7
Infrastructure

7.1 Introduction

The project site encompasses a total of approximately 46.6 acres consisting of second and third generation forest cover, residential development, and two man made ponds having a total combined water surface area of approximately 1.8 acres.

Topographically, the site contains several high points on the south central portion of the site with topographic undulations throughout the site. One on-site pond of approximately 1.6 acres is located on the western portion of the site. Another on-site pond having a water surface area of approximately 0.2 acres is located near the southeastern corner of the site and is believed to be an old ice pond. There are wetland areas are contiguous with these two ponds as well as additional wetland areas located along the northern and southern parcel boundaries. All of the runoff from the site eventually discharges to NYSDEC wetland W-3, to the north of the site and from there to the Hudson River via Sheldon Brook. All on-site wetlands whether local, State or federal have been flagged, survey located and shown on the project drawings. See Exhibit 7-1, Existing Conditions.

On-site elevations range from lows of approximately 165 feet on the north boundary to a high of approximately 390 feet on the southeast corner of the site.

On-site soils were determined utilizing the USDA SCS Soil Survey of Putnam and Westchester Counties, New York, issued September 1994. The soils found on site consist of Charlton loam (ChB, ChC, ClF), Charlton-Chatfield complex (CrC) and Chatfield-Charlton complex (CsD), and a small portion of Urban land-Charlton complex (UhC).

For the Jardim Estates East subdivision, the limits of disturbance encompass approximately 7.92 acres which include Charlton loam (ChB, ChC, ClF), Charlton-Chatfield complex (CrC) and Chatfield-Charlton complex (CsD) soils. The remaining soils on site are generally outside of the limits of disturbance. See Exhibit 7-1, Existing Conditions and Exhibit 7-2, Limits of Disturbance.

7.2 **Stormwater** (See Appendix 16.10 for the complete Stormwater Pollution Prevention Plan)

7.2.1 Pre-development Hydrologic Analysis

For information, including drainage basin delineation and design point locations, for the pre-development condition see Exhibit 7-3, Pre-Development Drainage.

The total drainage basin for this project is approximately 152.4 acres in size and includes off-site lands in the vicinity of the project site.

7.2.1.1 Basin B1 to Design Point 1

The contributing area for this basin is approximately 63.3 acres in size consisting of "B" soils or water surface area. 49.6 acres of this watershed originate from off site and consist of mostly wooded areas with some runoff from residentially developed areas to the south and residentially developed outparcels within the site. Approximately 13.7 acres of this watershed originates on site which includes a 1.57-acre on-site pond, wooded areas and residentially developed areas. This basin drains through the on-site pond whose outfall is considered as design point 1 for this analysis.

7.2.1.2 Basin B2 to Design Point 2

The contributing area for this basin is approximately 64.5 acres in size consisting of "B" soils or water surface area. 44.4 acres of this watershed originate off site from the south and consists mostly of wooded areas with a small portion of this off-site watershed coming from a residentially developed outparcel within the site. The remaining portion of this watershed (approximately 20.1 acres) originates on site and is mostly wooded with some minor remnants of previous development. This basin drains through a 24-inch concrete pipe on the property's eastern boundary which will be considered as design point 2 for this analysis.

7.2.1.3 Basin B3 to Design Point 3

The contributing area for this basin is approximately 24.6 acres in size consisting of "B" soils. 15.4 acres of this watershed originate off site and consist mostly of a densely populated residential area located to the northwest of the site. A single large wooded parcel also contributes a smaller portion of the off-site flows to this watershed. The on-site area of this watershed is wooded with a small area showing remains of previous development and a portion of an access road which serves existing residences to the east of the site. The locations where the flows from this watershed cross Sheldon Avenue to the north of the site are considered design point 3 for this analysis.

7.2.2 Pre-Development Peak Flows

Peak flows are based on a Type III, 24-hour rainfall distribution that yields a design rainfall of 2.8", 5.0", 6.0" and 7.5" for the 1, 10, 25 and 100-year recurrence storms, respectively. The drainage basin detailed data including the Cn calculations, time of concentration calculations, ground cover data and stormwater hydrographs are shown for the 100-year storm event are presented in Appendix C of the Stormwater Pollution Prevention Plan. Summary data is provided for all of the analyzed rainfall events.

Table 7.1 Pre-development Peak Discharges (cfs)

Basin	1-Year Storm	10-Year Storm	25-Year Storm	100-Year Storm
B1	5.3	44.4	69.0	110.0
B2	2.1	30.4	50.4	85.1
B3	4.5	29.7	44.5	69.1

It is the above peak discharge rates that are the basis to insure the post-development peak discharge rates are less than or equal to those presented above. All numbers shown have been rounded to the nearest tenth.

7.2.3 Post-development Hydrologic Analysis

For information, including drainage basin delineation and design point locations, for the post-development condition see Exhibit 7-4, Post Development Drainage.

In the post development condition, the overall drainage basin shapes, drainage patterns and areas change due to the construction of the "rain gardens" or small stormwater basins throughout the site. Many sub basins have been added within the larger pre development basins to account for the addition of these small stormwater basins that have been incorporated into this project. These small rain gardens act to provide the required water quality treatment for the project in addition to providing peak flow attenuation for the stormwater leaving the site. To see the proposed grading, details and relationship to other proposed and existing facilities within the project site, see Exhibit 7-5, Stormwater Draining Plan – Conventional Layout and

Exhibit 7-6, Road Profiles – Conventional Layout. This information is also illustrated on the full project site plans.

7.2.3.1 Basins B1, B1A, B1B, B1C, B1D and B1E to Design Point 1

These drainage basins generally consist of the area in pre development B1 with some minor adjustments due to site grading. The watersheds for drainage basins B1A, B1C, B1D and B1E will be the proposed developed portions within the original B1 watershed that will be provided with small stormwater basins or rain gardens equipped with control structures to impound or limit the stormwater flow from these smaller basins. In the stormwater analysis, the smaller basins are modeled as "subcatchments" which are then directed to "ponds" which simulate the proposed stormwater basins whose outflows are then directed through "reaches" which simulate the transport of stormwater from the basin outlet to the design point. The watershed for drainage basin B1B is comprised of existing developed offsite and onsite areas within the original B1 watershed which are not to be further developed and will therefore not be provided with stormwater treatment practices. The total watershed to design point 1 increases approximately 0.4 acre in size to approximately 63.7 acres in the post development condition with 2.9 of these acres receiving treatment from rain gardens. Design point 1 remains the same as in the pre development condition for basin B1.

7.2.3.2 Basins B2, B2A, B2B and B2C to Design Point 2

These drainage basins generally consist of the area in pre development B2 with some minor adjustments due to site grading. The watersheds for the smaller drainage basins (B2A, B2B and B2C) will be the proposed developed portions within the original B2 watershed that will receive similar treatment as the smaller watersheds tributary to design point 1 with the exception of basin B2C which will be provided stormwater treatment by means of underground infiltrators. The total watershed to design point 2 decreases approximately 0.1 acre in size to approximately 64.4 acres in the post development condition with 3.0 of these acres receiving treatment from rain gardens or underground infiltration devices. Design point 2 remains the same as in the pre development condition for basin B2.

7.2.3.3 Basins B3, B3A, B3B, B3C and B3D to Design Point 3

These drainage basins generally consist of the area in pre development B3 with some minor adjustments due to site grading. The watersheds for the smaller drainage basins (B3A, B3B, B3C and B3D) will be the proposed developed portions within the original B3 watershed that will receive similar treatment as the smaller watersheds tributary to design points 1. The total watershed to design point 3 increases approximately 0.2 acres in size to approximately 24.5 acres in the post development condition with 3.0 of these acres receiving treatment from rain gardens. Design point 3 remains the same as in the pre development condition for basin B3.

7.2.4 Post-development Peak Flows

Peak flows are based on a Type III, 24 hour rainfall distribution that yields a design rainfall of 2.8", 5.0", 6.0" and 7.5" for the 1, 10, 25 and 100 year recurrence storms, respectively. The drainage basin detailed data including the Cn calculations, time of concentration calculations, ground cover data and stormwater hydrographs are shown for the 100 year storm event are presented in Appendix D of the Stormwater Pollution Prevention Plan. Summary data is provided for the remainder of the analyzed rainfall events.

Table 7.2 Post-development Peak Discharges (cfs)

Basin	1-Year Storm	10-Year Storm	25-Year Storm	100-Year Storm
B1, B1A, B1B, B1C, B1D and B1E	5.2	43.3	67.1	106.8
B2, B2A, B2B and B2C	2.0	30.0	49.8	83.4
B3, B3A, B3B, B3C and B3D	4.5	27.5	41.0	64.8

The above values were rounded to the nearest tenth.

7.2.5 Summary of Pre- and Post-development Peak Flows

Table 7.3 below compares the pre development peak flow rates to the post development peak flow rates at design points 1, 2 and 3. As can be seen, for each of the design points, it can be expected that the post development rates are equal to or less than the pre development rates and that mitigation is accomplished.

Based on the above table, the Jardim Estates East subdivision as proposed with its stormwater management system will not result in an increase in the peak rate of discharge at design points 1, 2 and 3 for all of the analyzed storms in the post-development condition as compared to the pre development condition.

With respect to the NYSDEC General Stormwater SPDES Permit requirement, the above described post development peak flow attenuation satisfies the requirements for Overbank Flood Control (Qp), and Extreme Flood Control (Qf). The Stream Channel Protection Volume (Cpv) requirement is satisfied in that all of the proposed rain gardens and underground infiltrators are sized to contain the full 1 year rainfall event from their tributary areas, not just the required water quality volume.

Table 7.3 Pre- and Post- Development Peak Discharge Comparison

Design Point	Design Storm	Pre-development Discharge[1] (cfs)	Post-development Discharge[1] (cfs)	Net Change (cfs) % Reduction ( )
DP1	1	5.3	5.2	-0.1(1.9%)
	10	44.4	43.3	-0.1(0.2%)
	25	69.0	67.1	-1.9(2.8%)
	100	110.0	106.8	-3.2(2.9%)
DP2	1	2.1	2.0	-0.1(4.8%)
	10	30.4	30.0	-0.4(1.3%)
	25	50.4	49.8	-0.6(1.0%)
	100	85.1	83.4	-1.7(2.0%)
DP3	1	4.5	4.5	0.0(0.0%)
	10	29.7	27.5	-2.2(7.4%)
	25	44.5	41.0	-3.5(7.7%)
	100	69.1	64.8	-5.2(6.2%)

* The flow values shown in this table have been rounded to the nearest tenth from the values generated by the computer model and are appropriate for this type of analysis. See Appendix C of the Stormwater Pollution Prevention Plan for the pre-development stormwater summaries, calculations and hydrographs and Appendix D of the Stormwater Pollution Prevention Plan for the post-development stormwater summaries, calculations and hydrographs.

7.2.6 Pipe Capacities

All proposed pipe capacities will be checked prior to final approval for the project. In general, all piping within any proposed common driveways will be 15" or 18" diameter HDPE piping. The methodology to be employed in determining the pipe capacities are based on the Rational Method of determining the instantaneous peak rate of flow for those areas tributary to the pipe runs. In utilizing the Rational Method, a design storm of 10 years will be modeled based on the Intensity-Duration frequency Curves for either Central Westchester County, NY, whichever is greater. The rainfall intensity is based on the time of concentration for each tributary area.

The areas tributary to the pipe runs will be delineated, sized in acres and runoff coefficients, or C-values, will be tabulated for each pipe run. Times of concentration are assigned to each drainage basin using a minimum value of five minutes.

7.2.7 Stormwater Quality

The stormwater water quality program presented herein is designed to meet the NYSDEC required sizing criteria and pollutant removal goals. As part of the water quality program and sizing criteria, a water quality volume, WQv, shall be provided to capture and treat 90% of the average annual stormwater runoff volume. The WQv is directly related to the amount of impervious cover at a site. For these calculations, the existing offsite gravel driveway surfaces that may be tributary to a given water quality basin are considered to be impervious. Individual "rain gardens," or in the case of lot 5, an underground infiltration device, will be dispersed throughout the site to capture and treat the required WQv through infiltration practices. The control structures for these rain gardens will be designed to provide a minimum storage volume that is equal to or greater than the required WQv. The fact that the entire WQv is being treated by infiltration measures will also satisfy the Runoff Reduction Volume (RRv) requirements for this project. The following equation is utilized to determine the water quality volume in acre-feet of storage for the Jardim Estates East subdivision:

$$WQv = \frac{(P)\,(Rv)\,(A)}{12}$$

WQv	=	water quality volume	(acre-feet)
P	=	90% rainfall event Number	(figure 4.1 DEC Stormwater Manual)
Rv*	=	0.05 + 0.009 (I)	(I = percent of impervious cover)
A	=	site area	(acres)

* A minimum Rv value of 0.2 shall be used for all regulated sites.

For this site, the P value of the 90% rainfall for New York State in figure 4.1 of the NYSDEC Stormwater Management Design Manual is 1.3.

7.2.7.1 Water Quality Treatment – Drainage Basins

Post-development Basin B1A (Lot 3/4 common driveway to lot 3/4 front)

WQv	=	water quality volume (acre-feet)
P	=	1.3
Rv	=	0.392 (I = 38%)
A	=	0.55 acres
WQv	=	(1.3) (0.392) (0.55) / 12 = 0.02336 acre-feet
WQv	=	0.02336 acre-feet or 1,018 or 1,020 cubic feet

For post-development basin LB1A, a rain garden with its lowest control structure inlet 2.5 feet above the rain garden base will provide a water quality storage volume of 8,950 cubic feet, greater than the 1,020 cubic feet required for the basin it is proposed to serve.

Post-development Basin B1B (Outparcel "C", N.F. Maselli)

Post-development basin B1B consists mainly of an off-site parcel with some lawn area from lot 12 and some existing paved roadways which are to be removed and relocated to provide access to proposed lots 3 and 4. No stormwater quality practices are required or are to be provided here as the only substantial groundcover changes being performed within this watershed involve the removal of existing asphalt.

Post-development Basin B1C (Lot 12 to Lot 12 Pond)

WQv	=	water quality volume (acre-feet)
P	=	1.3
Rv	=	0.203 (I = 17%)
A	=	0.59 acres

WQv	=	(1.3) (0.203) (0.59) / 12 = 0.01298 acre-feet
WQv	=	0.01298 acre-feet or 565 cubic feet

For post-development basin B1C, a rain garden with its control structure inlet 2.8 feet above the garden base will provide a water quality storage volume of 1,025 cubic feet, greater than the 565 cubic feet required for the basin it is proposed to serve.

Post-development Basin B1D (Southern Portion of Roadway to Pond across Outparcel "A")

WQv	=	water quality volume (acre-feet)
P	=	1.3
Rv	=	0.482 (I = 48%)
A	=	1.02 acres

WQv	=	(1.3) (0.482) (1.02) / 12 = 0.05326 acre-feet
WQv	=	0.05326 acre-feet or 2,320 cubic feet

For post-development basin B1D, a rain garden with its lowest control structure inlet 2.2 feet above the garden base will provide a water quality storage volume of 3,750 cubic feet, greater than the 2,320 cubic feet required for the basin it is proposed to serve.

Post-development Basin B1E (Lot 10 to Lot 10 Pond)

WQv	=	water quality volume (acre-feet)
P	=	1.3
Rv	=	0.2 minimum (I = 15%)
A	=	0.71 acres

WQv	=	(1.3) (0.2) (0.71) / 12 = 0.01538 acre-feet
WQv	=	0.01538 acre-feet or 670 cubic feet

For post-development basin B1E, a rain garden with its lowest control structure inlet 2.0 feet above the garden base will provide a water quality storage volume of 2,675 cubic feet, greater than the 670 cubic feet required for the basin it is proposed to serve.

Post-development Basin B2A (East End Lots 3 & 4 to Lot 3 Pond)

WQv	=	water quality volume (acre-feet)
P	=	1.3
Rv	=	0.2 minimum (I = 13%)
A	=	2.03 acres
WQv	=	(1.3) (0.2) (2.03) / 12 = 0.04398 acre-feet
WQv	=	0.04398 acre-feet or 1,916 or 1,920 cubic feet

For post-development basin B2A, a rain garden with its lowest control structure inlet 3.5 feet above the garden base will provide a water quality storage volume of 2,355 cubic feet, greater than the 1,920 cubic feet required for the basin it is proposed to serve.

Post-development Basin B2B (Lot 7 to Lot 7 Pond)

WQv	=	water quality volume (acre-feet)
P	=	1.3
Rv	=	0.2 minimum (I = 10%)
A	=	0.87 acres
WQv	=	(1.3) (0.2) (0.87) / 12 = 0.01885 acre-feet
WQv	=	0.01885 acre-feet or 821 or 825 cubic feet

For post-development basin B2B, a rain garden with its lowest control structure inlet 1.8 feet above the garden base will provide a water quality storage volume of 1,055 cubic feet, greater than the 825 cubic feet required for the basin it is proposed to serve.

Post-development Basin B2C (Lot 5 to Lot 5 Infiltrators)

WQv	=	water quality volume (acre-feet)
P	=	1.3
Rv	=	0.95 (I = 100%)
A	=	0.11 acres
WQv	=	(1.3) (0.95) (0.11) / 12 = 0.01132 acre-feet
WQv	=	0.01132 acre-feet or 493 or 495 cubic feet

For post-development basin B2C, an infiltration system with a water quality storage volume of 1,115 cubic feet will be provided, greater than the 495 cubic feet required for the basin it is proposed to serve.

Post-development Basin B3A (Lot 9 to Lot 9 Pond)

WQv	=	water quality volume (acre-feet)
P	=	1.3
Rv	=	0.2 minimum (I = 10%)
A	=	1.19 acres

WQv	=	(1.3) (0.2) (1.19) / 12 = 0.02578 acre-feet
WQv	=	0.02578 acre-feet or 1,123 or 1,125 cubic feet

For post-development basin B3A, a rain garden with its control structure inlet 1.5 feet above the garden base will provide a water quality storage volume of 1,505 cubic feet, greater than the 1,125 cubic feet required for the basin it is proposed to serve.

Post-development Basin B3B (Lot 8 and Northern End of Road to Lot 8 Pond)

WQv	=	water quality volume (acre-feet)
P	=	1.3
Rv	=	0.302 (I = 28%)
A	=	1.38 acres

WQv	=	(1.3) (0.302) (1.38) / 12 = 0.04515 acre-feet
WQv	=	0.04515 acre-feet or 1,967 or 1,970 cubic feet

For post-development basin B3B, a rain garden with its lowest control structure inlet 3.2 feet above the garden base will provide a water quality storage volume of 3,065 cubic feet, greater than the 1,970 cubic feet required for the basin it is proposed to serve.

Post-development Basin B3C (Lot 6 Residence to Upper Lot 6 Pond)

WQv	=	water quality volume (acre-feet)
P	=	1.3
Rv	=	0.95 (I = 100%)
A	=	0.11 acres

WQv	=	(1.3) (0.95) (0.11) / 12 = 0.01132 acre-feet
WQv	=	0.01132 acre-feet or 493 or 495 cubic feet

For post-development basin B3C, a rain garden with its lowest control structure inlet 1.2 feet above the garden base will provide a water quality storage volume of 785 cubic feet, greater than the 285 cubic feet required for the basin it is proposed to serve.

Post-development Basin B3D (Lot 6 Driveway to Lower Lot 6 Pond)

WQv	=	water quality volume (acre-feet)
P	=	1.3
Rv	=	0.41 (I = 40%)
A	=	0.24 acres
WQv	=	(1.3) (0.41) (0.24) / 12 = 0.01066 acre-feet
WQv	=	0.01066 acre-feet or 464 or 465 cubic feet

For post-development basin B3D, a rain garden with its lowest control structure inlet 1.2 feet above the garden base will provide a water quality storage volume of 745 cubic feet, greater than the 465 cubic feet required for the basin it is proposed to serve.

7.2.8 Optional Water Quality Methods

Optional methods of achieving the required water quality storage volume for roof drainage and/or driveway drainage include the use of subsurface infiltrators; seepage pits (galleys) or dry swales. Examples for sizing these structures to provide the required storage volumes previously listed are as follows:

7.2.8.1 Infiltration chambers

Infiltrator chambers as manufactured by Cultec, model Recharger 330 or equivalent surrounded by gravel will provide 10.4 cubic feet of storage per linear foot. 20 linear feet of infiltrator chamber will therefore provide 208 cubic feet (10.4 x 20) of storage, or adequate capacity to replace one drywell. 40 linear feet of infiltrator chambers would provide 416 cubic feet of storage, and 80 linear feet of infiltrator chambers would provide 832 cubic feet of storage.

7.2.8.2 Seepage Pits (Dry Well)

Seepage pits having a diameter of 6′ or 8′ with varying depths from 4 feet to 6 feet, surrounded by gravel will provide from 113 cubic feet to 301 cubic feet of storage per pit.

7.2.8.3 Dry Swales

Dry swales having dimensions of a 4′ base x 10′ top width x 18″ depth will provide 10.5 cubic feet of storage per linear foot. 20 linear feet of dry swale will therefore provide 210 cubic feet (10.5 x 20) of storage. 40 linear feet of dry swale would provide 420 cubic feet of storage, and 80 linear feet of dry swale would provide 840 cubic feet of storage.

7.2.9 Additional Water Quality Features

While the above water quality practices meet or exceed the objectives of the stated water quality goals and NYSDEC requirements, the development of this site provides for additional non-standard water quality features to further protect downstream areas. These are discussed as follows:

- The proposed limits of disturbance encompass only 7.9 acres of the 46.6 acre site and are the least extensive as is practicable. The limits of disturbance will be staked prior to construction and delineated with orange construction fencing which shall remain throughout construction until areas are stabilized. The majority of the site which includes wetlands, buffers and more steeply sloped areas will be left undisturbed and the proposed development will be located substantially in previously developed areas of the site.
- Standard stormwater structures are sized beyond standard practice requirements to infiltrate at least the full runoff from the one year runoff events. The facility at the western end of the driveway serving lots 3 and 4 is sized to infiltrate up to the 100 year runoff event. This oversizing of the facilities will provide stormwater infiltration far beyond that required, providing additional groundwater recharge and maintaining stream baseflows in the vicinity.

7.2.10 Mitigation

Implementation of the proposed stormwater management plan will result in a net reduction of stormwater peak flows leaving the site and therefore a reduction a reduction of stormwater flow downstream than currently exists.

For the treatment of stormwater, the project will employ the use of several stormwater BMPs, which include an extended detention pond, a water quality basin, wooded filter strips with level spreaders preceding them, grassed swales, and other adjunct measures as previously described.

Proper implementation and construction of both the stormwater drainage and quality infrastructures and maintenance of the erosion and sediment control plan will mitigate any potential adverse impact to either upstream or downstream drainage areas or facilities due to stormwater quantity or quality as a result of the proposed development (See Exhibit 7-5, Stormwater Drainage Plan).

7.3 Public Water Supply (See Exhibit 7-7, Utility Plan)

7.3.1 Existing Conditions

The Village of Tarrytown has two sources of water and the Water District currently supplies approximately 1.586 million gallons of water per day to residents and other consumers within its service area. The primary, year round source of water is provided by the New York City Catskill Aqueduct system. The Village is connected to the Aqueduct just south of the Kensico Reservoir and due to its high quality of water, is not filtered.

As a supplemental supply in case of an emergency or if there are low flows or repairs in the Catskill Aqueduct, the Water District takes water from the New York City Croton Aqueduct. The Croton Aqueduct can supply 4 million gallons of water per day to the Village and is also not filtered.

Water from both the sources is disinfected by the Village with Chlorine and meets Federal and State microbiological standards.

The population of the Tarrytown Water District is approximately 12,000 people, which includes approximately 2,500 service connections. In 2009, the Tarrytown Water Department delivered over 580 million gallons of water from the Catskill Aqueduct only with the following flows:

- Annual Average: 1.586 mgd
- Maximum month: 1.804 mgd
- Maximum day: 3.425 mgd

The estimated unaccounted for water in the Tarrytown Water System is approximately 23% and is based on the amount of water pumped against the amount of water sold. The unaccounted water includes water lost due to water main breaks, fire fighting, street cleaning, hydrant flushing and other unrelated uses of water

The average household within the district uses approximately 11,200 cubic feet (83,776 gallons) of water per year. This is approximately 230 gallons per day per household.

The Village of Tarrytown Water District owns and operates two water storage tanks. The high service tank is located above 620 South Broadway and has a capacity of 4.4 million gallons and the 900,000 gallon low service tank is located north of Sunnyside Avenue off Neperan Road. There is also a 50,000 gallon air break tank located behind Shaft 10 by Tower Hill Road. All water is fed to the air break tank where it is chlorinated and chemically treated prior to distribution.

The project site is generally served by the high service tank. Based on the USGS Quadrangle, the tank base is at an elevation of approximately 410, with the water flow line at approximately elevation 470. The project site has elevations ranging from approximately 170 to 320.

A review of the report prepared for the Village of Tarrytown by ADS Environmental Services, static pressures in the project site range from approximately 110 psl to 144 psi. The following are descriptions of the fire hydrants in the project vicinity:

Date Tested	Hyd #	Location	Static Pressure psi	Residual Pressure, psi	Fire Flow Available at 20 psi	Hydrant Rating
01-10-08	303	End of Sheldon Ave	144	20	110	1
02-07-08	314	Gracemere	120	104	4170	1
01-15-08	327	Walnut St. at Woodlawn St	110	80	430	1
02-07-08	328	9 Gracemere (on-site)	120	100	290	2

Hydrant condition 1 is described as good and hydrant condition 2 is described as useable but needs maintenance. Overall, the water system in the project vicinity provides ample water pressures and flows to serve the proposed subdivision lots. The hydrant at the end of Sheldon Avenue does not provide sufficient fire flow, but it is not utilized by this development.

The proposed water mains on site will connect to the Gracemere water main. The water system serving the proposed lots will provide more than adequate flows for the new homes.

The western portion of the project site contains an existing 16″ ductile iron water main transmission line that runs from the high service tank westerly to Route 9 (through the Jardim Estates subdivision). Branching off of this 16″ main is a 12″ cast iron main that runs northwesterly through Browning Lane and an 8″ main that runs easterly into a small cul-de-sac, the Gracemere roadway and another spur that terminates in the private road. There is an existing 6″ water main end at the south end of Woodlawn Street (near the north property line of the project site).

7.3.2 Potential Impacts

The proposed subdivision will consist of three existing residences and nine new residences for a total of twelve residential units. However, one of the residences, lot 5, will be serviced with an individual drilled well since its isolated location makes it impractical, both environmentally and economically, to provide a municipal water service to it. Therefore, for the conventional subdivision plan there will be eleven houses that will utilize the Village water system. Lots 1 and 2 will retain their existing water service connections and the remaining nine lots will be supplied with water via individual water service taps and connections to the new water mains to be constructed for this subdivision.

The new water mains will be 8" ductile iron pipe, extending to the end of the proposed cul-de-sac and a spur running along the common driveway serving lots 3 and 4. Each run will dead-end and be provided with a fire hydrant near each end.

It is estimated that the daily domestic water demand for the proposed project is approximately 4,800 gallons per day (gpd) for all twelve lots (4,400 gpd without lot 5). This is based on an average of four people per household and a daily design rate of 100 gallons per person per day or 400 gpd per household for the twelve households.

The proposed water distribution system would also serve the fire protection needs of the project. The existing fire hydrants will be utilized and new fire hydrants will be installed along the new water mains. The National Fire Protection Association recommends that the water system be capable of delivering 750 gallons per minute at a minimum pressure of 20 pounds per square inch at each fire hydrant location. Based on the existing static pressures, pipe sizes in the project site and recorded residual flows, the new water mains will provide the necessary fire flows for each of the new residences in the project.

All new water mains and appurtenances will be constructed at no cost to the Tarrytown Water District or to any Village tax payers. All work will be paid for by the developer of the project and all construction will be in accordance with the requirements of the Westchester County Department of Health and the Village of Tarrytown Department of Public Works. Following acceptance of the water main system and appurtenances by the Westchester County Department of Health, the water main system will be offered for dedication to the Village at no cost.

It is believed that there will be no adverse impacts on the existing water system as a result of this project and the subsequent extension of the existing water main. In total for the conventional subdivision, there will only be an additional 8 new residences connected to the Village water system. it is anticipated that this modest number of homes will not have any impact to the Village of Tarrytown public water supply

7.3.3 Mitigation

The potential impacts of this project are minor and no adverse impacts are expected. The water main extension is relatively short and will serve only eight new homes. The water main extension will be constructed in accordance with all County and

Village requirements. However, with the use of water saving devices in the new residences, water demand will be kept to minimum levels.

7.4 Sanitary Sewer

7.4.1 Existing Conditions

Currently, all areas in the vicinity of the project site are served by Village of Tarrytown public sanitary sewers. The public sewer mains convey wastewater to the Tarrytown Pump Station which then conveys wastewater to the North Yonkers Pump Station. From this pump station, wastewater is conveyed to the Westchester County Yonkers Joint Wastewater Treatment Plant, located in the City of Yonkers.

The Yonkers Joint Wastewater Treatment Plant has a permitted capacity of 120 million gallons per day (mgd). Under the terms of an amended SPDES permit, the Plant is allowed to discharge approximately 120 mgd between the months of June through October. During the months of November through May, the Plant is allowed to discharge 145 mgd. The Plant's NYSDEC Permit number is 3-5518-00342/00002 and was issued on 03-22-06. Based on a conversation with the Plant Manager, the Plant operates at an average daily flow of approximately 103 mgd, which will not, under normal operating circumstances, exceed its permitted discharge allowance.

Presently there is an 8" sanitary sewer main in Woodlawn Street with an existing sewer manhole located at the intersection of Woodlawn Street and Walnut Street. However, access to this manhole would require disturbance to regulated steep slopes and therefore is not proposed as a connection for Jardim Estates East. The existing manhole at the entrance to Gracemere will be used for the sanitary sewer connection from Jardim Estates East.

Based on the recent construction of the first Jardim Estates Subdivision, there were no sewer line problems in the vicinity of the project or to the Tarrytown Pump Station. Upon successful completion of the sanitary sewer mains and appurtenances and acceptance by the Village, the sewer system will become owned and operated by the Village of Tarrytown as part of the municipal system.

Due to the irregular topography of the site where land slopes away from the homes in all directions, the use of a conventional large diameter gravity sewer system is not practical nor is the conventional gravity system realistic even with a sewage pump/lift station. This is also due to the irregular topography of the site. It is likely that a minimum of two sewage pump stations would be required to collect and convey the wastewater from the project to the existing sanitary sewer system.

The proposed sanitary sewer system for the Jardim Estates East subdivision is a low pressure sewer (LPS) system that is powered by grinder pumps. A low pressure sewer system uses small-diameter pipes and grinder pumps that will be installed at each residence. The grinder pump station collects all of the wastewater from the residence and grinds it into slurry. The wastewater will then be pumped to existing sewer manhole located in the roadway at Gracemere. The existing sewer manhole is owned and operated by the Village of Tarrytown.

While gravity sewer systems often use large mains that are installed in deep trenches, pressure sewer pipes will be as approximately 1 ¼" to 2 inches in diameter and follow the contour of the land. These small trenches will reduce the amount of soil disturbance required for installation. Low-pressure sewer systems are used in areas where a conventional gravity sewer system cannot be installed without excessive environmental impact as the land may be very flat, rocky, hilly, or wet.

The Grinder pumps discharge a finely ground slurry into the small diameter pressure piping. This is a completely pressurized collection and conveyance system that will discharge the wastewater into the existing sanitary manhole. All of the pipes are arranged in a branch network without loops. The actual system consists of conventional drain, waste and vent piping on the inside of the residence. The outlet from the house is connected to the grinder pumps station inlet. The grinder pump station will be installed below grade outside of the residences and are provided with the necessary overflow capacity. Additionally, these pump stations will be duplex units, with each station having two pumps that will operate on alternating cycles. The two pumps will ensure that in the event a pump fails that the pump station will still operate until the failed pump is replaced.

With all of the lands outside of the individual lots being owned and controlled by the proposed Home Owners Association, easements will be provided to the Village of Tarrytown for their ownership and maintenance of the LPS sanitary sewer system. Similarly, the individual pump stations and service lines to the street on each lot will be in an easement dedicated to the Village of Tarrytown for access to the pump stations for maintenance and repair.

All proposed sanitary sewer facilities for this project shall be constructed in accordance with the requirements of the Westchester County Department of Health and the Village of Tarrytown Village Engineer and Department of Public Works.

7.4.2 Potential Impacts

The development of Jardim Estates East will include the construction of a sanitary sewer system that will convey wastewater generated on site to the existing Village sanitary sewer system. Anticipated daily flows from the project site will be slightly less than the daily demand for public water as some of the water used by residents will not go into the sanitary sewer system. Water used for landscaping, lawn irrigation, car washing, pools, etc. will not be conveyed to the sanitary sewer system.

It is estimated that approximately 15% of the daily demand for water would be used for these operations. Therefore, from the daily demand rate of 4,800 gpd (12 homes x 4 bedrooms each x 100 gpd per bedroom) x 85%, 4,080 gpd of wastewater will be entering the sanitary sewer system.

The anticipated daily flow of 4,080 gpd represents approximately 0.004% (four one thousandths of 1 percent) of the daily wastewater flow received at the Yonkers Joint Wastewater Treatment Plant. The modest flows generated by the project would result in minimal effects, if any, on the capacity of the Wastewater Treatment Plant.

Additionally, the proposed LPS system provides significant environmental benefits compared to the conventional sanitary sewer construction. The advantages are as follows:

- A LPS system is not subject to infiltration from groundwater or surface water.
- Minimal disruption to the land as trenching follows the contours of the land.
- No preventative maintenance is required for the system.

7.4.3 Mitigation

It is anticipated that there will be no adverse impact to the existing sanitary sewer system as a result of this development. Therefore, no additional mitigating measures will be required for the proposed LPS sanitary sewer system.

7.5 Transfer of Ownership

All new water mains and appurtenances will be constructed at no cost to the Tarrytown Water District or to any Village taxpayers. All work will be paid for by the developer of the project and all construction will be in accordance with the requirements of the Westchester County Department of Health and the Village of Tarrytown Department of Public Works. Following acceptance of the water main system and appurtenances by the Westchester County Department of Health, the water main system will be offered for dedication to the Village at no cost.

Easements will be provided where the sanitary sewer mains are located on private property. The easements will ultimately become dedicated to and owned and operated by the Village of Tarrytown.

7.5.1 Responsibility for Infrastructure Elements

With regard to responsibility for infrastructure elements and maintenance, enforcement and subsequent capital expenditures, the existing private roadways within the site will remain privately owned. A homeowner's association will be responsible for their ownership and maintenance. The proposed public water distribution system will be dedicated to the Village of Tarrytown and easements provided for public water lines that are located on private property. The proposed wastewater collection system will be dedicated to the Village of Tarrytown and easements provided for the public sewer lines that are located on private property. The proposed stormwater drainage system will be owned, operated and maintained by the homeowner's association and easements provided for the drainage lines that are on private property. The use of all roads within the subdivision are subject to the use of others who hold easement rights.

Drainage easements over individual lots will be reserved by the Developer for the construction and maintenance of drainage facilities. All drainage facilities will be maintained by a homeowners association. As stated above, water and sewer facilities would be constructed by the Developer, according to Village standards and, once constructed, these facilities would be dedicated to the Village.

The Developer will be responsible for infrastructure elements until the Homeowner's Association is in full effect under New York State law.

SOIL DATA CHART– USDA (SOIL SURVEY OF PUTNAM AND WESTCHESTER COUNTIES, N.Y.)

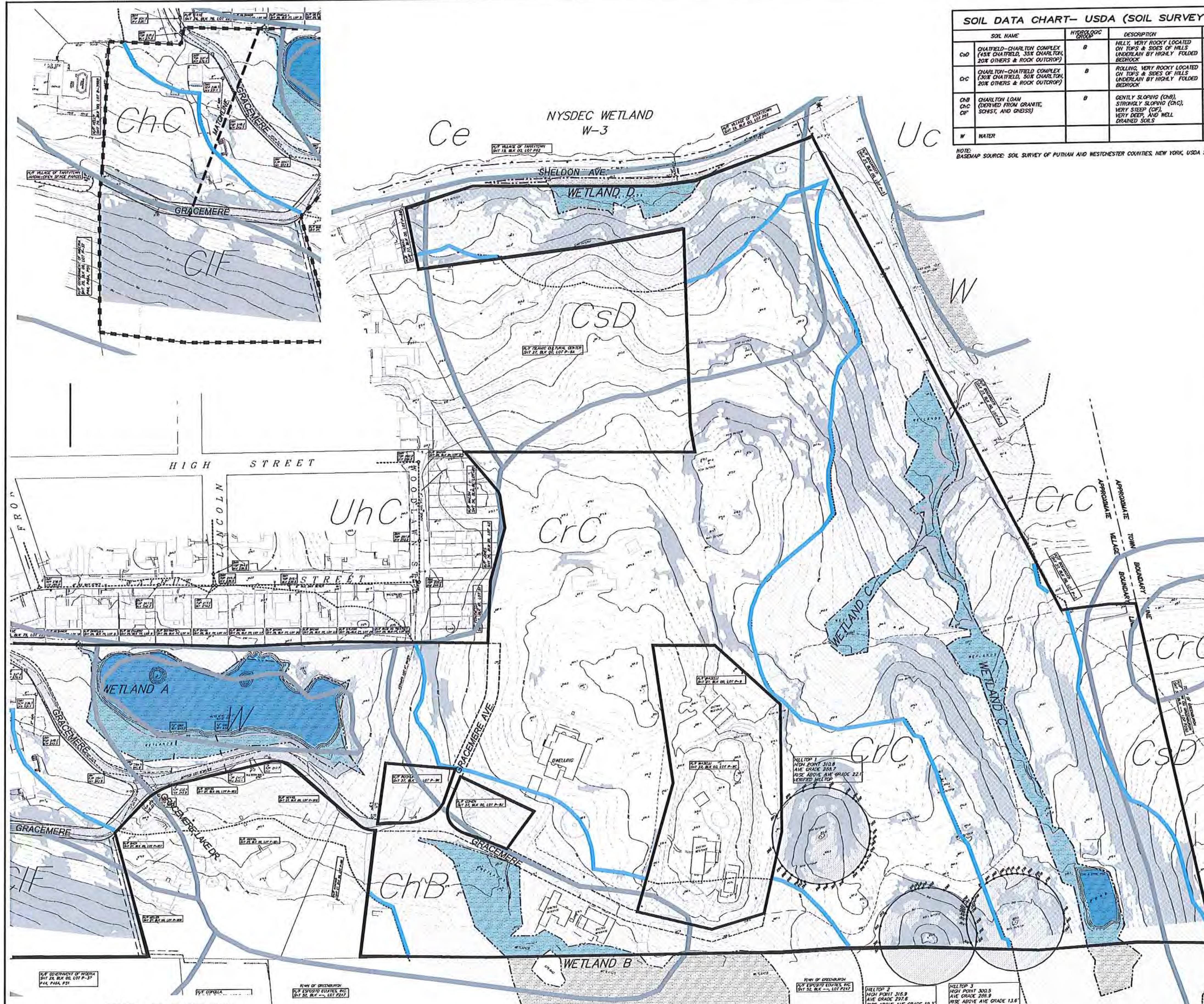
SOIL NAME		HYDROLOGIC GROUP	DESCRIPTION	COMPOSITION	SOIL PROPERTIES	SLOPE
CsD	CHATFIELD–CHARLTON COMPLEX (45% CHATFIELD, 35% CHARLTON, 20% OTHERS & ROCK OUTCROP)	B	HILLY, VERY ROCKY LOCATED ON TOPS & SIDES OF HILLS UNDERLAIN BY HIGHLY FOLDED BEDROCK	SURFACE: VERY DARK GRAYISH BROWN LOAM SUBSURFACE: DARK BROWN LOAM SUBSOIL (CHATFIELD): BR. FLAGGY SILT LOAM (CHARLTON) DARK YELLOWISH BR. S. LM. SUBSTRATUM / BEDROCK: (CHATFIELD) FRACTURED GRANITIC BEDROCK (CHARLTON) DRK. GRAYISH BR. S. LM. & LMY. SAND	WATER TABLE:>6' THROUGHOUT THE YEAR PERMEABILITY: MODERATE OR MOD. RAPID WATER CAPACITY: MODERATE OR LOW EROSION HAZARD: SEVERE DEPTH TO BEDROCK: 20 TO 40" (CHATFIELD SOIL) >60" (CHARLTON SOIL)	15 TO 35%
CrC	CHARLTON–CHATFIELD COMPLEX (30% CHATFIELD, 50% CHARLTON, 20% OTHERS & ROCK OUTCROP)	B	ROLLING, VERY ROCKY LOCATED ON TOPS & SIDES OF HILLS UNDERLAIN BY HIGHLY FOLDED BEDROCK			2 TO 15%
ChB ChC ClF	CHARLTON LOAM (DERIVED FROM GRANITE, SCHIST, AND GNEISS)	B	GENTLY SLOPING (ChB), STRONGLY SLOPING (ChC), VERY STEEP (ClF), VERY DEEP, AND WELL DRAINED SOILS	SURFACE: 0 TO 2", DARK GRAYISH BR. LOAM SUBSURFACE: 2 TO 8", DARK. BR. LOAM SUBSOIL: 8 TO 24", DARK YELLOWISH BR. S. LM. SUBSTRATUM: 24 TO 60", DK. GRAYISH BR. S. LM. W/. LENSES OF LOAMY SAND.	WATER TABLE:>6' THROUGHOUT THE YEAR PERMEABILITY: MODERATE OR MOD. RAPID. WATER CAPACITY: MODERATE EROSION HAZARD: MODERATE DEPTH TO BEDROCK: >60"	ChB–2 TO 8% ChC–8 TO 15% ClF–35 TO 40%
W	WATER					

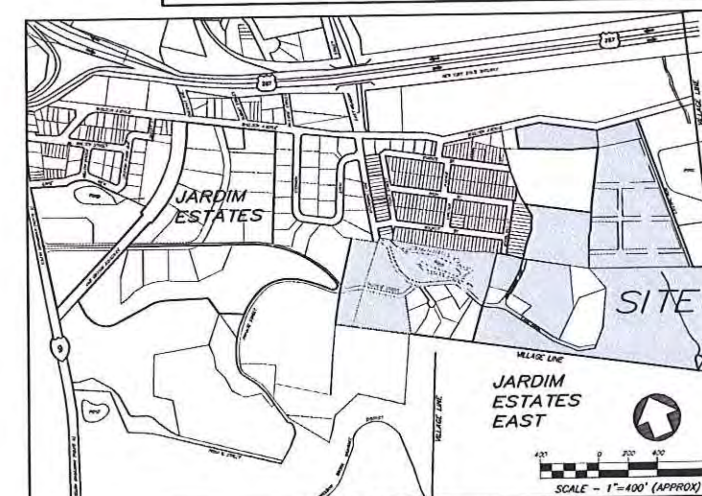
NOTE:
BASEMAP SOURCE: SOIL SURVEY OF PUTNAM AND WESTCHESTER COUNTIES, NEW YORK, USDA SOIL CONSERVATION SERVICE.

NOT TO SCALE

GENERAL NOTES

1. SURVEY AND TOPOGRAPHIC DATA SHOWN HEREON IS TAKEN FROM MAPS PREPARED BY RILEY LAND SURVEYORS, LLP.
2. WETLANDS SHOWN HEREON WERE FLAGGED BY TIM MILLER ASSOCIATES, INC. AND SURVEY LOCATED BY RILEY LAND SURVEYORS, LLP.
3. PROPERTY IS LOCATED IN THE HUDSON RIVER WATERSHED.
4. PRIOR TO COMMENCING ANY WORK THE CONTRACTOR IS TO CONTACT DIG SAFELY NEW YORK (FORMERLY UFPO) (CODE 753) AT 1-800-962-7962.
5. THE LOCATION OF UNDERGROUND UTILITIES SHOWN HEREON ARE APPROXIMATE, THEREFORE ACCURACY, COMPLETENESS AND/OR EXISTANCE OF SUBSURFACE INFORMATION CAN NOT BE CERTIFIED BY THE ENGINEER.





VICINITY MAP

WETLAND AREA CHART		
ITEM	SQUARE FEET	ACRES
WETLAND A	93,800	2.15
WETLAND B	28,337	0.65
WETLAND C	76,591	1.76
WETLAND D	20,428	0.47
TOTAL:	219,156	5.03

NOTE: AREAS GIVEN ARE FOR ONSITE ONLY

STEEP SLOPES CHART		
SLOPE CATEGORY	ACRES	% OF SITE AREA
0% TO 10% SLOPES	[illegible]	33.9
10% TO 15% SLOPES	8.77	20.3
15% TO 20% SLOPES	[illegible]	14.1
20% TO 25% SLOPES	4.21	9.8
25% AND GREATER SLOPES	[illegible]	[illegible]
TOTAL:	43.11*	100.0

* INCLUDES EASEMENT / RIGHT OF WAY AREAS

Exhibit 7-1

EXISTING CONDITIONS

JARDIM ESTATES EAST

Tarrytown, New York

CRONIN ENGINEERING
PROFESSIONAL ENGINEERING & CONSULTING

VHB Saccardi & Schiff

LIMITS OF DISTURBANCE	
DESCRIPTION	AREA
STEEP SLOPES >25%	0 SQ. FT.
	0 ACRES
WETLANDS	5,860 SQ. FT.
	0.13 ACRES
WETLAND 150' BUFFER	72,594 SQ. FT.
	1.67 ACRES
TOTAL	345,227 SQ. FT.
	7.93 ACRES

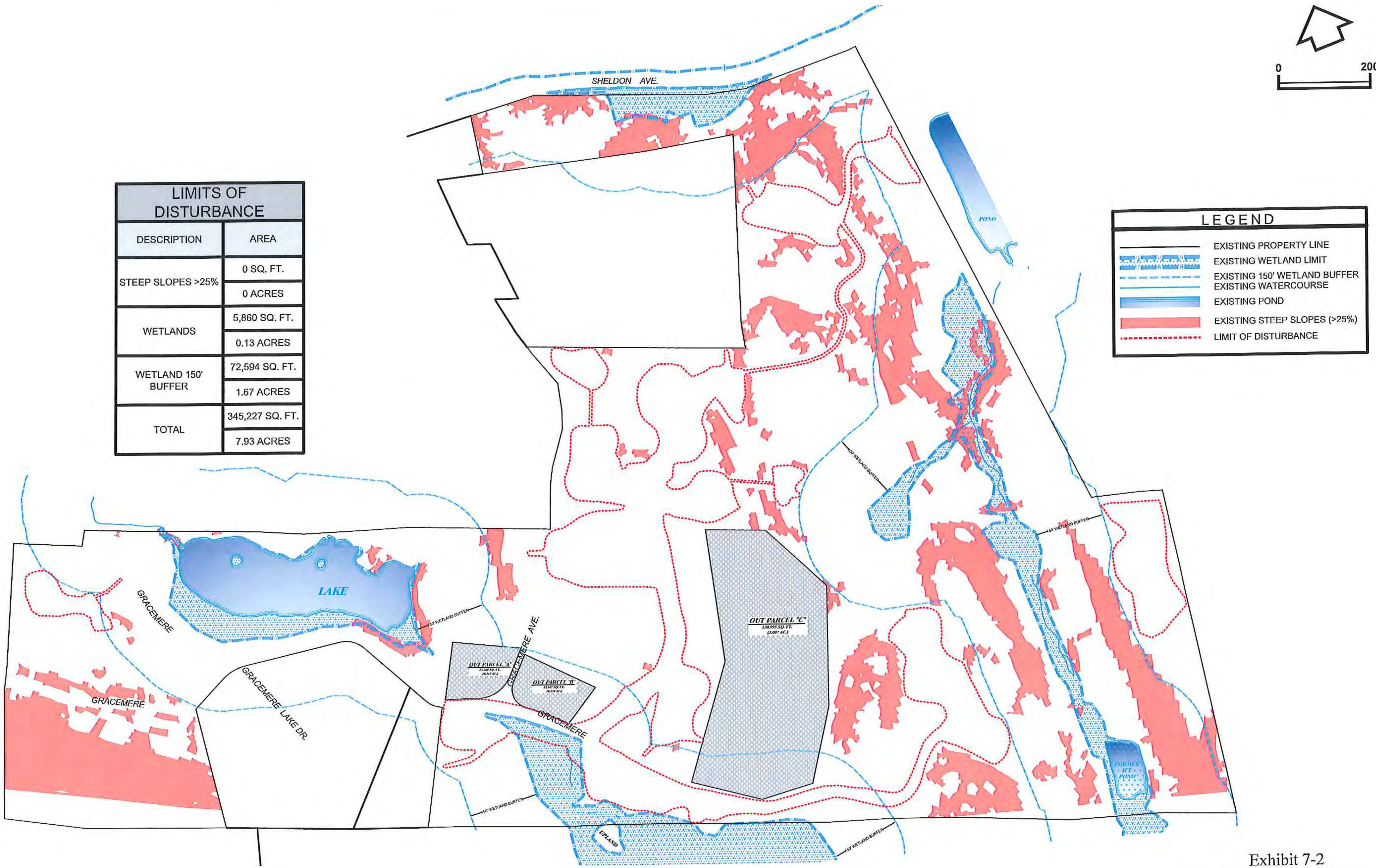


Exhibit 7-2

LIMITS OF DISTRUBANCE

JARDIM ESTATES EAST

Tarrytown, New York

VHB Saccardi & Schiff

CRONIN ENGINEERING

PROFESSIONAL ENGINEERING & CONSULTING

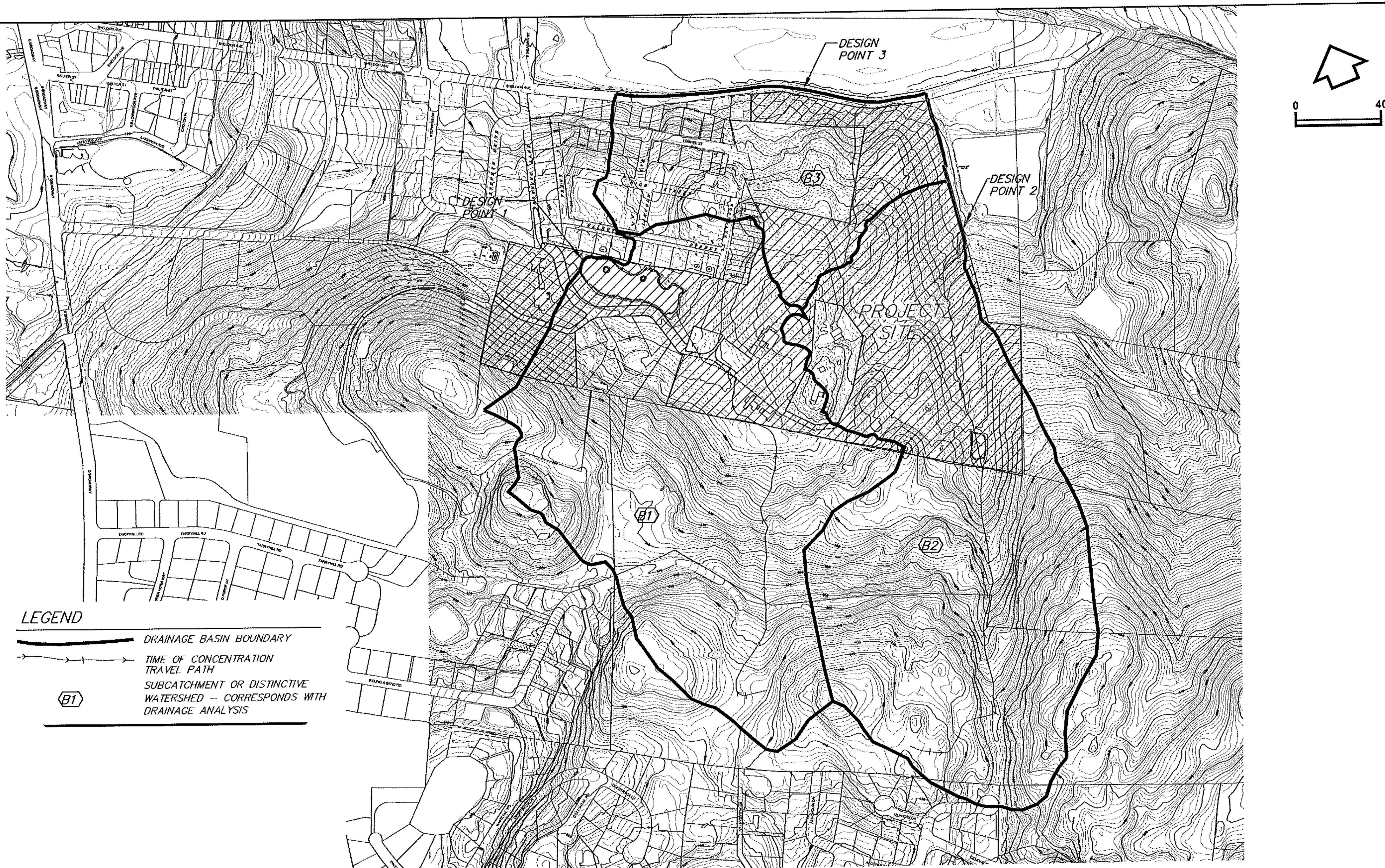


Exhibit 7-3

PRE-DEVELOPMENT DRAINAGE

JARDIM ESTATES EAST

Tarrytown, New York

VHB Saccardi & Schiff

CRONIN ENGINEERING

PROFESSIONAL ENGINEERING & CONSULTING

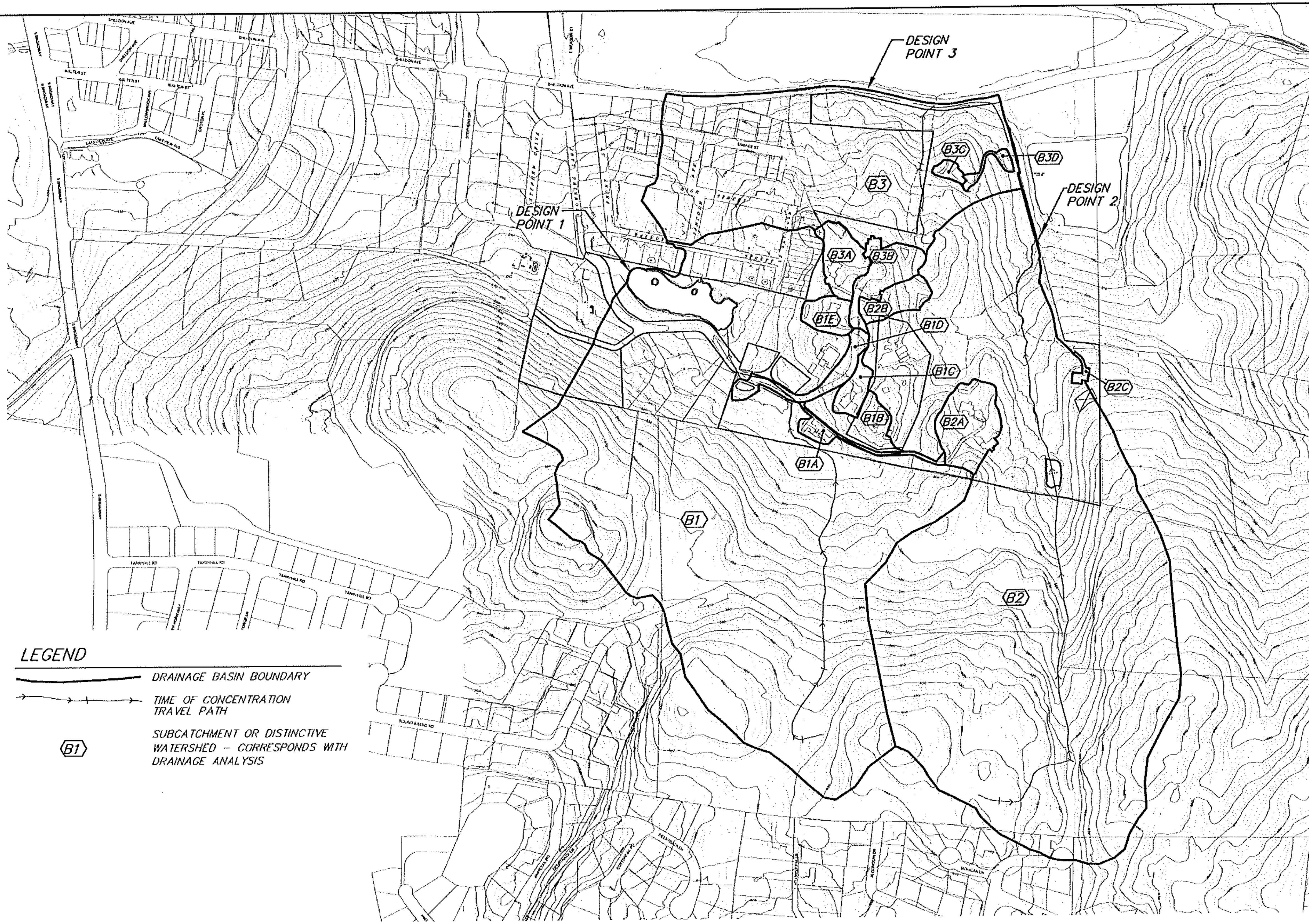


Exhibit 7-4

POST-DEVELOPMENT DRAINAGE

JARDIM ESTATES EAST

Tarrytown, New York

CRONIN ENGINEERING

PROFESSIONAL ENGINEERING & CONSULTING

VHB Saccardi & Schiff

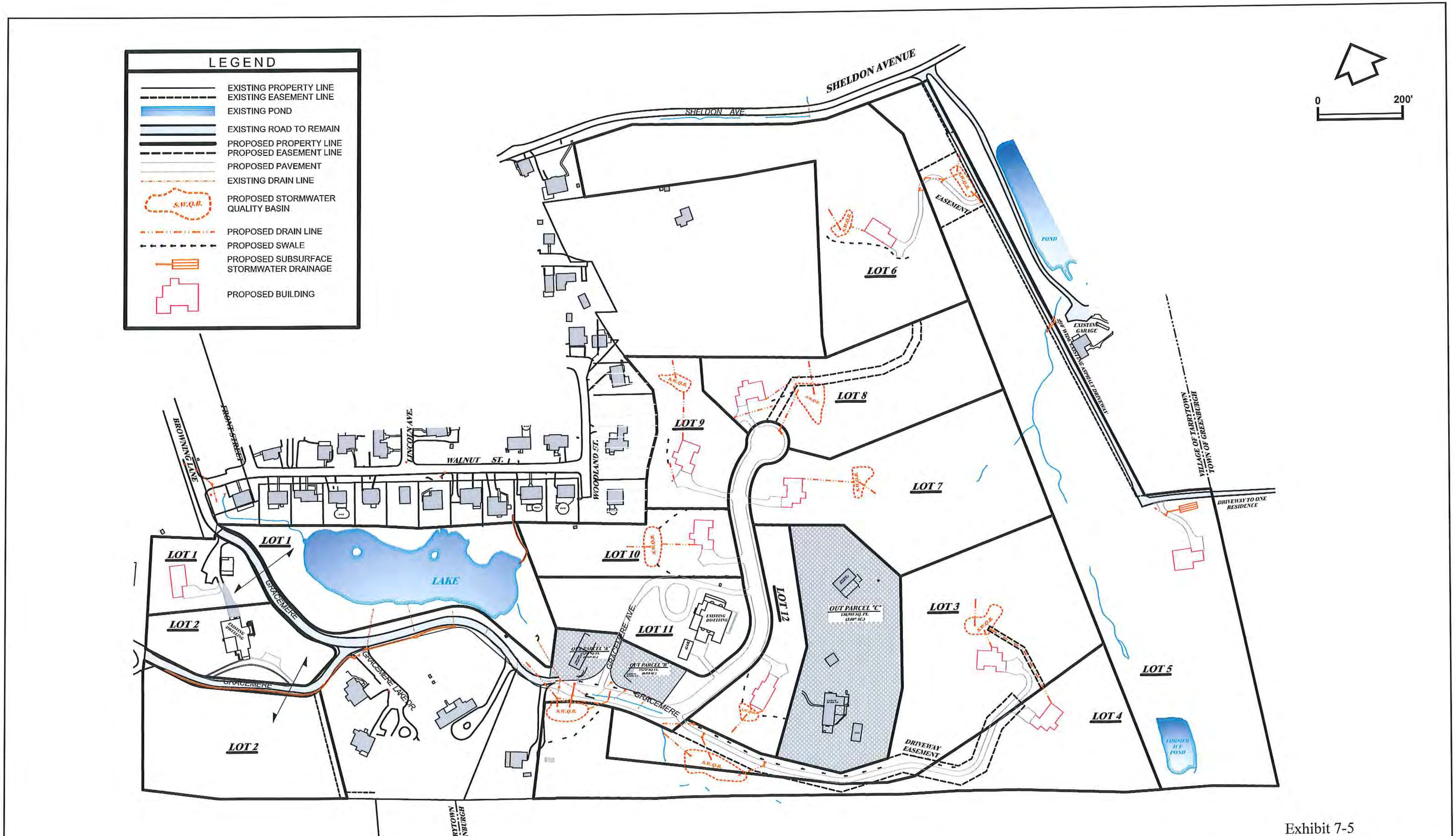


Exhibit 7-5

STORMWATER DRAINAGE PLAN

JARDIM ESTATES EAST

Tarrytown, New York

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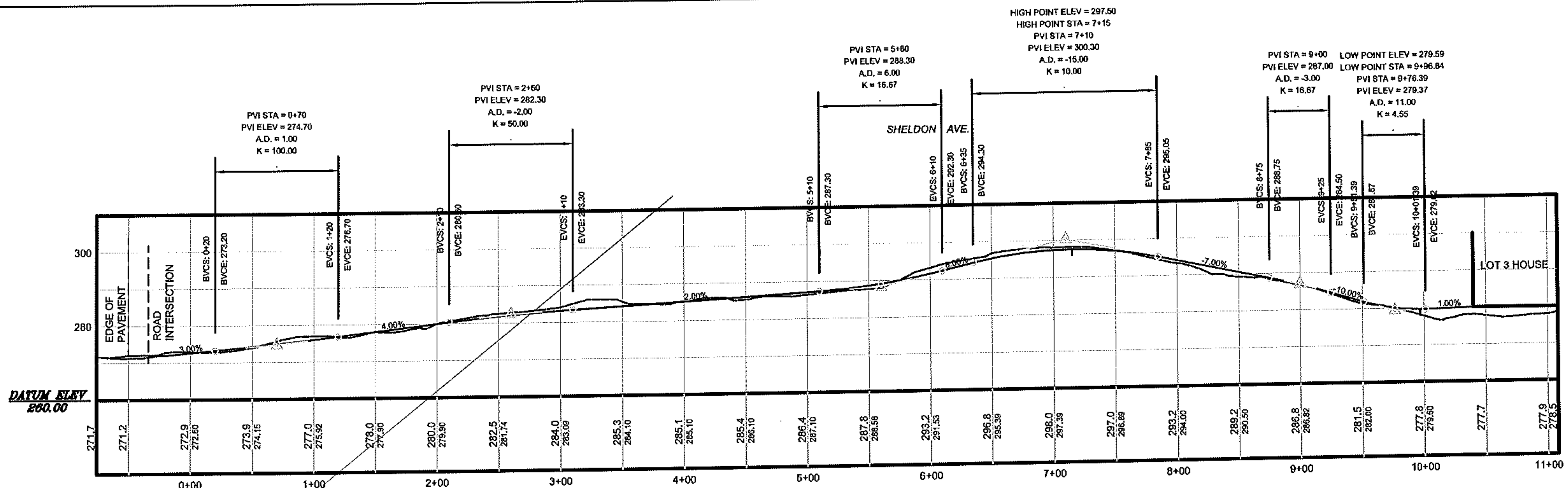
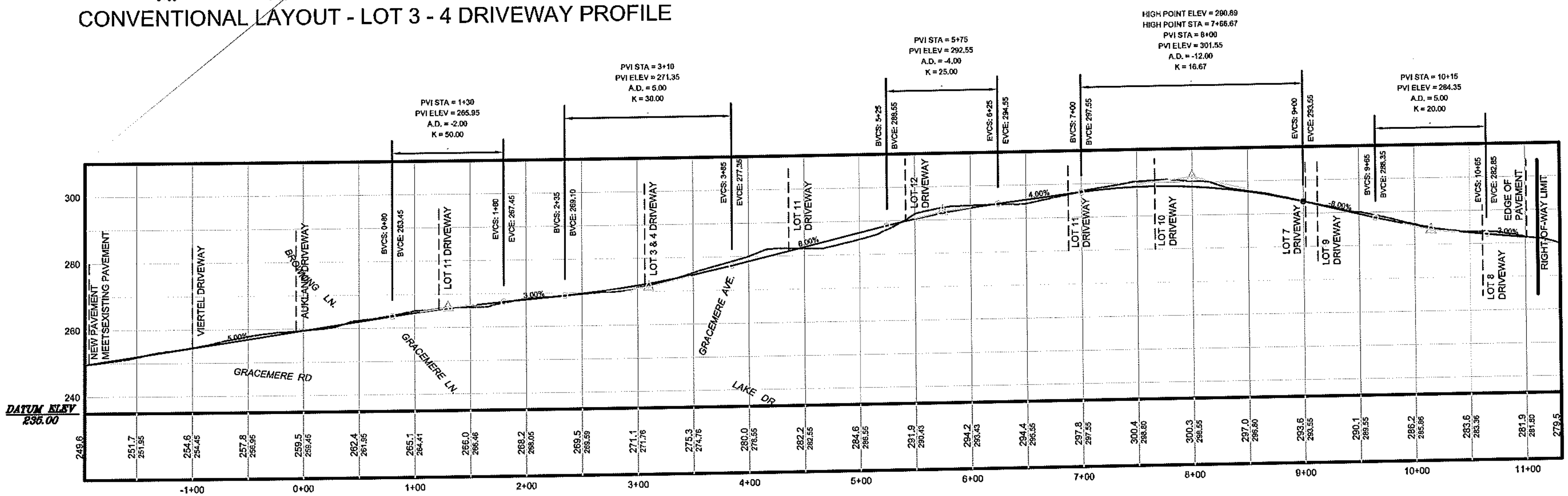


CONVENTIONAL LAYOUT - LOT 3 - 4 DRIVEWAY PROFILE



CONVENTIONAL LAYOUT - PROPOSED ROAD PROFILE

Exhibit 7-6

ROAD PROFILES

JARDIM ESTATES EAST

Tarrytown, New York

VHB Saccardi & Schiff

CRONIN ENGINEERING

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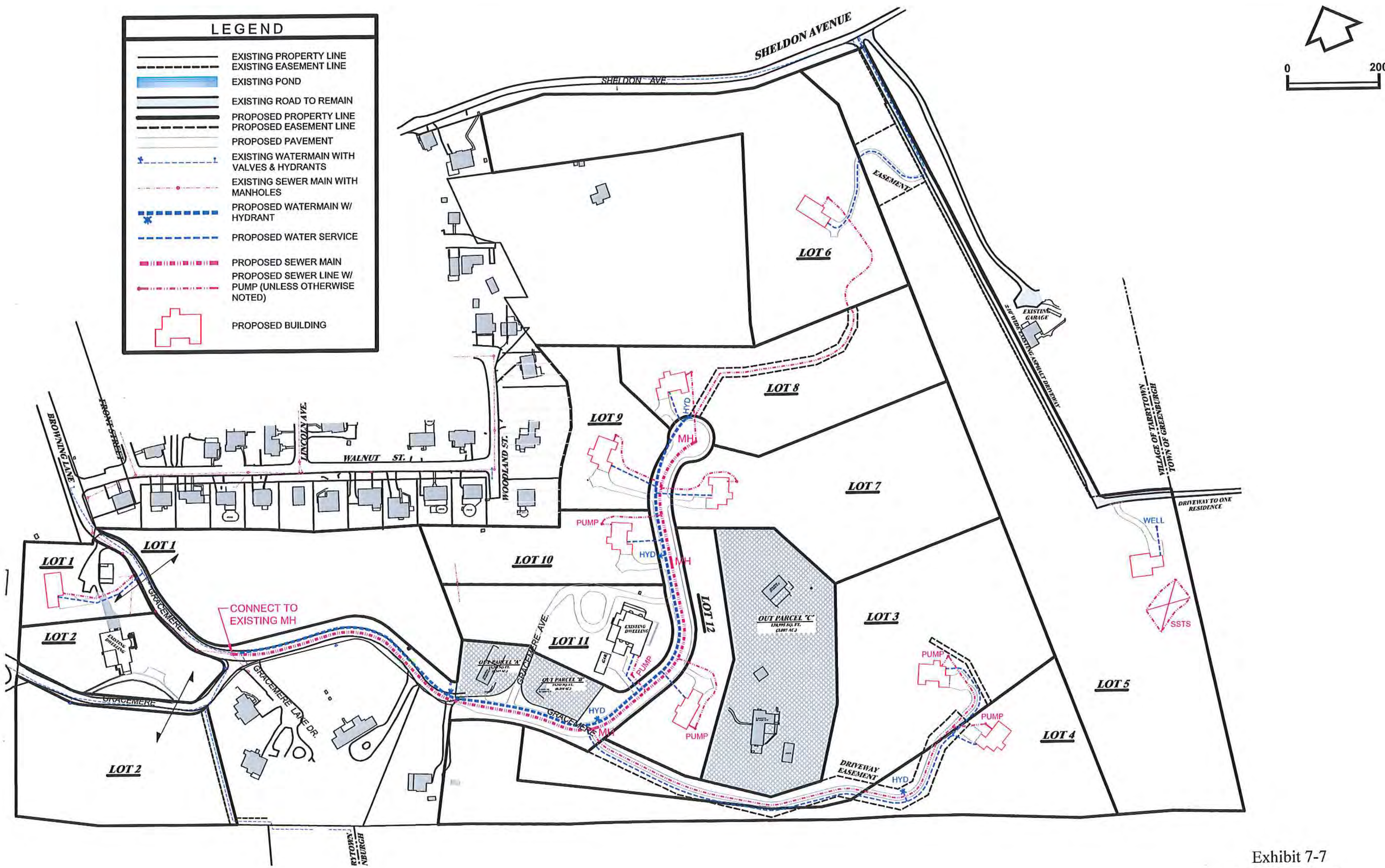

Exhibit 7-7

UTILITY PLAN

JARDIM ESTATES EAST

Tarrytown, New York

VHB Saccardi & Schiff

CRONIN ENGINEERING
PROFESSIONAL ENGINEERING & CONSULTING