Delaware River Basin Comprehensive Flood Risk Management Interim Feasibility Study and Integrated Environmental Assessment for New Jersey

Flooding in the Study Area, April 2005

June 2015

APPENDIX B: Draft Interior Drainage Analysis

U.S. ARMY CORPS OF ENGINEERS PHILADELPHIA DISTRICT

NEW JERSEY DEPARTMENT OF ENVIRONMENTAL PROTECTION

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APPENDIX B: INTERIOR DRAINAGE ANALYSIS

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APPENDIX B: INTERIOR DRAINAGE ANALYSIS

1.0 Introduction

1.1 Scope

Interior drainage facilities are required to safely store and discharge storm water runoff that collects on the protected side of the levees and floodwalls associated with the flooding risk reduction project. This appendix describes the interior drainage facilities for the proposed project locations in the Delaware River Basin Comprehensive Flood Risk Management Interim Feasibility Study for New Jersey and documents how these facilities were developed to manage interior runoff.

1.2 Study Location

The geographic area of the basin study encompasses the 1% annual chance of exceedance (ACE) (100-year) floodplain in multiple municipalities. The two proposed project areas include the unincorporated area of Gibbstown in Greenwich Township, Gloucester County, and the City of Lambertville in Hunterdon County.

1.2.1 Greenwich Township (Gibbstown), Gloucester County

Gibbstown is one of the topographically flat, low-lying communities near the banks of the Delaware River. The community itself is located approximately a mile from the banks of the river. Between the Gibbstown residential areas and the river are located the DuPont and Ashland/Hercules industrial properties and Paulsboro refinery, as well as some light industrial manufacturing facilities. Much of the rest of the area between the town and the river consists of herbaceous and forested wetlands crossed with a network of interconnected streams and ditches. The community location is shown in Figure 1-1.

An existing, historic agricultural levee along the river bank provides some protection from riverine flooding but has been breached several times. This levee, known as the Repaupo or Gibbstown Levee, runs for approximately 4.5 miles along the Delaware River and was originally constructed in the early 1800's to facilitate the farming of salt hay.

The project area consists of residential housing, commercial buildings, and municipal buildings and is crossed by major transportation routes including US Route 295/130 and a railway.

1.2.2 City of Lambertville, Hunterdon County

The City of Lambertville is a densely-developed historic community of approximately 3,906 people (Census 2010) in a 1.3 square mile area on the Delaware River. The city location is shown in Figure 1-2.

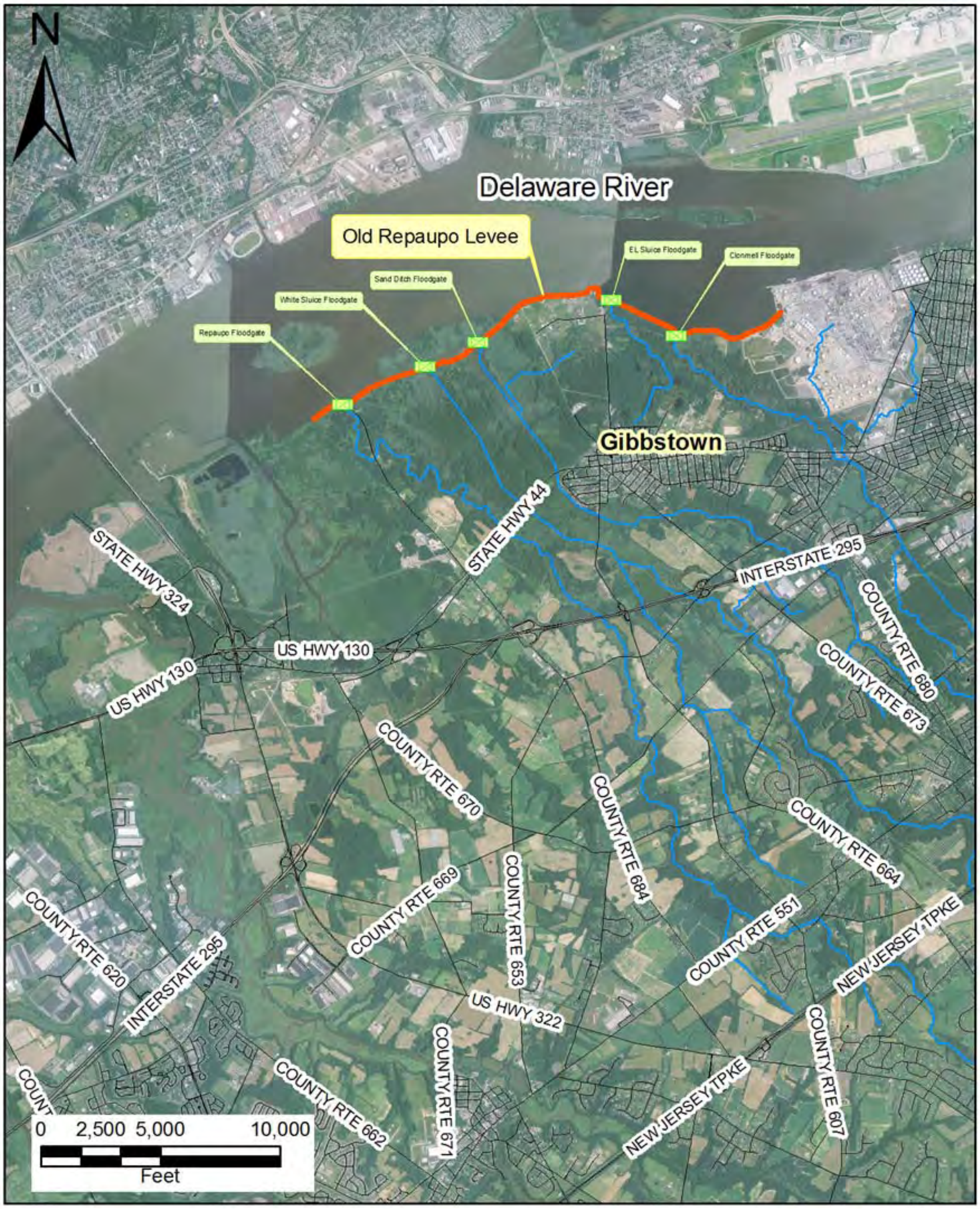
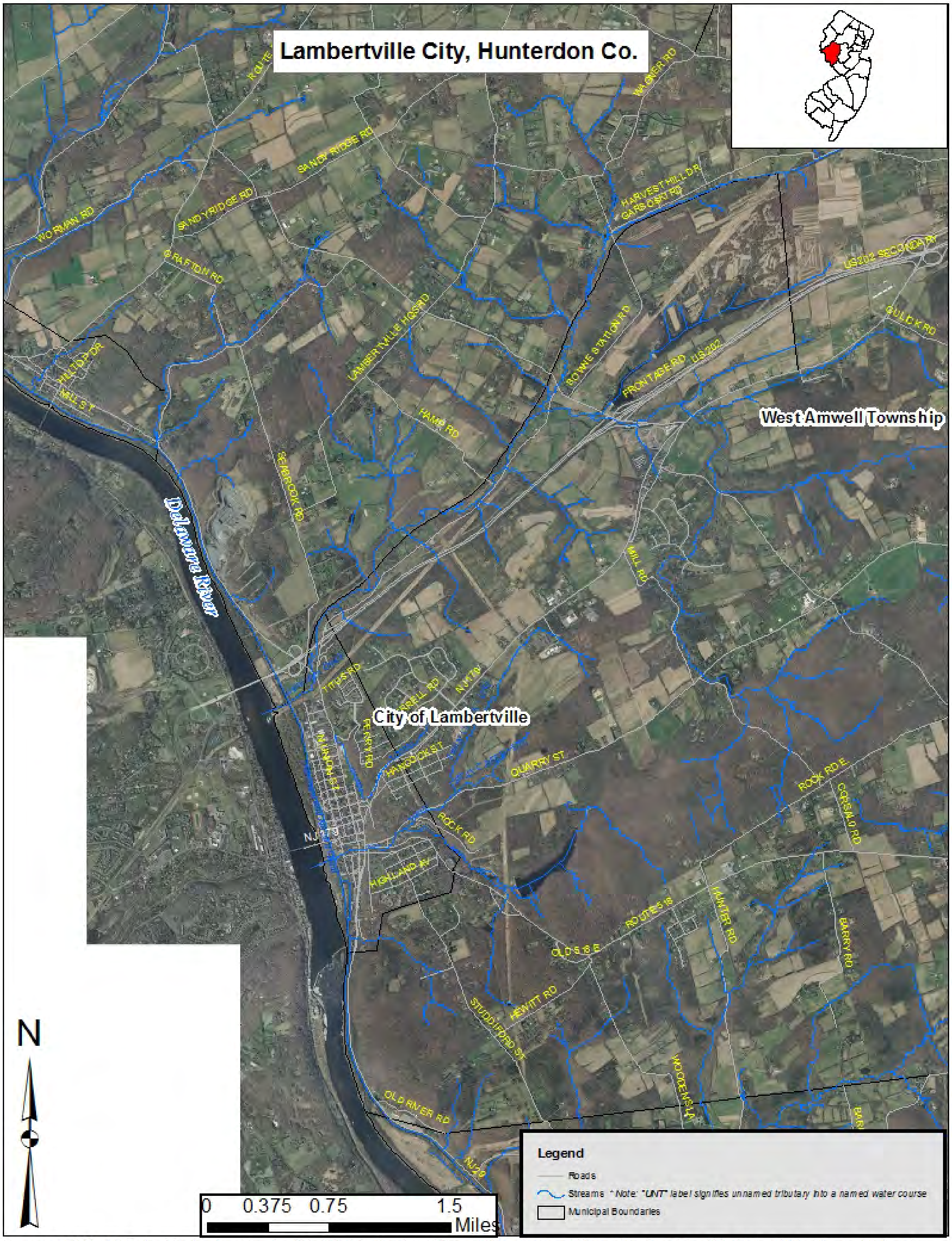


Figure 1-1: Gibbstown Project Location



Source: FEMA Q3 Data, Hunterdon County, NJ; NJDEP, Municipalities of New Jersey (Clipped to Coast by NJDEP), 2008, Stream Network (Upper Delaware Basin), 2008, 2002 Waters of New Jersey (Rivers, Bays, and Oceans); NJDOT, Roads 2008

Figure 1-2: City of Lambertville Project Location

1.3 Source of Flooding

1.3.1 Greenwich Township (Gibbstown), Gloucester County

Flooding occurs from the tidally influenced portion of the Delaware River where the existing ground elevations lay relatively close to sea level. Flooding of the local area has occurred when the existing levee has breached during major storm events. Additional nuisance flooding occurs during rainfall events due to poor interior drainage, which is exacerbated by the flat topography of the study area.

1.3.2 City of Lambertville, Hunterdon County

The main source of flooding in the interior areas of the town is backwater from the Delaware River affecting the tributaries Alexauken Creek, Ely Creek, and Swan Creek. Alexauken Creek lies upstream towards the city's northern border and has a 15 square-mile drainage area. Nearing the confluence with the Delaware River, Alexauken Creek goes under a railroad bridge and then is carried under the Delaware & Raritan (D&R) Canal aqueduct approximately 300 feet before it meets the Delaware River. In addition to overland flooding, floodwater from the Delaware River backflows into Alexauken Creek through the stormwater drainage system and floods area homes (primarily basements) and the nearby CVS Pharmacy. The area of flooding includes the northernmost section of North Union Street and north of Cherry Street.

Backwater flooding on Ely Creek causes it to overflow a 54" arched stone culvert that goes under the D&R Canal and flood the adjacent low-lying area bounded by the D&R Canal, Arnett Avenue, and Cherry Street.

1.4 General

Areas protected from exterior flood elevations (Lambertville) or storm surge elevations (Gibbstown) by the proposed line-of-protection are subject to interior flooding from stormwater runoff. Thus, interior drainage facilities are required to safely store and discharge the runoff to limit interior residual flooding. The interior areas were studied to determine the specific nature of flooding and to formulate drainage alternatives to maximize National Economic Development (NED) benefits.

In accordance with the Army Corps of Engineers Engineering Manual (EM) 1110-2-1413, *Hydrologic Analysis of Interior Areas*, the interior drainage facilities are evaluated separately from the line-of-protection. First, a minimum facility plan is identified. The minimum facility plan is considered the smallest plan that can be implemented as part of the line-of-protection that does not result in increased stormwater flooding. It is the starting point from which additional interior facilities planning commences.

Next, the benefits accrued from alternative interior drainage plans are attributable to the reduction in the residual flood damages which may have remained under the minimum facility condition. Finally, an optimum drainage alternative is selected based on meeting NED objectives.

The interior drainage facilities must be formulated to maximize NED benefits while meeting NED objectives to provide a complete, effective, efficient, and acceptable plan of protection.

- ***Completeness*** is defined in Engineering Regulation (ER) 1105-2-100 as the extent to which the alternative plans provide and account for all necessary investments or other actions to ensure the realization of the planning objectives, including actions by other Federal and non-Federal entities.

- ***Effectiveness*** is defined as the extent to which the alternative plans contribute to achieve the planning objectives.

- ***Efficiency*** is defined as the extent to which an alternative plan is the most cost-effective means of achieving the objectives.

- ***Acceptability*** is defined as the extent to which the alternative plans are acceptable in terms of applicable laws, regulations, and public policies.

2.0 Interior Drainage Analysis - Gibbstown

2.1 Existing Hydrology

2.1.1 Hydrologic Model

To correctly depict flood risk for the without-project conditions and to objectively evaluate the reduction of flood risk for alternatives screened in the with- project analysis and potential interior drainage features, the expected inundation areas that would result from a flood from the Delaware River or from interior drainage behind the Gibbstown Levee must be fully understood. A hydrologic model (Hydrologic Modeling System (HEC-HMS) version 3.5) of the Repaupo Creek Watershed was developed to evaluate the hydrologic conditions of the watershed. This model was the basis for modeling the interior drainage analysis. The individual hydrologic methods chosen to simulate the rainfall-runoff processes are discussed below.

2.1.2 Watershed Topography

LiDAR data from March/April 2007 for Gloucester County, NJ was clipped to the Repaupo Creek Watershed boundaries and used in the hydrologic model developed for the watershed. The 2007 LiDAR was originally used by FEMA and its Contractors to produce high accuracy 3D elevation based geospatial products for updating the floodplain mapping in Gloucester County, NJ. The data was compiled with horizontal positional accuracy of 1 meter at a 95% confidence level and with a vertical positional accuracy of 0.181 meters at the 95% confidence level.

The Repaupo Creek watershed is relatively flat north of I-295 except for the town of Gibbstown and at the northeast corner where the former Valero Refinery is located. Elevations north of I-295 in the wetland areas range from -3 ft. NAVD88, and rise only to 20 ft. NAVD88 within Gibbstown and 30 ft. NAVD88 at the former Valero Refinery. A majority of this area is wetlands and serves as a natural storage for elevated tides from the Delaware River and runoff coming down the watershed from its headwaters.

The terrain starts to rise up to its headwaters between I-295 and the NJ Turnpike near Swedesboro Ave. with elevations increasing from 15 ft. NAVD88 to 120 ft. NAVD88. The headwaters of the watershed have elevations around 155 ft. NAVD88. More than half of the watershed is at elevations below 30 ft. NAVD88.

2.1.3 Hydrologic Model Calibration

Sources of data that were used for calibration/validation purposes included qualitative data from various reports and frequency discharges computed from the original flood insurance study HEC-2 hydraulic model done by FEMA. As stated in the 2010 Flood Insurance Study, the peak discharges for Repaupo Creek, White Sluice Race, London Branch, Nehonsey Brook, and Clonmell Creek were developed using drainage area proportions using discharges calculated for Mantua Creek at two locations. The locations were dependent on the slope of the stream.

There are no streamflow gages within Repaupo Creek Watershed that recorded flows from historical events such as Hurricane Floyd in September 1999, and the significant rainfall events of July 2004 and April 2007. Also, no reliable high-water marks from any significant historical event could be found during the investigation. Due to the lack of direct flow data and high-water marks available for calibration/validation, discharges from the HEC-HMS hydrologic model for the frequency-based precipitation simulations were compared against previously computed discharges used in the original HEC-2 hydraulic model as summarized in the 2010 Flood Insurance Study. There was good agreement between the model results and flood insurance peak discharges specifically at the 1% ACE (100-yr) event for many locations within the watershed.

Alternatives and sensitivities portraying varied hydrologic conditions were examined as part of this effort. The following parameters were varied:

- Delaware River Stage.
- Local Rainfall Events.
- Cedar Swamp Inflow.
- Closed Floodgates.
- Initial Water Surface Elevation for Interior Ponds.
- Timing of Peak Delaware River Stage versus Precipitation Event

2.1.4 Without-Project Hydrologic Model Results

A final stage vs frequency of interior ponding elevations for the without- project conditions was derived based upon the alternative and sensitivity simulations conducted. Several assumptions and factors were considered in the development of the final stage frequency of interior ponding elevations. Inflow from Cedar Swamp and Klondike Ditch was incorporated into the interior pond stage frequency. The precipitation event over the watershed could happen on any random day of the year. Precipitation over the watershed is independent of Delaware River tidal conditions. It could happen when the tides on the Delaware River are normal or when the tides are elevated due to storm conditions. As the simulations showed, the difference in ponding elevations when the Delaware River is experiencing normal tidal conditions versus when a rare, low probability event occurs is between 0.5 ft and 1.3 ft. The difference in interior ponding elevations between 50% ACE (2-yr) and 0.2% ACE (500-yr) tailwater conditions decreases to only 0.5 ft. This small difference can be considered to be within the error of the model itself. A conservative estimate of tailwater conditions corresponding to a 1% annual chance of exceedence storm (100-yr) conditions was adopted for the final interior ponding elevation stage frequency. Table 2-1 summarizes the final “without” project interior pond stage frequency curves for the two areas examined; Repaupo Creek/White Sluice Race and Clonmell Creek.

Table 2-1: Peak Interior Pond Elevation – Gibbstown Existing Conditions

Annual Chance Rainfall Event/Recurrence Interval	Repaupo Creek/White Sluice Race (ft NAVD88)	Clonmell Creek (ft NAVD88)
99% / 1yr	0.35	1.00
50% / 2yr	0.58	1.47
20% / 5 yr	1.10	2.18
10% / 10yr	1.44	2.65
4% / 25yr	1.86	3.23
2% / 50yr	2.16	3.64
1% / 100yr	2.46	4.06
0.4% / 250yr	2.87	4.61
0.2% / 500yr	3.16	5.04

2.2 Proposed Protection Plan

The proposed project in Gibbstown consists of approximately 7,386 linear feet (LF) of levee and approximately 13,788 LF of floodwall forming the line of protection which generally follows the railway alignment along the north edge of town. The proposed line of protection elements are shown in Figures 2-1 through 2-3.

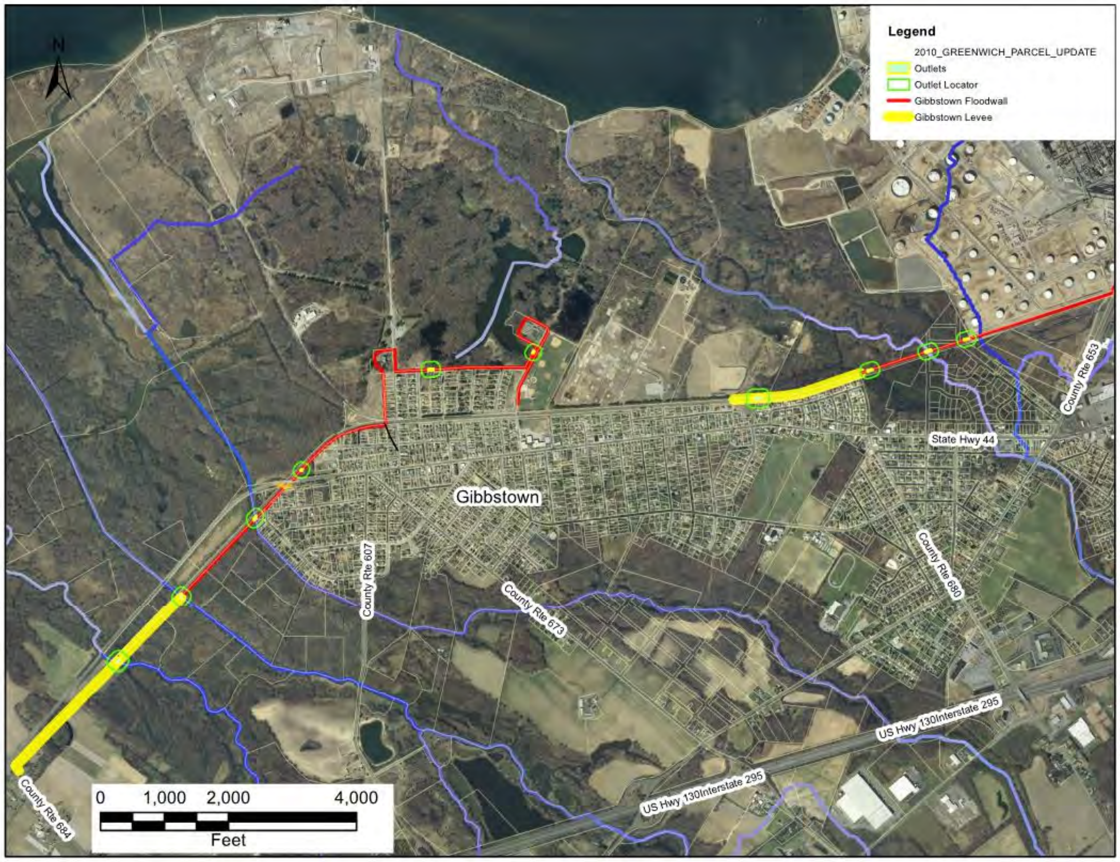


Figure 2-1: Gibbstown Line of Protection

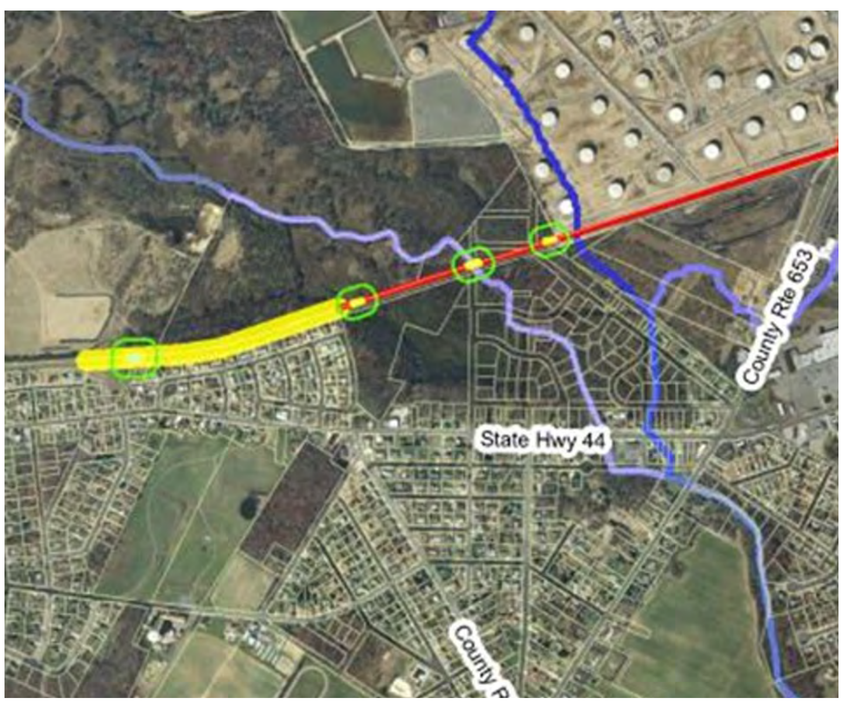


Figure 2-2: Gibbstown Line of Protection - East

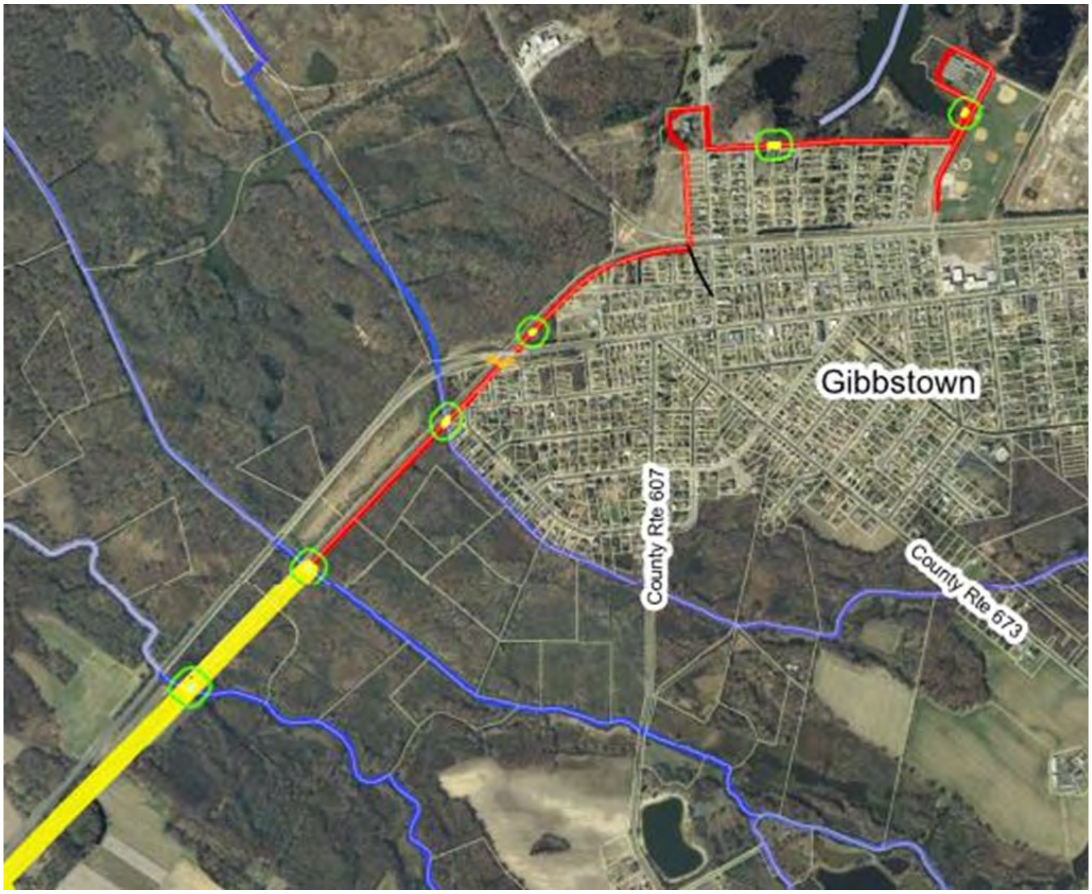


Figure 2-3: Gibbstown Line of Protection - West

2.3 Interior Drainage Model

The USACE's HEC-HMS (Hydrologic Modeling System) was used to analyze the runoff and interior drainage features' performance. The existing HEC-HMS model developed by the Philadelphia District, summarized in Section 2.1, was used as the basis for the interior drainage analysis. This model incorporated all of the drainage areas within the project area up to the Delaware River. The hydrologic model was modified to reflect the line of protection and adapted for use in this interior drainage analysis as described below.

The model parameters used in the existing conditions model were incorporated into the line of protection model for the interior drainage analysis. The proposed line of protection lies along the north side of the community, approximately one mile from the Delaware River. The drainage areas between the line of protect and the river do not contribute to the interior drainage runoff and were, therefore, removed or correspondingly adjusted in area where partially contributing. The area adjustments were performed on the GIS model of the existing drainage boundaries to form a GIS model of the interior drainage boundaries. This effort involved truncating and recalculating the drainage areas that saddled the proposed line of protection and truncating the hydrologic model to only contain the elements tributary to the line of protection. The resulting GIS delineation of the interior drainage areas is shown in Figure 2-4.

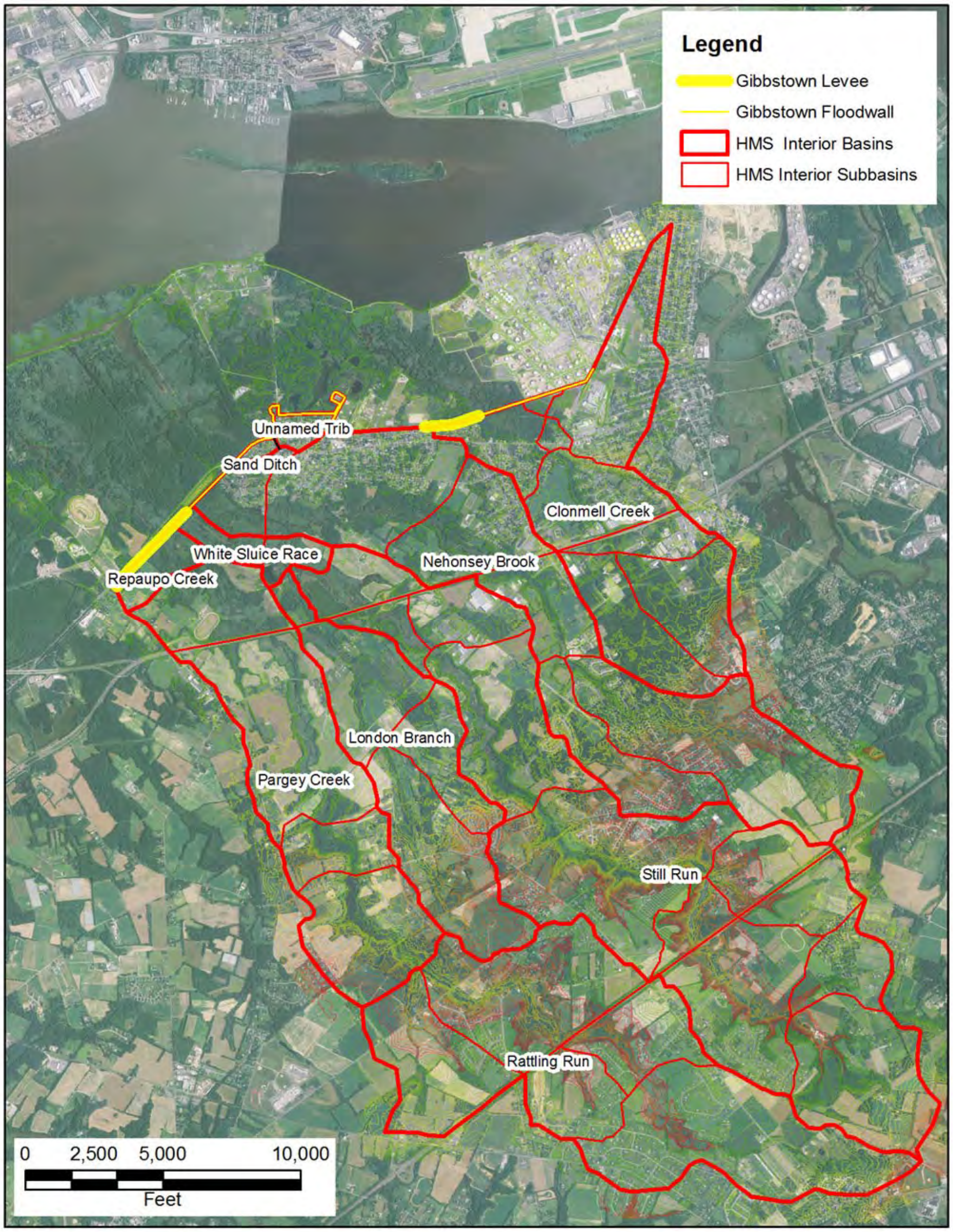


Figure 2-4: Gibbstown Drainage Areas

2.3.1 Boundary Conditions

The Delaware River tidal stage hydrographs developed for the existing conditions model for the various tidal storm event conditions were used as exterior boundary conditions. The development of the stage vs. frequency curves for the Delaware River is discussed in detail in Appendix A.

For purposes of modeling the interior drainage, it was assumed that the historic levee and gate structures are not effective in preventing damage beyond the normal tidal range. In reality, the structure may provide low levels of protection prior to breaching.

2.3.2 Precipitation

Precipitation data was obtained from New Jersey 24 Hours Rain Fall Frequency Data for 1, 2, 5, 10, 25, 50 and 100-year events and supplemented by NOAA Atlas 14, Volume 2, Version 3, for location name: Gibbstown, New Jersey, US Point Precipitation Frequency for the estimated 24 hour, 500-year event. The 250-year event was interpolated from average recurrence interval/precipitation depth chart in NOAA Atlas 14.

2.3.3 Interior Drainage Areas

There are four separate interior drainage areas that form local interior ponding areas behind (or leeward) of the proposed line of protection. The largest of these areas, the Repaupo/White Sluice (Repaupo) area to the western side of the project contains four streams that are interconnected with ditches in the low lying areas prior to reaching the line of protection and form a common ponding area.

The next largest area is the Clonmell Creek watershed (Clonmell), located at the eastern side of the project, where a single stream passes through the line of protection.

An approximately fifty-acre portion of the town center (Town Center) area of Gibbstown, adjacent to and sloping towards the line of protection will drain independently through the line of protection.

The smallest interior drainage area also lies within the town center (Town Center 2) and is a 22-acre area confined between the proposed line of protection adjacent to the railroad, and West Broad Street which rises up to overpass the railroad.

The four drainage areas are shown in Figure 2-5.

The proposed conditions HEC-HMS model overall schematic diagram is shown in Figure 2-6.

Figure 2-5: Gibbstown With-project Drainage Areas

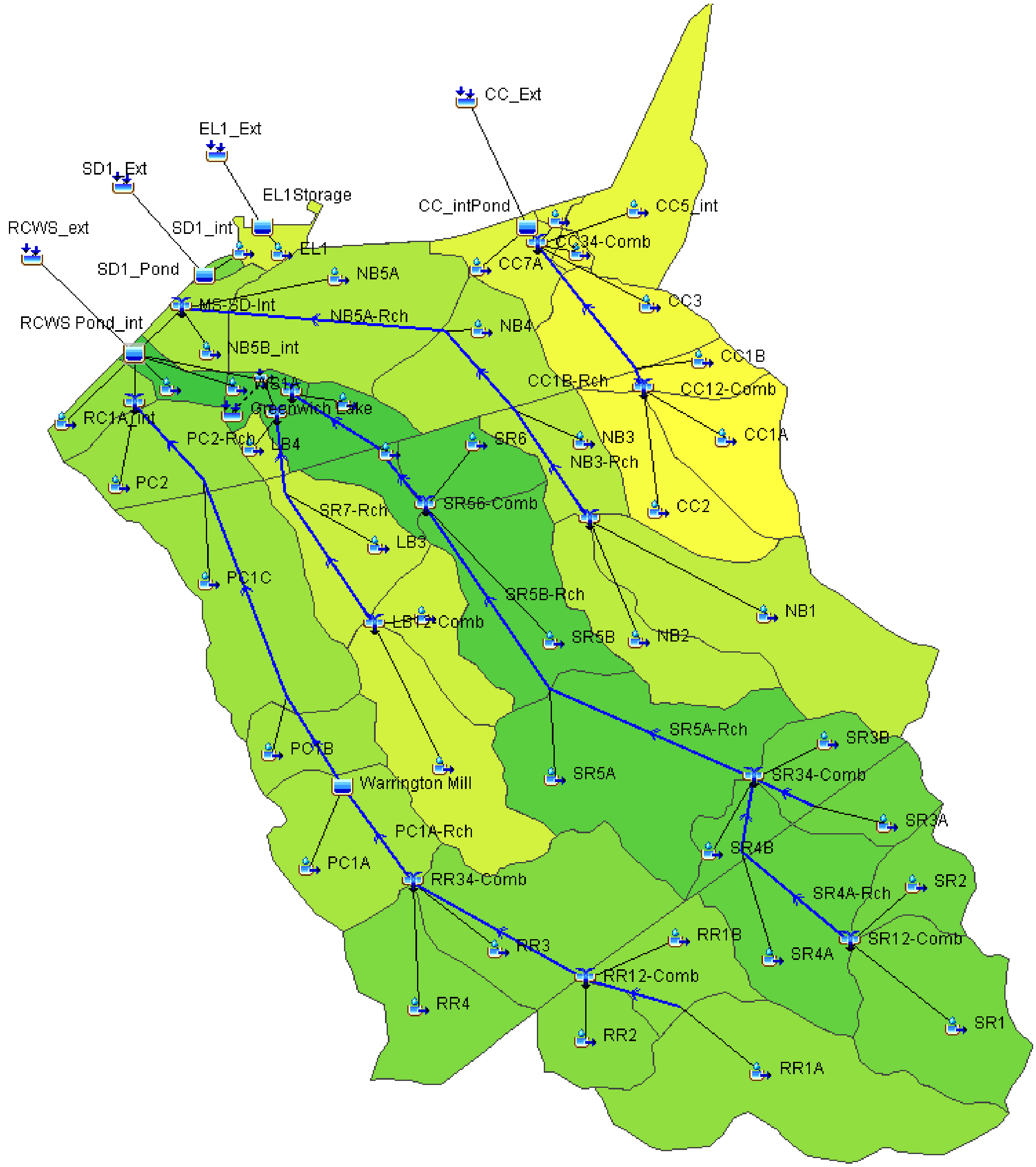


Figure 2-6: HEC-HMS Overall Model Schematic

2.4 Analysis Approach

Due to the limited correlation between rainfall/runoff events and tidal flooding events, it is considered most likely that only limited runoff will coincide with severe storm surge and significant storm surge will coincide with only moderately severe rainfall. Historical data indicate that the majority of interior runoff events will coincide with a storm surge level less than or equal to a 2-year storm. Similarly, the majority of significant storm surge events are likely to coincide with runoff equivalent to a 2-year event or less.

Therefore, the analysis was conducted for events with nine (9) recurrence intervals: the 1-yr, 2-yr, 5-yr, 10-yr, 25-yr, 50-yr, 100-yr, 250-yr, and 500-yr frequency events. In order to develop a stage-frequency relationship, the interior events were routed against exterior tidal marigrams. For the most likely flooding scenarios, the nine interior storm events were routed against a 2-yr exterior tide, and a 2-yr interior storm event was routed against the nine exterior events. The highest water surface elevation (WSEL) of corresponding coincidental frequencies (e.g., 2-yr interior and 10-yr exterior, or 10-yr interior and 2-yr exterior) was identified as the most damaging flood level for the coincidental frequency (as shown in Table 2-2).

2.5 Minimum Facilities

For purposes of modeling the interior drainage, it was assumed that the historic levee and gate structures are not effective in preventing damage beyond the normal tidal range. This is consistent with how the proposed project benefits have been calculated. The criteria utilized for sizing the gravity outlet conveyances was to target minimal head loss through the proposed line of protection such that interior flooding is reduced with a “no tailwater” condition. Through an iterative process, the size and configuration of interior to exterior drainage conveyances was determined for each for the four ponding areas.

The most likely interior ponding elevation for each area are summarized in Table 2-3 The description of the minimum facilities at each ponding area as described in the following paragraphs.

Table 2-2: Analysis Approach

Analysis Approach Combination of Interior and Exterior Conditions to be Analyzed											
Interior Flow	Exterior Stage	Time Condition	Peak Int. WSEL	Peak Ext. WSEL	Interior Flow	Exterior Stage	Time	Peak Int. WSEL	Peak Ext. WSEL	Max WS	Risk Condition
1yr	Normal	Current	……Model Output……		N/a			……Model Output……		Greatest WSEL for the Frequency Comb.	Lower Bound
2yr	Normal	Current			N/a						Lower Bound
5yr	Normal	Current			N/a						Lower Bound
10yr	Normal	Current			N/a						Lower Bound
25yr	Normal	Current			N/a						Lower Bound
50yr	Normal	Current			N/a						Lower Bound
100yr	Normal	Current			N/a						Lower Bound
250yr	Normal	Current			N/a						Lower Bound
500yr	Normal	Current			N/a						Lower Bound
1yr	2yr	Current			2yr	1yr	Current				Expected (1yr)
2yr	2yr	Current			2yr	2yr	Current				Expected (2yr)
5yr	2yr	Current			2yr	5yr	Current				Expected (5yr)
10yr	2yr	Current			2yr	10yr	Current				Expected (10yr)
25yr	2yr	Current			2yr	25yr	Current				Expected (25yr)
50yr	2yr	Current			2yr	50yr	Current				Expected (50yr)
100yr	2yr	Current			2yr	100yr	Current				Expected(100yr)
250yr	2yr	Current			2yr	250yr	Current				Expected(250yr)
500yr	2yr	Current			2yr	500yr	Current				Expected(500yr)
1yr	10yr	Current			10yr	1yr	Current				Upper Bound
2yr	10yr	Current			10yr	2yr	Current				Upper Bound
5yr	10yr	Current			10yr	5yr	Current				Upper Bound
10yr	10yr	Current			10yr	10yr	Current				Upper Bound
25yr	10yr	Current			10yr	25yr	Current				Upper Bound
50yr	10yr	Current			10yr	50yr	Current				Upper Bound
100yr	10yr	Current			10yr	100yr	Current				Upper Bound
250yr	10yr	Current			10yr	250yr	Current				Upper Bound
500yr	10yr	Current			10yr	500yr	Current				Upper Bound

Table 2-3: Summary of Gibbstown Minimum Facilities Analysis

Annual Chance of Exceedance / Recurrence Interval	Exterior Flood Elevation (ft NAVD88)	Most Likely Interior Elevation (ft NAVD88)			
		Repaupo	Clonmell	Town Center	Town Center 2
100% / 1 yr	3.26	-0.08	1.71	4.81	2.37
50% / 2 yr	5.48	0.29	1.81	5.03	2.38
20% / 5 yr	6.07	0.92	2.31	5.12	2.68
10% / 10 yr	6.43	1.27	2.68	5.18	2.75
4% / 25 yr	6.92	1.72	2.97	5.25	2.84
2% / 50 yr	7.24	2.03	3.17	5.30	2.93
1% / 100 yr	7.58	2.30	3.40	5.33	3.02
0.4% / 250 yr	8.07	2.67	3.73	5.36	3.12
0.2% / 500 yr	8.37	2.93	4.01	5.39	3.20

Repaupo/White Sluice Interior Ponding Area

For the Repaupo/White Sluice Interior Ponding area (HMS pond node: RCWS_POND_INT as shown in Figure 2-6), six 6-foot high by 10-foot wide box culverts with a length of approximately 100 feet through the levee and 10 feet through the floodwall conveying the flows of the 3 creeks through the levee were sufficient to meet minimum facility requirements. The size of these outlets appeared reasonable given the sizes of the creeks passing through the location of the proposed levee.

Clonmell Creek Interior Ponding Area

For the Clonmell Creek Interior Ponding area (HMS pond node: CC_INTPOND as shown in Figure 2-6), three 4-foot high by 10-foot wide box culverts with a lengths similar to the Repaupo culverts above conveying the flows of the creek through the levee were sufficient to meet minimum facility requirements.

Town Center Interior Ponding Area

For the main town center interior ponding area (HMS pond node: EL1STORAGE as shown in Figure 2-6), three 3-foot high by 4-foot wide box culverts with a lengths similar to the Repaupo culverts above conveying the flows of the local runoff through the levee were sufficient to meet minimum facility requirements.

Town Center 2 Interior Ponding Area

For the town center interior ponding area (HMS pond node: SD1_Pond as shown in Figure 2-6), a 3-foot diameter culvert with a lengths similar to the Repaupo culverts above draining the local runoff through the levee was sufficient to meet minimum facility requirements.

2.6 Comparison Against the Without-Project Conditions Model

The presence of the historic levee along the bank of the Delaware River presents a unique modeling situation. As noted previously, for purposes of modeling the interior drainage, it was assumed that the historic levee and gate structures are not effective in preventing damage beyond the normal tidal range. In reality, the structure may provide low levels of protection prior to breaching; therefore, it is important to look at the minimum facilities' performance compared with what residents may see 'on the ground' during storm events to ensure there is no real or perceived induced flooding.

Table 2-4 provides a comparison of the minimum facilities performance with exterior conditions against the without project interior drainage model, which incorporated some performance of the historic levee. As shown in the table, the Repaupo Minimum Facilities reduce the water surface elevations (WSELs) inside the LOP to below the existing exterior. For the Clonmell drainage area, the Minimum Facilities WSELs for the 1yr to 10yr events are slightly higher than the existing conditions. This is a result of modeling the interior elevations versus Delaware River exterior conditions. Blockage of the drainage features prevents sufficient water from exiting the interior system in the model; additional outlets will have minimal effect on further reducing the interior WSELs for those events; therefore, this represents the Minimum Facilities results.

Table 2-4: Comparison of Interior Drainage Results with Existing Conditions

Annual Chance of Exceedance/Recurrence Interval	Comparison of Interior Elevations (ft NAVD88)			
	Repaupo		Clonmell	
	Existing*	Min Fac.	Existing*	Min Fac.
99% / 1 yr	0.35	-0.08	1.00	1.71
50% / 2 yr	0.58	0.29	1.47	1.81
20% / 5 yr	1.10	0.92	2.18	2.31
10% / 10 yr	1.44	1.27	2.65	2.68
4% / 25 yr	1.86	1.72	3.23	2.97
2% / 50 yr	2.16	2.03	3.64	3.17
1% / 100 yr	2.46	2.30	4.06	3.40
0.4% / 250 yr	2.87	2.67	4.61	3.73
0.2% / 500 yr	3.16	2.93	5.04	4.01

*Existing: With historic levee in place.

Detailed results of the interior drainage analysis are shown in Tables 2-5 through 2-8 for the Repaupo/White Sluice, the Town Center and the Clonmell Interior Ponding areas respectively.

Gibbstown Interior Drainage **Repaupo Ponding Area** **Minimum Facility**

Summary of Results & Conditions Modelled

Alternative	Interior Flow	Exterior Stage	Time Condition	HEC-HMS Module/file	Peak WS	Interior Flow	Exterior Stage	Time	HEC-HMS Module/file	Peak WS	Max WS	Risk Condition
	1yr	Normal	Current	P001 TW_NORM	-0.45						-0.45	Lower Bound
	2yr	Normal	Current	P002 TW_NORM	-0.10						-0.10	Lower Bound
	5yr	Normal	Current	P005 TW_NORM	0.40						0.40	Lower Bound
	10yr	Normal	Current	P010 TW_NORM	0.77						0.77	Lower Bound
	25yr	Normal	Current	P025 TW_NORM	1.19						1.19	Lower Bound
	50yr	Normal	Current	P050 TW_NORM	1.47						1.47	Lower Bound
	100yr	Normal	Current	P100 TW_NORM	1.74						1.74	Lower Bound
	250yr	Normal	Current	P250 TW_NORM	2.08						2.08	Lower Bound
	500yr	Normal	Current	P500 TW_NORM	2.30						2.30	Lower Bound
	1yr	2yr	Current	P001 TW002	-0.08	2yr	Normal	Current	P002 TW_NORM	-0.10	-0.08	Most Likely
	2yr	2yr	Current	P002 TW002	0.29	2yr	2yr	Current	P002 TW002	0.29	0.29	Most Likely
	5yr	2yr	Current	P005 TW002	0.92	2yr	5yr	Current	P002 TW005	0.29	0.92	Most Likely
	10yr	2yr	Current	P010 TW002	1.27	2yr	10yr	Current	P002 TW010	0.29	1.27	Most Likely
	25yr	2yr	Current	P025 TW002	1.72	2yr	25yr	Current	P002 TW025	0.30	1.72	Most Likely
	50yr	2yr	Current	P050 TW002	2.03	2yr	50yr	Current	P002 TW050	0.31	2.03	Most Likely
	100yr	2yr	Current	P100 TW002	2.30	2yr	100yr	Current	P002 TW100	0.39	2.30	Most Likely
	250yr	2yr	Current	P250 TW002	2.67	2yr	250yr	Current	P002 TW250	0.40	2.67	Most Likely
	500yr	2yr	Current	P500 TW002	2.93	2yr	500yr	Current	P002 TW500	0.40	2.93	Most Likely
	1yr	10yr	Current	P001 TW010	-0.08	10yr	Normal	Current	P010 TW_NORM	0.77	0.77	Upper Bound
	2yr	10yr	Current	P002 TW010	0.29	10yr	2yr	Current	P010 TW002	1.27	1.27	Upper Bound
	5yr	10yr	Current	P005 TW010	0.95	10yr	5yr	Current	P010 TW005	1.30	1.30	Upper Bound
	10yr	10yr	Current	P010 TW010	1.32	10yr	10yr	Current	P010 TW010	1.32	1.32	Upper Bound
	25yr	10yr	Current	P025 TW010	1.77	10yr	25yr	Current	P010 TW025	1.34	1.77	Upper Bound
	50yr	10yr	Current	P050 TW010	2.08	10yr	50yr	Current	P010 TW050	1.39	2.08	Upper Bound
	100yr	10yr	Current	P100 TW010	2.36	10yr	100yr	Current	P010 TW100	1.45	2.36	Upper Bound
	250yr	10yr	Current	P250 TW010	2.75	10yr	250yr	Current	P010 TW250	1.48	2.75	Upper Bound
	500yr	10yr	Current	P500 TW010	3.02	10yr	500yr	Current	P010 TW500	1.65	3.02	Upper Bound

Table 2-5: Minimum Facility Results for the Clonmell Interior Drainage Area (Feet NAVD88)

Gibbstown Interior Drainage **Clonmell Ponding Area** **Minimum Facility**

Summary of Results & Conditions Modelled

Alternative	Interior Flow	Exterior Stage	Time Condition	HEC-HMS Module/file	Peak WS	Interior Flow	Exterior Stage	Time	HEC-HMS Module/file	Peak WS	Max WS	Risk Condition
	1yr	Normal	Current	P001 TW_NORM	1.34						1.34	Lower Bound
	2yr	Normal	Current	P002 TW_NORM	1.71						1.71	Lower Bound
	5yr	Normal	Current	P005 TW_NORM	2.27						2.27	Lower Bound
	10yr	Normal	Current	P010 TW_NORM	2.60						2.60	Lower Bound
	25yr	Normal	Current	P025 TW_NORM	2.90						2.90	Lower Bound
	50yr	Normal	Current	P050 TW_NORM	3.13						3.13	Lower Bound
	100yr	Normal	Current	P100 TW_NORM	3.36						3.36	Lower Bound
	250yr	Normal	Current	P250 TW_NORM	3.70						3.70	Lower Bound
	500yr	Normal	Current	P500 TW_NORM	4.00						4.00	Lower Bound
	1yr	2yr	Current	P001 TW002	1.45	2yr	Normal	Current	P002 TW_NORM	1.71	1.71	Most Likely
	2yr	2yr	Current	P002 TW002	1.81	2yr	2yr	Current	P002 TW002	1.81	1.81	Most Likely
	5yr	2yr	Current	P005 TW002	2.31	2yr	5yr	Current	P002 TW005	1.86	2.31	Most Likely
	10yr	2yr	Current	P010 TW002	2.68	2yr	10yr	Current	P002 TW010	1.88	2.68	Most Likely
	25yr	2yr	Current	P025 TW002	2.97	2yr	25yr	Current	P002 TW025	1.93	2.97	Most Likely
	50yr	2yr	Current	P050 TW002	3.17	2yr	50yr	Current	P002 TW050	1.97	3.17	Most Likely
	100yr	2yr	Current	P100 TW002	3.40	2yr	100yr	Current	P002 TW100	2.00	3.40	Most Likely
	250yr	2yr	Current	P250 TW002	3.73	2yr	250yr	Current	P002 TW250	2.05	3.73	Most Likely
	500yr	2yr	Current	P500 TW002	4.01	2yr	500yr	Current	P002 TW500	2.07	4.01	Most Likely
	1yr	10yr	Current	P001 TW010	1.48	10yr	Normal	Current	P010 TW_NORM	2.60	2.60	Upper Bound
	2yr	10yr	Current	P002 TW010	1.88	10yr	2yr	Current	P010 TW002	2.68	2.68	Upper Bound
	5yr	10yr	Current	P005 TW010	2.41	10yr	5yr	Current	P010 TW005	2.70	2.70	Upper Bound
	10yr	10yr	Current	P010 TW010	2.72	10yr	10yr	Current	P010 TW010	2.72	2.72	Upper Bound
	25yr	10yr	Current	P025 TW010	3.05	10yr	25yr	Current	P010 TW025	2.79	3.05	Upper Bound
	50yr	10yr	Current	P050 TW010	3.29	10yr	50yr	Current	P010 TW050	2.84	3.29	Upper Bound
	100yr	10yr	Current	P100 TW010	3.54	10yr	100yr	Current	P010 TW100	2.89	3.54	Upper Bound
	250yr	10yr	Current	P250 TW010	3.84	10yr	250yr	Current	P010 TW250	2.98	3.84	Upper Bound
	500yr	10yr	Current	P500 TW010	4.03	10yr	500yr	Current	P010 TW500	3.02	4.03	Upper Bound

Table 2-6: Minimum Facility Results for the Town Center Interior Drainage Area (Feet NAVD88)

Gibbstown Interior Drainage **Town Center Ponding Area** **Minimum Facility**

Summary of Results & Conditions Modelled

Alternative	Interior Flow	Exterior Stage	Time Condition	HEC-HMS Module/file	Peak WS	Interior Flow	Exterior Stage	Time	HEC-HMS Module/file	Peak WS	Max WS	Risk Condition
	1yr	Normal	Current	P001 TW_NORM	3.25						3.25	Lower Bound
	2yr	Normal	Current	P002 TW_NORM	3.25						3.25	Lower Bound
	5yr	Normal	Current	P005 TW_NORM	3.25						3.25	Lower Bound
	10yr	Normal	Current	P010 TW_NORM	3.25						3.25	Lower Bound
	25yr	Normal	Current	P025 TW_NORM	3.28						3.28	Lower Bound
	50yr	Normal	Current	P050 TW_NORM	3.43						3.43	Lower Bound
	100yr	Normal	Current	P100 TW_NORM	3.57						3.57	Lower Bound
	250yr	Normal	Current	P250 TW_NORM	3.77						3.77	Lower Bound
	500yr	Normal	Current	P500 TW_NORM	3.90						3.90	Lower Bound
	1yr	2yr	Current	P001 TW002	4.81	2yr	Normal	Current	P002 TW_NORM	3.25	4.81	Most Likely
	2yr	2yr	Current	P002 TW002	5.03	2yr	2yr	Current	P002 TW002	5.03	5.03	Most Likely
	5yr	2yr	Current	P005 TW002	5.12	2yr	5yr	Current	P002 TW005	5.08	5.12	Most Likely
	10yr	2yr	Current	P010 TW002	5.18	2yr	10yr	Current	P002 TW010	5.08	5.18	Most Likely
	25yr	2yr	Current	P025 TW002	5.25	2yr	25yr	Current	P002 TW025	5.10	5.25	Most Likely
	50yr	2yr	Current	P050 TW002	5.30	2yr	50yr	Current	P002 TW050	5.11	5.30	Most Likely
	100yr	2yr	Current	P100 TW002	5.33	2yr	100yr	Current	P002 TW100	5.13	5.33	Most Likely
	250yr	2yr	Current	P250 TW002	5.35	2yr	250yr	Current	P002 TW250	5.15	5.35	Most Likely
	500yr	2yr	Current	P500 TW002	5.39	2yr	500yr	Current	P002 TW500	5.17	5.39	Most Likely
	1yr	10yr	Current	P001 TW010	4.98	10yr	Normal	Current	P010 TW_NORM	3.25	4.98	Upper Bound
	2yr	10yr	Current	P002 TW010	5.08	10yr	2yr	Current	P010 TW002	5.18	5.18	Upper Bound
	5yr	10yr	Current	P005 TW010	5.21	10yr	5yr	Current	P010 TW005	5.25	5.25	Upper Bound
	10yr	10yr	Current	P010 TW010	5.30	10yr	10yr	Current	P010 TW010	5.30	5.30	Upper Bound
	25yr	10yr	Current	P025 TW010	5.40	10yr	25yr	Current	P010 TW025	5.33	5.40	Upper Bound
	50yr	10yr	Current	P050 TW010	5.47	10yr	50yr	Current	P010 TW050	5.35	5.47	Upper Bound
	100yr	10yr	Current	P100 TW010	5.54	10yr	100yr	Current	P010 TW100	5.38	5.54	Upper Bound
	250yr	10yr	Current	P250 TW010	5.64	10yr	250yr	Current	P010 TW250	5.43	5.64	Upper Bound
	500yr	10yr	Current	P500 TW010	5.73	10yr	500yr	Current	P010 TW500	5.46	5.73	Upper Bound

Table 2-7: Minimum Facility Results for the Town Center 2 Interior Drainage Area (Feet NAVD88)

Gibbstown Interior Drainage **Town Center 2 Ponding Area** **Minimum Facility**

Summary of Results & Conditions Modelled

Alternative	Interior Flow	Exterior Stage	Time Condition	HEC-HMS Module/file	Peak WS	Interior Flow	Exterior Stage	Time	HEC-HMS Module/file	Peak WS	Max WS	Risk Condition
	1yr	Normal	Current	P001 TW_NORM	2.27						2.27	Lower Bound
	2yr	Normal	Current	P002 TW_NORM	2.37						2.37	Lower Bound
	5yr	Normal	Current	P005 TW_NORM	2.58						2.58	Lower Bound
	10yr	Normal	Current	P010 TW_NORM	2.68						2.68	Lower Bound
	25yr	Normal	Current	P025 TW_NORM	2.81						2.81	Lower Bound
	50yr	Normal	Current	P050 TW_NORM	2.92						2.92	Lower Bound
	100yr	Normal	Current	P100 TW_NORM	3.02						3.02	Lower Bound
	250yr	Normal	Current	P250 TW_NORM	3.12						3.12	Lower Bound
	500yr	Normal	Current	P500 TW_NORM	3.20						3.20	Lower Bound
	1yr	2yr	Current	P001 TW002	2.29	2yr	Normal	Current	P002 TW_NORM	2.37	2.37	Most Likely
	2yr	2yr	Current	P002 TW002	2.38	2yr	2yr	Current	P002 TW002	2.38	2.38	Most Likely
	5yr	2yr	Current	P005 TW002	2.68	2yr	5yr	Current	P002 TW005	2.42	2.68	Most Likely
	10yr	2yr	Current	P010 TW002	2.75	2yr	10yr	Current	P002 TW010	2.44	2.75	Most Likely
	25yr	2yr	Current	P025 TW002	2.84	2yr	25yr	Current	P002 TW025	2.46	2.84	Most Likely
	50yr	2yr	Current	P050 TW002	2.93	2yr	50yr	Current	P002 TW050	2.49	2.93	Most Likely
	100yr	2yr	Current	P100 TW002	3.02	2yr	100yr	Current	P002 TW100	2.50	3.02	Most Likely
	250yr	2yr	Current	P250 TW002	3.12	2yr	250yr	Current	P002 TW250	2.53	3.12	Most Likely
	500yr	2yr	Current	P500 TW002	3.20	2yr	500yr	Current	P002 TW500	2.55	3.20	Most Likely
	1yr	10yr	Current	P001 TW010	2.34	10yr	Normal	Current	P010 TW_NORM	2.68	2.68	Upper Bound
	2yr	10yr	Current	P002 TW010	2.44	10yr	2yr	Current	P010 TW002	2.75	2.75	Upper Bound
	5yr	10yr	Current	P005 TW010	2.71	10yr	5yr	Current	P010 TW005	2.75	2.75	Upper Bound
	10yr	10yr	Current	P010 TW010	2.80	10yr	10yr	Current	P010 TW010	2.80	2.80	Upper Bound
	25yr	10yr	Current	P025 TW010	2.89	10yr	25yr	Current	P010 TW025	2.94	2.94	Upper Bound
	50yr	10yr	Current	P050 TW010	2.98	10yr	50yr	Current	P010 TW050	3.00	3.00	Upper Bound
	100yr	10yr	Current	P100 TW010	3.04	10yr	100yr	Current	P010 TW100	3.03	3.04	Upper Bound
	250yr	10yr	Current	P250 TW010	3.13	10yr	250yr	Current	P010 TW250	3.07	3.13	Upper Bound
	500yr	10yr	Current	P500 TW010	3.21	10yr	500yr	Current	P010 TW500	3.09	3.21	Upper Bound

2.7 Economic Criteria

2.7.1 Conditions

The analysis of benefits and costs for the formulation of the interior drainage plans was conducted using a discount rate of 3.5 % over a 50-year project life.

2.7.2 Costs

General

Interior drainage facility costs are based on incremental improvements and are additive to features integral to the line-of-protection (i.e., the Minimum Facilities). These costs consist of first construction costs, real estate costs, and annual operation and maintenance (O&M) expenses. Each of these is described below.

First Construction Costs

First construction costs assigned to interior drainage facilities include primary and secondary outlets, intake structures and gates associated with the outlet, pond excavation, diversion pipes, and pump stations. Interior drainage costs do not include major line-of-protection costs, but rather are limited to project features that may be altered by the interior drainage design. First costs for items were estimated based on prevailing unit costs. First costs include: (1) for structures other than pumps, Engineering and Design (15%), Supervision and Administration (10%), and Contingency (25%).

Real Estate Costs

Real estate acquisition costs were not included in the analysis.

Operation and Maintenance

Annual charges attributed to the operation and maintenance (O&M) of interior drainage facilities consist of labor charges for the care and cleaning of pond areas, outlets and pump stations, as well as anticipated energy charges and annualized replacement costs. A cost of $2 per foot was used for O&M costs.

Cost Estimate Assumptions

The following assumptions were made when developing the interior drainage facilities' estimated costs:

- Mobilization/Demobilization, dewatering: Construction of minimum facilities is assumed to occur simultaneously with the line-of-protection construction; therefore, these costs are not required as part of the interior drainage costs.

- Toe ditch construction: Toe ditch construction costs are assumed to be part of the line-of-protection costs.

- Outlet riprap is not required due to the concrete apron in the typical outlet.

2.7.3 Benefits

General

The benefits for interior drainage facilities are calculated as the difference between minimum facility residual damages and residual damages associated with the interior drainage plan alternative being evaluated.

Residual Flood Damages

The expected damage to structures and vehicles were calculated for various depths of interior or residual flooding, that is, flooding which occurs as a result of the line-of-protection preventing runoff. The residual damages with the minimum facility plan represent the starting point from which additional interior facilities planning commences. The benefits accrued from alternative interior drainage plans are attributable to the reduction in the residual flood damages which may have remained under the minimum facility condition.

Residual Annual Damages

Residual damages were calculated using risk based simulation techniques with HEC's Flood Damage Analysis Model (HEC-FDA). The damage analysis assumed that there will be no significant coincidence between the residual interior flooding from rainfall and residual flooding from storms exceeding the line-of-protection. In accordance with EM 1110-2-1413, interior damage was calculated for a full range of interior flood events up to and including the 500-year storm.

2.8 Alternatives Analysis

As previously stated, minimum facilities represent the minimum drainage required such that no induced flooding occurs during low exterior stages. In addition to the minimum facilities analysis, various alternatives that could potentially improve interior drainage were evaluated for each interior ponding area to determine their viability and cost effectiveness. The alternatives considered were:

1. Increased capacity of gravity outlets.
2. Pump stations to draw down interior ponding levels.
3. Excavated detention areas adjacent to the line-of-protection.
4. Construction of interior levees.

As noted previously, the presence of the historic levee along the bank of the Delaware River presents a unique modeling situation. For purposes of modeling the interior drainage, it was assumed that the historic levee and gate structures are not effective in preventing damage beyond the normal tidal range. In reality, the structure may provide low levels of protection prior to breaching. As a result, the minimum facilities for the Gibbstown line of protection are actually enhanced minimum facilities in that their performance exceeds the typical minimum facilities requirement (i.e., no induced flooding above exterior event frequencies) and provides additional reduction in residual damages beyond what is otherwise typically required by minimum facilities.

2.8.1 Minimum Facilities Residual Damages

Minimum facilities residual damages were calculated in HEC-FDA using the residual interior WSELs in Tables 2-5 through 2-8. The alternatives analysis included investigating additional gravity outlets, pump stations, and other structures, including interior levees. Based on the residual damages with the minimum facilities in place, shown in Table 2-9, it is clear that alternatives may only be viable for the Repaupo and Clonmell drainage areas. Reduction of the residual damages within the in-town (Areas 1 and 2) reaches is not practicable.

Table 2-8: Residual Annual Damages with Minimum Facilities

Drainage Area			
Repaupo	**Clonmell**	**Town Center 1**	**Town Center 2**
$ 84,428	$ 3,111	$ 4	$ 0

2.8.2 Alternatives Considered

The following was an initial assessment of interior drainage alternatives considered at the Repaupo and Clonmell drainage areas:

<u>Repaupo Interior Area</u>

Increase Outlet capacity	For consideration: additional outlets may prove effective.
Pump Station	Not practical: too costly*, a large pump station would be needed reduce WSELs by any measurable amount.
Excavated Ponding Area	Not practical: ground is already low and mostly wetlands.
Interior Levee	For consideration: may prove effective.

<u>Clonmell Interior Area</u>

Increase Outlet capacity	For consideration: Additional outlets may prove effective.
Pump Station	Not practical: too costly*, a large pump station would be needed reduce WSELs by measurable amounts.
Excavated Ponding Area	Not practical: ground is already low and mostly wetlands.
Interior Levee	Not practical: insufficient number of interior structures impacted by residual flooding and insufficient land available for construction.

*Typical pump station costs are expected to exceed $3 million for a 50 cfs pump station based on a recently constructed USACE project in New Jersey. A 50 cfs pump is a typical, minimum size pump needed to convey a measurable amount of runoff from a drainage area the size of the Clonmell drainage area. Annual costs for such a pump station would be approximately $160,000, which far exceeds the total residual damages in either the Repaupo or Clonmell drainage areas.

2.8.3 Analysis Results

A preliminary review of each ponding area was conducted to evaluate the physical practicality of the various alternatives. Below is a brief description of the alternatives evaluated for each drainage area and their physical viability.

Repaupo Interior Area

Increase Outlet capacity	Added one additional 6'H x 10'W gravity outlet.
Interior Levee	Evaluated two levees: elevation 2 feet and 3 feet NAVD.

Clonmell Interior Area

Increase Outlet capacity	Added one additional 6'H x 10'W gravity outlet.

Gravity Outlets. The Repaupo and Clonmell drainage areas were modeled in HEC-HMS with additional gravity outlets using the 50-year interior versus 2-year Delaware River tailwater condition. As shown in Table 2-10, additional gravity outlets do not provide any significant decrease in interior flood elevations. Therefore, additional gravity outlets are not viable interior drainage alternatives.

Table 2-9: Gravity Outlet Alternatives; Interior Water Surface Elevations

Interior Drainage Feature	Peak WSEL (feet, NAVD)	
	Repaupo	Clonmell
Minimum Facility	2.03	3.17
Addt Gravity Outlet	1.99	3.17
Decrease in WSEL (ft)	0.04	0.00

Interior Levee. Due to the low marshlands surrounding Gibbstown, excavation of additional ponding areas is not practical. However, forced storage in the form of an interior levee/floodwall to protect structures subject to residual flooding was considered. The levee/floodwall structure does not have to be high – to elevation 2 feet or 3 feet NAVD88 - to provide protection from residual flooding. These elevations were selected based on: (1) expected expense of levees/floodwalls based on height, (2) number and location of structures needing protection, and (3) available high ground near structures impacted by residual ponding.

Figures 2-7 and 2-8 below are plan layouts of the two interior levee/floodwall alternatives: to elevation 2 feet NAVD and elevation 3 feet NAVD. The reductions to interior area storage for the 2-foot NAVD elevation alternative were deemed insignificant by inspection and, therefore, the interior stage storage was not adjusted and a re-run of the HEC-HMS interior model was not warranted. The original minimum facilities runs were used, with adjustments to the structure inventory to reflect the presence of the levee.

APPENDIX B: INTERIOR DRAINAGE ANALYSIS

The interior area storage changes for the 3-foot elevation alternative were significant enough to re-run the HEC-HMS interior drainage model.

Figure 2-7: Repaupo 2ft NAVD Interior Levee Alternative

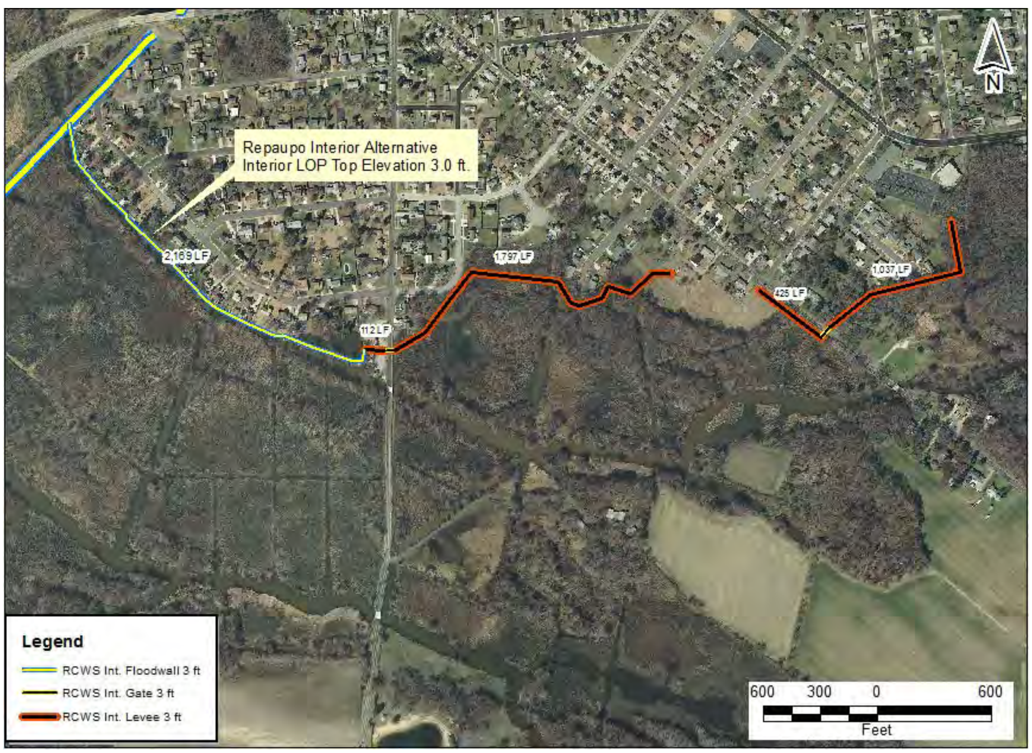


Figure 2-8: Repaupo 3ft NAVD Interior Levee Alternative

Construction cost estimates for the two Repaupo interior line-of-protection options and the resultant net benefits are shown in Table 2-11. Although both alternatives alleviate significant amounts of the residual damages, the costs far exceed the benefits. Therefore, they are not economically viable alternatives.

Table 2-10: Repaupo Area Alternatives Analysis

Basin Description	First Construction Cost(3)	Total Annual Cost(4)	Equivalent Annual Damage	Annual Damages Reduction	Annual Net Benefits
Minimum Facilities	n/a	n/a	$84,428	$0	$0
Elev 2’ – interior levee/FW(1)	$6,308,000	$274,000	$20,526	$63,902	($210,098)
Elev 3’ – interior levee/FW(2)	$14,010,000	$608,400	$3,668	$80,760	($527,640)

1) Approximately 2,500 feet of levee/floodwall (Figure 7).
2) Approximately 5,500 feet of levee/floodwall (Figure 8).
3) Includes E&D (15%), S&A (10%), and Contingency (25%), and land costs.
4) Annual Costs and Residual Damages are based on a discount rate of 3.5% and a 50-year project life; Annual Costs include Operation and Maintenance.

The results of the hydrologic analysis for the elevation 3 feet (NAVD88) levee alternative are shown in Table 2-12. The peak water surface elevation increases due to the loss of storage in this area were relatively minimal – up to only 0.06 feet. As noted in the table, the maximum modeled interior WSEL is 2.99 feet, at the top of the levee.

Table 2-11: Elevation 3.0 feet (NAVD88) Levee Results for the Repaupo/White Sluice Interior Drainage Area

Gibbstown Interior Drainage **Repaupo Ponding Area** **Alternative 3-ft. Elev. Interior LOP**

Summary of Results & Conditions Modelled

Alternative	Interior Flow	Exterior Stage	Time Condition	HEC-HMS Module/file	Peak WS	Interior Flow	Exterior Stage	Time	HEC-HMS Module/file	Peak WS	Max WS	Risk Condition
	1yr	Normal	Current	P001 TW_NORM	-0.44						-0.44	Lower Bound
	2yr	Normal	Current	P002 TW_NORM	-0.09						-0.09	Lower Bound
	5yr	Normal	Current	P005 TW_NORM	0.41						0.41	Lower Bound
	10yr	Normal	Current	P010 TW_NORM	0.78						0.78	Lower Bound
	25yr	Normal	Current	P025 TW_NORM	1.21						1.21	Lower Bound
	50yr	Normal	Current	P050 TW_NORM	1.49						1.49	Lower Bound
	100yr	Normal	Current	P100 TW_NORM	1.76						1.76	Lower Bound
	250yr	Normal	Current	P250 TW_NORM	2.10						2.10	Lower Bound
	500yr	Normal	Current	P500 TW_NORM	2.34						2.34	Lower Bound
	1yr	2yr	Current	P001 TW002	-0.07	2yr	Normal	Current	P002 TW_NORM	-0.09	-0.07	Most Likely
	2yr	2yr	Current	P002 TW002	0.31	2yr	2yr	Current	P002 TW002	0.31	0.31	Most Likely
	5yr	2yr	Current	P005 TW002	0.93	2yr	5yr	Current	P002 TW005	0.31	0.93	Most Likely
	10yr	2yr	Current	P010 TW002	1.29	2yr	10yr	Current	P002 TW010	0.31	1.29	Most Likely
	25yr	2yr	Current	P025 TW002	1.76	2yr	25yr	Current	P002 TW025	0.31	1.76	Most Likely
	50yr	2yr	Current	P050 TW002	2.06	2yr	50yr	Current	P002 TW050	0.32	2.06	Most Likely
	100yr	2yr	Current	P100 TW002	2.34	2yr	100yr	Current	P002 TW100	0.40	2.34	Most Likely
	250yr	2yr	Current	P250 TW002	2.72	2yr	250yr	Current	P002 TW250	0.42	2.72	Most Likely
	500yr	2yr	Current	P500 TW002	2.99	2yr	500yr	Current	P002 TW500	0.42	2.99	Most Likely
	1yr	10yr	Current	P001 TW010	-0.07	10yr	Normal	Current	P010 TW_NORM	0.78	0.78	Upper Bound
	2yr	10yr	Current	P002 TW010	0.31	10yr	2yr	Current	P010 TW002	1.29	1.29	Upper Bound
	5yr	10yr	Current	P005 TW010	0.97	10yr	5yr	Current	P010 TW005	1.32	1.32	Upper Bound
	10yr	10yr	Current	P010 TW010	1.34	10yr	10yr	Current	P010 TW010	1.34	1.34	Upper Bound
	25yr	10yr	Current	P025 TW010	1.80	10yr	25yr	Current	P010 TW025	1.37	1.80	Upper Bound
	50yr	10yr	Current	P050 TW010	2.12	10yr	50yr	Current	P010 TW050	1.41	2.12	Upper Bound
	100yr	10yr	Current	P100 TW010	2.40	10yr	100yr	Current	P010 TW100	1.47	2.40	Upper Bound
	250yr	10yr	Current	P250 TW010	2.81	10yr	250yr	Current	P010 TW250	1.50	2.81	Upper Bound
	500yr	10yr	Current	P500 TW010	3.07	10yr	500yr	Current	P010 TW500	1.68	3.07	Upper Bound

2.9 Gibbstown Interior Drainage Protection

As described above, alternative interior drainage plans were formulated to provide safe and reliable protection from interior flooding. Due consideration was given to evaluating only feasible alternatives, that is, alternatives that are implementable and provide equitable protection to properties within the line-of-protection. Selection of a recommended plan thus focused on economics, that is, providing the optimum reduction in damages for the cost of protection.

Using these criteria, the minimum facilities were selected for recommendation for the Gibbstown line of protection. The alternatives considered are not economically viable. Table 2-13 lists the interior drainage features.

Table 2-12: Gibbstown Interior Drainage Features

Drainage Area	Culvert Type	Nr. Of Culverts	Size
Repaupo/White Sluice	Box	6	6’ x 10’
Clonmell	Box	3	4’ x 10’
Town Center	Box	3	3’ x 4’
Town Center 2	RCP	1	36”

3.0 Interior Drainage Analysis – Lambertville

3.1 Proposed Protection Plan

The proposed project in the north end of the City of Lambertville includes approximately 516 LF of levee at Alexauken Creek and approximately 1,409 LF of floodwall along the D&R Canal. Alexauken Creek lies upstream towards the city's northern border and has a 15 square-mile drainage area. Nearing the confluence with the Delaware River, Alexauken Creek goes under a railroad bridge and is then carried under the D&R Canal aqueduct, approximately 300 feet before it meets the Delaware. The project area and proposed lines of protection are shown in Figure 3-1.

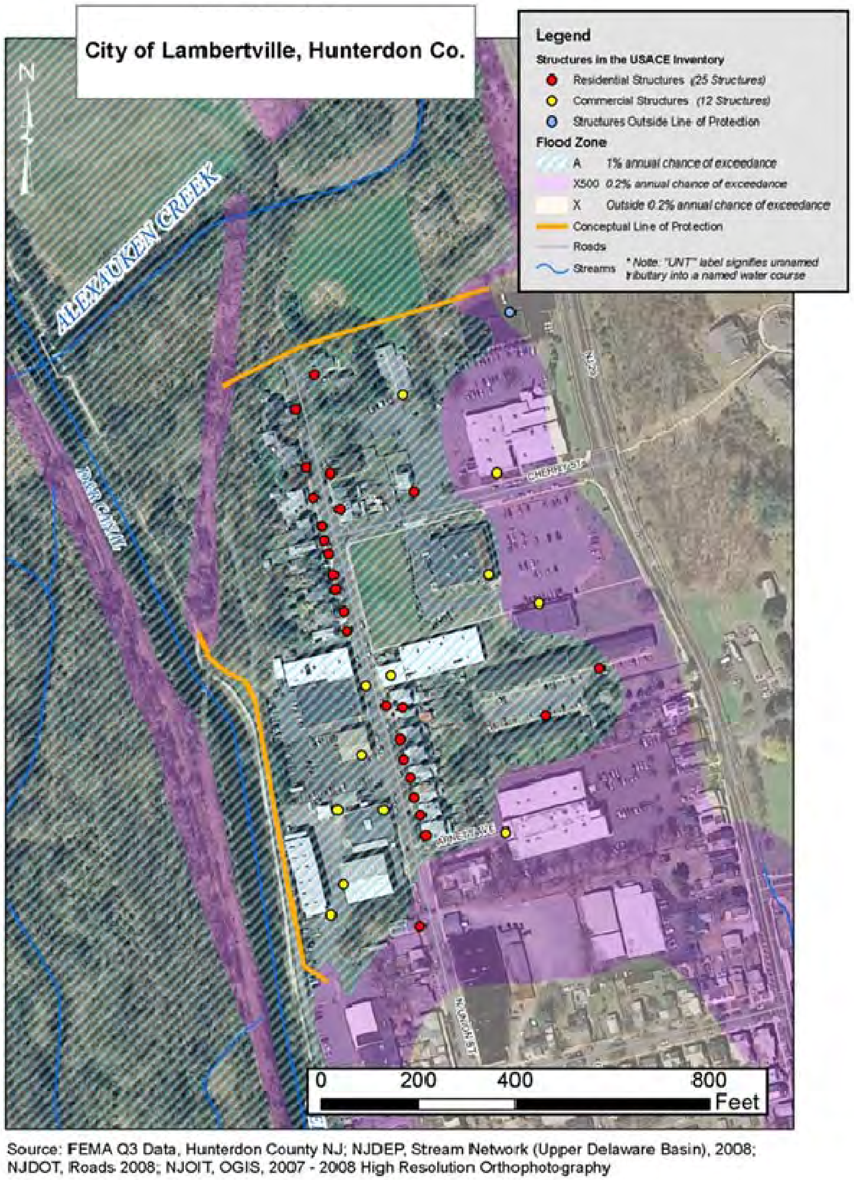


Figure 3-1: Lambertville Line of Protection

3.2 Existing Hydrology

3.2.1 Existing Studies

The City of Lambertville recently had a small mitigation project constructed using FEMA grant funds. The project was constructed on Ely Creek and consisted of a sluice gate, pump riser, and portable pump. The project is described in the *Engineering Design Report for Ely Creek Backflow Prevention Project*, conducted by Princeton Hydro, LLC, dated January 19, 2012. Much of the initial HEC-HMS model parameters in this interior drainage study were taken from the Princeton Hydro model. The model parameters were reviewed and incorporated into the complete line of protection model for the interior drainage analysis. Much of the description of the model parameters below were taken from the existing study.

3.2.2 Hydrologic Conditions

The drainage areas were previously established using the following three sources: 1) Taylor, Wiseman, & Taylor's "Drainage and Detention Calculations for Lambert's Hill P.R.D." prepared for Orleans Homebuilders, Inc. (February 5, 2003), 2) Stires Associates' "Drainage Report for the Estates at West Amwell, Lots 17.01, 17.02, & 20 Block 3" prepared for Calton Homes, Inc. (January 28, 2000), and 3) Parsons Brinckerhoff's "Engineer's Report for Stream Encroachment Application, NJ Route 29" (2005). Additional drainage areas outside the Ely Creek study area were developed and included with the above mentioned areas to create the HEC-HMS model's drainage basin. The drainage areas are shown in Figure 3-2.

3.3 Interior Drainage Model

3.3.1 Hydrologic Conditions

The USACE's HEC-HMS (Hydrologic Modeling System) was used to analyze the runoff and interior drainage features' performance. The model was created using parameters and drainage areas from the sources previously mentioned and supplemented with additional drainage areas and features as necessary in the project area behind the proposed lines of protection. The Lambertville HEC-HMS model parameters are shown in Attachment 1. A HEC-HMS schematic of the model is shown in Figure 3-3.

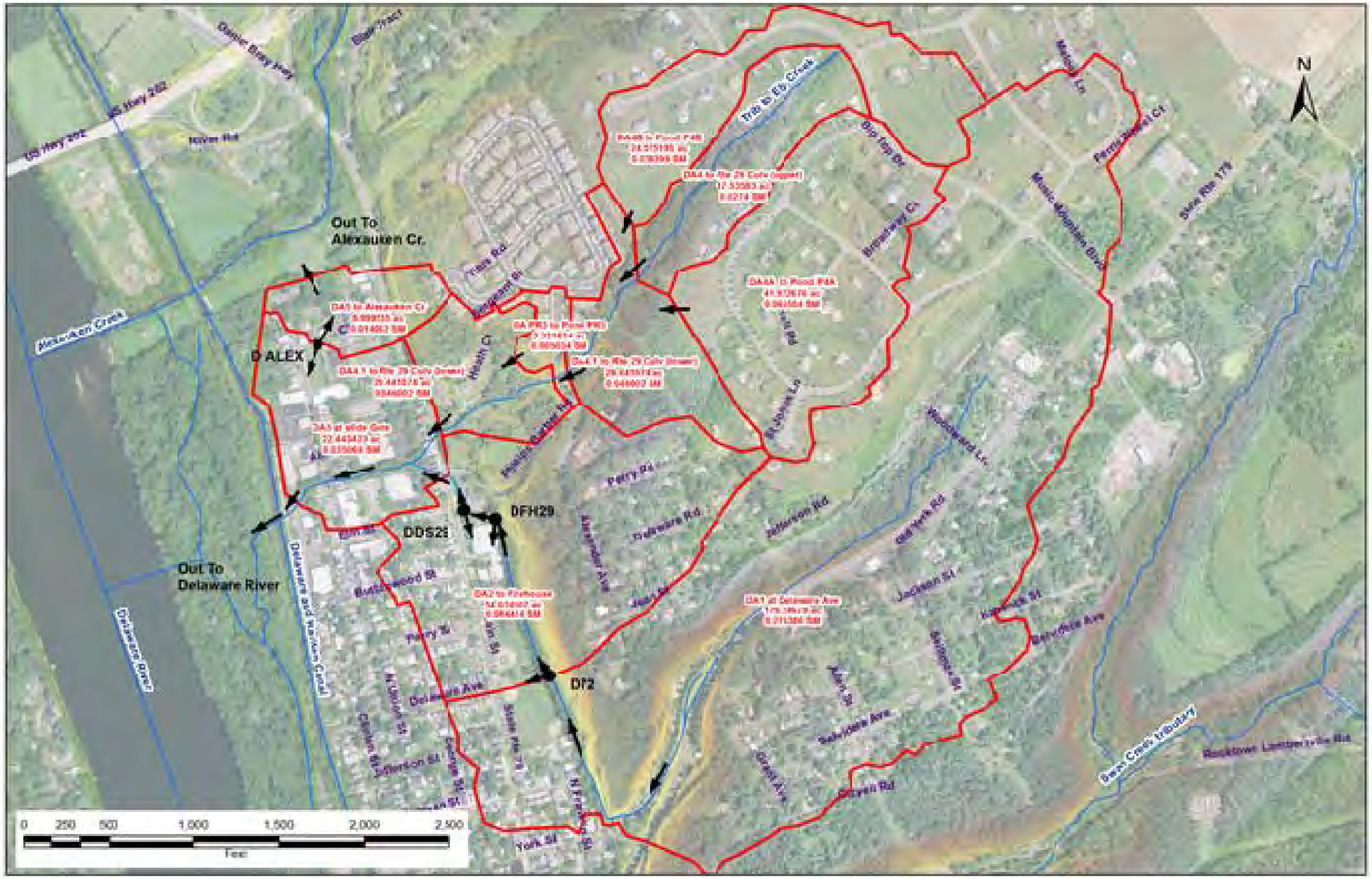


Figure 3-2: Lambertville Drainage Areas

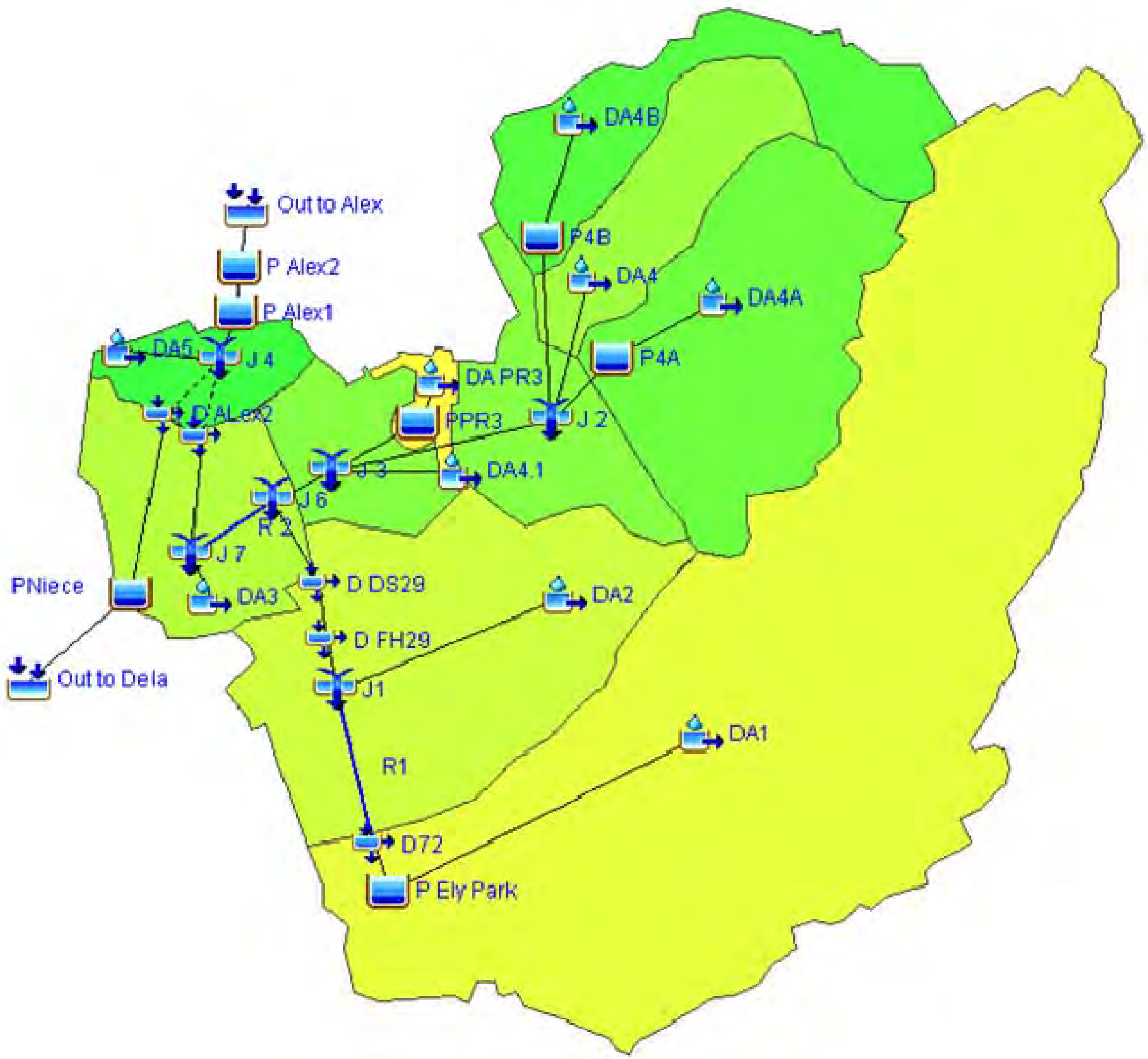


Figure 3-3: HEC-HMS Model Schematic

Ely Creek begins upstream of the site, flowing along New Jersey Route 179 until making a sharp turn at York Street, continuing in a northwesterly direction. At Delaware Avenue, flow is diverted to a 72" trunk line (D72) flowing beneath Delaware Avenue and discharging to Island Creek. A secondary diversion (DFH29) of Ely Creek includes inlets behind the Elementary School. The trunk line down Delaware Avenue was constructed in 2001 along with the primary and secondary diversions. A third diversion of Elk Creek (DDS29) consists of an overflow placed above the stream bed at the point where Ely Creek goes under the firehouse driveway which is located just upstream of the culvert under Route 29. The diversion (DDS29) is not diverting base flows in Ely Creek but only diverts flood flows.

Once Ely Creek water passes diversion D72 at Delaware Avenue it flows toward the school building on North Main Street where it merges with the outflow from drainage area DA2. This combined flow travels through channel R1 behind the Elementary School complex where it is impacted by a second diversion (DFH29) through the inlets behind Elementary School and short distance downstream for third diversion (DDS29) at Firehouse driveway. All diverted water is conveyed back through the trunk line along Delaware Avenue and ultimately to the Delaware River.

After the diversions, the remaining Ely Creek flow combines with outflow from drainage areas DA4, DA4a, DA4b, DAPR3 and DA4.1, upstream of Route 29 and is carried on R2 toward an area of natural detention (PNiece). An outflow structure consisting of a 6.5 ft W x 3 ft H culvert drains the creek under the rail road and D & R Canal. If flood flows exceed the storage capacity of the natural storage area, storm water flows over the railroad bed to the D&R canal.

As flood depths increase in the Niece ponding area (typically at lower frequency events) in the vicinity of Cherry Street just west of the CVS building, natural diversion occurs toward Alexauken Creek (DAlex and DAlex2). These diversions are presented in the HEC-HMS model as lateral spillways. This diverted flow is combined (J4) with the outflow from the local drainage area (DA5) and is carried through a smaller, natural detention area (PAlex), finally exiting toward Alexauken Creek. The existing ground configuration at the location of the proposed levee is modeled as a weir, later the location of a drainage outlet in the line of protection. The area is shown in more detail in Figures 3-4 and 3-5. The diversion structures are shown in Figures 3-6 and 3-7. The flood mitigation project (sluice gate) is shown in Figure 3-8.

3.3.2 Precipitation Data

Precipitation data was obtained from New Jersey 24 Hours Rain Fall Frequency Data for 1, 2, 5, 10, 25, 50 and 100-year events and supplemented by NOAA Atlas 14, Volume 2, Version 3, for location name: Lambertville, New Jersey, US Point Precipitation Frequency for the estimated 24 hour, 500-year event. The 250-year event was interpolated from average recurrence interval/precipitation depth chart in NOAA Atlas 14.

3.3.3 Boundary Conditions

Due to the significant difference in drainage areas, the Delaware River flood elevations were not used as boundary conditions due to the expected difference in peak flow timing. The Delaware River peak occurs almost 2 days after significant rainfall. By constrast, the Alexauken Creek drainage area is approximately 15 square miles while the Ely Creek/line of protection drainage area is only approximately 0.6 square miles. Due to this difference, the line of protection was modeled using free flowing outfalls. It is expected that Ely Creek flooding occurring as a result of precipitation on the Alexauken/Ely Creeks drainage areas would be well before the corresponding peak on Alexauken Creek.

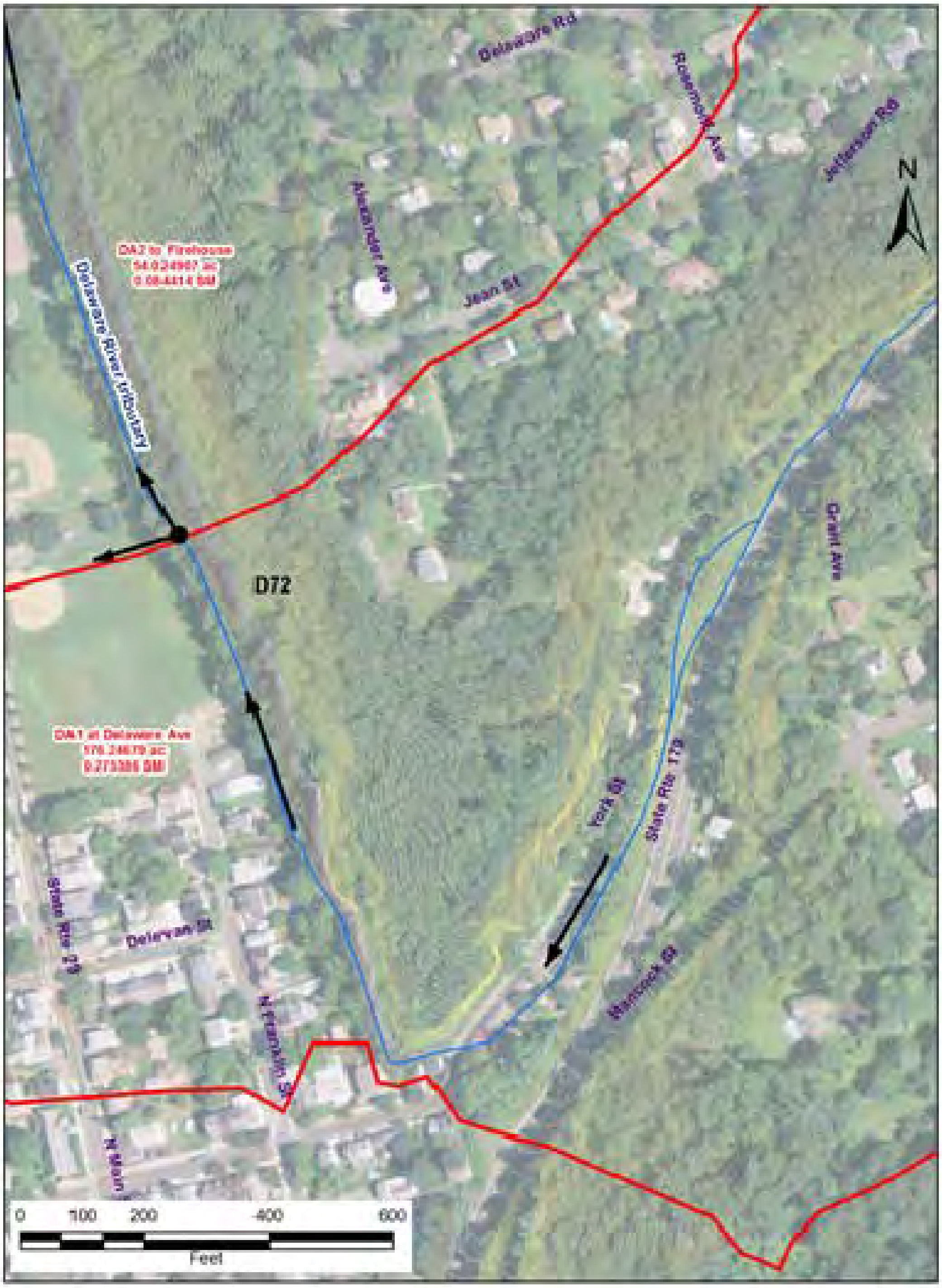


Figure 3-4: Ely Creek and D72 Diversion

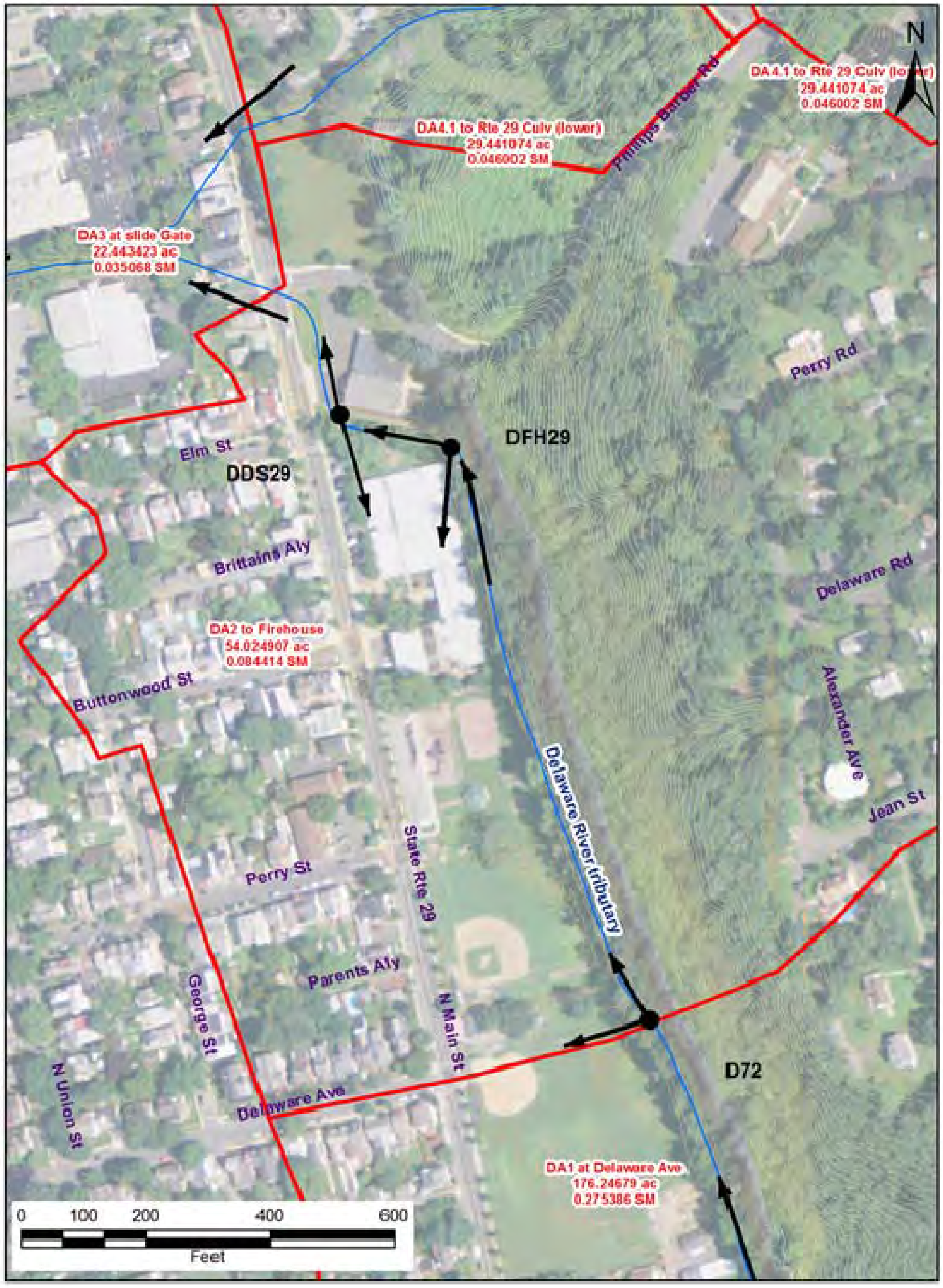


Figure 3-5: Ely Creek and DFH29 and DDS29 Diversions

Figure 3-5: Diversion D72

Figure 3-6: Diversion DDS2972 (on left)

Figure 3-7: FEMA Mitigation Project (sluice gate)

3.4 Minimum Facilities

Adjustments to the hydrologic model to account for the line of protection included:

1) the overflow weir condition along the railroad in ponding area PNiece was removed, and,

2) the outflow weir at PAlex (natural ground configuration, outflowing onto the adjacent ball field) was replaced with a culvert(s) through the proposed levee. The culvert includes a moderate drop structure consisting in a three-sided weir to focus outflow to the culvert.

The FEMA-mitigation project sluice gate remained in place as did all the upstream diversion from Ely Creek. Outflow from all pipes was assumed to be one directional.

As stated in the introduction, the minimum facilities should provide interior flood relief such that during low exterior stages the local storm drainage system functions essentially as it did without flood protection in place, up to that of the local storm sewer design. The minimum facilities represent the minimum drainage required such that no induced flooding occurs during low exterior stages.

APPENDIX B: INTERIOR DRAINAGE ANALYSIS

The existing diversions on Ely Creek as well as the Ely Creek outfall and sluice gate were expected to continue operating during precipitation events. With no backwater effects (see Boundary Conditions above), the Ely Creek sluice gate was not closed. Through an iterative process, it was determined that the Ely Creek outfall in conjunction with a 48” RCP culvert with inlet and outlet headwalls and scour protection on the north side of the levee were sufficient to meet minimum facility requirements. The minimum facilities conditions are shown in Figure 3-9.The results of the minimum facilities analysis are shown in Table 3-1.

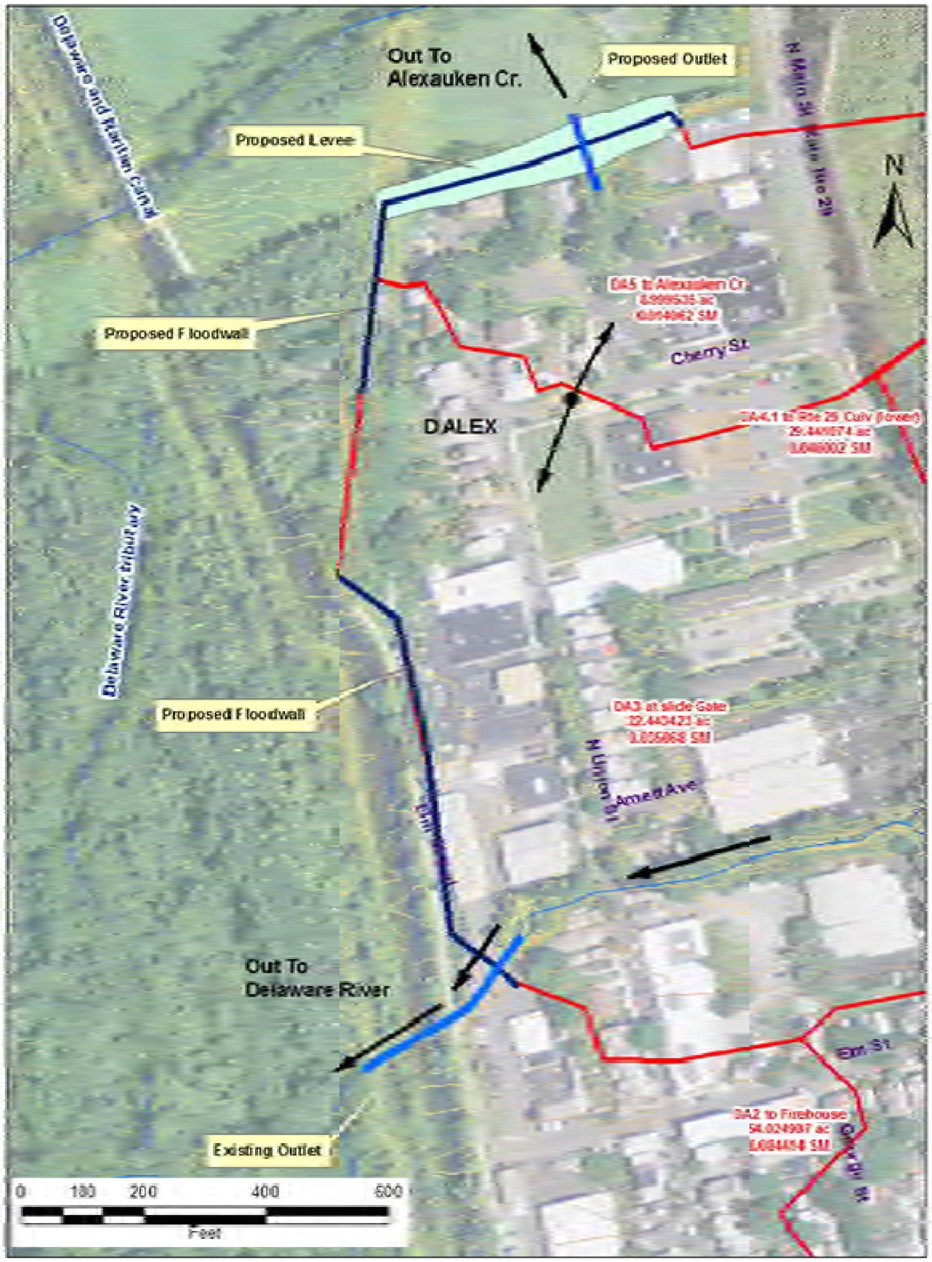


Figure 3-8: Lambertville Minimum Facilities Interior Drainage

Table 3-1: Lambertville Minimum Facilities Analysis Results

	Ely Creek			Alexauken Creek		
Annual Chance Exceedance/ Recurrence Interval	**Existing Peak (ft NAVD88)**	**Min Fac Peak (ft NAVD88)**	**Difference (ft)**	**Existing Peak (ft NAVD88)**	**Min Fac Peak (ft NAVD88**	**Difference (ft)**
99% / 1yr	58.7	58.7	0	64.8	64.5	-0.3
50% / 2yr	59.7	59.7	0	65.1	64.7	-0.4
20% / 5yr	60.8	60.8	0	65.5	64.8	-0.7
10% / 10yr	61.9	61.9	0	65.7	64.9	-0.8
4% / 25yr	64.2	64.2	0	66.0	65.1	-0.9
2% / 50yr	66.7	66.7	0	66.1	65.2	-0.9
1% / 100yr	68.6	68.6	0	66.3	65.4	-0.9
0.4% / 250yr	70.4	70.4	0	66.5	65.5	-1
0.2% / 500yr	71.0	71.0	0	67.1	67.1	0

As shown in Table 3-1, the 48" culvert at Alexauken Creek provides a small reduction in flood elevations in ponding area PAlex. This is primarily due to the three-sided concrete drop structure (lower weir coefficient than natural ground) and lower Manning's n-value for the culvert.

3.5 Alternatives Analysis

The economic analysis is also similar to that described in Section 2.7; however, upper and lower bound elevations were not calculated because the interior drainage features were modeled as free flowing (no backwater) as described above.

The alternatives analyses of interior drainage facilities for the Lambertville line of protection included the following alternatives:

1) Additional gravity outlets.
2) Pump stations.

3.5.1 Alternatives Considered

A preliminary review of each drainage area was conducted to evaluate the physical practicality of the various alternatives. Below are brief descriptions of the alternatives and the initial assessment of viability.

Alexauken Creek Interior Area

Increase outlet capacity	Not practical: due to low flows, additional outlets are unlikely to have a measurable impact on WSELs.
Pump Station	Not practical: area and flows are too small to support pump station costs.

Ely Creek Interior Area

Increase Outlet capacity	For consideration: additional outlets may prove effective; potential impacts to canal depending on installation method.
Pump Station	For consideration. May prove too costly.

As noted, there are no economically viable options for the Alexauken Creek interior drainage area. However, five alternatives were considered for the Ely Creek interior area. The outlets for all alternatives are parallel to the existing Ely Creek culvert under the D&R Canal, as shown in Figure 3-9. A pump station outlet would likely be across the canal, making installation difficult.

3.5.2 Ely Creek Interior Area Alternatives

Increased Gravity Outlet Capacity - 3 Alternatives

Alternative 1: Approximately 300 LF of 54" Steel Pipe Culvert

Alternative 2: Approximately 300 LF of 2x42" Steel Pipe Culvert

Alternative 3: Approximately 300 LF of 3x36" Steel Pipe Culvert

Pump Station - 2 Alternatives

Alternative 4: Pump Station, 2x25 cfs with approximately 250 LF of outflow, 36" Steel Pipe Culvert

Alternative 5: Pump Station, 2x50 cfs with approximately 250 LF of outflow 42" Steel Pipe Culvert

3.5.3 Hydrologic Results

The Ely Creek alternatives were evaluated using a no-tailwater condition and a 2-year Delaware River hydrograph tailwater for sensitivity (elevation 60.83 feet NAVD). A no-tailwater condition is the most likely condition given the significant delay in the Delaware River hydrograph when compared with the small watershed of Ely Creek. The results of the hydrologic analysis are shown in Table 3-2.

Table 3-2: Ely Creek Alternatives Analysis Results

	Ely Creek at Delaware River (PNiece Ponding Area)						
Annual Chance Exceedance/ Recurrence (yr)	**Existing Peak Elev. No TW (ft NAVD)**	**LOP MF Peak Elev. No TW (ft NAVD)**	**Alternative 1 Peak Elev. No TW (ft NAVD)**	**Alternative 2 Peak Elev. No TW (ft NAVD)**	**Alternative 3 Peak Elev. No TW (ft NAVD)**	**Alternative 4 Peak Elev. No TW (ft NAVD)**	**Alternative 5 Peak Elev. No TW (ft NAVD)**
99% / 1yr	58.7	58.7	58.4	58.3	58.2	58.7	58.7
50% / 2yr	59.7	59.7	59.1	58.9	58.8	59.7	59.7
20% / 5yr	60.8	60.8	59.7	59.4	59.2	59.9	60.0
10% / 10yr	61.9	61.9	60.1	59.7	59.5	60.9	60.1
4% / 25yr	64.2	64.2	61.0	60.3	60.0	61.8	61.0
2% / 50yr	66.7	66.7	62.1	61.3	60.9	65.0	62.4
1% / 100yr	68.6	68.6	64.3	63.1	62.5	68.1	66.4
0.4% / 250yr	70.4	70.4	68.4	68.0	67.4	70.1	69.6
0.2% / 500yr	71.0	71.0	69.8	69.2	68.9	70.8	70.6
Frequency (year)		**LOP MF Peak Elev. TW=60.83 (ft NAVD)**	**Alternative 1 Peak Elev. TW=60.83 (ft NAVD)**	**Alternative 2 Peak Elev. TW=60.83 (ft NAVD)**	**Alternative 3 Peak Elev. TW=60.83 (ft NAVD)**	**Alternative 4 Peak Elev. TW=60.83 (ft NAVD)**	**Alternative 5 Peak Elev. TW=60.83 (ft NAVD)**
99% / 1yr		61.0	60.9	60.9	60.9	60.8	60.1
50% / 2yr		61.7	61.0	61.0	61.0	61.0	60.9
20% / 5yr		62.6	61.2	61.2	61.2	61.3	61.0
10% / 10yr		63.6	61.4	61.3	61.3	61.8	61.0
4% / 25yr		66.0	61.9	61.8	61.8	63.5	61.8
2% / 50yr		67.7	62.8	62.7	62.6	66.2	64.1
1% / 100yr		68.9	64.8	64.5	64.4	68.3	67.5
0.4% / 250yr		70.6	68.4	68.3	68.2	70.3	70.0
0.2% / 500yr		71.1	69.8	69.6	69.5	70.9	70.7

As shown in Table 3-2, the pump station alternatives (4 and 5) did not provide significant reductions in interior WSELs. However, by increasing the gravity outlet capacity (Alts 1 through 3) there are measurable decreases in water surface elevations.

It should be noted that by implementing additional gravity outlets (Alt 1 – 3) in the Ely Creek PNiece Ponding Area that the water surface elevations for all studied events would drop such that natural diversion at elevation 70.15 feet NAVD88 at Cherry St from the Ely Creek area to the Alexauken Creek area would probably not occur. A reduction or elimination of diversion toward Alexauken Creek (north) would reduce residual flooding in the northern ponding area by a proportional amount.

3.5.4 Economic Analysis

The economic analysis of alternatives is described in Section 2.7.

Costs. Construction cost estimates for the five alternatives are presented in Table 3-3. The current Federal discount rate is 3.5%; the project life is 50 years.

Benefits. Using HEC-FDA, Average Annual Damages were calculated for the base year and future years with the minimum facilities and alternatives in place. Average Annual Damages were calculated for the 50-year period of analysis and a Federal discount rate of 3.50%. A summary of from Alternatives 1 to 5 and the resulting benefit-cost ratio in comparison with each other is presented in Table 3-4. As noted previously, Alternative 3 is slightly more costly than Alternative 1; however, it does provide the same net benefits and provides more annual benefits than Alternative 1.

Table 3-3: Construction Cost Estimates – Ely Creek Alternatives

ACCT #	CWBS FEATURE	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5
06	FISH & WILDLIFE FACILITIES	$0	$0	$0	$0	$0
11	LEVEES AND FLOODWALLS	$0	$0	$0	$0	$0
15	FLOODWAY CONTROL-DIVERTION STRUCTURE	$997,100	$1,201,000	$1,275,000	$4,232,200	$5,783,000
	TOTAL CONSTRUCTION COST	**$997,100**	**$1,201,000**	**$1,275,000**	**$4,232,200**	**$5,783,000**
01	LANDS & DAMAGES	$81,290	$81,290	$81,290	$81,290	$81,290
30	ENGINEERING & DESIGN	$129,000	$154,000	$163,000	$518,000	$704,000
31	CONSTRUCTION MANAGEMENT	$108,000	$128,000	$136,000	$431,000	$586,000
	TOTAL FIRST COST	$1,315,390	$1,564,290	$1,655,290	$5,262,490	$7,154,290
	Interest During Construction (IDC)	$56,253	$66,897	$70,789	$225,052	$305,956
	TOTAL INVESTMENT COST	**$1,371,643**	**$1,631,187**	**$1,726,079**	**$5,487,542**	**$7,460,246**
	ANNUAL COST	$58,478	$69,544	$73,589	$233,954	$318,058
	O&M	$5,542	$6,590	$6,974	$22,171	$30,141
	TOTAL ANNUAL COST	**$64,020**	**$76,134**	**$80,563**	**$256,125**	**$348,199**

Note: May 2014 price level, 3.5% Federal discount rate, 50-year project life.

Table 3-4: Lambertville Alternatives 1 to 5, Summary of BCRs

Alternative	Interior Drainage Area	Flood Damages[1] Minimum Facility	Flood Damages[1] With Alternative	Annual Benefits	Total First Cost[2]	Total Investment Cost[3]	Total Annual Cost[4]	Net Benefits	BCR
Alt 1	Alexauken	$38	$0	$38					
54" pipe	Ely	$196,127	$51,899	$144,228					
	Total	**$196,165**	**$51,899**	**$144,266**	**$1,315,400**	**$1,372,000**	**$64,000**	**$80,300**	**2.25**
Alt 2	Alexauken	$38	$0	$38					
2x42" pipe	Ely	$196,127	$41,573	$154,554					
	Total	**$196,165**	**$41,573**	**$154,592**	**$1,564,300**	**$1,631,000**	**$76,100**	**$78,500**	**2.03**
Alt 3	Alexauken	$38	$0	$38					
3x36" pipe	Ely	$196,127	$35,255	$160,872					
	Total	**$196,165**	**$35,255**	**$160,910**	**$1,655,300**	**$1,726,000**	**$80,600**	**$80,300**	**2.00**
Alt 4	Alexauken	$38	$1	$38					
50 cfs pump	Ely	$196,127	$143,395	$52,733					
	Total	**$196,165**	**$143,395**	**$52,770**	**$5,262,500**	**$5,488,000**	**$256,100**	**($203,300)**	**0.21**
Alt 5	Alexauken	$38	$0	$38					
100 cfs pump	Ely	$196,127	$97,939	$98,188					
	Total	**$196,165**	**$97,939**	**$98,227**	**$7,154,300**	**$7,460,000**	**$348,200**	**($250,000)**	**0.28**

Note: 1) Average Annual Damages.
2) Includes contingencies (35%),
3) Includes IDC,
4) Includes O&M

3.6 Lambertville Optimum Plan

As described above, alternative interior drainage plans were formulated to provide safe and reliable protection from interior flooding. Due consideration was given to evaluating only feasible alternatives, that is, alternatives that are implementable and provide equitable protection to properties within the line-of-protection. Selection of a recommended plan thus focused on economics; that is, providing the optimum reduction in damages for the cost of protection.

The results of the analysis for Lambertville indicate that additional gravity outlets at Ely Creek are the optimized plan for interior drainage. Multi-pipe outlets were evaluated due to the varying costs for jacking different size pipes. Installation of three 36" pipes is more costly than jacking one 54" pipe; however, the multi-pipe outlet yields greater benefits and results in the same net benefits as the single pipe.

The additional gravity outlet at Ely Creek includes a jacked pipe (RCP culvert) adjacent to the existing gravity outfall at the lumber yard (see Figure 3-8), with a sluice gate. The pipe must be jacked under the canal due to the cultural nature of the canal; a cut and fill method of installation under the canal would likely result in too many cultural resources impacts.

ATTACHMENT 1 – Lambertville Interior Drainage Model Parameters

HEC-HMS Report

BASIN MODEL: "Ely Creek LOP MF1"

Subbasin Table																	
Subbasin	**Area (Sq.Mi.)**	**Loss**	**% Imp**	**CN**	**Init Loss**	**Init Abst**	**Const Loss**	**Trans-form**	**Tc (min.)**	**LAG (min.)**	**Stor-Coeff**	**Base Flow**	**Rcn Fact**	**Flow Area**	**Flow Peak**	**Down Stream**
DA1	0.2754	SCS	0.0	73	-	-	-	SCS	- - -	24	- - -	None	-	-	-	P Ely Park
DA2	0.0844	SCS	0.0	68	-	-	-	SCS	- - -	15	- - -	None	-	-	-	J1
DA3	0.0351	SCS	0.0	82	-	-	-	SCS	- - -	11	- - -	None	-	-	-	J 7
DA4	0.0274	SCS	9.8	55	-	-	-	SCS	- - -	15.5	- - -	None	-	-	-	J 2
DA4.1	0.046	SCS	3.51	61	-	-	-	SCS	- - -	12	- - -	None	-	-	-	J 3
DA4A	0.0655	SCS	0.0	68	-	-	-	SCS	- - -	15.8	- - -	None	-	-	-	P4A
DA4B	0.0384	SCS	0.0	79	-	-	-	SCS	- - -	25.6	- - -	None	-	-	-	P4B
DA5	0.0141	SCS	0.0	82	-	-	-	SCS	- - -	6	- - -	None	-	-	-	J 4
DA PR3	0.005	SCS	0.0	78	-	-	-	SCS	- - -	6	- - -	None	-	-	-	PPR3

Storage Element Table					
Reservoir/Pond Area	**Route Meth.**	**Route Curve**	**Init. El. (ft)**	**El:A or V:Q table**	**Down Stream**
P4A	Modified Puls	Elevation-Area-Outflow	QIn=Out	P4A EL AR	J 2
P4A Info:	Detention Pond P4A				
P4B	Controlled Outflow	Elevation-Area	QIn=Out	P4B EL AR	J 2
P4B Info:	Detention Pond P4B				
P Alex1	Controlled Outflow	Elevation-Area	QIn=Out	P Alex ElAr	P Alex2
P Alex2	Controlled Outflow	Elevation-Area	QIn=Out	P Alex ElAr	Out to Alex
P Alex2 Info:	Ponding at future LOP				
P Ely Park	Modified Puls	Elevation-Area-Outflow	QIn=Out	ElyParkSASE	D72
P Ely Park Info:	Pond Ely Park at Delaware Ave				

PNiece	Controlled Outflow	Elevation-Area	QIn=Out	PNieceEIAR	Out to Dela
PNiece Info:	Ponding at Niece Lumber Property				
PPR3	Controlled Outflow	Elevation-Storage	QIn=Out	- - -	J 3
PPR3 Info:	Detention Pond PPR3				

Storage Element Tailwater Table

Reservoir/Pond Area	Main TW Condition	Main TW Table Name	Aux. TW Condition	Aux. TW Table Name
P4A	None	- - -	None	- - -
P4B	None	- - -	None	- - -
P Alex1	None	- - -	None	- - -
P Alex2	None	- - -	None	- - -
P Ely Park	None	- - -	None	- - -
PNiece	None	- - -	None	- - -
PPR3	None	- - -	None	- - -

Storage Element Outlet Table

Struct. Name/outlet type	Outlet Direction	Outlet Shape/Type	Dia Rise-SPan/Area (ft or sf)	No. Barrels	inv in /CL El	inv. out	Culv. Length (ft)	Mann n/Orif Coef
P4B								
Orifice	Main	Area	0.04906	1	200.125	-	-	0.06
Orifice	Main	Area	5.25	1	207	-	-	0.6
PPR3								
Culvert	Main	Circular	0.25	1	117	117	0.5	0.013
Culvert	Main	Box	0.25 x 2	1	118.6	118.6	0.5	0.013
Orifice	Main	Area	5.64	1	121.4	-	-	0.8
PNiece								
Culvert	Main	Arch	3 x 6.5	1	57.21	52.5	306	0.015
P Alex2								
Culvert	Main	Circular	5	1	60.5	60	90	0.013

Storage Element Spillway Table

Struct. Name/Spillway type	Outlet Direction	Length	Elevation(ft/sf)	Coefficient
PPR3				
Broad-Crested Spillway	Main	20	120.4	3
PNiece				
Broad-Crested Spillway	Main	600	72	2.68
Broad-Crested Spillway	Auxiliary	80	70.15	2.68
P Alex1				
Broad-Crested Spillway	Main	12	64.1	3

Junction Table

Junction	Downstream
R1	J1
R 2	J 7

Reach Table

Reach	Route Meth.	Lag	Ch Loss	No. Reachs	init flow equal	storage outflw table	Musk. k	Musk. x	Musk. Steps	Downstream
R1	Kine. Wave	-	None	- - -	No	None	- - -	- - -	- - -	J1
R 2	Kine. Wave	-	None	- - -	No	None	- - -	- - -	- - -	J 7

Diversion Table

Diversion	Downstream	Divert To	Diversion Method
D72	R1		Inflow-Diversion Table
D DS29	J 6		Inflow-Diversion Table
D FH29	D DS29		Inflow-Diversion Table

Current as of 5 May 2014 at 19:15:00