

Appendix G
Interior Drainage Report



**Battery Park
City Authority**

North/West Battery Park City Resiliency Project

Interior Drainage Report

March 4, 2025

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CONTENTS

Acronyms & Abbreviations	ix
1 Overview and Objectives	1
1.1 Introduction and Study Area	1
1.2 Drainage Analysis Goals	2
1.3 Decisions Made During Design Phase	2
2 Storm Surge Isolation Methodology and Design Approach	3
2.1 Existing Storm and Sanitary Sewer System	5
2.2 Conveyance System Modifications	8
2.2.1 Sanitary Sewer Modifications	9
2.2.1.1 Alternative 1: Gravity Flow to CSO	10
2.2.1.2 Alternative 2: Pumping Station	10
2.2.1.3 Alternative 3: Onsite Storage	10
2.2.1.4 Selected Alternative for Sanitary Sewer Modifications	11
2.2.1.5 90% Design of the Vesey Street Gravity Flow to CSO	12
2.2.1.6 90% Design of the West Thames Temporary Pumping Station	14
2.2.1.6.1 Recommended Site Layout	16
2.2.1.6.2 Influent Manual Gate Structure	16
2.2.1.6.3 Connection into the 84" Combined Sewer	17
2.2.1.6.4 Design Flow	18
2.2.1.6.5 Gravity Pipe Design	18
2.2.1.6.6 Force Main Design	19
2.2.1.6.7 Pump Design	20
2.2.1.6.8 Wet Well Design	22
2.2.1.6.9 Electric Design	23
2.2.1.6.10 Controls	24
2.2.1.6.11 Access and Security	24
2.3 Outfall Modifications	25
2.4 Post-construction Stormwater Management Practices	25
2.4.1 Green Infrastructure	26
2.5 Storage Design	30
2.6 Slide Gate Modifications	30
2.7 Tide Gate Modifications and CSO relocation	30
2.8 Pile Support Design	31
2.9 Other Modifications	32

2.9.1	Gateway Building Stormwater Improvements	32
2.9.1.1	90% Design of Gateway Plaza Sump Pump	34
2.9.1.1.1	Design Flow	34
2.9.1.1.2	Gravity Pipe Design	34
2.9.1.1.3	Force Main Design.....	34
2.9.1.1.4	Pump Design	35
2.9.1.1.5	Wet Well Design	35
2.9.1.1.6	Electric Design.....	35
2.9.1.1.7	Controls	36
2.9.1.1.8	Access and Security	36
2.9.2	BMCC Parking Lot Drainage Improvements	36
2.10	Erosion & Sedimentation Control.....	37
3	Stormwater Analysis Methodology	39
3.1	Modeling Software	39
3.2	Data Sources	39
3.3	Model Construction	40
3.4	Model Assumptions.....	41
3.5	Rain-On-Mesh (ROM) Methodology	41
4	Analysis Results and Key Findings	42
4.1	Rainfall and Surge Conditions Considered.....	42
4.2	Quantifying Existing Flood Risk	43
4.3	Proposed Solutions	45
4.4	Preliminary Pump Station Sizing Analysis	46
4.4.1	Pump Stations.....	46
4.4.2	Sensitivity Analysis: Impact of Timing of MOCEJ/DEP Surge Hydrograph.....	48
4.4.3	Cloudburst Analysis	49
4.5	Localized Ponding.....	53
4.5.1	Teardrop Park.....	53
4.5.2	Gateway Building	53
5	Future Work	55
6	References	56

TABLES

Table 2-1. Proposed Modifications to Drainage Infrastructure	3
Table 2-2. Force Main Pipe Capacities	19
Table 2-3. Wet Well Design Calculations	23
Table 2-4. Proposed Tide Gates at Existing Outfalls	25
Table 2-5. Water Quality Volume by Reach	26
Table 2-6. WQv Managed by Green Infrastructure	27
Table 3-1. Data Sources.....	39
Table 4-1. Approximate Joint Annual Probability of Rainfall and Surge Assuming Event Independence	42
Table 4-2. Interior Drainage Objectives and Criteria	42
Table 4-3. Additional Cloudburst Scenarios	50

FIGURES

Figure 1-1. Site Location Map	1
Figure 2-1. Proposed Isolation Approach	4
Figure 2-2. Existing Storm and Sewer System.....	7
Figure 2-3. Profiles at Sanitary Connection Locations in BPC	9
Figure 2-4. Onsite Storage for Vesey St (left) and West Thames St (right)	11
Figure 2-5: Vesey Street: Gravity Flow to CSO	12
Figure 2-6: Vesey Street: Gravity Flow to CSO.....	13
Figure 2-7. Existing Sewer Regulator Chamber Q	14
Figure 2-8. Record Drawing of Chamber Q (1982 - BPC South Residential Phase II Infrastructure).....	15
Figure 2-9. Recommended Site Layout of West Thames Pumping Station and Manual Gate Structure	16
Figure 2-10. Proposed New Manual Gate Structure	17
Figure 2-11. West Thames Pump Curve	21
Figure 2-12. Proposed Green Infrastructure Features	29
Figure 2-13. Gateway Flood Mitigation Options	33
Figure 2-14: Gateway Plaza without modification to drainage vs Gateway Plaza with modification to drainage and a sump pump	34
Figure 2-15. Overflow Pipe at BMCC Parking Lot.....	37
Figure 4-1. 5-year and 2-year Rainfall Hyetograph and 100-year Surge Conditions	43

Figure 4-2. Existing Conditions with No FBS and NSI Open (5-year 24-hour rainfall, 100-year surge) 44

Figure 4-3. Baseline Conditions for Present and Future (ROM Results) 45

Figure 4-4. Baseline (No Pump/Interior Drainage Work) (Left), Proposed Pumping and Conveyance (Right) 47

Figure 4-5. Hydraulic Connections included in Figure 4-4 47

Figure 4-6. Surge Conditions Used in Sensitivity Analysis..... 48

Figure 4-7. Sensitivity Analysis - MOCEJ/DEP Surge Hydrograph..... 49

Figure 4-8. Cloudburst Events (Intensity and Volume)..... 51

Figure 4-9. 100-year 3-hour Cloudburst Results with Proposed Work (Pump Station + Alternative 5 Conveyance) and Additional Modification..... 52

Figure 4-10. 100-year 1-hour Results with Proposed Work (Pump Station + Alternative 5 Conveyance) and Additional Modifications..... 53

APPENDICES

Appendix A. Hydraulic Analysis Calculations

Appendix B. Interior Drainage-Pumping Solutions

Appendix C. Cloudburst Pluvial Analysis

Appendix D. Data Sheets for 2,135 gpm Pumps

Appendix E. 90% Site Structural Design Calculations

Appendix F. Micropile Design Calculations

Appendix G. Support of Excavation Conceptual Drawings

Appendix H. Temporary Drainage for Reach 4

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
PANYNJ	Port Authority of New York and New Jersey
ROM	Rain-On-Mesh
SBPCR	South Battery Park City Resiliency
SLR	Sea Level Rise
USWR	Unified Stormwater Rule

1 Overview and Objectives

1.1 Introduction and Study Area

Climate change-induced sea level rise accompanied by more frequent, intense storm events and storm surges are a significant and increasing risk to coastal communities. In 2012, Superstorm Sandy demonstrated the impacts of these types of events in the New York City area and highlighted the City's vulnerability to coastal storm surges and severe weather. The Battery Park City Authority (BPCA) is proposing to implement the North/West Battery Park City Resiliency Project (referred to herein as the "NWBPCR Project" or "Project"). The primary goal of the NWBPCR Project is to protect Lower Manhattan from the effects of climate change by developing a Flood Barrier System (FBS) that will shield the area from current and future storm surges and rising sea levels caused by climate change. Located in lower Manhattan, the project involves installing a continuous FBS within Battery Park City (BPC) along the Hudson River waterfront, extending from 1st Street in the south (connecting to the SBPCR project) to BPC North Esplanade in the north (connecting to higher ground). The FBS continues across Route 9A/West Street and extends eastward along North Moore Street. This comprehensive FBS integrates various coastal flood protection elements, including floodwalls, closure structures, and enhancements to interior drainage infrastructure.

The project area is defined as the region loosely bounded by North Moore Street adjacent to the Borough of Manhattan Community College (BMCC) to the North and the South Cove on 1st Place to the South as shown in **Figure 1-1**.

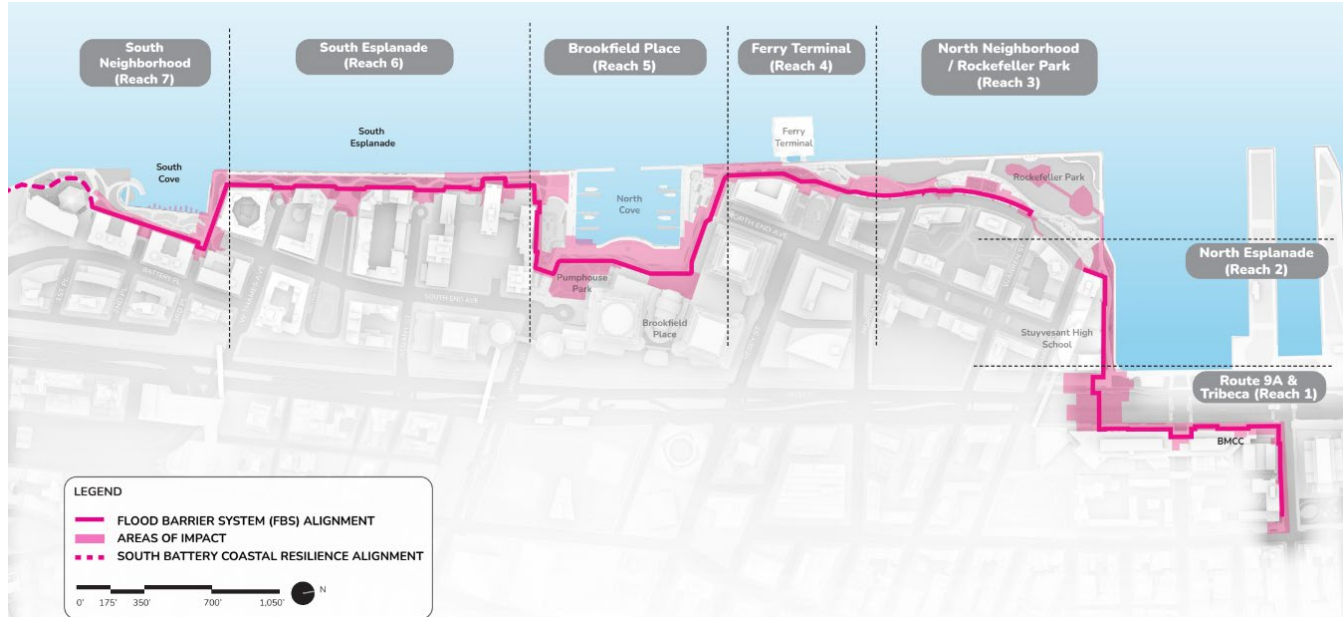


Figure 1-1. Site Location Map

In addition to a flood barrier system designed to protect against storm surges, interior drainage improvements will be implemented to prevent flooding from rainfall and coastal storm surge. Coastal storm surges entering the study area through the City's unprotected storm sewer system, are prevented by isolating BPC from the unprotected storm 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 storm 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 systems¹.

1.3 Decisions Made During Design Phase

The following were decisions made prior to the 90% 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 2.6**.
- Incorporated the proposed overflow gravity pipe option to manage stormwater at BMCC parking lot. See **Section 2.9.2**.

Below are design components that are included in the 90% design based on decisions made:

- Incorporated sanitary sewer flow management design for North (Vesey Street) and South (W. Thames Street) Areas. See **Section 2.2.1** for details on this.
- Incorporated Gateway Plaza stormwater management design. See **Section 2.9.1**.
- Relocated 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 2.7**. Revised tide gate design to incorporate future force main connection from proposed pump station connection
- Relocated Rector Place tide gate chamber west to avoid conflicts with the existing 54" stormwater outfall, which is located closer to the existing 96" CSO based on recent survey.
- Relocated River Terrace tide gate chamber east to avoid modifying the existing junction structure
- Replaced the proposed tide gate in the southbound Route 9A lane with a slide gate to avoid relocating the existing water mains
- Relocated hydrodynamic separators downstream of existing DEP catch basins based on comments from DEP
- Removed hydrodynamic separators on platform in Reach 5 Area 8 due to limited depth. Upsized the hydrodynamic separator by Vesey Street in Reach 4 Area 10.

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

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 2.6**) 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 improvements, as discussed with New York City Department of Environmental Protection (NYCDEP) and shown in the 90% 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 2.2.1 .
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. Three slide gates are proposed in Reach 1, see Figure 2-1 .
Rerouting of storm sewers	Where feasible, existing storm sewers are proposed to be disconnected and rerouted to prevent storm surge from entering the protected area through the sewer system.
Other floodproofed manholes along/adjacent to the FBS alignment	Prevents unprotected sewers from flooding into protected area.

North/West Battery Park City Resiliency Project – Interior Drainage Report

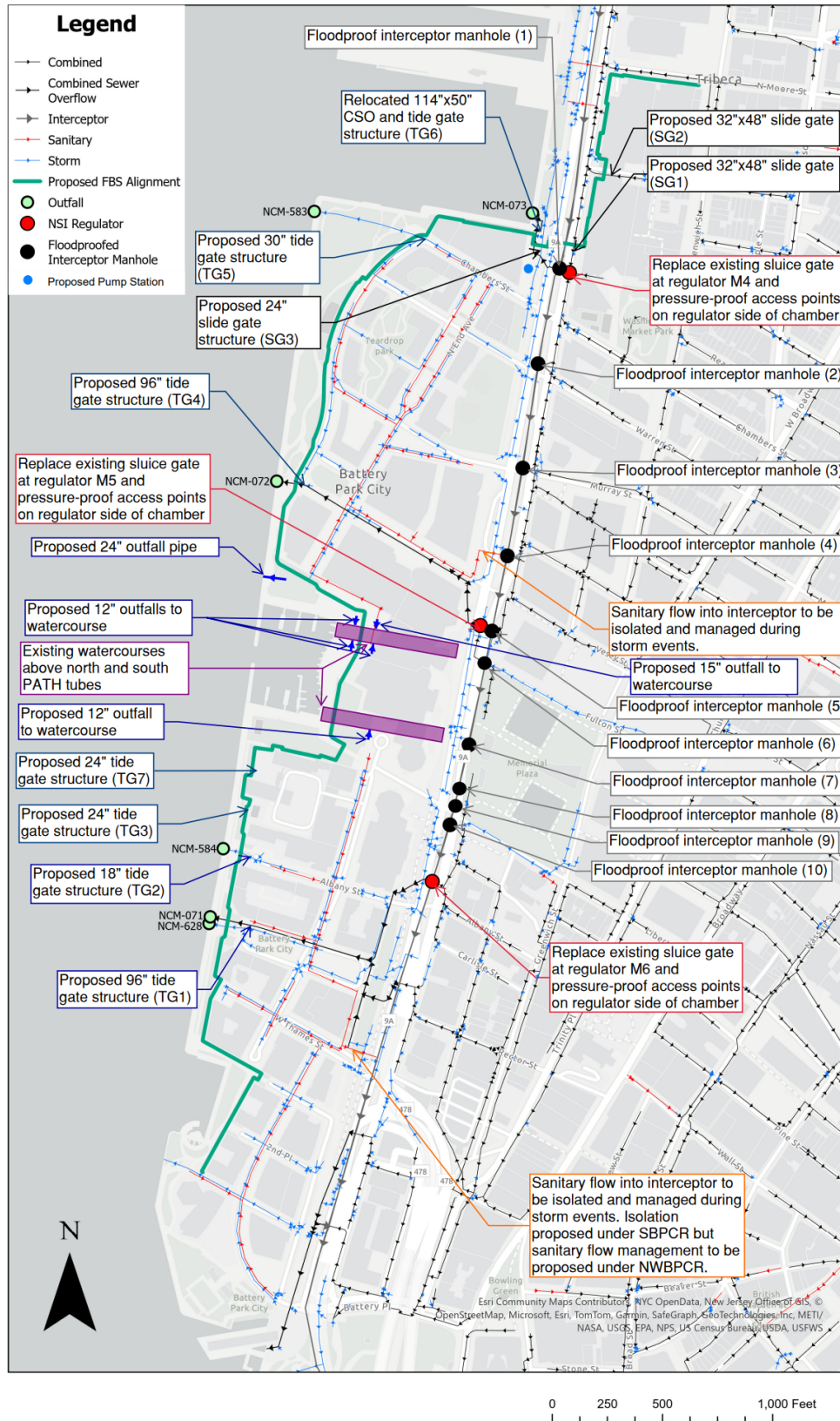


Figure 2-1. Proposed Isolation Approach

2.1 Existing Storm and Sanitary Sewer System

Battery Park City is served by separate sanitary sewer system and a municipal separate storm sewer system (MS4). In addition, six major outfalls traverse through the project area (three stormwater and three combined sewer overflow outfalls) that drain to the Hudson River. See **Figure 2-2** and **Table 2-2** below for a list of existing outfalls.

Table 2-2. Existing Outfalls

Outfall	Location	DEP ID#
114"x50" box culvert CSO	Stuyvesant Plaza	NCM-073
30" dia. Stormwater Outfall	North Esplanade/Chamber Street	NCM-583
96" dia. CSO	Vesey Street	NCM-072
18" dia. Stormwater Outfall	Albany Street	NCM-584
96" dia. CSO	Rector Place	NCM-071
54" dia. Stormwater Outfall	Rector Place	NCM-628

The Battery Park City sanitary sewer system within the N/W BPCR project area is divided into two areas: the north and south areas. The north sanitary sewer area serves buildings from Brookfield Place to the North Esplanade, discharging via a 30-inch pipe into the existing 72-inch interceptor near the Vesey St. and West St. intersection. The south sanitary sewer area serves buildings from Liberty St. to 1st Place, discharging via a 36-inch pipe into the existing interceptor near the W. Thames St. and West St. intersection. The existing MS4 system collects and conveys stormwater runoff from Battery Park City, discharging it into either the CSO or stormwater outfalls. It is understood that there are no tide gates within Battery Park City that would prevent water from backing up into the MS4 system during tidal conditions.

A portion of the FBS is located along West Street (Route 9A), Harrison Street, and North Moore Street. A portion of the roadway from these areas drain into the project area. These areas are primarily served by a combined sewer system that collects and conveys both sanitary sewer and stormwater. The combined sewer system directs flow to a series of regulator chambers, which divert the flow to the existing interceptor along West Street. Excess flow that cannot enter the interceptor overflows into the CSO outfalls. The combined sewer systems of Harrison Street and North Moore Street join the main combined sewer along West St. and discharge into a regulator further north of the project area. The remaining combined sewer system within the protected area discharges into the three regulator chambers located within the project area. Under dry weather conditions, it discharges from the regulators to the interceptor. However, when the sluice gates in the regulator are closed before an anticipated storm event, the flow overflows to the CSO.

Table 2-3. Existing Regulator Chambers

Regulator	Location	DEP ID#
M4	West St (Northbound lane) near south end of BMCC building	NCM04
M5	West St (Southbound lane) near intersection of Vesey St.	NCM05
M6	West St (Southbound lane) near intersection of Albany St.	NCM06

Each regulator includes a tide gate chamber located downstream along the CSO, which prevents water from entering the combined sewer system during tidal conditions.

Additionally, a series of smaller stormwater outfalls discharge into the Hudson River, serving local drainage areas within Battery Park City as outlined in the table below.

Table 2-4. Existing Stormwater Outfalls

Outfall	Location
12" outfall	West Thames St.
Two - 24" outfall	Gateway Plaza
12" outfall	Along esplanade near Gateway Building

In addition to the outfalls, there are two watercourses located above the north and south PATH tubes that discharge water into the Hudson River. According to record and as-built drawings, these watercourses consist of trapezoidal-shaped channels covered by a platform at the surface. It is understood that the drainage system within the Winter Garden and surrounding areas at Brookfield Place discharges stormwater runoff into the north watercourse. Pumphouse Park and the surrounding area discharges stormwater runoff into the south watercourse.

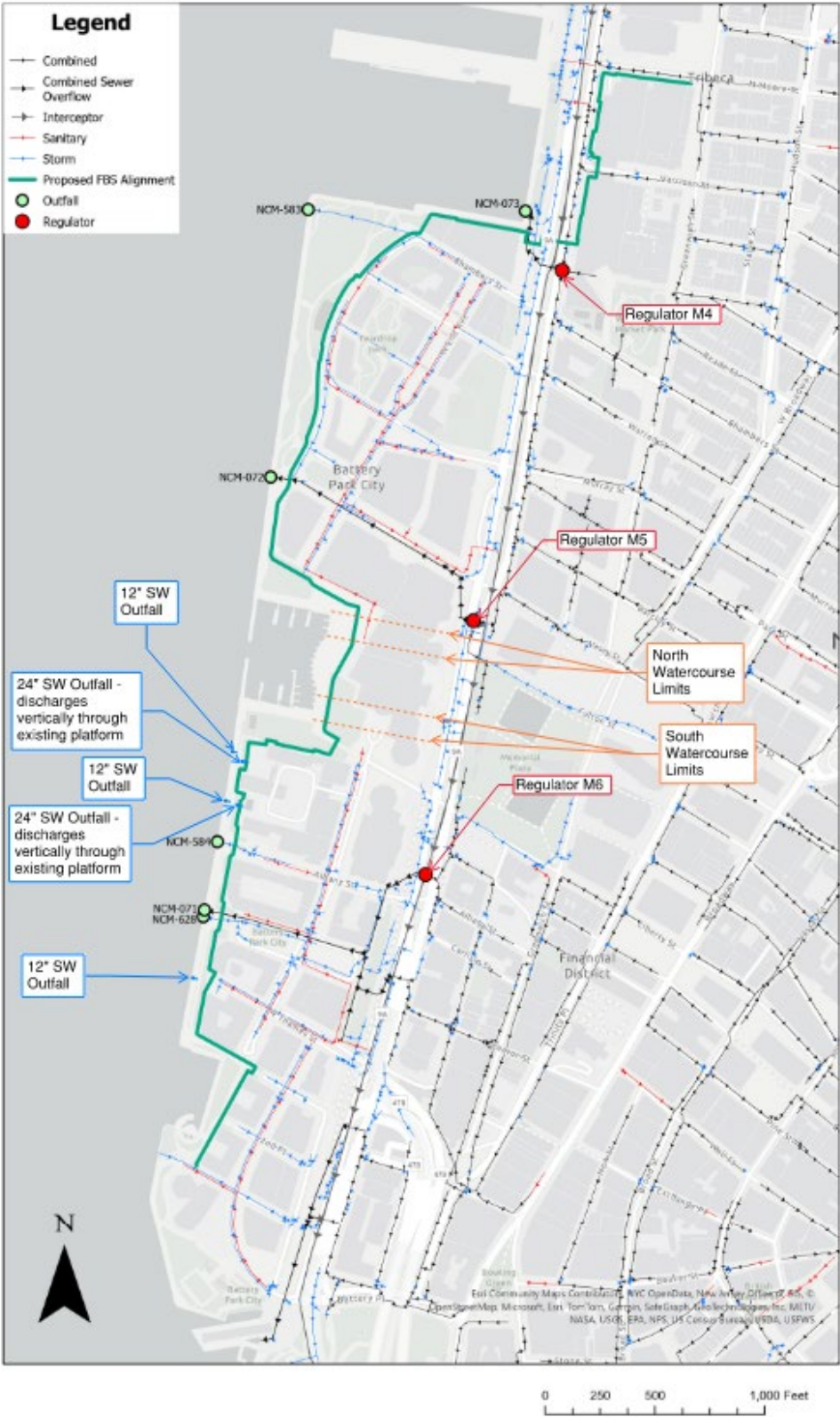


Figure 2-2. Existing Storm and Sewer System

2.2 Conveyance System Modifications

Modifications include reconfiguring the existing drainage infrastructure as part of the NSI system. As depicted on the 90% 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.

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 performed using the H&H modeling software described in described in **Section 3.1** to assess the capacity of proposed sewers in the project area. This analysis was performed for all proposed pipes in the project area, including pipes that will be owned by NYCDEP, NYSDOT, and BPC. The goal of the analysis was to determine whether the proposed sewers have the capacity to adequately convey the flow associated with the 5-year storm event, as required by NYCDEP. Mean Higher High Water (MHHW) levels were applied as a downstream boundary condition for all outfalls in the model.

Sewers are considered to have sufficient capacity if the hydraulic grade line (HGL) is below the inner top of the pipe under design flow conditions. The model results showed whether the pipes had the capacity to accommodate the expected flow from the 5-year storm with MHHW levels. The sewers were found to meet the requirements except for certain cases where the proposed sewers connected to an existing sewer network that was already surcharged, which affected the proposed sewer's ability to effectively convey flow.

The results of this hydraulic analysis can be found in **Appendix A**, which includes maximum HGLs and tables for the proposed sewers and existing sewers downstream. Any constraints in the sewer system identified here will be evaluated in future design iterations.

Maximum HGLs at key points in the sanitary system within BPC were also evaluated. The maximum HGLs for the West Thames and Vesey Street sanitary connections are presented in **Figure 2-3** below for the future design condition and are discussed in more depth in the proceeding section on sanitary flow management. A pump station at West Thames Street prevents pipe surcharging and maintains a low HGL of 3.1' NAVD88, minimizing risk to basement flooding. The gravity flow connection at Vesey Street also helps minimize risks for basement flooding due to pipe surcharges. A peak HGL of 9.3' NAVD88 is modeled for the future design condition at the Vesey Street gravity overflow.

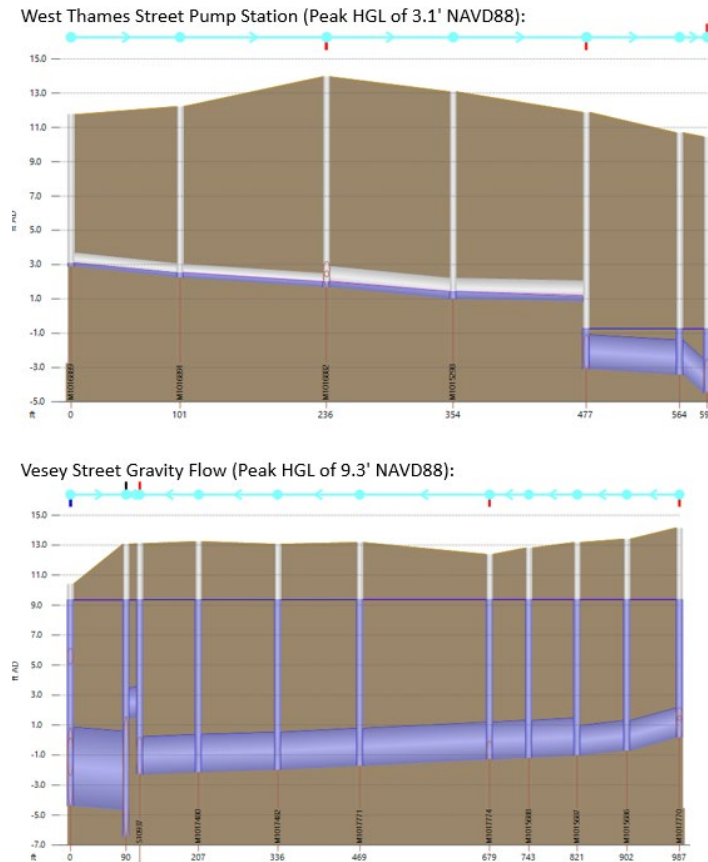


Figure 2-3. Profiles at Sanitary Connection Locations in BPC

2.2.1 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.

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:
 - o Average daily flow: 1.2 MGD
 - o 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

Flow monitoring is currently being conducted at the north and south sanitary sewer location to validate the estimated flows used in the design. Flow monitoring work started in late October 2024. Based on the sewer flow data to date, average sewer flow for the south and north sanitary sewer is 1.3 MGD and 1.0 MGD, respectively. Preliminary results from the flow monitoring can be found in Appendix T Existing Conditions Assessment Report.

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.

2.2.1.1 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 may 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.

2.2.1.2 Alternative 2: Pumping Station

This alternative involves the construction of two small pumping 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 pumping stations, resulting in a higher cost and moderate disturbance. This alternative would discharge combined sewage into the Hudson River during rare, storm surge events.

2.2.1.3 Alternative 3: Onsite Storage

This alternative involves the construction of two holding tanks near the sanitary flow connections to the interceptor, see **Figure 2-4**. 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.

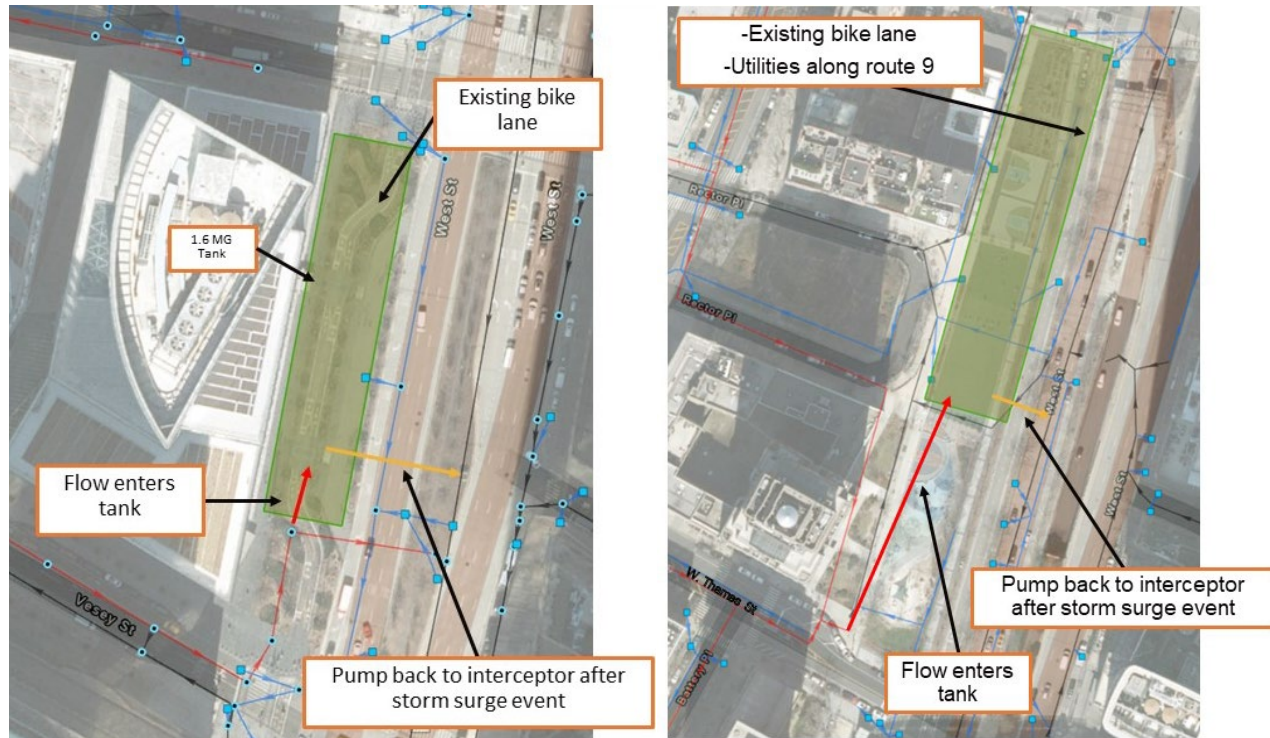


Figure 2-4. Onsite Storage for Vesey St (left) and West Thames St (right)

2.2.1.4 Selected Alternative for Sanitary Sewer Modifications

Based on the input from BPCA and DEP during 60% design, the selected alternatives for the 90% 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 (see **Figure 2-5**). The proposed concept is to redirect sanitary flow from the 30" Sanitary Sewer to the combined sewer network through an overflow structure during the storm surge condition as presented in **Figure 2-6**. See **Section 2.2.1.5** for further details on the gravity flow design for the 90% design.
2. W. Thames Street: Alternative 2: Pumping 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 the storm surge condition as presented under **Figure 2-9**. See **Section 2.1.1.6** for further details on the lift station design for the 90% design.

The PDB team developed conceptual drawings showing the approximate support of excavation limits for the sanitary sewer work in Vesey Street and W. Thames Street, see **Appendix G**.

2.2.1.5 90% Design of the Vesey Street Gravity Flow to CSO

The 90% design for Vesey Street includes a new slide 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 proposed structures are located within the NY-9A right-of-way as shown in **Figure 2-6**.

Under normal operating conditions sanitary flows enter the existing sanitary interceptor located within the NY-9A right of way from Chamber K located northwest of the intersection of Vesey Street and NY-9A. An existing emergency overflow exists in manhole "SS-13" which provides for discharge of sanitary flows to the existing CSO NCM-072. However, under storm surge conditions the proposed tide gate upstream of the CSO and closing of the slide gate in advance of the storm at the existing interceptor sewer under NY-9A will not allow for gravity discharge of sanitary flows. Therefore, a new slide gate chamber is proposed to reroute sanitary flows to the combined sewer network which will discharge to the new NW pumping station. See **Figure 2-5** showing the existing layout of the sanitary sewer system under Vesey Street.

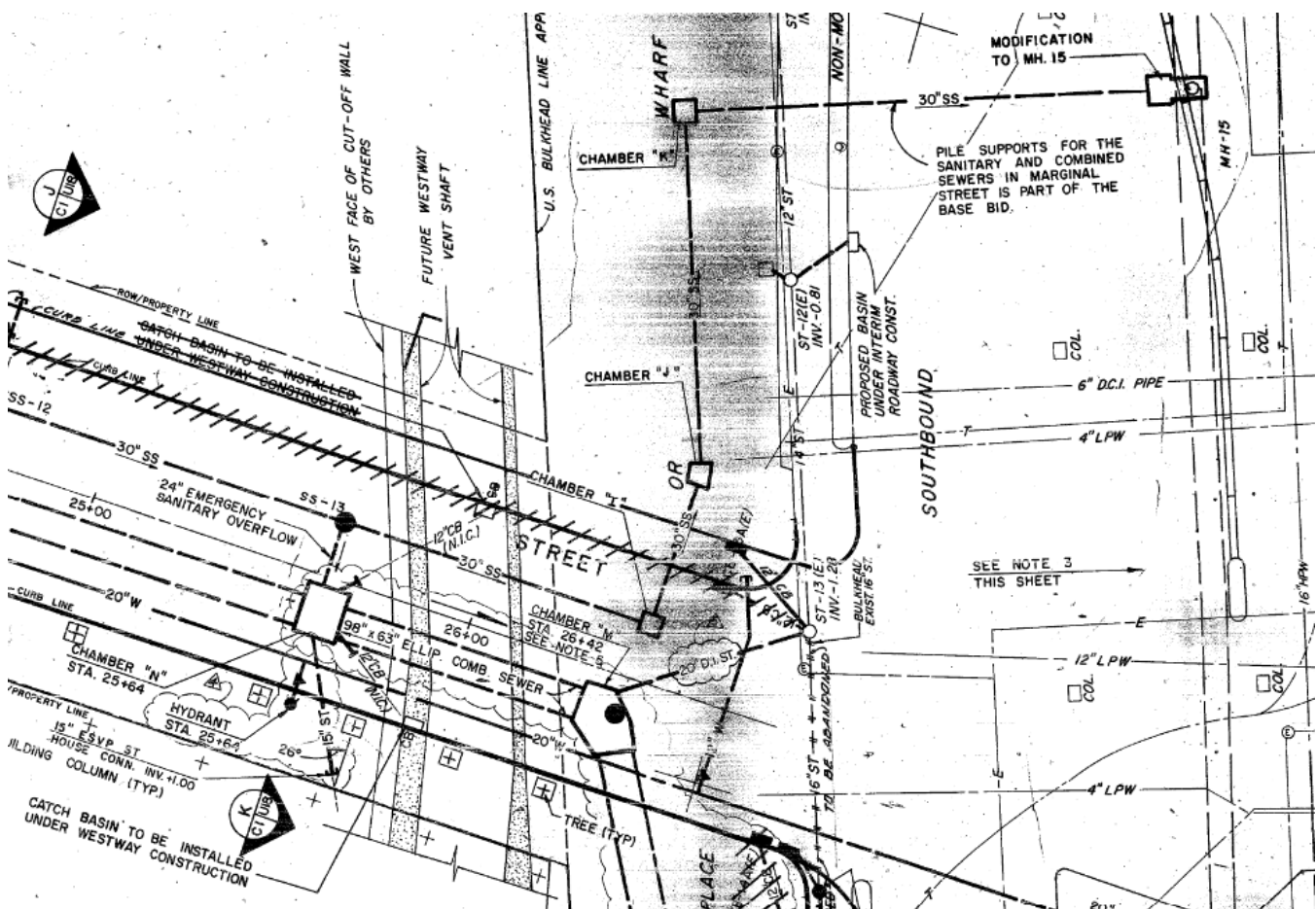


Figure 2-5: Vesey Street: Gravity Flow to CSO

In the proposed configuration, when the interceptor sewer slide gate is closed, sanitary flows will back up in the new gate chamber and overflow into the combined sewer network. It must be noted that the structure includes a manual slide gate controlling the overflow function. The new slide gate must be manually opened in advance of the storm to redirect the sanitary sewer flow to the combined sewer system. The viability of the gravity flow alternative requires the existing drainage system along Route 9A to be upsized to convey the flow to the proposed pump station during a storm surge event.

See **Figure 2-6**. For the full-scale drawings, refer to the 90% design drawings.

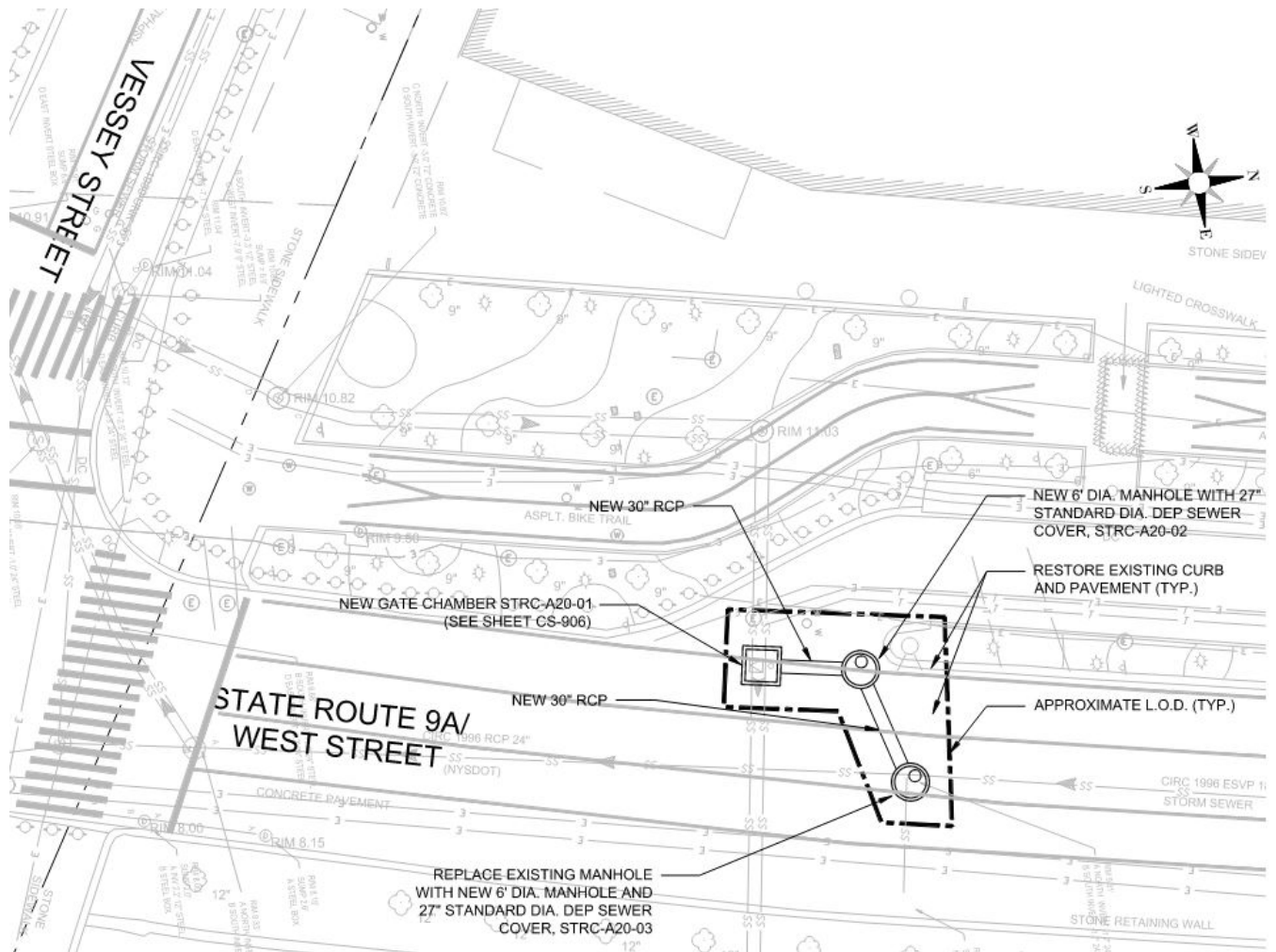


Figure 2-6: Vesey Street: Gravity Flow to CSO

2.2.1.6 90% Design of the West Thames Temporary 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. It must be noted that the structure includes a manual slide gate controlling the overflow function. The gate must be opened before any overflow can occur.

Refer to **Figure 2-7** and **Figure 2-8** below.

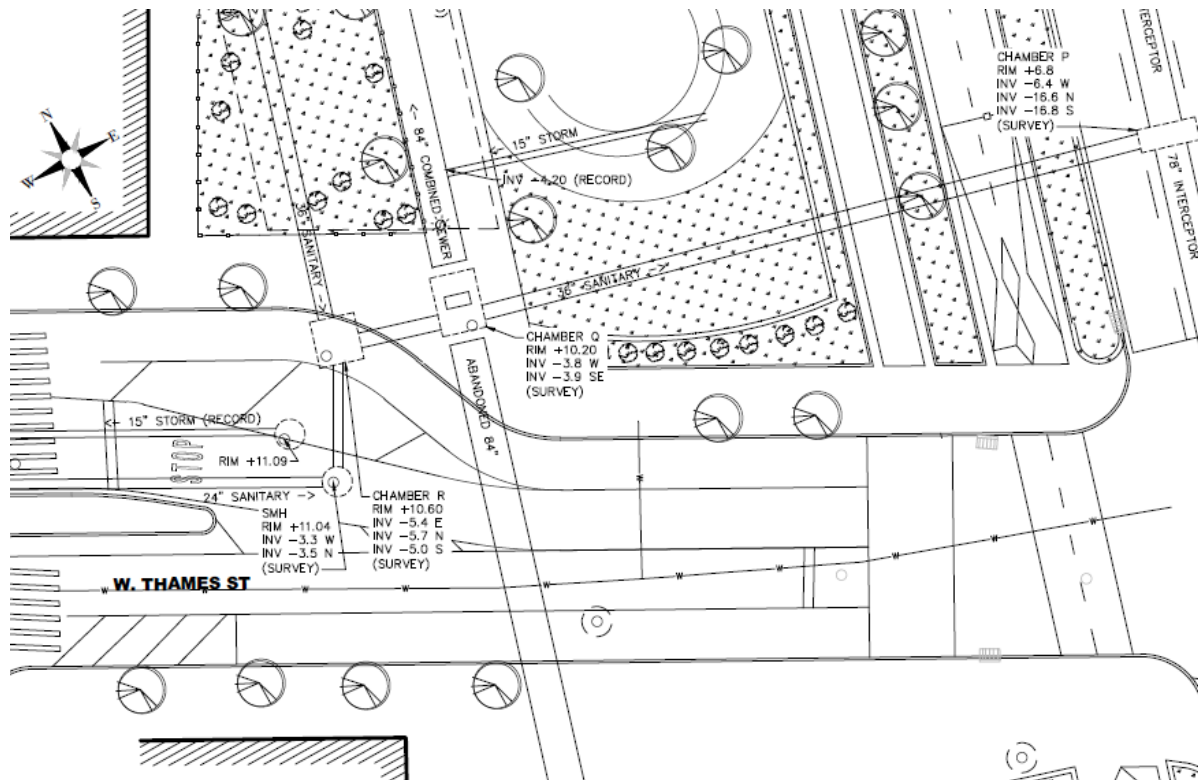


Figure 2-7. Existing Sewer Regulator Chamber Q

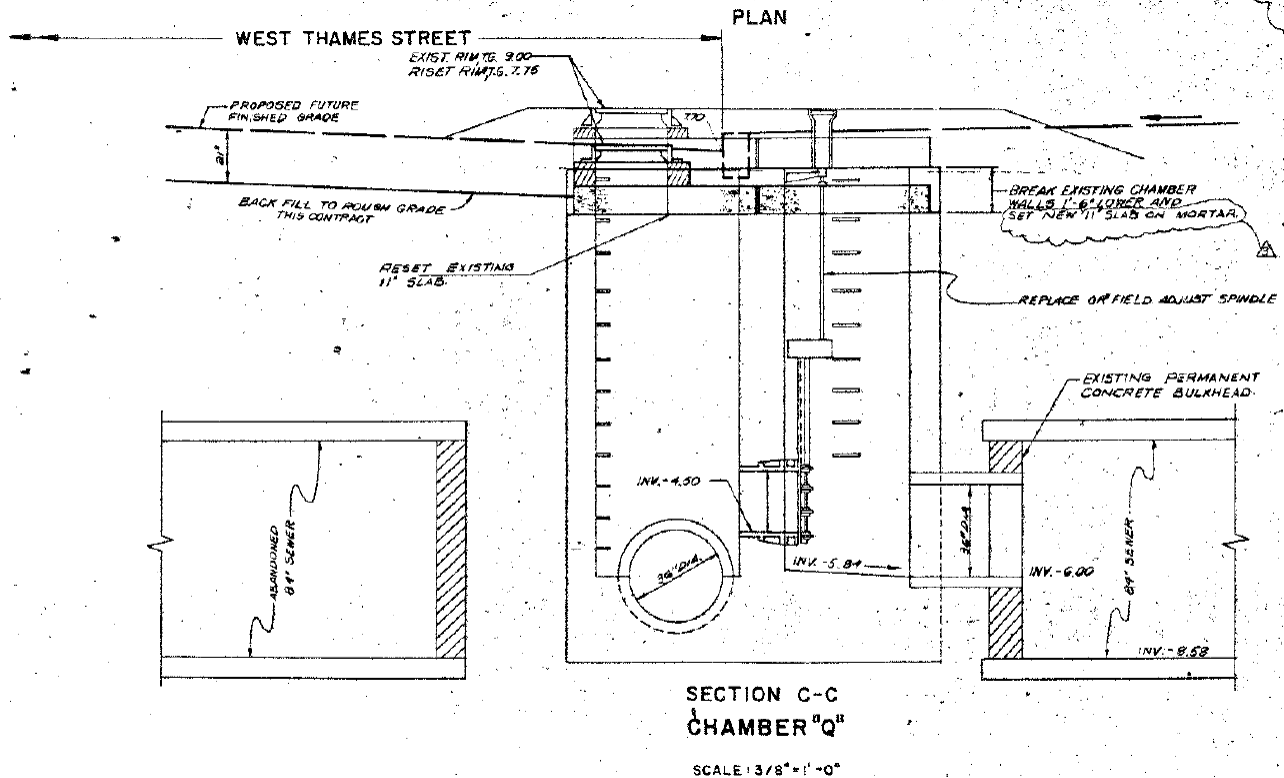


Figure 2-8. Record Drawing of Chamber Q (1982 - BPC South Residential Phase II Infrastructure)

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

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 to prevent a backup of the sewer system. Deployable pumps are proposed as part of the 90% design. For further information on the pumps, refer to **Section 2.2.1.6.7**.

2.2.1.6.1 Recommended Site Layout

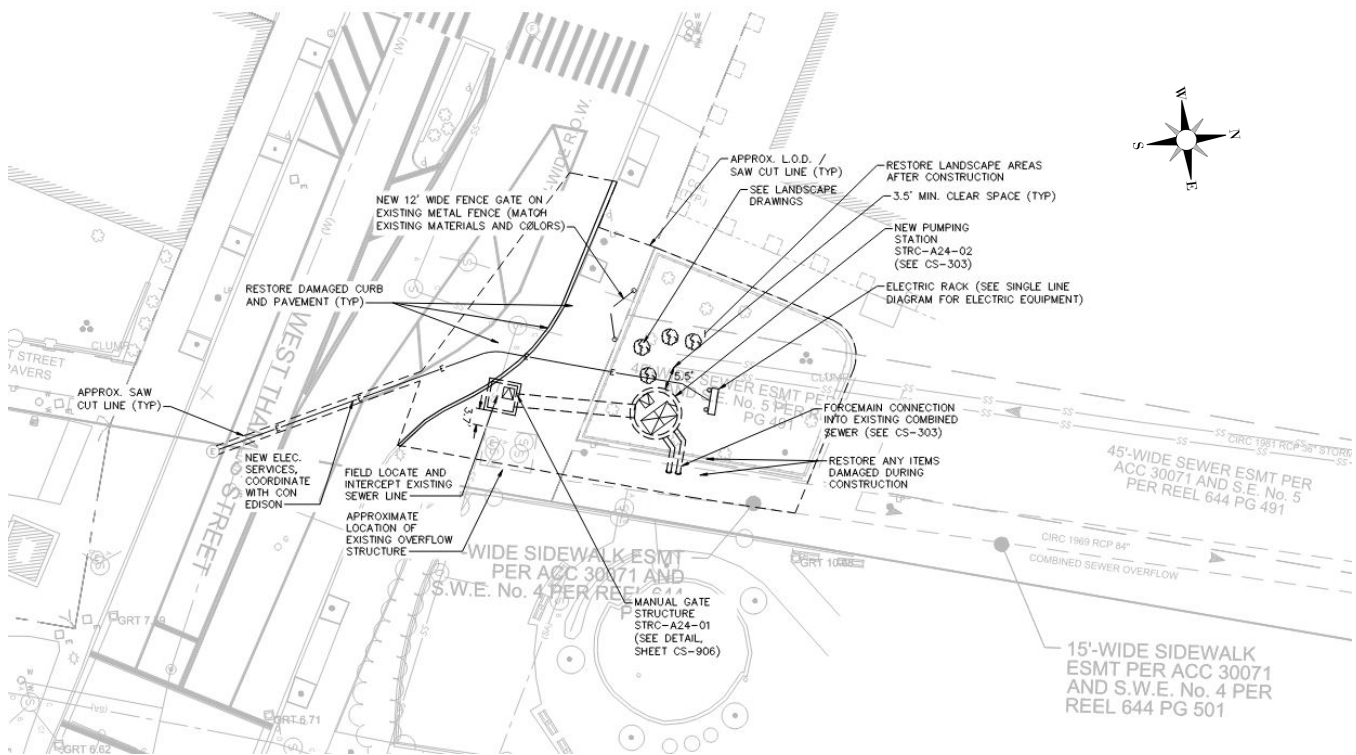


Figure 2-9. Recommended Site Layout of West Thames Pumping Station and Manual Gate Structure

The recommended site layout of the manual gate and pumping station is shown in **Figure 2-9**. Refer to the 90% design drawings for full size drawings.

The layout is based on consideration of the following:

- The proposed pumps will be temporary to allow the pumps to be exercised offsite.
- According to record drawings, the proposed structures and equipment are located within an existing easement, inside a landscaped area, and inside an existing fenced area which appears inaccessible to the general public. Note that prior to 100% design or commencement of construction, the existing easement record documents must be researched and evaluated to determine if the project scope is covered by the easement and to identify the specific requirements that need to be followed. The layout avoids existing trees to the maximum extent practicable, with the goal of preserving existing trees.
- Temporary sheeting and bracing will be required for support of excavations (SOE). The proposed structures are positioned to maximize clearance space to allow space for sheeting between structures and pipes. Specific means and methods will be determined by the contractor.
- Proposed structures and pipes must be supported on piles, in the same manner as how the existing structures are supported.
- The proposed location of the manual gate structure will allow for temporary bypass pumping during construction, from existing Chamber R to Chamber Q.

2.2.1.6.2 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 2-10** below. A small cylindrical valve box on top of the structure will allow access to the valve nut so that the gate can be operated from the surface level using a valve key or similar tool. There is also a separate access opening to allow for inspection and maintenance inside the structure. A corrosion resistant ladder and safety pole-up device are provided for maintenance access or inspection purposes. The slide gate will be mounted on a steel “thimble” provided by the gate manufacturer. The thimble must be provided to the precast manufacturer who will cast the thimble into the structure during precast production. The outside face of the thimble will accept a mechanical joint connection from the inbound sewer pipe.

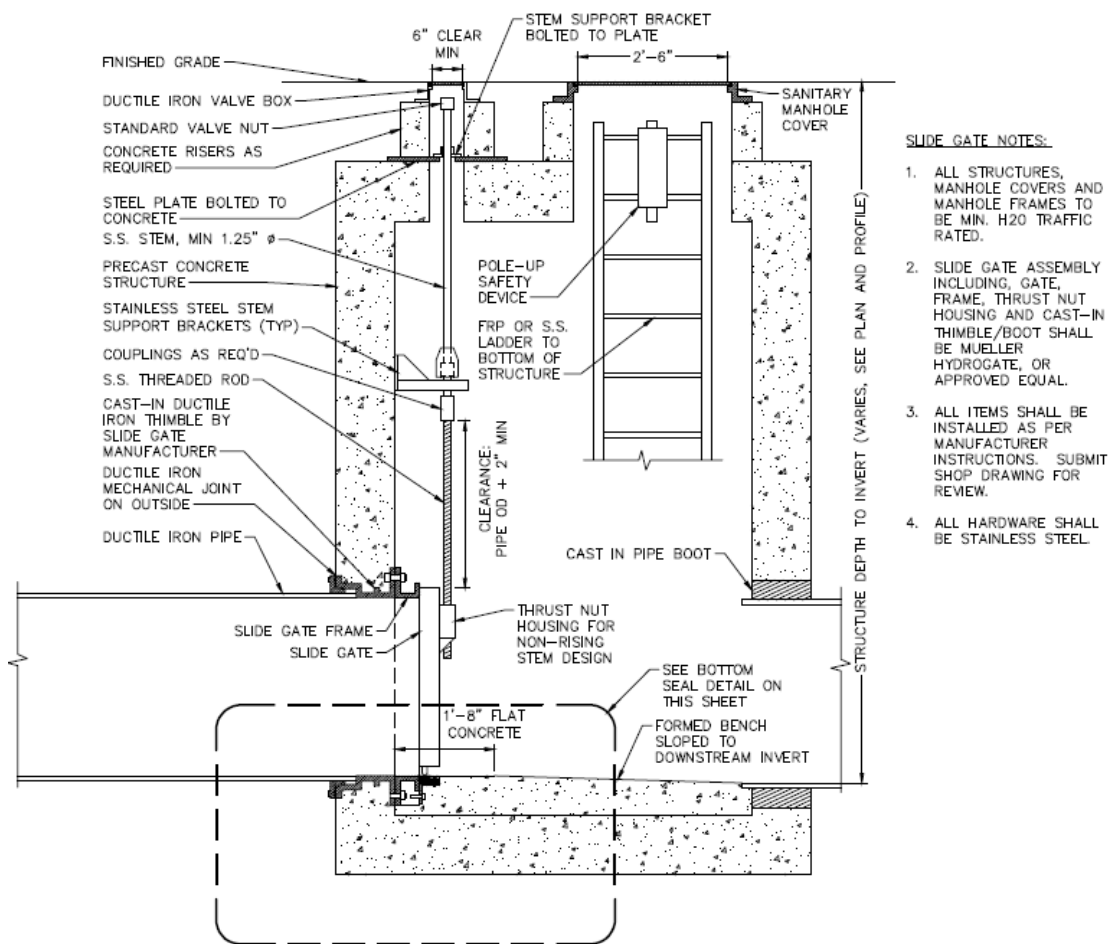


Figure 2-10. Proposed New Manual Gate Structure

2.2.1.6.3 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. The device will operate by releasing wastewater back into the wet well if pressures exceed the set point of the device.

Regarding the set point of the pressure relief valve, there are three (3) controlling considerations

- The pressure capacity of the downstream RCP pipe joints (typically 10 psi max per ASTM C443)
- The condition of the downstream RCP pipe (sound RCP condition shall be field verified by CCTV inspection prior to setting pressure limit.)
- The allowable HGL at the connection point (determined by the latest revision of the hydrologic model, see Appendix A)

Note that the allowable pressure measured at the valve is larger than the allowable pressure measured at the connection point, given that there is friction head-loss between the valve and the point of connection downstream.

2.2.1.6.4 Design Flow

The average daily flow for the tributary area of the pumping station was calculated as described in **Section 2.2.1**.

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 $910,000 \text{ gpd} / 100 \text{ gpcd} / 1000 = 9.1$

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

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

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

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

Requirements for Further Design:

Design flow assumptions to be further verified once information from flow monitoring is received. The pump selection will be adjusted, if necessary, based on the finalized flow calculations.

2.2.1.6.5 Gravity Pipe Design

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

Refer to **Figure 2-9** 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 suit 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 2-10** for additional information about the gate structure).

2.2.1.6.6 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. In addition, flow into the pumping station can be stopped using the upstream manual gate structure.

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 interconnect, 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 given that flow to the pump station is controlled by a manual gate, 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.

2.2.1.6.7 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 2-11**.

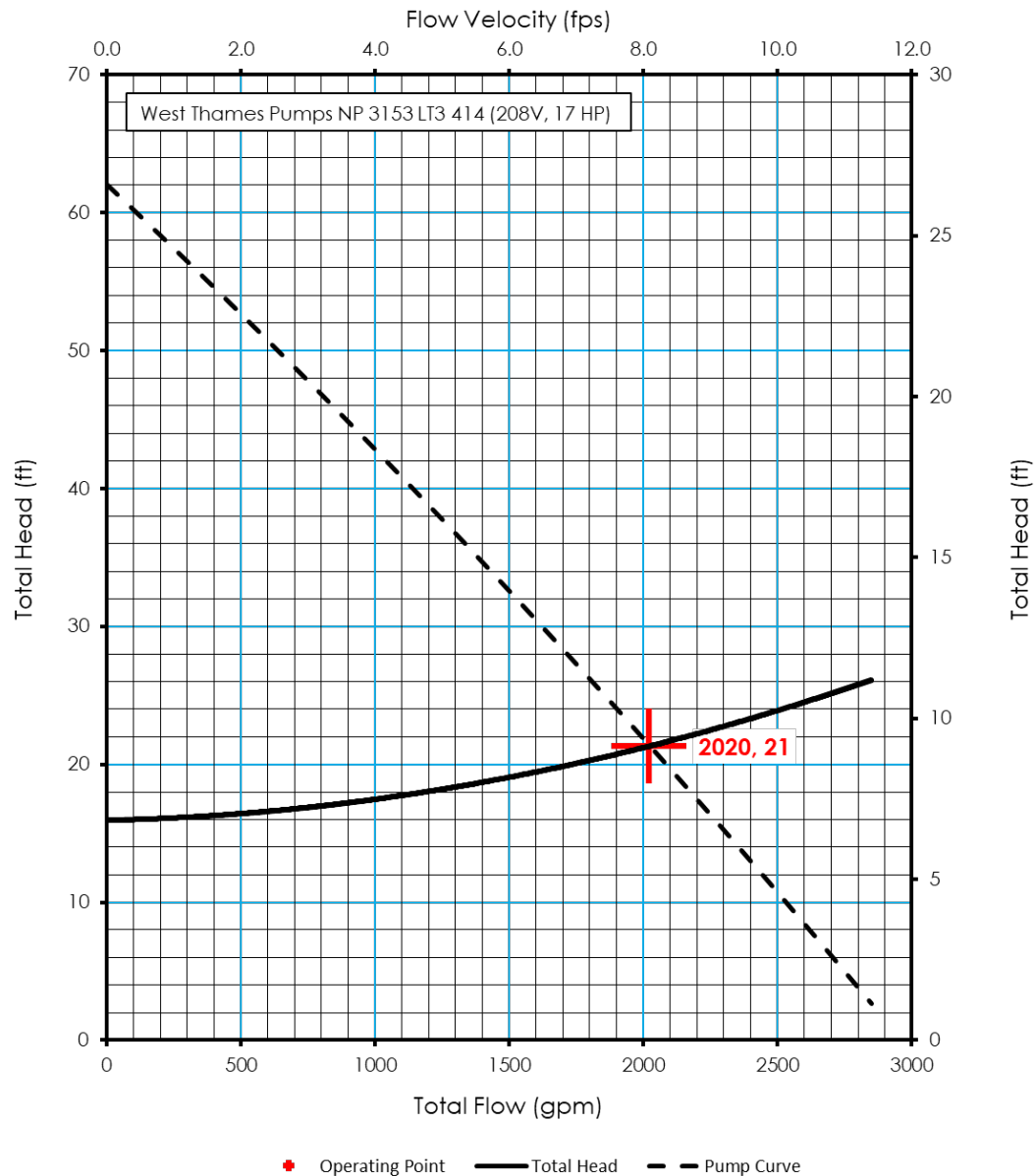


Figure 2-11. West Thames Pump Curve

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

System and pump 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 pump selection is the following:

- Flygt NP 3153 LT 3 414 or approved equal (one duty pump+one standby pump)
- 17HP, 208V, 3 phase, 1800 rpm
- Operating Point 2020 gpm @ 21' TDH
- 705 lbs pump weight
- Flow velocity in 10" force main = ~8 fps in one pipe

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.

2.2.1.6.8 Wet Well Design

Conventionally, wet wells are sized by engineers using “rules of thumb” provided in various reference books to determine the required “effective depth”. For example, a common approximation is $V = T \cdot Q_{\text{peak}}/4$. Where T is cycle time in minutes and Q_{peak} is peak flow. Note that “ $Q_{\text{peak}}/4$ ” 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 of a wet well 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 and schedule savings due to the difficulty of performing cast-in-place work in space constrained, 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 minimum structure size is determined by the allowable 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 calculated values. 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:

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

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).

2.2.1.6.9 Electric Design

The following are the design team's electrical design decisions and assumptions at the 90% milestone:

- 208V 3 phase electric service is available per confirmation by Con Edison.
- Electric is sized for the temporary pumps with one duty pump.
- Pumps are connected to an electric terminal box using a quick connect plug at end of the pump cable.
- 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. This switch is manual because connection of the portable generator is a manual operation that would require staff to be on-site.

2.2.1.6.10 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. For the permanent pump alternative, two pumps are installed inside the wet well at all times.
- 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 dialled out as it is assumed that the operator of the temporary pumps will be on-site during pump usage, or otherwise that an operator will be on site to open the manual gate structure. An "alarm test" button will be provided on the panel to simulate an alarm condition for testing or training purposes.

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 NYCDEP Bureau of Wastewater Treatment (BWT) standards. One consideration is that operations staff will be on-site to open the manual gate structure, connect a temporary generator, or to install the temporary pumps. In such a condition, with on-site staff present, it would be apparent whether one of the pumps failed or not. Additional spare temporary pumps may also be stored off-site to achieve the desired level of redundancy.

2.2.1.6.11 Access and Security

The following are the design team's access and security assumptions at the 90% milestone:

- The pump station and appurtenances will be enclosed inside the existing fencing surrounding the landscaped area, and not accessible to the public. A fence gate will be added to the existing fencing as required to allow maintenance access. The appearance of the gate will match that of the existing fence as close as practicable.

- 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 for 100% design).
- evergreen screen trees or shrubs will be provided as appropriate to screen equipment from view.
- 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 disconnect switches will not be provided as the control panel will be located very close to the pumps.
- Hatches on utility structures will be flush at grade and lockable.

2.3 Outfall Modifications

Outfall modifications will focus on backflow prevention as part of the NSI approach. As depicted in the 90% 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 90% interior drainage plans. The 90% plans also provide details for the new tide gate structures. **Appendix E** includes calculations for the tide gates and all related components.

Table 2-4. Proposed Tide Gates at Existing Outfalls

Outfall ID	Tide Gate #	Approximate Location	Proposed Tide Gate Size
NCM-073	TG6	Stuyvesant Plaza (Relocated CSO and tide gate)	60" x 60" (2)
NCM-583	TG5	Northwest End of Rockefeller Park (River Terrace)	30" x 30"
NCM-072	TG4	Under Ferry Terminal (Vesey Street)	60" x 60" (2)
-	TG3	South of Gateway Plaza	24" dia.
-	TG7	North of Gateway Plaza	24" dia.
NCM-584	TG2	West of Albany Street	18" dia.
NCM-071	TG1	West of Rector Place	60" x 60" (2)

2.4 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:

WQv: water quality volume (ft³)

A: contributing area (ft²)

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 53,700 ft³. 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 (SF)	Proposed Pervious Area (SF)	Proposed Impervious Area (SF)	% Impervious Cover, I	Runoff Coefficient, R _v	Water Quality Volume, WQv (CF)
1	138,850	15,690	123,160	88.7	0.85	14,800
2	35,260	14,920	20,350	57.7	0.57	2,600
3	37,610	6,900	30,710	81.7	0.78	3,700
4	55,400	20,350	35,050	63.3	0.62	4,300
5	180,170	48,000	132,180	73.4	0.71	16,000
6	107,670	42,390	65,280	60.6	0.60	8,100
7	57,200	20,830	36,380	63.6	0.62	4,500
Total	612,160	169,080	443,110	72.4	0.70	53,700

2.4.1 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 comeingle 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 ft², 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 90% 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 69,340 ft³ or 129% of the total WQv (53,700 ft³) for the project. **Table 2-6** summarizes the WQv managed by each feature. **Figure 2-12** 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)
<u>Reach 1</u>	HDS116-1	HDS	3,650
	HDS117-1	HDS	730
<u>Reach 2</u>	HDS114-1	HDS	6,140
<u>Reach 3</u>	RGHP112-1	High Performance Rain Garden	800
	RGHP113-1	High Performance Rain Garden	310
	RGHP113-2	High Performance Rain Garden	980
	RGHP113-3	High Performance Rain Garden	1,060
<u>Reach 4</u>	HDS110-1	HDS	27,920
	PP110-1	Permeable Pavers	190
	PP110-3	Permeable Pavers	260
<u>Reach 5</u>	HDS109-1	HDS	2,350
	PP107-1	Permeable Pavers	810
	PP107-2	Permeable Pavers	240
	PP107-3	Permeable Pavers	240
	PP108-1	Permeable Pavers	1,240
	PP108-2	Permeable Pavers	1,150
	PP108-3	Permeable Pavers	670
	PP108-4	Permeable Pavers	2,040
	PP109-1	Permeable Pavers	350
	PP109-2	Permeable Pavers	220

North/West Battery Park City Resiliency Project – Interior Drainage Report

Reach	Practice Name	Practice Type	WQv Managed (ft ³)
	PP109-3	Permeable Pavers	530
<u>Reach 6</u>	HDS103-1	HDS	1,380
	HDS104-1	HDS	3,790
	HDS104-2	HDS	3,740
	RGET103-1	Rain Garden	370
	RGET103-2	Rain Garden	520
	RGET104-1	Rain Garden	430
	RGET104-2	Rain Garden	270
	RGET104-3	Rain Garden	320
	RGET105-1	Rain Garden	810
<u>Reach 7</u>	HDS102-1	HDS	1,300
	HDS102-2	HDS	1,580
	RG101-1	Rain Garden	370
	RG101-2	Rain Garden	260
	RG101-3	Rain Garden	1,310
	RG102-1	Rain Garden	900
	PP101-1	Permeable Pavers	60
	PP101-2	Permeable Pavers	50
<u>Total</u>			69,340



Figure 2-12. 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 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 90% design drawings.

2.5 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.

2.6 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 the sanitary sewer structure where the BPC sanitary sewer connects to the existing interceptor near Vesey Street as shown in **Figure 2-1**.

Three new slide gate structures are being proposed within Reach 1, one along Harrison Street at the intersection of West Street, one further south along the northbound lane of West Street, and one along the southbound lane of Route 9A. The three 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 in the northbound lane of West Street 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 slide gate being proposed in the southbound lane of West Street is connected to the existing 24" sewer. See **Figure 2-1** for locations of slide gates.

The purpose of the three slide gate structures is to prevent floodwaters from entering the protected area during the storm surge. During the gate closure period, combined sewer flows in Harrison Street will be directed to the slide gate in the northbound lane of West Street via an overflow pipe and will eventually flow to the pump station. The slide gate in the northbound lane of 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. Similarly, the slide gate in the southbound lane of West Street will be closed to prevent flows from the wet side entering the structure.

2.7 Tide Gate Modifications and CSO relocation

Seven 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 90% 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 the CSO is being relocated, not the outfall. The proposed relocation of the existing CSO and tide gate structure is depicted on the 90% design drawings.

For the tide gates in Chambers Street, Albany Street, Vesey Street, Rector Place, and Stuyvesant Plaza (along with the CSO relocation), the PDB team has developed conceptual drawings depicting the approximate limits for the support of excavation. Refer to **Appendix G**.

The construction of the tide gate in Vesey St as part of Early Work Package #5 will require temporary drainage. Refer to **Appendix H**.

2.8 Pile Support Design

Rock-socketed and soil-bonded micropiles were considered for the majority of proposed interior drainage structure foundations at the project site. Due to the highly variable nature of the uncontrolled fills in terms of density and composition, that in most areas extend down to top of rock, and to meet all design requirements, the interior drainage structure foundations were designed as rock socketed piles within the limits of Battery Park City (Reaches 2 to 7). Outside of Battery Park City (in Reach 1), the piles were designed as soil-bonded micropiles based on the subsurface conditions encountered, especially considering the reported depths to top of rock. See **Appendix F** for detailed micropiles design calculations.

Rock-socketed micropiles consist of 13.375-inch casing outside diameter that extends through the overburden and a minimum 6 inches into NYCBC Class 1c rock or better (for seating the casing), a minimum rock socket diameter of 11-inches, Type A method of grouting (grout is placed under gravity head only), and a minimum required rock socket length of 5 feet into NYCBC Class 1c rock or better.

In consideration of the anticipated loads on structures in Reach 1, two (2) separate soil-bonded micropiles will be utilized. The first soil-bonded micropile is for NYCDEP standard structures (pipes, manholes, etc.) and assume an allowable capacity of 20 tons as indicated on the NYCDEP Standard Details. The 20-ton micropiles for NYCDEP standard structures were designed to satisfy the exceptions of NYCBC Section 1810.3.1 and so do not require load testing during construction. The 20-ton micropiles consist of 13.375-inch casing outside diameter that extends 30 to 50 feet below existing site grade to extend below uncontrolled fill and marsh (organic silt and clay) deposits, a minimum 30-foot pressure grouted soil bond length below the bottom of the casing, and Type B method of grouting (grout is placed into the hole under pressure – the hole can be unlined or a temporary drill casing may be withdrawn).

The second soil-bonded micropile is for non-standard NYCDEP structures (tide gates, slide gates, etc.) that will require a larger allowable load capacity. These micropiles consist of 13.375-inch casing outside diameter that extends 30 to 50 feet below existing site grade to extend below uncontrolled fill and marsh (organic silt and clay) deposits, a minimum 50 feet pressure grouted soil bond length below bottom of casing, and Type B method of grouting (grout is placed into the hole under pressure as the temporary drill casing is withdrawn).

Injection pressures typically range from 72 to 145 psi per FHWA NHI-05-039 and should meet the NYCBC 2022 Section 1812.8.6 criteria. For the soil-bonded micropiles in Reach 1, the length of the upper-cased portion of the micropiles was selected to increase the pile moment capacity and lateral capacity and those demands primarily determine the cased length. In addition, the cased length was also selected to extend through looser fill material and pockets of softer cohesive material encountered below ground. Except for micropiles in Reach 1, the micropiles derive their geotechnical capacity primarily from the rock sockets and are considered to be "Mini-caissons" for which

the provisions of Section 1812.7 Caisson Piles in the NYCBC 2022 apply. Micropiles in Reach 1 derive their geotechnical capacity from the grout-to-soil bond conforming to provisions of NYCBC 2022 Section 1812.8.

The allowable geotechnical rock socket capacities for rock-socketed micropiles were determined based on an allowable rock socket bond strength of 200 psi from Section 1812.7.3 of NYCBC 2022, assuming the rock sockets are installed in Class 1c rock or better as defined in Section 1803.3 of NYCBC 2022. The majority of the rock cores taken along the project site show Class 1b or Class 1a rock.

The allowable geotechnical capacities for larger capacity soil-bonded micropiles were determined based on an ultimate grout-to-ground bond strength of 22 psi from Table 5-3 in FHWA NHI-05-039 assuming fine to coarse medium dense sand with some silt and gravel. This assumed bond strength is based on geotechnical field and lab testing data at the soil bond zone area, approximately El. -25 feet to -75 feet. The allowable geotechnical capacities for the standard NYCDEP structure soil-bonded micropiles were determined based on an allowable grout-to-ground bond strength of 4 psi in accordance with Exception 4.1 of NYCBC Section 1810.3.1 and assumes the bond zone is formed entirely in NYCBC Class 3b or better soils. The resulting estimated axial capacities, anticipated range of top of rock elevations, suggested nominal top of rock elevation (for estimating purposes and not for construction), casing lengths below existing site grade, minimum rock socket lengths, and minimum soil bond lengths for various segments of the interior drainage structure foundations are summarized on sheets BP-004 through BP-006.

The allowable compressive capacity and uplift capacity for the higher capacity 13.375-inch O.D. soil bonded micropiles with 0.514-inch wall thickness and 80 ksi yield strength are 128 tons and 85 tons, respectively. A minimum 50-foot bond length in soil is recommended for the higher capacity 13.375-inch O.D. soil bonded micropiles. The allowable compressive capacity and uplift capacity for the standard NYCDEP structure 13.375-inch O.D. soil bonded micropiles with 0.514-inch wall thickness and 80 ksi yield strength are 20 tons and 20 tons, respectively. A minimum 30-foot bond length in soil is recommended for the standard NYCDEP structure 13.375-inch O.D. soil bonded micropiles within Reach 1. The allowable compressive capacity and uplift capacity for 13.375-inch O.D. rock socketed micropile with 0.514-inch wall thickness and 80 ksi yield strength are 135 tons and 90 tons, respectively. A minimum 5-foot bond length into a Class 1c bedrock or better is recommended for 13.375" O.D. rock socketed micropiles. The required minimum rock socket length is based on Section 1812.7.3 of the 2022 NYCBC that states the depth of the rock socket in Class 1c rock or better below the bottom of the casing shall not be less than 3 feet or the outside diameter of the casing, and also to accommodate uncertainty in the rock quality video inspection at all the proposed rock-socketed micropile locations. For constructability and cost considerations, the same micropile size is used for the project site for all interior drainage structures that are to be supported on micropiles. Optimization of the pile layouts and pile spacing has been underway and incorporated into this design submission. Optimization of the pile layout is based on the allowable compressive and uplift capacities of the micropiles as well as other factors such as the pile deflection limits, structural capacity of the piles, site space constraints, and existing site conditions.

Loss of thickness due to corrosion is in accordance with the recommended values in the Project Technical Criteria. It is our understanding from our review of the deep foundation designs by project geotechnical engineer (WSP) that there are no substantial grade changes that would result in settlements and/or downdrag on the piles.

2.9 Other Modifications

2.9.1 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 also be 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 2-13** for a diagram showing the two options.

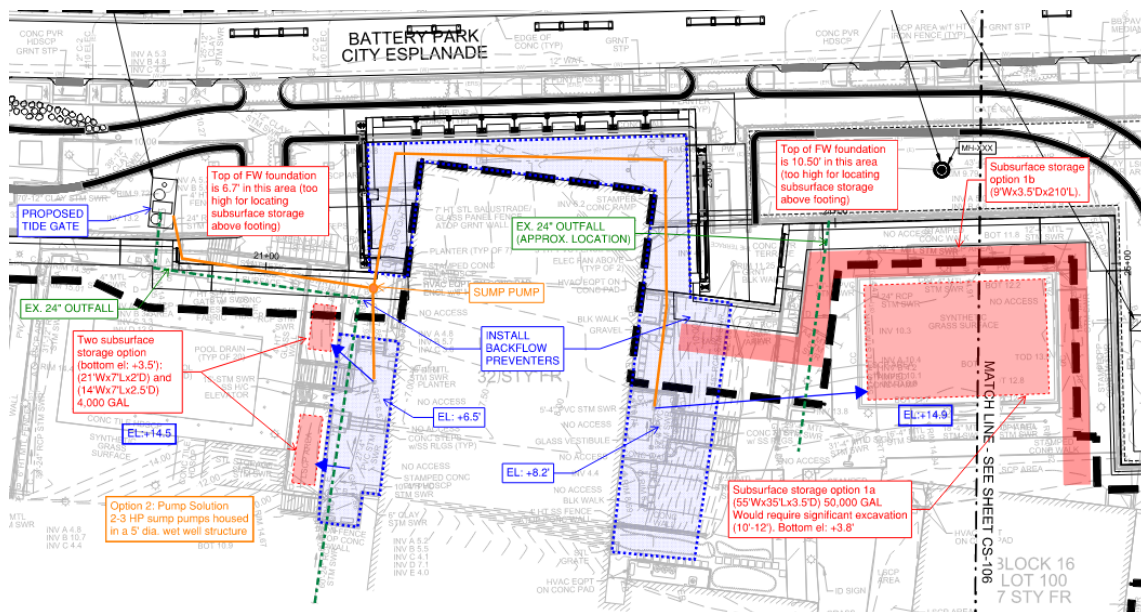


Figure 2-13. Gateway Flood Mitigation Options

To meet the goal of limiting flooding within the protective barrier, the second option was selected during 90% 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 2-14** below. Refer to **Section 2.9.1.1** for further details on the design.

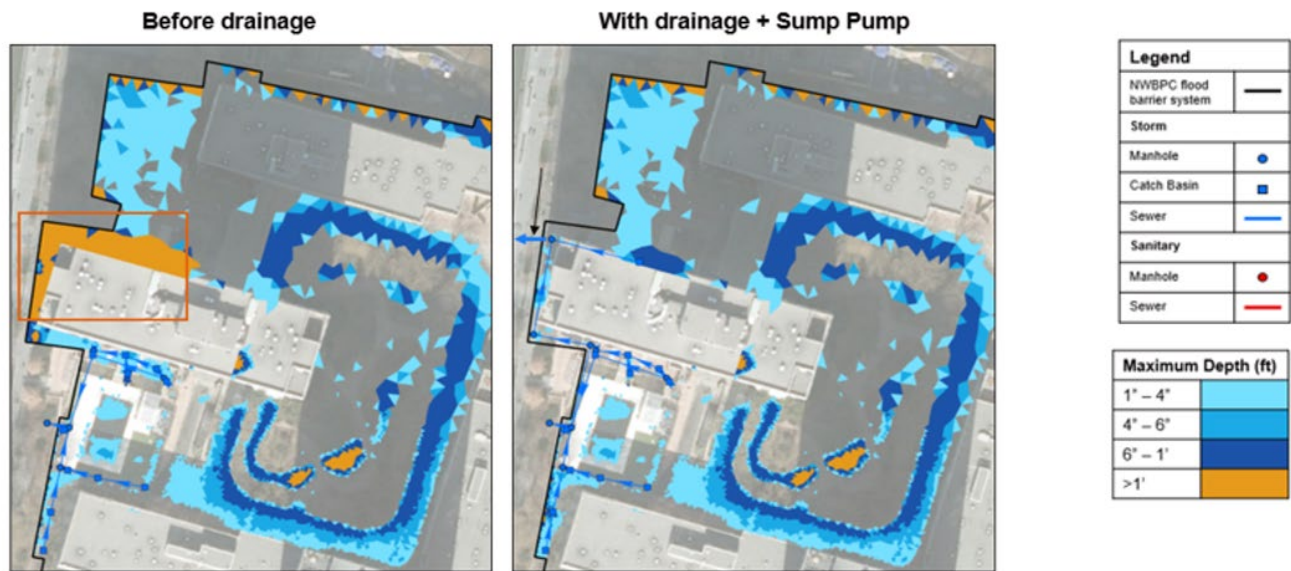


Figure 2-14: Gateway Plaza without modification to drainage vs Gateway Plaza with modification to drainage and a sump pump

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 90% design drawings for its location. North of the middle building, another 24” outfall was found during the CCTV pipe investigation. A tide gate chamber is proposed for that outfall as well and is incorporated in the 90% design.

2.9.1.1 90% Design of Gateway Plaza Sump Pump

2.9.1.1.1 Design Flow

The following are the design team’s flow assumptions at the 90% milestone:

The stormwater design flow was determined by the H&H modeling 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 after 90%.

2.9.1.1.2 Gravity Pipe Design

The 90% plans include drain basin structures to direct flow around the building from the North side to the South. CCTV inspection is still under review to verify hydraulic connectivity between structures within the patio areas.

2.9.1.1.3 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 enter the sidewall of the tide gate structure downstream of

the tide gate mechanism. A Link-Seal, or equal, will be utilized to maintain water tightness of the penetration and stabilize the pipe within the penetration.

2.9.1.1.4 Pump Design

The following are the design team's pump design assumptions at the 90% 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 N80-450 or approved equal (one duty, one standby)
- 3HP, 480V, 3 phase, 800-2095 rpm
- Operating Point 50 gpm @ 11' 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.

2.9.1.1.5 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.

2.9.1.1.6 Electric Design

The following are the design team's electric assumptions at the 90% milestone:

- 480V 3 phase electric service at the Gateway Plaza building, based on a site visit conducted at the Gateway Plaza sump pump location and conversations with building maintenance personnel.

- 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 90%)
- Power will be supplied from the building electrical room (location and conduit routing TBD after 90%)

Requirements for Further Design:

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.

2.9.1.1.7 Controls

The following are the design team's controls assumptions at the 90% 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 90%)
- 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 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.

2.9.1.1.8 Access and Security

The following are the design team's access and security assumptions at the 90% 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.

2.9.2 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 90% 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. A portion of the existing discharge pipe will be replaced, and it will cross the floodwall on the west end of the lot and tie into a manhole on the flood side of the floodwall, where the overflow pipe would then direct flow to the south. The overflow pipe would continue 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 combined sewer pipe on the protected side. The flow exiting

this structure would be managed by the pump station. See **Figure 2-15** for a schematic diagram of the overflow piping at the BMCC parking lot. Refer to the 90% design drawings for the latest configuration of the proposed piping and manholes.

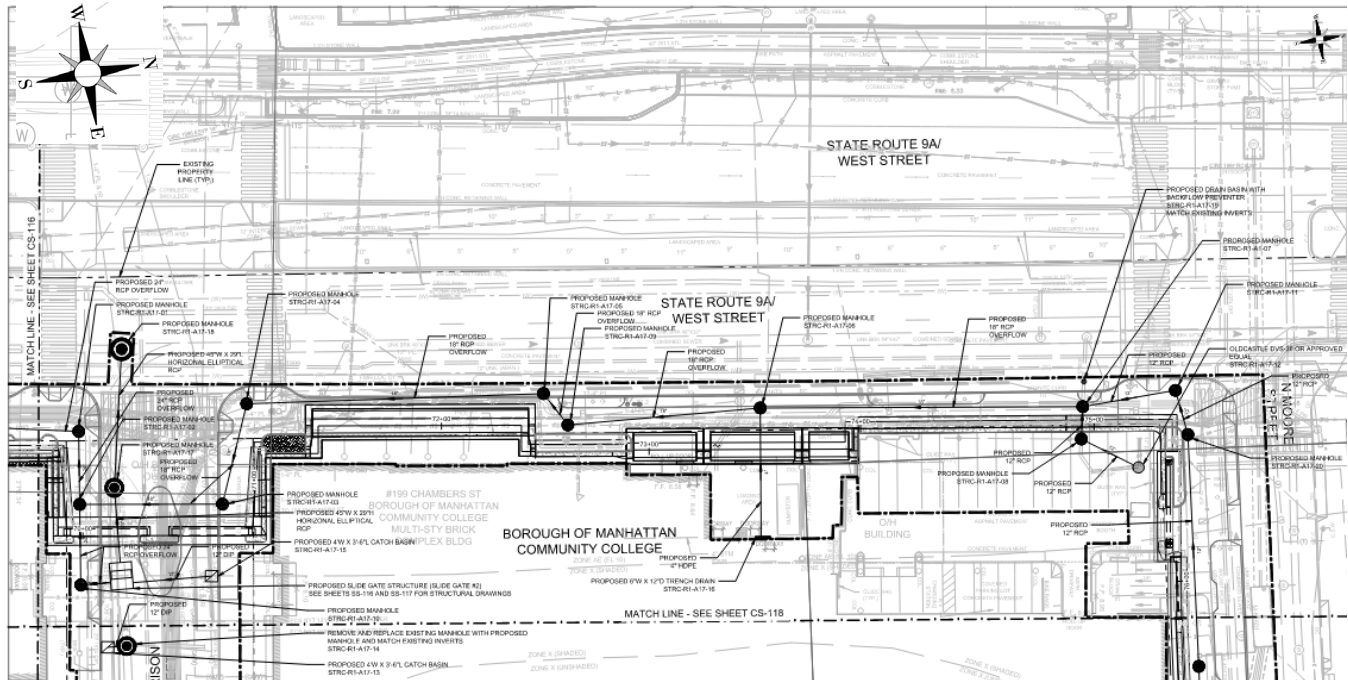


Figure 2-15. Overflow Pipe at BMCC Parking Lot

2.10 Erosion & Sedimentation Control

The NWPBPCR project requires effectively managing and mitigating the potential environmental impacts associated with soil disturbance during construction activities. With a solid 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 environmental 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 established, such as applying water to surfaces prone to dust generation, using dust suppressants, and applying covers to stockpiles of materials.

3. Soil Stabilization Measures:

To minimize soil erosion, stabilization techniques will be used. 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 90% 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 (Autodesk) 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.
Surface Elevation Data	New York City 2017 digital elevation model	Used to build a 2D flooding surface, or mesh, in the model. The mesh consists of finite,

Data Type	Source	Purpose
	(DEM)/Topobathymetric LiDAR from NYC Open Data ² Survey data (see above)	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, 50 th 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.

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 standards⁵. The LTCP model was built to assess the amount of water discharged into waterbodies through CSO outfalls. It was updated through the following steps:

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/>

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

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.
3. A rain-on-mesh (ROM) model was constructed within the project area, while a 1D2D model was utilized in nearby areas of lower Manhattan outside of the project area. See section **Section 3.5** for more detail on ROM Methodology.

Under this initial effort, no flow monitoring data was collected in the study area to use in validation of the model. Flow monitoring data is currently being collected as of November 2024. Insights from flow monitoring will be included in the next version of the Interior Drainage Report.

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 changed based on the latest design plans.
- PCSMPs identified to meet water quality requirements were not incorporated into the model.
- CCTV data was reviewed and incorporated in the model build-out.
- 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.

3.5 Rain-On-Mesh (ROM) Methodology

The model developed in Infoworks ICM uses a 1D2D base model from Financial District/Seaport Coastal Resiliency (FiDi), which extends throughout the Newtown Creek Manhattan (NCM) sewershed. The 2D area is limited to lower Manhattan. In this region, sewers loaded by hydrologic parameters may exceed capacity and flow may spill from manholes onto a surface created by a DEM. Within NWBPCR project extents, a rain-on-mesh model was constructed to help identify localized flooding and drainage issues that may prevent stormwater from reaching catch basins. In this area, rainfall is applied directly to a 2D mesh and flows overland to reach the sewer network rather than loading sewers with catchments.

In the ROM region, catch basins were incorporated into the model using BWSO and survey data and inlet assumptions from DEP and were verified using Google Street View and during site visits. To account for variability in runoff from building rooftops, separate subcatchments were created using building footprints and set to drain to the nearest street midpoint. Surface types were accounted for by creating roughness and infiltration areas where flow could be slowed or infiltrated into the ground (i.e. in pervious park space).

To summarize, the NWBPCR model includes an updated 1D hydraulic network throughout NCM sewershed. The model includes a 2D area in lower Manhattan where overland flows can be visualized and a smaller ROM zone within NWBPCR extents where rainfall is applied directly to the surface. This approach allows for greater resolution in the project area in identifying localized drainage solutions as well as capacity constraints.

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 $\geq 1'$ 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 90% 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.

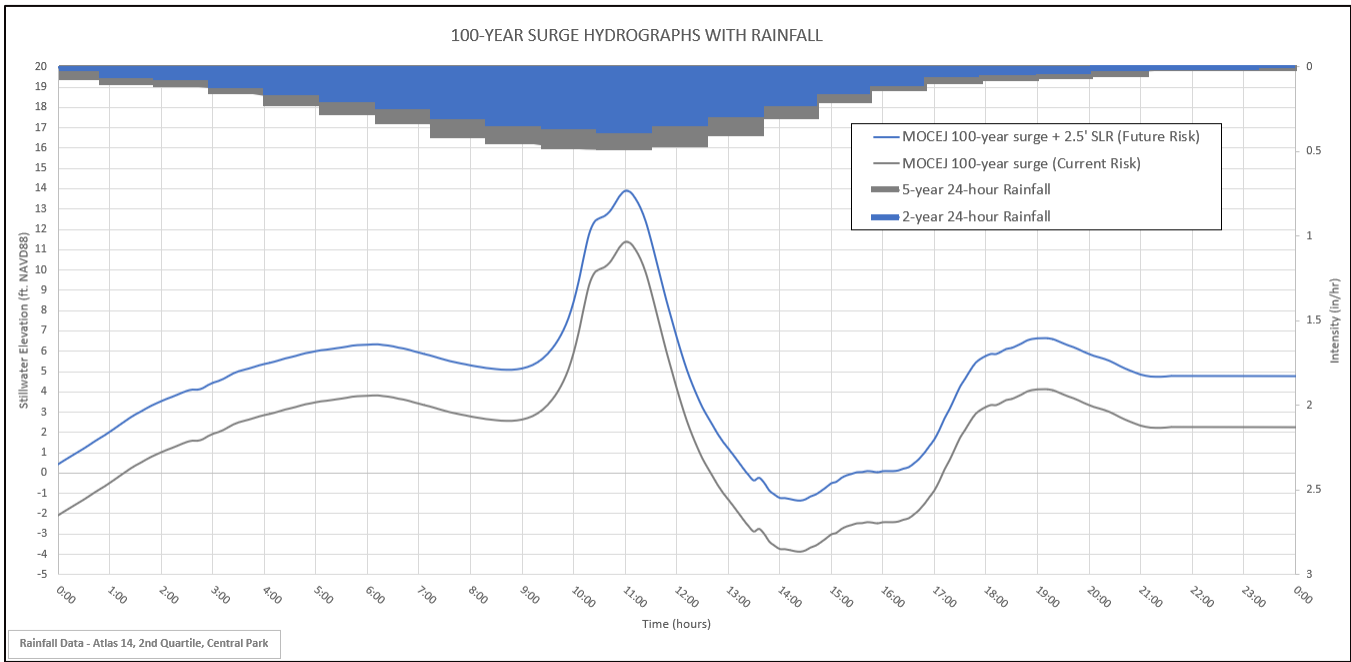
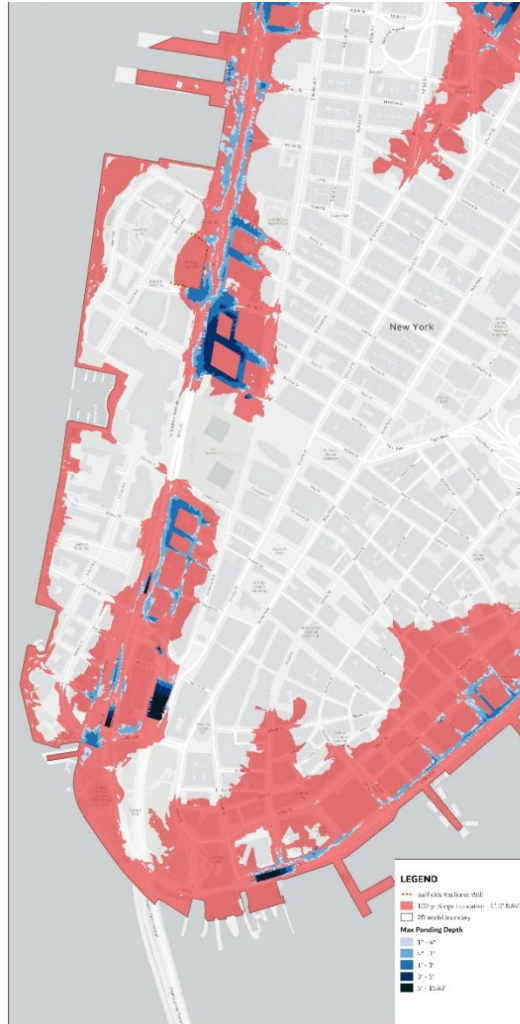


Figure 4-1. 5-year and 2-year Rainfall Hyetograph and 100-year Surge Conditions

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.



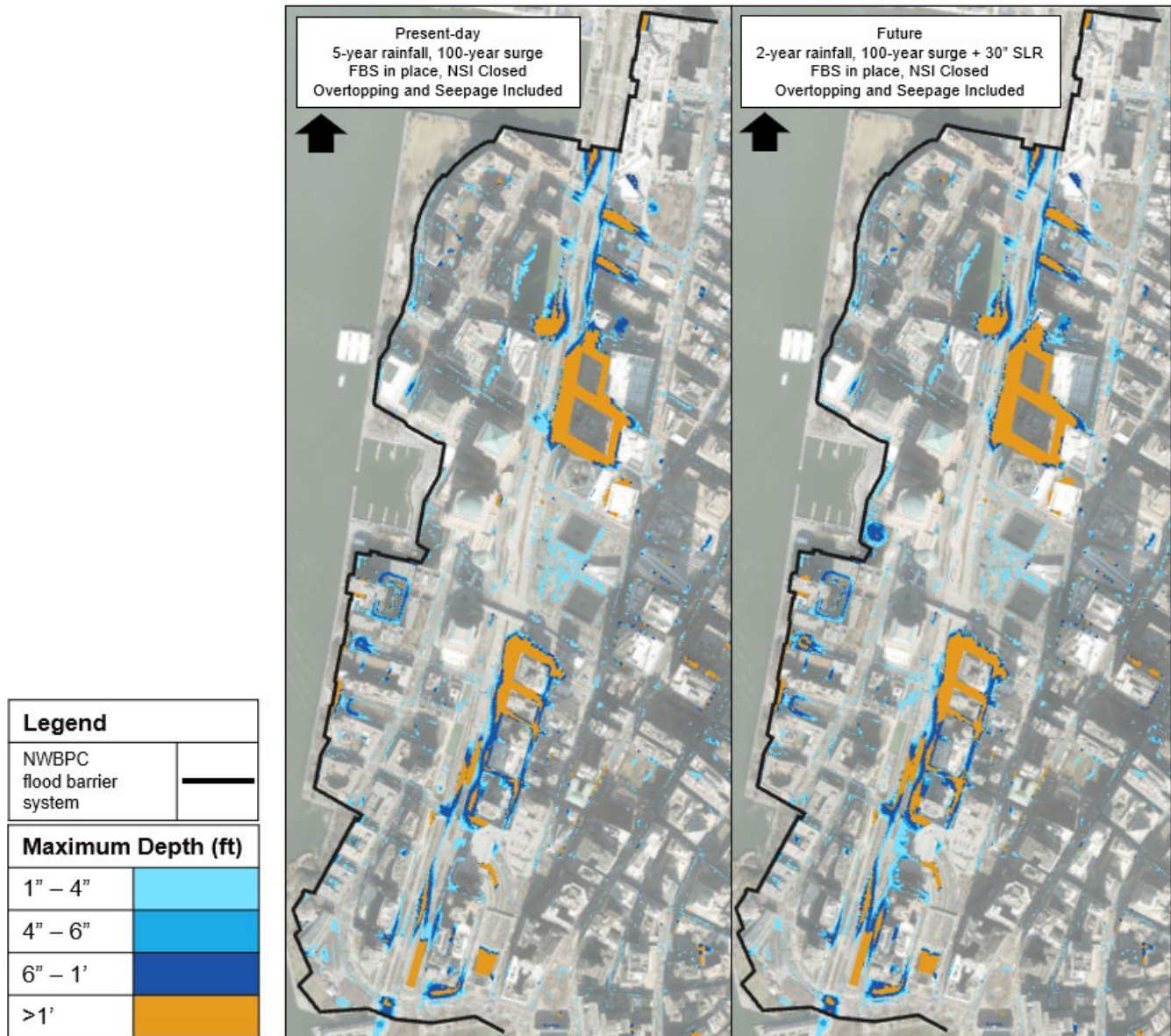


Figure 4-3. Baseline Conditions for Present and Future (ROM Results)

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 90% 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 project⁶ 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**. 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 90% 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/>

North/West Battery Park City Resiliency Project – Interior Drainage Report

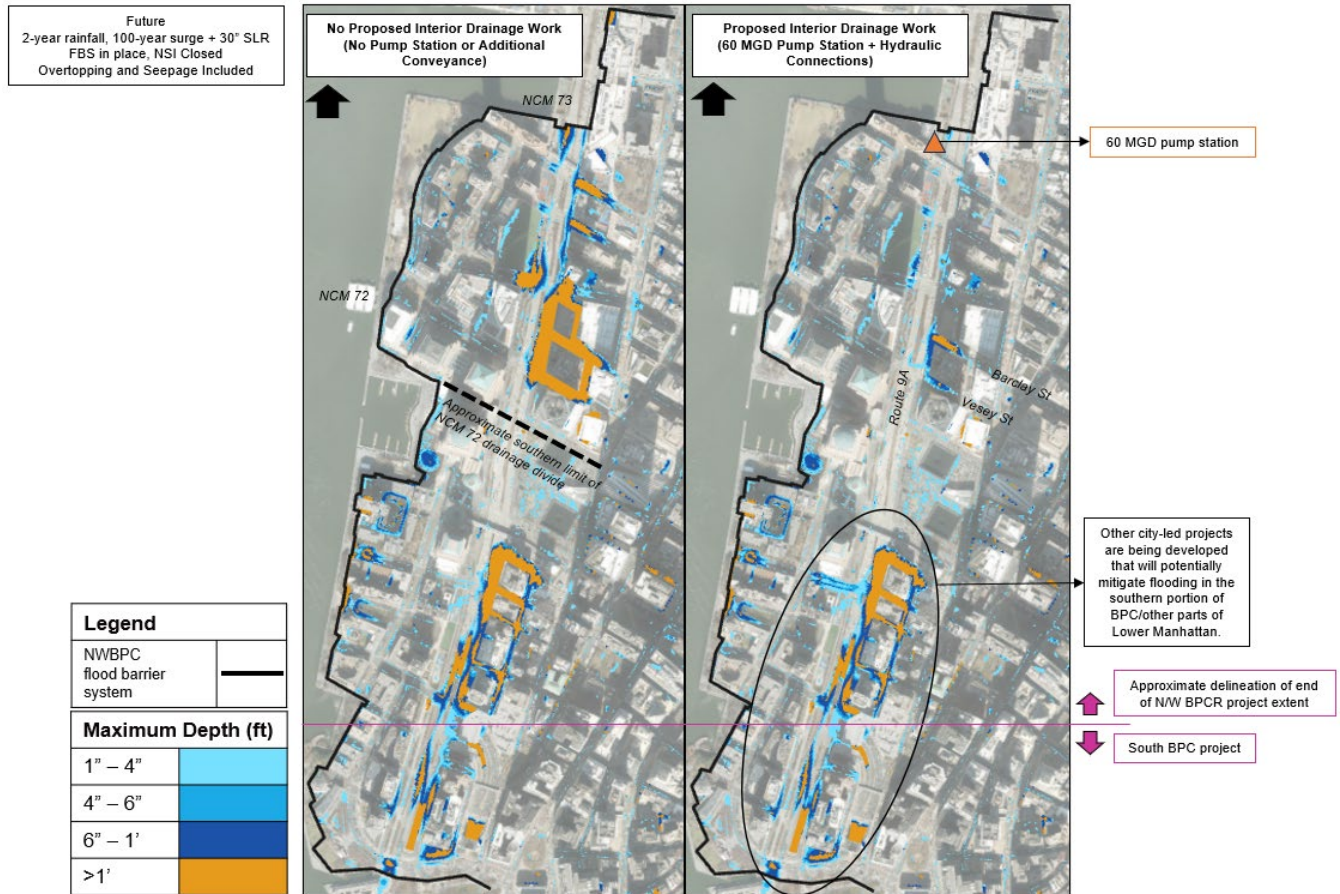


Figure 4-4. Baseline (No Pump/Interior Drainage Work) (Left), Proposed Pumping and Conveyance (Right)

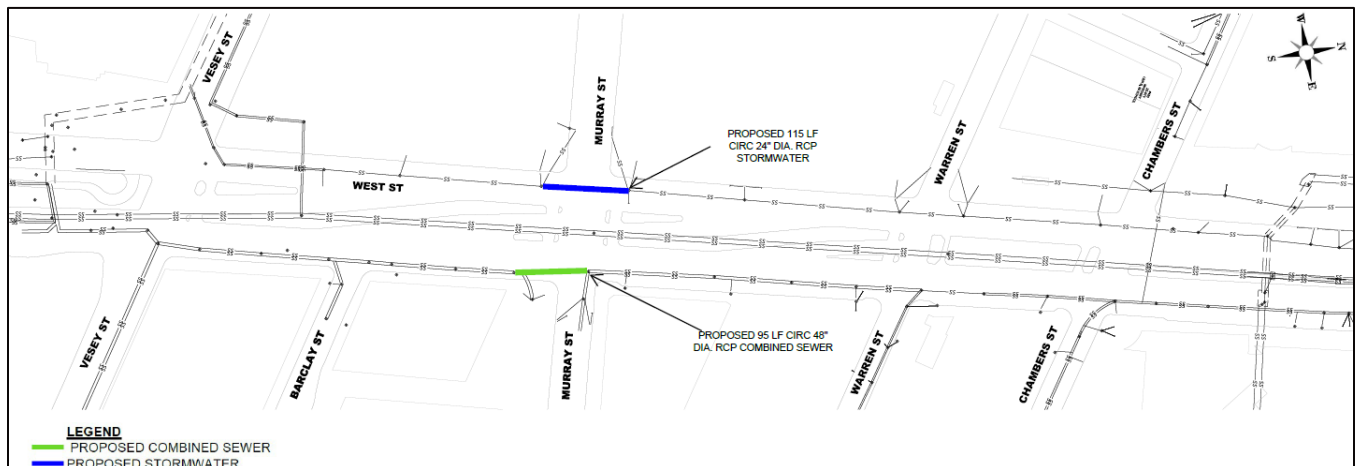


Figure 4-5. Hydraulic Connections included in Figure 4-4

In addition to coordinating with BPCA, the design team has been working with DEP on the pump station design through a series of technical memos (TMs), with the pump station design progressing on a separate track from the FBS design. A siting analysis was conducted as part of TM-1, resulting in the selection of Stuyvesant High School Plaza as the preferred location. TM-4 includes an analysis of conveyance alternatives which is included under pump station design work. A comprehensive cost and constructability analysis is currently underway 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.

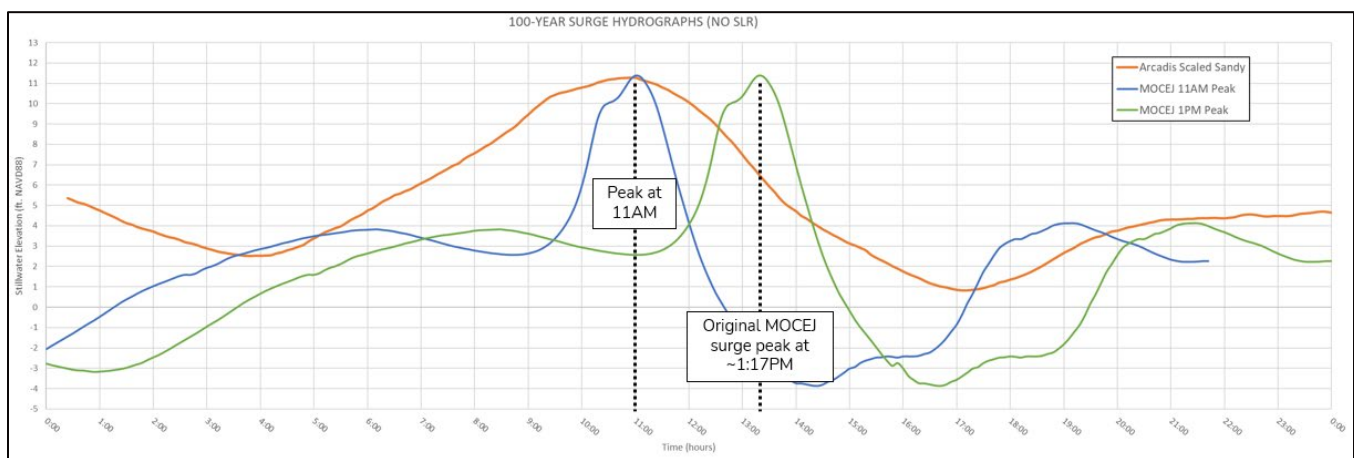


Figure 4-6. Surge Conditions Used in Sensitivity Analysis

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.

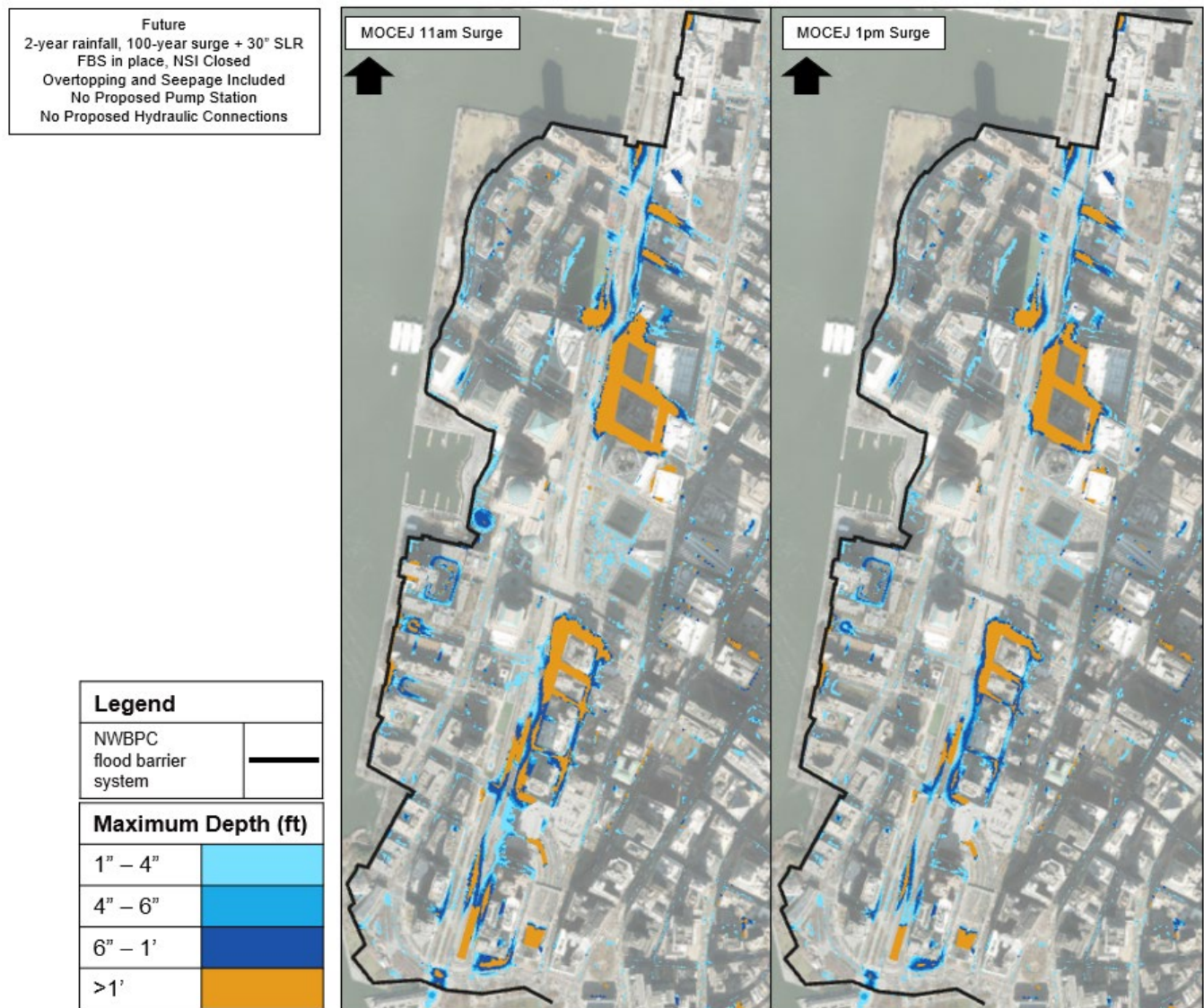


Figure 4-7. Sensitivity Analysis - MOCEJ/DEP Surge Hydrograph

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

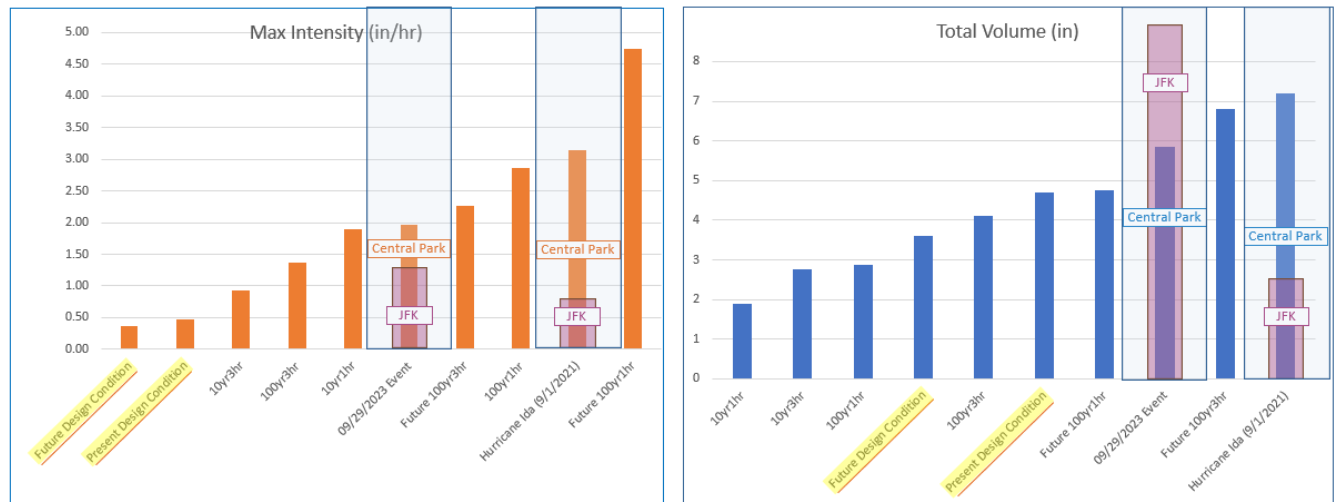
Rainfall Return Period	Durations (hr)	Assumed Boundary Conditions	Near-surface-isolation	OT and Seepage
Current 10-year (Baseline + Proposed)	1, 3	Present-day MHHW	Open	Not included
Current 100-year (Baseline + Proposed)	1, 3	Present-day MHHW	Open	Not included

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 and was not updated because the changes in the alignment have no discernible impact on the results; for details on the latest floodwall alignment, see the 90% design plans.

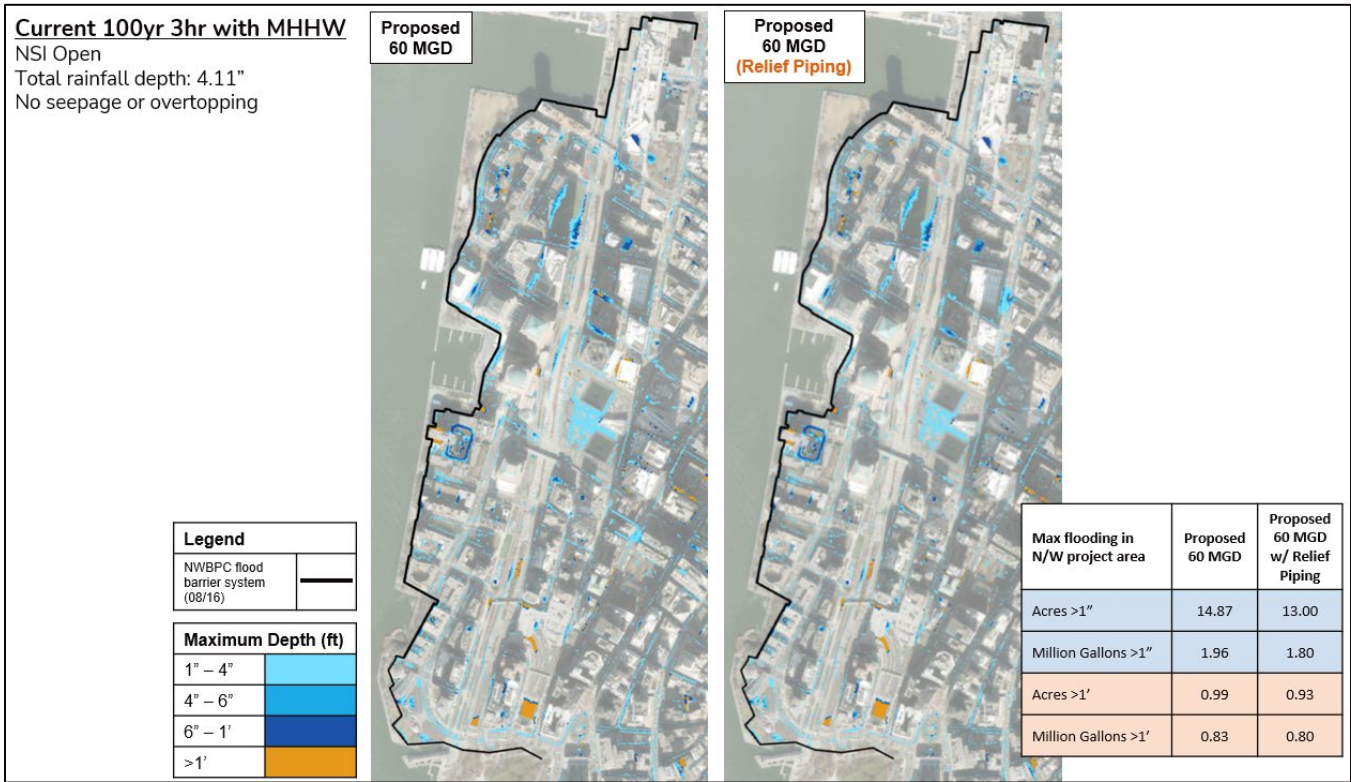


Figure 4-9. 100-year 3-hour Cloudburst Results with Proposed Work (Pump Station + Alternative 5 Conveyance) and Additional Modification

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.

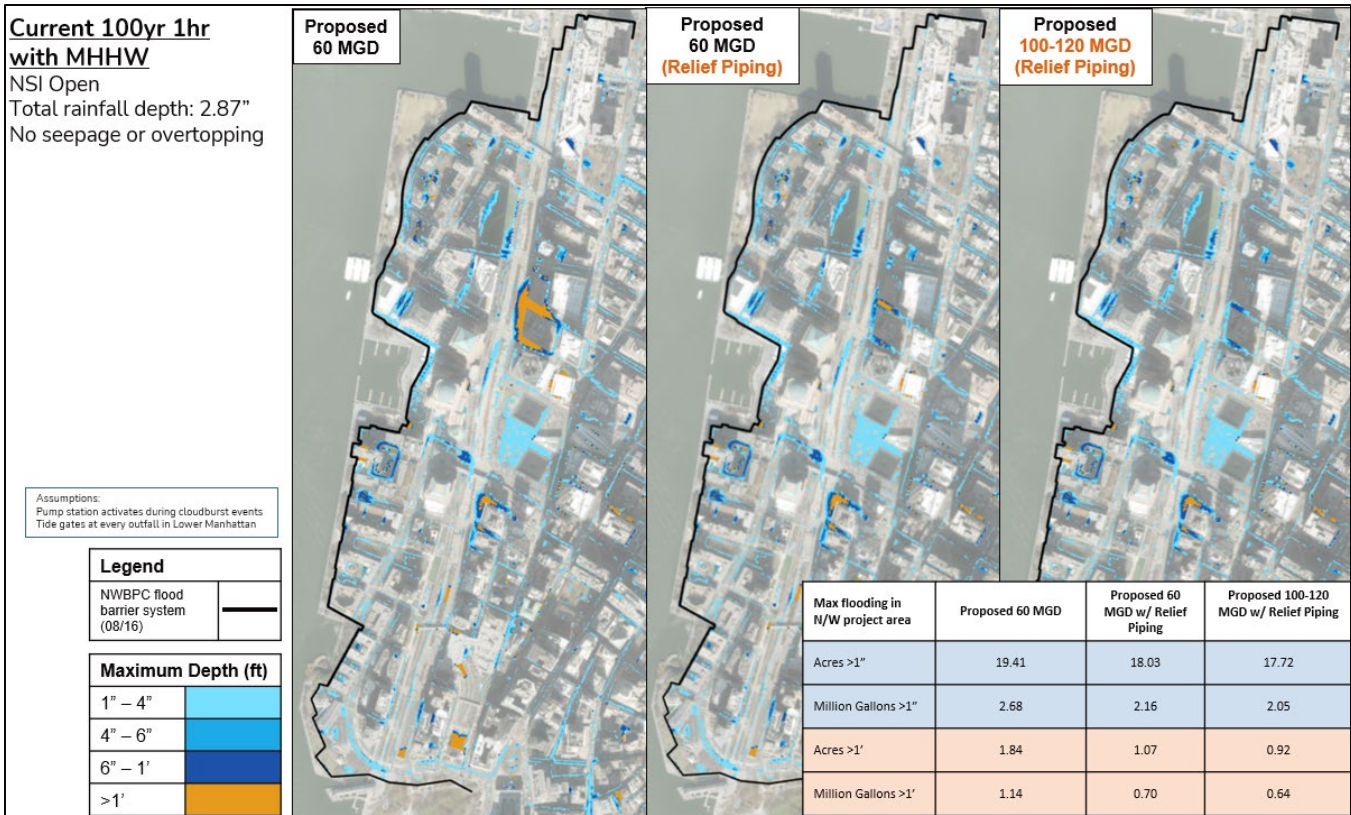


Figure 4-10. 100-year 1-hour Results with Proposed Work (Pump Station + Alternative 5 Conveyance) and Additional Modifications

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 90% 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 2.9.1** 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. Operation of future stormwater infrastructure is also a key consideration and will be assessed in future phases of work.

6 References

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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>.
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