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ENGINEERING, SURVEYING & LANDSCAPE ARCHITECTURE, P.C.

Preliminary Water Engineering Report

For

1 Croton Point

Croton Point Ave.
Croton-On-Hudson, New York

October 30, 2024

Prepared By

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1.0 INTRODUCTION

1.1 Project Description

The project consists of a proposed 100-unit, for sale affordable condominium development consisting of 46 one-bedroom and 54 two-bedroom homes in a to be constructed 5-story building with requisite parking and amenities (the "Development") to be located on lands fronting on Croton Point Avenue consisting of (i) Tax Map Parcels 79.17-1-5 & 3 owned by the Village ("Lot A"); and (ii) Tax Map Parcel 79.17-1-3 owned by Croton Point Realty Inc (the "CPR Parcel"). Lot A is currently in use as a Village commuter parking lot. The CPR Parcel is improved with an office building which is proposed to be demolished for the Development.

The proposed brick and metal panel building will be served by 100 parking spaces located in a below grade parking level and two outdoor parking areas, including five (5) Level 2 EV charging stations. Amenities within the building will include a community room, a fitness center, a co-working lounge, a bike storage room and a roof-top deck providing expansive scenic views overlooking Croton Point Bay and the Hudson River.

Sustainable building design features will likely include full electrification, solar readiness, high efficiency HVAC equipment (cold climate air source heat pumps) and appliances (Energy Star Multifamily New Construction Program) and low-flow water fixtures.

In total the property consists of 1.8 ± acres and is located in the LI (Light Industrial with a Transoriented Development Overlay) zoning district, the Village of Corton-on-Hudson Water District and the Ossining Sanitary Sewer District.

1.2 Existing Site Conditions

The subject project is located on Croton Point Avenue across from Veterans Plaza in the Village of Croton-On-Hudson. As previously stated, the site currently exists as a Village Parking lot and commercial development with several buildings, parking areas, walkways, and other appurtenances. It currently exists as some office buildings as well as overflow parking for the Croton-Harmon train station.

1.3 Proposed Site Conditions

It is proposed to construct a 21,500 sf ± (footprint area) affordable condominium development with associated parking and various site appurtenances. The new five-story building will contain one hundred units. There will be 46 one-bedroom apartments and 54 two-bedroom apartments. Amenity spaces will also be constructed which will be used by the residents of the building. As part of the proposed development, there will be a new water and fire service connection to the building. The new service lines will exit the building on the west side and will tie into the existing water main on Croton Point Ave. See Appendix A for Water Main mapping.

2.0 PROJECT DESIGN FLOWS

The project domestic maximum daily water demand used for design is anticipated to be the same as the maximum daily wastewater demand. As such the design maximum daily water flows for the proposed project, are based on the hydraulic loading rates given in the New York State Department of Environmental Conservation (NYSDEC) publication *Design Standards for Intermediate Sized Wastewater Treatment Works – 2014* (DEC 2014). The design maximum daily water demand is a conservative design flow on which the water infrastructure will be designed. This value does not represent the average daily demand, which is expected to be less.

The design flow rates for the residential units are based on the number of bedrooms per the tables below. The following table calculates the hydraulic loading rates, and the design flow rates (gallons per day or gpd) for the proposed project.

Proposed Use	# of Units	Hydraulic Loading Rate	Design Maximum Daily Flow (gpd)
One Bedroom Units	46 Units	110 gpd/bedroom	5,060
Two Bedroom Units	54 Units	110 gpd/bedroom	11,880
Total			**16,940**

The anticipated design average daily flows for the project are expected to be significantly less than the design maximum daily flow rate. The design maximum daily flows represent conservative flows to ensure that the proposed sewer works are designed with an ample factor of safety. The anticipated actual flows are based on anticipated occupancy rates and measured data for water use. Statistical data (obtained from *Rutgers University, Center for Urban Policy Research*, *Residential Demographic Multipliers*, June 2006) for the average number of occupants in rental units (based on number of bedrooms) was used to calculate the expected number of residents anticipated for the project as shown in the table below. Data from the American Water Works Association (AWWA) Water Conservation Division Subcommittee Report, Water Conservation Measurement Metrics Guidance Report, dated January 2010 shows that the average in home water use is 69.3 gpd per person. This number is reduced to 43.5 gpd per person when water saving fixtures are used, which is the case for this project. The following table below will be used to calculate flow to the existing sewer main. The 45 gpd per person was used to calculate the proposed anticipated average daily design as water saving fixtures will be used. See the table below for the calculation of the design average daily flow rates in the existing and proposed conditions.

Table 2.3: Croton Point - Proposed Design Average Daily Flow Rate

Proposed Use	Occupancy Rate	Total Anticipated Residents	Water Use Per Resident (gpd)	Water Use (gpd)
46 – One Bedroom Units	1.206 people/unit	56	43.5	2,436
54 –Two Bedroom Units	1.689 people/unit	91	43.5	3,959
			Total Anticipated Water Use (gpd)	**6,395**

As demonstrated above, through the use of water saving fixtures as required by current building code, a design maximum flow of 16,940 gpd is proposed for the project, while the actual anticipated flows are 6,395 gpd.

The peak hourly flow is calculated using a peaking factor that is based on the population of the subject project. A peaking factor of four will be used for the project based on Figure 1 from Recommended Standards for Wastewater Facilities.

Peak Hourly Domestic Flow

16,940 gpd ÷ (24 hr/day) ÷ (60 min/hr) = 11.8 gallons per minute (gpm)

Peak Hourly Flow = 11.8 gpm x 4 = 47 gpm

Although the anticipated flows (average daily design flow) for the project is significantly lower than the design maximum daily flows, the Peak Hourly Flow based on the design maximum daily flows are used for the design of the system. This provides a factor of safety in the proposed design.

The new building is required to have fire sprinklers. The requirements for fire sprinkler systems were preliminarily established for the project based on review by the MEP engineer. The fire sprinkler demand for the building is dictated by the required flow for the project and was provided 750 gpm for 60 minutes. The final fire sprinkler system will meet NFPA requirements and be designed by the MEP engineer during the building permit process.

3.0 PROPOSED CONNECTION TO VILLAGE OF CROTON-ON-HUDSON SYSTEM

3.1 System Characteristics

Based on mapping provided of the existing system there is an existing 6 inch main in Croton Point Avenue that services the existing structures on the subject property. There is also an existing 10 inch main in Wayne Street. See Appendix A for the Water Main mapping and hydrant flow test results.

3.2 Proposed Water Service Connection and Hydrant

The existing water main which the project proposes to connect to is in Croton Point Avenue. As previously discussed, two separate service lines, one for domestic and one for fire service are proposed. Based on the available mapping provided in Appendix A the existing watermain is a 6" diameter pipe in Croton Point Avenue.

All water service piping will be Class 52 DIP. Restrained joint connections will be provided at all pipe bends through the use of Mega-lug fittings or approved equal. In addition, thrust blocks will be provided at all bends. Upon completion of the water service installation pressure testing, disinfection, and flushing will be performed in accordance with AWWA standards. At the request of the fire department, a fire hydrant is proposed to connect to the existing water main in the vicinity of the project.

In addition to the water service connection proposed, a hydrant will be added to the existing main in front of the project site. The hydrant was requested by the Village Fire Department and will connect to the existing system.

Recommended Standards for Water Works (RSWW) recommends that the normal working pressure not be below 35 psi, and both the RSWW and the American Water Works Association (AWWA) M 31 recommend that a minimum of 20 psi be maintained at all points in the water distribution system during fire flows.

Hydrant flow tests were completed on October 15, 2024. Three hydrant flow tests were performed including Wayne Street, and two locations on Croton Point Avenue. See Appendix A for additional information regarding hydrant flow testing. The results are summarized in the following table.

Location	Flow (gpm)	Static Pressure (psi)	Residual Pressure (psi)
Wayne Street	578	121	91
Croton Point Ave. (Veterans Plaza)	391	149	39
Croton Point Ave.	427	146	58

Based on this testing the existing 6" water main in Croton Point Avenue can supply the domestic needs for the project. However, for the fire flow, the 6-inch water main that feeds the 6 inch line in Croton Point Avenue provides less than 500 gal/min at 20 psi thus not meeting the project fire flow demand. Improvements would be required to provide the 750 gpm sprinkler demand. Two alternatives are discussed below including:

1. Improvements to the distribution system via a connection between the existing Croton Point Ave Water Main and the Wayne Street Water Main (see calculations in Section 3.4 below).

2. Onsite water storage and pump system (see discussion in section 3.5 below).

3.3 Calculation for Residual Pressure at Service Connection for Peak Domestic Flow

The following calculations determine the residual pressure at the service connection to ensure that the minimum pressure of 35 psi is met at the highest service connection as required by RSWW. The calculations assume no distribution system improvements (Wayne Street Extension). The residual pressure will be calculated for peak domestic demand of 47 gpm as calculated in section 2.0:

1. Calculate Static Pressure at First Floor Elevation (FFE)

Static Pressure at Hydrant (Wayne Street)		=	121 psi
Elevation at Hydrant (Wayne Street)		=	88 ft
Elevation of Proposed Service Connection		=	35 ft
Difference in Elevation of Hydrant & FFE	(35 ft – 88 ft)	=	-53 ft
Equivalent pressure Associated with Change in Elev.	(53.0 ft / 2.31 psi)	=	23 psi
Static Pressure at FFE	(121 psi + 23 psi)	=	144 psi

2. Calculate Head Loss in System to Residual Pressure Hydrant at Peak Domestic Flow

The equation below is taken from AWWA M17. The equation is used to calculate flow available at different pressures or differences in the residual pressure that would result from different flow rates. Here the equation is used to calculate the residual pressure (at the observation hydrant) for the domestic design flow of the building, using the pressures and flow rates measured during the flow test. The proposed domestic water service lines will be sized for a flow of 47 gpm.

Domestic Flow Calculation:

$$Q_R = Q_F * h_r^{0.54} / h_f^{0.54}$$

Where:

- Q_R = peak flow (47 gpm)
- Q_F = flow from hydrant during test (391 gpm)
- h_r = the difference in pressure between the static pressure measured at the observation hydrant and the residual pressure at the total combined flow
- h_f = the difference between the static pressure and residual pressure measured at the observation hydrant during the flow test, (110 psi)

$$47 \text{ gpm} = 391 \text{ gpm} * h_r^{0.54} / 110 \text{ psi}^{0.54}$$

$$h_r = 2 \text{ psi}$$

3. Calculate Head Loss at Peak Domestic Flow from Residual Hydrant to Building

Total Head Loss Due to Friction (See Spreadsheets in Appendix B) = 4 psi

Domestic Flow Calculation:

As noted above the static pressure at the FFE is 144 psi. A head loss of 2 psi was calculated in the system at peak domestic flow to the residual hydrant. An additional 4 psi of head loss was calculated in the existing water main from the residual hydrant to the proposed service line and in the proposed service line to the building. This results in a calculated pressure at FFE of:

144 psi – 2 psi -4 psi = 138 psi

As noted above the 138 psi pressure under peak hourly flow conditions exceeds the RSWW requirement of 35 psi for peak hourly domestic flow conditions. The existing system can supply the peak hourly domestic flow without distribution system improvements. As the pressure exceeds 100 psi, a pressure reduction valve should be provided on the domestic plumbing entering the building.

3.4 Calculation for Residual Pressure at Service Connection for Combined Peak Domestic Flow Plus Fire Flow (with Wayne Street Connection)

It was noted by the Village a 10-inch water line was installed in Wayne Street and a stub installed in Croton Point Avenue to allow the extension of the 10-inch line in Wayne Street to Croton Point Avenue. The following flow and pressure calculations assume that connection is made.

The equation below is taken from AWWA M17. The equation is used to calculate flow available at different pressures or differences in the residual pressure that would result from different flow rates. Here the equation is used to calculate the residual pressure (at the observation hydrant) for the domestic design flow of the building, using the pressures and flow rates measured during the flow test. The proposed domestic water service lines will be sized for a flow of 47 gpm. The fire service line will be sized for a flow of 797 gpm which is the combination of the peak hourly domestic flow of 47 gpm and the required sprinkler flow of 750 gpm.

Head Loss in System at Fire Flow Calculation:

$$Q_R = Q_F * h_r^{0.54} / h_f^{0.54}$$

Where:

- Q_R = peak flow (797 gpm)
- Q_F = flow from hydrant during test (578 gpm)
- h_r = the difference in pressure between the static pressure measured at the observation hydrant and the residual pressure at the total combined flow
- h_f = the difference between the static pressure and residual pressure measured at the observation hydrant during the flow test, (30 psi)

$$797 \text{ gpm} = 578 \text{ gpm} * h_r^{0.54} / 30 \text{ psi}^{0.54}$$

$$h_r = 54 \text{ psi}$$

Fire Flow Calculation:

As shown in Appendix A, a head loss of 7 ft (3 psi) is calculated in the fire service line, 52 ft (22 psi) in the existing 6” diameter water main and 4 ft (2 psi) in the existing and proposed 10” diameter water main. The static pressure of 144 psi and system head loss of 44 psi to the Wayne Street Hydrant are calculated above. This results in a calculated pressure at building FFE of:

144 psi – 3 psi- 22 psi- 2 psi -54 psi = 63 psi

As noted above the 63 psi pressure under combined domestic and fire flow conditions exceeds the RSWW requirement for 20 psi throughout the system. The project MEP engineer will complete design of the fire sprinkler system as part of the building permit process.

3.5 Water Storage Tank Alternate for Fire Protection

As noted above, with an extension of the 10" watermain from Wayne Street to Croton Point Avenue, the existing system can supply the required water flow for the fire sprinkler system. While this connection has long been envisioned and desired by the Village the Wayne Street extension requires an easement from a private property owner. Should the easement not be able to be secured, and the Wayne Street extension cannot be completed, an onsite water storage tank and pump system would be used to supplement the available flows from the existing 6" watermain in Croton Point Avenue to meet the fire sprinkler requirements.

The MEP engineer has determined that a 60 minute duration is required for the 750 gpm sprinkler flow. For this flow rate and duration, a water use of 45,000 gallons is calculated. This water volume would be provided by a combination of the available flows from the existing system, the ability to supply the proposed hydrant, and supplemental onsite storage. Through the site plan process the applicant will work with the Village to determine the required size of the supplemental storage tank. The tank would be buried below grade.

Pump(s) would be provided to deliver the required supplemental flow from the storage tank to the sprinkler system. The pump(s) would be in the building mechanical room or in a vault below grade. A back up generator will be provided to power the pump(s) in case of utility power failure. All water tanks would be buried below grade.

APPENDICES

Appendix A Croton-On-Hudson Water Main Mapping & Hydrant Flow Data
Appendix B Head Loss Calculation Worksheets

Appendix A
Water Main Mapping & Hydrant Flow Data

 WB CROTON POINT (24154.100)

HYDRANT TEST

Flow Tests with Village - 10/15/2024

Manufacturer Mueller No.

	Date, time	Nozzle Size	Pressure			Flow *gpm*	Flow 20 *psi*	Time *min*	Water Used *gal*
			Stat.	Res.	Pitot				
(1)	1:30pm Wayne Street (See map)	2.5"	121psi	91psi	15psi	650gpm (578gpm actual)	1,112gpm	—	—
(2)	2:30pm Vet. Plaza (See map)	2.5"	149psi	39psi	7psi	440gpm (391gpm actual)	479.6gpm	—	—
(3)	2:45pm Croton Point Ave. (See map)	2.5"	146psi	58psi	8psi	480gpm (426.7gpm actual)	582gpm		

Square Nozzle C=0.8

Figure 5-5 Historical record.


Static / Residual Hydrant Test 1
Flow Hydrant Test 1
Static / Residual Hydrant Test 2 & Flow Hydrant Test 3
Static / Residual Hydrant Test 3
Flow Hydrant Test 2

Appendix B
Head Loss Calculation Worksheets

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WB Croton Point
Head Loss Calculations - Fire Flow

Head Loss in Service Line

C	110		Roughness coefficient for ductile iron pipe
d	6	in	**Diameter of water service line**
L	100	ft	Length of water service line
Q	750	gpm	Flow Rate
V	8.5	ft/s	Velocity
L_e	20	ft	Equivalent length to account for losses in valves and bends
L_t	120	ft	Total Length = $L + L_e$
HL	7	ft	$HL = \frac{10.44(L_t)(Q^{1.85})}{(C^{1.85})(d^{4.87})}$
HL (in service)	3	psi	

Headloss in existing 6" main in Croton Point Avenue

C	110		Roughness coefficient for ductile iron pipe
d	6	in	**Diameter of water main**
L	700	ft	Length of water main
Q	797	gpm	Flow Rate
V	9.0	ft/s	Velocity
L_e	80	ft	Equivalent length to account for losses in valves and bends
L_t	780	ft	Total Length = $L + L_e$
HL	52	ft	$HL = \frac{10.44(L_t)(Q^{1.85})}{(C^{1.85})(d^{4.87})}$
HL (in service)	22	psi	

Headloss in 10" watermain in Wayne Street

C	110		Roughness coefficient for ductile iron pipe
d	10	in	**Diameter of water main**
L	570	ft	Length of water main
Q	797	gpm	Flow Rate
V	3.3	ft/s	Velocity
L_e	80	ft	Equivalent length to account for losses in valves and bends
L_t	650	ft	Total Length = $L + L_e$
HL	4	ft	$HL = \frac{10.44(L_t)(Q^{1.85})}{(C^{1.85})(d^{4.87})}$
HL (in service)	2	psi	

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WB Croton Point
Head Loss Calculations - Domestic Flow

Head Loss in Service Line

C	110		Roughness coefficient for ductile iron pipe
d	2	in	Diameter of water service line
L	100	ft	Length of water service line
Q	47	gpm	Flow Rate
V	4.8	ft/s	Velocity
L_e	20	ft	Equivalent length to account for losses in valves and bends
L_t	120	ft	Total Length = $L + L_e$
HL	9	ft	$HL = \frac{10.44(L_t)(Q^{1.85})}{(C^{1.85})(d^{4.87})}$
	4	psi	

Headloss in existing 6" main in Croton Point Avenue

C	110		Roughness coefficient for ductile iron pipe
d	6	in	**Diameter of water main**
L	300	ft	Length of water main
Q	47	gpm	Flow Rate
V	0.5	ft/s	Velocity
L_e	50	ft	Equivalent length to account for losses in valves and bends
L_t	350	ft	Total Length = $L + L_e$
HL	0	ft	$HL = \frac{10.44(L_t)(Q^{1.85})}{(C^{1.85})(d^{4.87})}$
HL (in service)	0	psi	