discharge location is approximately 700 feet away from the end of where the analysis ends (as the crow flies, see Figure 8). This analysis is incorrect and incomplete because it does not capture the entire length of the flow path.

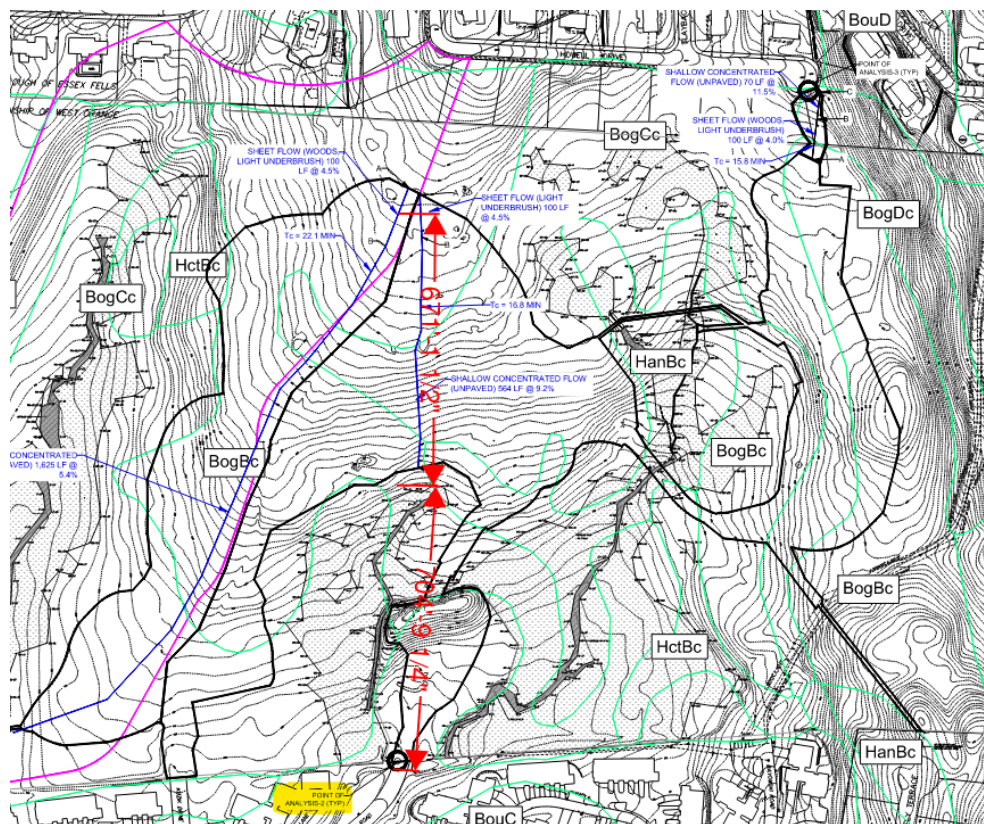


Figure 8: Existing Conditions Drainage Path for POA-2, demonstrating how the analysis stops approximately 700 feet short of the POA location

This is similar to using the older TC calculation, it artificially shortens TC so it lessens the existing peak flow rates and potentially risks the proposed improvements increasing the flow rate leaving the site instead of decreasing.

To understand the impact that the TC errors have on the design, Princeton Hydro calculated the TC for POA 2 using the correct method and entire flow path. The report has a total TC of 16.8 minutes. The Princeton Hydro calculations show a TC of 60.8 minutes, approximately 3.5 times longer. The resulting 2-year peak flow using the TC as calculated by PH is 7.52 cfs, whereas the applicant's model states 14.76 cfs (page 22) and the report states 17.53 cfs (page 22). In addition to severely overestimating the existing peak outflows, the values as listed in the report do not match the flow rates in the model, which must be corrected. Note that Princeton Hydro evaluated the applicant's calculations against the model results not what is in the body report. Based on the corrected time of concentration calculations, the maximum allowable peak flow rate for the 2 year storm is 3.76 cfs. Currently, the design's peak flow rate is 7.24 cfs, almost twice what should be permitted.

Princeton Hydro calculated the existing peak flow rate for the 100 year storm as 37.7cfs using the corrected TC value, but the applicant's model results in an existing 100-year peak outflow of 72.79 cfs and a proposed peak outflow of 38.99 cfs. As such, the proposed site will actually generate a higher peak flow rate than what currently exists for the 100 year storm. This is particularly alarming, especially considering there are flood prone areas further downstream. To meet the peak flow rate requirements set forth by the NJDEP and the local ordinance would require the volume storage for the BMPs to dramatically increase. This would have a noticeable impact on the site layout in general.

The analysis of the existing drainage area for POA 2 has yet another flaw. The topography within the area delineated for POA 2 has two clear discharge locations in the proposed condition, but it only assumes discharge location at Kuzik Road (see Figure 9). The design maintains both discharge locations, but the modeling assumes they are combined. The flows do eventually combine approximately 600 ft downstream into the adjacent neighborhood, not at the boundary of the site as currently assumed by the model. The drainage area analysis should be redone and the drainage area should be split into two separate drainage areas with two separate points of analysis to ensure the peak flow rate at both locations are compliant. It is difficult to predict how this will impact the site design, but it could be substantial. This re-evaluation is necessary to ensure that the peak flow rate of runoff from the site is compliant with the state and local regulations and to prevent downstream flooding.

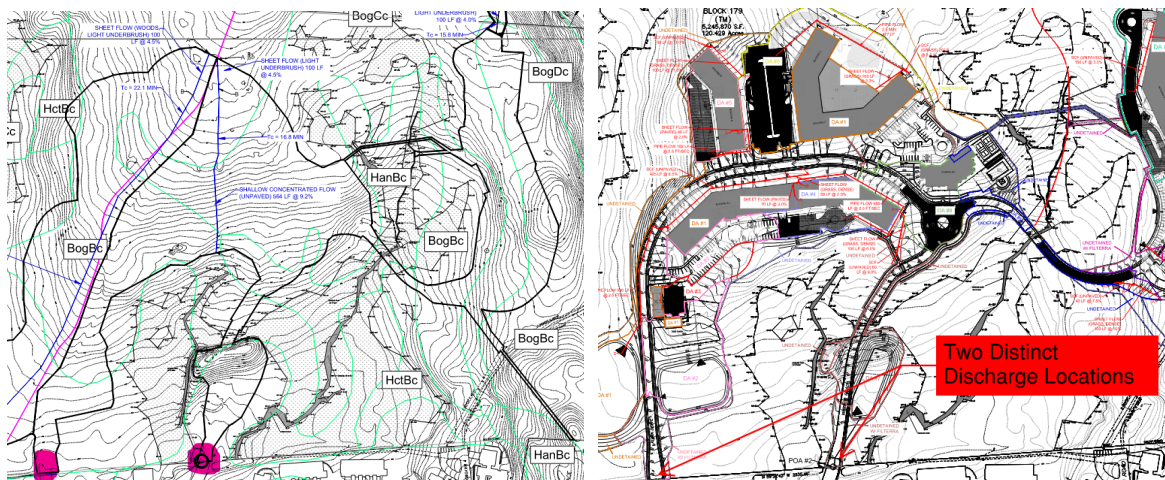


Figure 9: These are images of DA-1 and DA-2 from the Stormwater Report for the existing condition (left) and proposed condition (right) to demonstrate that the DA for POA 2 should be modeled as two separate DAs because there are two distinct points of discharge.

2.7 SEPARATION OF SANITARY INFRASTRUCTURE AND BMP

The design has a sanitary pipe traveling through porous pavement system (Figure 11). While Princeton Hydro could not find a specific rule against a sanitary line traveling through a subsurface basin, there are rules dictating adequate separation between stormwater piping and sanitary piping. Stormwater pipes and sanitary pipes should maintain a minimum horizontal distance of 10 ft and a minimum vertical distance of 1 ft, and the sanitary line should be below the stormwater pipe. The purpose of the separation is to prevent leaking sanitary pipes from entering the stormwater system and potentially causing an illicit discharge of raw wastewater. The design should be modified to relocate the sanitary pipe out of the subsurface system. This may require the project to have additional Flood Hazard and/or Wetlands Permitting depending on how the layout is impacted by this modification.

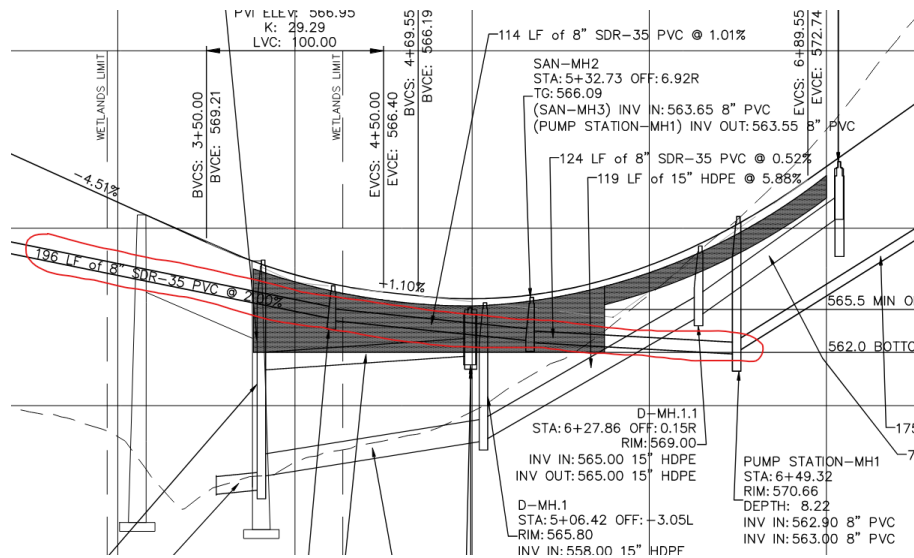


Figure 10: Profile of BMP 9 showing a sanitary line traveling through it.

3.0 SUMMARY

In conclusion, our detailed review has found that the application is not in compliance with applicable State and local stormwater requirements (§25-29). It is my professional opinion that the application does not meet all three central performance requirements of the local stormwater requirements.

Since the project has not demonstrated compliance with all three specific stormwater management requirements of the central goals of stormwater management planning as outlined in NJAC 7:8 and §25-29 of the local general ordinance, in my professional opinion, the application is non-approvable as designed. If the application is approved as designed, the development will increase the frequency and severity of damage expected during a flood in the adjacent and downstream areas.

I appreciate the opportunity to provide you with these engineering services for you. Please do not hesitate to contact me with any questions regarding this review.

Sincerely,

Sean Walsh, PE
 Senior Project Manager