Appendix G
Interior Drainage Report

North/West Battery Park City Resiliency Project

Interior Drainage Report

May 10, 2024

Version Control

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Note:

1) All = Turner EE Cruz, Arcadis and its subconsultants (SCAPE/BIG/WSP, etc.)
2) Arcadis team = Arcadis and its subconsultants (SCAPE/BIG/WSP, etc.)

CONTENTS

TABLES

FIGURES

APPENDICES

Acronyms & Abbreviations

AEP	Annual Exceedance Probability
BMCC	Borough of Manhattan Community College
BOD	Basis of Design
BPC	Battery Park City
BPCA	Battery Park City Authority
BWSO	Bureau of Water and Sewer Operations
BWT	Bureau of Wastewater Treatment
CCTV	Closed Circuit Television
CSO	Combined Sewer Overflow
DEM	Digital Elevation Model
FBS	Flood Barrier System
FEMA	Federal Emergency Management Agency
H&H	Hydrologic and Hydraulic
HDS	Hydrodynamic Separator
ICM	InfoWorks Integrated Catchment Model
LTCP	Long Term Control Plan
MGD	Million Gallons Per Day
MHHW	Mean Higher High Water
MOCEJ	NYC Mayor's Office of Climate and Environmental Justice
NOAA	National Oceanic and Atmospheric Administration
NPCC	New York City Panel on Climate Change
NSI	Near Surface Isolation
NWBPCR	North/West Battery Park City Resiliency
NYCDEP	New York City Department of Environmental Protection
PCSMP	Post-construction Stormwater Management Practice
PDB	Progressive Design Build
SBPCR	South Battery Park City Resiliency
SLR	Sea Level Rise
USWR	Unified Stormwater Rule

1 Overview and Objectives

1.1 Introduction and Study Area

Battery Park City Authority (BPCA) is currently undergoing a resilience program to protect the community from future storm events. This effort consists primarily of coastal protection, with the construction of a flood barrier system (FBS) starting in the southern area of Battery Park City (BPC) and continuing north. BPCA has engaged Turner EE Cruz in a Progressive Design Build contract (PDB Team) to advance the North/West Battery Park City Resiliency (NWBPCR) project, which stretches from First Place along the Esplanade then crossing West Street and tying in at the intersection of North Moore Street and Greenwich Street. See **Figure 1-1** for a site location map and location of the proposed coastal flood barrier system.

Figure 1-1. Site Location Map

In addition to a flood barrier system to protect against storm surge events, the PDB Team is responsible for managing interior flooding resulting from rainfall and coastal storm surge. Coastal storm surge entering the study area from the City's unprotected sewer system will be prevented by isolating BPC from the unprotected sewer system, including the interceptor that runs along Route 9A / West Street.

This technical memorandum provides details on the analyses and computer modeling Arcadis conducted as part of the PDB Team for the NWBPCR project, as well as discusses proposed modifications to the existing drainage infrastructure. A hydrologic and hydraulic (H&H) model was used to understand the magnitude and location of stormwater flooding that may occur with a flood barrier system in place around the BPCA project area but without new drainage infrastructure. Subsequently, the H&H model was used to develop solutions to reduce stormwater flooding under future conditions.

1.2 Drainage Analysis Goals

The two main drainage objectives of this project are as follows:

1. Isolate the protected area such that floodwater is prevented from entering through existing sewer connections during coastal storm surges. This is needed to meet Federal Emergency Management Agency (FEMA) certification requirements and to reduce flood risk in the area.
2. Manage interior flooding within BPC associated with a coincident coastal storm surge and rainfall event to minimize interior flooding. Interior flooding is defined as any areas that exceed 1 foot of average ponded water depth on the surface within the areas of BPC protected by the coastal barrier and adjacent areas along and east of Route 9A. This target depth is specified per FEMA certification requirements for levee systems[1].

1.3 Decisions Made During Design Phase

The following were decisions made prior to the 60% phase:

- Revised the proposed near surface isolation components at Harrison St. and West St. based on the change of the FBS wall alignment along BMCC. See **Section 2.5.**
- Incorporated the proposed overflow gravity pipe option to manage stormwater at BMCC parking lot. See **Section 2.7.2**

Below are design components that are included in the 60% design based on decisions made:

- Incorporated Sanitary Sewer Flow Management design for North (Vesey Street) and South (W. Thames Street) Areas. See **Section 2.1.1** for details on this.
- Incorporated Gateway Plaza stormwater management design. See **Section 2.7.1**.
- Relocate the existing tide gate and a section of the CSO located near Stuyvesant Plaza to enable the FBS wall to bridge over the existing historic bulkhead. See **Section 2.6**.

2 Storm Surge Isolation Methodology and Design Approach

To meet the FEMA certification requirements and to reduce flooding risk in the area, modifications to the existing drainage infrastructure will be required. Modifications must limit floodwater from entering the protected area through the existing combined sewer drainage system (the interceptor or street-level sewers) during coastal storm surges, limit floodwater entry through existing stormwater infrastructure and outfalls, and manage interior flooding caused by rainfall, FBS overtopping, and seepage volumes during coastal storm events. A near-surface-isolation (NSI) system is proposed to manage storm surge through the existing interceptor sewer. The NSI system would consist of a slide gate (see **Section 2.5**) within existing regulator structures, M4, M5, and M6 (see **Figure 2-1**), which would be closed in advance of a flood event to prevent storm surge from rising through each regulator's underflow connection to the interceptor line and flooding the protected area. Access manholes to these regulators and interceptor manholes along Route 9A/ West Street between Albany Street and Harrison Street would be floodproofed as necessary. **Table 2-1** summarizes these proposed modifications, along with other interior drainage

[1] https://www.ecfr.gov/current/title-44/chapter-I/subchapter-B/part-65/section-65.10

improvements, as discussed with New York City Department of Environmental Protection (NYCDEP) and shown in the 60% interior drainage plans.

Table 2-1. Proposed Modifications to Drainage Infrastructure

Component	Description
NSI at Regulators M4, M5, and M6	Remove and replace closure gate on regulator capture pipe and sanitary sewer structure connection to the interceptor. Prevents unprotected interceptor sewer from flooding into protected area.
New tide gates	Prevents storm surge from entering sewer system.
Management of sanitary flows during gate closure period	During normal conditions, sanitary flows typically enter the interceptor at two locations within the project area. These connections would be isolated in advance of a flood event and management of sanitary flows during the event will be necessary. Proposed management of sanitary flows are outlined in **Section 2.1.1**.
Floodproofed interceptor manholes	Prevents unprotected interceptor sewer from flooding into protected area. The floodproofing consists of a pressure cover that would be locked to minimize the volume of water from entering the protected area through the manholes.
Additional sewer gates and isolation under the FBS	Dependent on sewers passing under FBS alignment. Gate/closure to be designed to meet FEMA accreditation. Two slide gates are proposed in Reach 1, see **Figure 2-1**.
Other floodproofed manholes along/adjacent to the FBS alignment	Prevents unprotected sewers from flooding into protected area.

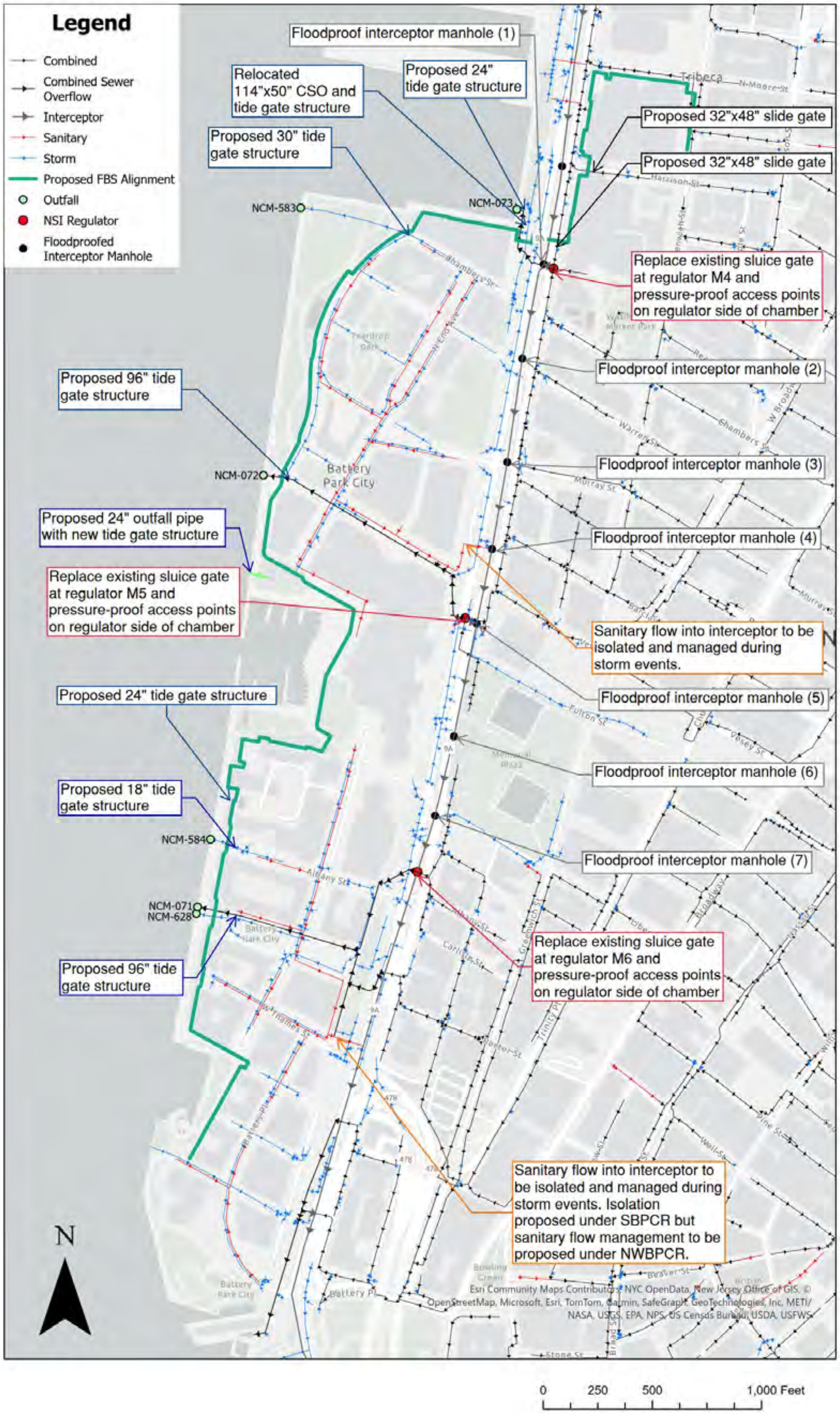


Figure 2-1. Proposed Isolation Approach

2.1 Conveyance System Modifications

Modifications include reconfiguring the existing drainage infrastructure as part of the NSI system. As depicted on the 60% interior drainage plans, new drainage sewers and manholes are proposed to be installed to isolate the protected area, preventing floodwaters from entering the protected areas through the City's combined sewer system or existing outfalls within the project area. Existing catch basins will be removed and replaced as necessary to accommodate these modifications. The PDB Team will evaluate and address any additional drainage connections entering the protected area as the FBS alignment gets further developed.

Any new conveyance infrastructure was designed in accordance with the NYCDEP drainage design criteria and consistent with NYCDEP specifications. Existing sewers downstream of new or reconfigured drainage infrastructure were evaluated for adequate sewer capacity based on NYCDEP drainage planning guidelines and replaced with larger infrastructure if additional capacity was necessary.

A hydraulic analysis was conducted to assess the suitability of newly installed sewers for the project area. This analysis aimed to preliminarily determine whether the sewers have the capacity to adequately manage the flow associated with the 5-year storm event as required by NYCDEP, particularly considering the unique characteristics of the area. In the process of evaluating the hydraulic capacity of the newly installed pipes and ensuring their compatibility with the intended flow, the analysis used the Manning's equation. The utilization of the Manning's equation signifies a systematic and established approach to assessing the sewers' ability to handle fluid flow within the given area.

The analysis provided insights into closed conduit flow characteristics, such as velocity and discharge, based on factors like pipe slope, cross-sectional area, and the Manning's roughness coefficient. By integrating the Manning's equation into the hydraulic analysis, a detailed understanding of how fluid flow behaves within the sewers was attained. By comparing the calculated hydraulic parameters against the design specifications and expected flow conditions, it was possible to ascertain whether the sewers possessed the required capacity to effectively accommodate the anticipated fluid volumes.

Furthermore, the utilization of the Manning's equation facilitated the identification of any potential constraints or shortcomings in the pipe system. If the calculated flow parameters exceeded the sewers' capacity, adjustments were proposed to ensure that the hydraulic performance aligns with the demands of the area. This involved considering modifications such as altering the pipe diameter or adjusting the slope to regulate the flow and maintain optimal hydraulic conditions. The hydraulic analysis calculations are presented in **Appendix A**.

A more detailed model analysis will be conducted for sizing new sewers using the same H&H modeling software described in **Section 3.1**. The H&H modeling, which will follow NYCDEP's drainage planning criteria, will account for hydraulic factors that the method described above does not.

2.1.1 Sanitary Sewer Modifications

The PDB Team estimated wastewater flows generated from BPC and determined if they need to be managed during coastal storm surge events, when BPC is expected to be isolated from the City's combined sewer system. An analysis of water uses and design flows in BPC was conducted using the following method:

- 2014 NYS Design Standards for Intermediate Sized Wastewater Treatment Systems

Assumptions for typical per-unit hydraulic loading rates and flow per square feet (office and retail spaces) were used to calculate the water use across commercial, retail, and residential spaces located within the protected area.. These assumptions will be refined if water use data is available from BPCA.

The work area was subdivided into two smaller geographical sections, south and north, each with their respective land usage characteristics. The average daily flow calculated for the north area equated to 1.4 MGD while the south area equated to 1.2 MGD. As described in Figure 1 of the Ten States Standard, the "Ratio of Peak Hourly Flow to Design Average Flow" was used to calculate the peak hourly flow rate. This method factors in the population of the areas under evaluation, which, as per the Block Level Census Data of 2020 Population, the northern portion of BPC equated to 5,847 people, while the southern area was 10,422 people. Since the flooding mitigation infrastructure will operate only during extreme weather conditions, it was assumed that commercial buildings, offices, retail stores, and schools will be closed during emergency conditions. Therefore, the calculated peak factor was only applied to residences.

- Southern portion of BPCA:
    - Average daily flow: 1.2 MGD
    - Peak factor: 2.9 (applied to residences only)
    - Peak hourly flow: 2.7 MGD

- Northern portion of BPCA
    - Average daily flow: 1.4 MGD
    - Peak factor: 2.9 (applied to residences only)
    - Peak hourly flow: 1.6 MGD

There are two sanitary flow connections to the interceptor: Vesey St. (estimated 1.6 MGD peak hourly flow) and West Thames St. (estimated 2.7 MGD peak hourly flow). Both connections have existing sluice gates inside the manhole connection. The scope of NWBPCR includes replacement of the existing sluice gate at Vesey St. and management of sewer flow for both sewer connections. Replacement of the existing sluice gate at W Thames Street sewer connection is part of the South Battery Park City Resiliency Project (SBPCR). Three alternatives were analyzed for managing wastewater flows during coastal storm surge events.

2.1.1.1 Alternative 1: Gravity Flow to CSO

This alternative utilizes the current configuration of the sanitary connections. Prior to joining with the interceptor, both sanitary flow connections pass through a chamber with a closed emergency overflow gate leading to a CSO. In Alternative 1A, the emergency overflow gate is opened, allowing flow to enter the CSO by gravity as it cannot enter the interceptor. In Alternative 1B, the gates are opened, but a backwater valve is added in the chamber to ensure a fully surcharged CSO does not flow back into BPC through the sanitary connection. While Alternatives 1A and 1B are low in cost, have few schedule impacts, and require little additional infrastructure, it is likely that sanitary flows in both alternatives will back up into service laterals, which will likely result in basement flooding. Preliminary modeling results show that installing backwater valves in the emergency overflow gate chambers (Alternative 1B) would have little impact on lowering the hydraulic grade line of the system under the storm surge condition in BPC. Alternatives 1A and 1B would discharge combined sewage into the Hudson River during rare, storm surge events.

2.1.1.2 Alternative 2: Lift Station

This alternative involves the construction of two small lift stations (1.6 MGD and 2.7 MGD) near the current sanitary flow connections to convey flow to the CSOs. Based on preliminary modeling results, this alternative would lower the hydraulic grade line of the system in BPC. This would likely minimize backups in service laterals, reducing the risk of basement flooding. This alternative requires siting and construction of two small lift stations, resulting in a higher cost and moderate disturbance. This alternative would discharge combined sewage into the Hudson River during rare, storm surge events.

2.1.1.3 Alternative 3: Onsite Storage

This alternative involves the construction of two holding tanks near the sanitary flow connections to the interceptor, see **Figure 2-2**. These tanks would be designed to handle flow for a 24-hour duration (1.6 MG and 2.7 MG). The tanks would be situated below ground to allow flow to enter by gravity. The approximate size of the storage tanks would be 67’W x 320’L x 10’D for Vesey St and 88’W x 410’L x 5’D for West Thames St. A small lift station would pump flow back into the interceptor after the storm surge condition ends. Unless the tanks reach their capacity, this alternative does not discharge raw sewage to the Hudson and would result in reduced risk of basement flooding in BPC. However, it would involve temporary disturbance of a bike lane, turf field, courts, playground, and a park, possibly requiring an easement. Considerable excavation and construction and moderate utility relocation would be necessary. Holding tanks would need to be customizable in dimensions and geometry to fit within a limited area footprint. Concerns due to odors, leaks, contamination, and dewatering would need to be accounted for. This alternative would result in significant schedule impacts and a very high cost.

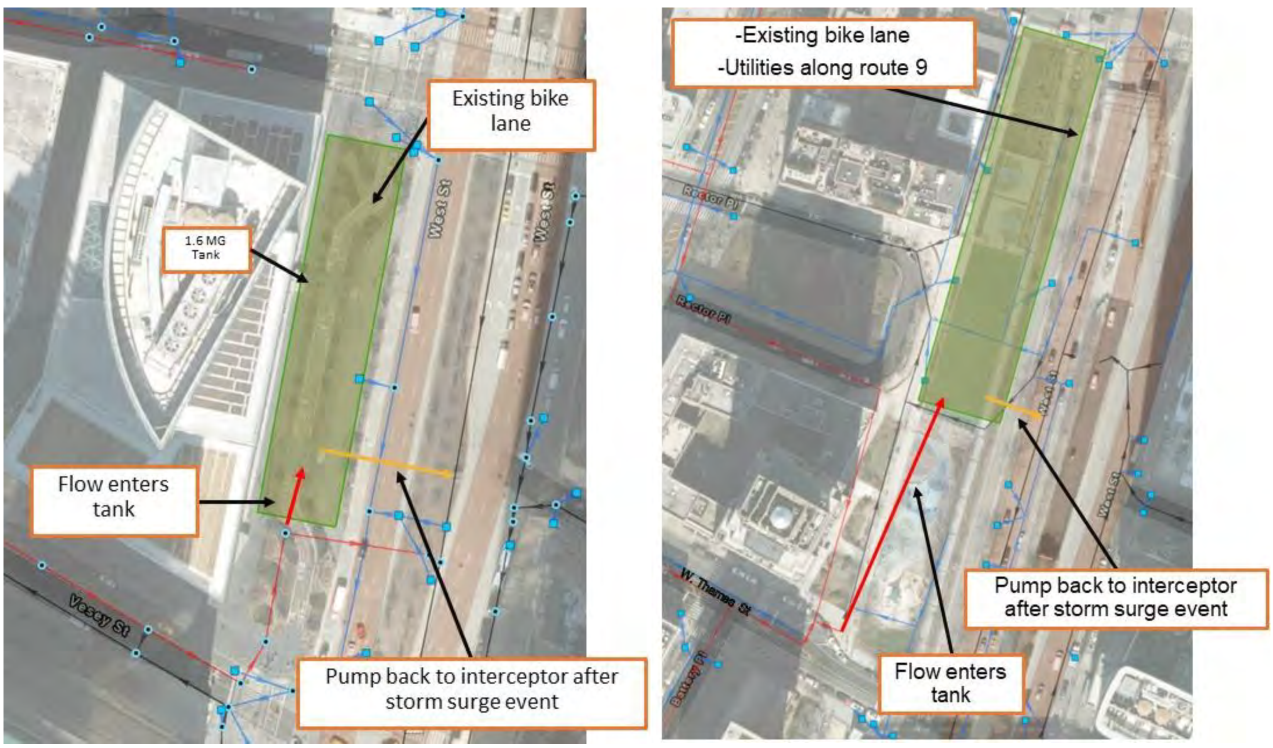


Figure 2-2. Onsite Storage for Vesey St (left) and West Thames St (right)

2.1.1.4 Selected Alternative for Sanitary Sewer Modifications

Based on the input from BPCA and DEP, the selected alternatives for the 60% Design are shown below for both the Vesey Street and W. Thames Street sanitary connections.

1. Vesey Street: Alternative 1: Gravity Flow to CSO
    - Prior to joining with the interceptor, the sanitary flow connection passes through a chamber with a closed emergency overflow gate leading to a CSO. The proposed concept is to redirect sanitary flow from the 30" Sanitary Sewer to the combined sewer network through an overflow structure during storm surge condition as presented in **Figure 2-3**. See **Section 2.1.1.5** for further details on the gravity flow design that was in the 60% design.

2. W. Thames Street: Alternative 2: Pump Station to Pump Flow to CSO
    - This alternative includes a small temporary pumping station with submersible pumps to redirect sanitary flow to the existing 84" CSO during storm surge condition as presented under **Figure 2-6**. See **Section 2.1.1.6** for further details on the lift station design that was included in the 60% design.

2.1.1.5 60% Design of the Vesey Street Gravity Flow to CSO

The 60% design for Vesey Street includes a new gate chamber on the existing 30" sanitary sewer. Two new 6' diameter manholes are proposed to redirect the flow to the existing storm sewer network. The existing 18" storm sewer on the downstream manhole will be removed and replaced with new upsized storm piping, which is being designed as part of the pump station conveyance design. See **Figure 2-3.** For the full-scale drawings, refer to the 60% design drawings.

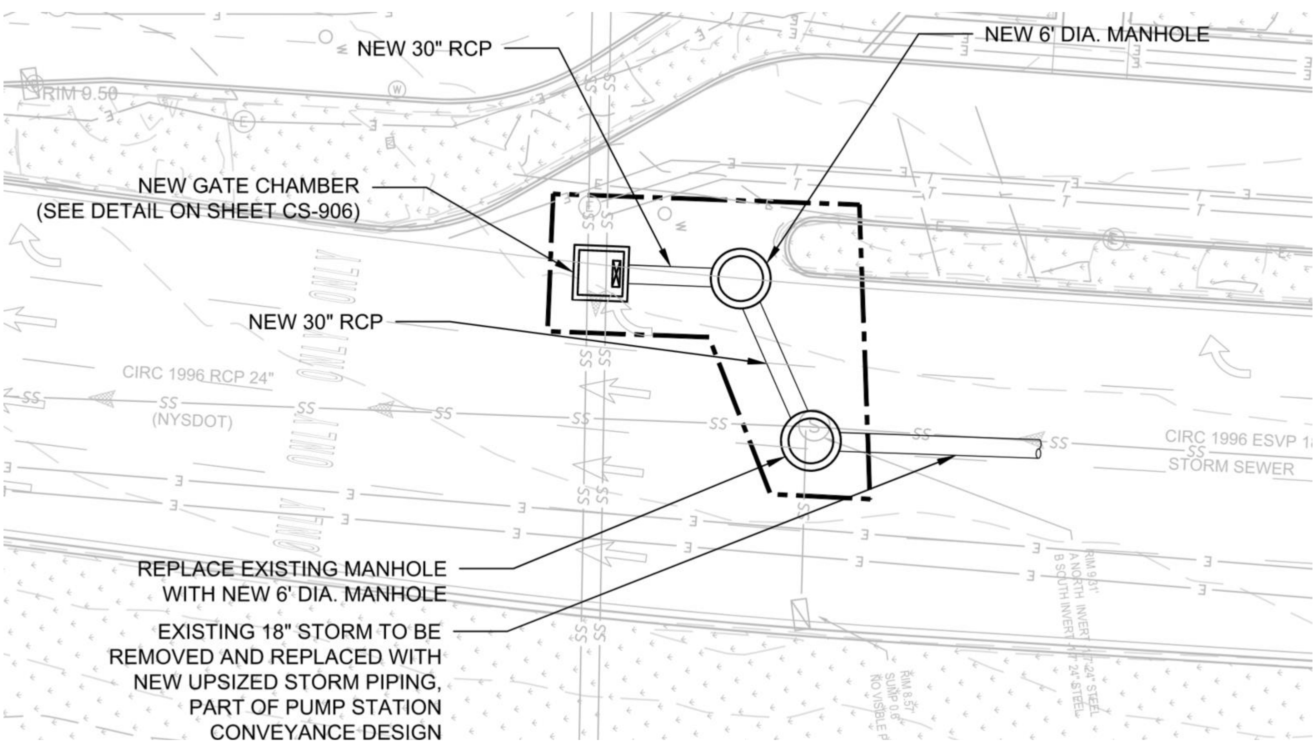


Figure 2-3: Vesey Street: Gravity Flow to CSO

2.1.1.6 60% Design of the West Thames Pumping Station

Chamber Q is an existing sewer overflow chamber which is located on W. Thames Street, west of the intersection with route NY-9A. The function of the overflow chamber is such that under normal operating conditions sanitary flows enter the structure from the west and exit east towards the existing sanitary interceptor located within the NY-9A right of way. If the sewer system backs up and the hydraulic grade line raises higher than the design overflow elevation, the excess sanitary flows overflow into an 84" combined sewer pipe by manually opening the existing slide gate. It must be noted that the structure includes a manual slide gate controlling the overflow function. The gate must be manually opened before any overflow can occur.

Refer to **Figure 2-4** and **Figure 2-5** below.

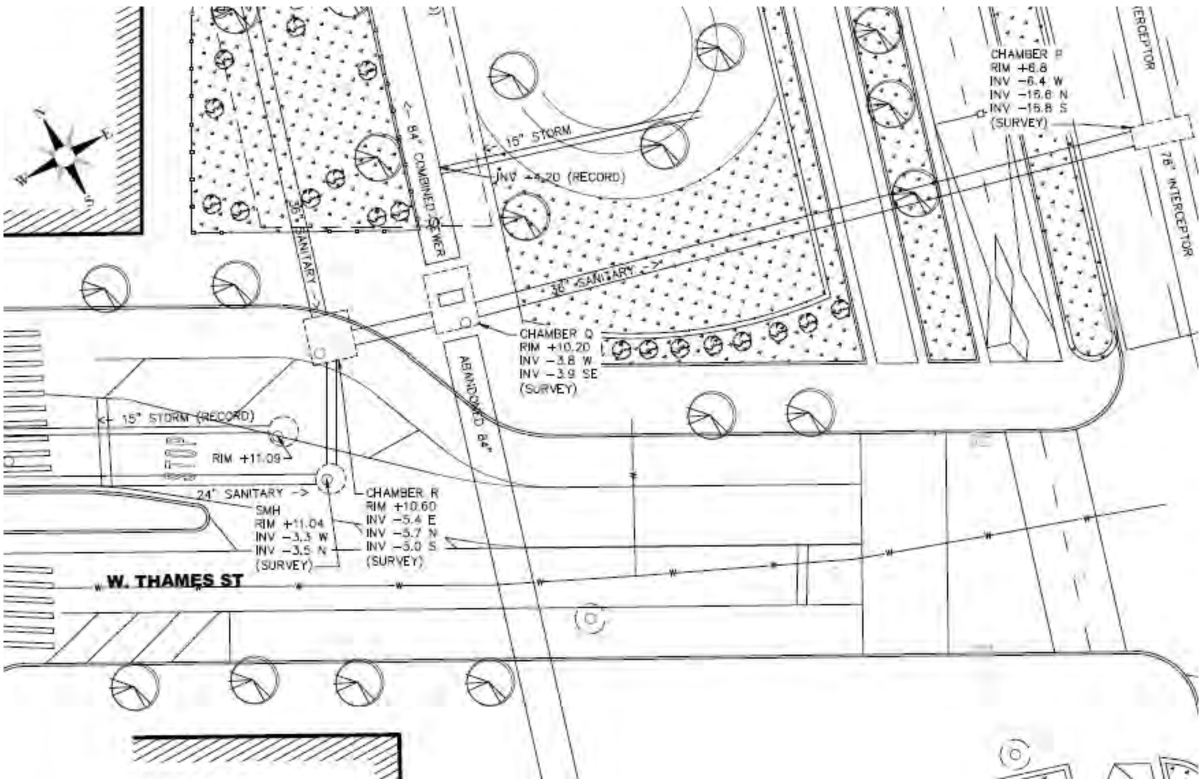


Figure 2-4. Existing Sewer Regulator Chamber Q

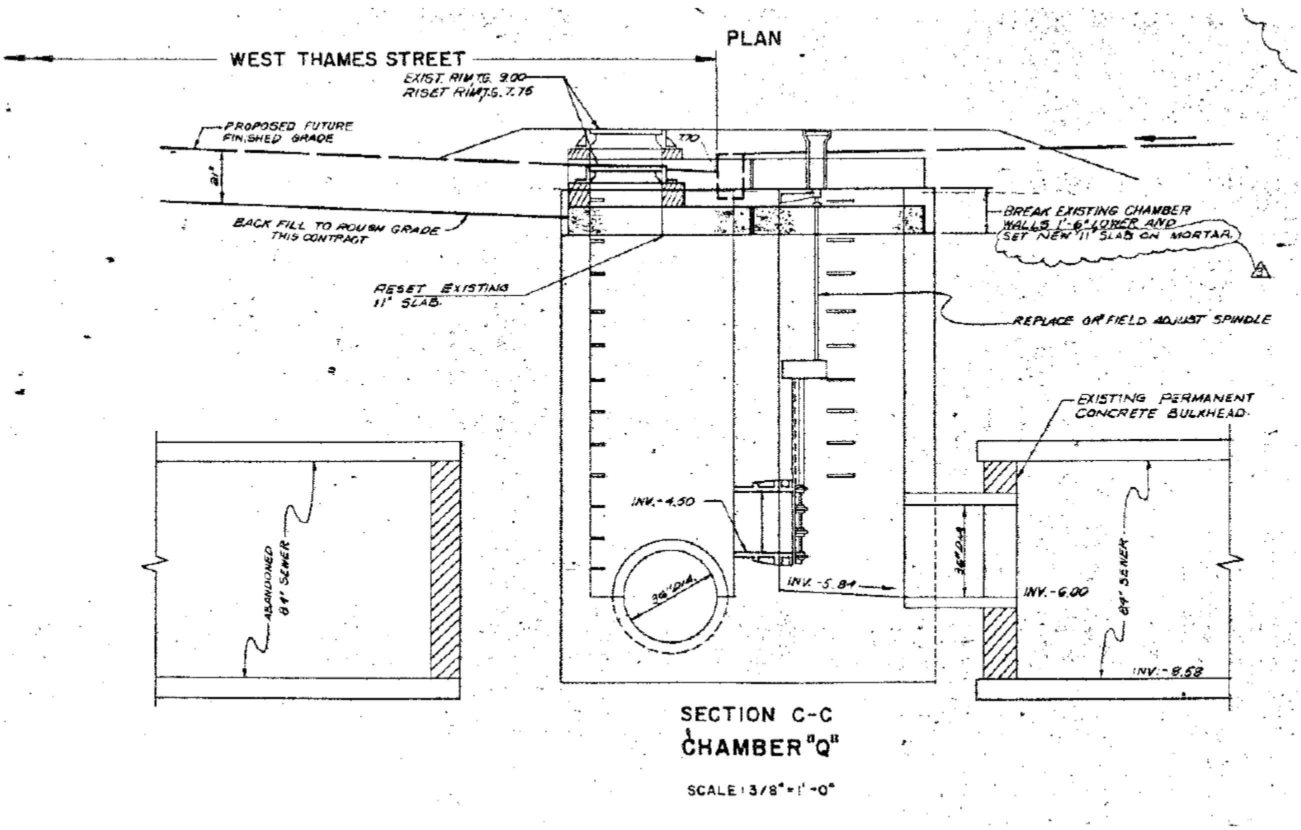


Figure 2-5. Record Drawing of Chamber Q (1982 - BPC South Residential Phase II Infrastructure)

Hydraulic modeling was performed to examine the impact of proposed infrastructure improvements as part of the BPCA Resiliency project indicates that the hydraulic grade line (HGL) at this location is problematic. The existing regulator only functions passively under gravity flow conditions and, as a result, calculations indicate that the HGL would need to rise to unacceptably high levels before gravity flow can occur.

The design intent is to install a new pumping station and new manual gate structure to replace the functionality of the existing chamber Q, while providing sufficient control to maintain the system HGL within acceptable bounds. Similar to the existing structure, the new pumping station will only receive flows when a manual gate is open. After manual activation, the pumping station will convey excess sanitary flows into the combined sewer in a similar manner as the existing regulator to prevent a backup of the sewer system. Deployable pumps are proposed as part of the 60% design. For further information on the pumps, refer to **Section 2.1.1.6.7.**

2.1.1.6.1 Recommended Site Layout

The recommended site layout of the manual gate and pumping station is shown in **Figure 2-6** below. Refer to the 60% design drawings for full size drawings.

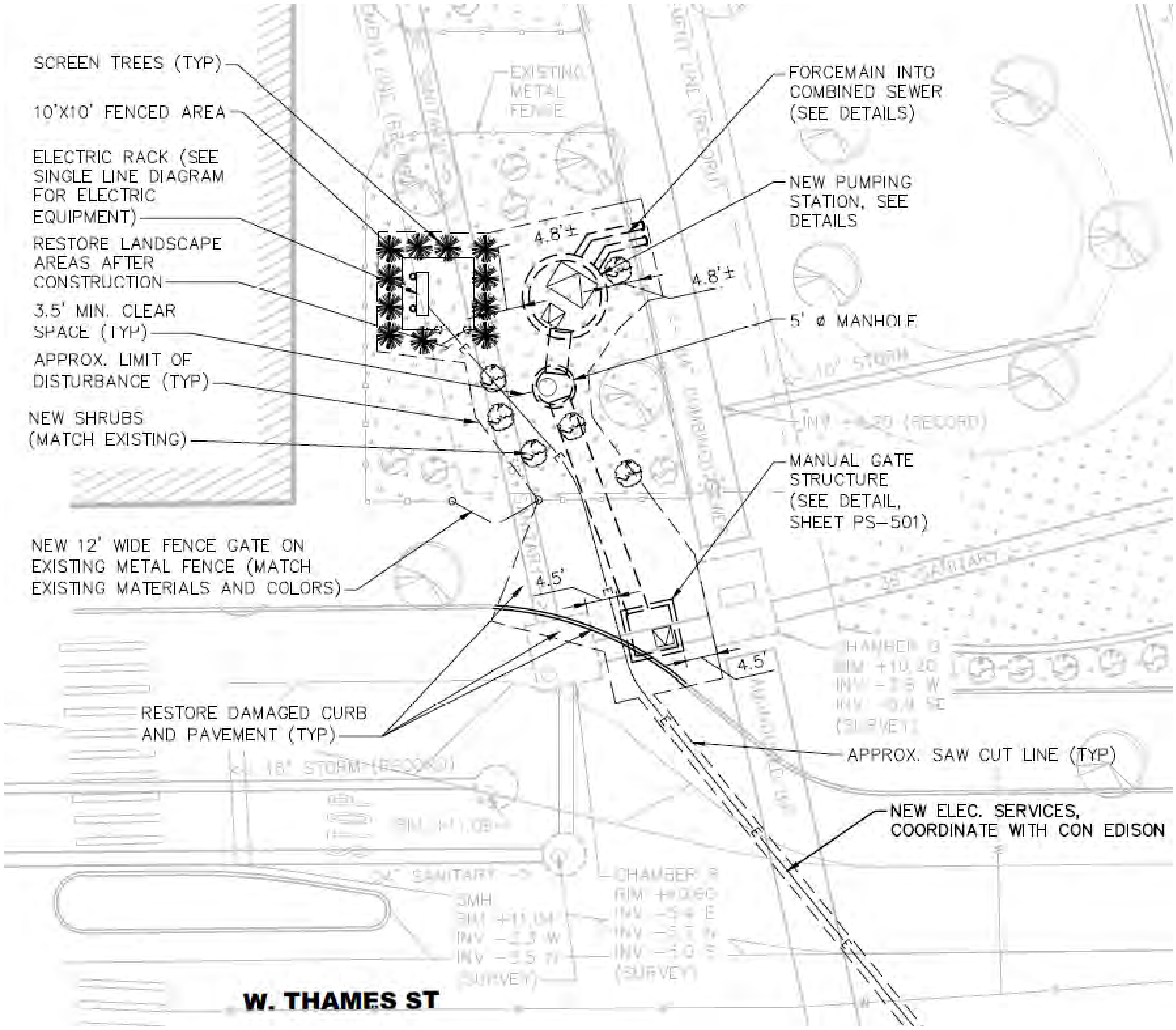


Figure 2-6. Recommended Site Layout of Pumping Station and Manual Gate Structure

The layout is based on consideration of the following:

- The proposed structures and equipment are located inside an existing easement, inside a landscaped area, and inside an existing fenced area which appears inaccessible to the general public.
- The layout avoids existing trees to the maximum extent practicable, with the goal of preserving existing trees.
- Temporary sheeting and bracing will be required. The proposed structures are positioned to maximize clearance space to allow space for sheeting between structures and pipes.
- The proposed location of the manual gate structure will allow for temporary bypass pumping during construction, from existing Chamber R to Chamber Q.

2.1.1.6.2 Influent Manual Gate Structure

Flow into the pumping station will be controlled by a manual gate structure. The structure will not automatically allow sanitary flows into the pumping station but will require an operator to open the gate. The structure is shown in **Figure 2-7** below.

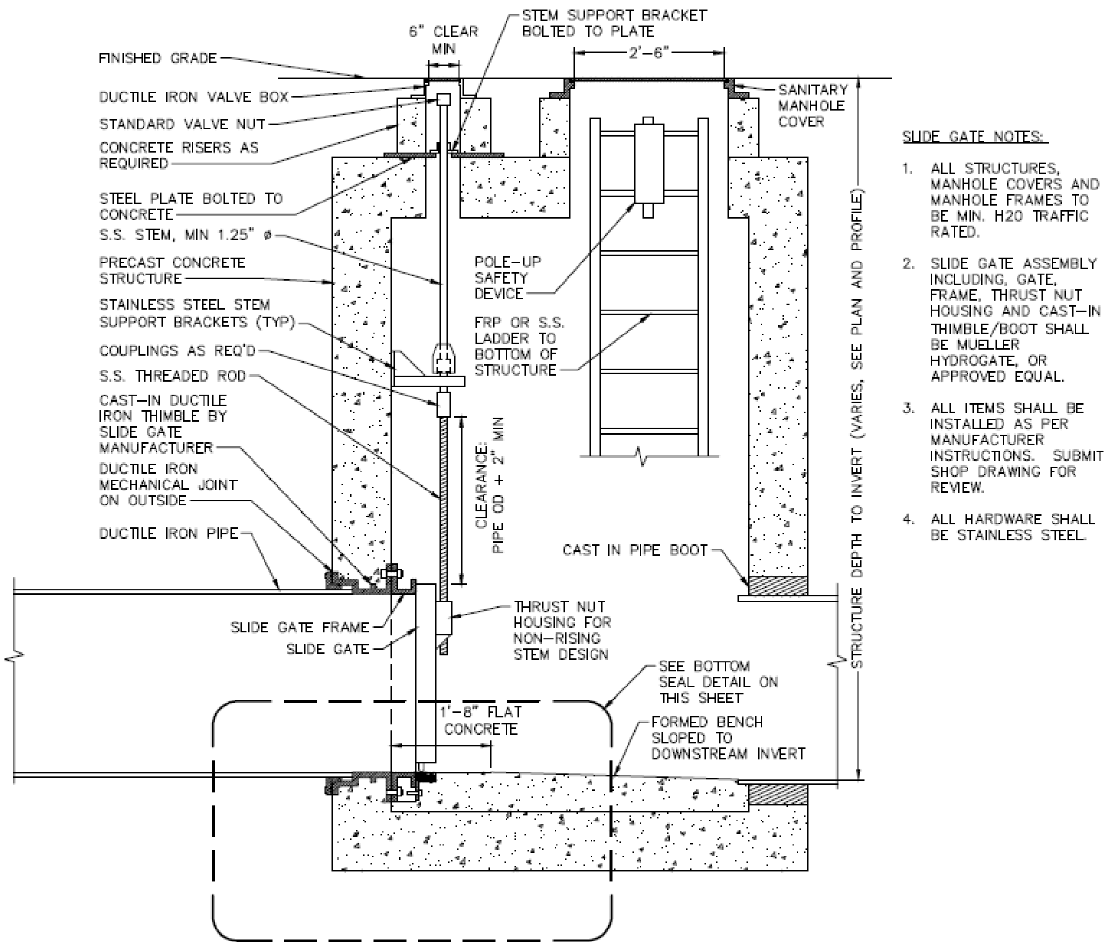


Figure 2-7. Proposed New Manual Gate Structure

2.1.1.6.3 Connection into the 84" Combined Sewer

Discharge force mains from the pumping station will discharge directly into the existing combined sewer. To accommodate this connection, the existing concrete pipe will be core drilled and a "Link-Seal" style penetration sealing system will be utilized such that the connection will be watertight within the operating pressure limits.

The maximum discharge pressure will be controlled by a pressure relief valve located inside the wet well, or by other equivalent means. This provision will allow control over the maximum allowable pressure at the connection point.

<u>Requirements for Further Design:</u>

It is necessary to determine the maximum allowable pressure of the combined sewer in this location and to finalize hydraulic modelling of the impact of the new pumps on that existing system. A pressure limiting device may be required depending on what design limit is determined to be acceptable.

One alternative to the “pressure connection” to the combined sewer design would be a “gravity connection” design. In this alternative, the force mains would terminate at a shallow depth weir box located above the elevation of the combined sewer to transition to gravity flow. One disadvantage of the gravity flow alternative is that the forward flow rate is entirely constrained by downstream hydraulic conditions in the combined sewer. If the downstream HGL is high enough, gravity forward flow out of the weir box would be impaired and the pumping station would back up and not be able to serve its intended function. The downstream HGL will need to be further evaluated to determine if a “gravity connection” design is feasible.

2.1.1.6.4 Design Flow

The average daily flow for the tributary area of the pumping station was calculated as described in **Section 2.1.1**.

Peak demand is assumed for residential uses (as many people would stay home during a severe storm). It is assumed that during the design scenario (a severe storm) peak commercial activity would not occur. An average sanitary demand is assumed for commercial uses.

An average utilization of 100 gallons/person/day is assumed for the residential component. The equivalent residential population, in thousands, is approximated as 910,000 gpd / 100 gpcd /1000 = 9.1

The peaking factor for the residential component is calculated below using the formula in Ten State Standards.

$$Residential\ Peaking\ Factor, PF = \frac{18+\sqrt{9.1}}{4+\sqrt{9.1}} = 3.0$$

$$Residential\ Peak\ Flow, Q_{Peak} = Q \cdot PF = 0.91\ MGD \cdot 3.0 = 2.73\ MGD$$

$$Design\ Flow = Peak\ Residential + Average\ Commercial$$
$$Design\ Flow = 2.73MGD + 0.28 = 3.0\ MGD\ (2{,}083\ gpm)$$

<u>Requirements for Further Design:</u>

Design flow assumptions to be further verified once information from flow monitoring is received (anticipated before 60% design consensus).

2.1.1.6.5 Gravity Pipe Design

The term “gravity pipe” refers to pipes where fluid is driven by gravity rather than by pumps.

Refer to **Figure 2-6** for the proposed gravity pipe locations. New gravity pipes are proposed to connect the new manual gate structure to the new permanent pumping station located at Stuyvesant Plaza. An intermediate junction manhole is located between the two structures to adjust the route of the gravity pipe to suite site conditions.

Regarding the sizing of these gravity pipes, a 36” gravity pipe size is selected to match the size of the existing sanitary sewer. This will ensure that the proposed gravity pipes have at least as much capacity as the existing gravity sewer does.

If a 1% minimum pipe slope is assumed, the capacity of the 36” gravity pipe is approximately 30,000 gpm, using the Mannings equation with n = 0.013. Note that this Manning’s n value is conservative. Newly installed sewer pipes will start out with a relatively smooth inside wall; however, over time the pipes become rougher due to wear and fouling of the interior surfaces.

For the pipe material, Class 53 cement lined ductile iron pipe is selected. The reason for this is that a ductile iron mechanical joint is desirable for the connection between the gravity pipe and the manual gate structure. At the penetration into the gate structure side wall, a ductile iron "thimble" must be cast into the concrete wall to provide a firm attachment point for the gate on the structure interior. The thimble is designed to receive a ductile iron mechanical joint from the outside of the structure (refer to **Figure 2-7** for additional information about the gate structure).

2.1.1.6.6 Force Main Design

The approximate flow capacity of ductile iron force main pipe is shown in **Table 2-2** below, assuming a maximum of 8 feet per second velocity:

Table 2-2. Force Main Pipe Capacities

Force Main Pipe Capacities (Approximate)			
Nominal Diameter	Actual ID (in)	Area (sf)	Capacity (gpm)
4	4.04	0.09	310
6	6.1	0.20	720
8	8.21	0.37	1,310
10	10.22	0.57	2,040
12	12.28	0.82	2,940
14	14.27	1.11	3,980
16	16.35	1.46	5,220
18	18.43	1.85	6,630
20	20.51	2.29	8,220
24	24.67	3.32	11,890
30	30.73	5.15	18,450

Calculation notes:

1) Inside diameters based on class 53 cement lined ductile iron pipe.

2) Calculation assumes a max allowable flow velocity of 8 fps. Higher velocities are theoretically possible but may result in energy waste and additional wear on pipes and equipment.

The design team selected two (2) 10" ductile iron force main pipes in parallel which provides a total capacity of approximately twice the design flow, when both pipes are used simultaneously.

To minimize site disturbance, and due to site space constraints, no valve chamber will be provided outside the wet well. Part of the reasoning for this, is that the pumping station will not be in frequent use but only used rarely during extreme storm events.

The design does not require plug valves for isolation, as each pump has its own separate discharge force main. Plug valves are typically provided when two pipes connect into a single pipe, because it is desirable to be able to stop flow from one branch of the pipe system, using a valve, while the other branch continues to operate. This scenario is not applicable to the proposed design.

Check valves are required to prevent backflow and will be provided inside the wet well near the top of the structure. As previously discussed, they will be used infrequently, and it is not expected that they will need any maintenance during their service lifespan. To replace them at the end of their service life (estimated at 50 years +/-), flow to the wet well would be shut off using the manual gate structure, electricity would be disconnected, and a confined space entry of the wet well would be performed to replace the check valve using appropriate PPE, ventilation, and safety procedures.

Requirements for Further Design:

Having two (2) 10" pipes allows the capability of two (2) pumps to be utilized. This provides operational flexibility (i.e., either the use of one large pump, two large pumps, or two smaller pumps). An alternative under consideration at the 60% milestone is whether temporary or permanent pumps will be utilized. If permanent pumps are utilized, it is a best practice to have one duty pump and one standby pump for a minimum of two (2) pumps. For the temporary pump alternative, a single duty pump could theoretically be utilized under the assumption that it would be tested and known to be functional before being brought out to the site and installed. Note, however, that even in the temporary pump alternative it may be desirable to have the option of using two smaller duty pumps rather than a single large duty pump.

2.1.1.6.7 Pump Design

A system curve describes the pressure characteristics of a system of pipe and fittings at various flow rates. Pump curves describe the characteristics of the pump and are provided by the pump manufacturer. The intersection of the two curves determines the operating point of the pump (i.e., flow rate and total dynamic head).

The estimated system curve of the pipe and fittings is shown in **Figure 2-8**:

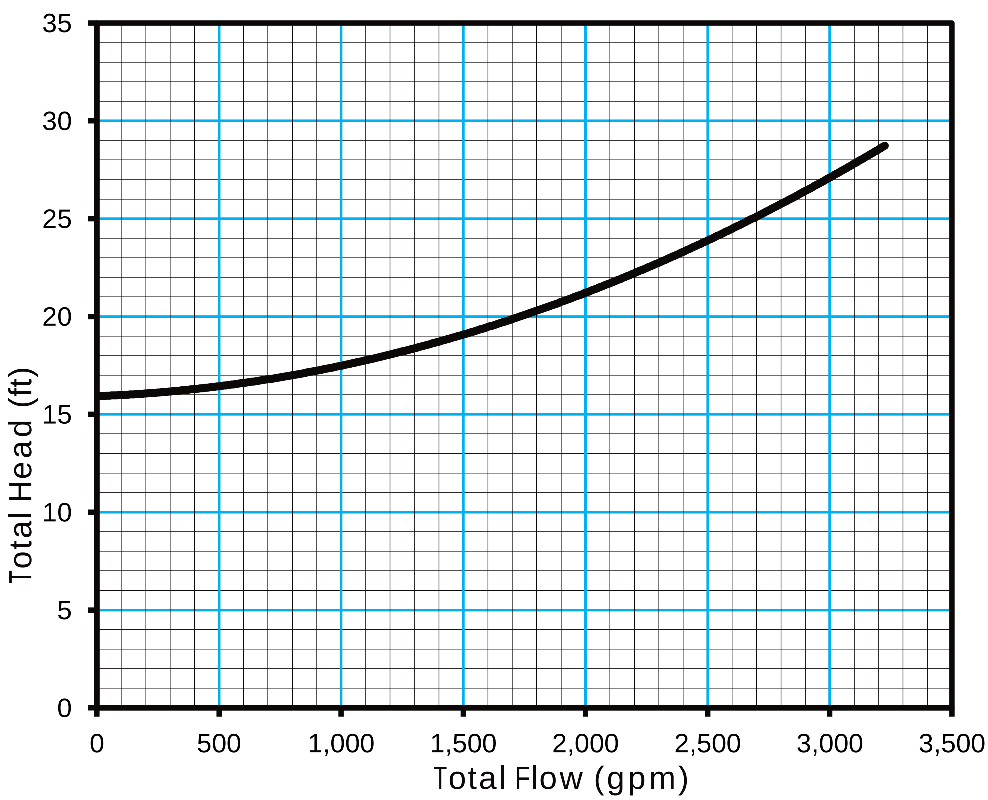


Figure 2-8. Estimated System Curve

Refer to **Appendix D** for the Pump Curve and intersection point.

System curve assumptions:

- Hazen-Williams Equation
- 10” diameter C53 D.I. pipe, 40 long, C=130
- 221’ equivalent length of fittings
- 16’ static head

The design team’s selection for the “temporary pumps with one duty pump” alternative is the following:

- Flygt NP 3171 LT 3 615 or approved equal (one duty pump)
- 18HP, 460V, 3 phase, 1180 rpm
- Operating Point 2140 gpm @ 22’ TDH
- 1,050 lbs pump weight
- Flow velocity in 10” force main = ~8 fps in one pipe

Requirements for Further Design:

Another temporary pump alternative is provided below which is two (2) smaller pumps in parallel, both duty pumps, using two (2) 10” force mains:

- Flygt NP 3127 LT 3 426 or approved equal (two duty pumps)
- 8.5 HP, 480V, 3 phase, 1800 rpm
- Operating Point 1,022 gpm @ 18.1’ TDH
- 355 lbs pump weight
- Flow velocity in 10” force main = 4 fps in both pipes

For the permanent pumps alternative, the selection would be the same as the above noted 18HP pump, however two would be installed, with one reserved as a standby unit.

Another important consideration is that pumps must be exercised regularly, regardless of whether they are permanent or temporary. According to the manufacturer, rotating the impeller once per month is recommended and it does not matter if the pumps are wet or dry. In the case of temporary pumps, it will be necessary to have staff go to the storage location monthly to perform this maintenance.

2.1.1.6.8 Wet Well Design

Conventionally, wet wells are often sized by engineers using “rules of thumb” provided in various reference books to determine the required “effective depth”. For example, a common approximation is V = T*Qpeak/4. Where T is cycle time in minutes and Qpeak is peak flow. Note that “Qpeak/4” will generally be a value roughly similar to the average flow. A common, conservative, value for T is 15 minutes, assuming there is ample space on site to accommodate the required wet well size. A conservative size can be desirable to allow the pumps to run at a steady pace for a longer time duration which is more efficient than frequent stops and starts.

In the Ten State Standards 2014 edition, Section 42.62 (Wet Well Size), there is no minimum size stated, only that the maximum “fill time” shall not exceed 30 minutes to prevent septicity.

As a practical matter, the minimum size is constrained by how many starts per hour the pumps can withstand. Modern sewage pumps are typically rated for up to 30 starts per hour (2 minute cycle time, where one cycle is the total of time to fill plus time to empty). In addition, this condition must be satisfied at all flow rates, not just average flow. In fact, the worst case (fastest cycle) generally occurs when influent flow rate is equal to about half of the pump discharge rate.

For this area, space and depth are highly constrained (as the wet well must be installed between two (2) existing pipes and in high groundwater conditions) so the design must be optimized as far as practicable for space efficiency rather than using conventional "rule of thumb" sizing guidelines.

The design team selected a 10' inside diameter concrete wet well. The reasoning for this size is that it is the largest circular structure which can fit within the available space on site. In addition, it is the largest commercially available size of precast concrete circular wet well that can be transported over roadways. Use of precast rather than cast-in-place concrete will result in significant cost savings due to the difficulty of performing cast-in-place work in high groundwater conditions. The design team assumes that the wet well will need to be supported on piles, given the known poor-quality soil in the project area.

As previously discussed, the lower bound of structure size is determined by pump cycle time. To be conservative, the design team will target a 4-minute minimum cycle time. This will provide a safety margin (as compared to the rated minimum value of 2 minutes) in case the actual pumping rate is higher or lower than the theoretical values determined.

The effect of various structure depths on cycle time was evaluated using a spreadsheet methodology. The spreadsheet solution is shown in **Table 2-3**:

Table 2-3. Wet Well Design Calculations

Design Inputs	
Wet Well Effective Volume (gal)	2350
Pump Rate (gpm)	2140

Pump Cycle Time				
Flow, Q (gpm)	Time to Fill (min)	Time to Empty (min)	Total Cycle Time (min)	Starts/ Hour
100	23.50	1.15	24.65	2.4
200	11.75	1.21	12.96	4.6
300	7.83	1.28	9.11	6.6
400	5.88	1.35	7.23	8.3
500	4.70	1.43	6.13	9.8
600	3.92	1.53	5.44	11.0
700	3.36	1.63	4.99	12.0
800	2.94	1.75	4.69	12.8
900	2.61	1.90	4.51	13.3
1000	2.35	2.06	4.41	13.6
1100	2.14	2.26	4.40	13.6
1200	1.96	2.50	4.46	13.5
1300	1.81	2.80	4.61	13.0
1400	1.68	3.18	4.85	12.4
1500	1.57	3.67	5.24	11.5
1600	1.47	4.35	5.82	10.3
1700	1.38	5.34	6.72	8.9
1800	1.31	6.91	8.22	7.3
1900	1.24	9.79	11.03	5.4

The calculation indicates that an effective volume of 2,350 gallons is required, which is equivalent to an effective depth of 4' inside a 10' diameter wet well.

For reference, the effective volume calculation is shown below:

$$Effective\, Volume, V = A \cdot h = \frac{\pi \cdot (10\,ft)^2}{4} \cdot 4' \cdot \frac{7.48\,gal}{ft^3} = \ 2{,}350\,gal$$

Lastly, the conclusion which can be drawn from the spreadsheet solution is that the worst-case cycle time is 4.4 minutes at an influent flow rate of 1,000 gpm, which is greater than the minimum value selected for design (4 minutes).

2.1.1.6.9 Electric Design

The following are the design team's electric assumptions at the 60% milestone:

- Assumed 480V 3 phase electric service, approximately 100 amp size (available service to be verified with utility.)
- Assumed Temporary pumps alternative
  - Assuming one large temporary pump rather than two (2) small ones.
- Temporary pump is connected to electric by special quick connect plug at end of pump cable.
- Assumed two (2) redundant electric services from Con Edison
- Assumed a terminal box will be provided for optional connection of portable generator.
- Two (2) transfer switches are assumed:
  - An automatic transfer switch to switch between the two (2) Con Edison services.
  - A manual transfer switch to switch between Con Edison or portable generator as the source of power.

2.1.1.6.10 Controls

- Assumed basic on/off functionality controlled by floats. No advanced remote control or telemetry. No variable frequency drives (VFDs).
- Redundant floats will be provided to ensure continued operation in the event that a float malfunctions.
- For the temporary pump alternative, the control panel will normally be disconnected from power using the panel's built-in disconnect switch and there are normally no pumps in the wet well.
- To use the temporary pump(s):
  - First verify that the control panel is disconnected from power.
  - Next visually check the condition of the floats in the wet well. Check that they are not obstructed, or fouled with debris. If required, they can be pulled up for inspection or replacement.
  - Next plug in the power cable of the temporary pump(s) and slide them down the guide rail in the wet well. Moving the pumps could be done using a work truck which has a suitable hoist on it.
  - Next, verify the settings of the electric transfer switches (which source of power should be utilized)
  - Lastly, turn the control panel disconnect to the on position to energize the panel. The pumps will operate automatically based on floats. If desired by the operator, the pumps may also be run manually using the HOA switch on the panel.
- If any faults or alarm conditions are detected, they will be displayed on the front of the control panel and a red alarm light will turn on (i.e., high water level, low water level, failed seal, etc.) Alarms can be reset

using a reset button on the control panel. There is no siren and alarms are not dialed out as it is assumed that the operator of the temporary pumps will be on-site during pump usage.

Requirements for Further Design:

The 10' diameter wet well size can only accommodate two pumps (N+1) redundancy. It should be considered whether N+1+1 redundancy is appropriate to satisfy NYCDEC Bureau of Wastewater Treatment (BWT) standards. One consideration is that if the temporary pump alternative is selected, a lesser level of redundancy may be appropriate. Note that the pumping station is normally not being used and the gate structure is normally closed. Failure of a redundant system while the gate is closed, would cause no issue as there is no flow in the wet well and no pumps installed. Any issues would be noticed and corrected by on-site operations personnel as part of installing and starting up the temporary pumps.

2.1.1.6.11 Access and Security

The following are the design team's access and security assumptions at the 60% milestone:

- Assumed the electric equipment will be located in the grass area near the pumps and within the existing record easement. (easement details to be verified).
- Assumed 10' x 10' locked fenced area with electric equipment, controls and 480V panels inside.
- The fence will be surrounded by evergreen screen trees for aesthetics.
- Lock-out-tag-out procedure will be to lock the control panel disconnect switch in the off position before any work on pumps (including plugging them in or unplugging them). Local disconnects will not be provided as the control panel will be very close to the pumps.
- Hatches on utility structures will be flush at grade and lockable, but will not be enclosed in the fence.
- The pump cable will be run from the fenced electric area into the hatch of the wet well.

2.2 Outfall Modifications

Outfall modifications will focus on backflow prevention as part of the NSI approach. As depicted in the 60% interior drainage plans, a new outfall is proposed to redirect flow entering the protected area in Belvedere Plaza to discharge to the Hudson River. Additionally, the PDB Team is proposing to install tide gates at the existing outfalls listed in **Table 2-4** and shown in **Figure 2-1**. Further information can be found in the Existing Outfall Schedule, which is part of the 60% interior drainage plans. The 60% plans also provide details for the new tide gate structures.

Table 2-4. Proposed Tide Gates at Existing Outfalls

Outfall ID	Approximate Location	Proposed Tide Gate Size
NCM-073	Stuyvesant Plaza (Relocated CSO and tide gate)	60" x 60" (2)
NCM-583	Northwest End of Rockefeller Park	30"
NCM-072	Under Ferry Terminal	60" x 60" (2)
-	West of Gateway Plaza	24"
NCM-584	West of Albany Street	18"
NCM-071	West of Rector Place	60" x 60" (2)

2.3 Post-construction Stormwater Management Practices

The PDB Team calculated preliminary water quality volumes based on requirements and criteria established by the City's Unified Stormwater Rule. Post-construction stormwater management practices (PCSMPs) were designed to meet necessary water quality requirements, based on the amount of land disturbance anticipated by the NWBPCR project.

The water quality volume (WQv) that must be treated by Post-construction Stormwater Management Practices (PCSMPs) is the total runoff produced by the 90th percentile, 24-hour storm and is calculated using the following equation:

$$WQ_V = \frac{1.5"}{12} * A * R_V$$

Where:
WQ_V: water quality volume (ft^3)
A: contributing area (ft^2)
R_V: runoff coefficient relating total rainfall and runoff
R_V: 0.05 + 0.009(I),
I: percent impervious cover

WQv is the volume produced by the first 1.5 inches of rain. The contributing area was taken as the limits of work within each reach since the limits of disturbance were not readily available. The preliminary WQv required for this project is 40,300 ft^3. Although the WQv is calculated holistically for the project, it can be divided by reach to highlight the different sections of the project. Preliminary WQv results are summarized in **Table 2-5.** These calculations will be further refined as the design progresses.

Table 2-5. Water Quality Volume by Reach

Reach	Contributing Area, A (ft^2)	Total Pervious Area (ft^2)	Total Impervious Area (ft^2)	% Impervious Cover, I	Runoff Coefficient, Rv	Water Quality Volume, WQv (ft^3)
1	127,900	13,400	114,600	89.6	0.86	13,700
2	33,300	27,100	6,200	18.6	0.22	1,000
3	55,700	30,600	25,200	45.1	0.46	3,200
4	60,600	30,900	29,700	49.1	0.49	3,800
5	158,400	84,600	73,800	46.6	0.47	9,300
6	108,600	59,500	49,200	45.2	0.46	6,300
7	54,200	28,100	26,200	48.3	0.48	3,300
Total	598,700	274,200	324,900	54.2	0.54	**40,300**

2.3.1 Green Infrastructure

Green infrastructure PCSMPs are needed to meet the runoff reduction and water quality stormwater regulations defined by NYC DEP in the Unified Stormwater Rule (USWR). The project area within Battery Park City is serviced by a municipal separate storm sewer system (MS4) meaning that runoff is captured by storm sewers that do not comingle with sanitary sewer flows until downstream of combined sewer overflow devices. The project area east of Battery Park City (Reach 1) is within the combined sewer system (CSS) meaning that rainfall runoff is captured by sewers that convey both stormwater and sewage.

The runoff reduction requirements of the USWR are calculated by comparing the total area of impervious surfaces from pre- to post-development. Since the project is reducing the overall impervious cover by over 50,000 ft^2, the runoff reduction requirements are assumed to be met without consideration of green infrastructure practices.

To meet the water quality requirements, green infrastructure was designed following the PCSMP hierarchy of practices where retention is preferred over filtration (for MS4 areas) or detention (for CSS areas). Within each category of practice, vegetated PCSMPs are preferred over non-vegetated PCSMPs.

All practices currently in the MS4 area for the 60% submission are designed as treatment PCSMPs with the potential for some to be converted to infiltration practices upon further geotechnical investigations. The vegetated practices include standard and high-performance rain gardens, and the non-vegetated practices include porous pavers and hydrodynamic separators (HDS). These are designed to meet the requirements of the NYC Stormwater Manual and include underdrains to convey treated runoff to the MS4 network. Treatment practices were initially selected over retention practices because the preliminary geotechnical report indicated Hydrologic Soil Group (HSG) Type D soils throughout the project area, except where the project rests on top of structural platforms overlying the Hudson River. Treatment practices were selected over retention practices due to the poor infiltration capacity of HSG Type D soils across the project area, as indicated by the geotechnical report, and the infeasibility of infiltration through structural platforms overlying the Hudson River.

Green infrastructure features were designed to manage 41,280 ft^3 or 100% of the total WQv (40,300 ft^3) for the project. **Table 2-6** summarizes the WQv managed by each feature. **Figure 2-9** shows the locations of the features throughout the project alignment.

Table 2-6. WQv Managed by Green Infrastructure

Reach	Practice Name	Practice Type	WQv Managed (ft^3)
Reach 1	HDS116-1	HDS	5,500
	HDS117-1	HDS	720
Reach 2	HDS114-1	HDS	1,760
Reach 3	GS111-1	Grass Filter Strip (Existing)	170
	GS113-1	Grass Filter Strip (Existing)	3,360
	RGHP112-1	High Performance Rain Garden	2,020
	RGHP113-1	High Performance Rain Garden	180

Reach	Practice Name	Practice Type	WQv Managed (ft^3)
	RGHP113-2	High Performance Rain Garden	910
	RGHP113-3	High Performance Rain Garden	880
Reach 4	HDS110-1	HDS	8,000
Reach 5	HDS108-1	HDS	560
	HDS108-2	HDS	900
	HDS108-3	HDS	830
	HDS108-4	HDS	1,390
	HDS109-1	HDS	1,860
Reach 6	HDS103-1	HDS	1,360
	HDS103-2	HDS	1,730
	HDS104-1	HDS	1,520
	HDS104-2	HDS	3,620
Reach 7	RG101-1	Rain Garden	290
	RG101-2	Rain Garden	250
	RG101-3	Rain Garden	670
	RG102-1	Rain Garden	310
	HDS102-1	HDS	1,090
	HDS102-2	HDS	1,400
Total			**41,280**

Figure 2-9. Proposed Green Infrastructure Features

Treatment practices were placed where feasible across the project area. Despite many site constraints (limited space, locations on structural decking, urban design elements, direct drainage to the river, Route 9A, etc.) stormwater practices were maximized to improve water quality. In some areas opportunities were identified to treat offsite runoff as well as runoff produced within the limits of disturbance. For example, high-performance rain gardens were proposed to treat road runoff along River Terrace. Also, some hydrodynamic separators are accepting flow from upstream MS4 networks that extend beyond the limits of disturbance. As a result, the project is managing slightly more than 100% of the overall required WQv.

Permeable pavement is also proposed in Reaches 2, 4, 5, and 7 to reduce site imperviousness, reduce peak flows, and provide some pretreatment of runoff. The locations of the permeable pavement are depicted in the 60% design drawings. The PDB Team is working with BPCA to determine the final strategy on permeable pavers.

2.4 Storage Design

The assessment of storage options for flood mitigation included considerations of storage design (volume capacity) and feasibility of construction. Upon thorough analysis, it was concluded that a pump station would offer a more efficient solution due to its ability to manage larger volumes effectively, see **Section 4.4**. Therefore, any additional investigation into storage design has been eliminated from further exploration.

2.5 Slide Gate Modifications

The NSI system consists of removing and replacing existing sluice gates at regulators M4, M5, and M6 including removing and replacing an existing slide gate at sanitary sewer structure where the BPC sanitary sewer connects to the existing interceptor near Vesey Street.

Two new slide gate structures are being proposed within Reach 1, one along Harrison Street at the intersection of West Street, and another further south along West Street. Both slide gates are connected to one another to redirect flow to the NWBPCR Pump Station (See **Section 4.4.1**) along West Street during peak storm surge conditions, given that they are positioned in the protected area just outside of the floodwall. The slide gate along Harrison Street is connected to the 32”x48” combined sewer that picks up flow from Harrison Street, which under normal conditions would eventually flow north along West Street. The slide gate being proposed further south is connected to the existing 32”x48” combined sewer that flows along West Street, and this flow is transported further south to the M4 regulator structure.

The purpose of the two slide gate structures is to prevent floodwaters from entering the protected area during the storm surge. During gate closure period, combined sewer flows in Harrison Street will be directed to the slide gate further south via an overflow pipe and will eventually flow to the pump station. The slide gate further south along West Street will also be closed during peak conditions to prevent the original 32”x48” combined sewer pipe that is now inundated to enter the structure and will only redirect the incoming flow from Harrison Street.

2.6 Tide Gate Modifications and CSO relocation

Six new tide gates are positioned along the flood barrier route, spanning Chambers Street, Albany Street, south of the Irish Hunger Memorial at Vesey Street, in proximity to Rector Park, and close to Stuyvesant Plaza just north of Stuyvesant High School. Of these, three will be installed at CSOs underground pipes, with one near Rector Park, another positioned just south of the Irish Hunger Memorial at Vesey Street, and the third located near Stuyvesant Plaza, immediately east of Stuyvesant High School. The sizing and placement of these tide gate structures have been accurately depicted on the 60% design plans.

The proposed FBS wall will bridge over the historic bulkhead by Stuyvesant High School to limit disturbance within the historic bulkhead. However, the bridging of the FBS wall and constructing the pump station will require relocation of the existing 114”x50” CSO and tide gate structure to provide space for both the pump station and wall foundation. The CSO situated in the protected side will merge with the Pump Station to manage flow redirection, whereas the CSO located on the wet side will be integrated into the FBS design. It’s important to note that only CSO is being relocated, not the outfall. The proposed relocation of the existing CSO and tide gate structure is depicted on the 60% design drawings.

2.7 Other Modifications

2.7.1 Gateway Building Stormwater Improvements

As discussed in **Section 4.5.2**, the H&H modeling results show flooding greater than one foot near the Gateway Building. To mitigate flooding in these areas, two options were conceptually evaluated:

- Option 1: The first option is to install a passive underground storage system, one at the north side and one at the south side. The underground storage system would hold the stormwater and drain by gravity after the storm passes. Approximately 4,000 gallons and 50,000 gallons of storage would be required for the north area and south area, respectively. The storage option would result in a larger disturbance to facilitate construction compared to the sump pump option. Additionally, installation of a backflow prevention device would be necessary for this option. This would be also designed to mitigate flooding of less than 1 foot.
- Option 2: The second option is to install sump pumps located on the south side of the building to manage stormwater for both patio areas. The pump option would result in a small disturbance to install the necessary infrastructure for the sump pump. The water would then be pumped downstream of the proposed tide gate structure. Additionally, installation of a backflow prevention device would be necessary for this option. This option would be designed to mitigate flooding of less than 1 foot.

See **Figure 2-10** for a diagram showing the two options.

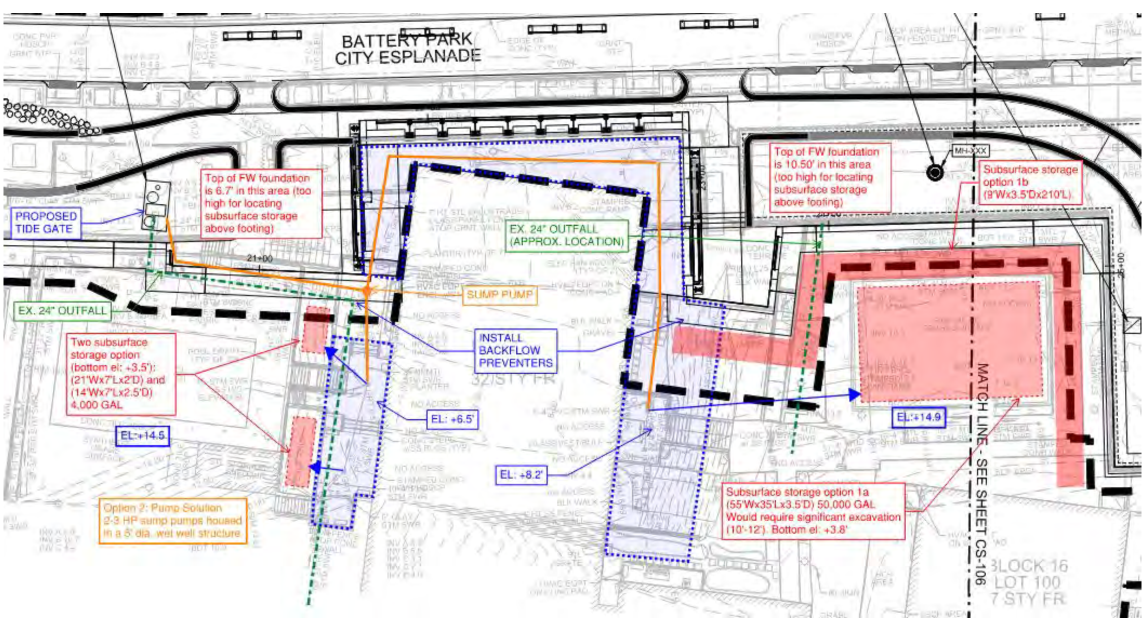


Figure 2-10. Gateway Flood Mitigation Options

To meet the goal of limiting flooding within the protective barrier, the second option was selected for 60% design. A sump pump added to the Gateway Plaza areas reduces all ponding over 1 foot in this area along the floodwall as depicted in **Figure 2-11** below. Refer to **Section 2.7.1.1** for further details on the design.

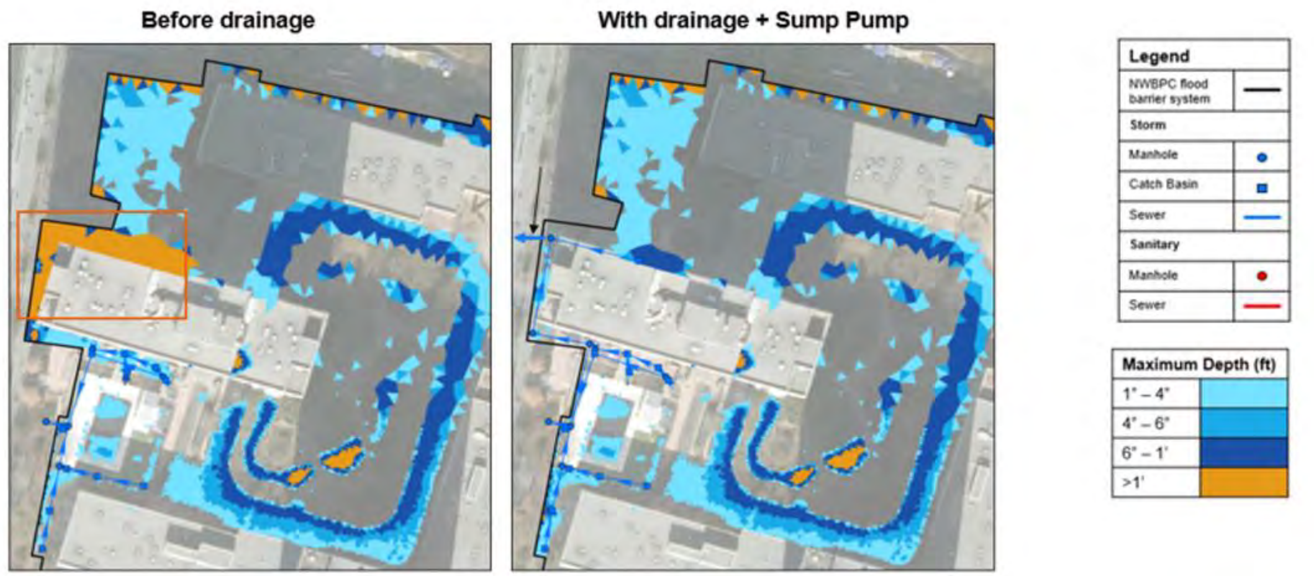


Figure 2-11: Gateway Plaza without modification to drainage vs Gateway Plaza with modification to drainage and a sump pump

Along with pumping, a tide gate chamber is proposed for the existing 24” outfall south of the middle building in Gateway Plaza. Refer to the 60% design drawings for its location. North of the middle building, another 24” outfall was found during the CCTV pipe investigation. A tide gate chamber will be proposed for that outfall as well and will be incorporated by the 60% design consensus.

2.7.1.1 60% Design of Gateway Plaza Sump Pump

2.7.1.1.1 Design Flow

The following are the design team’s flow assumptions at the 60% milestone:

The stormwater design flow was determined by the design team to be 40-50 gpm. The selected Flygt Concertor pumps (or equal) have a variable speed design which allows them to operate over a wide range of heads and flow rates. If flow is determined to be higher (up to about 300 gpm), the selected pumps should still work.

Requirements for Further Design:

Design flow assumptions to be further verified based on flow monitoring to be conducted before 60% consensus.

2.7.1.1.2 Gravity Pipe Design

The 60% plans include manhole structures to direct flow around the building from the North side to the South. CCTV inspection is still under review to verify hydraulic connectivity.

2.7.1.1.3 Force Main Design

The force main selected is 3"Ø SDR-9 HDPE. Based on the 50 gpm flow rate, a fluid velocity of about 2.3 ft/s will be achieved using a single force main with a single pump operating. If the flow rate is determined to be significantly higher dual force mains may be selected to allow for increased capacity.

The force main routing was selected to minimize length and disturbance to existing facilities. The routing is proposed to be above the footing of the flood wall. The pipe will need to penetrate the wall near the tide gate structure. A Link-Seal, or equal, will be utilized to maintain water tightness of the penetration and stabilize the pipe within the penetration. The pipe will enter the sidewall of the tide gate structure downstream of the tide gate mechanism.

2.7.1.1.4 Pump Design

The following are the design team's pump design assumptions at the 60% milestone:

System curve assumptions:

- Hazen-Williams Equation
- 3" diameter SDR-9 IPS HDPE pipe, ±60' long, C=100
- 73' equivalent length of fittings
- 8' static head

For this application, the design team selected the following:

- Flygt N80-600 or approved equal (one duty pump)
- 3HP, 208V, 3 phase, 800-2095 rpm
- Operating Point 40 gpm @ 17' TDH
- 267 lbs pump weight
- Flow velocity in 3" force main = 2.3 fps in one pipe

Another important consideration is that pumps must be exercised regularly. According to the manufacturer, rotating the impeller once per month is recommended and it does not matter if the pumps are wet or dry. It may be possible to include an exercise circuit within the control panel to "bump" the pumps once per month to ensure they are exercised.

2.7.1.1.5 Wet Well Design

The design team selected a precast concrete wet well product produced by Oldcastle Precast known as the "Onelift". The Onelift integrates a wet well and drywell into one structure reducing the size of the excavation and area of disturbance. The system can be pre-assembled with pumps, piping, and valves off site reducing the duration of disturbance to the building residents. The wet well will need to be supported on piles, given the known poor-quality soil in the project area.

An important consideration in wet well sizing is cycle time; if the wet well is too small this will result in excessive on/off cycling of the pumps. The selected pumps are rated for a maximum of 60 starts per hour (1 minute cycle time). Based on the wet well geometry of 297 gal/vertical foot and the 0.5' distance between the lead pump on and pumps off elevation, the minimum time to empty will be 2.9 minutes at 50 gpm.

Regarding the design scenario for cycle time, the worst case occurs when the influent flow rate is approximately one half of the pump discharge flow rate. In contrast, for very low influent flow rates it takes significant time to fill the wet well and at high influent flow rates the well fills quickly but takes a long time for the pumps to empty it.

The wet well location was selected as the patio on the south side of the building due to its proximity to existing drainage infrastructure.

2.7.1.1.6 Electric Design

The following are the design team's electric assumptions at the 60% milestone:

- Assumed 208V 3 phase electric service at the Gateway Plaza building (Voltage TBD after 60%)
- Assumed that a sufficient excess capacity within the existing building electrical system exists to provide for the sump pump. (The design team will need to coordinate with building maintenance personnel after 60%)
- Power will be supplied from the building electrical room (location and conduit routing TBD after 60%)

<u>Requirements for Further Design:</u>

Operation during power outage should be evaluated. A portable generator will most likely not be used if the patio area is flooded as the generator would likely need to be located on the flooded patio. Coordination with building maintenance is recommended to determine if the building has its own emergency power, or if a location exists where a portable generator could be deployed during power outages.

2.7.1.1.7 Controls

The following are the design team's controls assumptions at the 60% milestone:

- Assumed basic on/off functionality controlled by floats. No advanced remote control. No variable frequency drives (VFDs), though the pumps do contain a variable speed function which provides functionality similar to a VFD.
- Based on final determination of design flow (see above), a decision will be made as to whether variable speed pumps are utilized, as a single speed pump may be more desirable in certain situations. (To be determined after 60%)
- Due to the control panel's proximity to public space, if any faults or alarm conditions are detected, they will **NOT** be displayed on the front of the control panel **NOR** will an alarm light will turn on (i.e., high water level, low water level, failed seal, etc.). Alarms will be transmitted to building maintenance via telemetry, or a remote alarm panel located in a building maintenance area (if applicable). Alarms can be reset using a reset button on the control panel.

2.7.1.1.8 Access and Security

The following are the design team's access and security assumptions at the 60% milestone:

- Assumed the electric equipment will be located adjacent to the building north of the patio area where currently there is HVAC equipment. The panel will be a dead front panel with controls on the inner door accessible only after unlocking and opening the front panel.
- Hatches on wet well will be flush at grade and lockable.

2.7.2 BMCC Parking Lot Drainage Improvements

Under existing conditions, the BMCC parking lot discharges to a combined sewer in Route 9A that would be on the wet side of the floodwall. Without any drainage modifications, storm surge from the combined sewer could backflow into the parking lot. A backflow preventer will prevent storm surge from entering the lot, but the runoff produced from stormwater and seepage collected within the lot would still need to be discharged at a new location during storm surge events. As depicted on the 60% design drawings, an overflow pipe is proposed to allow runoff to be rerouted south along Route 9A. This pipe would be connected downstream of the parking lot's existing catch basin and would be set at a higher invert than the existing discharge pipe. The overflow pipe would be directed towards the south end of the lot, before crossing under the floodwall on the west. The pipe would then go south, cross Harrison Street and continue along Route 9A, until it goes under the floodwall for the second time. After going under the floodwall, it would connect to the existing combine sewer pipe on the protected side. The flow exiting this structure would be managed by the pump station. See **Figure 2-12** for a schematic diagram of the overflow piping at the BMCC parking lot. Refer to the 60% design drawings for the latest configuration of the proposed piping and manholes.

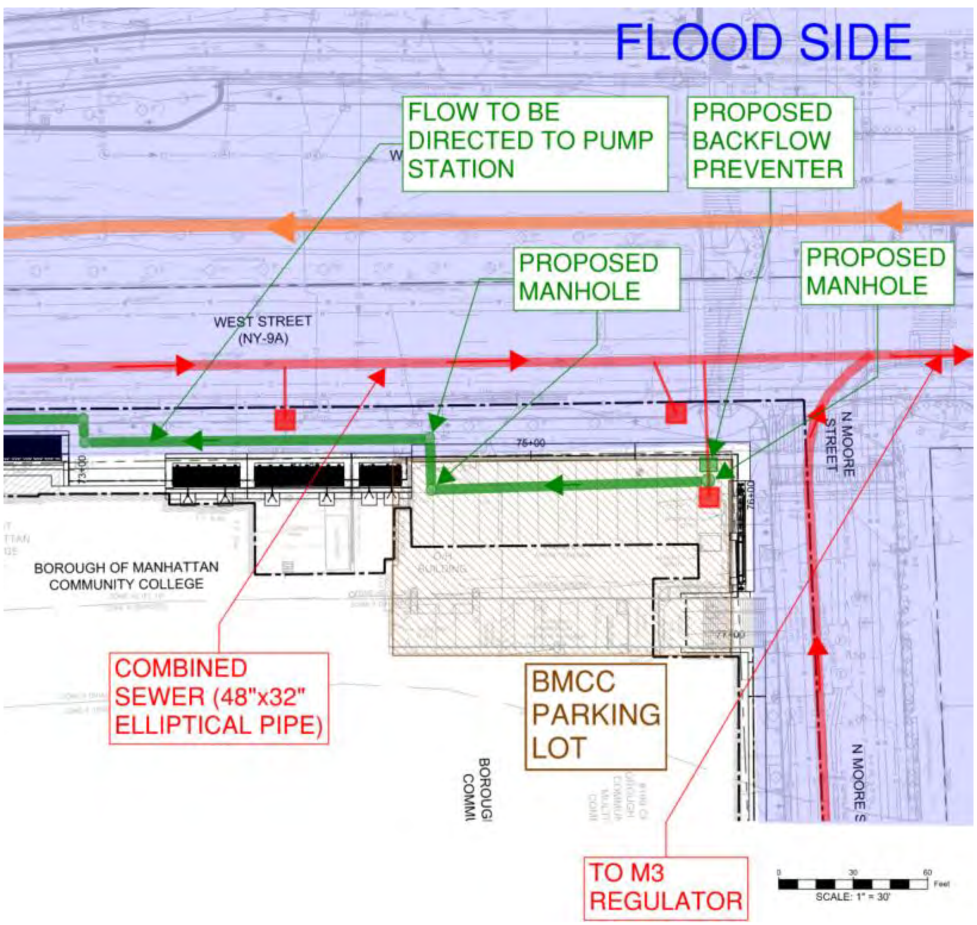


Figure 2-12. Overflow Pipe at BMCC Parking Lot

2.8 Erosion & Sedimentation Control

The NWBPCR project requires effectively managing and mitigating the potential environmental impacts associated with soil disturbance during construction activities. With a keen focus on erosion and sediment control, the project aims to uphold the highest environmental standards and comply with the guidelines set forth by the New York State Standards and Specifications for Erosion and Sediment Control.

The significance of this project lies in its commitment to preserving the integrity of surrounding ecosystems and water bodies, preventing the transportation of sediments into nearby waterways, and ensuring that the construction activities do not contribute to air and water pollution. Erosion and sediment control measures play a pivotal role in achieving these goals.

The implementation of erosion and sediment control measures encompasses a comprehensive range of strategies, both structural and procedural, to effectively address the challenges posed by soil disturbance. These measures can be categorized into various aspects:

1. Structural Controls:
   The project will involve the installation of sediment barriers strategically placed to intercept and contain sediment-laden runoff. These barriers, often consisting of silt fences or sediment ponds, serve as the first line of defense against sediment migration. Dewatering measures will also be incorporated to manage excess water accumulation on the construction site, preventing soil erosion and the release of sediment-laden water.
2. Dust Control:
   Dust generated from construction activities can have adverse effects on both air quality and nearby ecosystems. Effective dust control measures will be employed, such as applying water to surfaces prone to dust generation, using dust suppressants, and employing covers for stockpiles of materials.
3. Soil Stabilization Measures:
   To minimize soil erosion, stabilization techniques will be employed. This could involve revegetation with native plants to anchor the soil, the application of erosion control blankets, and the use of geotextiles to reinforce soil structure.
4. Pollution Prevention Measures:
   Various practices will be implemented to prevent pollution at its source. These measures include proper management of construction materials, containment of potential pollutants, and ensuring that equipment and vehicles are well-maintained to minimize leaks and spills.

At the current stage of the project, erosion and sediment control plans have been developed for the 60% design. These plans are essential in laying the groundwork for the construction activities to come. They detail the proposed erosion and sediment control measures, and their intended locations. This phase also allows for crucial feedback and input from relevant stakeholders, environmental agencies, and experts to fine-tune the plans before the project advances further.

3 Stormwater Analysis Methodology

Arcadis used an H&H model to understand the magnitude and extent of flooding within the study area. After creating a baseline conditions model in which the isolation strategies described in **Section 2** are implemented along with the FBS, various drainage solutions were tested to evaluate the ability of each to reduce remaining flooding.

3.1 Modeling Software

H&H modeling software can be used to simulate the rainfall-runoff relationship and the resulting amount of stormwater runoff to be managed by drainage infrastructure. The hydraulic component of the software estimates the water levels and flow rates within sewers (1-dimensional, or 1D) and at the surface, such as street flooding (two-dimensional, or 2D). 2D representation is important as water can travel in many paths when at the surface; the software can estimate the quantity and direction of overland flows, and ponding over time at low points. 2D representation also allows for the visualization of flooding extents and depths. Innovyze's (an Autodesk Company) InfoWorks Integrated Catchment Model (ICM) software was used to conduct the study.

3.2 Data Sources

Table 3-1 lists the data sources used for the modeling effort:

Table 3-1. Data Sources

Data Type	Source	Purpose
NYCDEP Long Term Control Plan (LTCP) model	New York City Department of Environmental Protection and South Battery Park City Resiliency Project (SBPCR)	Used as the basis for the updated North/West model and includes model updates made under the SBPCR.
Sewer GIS Network (i.e., existing sewers, sewer structures, outfalls, pumps)	New York City Department of Environmental Protection	Used to build the sewer network representation in the model environment.
Survey Data	North BPC Survey (AECOM/NAIK), from 3/21/2021, North Battery Park City Resiliency Project North/West Survey (AECOM/Matrix), from 10/13/2022, North/West Battery Park City Resiliency Project	Used to refine the sewer network representation in the model environment.
Sewer As-builts	New York City Department of Environmental Protection, New York City Department of Transportation	Used to verify and refine the sewer network representation in the model environment.

Data Type	Source	Purpose
Surface Elevation Data	New York City 2017 digital elevation model (DEM)/Topobathymetric LiDAR from NYC Open Data[2] Survey data (see above)	Used to build a 2D flooding surface, or mesh, in the model. The mesh consists of finite, triangular elements that, when grouped together, form a continuous surface on which water can pond and flow. Used survey data to refine the mesh.
Rainfall Hyetographs	National Oceanic and Atmospheric Administration (NOAA) Atlas 14, Volume 10, 2nd Quartile, 50th Percentile[3]	Used to specify the amount of rainfall over time for storm conditions.
Sea Level Rise Conditions	New York City Panel on Climate Change (NPCC) 2019 Report Chapter 3: Sea Level Rise[4]	Used to estimate the effect of climate change, resulting in sea level rise, on the tide and surge conditions simulated in the model.
Wastewater Flows	2014 New York State Design Standards for Intermediate Sized Wastewater Treatment Systems and NYC's Upcoming City Environmental Quality Review Water and Sewer Manual	Used to account for wastewater flows conveyed by the sewers that occupy a portion of the sewer conveyance capacity.
Peak Storm Surge Stillwater Elevation	FEMA Preliminary Flood Insurance Rate Maps (PFIRM)	Used to establish the peak stillwater elevation for the surge condition.
Storm Surge Hydrograph	Mayor's Office of Climate and Environmental Justice (MOCEJ)/Department of Environmental Protection (DEP) Surge Hydrograph	Used to establish the shape of the storm surge hydrograph for the 100-year surge event.

[2] https://data.cityofnewyork.us/City-Government/Topobathymetric-LiDAR-Data-2017-/7sc8-jtbz
[3] https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_temporal.html
[4] https://www.nyas.org/annals/special-issue-advancing-tools-and-methods-for-flexible-adaptation-pathways-and-science-policy-integration-new-york-city-panel-on-climate-change-2019-report-vol-1439/

3.3 Model Construction

The basis for the model was the existing Long Term Control Plan (LTCP) Infoworks ICM model developed by NYC DEP. The goal of the LTCP is to identify appropriate CSO controls necessary to achieve water quality standards[5]. The LTCP model was built to assess the amount of water discharged into waterbodies through CSO outfalls. It was updated through the following steps:

1. The LTCP model was expanded with available DEP GIS data of sewers located in the street roadbed, survey information, and as-builts to add higher resolution within the study area. Individual building connections, sewer laterals, site connections, and roof leaders are not explicitly represented in the H&H model.
2. High-resolution catchment areas were delineated for the expanded network to identify which areas contribute stormwater runoff to individual sewers.

Under this initial effort, no flow monitoring data was collected in the study area to use in validation of the model. Flow monitoring data will be considered for collection in a future phase of work to validate the model.

3.4 Model Assumptions

For this analysis, the following assumptions were made:

- The peaks for rainfall and storm surge occur at the same time. See **Section 4.4.2** for more detail on this assumption.
- Except for the proposed FBS improvements, it is assumed that there are no changes in the built environment between current and future conditions (e.g., same buildings, population, acreage of open space, etc.).
- Additional flows into the protected area over (i.e. overtopping) or under (i.e. seepage) the FBS were incorporated into the model. Grading on the interior of the FBS was not changed (i.e., reflects current topography) but will be refined in a future step to account for proposed changes.
- PCSMPs identified to meet water quality requirements were not incorporated into the model.
- Private drainage and sewers were not incorporated in the model build-out but will be added once data is obtained through future survey and CCTV efforts.
- Coastal storm surge protection is assumed to already be in place as part of the SBPCR and other resiliency projects to prevent storm surge flood pathways south of the NWBPCR project.

[5] More information about the LTCP program can be found here: https://www1.nyc.gov/site/dep/water/citywide-long-term-control-plan.page

4 Analysis Results and Key Findings

4.1 Rainfall and Surge Conditions Considered

Storm and surge conditions that result in the 1% joint annual exceedance probability (AEP) were considered for the H&H analysis following FEMA certification guidelines. The following **Table 4-1** shows the combinations of rainfall and surge considered, with the 1% storm combinations highlighted in blue. All events below the 1% line have a lower AEP. For this phase of work, all storm events were assumed to have a duration of 24-hours to represent a hurricane-type event, consistent with other ongoing New York City coastal resilience projects. In addition to the 1% storms, two design storms listed in the scope of work were also considered, specified in **Table 4-2.**

Table 4-1. Approximate Joint Annual Probability of Rainfall and Surge Assuming Event Independence

Joint Annual Exceedance Probability (AEP) of Rainfall and Surge Assuming Event Independence								
		Rainfall Return Period (x-yr)						
		1	2	5	10	20	50	100
Storm Surge Return Period (x-yr)	1	100.0%	50.0%	20.0%	10.0%	5.0%	2.0%	1.0%
	2	50.0%	25.0%	10.0%	5.0%	2.5%	1.0%	0.5%
	5	20.0%	10.0%	4.0%	2.0%	1.0%	0.4%	0.2%
	10	10.0%	5.0%	2.0%	1.0%	0.5%	0.2%	0.1%
	20	5.0%	2.5%	1.0%	0.5%	0.25%	0.1%	0.05%
	50	2.0%	1.0%	0.4%	0.2%	0.1%	0.04%	0.02%
	100	1.0%	0.5%	0.2%	0.10%	0.05%	0.02%	0.01%
Sample 1% AEP Events for FEMA Base Flood Mapping Sensitivity Analysis								

Table 4-2. Interior Drainage Objectives and Criteria

Objective Component	Criteria
Primary Objective: Minimize Flooding >=1' Average Depth	Current risk: 100-year coastal storm, 5-year NOAA2Q 50th Percentile 24-hour rainfall, Present Day Sea Level. 0.2% AEP Future risk: 100-year coastal storm, 2-year NOAA2Q 50th Percentile 24-hour rainfall, 30" Sea Level Rise (SLR). 0.5% AEP

4.2 Quantifying Existing Flood Risk

The H&H baseline model with all isolation components and the proposed FBS in place (but not fully inclusive of proposed work reflected in the 60% design drawings) was used to simulate the above noted scenarios and develop maps of predicted flood depths and extents. The combination of the 2-year rainfall and the 100-year surge (a 0.5% storm) used to simulate future conditions along with modeled overtopping and seepage was a more conservative event and resulted in the most flooding greater than 1 foot within the study area, when compared against the 1% FEMA storms considered. All flood maps and design options discussed in the following sections therefore reference the future condition. See **Figure 4-1** for the rainfall and surge conditions used to create the proceeding flood maps in the following sections.

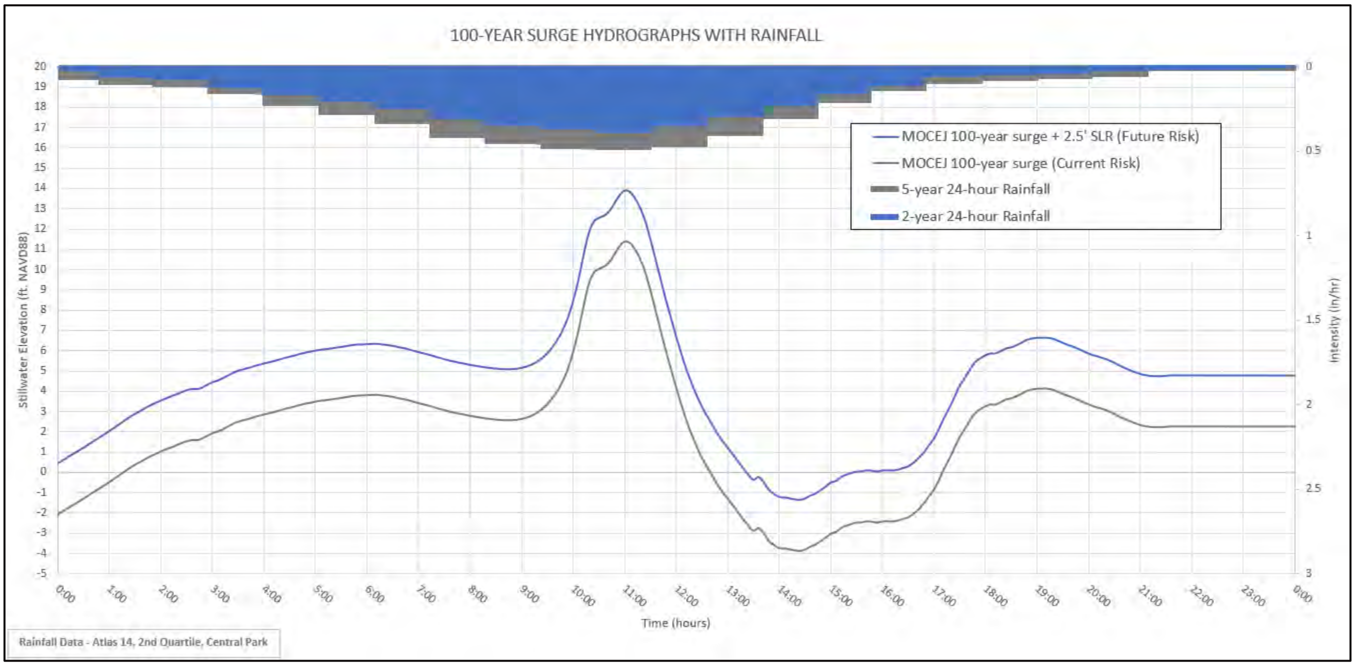


Figure 4-1. 5-year and 2-year Rainfall Hyetograph and 100-year Surge Conditions

Figure 4-2 shows the existing conditions with no FBS and NSI open for a 5-year 24-hour rainfall with the 100-year surge.

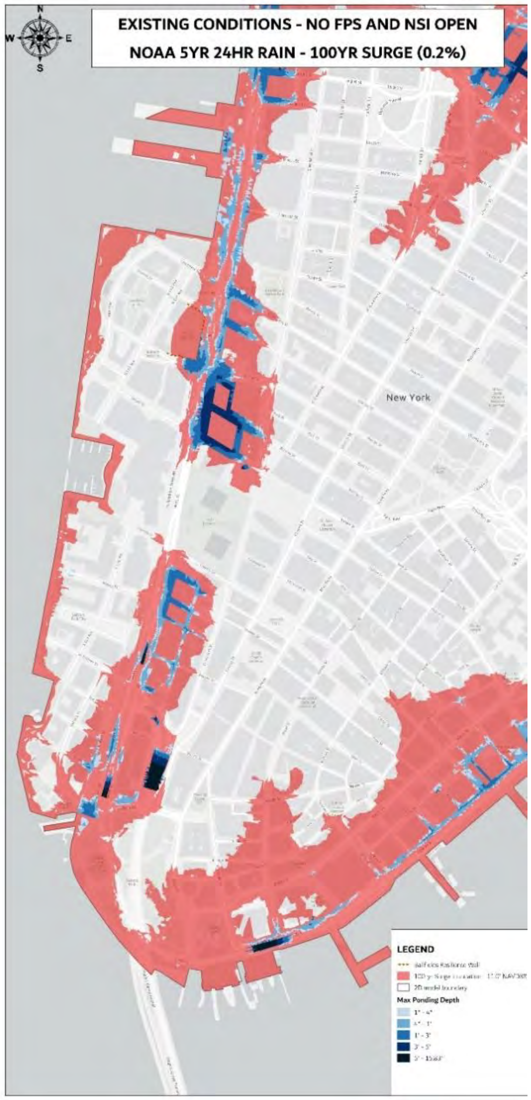


Figure 4-2. Existing Conditions with No FBS and NSI Open

Figure 4-3 shows the depths and extents of flooding under the present and future design conditions with the FBS and isolation components in place. The future condition simulations show more ponding than the present-day conditions due to more overtopping and seepage likely to occur in the future with sea level rise.

The 60% Interior Drainage Report showcases results from a 1D-2D model created as part of the N/WBPCR project. A separate rain-on-mesh (ROM) model in Infoworks ICM was constructed to help identify localized surface drainage issues within the NWBPCR project. In rain-on-mesh modelling, rainfall is applied directly to the 2D surface and flows overland to reach the sewer network. This approach allows for the assessment of localized flooding and drainage issues that may prevent water from reaching sewer catch basins. This differs from the 1D-2D model approach, which first calculates runoff based on hydrologic parameters and then loads it directly into the underground sewer system at specific manholes.

Although the modelling methodologies, as described above, are different and not calibrated, the ROM in-sewer flows, and flood extents closely match the 1D-2D approach. Documentation of the assumptions and inputs that were used to build the ROM model will be produced in a future phase of work. Flood maps and design options discussed in the following sections therefore reference the latest ROM results. Solutions were tested and compared against the baseline scenario to mitigate any flooding over 1 foot within the protected area. No proposed interior drainage infrastructure was modeled in SBPCR.

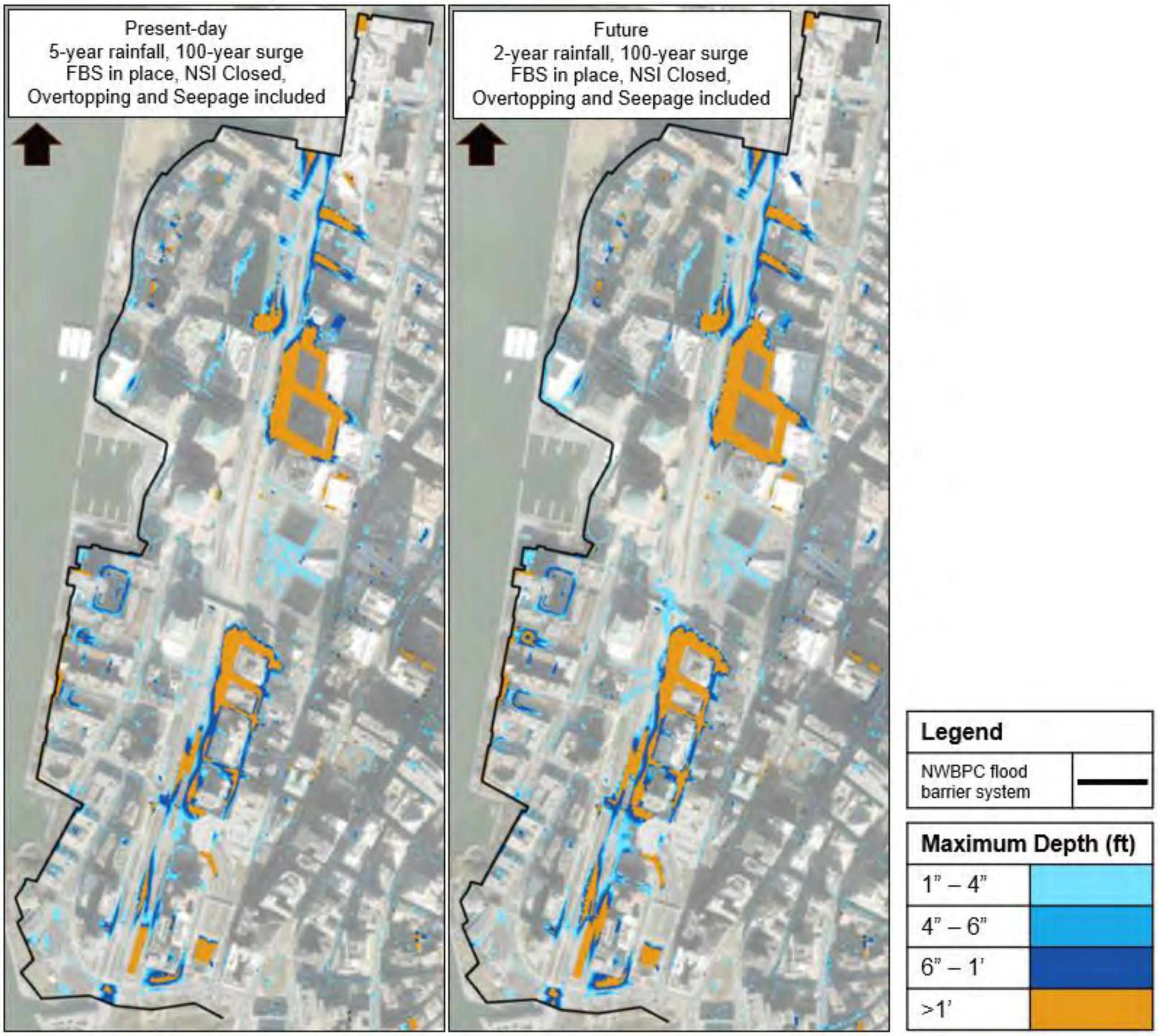


Figure 4-3. Baseline Conditions for Present and Future (ROM Results)

4.3 Proposed Solutions

The following drainage solutions were evaluated to mitigate stormwater flooding within the protected area:

- Permanent pumping stations
  - Preliminary model results showed that pumping is needed to mitigate flooding within the project area. A white paper (see **Appendix B**) was written on the considerations of temporary vs. permanent pumping. Discussion among the PDB team, BPCA, and key stakeholders led to a decision to proceed with a permanent pump station, which is advancing through design. The NWBPCR Pump Station is following an alternative design timeline that does not synchronize with

the 60% design. Initial modeling efforts focused on identifying proposed ranges of pumping rates and locations of pump stations. Further detailed modeling is now underway as part of the NWBPCR Pump Station design process to refine and finalize pumping conditions

- Additional Drainage Infrastructure/Conveyance
  - Conveyance refers to using gravity to convey water from one location to another, often through additional drainage infrastructure like catchbasins, trench drains, or underground sewers. Conveyance is a useful strategy to bring water into the sewer system but must be paired with other drainage strategies, like pumping, to discharge water out of the protected area. Additional conveyance is being proposed as part of this project. See the following sections for more detail.

4.4 Preliminary Pump Station Sizing Analysis

4.4.1 Pump Stations

The Arcadis team modeled potential pumping solutions for mitigating flooding under the present and future rainfall conditions with coincident storm surge. Flooding was quantified in terms of depth and extent of ponding averaged across the individual grid cells of the 2D surface. All figures in this report show the maximum water depth over a 24-hour event. The Arcadis team only sought to mitigate flooding in the northern BPC area where NCM-72 and NCM-73 are located as other city-led projects are being developed that will potentially mitigate flooding in the southern portion of BPC and in other parts of Lower Manhattan. As an example, the Financial District and Seaport Climate Resilience Master Plan project[6] would involve the construction of a 100-300 million gallon per day (MGD) pump station (referred to here as FiDi Pump Station) that would mitigate flooding in the Financial District and Seaport while having the potential to mitigate flooding in the southern area of BPC with some additional drainage improvements within the BPC drainage area to convey additional flows into the interceptor. With a Lower Manhattan solution in place, connections to the interceptor are assumed to convey flows to a FiDi Pump Station and would require opening and/or bypassing of the NSI gates, which may result in BPC no longer being isolated from the interceptor. In this event, FEMA accreditation would need to be reassessed.

A consolidated pump station was considered to mitigate flooding up to 1 foot in the NWBPCR project area. To consolidate flows into one pump station, new stormwater and combined hydraulic connection sewers are required to connect the areas contributing to outfalls NCM-72 and NCM-73. **Figure 4-4** showcases model simulations in which the proposed 60 MGD pump station and hydraulic connection sewer mitigate flooding in Battery Park City and east of 9A to a degree. See new connection sewers highlighted in **Figure 4-5** and in more detail within the 60% design plans. Two pump stations at NCM-72 and NCM-73 were also considered but a single pump station and associated conveyance infrastructure was found to achieve the same benefit as a two-pump station option. A single, consolidated pump station is therefore being further developed in lieu of a two-pump station option to reduce operational requirements.

A 60 MGD pump station was modeled near Stuyvesant High School along with the identified drainage improvements. The 60 MGD pump station flow rate is specific to the assumptions of the 60% design stage and will be continually refined during future phases of design. With the proposed consolidated pump station in place, most of the flooding over 1 foot is mitigated within the protected area north of Vesey Street, under the present and future design events, as shown in **Figure 4-4**.

[6] https://fidiseaportclimate.nyc/

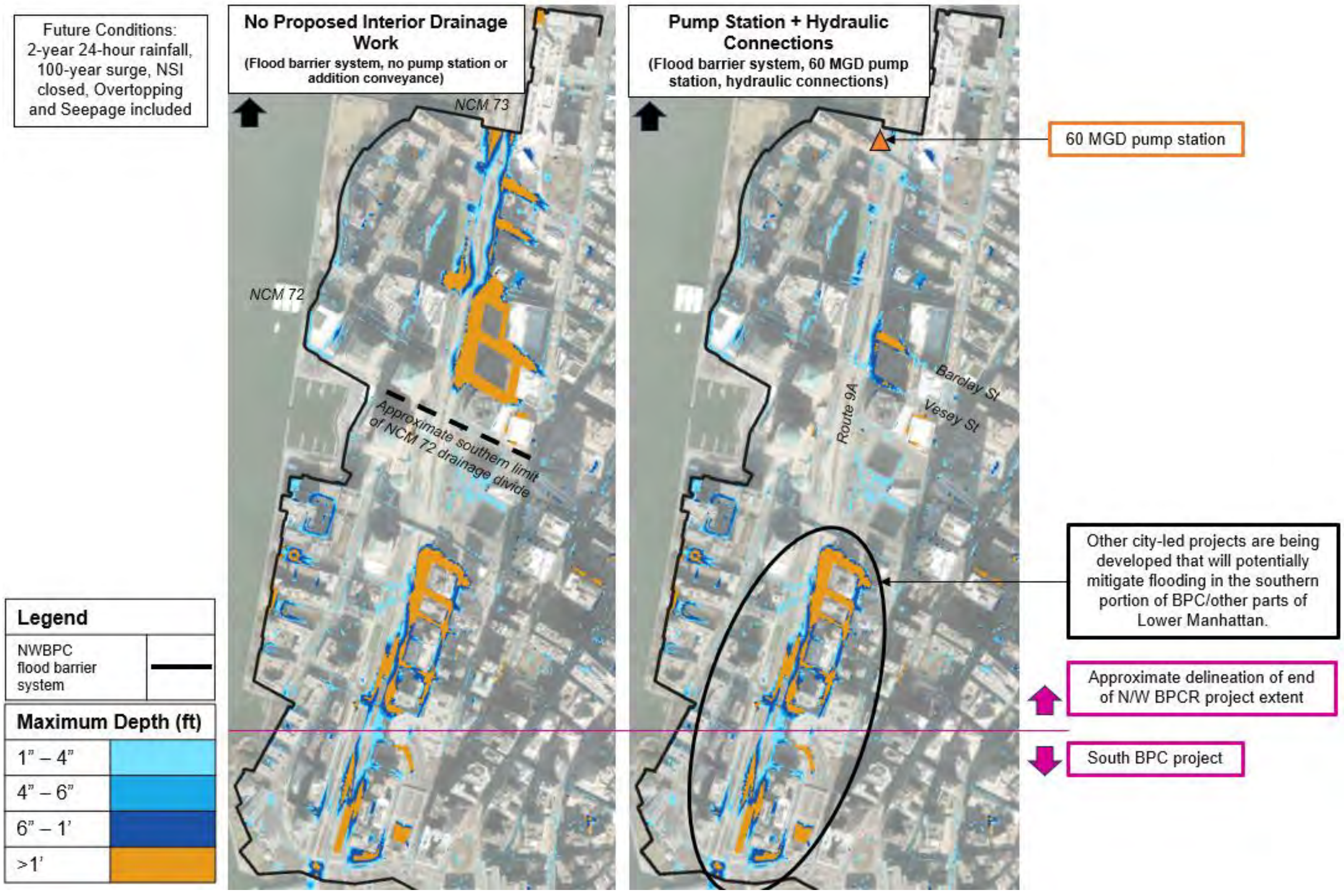


Figure 4-4. Baseline (No Pump/Interior Drainage Work) (Left), Proposed Pumping and Conveyance (Right)

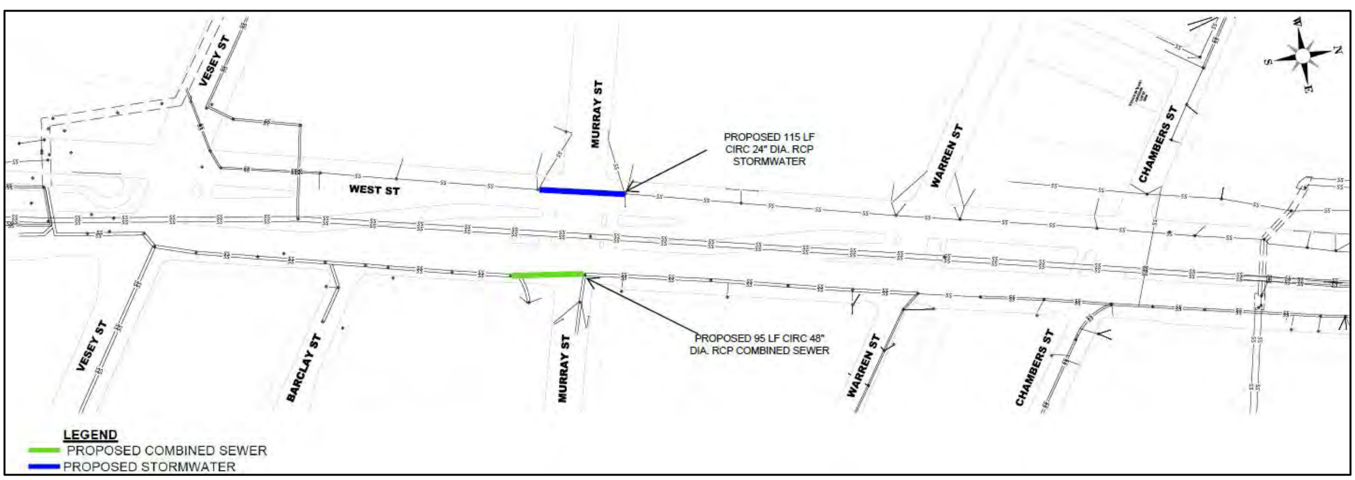


Figure 4-5. Hydraulic Connections included in Figure 4-4

Along with coordinating with BPCA, the design team has been coordinating the pump station design with DEP through a series of technical memos (TMs). A siting analysis was performed as part of TM-1 and Stuyvesant High School Plaza was selected as the preferred location. See TM-2 and TM-3 as part of the pump station facility planning process for the preliminary design of the pump station, and TM-4 for an analysis of conveyance alternatives available. A full cost and constructability analysis is ongoing and will be presented in future phases of work.

4.4.2 Sensitivity Analysis: Impact of Timing of MOCEJ/DEP Surge Hydrograph

A sensitivity analysis was completed to identify if the peak of the MOCEJ/DEP surge hydrograph should coincide with the peak of modelled rainfall or maintain an approximate 1PM peak as shown in previous modelling efforts (such as in the South Battery Park City Resiliency Project). The 30% Interior Drainage Report outlines a sensitivity analysis that was completed to identify if the proposed pumping and conveyance solution would need to be altered based on the 100-year surge condition applied in the model. The different surges considered were the observed Hurricane Sandy surge hydrograph scaled to the FEMA 100-year surge stillwater elevation, a design bell curve hydrograph developed by Arcadis based on the FEMA 100-year surge stillwater elevation, and a separate hydrograph developed by the Mayor's Office of Climate and Environmental Justice (MOCEJ) with DEP and used by AECOM in previous analyses. Since 30%, the project has shifted to modeling the MOCEJ/DEP surge to achieve consistency between NWBPCR and previous projects utilizing that surge hydrograph. However, additional modelling was needed to decide which was more conservative – coinciding the peaks of rainfall and surge or having the peak of surge occur after the peak of rainfall. **Figure 4-6** shows the two MOCEJ/DEP hydrographs and the timing of the peaks, as simulated, relative to the peak rainfall timestep of 11 AM.

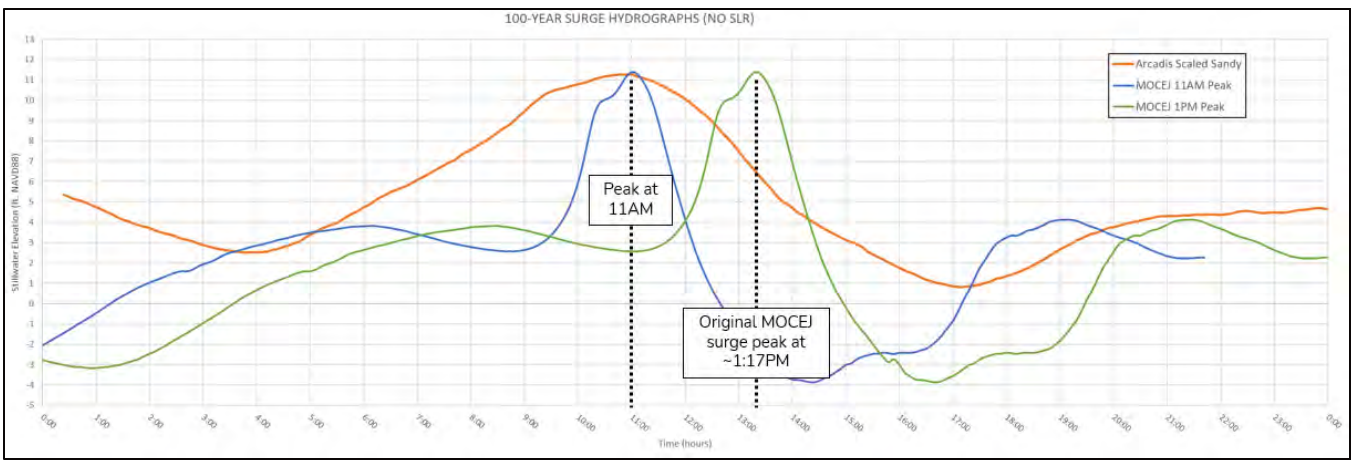


Figure 4-6. Surge Conditions Used in Sensitivity Analysis

Figure 4-7 shows the baseline flooding conditions with the 2-year rainfall and two surge conditions applied. The MOCEJ/DEP surge hydrograph with the peak at 11AM coinciding with the peak of rainfall resulted in the most conservative baseline, with 13.8 acres of flooding over 1 foot. The MOCEJ/DEP surge hydrograph with the peak near 1PM resulted in one less acre of flooding over 1 foot (12.8 acres). The 11AM surge hydrograph was therefore selected for the remaining modelling tasks associated with the NWBPCR project.

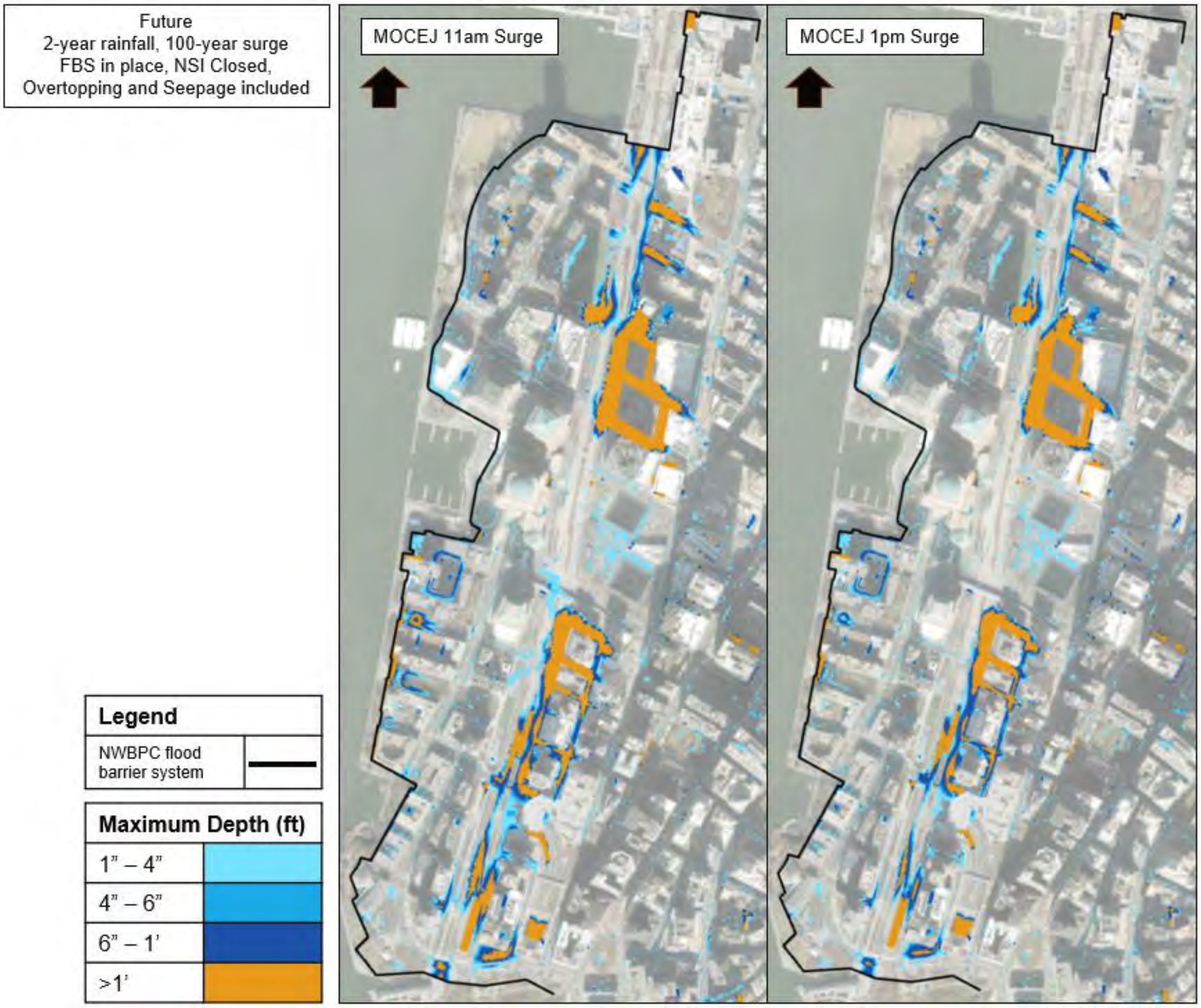


Figure 4-7. Sensitivity Analysis - MOCEJ/DEP Surge Hydrograph

4.4.3 Cloudburst Analysis

BPCA requested a more robust rainfall analysis to evaluate flooding under less frequent and more intense rainfall events (i.e., Cloudbursts) due to the significant investment of the proposed work. The purpose of the additional Cloudburst analysis was to evaluate the impact to BPC under more extreme events and consider minor adjustments to planned infrastructure that can extend the effectiveness of the mitigation to reduce flooding under a broader range of storm events.

The additional analysis was divided into current and future conditions, both of which use the rain-on-mesh model developed for this project. The current scenarios considered the 1-hour and 3-hour durations for the 10-year and 100-year rainfall events. The 10-year, 1-hour duration storm is currently being considered by NYCDEP as part of the City's Cloudburst program; including the 10-year, 3-hour duration event along with the 1-hour and 3-hour, 100-year events helped to provide a more detailed understanding of the impacts of high intensity events. These were evaluated under MHHW to focus on rainfall risk independent of storm surge.

Future conditions considered projected increases to the 100-year rainfall to account for climate change, based on projections from the Northeast Regional Climate Center (NRCC)[7]. These extreme rainfall conditions were evaluated assuming projected SLR as well as a separate scenario that also includes the 100-year surge to provide an anticipated "bookend" on potential interior flooding risk. While it is possible that the City would experience even more extreme conditions, the joint probability of this event occurring would be less than 0.01% (or a 1 in 10,000-year event).

Sizing infrastructure to manage this type of extreme event is not likely to be cost effective. However, analyses like these can be used to examine the extent of potential residual interior flooding risk. Maps were prepared for these future scenarios to be presented to BPCA but no adjustments to planned mitigation infrastructure were recommended.

Table 4-3 below outlines the additional modeling scenarios for more extreme rainfall events. "Baseline" conditions included the existing network but with near-surface-isolation included based on the notes within the table. "Proposed" included the proposed NW BPCR 60 MGD Pump Station as well as proposed conveyance, also described as Alternative 5 within TM-4 as part of the pump station design effort. The Set 1 scenarios were used to propose minor design modifications to mitigate ponding over 1 foot, while Set 2 scenarios were only used to examine the extent of residual interior flooding risks in extreme conditions. Activation levels for the pump station were not assessed as part of this analysis, but an assumption was made that the pump station would be set to activate during cloudburst events. Future work through the design of the pump station will develop the pump station controls in more detail. Other key assumptions were that the flood barrier system is in place and that there are tide gates at every outfall in Lower Manhattan.

Table 4-3. Additional Cloudburst Scenarios

Set 1: Pluvial Analysis

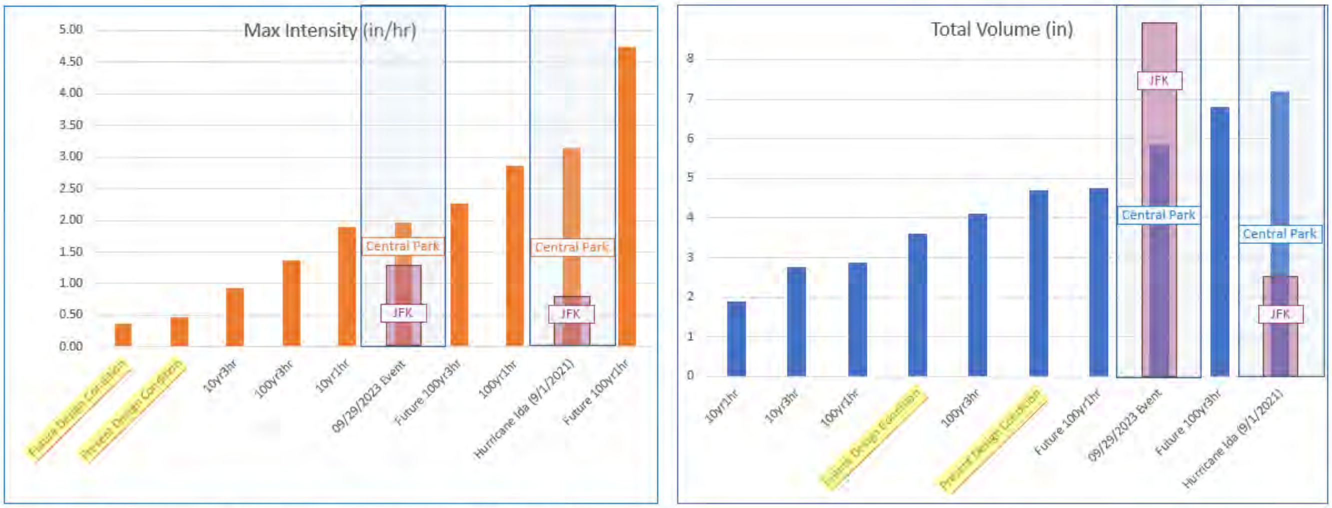
Rainfall Return Period	Durations (hr)	Assumed Boundary Conditions	Near-surface-isolation	OT and Seepage
Current 10-year (Baseline + Proposed)	1, 3	Present-day MHHW	Open	Not included
Current 100-year (Baseline + Proposed)	1, 3	Present-day MHHW	Open	Not included

Set 2: Future Pluvial + Surge/SLR Analysis

Rainfall Return Period	Durations (hr)	Assumed Boundary Conditions	Near-surface-isolation	OT and Seepage
Future 100-year (Baseline + Proposed)	1, 3	2050s SLR	Open	Not included
Future 100-year (Baseline + Proposed)	1, 3	100-year surge (MOCEJ) + 2050s SLR	Closed	Included

The following graphs in **Figure 4-8** show the proposed rainfall events as part of the Cloudburst analysis in terms of maximum intensity in inches per hour and total rainfall depth in inches. The NWBPCR present and future design conditions are also included for comparison.

[7] https://ny-idf-projections.nrcc.cornell.edu/

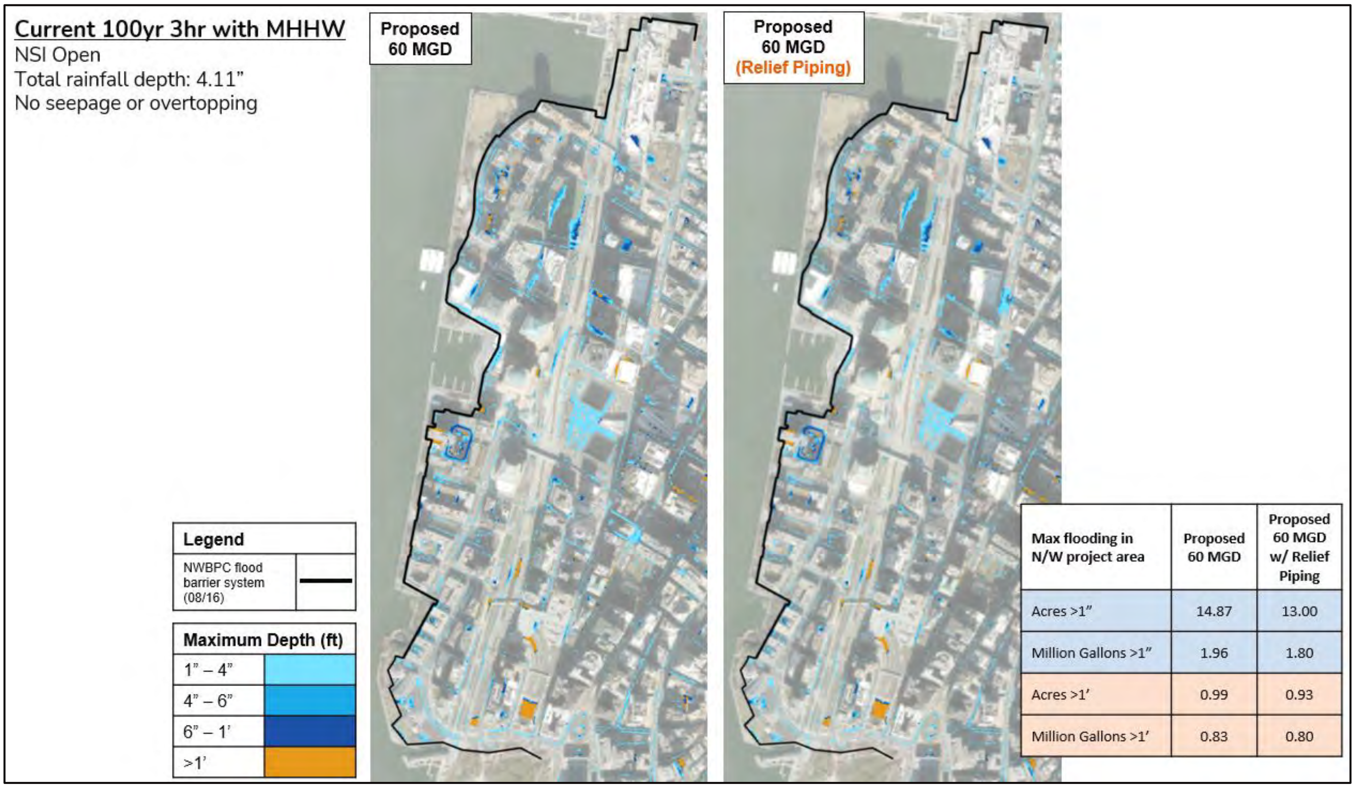


Source for Pluvial Analysis storms: Atlas 14 Central Park (assumed constant intensity for cloudburst events)
Source for Ida and 9/29: ASOS Central Park or JFK rain gauge data

Figure 4-8. Cloudburst Events (Intensity and Volume)

See **Appendix C** for all the maps created for Set 1 and 2 scenarios as part of the Cloudburst analysis. The proposed NW BPCR pump station continued to mitigate all ponding over 1 foot in the northwest portion of the protected area for the 10-year, 1-hour and 3-hour events. For the 100-year, 1-hour and 3-hour event, the pump station helped mitigate ponding but some ponding over 1 foot remained without additional modifications to the design. All the future pluvial Set 2 scenarios showed considerable ponding over 1 foot even with the proposed drainage improvements in place; in each of the four scenarios, however, the pump station as proposed continued to reduce some of the surge ponding.

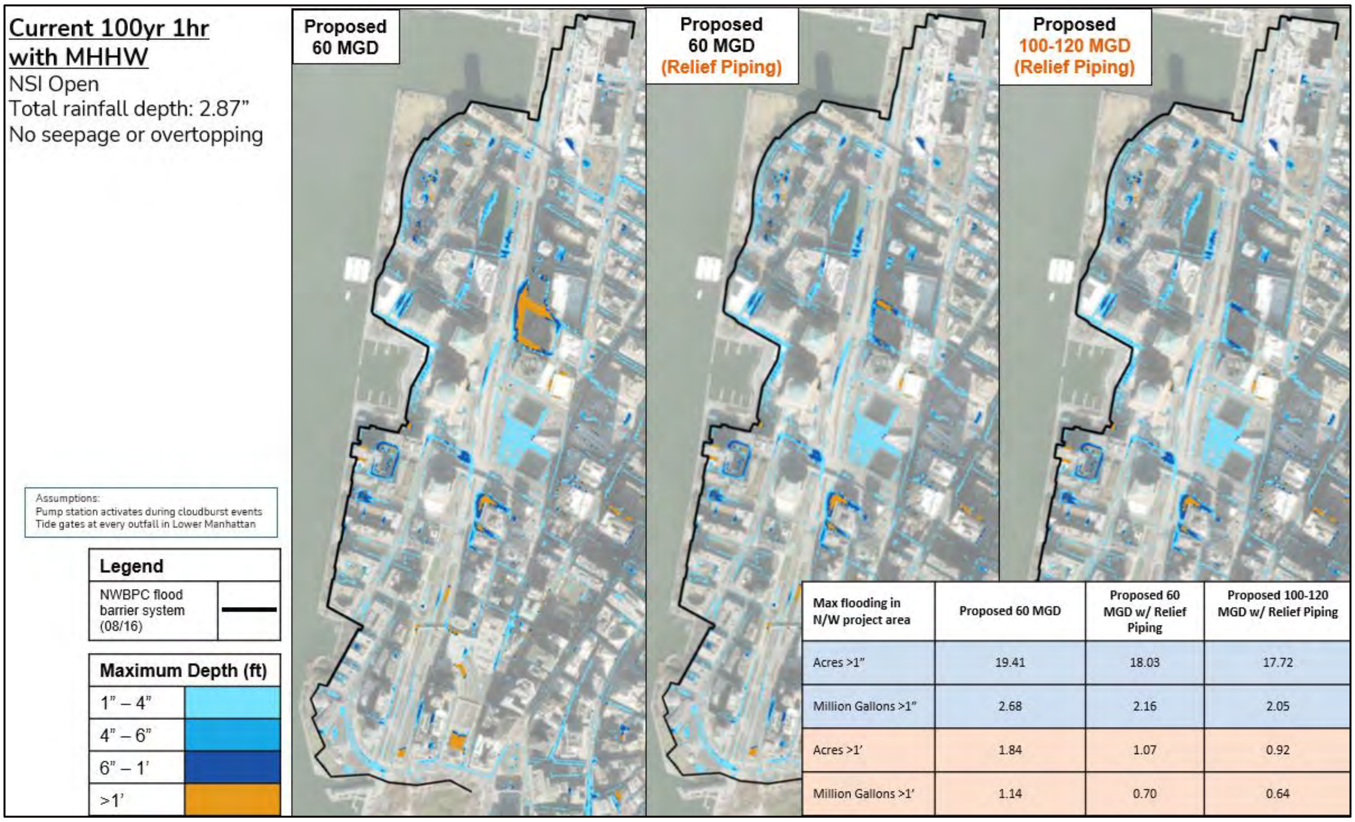
To mitigate all of the flooding over 1 foot in the current 100-year 3-hour scenario (the current scenario with the second most surface ponding), a restriction at the regulator would need to be bypassed with relief piping to bring more water to the pump station. If relief piping is in place, the pump station as proposed would mitigate all ponding over 1 foot. See **Figure 4-9** for the results of the 100-year 3-hour scenario with and without this additional modification in place. The Cloudburst analysis was performed prior to the shift in the floodwall alignment in Reach 1; for details on the latest floodwall alignment, see the 60% design plans.



Max flooding in N/W project area	Proposed 60 MGD	Proposed 60 MGD w/ Relief Piping
Acres >1”	14.87	13.00
Million Gallons >1”	1.96	1.80
Acres >1’	0.99	0.93
Million Gallons >1’	0.83	0.80

Figure 4-9. 100-year 3-hour Cloudburst Results with Proposed Work (Pump Station + Alternative 5 Conveyance) and Additional Modification

To mitigate all of the flooding over 1 foot in the current 100-year 1-hour scenario (the current scenario with the most surface ponding), the pump station capacity would need to be nearly doubled to mitigate ponding over 1 foot in addition to including bypass piping. This pump station capacity would likely not be cost effective and is not being recommended for implementation but stated to give an idea of the extent of work needed to be constructed to mitigate for such an intense rain event. The pump station as proposed and with relief piping still provides benefit to BPCA and reduces most of the ponding over 1 foot. See **Figure 4-10** for the results of the 100-year 1-hour scenario with and without these additional modifications in place.



Max flooding in N/W project area	Proposed 60 MGD	Proposed 60 MGD w/ Relief Piping	Proposed 100-120 MGD w/ Relief Piping
Acres >1"	19.41	18.03	17.72
Million Gallons >1"	2.68	2.16	2.05
Acres >1'	1.84	1.07	0.92
Million Gallons >1'	1.14	0.70	0.64

Figure 4-10. 100-year 1-hour Results with Proposed Work (Pump Station + Alternative 5 Conveyance) and Additional Modifications

4.5 Localized Ponding

The design team has reviewed the H&H results to identify areas where ponding exceeds 1 foot or where existing problems have been identified to evaluate if localized drainage improvements are required. Areas that were evaluated include Teardrop Park and the Gateway Building. See below for further details.

4.5.1 Teardrop Park

The model assumes that the Teardrop conveyance infrastructure is adequately maintained and free from blockages. The results indicate that there is no significant ponding exceeding 1 foot in Teardrop Park to the north/west or south areas. This underscores that NWBPCR project is not contributing to flooding in Teardrop Park above 1 foot. However, there is still some residual ponding ranging between 1 inch and 1 foot.

Since there is no ponding exceeding 1 foot, no drainage improvements in Teardrop Park are proposed as part of the 60% design.

4.5.2 Gateway Building

The H&H modeling results show flooding near the Gateway Building. The extent of flooding largely occurs on the west side and at two existing patios located on the north and south sides of the Gateway building. These patios are sunken areas that lead into the building through egress doors and are approximately seven feet below the surrounding areas. The patios consist of area drains that discharge stormwater runoff to an existing 24-inch outfall located below the south patio. During storm surge conditions when water in the outfall is unable to drain, water will back up into the system and subsequently into the patio areas through the area drains. This results in flooding greater than one foot, which is mainly due to rainfall and seepage.

See **Section 2.7.1** for the proposed drainage improvements to mitigate the flooding.

5 Future Work

Stormwater management components will continue to be iteratively evaluated for constructability and cost. Details on the sizing of individual drainage design components are ongoing. Operation of future stormwater infrastructure is also a key consideration and will be assessed in future phases of work.

6 References

1. Topobathymetric LiDAR Data 2017, https://data.cityofnewyork.us/City-Government/Topobathymetric-LiDAR-Data-2017-/7sc8-jtbz.
2. NOAA Precipitation Frequency Data Server, https://hdsc.nws.noaa.gov/hdsc/pfds/pfds_temporal.html.
3. Advancing Tools and Methods for Flexible Adaptation Pathways and Science Policy Integration – NYAS, (https://www.nyas.org/annals/special-issue-advancing-tools-and-methods-for-flexible-adaptation-pathways-and-science-policy-integration-new-york-city-panel-on-climate-change-2019-report-vol-1439/.
4. NYC Department of Environmental Protection - LTCP Program, https://www1.nyc.gov/site/dep/water/citywide-long-term-control-plan.page.
5. Electronic Code of Federal Regulations (e-CFR) - Title 44, Chapter I, Subchapter B, Part 65, Section 65.10, https://www.ecfr.gov/current/title-44/chapter-I/subchapter-B/part-65/section-65.10.
6. Department of Environmental Protection. *New York City Stormwater Manual (Appendix to Chapter 19.1 of Title 15 of the Rules of the City of New York)*. February 15, 2022.
7. Department of Environmental Protection. *Stormwater Management Unified Stormwater Rule*. February 15, 2022.
8. New York State Department of Environmental Conservation. *2014 NYS Design Standards for Intermediate Sized Wastewater Treatment Systems*. March 5, 2014.

Appendix A

Hydraulic Analysis

Introduction:

The purpose of this calculation is to provide an overview of the conveyance capacity of the proposed piping that's being installed for interior drainage purposes. Tributary drainage areas (TDAs) were delineated for individual drainage networks by referencing spot elevation data provided by survey work of existing conditions and by referencing proposed grading and landscaping plans. Once the peak flow was calculated for the TDA, Manning's Equation was utilized to assess the compatibility of the pipes of that given area with the intended flow. Typically, the hydraulic capacity of the most downstream component of a piping network was analyzed for its respective TDA. The peak flow for each TDA was calculated using the Rational Method, taking into account a rainfall intensity of 5.95 inches per hour, as required by NYCDEP.

Preliminary calculations were performed in order to provide basic parameters for evaluating piping capacity for (1) a given TDA, in this case, half-an-acre and one-acre; (2) a given pipe material, either RCP or HDPE, and (3) a given minimum slope of 1%. When determining the peak flows, all calculations assumed a runoff coefficient of 0.85 for completely impervious areas as a conservative approach. Peak flows calculated for individual TDAs were compared to the previously determined pipe capacity parameters in order to easily assess the compatibility of the most downstream component of the network, as this ensures the rest of the upstream proposed piping within the TDA's drainage network can adequately accommodate the flow.

Conveyance Capacity (Manning's Equation):

$$v = \left(\frac{1.49}{n}\right) R^{2/3} \sqrt{S}$$

where v = velocity, ft/sec
n = Manning's roughness coefficient, based on material and age of pipe
R = hydraulic radius, ft
S = slope of the pipe

Peak Flow (Rational Method):

$$Q_P = \frac{C_w TDA}{7320}$$

where Q_P = the developed peak flow, cfs
C_w = the weighted runoff coefficient
TDA = tributary drainage area, ft^2
7320 = 43,560 ft^2/ac divided by the rainfall intensity for the design storm and rainfall

Design Storm and Rainfall:

where Return period = 5 year
Rainfall intensity = 5.95 in/hr
Time of concentration = 6 minutes

The calculations for the hydraulic analysis are shown on pages 3 and 4. The orange highlight indicates an inputted value.

Hydraulic Computations (half-acre):

Drainage Area to Piping (A_s) =	21780	sf
Impervious Area =	21780	sf
Impervious Area =	0.5	ac

Peak Flow Computations:

C_w =	0.85		
Q_P =	2.53	cfs	$Q_P = C_w \times TDA / 7320$

Conveyance Capacities Calculations (RCP):

n =	0.013		
d =	12	in	Assume circular pipe
d =	1	ft	
A =	0.79	sf	$A = \pi \times r^2$
R_H =	0.25	ft	$R_H = A / P$ assuming full flow (d / 4)
S =	0.01		
v =	4.55	fps	$v = (1.49 / n) \times R^{(2/3)} \times \sqrt{S}$
Q =	3.57	cfs	Mean flow $Q = v \times A$

Conveyance Capacities Calculations (HDPE):

n =	0.009		
d =	10	in	Assume circular pipe
d =	0.83	ft	
A =	0.55	sf	$A = \pi \times r^2$
R_H =	0.21	ft	$R_H = A / P$ assuming full flow (d / 4)
S =	0.01		
v =	5.82	fps	$v = (1.49 / n) \times R^{(2/3)} \times \sqrt{S}$
Q =	3.17	cfs	Mean flow $Q = v \times A$

12” is the minimum diameter required for a reinforced concrete pipe to accommodate the peak flow of a half-sized acre TDA.

10” is the minimum diameter required for an HDPE pipe to accommodate the peak flow of a half-acre sized TDA.

Hydraulic Computations (one-acre):

Drainage Area to Piping (A_s) =	43560	sf	
Impervious Area =	43560	sf	
Impervious Area =	1	ac	

Peak Flow Computations:

C_w =	0.85		
Q_P =	5.06	cfs	$Q_P = C_w \times TDA / 7320$

Conveyance Capacities Calculations (RCP):

n =	0.013		
d =	15	in	Assume circular pipe
d =	1.25	ft	
A =	1.23	sf	$A = \pi \times r^2$
R_H =	0.31	ft	$R_H = A / P$ assuming full flow (d / 4)
S =	0.01		
v =	5.28	fps	$v = (1.49 / n) \times R^{(2/3)} \times \sqrt{S}$
Q =	6.48	cfs	Mean flow $Q = v \times A$

Conveyance Capacities Calculations (HDPE):

n =	0.009		
d =	12	in	Assume circular pipe
d =	1	ft	
A =	0.79	sf	$A = \pi \times r^2$
R_H =	0.25	ft	$R_H = A / P$ assuming full flow (d / 4)
S =	0.01		
v =	6.57	fps	$v = (1.49 / n) \times R^{(2/3)} \times \sqrt{S}$
Q =	5.16	cfs	Mean flow $Q = v \times A$

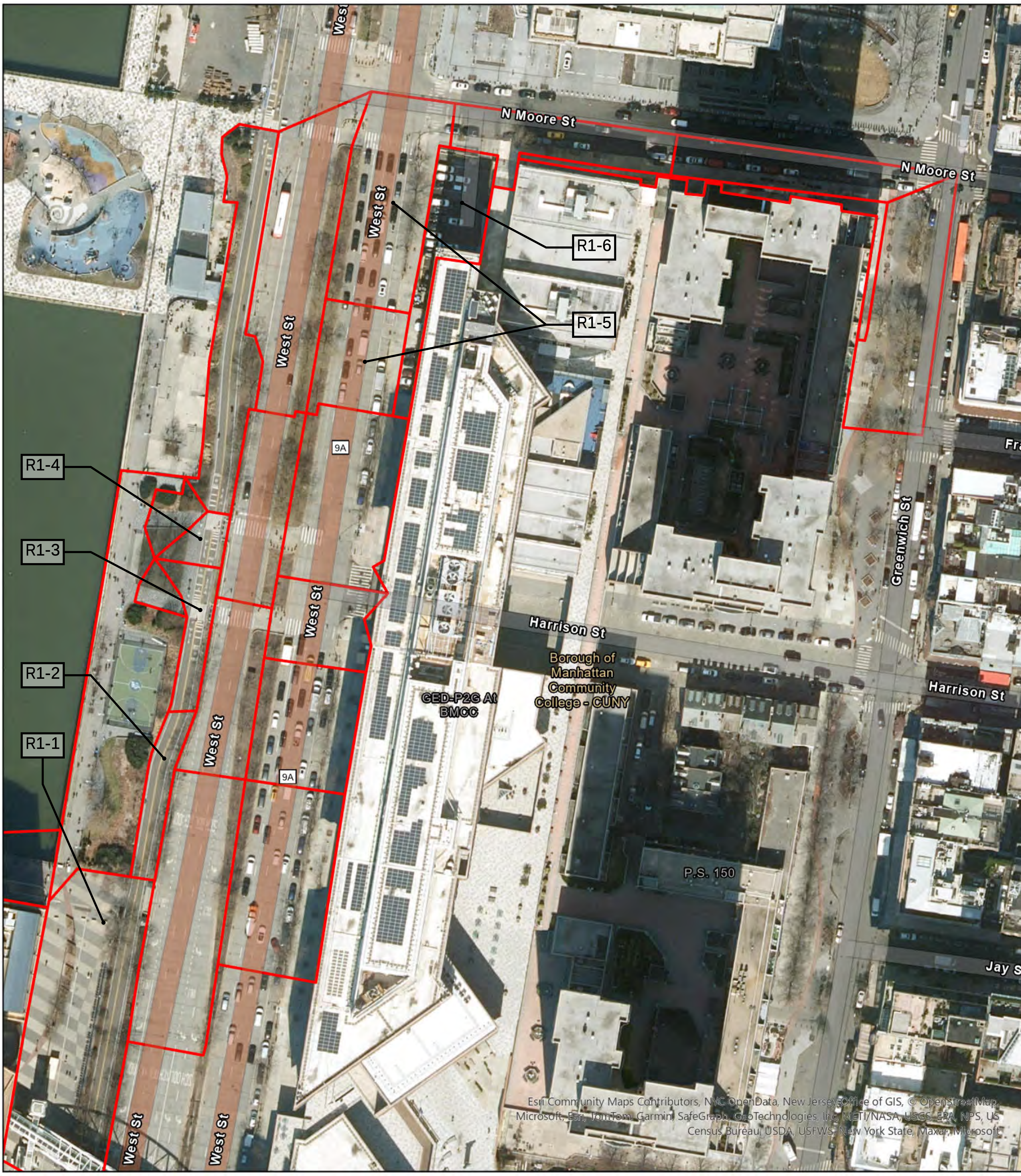
15” is the minimum diameter required for a reinforced concrete pipe to accommodate the peak flow of a half-sized acre TDA.

12” is the minimum diameter required for an HDPE pipe to accommodate the peak flow of a half-acre sized TDA.

The following table summarizes the peak flow required for the downstream pipe to accommodate within various TDAs, along with the piping characteristics. As shown, all pipes meet the minimum size requirements for the intended flows. TDA delineation outlines are shown on the following pages.

TDA ID	TDA (sf)	Peak Flow (cfs)	Pipe Material	Downstream Pipe Diameter	Notes
R1-1	11754	1.36	RCP	12"	
R1-2	6695	0.78	RCP	12"	
R1-1 + R1-2	18449	2.14	RCP	18"	
R1-3	6924	0.80	RCP	12"	
R1-4	4288	0.50	RCP	12"	
R1-3 + R1-4	11212	1.30	RCP	12"	
R1-5	18502	2.15	RCP	12"	
R1-6	9914	1.15	RCP	12"	
R1-5 + R1-6	28416	3.30	RCP	12"	
R3-1	8804	1.02	HDPE	12"	
R3-2	3516	0.41	RCP	12"	
R3-3	3646	0.42	HDPE	15"	
R4-1	15274	1.77	HDPE	12"	
R4-2	5152	0.60	HDPE	8"	8” HDPE can accommodate a minimum peak flow of 1.75 cfs; 8” conveyance is satisfactory.
R4-3	9260	1.08	HDPE	8"	
R4-1 + R4-2	20426	2.37	RCP	36"	
R4-2 + R4-3	14412	1.67	HDPE	15"	
R4-4	7900	0.92	RCP	18"	
R5-1	11511	1.34	RCP	24"	
R5-2	17044	1.98	HDPE	12"	
R5-3	16686	1.94	RCP	12"	
R5-4	26541	3.08	RCP	15"	
R5-5	13884	1.61	RCP	12"	
R5-6	6893	0.80	RCP	12"	
R5-7	2822	0.33	RCP	12"	
R6-1	565	0.07	RCP	12"	
R6-2	10363	1.20	RCP	12"	
R6-3	11847	1.38	RCP	12"	
R7-1	5263	0.61	HDPE	8"	
R7-2	3084	0.36	HDPE	8"	
R7-3	3823	0.44	RCP	12"	
R7-4	1907	0.22	HDPE	6"	6” HDPE can accommodate a minimum peak flow of 0.81 cfs; 6” conveyance is satisfactory.
R7-5	2641	0.31	HDPE	8"	
R7-6	11574	1.34	HDPE	8"	
R7-6 + R7-7	15132	1.76	RCP	12"	
R7-8	3421	0.40	HDPE	8"	
R7-6 + R7-7 + R7-8	18554	2.15	RCP	12"	
R7-9	6041	0.70	HDPE	8"	
R7-6 + R7-7 + R7-8 + R7-9	24595	2.86	RCP	15”	
R7-10	3267	0.38	HDPE	6"	

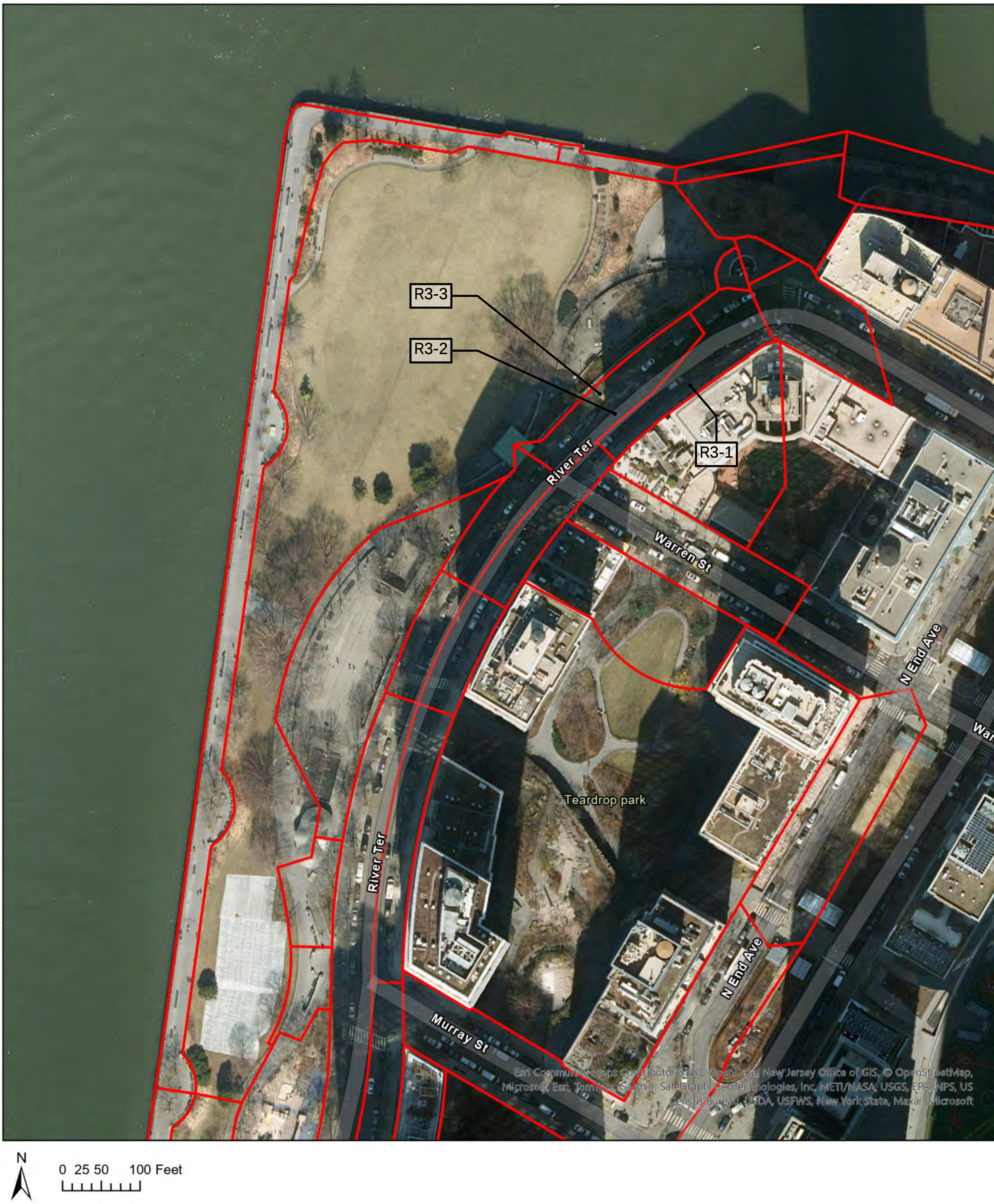
N/W Battery Park City Resiliency Project
Reach 1 - Tribeca / BMCC / HRPK
Drainage Areas



N/W Battery Park City Resiliency Project

Reach 3 - Rockefeller Park

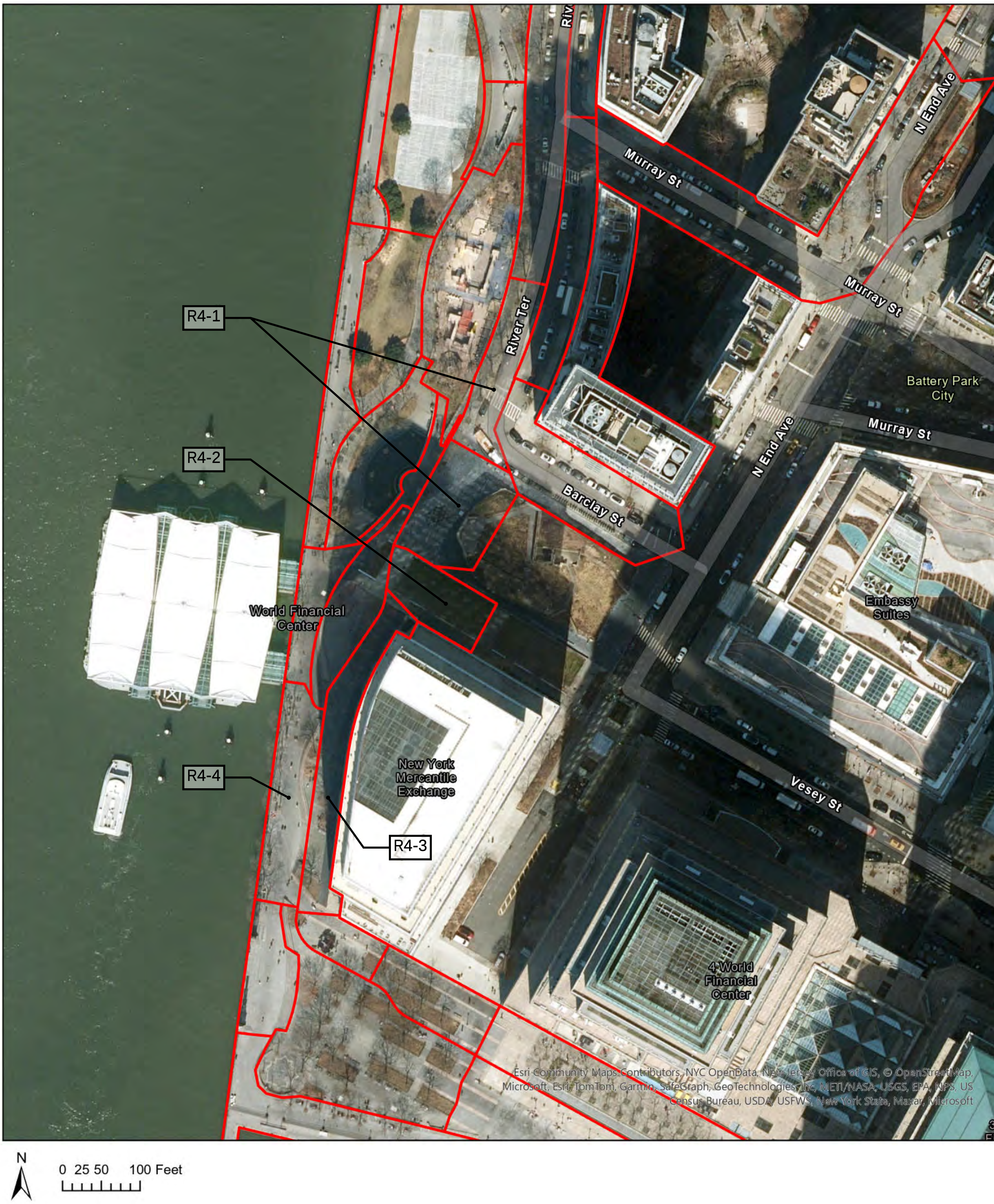
Drainage Areas



N/W Battery Park City Resiliency Project

Reach 4 - Belvedere Plaza

Drainage Areas



N/W Battery Park City Resiliency Project

Reach 5 - North Cove

Drainage Areas

N

0 25 50 100 Feet

N/W Battery Park City Resiliency Project

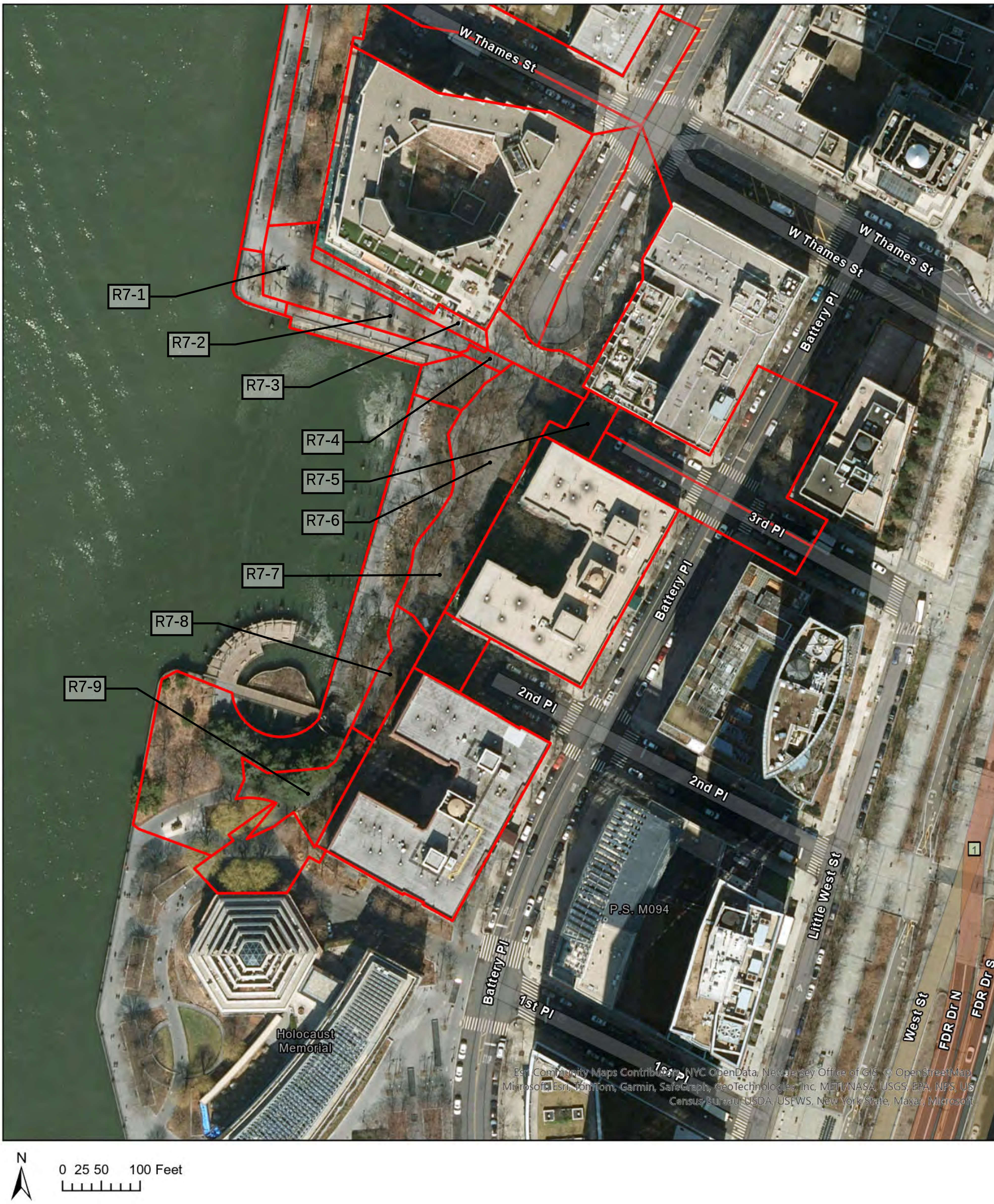
Reach 6 - South Esplanade

Drainage Areas

N/W Battery Park City Resiliency Project

Reach 7 - South Cove

Drainage Areas



Appendix B

Interior Drainage - Pumping Solutions

NEW YORK STATE OF OPPORTUNITY | Battery Park City Authority

North/West Battery Park City Resiliency Project

Interior Drainage - Pumping Solutions

August 18, 2023

Turner · CRUZ ARCADIS SCAPE BIG WSP

Version Control

Document Dated	Version No.	Revision No.	Date Issued	Description	Reviewed By
6/13/2023	0 (Draft)	0	6/13/2023	Submit to Turner EE Cruz for review	Peter Glus, Russ Dudley
6/28/2023	1	0	6/28/2023	Addressed Turner EE Cruz's comments	Peter Glus, Dan Rozell
7/3/2023	2	0	7/3/2003	Addressed Turner EE Cruz's backchecked comments	Dan Rozell, Russ Dudley
7/21/2023	3	0	7/21/2023	Addressed AECOM's comments	Russ Dudley
8/18/2023	4	0	8/18/2023	Updated language to address BPCA comments	Russ Dudley

Note:

1) All = Turner EE Cruz, Arcadis and its subconsultants (SCAPE/BIG/WSP, etc.)
2) Arcadis team = Arcadis and its subconsultants (SCAPE/BIG/WSP, etc.)

CONTENTS

TABLES

FIGURES

Acronyms & Abbreviations

AEP	Annual Exceedance Probability
BMCC	Borough of Manhattan Community College
BPCA	Battery Park City Authority
CSO	Combined Sewer Overflows
CLOMR	Conditional Letter of Map Revision
FBS	Flood-Barrier System
FEMA	Federal Emergency Management Agency
FIRMs	Flood Insurance Rate Maps
LOMR	Letter of Map Revision
N/W BPCR	North/West Battery Park City Resiliency
NYCDEP	New York Department of Environmental Protection
O&M	Operations and Maintenance
PDB	Progressive Design Build

1 Executive Summary

Through the North/West Battery Park City Resiliency (NWBPCR) project, Battery Park City is investing in protection from coastal storm surge and rain events. In addition to a coastal storm surge barrier, the resiliency project also involves isolation of the existing sewer system to prevent storm surge from entering the protected area through the sewer system. Interior flooding results from the inability of the sewer system to drain stormwater during storm surge events and rain events, and a pumping solution is required to manage the interior flooding.

A planned pump station in the Financial District, in combination with other conveyance improvements in the project area, currently being advanced through the concept design stage by the New York City Economic Development Corporation and the Mayor's Office of Climate and Environmental Justice, is shown to reduce the flooding in the southern portion of the project area based on preliminary modeling results. A single pump station location is proposed in the northern portion of the project area to address the remaining interior flooding not addressed by the Financial District pump station and associated conveyance improvements.

The combination of the 24-hour 5-year rainfall and the 100-year surge (considered a 0.2% joint annual probability storm) was used to size the proposed pump station which was found to be a more conservative approach than the 1% joint annual exceedance probability (AEP) required by FEMA certification guidelines. The proposed capacity of the pump station for the northern section of BPC within NWBPCR project, based on preliminary modeling results is approximately 60 MGD.

This report provides a detailed analysis of the evaluation conducted to compare the benefits and challenges between a temporary pumping solution and a permanent pumping solution. The evaluation includes a discussion of the permanent pumping solution as recommended solution by the PDB Team.

One temporary pumping solution assumes the use of portable diesel pumps and a permanent wet well to be constructed on site. The pumps will drain the CSO flow out of the permanent wet well. The portable pumps will be brought on site in advance of a flood event. To handle the proposed design flow, approximately eleven, 15-ft x 8-ft (approximate dimension of The Godwin Dri-Prime CD300M pump) diesel pumps would need to be positioned on the densely populated streets during an emergency condition ahead of a storm, which would be a challenging scenario to manage. Additionally, priming these units is known to be an arduous task, adding yet another challenge during an emergency condition, thus increasing the operating risks of the temporary pump station. The second temporary option assumed the installation of a permanent wet well (with additional valve pit and piping connections) with removable submersible centrifugal pumps to be temporarily installed during emergency events. Similar to the first option, the pumps and electrical equipment would need to be stored at a remote designated location for most of the year and brought on site in advance of a flood event on trailer trucks. Given the size and quantity of equipment required to handle the proposed design flow, this operation would demand a tremendous amount of planning and coordination in addition to the need for equipment maintenance during long-term storage.

As a result, the permanent pumping solution has the advantage of being more reliable and efficient to operate during a flood event and as such it is the recommended solution by the Design Team. Although a comprehensive site analysis will still have to be performed during design stages, the modeled solution used in this report is located at the Stuyvesant High School Plaza. This option proposes a pumping system that reduces flooding in the northern section of BPC. This system will consist of an influent chamber with a screening mechanism, an underground wet well equipped with seven (7) permanently installed submersible centrifugal pumps, a valve pit, a 48-inch force main that discharges into a CSO connecting vault downstream of the tide gates, and an elevated electrical room to power the pump station. The electrical room was rendered to match the existing area building architecture at the Stuyvesant High School Plaza. The management of the pump station will follow a periodic maintenance program to maintain the screening mechanism and the pumps during periods of non-usage. Preliminary siting evaluations have been outlined in the report. Detailed design with more comprehensive cost estimates will be evaluated in further design phases.

2 Introduction

The Battery Park City Authority (BPCA) is currently undergoing a resilience program to protect the community from future coastal surge and storm events and to comply with FEMA accreditation requirements. FEMA Accreditation is required to update the Flood Insurance Rate Maps (FIRM) for the study area and potentially lower associated flood insurance costs. As shown in Figure 2-1, this effort consists primarily of coastal protection, with the construction of a structural barrier starting in the southern area of Battery Park City (BPC) and continuing north to the North Moore intersection with Greenwich Street. BPCA has engaged the Progressive Design Build Team (PDB Team) to advance the North/West Battery Park City Resiliency (NWBPCR) project, which stretches from First Place along the Esplanade before crossing West Street and tying in north of Chambers Street.

In addition to a coastal barrier to protect against storm surge events, the PDB Team is responsible for the design of a system which can manage interior flooding resulting from rainfall generated within the City's sewer sheds. Flooding greater than an average of 1-foot above ground level within the interior area protected by the wall will need to be managed, by means of storm sewer improvements as well as the pumping outside the protected area.

The PDB team is developing a flooding mitigation solution only for the northern section (Reaches 1-4) of the BPC study area (referred to here as BPCA Pump Station, as shown in Figure 2-1), as part of the NWBPCR project, as other city-led projects are being developed that will potentially mitigate flooding in the southern portion of BPC and in other parts of Lower Manhattan (outside of NWBPCR project). As an example, the Financial District and Seaport Climate Resilience Master Plan project[1] would involve the construction of a 100-300 million gallon per day (MGD) pump station (referred to here as FiDi Pump Station) that would mitigate flooding in the Financial District and Seaport while having the potential to mitigate flooding in the southern area of BPC with some additional drainage improvements within the BPC drainage area to convey additional flows into the interceptor. **Figure 2-1** shows baseline flooding (considering 5-year 24-hour rainfall with 100-year surge and present-day sea level) in BPC with a flood protection system in place (left) with flooding over 1-foot mitigated through the implementation of pumping and drainage improvement solutions as part of the BPCA and Lower Manhattan Pump Stations (right). As proposed, the BPCA Pump Station would work in conjunction with the NSI system by pumping from a combined sewer overflow downstream of the regulator. This would isolate the pumping solution from CSOs in the southern portion of the project area and would not increase flooding within the SBPCR project area. With a Lower Manhattan solution in place, connections to the interceptor are assumed to convey flows to a future FiDi Pump Station and would require opening and/or bypassing of the NSI gates, which would result in BPC no longer being isolated from the interceptor. Since the extent and timeframe of a FiDi Pump Station are not known at this time, FEMA accreditation for the southern portion of the project area would need to be reassessed in the future.

[1] https://fidiseaportclimate.nyc/

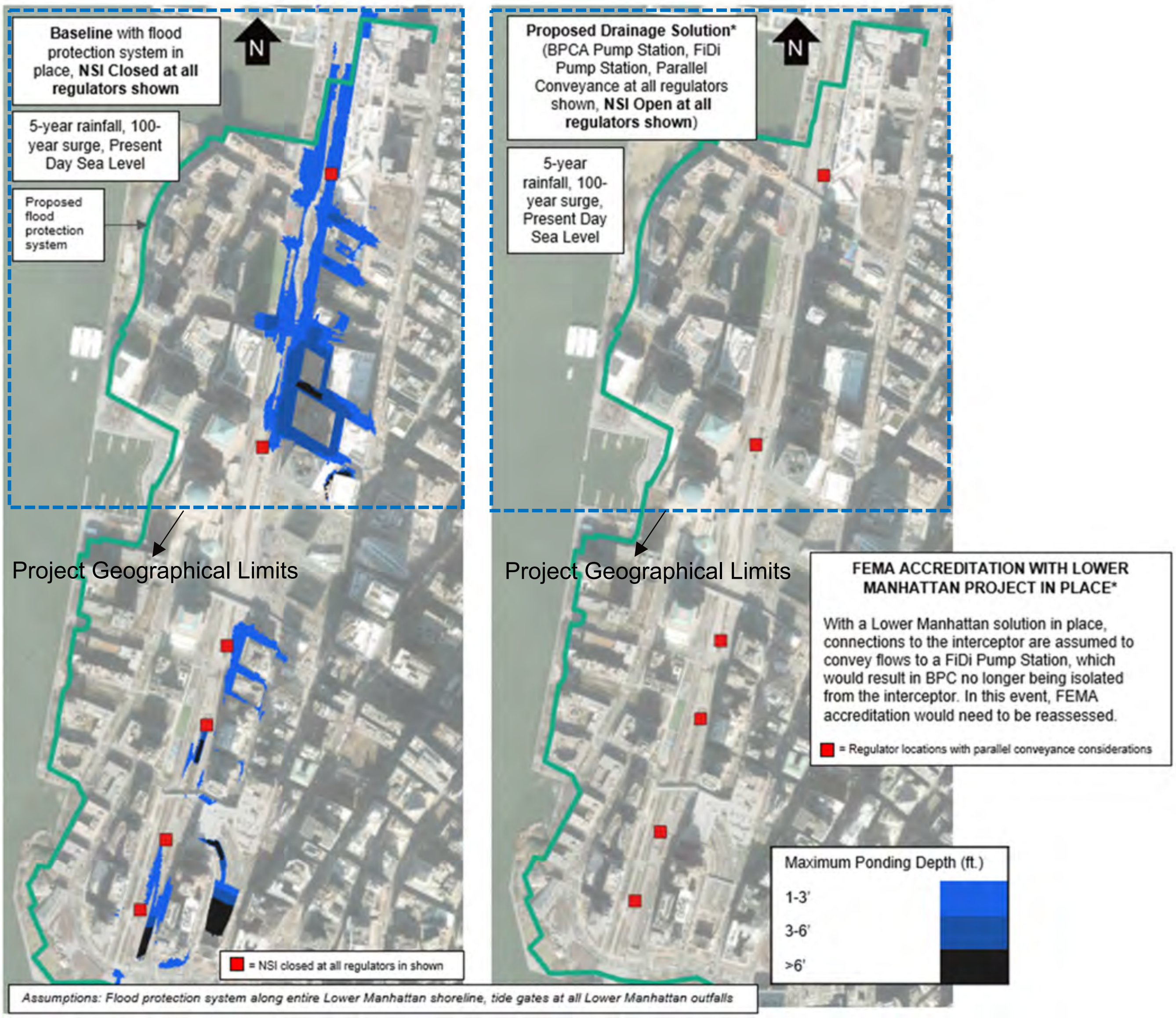


Figure 2-1. Baseline Flooding vs. Flooding Mitigation through Proposed BPCA and FiDi Pump Stations (Preliminary Modeling Results)

Prior to the PDB Team's contract in November 2022, BPCA, AECOM, and DEP explored different design scenarios and had indicated a preference toward accreditable temporary pumping systems. As part of the NWBPCR project, between November 2022 and May 2023, in consultation with DEP the PDB team updated existing drainage models for the area and evaluated numerous flooding scenarios, including the specified design storms. Based on the updated models, designs were developed, and evaluations conducted for both the FEMA accreditable temporary and permanent pumping options presented in detail in the sections below.

The Project Geographical Limit for the proposed pumping solution is limited to the northern portion of the NWBPCR project, defined as the region loosely bounded by Reach 4 and North Moore Street as shown in Figure 2-2. As discussed previously, this is the area outside of the zone anticipated to be mitigated by a FiDi Pump Station.

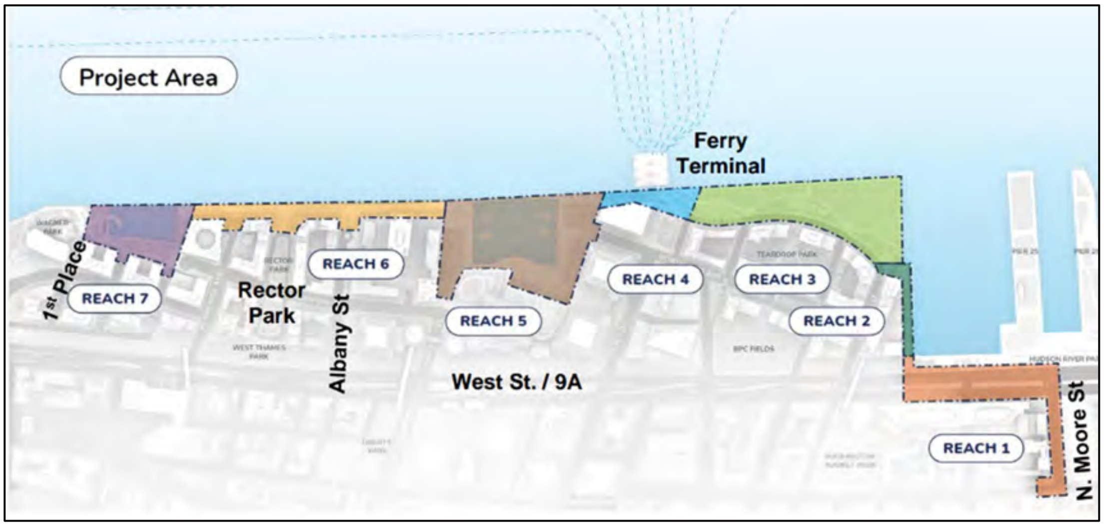


Figure 2-2. BPCA Project Area

2.1 Overview of Hydrologic & Hydraulic Analysis

A hydrologic and hydraulic (H&H) model was used to understand the magnitude and location of stormwater flooding that may occur when a flood-barrier system (FBS) is in place around the BPCA Project Area and develop pumping solutions to mitigate stormwater flooding under current and future conditions. Storm and surge conditions that result in the 1% joint annual exceedance probability (AEP) were considered for the H&H analysis following FEMA certification guidelines, as shown in Table 2-1. For this phase of work, all storm events in the protected area behind the flood barrier system were assumed to have a duration of 24-hours to represent a hurricane-type event, consistent with other ongoing New York City coastal resilience projects. In addition to the 1% storms, two design storms listed in the scope of work were also considered and are specified in Table 2-1 and Table 2-2.

Table 2-1 Approximate Joint Annual Probability of Rainfall and Surge Assuming Event Independence

Joint Annual Exceedance Probability (AEP) of Rainfall and Surge Assuming Event Independence								
		Rainfall Return Period (x-yr)						
		1	2	5	10	20	50	100
Storm Surge Return Period (x-yr)	1	100.0%	50.0%	20.0%	10.0%	5.0%	2.0%	1.0%
	2	50.0%	25.0%	10.0%	5.0%	2.5%	1.0%	0.5%
	5	20.0%	10.0%	4.0%	2.0%	1.0%	0.4%	0.2%
	10	10.0%	5.0%	2.0%	1.0%	0.5%	0.2%	0.1%
	20	5.0%	2.5%	1.0%	0.5%	0.3%	0.1%	0.1%
	50	2.0%	1.0%	0.4%	0.2%	0.1%	0.04%	0.02%
	100	1.0%	0.5%	0.2%	0.1%	0.1%	0.02%	0.01%
Sample 1% AEP Events for FEMA Base Flood Mapping Sensitivity Analysis								

Table 2-2 Interior Drainage Objectives and Criteria

Objective Component	Criteria
Primary Objective: Minimize Flooding >=1' Average Depth	Current risk: 100-year coastal storm, 5-year NOAA2Q 50th Percentile 24-hour rainfall, Present Day Sea Level. 0.2% AEP Future risk: 100-year coastal storm, 2-year NOAA2Q 50th Percentile 24-hour rainfall, 30" Sea Level Rise (SLR). 0.5% AEP

The combination of the 5-year rainfall and the 100-year surge (a 0.2% storm) was a more conservative event compared to the 1% FEMA storms considered and resulted in the most flooding greater than 1 foot within the project area. The 5-year rainfall and the 100-year surge were therefore used to size proposed pumping solutions to mitigate flooding greater than 1 foot within the project area.

Model simulations, as shown in **Figure 2-3**, indicate that for the northern section, the installation of a new pump station at NCM-73 with additional piping to consolidate flows contributing to NCM-72 and NCM-73 is required to alleviate the flooding under the 5-year rainfall and the 100-year surge (considered a 0.2% joint annual probability storm). The additional piping to consolidate flows into one pump station involves new stormwater and combined connection sewers that are required to connect the areas contributing to outfalls NCM-72 and NCM-73, as highlighted by the orange boxes in **Figure 2-3**. In addition to new sewers, some existing sewers will also need to be upsized to convey additional flows also shown in **Figure 2-3**. The proposed capacity of the new pump station based on modeling results would be approximately 60 MGD. This pump station flow rate is specific to the assumptions of the 30% design stage and will be continually refined during future phases of design. This flow rate was used to inform initial design stages and decision-making regarding a temporary vs. permanent pump station. With the proposed pump station in place, all flooding over 1 foot is mitigated in the northern portion of BPC, and in the area east of BPC and beyond Route 9A/West Street.

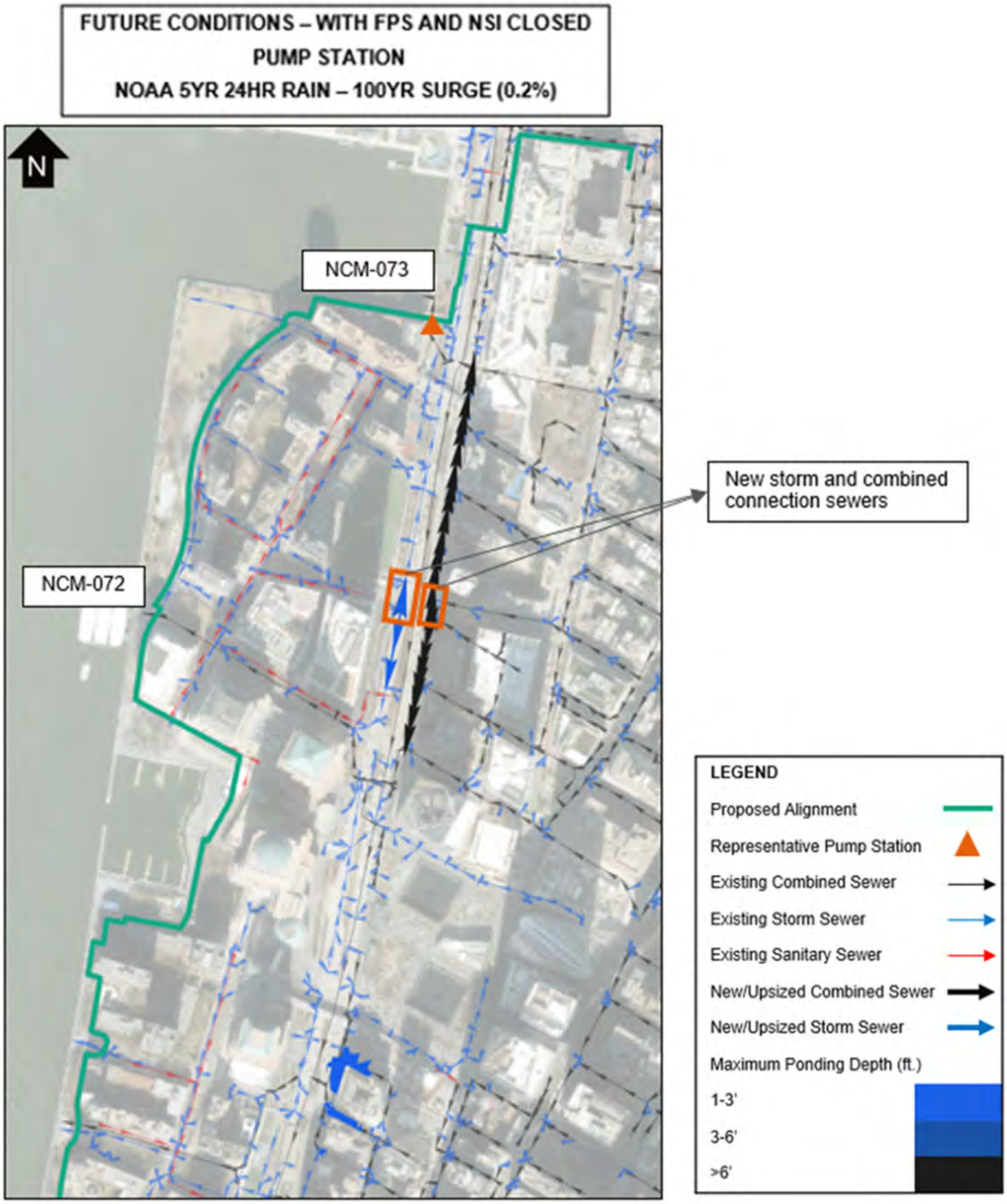


Figure 2-3. Proposed BPCA Pump Station and Sewers

2.2 Pump Station Siting

Various pump station locations serving the northern section areas that contribute to outfalls NCM-72 and NCM-73 were explored for the preliminary siting assessment. Three potential locations were evaluated as highlighted in **Figure 2-4**.

1. **Stuyvesant Plaza:** The first potential location is the plaza along Stuyvesant High School. While this area presents more public space available between the Plaza and Route 9A/West Street, it would also have a heavy impact on potential pedestrian usage. A high-level evaluation of an underground infrastructure was done at this stage using existing site surveys.
2. **BPCA Ball Fields:** The second potential location for a pump station would be at the southeast corner of the ball fields at Murray Street, where some infrastructure already exists. This could most probably require an adjustment of the existing interim resiliency project in place, and also will potentially impact pedestrian usage to minimize impacts to the Ball Fields. The influent line to the pump station at the Ball Fields is proposed to receive flows from an existing sanitary sewer line running along Route 9A/West Street. This existing line consists of a 156- ft long 18-inch line connected to a 700-feet long 24-inch line that eventually discharges into the existing 72-inch RCP CSO and ultimately into outfall NCM-073. Assuming surcharged flow at a 2% slope and 8.41ft/s, the 18-inch line will be able to handle approximately 9.6 MGD of flow and at an assumed 1.5% slope and 8.82 ft/s, the 24-inch line can handle approximately 18 MGD of flow. Therefore, both these lines will need to be upsized. The final diameter will depend on the estimated flow rate traveling back from the northern area of the sewershed. A sizable quantity of flow will be captured by the catch basins located at the surface area around the ball fields, where most of the ponding is expected to occur. However, the sizes of the new pipes could be as large as 42-inch diameter pipe at 0.90% slope to be able to handle the 60 MGD design flow. Further evaluation on the grading will need to be completed. Additionally, the area proposed for the location of the pump station is outside of the boundaries of BPCA property; therefore, negotiations with New York City will need to be arranged to determine the feasibility of this alternative. A high-level evaluation of underground infrastructure was done at this stage using existing site surveys.

Figure 2-4. Potential Pump Station Locations

3. **Public School IS 289 Schoolyard**: The third potential location is at the schoolyard associated with IS 289. The footprint required for installation of the pump station would encompass a significant portion of the play area and possibly extend beyond the play area and impact the pedestrian walkway. This option was not assessed in this stage and will be given a more in-depth evaluation during the preliminary design phase. The constraints of working in an area within the perimeter of a public school proposes significant potential permitting obstacles and it is beyond the limits of this report.

Options 1 and 2 were further evaluated, each for temporary and permanent pump stations.

3 Interior Drainage Pumping Solutions

3.1 Temporary Pumping Solutions

The purpose of a temporary pump station is to provide the flexibility to mobilize pumps and associated electrical equipment in advance of a flood event while reducing initial capital expenditures associated with a permanent pump station infrastructure, reduce on-site maintenance of the equipment, and reduce the overall footprint of the pump station. The pumping units and electrical equipment would be stored in a remote location outside of the Project Area.

A preliminary design with a focus on the mobilization and maintenance aspect was developed at this stage to determine the feasibility of this option. Mobilization of equipment and operating a temporary pump station is dependent on various factors such as the design flowrate, the location, location of equipment storage and the type of equipment needed.

Two major alternatives were identified associated with temporary solutions:

1. **Portable Diesel Pumps and Permanent Wet Well.**
2. **Removable Centrifugal Pumps and Permanent Wet Well.**

3.1.1 Portable Diesel Pumps and Permanent Wet Well

3.1.1.1 Pumping Machinery

Godwin Diesel Pumps were initially considered for evaluation as the pumps can be mobilized and operated without the need for installation. However, on further evaluation, it was deemed unsuitable for this application. The largest Godwin Pump can handle a flow of 5 MGD, which means at least eleven 15-ft x 8-ft (approximate dimension of The Godwin Dri-Prime CD300M pump) Godwin pumps would be required to be deployed during emergencies thereby resulting in the need of a large amount of at-grade space, a large storage area required outside the Project Area, and challenging logistics to mobilize the equipment in time.

Although several other manufacturers offer units with greater pumping capacity, the amount of space required is somewhat equivalent to the Godwin units. Additionally, since the Godwin pumps are at-grade centrifugal pumps, priming i.e, filling the volute of the pump with the liquid to be pumped, will be required prior to operating the pumps, to protect the pumps from running dry. Lack of priming can result in operating failures and high maintenance costs. Any challenge that occurs during priming can cause pumping delays which is a major concern during an emergency event where time is of critical importance.

This option will also require a permanent wet well from where to draw the CSO flow from. This infrastructure would need to be located within the pump station locations described above. Piping, electrical power, and associated miscellaneous components would still need to be provided at least on a temporary basis as needed.

3.1.2 Removable Centrifugal Pumps and Permanent Wet Well

3.1.2.1 Concept and Layout

The overall concept of this alternative is to provide sufficient permanent infrastructure to ensure reliability of the pump station during emergencies and to store the pumping units in a dedicated storage location to minimize site maintenance. Most of the pump station infrastructure would be constructed similar to a permanent functioning system including wet well, piping, screening units, and other miscellaneous components. The pumping units and the electrical powering system, however, would be stored at a remote location outside the project area, and will be brought into the site as needed and within the emergency protocol. This option demands careful coordination to deploy the necessary equipment at the time of an emergency.

For the initial evaluation, the preliminary sizing of the temporary pump station was assumed to be 40 ft x 60 ft. As shown in the preliminary layout in **Figure 3-1**, the 40 ft x 60 ft pump station includes a combination of permanent and temporary components. The permanent installation includes an in-line screening mechanism to manage solids in CSO flow, (Not shown in **Figure 3-1**, refer **Section 3.2.1.2** for screen details), subgrade wet well, valve pit, the piping connections at inlet and discharge, and a staging area for the electrical equipment and machinery trailer. The temporary components include the process-mechanical equipment such as the pumps and the electrical equipment. Access hatches will be provided at grade level to the pumping equipment and valve pit. The sizing of the piping connections at inlet and discharge is discussed in detail in **Section 3.2**.

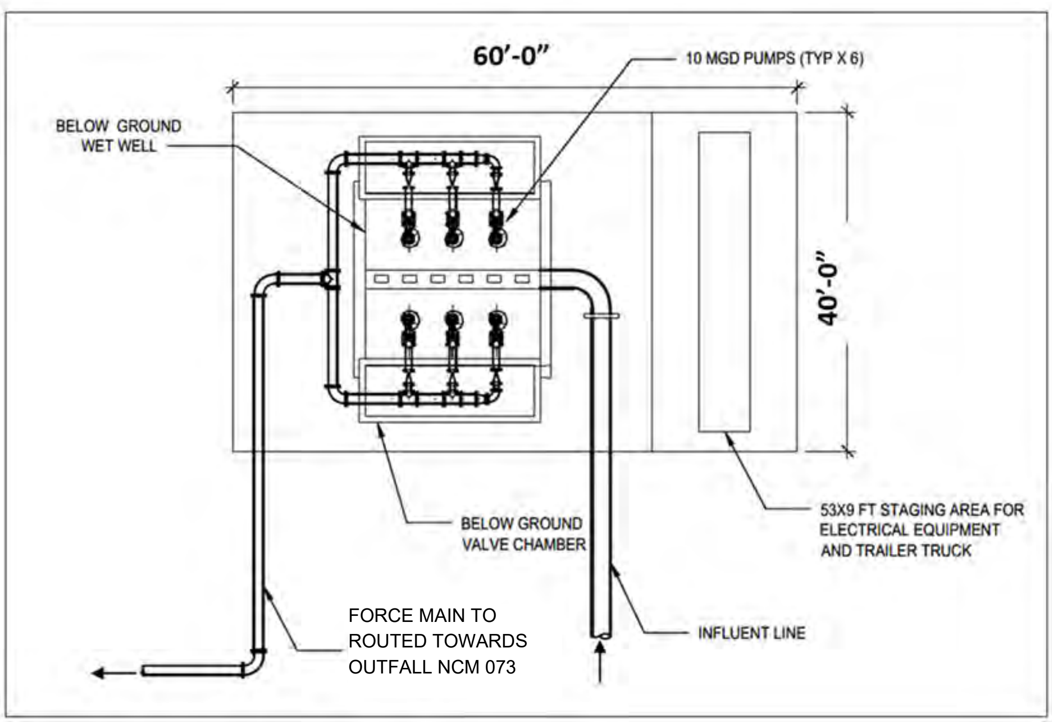


Figure 3-1. Temporary Pump Station – Preliminary layout

3.1.2.2 Pumping Machinery

The pumps most suitable for this application were found to be the submersible centrifugal pumps operating at constant speed. These types of pumps were favored due to their ability to handle a wide range of flows, the need for less at-grade space, the minimal space requirement below grade, and the ability to pass 2 to 3 inches diameter sized solids. Three sizing options were evaluated for the submersible centrifugal pumps as shown in Table 3-1. One of the advantages compared to diesel pumps of using submersible centrifugal pumps is that when needed, the pumps can be slid down guide rails and be ready for operation after electrical connections are made, thereby requiring less at grade space throughout the pumping operation.

Table 3-1. Submersible Centrifugal Pumps Sizing Options

Pump Sizing Option	Pump Type	Pump Size	Pump Quantity	Weight of each pump	Total Weight	Height of each pump	Width of each pump
1	Submersible	5 MGD/ 44HP	12	~3000 lbs/ 1.3 tons	~36,000 lbs	5-feet	6-feet
2	Submersible	10 MGD/ 90 HP	6	~4000 lbs/ 1.8 tons	~ 24,000 lbs	6.5-feet	6.5-feet
3	Submersible	20 MGD/ 150 HP	3	~10,000 lbs/4.5 tons	~30,000 lbs	8.75-feet	9-feet

Although Pump Sizing Option 3 offers a lesser number of pumps to install, the weight of each pump would require a more robust mobilization and installation setup. While in Option 1, the weight of each pump is much less compared to Option 2 and 3, the quantity of pumps required will likely require a greater number of flatbed trucks for mobilization. For these reasons, the six (6) 10 MGD submersible pumps were selected for the temporary pump station with the pumps operating under maximum pumping condition.

3.1.2.3 Electrical System for Temporary Pump Station

The temporary electrical system will consist of a trailer equipped with all the components needed to run the pump station including the motor control center and switchgear. A permanent power supply will need to be designed and installed on site to power the temporary electrical equipment for the six (6) 10 MGD/ 90 HP constant speed pumps. The pump motors will be 200V to be able to operate on ConEdison's standard 208/120 Volt supply and the motors will be started using electronic soft starters. Emergency backup power will be provided by portable generators. The electrical equipment along with the portable generator will need to be mobilized on a trailer at the same time as the pumping machinery.

3.1.2.4 Mobilization And Operations & Maintenance (O&M)

3.1.2.4.1 Mobilization

In advance of a flood event, the six (6) 10 MGD pumps along with electrical equipment will need to be mobilized from the dedicated storage location outside the Project Area and transported to the pump station location on flatbed trucks. As part of the O&M protocol, a written procedure will be included for assembling and mobilizing equipment at least two days in advance of a flood event or as soon as an emergency event is known to occur. A 22-foot flatbed truck with crane, as shown in **Figure 3-2**, is typically used for mobilization of the pumps to the project site. Given the width of each pump and the capacity of the flatbed truck, approximately three (3) flatbed trucks would be required for the six 10 MGD pumps, with an additional trailer for the electrical equipment. Floor crane installation such as a jib crane was also considered, but this would require an at-grade permanent installation which would not be feasible, given the temporary pump locations open to the public when not in use. Given the scale of the equipment being handled, the location will need to be secured from public access before mobilization begins. At the pump station, the cranes would need to lift coverings off wet wells, lift pumps into place, and assemble piping and electrical connections. At the end of the flood event, all the connections will be disconnected, and the pumping units transported back to the storage location on or near BPCA property. To be accredited by FEMA, when in storage, the pumps will need to be dry-tested, which would involve rotation of the impellers, presumably on a monthly basis as a minimum.

Figure 3-2. Example Of Flatbed Truck For Mobilization

O&M of a temporary pump station of such large scale will require considerable amount of coordination, planning and a dedicated crew.

Additional factors that were considered but not evaluated in more detail include,

1. The location of a remote storage facility outside of the Project Area.
2. Mobilization time from the storage facility to the temporary pumping site.
3. Logistics of navigating multiple trailer trucks in advance of and after a storm event. Some of the factors include:
    a. Turning radius of the truck to maneuver into the staging location.
    b. Navigating the trucks in and out of the staging locations.
    c. Acquiring a large property size to account for staging area for the multiple trucks
    d. Traffic control plans.
4. Health and safety concerns with deployment of large equipment in locations so close to public access.

3.1.2.4.2 O&M Requirements

The O&M requirements for the temporary pump station will be a combination of routine procedures followed at traditional pump stations with additional requirements due to it being a temporary facility. O&M requirements specific to a temporary pump station will include:

1. Following manufacturer recommended maintenance procedures for the equipment when in storage which will require manual intervention that cannot be performed remotely.
2. Regular Training for Emergency Crews (at least annually).
3. Periodic “live” deployment test (at least every two years).

3.1.3 Pumping Station Siting

The following factors were taken into consideration when siting the temporary pump station:

1. Mobilization logistics such as the size of the staging area to accommodate the trailer trucks, vehicular accessibility and maneuvering into the staging location.
2. Type of crane for installation –
   a. Mobile crane included with the flatbed truck.
   b. Crane reach and load tables.
3. Health and safety concerns

3.1.3.1 Location 1 – Stuyvesant High School

As shown in **Figure 3-3**, the plaza adjacent to Stuyvesant High School has high pedestrian usage and is also in close proximity to the Hudson River Greenway Bike Path. This location makes vehicular staging very challenging, especially for the temporary pump station that relies heavily on equipment mobilization for efficient handling of flood impact. In addition, the deployment and handling of large pumping equipment and electrical equipment, during a storm event, so close in proximity to the public and students poses considerable health and safety concerns. Although this location has the advantage of a shorter length of force main pipe discharging into the outfall CSO 73, for the reasons discussed above, this location is not recommended to be considered for the temporary pump station.

Figure 3-3. Temporary Pump Station – Proposed Location 1

3.1.3.2 Location 2 – BPCA Ball Fields

The BPCA Ball Fields are located along Route 9A/West Street between Warren Street and Murray Street. The flooding at Murray Street poses the greatest impacts on BPCA property. Therefore, a pump station in this location would yield the most benefits to mitigate flooding concerns within the area. As shown in **Figure 3-4**, the potential location for the pump station would be at the corner of the ball fields where other infrastructure exists. This infrastructure, assumed to consist of electrical equipment, may need to be modified per future proposed design. Being in the corner lot, however, provides more flexibility in terms of vehicular access when compared to Location 1 discussed above. This location is currently fenced off from the main ball field, making it easier to secure it off from public access during the mobilization of equipment in advance of a flood event. It is estimated, however, that additional property adjacent to the existing site would be needed for the complete installation of the various necessary components.

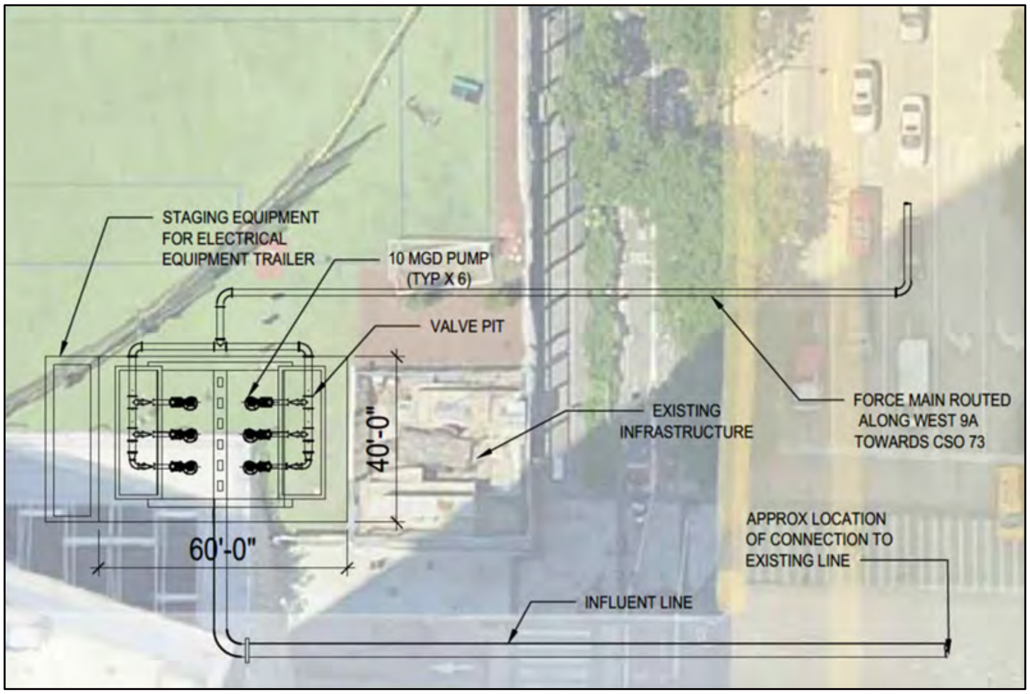


Figure 3-4. Temporary Pump Station – Preliminary Layout

The influent line to the pump station would be connected to an existing 18-inch sanitary sewer line along Route 9A/West Street, ultimately discharging into the existing 114-in X 90-in RCP Box culvert during an overflow event. Provision will be made for a gate or other suitable flow- regulation device, to allow overflow of CSO flows into the existing RCP box culvert. The hydraulic capacity of this existing sanitary pipe will need to be further evaluated to determine if it holds enough cross-sectional area and slope to convey the 60 MGD of flow anticipated. The force main discharge line runs approximately 1,000 linear feet from the pump station in the direction of the CSO 73 outfall.

3.1.4 Summary

Although a temporary pump station has the advantages of reduced initial capital investment, on-site maintenance and a reduced footprint, O&M would require a dedicated crew and equipment for mobilization, outside facility storage, and a significant amount of planning and coordination to ensure reliable operation during storm events. Temporary solutions will also require significant training efforts to educate crews for day "X" and ensure everybody is familiar with each step and their role during mobilization, deployment, and operation during the storm event. These challenges become exacerbated if the pump station flowrates increase in future phases of design, as higher flowrates require more or larger equipment.

3.2 Permanent Pumping Solutions

As discussed above, the deployment of machinery before and after a flood event requires a dedicated crew and equipment, and significant amount of planning to make sure the operation runs smoothly, and deployment happens on time after a storm event is known. For this reason, a permanent pump station approach was evaluated as an alternative as it offers a more reliable approach and is relatively simple to operate. The permanent pumping solution eliminates the need to mobilize machinery required to run the pump station during a flood event, eliminating the risk associated with the mobilization operation and response time upon identification of a storm event. All the pump station components would be permanently installed with operator intervention needed only for operation and maintenance.

3.2.1 Pump Station Design

The permanent pump station is configured as a wet-well-type station with six (6) 10 MGD submersible pumps and one (1) standby unit. Wet well sizing will be based on Hydraulic Institute Standards and will be equipped with a valve pit for easy access to the valves. A new CSO connecting vault will serve as a chamber to connect the new influent line into the pump station to an existing conduit, to contribute flows into the pump station. To manage the larger solids present in the CSO flows, an in-line screening mechanism will be considered for the CSO connecting vault (further analysis will dictate if the best location for the screening equipment is at the pump station, upstream of the wet well, or as separate infrastructure as assumed in this report). The type of screening mechanism proposed is being evaluated and will be studied in detail during the design phase. The seven (7) 18-inch-diameter discharge connections from each pump will discharge into a 48-inch-diameter ductile iron force main, which will be connected to outfall NCM-073.

Access hatches will be installed at grade level for maintenance and repair of each pump and valve and at the CSO connecting vault to remove and/or repair the screening mechanism. An electrical building to house the electrical equipment will be located adjacent to the pump station and is further discussed in **Section 3.2.1.3**. The exact location of the electrical building will be further evaluated during the design phase.

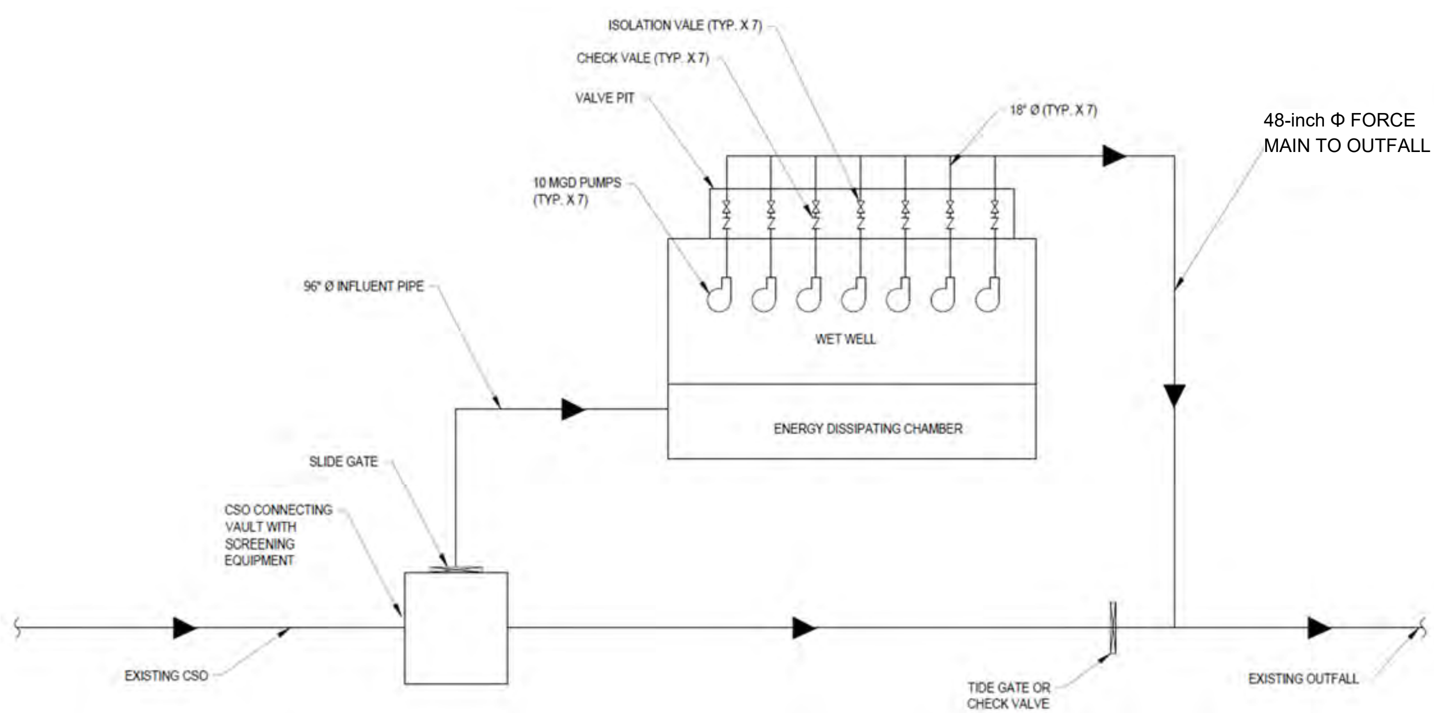


Figure 3-5. Permanent Pump Station – Schematic layout

3.2.1.1 Pumping Machinery

The same submersible centrifugal pumps evaluated for the temporary pump station will also be used for the permanent pump station as the design flow remains unchanged as of now. The mid-size 10 MGD pump will be used.

Table 3-2 Permanent Pump Station - Pump Machinery

Parameter	Value
Pump Type	Submersible centrifugal
Pump Size	10 MGD/ 90 HP
Number of pumps	6 (lead/lag for maximum pumping condition) + 1 standby
Weight	~4000 lbs each
Pump Dimensions	6.5 x 6.5 ft

3.2.1.2 Screening Equipment

The use of an inlet bar screen, grid, or some type of trash rack is recommended to protect the pumping units from the solids and other debris in the incoming CSO flow. The manufacturers suggest a minimum 2 to 3-inch bar spacing for this application. In this initial stage, an evaluation was performed on the use of inline screening to be mounted at the CSO Connecting Vault, upstream of the wet well. Other alternatives are expected to be evaluated, including bar racks and mechanical bar screens located inside of the pumping station during the design phase.

3.2.1.2.1 In-line Screening

The in-line screening concept is designed specifically for the removal of solids in the CSO system during wet weather events and it would be installed at the CSO Connection Vault. Solids and floatables that are captured by the screens, are kept in the CSO conduit, and move laterally along the surface of the screen towards the end of the equipment where they are directed back into the CSO conduit. The material then travels in the direction of flow towards the CSO outfall 73. These types of systems are usually equipped with a hydraulically driven raking assembly that prevents blinding.

(i) ALTERNATIVE 1 – Vertical Static Bar Screen

The inline vertical static bar screen alternative is a manual bar screen with bar space between 2 to 3 inches set at 20 degrees from the vertical, 24 ft long and 8.5 ft tall as shown in **Figure 3-6**. The screen is intended to capture larger material that would otherwise damage the pumping units and keep the solids in the CSO conduit.

The screen will need to be cleaned thoroughly after every event by manually raking the material down from the screen into the CSO conduit. Operators will access the underground CSO Connection Vault through an access hatch leading to a ladder and a platform located above the screen. If enough material accumulates at the screen during a storm event, blinding the passage of water, the water level will rise above the screen, to an overflow weir that will direct the unscreened flow into the wet well.

The screen will be sectioned in smaller pieces to ease the installation and removal from the vault. Each section shall be removed separately through a hatch of at least 5’ x 5’ that will be located at the surface elevation of the vault.

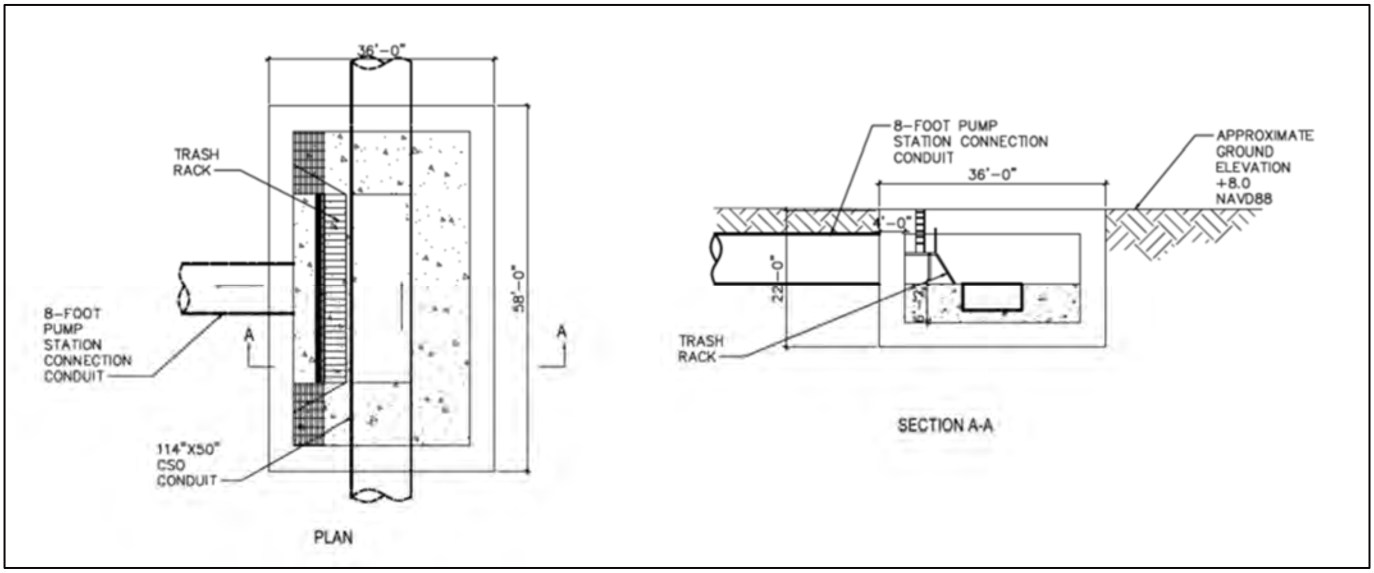


Figure 3-6. Vertical Static Bar Screen

(ii) ALTERNATIVE 2 – Horizontal Hydraulically Raked Screen

The horizontal hydraulically raked screen alternative is a proprietary screen product from PWTech. The screen will be set horizontally above an opening at the CSO conduit’s crown in the CSO Connection Vault. An automatic hydraulically driven raking mechanism will be set to operate during storm events removing material from the screen on a timing cycle as shown in **Figure 3-7**. If the automatic raking mechanism fails to operate, and the screen blinds during a wet weather event, the water level will rise within the vault to an overflow weir that will direct the unscreened flow to the wet well. In normal operation, raked material will be removed from the screen and kept in the CSO conduit where it will follow the direction of the flow and to outfall 73. The hydraulic system, including hose assembly, hydraulic power unit, hydraulic pumps, and all the electrical components, will be located in the Electrical Building.

Periodic visits to visually inspect the equipment shall be considered as part of the regular maintenance schedule, and once every quarter is recommended. Additional visits will be performed after each wet weather event. The raking mechanism shall be set to operate regularly, in dry conditions, to ensure the equipment will function properly when needed.

The screen consists of two (2) sections of 50 inches by 48 inches to handle the total flow of 60 MGD. Each section can be installed and removed separately and therefore a hatch of 5’ x 5’ will be needed at the vault.

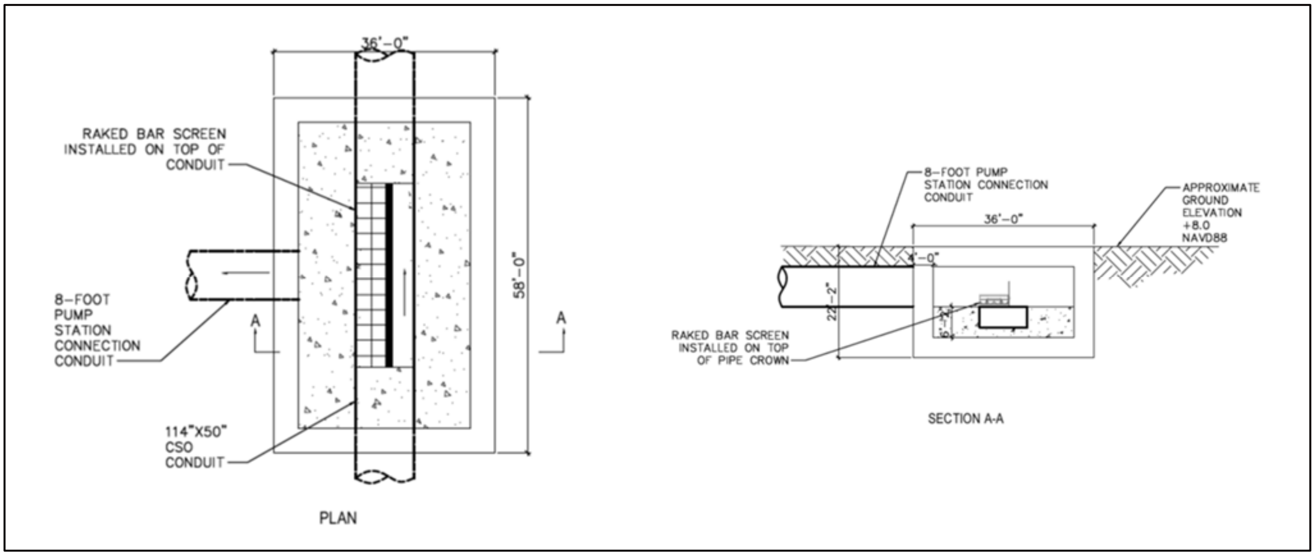


Figure 3-7. Horizontal Hydraulically Raked Screen

3.2.1.2.2 Screening Removal

If an inline screen system is selected, the maintenance of the equipment will not require collection and disposal of material. All the debris will be placed back into the CSO conduit to follow the direction of the flow into outfall 73.

Grit, although not considered in this evaluation, will need to be removed from the wet well of the pump station as part of a scheduled maintenance procedure.

3.2.1.2.3 In-line Screening and Bypass Removal

With the installation of inline screen equipment at the CSO Connection Vault, the removal and disposal of screenings is not required. The inline screen will operate by blocking the solids and floatables to enter the wet well, keeping them in the CSO conduit. The material will be carried on into the CSO outfall.

3.2.1.3 Electrical and Instrumentation Components

The electrical equipment required to provide the power supply to the seven (7) 10 MGD / 90 HP constant speed pumps, is proposed to be housed in an elevated electrical building located adjacent to the pump station as shown in **Figure 3-8**. The building is proposed to be elevated with an exterior façade that matches with the surrounding infrastructure, as this is a location with high pedestrian usage and installing an electrical building at grade level will create a surface expression that impacts urban design. However, the feasibility of this location and additional considerations such as access, visual impacts, elevated vs at-grade etc. will be further evaluated.

The pump motors will be 200V for operation on ConEdison's standard 208/120 Volt supply. The motors will be started using electronic soft starters. Emergency backup power will be provided by diesel- powered portable generators. The power requirements for the screening equipment will be incorporated in the evaluation once the screening mechanism is finalized.

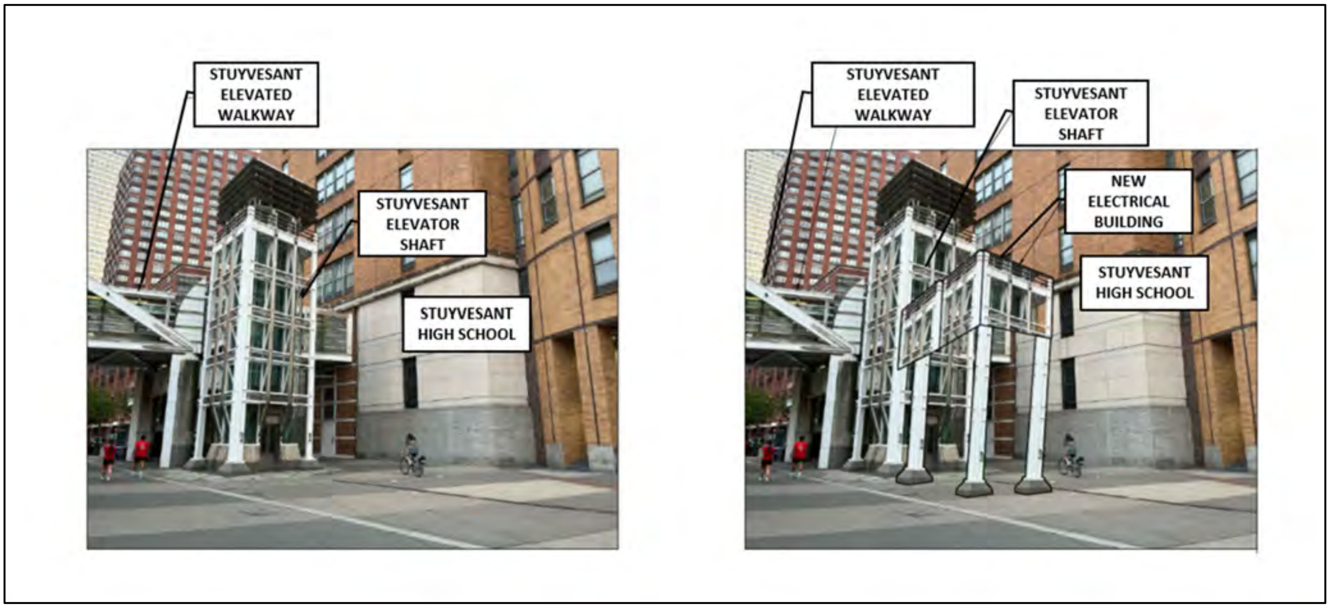


Figure 3-8. (Left) Existing Infrastructure (Right) Proposed Infrastructure for Electrical Building

3.2.1.3.1 Variable Frequency Drive vs Constant Speed

The primary goal of a variable frequency drive (VFDs) is to keep the discharge rate of the pump station as close to constant as possible all the time. The station then operates as if it was discharging by gravity. By doing this the storage volume needed in the wet well is minimized and the pump cycling reduced. This type of approach is practical for influent pump stations at wastewater treatment plants because the incoming flow show minimal fluctuations and lessens treatment upsets that could be caused by sudden surges in the influent flow.

The nature of the BPCA Pump Station, however, is to mitigate flooding conditions that occur only during storm surges and wet weather events. As a result, the pumps will only be active on a seasonal basis and the discharge rate does not play a role in the overall efficiency of the system. The wet well size will be sufficiently large to ensure that the pumps cycles do not exceed the maximum starts per hour recommended by the manufacturer.

Additionally, VFDs add significant cost, are less efficient than constant speed pumping systems, and add substantial complexity to the system. VFDs also requires more equipment, air-conditioned electrical rooms, and more maintenance. Personnel will need to be trained to operate this type of system and be ready to adjust settings as needed. Since the BPCA Pump Station is planned to be an unmanned facility, these conditions will be challenging to meet.

A system set up with constant speed pumps, on the other hand, can be set to operate with a wide range of flows, is more efficient than VFDs and are more reliable and less expensive than VFDs. The pumps shall be equipped with soft starters that are used to protect the motor from the stress produced by a large amount of electrical surge when the motor is started or stopped.

3.2.1.3.2 Electrical Spacing

A preliminary evaluation was conducted specifically to estimate the electrical room size needed to support seven (7) 90 HP 10 MGD (six (6) centrifugal pumps plus one (1) standby unit) constant speed pumping units plus the auxiliary loads at the BPCA Pump Station. One (1) Con Edison supply feeder was also included in this preliminary evaluation. The preliminary size of the electrical room is expected to be 50’ x 18’.

The electrical building size includes one platform, located outdoors outside of the building. A canopy above the platform may be desirable. A docking station for connection of the portable generator would be located on the platform where additional space has been allocated.

3.2.1.3.3 Redundancy

Since this pumping station will be designed to operate in emergency conditions only (as described above), the redundancy of critical equipment has been considered accordingly. Although DEP Design Guideline – Main Sewage Pump (MSP) Systems recommends including N+1+1 pump system sizing based on the entire required operating flow range, the PDB Team recommends including only one redundant pumping unit, pending further discussion with NYCDEP. Since the design criteria is focused on a 0.2% joint annual probability storm (i.e. a 500-year event), a second redundant unit will be an unjustifiable capital expense that may never prove necessary. Additionally, since the pumps will need to be exercised on a regular basis, having an extra unit in the station will demand unnecessary maintenance work.

Additionally, a more in-depth discussion will be held during the design phase to decide on the need of a permanent generator. At this stage it was assumed that use of a portable generator would be brought to the site as needed. As noted above, provisions are being made to ensure this first draft includes a docking station for connecting to the portable generator.

3.2.2 Pump Station Siting

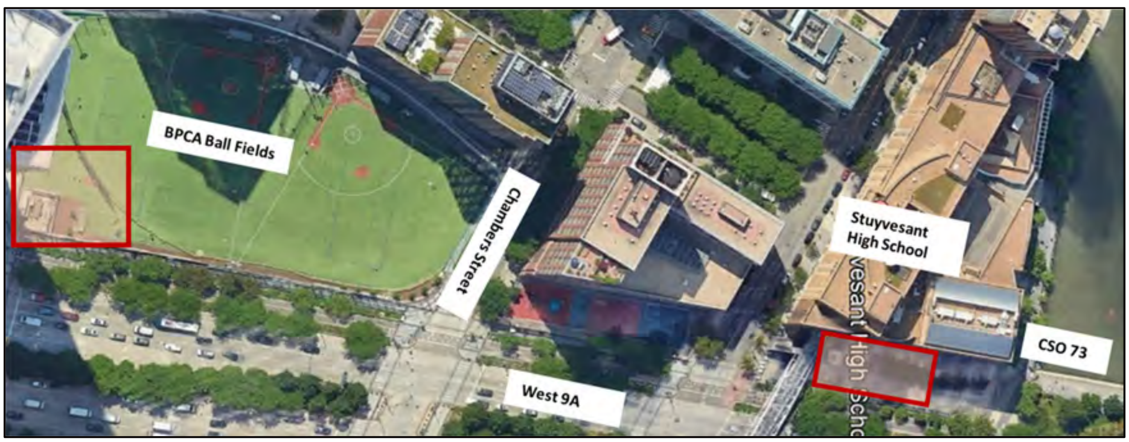


Figure 3-9. Permanent Pumping Station – Proposed Locations

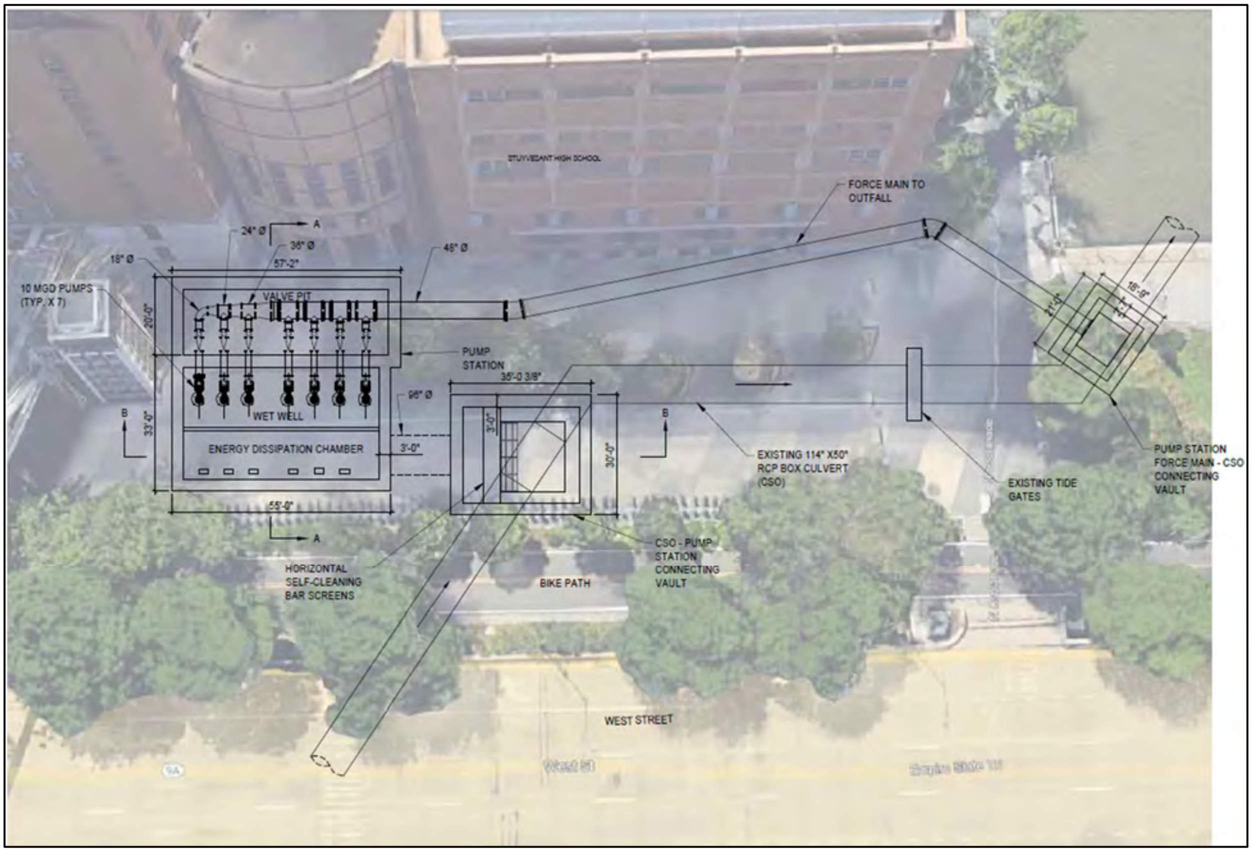
The following factors were taken into consideration when evaluating these locations:

1. Increase in the footprint of the pump station to include a permanent electrical building.
2. Vehicle access for maintenance and repair purpose.

3.2.2.1 Location 1 – Stuyvesant High School

The **Figure 3-10** below provides a preliminary layout of the permanent pump station location near the Stuyvesant High School. The pump station is located close to outfall CSO 73, upstream of the proposed flood wall barrier and the existing tide gates. The figure shows the CSO Connecting Vault structure proposed to connect to the new 96-inch influent line to the existing 114 in x 90-in RCP box culvert. The vault will be equipped with the inline screening mechanism described in **Section 3.2.1.2**. The 48-inch diameter force main was sized to maintain a velocity of 6 to 8 fps. The electrical building is proposed to be located on an elevated structure adjacent to the pump station. Wet well sizing, based on Hydraulic Institute Standards resulted in a 35-ft x 55-ft wet well and 57-ft by 20-ft valve pit. To manage the solids present in the CSO flows, an in-line screening mechanism will be installed in the CSO Connecting Vault as shown in the figure. The type of screening mechanism proposed is being evaluated and will be studied in detail during the design phase. The seven (7) 18-inch diameter discharge connection from each pump will discharge into a 48-in diameter ductile iron force main which will be connected to the existing 114-in x 50-in RCP box culvert downstream of the tide gates at the existing CSO 73 outfall. As shown in **Figure 3-10**. Access hatches will be installed at grade level to provide access for maintenance and repair for each pump and valve, and at the CSO Connecting Vault to remove and/or repair the screening mechanism.

With this option, a new connection will be made between the new force main and the existing 114-inch x 50-inch line. The existing line will need to be evaluated to ensure it can handle the new pressure from the pumped flow.



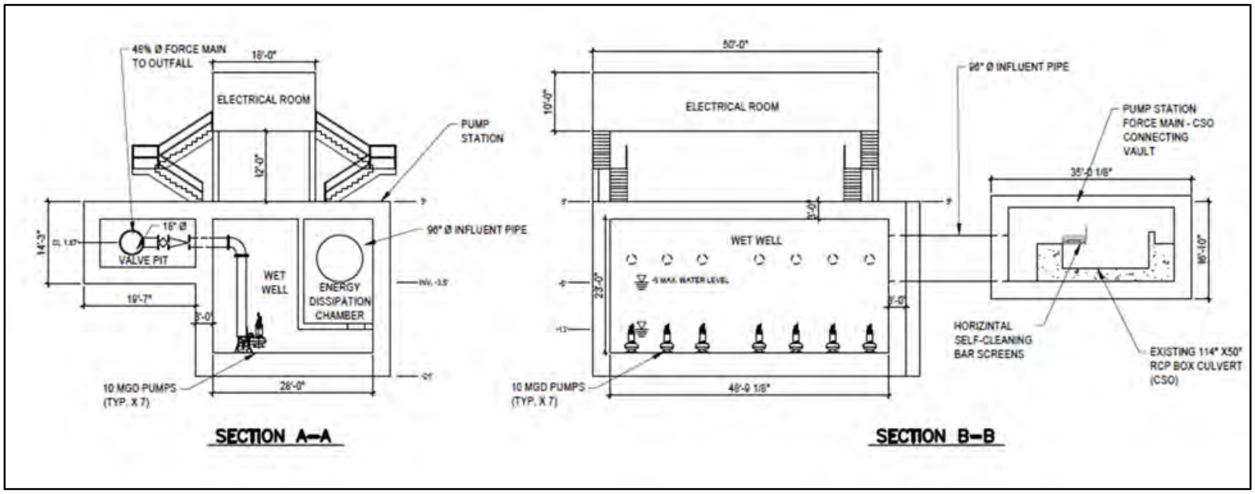


Figure 3-10. Permanent Pump Station – Location 1

The advantage of this location is the proximity to the final discharge location which allows for shorter length of force main piping to be installed. The potential challenges to this location would be the location of the electrical building and vehicular access for maintenance and repair purposes.

3.2.2.2 Location 2 – BPCA Ball Fields

Figure 3-11 below, provides a preliminary layout of the permanent pump station location at the BPCA Ball Fields. The permanent pump location would occupy the same location discussed in 3.1.3.2.

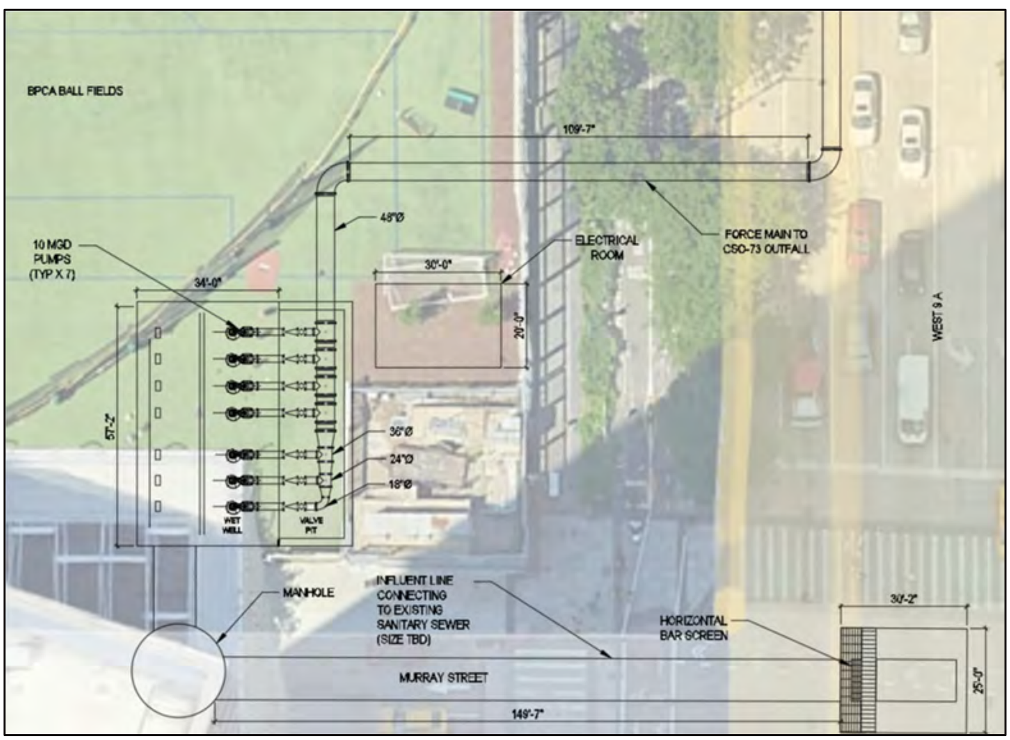


Figure 3-11. Permanent Pump Station – Location 2

Based on existing survey plans, the influent flow into the pump station will be received from an existing 18-inch sanitary sewer line located on Route 9A/West Street. Further evaluation will be necessary to determine the feasibility of handling large flows through this approach. If this location is pursued, hydraulic modelling will be conducted to determine the flow routing and any potential reconfigurations of the existing drainage network required.

The components of this pump station are similar to the one designed for Location 1. However, the force main from the pump station will need to be routed along Route 9A/West Street to be connected to the NCM-73 outfall for final discharge. The approximate length of the force main to the outfall is 1000-ft. Location 2 can immediately address the flooding impacts on Murray Street and is relatively accessible for maintenance vehicles. Nevertheless, the challenge with this location is with respect to the installation of the 1000-ft force main along Route 9A/West Street, that substantially increases initial infrastructure costs and adds permitting and schedule challenges. Construction will also cause significantly more disruption.

3.2.3 Operations and Maintenance

The operation and maintenance requirements for the permanent pump station include routine inspection and will require additional inspections after each event to clean the screens and visually inspect the pumping units at the wet well. Additionally, the pumps and screens will need to be exercised periodically. The following depicts the anticipated maintenance requirements for the pump station:

1. Following manufacturer recommended routine maintenance procedures for the equipment:
    a. Procedures to maintain equipment when not in use such as exercising the pumps which can be done remotely. The pumps shall be exercised as indicated by the manufacturer. The minimum recommended schedule is once a month.
2. Clean-up of the screening mechanism after each event. If the screens, are exposed to the CSO flow on a more frequent basis, the screens should be inspected and cleaned in anticipation of an emergency event.
3. Addressing repairs:
    a. In the event that a pump needs to be removed or replaced for repairs, a flatbed trailer with a crane can be employed similar to the type discussed in **Section 3.1.2.4**. Floor cranes like gantry cranes and jib cranes were also evaluated. However, a permanent installation of these type would not be feasible as both locations are open to public access.

4 FEMA Certification and Accreditation Concerns

4.1.1 Certification Concerns

A temporary pumping solution can be more challenging to certify through FEMA. Since the pumping equipment is not in place permanently, a rigorous operations and maintenance plan will be required potentially including assurances that the equipment could be mobilized in time to prevent flooding from storm events. During non-storm events, the pumps will also need to be exercised periodically, which will be much more challenging to accomplish while the pumps are in storage without access to flow. A comprehensive standard of procedure (SOP) will be studied and recommended for the pumps' periodic exercise program. This program is anticipated to be needed monthly and may include the demand for manual operation of slide gates and other miscellaneous items needed to flood wet well. The pumps may also be operated by local controls. All pumps will need to be operated during the procedure to ensure that all pumps are run for an equivalent number of hours per month. Operations personnel will be needed at the pump station to ensure the procedure is properly followed. The outflow resulting from this exercise will also discharge into the outfall NCM -073.

It could also be more difficult to provide necessary redundancy components to achieve FEMA certification for a temporary pumping solution. Backup power supply, in particular, could be exceptionally challenging for equipment that is mobilized onto the site temporarily only during a storm event.. These issues make the pump station more difficult to achieve FEMA certification for a temporary solution.

4.1.2 Schedule Considerations

4.1.2.1 Pump Station Design Schedule

The PDB Team has prepared an initial schedule from preliminary design to 100% design, to complete the design task of a pump station. Since the BPCA Pump Station will be operated by NYCDEP Personnel, the design process shall follow the typical design stages required by the agency. The following schedule summary depicts the steps followed in the design of a permanent pumping solution, however, the Removable Centrifugal Pumps on a Permanent Wet Well, would follow a similar timeframe. This typical DEP process requires a Facility Planning Phase that would advance into detailed design.

The Facility Planning Phase would involve topographic survey, geotechnical investigation, utility survey, and other site investigation activities such as a Preliminary Design Phase Power Distribution Study which focuses on the current utility power sources in the area, the options to include communication with the electrical utility, and on developing an electrical sourcing plan for the Project. Several workshops will be scheduled at the end of Facility Planning to discuss our findings with the DEP. Furthermore, the following list of technical reports are anticipated to be included to comply with DEP's technical design guidelines:

- Technical Report – 1: Hydraulic Conditions
- Technical Report – 2: Location of BPCA CSO Pump Station, Alternative Analysis
- Technical Report – 3: Pump Station Configuration. Selection of Equipment, Removal of Equipment Analysis, and Maintenance Access
- Technical Report – 4: Parallel Conveyance Layout, Connecting Chamber Configuration
- Technical Report – 5: Pump Station Control Strategy and System Layout
- Technical Report – 6: Configuration of Electrical Space for Pump Station Drive and Control System
- Technical Report – 7: Constructability, Sequencing, Schedule, and Cost

In addition, a rigorous siting analysis will need to be prepared to comply with DEP's typical alternative screening evaluation. Since there will be significant below and above ground infrastructure, the final location will be assessed based on its ultimate purpose, mitigate flooding, and on DEP's maintenance personnel and equipment access. The site selection process will require the PDB Team to evaluate several locations to provide a detailed comparison and assess each location's advantages and disadvantages.

A Basis of Design Report would be required including an energy profile, a sustainability assessment, and material and waste analysis, involving the evaluation of potential locations and design variations. Additional permitting, such as ULURP, PDC, and support for ongoing CEQR or FEMA review would also be needed.

The following summarized list of activities and their respective duration depicts each step from beginning to end of design:

Table 4-1 Preliminary Schedule

Tasks		Duration
Preliminary Evaluation Phase	Initial Pump Station Evaluation	**60 days (completed)**
Facility Planning Phase		**303 days**
	Existing Condition Assessment	120 days
	Facility Plan Report and Workshops	150 days
	Basis of Design Report and Workshops	33 days
Design Phase		**430 days**
	Technical Investigations and Report	60 days
	30% Design	120 days
	60% Design	160 days
	100% Design	90 days

As shown in Table 4-1, the design of the BPCA Pump Station will take approximately 733 consecutive days (24 months) to complete.

4.1.2.2 Schedule Impact on FEMA Accreditation

The PDB Team's current schedule indicates that the Conditional Letter of Map Revision (CLOMR) is scheduled to be sent to FEMA for review along with the North/West Battery Park City Resiliency Project's 60% design documents, in September 2023. The submission of a CLOMR is the first step in certifying that property is protected from a 100-year event. It is expected that the FEMA review process for CLOMR will take possibly up to one year, if revisions are necessary. Even though building permits cannot be issued based on a CLOMR, because a CLOMR does not change the NFIP map, this step is recommended prior to receiving a Letter of Map Revision (LOMR), which reflects an official change to an effective Flood Insurance Rate Map. These letters are issued by FEMA to revise or amend an existing flood map and can remove a property or reflect a change in flooding condition.

Regardless of the pump station solution, the schedule will be dictated by aligning with NYCDEP's typical design processes. The current draft schedules indicate a gap of approximately 16 months between the North/West Battery Park City Resiliency Project's 60% design documents and the NWBPCR Pump Station 60% documents (assuming that the design of the pump station starts in July of this year). Although there may be options to compress the pump station schedule, the role of NYCDEP in reviewing the documents is not a variable that the PDB Team can control.

We recommend a discussion with BPCA to explore how a pumping solution can be effectively integrated into the CLOMR process and what information would be needed for FEMA accreditation.

5 Summary and Recommendations

Permanent vs. Temporary

Two alternatives to interior drainage pumping solutions were evaluated to provide relief to the flooding impacts occurring within the BPC project area, under the ongoing resiliency program led by BPCA. Flooding greater than 1-foot above ground level will be pumped outside the project area.

The two alternatives – temporary pumping solutions and permanent pumping solutions were analyzed to determine feasibility of implementation based on siting impacts, operations and maintenance impacts, and overall cost impacts. As described in **Section 2.1**, the proposed capacity of the pump station based on modeling results is approximately 60 MGD. Based on the evaluation conducted, the PDB Team recommends implementation of a permanent pumping solution as a more reliable and efficient approach.

Location assessment

To assess the siting impacts, three locations were identified as outlined in **Section 2.2** out of which two locations were further evaluated. The Stuyvesant Plaza was identified as the preferred location for the permanent pump station mainly due to its proximity to the final discharge location at the outfall. This location was used to model the draft layout. The BPCA Ball Fields location was used to model both temporary and permanent solutions mainly due to the availability of space within the Ball Fields, however it is not the PDB Team's recommended alternative for a permanent pump station due to heavy disruption of the public space, difficult access during temporary construction activities and for recurring maintenance and the need to install in the streets approximately 1000 linear foot of force main discharge to the outfall.

Temporary Pumping Options

Two types of temporary pumping solutions were evaluated, Portable Diesel Pumps on a Permanent Wet Well, and Removable Centrifugal Pumps on a Permanent Wet Well. Although portable diesel pumps were initially favored over other alternatives due to operation simplicity and few permanent structures needed, this option was quickly dismissed due to the magnitude of the flow rate. To manage the anticipated 60 MGD obtained from modeling, approximately eleven (11) 15 feet long by 8 feet wide units would be needed. Mobilization and space requirements for this number of pumps make this option unfeasible and thus ruled out from further examination.

The Removable Centrifugal Pumps on a Permanent Wet Well alternative consists of providing basic permanent infrastructure to support the pumping solution while the pumping units are transported to the site in advance of a flood event. The assessment, however, showed that this alternative has a number of flaws that put the operation at a high risk. The proposed submersible centrifugal pumps are temporarily stored at a remote dedicated area and brought in and installed at the permanent wet well during emergency events. This operation demands considerable amount of planning and coordination in addition to the need for equipment maintenance during long-term storage. The electrical equipment to power and control the pumps would also be brought in on a trailer truck that needs to be situated at a predetermined area near the wet well. Ensuring access to the predetermined location at the BPC Ball Field or Stuyvesant High School Plaza proposes a coordination challenge that will need an in-depth analysis. The size of the pumps used in this evaluation, 10 MGD units weighting approximately 4,000 pounds, demand several trips of 22-foot flatbed trucks to transport the six (6) units and a portable crane capable of handling and installing each pump in place, requiring coordination among several agencies within the City. In addition, this alternative would require a dedicated contractor and crew, outside of NYCDEP to operate and maintain the equipment, creating a weak link in the operation.

Benefits of the permanent pumping solution

The permanent solution, on the other hand, provides a reliable method of pumping out the flooding from the project area and the least amount of coordination prior to and after severe wet weather events. The evaluation used in this assessment included both the BPC Ball Fields and the Stuyvesant High School Plaza, with the latter being the preferred preliminary site recommended by the PDB Team. This system will consist of an influent chamber with a screening mechanism, an underground wet well equipped with seven (7) permanent installed submersible centrifugal pumps (6 active, 1 standby), a valve pit, a 48-inch force main that discharges into a CSO connecting vault downstream of the tide gates, and an elevated electrical room to power the pump station. The management of the pump station will follow a periodic maintenance program to maintain the screening mechanism and the pumps during periods of non-usage. This alternative will not require external contactors to perform maintenance, however pursuant to further discussions, it is assumed for now that NYCDEP will be responsible for the functioning of the station.

Schedule Impact

Since the current schedule of the ongoing North/West Battery Park City Resiliency (NWBPCR) project will be impacted by the design of this facility whether it is a temporary or a permanent solution, the approach proposed by the PDB Team is the same regardless of the option chosen. The current schedule shows the submission of the CLOMR along with the 60% design document in September of this year. We recommend a discussion with BPCA to explore the scheduling of the CLOMR and how it relates to the permanent pump station design.

Appendix C

Cloudburst Pluvial Analysis

Pluvial Analysis

Pluvial Analysis Criteria

Set 1: Pluvial Analysis

Rainfall Return Period	Durations (hr)	Assumed Boundary Conditions	Near-surface-isolation	OT and Seepage
Current 10-year (Baseline + Proposed)	1, 3	Present-day MHHW	Open	Not included
Current 100-year (Baseline + Proposed)	1, 3	Present-day MHHW	Open	Not included

Set 2: Future Pluvial + Surge/SLR Analysis

Rainfall Return Period	Durations (hr)	Assumed Boundary Conditions	Near-surface-isolation	OT and Seepage
Future 100-year (Baseline + Proposed)	1, 3	2050s SLR	Open	Not included
Future 100-year (Baseline + Proposed)	1, 3	100-year surge (MOCEJ) + 2050s SLR	Closed	Included

- Baseline = existing network (with NSI open or closed based on table above)
- Proposed = proposed 60 MGD PS and conveyance (with NSI open or closed based on table above)
- Set 1 → to be used to propose design modifications to mitigate ponding over 1 foot
- Set 2 → only for mapping purposes

Other notes:
- Rain-on-mesh model used for both Set 1 and 2 simulations
- Activation levels for pump station have not been assessed; assumed that the pump station activates during cloudburst events
  - Requires further development of pump station representation and controls
- Tide gates at every outfall in Lower Manhattan

Rainfall Comparison

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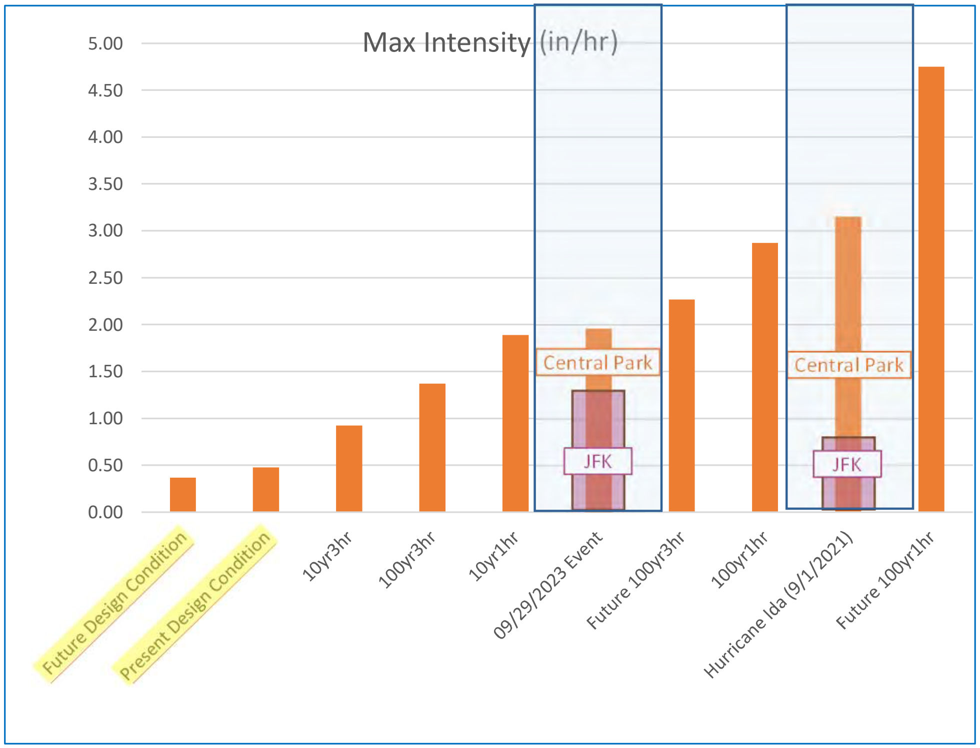

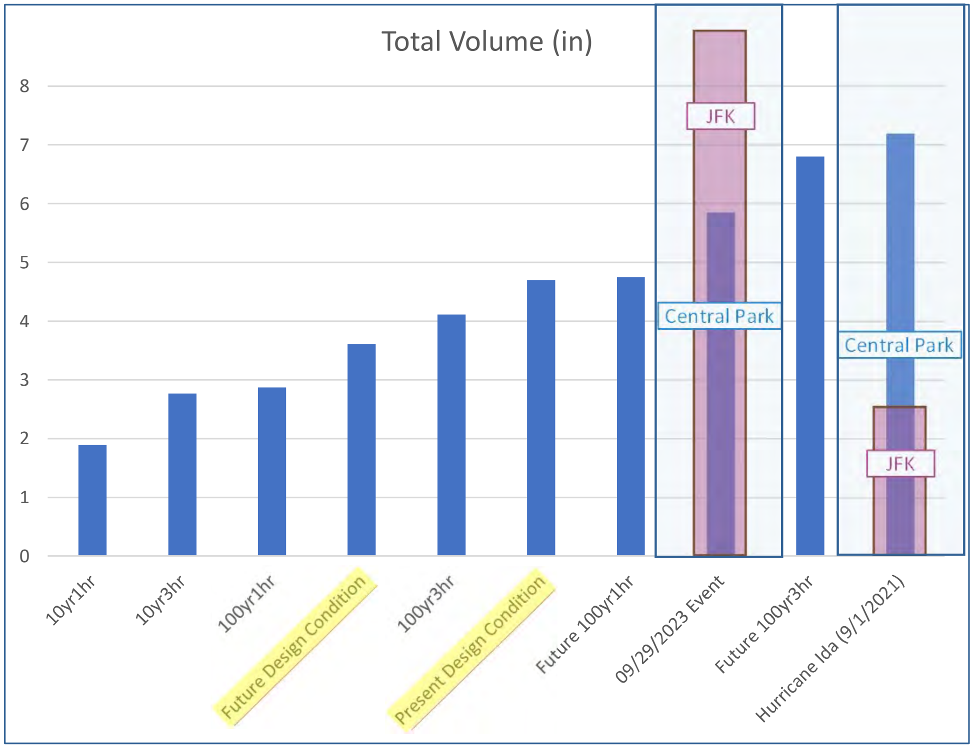

Source for Pluvial Analysis storms: Atlas 14 Central Park (assumed constant intensity for cloudburst events)
Source for Ida and 9/29: ASOS Central Park or JFK rain gauge data

Rainfall Intensities

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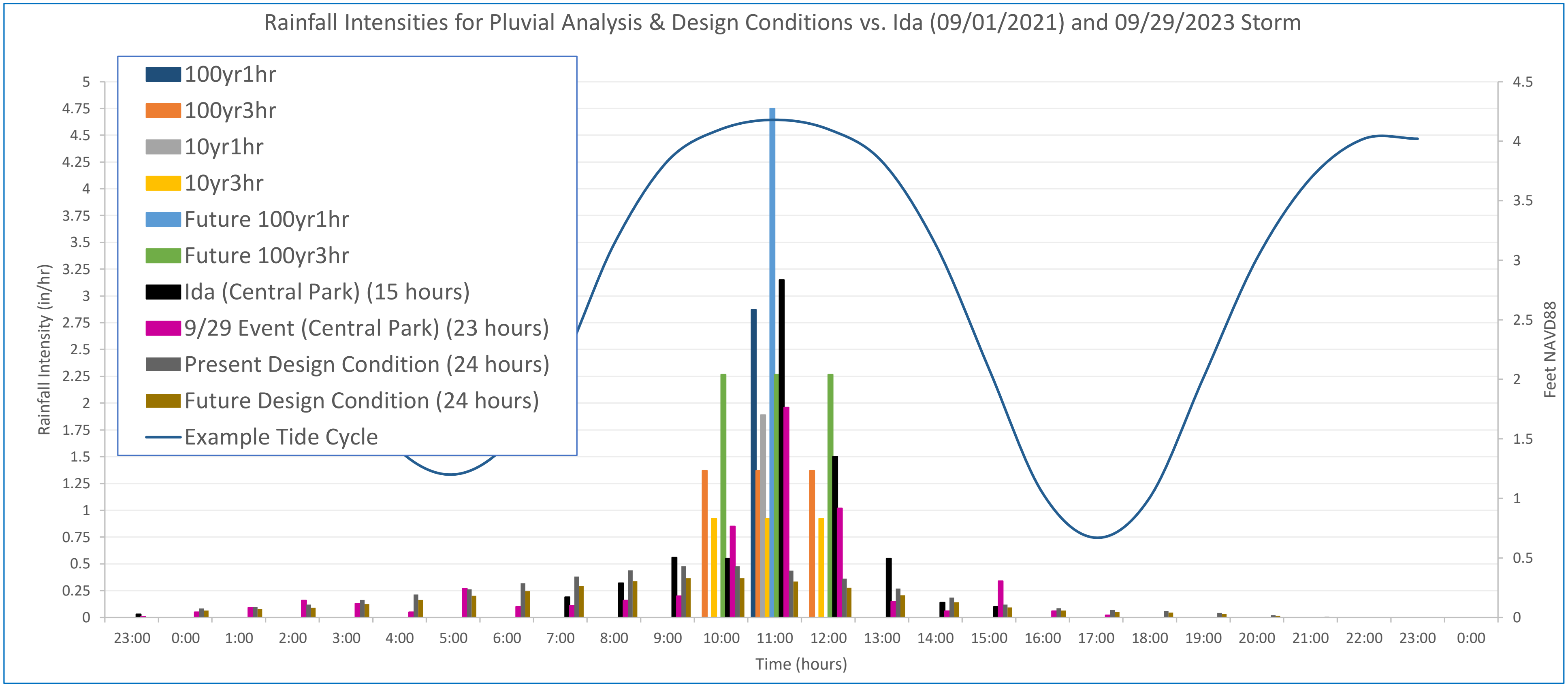
a joint venture



Source for Pluvial Analysis storms: Atlas 14 Central Park (assumed constant intensity for cloudburst events)
Source for Ida and 9/29: ASOS Central Park or JFK rain gauge data

Cumulative Rainfall

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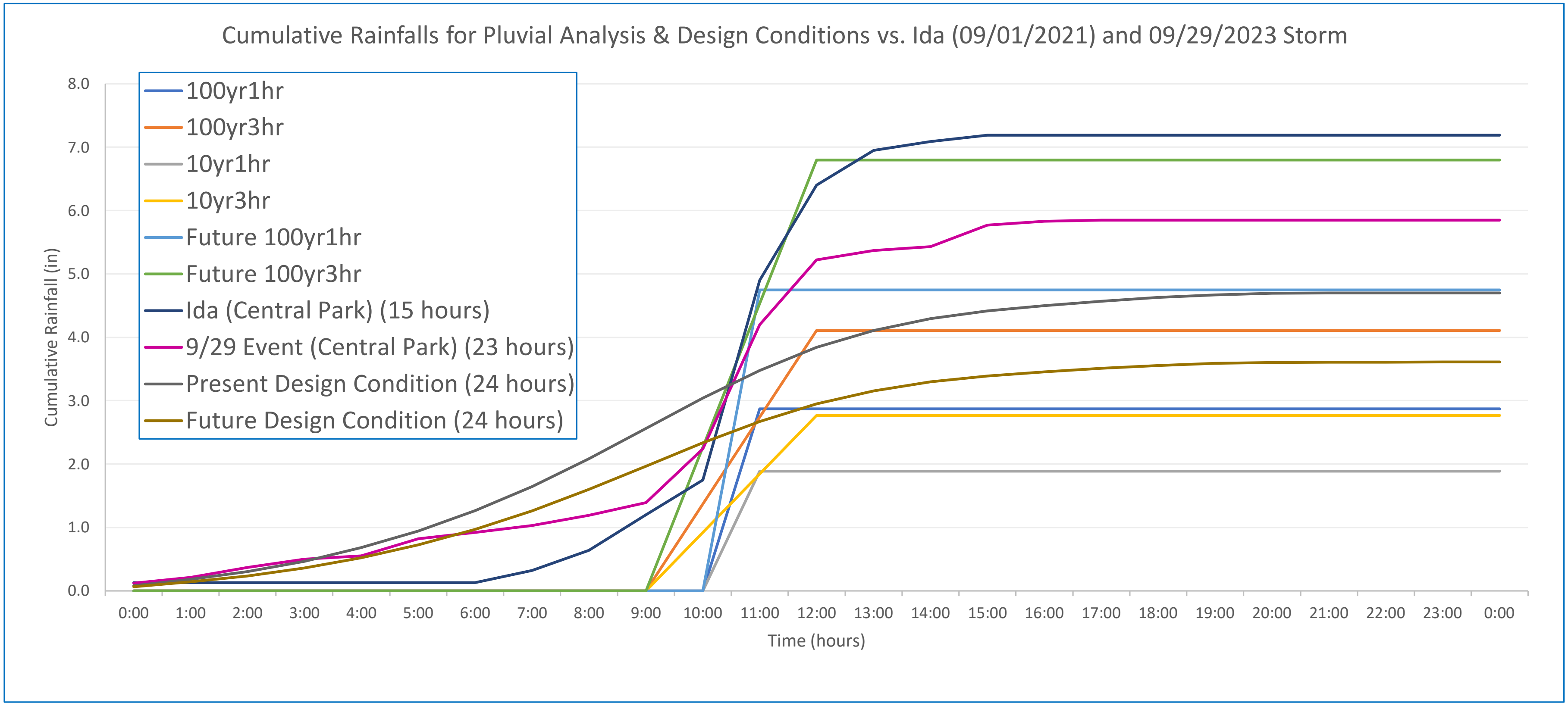
a joint venture



Source for Pluvial Analysis storms: Atlas 14 Central Park (assumed constant intensity for cloudburst events)
Source for Ida and 9/29: ASOS Central Park or JFK rain gauge data

Annual Exceedance Probabilities

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Rainfall (No Surge)	
Current 10-year (1, 3 hr)	10%
Current 100-year (1, 3 hr)	1%
Future 100-year (1, 3 hr)	<1%

Design Conditions

Rainfall (With Surge)

Joint Annual Exceedance Probability (AEP) of Rainfall and Surge Assuming Event Independence

		Rainfall Return Period (x-yr)						
		1	**2**	**5**	**10**	**20**	**50**	**100**
Storm Surge Return Period (x-yr)	**1**	100.0%	50.0%	20.0%	10.0%	5.0%	2.0%	1.0%
	2	50.0%	25.0%	10.0%	5.0%	2.5%	1.0%	0.5%
	5	20.0%	10.0%	4.0%	2.0%	1.0%	0.4%	0.2%
	10	10.0%	5.0%	2.0%	1.0%	0.5%	0.2%	0.1%
	20	5.0%	2.5%	1.0%	0.5%	0.3%	0.1%	0.1%
	50	2.0%	1.0%	0.4%	0.2%	0.1%	0.04%	0.02%
	100	1.0%	0.5%	0.2%	0.1%	0.1%	0.02%	0.01%

Sample 1% AEP Events for FEMA Base Flood Mapping Sensitivity Analysis

Current 10yr 1hr with MHHW

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Model Inputs

NSI Open
Total rainfall depth: 1.89”
No seepage or overtopping

No ponding over 1’ in drainage area to PS with proposed work

No proposed modifications pertaining to the 10yr1hr event

Assumptions:
Pump station activates during cloudburst events
Tide gates at every outfall in Lower Manhattan

Legend	
NWBPC flood barrier system (08/16)	▬

Maximum Depth (ft)	
1” – 4”	
4” – 6”	
6” – 1’	
>1’	

Max flooding in N/W project area	Baseline	Proposed
Acres >1”	13.03	12.31
Million Gallons >1”	1.42	1.20
Acres >1’	0.73	0.48
Million Gallons >1’	0.46	0.34

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Current 10yr 3hr with MHHW

Model Inputs

NSI Open
Total rainfall depth: 2.77”
No seepage or overtopping

Assumptions:
Pump station activates during cloudburst events
Tide gates at every outfall in Lower Manhattan

Legend	
NWBPC flood barrier system (08/16)	(solid black line)

Maximum Depth (ft)	
1” – 4”	light blue
4” – 6”	blue
6” – 1’	dark blue
>1’	orange

Baseline

Proposed
60 MGD

No ponding over 1’ in drainage area to PS with proposed work

No proposed modifications pertaining to the 10yr3hr event

Max flooding in N/W project area	Baseline	Proposed
Acres >1”	11.51	11.34
Million Gallons >1”	1.36	1.33
Acres >1’	0.66	0.64
Million Gallons >1’	0.51	0.51

Battery Park City Authority

Current 100yr 1hr with MHHW

Model Inputs

NSI Open

Total rainfall depth: 2.87”

No seepage or overtopping

Assumptions:
Pump station activates during cloudburst events
Tide gates at every outfall in Lower Manhattan

Legend	
NWBPC flood barrier system (08/16)	

Maximum Depth (ft)	
1” – 4”	
4” – 6”	
6” – 1’	
>1’	

Ponding extents decrease in this area under proposed scenario

Area of concern requiring changes to the proposed design

Max flooding in N/W project area	Baseline	Proposed
Acres >1”	20.13	19.41
Million Gallons >1”	3.21	2.68
Acres >1’	2.53	1.84
Million Gallons >1’	1.59	1.14

Current 100yr 3hr with MHHW

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Model Inputs

NSI Open
Total rainfall depth: 4.11”
No seepage or overtopping

Assumptions:
Pump station activates during cloudburst events
Tide gates at every outfall in Lower Manhattan

Legend	
NWBPC flood barrier system (08/16)	▬

Maximum Depth (ft)	
1” – 4”	light blue
4” – 6”	medium blue
6” – 1’	dark blue
>1’	orange

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Baseline

Proposed 60 MGD

Ponding extents decrease in this area under proposed scenario

Changes to proposed work still required to mitigate remaining flooding over 1’

Max flooding in N/W project area	Baseline	Proposed
Acres >1”	15.50	14.87
Million Gallons >1”	2.30	1.96
Acres >1’	1.49	0.99
Million Gallons >1’	1.09	0.83

Future 100yr 1hr with 2050s SLR

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Model Inputs

NSI Open
Total rainfall depth: 4.75”
No seepage or overtopping

Ponding extents decrease in this area under proposed scenario

Assumptions:
Pump station activates during cloudburst events
Tide gates at every outfall in Lower Manhattan

Legend	
NWBPC flood barrier system (08/16)	

Maximum Depth (ft)
1” – 4”
4” – 6”
6” – 1’
>1’

Max flooding in N/W project area	Baseline	Proposed
Acres >1”	42.42	40.55
Million Gallons >1”	13.82	12.49
Acres >1’	13.76	12.06
Million Gallons >1’	10.25	8.99

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Future 100yr 3hr with 2050s SLR

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Model Inputs
NSI Open
Total rainfall depth: 6.80”
No seepage or overtopping

Assumptions:
Pump station activates during cloudburst events
Tide gates at every outfall in Lower Manhattan

Legend	
NWBPC flood barrier system (08/16)	

Maximum Depth (ft)	
1” – 4”	
4” – 6”	
6” – 1’	
>1’	

Ponding extents decrease in this area under proposed scenario

Max flooding in N/W project area	Baseline	Proposed
Acres >1”	34.82	32.55
Million Gallons >1”	10.87	9.59
Acres >1’	9.97	8.69
Million Gallons >1’	7.95	6.92

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Future 100yr 1hr with 100yr Surge and 2050s SLR

Model Inputs

NSI Closed
MOCEJ Surge (Peak at 11AM)
Surge peak and rainfall peak aligned
Total rainfall depth: 4.75”
Seepage* included
Overtopping** included

*Seepage model from WSP used Scaled Hurricane Sandy surge hydrograph with peak surge at 13.8’ NAVD88
*Overtopping updated as of 10/9/23 from Arcadis coastal modeling team

Assumptions:
Pump station activates during cloudburst events
Tide gates at every outfall in Lower Manhattan

Legend	
NWBPC flood barrier system (08/16)	

Maximum Depth (ft)	
1” – 4”	
4” – 6”	
6” – 1’	
>1’	

Ponding extents decrease in this area under proposed scenario

Max flooding in N/W project area	Baseline	Proposed
Acres >1”	52.68	51.03
Million Gallons >1”	26.19	22.57
Acres >1’	27.26	24.75
Million Gallons >1’	23.04	19.31

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Future 100yr 3hr with 100yr Surge and 2050s SLR

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Model Inputs

NSI Closed
MOCEJ Surge (Peak at 11AM)
Surge peak and rainfall peak aligned
Total rainfall depth: 6.80”
Seepage* included
Overtopping** included

*Seepage model from WSP used Scaled Hurricane Sandy surge hydrograph with peak surge at 13.8’ NAVD88
*Overtopping updated as of 10/9/23 from Arcadis coastal modeling team

Assumptions:
Pump station activates during cloudburst events
Tide gates at every outfall in Lower Manhattan

Legend	
NWBPC flood barrier system (08/16)	▬

Maximum Depth (ft)	
1” – 4”	light blue
4” – 6”	blue
6” – 1’	dark blue
>1’	orange

Ponding extents decrease in this area under proposed scenario

Max flooding in N/W project area	Baseline	Proposed
Acres >1”	52.83	50.99
Million Gallons >1”	32.12	27.48
Acres >1’	31.13	28.51
Million Gallons >1’	29.45	24.69

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Design Modifications

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- Use Set 1 simulations to propose design modifications to mitigate ponding over 1 foot
  - 100yr 3hr MHHW
  - 100yr 1hr MHHW

- Toolbox:
  - Modify pump station capacity
  - Modify proposed conveyance infrastructure

Current 100yr 3hr with MHHW

Model Inputs

NSI Open
Total rainfall depth: 4.11”
No seepage or overtopping

Assumptions:
Pump station activates during cloudburst events
Tide gates at every outfall in Lower Manhattan

Legend	
NWBPC flood barrier system (08/16)	

Maximum Depth (ft)	
1” – 4”	
4” – 6”	
6” – 1’	
>1’	

Proposed
60 MGD

Proposed
60 MGD
(Relief Piping)

Would need to bypass restriction at the regulator and bring more water to the PS if system were to be designed for cloudburst events.

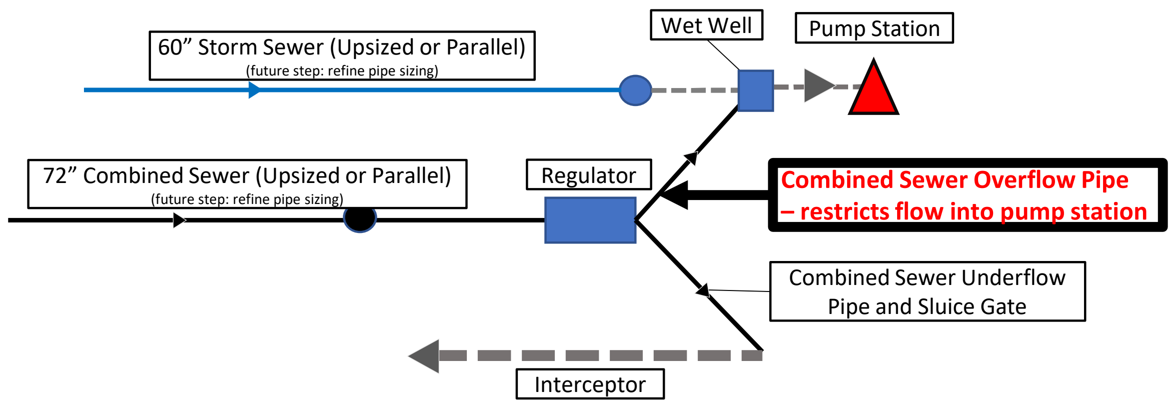
No ponding over 1’ in drainage area to PS with additional pumping capacity

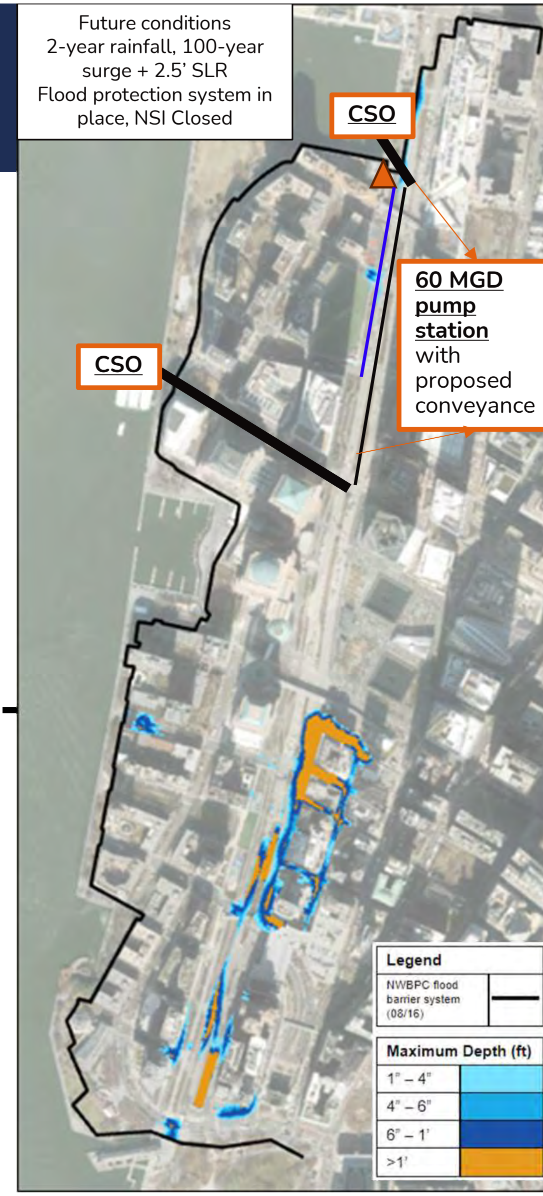
Max flooding in N/W project area	Proposed 60 MGD	Proposed 60 MGD w/ Relief Piping
Acres >1”	14.87	13.00
Million Gallons >1”	1.96	1.80
Acres >1’	0.99	0.93
Million Gallons >1’	0.83	0.80

Battery Park City Authority

Proposed Additional Conveyance

Initial Proposed System:





Proposed Additional Conveyance

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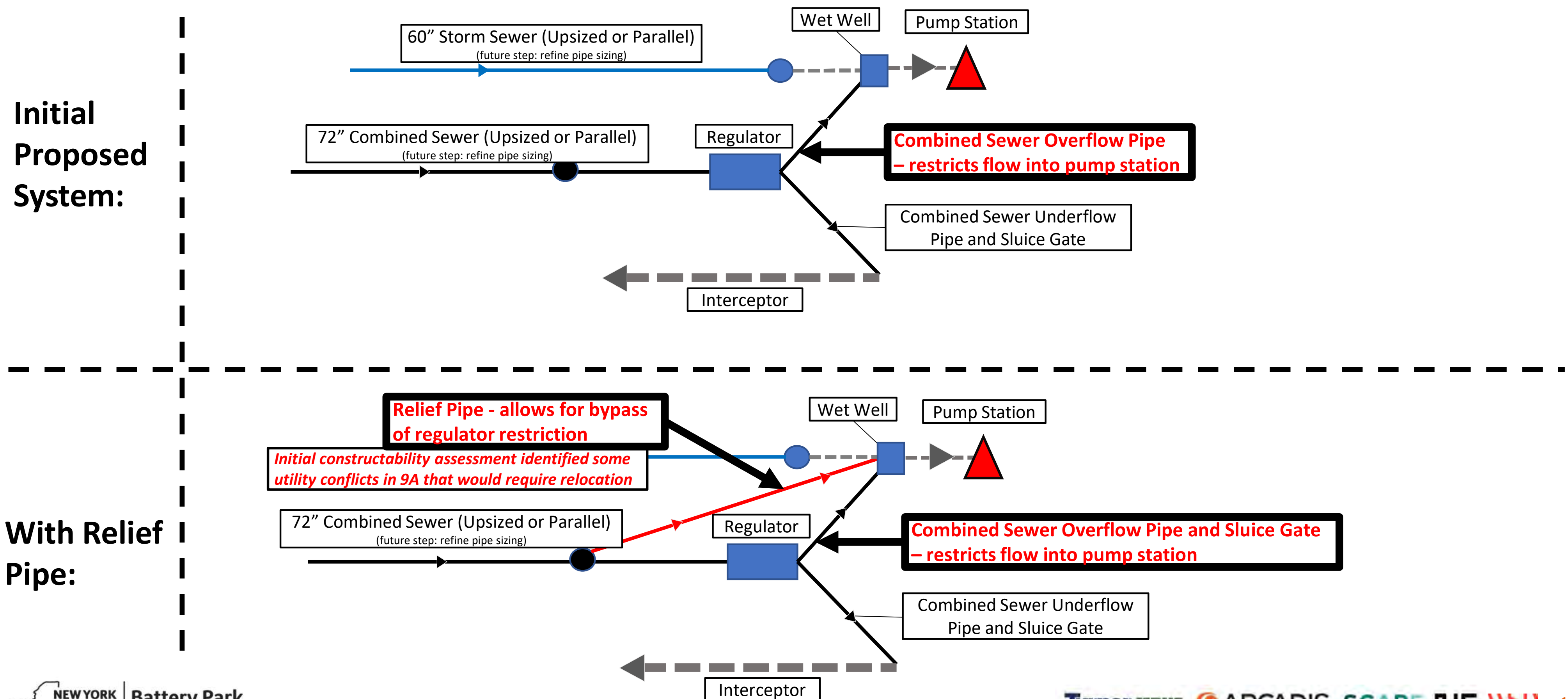

Current 100yr 1hr with MHHW

Model Inputs

NSI Open
Total rainfall depth: 2.87”
No seepage or overtopping

No ponding over 1’ in drainage area to PS with additional pumping capacity

Assumptions:
Pump station activates during cloudburst events
Tide gates at every outfall in Lower Manhattan

Legend	
NWBPC flood barrier system (08/16)	▬

Maximum Depth (ft)	
1” – 4”	light blue
4” – 6”	blue
6” – 1’	dark blue
>1’	orange

Max flooding in N/W project area	Proposed 60 MGD	Proposed 60 MGD w/ Relief Piping	Proposed 100-120 MGD w/ Relief Piping
Acres >1”	19.41	18.03	17.72
Million Gallons >1”	2.68	2.16	2.05
Acres >1’	1.84	1.07	0.92
Million Gallons >1’	1.14	0.70	0.64

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Conclusions

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- Proposed 60 MGD pump station is effective at reducing flooding under the initial design criteria
- Potential adjustments to the conveyance infrastructure can provide additional flexibility to manage a broader range of storm events
- Increases in pumping capacity have limited additional benefits for cloudburst events
- Pump operations will ultimately be determined by DEP, who will be responsible for operations and maintenance of the pump station

Appendix D

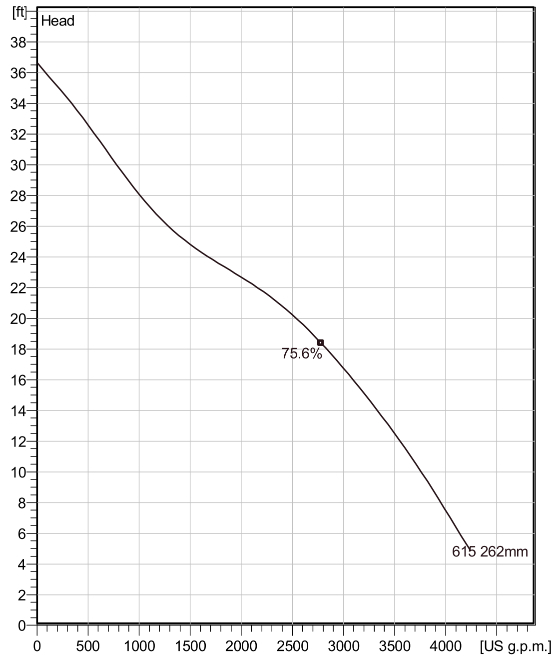
Data Sheets for 2,135 gpm Pumps

NP 3171 LT 3~ 615

Patented self cleaning semi-open channel impeller, ideal for pumping in waste water applications. Modular based design with high adaptation grade.

Technical specification

Curves according to: Water, pure Water, pure [100%],39.2 °F,62.42 lb/ft³,1.6891E-5 ft²/s



Curve: ISO 9906

Nominal (mean) data shown. Under- and over-performance from this data should be expected due to standard manufacturing tolerances.
Please consult your local Flygt representative for performance guarantees.

Configuration

Motor number
N3171.830 25-32-6KE-W
IE3 18hp

Installation type
P - Semi permanent, Wet

Impeller diameter
262 mm

Discharge diameter
10 inch

Pump information

Impeller diameter
262 mm

Discharge diameter
10 inch

Inlet diameter
250 mm

Maximum operating speed
1180 rpm

Number of blades
2

Max. fluid temperature
40 °C

Material

Impeller
Hard-Iron ™

Project	Xylect-22043434	**Created by**			
Block		**Created on**	3/22/2024	**Last update**	3/22/2024

NP 3171 LT 3~ 615

Technical specification

Motor - General

Motor number N3171.830 25-32-6KE-W IE3 18hp	**Phases** 3~	**Rated speed** 1180 rpm	**Rated power** 18 hp
ATEX approved IEC	**Number of poles** 6	**Rated current** 21 A	**Stator variant** 7
Frequency 60 Hz	**Rated voltage** 460 V	**Insulation class** H	**Type of Duty** S1
Version code 830			

Motor - Technical

Power factor - 1/1 Load 0.85	**Motor efficiency - 1/1 Load** 92.6 %	**Total moment of inertia** 7.89 lb ft²	**Starts per hour max.** 30
Power factor - 3/4 Load 0.80	**Motor efficiency - 3/4 Load** 92.8 %	**Starting current, direct starting** 159 A	
Power factor - 1/2 Load 0.70	**Motor efficiency - 1/2 Load** 92.2 %	**Starting current, star-delta** 53 A	

Project	Xylect-22043434	**Created by**			
Block		**Created on**	3/22/2024	**Last update**	3/22/2024

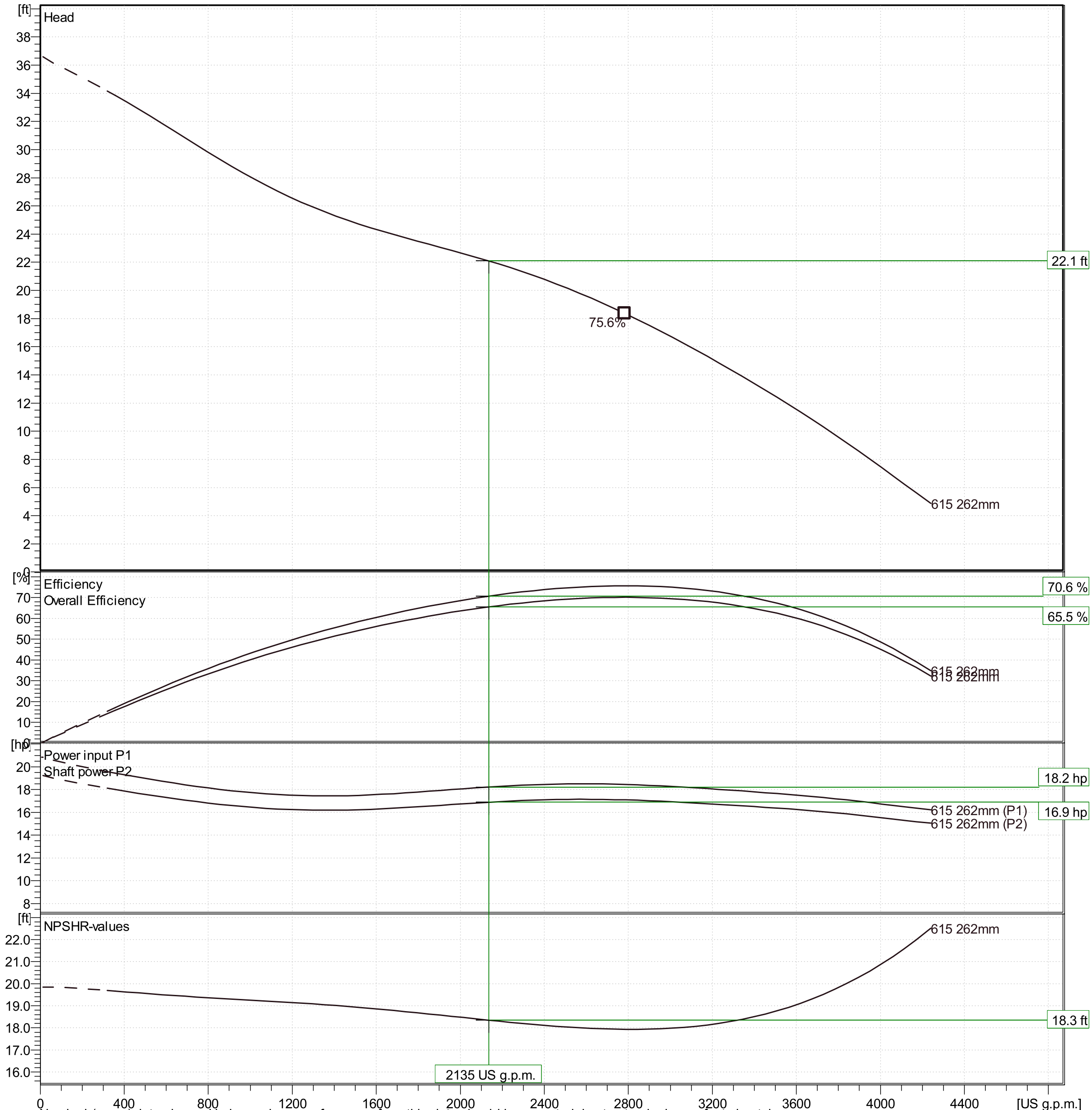
NP 3171 LT 3~ 615

Performance curve

Duty point

Flow	Head
2140 US g.p.m.	22.1 ft

Curves according to: Water, pure Water, pure [100%],39.2 °F,62.42 lb/ft³,1.6891E-5 ft²/s



Nominal (mean) data shown. Under- and over-performance from this data should be expected due to standard manufacturing tolerances.
Please consult your local Flygt representative for performance guarantees.

Curve: ISO 9906

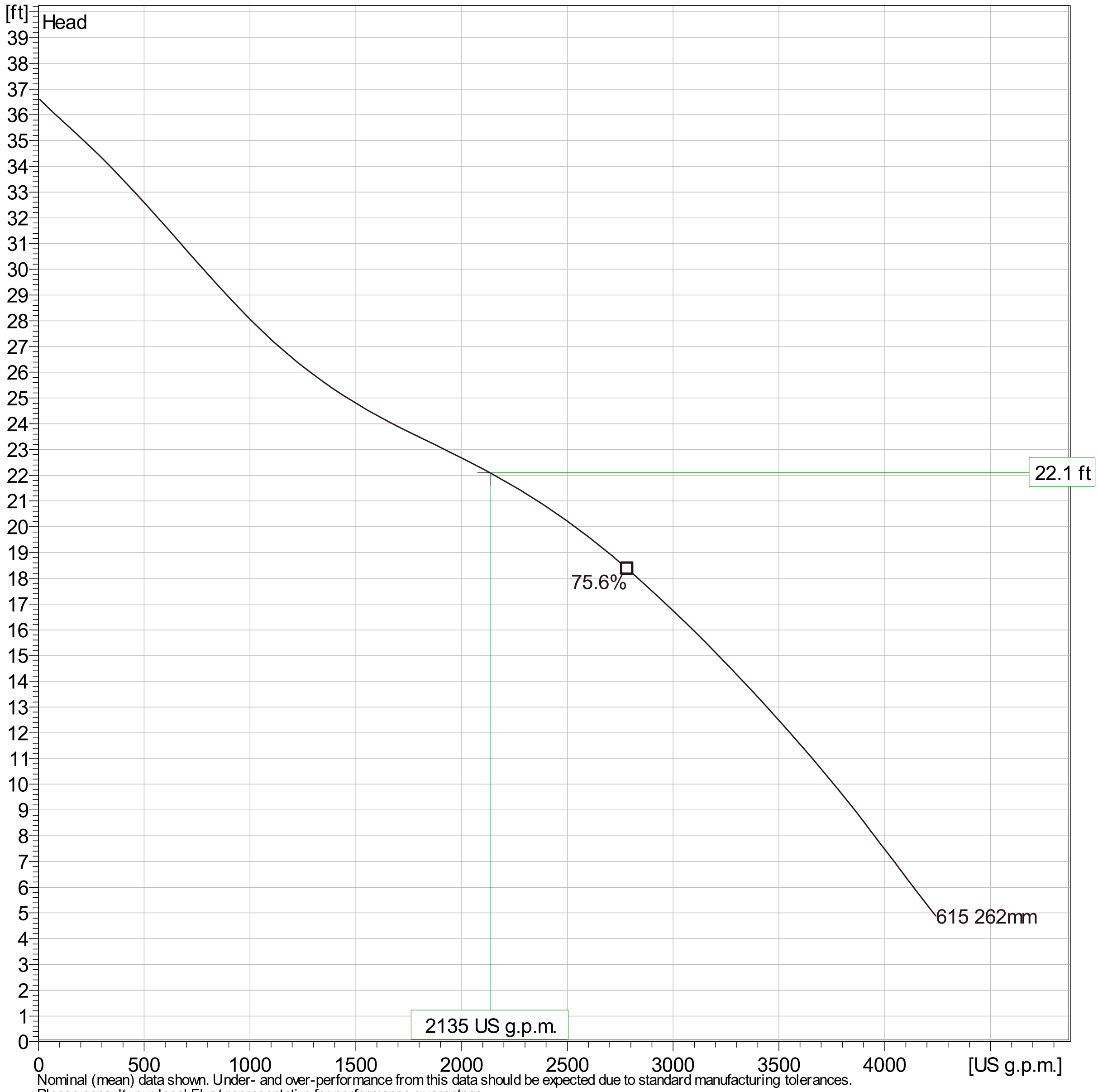
Xylect-22043434

Created on 3/22/2024 **Last update** 3/22/2024

NP 3171 LT 3~ 615

Duty Analysis

Curves according to: Water, pure [100%] ; 39.2°F; 62.42lb/ft³; 1.6891E-5ft²/s



Nominal (mean) data shown. Under- and over-performance from this data should be expected due to standard manufacturing tolerances.
Please consult your local Flygt representative for performance guarantees.

Operating characteristics

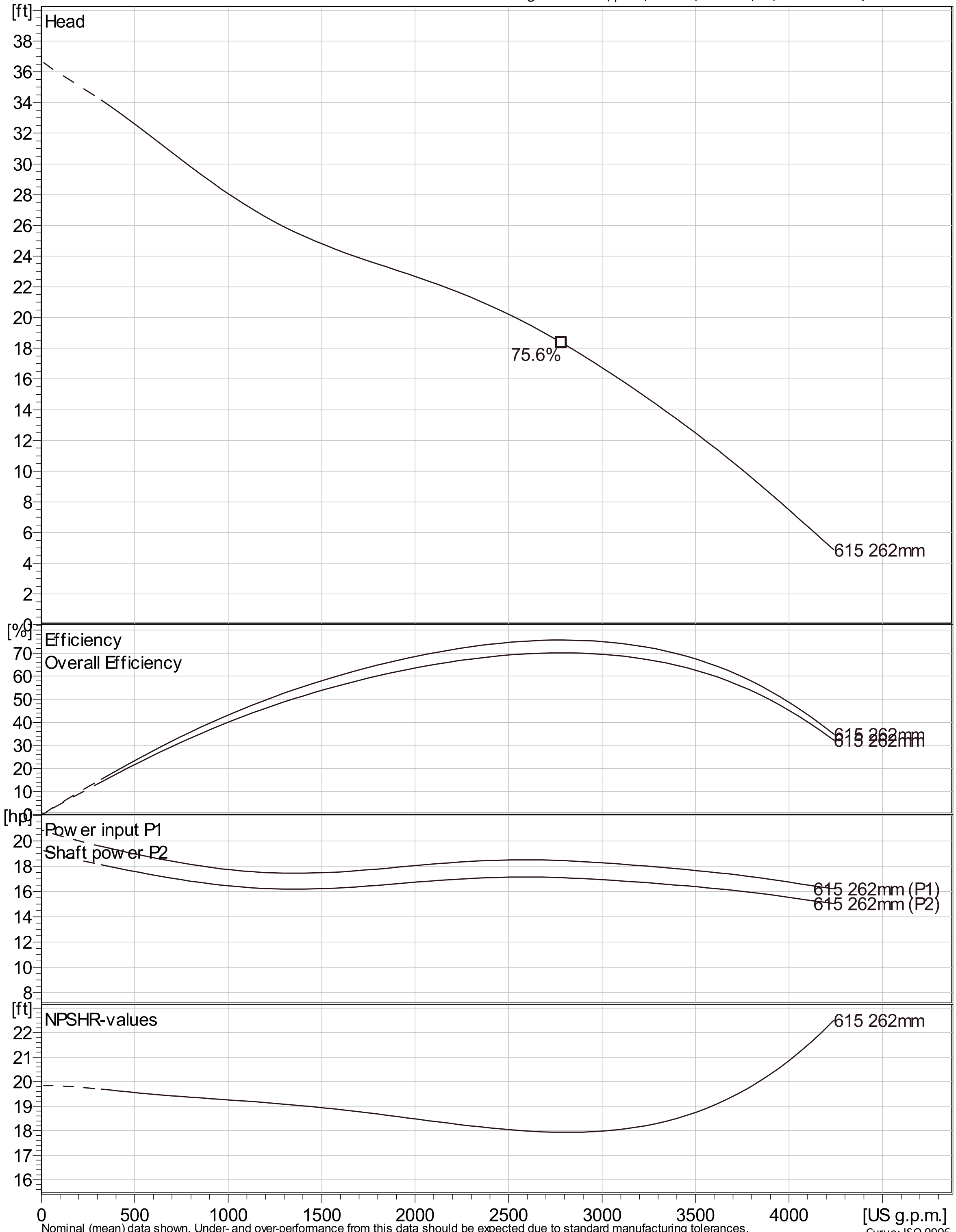
Pumps / Systems	Flow	Head	Shaft power	Flow	Head	Shaft power	Hydr.eff.	Spec. Energy	NPSHre
	US g.p.m.	ft	hp	US g.p.m.	ft	hp		kWh/US MG	ft
1	2140	22.1	16.9	2140	22.1	16.9	70.6 %	106	18.3

Project		**Created by**			
Block	Xylect-22043434	**Created on**	3/22/2024	**Last update**	3/22/2024

NP 3171 LT 3~ 615

VFD Curve

Curves according to: Water, pure ,39.2 °F,62.42 lb/ft³,1.6891E-5 ft²/s



Nominal (mean) data shown. Under- and over-performance from this data should be expected due to standard manufacturing tolerances. Please consult your local Flygt representative for performance guarantees.

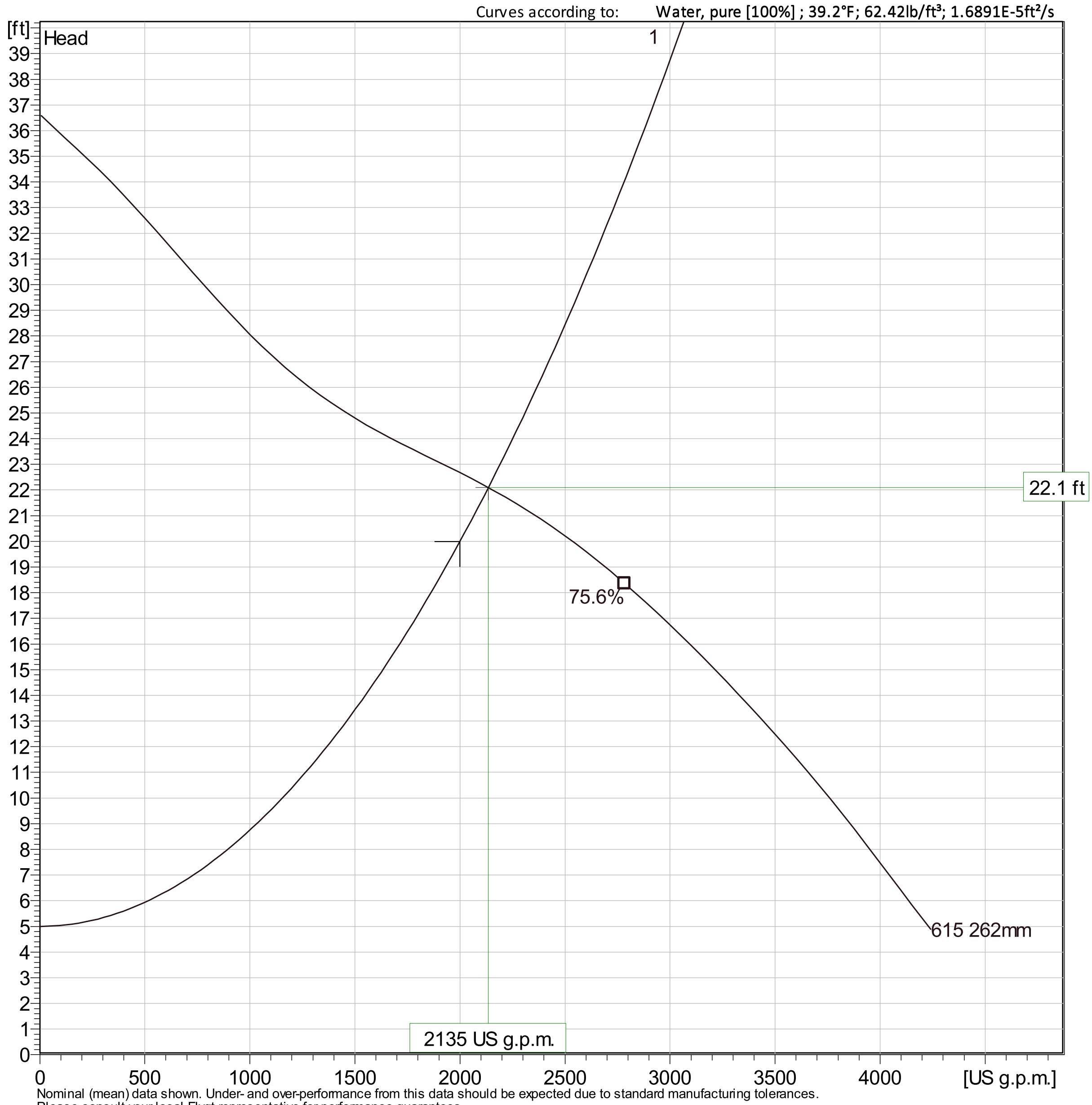
Curve: ISO 9906

Project	Xylect-22043434	**Created by**			
Block		**Created on**	3/22/2024	**Last update**	3/22/2024

NP 3171 LT 3~ 615

VFD Analysis

Curves according to: Water, pure [100%] ; 39.2°F; 62.42lb/ft³; 1.6891E-5ft²/s



Nominal (mean) data shown. Under- and over-performance from this data should be expected due to standard manufacturing tolerances.
Please consult your local Flygt representative for performance guarantees.

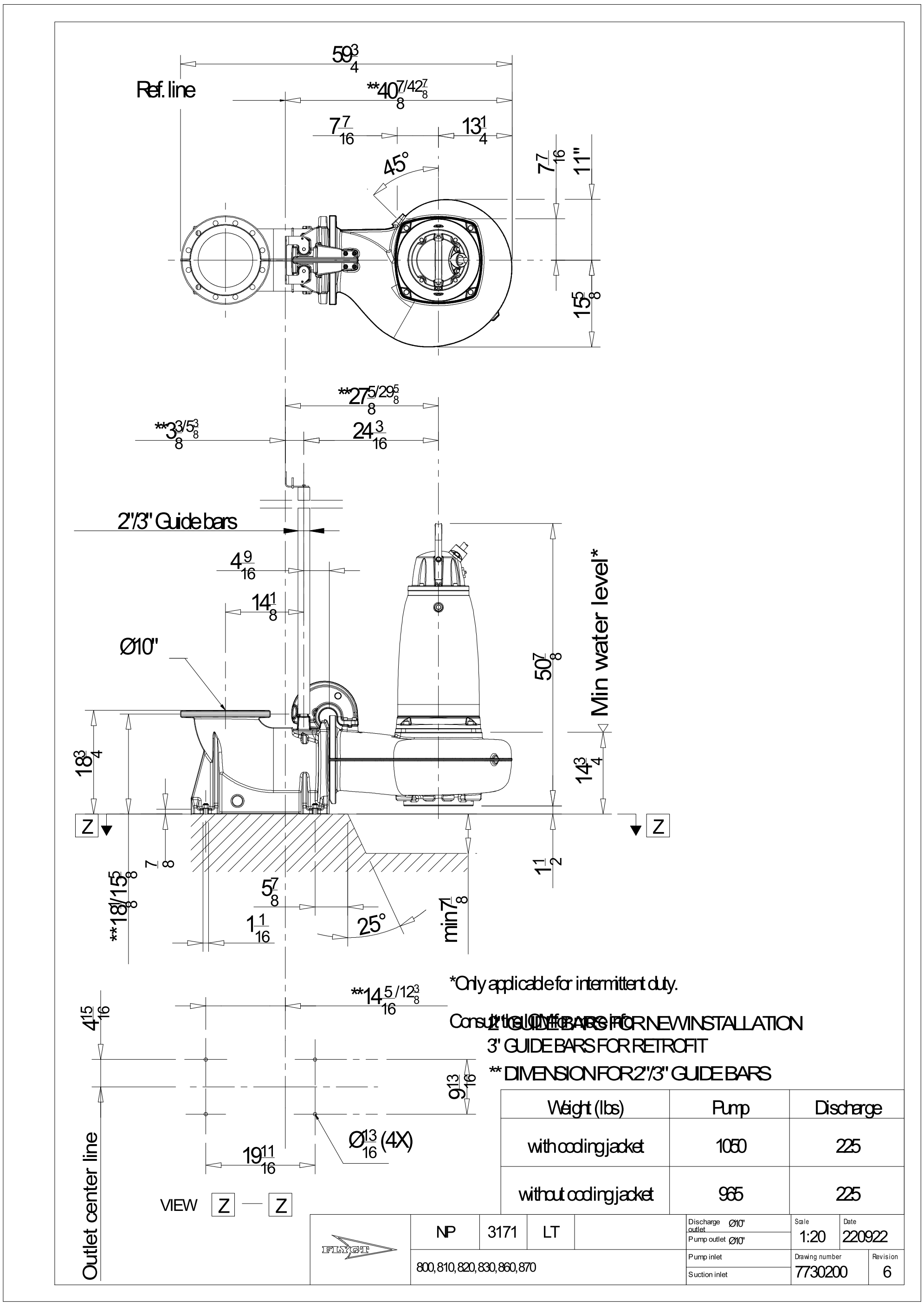
Operating Characteristics

Pumps / Systems	Frequency	Flow	Head	Shaft power	Flow	Head	Shaft power	Hydr.eff.	Specific energy	NPSHre
		US g.p.m.	ft	hp	US g.p.m.	ft	hp		kWh/US MG	ft
1	60 Hz	2140	22.1	16.9	2140	22.1	16.9	70.6 %	106	18.3

Project	Xylect-22043434	**Created by**			
Block		**Created on**	3/22/2024	**Last update**	3/22/2024

NP 3171 LT 3~ 615

Dimensional drawing



Weight (lbs)	Pump	Discharge
with cooling jacket	1050	225
without cooling jacket	965	225

Project	Xylect-22043434	**Created by**	
Block		**Created on**	3/22/2024
		Last update	3/22/2024