Open Space Institute

Essex Hudson Greenway

Stormwater Feasibility Study

Draft | June 24, 2022

This report takes into account the particular instructions and requirements of our client. It is not intended for and should not be relied upon by any third party and no responsibility is undertaken to any third party.

Job number 277106

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Executive Summary

The Essex Hudson Greenway project involves converting approximately 7 miles of old unused railroad to a greenway in order to improve community engagement and the environment. Within this overarching goal, there are many opportunities to implement better stormwater management practices that would increase stormwater capture and decrease excessive discharge rates during major storm events. Various locations for potential underground stormwater storage tanks have been identified in the cities of Newark, Kearny, Belleville, and Jersey City. Within these four locations, feasible options for storage sites have been identified by Mathews Nielson Landscape Architects (MNLA) based on preliminary topography analysis and capacity demands.

The goal of this report seeks to further analyze the feasibility of these options in Newark and Kearny by identifying suitable connection points to existing utility mains in the area. Catchment areas were identified for the various locations based on topography and the existing underground utility network, and the Rational Method was used to find the volume of storage that would be present at each location during a 10-year Design Storm. Using this information, the size and configuration of various storage systems were developed along with their respective connections to and from the existing municipal lines.

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1. Project Description

This feasibility report is a continuation of the Essex Hudson Greenway Framework Plan that illustrates opportunities to transform the Old Boonton Line in New Jersey from a railroad to a recreation trail. There are many opportunities to establish underground stormwater tanks along the rail line to help reduce peak discharge volumes during large storm events. The goal of this report is to analyze the feasibility of installing stormwater detention tanks in various locations in both Newark and Kearny. These cities were both included in the original scope of this project for stormwater feasibility analysis, however after encountering an impasse with the City of Kearny, the scope has narrowed to the City of Newark.

To conduct the feasibility analysis, we obtained public record documents from various sources located in the City of Newark. These drawings delineated existing utilities in the area, combined sewer overflow (CSO) locations and ground topography. Using this information 4 distinct locations were analyzed for stormwater detention. Each location considered a range of storage options and connections to the existing municipal systems.

2. Agency Outreach

Public record information was requested from various sources to establish base maps and determine the existing utilities in the area. Initially, two OPRA (Open Public Records Act) forms\(^1\) were sent to the City of Kearny and the City of Newark. The City of Kearny denied the request, stating that providing access to this information would cause a risk to public safety. As a result, records from Kearny municipalities were not obtained, and the scope narrowed to the City of Newark singularly.

Newark Sewer and Water (NSW) provided GIS images and hand drawn pdfs showing various utility main locations in the public Right-of-Way. Systems shown include combined storm and sanitary systems (CSS), storm drains (ST), and sanitary sewers (SS). Pipe sizes, materials and flow direction information were obtained from these GIS images. In addition to agency outreach, Open Space Institute (OSI) provided survey data and MNLA coordinated the public database GIS information. Essex and Hudson County and Passaic Valley Sewer Commission were contacted, however no information was obtained.

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\(^1\) OPRA | How to Request Government Records? (nj.gov)
3. Existing Conditions

3.1 Existing Utilities

Existing utilities in the area include water, sanitary sewer, storm drain and combined storm and sanitary. The sizes and locations of these systems are shown Figure 1 below. Additional information can be found in Appendix A.

Figure 1 Existing Utility Plan
3.2 Catchment Areas

Using the existing pipe network data from Newark Sewer and Water and available GIS and Lidar cloud points, catchment areas were delineated. The natural topography of the land in this area slopes east to the Passaic River. All 4 study locations lie within the same catchment basin, which eventually discharges into the Passaic River or CSO location. The overall catchment area has been divided into sub-catchments based on the type of system (CSS or Storm) and the downstream storage location. These sub catchments can be seen in Figure 2 below. Refer to Appendix B for the full catchment area exhibit.

![Figure 2 Catchment Area Exhibit](image-url)
3.3 Existing Land Cover

The existing site is primarily residential with sparse vegetation and landscaping. The abandoned railway itself is a mix of pavement, dirt, and gravel. For the purposes of this study, the gravel, dirt, and landscaping areas are assumed to be pervious, while the asphalt, pavement and residential properties are assumed to be impervious. The land cover areas are highlighted in Figure 3 below. Using information from Title XLI Zoning and Land Use, the impervious runoff coefficient is taken as 0.95 and the pervious runoff coefficient as 0.1. Refer to Appendix C for additional information.

Figure 3 Land Cover Exhibit
4. Stormwater analysis

4.1 Rational Method Analysis

The Rational Method was utilized in this analysis to generate the volume of storage that each study area would be able to produce. The following equation is used to predict the peak surface runoff for each catchment area:

\[ Q = cIA \]

Where:

- \( Q \) = Peak surface runoff (cubic feet per second)
- \( c \) = surface runoff coefficient
- \( I \) = rainfall intensity (inches per hour)
- \( A \) = catchment basin area (acres)

A conservative value of 0.95 is used for the impervious surface runoff coefficient \((c_i)\) and a value of 0.1 for the pervious surface runoff coefficient \((c_p)\). These values provide conservative results for \( Q \). From these values, a weighted surface runoff coefficient was calculated for each catchment area using the following equation:

\[ \frac{(c_iA_i) \times (c_pA_p)}{A_T} = c_w \]

Where:

- \( c_w \) = weighted coefficient
- \( A_T \) = total area of catchment (acres)
- \( A_i \) = impervious area of catchment (acres)
- \( A_p \) = pervious area of catchment (acres)

According to the City of Newark Title XLI Zoning and Land Use Regulation, the amount of runoff shall be calculated based on a Design Storm with a ten-year return frequency for Essex County with a minimum initial time of concentration of 10 minutes. Consequently, a value of 0.837 in/hr was used for \( I \) as calculated by NOAA tables (see Appendix G).

To obtain a value for \( A \) (acres), the project area was divided into approximate catchment areas based on the local topography shown by Lidar GIS data and directional flow in the existing pipe networks. After

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3 City of Newark, NJ Storm Drainage (ecode360.com), City of Newark Chapter 41:17 Storm Drainage, 06/15/2022
4 City of Newark, NJ Storm Drainage (ecode360.com), City of Newark Chapter 41:17 Storm Drainage, 06/15/2022
5 PF Map: Contiguous US (noaa.gov)
performing an iterative calculation, values for $V$ (volume of runoff in gallons) were obtained using the following equations and conversions:

$$V = Q \times \frac{7.48 \text{ gal}}{\text{cf}} \times T_c$$

A time constant ($T_c$) of 600s was used per Design Storm length. Using these values for $V$, the size and configuration of suitable stormwater systems were chosen. These calculations are summarized in Table 1 below. Additional calculations can be found in Appendix D.

<table>
<thead>
<tr>
<th>Sub-Catchment Area</th>
<th>A (ac)</th>
<th>Cw</th>
<th>I (in/hr)</th>
<th>Q (cfs)</th>
<th>Tc (s)</th>
<th>V (cf)</th>
<th>V (gal)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>34.1</td>
<td>0.88</td>
<td>0.837</td>
<td>25.0</td>
<td>600</td>
<td>15016</td>
<td>112318</td>
</tr>
<tr>
<td>2</td>
<td>60.9</td>
<td>0.94</td>
<td>0.837</td>
<td>47.9</td>
<td>600</td>
<td>28739</td>
<td>214968</td>
</tr>
<tr>
<td>3</td>
<td>4.1</td>
<td>0.95</td>
<td>0.837</td>
<td>3.3</td>
<td>600</td>
<td>1975</td>
<td>14774</td>
</tr>
<tr>
<td>4</td>
<td>5.2</td>
<td>0.95</td>
<td>0.837</td>
<td>4.1</td>
<td>600</td>
<td>2481</td>
<td>18557</td>
</tr>
<tr>
<td>5</td>
<td>12.2</td>
<td>0.79</td>
<td>0.837</td>
<td>8.1</td>
<td>600</td>
<td>4848</td>
<td>36265</td>
</tr>
<tr>
<td>6</td>
<td>9.9</td>
<td>0.91</td>
<td>0.837</td>
<td>7.5</td>
<td>600</td>
<td>4505</td>
<td>33697</td>
</tr>
<tr>
<td>TOTAL</td>
<td>126.5</td>
<td>0.91</td>
<td>0.837</td>
<td>95.9</td>
<td>600</td>
<td>57564</td>
<td>430578</td>
</tr>
</tbody>
</table>

5. Design Considerations

5.1 Nature of Storage

One of the design considerations is the nature of the storage. There are many different options for underground stormwater detention. Examples include subsurface vaults or tanks, large-diameter pipes, and infiltration systems. For this project, HDPE pipes have been considered a suitable material for storage, due to its widespread availability and cost-effectiveness. Infiltration systems were not considered feasible due to the combined sewer system.

5.2 Pipe Configurations

The specific layout for the pipes will depend on several factors. Firstly, the pipe diameter will determine not only the amount of capacity held in each pipe, but also the depth requirements for the system. Larger pipes will require more cover. See Error! Reference source not found. on page Error! Bookmark not defined. for minimum and maximum coverage for this specific project. This information was taken from MNLA Stormwater Siting Report published on August 19, 2020. Geotechnical guidance will be needed
to analyze the specific chosen area before a certain depth can be confirmed to avoid any bedrock or other obstructions for these utilities. In general, larger pipes will be most cost-effective.

Table 2 HDPE System Sizing (from Essex Hudson Greenway Stormwater Storage Siting Studies by MNLA in 2020)

<table>
<thead>
<tr>
<th>NOM. PIPE DIA. (IN)</th>
<th>GALLONS PER FOOT (GPF)</th>
<th>MIN COVER (FT)</th>
<th>MAX COVER (FT)</th>
<th>COVER</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>36</td>
<td>1.0</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>36</td>
<td>51</td>
<td>1.0</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>42</td>
<td>70</td>
<td>1.0</td>
<td>50</td>
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<tr>
<td>48</td>
<td>91</td>
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<td>30</td>
<td></td>
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<tr>
<td>54</td>
<td>115</td>
<td>1.0</td>
<td>30</td>
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<tr>
<td>60</td>
<td>143</td>
<td>1.0</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>66</td>
<td>172</td>
<td>1.5</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>72</td>
<td>205</td>
<td>1.5</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>84</td>
<td>279</td>
<td>2.0</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>96</td>
<td>364</td>
<td>2.0</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>108</td>
<td>476</td>
<td>2.5</td>
<td>25</td>
<td></td>
</tr>
<tr>
<td>120</td>
<td>569</td>
<td>3.0</td>
<td>25</td>
<td></td>
</tr>
</tbody>
</table>

Secondly, the location chosen for the storage pipes must not have too many space constraints, as to allow for a suitable number of parallel pipes to the rail line centerline. Minimum 15’ offsets will be provided for surrounding buildings, existing utilities and trees. Furthermore, if a pump is needed at a certain location there must be adequate space and access to the site.

5.3 Pump Consideration

There are configuration options in certain locations that may require a pump to increase the amount of runoff available to enter the underground system. This may also happen if an invert of an existing utility is too high to serve as an outfall for the storage system. A pump is not an ideal solution as it would require additional maintenance and adequate space for installation. It will increase the lifetime costs of the project, but could be effective at reducing overflows and consequent fines.
5.4 Location Options

This section will highlight the four options and the opportunities and constraints of each location. Recommended designs are based on the available utility connection points nearby and the existing topography in the area. In each location, there are endless possibilities and iterations of configurations, as pipe diameter, length and number are all changeable. Our recommendation is based on a system that would be cost-effective and collects an adequate amount of discharge. Alternate designs based on reduced pipe diameter are also suggested.

It is assumed that we can only outfall into combined sanitary system and storm mains and not sewer mains. Additionally, it is assumed that we can store storm and combined flows separately in the tanks which have a potential inflow from both storm and combined mains. This will allow the system to outflow to the storm system as well as the combined system to reduce overflow and avoid contamination of untreated storm mains. If there is no feasible storm main outfall connection in the location’s vicinity, it is assumed all flow will enter the combined system.

5.4.1 Location A

Location A is located south of Highland Avenue and Greenwood Lake St within the railroad corridor. Below is an outline of the location’s constraints and design recommendations. Figure 4 outlines the proposed location and the required connections to the existing systems. (Refer to Appendix E for large scale exhibit).

Constraints and Location Features

- Location A has space constraints of trees, existing sanitary sewer (SS), existing water (W) with a 15’ offset from these constraints. Within 100’ ROW, there is a 70’ width and 190’ length to place storage system
- Connections from municipal system into the storage system
  - Highland Avenue and Verona Avenue (Combined Storm and Sanitary (CSS))
  - Highland Avenue and Tiffany Boulevard (Storm (ST))
- Connections from storage system back into municipal system
  - Mt Prospect Avenue and Verona Avenue (CSS)
  - Ruby Place and Tiffany Boulevard (ST)
  - This connection may require a pump due to higher municipal inverts
- Existing ground elevation of the tank location is 30’ above sea level.
- According to national standards, the self-cleansing minimum slope of a 10” pipe is 0.24% which is the critical scenario (pipes could be increased in size and therefore, decrease slope and invert). Setting the pipe connecting into the tank at this slope (0.3%) we obtain a maximum invert for a gravity system. Therefore, the maximum invert is approximately 20.75’ above MSL.
- This invert allows for a maximum diameter of 7’ including the 2.0’ minimum clearance requirement.

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**Recommended Design**
- Given all the above listed constraints, a system designed with (6) 7’ Dia. X 190’ pipes will allow a storage of 188,000 gallons.
- Connect storage system outlet pipe to the CSS at Verona Avenue and Mt Prospect Avenue.
- Possible connection to ST in Tiffany Blvd, but pump will be required.

**Alternative Design Considerations**
- Smaller diameter storage pipes may be used within the stated boundaries and constraints. If needed, system may be given steeper inlet pipes. The minimum recommended size is 3’, as beyond this size the capacity becomes less cost-effective per sq ft pipe.
  - (14) 3’ Dia. X 190’ diameter pipes give capacity of 188,000 gallons (same as 10’) but will be ultimately more expensive due to increased material usage and similar depth.
- The system could connect the outfall to the existing ST system at Tiffany Boulevard and Ruby Place however, this option would require a pump (approx. 4’ head).
Figure 4 Location A Plan View
5.4.2 Location B

Location B is located northeast of Verona Ave and Mt Prospect Ave within the railroad corridor. Below is an outline of the location’s constraints and design recommendations. Error! Reference source not found.5 outlines the proposed location and the required connections to the existing systems. (Refer to Appendix E for large scale exhibit).

Constraints and Location Features
- Constraints include 15’ offset from buildings and from Mt Prospect Avenue.
- Connections from municipal system into the storage system
  - Mt Prospect Avenue and Verona Avenue (CSS)
  - Mt Prospect Avenue and Tiffany Boulevard (ST)
    - This connection may require a pump due to higher municipal inverts
- Connections from storage system back into municipal system
  - Summer Avenue and Verona Avenue (CSS)
  - There are no feasible outfall connections to ST or SS available in this area.
- Existing ground elevation of the tank location is 24.5’ above sea level.
- The connecting inverts in this location may be difficult because this location is a higher point in the system.
- Given minimum slope of 0.2% from point of connection in Summer Avenue, maximum invert depth is 15.80’.

Recommended Design
- (6) 6’X275’ provides 230,000 gallons storage. This location is relatively flexible and the length can be extended if desired.
- Possible inlet from ST in Mt Prospect Ave, but pump will be required.

Alternative Design Considerations
- Depending on minimum slopes and potholing inverts, 3’ diameter pipes could be installed if a steeper connecting pipe slope is required to reach self-cleansing velocity without losing capacity
  - (14) 3’X275’ = 271,000 gallons.
  - To include storm system connection at Mt Prospect Avenue and Tiffany Boulevard, a pump may be required with 2’ head and 600’ pipe. This would add between 36,000 and 70,000 gallons of stormwater flow.
Figure 5: Location B Plan View
5.4.3 Location C
Location C is located northeast of Verona Ave and Summer Ave within the railroad corridor. Below is an outline of the location’s constraints and design recommendations. Error! Reference source not found.6 outlines the proposed location and the required connections to the existing systems. (Refer to Appendix E for large scale exhibit).

Constraints and Location Features
- Flexibility in this location with pipe sizes and length of storage pipes
- Connection from municipal system into the storage system
  - Summer Avenue and Verona Avenue (CSS)
- Connection from storage system back into municipal system
  - Broadway Avenue and Verona Avenue (CSS)
- Existing ground elevation of the tank location is 21.5’ above sea level.
- For 10’pipe diameter, invert of 8’ needed.

Recommended Design
- (4) 10’X 200’ pipe system retains 188,000 gallons. Again, location space is abundant, and design is flexible. Pipe length can be added for more storage.

Alternative Design Considerations
- Depending on minimum slopes and potholing inverts, 3’ diameter pipes could be installed if a steeper connecting pipe slope is required to reach self-cleansing velocity without losing capacity
  - (14) 3’X200’ = 200,000 gallons.
5.4.4 Location D

Location D is located northwest of Verona Ave and Broadway Ave within the railroad corridor. Below is an outline of the location’s constraints and design recommendations. Error! Reference source not found. 7 outlines the proposed location and the required connections to the existing systems. (Refer to Appendix E for large scale exhibit).

**Constraints and Location Features**
- Availability to be flexible with length to achieve much higher capacity.
- Connection from municipal system into the storage system
  - Lincoln Avenue and Verona Avenue (CSS)
- Connection from storage system back into municipal system
  - Broadway Avenue and Verona Avenue (CSS)
- Existing ground elevation of the tank location is 17’ above sea level.
- This location will clash with construction of Location C due to invert and connection locations.

**Recommended Design**
- (4) 10’X400’ = 360,000 gallon capacity.

**Alternative Design Configuration**
- 10’ diameter pipes are the recommendation here due to the abundance of space and ground cover.
Figure 5 Location D Plan View
6. Bill of Quantities

The following table outlines the approximate bill of quantities for each location:

Table 3 Bill of Quantities Estimate

<table>
<thead>
<tr>
<th>Material</th>
<th>Location A</th>
<th>Location B</th>
<th>Location C</th>
<th>Location D</th>
</tr>
</thead>
<tbody>
<tr>
<td>PVC Pipe (LF)</td>
<td>2250</td>
<td>870</td>
<td>1525</td>
<td>410</td>
</tr>
<tr>
<td>PVC Forced Pipe Main (LF)</td>
<td>660</td>
<td>600</td>
<td></td>
<td></td>
</tr>
<tr>
<td>HDPE Storage Pipe (7’) (LF)</td>
<td>1140</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HDPE Storage Pipe (6’) (LF)</td>
<td></td>
<td>1650</td>
<td></td>
<td></td>
</tr>
<tr>
<td>HDPE Storage Pipe (10’) (LF)</td>
<td></td>
<td></td>
<td>760</td>
<td>1600</td>
</tr>
</tbody>
</table>

7. Recommendations & Conclusion

In conclusion, all four locations are feasible stormwater collection sites. As depicted on the exhibits in Appendix E, the inverts of the existing utility municipal system are such to allow gravity flow for the CSS incoming and outgoing pipes. In most cases a pump will be needed to include stormwater in the collection. Consequently, it will be more costly to install a system with only stormwater and it is recommended to design for underground storage of both sanitary sewage and stormwater. Additionally, stormwater is roughly only 15% of overall flows that the system could potentially capture. Including combined flows in this project will increase the effectiveness of reducing overflows, however these flows should be treated prior to entering the tanks.

Many different configurations and combinations are possible in each location. A cost-based analysis in combination with a Geotechnical Report is recommended to determine which configurations are feasible. In general, of the approximate 430,000 gallons of flow the system could potentially capture, each location will be able to store at least 188,000 gallons. Designs can be sized up in locations B, C, and D for increased capacity, should this be required and within budget. Generally, larger diameter pipes will be more cost-effective per gallon of storage.

Due to size and space constraints, this report recommends location D as the most effective location for a stormwater storage system. Its convenient downstream location means it will pick up majority of the flows in the basin. It is possible to capture all the flows of the 4 other systems in this location given a certain configuration. This location has ample space to size up a storage system, and inverts in this area are such that gravity flow is possible with forgiving pipe slopes. However, only combined flow capture is possible in this area, as there is no existing storm drain infrastructure near pick up.
An approximate bill of quantities for this area is included in Table 4:

**Table 4 Bill of Quantities Estimate Location D Expansion**

<table>
<thead>
<tr>
<th>Material</th>
<th>Location A</th>
<th>Location B</th>
<th>Location C</th>
<th>Location D</th>
<th>Location D Only</th>
</tr>
</thead>
<tbody>
<tr>
<td>PVC Pipe (LF)</td>
<td>2250</td>
<td>870</td>
<td>1525</td>
<td>410</td>
<td>410</td>
</tr>
<tr>
<td>PVC Forced Pipe Main (LF)</td>
<td>660</td>
<td>600</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HDPE Storage Pipe (7') (LF)</td>
<td>1140</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HDPE Storage Pipe (6') (LF)</td>
<td></td>
<td></td>
<td>1650</td>
<td></td>
<td></td>
</tr>
<tr>
<td>HDPE Storage Pipe (10') (LF)</td>
<td></td>
<td></td>
<td></td>
<td>760</td>
<td>1600</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2000</td>
</tr>
</tbody>
</table>

Only adding (1) 400 linear foot 10’ diameter HDPE pipe to the configuration laid out in Section 5.4.4, the entire basin upstream could be captured. See Figure 8 below for configuration.
Figure 7 Location D Expansion Catchment Area
Appendix A: Existing Utility Exhibits
Appendix B: Catchment Area Exhibit
Appendix C: Existing Land Cover Exhibit
Appendix D: Calculations
Calculations:

Pipe Slope Calculations:
The pipe slope is calculated in the following manner:

$$\frac{\text{Rise}}{\text{Run}} \times 100 = \text{slope} \%$$

The critical pipe slope is known as the ‘self-cleansing velocity’. This is the minimum pipe slope allowed before blockages will occur. The following equation determines the velocity of a pipe:

$$\frac{1.49 \times (R^2 \times S^\frac{1}{2})}{n} = v$$

Where

- $v$ = velocity (m/s)
- $R$ = hydraulic radius pipe (m)
- $S$ = pipe slope
- $n$ = Manning’s coefficient (0.011 for PVC)

Critical slope for 10” pipe is 0.6 m/s for sewer and 0.725 m/s for storm.

Location Capacity Calculations:
Overall capacity equation:

$$V = 2\pi l \times \frac{d}{2} \times \text{number of pipes} \times 7.48$$

Where

- $V$ = volume of system (gal)
- $d$ = diameter of pipe (ft)
- $l$ = length of pipe (ft)
- 7.48 = conversion factor from cf to gallon

<table>
<thead>
<tr>
<th>Location</th>
<th>Pipe Dia (ft)</th>
<th>Pipe Length (ft)</th>
<th>No. Pipes</th>
<th>Total Capacity (gallons)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>7</td>
<td>190</td>
<td>6</td>
<td>187,517</td>
</tr>
<tr>
<td>B</td>
<td>6</td>
<td>275</td>
<td>6</td>
<td>232,634</td>
</tr>
<tr>
<td>C</td>
<td>10</td>
<td>200</td>
<td>4</td>
<td>187,987</td>
</tr>
<tr>
<td>D</td>
<td>10</td>
<td>400</td>
<td>4</td>
<td>375,975</td>
</tr>
</tbody>
</table>
Appendix E: Location Configurations & Cross Sections
Appendix F: Location D Expansion Exhibit
Appendix G: Rainfall Intensity Table
### NF Tabular

<table>
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1 Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at upper and lower bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

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