City of Alexandria, Virginia
Waterfront Implementation

Technical Memorandum 6
PUMP STATION CAPACITY AND SIZING

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Abbreviations

ASCE	American Society of Civil Engineers
ATS	automatic transfer switch
BFE	base flood elevation
Carollo	Carollo Engineers, Inc.
cfs	cubic feet per second
CFD	computational fluid dynamics
City	City of Alexandria
El.	elevation
ft	feet
GHG	greenhouse gas emissions
hp	horsepower
KW	kilowatt
I&C	instrumentation and controls
IDF	Intensity-Duration-Frequency
M	million
MCC	motor control center
MG	million gallons
mgd	million gallons per day
MSWMP	Master Storm Water Management Plan
O&M	operation and maintenance
PDB	progressive design build
PDB-O	progressive design build and operations
PS	pump station
PS1	Southern Pump Station
PS2	Northern Pump Station
TM	technical memorandum
VFD	variable frequency drive
WFI	Waterfront Implementation
XPSWMM	Modeling Software

Technical Memorandum 6

PUMP STATION CAPACITY AND SIZING

1.1 Purpose

The City of Alexandria (City) currently experiences localized street flooding within the core area, located between Union Street and the waterfront from Duke Street to Queen Street, caused by rainfall and the limited capacity of the existing storm sewer collection system. Street flooding affects businesses and residents and reduces mobility within the core area. In addition, nuisance street flooding unrelated to local rainfall affects the core area due to high tide in the Potomac River and storm surge events caused by offshore low pressure that raises river surface elevations. In 2018, the Baseline Project was formalized which included two stormwater pumping stations to mitigate the street flooding impacts and discharge stormwater to the Potomac River.

Carollo Engineers, Inc. (Carollo) evaluated the pumping capacities of the proposed Baseline Project stormwater pump stations (PS) to identify opportunities to reduce their size and the number of pump stations required to alleviate flooding. This memorandum describes the stormwater pump stations proposed by the 2018 Baseline Project, followed by Carollo's evaluation of the Baseline Project, and proposed modifications to reduce the recommended stormwater pumping requirements, including pump station footprint, location, and pump sizes.

1.2 Baseline Project Pump Stations

Stantec's 2018 *Master Storm Water Management Plan* (MSWMP) analyzed various pumping alternatives to discharge stormwater from the stormwater collection system into the Potomac River to mitigate street flooding within the Waterfront Implementation (WFI) project core area. The two pump stations, Waterfront Park Pump Station (PS1) and Thompsons Alley Pump Station (PS2)[1], were sized based on the peak flow rate of the 10-year City storm, with an intensity of 9-inches per hour, during a 5-minute peak period. The MSWMP recommended design is part of the Baseline Project with the main design components outlined in Table 1.

Table 1 Baseline Project Pump Stations Design Requirements

Description	Waterfront Pump Station (PS1)	Thompsons Alley Pump Station (PS2)
Rated capacity, cubic feet per second (cfs)	198	132
Rated capacity, million gallons per day (mgd)	130	85
Number of pumps	2 high-flow + 1 standby 2 low-flow	2 high-flow + 1 standby 2 low-flow

[1] Stormwater pumping station nomenclature for Waterfront Park Pump Station (PS1) and Thompsons Alley Park Pump Station (PS2) is specific to the Baseline Project as documented in the 2018 Stantec MSWMP. However, for the purposes of this TM and to be consistent with the Design-Builder Request for Proposal (February 2023), all future references to stormwater pumping stations are as follows: Southern Pump Station (PS1) and Northern Pump Station (PS2).

Description	Waterfront Pump Station (PS1)	Thompsons Alley Pump Station (PS2)
Pump motor, horsepower (hp)	270 (high flow) 60 (low flow)	200 hp (high flow) 60 hp (low flow)
Emergency power generator, kilowatts (kw)	1500	1200
Proposed location	SW corner of Waterfront Park	Thompsons Alley waterfront
Proposed aboveground footprint	30'x30'	30'x30'

1.3 Baseline Project Evaluation

Carollo's team evaluated the Baseline Project to identify phasing, innovative solutions, and value-engineering modifications that can cost-effectively accomplish project goals. The team considered and analyzed the following considerations related to the pump stations:

- Confirmation of Baseline Project Sizing and Design Assumptions.
- Evaluating the number and size of the pump stations:
  - Eliminating the pump stations.
  - Reducing the number of pump stations from two to one, to be located at the Waterfront Park.
  - Increasing the number of pump stations from one to four smaller pump stations, each with a dedicated stormwater outfall to the Potomac River, at four existing permitted outfall locations within the core area.
- Evaluating the design storm criteria to use less stringent criteria that predicts reduced inflow to the pump stations, thus reducing the required pumping capacities.
- Optimizing the storm sewer collection system layout to maximize its carrying capacity and attenuate the inflow to the pump stations, thus reducing the required pumping capacities.
- Using green infrastructure underground stormwater detention chambers to attenuate inflow to the pump stations, thus reducing the pumping capacities.

1.3.1 Confirmation of Baseline Project Sizing and Design Assumptions

The MSWMP calculated a 30-feet by 30-feet building size for PS1 and PS2; Figure 1 shows the PS1 aboveground layout. For preliminary design and cost estimating, this assumption is reasonable, however, detailed design may slightly expand the electrical room. Some reconfigurations of the room and/or a smaller rated pump station may offset any added spatial requirements revealed later in design.

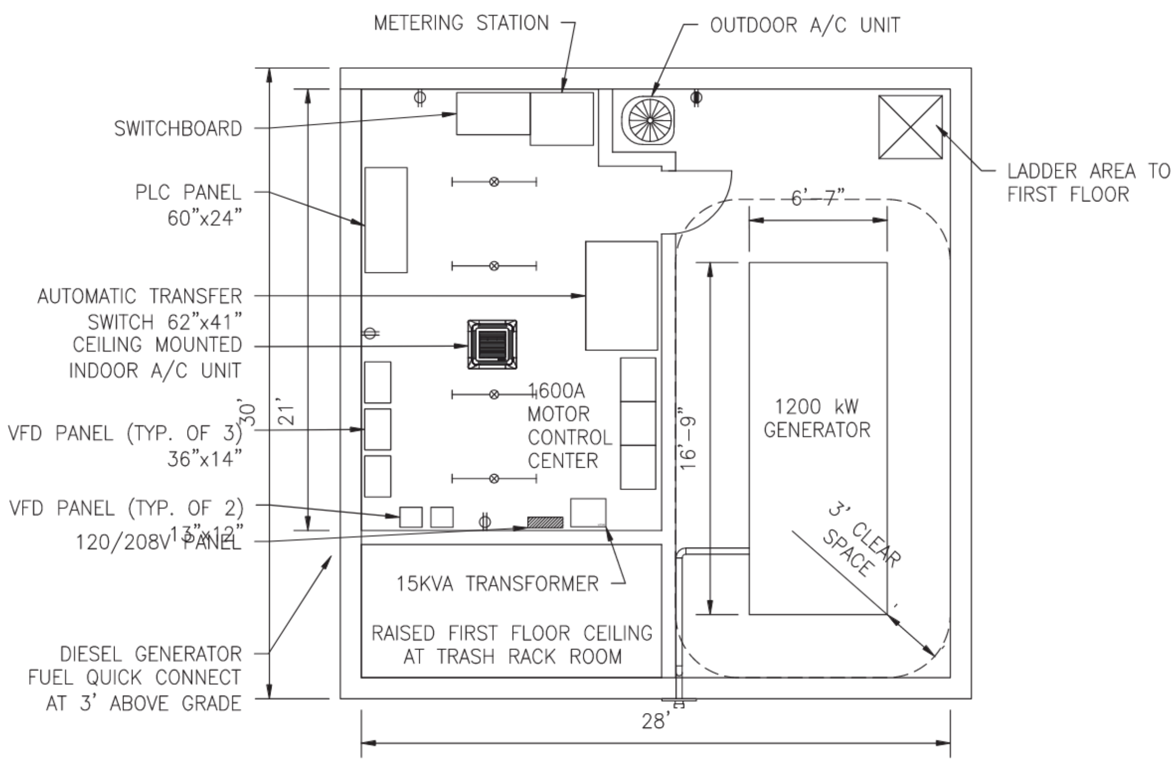


Figure 1 Southern Pump Station (PS1) Aboveground Layout

Given the proposed layout, it may be important to keep the following operational considerations in mind while proceeding in design:

- The current electrical room layout does not allocate space to a workstation for an operator. Slight configuration of the space could accommodate a corner design between the PLC panel and switchboard. This would require moving the outdoor A/C unit towards the ladder area, extending the wall length of the electrical, and shifting the switchboard and metering station.
- Only outside access to the pumps is permitted. An operator and/or maintenance crew will need to bring a portable crane.
- Upstream and downstream isolation of the screens is not shown. Isolation can be done via stop logs or gate operations. In either case, ingress and egress of staff must be considered. Stop logs will require on-site storage. Collectively the considerations could modify the building footprint, but not significantly.

1.3.2 Evaluation of Pump Stations Number and Sizes

1.3.2.1 Eliminating the Pump Stations

Carollo evaluated the feasibility of eliminating the pump stations proposed by the Baseline Project by conducting a cursory reviewing the following information:

- Stormwater runoff volumes projected by the 2018 MSWMP.
- Existing topographic and utility survey, including street grade elevations, location and elevations of stormwater inlets and manholes, and existing bulkhead elevation.
- Proposed street grading elevations for the Baseline Project.
- Existing and proposed stormwater sewer collection system carrying capacity.

- Historical rainfall data and Potomac River tidal elevations.
- Historical flooding depths and occurrences within the core area.
- Available geotechnical information from the Phase I geotechnical investigation.

Carollo's Technical Memorandum (TM) 2: *Potomac River Flood Frequency Analysis* recommended proceeding with elevation (El.) +6.0-feet bulkhead to protect the core area from flooding caused by varying river water surface elevations.

Although flooding due to river water surface elevations will be largely averted at the recommended bulkhead elevation, stormwater runoff is expected to accumulate within the core area without the ability to drain by gravity into the Potomac River, thus causing rainfall-induced flooding. The 2018 MSWMP projects stormwater runoff peak flows of up to 330 cfs between the two pumping stations, which correspond to the volume of almost four Olympic swimming pools poured into the core area in one second.

The existing stormwater system drains by gravity to the Potomac River, and thus, is tidally dependent. Given the high-water table and proximity to the river, the existing outfall inverts are almost always below the Potomac River level, so the stormwater network almost never fully drains.

Lastly, the project is in a highly developed waterfront area consisting of largely impervious surfaces, pool fill, and soil conditions that are not conducive to infiltration.

Given these site conditions and considerations, **Carollo's review concluded that it is hydraulically impossible to drain any flooding waters by gravity into the river or evacuate them through infiltration, making pumping strictly necessary to mitigate street flooding within the core area**.

1.3.2.2 One Pump Station

Carollo's evaluation considered reducing the number of pump stations from two to one, with the goal of reducing pump station operation and maintenance (O&M) requirements for the City and the number of buildings occupying public space. For this evaluation, Carollo considered retaining PS1 at Waterfront Park due to its central location within the core area and its proximity to the areas of greater street flooding, such as the King Street and Strand Street intersection, as discussed in Carollo's *Storm Sewer Collection System Upgrades* memorandum. This scenario considered eliminating PS2, which at the location proposed by the Baseline Project, would require dredging, filling, and extension of the existing bulkhead to accommodate the PS2 footprint (Figure 2).

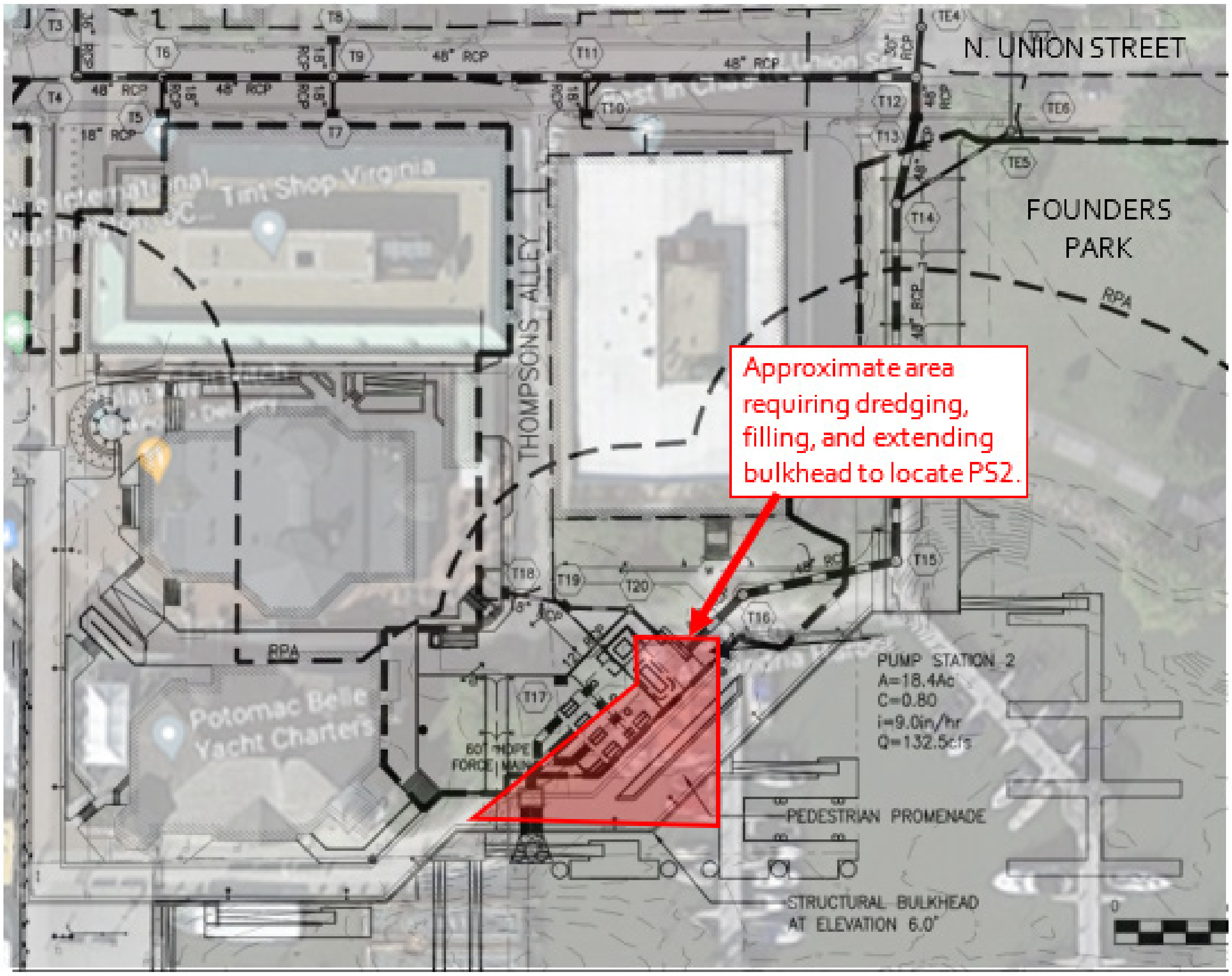


Figure 2 Required Site Work at PS2 per the Baseline Project

Although eliminating PS2 has some benefits, Carollo determined that the single PS1 would need to be enlarged to a rated capacity of approximately 330 cfs (214 mgd) and almost double the required aboveground footprint, which would take away from park space. This increased capacity would also require:

- Re-routing the storm sewer pipes currently located north of King Street, to run south towards PS1 at Waterfront Park, which would result in deeper sewer pipe elevations in the northern end of the core area, given that the ground elevations north of King Street are generally higher than those at Waterfront Park.
- Adjustment or replacement of manholes and stormwater inlets north of King Street, to accommodate the modified sewer depths mentioned above.
- Re-grading of North Union Street to minimize sewer depths.
- Installing a 3,000-kW emergency power generator within public park space which, compared to the 1500 kW generator proposed by the Baseline Project would be more costly, require a larger installation area, and require longer periods of time for refueling or more frequent fueling events.

In addition, a single pump station would increase the operational risk for the City. While the pump station would include redundant pumps and a standby generator to minimize the probability of failure, there would be no redundancy or an alternate system to evacuate the stormwater from the entire core area in the event of a catastrophic pump station failure.

In conclusion, Carollo's analysis determined that the increased pump station capacity requirements along with the additional infrastructure required to support the single pump station option, would result in an overall greater cost and operational risk to the City when compared to the Baseline Project. Therefore, **it is not recommended to proceed with a single pump station,** and as such, did not pursue further analysis of this option through subsequent hydraulic modeling.

1.3.2.3 Four Pump Stations

Carollo evaluated increasing the number of pump stations from two proposed by the Baseline Project to four pump stations, each discharging directly to each of four existing permitted stormwater outfalls to the Potomac River located within the core area: the Duke Street outfall, the Prince Street outfall, the King Street outfall, and the Queen Street outfall. Figure 3 depicts the potential locations of the four pump stations and their dedicated stormwater outfalls.

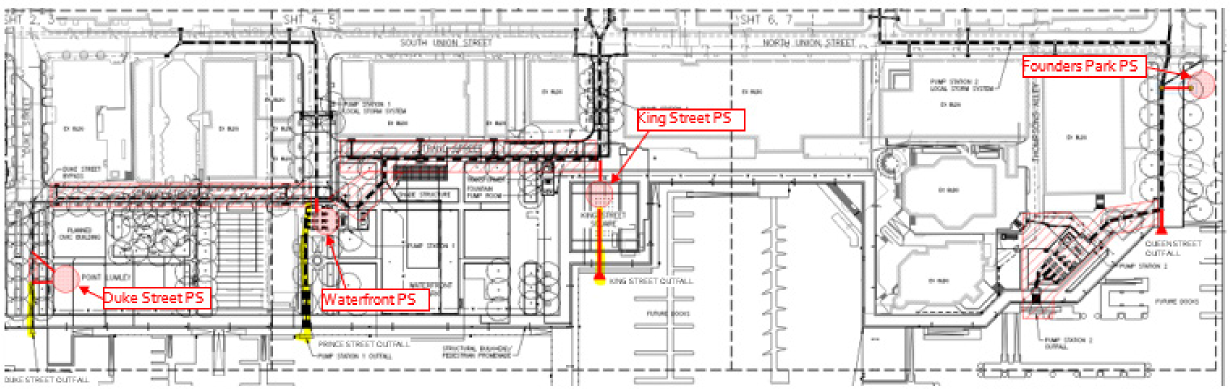


Figure 3 Location of Four Pump Station and Existing Core Area Stormwater Outfalls

In contrast to the one-pump-station evaluation discussed previously, increasing the number of pump stations to four would reduce the operational risk for the City given that each pump station will manage smaller drainage areas, which would limit the impacts of an unlikely pump station failure. Conversely, this benefit would be offset by the increased O&M requirements of multiple pump stations.

Each of the four pump stations pumping capacities will be smaller when compared to the capacities of PS1 and PS2 from the Baseline Project, with each rated at approximately 83 cfs (53 mgd); however, given that each location would still require an emergency power generator, standard electrical equipment and controls, bar screens, and solids collection equipment, the reduction in footprint will not be proportional to the reduction in pumping capacity and four aboveground structures will need to be accommodated within public space, not only reducing space for recreation and amenities, but also increasing construction cost.

Given the anticipated increases in cost, O&M requirements, and reduction of public space for recreation and amenities, **Carollo concluded that revising the number of pump station facilities to one or four pump stations, would not meet the project goals in a cost-effective manner**, and as such, did not pursue further analysis of these options through subsequent hydraulic modeling.

1.4 Evaluation of the Storm Design Criteria

The design storm is a critical planning factor for evaluating stormwater system performance and determining infrastructure sizing. The design storm's parameters (particularly total volume and peak rainfall intensity) were selected to balance existing rainfall patterns with projected climate change and the City of Alexandria's desire to reduce waterfront flooding to the maximum extent possible. The memorandum, *Design Storm Selection for Hydrologic and Hydraulic Modeling*, recommends a peak rainfall intensity from the City of Alexandria's intensity-duration-frequency (IDF) Curve for the **10-year 5-minute time of concentration, which corresponds to 9-inch per hour with a 2-hour storm duration and total rainfall depth of 2.53 inches**. Routine checks using more current information and climate model results are also recommended to confirm that these design guidelines appropriately account for climate change effects. For more information and detail on the evaluation of the design criteria, assumptions, and justification of recommendation, refer to the *Design Storm Selection for Hydrologic and Hydraulic Modeling* memorandum.

1.5 Optimization of the Storm Sewer Collection System Layout

Optimizing the storm sewer collection system layout is essential to efficiently manage flooding events during wet weather; if pipes do not have the capacity to transport stormwater to pump stations, street flooding will occur regardless of the pump sizes. As summarized in *Storm Sewer Collection System Upgrades* memorandum, Carollo evaluated the Baseline Project stormwater conveyance system against the recommended design storm. Modeling results demonstrated an opportunity for upsizing stormwater pipes to further manage inland flooding and raise the storm sewer depth in the core area to reduce overall project costs. The memorandum also touches on the benefits of subsurface stormwater detention chambers and pumping stations for the comprehensive project to manage flooding.

1.6 Installation of Underground Stormwater Detention Chambers

As described in *Parkspace and Streetscape Attenuation Solutions*, Carollo's evaluation of state-of-the-art green stormwater attenuation solutions included the installation of subsurface stormwater detention chambers in both Waterfront and Founders Park. The chambers are intended to attenuate the peak flows into the pumping station, thereby reducing the total rated capacity of each pumping station. Streetscape solutions and bioretention were also evaluated; while they can provide some benefits to the Project, these elements do not have a significant impact on pump station sizing. It was recommended to further evaluate the feasibility of subsurface detention by the geotechnical subconsultant (and documented in the *Geotechnical Design Memorandum)* and the cost-benefit by the Design Builder during Phase I services. Refer to *Parkspace and Streetscape Attenuation Solutions* for a detailed evaluation of potential green enhancements to the Baseline Project.

1.7 Proposed Pump Stations Modifications

The Baseline Project evaluation established the following parameters for the pumping station analysis:

- The use of two stormwater pump stations.
- The use of the design storm: 10-year return period, 5-minute peak, 9-inch per hour precipitation intensity, 2-hour storm duration, 2.53-inch storm depth.
- The proposed modifications to the Baseline Project storm sewer collection system, including revised sewer depths and localized modifications to storm sewer pipe sizes.
- The use of aboveground bioretention and subsurface stormwater detention chambers at Waterfront Park and Founders Park.

1.7.1 Pump Station Inflow

Figure 4 is a detailed schematic illustrating how the water will flow to both the subsurface detention chambers and pump stations during storm events. The sections below breakdown the different parts of the model that was used to visualize the flow at the design storm that informed the sizing and operations of the pump station.

1.7.1.1 Stormwater Runoff Peak Flow

The basis of design for sizing the Baseline Project stormwater pump stations corresponded to the peak runoff rate of the 10-year City design storm, as stated in the 2018 MSWMP. The Rational Method used to calculate the peak runoff rate, defines it as the product of the runoff coefficient (percentage of impervious area), the rainfall intensity, and the drainage area. The peak runoff rates calculated for both pump stations are 198 cfs for PS1 and 132.5 cfs for PS2, as noted in Table 1 earlier in this memo.

Using the runoff peak rate as the basis of design to size the pump station allows for immediate evacuation of the stormwater runoff without the need for storage or a large wet well. For example, the Baseline Project PS1 can pump up to 198 cubic feet of runoff volume in just one second. This design is advantageous when space for the pump station wet well is limited; however, it also results in overly large pump stations that will only be utilized at full capacity during runoff peak conditions. In contrast, providing a large wet well, or stormwater detention upstream of the wet well, allows for the stormwater to remain longer in the wet well without the need for immediate pumping, and to be drawn by the pumps at a slower rate, thus providing peak attenuation and reducing the pump station pumping capacity. Carollo's review of the design storm criteria confirmed the use of the design storm selected for the Baseline Project, such that the peak runoff rates stated above cannot be modified to reduce the pump station size. However, as noted above, the size of the pump station assumes the need to fully pump the runoff peak flow as it enters the pump station without a large wet well or stormwater detention structure that allows for peak attenuation. It then follows, that providing either a larger wet well or stormwater detention upstream of the pump station wet well would attenuate the runoff peak flow such that the entire peak volume would not need to be pumped in full as it enters the pump station.

1.7.1.2 Stormwater Runoff Peak Flow Attenuation

Carollo analyzed the effect the underground stormwater detention chambers would have on attenuating the runoff peak flow entering the pump stations. Carollo used XPSWMM to model the design storm for a period of 24 hours to observe the accumulation of stormwater in the detention chambers and to simulate the release of the stormwater from the detention chambers to the pump stations wet wells.

Per the *Storm Sewer Collection System Upgrades* memorandum, the model added subsurface detention in Waterfront Park and Founders Park to attenuate the King Street sub-catchment area and Queens Street sub-catchment (as defined in the 2018 MSWMP), respectively. The results of the model showed that the stormwater detention chambers can receive and retain the runoff peak flow for longer than the entire duration of the peak event (5 minutes). In this way, the peak flow is attenuated; such that the rate at which the stormwater detained in the chambers is released to the wet well, is much slower than the rate at which the runoff peak enters the chambers or pump station wet well directly from the storm sewer collection system.

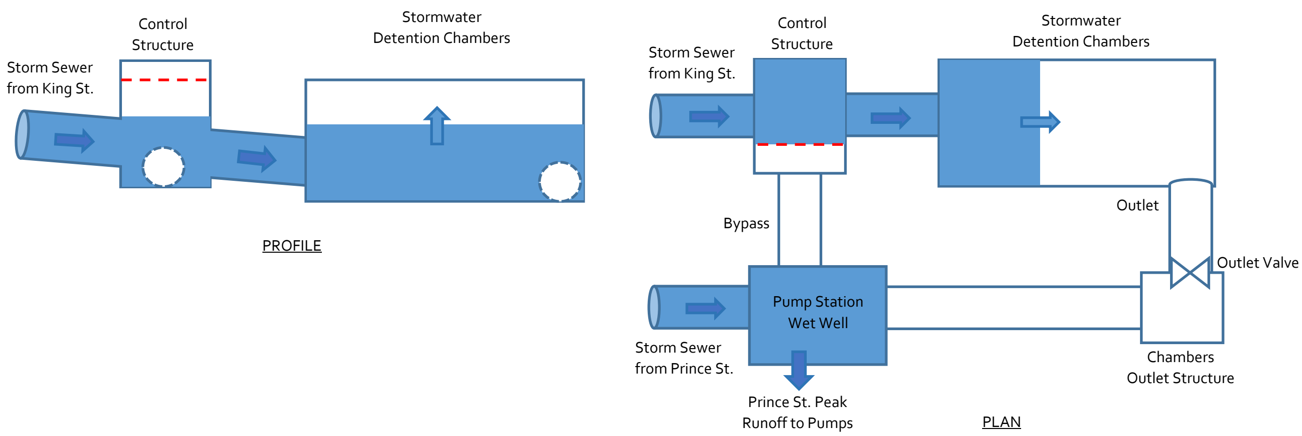
1.7.1.3 Stormwater Bypass and Inflow to Pump Stations

While most of the stormwater collected through the sewer system was modeled to flow into the proposed chambers at both parks, select drainage areas were routed directly into the pump station wet wells due to topography and the system's hydraulic limitations. For instance, the drainage area along and south of Prince Street is at a lower elevation than the proposed PS1 site at Waterfront Park, thus making it hydraulically easier to flow directly into the PS1 wet well than to the chambers. For PS2, the flow from the drainage collected near the Torpedo Factory and Chart House restaurant is also fed directly into the PS2 wet well instead of being routed through the chambers under Founders Park, given this area is closer to PS2 than to the park. As such, the flows that are directly routed to each of the pump stations wet wells do not benefit from the flow attenuation effect explained above and enter the pump station at the design storm runoff peak rate corresponding to those smaller "uncontrolled" areas.

Simultaneous to the peak runoff entering the pump stations wet wells from the uncontrolled areas, the stormwater will be entering and filling up the chambers. Once the chambers reach capacity, the stormwater backs up into a flow control structure. At this junction the weir height is set at the maximum water surface elevation of the chambers, such that stormwater runoff will only spill over the weir and into the bypass pipe once the chambers are full. The bypass pipe will deliver the excess flow directly to the pump station wet well. Once the stormwater peak has passed and the wet well stops receiving the peak runoff from the uncontrolled area(s) and bypass, an isolation valve located on the chambers' outlet pipe, will be open for the detained stormwater to be released (attenuated peak) into the wet wells. Figure 4 illustrates the bypass operation and inflows to the pump station wet well.

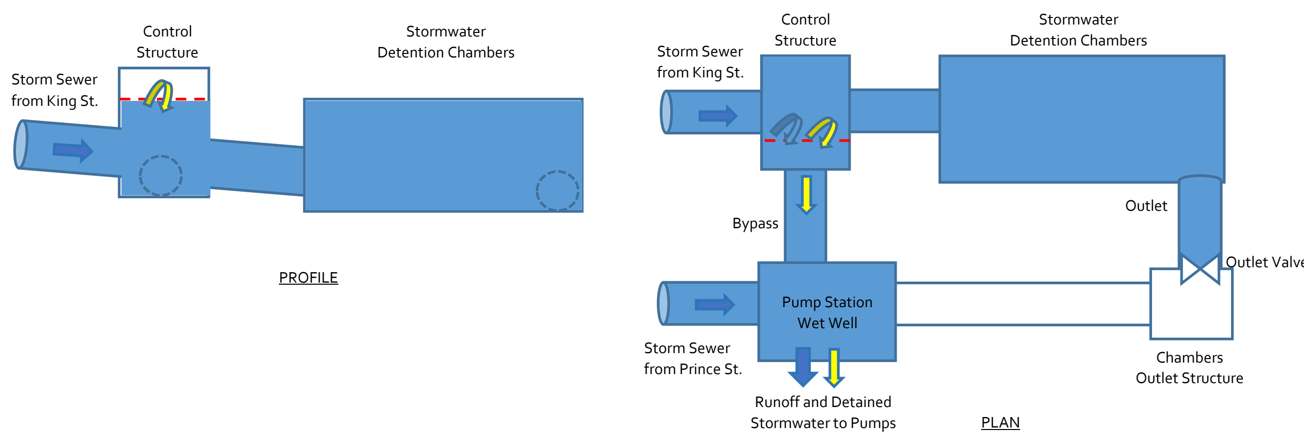
A. Storm Starts: Stormwater Runoff enters Pump Station Wet Well and Detention Chambers

At the onset of a storm event, stormwater runoff collected by the King Street drainage divide enters the detention chambers while stormwater from the Prince Street drainage divide enters the pump station wet well directly. The chambers outlet valve is closed.



B. Storm Peaks: Detention Chambers Fill-up and Excess Flow runs through Bypass

Once the stormwater detention chambers fill up, the stormwater continues to accumulate in the control structure until it flows over the control weir and runs through the bypass to the pump station wet well. The chambers outlet valve remains closed.



C. Storm Subsides: Storm Sewers Flow Decreases and Detention Chambers are Dewatered

After the storm event subsides the runoff collected by the storm sewer collection system decreases to an end and the chambers outlet valve is open to release the detained stormwater into the pump station wet well, causing a "dewatering peak" to enter the wet well.

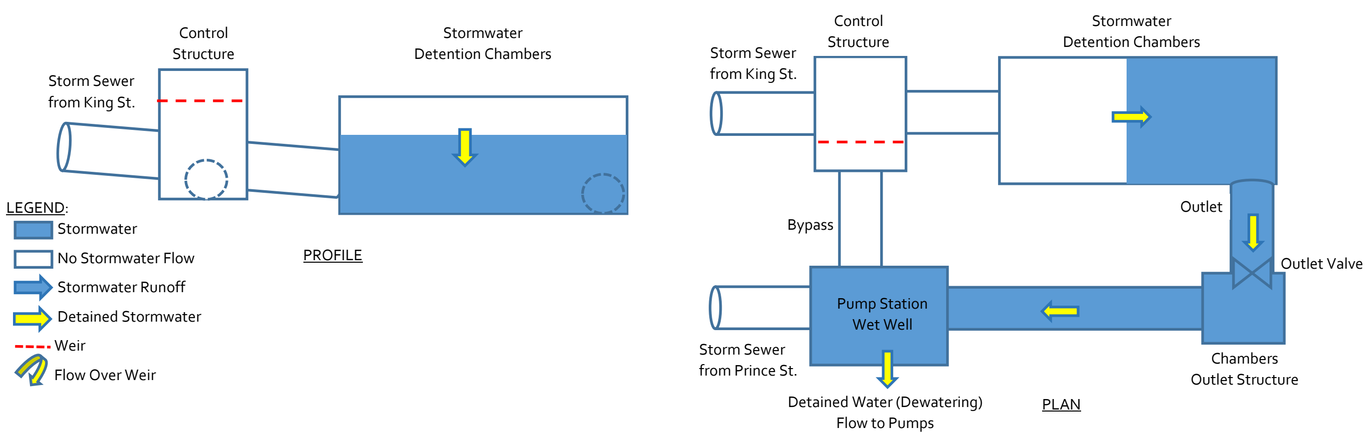


Figure 4 Stormwater Bypass Operation and Pump Station Inflows

Note that the pump operations described above and in accordance with Figure 4 is not the only operational mode available. As described in Potomac River Flood Frequency Analysis, a pump station that is sized for the design storm with this operational model in mind is capable of also managing larger storms with operational modifications. In this way, an operator may elect to leave the chamber outlet valve open for the duration of the event so that the chambers function as an extended wet well. This operational setting would allow the pump station to manage storms as big as 10-year 24-hour or a larger storm depth (with the same peak rainfall intensity) than 2.53-inches.

1.7.1.4 Southern Pump Station, PS1

The pump station analysis summarized below assumes the operational settings described in Figure 4. Runoff be conveyed mainly through the storm sewer collection system into the underground detention chambers in Waterfront Park; however, the runoff peak from the uncontrolled drainage area along Prince Street (and south of Prince St.) will enter the wet well directly. According to the XPSWMM model results the runoff peak rate for the smaller uncontrolled area corresponds to approximately 40 mgd. The second 60 mgd peak flow is the result of the King St. runoff bypassing the underground storage once they are full. Therefore, the new peak flow entering the pump station wet well is 60 mgd, with the difference (compared to the Baseline Project peak flow) being attenuated by the stormwater chambers. The new pump station capacity is defined by this new peak, such that it can handle the immediate discharge of the runoff from the uncontrolled area. Figure 5 below illustrates the inflows into the PS1 wet well estimated by the model, thus **demonstrating that including detention upstream of the pump station to handle the design storm runoff peak rate from the King Street drainage will considerably reduce the required pump station capacity.**

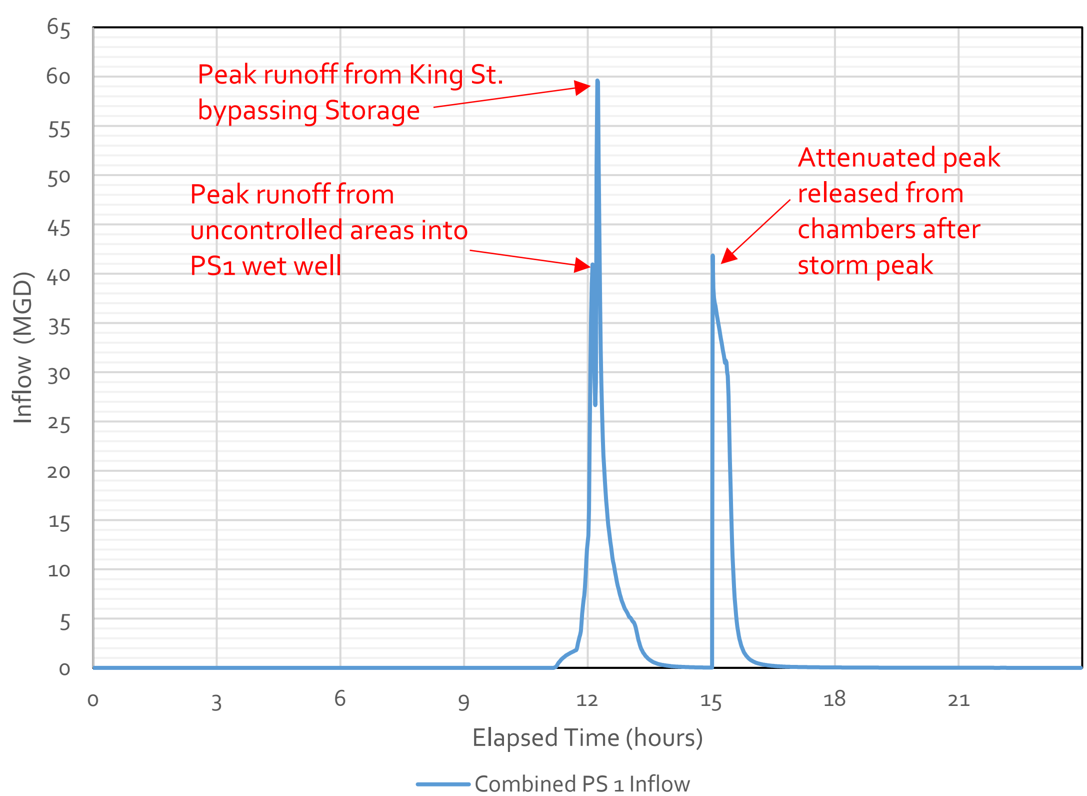


Figure 5 Southern Pump Station (PS1) Inflow and Stormwater Pumping

Table 2 summarizes key modifications to PS1. With regards to pump operation, 2-20 mgd and 2-10 mgd pumps are needed to handle the full peak flow. The attenuated peak released from chambers – approximately 42 mgd – is calculated based on a 2-foot diameter outlet pipe under full flow conditions. Once the outlet valve is opened, collected stormwater is released to the pumping station, whereas the outlet pipe size from the chambers to the PS1 wet well will dictate the maximum flow rate.

Table 2 Southern Pump Station (PS1) Key Design Parameters

Description	Project Baseline	Baseline Modifications
Pump station rated capacity (cfs)	198	93
Pump station rated capacity (mgd)	130	60
Pump Configuration	2 high-flow + 1 standby 2 low-flow	2 high-flow + 1 standby 2 low-flow
Pump Operation	2 high-flow duty	1 low-flow duty Peak: 2 high-flow +2 low-flow
Pump Type	submersible axial with VFDs	submersible axial with VFDs
Flow Capacity	(3) 65 mgd (high flow) (2) 10 mgd (low flow)	(3) 20 mgd (high flow) (2) 10 mgd (low flow)
Pump motor, horsepower (hp)	270 (high flow) 60 (low flow)	150 (high flow) 60 (low flow)
Emergency generator capacity (kw)	1500	500
Proposed location	SW corner of Waterfront Park	No Change from Baseline
Aboveground dimensions	30’ x 30’	29.5’ x 28’
Underground dimensions	72’ L x 37’ W x 20’ depth	No Change from Baseline

Although the rated capacity of the pump station reduces, there is no significant reduction in the above or belowground footprint of PS1.The belowground footprint is driven by the wet well dimensions. Due to the required spacing between the centerline of the 20 mgd and 10 mgd pumps along with the clearance from walls, the wet well dimensions are consistent with the Baseline Project. Consequently, the width of the check valve vault and the screenings room will remain the same. Therefore, the change to the underground footprint will be negligible.

The electrical equipment, bar screens, and generator will be located aboveground. Because of the space requirements of variable frequency drives (VFDs) for pump motors rated at 150 HP (high flow pump) and 70 HP (low flow pump), downsizing the electrical room is not advised. Additionally, the horizontal footprint required for two bar screens sized for the total flow rate of 60 mgd is approximately 12-feet by 8-feet. The screens cannot be completely belowground since the screenings need to be discarded in a bin and vehicles need easy access to the bin for hauling the screenings off. There is potential for further reduction of the horizontal footprint if the screens are installed such that the top of the screens is below the finished floor elevation of the electrical room or the generator room. However, this is dependent on several factors including the angle of screen installation, general screen and channel layout, and orientation of the influent pipe. Therefore, Carollo made a conservative assumption that the horizontal footprint of screens will not be reduced. The space requirement for the emergency generator, however, is lower since the capacity is reduced from 1500 KW to 500 KW which slightly reduces the footprint. **Overall, the PS1 aboveground footprint may be reduced up to 10 percent.**

1.8 Northern Pump Station (PS2)

The pump station analysis summarized below assumes the operational settings described in Figure 4. Siting underground storage chambers in Founders Park would retain at least 1,700,000 gallons (1.7 MG) of stormwater. According to the results of the hydraulic model, this additional storage capacity will reduce the storm peak from approximately 85 mgd to 2.5 mgd. Since the 1.7 MG-sized detention chamber can store the predicted design storm runoff volume from the Queen Street drainage divide, the resultant 2.5 mgd flow is attributed to the uncontrolled flow at the Torpedo Factory and Chart House restaurant. In other words, under design storm conditions, the XPSWMM model results do not utilize the bypass pipe from the Queens Street drainage. Figure 6 illustrates PS2's anticipated inflow on the left side of the chart from the uncontrolled area and underground chamber dewatering on the right.

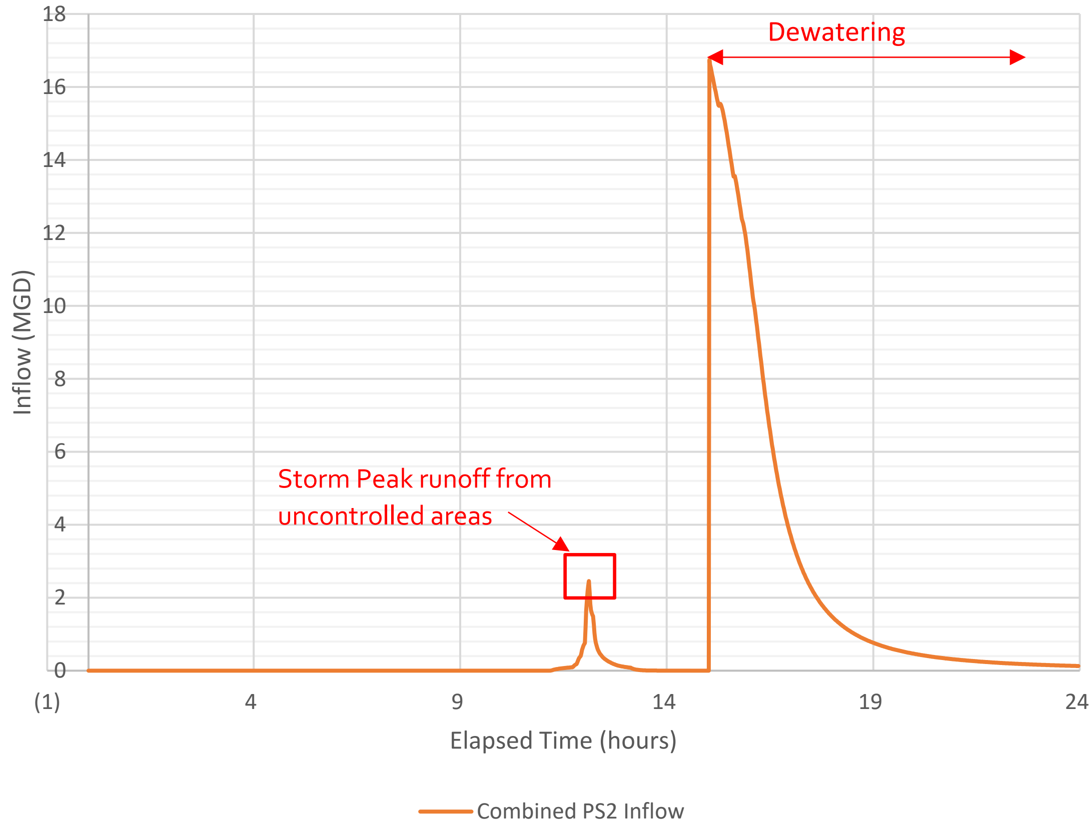


Figure 6 Northern Pump Station (PS2) Inflow and Dewatering

In contrast to PS1, the PS2's rated capacity is driven by the uncontrolled peak runoff. Again, the peak dewatering flow rate is calculated based on a 2-foot outlet pipe under full pipe flow conditions. While it may seem counterintuitive, the PS2 rated capacity does not need to satisfy the 16 mgd dewatering peak flow because the 16 mgd inflow rate will not cause PS2 flooding. This is because the PS2 finished floor is El. +5.0-feet and the chamber's maximum water surface elevation is El. +1.1-feet. Thus, when outlet valve is open and the wet well and chambers are hydraulically connected, the water surface elevation will not exceed El. +1.1-feet and therefore, not cause building flooding. As the dewatering inflow rate exceeds the PS2 rated capacity water will continue to fill the wet well up to the water surface elevation in the chambers, e.g., approximately El. +1.1-feet, until the two water elevations are equalized. The pumps will continue to run until the chambers are completely dewatered.

As noted in Table 3, PS2 is configured such that one low-flow pump can effectively manage the uncontrolled peak inflow, whereas two low-flow pumps (and one on standby) can dewater 1.7 MG of water in less than 8 hours. The PS2 sizing and three low-flow pump configuration offers flexibility to the PS operator to dewater the chambers between 6 and 12 hours.

Table 3 summarizes key modifications to PS2. However, any PS, regardless of rated capacity, still requires an emergency power generator, standard electrical equipment and controls, bar screens, and solids collection equipment, so the reduction in footprint will not be proportional to the reduction in pumping capacity.

Table 3 Northern Pump Station (PS2) Key Design Parameters

Description	Project Baseline	Baseline Modifications
Pump station rated capacity (cfs)	132	8
Pump station rated capacity (mgd)	85	5
Pump Configuration	2 high-flow + 1 standby 2 low-flow	2 low-flow + 1 standby
Pump Operation	2 high-flow duty	2 low-flow duty + 1 standby
Pump Type	submersible axial with VFDs	submersible with VFDs
Flow Capacity	(3) 50 mgd (2) 5 mgd	(3) 2.5 mgd
Pump motor, horsepower (hp)	200 hp (high flow) 60 hp (low flow)	25 hp
Emergency generator capacity (kw)	1200	100
Proposed location	Thompsons Alley waterfront	211 N. Union St. parking lot
Aboveground dimensions	30' x 30'	29.5' x 26'
Underground dimensions	68' L x 37' W x 22.5' depth	45' L x 12' W x 20' depth

Figures 7 and 8 show two proposed layouts for the aboveground area of PS2; Figure 7 maintains the existing emergency generator and fuel tank location whereas Figure 8 assumes these components can be relocated to an alternative location. As shown in both figures, opportunities to resize and reconfigure the aboveground dimensions are a direct result of reduced electrical demand, including smaller electrical distribution equipment and smaller bar screens. The resized electrical room – 17'-8" x 16' – consists of the three VFDs assumed to be internal to the three-section motor control center (MCC) and a smaller automatic transfer switch (ATS) panel (in comparison to the baseline). In Figure 7, the space requirement of the emergency generator is less due to the capacity reduction from 1200 KW to 100 KW. The two bar screen screens sized for a peak flow of about 16 mgd into the wet well require a horizontal footprint up to 11-feet by 8-feet, which is the most conservative estimate. **Overall, the PS2 aboveground footprint may be reduced by approximately 15 percent in comparison to the baseline assuming the emergency generator and fuel tank remain at PS2**.

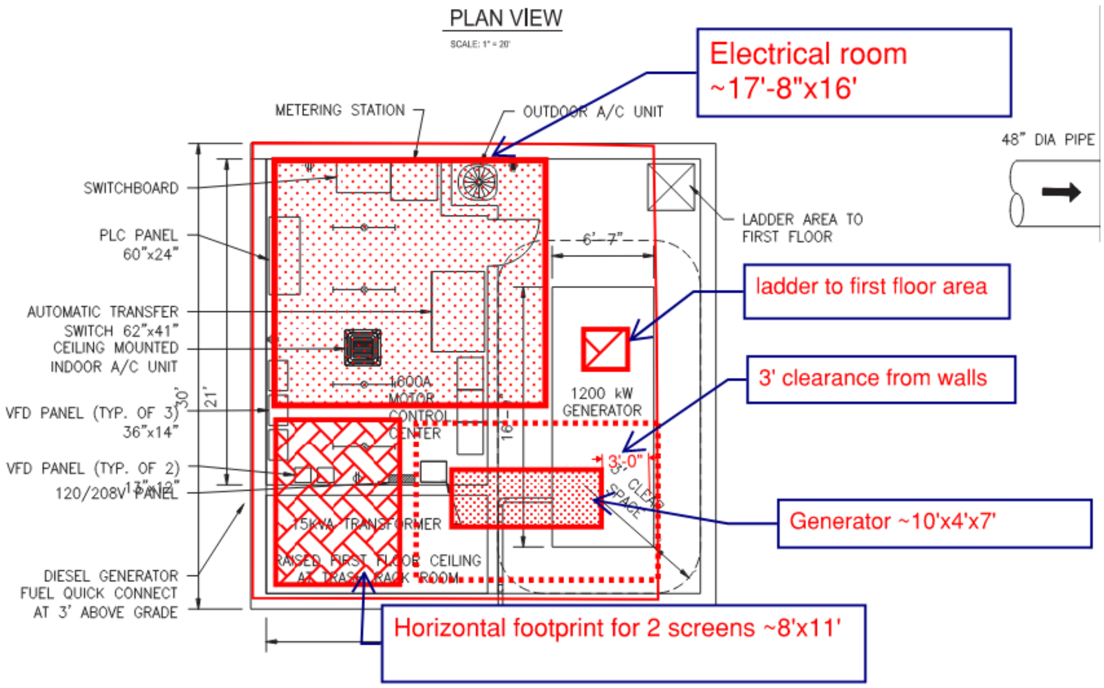


Figure 7 Layout Showing Modifications to the Aboveground PS2 Footprint

One additional opportunity to reduce the aboveground footprint of PS2 includes relocating the emergency generator to an alternative location. There are no known regulations, codes or laws that stipulate the distance requirements between the ATS (in the PS2 electrical room) and the generator, however, the selected siting location shall keep the following in mind:

- Site the ATS and emergency generator as close as possible due to the required cable/conduit run between the two. Increasing the distance between will require additional cable/conduit and associated site work for the newly buried electrical line.
- Provide adequate access for a fuel truck, and therefore, site as close to a roadway as possible.
- Ensure sufficient clearance around the emergency generator and fuel tank. Clearances are based upon necessary maintenance space, fire testing of the generator enclosure and the manufacturer's requirement for air flow for proper operation, and national and local codes.
- Comply with Alexandria (Fairfax County) noise ordinances. Carollo confirmed that siting a Generac generator with a Level 2 enclosure at the 211 N Union Street Parking Lot would not be violation of the ordinance[2].
- Installed at the base flood elevation (BFE) + 1-foot or El. + 11.2-feet. The Baseline proposed El. +10.2 -feet is insufficient per the adapted 2018 Virginia Building Code. This update enforces American Society of Civil Engineers (ASCE) 24-14 such that the outdoor emergency generator must be higher under a Class 3 category.
- Provide adequate security for safety purposes around the emergency generator and fuel tank.
- Keep level and secure since the generator may vibrate during operation. It must be secured to a permanent structure, e.g., concrete pad, that can support the weight of the emergency generator and fuel tank.

[2] The Fairfax County residential noise ordinance is 80 dB(A) 10pm to 7am. The Generac generator with a Level 2 enclosure has a maximum sound rating of 77 dB(A) at 23-feet away. The residential houses across N Union Street are approximately 50-feet away, and therefore, the generator would not violate the ordinance.

For planning purposes Carollo recommends a minimum footprint of 20-feet by 12-feet and a vertical height of 15-feet to accommodate the generator and subbase fuel tank. It is assumed this area is also free of trees, shrubs, and vegetation that could obstruct air flow. The finished grade of the location must be at El. +11.2 feet or higher.

Figure 8 assumes the emergency generator is relocated offsite. Since the space below the generator was unoccupied, both the first and second floor of PS2 can be removed, and an access hatch shall be installed at grade to facilitate the removal of pumps from the wet well as needed. **If an appropriate location for the generator (and diesel fuel tank) can be identified and secured, then the aboveground footprint of PS2 can be reduced by one-third or 33 percent in comparison to the baseline.**

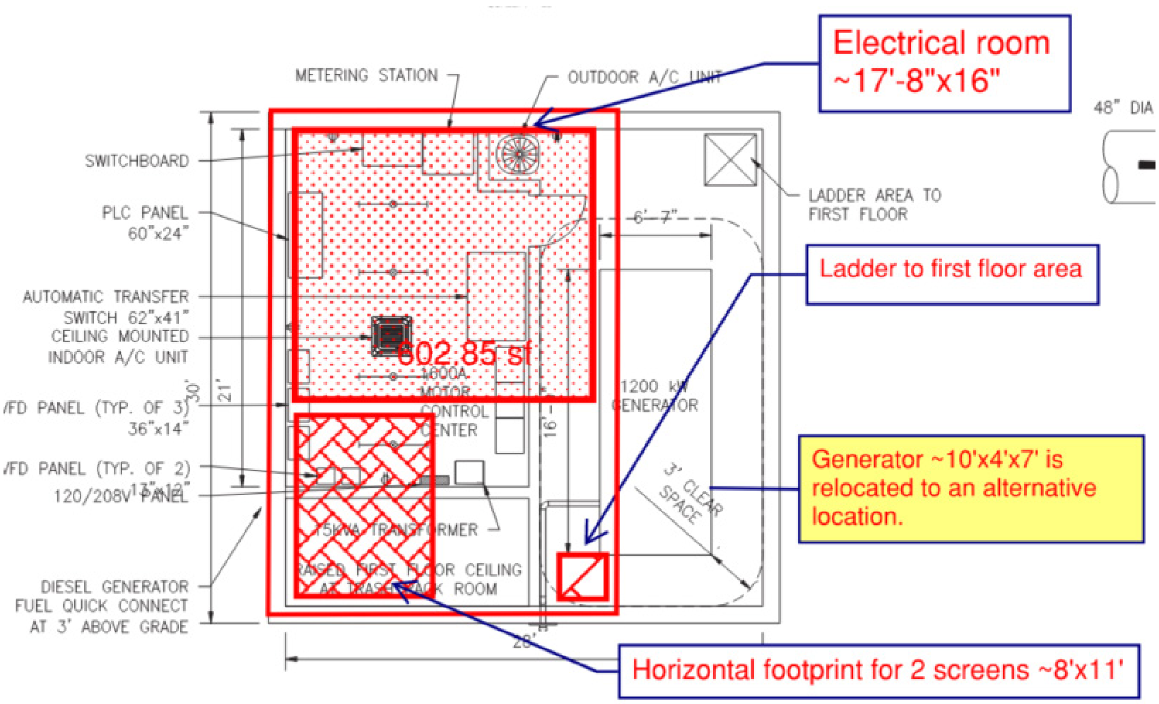


Figure 8 Layout Showing Modifications to Aboveground PS2 Footprint with Generator Relocated

Figure 9 shows the proposed layout for the belowground area of PS2, including the revised dimensions of the wet well, check valve vault and screenings room. The width of the wet well can be reduced from 37-feet in the Baseline Project to 12-feet. This is because Carollo recommends three smaller, standard submersible pumps as opposed to the five submersible axial pumps in the Baseline Project. The decreased width of the wet well subsequently decreases the width of the room that houses the screens as well as the check valve vault. The diameter of the discharge pipes required for the low flow pumps is 12-inches and the outfall pipes can be reduced from 60-inches to 16-inches because of the lower discharge rate. Due to the reduced pipe sizes, the 13-feet (L) by 10-feet (W) energy dissipator can also be reduced to 6-feet (L) by 6-feet (W). **Overall, the underground footprint is reduced by 70 percent, and the modified layout offers greater siting flexibility in comparison to the baseline.**

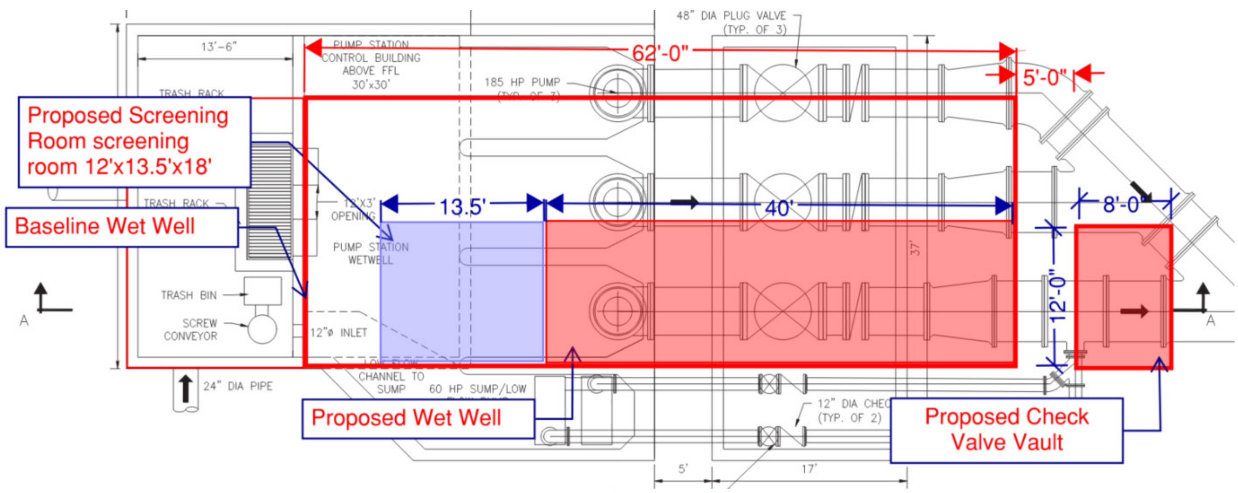


Figure 9 Layout Showing Modifications Made to the Belowground PS2 Footprint

As shown in Figure 10, it is anticipated that the modified pump station footprint will allow for installation of the pump station and wet well within the existing filled area west of the bulkhead. This will eliminate the need for dredging and filling of a section of the river to accommodate the baseline pump station and limiting site disturbance during construction.

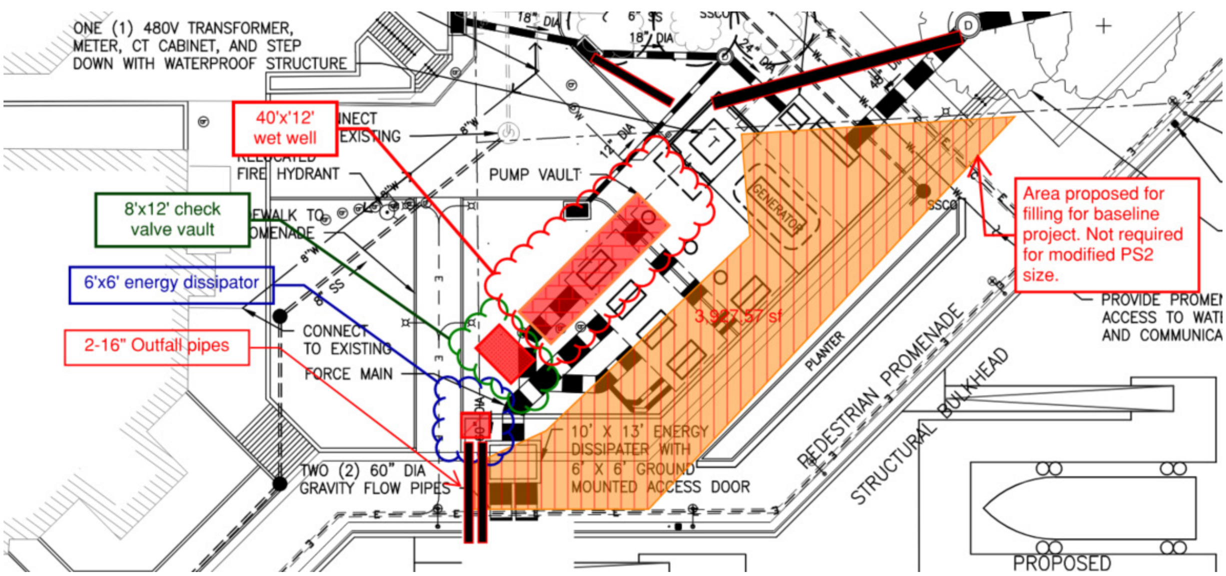
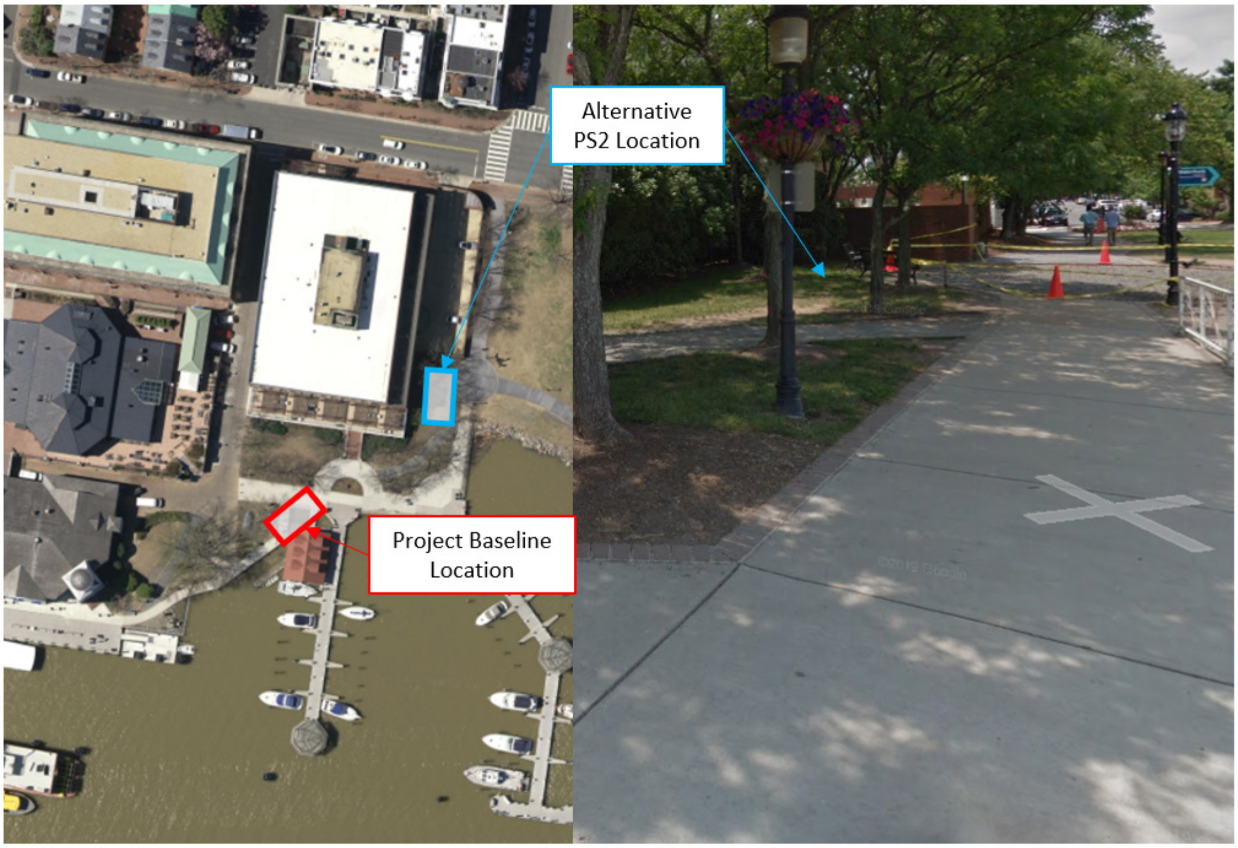


Figure 10 Northern Pump Station (PS2) Layout within Existing Shoreline

Alternatively, the pump station could maintain a similar configuration to the baseline project but shift the location more landward. The potential alternative location is adjacent to the Office Building (211 N Union Street) at the foot of Queen Street. This alternative location is under consideration since it will conceal this smaller dewatering pump station from public view. Figure 11 shows PS2's project baseline and alternative location.



Source: City of Alexandria, GIS Open Data Hub; Google Maps

Figure 11 Northern Pump Station (PS2) Alternative Location

This alternative location for PS2 requires additional evaluation and validation by the field investigations to be executed under this project, e.g., by the Phase II geotechnical investigation to confirm the soils can support the structure and by the survey to confirm property line boundaries. In addition, this alternative requires a review by the City of Alexandria to confirm easements and land use, and coordination with the adjacent property owners to formulate access for pump station operation and maintenance and for fueling of the standby generator.

1.9 Operation & Maintenance

The City is considering various long-term O&M options for the stormwater pumping stations that will be implemented as part of the Project:

1. Training of existing City staff or hiring new staff to perform O&M.
2. Training of existing City staff or hiring new staff to perform operations and contracting with equipment suppliers or other third-party entity to perform maintenance services, e.g., maintenance service contract.
3. City procuring and contracting with a third-party service provider to perform O&M services, e.g., fee for service utility.
4. City entering into an agreement with another local municipality to perform O&M.

To fully understand the feasibility of each option, the City is connecting with its internal operations group (Department of Transportation and Environmental Services) and various external entities that could serve in one of these roles. A separate memorandum will document the findings and recommendations for long-term O&M.

Note that the Project is not pursuing a progressive design build and operations (PDB-O) delivery; in this delivery method, the design builder would be responsible for long-term operations. However, under a progressive design build (PDB) delivery the design-builder may procure, or assist the City to procure, a third-party O&M service provider and/or amend its contract to provide short-term O&M (to occur through negotiations during Phase 1).

1. Regardless of who or what entity assumes responsibility for operations and/or maintenance, the operational intent is as follows:
2. The pumping stations will operate on an intermittent basis, i.e., during a wet weather event and/or following a wet weather event. The exact timing of operation will depend on rainfall-runoff characteristics of the storm, the extent of a floodwall breach (if any), and the inclusion of underground stormwater detention chambers (if any).
3. The pumping stations are responsible for evacuating the water that drains to the "Core Area"; it will operate with each wet weather event and/or extreme tide event to fully evacuate the water.
4. The pumping stations are unmanned.
5. The pumping stations will incorporate instrumentation and controls (I&C), so pumps, screens, and other critical equipment will not require human intervention to turn on/off/adjust.
6. The pumping stations will incorporate remote operation so monitoring the wet well, pumps, and other critical equipment may occur offsite.
7. Major equipment that requires O&M are:
    a. Pumps
    b. Screens
    c. Generator
8. Other O&M considerations are I&C/programming, electrical equipment, security and general building maintenance.

Table 4 outlines some general O&M activities as follows:

Table 4 Operations and Maintenance Expectations Relative to a Storm Event

Time Relative to a Storm Event	Operations and Maintenance Expectations
Before	Maintenance staff should visit each pump station and storage chamber access points to ensure valves, sensors, and pumps are in place and functional.
During	Monitoring of equipment including pressure gages, wet well levels, valves, and pumps.
After	Visit the pump station during a 24-hour period following the storm event, to open the gates of the storage chambers, attend fuel delivery to the standby generator as needed, and to remove screenings collected from the screens.

Time Relative to a Storm Event	Operations and Maintenance Expectations
Routine Maintenance	Exercise pumps, valves, and standby generator per manufacturer's recommendations. Monthly visit to each pump station and chamber access points to verify functionality of equipment and controls, and verify generator is fueled. Inspect storage chambers isolator row bi-annually, with one inspection occurring in the spring after the winder loading of salt/sand. Inspections are to be performed through the access manholes and inspection ports along the length of the isolator row. Clean the underground storage chambers system using a vacuum system when approximately 3-inches of sediment has accumulated throughout the length of the isolator row.

O&M Notes:
(1) Use of telemetry, automated, and remote operation shall be considered and discussed with the operating entity.
(2) Operation of the underground storage chambers is detailed in a separate memorandum describing the design parameters. In general, it is intended that automated or manual gate valves are used to control release of the stored volume into the pump station wet well.
(3) Correct functioning of the underground storage chambers, including proper maintenance to keep them free from obstructions, will be critical to the proper operation of the pump stations.

1.10 Cost

Carollo reviewed the year 2020 Class 4 direct construction costs for the project baseline and introduced the modifications proposed in this memorandum for comparison. Table 5 summarizes the costs for both pump stations. Costs for the storm sewer optimization and underground storage chambers are included in the memoranda for each component.

Table 5 Pump Stations Direct Cost Summary

Description	Project Baseline	Proposed Modifications
Southern Pump Station (PS1)	$7,700,000	$6,100,00
Northern Pump Station (PS2)	$8,200,000	$3,900,000
Total	**$15,900,000**	**$10,000,000**

1.11 Conclusions and Recommendations

Carollo's review determined that pump stations are required to evacuate the stormwater collected and conveyed to City of Alexandria's Waterfront and cannot be eliminated. Given that the project baseline pump stations are sized to handle the storm peak as it enters the pump stations, adding stormwater storage capacity prior to entering the pump station will reduce the inflow storm peak, which will require a smaller pumping capacity.

Hydraulic modeling results indicate that adding underground storage chambers at the Waterfront Park and at Founders Park will effectively reduce the sizes of PS1 and PS2 with the most significant reduction anticipated for PS2. Direct constructions costs for each pump station are also reduced; however, the cost of the underground storage chambers needs to be factored in along with the cost of other project elements, to determine its cost-effectiveness.

However, the environmental benefits of incorporating the chambers and thus, reducing the pumping station capacities shall not be overlooked. Simply put, a reduced peak energy demand for PS1 and PS2 directly translates to lower demand charges, lower energy costs, and less greenhouse gas emissions (GHG) emissions. **Therefore, the recommended comprehensive stormwater management solution, inclusive of stormwater detention chambers and pumping stations among other improvements, demonstrates a more resilient, forward thinking approach to mitigating rainfall induced flooding.**

Installing underground storage chambers at the Waterfront and Founders parks to reduce the stormwater pump station capacities and ultimately alleviate street flooding will require the following operational considerations:

- A controlled release of storm water to PS2 after a storm event will keep its footprint small.
- The PS1 footprint could be made slightly smaller, but overall footprint will be closer to the Baseline Project.
- Additional adjustments to the location, depth, and size of the underground storage chambers require validation through hydraulic modeling to confirm PS sizing.
- Coordination with the City of Alexandria and other stakeholders is required to vet the alternative location for PS2.

Lastly and during design, it is recommended that the Design Builder model each pump station with computational fluid dynamics (CFD) modeling which could further optimize the pump station footprint and reduce overall costs.

1.12 References

City of Alexandria IDF Curves:

1. "Comparison of Alexandria's Storm Design Criteria to Neighboring Jurisdictions," May 1, 2009, CH2M Hill. See Appendix A.

City of Alexandria Stormwater Model Documentation:

2. Stantec. November 2018. "Master Storm Water Management Plan." Alexandria VA: The City of Alexandria.

Other Referenced Technical Documentation Prepared by Carollo Engineers:

3. Carollo Engineers. May 2022. *Design Storm Selection for Hydrologic and Hydraulic Modeling*. Alexandria VA: The City of Alexandria.
4. Carollo Engineers. May 2018. *Storm Sewer System Collection Upgrades.* Alexandria VA: The City of Alexandria.
5. Carollo Engineers. April 2018. *Parkspace and Streetscape Storage Solutions.* Alexandria VA: The City of Alexandria.
6. Carollo Engineers. May 2022. *Potomac River Flood Frequency Analysis.* Alexandria VA: The City of Alexandria.
7. Carollo Engineers. Forthcoming 2022. *Geotechnical Design Memorandum.* Alexandria VA: The City of Alexandria.