**FINAL SUPPLEMENTAL ENVIRONMENTAL IMPACT STATEMENT FOR THE
CROTON WATER TREATMENT PLANT
AT THE MOSHOLU SITE**

6.15. WATER RESOURCES

6.15.1. Introduction

This section examines the existing and potential impacts of the proposed Croton Water Treatment Plant project (Croton project) on the existing surface water, stormwater runoff, and groundwater resources in the vicinity of the Mosholu Site. The Mosholu Site is located at the Mosholu Golf Course in Van Cortlandt Park, in the Bronx, New York. The methodology used to prepare this analysis is presented in Section 4.15, Data Collection and Impact Methodologies, Water Resources.
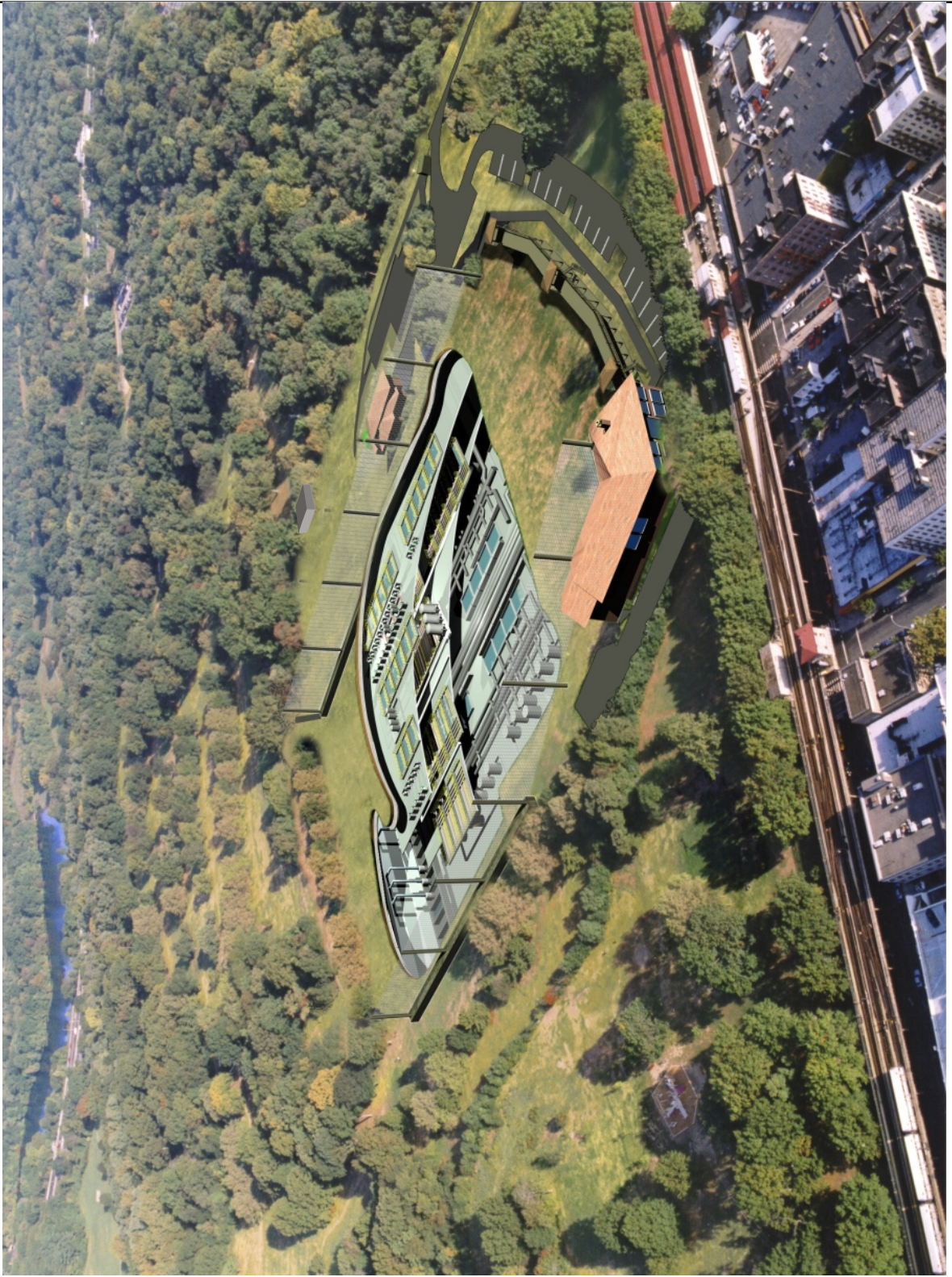
6.15.2. Baseline Conditions

6.15.2.1. *Existing Conditions*

A complete description of the land uses at the water treatment plant site and study area is presented in Section 6.2, Land Use, Zoning and Public Policy. The water treatment plant site for the proposed plant at the southeast corner of Van Cortlandt Park (Park) covers the existing driving range and clubhouse area and extends to a portion of the Shandler Recreation Area to the north (Figure 6.15-1 and Figure 6.15-2). The water treatment plant site slopes gently from west to east, from about Elevation 200 to Elevation 170 feet. The natural resources which could be affected by alterations in hydrological conditions include the small (0.30 acre) wetland adjacent to the northwestern side of the proposed footprint, the forested wetland north of the water treatment plant site access road in the Shandler Recreation Area, and trees which are outside the potential impact area but subject to potential changes in soil moisture.
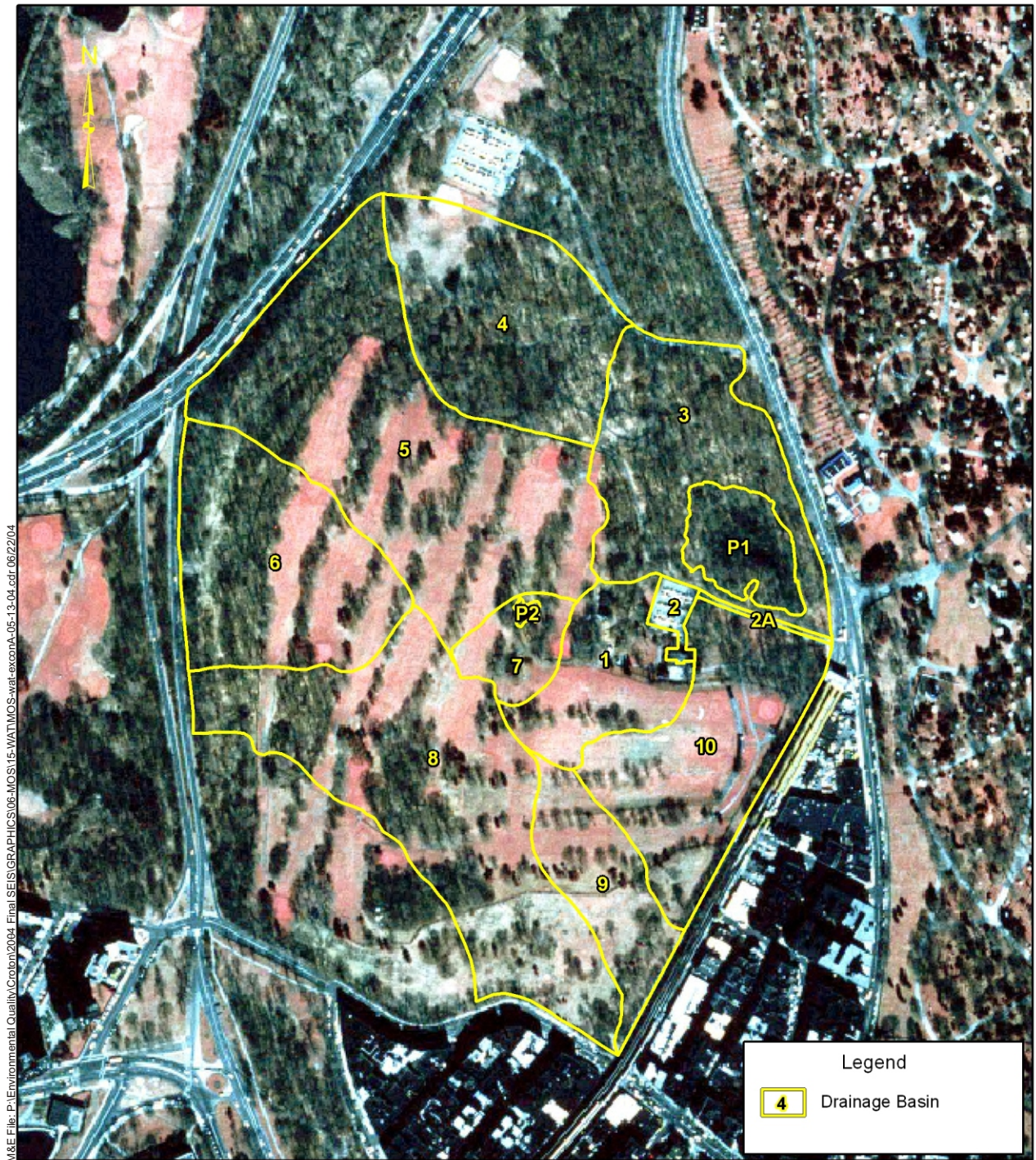
6.15.2.1.1.   *Surface Water*

There are no surface water bodies on the water treatment plant site. The following two wetlands were analyzed to determine if any potential changes to the groundwater elevations caused by the construction and operation of the proposed water treatment plant would result in impacts to these resources. Other wetlands farther away from the impact area were not quantitatively evaluated because it was determined in the preliminary screening that the impacts to the near by wetlands could be mitigated or were not significant. Information relating to these other wetlands is presented in Section 6.14, Natural Resources. A discussion on water quality was not included because surface water does not occur on this site, and hence no specific surface water applicable standards.

Small Wetland.   The small wetland is approximately 0.3 acres in size and is located adjacent to the area of potential construction (Figure 6.15-2). It receives runoff from the higher elevations to the south, west, and east. This isolated wetland is located in the lower portion of a very small basin that is in a relatively high topographic position on the golf course. The elevation of the wetland is above the groundwater table. Groundwater in this vicinity is estimated to be between elevations 180 and 185, whereas the lowest elevation of the wetland is approximately 188 (Figure 6.15-2). Once surface water in the wetland reaches approximately Elevation 190, the water drains slowly out of the wetland and flows to the northwest.

Aerial View of the Mosholu Site

Figure 6.15-1

N

4

3

5

6

P1

P2

2

2A

7

1

8

10

9

Legend

4   Drainage Basin

0   300   600   1200
Feet

SCALE IN FEET

**Stormwater Drainage Basins
Mosholu Site**

Croton Water Treatment Plant

Figure 6.15-2

Forested Wetland. The forested wetland is approximately 5 acres in size with 2.7 acres of standing water, and is located in the forest north of the existing golf course driveway in the Chandler Recreation Area (Figure 6.15-2). A footpath to the east, parallel to Jerome Avenue, and another footpath bound it on the west. A prominent hill lies outside the northern edge of the wetlands. There is a northwest finger of the forested wetland that extends along a drainage channel. This small, rock-lined channel drains into the northwest corner of the main part of the forested wetland. This channel drains from a small shallow depression at an elevation of about 178 feet that is dominated by small wetland saplings, shrubs, and herbaceous plants. The southern edge of the forested wetland is in the shape of a horseshoe, with the open end of the horseshoe directed toward the access road that leads from Jerome Avenue to the existing parking lot for the Mosholu Golf Course.

The lowest elevation of the wetland is approximately Elevation 164.6 feet, and the groundwater elevation in this area is estimated to be at approximately 165 based on the geotechnical borings completed at the golf course (Figure 6.15-2). Therefore, it is likely that groundwater is the primary hydrologic factor supporting this wetland. The maximum elevation of surface water in the wetland is limited to Elevation 164.6 ft by a drainage ditch that empties into an 18-inch combined sewer line through a concrete headwall.

6.15.2.1.2.   *Stormwater Runoff*

Model Description. Existing stormwater runoff at the water treatment plant site was simulated using HydroCAD® Version 7 stormwater modeling software.[1] The model provides hydrograph generation and routing based on the Soil Conservation Service (SCS, now known as Natural Resources Conservation Service) TR-20 procedures. In order to model stormwater flows, the drainage area of the water treatment plant site was divided into eleven sub-catchments or basins (Figure.5.15-2). Stormwater runoff volumes and rates were estimated for basins tributary to both the forested and small wetlands.

Soils are classified into hydrologic soil groups (HSGs) to indicate the minimum rate of infiltration obtained for bare soils after prolonged wetting. The HSGs, which are A, B, C, and D, are one element used in determining runoff curve numbers (CN). Based on the recently completed geotechnical boring program, all on- and off-site soils were considered to be Type B. These soils have moderate infiltration rates when thoroughly wetted, and consist chiefly of moderately well-to-well drained soils with moderately fine to moderately coarse texture. These soils have a moderate rate of water transmission (0.15-0.30 in/hr).[2] Infiltration and runoff rates were calculated using runoff curve numbers for each cover type, as provided in the Soil Conservation Service Technical Release 55 (SCS TR-55) and reprinted in Appendix A of the model manual[3]. For portions of the watershed outside the golf driving range, conservative assumptions of soil type were made based on the geologic history of the area.

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[1] Applied Microcomputer Systems (AMS), 2001. HydroCAD Stormwater Modeling System. Owners Manual, Version 7. Chocorua, New Hampshire.
[2] AMS, 2001.
[3] AMS, 2001

The on-site watershed draining the water treatment plant under existing conditions was delineated based on March 1996 topographic survey data identifying two-foot contour intervals. The two-foot contours were first converted into a digital elevation model, and then a Geographical Information System (GIS) (ArcView 3-D Analyst) was used to delineate drainage areas, and determine slopes and hydraulic lengths (i.e. the longest distance stormwater runoff will travel in each basin). In addition to the overall area of each basin, the area of various cover types in each basin was determined with GIS in order to facilitate an assessment of infiltration and runoff rates for each cover type. For the existing conditions model, the acreages of wetland, wooded, grass, brush, and impervious areas were approximated using the GIS database. It should be noted that for the purposes of the stormwater analysis, wetland acreages from the GIS database were measured based on the topography as determined from the 2-foot topographic map. Therefore, the wetland acreages used for the model input do not necessarily correspond to the wetland acreages reported in project impacts below, which represent state and federal regulated wetland acreages based on the presence of hydrophytic vegetation and hydric soils in addition to wetland hydrology.

The forested wetland in the Shandler Recreation Area, located immediately north of the existing golf course access road, is a sensitive resource; therefore, the basin contributing to this wetland was assessed. This basin was divided into three sub-basins based on hydrologic drainage patterns. One of the three sub-basins contributing to the forested wetland (Basin 1) includes three catchbasins near the existing clubhouse that collect runoff from portions of the driving range and fairways. Based on field inspection, the flow from these catchbasins is directed to the east, toward Jerome Avenue. Field measurements of the catchbasins and the pipes discharging from each basin indicated that each of these three catchbasins has a capacity of approximately one cubic feet per second (cfs). Therefore, the HydroCAD® model was configured to divert flow exceeding 1 cfs overland to the forested wetland. The outlet of the forested wetland was modeled as an orifice, the dimensions of which were identified as 20-inches wide by 12-inches high based on field measurements. A fourth catchbasin is located in the northeast corner of the existing clubhouse parking lot (Basin 2). Runoff is directed to the forested wetland through an eight-inch pipe. Runoff exceeding the capacity of the catchbasin exits the site via the curbed access road (Basin 2A) and discharges to the combined sewer along Jerome Avenue. Although there is a drainage channel discharging from the wetland into the orifice, sensitivity analyses with the HydroCAD® model indicated that this channel is overwhelmed during storms and that the downstream orifice is instead the controlling factor.

The small wetland was a second area of concern. This isolated wetland is located in the lower portion of a very small basin that is in a relatively high topographic position on the golf course. The outlet of the small wetland was modeled as a culvert with an invert of 190 feet.

A Type III storm was used to model each of five storm events. This is the most common type of storm in the City and is typical of eastern coastal areas of the U.S. where large 24-hour rain events are typically associated with tropical storms. The 24-hour design storms that were modeled included the 3-month (1.5 inches), 2-year (3.5 inches), 5-year (4.5 inches), 10-year (5.1 inches), and 100-year (8.1 inches).[4] For each of the modeled storms existing peak flows and 24-
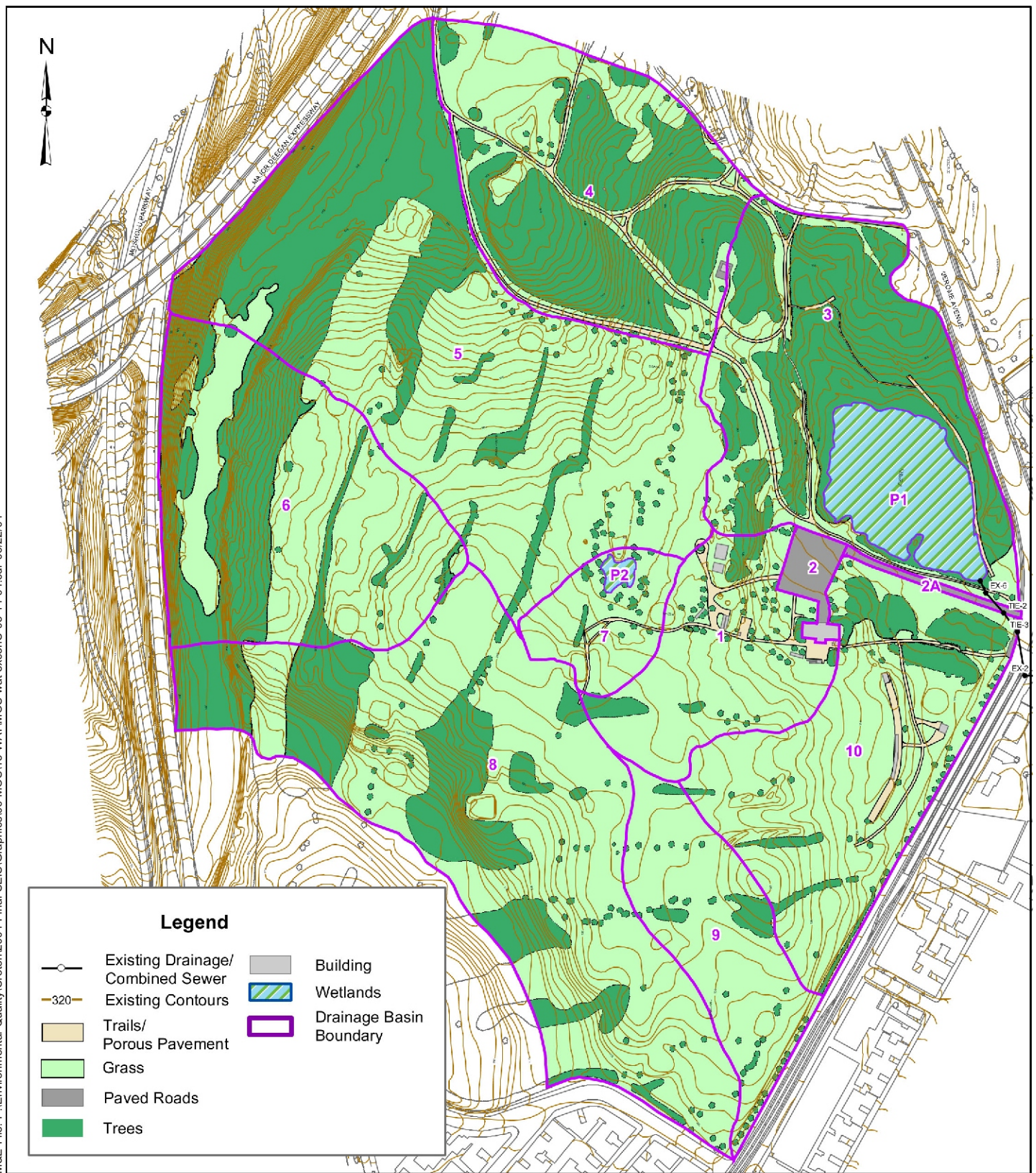
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[4] This rainfall data is from the U.S. Weather Bureau. 1961. Technical Paper No. 40-Rainfall Frequency Atlas of the United States (TP 40)

hour runoff volumes were estimated. Assessment of peak flow rates is important because an increase in peak flows could result in erosion along drainage paths in both upland and wetland areas. Although analysis of the 3-month storm is not required from a regulatory basis, this return period represents a storm event that will generally provide water at frequent enough intervals to support a surface water dependent wetland. The 2-year and 5-year storms were simulated to determine the existing peak flows and 24-hour runoff volumes under these conditions, which are parameters relevant to both wetlands and upland resource areas in the basins draining from the water treatment plant site. The 5-year design storm is also the standard design storm used by the NYCDEP to size infrastructure needed to dissipate peak flows and maintain existing 24-hour runoff volumes. The 10-year storm was analyzed for potential water resource impacts because this storm is anticipated to have a greater influence on the site natural resources. In order to comply with requirements associated with the NY State Pollutant Discharge Elimination System (SPDES) general permit for stormwater discharges from construction activity, the modeling effort also included assessment of the 100-year storm and the potential for downstream flooding.

Existing Stormwater Runoff. The HydroCAD® model used to predict storm flows is described in the Section 4.15, Data Collection and Impact Methodologies, Water Resources. Figure 6.15-3 illustrates the delineation of basins for existing conditions. The input parameters used to simulate existing stormwater flows in the basins are summarized in Table 6.15-1. Appendix G provides the complete model results for the existing conditions. Basin 1 discharges into catchbasins that flow to the combined sewer along Jerome Avenue. During large storms, the capacity of the catchbasins is overwhelmed and stormwater can flow overland into the forested wetland (P1). Basins 2 and 3 discharge into the forested wetland (P1) and basin 7 discharges into the small wetland (P2). Runoff from the remaining basins flows overland to the border of the project area. Basin 5 discharges to the Major Deegan Expressway, Basin 6 discharges to the Mosholu Parkway, Basin 8 discharges to the southwest border of the water treatment plant site, Basins 2A, 9 and 10 discharge to Jerome Avenue, and Basin 4 discharges to the northwest border of the water treatment plant site (Figure 6.15-3). Table 6.15-2 summarizes the peak runoff rate and total runoff volume for each basin for the 3-month, 2-year, 5-year, and 10-year storm events under existing conditions.

The modeling results indicate that the runoff volume entering the forested wetland during a 3-month storm event is approximately 0.1 acre-ft. As discussed above, this is generally the storm event most likely to be providing a regular source of surface water for the wetland. However, since only a very small amount of overland runoff reaches the wetland during the 3-month storm event, the HydroCAD® results confirm that in fact stormwater runoff is not the primary source of water supporting the hydrology of this wetland.

Legend

—○—	Existing Drainage/ Combined Sewer	�\[gray\]	Building
—320—	Existing Contours	▨\[hatched\]	Wetlands
▨\[tan\]	Trails/ Porous Pavement	▭\[purple\]	Drainage Basin Boundary
▩\[light green\]	Grass		
▩\[dark gray\]	Paved Roads		
▩\[dark green\]	Trees		

200 100 0   200   400   600   800

Scale in Feet

**Water Resources
Existing Conditions
Mosholu Site**

Croton Water Treatment Plant

Figure 6.15-3

TABLE 6.15-1. EXISTING CHARACTERISTICS FOR PROPOSED CROTON PROJECT MOSHOLU SITE - EXISTING CONDITIONS

Basin	Grass (Good, B)[1] Area (acres)	CN	Wetland Area (acres)	CN	Wooded Area (acres)	CN	Building/Paved Area (acres)	CN	Porous Pavement Area (acres)	CN	Composite Total Area (acres)	CN	Tc (min)
Basins Tributary to Forested Wetland													
1	5.77	61	-	-	0.54	55	0.09	98	0.41	85	6.8	62	20.1
2	-	-	-	-	-	-	0.80	98	-	-	0.8	98	7.4
3	3.17	61	2.45[a] 1.15[b]	98 55	8.6	55	-	-	0.80	85	16.2	64	20.2
Basin Tributary to Small Wetland													
7	2.73	61	0.15	98	-	-	-	-	0.08	85	3.0	64	10.4
Other Basins													
2A	-	-	-	-	-	-	0.25	98	-	-	0.25	98	20.6
4	5.48	61	-	-	7.44	55	0.03	98	0.69	85	13.6	59	25.2
5	16.29	61	-	-	10.39	55	0.01	98	0.01	85	26.7	59	26.9
6	8.09	61	-	-	5.94	55	-	-	-	-	14.0	58	20.6
8	16.13	61	-	-	7.02	55	-	-	-	-	23.2	59	19.5
9	5.77	61	-	-	0.38	55	-	-	-	-	6.2	61	35.9
10	10.62	61	-	-	1.73	55	0.09	98	0.45	85	12.9	61	23.8

CN = Curve Number, a factor describing the surface permeability; higher numbers are assigned to areas of lower permeability.

Tc = Time of Concentration, the time in minutes required for a particle of water to flow from the most hydrologically remote point in the watershed to the receiving area

1.) Grass (Good B) = grass cover > 75% and CN of 61

a.) Wetland, standing water, El. 164; b.) Wetland, El. 164 to 166

TABLE 6.15-2. EXISTING RUNOFF CHARACTERISTICS IN 3-MONTH, 2-YEAR, 5-YEAR, AND 10-YEAR STORMS AT THE MOSHOLU SITE

Basin	3-Month Storm		2-Year Storm		5-Year Storm		10-Year Storm	
	Peak Runoff Rate (cfs)	Runoff Volume (acre-ft)	Peak Runoff Rate (cfs)	Runoff Volume (acre-ft)	Peak Runoff Rate (cfs)	Runoff Volume (acre-ft)	Peak Runoff Rate (cfs)	Runoff Volume (acre-ft)
Basins Tributary to Forested Wetland								
1	0.0	0.01	2.5	0.35	5.4	0.64	7.4	0.85
2	1.0	0.09	2.5	0.22	3.3	0.28	3.7	0.32
3	0.1	0.03	7.2	0.95	14.6	1.70	19.6	2.21
Total		0.13		1.52		2.62		3.38
Basin Tributary to Small Wetland								
7	0.0	0.01	1.7	0.17	3.4	0.31	4.6	0.41
Other Basins								
2A	0.2	0.03	0.6	0.07	0.7	0.09	0.8	0.10
4	0.0	0.00	3.3	0.55	7.8	1.09	11.2	1.46
5	0.0	0.00	6.2	1.08	15.0	2.12	21.3	2.85
6	0.0	0.00	3.1	0.53	8.0	1.05	11.6	1.42
8	0.0	0.00	6.0	0.94	14.7	1.85	21.0	2.48
9	0.0	0.00	1.6	0.29	3.5	0.53	4.9	0.73
10	0.0	0.01	4.0	0.61	8.9	1.15	12.3	1.53

The model indicates that the small wetland receives essentially no runoff during the 3-month storm under existing conditions. This result is most likely due to the fact that the watershed contributing to the small wetland is extremely small (approximately 3.0 acres) and entirely covered with pervious material (grass and isolated trees). Therefore, no runoff occurs and rainwater instead infiltrates and supports a perched water table. During larger storms, small amounts of runoff from the contributing basin occur and contribute to standing water in the wetland.

For the other basins draining the water treatment plant site, the results indicate that virtually all stormwater is infiltrated during the 3-month storm, but that runoff occurs during the 2-year, 5-year, and 10-year storms. Peak runoff rates and total runoff volumes for Basins 4 through 10 are summarized in Table 6.15-2. Complete model output for the existing site is included in Appendix G. As required by the SPDES regulations, downstream analysis of the 100-year, 24-hour event including peak discharge rates, total runoff volumes, was also conducted to assess potential impacts to the wetlands P1 and P2.
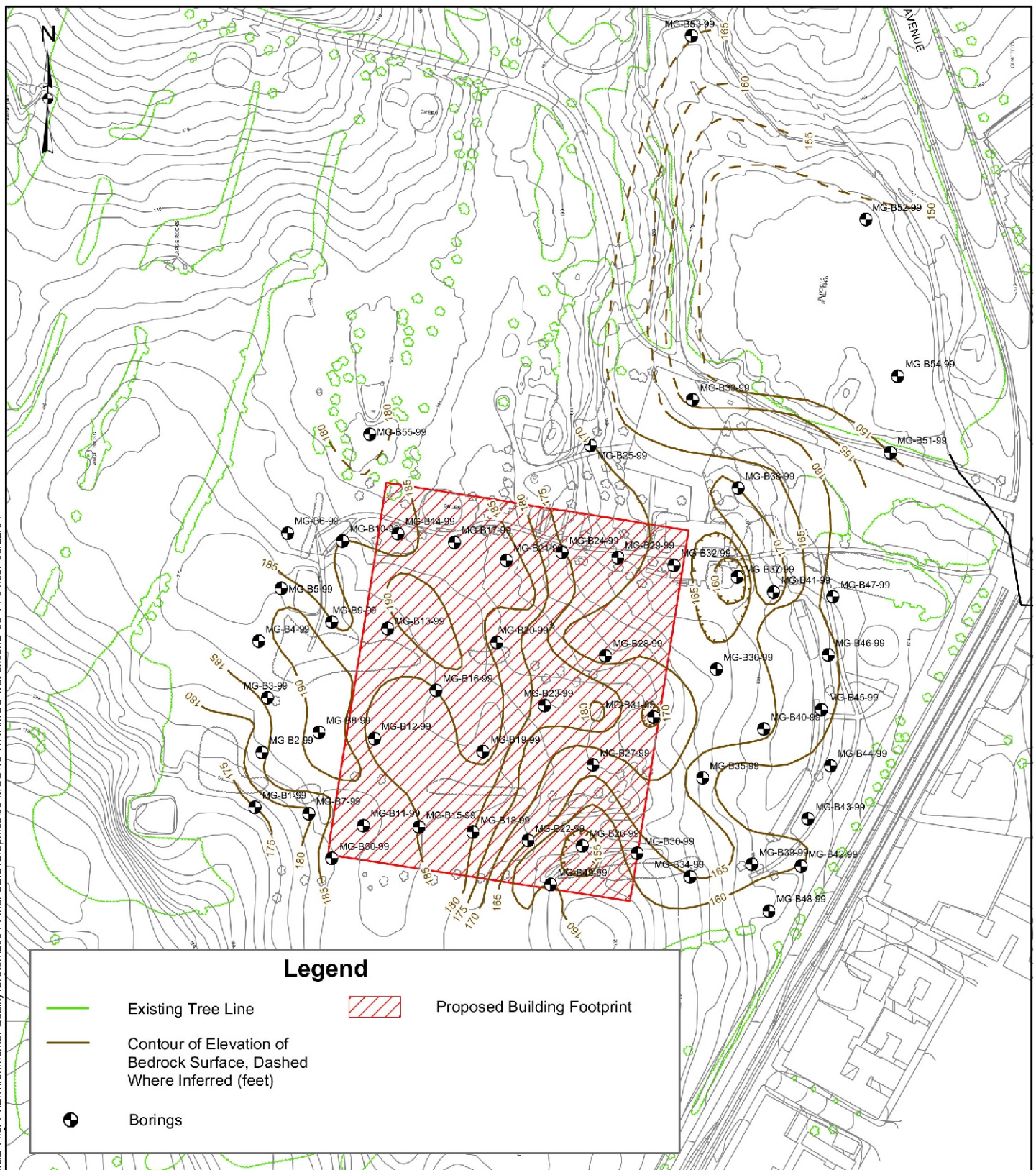
6.15.2.1.3. Groundwater

Groundwater in the study area occurs within the saturated portions of the overburden and bedrock. The groundwater beneath the study area originates exclusively as precipitation that falls directly on the study area and infiltrates.

Geology. Fifty geotechnical borings were drilled on a grid superimposed on the proposed footprint to support the design of the proposed structure. Another 35 holes were drilled within this area with an AirTrack drill rig to determine the top of bedrock between the borings. Four borings were drilled to obtain subsurface data on the soils in the vicinity of the forested wetland, and a fifth was drilled to provide the same information at the small wetland. A top of rock contour map was generated from these data (Figure 6.15-4). The bedrock surface is highest (about 190 feet) along the western side of the building footprint and slopes downward to elevations of about 150 to 155 feet beneath the forested wetland and along Jerome Avenue.

The water treatment plant site is generally underlain by a layer of unconsolidated deposits comprised of silty fine to medium sand with occasional gravel, identified as glacial till. Based on the conditions encountered during the geotechnical investigation, the till at the water treatment plant site is generally 15 feet to 20 feet thick, with thicknesses of less than five feet occurring locally.

The till is underlain by crystalline (i.e., of metamorphic and igneous origin) bedrock classified as gneiss and locally identified as the Fordham Gneiss. This bedrock is characterized by its dark and light "banded" appearance. The gneiss underlying the water treatment plant site exhibits evidence of fracturing (identified in outcrops as being typically subvertically to vertically inclined), and weathering. The weathering typically occurs along the contact between the overlying till and the bedrock surface, as well as along certain fracture surfaces. Weathering of the upper bedrock surface at the water treatment plant site, as determined from the geotechnical investigation borings, was observed to extend vertically downward as much as 10 to 15 feet, before grading to more competent bedrock.

Contour Map of Bedrock Surface Elevations Mosholu Site

Legend

Existing Tree Line	Proposed Building Footprint
Contour of Elevation of Bedrock Surface, Dashed Where Inferred (feet)	
Borings	

0   125   250   500 Feet

SCALE IN FEET

N

Croton Water Treatment Plant

Figure 6.15-4

In some of the borings, extremely weathered bedrock was encountered beneath the overlying till. This material, called saprolite, is so weathered that it can actually resemble the overlying till deposits more than the underlying gneissic bedrock. Due to the commonality between these two materials with respect to their unconsolidated consistency, both the till and saprolite are treated collectively as overburden.

Hydraulic Conductivities. Field permeability tests were conducted in soil and bedrock at the water treatment plant site to estimate the hydraulic conductivities of some subsurface materials. Soil hydraulic conductivities were tested in boring MG-B51-99, located to the northeast of the proposed footprint. The hydraulic conductivity of the overburden was found to be about 2×10^{-4} cm/sec (0.5 ft/day).

Field permeability testing of the bedrock was done at several borings. Where water could be pumped into the rock, the hydraulic conductivities typically ranged from 10^{-4} to 10^{-5} cm/sec (0.3 to 0.03 ft/day). However, since a number of the tested zones showed essentially no water intake, the bulk permeability of the bedrock is probably lower than these values.
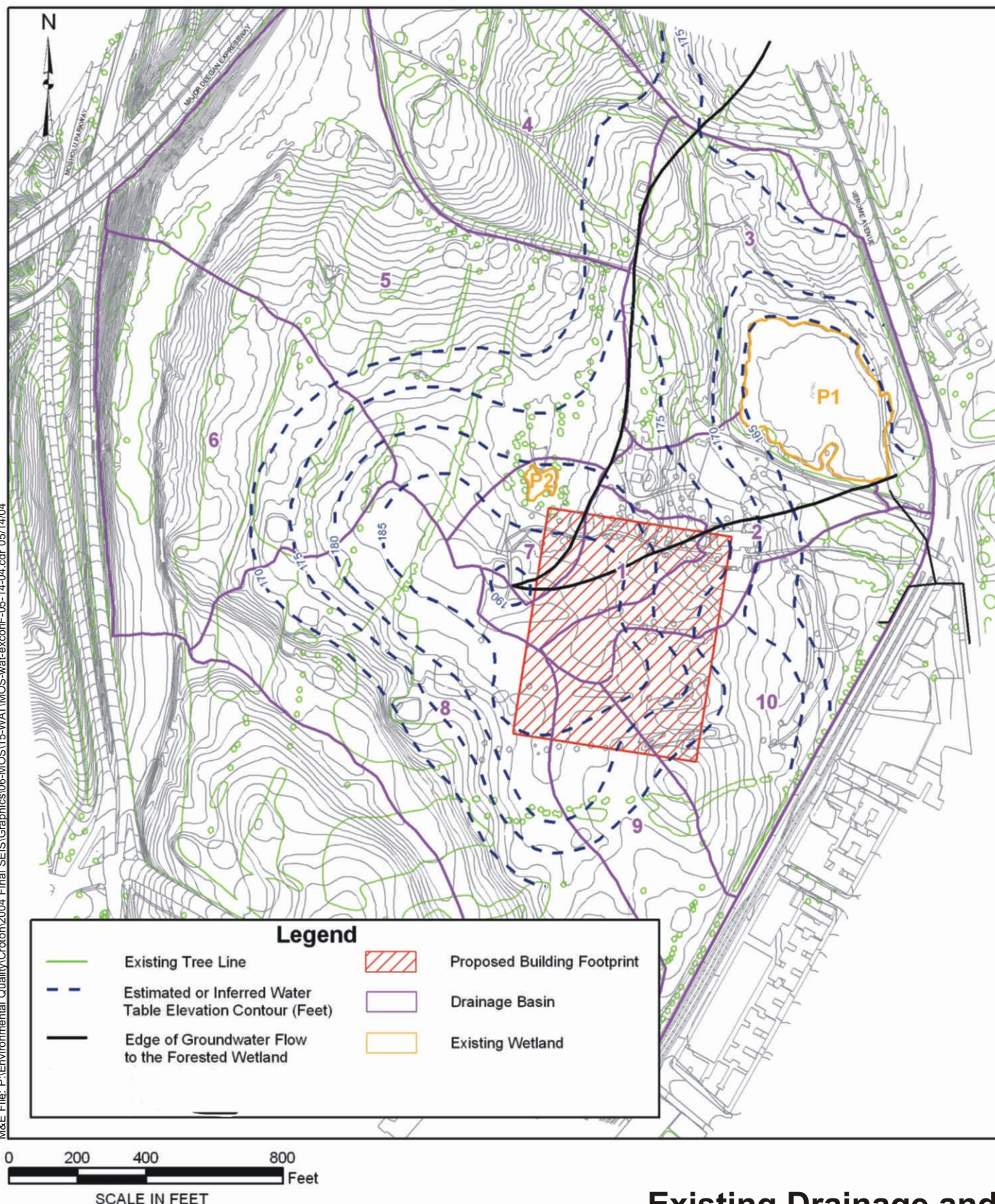
Groundwater Flow. As part of the geotechnical investigation, twenty observation wells (or piezometers) were installed at 17 of the boring locations in one or more (i.e., cluster wells) of the geologic units underlying the water treatment plant site. Subsequently, groundwater levels were measured on several occasions to determine the depth to water and the configuration of the groundwater surface in the overburden and bedrock units underlying the water treatment plant site. Water table contours were plotted from these data and inferred from the topography in areas of interest beyond the wells (Figure 6.15-5).

Though groundwater occurs in each of the geologic units underlying the water treatment plant site, its method of movement varies. In the overburden (till and saprolite), groundwater is stored and moves primarily though the pore spaces intervening between the comprising granular constituents (e.g., sand grains, gravel, and bedrock fragments). In the bedrock, groundwater is stored and moves primarily through individual and networks of fractures. This characteristic explains the generally low yield of groundwater exhibited by wells and tunnels (e.g., Shaft 3B of City Tunnel No. 3, which is several hundred feet north of the forested wetland in the Shandler Recreation Area) completed in the Fordham Gneiss which do not penetrate many such fractures.

Groundwater in the till deposits occurs under unconfined or "water table" conditions. Depending on the inclination and depth of occurrence of groundwater bearing fractures, the groundwater in the bedrock can occur from unconfined to confined ("artesian") conditions. Confined conditions in the bedrock are anticipated to be more prevalent with depth, while semi-confined to unconfined conditions are typically encountered in the upper portions of the bedrock including the water treatment plant site. Based on the geotechnical investigation results, groundwater in the saprolite and upper weathered portions of the bedrock in the water treatment plant site area occurs under unconfined to semi-confined conditions.

Existing Drainage and Season Water Table Contours Mosholu Site

Figure 6.15-5

Legend

Existing Tree Line	Proposed Building Footprint
Estimated or Inferred Water Table Elevation Contour (Feet)	Drainage Basin
Edge of Groundwater Flow to the Forested Wetland	Existing Wetland

SCALE IN FEET
0 200 400 800 Feet

The water table occurs either within the overburden and/or the upper bedrock depending on the encountered depth of groundwater relative to the penetrated geologic unit. The water table contour map for the water treatment plant site (Figure 6.15-5) shows the highest groundwater elevations (190+ feet) in the northwest corner of the footprint of the main treatment building.

This area of maximum water table elevation corresponds with the crest of a ridge on the bedrock surface that crosses the western edge of the footprint in a northwest-southwest orientation. Groundwater in the overburden and uppermost bedrock flows more or less radially outward in all directions from this area. The water table configuration indicates that, under existing conditions, only a small portion of the footprint of the main treatment building (in the northwest quadrant) is within the area that contributes groundwater flow to the forested wetland (see Figure 6.15-5).

Groundwater levels at multiple well installations indicate the vertical hydraulic gradient relationship between the overburden and underlying bedrock at the water treatment plant site. In both instances the vertical gradient is downward; indicating that groundwater in the overburden naturally flows into the underlying bedrock and is a natural source of recharge. As such, groundwater in the bedrock would not be generally considered to be a source of recharge to the overburden. This characterization is consistent with the local geology relative to the water treatment plant site topography (i.e., groundwater flows downhill).

The elevation of the water table near the forested wetland during this investigation strongly suggests that, during the high groundwater conditions that are typical in winter and spring, the water surface in the wetland is representative of the water table. Conversely, the upper foot of the wetland was unsaturated during summer and early Fall 1998, indicating that the water table falls below the wetland surface during typical summer conditions.

The drainage basin that contributes groundwater to the forested wetland encompasses about 25 acres. Assuming an average recharge rate of 15 inches/year for the basin (based on literature infiltration values for till soils), the annual average groundwater discharge rate from the overburden to the wetlands is estimated to be about 20 gallons per minute (gpm).

6.15.2.2. *Future Without the Project*

The Future Without the Project considerations include the anticipated year of peak construction (2010) and the anticipated year of operation (2011) for the proposed project. The anticipated peak year of construction is based on the peak number of workers because such inputs to the community would likely cause the most noticeable land use changes.

It is anticipated that the Mosholu Golf Course and Driving Range and the Allen Shandler Recreation Area would continue to operate as recreational facilities within Van Cortlandt Park. The Lew Rudin Youth Golf Center is still evolving and there would be continued need for improvements and space allocated on the golf course for these new golfers to learn and play.

6.15.3. Potential Impacts

6.15.3.1. *Potential Project Impacts*

The anticipated year of operation for the proposed project is 2011. Therefore, potential project impacts have been assessed by comparing the Future With the Project conditions against the Future Without the Project conditions for the year 2011.
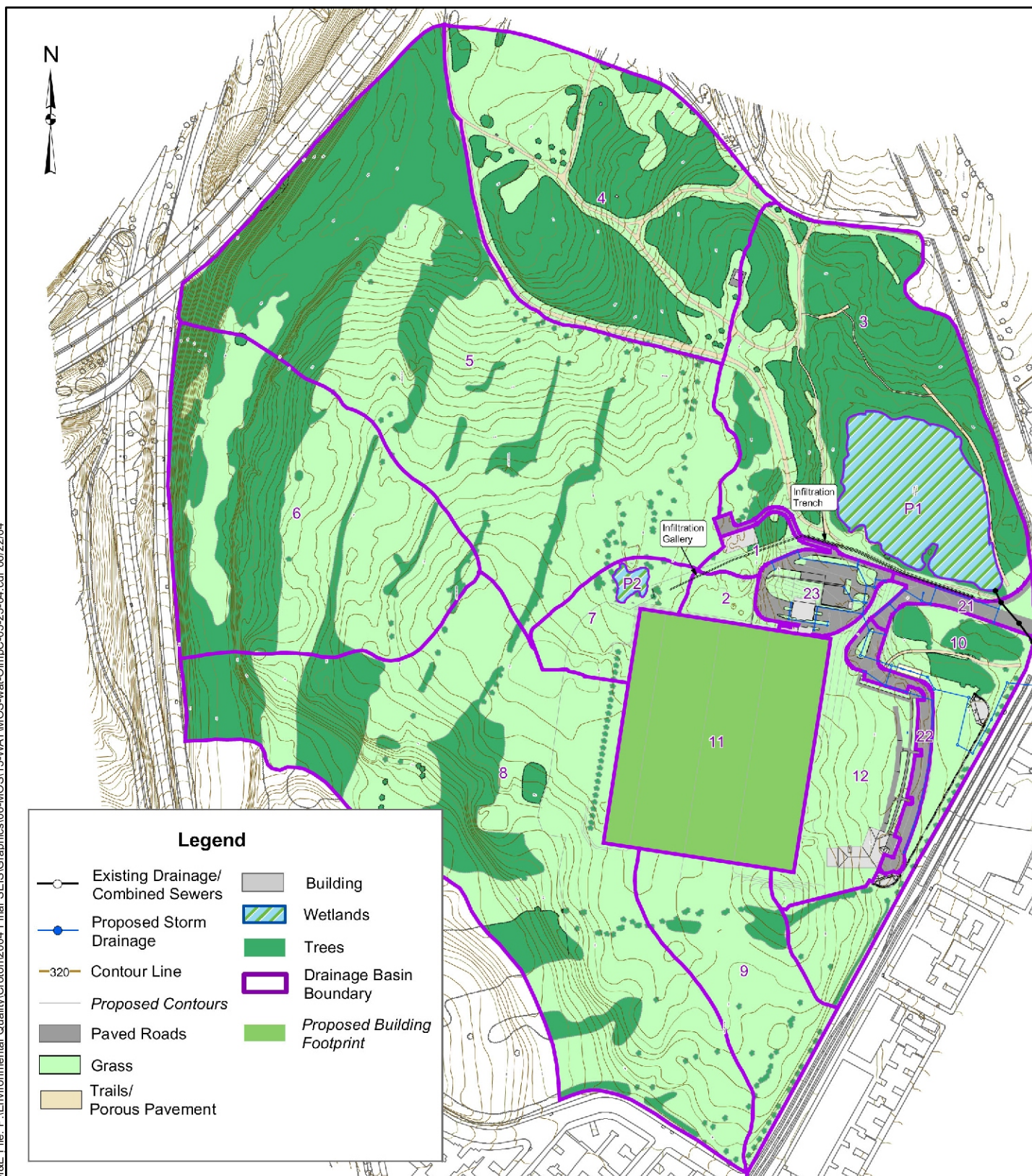
6.15.3.1.1. *Stormwater Runoff*

The stormwater management plan would provide long-term control and treatment of stormwater runoff from the water treatment plant site, to the maximum extent practicable. This includes landscaping to stabilize the water treatment plant site, provide treatment of stormwater runoff from all impervious services, and maintain flows to adjacent natural resource areas at or near the existing conditions rates and volumes.

The proposed site plan during operation of the proposed water treatment plant is shown in Figure 6.15-6. The permanent structure would consist of the proposed water treatment plant building situated in an area that has been excavated down into bedrock. Porous structural fill would be backfilled around and beneath the building. The surface of the proposed water treatment plant would be covered with about two feet of crushed stone, sand, topsoil, and sod or artificial turf, and then returned to use as part of the driving range.

The building would have drainage pipes at three levels. First, a rooftop system would collect water striking the top surface. The two feet of grass, soil, sand, and stone would provide considerable detention; this water would be passed by gravity to the combined sewer. Second, water would be collected at the top of the bedrock at an Elevation 170-180 feet. This water would be conveyed by gravity to the combined sewer. Finally, an underdrain would collect water at an elevation of approximately Elevation 104 feet and would be pumped to the combined sewer on Jerome Avenue. Best Management Practices (BMPs) would be designed to remove oil and sediment from stormwater during frequent wet weather events prior to discharging to the combined sewer system along Jerome Avenue. It is expected that groundwater flows prior to grouting would be up to approximately 55 to 60 gpm (pumping designed for 500 gpm).

The water collected from the western and northwestern side of the building at Elevation 180 feet (see Figure 6.15-6) represents flow that currently flows through the driving range at the base of the overburden toward the forested wetland. This flow would be conveyed by gravity around the building to a series of infiltration galleries, which would be built along the northern side of the construction impact area, between the forested wetland and the proposed water treatment plant. The gallery would extend westward 400 feet from the western edge of the existing parking area. Where the finished grade drops below Elevation 168 feet, the gallery would open into an infiltration trench immediately north of the parking area. See Figure 6.15-7 for a depiction of the infiltration trench.

Legend

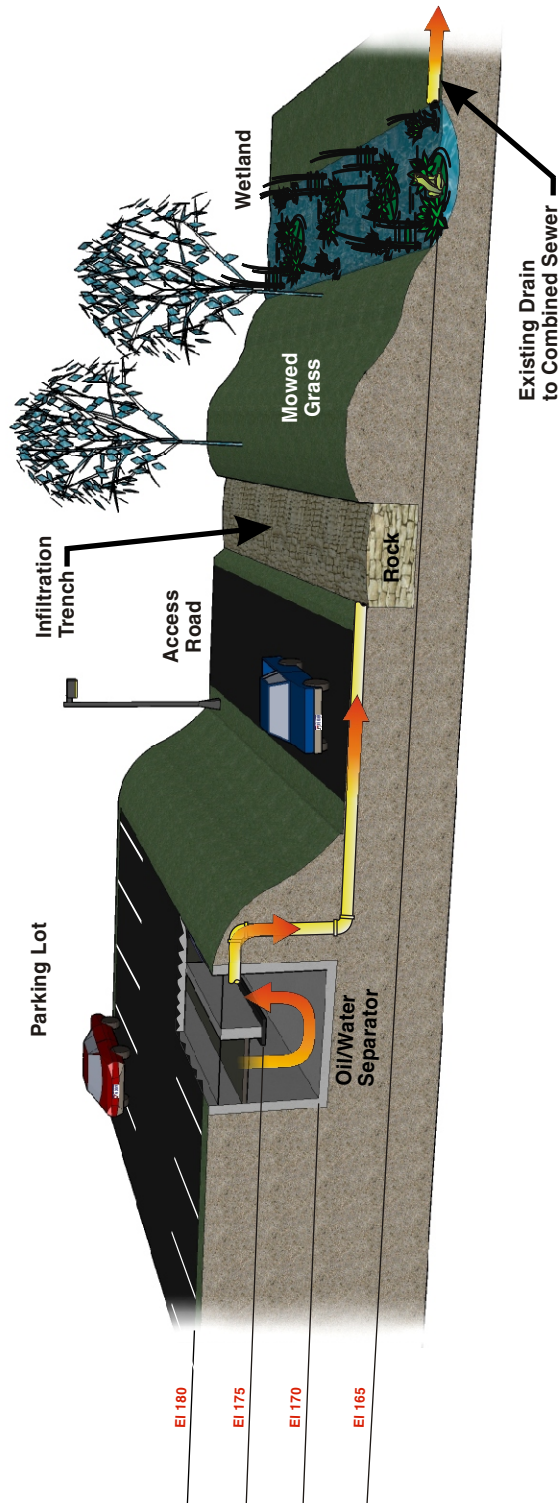|				
Existing Drainage/ Combined Sewers		Building		
Proposed Storm Drainage		Wetlands		
320 Contour Line		Trees		
Proposed Contours		Drainage Basin Boundary		
Paved Roads		Proposed Building Footprint		
Grass				
Trails/ Porous Pavement				

200 100 0 200 400 600 800

Scale in Feet

Operating Conditions at the Mosholu Site

Croton Water Treatment Plant

Figure 6.15-6

**Schematic of Proposed Infiltration Trench
For the Croton WTP -
Mosholu Site**

Croton Water Treatment Plant

Figure 6.15-7

Model Description and Results. The HydroCAD® stormwater model was used to predict runoff from the water treatment plant site for similar design storms, as shown in the existing conditions section above (3-month [1.5 inches], the 2-year [3.5 inches], the 5-year [4.5 inches], the 10-year [5.1 inches], and the 100-year [8.1 inches])[5], and also to size individual long-term pollution prevention devices, located to treat runoff from impervious areas.

The future conditions at the water treatment plant site were simulated by modifying the curve numbers and acreages in the HydroCAD® model to reflect the conditions illustrated in Figure 6.15-6. Table 6.15-3 summarizes the input parameters used in the model to simulate future stormwater conditions. Table 6.15-4 summarizes the runoff characteristics in 3-month, 2-, 5-, and 10-year storms.

The drainage system under the facility footprint (Basin 11) would intercept the storm flow and pass it through an oil/water separator, which would then be discharged to the combined sewer on Jerome Avenue. This volume of water is estimated to be 0.33 acre-ft for the 3-month storm, 1.34 acre-ft for the 2-year storm, and 1.93 acre-ft for the 5-year storm. Peak flows would be partially attenuated by the large volume of soil on the roof of the building. Under the proposed project design, the forested wetland would no longer receive direct untreated runoff from the Mosholu Golf Course club house parking lot. As a result, the stormwater model predicts a decrease in peak runoff flows and volumes to the forested wetland. However, as noted previously in the discussion related to existing stormwater flows at the site, only a very small amount of overland runoff reaches the wetland during the 3-month storm event. The stormwater model results confirm that groundwater rather than stormwater runoff is the primary source of water supporting the hydrology of this wetland. As described in Section 6.15.3.1.3, the forested wetland would be recharged via City water added to the proposed infiltration trench. If monitoring indicates a need to counteract the decreased stormwater flow, water would be added to the infiltration trench (in addition to the proposed 20 gpm) to maintain the hydrology of the forested wetland. Thus, the total volume of water reaching the forested wetland would remain essentially unchanged during operation of the water treatment plant. In addition, the stormwater model predicts a decrease in stormwater runoff volumes to the small wetland.

3-Month Storm. Stormwater model results for the 3-month, 24-hour storm indicate that the operation of the proposed water treatment plant would result in a decrease in flow rates and total runoff volume to the forested wetland compared to existing conditions. The model results indicate very little runoff is generated from the pervious areas of the project site. Current model results indicate that the total runoff volume to the forested wetland is predicted to decrease from 0.13 to 0.03 acre-ft. As noted previously, this is due to the fact that untreated stormwater runoff from the parking lot and clubhouse roof would no longer discharge directly to the forested wetland. Instead, this runoff would be intercepted and treated through an oil/water separator before flowing to the existing combined sewer.

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[5] This rainfall data is from the U.S. Weather Bureau. 1961. Technical Paper No. 40-Rainfall Frequency Atlas of the United States (TP 40).

TABLE 6.15-3. MOSHOLU SITE STORMWATER BASINS OPERATING CONDITIONS

Basin	Grass (Good, B)[1] Area (acres)	CN	Wetland Area (acres)	CN	Wooded Area (acres)	CN	Building/Paved Area (acres)	CN	Porous Pavement Area (acres)	CN	Composite Total Area (acres)	CN	Tc (min)
Basins Tributary to Forested Wetland													
1	0.45	61	-	-	0.15	55	0.11	98	-	-	0.7	65	3.5
2	0.67	61	-	-	-	-	0.01	98	-	-	0.7	62	14.4
3	2.96	61	2.45[a] 1.15[b]	98 55	8.40	55	-	-	0.73	85	15.7	64	20.2
Basin Tributary to Small Wetland													
7	2.05	61	0.15	98	-	-	-	-	-	-	2.2	64	10.4
Other Basins													
2A	0.12	61	-	-	-	-	0.63	98	-	-	0.8	92	4.5
22	0.05	61	-	-	-	-	0.88	98	-	-	0.9	96	6.0
23	0.46	61	-	-	-	-	1.32	98	-	-	1.8	88	25.3
4	5.48	61	-	-	7.44	55	0.03	98	0.69	85	13.6	59	25.2
5	16.29	61	-	-	10.40	55	0.01	98	0.01	85	26.7	59	26.9
6	8.09	61	-	-	5.94	55	-	-	-	-	14.0	58	20.6
8	16.11	61	-	-	6.88	55	-	-	-	-	23.0	59	19.5
9	4.90	61	-	-	0.33	55	0.06	98	-	-	5.2	61	31.2
10	3.61	61	-	-	1.15	55	0.20	98	0.08	85	5.0	61	16.3
11[c]	-	-	-	-	-	-	8.71	98	-	-	8.7	98	d
12	3.87	61	-	-	-	-	0.6	98	-	-	4.5	66	7.1

CN = Curve Number, a factor describing the surface permeability; higher numbers are assigned to areas of lower permeability.

Tc = Time of Concentration, the time in minutes required for a particle of water to flow from the most hydrologically remote point in the watershed to the receiving area; 1.) Grass (Good B) = grass cover > 75% and CN of 61; a.) Wetland, standing water, El. 164; b.) Wetland, El. 164 to 166; c.) There is no runoff from Basin 11. Rainfall would be collected and discharged to the combined sewer system; (d) varies

TABLE 6.15-4. OPERATION RUNOFF CHARACTERISTICS IN 3-MONTH, 2-, 5-, AND 10-YEAR STORMS

Basin	3-Month Storm		2-Year Storm		5-Year Storm		10-Year Storm	
	Peak Runoff Rate (cfs)	Runoff Volume (acre-ft)	Peak Runoff Rate (cfs)	Runoff Volume (acre-ft)	Peak Runoff Rate (cfs)	Runoff Volume (acre-ft)	Peak Runoff Rate (cfs)	Runoff Volume (acre-ft)
Basins Tributary to Forested Wetland								
1	0.0	0.00	0.6	0.04	1.1	0.08	1.5	0.10
2	0.0	0.00	0.3	0.04	0.6	0.07	0.8	0.09
3	0.1	0.03	7.0	0.92	14.2	1.65	19.0	2.14
Total		0.03		1.0		1.8		2.3
Basin Tributary to Small Wetland								
7	0.0	0.00	1.3	0.13	2.5	0.23	3.4	0.30
Other Basins								
2A	0.7	0.05	2.3	0.17	3.1	0.23	3.6	0.26
22	1.0	0.09	2.7	0.24	3.5	0.31	4.0	0.36
23	1.1	0.09	4.3	0.34	6.0	0.47	7.0	0.56
4	0.0	0.00	3.3	0.56	7.8	1.09	11.2	1.46
5	0.0	0.00	6.2	1.09	14.9	2.12	21.3	2.85
6	0.0	0.00	3.1	0.53	8.0	1.05	11.6	1.42
8	0.0	0.00	5.9	0.94	14.6	1.83	20.8	2.47
9	0.0	0.00	1.5	0.25	3.2	0.47	4.5	0.62
10	0.0	0.00	1.7	0.24	3.9	0.45	5.4	0.60
11[a]		0.64		2.01		2.72		3.15
12	0.0	0.02	3.4	0.30	6.5	0.52	8.5	0.67

a.)   There is no runoff from Basin 11. Rainfall would be collected and discharged to the combined sewer system.

The bulk of the recharge for the forested wetland would occur along the proposed infiltration trench; however, the infiltration gallery is also needed to provide flexibility in case the trench cannot infiltrate the required flow, as well as properly distributing the recharge. Therefore, storm flows were modeled with the infiltration trench alone. It was assumed that the portion of the infiltration trench and French drain system, which discharges the groundwater, would be 10,000 feet^2. With a hydraulic conductivity of 1ft/day, the proposed trench could infiltrate approximately 52 gpm. This would be sufficient to infiltrate the 20 gpm groundwater flow, as well as surface runoff from areas immediately adjacent.

The model results predict that future runoff rates and total runoff volumes in the basin tributary to the small wetland would remain essentially the same as those characterizing the existing conditions at the project site. Similarly, no significant changes are predicted for any of the remaining basins.

2-Year Storm. As expected, model results for the 2-year, 24-hour storm indicate that the operation of the proposed water treatment plant would result in a decrease in flow rates and total runoff volume to the forested wetland compared to existing conditions (Table 6.15-4). Current model results indicate a decrease in peak runoff rate and runoff volume from 9.1 cfs and 1.5 acre-ft during existing conditions, to 7.3 cfs and 1.0 acre-ft, respectively during facility operation. This decrease is due to the modification of the facility design which would result in the diversion of water from modified Basins 1 and 2 into the combined sewer. Although this represents a significant difference in storm flows to the forested wetland, high flows during storm events currently drain out the overflow to the sewer after rapidly passing through the forested wetland. The future flows would likely raise the water table to the height of the wetland drain and, therefore, maintain the existing water table. Thus the difference in storm flows between current conditions and those anticipated in the future would not have a significant impact on the hydrology. As noted above, if monitoring indicates a need to counteract the decreased flow, water would be added to the infiltration trench to maintain the hydrology of the forested wetland.

Runoff volume to the small wetland decreases from 0.17 acre-ft to 0.13 acre-ft. For the 2-year storm, the peak flow entering the combined sewer from the Croton WTP roof and the mainly impervious areas adjacent to the facility is 2.0 cfs with a volume of 1.3 acre-ft.

5-Year Storm. As a result of the reduction in area of Basins 1 and 2, stormwater runoff volumes to the forested wetland resulting from the 5-year, 24-hour storm would decrease during operation as compared to existing conditions (Table 6.15-4). Runoff volume to the forested wetland would decrease from 2.6 acre-ft to 1.8 acre-ft. Runoff volume to the small wetland would decrease by approximately 26 percent (from 0.31 acre-ft to 0.23 acre-ft). The effect of these changes on wetlands is discussed in Section 6.15.3.1.3.

The peak flows entering all other basins in the project area are expected to decrease slightly or remain the same. The exception to this is Basin 9 which, when compared to existing conditions, would have a decreased runoff volume of 0.06 acre-ft. Since the flow during the operation phase would be within 15 percent of the volume during existing conditions, there would be no significant change in flow in Basin 9.

The primary interest in the 5-year storm is for sizing stormwater facilities. This storm, and even larger storms, would rapidly fill the wetland and flow out the outlet structure to the sewer. The model results indicate that the peak flow entering the combined sewer from the water treatment plant roof and adjacent areas is 3.0 cfs with a volume of 1.9 acre-ft.

10-Year Storm. Under the 10-year 24-hour storm, the total flow rate entering the forested wetland from Basins 1, 2 and 3 is predicted to decrease as compared to existing conditions, from 21.5 to 20.5 cfs. The peak runoff rate in the basin tributary to the small wetland is also predicted

to decrease during this event from 4.6 cfs to 3.4 cfs. Therefore, no significant increase in erosion is anticipated in the small or forested wetland during long-term operation. As expected, runoff volume to the forested and small wetlands during a 10-year storm is predicted to decrease during operation, from 3.4 to 2.3 acre-ft, and 0.41 to 0.30 acre-ft, respectively. Therefore, no significant change in the hydrology of these wetlands due to a prolonged period of inundation is anticipated.

100-Year Storm. The 100-year 24-hour storm was also modeled for the operation phase of the project to assess the potential for downstream flooding of roadways and impacts to the wetlands. As expected, when compared to existing conditions under the same rainfall event, the total flow rate entering both wetlands during this event is predicted to decrease slightly during operation. The peak runoff rate in the basins tributary to the forested wetland is predicted to decrease during the 100-year storm from 50.7 to 48.7 cfs. The peak runoff rate in the basin tributary to the small wetland is predicted to decrease during the 100-year storm from 11.4 to 8.5 cfs. Therefore, no significant increase in erosion is anticipated in the small or forested wetland during operation. Runoff volume to the forested and small wetlands during a 100-year storm is also predicted to decrease during operation. Therefore, no significant change in the hydrology of these wetlands due to a prolonged period of inundation is anticipated. Based on HydroCAD® results for this storm event there would be no downstream flooding during construction as a result of this project.

Structural BMPs.   Stormwater runoff from the facility parking area and the associated access roads (2.7 acres) would be collected by the site drainage system, routed through a 50,000 gallon underground oil/water separator located northeast of the facility, and ultimately discharged to the combined sewer on Jerome Avenue. The design flow rate of the oil/water separator is 5,000 gpm (11cfs). In addition, street sweeping would also be employed to treat the new pavement.   No significant adverse impacts are anticipated from the quality of the stormwater runoff into the drainage following the pre-treatment by the structural BMP.

6.15.3.1.2.   *Groundwater*

The removal of groundwater from the underdrains and the associated envelope of gravel or other permeable material beneath the structure would lower water levels over an area larger than the excavation area. The magnitude and extent of drawdown in bedrock in any one direction would depend on the hydraulic conductivity of the bedrock fractures that extend from the structure in that direction. Although the results from the subsurface design investigation suggest that the bedrock is not highly fractured and has a relatively low hydraulic conductivity, significant drawdown within several hundred feet of the structure is possible. Water levels in the bedrock beneath the forested wetland, as well as beneath the western part of the drainage basin that contributes groundwater to the forested wetland, could be lowered in comparison to existing conditions.

Groundwater modeling was performed to evaluate the impacts of construction and long-term dewatering on groundwater and nearby wetlands. The model was developed using information available from site investigations such as boring logs, bedrock packer tests, slug tests, and visual observations. Since the model domain is much larger that the site limits, model characteristics outside the model domain were estimated through interpolation, extrapolation, and professional

judgment. The model was constructed to simulate long-term dewatering to predict drawdown impacts on nearby wetlands and determine potential recharge flow rates, if necessary.

The USGS three-dimensional finite difference flow model, MODFLOW, was used in this modeling evaluation. The model has five layers. The top layer (layer 1) is 15 feet thick and represents the unconsolidated materials above fractured bedrock. This layer consists of the till and weathered bedrock (saprolite). Layer 2 represents the top 10 feet of the fractured bedrock. The top of layer 2 coincides with the top of fractured bedrock. Layer 3 represents 20 feet of fractured bedrock below layer 2. Layer 4 represents 45 feet of fractured bedrock below layer 3. Layer 5 is the bottom layer of the model and represents deeper, more competent bedrock.
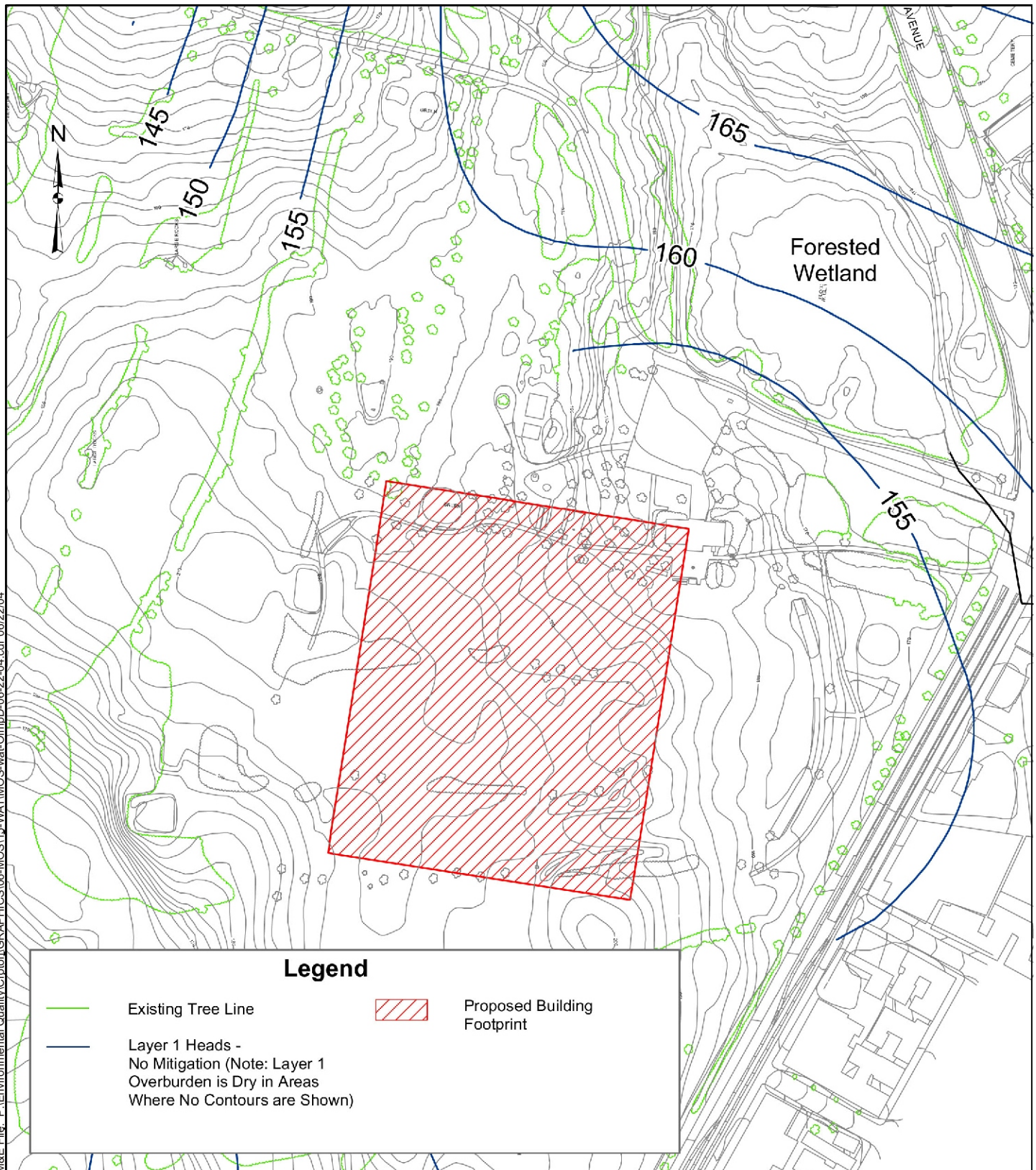
The forested wetland was represented by a cluster of drain nodes. Drains are used in groundwater modeling to simulate a case in which water is drained away (e.g., via surface water or pumping) when the piezometric surface rises above the ground surface or the bottom of a trench or pond. At this site, groundwater discharges to the forested wetland located north of the proposed treatment plant and is drained away at the southeast corner of the wetland by a culvert pipe. Model drain nodes were assigned appropriate elevations so that when the head in the aquifer in the wetland area increases above those elevations, water is drained away.

For calibration, the model was compared to field data collected during an average rainfall period, in this case assumed to be February 2000. The model head results were compared to observed heads measured in each observation well, and input parameters were adjusted where necessary to result in the best match. The resulting ranges of K_H for each layer, in ft/day, were 1.3 to 7 for layer 1; 0.03 to 0.7 for layer 2; 0.04 to 0.7 for layer 3; and 0.015 to 0.67 for layers 4 and 5. The calibrated model was used to simulate long-term dewatering during operation at the water treatment plant. Drain nodes were used to specify the groundwater elevation that would be maintained below the facility. The model calculates the flows necessary to maintain these heads, which are essentially the dewatering rates.

The model was first used to estimate the average rate of flow into the water treatment plant drains and to determine the resulting drawdowns. The dewatering flow associated with this scenario was approximately 55 gpm. Drawdowns of 5 to 10 feet, which lowered the water table to elevations of 155 to 160 feet, were predicted in the bedrock and the overburden beneath the forested wetland (Figure 6.15-8).

Since drawdowns of this magnitude could alter the wetland hydrology, another simulation was performed to estimate the amount of recharge which would be needed to maintain water in the wetland if an infiltration trench or gallery were placed between the proposed Croton project and the forested wetland. In this simulation, constant head cells were placed in layer 1 in a row along the proposed access road. The model then calculated the recharge flow necessary to maintain the head at these trench nodes and a zero drawdown condition in the wetland. An estimated recharge flow of about 20 gpm was found to be necessary to attain this goal. With this added recharge, the building underdrain system would collect approximately 60 gpm.

Contour Map of Groundwater Elevations With Building Underdrains and No Infiltration Trench Mosholu Site

Forested Wetland

145
150
155
160
165
155

N

Legend

— Existing Tree Line

— Layer 1 Heads -
No Mitigation (Note: Layer 1
Overburden is Dry in Areas
Where No Contours are Shown)

⬜ Proposed Building
Footprint

0   125   250   500
Feet
SCALE IN FEET

Croton Water Treatment Plant

M&E File: P:\Environmental Quality\Croton\GRAPHICS\06-MOS\15-WATMOS-wat-OimpD-06-22-04.cdr 06/22/04

AVENUE

Figure 6.15-8

The model showed that the drawdown impacts to the nearby forested wetland caused by long-term dewatering for the proposed water treatment plant can be mitigated by recharge to groundwater via an infiltration trench or gallery (Figure 6.15-9). The average flow into the water treatment plant dewatering system was predicted to be approximately 60 gpm, and the average flow rate necessary to maintain a saturated forested wetland is approximately 20 gpm. The model predictions are sensitive to changes in recharge and hydraulic conductivity. Although reasonable values for these parameters were either measured in the field or estimated during calibration, the rate of recharge to the infiltration trench or gallery would have to be fine-tuned during construction and operation, since small-scale groundwater flow patterns near and within the wetland may affect water movement in ways that cannot be predicted. In any event, natural perturbations in groundwater resulting from normal seasonally variability and year-to-year climactic variability would exceed the magnitude of the flow resulting from uncertainty in the model output.
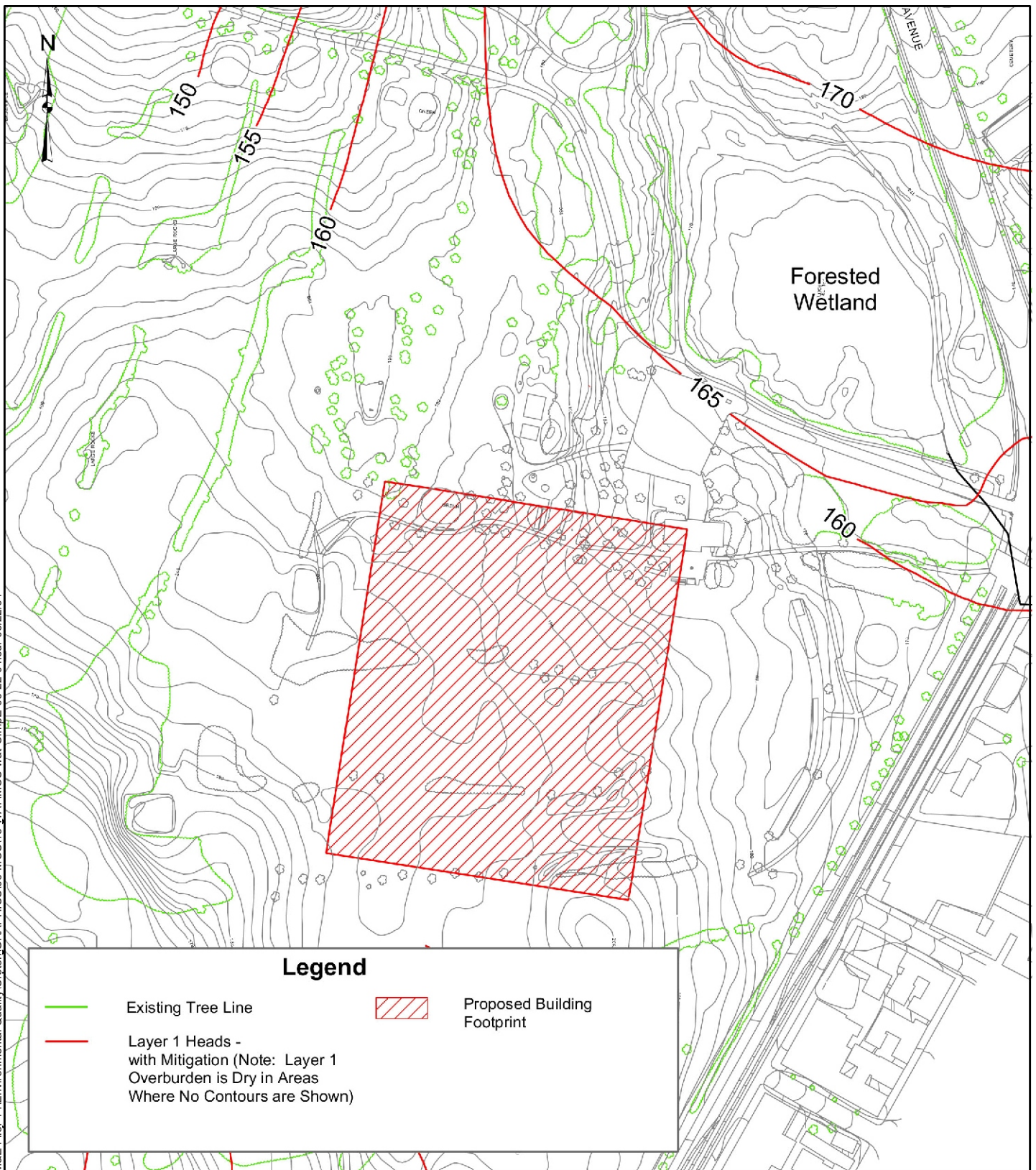
6.15.3.1.3.   Surface Water

Small Wetland. As discussed in the Methods of Analysis (Section 4.15), and Section 6.15.2.1.1 above, the 3-month storm generally represents a storm recurrence interval that would be most likely to provide sustaining hydrology for a wetland area. The model results indicate that there would be no change in total storm runoff to the small wetland under 3-month storm conditions during operation as compared to existing conditions. Therefore, stormwater runoff volume changes associated with the operation of the proposed water treatment plant are anticipated to have no significant impact on the wetland.

Total runoff volumes to the small wetland for the 2-year and 5-year storms are predicted to decrease during operation as compared to existing conditions. As indicated above, these are not the most critical storms for sustaining the hydrology of a wetland area. For the 2-year storm, total runoff to the small wetland is predicted to decrease by approximately 30 percent (from 0.17 acre-ft to 0.13 acre-ft). For the 5-year storm, total runoff is predicted to decrease by approximately 35 percent (from 0.31 acre-ft to 0.23 acre-ft) during operation. The decrease in the storm water flow from these storm events would not adversely affect this wetland since the flows are not currently detained by the topography of this system and essentially it operates as a flow through system for this volume of flow. The monitoring program included in the project plan would also assess the future function of this wetland, and the enhanced irrigation planned for the golf course renovation would be sized to maintain this wetland.

For all storms modeled, the total future peak runoff rate of water entering the small wetland is predicted to be the same or less than the existing runoff rate. Consequently, the proposed facility is anticipated to result in no increased erosion in the small wetland. Therefore, stormwater runoff rate changes associated with the operation of the proposed water treatment plant would result in no significant impact to the wetland. As discussed in the existing conditions section, the existing boring data suggest that the hydrology for this wetland is provided by infiltration of stormwater, which perches above the groundwater table in the overburden. Therefore, groundwater drawdown in the bedrock and overburden is expected to have no impact on the hydrology of the small wetland.

Legend

—— (green)	Existing Tree Line	
—— (red)	Layer 1 Heads - with Mitigation (Note: Layer 1 Overburden is Dry in Areas Where No Contours are Shown)	
(red hatched box)	Proposed Building Footprint	

Forested Wetland

150
155
160
165
170
160

0   125   250   500   Feet

SCALE IN FEET

Contour Map of Groundwater Elevations With Building Underdrains and Infiltration Trench Mosholu Site

Croton Water Treatment Plant

Figure 6.15-9

Forested Wetland. Potential impacts to the groundwater by the proposed project are very similar during both the construction and operational phases. There would be dewatering to as low as El. 115 feet during construction, and the permanent dewatering would extend to only about El. 116 feet.

The stormwater model results indicate that there would be a consistent decrease in total storm runoff to the forested wetland under 3-month, 2-, 5-, and 10-year storm conditions in the future as compared to existing conditions. However, modeling results for the existing site conditions indicated that very little runoff occurs from the largely pervious basins contributing to the forested wetland during the 3-month storm. As noted previously, since only a very small amount of overland runoff reaches the wetland during the 3-month storm event, the HydroCAD® results confirm that groundwater rather than stormwater runoff is the primary source of water supporting the hydrology of this wetland. Therefore, the diversion of stormwater flow into the proposed water treatment plant facility (Basin 11) would have a negligible effect on the forested wetland hydrology, vegetation, and function. For all storms modeled, the total future peak runoff rate of water entering the forested wetland is predicted to be the same or less than the existing runoff rate. Consequently, the proposed facility is anticipated to result in no increased erosion in the forested wetland. Therefore, stormwater runoff rate and volume changes associated with the operation of the proposed water treatment plant would result in no significant impact to the wetland.

The groundwater analysis indicates that the only probable mechanism by which groundwater levels in the forested wetland could be lowered would be migration through bedrock. Due to the low porosity of the bedrock, it is unlikely that there would be enough groundwater drawdown in the wetland soils to result in a significant impact on the hydrology of the forested wetland. The infiltration technology incorporated into the project plan would allow for recharge of the groundwater table in the wetland through discharge of either surficial groundwater derived from rainwater through the proposed infiltration gallery and trench from the permanent excavation, or from City drinking water. The infiltration trench has been sized such that a sufficient volume of water could be used to recharge the groundwater and sustain the hydrology of the wetland. Therefore, groundwater changes associated with the proposed water treatment plant are anticipated to have no significant adverse impact on the hydrology of the forested wetland. The infiltration trench is designed to release water to the combined sewer in excess of the three month storm event, thus preventing the over-watering of the wetland via surface flows except in the case of the 5-year storm, in which case surface flows would occur. These surfaces flows mimic natural conditions in a severe storm, and the existing drain in the wetland would still maintain the maximum surface level of the wetland.

6.15.3.2. *Potential Construction Impacts*

The anticipated peak year of construction for the proposed project is 2011. Therefore, potential construction impacts have been assessed by comparing the Future With the Project conditions against the Future Without the Project conditions for the year 2011.

The potential construction impact area including the potential staging area for the proposed plant would be approximately 28.6 acres. The approximate finished water treatment plant site area

would be 8.7 acres. During construction, the construction impact area would be cleared and graded to accommodate the storage and daily activities of construction vehicles and equipment.

6.15.3.2.1. *Stormwater Runoff*

The proposed stormwater controls incorporate measures specified by New York City,[6] New York State[7,8] and USEPA,[9] and the requirements of the NYSDEC general SPDES permit for stormwater discharges associated with construction activity[10]. Stabilization and structural best management practices (BMPs) would be included in the project design to dissipate peak flows to avoid on-site erosion, and maintain total storm volumes to avoid significant adverse impacts on wetland hydrology.

Construction Sequencing. The anticipated construction period for the proposed project would be a seven year period from August 2004, through the start of operations by October 2011. Stormwater management, erosion and sedimentation control measures would be implemented in a phased approach during construction. Phase I of construction would include demolition of existing facilities in the project area, site clearing and grubbing, construction of site access roads, and installation of erosion and sedimentation controls; Phase II would include construction of an infiltration trench and gallery, and building excavation; and Phase III would include building construction. For each phase of construction, the following topics are described: the sequence of construction and a summary of work to be conducted; erosion and sediment control measures to be implemented; and a description of on-site activities. Operation and maintenance of the proposed controls is described in Potential Project Impacts above.

Phase I (Initial Site Preparation). Prior to the start of significant construction activities at the water treatment plant site, the entrance to the project area from Jerome Avenue would be developed. Early in the construction phase, the entire water treatment plant site would be fenced, noise barriers erected, and concrete jersey barriers placed to demarcate the limits of construction and protect designated trees. A silt fence and double row of hay bales, as well as temporary sedimentation basins, would be installed inside the jersey barriers to assist in erosion and sedimentation control. The existing golf course clubhouse and associated structures would be demolished and temporary facilities for construction management and site security installed in anticipation of the excavation work. There would be very limited parking facilities on site during construction of the water treatment plant; workers would be transported to the site from a remote (off-site) parking area. The final step before excavation is initiated is the clearing and grubbing of trees within the proposed building footprint. This material would be removed from the site and transported to an authorized site.

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[6] City Environmental Quality Review (CEQR, 2001). CEQR Technical Manual.

[7] New York State Guideline for Urban Erosion and Sediment Control and New York State Soil and Water Conservation Committee (1997).

[8] New York State Stormwater Management Design Manual, New York City DEC, (NY, 2001)

[9] Storm Water Management for Construction Activities: Developing Pollution Prevention Plans and Best Management Practices (EPA B32-R-92-005)

[10] New York State Department of Environmental Conservation. State Pollution Discharge Elimination System (SPDES). 2003.

Phase II (Building Excavation). The first step in the excavation process is the removal of topsoil and overburden to expose the top of rock in the area to be excavated. The permeable material would be stored on-site for use during construction. Before the excavation proceeds into the bedrock, an infiltration gallery and trench would be installed along the northern boundary of the proposed construction impact area. The purpose of this system is to provide adequate infiltration during construction and operation to maintain the hydrology of the forested wetland.

Excavation is expected to start at the west end of the water treatment plant footprint. Final stripping is expected to progress from west to east, using bulldozers equipped with rippers. Soil excavation would continue until the top of the rock surface is exposed. Once the bedrock is exposed, drilling and blasting would continue through the remainder of the excavation period to remove the rock. As this phase of the project progresses, develop haul roads, and truck queuing and washing areas would be developed. Once the bedrock is exposed, drilling and blasting would continue through the remainder of the excavation period to remove the rock. All excavated rock would be trucked off site for disposal. A laydown/staging area would be established along the western boundary of the site, between the excavation and the limit of construction.

Phase III (Building Construction). After the excavation is completed the building construction would proceed, with underdrain installation and initial concrete construction (rebar and form placement at the foundation level). In general, building construction would proceed both horizontally and vertically starting at the northwest section, moving to the southeast section, and proceed in a similar fashion westward. Site excavation beyond the building footprint would be conducted for electrical ductbanks, plant water and sewer connections, and the proposed residuals piping to Jerome Avenue for transmission to Hunts Point WPCP. Erosion control measures from the previous contract would be used where practicable and modified where necessary.

Erosion and Sediment Control Measures. During construction, the sedimentation and erosion controls and stormwater management practices described in this section may potentially be employed to minimize erosion, and prevent sedimentation of the adjacent wetlands. However, the final design of the erosion and sedimentation control measures during construction of the proposed plant would be the responsibility of the contractor. Control measures would include stabilization for disturbed areas, and structural controls to divert runoff and remove sediment. In addition to managing stormwater runoff and erosion, BMPs would help to prevent accidental releases of fuels, lubricating fluids, or other hazardous materials. More detail related to stormwater management and erosion and sediment control, during construction and post-construction phases of the project, is provided in the Stormwater Pollution Prevention Plan (SWPPP) for the Mosholu site (Appendix G).

Phase I. The proposed erosion and sedimentation control plan would be developed to prevent waterborne sediment from entering surface water and wetland resource areas adjacent to the site during Phase I of construction. The location of erosion and sedimentation control measures would serve as an absolute limit of work. Under no circumstances would any work occur on the resource side of the erosion control barriers.

During this phase of construction, control of stormwater runoff to the Small Wetland and the Forested Wetland would be provided primarily using haybales, sediment fences, and temporary sedimentation basins/rock filters. Runoff from cleared areas would be collected via diversion berms and swales, each leading to filtration devices. A line of toed-in and staked silt fence and haybales would define the limits of work up gradient.

Phase II. The depression created by the initial removal of topsoil and overburden would create a depression which, following regrading within Basin 11, would serve as a large detention basin to capture runoff. Construction laydown and staging, and truck queuing/turn around areas, would where possible be surfaced with porous pavement. All catch basins within the drainage system would be equipped with inlet protection. Dust generation would be minimized by the use of water trucks and street sweeping. Stabilization of open soil surfaces would be conducted immediately after clearing and when seeding or mulch applications would be effective in stabilizing slopes. Stabilization of exposed areas and stockpiled soils would consist of hydroseeding, or straw or grass mulch.

The main activity during this phase of construction, rock excavation, would probably result in an increase in on-site traffic. The erosion and sediment control devices established in Phase I would remain in place. A regular program of inspections and maintenance would be conducted.

Phase III. At this stage of project construction, the site would be established and stabilized. The emphasis of stormwater management at this phase of the work would be on operation and maintenance of structural BMPs, and control of runoff from increased on-site activities. Runoff from the water treatment plant parking areas and access roads would be routed through the permanent subsurface 5,000 gpm oil/water separator located described previously.

6.15.3.2.2.   *Groundwater*

Construction of the proposed plant would require dewatering. Elements of the proposed project that would be constructed partially or completely below the water table include the main treatment building, the pumping station, and the raw water and finished water tunnels. The excavations for all of these facilities would require dewatering to remove groundwater that flows into the excavations. Although not anticipated because of the rock types, if inflows of water occur that could disrupt or endanger the construction, the Contractor would inject grout under pressure to seal the source of groundwater. Some groundwater would still enter those parts of the construction excavation that are below the groundwater table. The removal of this water would cause local groundwater to flow toward the excavation, resulting in a localized depression of the groundwater. The water treatment plant process building and pump station are the only excavations that would produce groundwater volumes that could influence the water table. The shafts and tunnels would be sealed as they were excavated, so groundwater is not expected to flow into the shafts and tunnels at rates that would influence the surroundings.

A forested wetland, generally without standing water, is to the north of the site and is higher in elevation than the lowest part of the water treatment plant excavation. There is the potential for water to flow from the wetland toward the excavation. This potential impact would be avoided by the construction of an infiltration gallery and trench, as described above in Section 6.15.3.1.3.

This infiltration gallery and trench would be built at the start of the construction sequence, before rock excavation is continued below the groundwater level (Phase II). This infiltration gallery and trench would prevent groundwater from exiting the wetland because the water table between the wetland and the excavation would be maintained at a level higher than the wetland. Because of these measures, and the naturally low transmissivity of the local rock, there would be no significant impact to groundwater during construction.

6.15.3.2.3. *Surface Water*

The period of construction that could have the greatest potential impacts on both the small and forested wetland would be during the initial activities and peak excavation. Early installation of erosion control measures and other stormwater BMPs would prevent potential untreated-stormwater runoff and equipment wash water to enter these resource areas. The erosion control measures and BMPs would include haybales, sediment fences, and temporary sedimentation basins/rock filters. These BMPs would be maintained as specified, and inspected on a regular basis, to prevent the existing wetland water quality from being affected during the construction phase of the project.

Based on the analyses presented above, the proposed Croton project at the Mosholu Site would have no significant adverse impacts on Water Resources. For comparison purposes, this is true of the Eastview and Harlem River sites as well.