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 A: Engineering Technical Appendix

Section 2: Hydrology and Hydraulics

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PHILADELPHIA DISTRICT

NEW JERSEY DEPARTMENT OF ENVIRONMENTAL PROTECTION

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APPENDIX A, Section 2: Hydrology and Hydraulics

Introduction

The Delaware River floodplain, as well as those of its tributaries, has been subject to both local and widespread damage caused by excessive rainfall leading to the flooding of lands and property adjacent to its streams. The Delaware River has a long history of flooding dating back to the late 1800's. Watersheds adjacent to the Delaware River, like many other watersheds have been impacted by flooding because the people live, work, travel, and recreate in floodplains, and because their land use activities have increased the runoff from watersheds and changed the hydraulics of the floodplain itself. This appendix presents the hydrology and hydraulic analyses which were conducted in order to establish existing conditions, define the flooding problems, and evaluate potential solutions.

A.1.0. Climatology of the Delaware River Basin

The Delaware River floodplain climate is largely continental even though it is very near the Atlantic coast. The air masses that influence the climate in the region move predominantly from the interior of North America and are modified by the influences of the Great Lakes and the Appalachian Mountains to the west. Generally, west to southwest airflow with extended overland travel brings the hot dry weather which is responsible for occasional summer droughts. North to south airflow occurs in winter, originating in the cold highs over Canada and bringing arctic air into the basin. Summer totals of precipitation are slightly higher than in the winter. Showers and thunderstorms produce most of the precipitation during the warm months. During the cool months, coastal storms account for most of the precipitation. The heaviest and most extended rains in the region are experienced with storms of tropical origin occurring during summer and autumn months. Winds of damaging force accompany hurricanes, nor'easters, and occasionally the severe thunderstorms during the summer months.

A.1.1. Temperature

The average monthly temperatures in the basin tend to increase from the headwaters of the Delaware River to the mouth. This trend can be seen in Table A.1.1. The table lists the monthly and annual mean temperatures compiled at several National Climatic Data Center (NCDC) stations within the river basin. The average winter temperatures vary from 28°F in Port Jervis, NY to 35°F in Philadelphia, PA and the summer average temperatures can vary from 70°F in Port Jervis, NY to 75°F in Philadelphia, PA. Philadelphia is approximately 145 miles south of Port Jervis, NY.

A.1.2. Precipitation

Average annual precipitation within the region is approximately 45 inches per year. Along the Delaware River in the study area it can vary from approximately 46 inches per year at Lambertville, NJ to 42 inches in the lower portion of the region at Philadelphia, PA. North of the study area in the state of New York at the Delaware River headwaters, the annual precipitation can approach 60 inches along the ridges in some of the mountainous regions. The highest monthly rainfall, approximately ten percent of the total annual amount generally occurs in the months of July and August, and the months of January and February have the lowest average monthly precipitation. Monthly and yearly precipitation averages at several NCDC stations are shown in Table A.1.2.

Table A.1.1: Mean Monthly Temperatures at Selected NCDC Stations

Month	Port Jervis (ºF)	Belvidere (ºF)	Lambertville (ºF)	Trenton (ºF)	Philadelphia (ºF)
January	26.3	28.1	30.8	31.9	32.3
February	28.2	30.1	32.5	33.3	34.4
March	37.2	38.6	40.5	40.9	42.6
April	48.9	49.3	50.8	51.2	53.4
May	58.8	59.7	61.5	61.1	63.3
June	68.1	67.6	70.1	69.4	72.5
July	72.6	72.9	74.7	74.5	77.3
August	70.7	71.2	73.1	72.6	75.9
September	63.1	64.3	66.3	66.5	68.6
October	51.9	53.3	55.4	55.4	57.1
November	41.5	42.7	45.1	47.2	46.9
December	30.2	32.4	34.7	36.4	36.7
Mean Annual	**49.8**	**50.8**	**53.0**	**53.4**	**55.1**

Table A.1.2: Mean Monthly Precipitation at Selected NCDC Stations

Month	Port Jervis (in.)	Belvidere (in.)	Lambertville (in.)	Trenton (in.)	Philadelphia (in.)
January	3.05	3.25	3.50	3.35	3.20
February	2.68	2.69	2.78	2.92	2.76
March	3.57	3.74	4.08	4.16	3.80
April	3.75	3.85	3.80	3.63	3.51
May	3.99	4.03	4.05	3.72	3.59
June	4.02	4.30	4.00	3.58	3.61
July	4.19	4.75	4.73	4.65	4.16
August	3.86	4.36	4.45	4.94	3.76
September	3.92	4.26	3.97	3.96	3.60
October	3.65	3.97	3.49	3.11	2.94
November	3.62	3.62	3.77	3.45	3.21
December	3.45	3.59	3.82	3.33	3.56
Mean Annual	**43.74**	**46.39**	**46.43**	**44.81**	**41.69**

A.1.3. Storm Types

Storm occurrences in the Delaware Basin are of two general types, namely, storms of tropical origin (hurricanes) and storms of extra-tropical origin such as thunderstorms and nor'easters. These storms occur separately and together, with the most intense precipitation resulting from a combination of both types. Movement of warm moist air into contact with surrounding air of lower temperature produces the violent thunderstorms and intense precipitation of the summer months in this area, and the nor'easters of the cool months. The latter are of coastal origin and are accompanied by severe winds and flood-producing precipitation. Some of the worst floods of record, however, have been associated with hurricanes. Records show that the most severe storms occur in the Delaware River basin when a hurricane joins an extra-tropical storm and the two storms travel together exhibiting a characteristic isohyetal storm pattern having two major storm centers.

A.1.4. Climate Variability Trends

A literature review was conducted to review the current state of knowledge on climate variability in the Delaware River Basin. There was little consensus among the articles as to what degree future climate variability will impact streamflow and groundwater in the region. Most of the articles projected longer-term climate variability trends well beyond the year 2065 with very little information given up to year 2065.

During the literature review, articles were found that summarized results for the Mid-Atlantic region from several different climate models and emission scenarios. Many of the articles predicted earlier peaks in streamflow in the spring and later peaks in the autumn. As for the low-flow period in the summer, the current state of knowledge is suggesting that its period could be extended but this probably would not be observable until the end of the century and not by the year 2065. All of these conclusions are dependent upon future trends in emissions. Lower emission scenarios produce less dramatic results in the computer models than higher emission scenarios.

Generally speaking, some other trends that many articles agreed upon were:

- Average annual temperatures will increase by the end of the century by 2-4 degrees Celsius and will depend upon carbon dioxide emissions.
- More warming is expected in the summer months. Extreme summer heat days are expected to rise by the end of the century.
- Annual mean precipitation is predicted to increase by 7-9% by the end of the century. The winter months are predicted to see higher increases than any other time of the year.

A.1.5. Tides

The Delaware River in the Gibbstown area is tidally influenced and is a distinct hydraulic zone from the Delaware River at Trenton and North. The tides affecting the Gibbstown study area are classified as semi-diurnal with two nearly equal high tides and two nearly equal low tides per day. The average tidal period is actually 12 hours and 25 minutes, such that two full tidal periods require 24 hours and 50 minutes. Thus, tide height extremes (highs and lows) appear to occur almost one hour (average is 50 minutes) later each day. Table A.1.3 summarizes the tidal benchmarks by River Marker (RM) at Delaware River tidal stations near Gibbstown.

Table A.1.3: Station Datum Elevations Summary

Datum	Description	Value (feet)			
		@ RM 99	@ RM 79	@ RM 54.1	@ RM 0
MHHW	Mean Higher-High Water	10.50	27.74	7.19	7.44
MHW	Mean High Water	10.10	27.38	6.87	7.01
MSL	Mean Sea Level	7.30	24.73	4.32	5.41
DTL	Mean Diurnal Tide Level	7.16	24.67	4.27	5.11
MTL	Mean Tide Level	7.05	24.58	4.27	5.01
NAVD88	North American Vertical Datum of 1988	6.94	-	4.20	4.98
MLW	Mean Low Water	4.00	21.78	1.53	2.94
MLLW	Mean Lower-Low Water	3.81	21.60	1.35	2.78
STND	Station Datum	0.00	0.00	0.00	0.00
GT	Great Diurnal Range	6.69	6.14	5.84	4.65
MN	Mean Range of Tide	6.10	5.59	5.34	4.08

A.1.6. Historic Sea Level Change & Stage Frequency Curves

Relative sea level change must be considered in every United States Army Corps of Engineers (USACE) coastal activity as far inland as the extent of estimated tidal influence. The National Oceanic and Atmospheric Administration (NOAA) publishes monthly mean sea level trends without the seasonal fluctuations. The following text and Figures A.1.1 – A.1.3 present the sea elevation trends near the study area. The Philadelphia station recorded an increase of 2.79 mm/yr with a 95% confidence interval of +/- 0.21 mm/yr based on monthly mean sea level data from 1900 to 2006. This trend is equivalent to a change of 0.92 feet in 100 years. At the Reedy Point, DE station, the mean sea level trend is 3.46 mm/yr with a 95% confidence interval of +/- 0.66 mm/yr based on monthly mean sea level data from 1956 to 2006. At the Lewes, DE station, the mean sea level trend is 3.20 mm/yr with a 95% confidence interval of +/- 0.28 mm/yr based on monthly mean sea level data from 1919 to 2006. This trend is equivalent to a change of 1.05 feet in 100 years. This trend is equivalent to a change of 1.05 feet in 100 years.

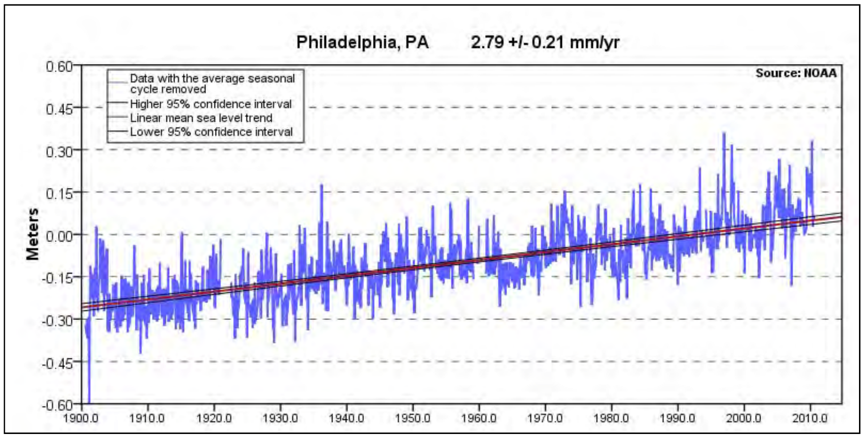

Figure A.1.1: Mean Sea Level Trend for Philadelphia, PA (from NOS)

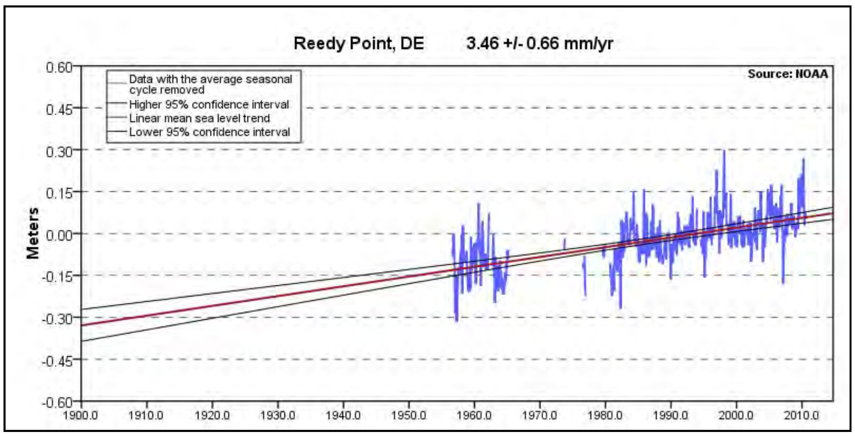

Figure A.1.2: Mean Sea Level Trend for Reedy Point, DE (from NOS)

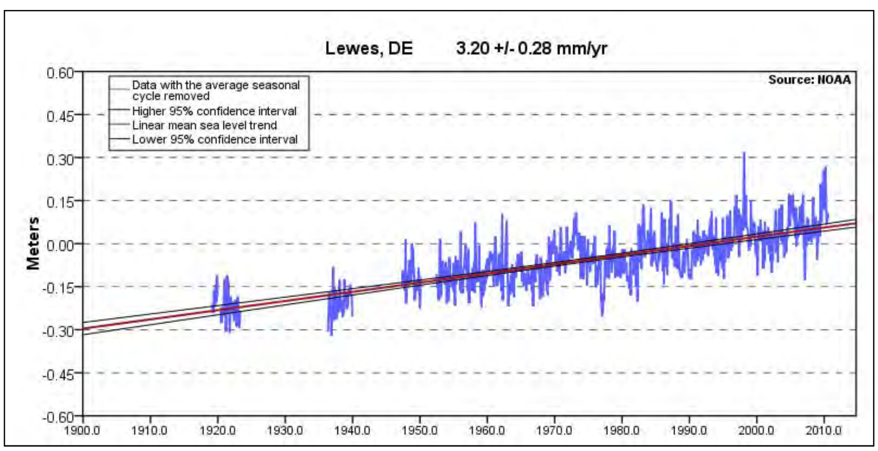

Figure A.1.3: Mean Sea Level Trend for Lewes, DE (from NOS)

Observed annual peak stages up to the year of 2006 for these tidal stations were adjusted by NOAA to incorporate the published sea-level trends. Stage frequency curves were then developed by NOAA for the Philadelphia and Lewes tidal stations based on a graphical frequency analysis of the adjusted annual peak stages. The stage frequency values at Philadelphia and Lewes were interpolated by river mile to obtain the stage frequency for the Gibbstown area which runs from river mile 82 to river mile 88.5. Table A.1.4 summarizes the adopted stage frequency used along the Gibbstown area at 0.5 river mile increments.

Table A.1.4: Delaware River Stage Frequency near Gibbstown

Event	ACE	RM 82	RM 82.5	RM 83	RM 83.5	RM 84	RM 84.5	RM 85
2-year	50%	5.46	5.46	5.47	5.48	5.48	5.49	5.50
5-year	20%	6.05	6.06	6.07	6.07	6.08	6.09	6.09
10-year	10%	6.41	6.41	6.42	6.43	6.43	6.44	6.44
25-year	4%	6.90	6.91	6.91	6.92	6.92	6.93	6.93
50-year	2%	7.22	7.23	7.23	7.24	7.24	7.25	7.25
100-year	1%	7.83	7.84	7.84	7.85	7.86	7.86	7.87
250-year	0.4%	9.32	9.32	9.33	9.33	9.34	9.34	9.35
500-year	0.2%	10.46	10.46	10.47	10.47	10.48	10.48	10.49

Datum: feet NAVD 88
RM = River Mile

Table A.1.4 (Continued): Delaware River Stage Frequency near Gibbstown

Event	ACE	RM 85.5	RM 86	RM 86.5	RM 87	RM 87.5	RM 88	RM 88.5
2-year	50%	5.50	5.51	5.52	5.52	5.53	5.53	5.54
5-year	20%	6.10	6.11	6.11	6.12	6.13	6.13	6.14
10-year	10%	6.45	6.46	6.46	6.47	6.48	6.48	6.49
25-year	4%	6.94	6.95	6.95	6.96	6.96	6.97	6.98
50-year	2%	7.26	7.26	7.27	7.28	7.28	7.29	7.29
100-year	1%	7.87	7.88	7.88	7.89	7.89	7.90	7.90
250-year	0.4%	9.35	9.36	9.36	9.37	9.38	9.38	9.39
500-year	0.2%	10.49	10.50	10.50	10.51	10.51	10.52	10.53

Datum: feet NAVD 88
RM = River Mile

A.2.0. FLOOD HISTORY ALONG THE DELAWARE RIVER

A.2.1. Knowlton Township to Trenton

The Delaware River Basin from the headwaters in New York State to Trenton is subject to significant flood damages on an annual basis. The highest flows recorded at a number of selected USGS gage locations (as shown in Figure A.2.1) situated on the Delaware River from Port Jervis, NY to Trenton, NJ are shown in Tables A.2.1 – A-2.6.

Table A.2.1: Highest Flood Peaks at Port Jervis, NY

Rank	Date	Streamflow (cfs)	Stage (feet)
1	8/19/1955	233,000	23.91
2	10/10/1903	205,000	23.10
3	6/28/2006	189,000	21.47
4	4/3/2005	166,000	20.52
5	9/18/2004	151,000	19.52
6	1/20/1996	134,000	18.37
7	5/23/1942	140,000	17.76
8	3/18/1936	137,000	17.55
9	6/29/1973	108,000	16.48
10	3/15/1986	102,000	16.02

Table A.2.2: Highest Flood Peaks at Montague, NJ

Rank	Date	Streamflow (cfs)	Stage (feet)
1	8/20/1955	250,000	35.15
2	10/11/1903	217,000	31.10
3	6/28/2006	212,000	32.15
4	4/3/2005	206,000	31.69
5	9/18/2004	168,000	28.37
6	3/19/1936	164,500	28.45
7	1/21/1996	149,000	26.66
8	5/24/1942	136,500	25.70
9	4/2/1940	123,100	24.33
10	7/1/1973	115,000	23.40

Table A.2.3: Highest Flood Peaks at Delaware Water Gap, PA

Rank	Date	Streamflow (cfs)	Stage (feet)
1	8/20/1955	260,000	37.40
2	6/29/2006	225,000	33.87
3	4/3/2005	215,000	33.25
4	9/19/2004	176,000	30.32
5	1/21/1996	155,000	28.40
6	3/16/1986	110,000	24.00
7	7/1/1973	103,000	23.82
8	5/31/1984	97,300	21.77
9	4/18/1983	88,100	20.97
10	4/3/1993	86,300	19.89

Table A.2.4: Highest Flood Peaks at Belvidere, NJ

Rank	Date	Streamflow (cfs)	Stage (feet)
1	8/19/1955	273,000	30.21
2	10/10/1903	220,000	28.60
3	4/4/2005	226,000	27.22
4	6/29/2006	225,000	27.16
5	3/19/1936	179,000	25.00
6	9/19/2004	184,000	24.80
7	1/20/1996	158,000	22.96
8	4/1/1940	138,300	21.40
9	5/24/1942	133,700	20.97
10	3/16/1986	126,000	20.34

Table A.2.5: Highest Flood Peaks at Riegelsville, NJ

Rank	Date	Streamflow (cfs)	Stage (feet)
1	8/20/1955	340,000	38.85
2	10/11/1903	275,000	35.90
3	4/4/2005	262,000	34.07
4	6/29/2006	254,000	33.62
5	1/8/1841	250,000	
6	3/20/1936	237,000	32.45
7	9/19/2004	216,000	30.95
8	1/21/1996	187,000	28.72
9	5/25/1942	164,000	27.50
10	4/2/1940	154,000	26.47

Table A.2.6: Highest Flood Peaks at Trenton, NJ

Rank	Date	Streamflow (cfs)	Stage (feet)
1	8/20/1955	329,000	28.60
2	10/11/1903	295,000	28.50
3	4/4/2005	242,000	25.33
4	6/29/2006	237,000	25.09
5	3/19/1936	227,000	24.43
6	3/2/1902	214,000	23.60
7	9/19/2004	201,000	23.41
8	1/20/1996	179,000	22.20
9	5/24/1942	161,200	21.12
10	3/28/1913	160,000	21.10

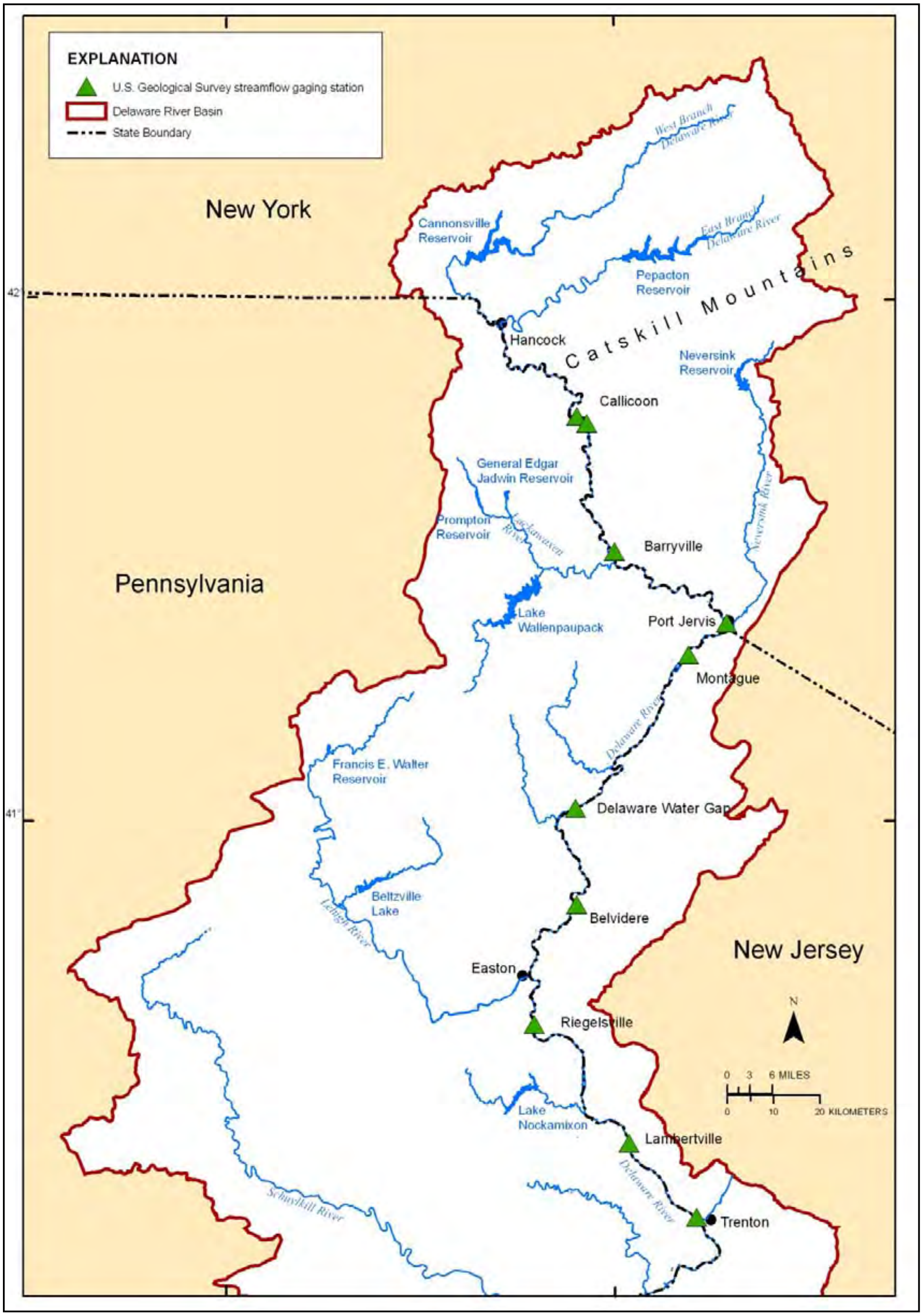

Figure A.2.1: Selected USGS Gage Locations on the Delaware River

October 1903: The flood occurred as a result of a hurricane associated storm which centered east of the upper Delaware River basin. These records remained unbroken until 52 years later in August of 1955 when flood crests several feet higher were recorded in much of the Delaware River.

March 1936: The flood of March 1936 resulted from a combination of precipitation and appreciable snowmelt. The storm had two periods of precipitation, the first on the 11th and 12th and the second on the 17th to 21st. Runoff from the second storm was greater than that from the first storm on the Delaware River.

May 1942: The storm of May 19-23 1942 traveled generally north eastward across eastern Pennsylvania and into New York and produced heavy flows along the Delaware River for several days.

August 1955: The flood of August 1955 was the result of two hurricanes, “Connie” and “Diane”, passing over the basin separated by a few days. Hurricane “Connie”, encountered the extremely dry conditions which prevailed through July and early August. Most of the precipitation from “Connie” was absorbed by the dry soil and resulted in relatively little runoff. “Connie” did, however, saturate the basin and consequently contributed toward increased runoff from “Diane” which quickly followed. The high intensity rainfall during hurricane “Diane” caused rapid flooding of record breaking proportion. Most of the drainage area above Trenton was in major flood. Along the mainstem Delaware River, the flooding exceeded the previous record flood levels for all points above Trenton.

A.2.2. Repaupo Creek Watershed in Greenwich and Logan Townships

The Delaware River in the Repaupo Creek Study Area is tidally influenced and when flooding occurs the low-lying tidal tributaries of the Delaware River Estuary are vulnerable to local and regional flooding. The highest stages recorded at selected tide station situated on the Delaware River in the vicinity of Greenwich and Logan Townships are shown in Tables A.2.7 – A.2.9

Table A.2.7: Highest Stages at NOS Stations 8545240 & 8545530, Philadelphia, PA

Rank	Date	Stage (ft. NAVD88)
1	11/25/1950	7.36
2	4/17/2011	7.34
3	10/25/1980	7.04
4	2/26/1979	6.74
5	4/3/2005	6.65
6	12/11/1992	6.59
7	6/30/1973	6.45
8	9/19/2003	6.35
9	4/19/2007	6.34
10	11/28/1993	6.33

Station 8545530 was removed Feb. 1989 and replaced by Station 8545240
Station 8545530 was one mile upstream of Station 8545240
Highest Stages Ranked Starting from Jul. 1900 (Station 8545530)
Station8545530 is 15.5 river miles upstream from Repaupo Creek Confluence

Table A.2.8: Highest Stages at NOS Station 8540433, Marcus Hook, PA

Rank	Date	Stage (ft. NAVD88)
1	4/17/2011	6.83
2	9/19/2003	6.19
3	8/28/2011	6.11
4	4/2/2005	5.76
5	4/19/2007	5.74
6	9/29/2011	5.67
7	3/29/2010	5.59
8	5/12/2008	5.58
9	3/13/2010	5.53
10	9/9/2011	5.53

Highest Stages Ranked Starting from Sept. 1981
Station is 4.5 river miles downstream from Repaupo Creek Confluence

Table A.2.9: Highest Stages at NOS Station 8551910, Reedy Point DE

Rank	Date	Stage (ft. NAVD88)
1	4/17/2011	6.27
2	10/25/1980	5.91
3	9/19/2003	5.69
4	5/12/2008	5.36
5	12/11/1992	5.36
6	11/28/1993	5.31
7	4/2/2005	5.07
8	8/28/2011	5.06
9	3/29/2010	5.04
10	9/29/2011	5.00

Highest Stages Ranked Starting from Jul. 1956
Station is 22.5 river miles downstream from Repaupo Creek Confluence

March 1962: A massive northeaster storm stalled over the Mid-Atlantic from March 6-8, 1962 for almost 3 days and lingered through five high tides. The storm was caused by an unusual combination of three pressure areas, combined with atmospheric conditions of the Spring Equinox.

September 1999: On September 16, 1999, Hurricane Floyd produced 6 to 10 inches of rain in many parts of Southeast Pennsylvania, New Jersey and Delaware during an 18 hour period. The Delaware River crested ten feet higher than usual and started to spill over the top of levee. The township also evacuated 14 homes during the storm which opened several breaches in the levee from the torrents of rain.

September 2003: Surge from Hurricane Isabel brought the Delaware River level to within 1 foot of the crest of the Repaupo Floodgate Structure. In advance of the approaching hurricane, county and municipal resources were used to place sandbag walls across both the Repaupo Floodgate Structure, which was lower than the levee crest and a low spot in the levee crest where the access road ramps up to the levee. In addition, when the river was at its highest range, a leak developed near the top of the Repaupo Floodgate structure. This was reported to have happened during similar high water events in the past. No evacuation was ordered, and the levee and floodgate structure held.

A.3.0. DELAWARE RIVER HYDROLOGY AND TIDAL STAGES

A.3.1. Trenton and North (Non-Tidal)

To facilitate accurate problem identification and subsequent information, a complete investigation of the hydrology of the main stem Delaware River from Trenton to New York State was performed by the USGS and the Corps using the latest existing data which was supplemented and updated as necessary.

Three HEC-1 rainfall-runoff hydraulic models previously developed by the Corps HEC for the calculation of the Standard Project Flood (SPF) for the Delaware River Basin were used in the discharge frequency analysis (HEC, Special Projects Memo No 82-9, 1982). The three models were divided up by major basins. The Upper Delaware Basin model went from the headwaters to the USGS gage at Montague, NJ. The Lower Delaware Basin model went from the Montague to Trenton, NJ gage, and the third model was for the Lehigh River Basin in Pennsylvania.

The models were modified from their original state to simulate multiple storms, and reservoirs coded in the input files of these three models were removed in order to simulate unregulated or natural flow conditions. The water year when storage started was obtained for each reservoir in the model and a new simulation was done as each individual reservoir started to store water. Summaries of the reservoirs used are shown in Table A.3.1.

Table A.3.1: Reservoirs Simulated in Rainfall-Runoff Models

Reservoir	Model	State	Storage Start Water Year
Hopatcong	Lower Delaware	NJ	1825
Wallenpaupack	Upper Delaware	PA	1926
Rio	Upper Delaware	NY	1926
Toronto	Upper Delaware	NY	1926
Swinging Bridge	Upper Delaware	NY	1930
Neversink	Upper Delaware	NY	1953
Pepacton	Upper Delaware	NY	1954
Wild Creek1	Lehigh	PA	1959
Penn Forest1	Lehigh	PA	1959
Jadwin	Upper Delaware	PA	1960
Prompton	Upper Delaware	PA	1960
FE Walter	Upper Delaware	PA	1961
Cannonsville	Upper Delaware	NY	1963
Beltzville	Lehigh	PA	1971
Nockamixon	Lower Delaware	PA	1974

[1]Penn Forest & Wild Creek are combined in Models as one reservoir

Simulations were conducted with and without the reservoirs in place in order to develop a relationship between regulated and unregulated annual peak discharges. It was assumed for the regulated analysis that the reservoirs were full at the beginning of each storm. A summary of the regulated discharge frequency data from the HEC-1 analysis are shown in Table A.3.2.

Table A.3.2: Regulated Discharge Frequency Values for the Delaware River

USGS Station ID	Station Name	Annual Chance Exceedance/Recurrence Interval Discharges (cfs)								
		50.0% (2-yr)	20.0% (5-yr)	10.0% (10-yr)	4.0% (25-yr)	2.0% (50-yr)	1.0% (100-yr)	0.4% (250-yr)[2]	NJFH AF[3]	0.2% (500-yr)
01438500	Delaware River at Montague, N.J.	65,200	101,000	127,000	164,000	194,000	226,000	270,000	282,000	308,000
01440200	Delaware River near Delaware Water Gap, PA.	71,800	110,000	139,000	178,000	210,000	244,000	291,000	305,000	332,000
01446500	Delaware River at Belvidere, N.J.	76,900	116,000	145,000	184,000	215,000	248,000	294,000	310,000	334,000
01457500	Delaware River at Riegelsville, N.J.	92,300	136,000	167,000	208,000	241,000	274,000	319,000	342,000	358,000
01463500	Delaware River at Trenton, N.J.	94,900	138,000	169,000	211,000	245,000	280,000	329,000	350,000	372,000

Additionally, exceedance probabilities and return periods were calculated by an analysis of historical annual peak discharge data at each gage location.

USGS has a database of the peak annual flows, monthly mean flows available on their Web site: http://waterdata.usgs.gov/nwis/uv/?referred_module=sw.

Annual peak streamflows from 1903 through 2009 were used in the analysis indicates that the maximum peak flow recorded at the USGS gage location occurred in 1955. However, high peak flows were also observed in 2004, 2005 and 2006. The selected peak flows used in the analysis are presented below in Table A.3.3.

Table A.3.3: Selected Peak Flows for Regression Analysis

Location	Peak Flow (cps) 1955	Peak Flow (cps) 2004	Peak Flow (cps) 2005	Peak Flow (cps) 2006
Port Jervis, NY	233,000	151,000	166,000	189,000
Montague, NJ	250,000	168,000	206,000	212,000
Belvidere, NJ	273,000	184,000	226,000	225,000
Riegelsville, NJ	340,000	216,000	262,000	254,000
Trenton, NJ	327,000	201,000	242,000	237,000

Results of the regression analysis were compared to a similar regression analysis summarized in the Delaware River Basin Study Survey Report from 1984. The comparison showed that the updated analysis agreed very closely with the original analysis done for the "1984 Report". A summary of the discharge frequency data from the analysis are shown in Table A.3.4.

Table A.3.4: Unregulated Discharge Frequency Values for the Delaware River

USGS Station ID	Station Name	Annual Chance Exceedance/Recurrence Interval Discharges (cfs)							
		50.0% (2-yr)	20.0% (5-yr)	10.0% (10-yr)	4.0% (25-yr)	2.0% (50-yr)	1.0% (100-yr)	0.4% (250-yr)	0.2% (500-yr)
01438500	Delaware River at Montague, N.J.	78,000	120,000	149,000	190,000	224,000	260,000	315,000	352,000
01440200	Delaware River near Delaware Water Gap, PA.	84,800	130,000	161,000	205,000	242,000	282,000	341,000	381,000
01446500	Delaware River at Belvidere, N.J.	90,800	135,000	167,000	212,000	248,000	286,000	345,000	385,000
01457500	Delaware River at Riegelsville, N.J.	104,000	150,000	183,000	230,000	267,000	305,000	365,000	402,000
01463500	Delaware River at Trenton, N.J.	106,000	152,000	186,000	233,000	271,000	311,000	375,000	417,000

Given the nature of the analysis, the different methodologies used and the assumptions that went into the HEC-1 simulations, the five percent difference between the results the two agencies computed for regulated discharges would be expected. At the conclusion of the work, results were compared between the Corps and USGS and a consensus was developed between both agencies on the final values.

A.3.2. Tidal Area (Gibbstown)

To facilitate accurate problem identification and subsequent information for the tidal portion of the study area, an investigation of the peak tides of the Delaware River in the vicinity of the Gibbstown area was performed by the National Ocean Service (NOS) and the Corps using the latest existing data which was supplemented and updated as necessary. Table A.3.5 summarizes the original observed monthly peak stages used in the analysis at the Philadelphia and Lewes tide stations respectively, relative to the 1983-2001 tidal epoch and NAVD 88 datum.

Table A.3.5: Long-term NOAA Tide Stations near the Gibbstown Area

Station ID	Name	Latitude	Longitude	River Mile	Established
8557380	Lewes, DE	38.7817	75.1200	0	1919
8551910	Reedy Point, DE	39.5583	75.5733	59	1956
8545240	Philadelphia, PA	39.9333	75.1417	99	1900

Note: Gibbstown Area located from RM 82 to RM 88.5

NOAA has published monthly mean sea level trends without the seasonal fluctuations for Philadelphia, Reedy Point, and Lewes. NOAA keeps a database record of the monthly peak tidal stages at gage locations on their Web site: http://tidesandcurrents.noaa.gov/stations.html?type=Water+Levels. At the Philadelphia station, the mean sea level trend is 2.79 mm/yr with a 95% confidence interval of +/- 0.21 mm/yr based on monthly mean sea level data from 1900 to 2006. This trend is equivalent to a change of 0.92 feet in 100 years. At the Reedy Point station, the mean sea level trend is 3.46 mm/yr with a 95% confidence interval of +/- 0.66 mm/yr based on monthly mean sea level data from 1956 to 2006. This trend is equivalent to a change of 1.14 feet in 100 years. At the Lewes station, the mean sea level trend is 3.20 mm/yr with a 95% confidence interval of +/- 0.28 mm/yr based on monthly mean sea level data from 1919 to 2006. This trend is equivalent to a change of 1.05 feet in 100 years.

Observed annual peak stages up to the year of 2006 for these tidal stations were adjusted by NOAA to incorporate the published sea-level trends. Stage frequency curves were then developed by NOAA for the Philadelphia and Lewes tidal stations based on a graphical frequency analysis of the adjusted annual peak stages. See Appendix B to the main report for the stage frequency curves at Philadelphia and Lewes relative to the NAVD 88 datum.

The stage frequency values at Philadelphia and Lewes were interpolated by river mile to obtain the stage frequency near Gibbstown, which runs from river mile 82 to river mile 88.5. In addition to the Stage Frequency Curves developed by NOAA, the Corps has developed new Stage Frequency Curves for FEMA.

A.4.0. WITHOUT PROJECT HYDRAULIC MODEL FOR TRENTON AND NORTH

A.4.1. Development and Calibration of Hydraulic Model

A hydraulic analysis was conducted for the Department of Homeland Security's Federal Emergency Management Agency (FEMA) by Medina Consultants in the wake of the record flooding caused by three major storms in three successive years in September 2004, April 2005, and June 2006. The hydraulic analysis was used to quantify the flood hazard risk along the Delaware River from the Sussex County, NJ/NY political boundary to the Mercer/Burlington County, NJ split. The hydraulic analysis resulted in new technical information to support mitigation and recovery efforts through the production of updated hydrologic and hydraulic models and flood hazard area work maps. FEMA's consultants also used this information to update the Flood Insurance Studies (FISs) and Flood Insurance Rate Maps (FIRMs) for the counties along the Delaware River in the State of New Jersey. This effort represented the most up to date hydraulic analysis of the Delaware River in the study area and superseded the previous hydraulic analysis on the Delaware River done by the District in 1991. Upon completion of the work, the District was requested by FEMA to review the hydraulic analysis done by Medina Consultants. During the review no major technical issues were found in the analysis, and it was approved by the District and subsequently adopted for this Study.

The U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center-River Analysis System (HEC-RAS) 4.0 was used to perform the hydraulic analyses. The HEC-RAS model consisted of 126 stream miles, 30 bridge structures and 2 Inline Structures within the study limits. The analysis was conducted in accordance with the requirements of Appendix C of FEMA's *Guidelines and Specifications for Flood Hazard Mapping Partners,* dated April 2003 (G&S).

<u>Data Sources, Projections, and Datum</u>: Topographic data for hydraulic modeling were obtained from two sources; Light Detection and Ranging (LiDAR) and field surveys during the spring of 2007. LiDAR data collected was scoped to meet FEMA's Guidelines and Specifications (G&S). Field surveys of 28 bridges, 2 inline structures, and 450 cross-sections were collected in accordance with FEMA's G&S for Flood Hazard Mapping Partners. These sources were combined to create an accurate representation of the ground surface within the floodplain area for hydraulic analysis and floodplain delineation purposes. Horizontal projections were referenced to North American Datum (NAD) of 1983 and New Jersey State Plane Coordinate system. Vertical elevations were referenced to North American Vertical Datum (NAVD 88).

<u>Terrain Development</u>: The bare-earth LiDAR data was used to develop a digital terrain model in the form of a Triangulated Irregular Network (TIN) using the Geographic Information System

(GIS) software ArcView. The TIN surface was used as the source of ground elevations for the hydraulic model's preparation and mapping work.

River Alignment: Field survey data and 2008 aerial photographs from the State of New Jersey and Commonwealth of Pennsylvania were used to delineate the main channel. The streamlines were digitized in a GIS by snapping vertices to the lowest survey point at each surveyed cross-section. In between surveyed cross-sections, the stream lines were interpolated from the aerial photography and verified using the digital terrain model from the LiDAR data.

Cross Sections: Over 550 hydraulic cross-sections were cut from the digital terrain model for the HEC-RAS model. Spacing between cross-sections varied from 2,000 feet down to 35 feet. Cross-section alignment was created by drawing sections in the GIS from the left overbank to the right overbank looking downstream. They were adjusted to extend across the entire anticipated floodplain width and were placed at right angles to the anticipated direction of flow in both the Delaware River and the overbank areas. They were also realigned manually as needed to avoid swales, to tie them into the high ground, and to make sure they were not intersecting with each other. Sections were generally drawn through each field surveyed channel section so that the detail would be captured accurately and to verify the LiDAR data with the field collected sections. Generally, the field survey data were used to develop the channel portion of the cross-section geometry and the TIN was the source of the overbank topography. Cross-section geometries located at the immediate upstream and downstream faces of bridges were blended with the field surveys within the TIN. In addition to bridge crossings, the surveyed channel portion and digital overbank LiDAR data were also blended at select locations. Figures A.4.1 - A.4.8 show the cross section layout map from the GIS for Knowlton Township to Trenton.

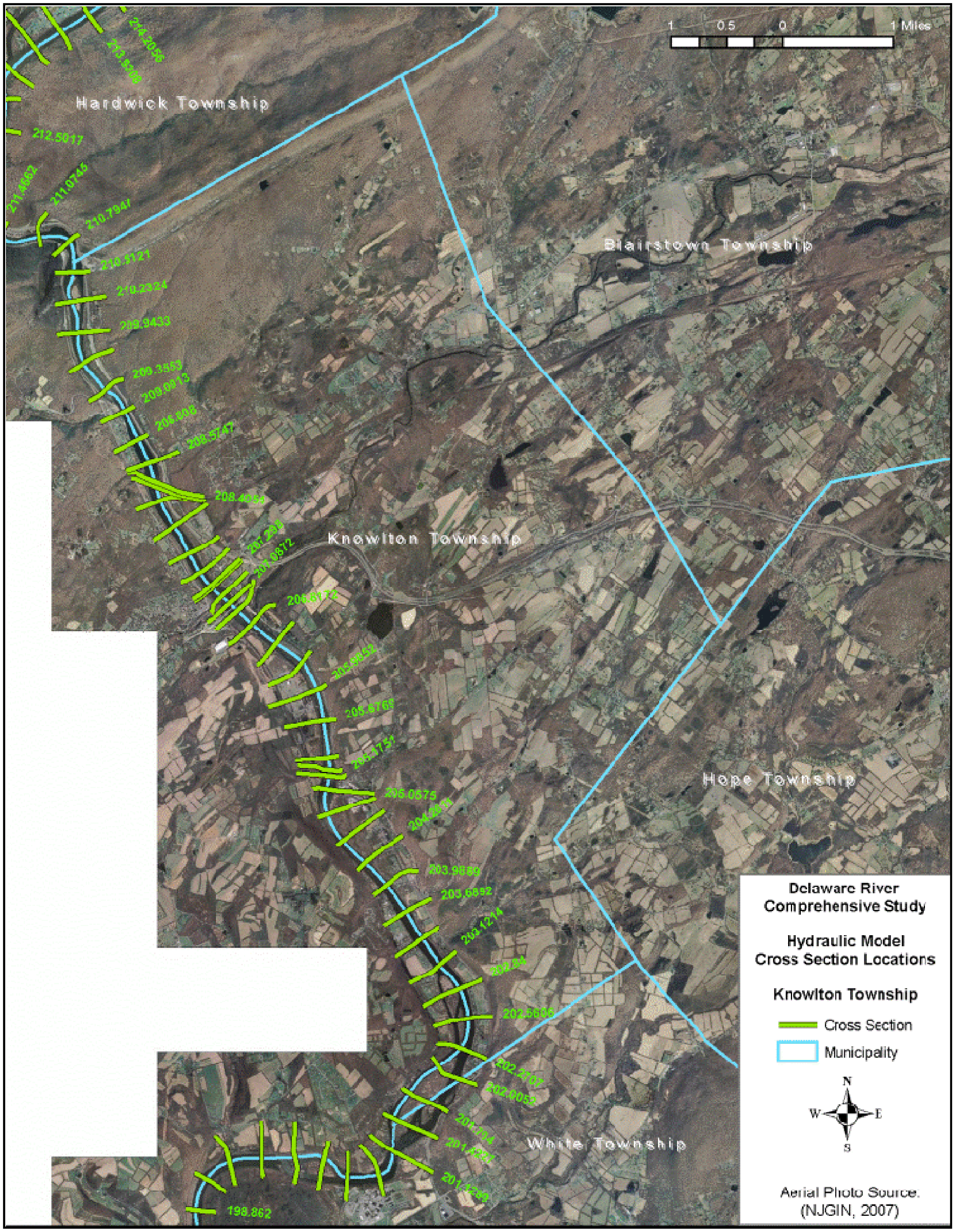

Figure A.4.1: Hydraulic Model Cross Section Locations for Knowlton Township

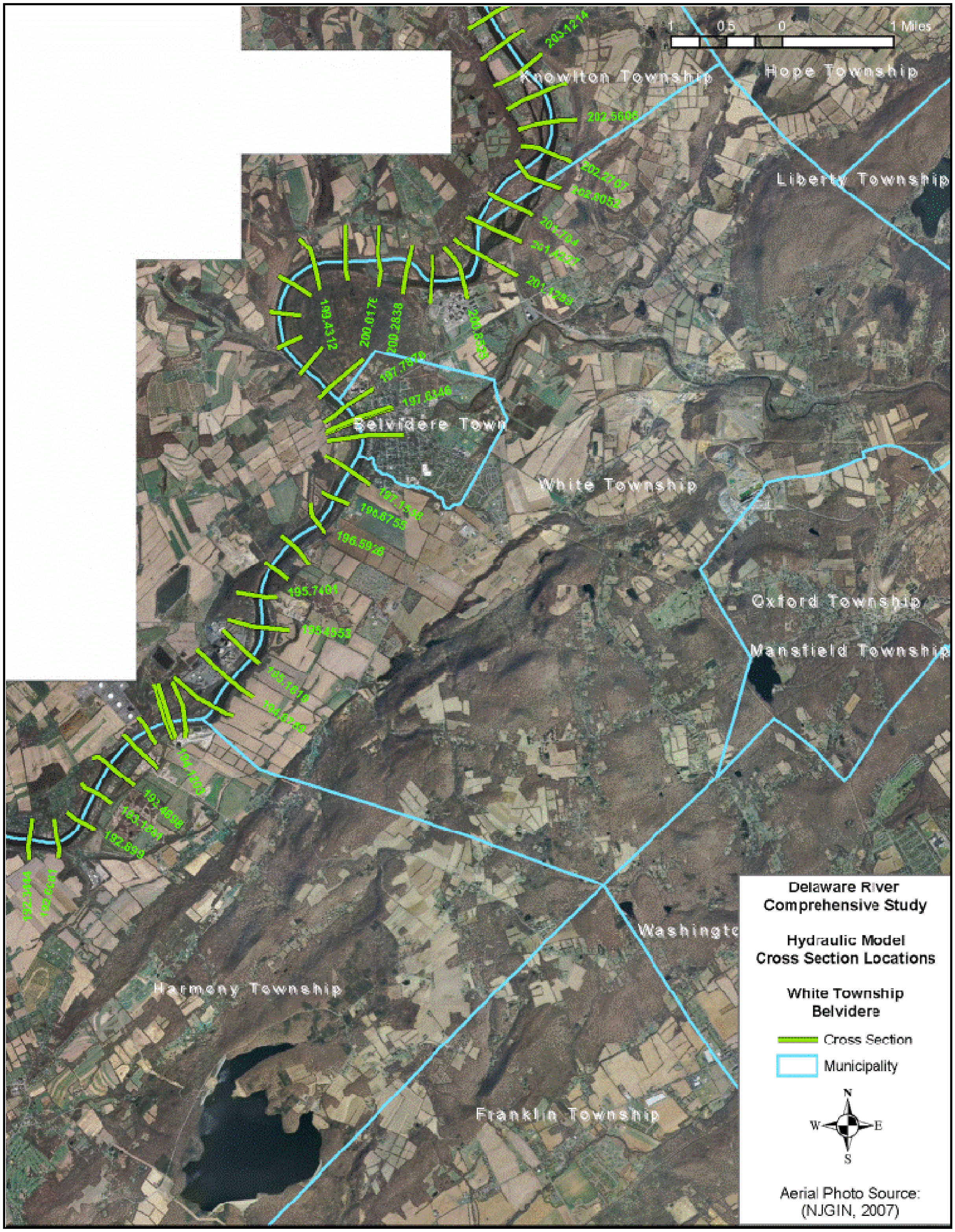

Figure A.4.2: Hydraulic Model Cross Section Locations for White Township & Belvidere

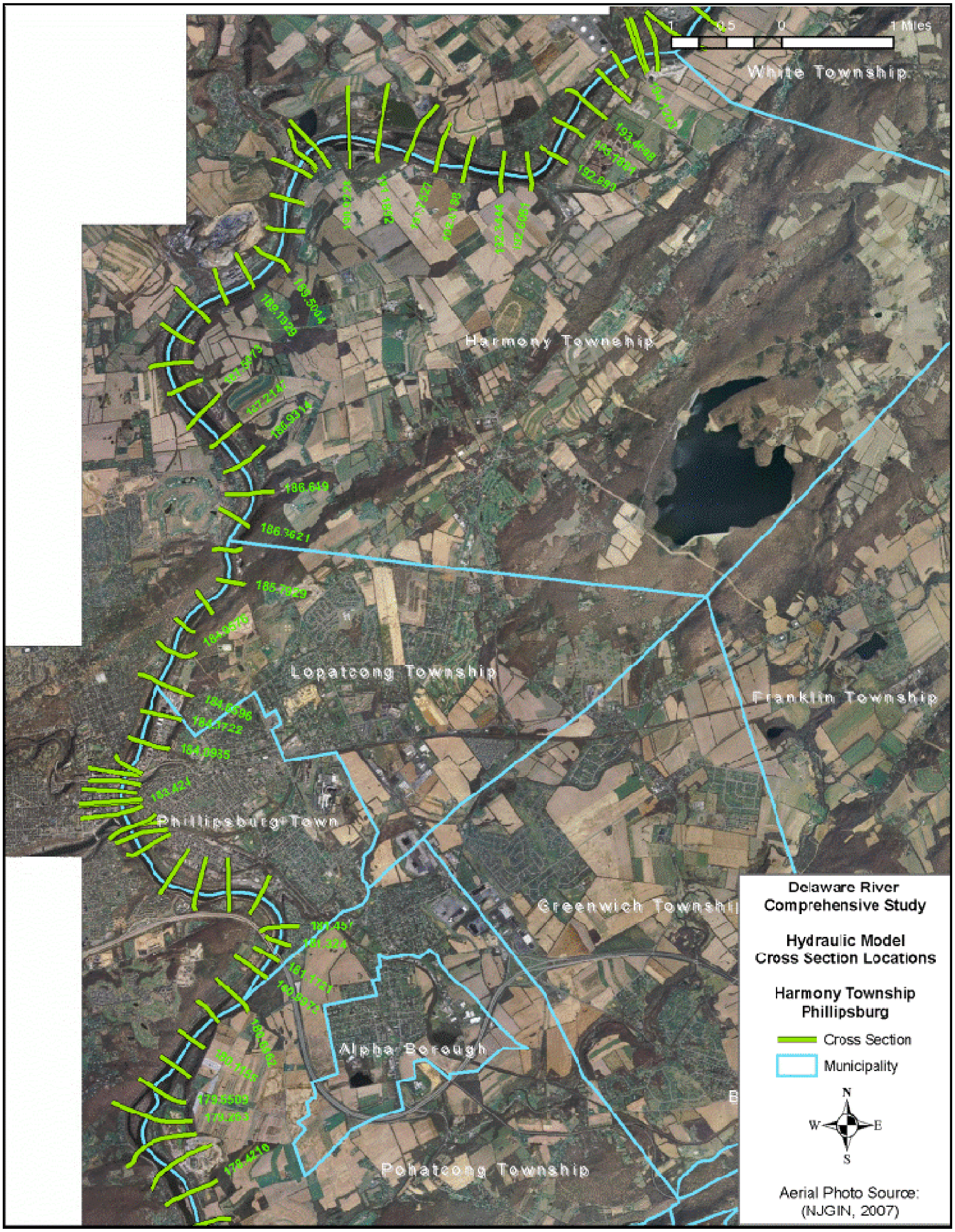

Figure A.4.3: Hydraulic Model Cross Section Locations for Harmony Township & Phillipsburg

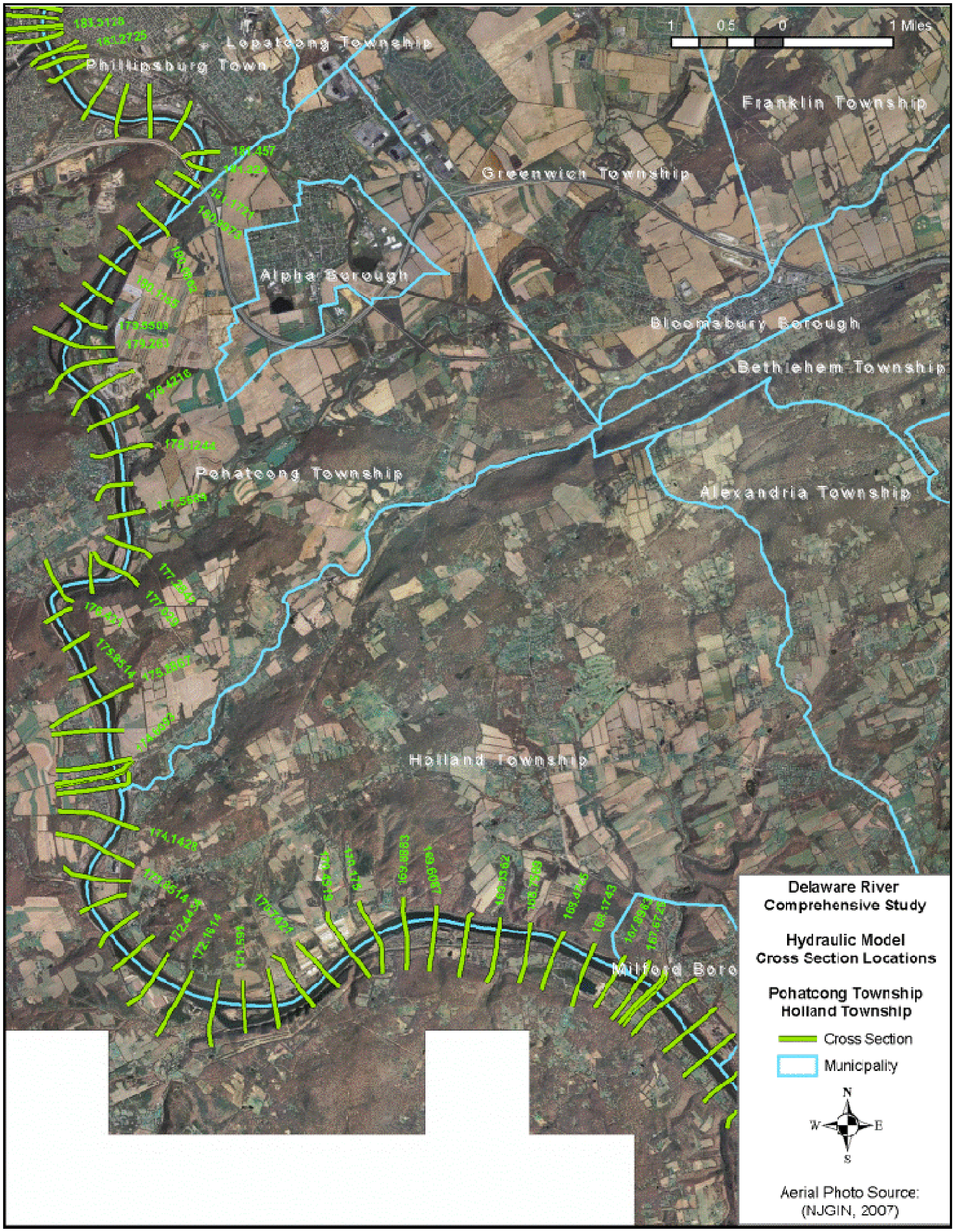

Figure A.4.4: Hydraulic Model Cross Section Locations for Pohatcong & Holland Townships

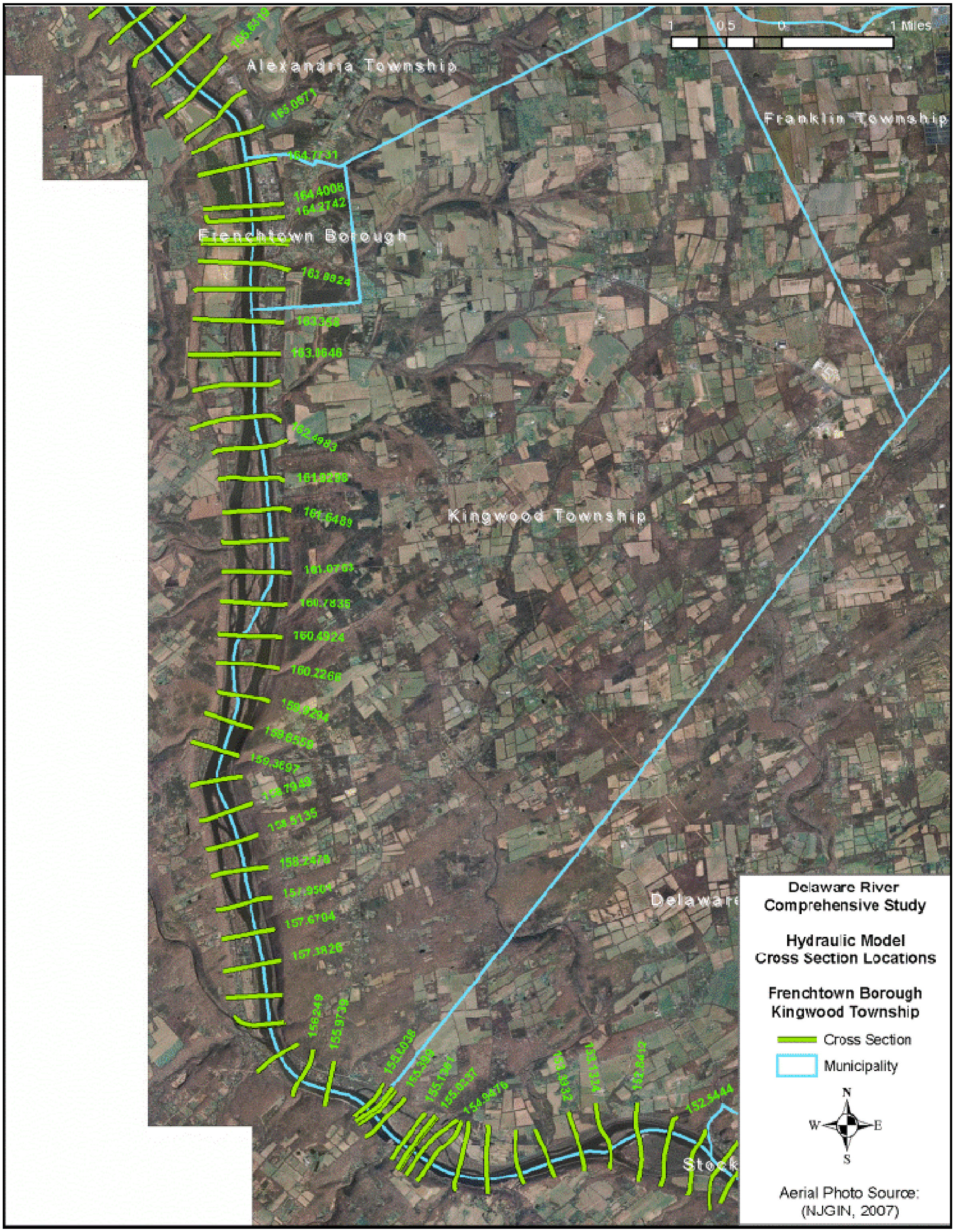

Figure A.4.5: Hydraulic Model Cross Section Locations for Frenchtown & Kingwood Township

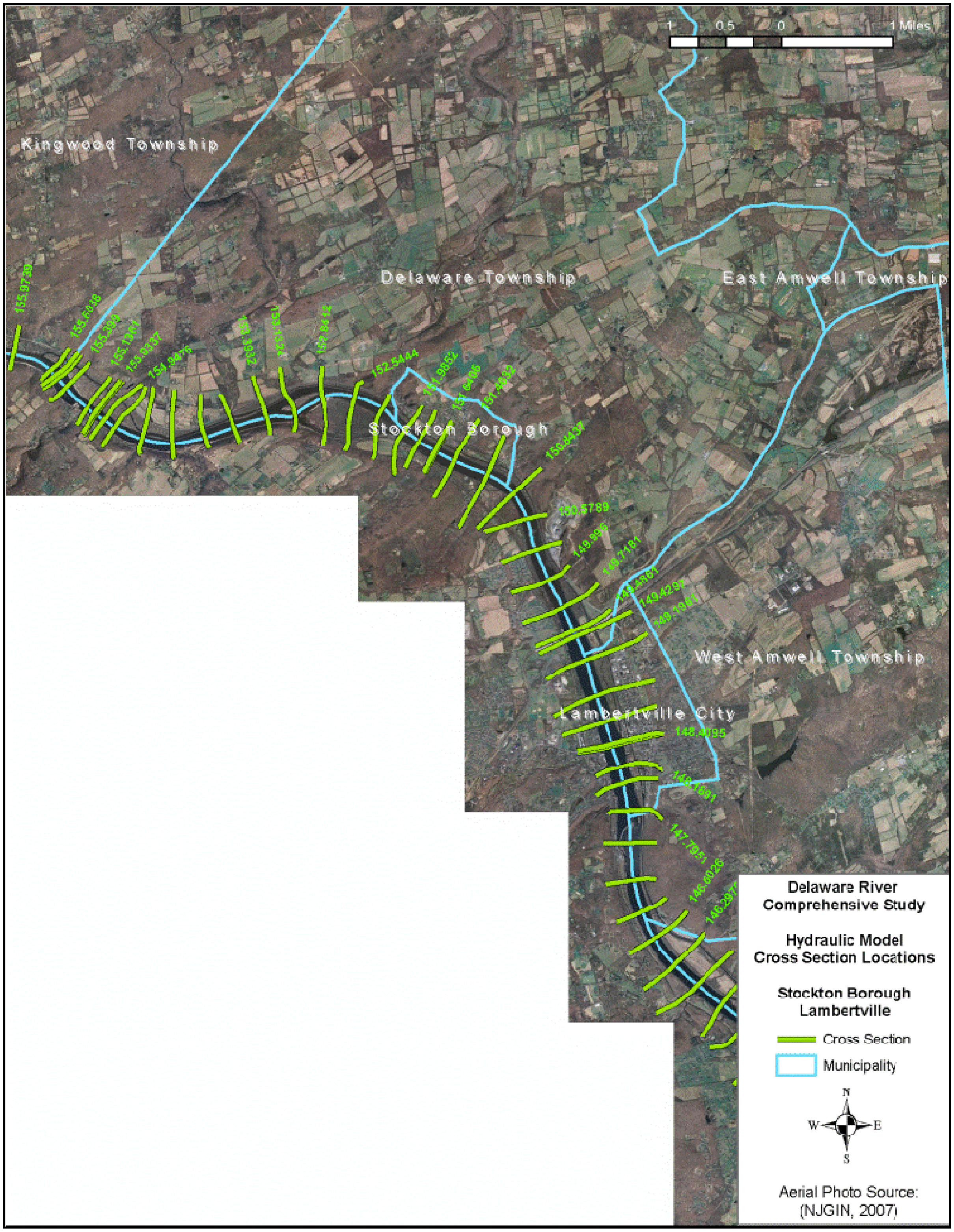

Figure A.4.6: Hydraulic Model Cross Section Locations for Stockton & Lambertville

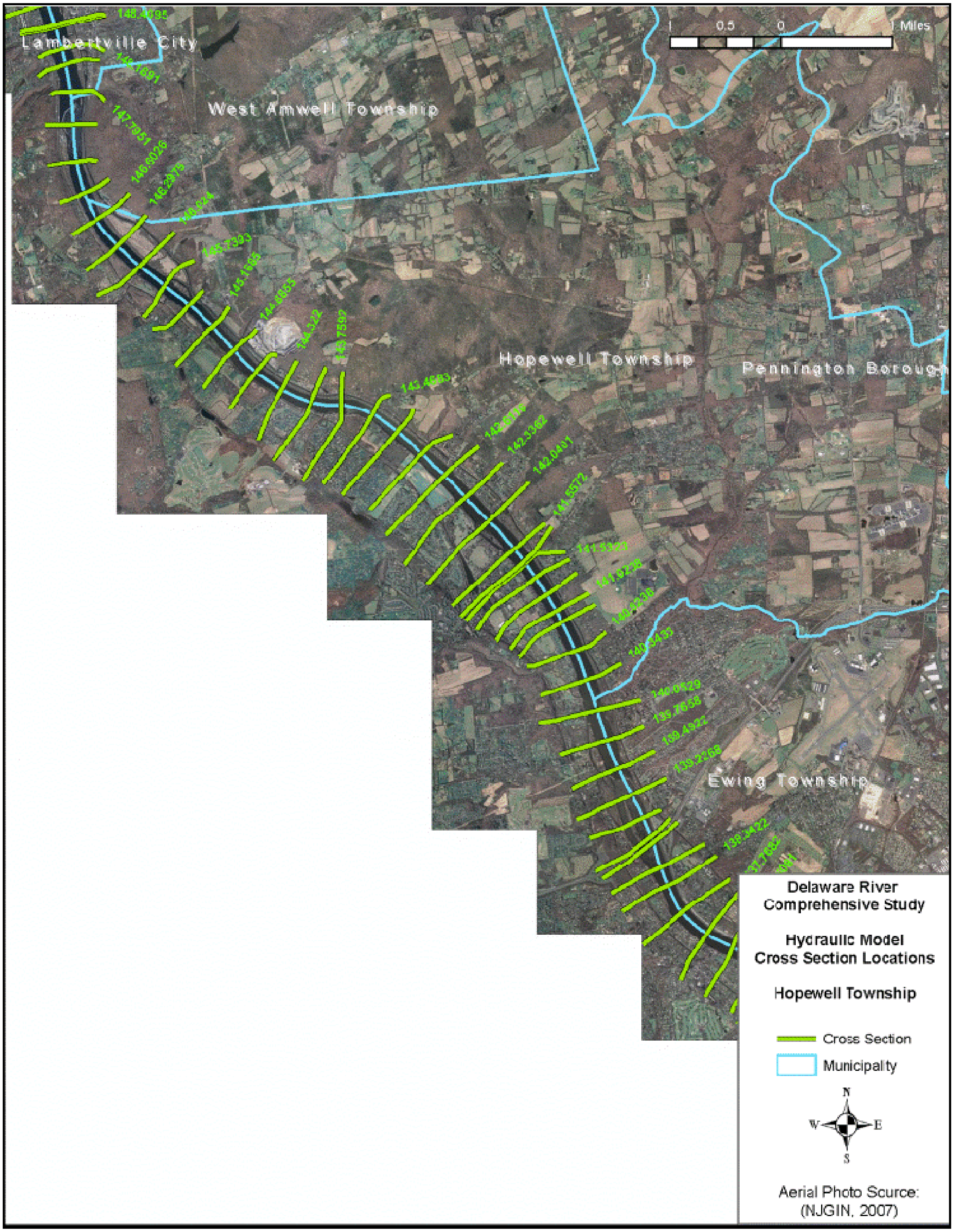

Figure A.4.7: Hydraulic Model Cross Section Locations for Hopewell Township

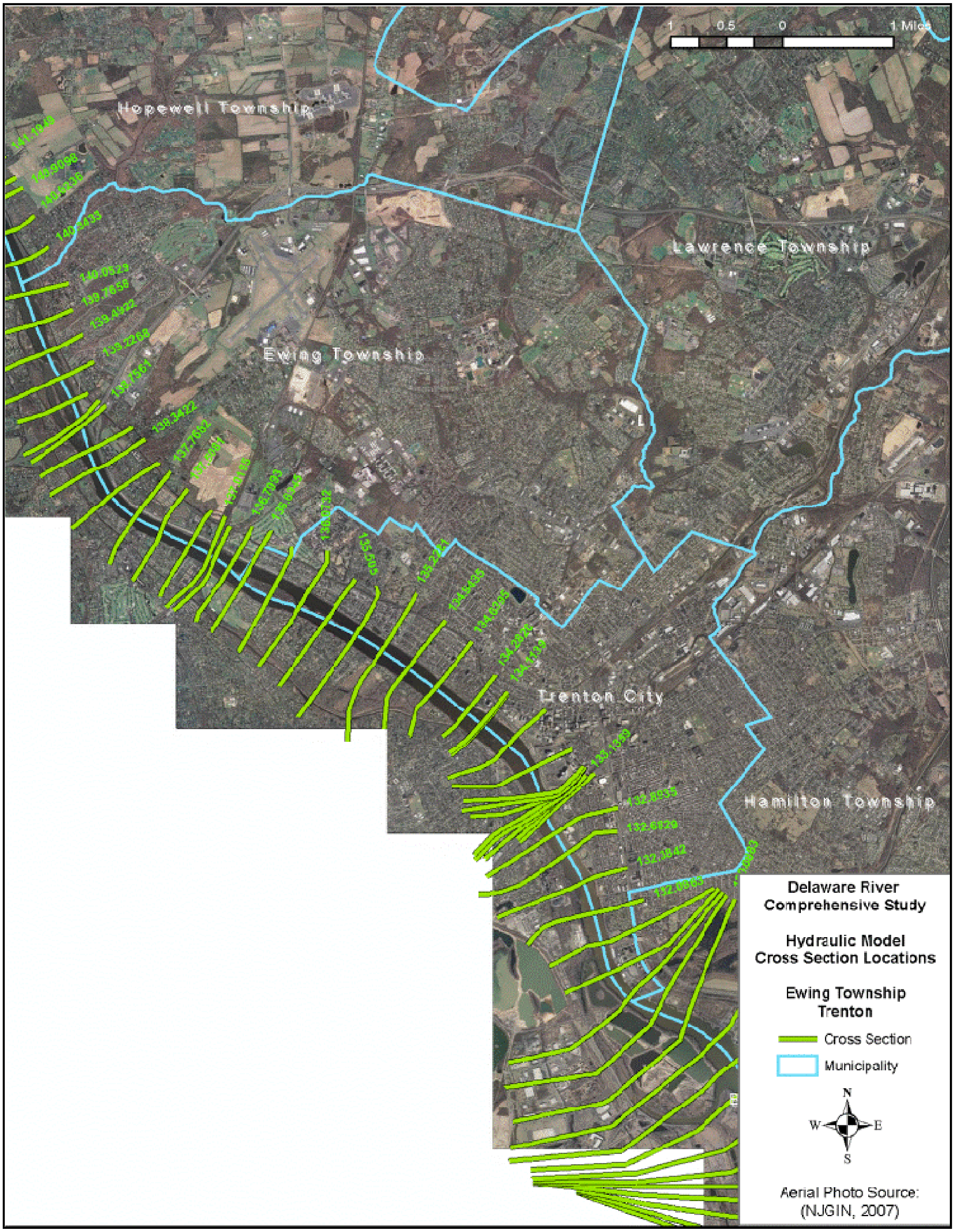

Figure A.4.8: Hydraulic Model Cross Section Locations for Ewing Township & Trenton

Bridge Crossings: There are 30 bridge crossings coded in the HEC-RAS model. All of the bridges that cross the Delaware River were modeled using the HEC-RAS model. The modeled bridges are identified by their stationing and names as shown in Table A.4.1.

Table A.4.1: HEC-RAS Bridge Crossings

River Station	Name	New Jersey Township
246.2295	Milford-Montague Bridge	Montague Township
238.5628	Dingman's Ferry Bridge	Sandystone Township
211.8843	Delaware Water Gap Bridge	Hardwick Township
208.3908	Railroad Bridge	Knowlton Township
207.3096	Portland-Columbia Pedestrian Bridge	Knowlton Township
207.0395	Portland-Columbia Bridge	Knowlton Township
205.2017	Railroad Bridge	Knowlton Township
197.6392	Riverton-Belvidere Bridge	Belvidere
194.1478	Railroad Bridge	Harmony Township
190.4536	Railroad Bridge	Harmony Township
183.7301	Easton-Phillipsburg Bridge	Phillipsburg
183.5176	Northampton Street Bridge	Phillipsburg
183.3429	Railroad Bridge	Phillipsburg
183.2479	Railroad Bridges (2 Structures)	Phillipsburg
181.2249	I-78 Bridge	Phillipsburg
174.5983	Riegelsville Bridge	Riegelsville
167.5245	Milford Bridge	Milford Boro
164.0843	Uhlerstown-Frenchtown Bridge	Frenchtown
155.0387	Lumberville-Raven Rock Ped. Bridge	Delaware Township
151.647	Stockton Bridge	Stockton
149.4579	U.S. Route 202 Toll Bridge	Delaware Township
148.3997	New Hope-Lambertville Bridge	Lambertville
141.5447	Washington's Crossing Bridge	Hopewell Township
138.7832	Scudder Falls I-95 Bridge	Ewing Township
136.9883	Railroad Bridge	Ewing Township
134.1065	Calhoun Street Bridge	Trenton
133.2673	Lower Trenton Bridge	Trenton
133.1563	Trenton-Morrisville Bridge	Trenton
133.0709	Railroad Bridge	Trenton

Manning's Roughness Values: Initial Manning's coefficient values (n-values) for the Delaware River and overbanks at each cross-section were estimated based on the field reconnaissance notes and photographs taken during field work and aerial photographs of the overbanks. The estimated values considered several factors such as channel bed material, density of overbank development, and overbank land use. Land use classification data was developed for the overbank areas by digitizing polygons of different land use types and attributing each polygon with the appropriate Manning's n-value. Changes in land cover across a given cross-section were captured by using appropriate multiple Manning n-values. Final Manning n-values ranged from 0.020 to 0.100 for the main channel and varied from 0.035 to 0.100 for the overbanks.

Contraction and Expansion Coefficients: Typical contraction and expansion coefficients of 0.1 and 0.3, respectively, were used for natural valley cross-sections to account for losses due to the changing width in the river channel. Typical contraction and expansion coefficients of 0.3 and 0.5, respectively, were used at appropriate locations upstream and downstream of bridge crossings to account for additional energy losses.

Ineffective Flow Areas: Ineffective flow areas are used to describe areas where water is not actively being conveyed from one cross-section to another. These areas are used to describe portions of a cross-section in which water will pond and the velocity of the water in downstream direction is close to zero. Typical locations of these areas are at bridge abutments. In the HEC-RAS model, ineffective flow areas were established outside of bridge openings and at appropriate locations in the left or right overbanks. They were verified for reasonableness by examining the aerial photographs.

Starting Water Surface Conditions: The Delaware River is under tidal influence downstream of Trenton, NJ. Starting water-surface elevations were set per tidal conditions established in the Bucks County, PA Flood Insurance Study dated 2 April 2004

Model Calibration: Initially, the HEC-RAS model was run with the historical recorded flows from 1955, 2005, and 2006. Historical recorded flows were obtained at the five USGS streamflow gages within the model boundaries and entered into the model at the appropriate gage locations. The model was calibrated against high watermarks (HWMs) surveyed by the USGS after the extreme flood event of April 2005. The HWMs are documented in Scientific Investigations Report (SIR) Report 2007-5067, entitled "*Flood of April 2-4, 2005, Delaware River Main Stem from Port Jervis, New York to Cinnaminson, New Jersey*". A total of 132 observed high-water marks were collected from the 2005 flood event within the HEC-RAS model boundaries. The calibration process entailed adjusting Manning n-values within a reasonable window and making minor adjustments to cross-section geometry so that the computed water surface elevations generally matched the recorded high water marks. Comparing the computed water surface elevations with the HWMs found that the majority of the computed elevations matched closely with the surveyed HWM from the April 2005 event. Table A.4.2 summarizes the model calibration at selected cross-section locations.

Table A.4.2: Computed Water-Surface Elevations Against 2005 HWMs at Select Cross-Sections

River Station	Community	Computed WSEL (ft)	Observed WSEL (ft)	WSEL Difference (ft)
211.4662	Hardwick	316.88	316.02	0.86
207.4735	Knowlton	294.83	293.74	1.09
206.8172	Knowlton	291.18	290.94	0.24
203.1214	Knowlton	277.99	276.73	1.26
202.84	Knowlton	277.62	276.8	0.82
198.0087	White	256.26	255.55	0.71
197.6346	Belvidere	254.33	252.96	1.37
196.0232	White	236.99	235.04	1.95
195.7401	White	235.72	235.12	0.6
194.1531	Harmony	228.26	227.94	0.32
193.1881	Harmony	226.28	225.02	1.26
192.899	Harmony	225.42	225.07	0.35
190.5962	Harmony	216.07	215.08	0.99
189.1929	Harmony	210.65	208.96	1.69
188.9203	Harmony	209.92	208.56	1.36
185.5061	Lopatcong	197.72	196.83	0.89
184.9576	Lopatcong	196.56	195.36	1.2
183.5281	Phillipsburg	192.77	191.48	1.29
183.0488	Phillipsburg	188.3	187.1	1.2
182.6719	Phillipsburg	187.61	187.1	0.51
181.8066	Phillipsburg	185.55	185.51	0.04
178.1244	Pohatcong	171.18	170.22	0.96
174.6959	Pohatcong	159.17	158.6	0.57
167.3755	Milford	133.31	133.57	-0.26
164.4008	Frenchtown	123.96	122.77	1.19
164.2742	Frenchtown	123.53	122.65	0.88
152.2657	Delaware	81.64	79.36	2.28
151.9852	Stockton	80.18	78.62	1.56
149.996	Delaware	72.26	70.98	1.28
148.8668	Lambertville	69.08	67.98	1.1
148.6181	Lambertville	68.46	68.08	0.38
147.7951	Lambertville	64.78	64.19	0.59
142.0401	Hopewell	50.48	48.29	2.19
141.5323	Hopewell	48.39	48.24	0.15
139.2268	Ewing	42.04	41.45	0.59
138.9964	Ewing	41.57	41.41	0.16
137.7682	Ewing	37.55	36.95	0.6
136.0732	Trenton	31.4	30.55	0.85
135.7964	Trenton	30.58	29.87	0.71
134.6395	Trenton	26.71	25.65	1.06
133.5221	Trenton	21.52	20.21	1.31
132.8935	Trenton	15.52	16.22	-0.7

Steady Flow Data: Once the HEC-RAS model was calibrated satisfactorily against the April 2005 storm event, plans representing the flood flow frequency values were entered into the model. The final peak flows for the model were obtained from the updated discharge frequency analysis done by the USGS.

A.4.2 Uncertainty Analysis of Hydraulic Data

A risk and uncertainty analysis examining hydrologic and hydraulic parameters was performed for the without project conditions using the HEC's programs, HEC-RAS and HEC-FDA. EM 1110-2-1619, "Risk-Based Analysis for Flood Damage Reduction Studies", dated 1 August 1996 and ER 1105-2-101, "Risk Analysis for Flood Damage Reduction Studies", dated 3 January 2006 were used as guidance.

Derivation of Discharge Uncertainty: The uncertainty of flow frequency results can be derived using two approaches. When the flow frequency values are thought to fit a log Pearson Type III distribution, the uncertainty can be derived analytically from the mean, standard deviation, skew, and representative record length. Conversely, the order statistics approach is preferred for deriving uncertainty when the log Pearson distribution is not applicable.

Because results of the USGS log Pearson III analysis and the Corps analysis were similar (less than 5% difference), it was assumed to be appropriate to derive the uncertainty of flow frequency results by the Log Pearson Type III approach. Table A.4.3 summarizes the gage statistics used in the log Pearson Type III approach.

Table A.4.3: Log Pearson III Statistics for Delaware River Gages

	USGS Station Name				
	Trenton	**Riegelsville**	**Belvidere**	**Delaware Water Gap**	**Montague**
USGS Station ID	01463500	01457500	01446500	01440200	01438500
Years of Record	109	109	103	82[a]	101[a]
Mean	4.9843	4.9634	4.8969	4.86094	4.81894
Standard Deviation	0.1874	0.1944	0.2045	0.21664	0.22069
Skew	0.218	0.02	0.087	0.14	0.13

[a] Years of Record extended by doing a two station comparison

Derivation of Stage Uncertainty: The stage-discharge relationship is not known with certainty, due to uncertainty in estimating Manning n-values, in defining the exact cross-section geometry, in measuring distances, surveying in the channel and overbanks, in estimating losses at expansion and contractions, and natural variability of the stream. Guidance from EM-1110-2-1619 and consultation with personnel at the HEC was used to quantify stage discharge uncertainty for several reaches along the Delaware River.

Natural variations include such factors as seasonal vegetation changes, debris constrictions, and unsteady flow effects. Equation 5-5 from EM-1110-2-1619 was used to estimate the standard deviation of stage uncertainty due to these natural effects for several reaches along the Delaware River. The downstream end of each reach was established at USGS streamflow station locations. Table A.4.4 summarizes the natural variability values for these reaches along the Delaware River.

Table A.4.4: Natural Variability Uncertainty

Reach	Bed Identifier (Ibed)	Basin Area (A) (sq. mi)	Stage Range for 100-yr Event (ft)	100-year Discharge (cu ft / sec)	Stream Slope (ft/ft)	Std Dev (ft)
Montague	2 (cobbles)	3,480	65	226,000	0.001	0.14
Water Gap	2 (cobbles)	3,850	72	244,000	0.0004	0.16
Belvidere	3 (gravels)	4,535	78	248,000	0.0008	0.21
Riegelsville	3 (gravels)	6,328	105	274,000	0.0008	0.32
Trenton	3 (gravels)	6,780	133	280,000	0.0005	0.46

Hydraulic Model Inaccuracies: Potential hydraulic modeling inaccuracies include errors in estimating roughness values, errors in cross section topography, and errors in defining effective flow area. EM 1110-2-1619 was used along with conducting a hydraulic model sensitivity analysis to determine a minimum standard deviation of error in stage attributable to the model. Manning's reliability was judged to be good since both stream gages and high-water marks were used to set roughness value. The cross sections for the Delaware River hydraulic model were based on field surveys for the mainstream channel and on digital terrain data (equivalent to a 2-foot contour map) for the overbank portions. With this information, the standard deviation due to model limitations was determined to be 0.3 feet for all reaches along the river.

As an additional measure of modeling uncertainty, a series of tests were conducted to determine the sensitivity of the model to several parameters for the 0.01 probability exceedance discharge. A low and high risk estimate of Manning's n and bridge contraction/expansion coefficients were developed and tested in the hydraulic model. These estimates are as follows:

High Risk Estimate:
Increase all Manning's n values by 25%
Increase bridge contraction and expansion coefficients to 0.6 and 0.8, respectively.

Low Risk Estimate:
Decrease all Manning's n values by 25%
Keep the bridge contraction and expansion coefficients as is (0.3 and 0.5, respectively).

The resultant profile differences for the 0.01 probability exceedance discharge were tabulated for these two estimates and are shown in Table A.4.5. For the risk and uncertainty analysis, it was assumed that these estimates capture 95% of the distribution of the variability. Therefore, the standard deviation about the mean (best) stage estimate is calculated by:

Standard Deviation = 95% band / 4 = (high estimate stage – low estimate stage) / 4

Table A.4.5: Stage Uncertainty Due to Hydraulic Model Inaccuracies

Reach	Model Limitations		Model Sensitivity	
	Manning's n Value Reliability	Std. Dev. (ft)	Avg. Stage Diff (ft)	Std. Dev. (ft)
Montague	Good	0.3	5.75	1.44
Water Gap	Good	0.3	7.25	1.81
Belvidere	Good	0.3	7.28	1.82
Riegelsville	Good	0.3	8.12	2.03
Trenton	Good	0.3	6.97	1.74

Combined Stage Discharge Uncertainty: Combined stage uncertainty was determined for each reach by combining the natural variability and the modeling uncertainty into one value using equation 5-6 from EM 1110-2-1619. See Appendix B to the main report for a summary of the combined stage discharge uncertainty at the 0.01 probability exceedance discharge.
The standard deviation for discharge values greater than the 0.01 exceedance probability discharge was assumed to be the same as the one associated with 0.01 exceedance probability discharge. Table A.4.6 summarizes the combined stage discharge uncertainty at the 0.01 probability exceedance discharge

Table A.4.6: Combined Stage Uncertainty

Reach	Std. Dev. (feet)
Montague	1.48
Water Gap	1.85
Belvidere	1.86
Riegelsville	2.07
Trenton	1.81

For discharge values less than the 0.01 exceedance probability discharge, the standard deviation of error was computed by taking the standard deviation from the 0.01 probability exceedance discharge and multiplying it by the ratio of the given discharge to the 0.01 exceedance probability discharge. See Table 5.4 in the Main Report for a summary of the stage discharge uncertainty for all annual chance of exceedance events in each reach examined.

Uncertainty of Hydraulic Data for Gibbstown Area: The stage frequency developed for the year 2015 for all eight exceedance probability flood events was imported to HEC-FDA as a stage-probability table. HEC-FDA was used to perform an analytical analysis to compute synthetic statistics from the given stage frequencies. The length of record at the Philadelphia and Lewes tide stations was used by HEC-FDA to compute the order statistics. See Economic Appendix C to the main report for further discussion of HEC-FDA.

A.4.3 Results of Hydraulic Model

Water surface elevation profiles were generated in HEC-RAS for the Annual Chance of Exceedance (ACE) values of 50%, 20%, 10%, 4%, 2%, 1%, 0.4%, and 0.2%. The water surface elevation profiles were used to relate flood damage at structures to predicted flood levels. This was done by utilizing the economic forecasting model HEC-FDA (Flood Damage Reduction Analysis). Additional information about HEC-FDA and the "without" project average annual damage amounts per community can be found in the Economics Technical Appendix. Standard HEC-RAS profile plots from Knowlton Township to Trenton are presented in Figures A.4.9 – A.4.12, respectively.

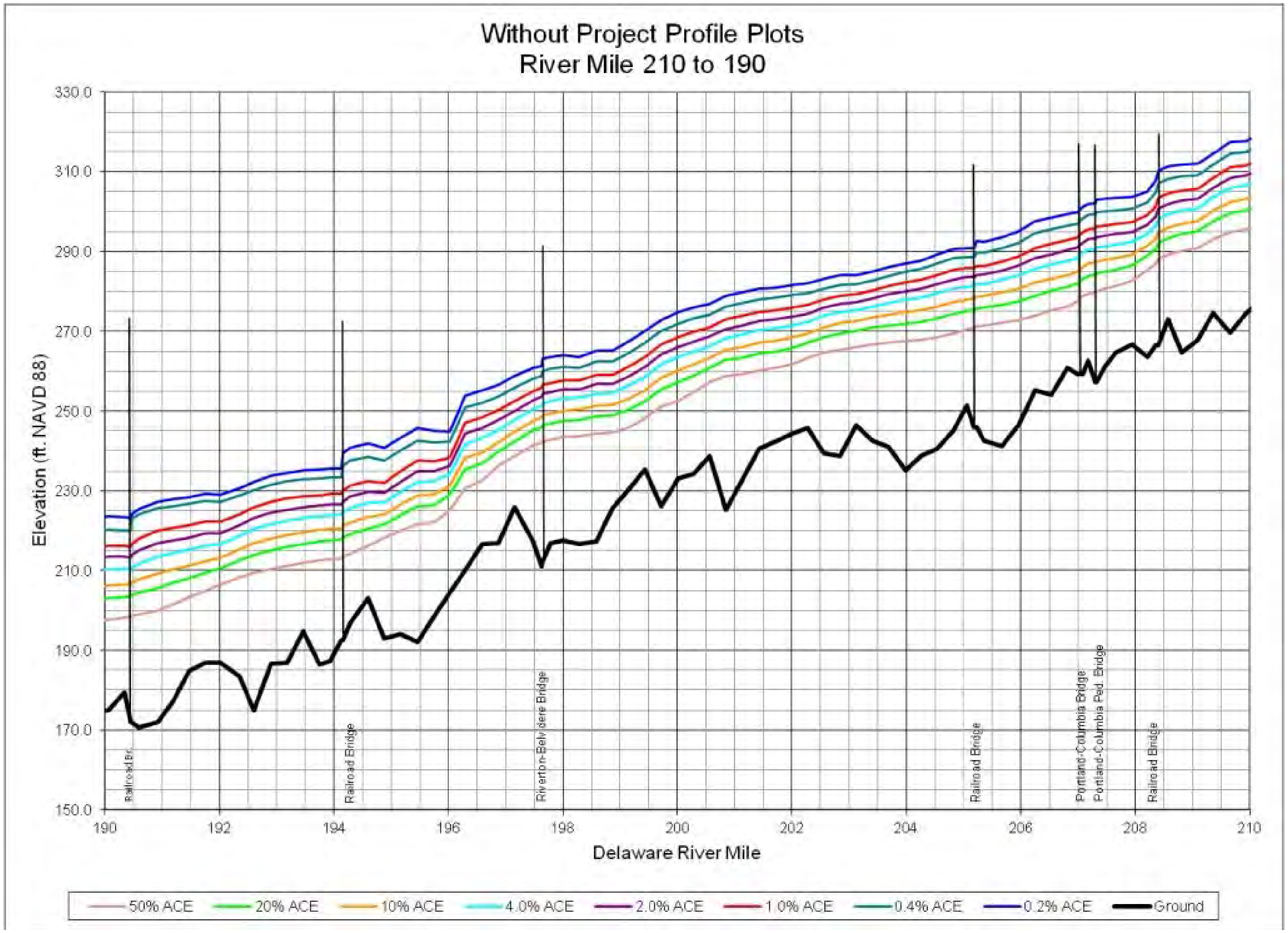

Figure A.4.9: HEC-RAS Water Surface Profiles for "Without" Project Conditions from River Mile 210 to 190.

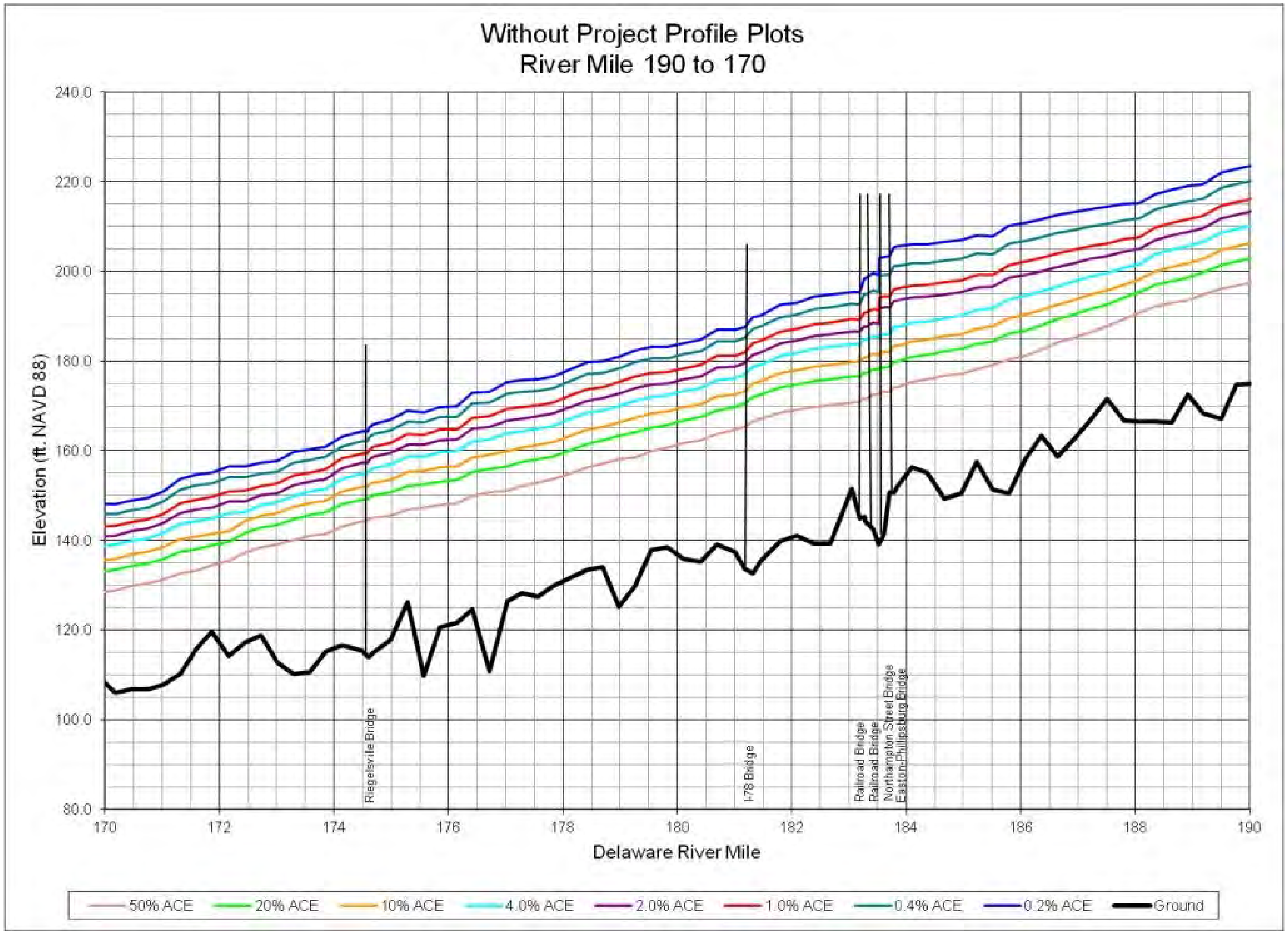

Figure A.4.10: HEC-RAS Water Surface Profiles for "Without" Project Conditions from River Mile 190 to 170.

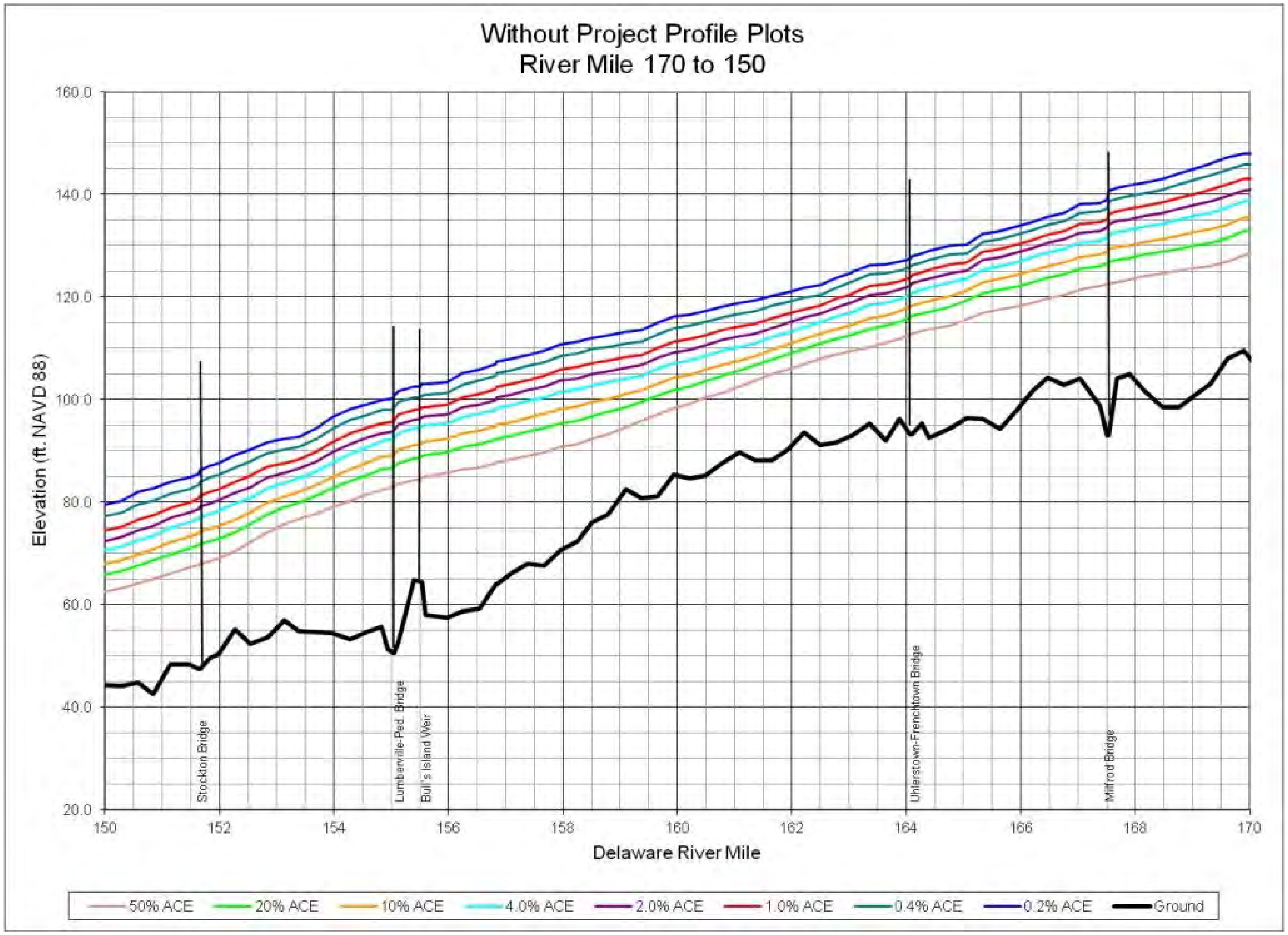

Figure A.4.11: HEC-RAS Water Surface Profiles for Without Project Conditions from River Mile 170 to 150.

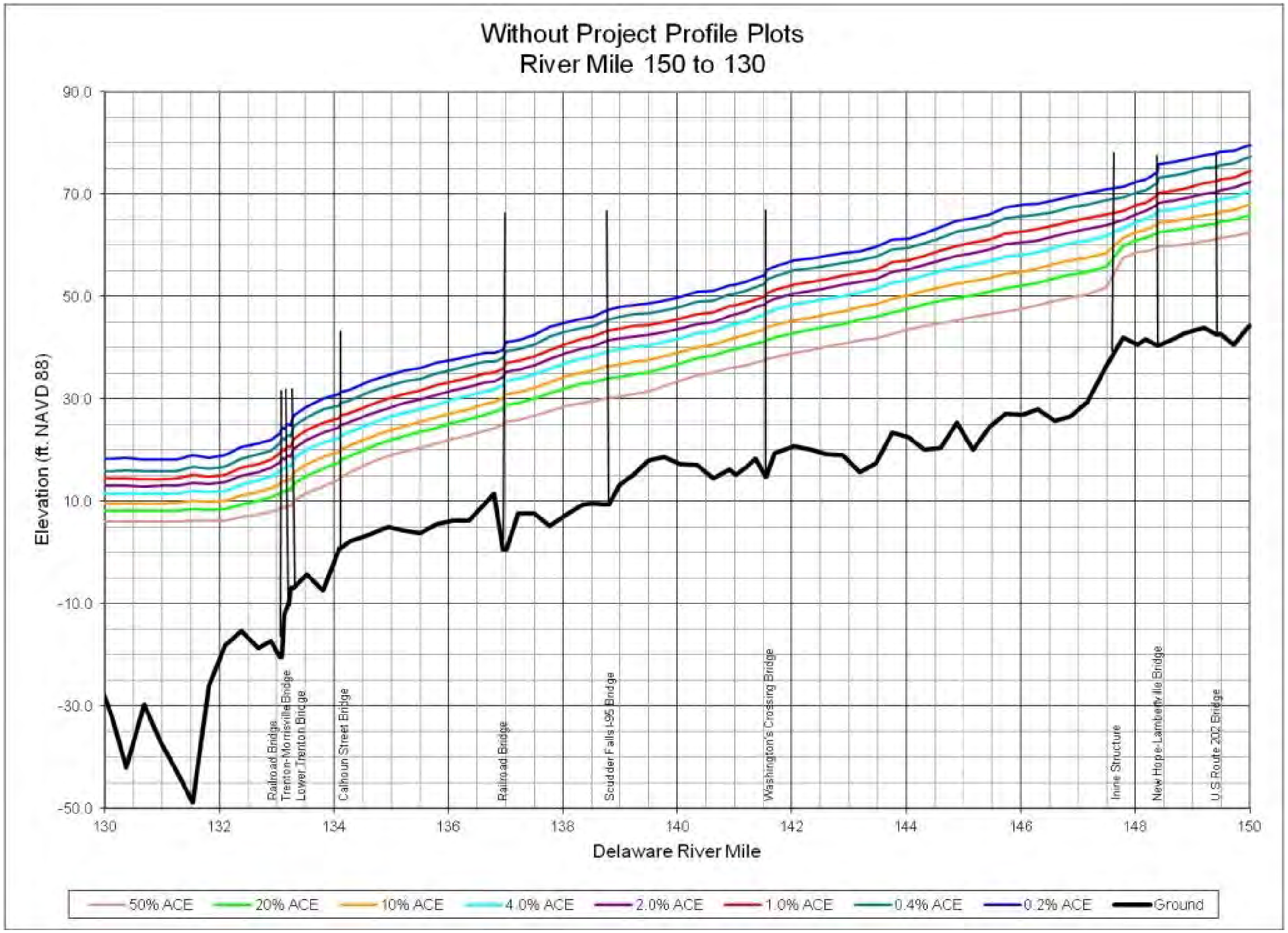

Figure A.4.12: HEC-RAS Water Surface Profiles for "Without" Project Conditions from River Mile 150 to 130.

A.5.0. FUTURE WITHOUT PROJECT CONDITIONS FOR KNOWLTON TOWNSHIP TO TRENTON

A.5.1. Future "Without" Project H&H Assumptions

The future conditions "without" project HEC-RAS hydraulic model represents the probable stage-discharge relationship at the year 2065 based on the best available current data, the incorporation of any known projects planned to be completed within the study reach, and any long term natural river processes that may affect future stages.

It has been shown that hydrologic conditions along the Delaware River in the past have been relatively static. This conclusion was based upon the work done by USACE and USGS during the gage analysis of peak annual streamflows for several gages along the Delaware River many of which have a period of record of over 100 years (Schopp & Firda, 2008). The work USGS did showed no long -term trends in the annual peak streamflow data over the course of the past 100 years.

However; in order to account for future potential basin development and climate variability, an additional analysis was done of the Delaware River Basin upstream of Trenton by USACE and URS. This analysis examined population projections into the future, and any resulting land use (imperviousness) changes that could impact the hydrology of the basin. The analysis also examined potential future climate variability according to the current state of knowledge in the scientific community. Based upon these two factors which are described in the next sections, the annual chance of exceedance streamflows for the future "without" project conditions for year 2065 were increased by 10% from the base-year conditions.

A.5.2. Future Percent Imperviousness Trends

An estimation of the future degree of impervious coverage was developed for the study area. This information was then be utilized to adjust streamflow discharges as necessary used in the "without" project HEC-RAS model.

To identify the extent and degree of flood risk, an estimate of the rainfall and runoff in the study area is required. The runoff volume is affected by the extent of impervious surfaces in the drainage area of the entire river basin. The USGS/HEC Flood Analysis Model for the Delaware River Basin upstream of Trenton was used to examine future changes to percent imperviousness. The model divided the Delaware River watershed above Trenton into 869 small watersheds called Hydrologic Response Units (HRUs). These HRUs are located in New York, Pennsylvania, and New Jersey (see Figure A.5.1). The USGS model called, PRMS (Precipitation Runoff Modeling System) was used for the rainfall/runoff portion of the Flood Analysis Model. HEC developed the HEC-ResSim (Reservoir System Simulation) portion of the modeling suite.

The PRMS model included an estimation of the degree of impervious coverage for each of the HRUs. These products were produced by the Multi-Resolution Land Characteristics (MRLC) Consortium, a group of thirteen federal programs in ten agencies that partner to purchase Landsat imagery and create land cover products for the Nation.

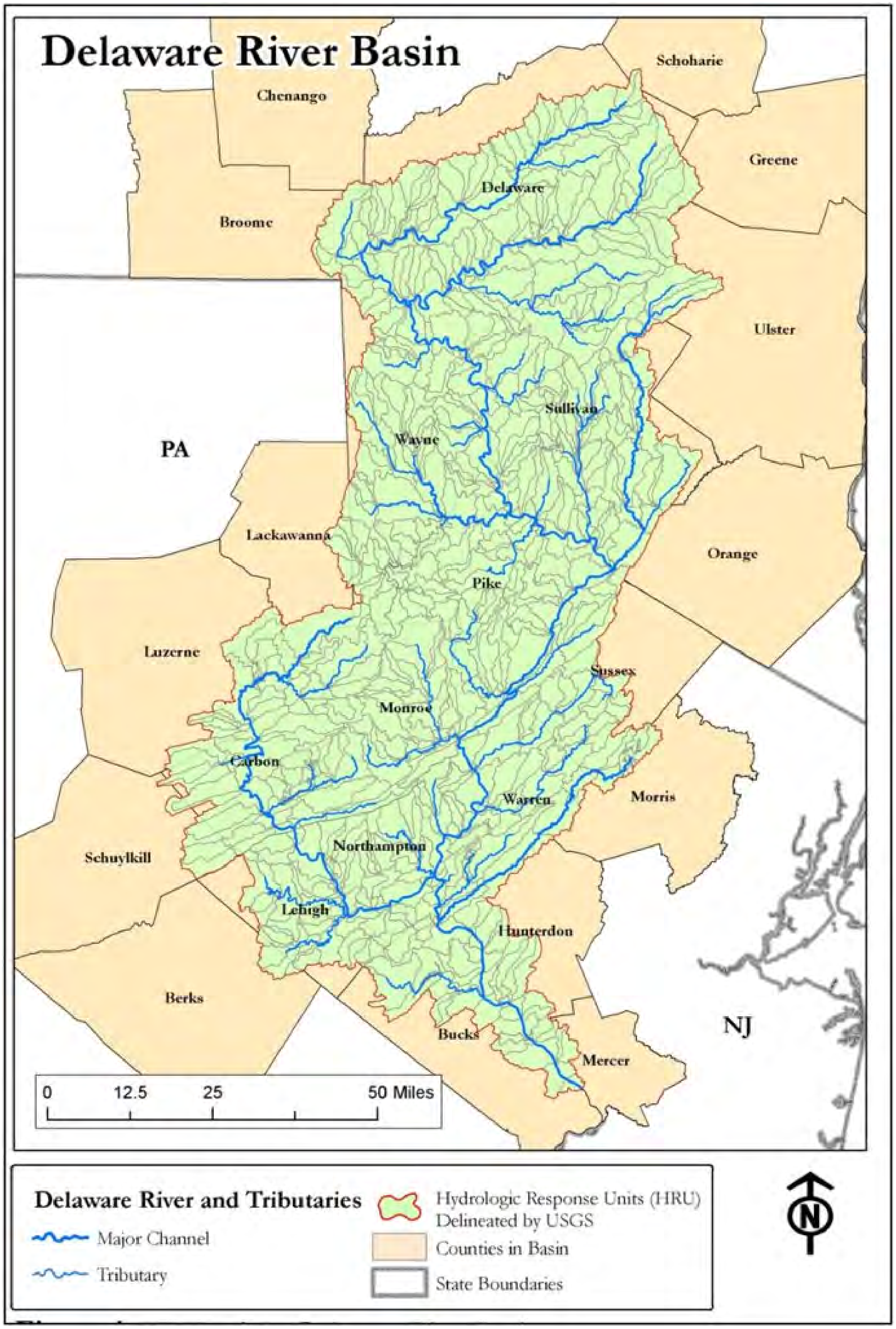

Figure A.5.1: Delaware Basin HRUs Defined by USGS (Goode, 2008)

The assessment of impervious coverage was based on the 2001 National Land Cover Dataset (NLCD), which characterized the land cover, impervious surface and canopy density of the nation (Homer et al, 2007). The degree of impervious coverage was determined through photo interpretation. Low percentage impervious is in light gray with increasing values depicted in darker gray and the highest value in pink and red. White areas have no impervious surface" (MRLC, 2010).

The land cover and impervious surface for the study area are shown in Figures A.5.2 and A.5.3, and the resultant percent imperviousness values assigned to each HRU are shown in Figure A.5.4. It should be noted that a significant number of HRUs (251 of 869) were assigned a 0% level of impervious coverage by the MRLC, and 576 have values between 0.1% and 5%.

As can be observed in the study area, there is a general correlation between the population density of a given area and its degree of impervious coverage. Typically as population increases, the degree of impervious coverage will increase, due to construction of roads, parking areas, buildings, and in cases of greater density, connected stormwater systems. However; the specific percentages of any given area will vary based on underlying soil type, topography, land use transportation patterns, and drainage infrastructure. To forecast impervious coverage levels in 2015 and 2065, projected population levels were evaluated and ultimately selected for use as a proxy variable, based on the general correlation between the two factors.

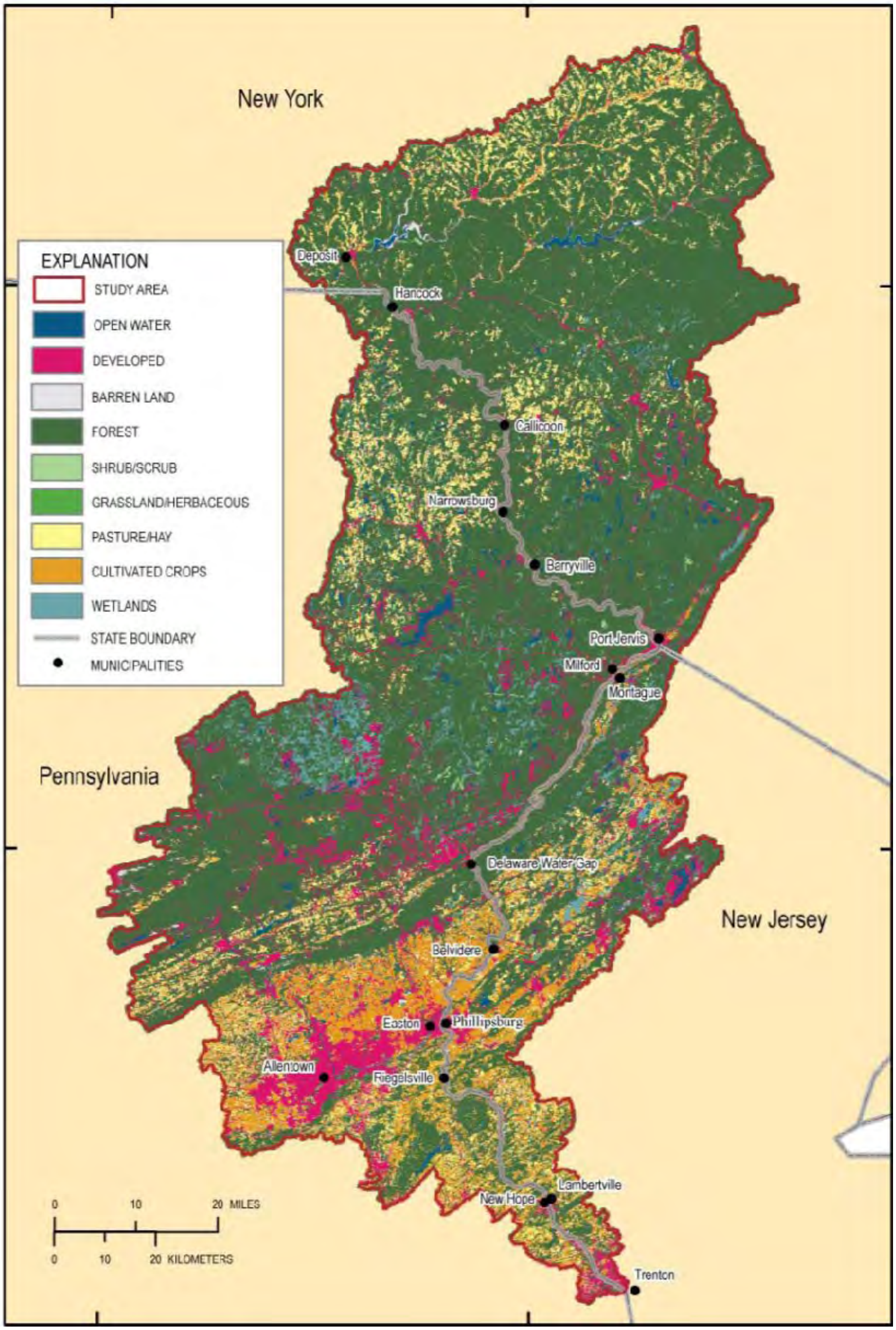

Figure A.5.2: Delaware Basin Land-Cover Data (Goode, 2008 & MRLCC, 2001)

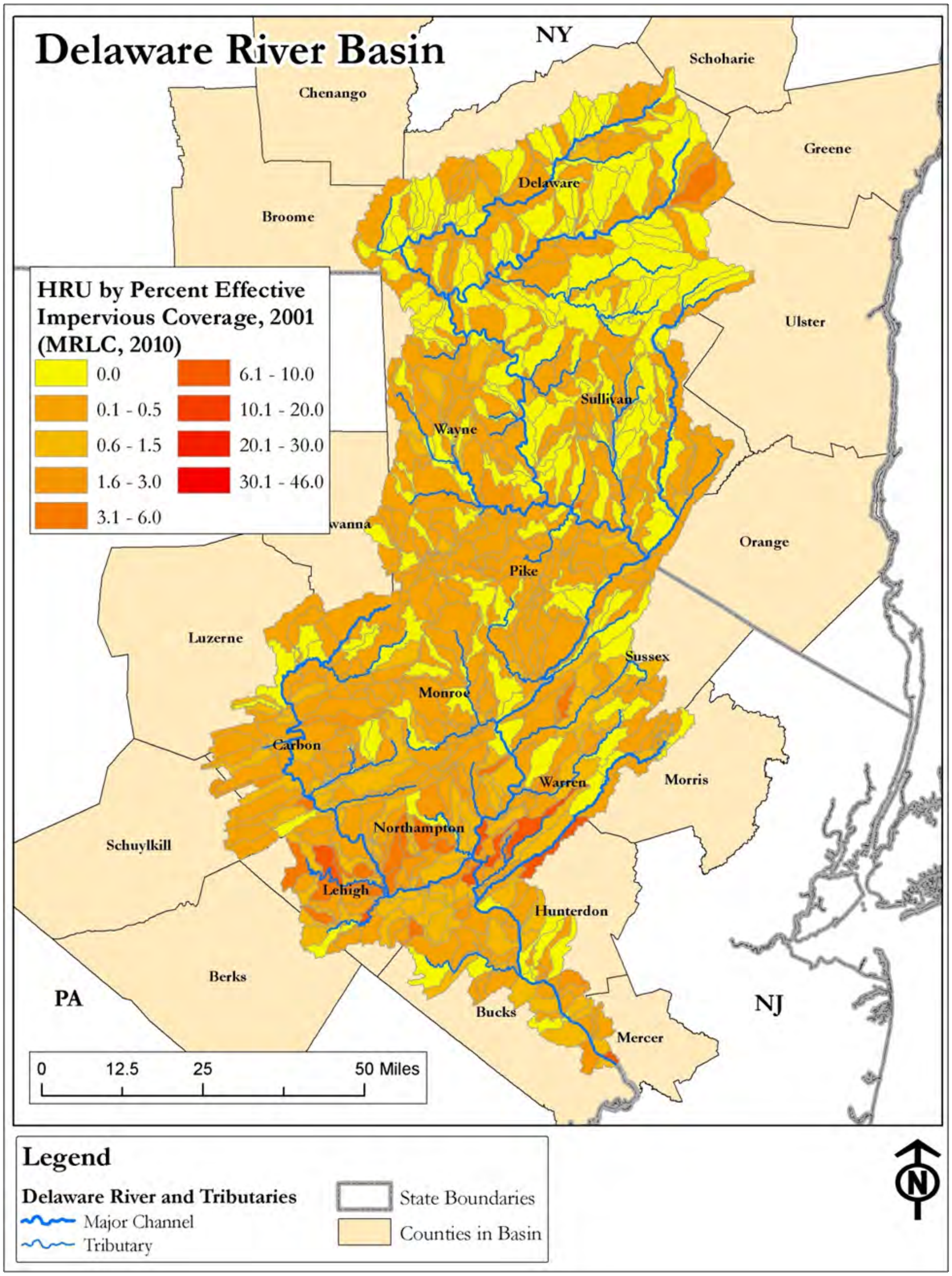

Figure A.5.3: Delaware Basin HRUs by Percent Effective Impervious Coverage (Goode, 2008)

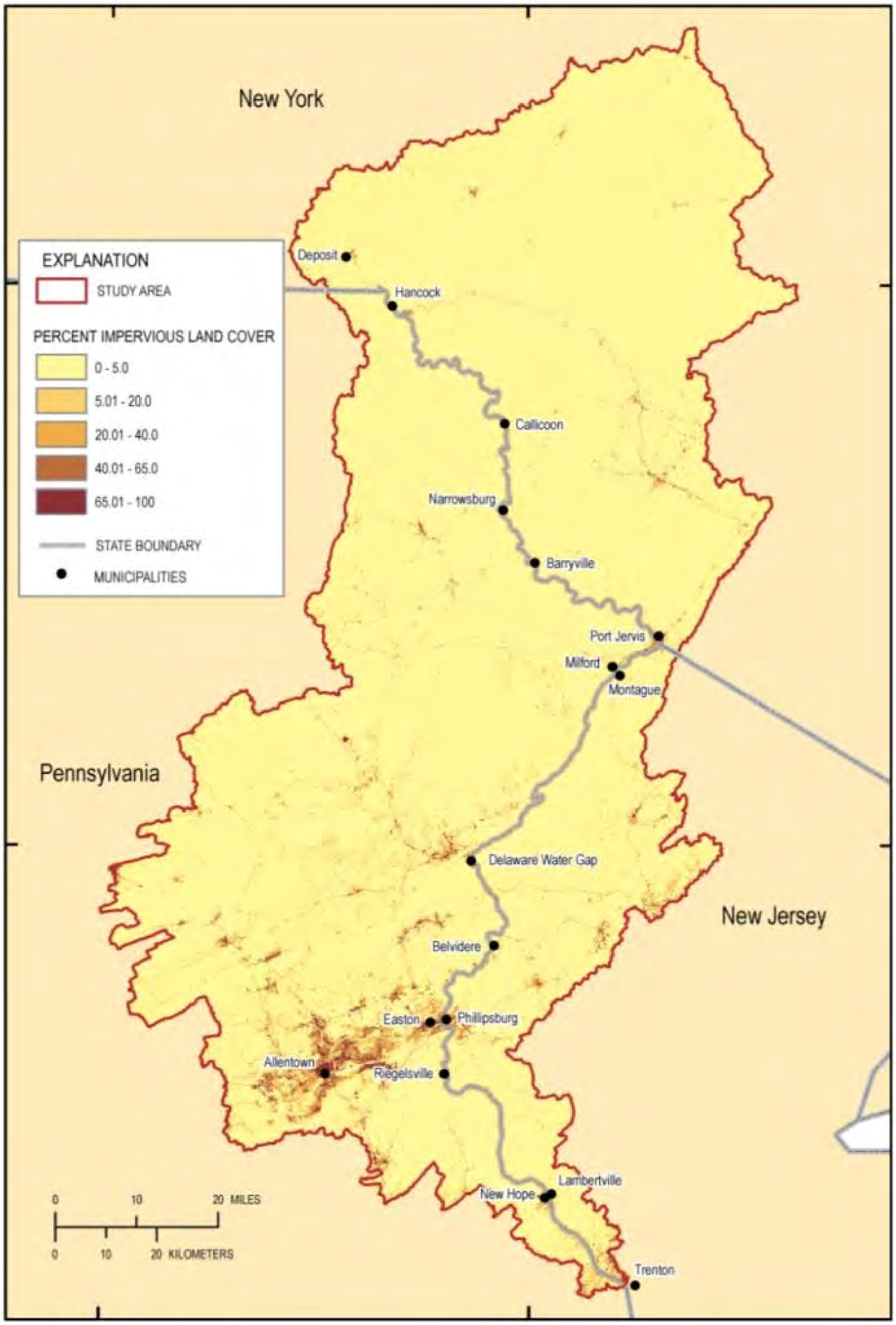

Figure A.5.4: Delaware Basin Impervious-Surface Data (Goode, 2008 & MRLCC, 2001)

At a meeting of USACE staff and consultants in June 2010, it was agreed the approach would require a great deal of subjective decision-making to forecast conditions in 2015 and 2065. Because all of the HRUs are part of the overall Delaware River Basin, meeting attendees agreed that overall population change (and the associated increase in impervious coverage) in the basin is more important for the purposes of modeling future discharges and stages than change within specific HRUs.

Thus, forecasted population change was used to predict increases in impervious land cover. Major non-developable areas, such as State and National parks, open water, and overlay districts with development restrictions were identified and accounted for in the estimation of future land cover change. The percentages of imperviousness in these areas were held constant at its 2001 level.

In areas where population is expected to decrease, it was assumed that the level of impervious coverage will remain constant. Reductions in this level would require the large-scale removal of roads, paved areas, and structures; none of these activities can be guaranteed to occur in an area with declining population, particularly if the level of decline is gradual.

A.5.3. Development of Population Forecasts, 2015 and 2065

The most recent population statistics (including projections for future years) for the study area counties were obtained by URS from the best available public sources for use as the basis for forecasting populations in 2015 and 2065. 2015 is the base year for the interim study, and 2065 represents the end of the 50-year planning horizon. The public projections selected provided forecasts for 2010 through 2030 in Pennsylvania, and from 2010 to 2035 for New York and New Jersey. The 2010 United States Census “Demographic Profile” product containing “selected population and housing characteristics” was not released at the time of the analysis and thus was not available for use in this report.

The following counties in New York, New Jersey, and Pennsylvania are included in whole or in part in the Delaware River Basin above Trenton, NJ (in approximate order heading downstream):

<u>New York</u>
Broome, Chenango, Delaware, Greene, Orange, Schoharie, Sullivan, Ulster

<u>New Jersey</u>
Sussex, Warren, Morris, Hunterdon, Mercer

<u>Pennsylvania</u>
Berks, Bucks, Carbon, Lackawanna, Lehigh, Luzerne, Monroe, Northampton, Pike, Schuylkill, Wayne

URS calculated the arithmetic average of the 2010 and 2020 projections to use for the 2015 population. In this report, data is presented at the county and State level; the spreadsheet in Appendix 1 provides municipal-level detail for New Jersey and Pennsylvania.

A.5.4. Review of Population Forecasting Methods

As described, publicly prepared population forecasts were obtained for use as the basis for future-year forecasts. A number of functions are included in Microsoft Excel® for preparing such forecasts, including TREND, GROWTH, and FORECAST. The TREND function returns values along a linear trend, and fits a straight line using the method of least squares. Given the nature of the historical data and tendency for populations to follow distinct cycles of increase and decrease, projection along a straight line was not expected to provide a realistic future projected population. Thus, the TREND function was discarded.

The GROWTH function calculates exponential growth by using existing data. GROWTH returns the y-values (population) for a series of new x-values (future years) after it is trained with an existing set of known x and y-values. The results for the study area counties at five-year intervals, using the GROWTH function, are available upon request. Summary results of the public 2015 forecast and the calculated 2065 population using GROWTH are provided in the Table A.5.1 below:

Table A.5.1: GROWTH Function Projections for 2065

	2000(1)	Population Forecast for 2015(2)(3)(4)	2065 Population Forecast w/GROWTH Function
New York Counties			
Broome County	200,536	183,115	121,912
Chenango County	51,401	49,395	34,828
Delaware County	48,055	44,644	26,684
Greene County	48,195	50,434	52,834
Orange County	341,367	411,911	650,901
Schoharie County	31,582	31,265	23,227
Sullivan County	73,966	78,329	82,368
Ulster County	177,749	187,097	193,060
NY Total	**972,851**	**1,036,190**	**1,185,814**
New Jersey Counties			
Hunterdon County	121,989	135,435	169,874
Mercer County	350,761	382,692	441,468
Morris County	470,212	497,361	572,594
Sussex County	144,166	170,258	245,906
Warren County	102,437	123,529	154,990
NJ Total	**1,189,565**	**1,309,275**	**1,584,833**
Pennsylvania Counties			
Berks County	373,646	418,465	489,567
Bucks County	597,635	680,975	886,968
Carbon County	58,802	62,850	68,197
Lackawanna County	213,295	202,505	188,786
Lehigh County	312,090	338,419	379,740
Luzerne County	319,250	301,480	279,201
Monroe County	138,687	203,041	518,221
Northampton County	267,066	303,484	401,135
Pike County	46,302	81,134	303,855
Schuylkill County	150,336	142,937	134,659
Wayne County	47,722	57,258	73,384
PA Total	**2,524,831**	**2,792,545**	**3,723,715**
Total--All Counties	**4,687,247**	**5,138,009**	**6,494,361**

Sources:
(1) Census 2000, U.S. Census of Population, 2000
(2) Cornell Program on Applied Demographics, 2009
(3) North Jersey Transportation Planning Agency (NJTPA), 2009, and Delaware Valley Regional Planning Commission (Mercer County), 2007
(4) Pennsylvania Department of Environmental Protection, State Water Plan, 2006

The next method evaluated for development of population projections was the Microsoft Excel® FORECAST function, a least squares trending/regression function. As described in Microsoft Excel®, the FORECAST function "calculates, or predicts, a future value by using existing values. The predicted value is a y-value for a given x-value. The known values are existing x-values and y-values, and the new value is predicted by using linear regression." This function was identified after a review of methods employed by different State and county agencies to do their projections. The PADEP Bureau of Watershed Management prepared its population projection for 2010, 2020 and 2030 using the FORECAST function (PADEP, 2006b).

The results for the study area counties at five-year intervals using the FORECAST function can be provided upon request. Summary results of the public 2015 forecast and the calculated 2065 population using FORECAST are provided in the Table A.5.2 below, and a comparison of results between the GROWTH and FORECAST functions are provided in Table A.5.3 and Table A.5.4.

Table A.5.2: FORECAST Function Projections for 2065

	2000(1)	Population Forecast for 2015(2)(3)(4)	2065 Population Forecast w/FORECAST Function
New York Counties			
Broome County	200,536	183,115	113,821
Chenango County	51,401	49,395	33,269
Delaware County	48,055	44,644	23,813
Greene County	48,195	50,434	52,802
Orange County	341,367	411,911	616,168
Schoharie County	31,582	31,265	22,510
Sullivan County	73,966	78,329	82,339
Ulster County	177,749	187,097	193,042
NY Total	**972,851**	**1,036,190**	**1,137,766**
New Jersey Counties			
Hunterdon County	121,989	135,435	165,347
Mercer County	350,761	382,692	433,080
Morris County	470,212	497,361	584,471
Sussex County	144,166	170,258	232,594
Warren County	102,437	123,529	147,998
NJ Total	**1,189,565**	**1,309,275**	**1,563,490**
Pennsylvania Counties			
Berks County	373,646	418,465	481,732
Bucks County	597,635	680,975	862,575
Carbon County	58,802	62,850	67,805
Lackawanna County	213,295	202,505	188,912
Lehigh County	312,090	338,419	376,159
Luzerne County	319,250	301,480	279,334
Monroe County	138,687	203,041	416,046
Northampton County	267,066	303,484	389,360
Pike County	46,302	81,134	207,599
Schuylkill County	150,336	142,937	134,738
Wayne County	47,722	57,258	70,983
PA Total	**2,524,831**	**2,792,545**	**3,475,244**
Total--All Counties	**4,687,247**	**5,138,009**	**6,176,500**

Sources:

(1) Census 2000, U.S. Census of Population, 2000

(2) Cornell Program on Applied Demographics, 2009

(3) North Jersey Transportation Planning Agency (NJTPA), 2009, and Delaware Valley Regional Planning Commission (Mercer County), 2007

(4) Pennsylvania Department of Environmental Protection, State Water Plan, 2006

Table A.5.3: Comparison of GROWTH and FORECAST Function Results, 2015-2065 (NJ/NY)

	2000[1]	Population Forecast for 2015[2][3][4]	2065 Population Forecast w/GROWTH Function	2065 Population Forecast w/FORECAST Function	Numeric Difference in 2065 btw. GROWTH and FORECAST levels[5]	% Difference in 2065 btw. GROWTH and FORECAST levels[5]
New York						
Broome Co.	200,536	183,115	121,912	113,821	8,090	6.64%
Chenango Co.	51,401	49,395	34,828	33,269	1,559	4.48%
Delaware Co.	48,055	44,644	26,684	23,813	2,871	10.76%
Greene Co.	48,195	50,434	52,834	52,802	32	0.06%
Orange Co.	341,367	411,911	650,901	616,168	34,733	5.34%
Schoharie Co.	31,582	31,265	23,227	22,510	716	3.08%
Sullivan Co.	73,966	78,329	82,368	82,339	28	0.03%
Ulster Co.	177,749	187,097	193,060	193,042	18	0.01%
NY Total	972,851	1,036,190	1,185,814	1,137,766	48,048	4.05%
New Jersey						
Hunterdon Co.	121,989	135,435	169,874	165,347	4,527	2.66%
Mercer Co.	350,761	382,692	441,468	433,080	8,388	1.90%
Morris Co.	470,212	497,361	572,594	584,471	-11,876	-2.07%
Sussex Co.	144,166	170,258	245,906	232,594	13,312	5.41%
Warren Co.	102,437	123,529	154,990	147,998	6,992	4.51%
NJ Total	1,189,565	1,309,275	1,584,833	1,563,490	21,343	1.35%

Table A.5.4: Comparison of GROWTH and FORECAST Function Results, 2015-2065 (PA)

	2000[1]	Population Forecast for 2015[2][3][4]	2065 Population Forecast w/GROWTH Function	2065 Population Forecast w/FORECAST Function	Numeric Difference in 2065 btw. GROWTH and FORECAST levels[5]	% Difference in 2065 btw. GROWTH and FORECAST levels[5]
Pennsylvania						
Berks Co.	373,646	418,465	489,567	481,732	7,835	1.60%
Bucks Co.	597,635	680,975	886,968	862,575	24,393	2.75%
Carbon Co.	58,802	62,850	68,197	67,805	392	0.57%
Lackawanna Co.	213,295	202,505	188,786	188,912	-126	-0.07%
Lehigh Co.	312,090	338,419	379,740	376,159	3,582	0.94%
Luzerne Co.	319,250	301,480	279,201	279,334	-133	-0.05%
Monroe Co.	138,687	203,041	518,221	416,046	102,176	19.72%
Northampton Co.	267,066	303,484	401,135	389,360	11,776	2.94%
Pike Co.	46,302	81,134	303,855	207,599	96,255	31.68%
Schuylkill Co.	150,336	142,937	134,659	134,738	-79	-0.06%
Wayne Co.	47,722	57,258	73,384	70,983	2,401	3.27%
PA Total	2,524,831	2,792,545	3,723,715	3,475,244	248,471	6.67%
Total--All Counties	**4,687,247**	**5,138,009**	**6,494,361**	**6,176,500**	**317,862**	**4.89%**

Sources:
[1] Census 2000, U.S. Census of Population, 2000
[2] Cornell Program on Applied Demographics, 2009
[3] North Jersey Transportation Planning Agency (NJTPA), 2009, and Delaware Valley Regional Planning Commission (Mercer County), 2007
[4] Pennsylvania Department of Environmental Protection, State Water Plan, 2006
Note:
[5]The numeric difference and % difference is calculated by comparing GROWTH function results to FORECAST function results.

As shown in Table A.5.3 and Table A.5.4, the total population forecasts derived using GROWTH and FORECAST functions for the counties in the Delaware River Basin are within 5% of each other, with the GROWTH function forecasting an additional 317,862 people for a total population of 6,494,361 in the year 2065.

Marked differences between the GROWTH and FORECAST population levels are seen in the forecasts for three counties, with GROWTH generating the following variances compared to FUNCTION:

- Delaware County, NY: -10.76%
- Monroe County, PA: 19.72%
- Pike County, PA: 31.68%

These three counties are forecast to experience greater rates of population change during the years 2015 to 2035 than the other basin counties; as can be expected from its exponential growth rate formula; the GROWTH function extrapolates these rates into greater population change than the FORECAST function.

Of the 21 other counties in the Delaware River Basin, eighteen have a forecast population variation between the functions of less than 5%, and three between 5% and 10%. The greatest growth in percentage terms is forecast for Monroe and Pike counties, PA.

All five of the New Jersey counties are expected to increase in population. Four of the New York counties (Broome, Chenango, Delaware, and Schoharie) are forecast to have significant decreases in population in 2065 compared to their 2015 levels. The greatest percentage decrease is forecast in Delaware County, which would have a 2065 population only 59% of its 2015 level. Broome County follows at 66%, Chenango County at 70%, and Schoharie County at 74%. (All percentages based on GROWTH forecast). Three Pennsylvania counties are forecast to have decreased by 2065, but at relatively minor levels (Lackawanna at 93% of its 2015 population, Luzerne at 92%, and Schuylkill at 94%).

In summary, both the GROWTH and FORECAST functions appear to provide reasonable forecasts of future population levels. The GROWTH function forecast, which provides an estimated population of 6,494,361 for the twenty-one basin counties, can be considered an “upper-bound” estimate. In estimating future levels of impervious coverage, as driven by population increases, the GROWTH function would provide a conservatively higher forecast, suitable for planning purposes.

Of the twenty-one counties evaluated, only four fall entirely within the boundaries of the Delaware River Basin: Pike, Monroe, and Northampton in Pennsylvania, and Warren in NJ. Five more counties have the vast majority of their area within the basin, while the remainder have 60% or less within. Thus, not all of the forecast population change for the 21 counties will affect impervious coverage percentages within the basin. Using ArcMap Geographic Information System (GIS) software, the 869 HRUs were assigned to the municipality in which their centroids are located. The forecast population change rates for that municipality (or county, in the case of New York State) were assigned to the HRU. As described in the following section, these rates of forecast population change were used as a proxy variable to increase the percentage of impervious coverage within the HRU.

A.5.5. Application of Population Projections to Percent Imperviousness

Examples of similar population-driven analyses of future levels of impervious coverage were sought. A forecast of future impervious coverage was done for the Clear Creek Basin in Texas, as described in the *Clear Creek General Reevaluation Report, Hydrologic Analysis, Without Project Conditions* (USACE-Galveston, 2003). Developed areas within the watershed were delineated as 50% or 100% developed. The extent of developed area within each Census tract was determined through GIS overlay analysis. This information was used to determine a “population/developed” area ratio for each Census tract. For the future years of 2010 and 2060, it was assumed that the “population/developed area” ratio would remain constant. The population projections for Census tracts were used to adjust the impervious coverage percentage. Thus, the amount of development for each tract could be projected for 2010 and 2060 conditions based upon the Census population projections for these years. Based on literature review and discussion at the June 2010 meeting at USACE, a similar approach was selected for the estimation of future conditions in the Delaware River Basin. The percentage increases in population for a given area were assumed to result in the same percentage increase in its impervious coverage. As noted previously, areas with declining forecast populations for 2065 were assumed to have the same level of impervious coverage as in the base year.

URS used the ratio index approach described above, and allocated projected population growth and subsequent increases in impervious coverage. The increased percentages of impervious coverage were then used as input into the rainfall/runoff model developed by USGS/HEC for use in forecasting the percentage increase in discharge at a given gage station (e.g., Trenton) in 2015 and 2065 over present-day discharges. Table A.5.5 summarizes the changes in percent imperviousness by watershed at gage location on the Delaware River.

Table A.5.5: Projected Composite HRU Percent Imperviousness Values at Gage Locations

USGS Gage	Year 2001 Composite HRU % Imperviousness	Year 2015 Projected Composite HRU % Imperviousness	Year 2065 Projected Composite HRU % Imperviousness
Trenton	0.0058	0.0068	0.0107
Riegelsville	0.0058	0.0068	0.0106
Belvidere	0.0017	0.0021	0.0042
Tocks Island	0.0012	0.0015	0.0028
Montague	0.0012	0.0014	0.0023
Port Jervis	0.0011	0.0012	0.0019

A.5.6. Future “Without” Project Conditions Flood Analysis Model Results

The USGS/HEC flood analysis model was used to examine any potential streamflow changes at the USGS Delaware River gage locations for the future “without” project conditions in year

2065. The suite of models used in the Flood Analysis Model package simulated the Delaware River Basin's hydrologic response during the three high-flow events between September 2004 and June 2006.

The PRMS model simulates reservoir inflow and watershed runoff for use as input into HEC-ResSim for the purpose of evaluating and comparing the effects of different watershed conditions on main-stem flooding in the Delaware River watershed draining to Trenton, N.J. USGS's PRMS model is a modular, physically based, distributed-parameter modeling system developed to evaluate the impacts of various combinations of precipitation, climate, and land use on surface water runoff and general basin hydrology.

The HEC-ResSim model simulated reservoir operations and routing of the flood flows through the river system that occurred during the three big storm events from 2004 to 2006. Both models are useful as planning tools to simulate the effects of land-use changes, different antecedent conditions on local runoff, reservoir inflows, reservoir operations, reservoir outflows and flow routing.

The percent imperviousness values for each HRU were changed in the PRMS model to match the values calculated by URS. Both models were then run to simulate the basin's response to the Sept. 2004, Apr. 2005, and Jun. 2006 storm events as if they occurred in the year 2065. No other changes to the input for either model were made for this simulation. It was the goal of the simulation to ascertain any changes in flows at the USGS gage locations on the Delaware River based solely on changes in percent imperviousness that would be representative of the year 2065.

The calculated flows at some of the USGS stream gage locations on the Delaware River increased only slightly as a result of increasing the percent imperviousness. Many other locations did not show an increase in flows. Tables A.5.6 – A.5.8 summarize the results.

Table A.5.6: Simulated Peak Flows for Sept. 2004 Storm in Year 2065

USGS Gage	Sept. 2004 Simulated PRMS/RES-SIM Peak Flow (cfs)	
	% Imperviousness Values from 2001	% Imperviousness Values Projected in 2065
Trenton	185,000	190,000
Riegelsville	181,000	184,000
Belvidere	148,000	150,000
Tocks Island	137,000	139,000
Montague	133,000	133,000
Port Jervis	126,000	126,000

Table A.5.7: Simulated Peak Flows for April 2005 Storm in Year 2065

USGS Gage	Apr. 2005 Simulated PRMS/RES-SIM Peak Flow (cfs)	
	% Imperviousness Values from 2001	% Imperviousness Values Projected in 2065
Trenton	235,000	240,000
Riegelsville	233,000	237,000
Belvidere	207,000	209,000
Tocks Island	177,000	178,000
Montague	173,000	174,000
Port Jervis	143,000	143,000

Table A.5.8: Simulated Peak Flows for June 2006 Storm in Year 2065

USGS Gage	Jun. 2006 Simulated PRMS/RES-SIM Peak Flow (cfs)	
	% Imperviousness Values from 2001	% Imperviousness Values Projected in 2065
Trenton	260,000	263,000
Riegelsville	254,000	256,000
Belvidere	225,000	226,000
Tocks Island	207,000	207,000
Montague	209,000	209,000
Port Jervis	198,000	198,000

A.5.7. Climate Variability Effects on Future "Without" Project Conditions Flows

In conjunction with the application of the flood analysis model to future "without" project conditions, a review of the current state of knowledge in the scientific community of how future climate variability could impact peak flood streamflows on the Delaware River was conducted. As summarized in Section A.1.4., it is projected that rainfall events could become more frequent and intense in nature in the basin. Snow melt due to temperature increases could happen quicker in the spring resulting in higher peaks, as well.

Given these projections along with the slight increase in flow due to increased imperviousness from the flood analysis model, it was determined that an increase of ten percent would be a reasonable estimate of future streamflows on the Delaware River. This increase was applied uniformly at all USGS gage locations along the Delaware River. Table A.5.9 summarizes the peak streamflows on the Delaware River used for the future "without" project conditions.

Table A.5.9: Peak Delaware River Streamflows for Future "Without" Project Conditions

	Annual Chance of Exceedance / Recurrence Interval							
	50%	20%	10%	4%	2%	1%	0.40%	0.20%
Location	2-year	5-year	10-year	25-year	50-year	100-yr	250-yr	500-yr
Montague, NJ	71,720	111,100	139,700	180,400	213,400	248,600	296,516	338,800
Water Gap, PA	78,980	121,000	152,900	195,800	231,000	268,400	319,836	365,200
Belvidere, NJ	84,590	127,600	159,500	202,400	236,500	272,800	323,191	367,400
Riegelsville, NJ	101,530	149,600	183,700	228,800	265,100	301,400	350,966	393,800
Trenton, NJ	104,390	151,800	185,900	232,870	269,500	308,000	362,076	409,200

A.5.8. Future "Without" Project Hydraulic Model Results

Streamflows for the HEC-RAS model developed for the "without" project conditions were modified to represent the future "without" project conditions. It was assumed that development along the Delaware River in the floodplain would not change from 2015 to 2065 (no changes in roughness coefficients) and that the cross-section geometries in the HEC-RAS model would also be static and remain unchanged. Surface elevation profiles were generated in HEC-RAS for the ACE values of 50%, 20%, 10%, 4%, 2%, 1%, 0.4%, and 0.2%. The water surface elevation profiles were used to relate flood damage at structures to predicted flood levels in the future. The economic forecasting model HEC-FDA (Flood Damage Reduction Analysis) was used for the future conditions model just like it was used for the base year model. Additional information about HEC-FDA and the future "without" project average annual damage amounts per community can be found in the Economics Technical Appendix. Standard HEC-RAS profile plots from Knowlton Township to Trenton are presented in Figures A.5.5 – A.5.8.

HEC-RAS results indicate that the ten percent increase in flows resulted in a maximum water surface elevation increase at the 1.0% ACE of 5.95 feet at river mile 190.46 which is the upstream cross-section to the railroad bridge in Harmony Township. Another significant increase of 5.77 feet in water surface elevations occurred at the 4.0% ACE event at river mile 183.53 which is the upstream cross-section of the Northampton Street Bridge in Phillipsburg. Generally speaking as would be expected, the majority of the increases in water surface elevations occur immediately upstream of many bridges. This occurs because the increased flows for a given event results in an elevation that places the water surface in contact with the bridge structure. Once that occurs, the flow regime changes at the bridge, resulting in the increase in water surface elevation.

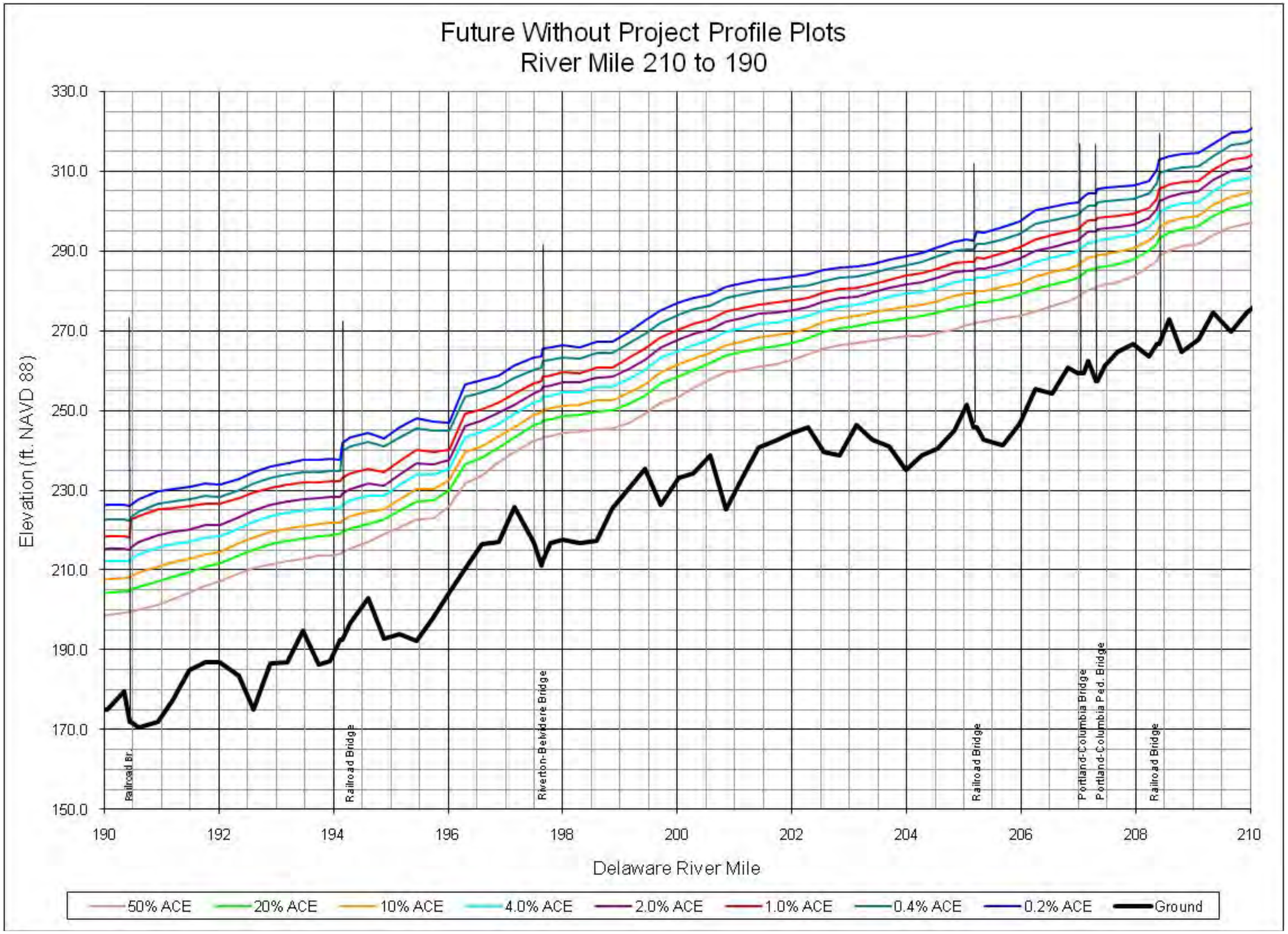

Figure A.5.5: HEC-RAS Water Surface Profiles for "Without" Project Conditions from River Mile 210 to 190.

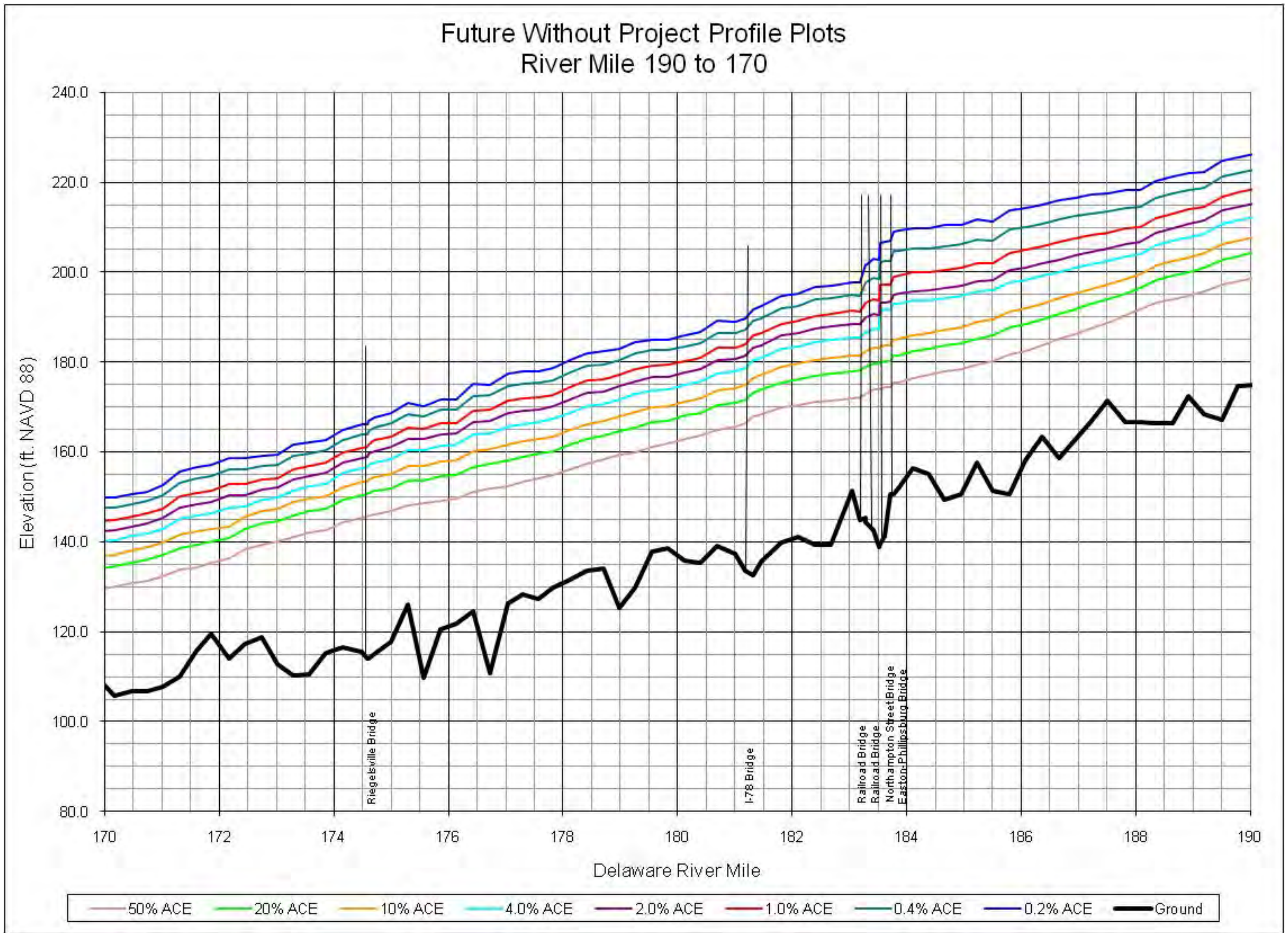

Figure A.5.6: HEC-RAS Water Surface Profiles for "Without" Project Conditions from River Mile 190 to 170.

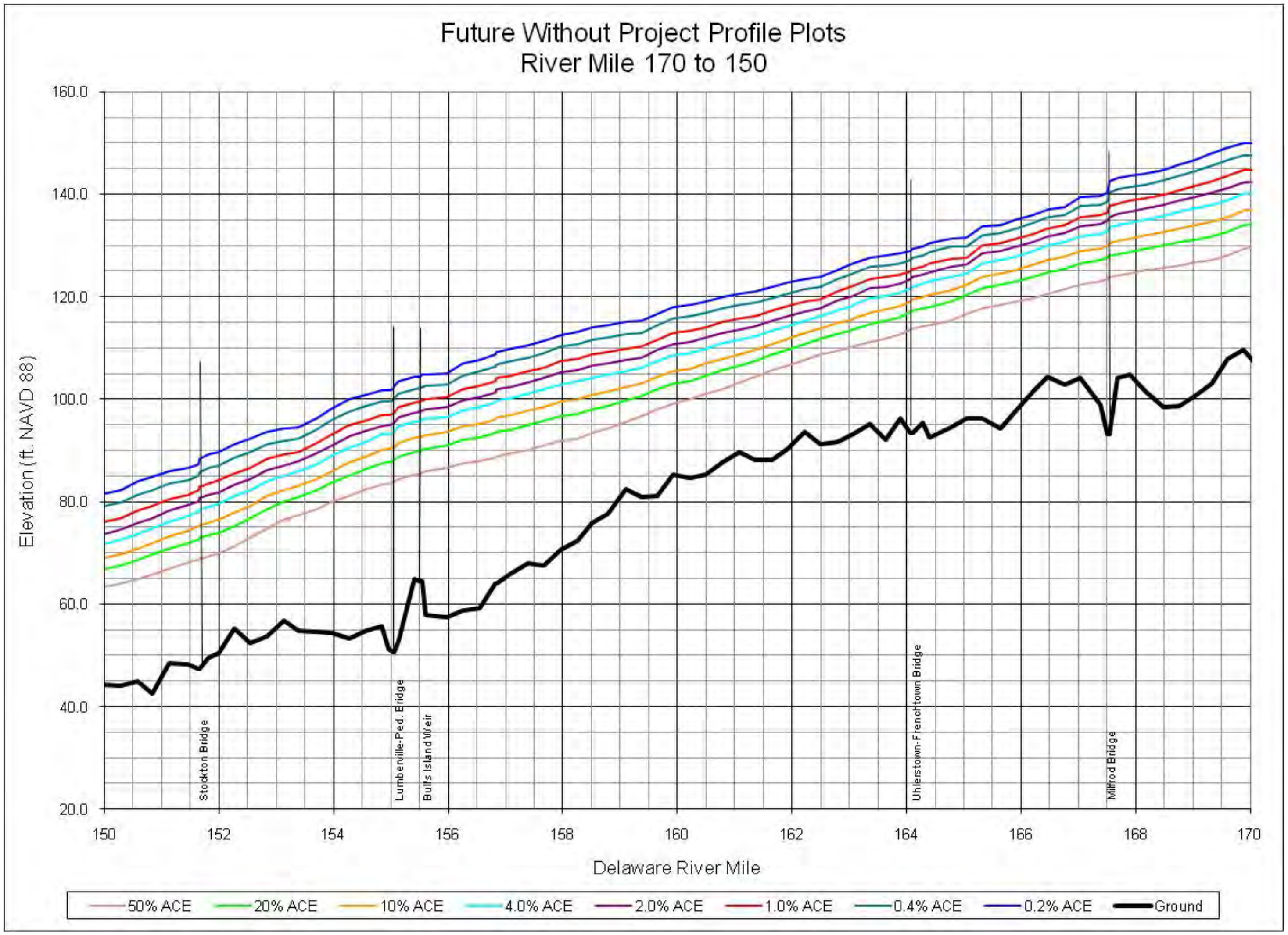

Figure A.5.7: HEC-RAS Water Surface Profiles for Without Project Conditions from River Mile 170 to 150.

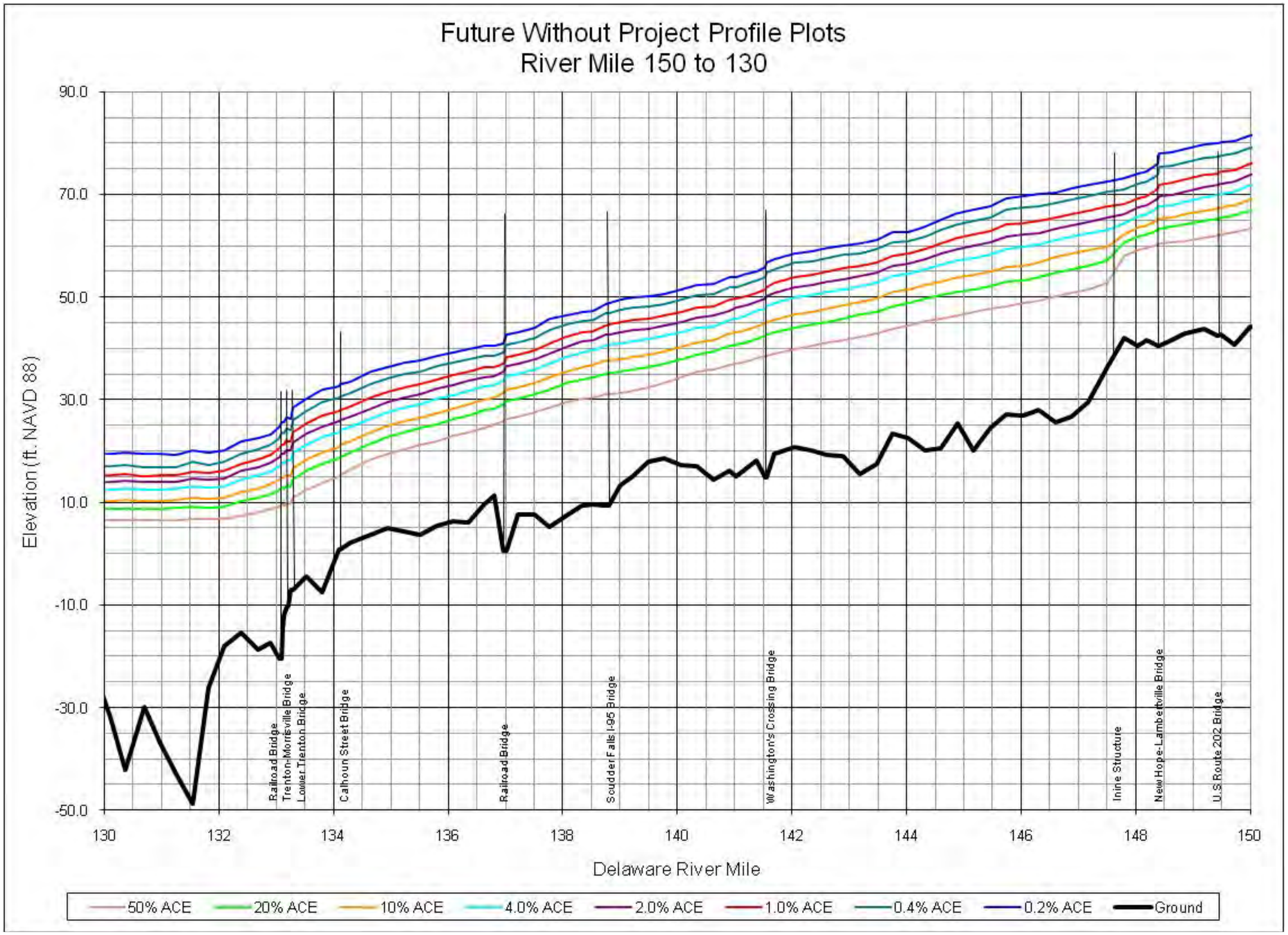

Figure A.5.8: HEC-RAS Water Surface Profiles for "Without" Project Conditions from River Mile 150 to 130.

A.6.0. WITHOUT PROJECT HYDROLOGIC MODEL FOR REPAUPO CREEK WATERSHED

A.6.1. Repaupo Creek Watershed Overview

The Repaupo Creek Watershed is located in Gloucester County, NJ and includes the townships of: Greenwich, East Greenwich, Logan, Woolwich, Harrison and Mantua. The watershed is approximately 26 square miles in size and has elevations that vary from 155 ft. NAVD 88 down to -3 ft. NAVD 88. The Gibbstown Levee runs along the Delaware River and is approximately 4.5 miles in length and provided some level of protection for the towns of Logan and Greenwich from storm surges and tidal flooding from the Delaware River. Several small tributaries drain the Repaupo Watershed. They include: Repaupo Creek, White Sluice Race, Sand Ditch, Clonmell Creek, London Branch, Nehonsey Brook, Pargey Creek, Rattling Run, and Still Run. Four of the tributaries (Repaupo, White Sluice, Sand Ditch, and Clonmell) have floodgates at their confluences with the Delaware River at the Gibbstown Levee. A fifth floodgate commonly referred to as the EL Sluice floodgate drains surface flow from the DuPont Industrial Site.

West of the Repaupo Creek Watershed and out of the influence of the levee is an area called Cedar Swamp. It is drained by a man-made canal named Klondike Ditch. The Little Timber Creek Watershed and tides from the Delaware River itself contributes flow to Klondike Ditch. It has been reported by local officials and residents that in addition to draining back into the Delaware River during low tides, portions of Cedar Swamp drain into the Repaupo Creek Watershed leeward of the Gibbstown Levee through the Godwin Pump Property. This potential connection to the Repaupo Creek Watershed was investigated as part of the hydrologic model and is discussed later. Figure A.6.1 show a view of the Repaupo Creek Watershed and the adjacent Cedar Swamp.

A.6.2. Local Precipitation

There are several NCDC (National Climatic Data Center) precipitation stations located in Southern New Jersey, Southeast Pennsylvania, and Northern Delaware that are in the vicinity of the study area. The closest long-term station is five miles away and is located at the Philadelphia International Airport. Hourly data and 15-minute data (where available) from the year 1900 to the year 2011 was used to evaluate historical precipitation events that impacted the study area at several station locations. Table A.6.1 summarizes the ten highest 24-hour precipitation events recorded at the Philadelphia International Airport Weather Station.

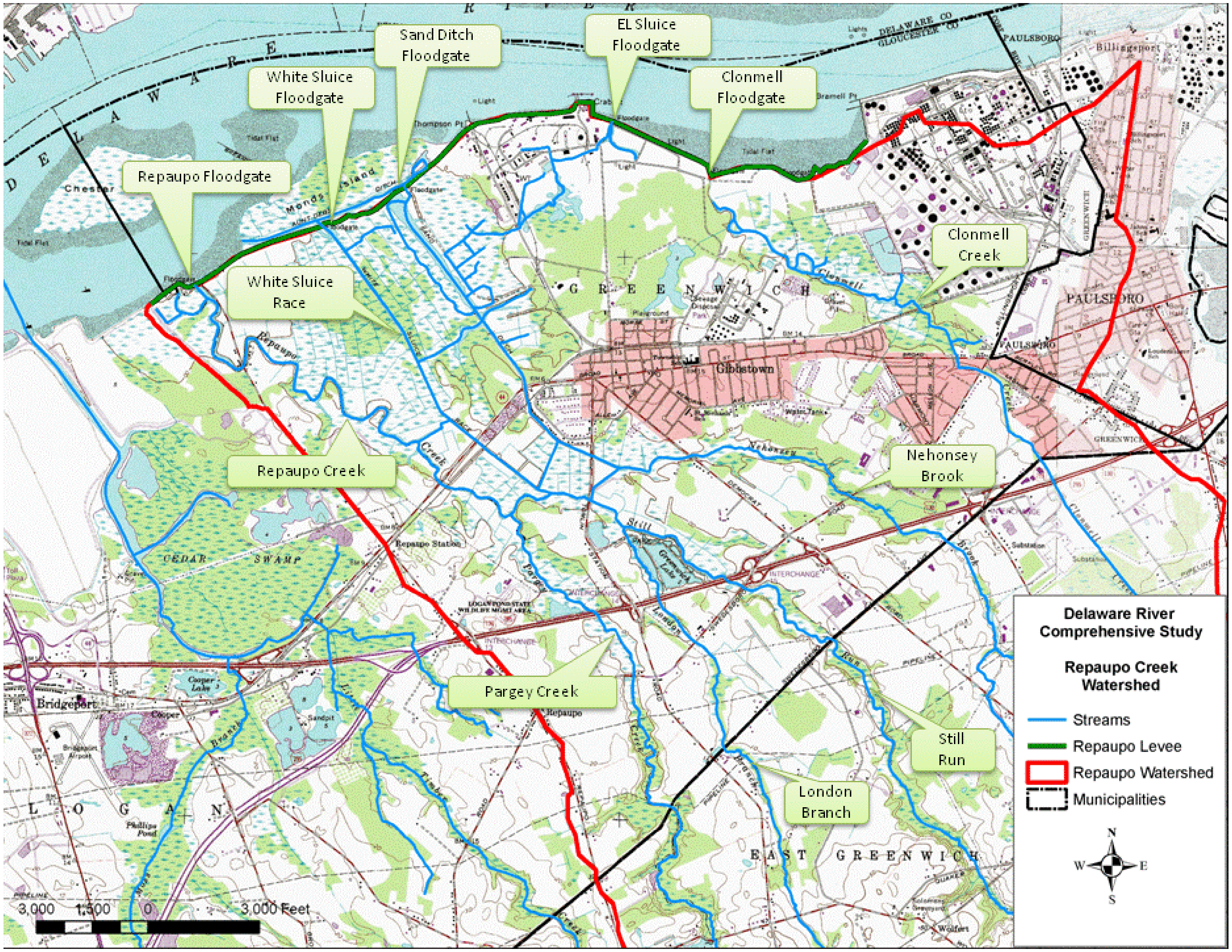

Figure A.6.1: Repaupo Creek Watershed at the Delaware River

Table A.6.1: Highest 24-hr Precipitation Totals Recorded at Philadelphia Int'l Airport

Rank	Date	24-hr Precipitation Total (inches)	Annual Chance of Exceedance
1	9/16/1999	6.77	2%
2	10/8/2005	5.94	4%
3	7/28/1971	5.68	5%
4	9/15/2004	5.59	5%
5	9/12/1960	5.45	5%
6	8/31/2011	4.93	9%
7	8/13/1955	4.84	10%
8	9/14/1966	4.69	11%
9	7/12/2004	4.68	11%
10	9/27/1985	4.64	12%

Three storm events post 1990 were selected for use in the hydrologic model based upon their 24-hour precipitation totals at the Philadelphia Airport weather station; the nearby stations of Mount Holly, NJ and Wilmington, DE; and ancillary data from other sources. The three storm events were Hurricane Floyd on September 15-17, 1999, a flash flood from July 12-13, 2004, and an April 14-17, 2007 nor'easter. A fourth storm which occurred on March 13-18, 2010 was also selected with moderate precipitation totals but with elevated Delaware River storm tides in the region over several tidal cycles. Summary narratives of the four storms are as follows:

<u>September 15-17, 1999</u>: Hurricane Floyd battered New Jersey on September 16th and brought with it torrential and in some areas, unprecedented and record breaking rains and damaging winds. The combination of winds funneling into the Delaware Bay and the Delaware River and the record runoff from inland waterways produced minor to moderate tidal flooding at the times of high tide in Cumberland, Salem, Gloucester, Camden and Burlington Counties. Evacuations occurred in low-lying areas near the river. The evening of the 16^{th}, high tide also slowed the discharge of streams into the Delaware. Precipitation totals in the county were as follows: 8.54 inches in Pitman, 7.80 inches in West Deptford, 7.60 inches in Washington Township, and 7.24 inches in Verga.

<u>July 12-13, 2004</u>: A series of thunderstorms with very heavy rain caused widespread poor drainage flooding and scattered stream and lake flooding. Hardest hit were townships west of the New Jersey Turnpike in Gloucester County. Storm totals included 6.14 inches in West Deptford and Doppler Radar storm total estimates exceeded six inches in Greenwich and Logan Townships.

<u>April 14-17, 2007</u>: The combination of the fresh water run-off and the onshore flow associated with the nor'easter caused minor tidal flooding at the time of high tide along tidal sections of the Delaware River and its tributaries overnight on the 15th. This also slowed the run-off of streams, creeks and rivers that empty into the Delaware River. In Greenwich Township, tidal flood waters along the Repaupo Creek were forced into the Delaware River. The high tide at the

Delaware River in Philadelphia reached 8.72 feet above MLLW at 1230 a.m. EDT on the 16th. Minor tidal flooding in the region begins at 8.2 feet above MLW. Over 6 inches of rain fell in 24-hours at the Mount Holly, NJ station and at nearby Glassboro, NJ. Severe flooding caused over $3 million dollars in damage according to the Gloucester County Multi-Jurisdictional Hazard Mitigation Plan.

<u>March 13-18, 2010</u>: Four days of rain, heaviest on the 13th, culminated in major flooding throughout New Jersey. The heaviest rain fell during the morning in the southern third of the state followed by periods of lighter rain which persisted into the 14th and 15th which slowed the recession of streams and rivers in the area. The heavy rain was caused by a slow moving low pressure system that had a tremendous fetch of moist air from the Atlantic Ocean.

A.6.3. Frequency Precipitation

Precipitation duration frequency estimates for the maximum observed rainfall intervals were determined using the Precipitation Frequency Data Server (NOAA Atlas 14) for Gloucester County as shown in Table A.6.2. Maximum observed precipitation amounts for various durations were extracted from the time-series data and are summarized in Tables A.6.3 – A.6.5 for the Philadelphia Airport, Wilmington, DE and Mt. Holly, NJ stations, respectively.

All three high precipitation storms had much larger return periods for longer rainfall durations than they did for shorter return periods. These durations (12 hrs. and 24 hrs.) exceed the times of concentration for the study area. This suggests that runoff internal to the watershed from just precipitation events should have had enough time to drain without causing any significant flooding problems during these events. However, this does not take into account tidal flooding from the Delaware River during these events which is the predominate flooding source for the study area.

Table A.6.2: Frequency-Based Precipitation Estimates for Gibbstown, NJ and Surrounding Area from NOAA Atlas 14

Duration	Annual Chance of Exceedance / Recurrence Interval (Precipitation in inches)							
	50% (2-yr)	20% (5-yr)	10% (10-yr)	4% (25-yr)	2% (50-yr)	1% (100-yr)	0.5% (200-yr)	0.2% (500-yr)
5-min	0.414 (0.380-0.452)	0.486 (0.445-0.530)	0.538 (0.492-0.586)	0.599 (0.545-0.653)	0.642 (0.580-0.700)	0.683 (0.615-0.748)	0.719 (0.642-0.790)	0.760 (0.672-0.841)
10-min	0.663 (0.608-0.722)	0.779 (0.713-0.848)	0.860 (0.786-0.937)	0.955 (0.868-1.04)	1.020 (0.923-1.12)	1.090 (0.977-1.19)	1.140 (1.02-1.25)	1.200 (1.06-1.33)
15-min	0.833 (0.765-0.908)	0.985 (0.902-1.07)	1.090 (0.994-1.19)	1.210 (1.10-1.32)	1.290 (1.17-1.41)	1.370 (1.23-1.50)	1.440 (1.28-1.58)	1.510 (1.34-1.68)
30-min	1.150 (1.06-1.25)	1.400 (1.28-1.52)	1.580 (1.44-1.72)	1.79 (1.63-1.96)	1.95 (1.76-2.13)	2.10 (1.89-2.30)	2.24 (2.00-2.46)	2.41 (2.13-2.67)
60-min	1.44 (1.33-1.57)	1.80 (1.64-1.95)	2.05 (1.88-2.24)	2.39 (2.17-2.60)	2.64 (2.39-2.88)	2.89 (2.60-3.17)	3.14 (2.80-3.45)	3.46 (3.06-3.82)
2-hr	1.74 (1.59-1.90)	2.17 (1.97-2.38)	2.50 (2.27-2.74)	2.93 (2.64-3.22)	3.27 (2.93-3.60)	3.61 (3.22-3.98)	3.95 (3.49-4.37)	4.40 (3.84-4.90)
3-hr	1.89 (1.73-2.07)	2.36 (2.16-2.59)	2.73 (2.48-3.00)	3.22 (2.91-3.53)	3.61 (3.24-3.96)	4.01 (3.57-4.41)	4.41 (3.88-4.87)	4.95 (4.29-5.50)
6-hr	2.33 (2.13-2.56)	2.90 (2.65-3.19)	3.37 (3.06-3.70)	4.02 (3.62-4.42)	4.55 (4.06-5.01)	5.11 (4.52-5.63)	5.69 (4.97-6.30)	6.50 (5.57-7.25)
12-hr	2.81 (2.57-3.11)	3.53 (3.21-3.90)	4.13 (3.74-4.55)	5.01 (4.48-5.52)	5.75 (5.10-6.34)	6.55 (5.73-7.25)	7.4 1 (6.40-8.25)	8.67 (7.31-9.71)
24-hr	3.26 (3.00-3.55)	4.13 (3.79-4.51)	4.87 (4.45-5.31)	5.95 (5.42-6.48)	6.88 (6.22-7.47)	7.89 (7.09-8.55)	9.01 (8.03-9.74)	10.7 (9.38-11.5)
2-day	3.74 (3.43-4.07)	4.74 (4.35-5.17)	5.58 (5.10-6.08)	6.79 (6.18-7.39)	7.82 (7.08-8.50)	8.92 (8.03-9.69)	10.1 (9.05-11.0)	11.9 (10.5-12.9)
3-day	3.95 (3.63-4.30)	5.00 (4.59-5.44)	5.86 (5.37-6.38)	7.12 (6.49-7.73)	8.17 (7.42-8.87)	9.31 (8.40-10.1)	10.5 (9.45-11.4)	12.3 (10.9-13.4)
4-day	4.16 (3.83-4.54)	5.25 (4.83-5.72)	6.15 (5.64-6.68)	7.44 (6.80-8.08)	8.53 (7.75-9.24)	9.70 (8.77-10.5)	11.0 (9.84-11.9)	12.8 (11.4-13.9)
7-day	4.78 (4.43-5.18)	5.95 (5.52-6.44)	6.92 (6.40-7.49)	8.33 (7.67-9.01)	9.51 (8.71-10.3)	10.8 (9.81-11.6)	12.1 (11.0-13.1)	14.1 (12.6-15.2)
10-day	5.34 (4.98-5.75)	6.56 (6.11-7.06)	7.54 (7.02-8.12)	8.94 (8.28-9.60)	10.1 (9.30-10.8)	11.3 (10.3-12.1)	12.5 (11.4-13.4)	14.3 (13.0-15.4)

Notes: Point Precipitation Point Estimates with 90% Confidence Intervals (inches) based upon Partial-Duration Series

Table A.6.3: Precipitation Analysis at Philadelphia Int'l Airport, PA NCDC Station

	Hurricane Floyd Sept. 15-17, 1999		July 12-13, 2004 Storm Event		April 14-17, 2007 Storm Event	
Duration	Max Precip. (in.)	Annual Chance of Exceedance	Max Precip. (in.)	Annual Chance of Exceedance	Max Precip. (in.)	Annual Chance of Exceedance
15	n/a	n/a	n/a	n/a	n/a	n/a
30	n/a	n/a	n/a	n/a	n/a	n/a
60 (1 hr)	0.95	> 50%	0.73	> 50%	0.43	> 50%
120 (2 hrs)	1.49	> 50%	1.20	> 50%	0.77	> 50%
240 (3 hrs)	2.15	27%	1.65	> 50%	1.06	> 50%
360 (6 hrs)	3.51	8%	2.74	24%	1.91	> 50%
720 (12 hrs)	6.17	1%	4.28	8%	3.07	32%
1440 (24 hrs)	6.77	2%	4.68	11%	4.36	15%

Table A.6.4: Precipitation Analysis at Wilmington, DE NCDC Station

	Hurricane Floyd Sept. 15-17, 1999		July 12-13, 2004 Storm Event		April 14-17, 2007 Storm Event	
Duration	Max Precip. (in.)	Annual Chance of Exceedance	Max Precip. (in.)	Annual Chance of Exceedance	Max Precip. (in.)	Annual Chance of Exceedance
15	n/a	n/a	n/a	n/a	n/a	n/a
30	n/a	n/a	n/a	n/a	n/a	n/a
60 (1 hr)	1.14	> 50%	0.99	> 50%	0.59	> 50%
120 (2 hrs)	1.63	> 50%	1.16	> 50%	0.72	> 50%
240 (3 hrs)	1.97	40%	1.23	> 50%	1.08	> 50%
360 (6 hrs)	2.43	40%	1.60	> 50%	1.85	> 50%
720 (12 hrs)	2.96	38%	1.93	> 50%	2.04	> 50%
1440 (24 hrs)	4.50	13%	2.05	> 50%	2.62	> 50%

Table A.6.5: Precipitation Analysis at Mt. Holly, NJ NCDC Station

	Hurricane Floyd Sept. 15-17, 1999		July 12-13, 2004 Storm Event		April 14-17, 2007 Storm Event	
Duration	Max Precip. (in.)	Annual Chance of Exceedance	Max Precip. (in.)	Annual Chance of Exceedance	Max Precip. (in.)	Annual Chance of Exceedance
15	0.30	>50%	n/a	n/a	0.20	>50%
30	0.50	>50%	n/a	n/a	0.40	>50%
60 (1 hr)	0.90	>50%	n/a	n/a	0.60	>50%
120 (2 hrs)	1.50	>50%	n/a	n/a	0.80	>50%
240 (3 hrs)	2.10	>50%	n/a	n/a	1.00	>50%
360 (6 hrs)	3.20	12%	n/a	n/a	1.70	>50%
720 (12 hrs)	5.10	4%	n/a	n/a	3.20	28%
1440 (24 hrs)	6.00	4%	n/a	n/a	6.20	3%

A.6.4. Tidal Flows During High Precipitation Events

The NOAA tidal station at Philadelphia is the closest long-term tide station to the study area and is located approximately 16 miles upstream on the Delaware River. It is also located only approximately 5 miles from the NCDC Philadelphia Airport precipitation station. The tidal stage frequency analysis used for this study was obtained from NOAA.

Tidal elevations recorded every hour at the station were analyzed for the four storm events previously selected for the precipitation analysis. The peak tidal elevations from the time series, and the corresponding annual chance of exceedance for each of the four storms are summarized in Table A.6.6.

Table A.6.6: Delaware River Peak Stage Elevations at the Philadelphia Tidal Station for Selected Storm Events

Storm Event	Peak Elevation (ft. NAVD 88)	Annual Chance of Exceedance
Hurricane Floyd (Sept. 1999)	5.87	34%
July 12-13, 2004	5.02	<50%
April 13-20, 2007	6.16	23%
March 12-18, 2010	5.92	30%

A peak analysis was done to compare Delaware River tides with the recorded precipitation data at the nearby NCDC Station for the four storm events. From the analyses if can be concluded that the peak storm tide on the river lagged the peak hourly precipitation recorded by 8-11 hours for Hurricane Floyd and the July 2004 storm. That time difference is approximately one tidal cycle in length. The peak tide after the April 2007 storm occurred 3.5 days after the peak hourly precipitation. However, that peak may have been associated with other factors within the

Delaware River Basin, while the actual storm peak seemed to occur 10 hours after the maximum hourly precipitation. This 10 hour lag is consistent with the lag shown for the other two storms. The lag for the March 2010 storm was greater. The peak tide occurred 28 hours after the peak precipitation for that event.

In addition to the peak analysis corresponding to the high precipitation storm events; historical peak tidal elevations over the entire period of record on the Delaware River at the Philadelphia tidal station were examined. The storms that resulted in the highest peak tidal elevations are presented in Table A.6.7 while Table A.6.8 summarizes the highest historical river peaks recorded at the Philadelphia tidal station and the corresponding precipitation that occurred at the nearby Philadelphia Airport weather station. If compared to the high precipitation storms listed in Table A.6.1, it can be seen that there was no single storm in common between the three top 10 rankings. Only the June 30, 1973 storm produced both a high 24-hr precipitation total ranked just outside of the top ten and a top ten storm surge elevation on the Delaware River.

Table A.6.7: Highest Recorded Elevations at Philadelphia Tidal Station with Corresponding 24-hr Precipitation

		NOAA Station 8545240 Philadelphia, PA		NCDC Station 366889 Philadelphia Int'l Airport, PA	
Rank	Date	Peak River Elevation (ft. NAVD 88)	River Elevation Annual Chance of Exceedance	Total 24-hr Precipitation (in.)	24-hr Precipitation Annual Chance of Exceedance
1	11/25/1950	7.36	2%	3.46	37%
2	4/17/2011	7.34	2%	3.12	>50%
3	10/25/1980	7.04	4%	3.85	25%
4	2/26/1979	6.74	7%	1.55	>50%
5	4/3/2005	6.65	9%	2.83	>50%
6	12/11/1992	6.59	11%	3.03	>50%
7	6/30/1973	6.45	13%	4.62	12%
8	9/19/2003	6.35	16%	1.14	>50%
9	4/19/2007	6.34	17%	4.36	15%
10	11/28/1993	6.33	17%	0.55	>50%

Storm events on the Delaware River were also analyzed at a selected low elevation in order to relate storm duration to flood gate operations at Repaupo Creek and White Sluice Race as summarized in Table A.6.8. Tidal storms from March 1989 to Oct. 2011 were analyzed against a tidal threshold value of -1.0 ft. NAVD 88. Only the March 18, 2010 storm ranked in the top ten of longest duration storms above the -1.0 ft. NAVD 88 threshold. Also the peak analysis comparison indicated above shows that during Hurricane Floyd the tides were above the threshold for 36 consecutive hours; the July 11-15, 2004 tides were above the threshold for 10 hours; and the April 15-19, 2007 tides were above the threshold for 36 hours consecutively. The durations in Table A.6.8 do not indicate how long the floodgates were closed during those events. The tide gates would have remained open during these times if the interior pond elevation was greater than the tidal elevation on the Delaware River.

Table A.6.8: Longest Duration Storms at Philadelphia Tidal Station with Corresponding 24-hr Precipitation

Rank	End Date of Storm	Duration of Tides Above -1.0 ft. NAVD 88 (hrs)	Peak Tidal Elevation (ft. NAVD 88)	River Elevation Annual Chance of Exceedance	Peak Total 24-hr Precipitation (in)	24-hr Precipitation Annual Chance of Exceedance
1	9/12/2011	148	6.28	20%		>50%
2	3/18/2010	109	5.92	34%	2.77	>50%
3	7/1/2006	98	6.24	21%	0.53	>50%
4	5/15/1998	97	5.93	34%	2.57	>50%
5	4/5/2005	88	6.65	9%	2.83	>50%
6	12/18/1996	86	5.72	48%	0.18	>50%
7	4/3/1993	74	5.75	45%	1.00	>50%
8	3/13/2011	73	5.73	45%		>50%
9	12/13/1992	61	6.59	11%	3.03	>50%
9	10/22/1996	61	5.72	48%	2.22	>50%
9	2/7/1998	61	5.74	45%	0.18	>50%

Based upon the analysis and knowledge of the region, the relationship between the precipitation frequency over the watershed and the stage frequency of the Delaware River was assumed to be independent of each other. The 1% ACE precipitation event over the watershed is equally likely to happen with a low, intermediate, or high Delaware River tidal elevation. A sensitivity analysis of differing tailwater conditions on the Delaware River and the effects on the interior pond peak elevations was done. Ultimately, a reasonable tailwater condition was selected based upon the results of the sensitivity analysis.

A.6.5. Watershed Geology and Soils

The Repaupo Creek Watershed is underlain by two major aquifers. These are the Mt. Laurel Wenonah aquifer and the Kirkwood-Cohansey aquifer. The Kirkwood-Cohansey is unconfined and the Mt. Laurel-Wenonah aquifer is confined except where it reaches the surface or is overlain by permeable surface material

Gloucester County is in the southwestern part of New Jersey and lies within the Coastal Plain Physiographic Province of New Jersey. Raccoon Creek is the watershed adjacent to the Repaupo Creek Watershed in the County. They are very similar to each other. The following geology and soil description is from the *Raccoon Creek Watershed Characterization and Assessment Report* authored by the Gloucester and Camden County Soil Conservation Districts:

> *"All the soils have formed on unconsolidated beds of either sand or clay mixed with silt or gravel (SCS, 1962). These beds were laid down in a succession of ocean or river deposits and then tilted to the southeast. Glacial waters brought deposits of rounded quartzes gravel. During periods of low water, wind and water erosion reworked the*

original deposits. Climate and topography contributed significantly to the formation of the soils. More recently, human activities have begun to affect the soil, as large areas have been drained, stripped of topsoil, re-graded, filled, borrowed from and otherwise altered."

The Natural Resources Conservation Services (NRCS) classifies most soils into one of four Hydrologic Soil Groups (HSG) based on their minimum infiltration (transmission) rates when thoroughly wetted. Table A.6.9 summarizes the acreage and percentage of each HSG within the watershed.

Table A.6.9: Repaupo Creek Watershed Hydrologic Soil Groups

HSG	Area (acres)	Percent of Watershed
A	153.32	0.98
B	7,272.13	46.61
B/D	2,113.58	13.55
C	2,390.94	15.32
C/D	301.77	1.93
D	3,370.72	21.60

A.6.6. Watershed Land Cover and Land Use

The Repaupo Creek Watershed is predominantly agricultural in nature south of Interstate 295. Several data sources were available to examine land use and land cover changes from 1986 to 2007. The datasets included the Land Use/Land Cover Databases for years of 1986, 1995, 2002, and 2007 from NJDEP, and the National Land Cover Data (NLCD) dataset from the year of 2006 from the National Multi-Resolution Land Characteristics Consortium (MRLC). Maps clipped to the Repaupo Creek Watershed were developed and the data was analyzed from these datasets in order to characterize and evaluate changes within the watershed. Generally there was a high level of agreement between the NLCD 2006 and NJDEP 2007 datasets.

Land use and land cover changes within the Repaupo Creek Watershed were analyzed in order to evaluate potential runoff changes that have occurred in the watershed. The NJDEP's datasets from 1986 to 2007 were used in the analysis. The Repaupo Creek Watershed was divided into three regions for the analysis. The regions were: (1) South of the NJ Turnpike; (2) Between the NJ Turnpike and I-295; and (3) North of I-295. These regions are shown in Figure A.6.2. Trends in acreage reclassified from one category to another were compiled and graphed for comparison purposes. ArcMap software was used in the analysis to aggregate similar land cover classifications by watershed region. Results of the analysis are shown in Tables A.6.10 – A.6.13.

Table A.6.10: Repaupo Creek Watershed Land Cover Analysis for the Year 1986

Region	Area (acres)	Agriculture (acres)	Forest (acres)	Urban (acres)	Barren Land (acres)	Water (acres)	Wetlands (acres)	Percent Imperviousness
1	2,644.05	1,971.13	193.21	296.07	21.46	3.41	158.77	n/a
2	7,114.75	4,179.82	640.10	1,083.15	64.34	67.72	1,079.62	n/a
3	5,843.65	688.15	373.31	2,024.27	301.62	265.02	2,191.28	n/a

Table A.6.11: Repaupo Creek Watershed Land Cover Analysis for the Year 1995

Region	Area (acres)	Agriculture (acres)	Forest (acres)	Urban (acres)	Barren Land (acres)	Water (acres)	Wetlands (acres)	Percent Imperviousness
1	2,644.05	1,853.16	246.59	312.04	19.97	3.41	208.87	2.79
2	7,114.75	3,594.93	727.27	1,339.45	11.85	67.90	1,373.34	4.50
3	5,843.65	540.96	251.11	2,107.56	19.15	276.65	2,648.21	10.99

Table A.6.12: Repaupo Creek Watershed Land Cover Analysis for the Year 2002

Region	Area (acres)	Agriculture (acres)	Forest (acres)	Urban (acres)	Barren Land (acres)	Water (acres)	Wetlands (acres)	Percent Imperviousness
1	2,644.05	1,760.70	262.44	366.41	37.15	7.80	209.55	3.21
2	7,114.75	3,268.45	770.06	1,584.72	23.49	92.14	1,375.89	5.64
3	5,843.65	464.57	233.36	2,210.99	24.51	320.58	2,589.62	10.91

Table A.6.13: Repaupo Creek Watershed Land Cover Analysis for the Year 2007

Region	Area (acres)	Agriculture (acres)	Forest (acres)	Urban (acres)	Barren Land (acres)	Water (acres)	Wetlands (acres)	Percent Imperviousness
1	2,644.05	1,335.86	220.96	824.16	52.00	10.03	201.03	5.69
2	7,114.75	2,650.46	760.20	1,875.83	227.63	259.57	1,341.06	6.23
3	5,843.65	454.99	226.20	2,261.48	18.17	336.61	2,546.20	11.12

Region 1 is predominately agricultural in nature but some of that land is converting to residential development. As an example, Region 1 went from 75% agricultural in 1986 down to 51% in 2007, and went from 11% residential development up to 31%. This change in land use could result in higher rainfall runoff coming down the watershed into the other regions and out to the Delaware River. Local officials and residents have stated that the natural storage area behind the levee in Region 3 has seen increased water levels the past twenty years. This land use change in Region 3 is one possible explanation for those elevated water levels. The hydrologic model discussed later in the appendix investigated this potential source along with other sources which included tidal flooding from the Delaware River.

Land use for Region 2 is mixed split between several classifications. Just like with Region 1, it too has undergone a change from 1986 to 2007. The agricultural land has decreased over time while the residential development has increased. Its increase in development is not as dramatic as Region 1, it has gone from 15% in 1986 up to 26% in 2007.

Region 3 has been relatively unchanged over time from 1986 to 2007 as Tables A.6.10 – A.6.13 show. The Region is predominantly urban (with the town of Gibbstown within its boundaries) and wetlands. The wetlands areas are used as natural storage for runoff coming down the watershed and from tidal flooding from the Delaware River.

The percent imperviousness from the NLCD 2006 dataset was mapped and the percent imperviousness trends from 1995 to 2007 were also analyzed from the NJDEP's datasets. NJDEP datasets estimate percent imperviousness on land use classifications and does not distinguish between connected and disconnected areas. Runoff from disconnected areas goes to areas where some of the runoff can infiltrate into the ground. Connected areas do not discharge into any such areas.

Increased development in the upper portion of the watershed would translate to higher percent imperviousness numbers. This increase could potentially lead to higher runoff volumes coming down the watershed to Gibbstown and the surrounding area if local stormwater management measures were not adequate. This would be the case for smaller more frequent storms while storms of larger magnitude (say 1% ACE) would not be impacted because the ground would be saturated and relatively impervious due to the large amount of rainfall. Tables A.6.10 – A.6.13 summarize the composite percent imperviousness for each region for the analysis years. As the tables show, percent imperviousness increased slightly from 1995 to 2007 for Regions 1-2 and remained relatively unchanged in Region 3.

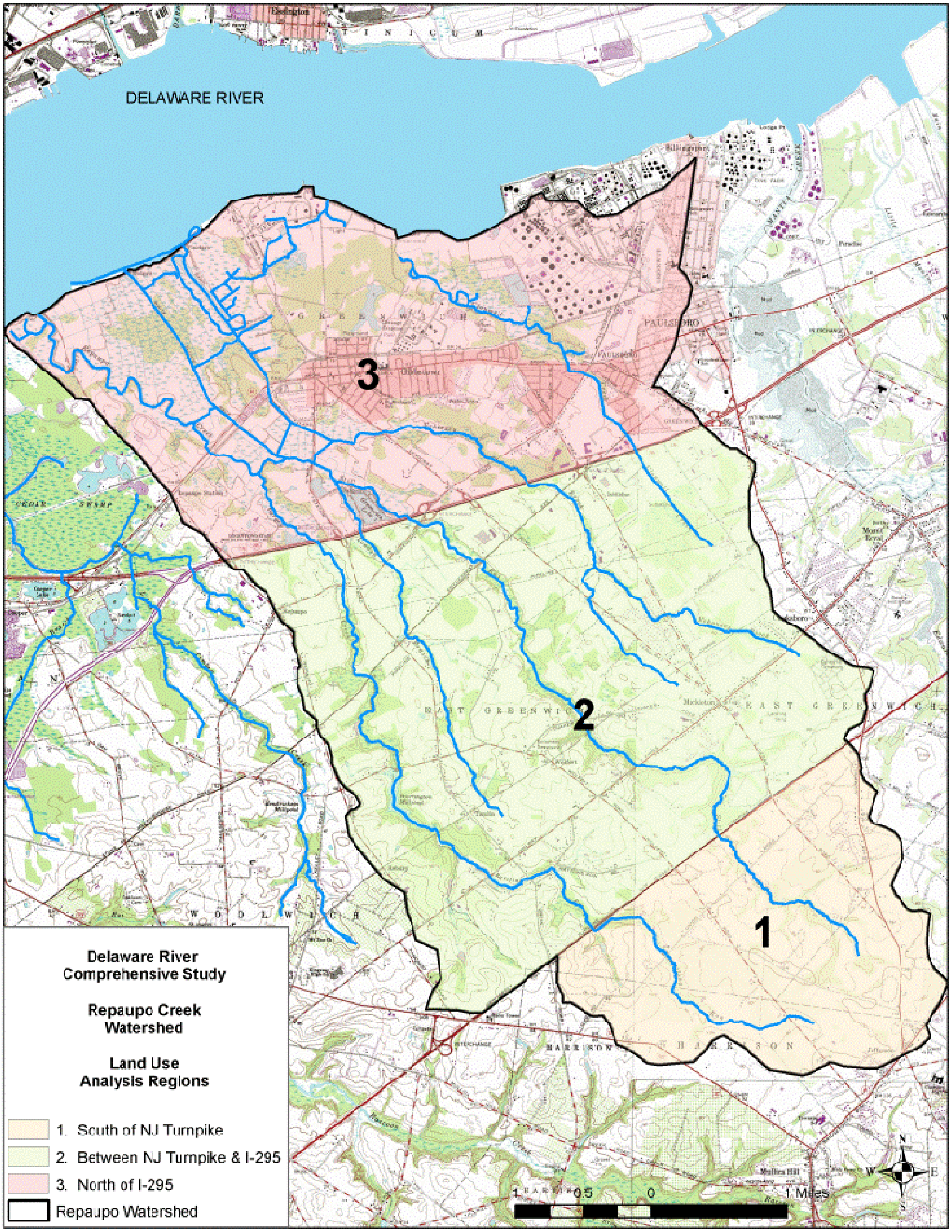

Figure A.6.2: Repaupo Creek Watershed Land Cover Analysis Regions

A.6.7. 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 models 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. NAVD 88, and rise only to +20 ft. NAVD 88 within Gibbstown and +30 ft. NAVD 88 at the former Valero Refinery. 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. NAVD 88 to +120 ft. NAVD 88. The headwaters of the watershed have elevations around + 155 ft. NAVD 88. As Figure A.6.3 shows more than half of the watershed is at elevations below +30 ft. NAVD 88.

A.6.8. Hydrologic Modeling Methods

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, 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 of the Repaupo Creek Watershed was developed to evaluate several alternatives which portrayed varied hydrologic conditions including such things as frequency-based tailwater conditions on the Delaware River and frequency-based local rainfall events over the watershed.

A hydrologic model simulates precipitation runoff and routing procedures, both natural and man-made. The essence of a hydrologic model is to transform precipitation (known) into runoff (unknown) at a given location. In this case, the runoff originating within the watershed was quantified using HEC’s Hydrologic Modeling System (HEC-HMS) version 3.5.

HEC-HMS has many capabilities for enumerating precipitation derived runoff ranging from lumped parameter, empirical unit hydrograph methods to quasi-distributed parameter, conceptual methods. HEC-HMS also has the ability to simulate runoff on an event or long term basis. For the Repaupo Creek Watershed model, a combination of lumped and conceptually-based modeling approaches was used on an event time scale. These modeling choices were made in anticipation of utilizing frequency-based precipitation to determine frequency flow rates and water surface elevations (WSEL) which would in turn be used to analyze various flood reduction measures. The individual hydrologic methods chosen to simulate the rainfall-runoff processes are discussed below.

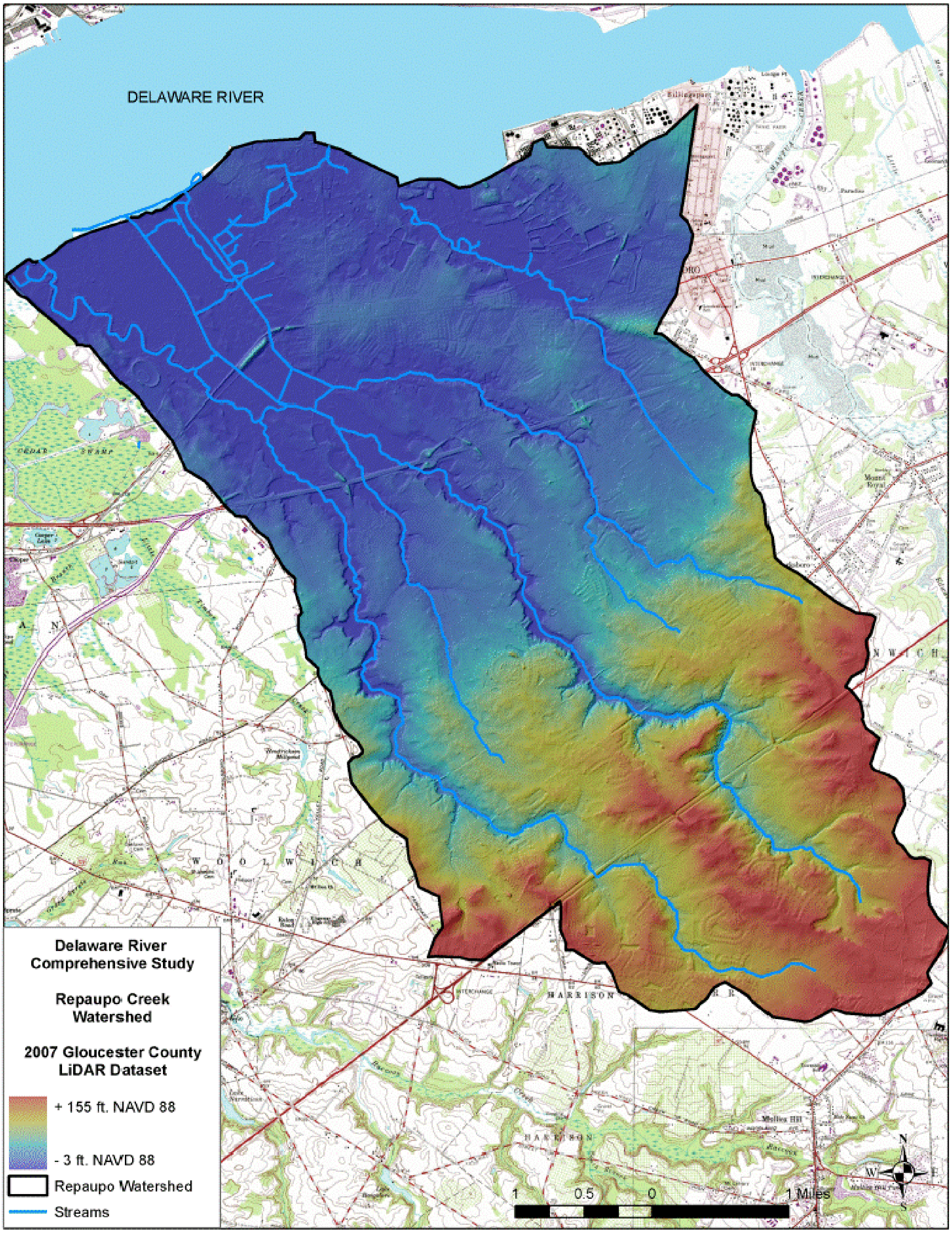

Figure A.6.3: 2007 LiDAR Topography for Repaupo Creek Watershed

Infiltration: Precipitation that falls onto pervious (non-paved) surfaces is subject to losses within HEC-HMS. These losses were quantified using the Green and Ampt infiltration routine. This method was chosen for two reasons: (1) Most robust conceptually- and event-based infiltration method within HEC-HMS; (2) Infiltration parameters can be estimated for un-gaged watersheds like this one based upon surface soil information.

Transform: To transform excess precipitation into surface runoff the SCS unit-hydrograph method was selected for the model. The SCS unit hydrograph method was originally developed for small agricultural watersheds which is what the Repaupo Creek Watershed can be described as. The DelmarVa shape SCS unit-hydrograph was selected based upon its wide acceptability in the region for small rural coastal-plain watersheds as discussed letter.

Baseflow: Baseflow within the watershed was modeled using the Exponential Recession rates in the South Jersey region.

Channel Routing: HEC-HMS has the ability to simulate hydrologic channel routing using the physically-based Muskingum-Cunge method. This channel routing method was chosen due to its ability to define eight-point cross sections. It is also arguably the most realistic method within HEC-HMS.

A.6.9. Hydrologic Model Setup

The HEC-HMS model was developed in a GIS using the program HEC-GeoHMS. HEC-GeoHMS is an extension to ArcGIS that develops a number of hydrologic modeling inputs for HEC-HMS. It analyzes digital terrain data and calculates watershed boundaries and drainage paths and transforms them into a hydrologic data structure the represents the drainage network for a given watershed. It assists in visualizing spatial information, documents watershed characteristics, and performs spatial analysis. The steps used to develop the Repaupo Creek Watershed hydrologic model are outlined below.

Data Collection: Spatial datasets utilized in the hydrologic model are summarized in Table A.14. 15. An extensive literature review was conducted for informational purposes and to supplement the collected digital data. Data was pulled from these reports and used in the hydrologic model. Data included: floodgate dimensions and elevations, overbank and channel roughness factors, and baseflow parameters. Several field work investigations were also conducted in order to ground truth some of the digital data, verify watershed delineations and current conditions of drainage flow paths, and significant drainage structures.

Terrain Preprocessing: The first step in model development was to preprocess the 2007 LiDAR topographic dataset within HEC-GeoHMS. Before the LiDAR data was used to delineate drainage paths and subbasins, it was hydrologically corrected. Due to the flat terrain of the northern portion of the watershed, an iterative process was done in order to best represent the movement of water through the watershed. The high resolution LiDAR data accurately captured

Table A.6.14: Summary of Data Sources for Repaupo Creek Watershed Hydrologic Model

Terrain	FEMA LiDAR Dataset Collected for Gloucester County, NJ (2007)
	National Elevation Data (NED) 3 meter data Compiled by USGS
	USGS 7.5-min Topographic Digital Raster Graphics
Streams	National Hydrography Dataset (NHD) Collected by USGS
	Supplemented by Digitizing Aerial Photography
Aerials	New Jersey High-Resolution Orthophotography (2007)
Street	New Jersey Roadway Network Collected by New Jersey Dept. of Transportation (2010)
Soils	The Soil Surveys Geographic Database (SSURGO) for Gloucester County, NJ Collected by USDA
Land Use	National Land Cover Database (NLCD) Collected by USGS (2006)
	NJDEP Land Use/Land Cover Datasets (1986, 1995, 2002, & 2007)
Tidal	Delaware River NOS Stations at Philadelphia, Marcus Hook, PA, Reedy Point DE, and Lewes, DE
Precipitation	Event-Based: NCDC Stations at Philadelphia International Airport, PA, Mt. Holly, NJ and Wilmington, DE
	Frequency-Based: Precipitation Data Server (NOAA Atlas 14) for Gloucester County, NJ
Stream Cross-Sections	HEC-2 Input File from the Hydraulic Analysis Used by FEMA for Floodplain Mapping of Greenwich and Logan Townships (1981)
Previous Report	Delaware River Basin, Gibbstown, NJ Flood Control, Restoration of Levee Report by Corps' of Engineers (1967)
	Hydrogeology of the Region of Greenwich Township, Gloucester County, NJ by USGS (1991)
	Preliminary Estimates of Costs and Benefits of Alternative Solutions for Flood Damage Reduction: Repaupo Creek Watershed by NRCS (1996)
	Repaupo Creek Watershed Hydrologic and Hydraulic Report by NRCS (1999)
	Upper Mantua Creek Watershed Characterization and Assessment Report by Gloucester & Camden County Soil Conservation Districts (2007)
	Repaupo Creek Watershed Floodgate Replacement Review by T&M Associates (2008)
	Reconstruction of Repaupo Creek Floodgate Plans Prepared by T&M Associates (2008)
	Raccoon Creek Watershed Characterization and Assessment Report by Gloucester & Camden County Soil Conservation Districts (2008)
	Gloucester County, NJ Multi-jurisdictional Hazard Mitigation Plan (2009)
	Methodology for Estimation of Flood Magnitude and Frequency for New Jersey Streams by USGS (2009)
	Flood Insurance Study by FEMA for Gloucester County, NJ (2010)
	South Jersey Levee Inventory Report and Database by NRCS (2010)
	Preliminary Drainage Improvement Study - Portions of Greenwich and Logan Townships by T&M Associates (2010)

many of the details in the watershed, but some additional steps were done to eliminate features that did not allow for movement of water within the watershed. These steps included eliminating local depressions or sinks so that water could move across the terrain towards the Delaware River, and additional editing in order to force proper drainage in the flat areas of the watershed. Eight additional datasets that collectively describe the drainage patterns of the watershed were derived from the hydrologically-corrected LiDAR terrain model. During this process streams and subbasins are delineated along with flow direction, flow accumulation, stream network, and stream segmentation. Based upon the 2007 LiDAR topographic dataset, the Repaupo Creek Watershed was determined to be 25.56 square miles in size (16,360 acres).

The Repaupo Creek Watershed was delineated into 51 subbasins and drainage reaches based upon several factors. These factors included: stream confluences; abrupt changes in channel, overland geometry and land use changes; the major road crossings of I-295 and NJ Turnpike; and locations of possible “with project” flood reduction measures. The assorted subbasins with the naming convention used in the model are shown in Figure A.6.4 and summarized with their areas in Table A.6.15.

Table A.6.15: Repaupo Creek HEC-HMS Subbasin Areas

Subbasin	Area (sq. mi.)	Subbasin	Area (sq. mi.)
CC1A	0.4651	RC1A	0.2318
CC1B	0.1858	RC1B	0.3982
CC2	0.4522	RC2	0.7045
CC3	0.4525	RR1A	1.2537
CC4	0.0357	RR1B	0.3982
CC5	0.7599	RR2	0.3387
CC6	0.6324	RR3	0.9480
CC7A	0.2531	RR4	0.6300
CC7B	0.1577	SD1	0.4582
CC8	0.5833	SD2	0.8591
EL1	0.2986	SR1	0.8144
EL2	0.3103	SR2	0.2833
LB1	0.7395	SR3A	0.2604
LB2	0.3373	SR3B	0.3009
LB3	0.5004	SR4A	0.7825
LB4	0.0652	SR4B	0.1848
NB1	1.1460	SR5A	1.2703
NB2	0.4764	SR5B	0.7755
NB3	0.3418	SR6	0.2936
NB4	0.5417	SR7	0.1382
NB5A	0.8780	SR8	0.2723
NB5B	0.2315	WS1A	0.1123
PC1A	0.6513	WS1B	0.1295
PC1B	1.2799	WS2A	0.0998
PC1C	0.9855	WS2B	0.4104
PC2	0.4501		

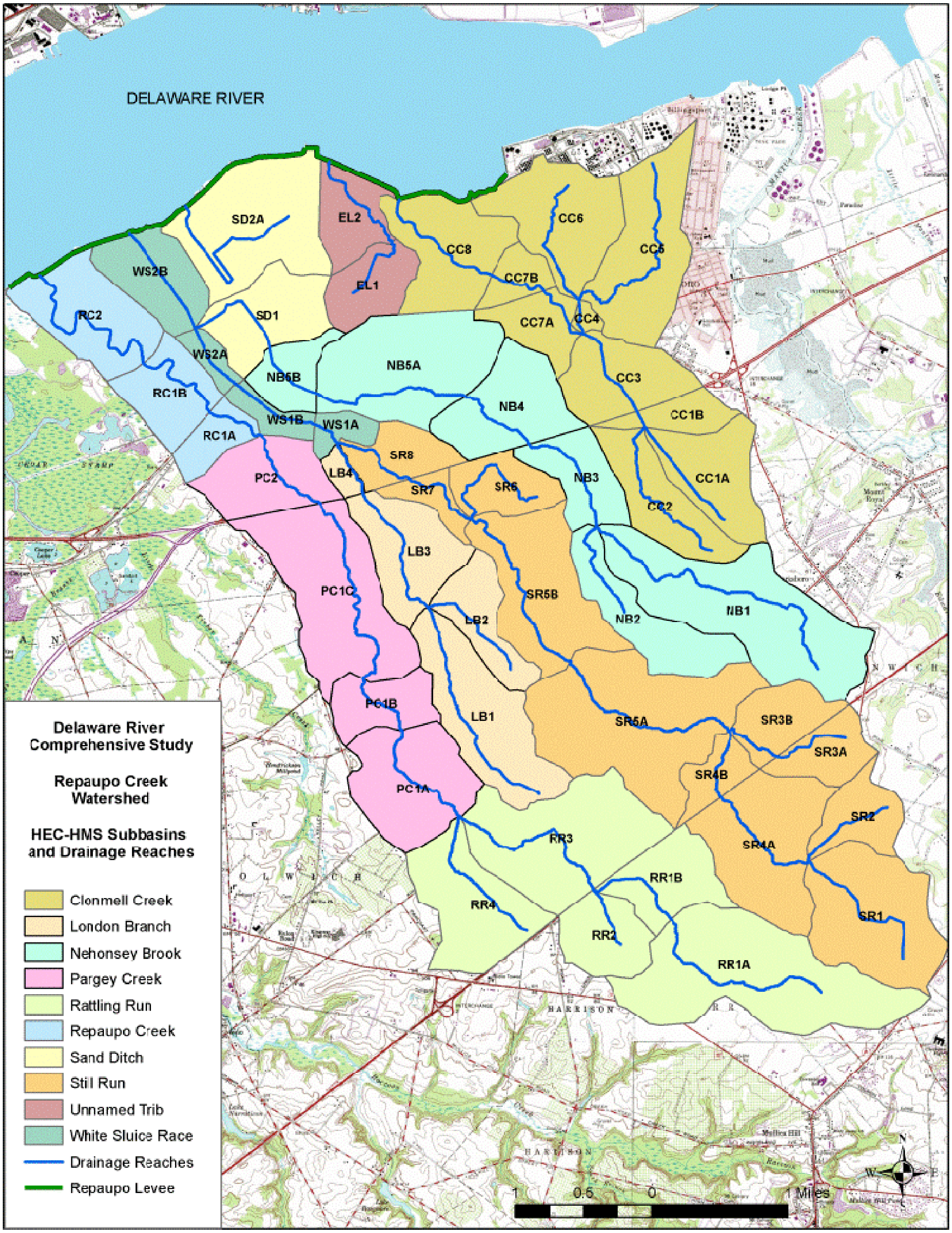

Figure A.6.4: Repaupo Creek Watershed HEC-HMS Subbasins

Infiltration Loss Parameters: Physical parameters that were required by the Green and Ampt infiltration routine for each subbasin included: Hydraulic Conductivity; Wetting Front Suction soil types using the SSURGO database previously discussed. Impervious area was estimated using the NLCD 2006 land use classification coverage and verified against the NJDEP 2007 Land Cover Land Use coverage. Infiltration calculations are ignored over the percentage of the basin where impervious areas exist. The initial soil moisture parameter was uniformly assigned a uniform value in anticipation of being used as a calibration factor.

Several different soil types were identified within the watershed. Each type was paired with suggest Green and Ampt infiltration values from EM 1110-2-1417 as shown in Table A.6.16 Utilizing the land use and soil coverages within ArcGIS, area-weighted Green and Ampt infiltration values per subbasin were calculated.

Table A.6.16: Green and Amp Infiltration Parameters by Soil Type

Description	Porosity (% of Volume)	Effective Porosity (% of Volume)	Wetting Front Soil Suction Head (in.)	Hydraulic Conductivity (in./hr.)
Loam	0.463	0.434	3.500	0.134
Loamy Sand	0.437	0.401	2.413	1.177
Sand	0.437	0.417	1.949	4.638
Sandy Loam	0.453	0.412	4.335	0.429
Silt Loam	0.501	0.486	6.567	0.256

Baseflow Parameters: Parameters that were required for the Exponential Recession baseflow routine included: initial discharge rate; recession constant; and ratio to peak. Mean annual discharge for each subbasin was estimated at 1.5 cfs per square mile. This value was obtained from the NRCS Report "Repaupo Creek Watershed Hydrologic and Hydraulics Report" (1999). An exponential decay constant of 0.5 was used for each subbasin in accordance with HEC's "Hydrologic Modeling System Technical Reference Manual". Baseflow was estimated to dominate the receding limb of the runoff hydrograph generated by a subbasin when 10% of the peak flow rate for that subbasin was reached; therefore a "ratio to peak" value of 0.1 was used for each subbasin.

Transform: The SCS Dimensionless Unit Hydrograph was used to distribute the runoff volume to a unit hydrograph. The determination of an SCS lag time was required for this method. The tools within HEC-GeoHMS were used to determine necessary hydrologic parameters needed for the transform method. The procedure within HEC-GeoHMS is consistent with the methodology of the SCS's Technical Release-55 Urban Hydrology for Small Watersheds published June 1986. The time of concentration was defined as the time required for water to travel to the subbasin outlet from the most hydraulically distant point in the subbasin (longest flow path). The longest flow path was calculated within HEC-GeoHMS based upon the 2007 LiDAR topographic dataset and divided into three segments: sheet flow; shallow concentrated flow; and channel flow. The break between sheet flow and shallow concentrated flow was no larger than 100 feet. This maximum is consistent with the latest guidance for maximum sheet flow length. The break between shallow concentrated flow and channel flow occurred where the longest flow path intersected the channel for each subbasin. These break point locations were manually edited as necessary from the locations automatically computed by HEC-GeoHMS.

Physical data were required to calculate the travel time associated with all three flow regimes, including flow length, slope, and a roughness coefficient. Composite overland flow roughness values for sheet flow and shallow concentrated flow were estimated by calculating a weighted roughness value using typical literature values for each surface condition and the flow length associated with each surface condition. The surface conditions were determined from the aerial photos and site investigations. For channel flow, travel time was calculated based on channel length and velocity associated with the 2-year 24-hour rainfall event for the watershed which was 3.26 inches. The velocity, in turn, was estimated based on channel slope and assumed flow depth and cross-sectional geometry. Slope data were calculated by using the upstream and downstream elevations and the stream length. Cross-section geometries were assigned based on review of the 2007 LiDAR topographic dataset and the wet-sections obtained from the HEC-2 hydraulic model used in the original flood insurance study for streams in the watershed.

The travel times associated with each of the three elements were added to calculate the time of concentration for each subbasin, and the lag time for a subarea was assumed to equal 0.6 times the time of concentration. This assumption is consistent with the methodology of the SCS's "Technical Release-55 Urban Hydrology for Small Watersheds" published June 1986. Lag time was used as a calibration parameter, and where necessary values were raised or lowered using engineering judgment. Any modification made to the lag times were done within the error range of all the variables used to calculate it for any given subbasin. The final lag time used for each subbasin was within 60-70% of the calculated time of concentration. Table A.6.17 summarizes the final lag times used in the hydrologic model for each subbasin.

The DelMarVa shape for the unit-hydrograph was chosen to be more appropriate for all subbasins in the watershed. The DelMarVa hydrograph has been adopted by the State of New Jersey where appropriate as of September 8,2005, and pursuant to the NRCS, Technical Bulletin NJ210-3-1. The Repaupo Creek Watershed is located in the outer coastal plain zone where the DelMarVa hydrograph has been proven in the past and is widely accepted as the most appropriate shape to use. The peak discharge computed using a DelMarVa hydrograph is smaller than the peak discharge computed by a standard shape hydrograph. This reduction in peak is more appropriate for flat watersheds such as Repaupo Creek Watershed. HEC-HMS version 3.5 allows users to select the DelMarVa hydrograph directly; older versions of HEC-HMS did not allow users to select it as a shape for the unit-hydrograph.

Table A.6.17: SCS Unit-Hydrograph Time of Concentration and Lag Time Summary by Subbasin

Subbasin Name	Time of Travel (min)	Lag Time (min)
CC1A	43.10	29.63
CC1B	36.66	24.93
CC2	49.25	34.10
CC3	64.82	42.52
CC4	29.15	20.23
CC5	53.84	32.35
CC6	62.32	39.45
CC7A	52.83	35.50
CC7B	34.99	21.2
CC8	100.82	69.59
EL1	93.09	64.76
EL2	101.28	48.49
LB1	70.57	33.72
LB2	44.21	30.68
LB3	89.82	61.69
LB4	46.23	32.45
NB1	69.73	48.06
NB2	94.92	64.9
NB3	52.93	35.45
NB4	99.91	67.22
NB5A	92.66	55.7
NB5B	47.3	32.1
PC1A	25.76	17.9
PC1B	24.82	17.3
PC1C	59.03	40.5
PC2	72.55	49.38
PC1A	40.87	24.9
PC1B	49.76	30.40
RC2	141.69	96.67
RR1A	66.68	46.3
RR1B	29.25	18.2
RR2	38.38	25.54
RR3	63.55	40.11
RR4	35.64	23.47
SD1	82.94	50.45
SD2A	100.87	69.54
SR1	54.95	33.18
SR2	29.05	17.94
SR3A	29.04	17.5

Subbasin Name	Time of Travel (min)	Lag Time (min)
SR4A	39.19	37.62
SR4B	15.19	17.3
SR5A	83.78	59.08
SR5B	35.58	23.61
SR6	54.15	36.71
SR7	25.53	17.5
SR8	102.61	70.90
WS1A	68.18	45.5
WS1B	81.82	56.70
WS2A	78.67	48.5
WS2B	99.43	68.6

Channel Routing: Physical parameters that were required by the Muskingum-Cunge routing routine for each subbasin channel include: channel length; channel slope; channel roughness. The channel length and slope were determined from HEC-GeoHMS processing of the 2007 LiDAR topographic data. Overbanks for the touting cross-sections were extracted from the LiDAR data and the wet sections which were not captured by the LiDAR data were obtained by utilizing the cross-sections used in the original HEC-2 hydraulic model developed for FEMA for floodplain delineations in Logan and Greenwich Townships. The overbanks and wet-sections were merged together to come up with an eight point cross-section used for routing purposes. Manning "n" values for the overbanks were estimated from site visits and aerial photography of the watershed, and the original HEC-2 hydraulic model was used for Manning "n" values for the channels. Manning "n" values for the left and right banks vary between 0.06 and 0.11 and for the main channel between 0.030 and 0.040. HEC-HMS does not allow negative elevations when defining a cross-section. All cross-sections were elevated by a fixed amount which avoided having any negative values. The fixed amount was typically the maximum depth at the most downstream cross-section for each stream.

Floodgate Structures: The hydrologic model included the five floodgate structures at the Gibbstown Levee for Repaupo Creek, White Sluice Race, Sand Ditch, El Sluice floodgate on the DuPont property and Clonmell Creek. The floodgates discharge fluvial flow from the streams and ditches to the Delaware River when water surface elevations in the streams are higher than the tidal elevation in the river. Also, the floodgates prevent tidal flows and storm surges from getting into streams and natural storage area leeward of the levee. Data for the older floodgates of White Sluice Race, Sand Ditch, El Sluice, and Clonmell were obtained from a literature review. Plans for the new Repaupo Creek Floodgate, which was completed in 2008, were provided by the project's engineering firm (T&M Engineering). Data needed by the hydrologic model included: number of gates; gate dimensions; and opening elevations.

Outflow through the floodgates was computed in the hydrologic model by using the orifice equation. By using the orifice equation it was assumed that the outlets were fully submerged for all time steps. Table A.6.18 summarizes the floodgate parameters used in hydrologic model.

Table A.6.18: Gibbstown Levee Floodgates Geometry

Floodgate	Number of Gate Openings	Center Elevation (ft. NAVD 88)	Area Per Opening (sq. ft.)	Discharge Coefficient
Repaupo Creek	3	-5	40	0.6
White Sluice Race	3	-6.1	28	0.6
Sand Ditch	4	-1.7	12.75	0.6
El Sluice	3	-4.85	9.62	0.6
Clonmell	3	-2.1	9.62	0.6

Cedar Swamp: West of the Repaupo Creek Watershed is Cedar Swamp and Klondike Ditch. Cedar Swamp is a tidal marsh and is subject to inflows from the Delaware River since there is no floodgate structure on Klondike Ditch. It fills as tide increases and empties back into the Delaware River as tide decreases. The Little Timber Creek drains into Cedar Swamp which in turn flows out to the Delaware River through Klondike Ditch. It has been observed in the past that during extreme storm events, Cedar Swamp has a secondary outlet through the Godwin Pump Facility Property and ultimately into the Repaupo Creek Watershed. Historically, the connection between Cedar Swamp and the Repaupo Creek Watershed was blocked by a series of internal berms on the Godwin Pump facility. It has been reported that portions of these internal berms washed out in April and July 2005 due to high water events during those months. A subsequent inspection done by Corps' of Engineers personnel in November 2005 found them to be overgrown with trees and other vegetation and that they were barely discernable or non-existent in some areas. The berms were found to be no more than 3 feet high, as well. Floodgate Road which runs north and south and borders the Godwin Pump Facility has historically been the western edge of the Repaupo Creek Watershed. Topography in the area is very flat, and it has been observed in the past that water can flow in either direction at Floodgate Road depending upon tidal and storm conditions. Upon review of this information, a HEC-RAS model was developed in order to estimate an inflow hydrograph to the Cedar Swamp storage area from tidal flows on the Delaware River. The hydrograph computed in HEC-RAS was then used in the HEC-HMS hydrologic model as an inflow hydrograph to the Cedar Swamp storage area. Based upon the hydrograph computed in HEC-RAS and the storage-elevation curve developed for Cedar Swamp (based upon the 2007 LiDAR dataset), two outflow hydrographs from Cedar Swamp were computed in HEC-HMS. One outflow hydrograph was directed back to the Delaware River based upon timing of the tidal signal, and the other served as inflow into the Repaupo Creek Watershed across Floodgate Road. This scenario of an inflow to the Repaupo Watershed from Cedar Swamp was added as an alternative to investigate in the hydrologic model. The subsequent effects on interior ponding elevations behind the Gibbstown Levee were quantified and compared to the ponding elevations behind the levee when flow from Cedar Swamp was not allowed within the Repaupo Creek Watershed.

Interior Storage: Leeward of the Gibbstown levee is a large wetland area that serves as a natural storage area for any tidal flooding from the Delaware River or runoff from the headwaters of the watershed. Storage-elevation curves were developed for this area up to elevation +5.0 ft. NAVD 88 using the 2007 LiDAR topographic dataset. The surface volume routine within ArcGIS was used which calculates the area and volume of a raster or tin surface above or below a given reference plane. Initially, the larger storage area was divided into several smaller ones for the hydrologic model. The smaller storage areas were: (1) Repaupo Creek/White Sluice Race; (2) Sand Ditch; (3) El Sluice Floodgate; and (4) Clonmell Creek. Repaupo Creek and White Sluice Race are interconnected by several ditches and there are no significant topographic high points between the two streams; so it was assumed that flow from these two streams and their subbasins would be merged into a single storage area. Additional storage and flow from Sand Ditch and its subbasins into the Repaupo/White Sluice storage area was assumed to be minimal for the hydrologic model because of several factors. These factors included: there is a line of small internal levees with top elevations of +2.0 ft. to +4.0 ft. NAVD 88 that run in between Sand Ditch and White Sluice Race, a dam built by DuPont that diverts flow from Nehonsey Brook to

White Sluice Race, and that historically DuPont has pumped into the Delaware River during extreme events. The same assumption of insignificant flow and storage area was also made for the EL Sluice Floodgate storage area. Surface water in the EL Sluice drainage area is primarily storm water runoff from the DuPont property and tide swell from the Delaware River. Its minimal area was kept separate from the main Repaupo/White Sluice storage area. Also a storage elevation curve was developed for Clonmell Creek storage area. The hydrologic model assumed that due to the topographic divides and distance between the Clonmell Creek and the Repaupo Creek/White Sluice storage areas they are prevented from merging into a single larger storage area. As previously mentioned in order to evaluate the effects of Cedar Swamp on ponding elevations leeward of the Gibbstown Levee, an additional storage area was calculated.

A.6.10. Development of Inflow Hydrograph from Cedar Swamp

It has been observed in the past that probably due to the disrepair of several internal small dikes on the Godwin Pump Property, flow can enter the Repaupo Watershed through the property from Cedar Swamp and over Floodgate Road. In order to replicate this process of flow coming from the Delaware River through Klondike Ditch; into Cedar Swamp and ultimately into the Repaupo Watershed, a hydraulic model was developed using HEC-RAS 4.1. The HEC-RAS model consisted of two miles of Klondike Ditch and Little Timber Creek, eleven cross-sections, and an internal storage area representing Cedar Swamp within the model limits.

Topographic data for hydraulic model were obtained from two sources; the 2007 Gloucester County LiDAR dataset, and a 1981 bathymetric survey done of Klondike Ditch retrieved from the National Ocean Service (NOS)'s online GEODAS database of survey data. These sources were combined to create a representation of the ground surface for the hydraulic analysis. Horizontal projections were referenced to NAD 83 and New Jersey State Plane Coordinate system. Vertical elevations were referenced to NAVD 88.

Eleven hydraulic cross-sections were cut from the topographic data for the HEC-RAS model. Spacing between cross-sections varied from 1,400 feet down to 400 feet. Cross-section alignment was created by drawing sections in HEC-RAS from the left overbank to the right overbank looking downstream using an aerial photograph from 2007 as reference. They were placed at right angles to the anticipated direction of flow in both Klondike Ditch and the overbank areas. The NOS bathymetric survey data were used to develop the channel portion of the cross-section geometry and the 2007 LiDAR dataset was the source of the overbank topography. Figure A.6.5 shows the cross section layout map within the modeled area.

Manning's coefficient values (n-values) for Klondike Ditch and overbanks at each cross-section were estimated based on values used for other streams and overbank areas at nearby locations and aerial photographs. Manning n-values of 0.028 for Klondike Ditch and 0.085 for the overbanks were used in the model.

The model's unsteady flow boundary conditions consisted of downstream starting water surface elevations of stage hydrographs of the Delaware River. Eight stage hydrographs (tidal signals) from the 50% (2-year) to the 0.2% (500-year) were simulated in the model. The same storm tidal signals were used for both HEC-RAS and HEC-HMS models. It was assumed for the model that there was no lateral inflow to the Cedar Swamp storage area from other sources at the upstream boundary of the model. This assumption was made based on the fact that the contributing drainage area upstream to Cedar Swamp is relatively small and that tidal flows filling the swamp are more significant than drainage to the swamp from upstream sources. An initial water surface elevation of -1.5 ft. NAVD 88 as assumed for the Cedar Swamp storage area. This elevation is consistent with the starting conditions used in the HEC-HMS model. Inflow hydrographs were generated for the Cedar Swamp storage area for the eight Delaware River stage frequencies. Inflow and outflow to/from Cedar Swamp storage area occurred depending on the tidal signal. As tides on the Delaware River increase, flow enters Klondike Ditch and into Cedar Swamp. When the tides decrease, flow changes direction and Cedar Swamp drains back into the Delaware River through Klondike Ditch.

Figure A.6.5: Cross-Section Layout of Cedar Swamp HEC-RAS Model

For the HEC-HMS model the time-series was split into two separate hydrographs. One represented inflow into the Cedar Swamp storage area from Klondike Ditch and the Delaware River, and the other represented outflow from Cedar Swamp back into Klondike Ditch and the Delaware River.

A.6.11. Hydrologic Model Calibration

The calibration/validation process included qualitative data from the frequency discharges computed from the original flood insurance study HEC-2 hydraulic model done by FEMA. As stated in FEMA's 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. The hydrology for Mantua Creek was developed using the two USGS stream gaging stations and the "Generalized Skew Study for the State of New Jersey".

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. Table A.6.19 compares peak discharges at several locations computed by the HEC-HMS hydrologic model against the corresponding peak discharges found in the 2010 Flood Insurance Study for the 2% (50-yr), 1% (100-yr), and 0.2% (500-yr) ACE events, respectively. As the table shows, 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.

Event-based simulations were also conducted for the three storms previously mentioned and the results at several locations were qualitatively compared to observations made by local officials, newspaper reports, and police reports describing flooded roads and properties. Lastly, input parameters were compared to parameters used in nearby hydrologic models for the Raccoon and Mantua Creek Watersheds. These watersheds are very similar to the Repaupo Creek Watershed in Gloucester County, NJ.

Parametric changes made to the hydrologic model to achieve the desired level of calibration included: varying hydraulic conductivity per subbasin by +/- 25% where necessary; altering the assumed initial moisture content; varying the impervious area as necessary to account for connected areas; subbasin lag time; and manning "n" values used for the channel routing reaches as necessary. Any changes made to the initial parameter estimates were done a reasonable manner and were within the error of the calculated parameter.

Table A.6.19: Hydrologic Model Peak Discharge Comparison Against Flood Insurance Study Hydrology

			2% ACE (50-yr)	1% ACE (100-yr)	0.2% ACE (500-yr)
Location	Source	Drainage Area (sq. mi.)	Peak Discharge (cfs)	Peak Discharge (cfs)	Peak Discharge (cfs)
Repaupo Creek @ Delaware River	Model	8.27	1,580	1,815	2,617
	From FIS	7.00	1,275	1,825	4,030
Pargey Creek @ I-295	Model	6.49	1,611	2,124	2,927
	From FIS	4.50	910	1,305	2,890
White Sluice Race @ Deb's Ditch	Model	11.84	2,714	3,159	4,357
	From FIS	12.00	1,905	2,725	6,025
White Sluice Race @ Nehonsey Brook	Model	7.36	2,056	2,436	3,382
	From FIS	7.60	1,330	1,905	4,220
Nehonsey Brook @ White Sluice Race	Model	4.07	817	958	1,358
	From FIS	3.90	825	1,180	2,610
Nehonsey Brook @ Tomlin Station Rd.	Model	1.96	861	997	1,333
	From FIS	3.30	715	1,025	2,265
Still Run @ London Branch	Model	5.38	1,351	1,756	3,753
	From FIS	5.60	1,070	1,535	3,400
London Branch @ Still Run	Model	1.64	640	728	992
	From FIS	1.60	425	610	1,355
Clonmell Creek @ Delaware River	Model	3.98	1,001	1,176	2,176
	From FIS	4.00	830	1,190	2,630
Clonmell Creek @ I-295	Model	1.10	380	438	974
	From FIS	0.90	260	375	840

A.6.12. Hydrologic Model Alternatives

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

Delaware River Stage: Eight different annual chances of exceedance events from a 50% ACE (2-year) to 0.2% (500-yr) were simulated. The eight frequency tidal events assumed a 6.1 foot tidal range with a peak corresponding to the Delaware River Stage Frequency adopted for the study. Also, a ninth alternative representing a “normal” tidal signal with a 6.1 foot tidal range from mean high water (MHW) down to mean low water (MLW) was simulated.

Local Rainfall Event: Nine different annual chance of exceedance rain events from a 99% ACE (1-year) to 0.2% (500-yr) were simulated based upon values obtained from NOAA Atlas 14 as previously mentioned.

Cedar Swamp Inflow: An additional 72 simulations were done in order to represent potential inflow into the Repaupo Creek Watershed from Cedar Swamp. The eight Delaware River annual chances of exceedance events in conjunction with the nine local precipitation annual chance of exceedance events made up the 72 simulations. Comparisons of ponding elevations in the natural storage area for Repaupo Creek and White Sluice Race for these simulations “with” versus “without” cedar swamp inflow were made.

Closed Floodgates: Nine simulations representing the nine different annual chance of exceedance rain events with the five Gibbstown Levee floodgates closed for 5 days were also simulated. This alternative represented a “worst-case” scenario of a maximum ponding elevation in the natural storage area. The 5 day duration was selected based upon analysis of historical long-duration storm tidal events for the Delaware River. It was assumed for this alternative that the Delaware River stages were higher than the interior water surface elevations for the entire simulation resulting in the floodgates to be closed.

Initial Water Surface Elevation for Interior Ponds: The initial water surface for the interior pond areas was varied from -2.0 ft. NAVD 88 to +2.0 ft. NAVD 88 in order to evaluate impacts to the peak ponding elevations.

Timing of Peak Delaware River Stage versus Precipitation Event: The timing of the Delaware River tidal signal in relation to the 24-hour frequency precipitation events was also examined. Floodgate openings and the resultant outflow from the interior pond areas were compared when peak tidal elevations coincided with peak precipitation and when peak tidal elevations lagged behind peak precipitation by 12, 18, and 24 hours.

The time durations for each simulation varied from five days to ten days, depending upon how long it took for the interior ponds leeward of the Gibbstown Levee to drain down to the initial water surface elevation of -1.5 ft. NAVD 88 through the floodgates. Based upon these alternatives and sensitivity simulations a final single interior pond stage frequency was derived for the “without” project conditions.

A.6.13. Hydrologic Model Alternatives Results

An initial water surface elevation of -1.5 ft. NAVD 88 was used for the interior pond areas. This elevation is a typical water surface elevation for Repaupo Creek and White Sluice Race under “normal” conditions. Sensitivity simulations showed that by increasing the water surface elevation for Repaupo Creek and White Sluice Race by 3.5 ft. to +2.0 ft. NAVD 88 within the HEC-HMS model increased the peak ponding elevation by only 1 ft for the 1% ACE (100-yr) precipitation event in conjunction with a 1% ACE (100-yr) Delaware River tailwater.

A conservative assumption was made to lag the peak Delaware River tidal elevation by 12 hours behind the 24-hour precipitation events. A series of sensitivity runs were done and the results showed that the floodgate openings and resultant outflow from the ponding areas was the smallest when the tidal signal lagged the precipitation by 12 hours rather than when they coincided or lagged by greater than 12 hours.

Upon completion of the sensitivity runs, the alternatives outlined in the previous section were simulated. The HEC-HMS hydrologic model computed the following for the interior areas leeward of the Gibbstown Levee:

- Peak interior pond elevation.
- Peak interior pond storage volume.
- Peak outflow through the floodgates.
- Total outflow volumes through the floodgates.
- Peak inflow to the interior areas from the Repaupo Creek Watershed.
- Total inflow volume to the interior areas from the Repaupo Creek Watershed
- Time to drain pool elevation in the interior areas back to initial conditions.

Outflows from the interior ponding areas and the durations to drain the interior ponding areas assumed no pumping over the levee. Computed outflows and times based upon capacity of floodgate openings only with Delaware River tailwater effects.

Results of the HEC-HMS of the interior areas indicates that the peak ponding elevations do not vary greatly at only a 0.5 ft. difference between the low tailwater condition of 50% ACE (2-yr) and a high 0.2 ACE (500-yr) tailwater condition. However; once inflow from Cedar Swamp is accounted for in the model, the spread between the various tailwater conditions increases to 2.7 ft. This increase in the spread can be attributed to the fact that there is no floodgate on Klondike Ditch; and the interior area is therefore subjected to the varying tidal heights of the Delaware River. The spread in elevation is more pronounced for the more frequent precipitation events because there is a higher interior tailwater effect for the less frequent precipitation events.

The highest interior ponding elevation with operational floodgates was calculated to be 2.84 ft. NAVD 88 with no inflow from Cedar Swamp for the 0.02% (500-yr) precipitation event in conjunction with a 0.02% (500-yr) Delaware River stage frequency. This peak elevation increased to 4.59 by the additional inflow from Cedar Swamp. The largest impact of inflow from Cedar Swamp occurred during the 99% ACE (1-yr) precipitation event in conjunction with a high Delaware River tailwater of 0.2% ACE (500-yr). Again the reason for this was tailwater effects within the interior pond area limiting flow coming across Floodgate Road for the high precipitation events.

Pumping over the levee was not assumed for any these cases, and outflows from the interior areas were based upon the capacities of the floodgates themselves. It was calculated that the longest time to drain the Repaupo Creek and White Sluice Race interior storage area peaked at approximately 7.5 days. That was under the conditions of a 0.2% (500-yr) precipitation event in conjunction with a 0.2% (500-yr) Delaware River stage frequency with no inflow from Cedar Swamp. The corresponding inflow from Cedar Swamp for the same scenario added a half of a day to the calculated time.

The Clonmell Creek interior storage area had a calculated peak elevation of 5.28 ft. NAVD 88 for the 0.2% (500-yr) precipitation event and the matching Delaware River stage frequency. It was assumed any inflow from Cedar Swamp would not impact the interior storage area associated with Clonmell Creek due to the distance and higher ground between the two areas. Since the Clonmell Creek floodgate is smaller than the Repaupo and White Sluice floodgates it took a longer time to drain back to initial conditions. The longest calculated time was 9.5 days.

A final stage 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 exceedance storm (100-yr) conditions was adopted for the final interior ponding elevation stage frequency. Table A.6.20 summarizes the final "without" project interior pond stage frequency curves for the two areas examined; Repaupo Creek/White Sluice Race and Clonmell Creek.

Table A.6.20: Final "Without" Project Interior Pond Stage Frequency Curves

Peak Interior Pond Elevation (feet NAVD 88)		Repaupo Creek / White Sluice Race	Clonmell Creek
Repaupo Watershed Precipitation Event	99% ACE (1-yr)	0.35	1.00
	50% ACE (2-yr)	0.58	1.47
	20% ACE (5-yr)	1.10	2.18
	10% ACE (10-yr)	1.44	2.65
	4% ACE (25-yr)	1.86	3.23
	2% ACE (50-yr)	2.16	3.64
	1% ACE (100-yr)	2.46	4.06
	0.4% ACE (250-yr)	2.87	4.61
	0.2% ACE (500-yr)	3.16	5.04

Time-series graphs for each frequency precipitation event were generated for the Repaupo/White Sluice and Clonmell HEC-HMS interior pond elements within the program HEC-DSSVue. HEC-DSSVue is a program that stored the time-series results generated by HEC-HMS for each alternative simulated for each element in the HEC-HMS hydrologic model.

The peak interior pond elevation occurs several hours after the peak 24-hour precipitation and storm tides pass. Post storm, the ponding elevation deceases in a "step-like" fashion.

As the floodgates open during times of lower tide ponding decreases, but once the tide level increases the floodgates close and the ponding elevation remains static or increases slightly until the tides get lower again.

A.7.0. FUTURE WITHOUT PROJECT HYDROLOGY FOR REPAUPO CREEK WATERSHED

A.7.1. Future Without Project Hydrologic Model Assumptions

A future “without” project analysis was done based upon the hydrologic model developed for the base year. Modifications to the model were made based upon assumed future watershed conditions in the year 2065. The assumed conditions were as follows:

- Future Delaware River Stage Frequencies were simulated based upon three different sea-level rise projections as outlined in guidance document EC 1165-2-211 (most recent guidance document at the time of analysis) (a) a Low Rate based upon the historical rate calculated at nearby tidal stations; (b) an Intermediate Rate based upon the Modified NRC Curve I in the guidance; and (c) a High Rate based upon the Modified NRC Curve III in the guidance.
- Continuation of percent imperviousness increases as appropriate in subbasins that exhibited increased development from 1995 to 2007 based upon the annual rate of change computed from the NJDEP land use/land cover datasets.
- Assumed no decreases in percent imperviousness for any subbasin.
- Assumed that the annual rate of change in percent imperviousness would cease after 20 years and not continue for a full 50 years to the year 2065 due to Local and or State intervention.

A.7.2. Potential Future Sea-Level Rise Trends

An analysis of future potential magnitudes of sea-level rise was conducted following the guidelines set forth in EC 1165-2-211 as previously mentioned. What effect higher relative sea-level rise rates could have on design alternatives, economic and environmental evaluation, and risk were considered in the study for the Gibbstown area. A low, intermediate and high rate as set forth in EC 1165-2-211 were calculated at the Philadelphia, PA, Reedy Point, DE, and Lewes, DE tidal stations as mentioned above.

NOAA has published monthly mean sea-level historical trends without seasonal fluctuations for Philadelphia, PA, Reedy Point DE, and Lewes, DE. These trends can be seen at http://tidesandcurrents.noaa.gov/sltrends/sltrends.shtml. At the Philadelphia station, the mean sea-level trend is 2.79 mm/yr with a 95% confidence interval of +/- 0.21 mm/yr based on monthly mean sea-level data from 1900 to 2006. This trend is equivalent to a change of 0.92 feet in 100 years. At the Reedy Point station, the mean sea-level trend is 3.46 mm/yr with a 95% confidence interval of +/- 0.66 mm/yr based on monthly mean sea-level data from 1956 to 2006. This trend is equivalent to a change of 1.14 feet in 100 years. At the Lewes station, the mean sea- level trend is 3.20 mm/yr with a 95% confidence interval of +/- 0.28 mm/yr based on monthly mean sea-level data from 1919 to 2006. This trend is equivalent to a change of 1.05 feet in 100 years. The NOAA published historical sea-level rise rates at the nearby stations were used to derive the value of 2.87 mm/year at the Repaupo Creek confluence with the Delaware River.

Based upon guidance in EC 1165-2-211, both an “intermediate” and “high” accelerated rate of local sea-level change was calculated at the nearby tidal stations. The “intermediate” rates of local mean sea-level change using the modified NRC Curve I and equations 2 and 3 in Appendix B in EC 1165-2-211 were derived for the Philadelphia, Reedy Point, and Lewes tidal stations. Consideration was given to both the most recent IPCC projections and the modified NRC projections. Likewise, estimates of the “high” rate of local sea-level change using the modified NRC Curve III and equations 2 and 3 in Appendix B were also calculated for the Philadelphia, Reedy Point, and Lewes tidal stations. Considerations were given to both the most recent IPCC projections and the modified NRC projections and were added to the local rate of vertical land movement. Table A.7.1 summarize the cumulative average rate of change of the three different annual sea-level rise projections for each tidal station out to year 2065.

Future sea-level rise projections near the Gibbstown area along the Delaware River were then derived from the nearby tidal stations. The rise in mean sea-level that follows the intermediate rate was calculated to be 5.41 mm/year and the rise in mean sea-level that follows the high rate was calculated to be 13.72 mm/year. Table A.7.1 summarizes the three different annual sea-level rise projections calculated at Repaupo Creek on Delaware River out to year 2065. Graphs of the three different projections near the Gibbstown area on the Delaware River are shown in Figures A.7.1 through A.7.4.

Table A.7.1: Annual Sea-Level Rise Projections

	Cumulative to Year 2065		
NOAA Tidal Station	**Low Rate (mm)**	**Intermediate Rate (mm)**	**High Rate (mm)**
8545240 Philadelphia, PA	2.79	5.34	13.64
8551910 Reedy Point, DE	3.46	6.01	14.31
8557380 Lewes, DE	3.20	5.75	14.05
Delaware River at Repaupo Creek	2.87	5.41	13.72

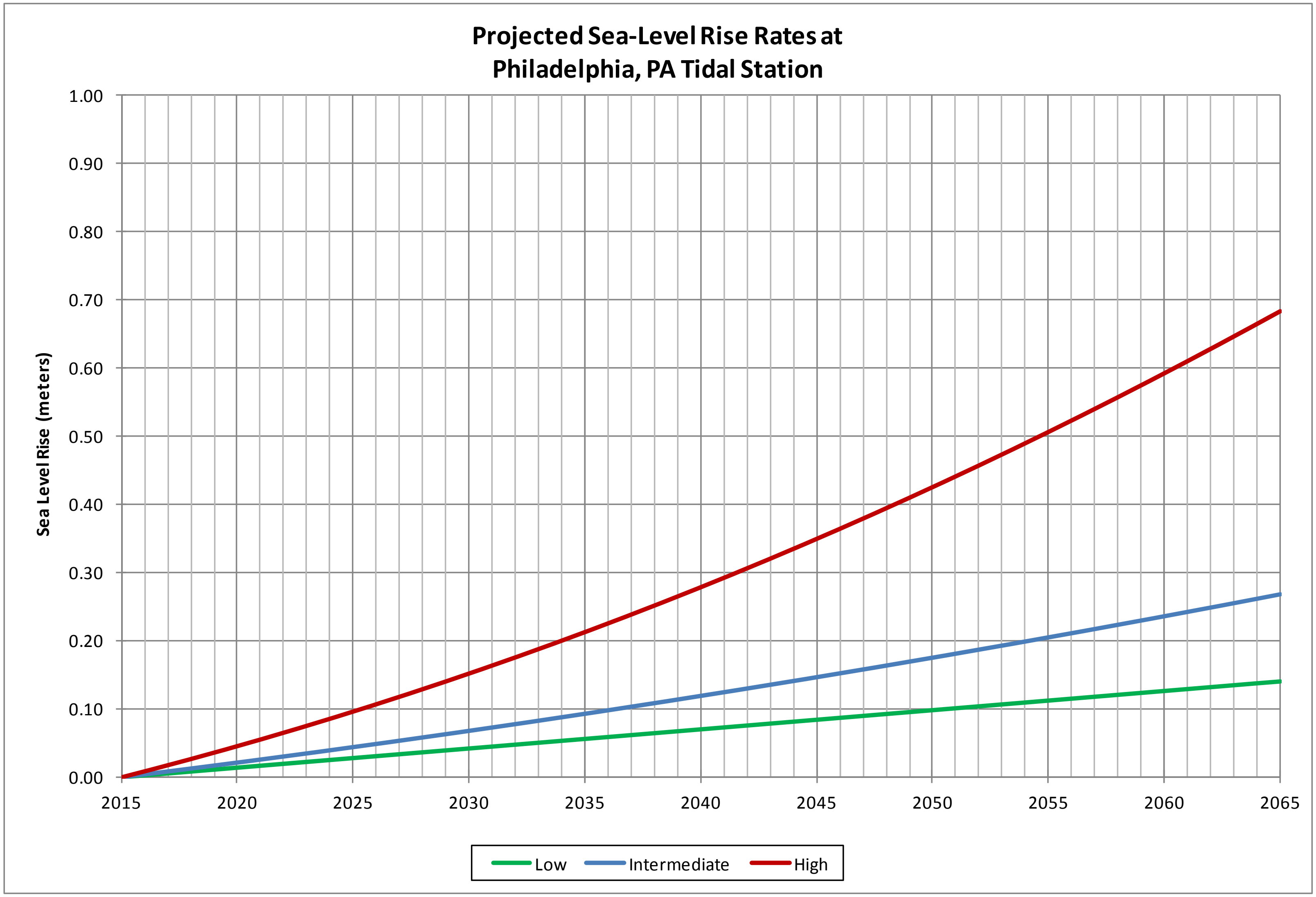

Figure A.7.1: Projected Sea-Level Rise Rates at Philadelphia, PA Tidal Station

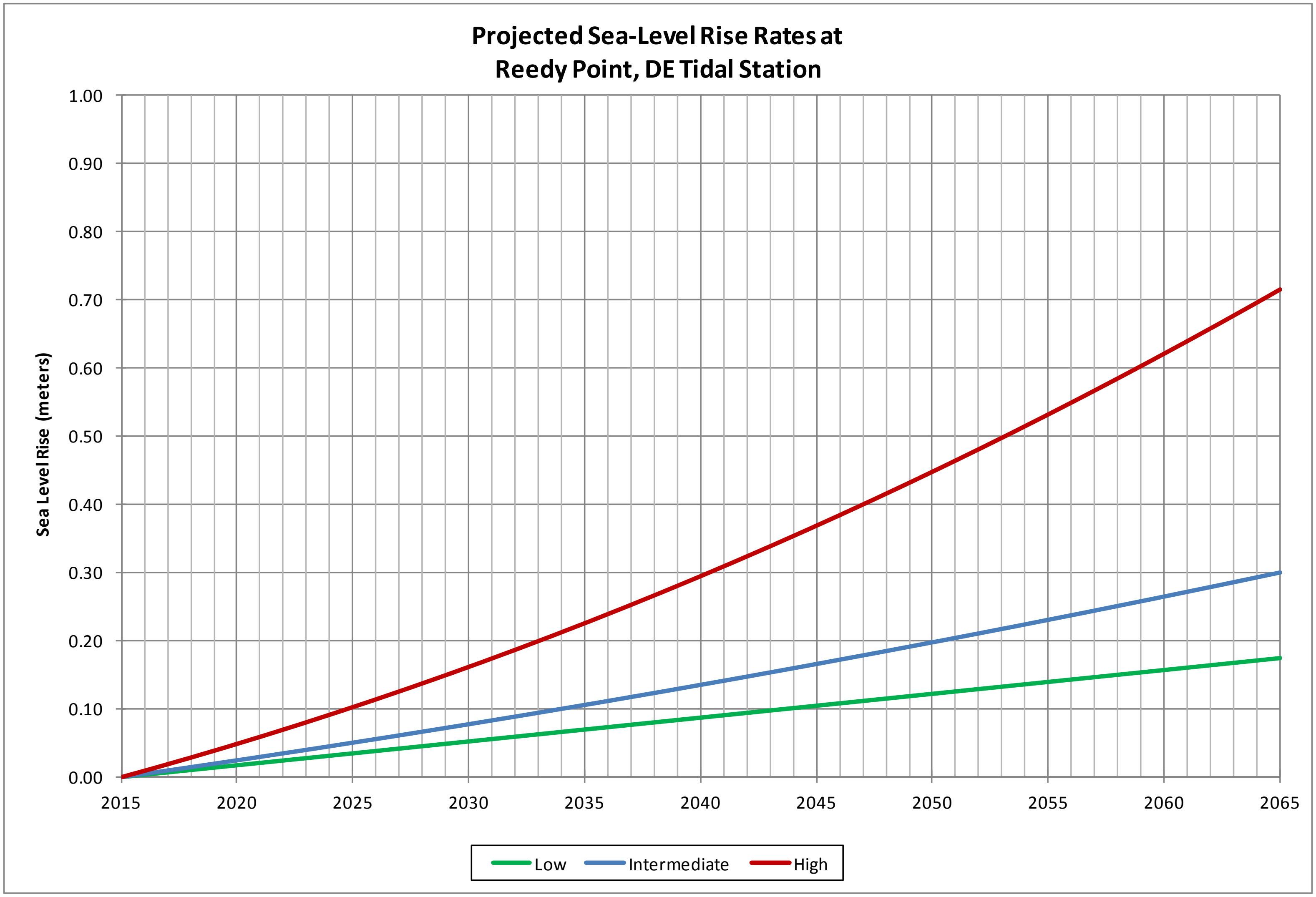

Figure A.7.2: Projected Sea Level Rise Rates at Reedy Point, DE Tidal Station

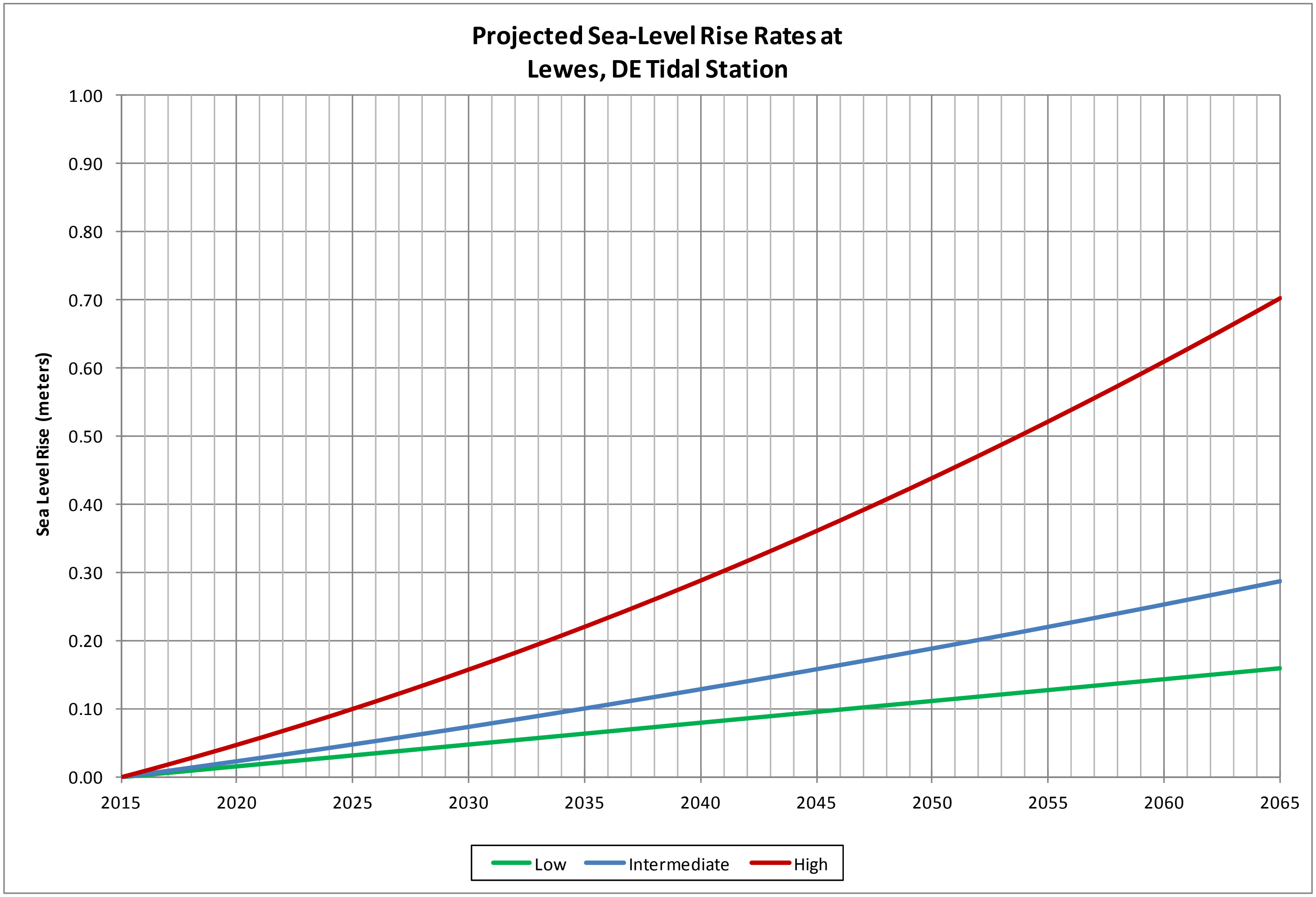

Figure A.7.3: Projected Sea-Level Rise Rates at Lewes, DE Tidal Station

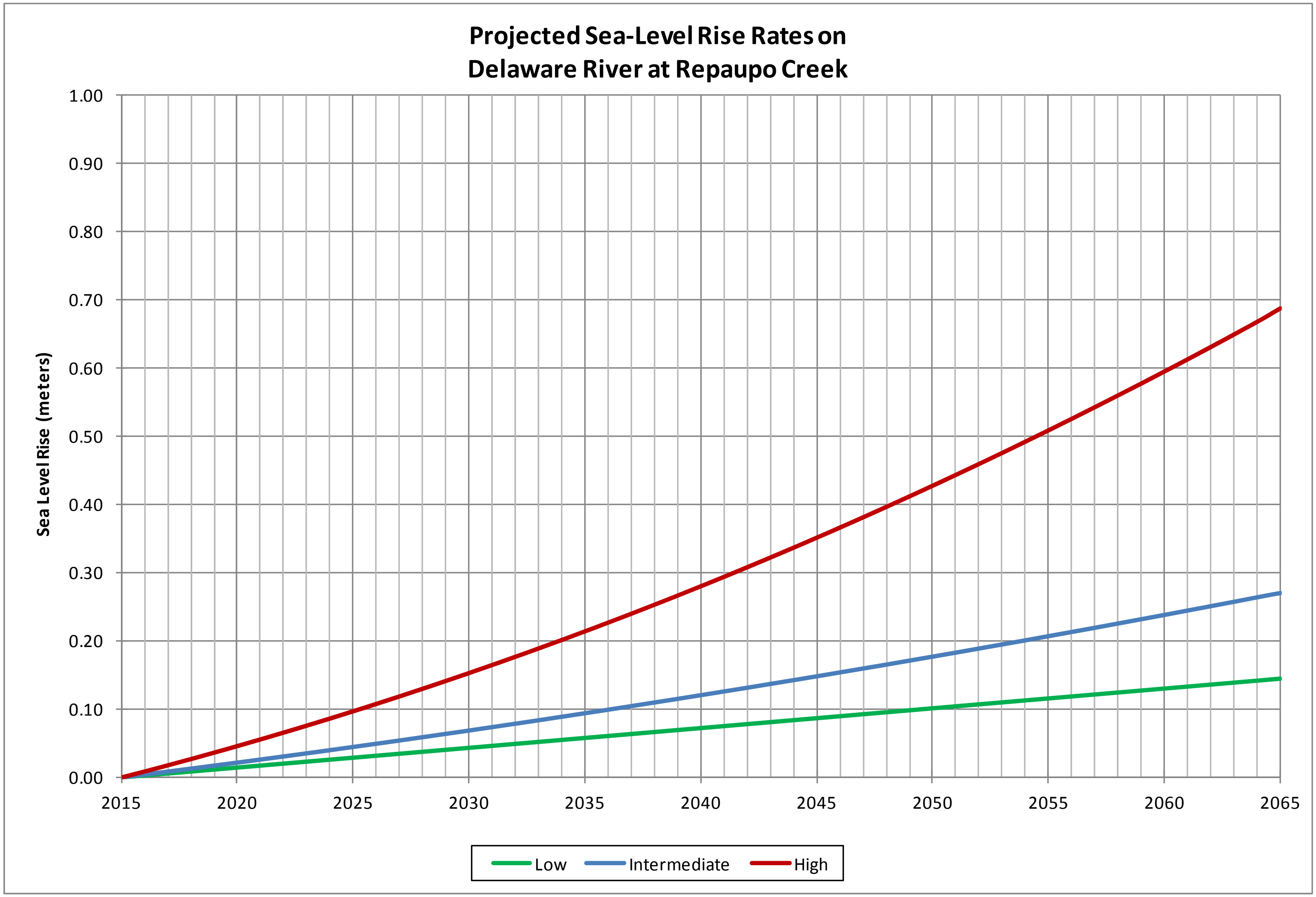

Figure A.7.4: Projected Sea-Level Rise Rates on Delaware River at Repaupo Creek

A.7.3. Future Stage Frequency Curves for Delaware River

The stage frequency for existing conditions adopted for the Delaware River near Gibbstown was based upon an analysis completed by NOAA of nearby long-term tide gages on the Delaware River at Philadelphia PA, and Lewes DE up to year 2006. The stage frequency curves developed for existing conditions were modified accordingly based upon the annual sea level rise estimates projected out 50 years for each future projection as shown in Tables A.7.2-A.7.3. The adopted stage frequencies for the three future sea-level rise projections were then computed based upon the adjacent tidal stations on the Delaware River. The stage frequencies for the three different sea level projections are shown in Tables A.7.4-A.7.9.

Table A.7.2: Stage Frequency at Repaupo Creek Based Upon Low Rate

Event	ACE	RM 82	RM 82.5	RM 83	RM 83.5	RM 84	RM 84.5	RM 85
2-year	50%	5.93	5.93	5.94	5.95	5.95	5.96	5.96
5-year	20%	6.51	6.51	6.52	6.53	6.53	6.54	6.54
10-year	10%	6.87	6.88	6.89	6.89	6.90	6.90	6.91
25-year	4%	7.35	7.35	7.36	7.37	7.37	7.38	7.38
50-year	2%	7.69	7.70	7.70	7.71	7.72	7.72	7.73
100-year	1%	8.30	8.31	8.31	8.32	8.33	8.33	8.34
250-year	0.40%	9.79	9.79	9.80	9.80	9.81	9.81	9.82
500-year	0.20%	10.93	10.93	10.94	10.94	10.95	10.95	10.96

Datum: feet NAVD 88
RM = River Mile

Table A.7.2 (Continued): Stage Frequency at Repaupo Creek Based Upon Low Rate

Event	ACE	RM 85.5	RM 86	RM 86.5	RM 87	RM 87.5	RM 88	RM 88.5
2-year	50%	5.97	5.98	5.98	5.99	5.99	6.00	6.01
5-year	20%	6.55	6.56	6.56	6.57	6.58	6.58	6.59
10-year	10%	6.92	6.92	6.93	6.94	6.94	6.95	6.95
25-year	4%	7.39	7.40	7.40	7.41	7.41	7.42	7.42
50-year	2%	7.73	7.74	7.74	7.75	7.75	7.76	7.76
100-year	1%	8.34	8.35	8.35	8.36	8.36	8.37	8.37
250-year	0.40%	9.82	9.83	9.83	9.84	9.85	9.85	9.86
500-year	0.20%	10.96	10.97	10.97	10.98	10.98	10.99	11.00

Datum: feet NAVD 88
RM = River Mile

Table A.7.3: Stage Frequency at Repaupo Creek Based Upon Intermediate Rate

Event	ACE	RM 82	RM 82.5	RM 83	RM 83.5	RM 84	RM 84.5	RM 85
2-year	50%	6.34	6.35	6.36	6.36	6.37	6.37	6.38
5-year	20%	6.92	6.93	6.94	6.94	6.95	6.96	6.96
10-year	10%	7.29	7.30	7.30	7.31	7.31	7.32	7.33
25-year	4%	7.77	7.77	7.78	7.78	7.79	7.79	7.80
50-year	2%	8.11	8.12	8.12	8.13	8.13	8.14	8.14
100-year	1%	8.72	8.73	8.73	8.74	8.75	8.75	8.76
250-year	0.40%	10.21	10.21	10.22	10.22	10.23	10.23	10.24
500-year	0.20%	11.35	11.35	11.36	11.36	11.37	11.37	11.38

Datum: feet NAVD 88
RM = River Mile

Table A.7.3 (Continued): Stage Frequency at Repaupo Creek Based Upon Intermediate Rate

Event	ACE	RM 85.5	RM 86	RM 86.5	RM 87	RM 87.5	RM 88	RM 88.5
2-year	50%	6.39	6.39	6.40	6.40	6.41	6.42	6.42
5-year	20%	6.97	6.97	6.98	6.99	6.99	7.00	7.01
10-year	10%	7.33	7.34	7.35	7.35	7.36	7.37	7.37
25-year	4%	7.81	7.81	7.82	7.82	7.83	7.84	7.84
50-year	2%	8.15	8.15	8.16	8.16	8.17	8.17	8.18
100-year	1%	8.76	8.77	8.77	8.78	8.78	8.79	8.79
250-year	0.40%	10.24	10.25	10.25	10.26	10.27	10.27	10.28
500-year	0.20%	11.38	11.39	11.39	11.40	11.40	11.41	11.42

Datum: feet NAVD 88
RM = River Mile

Table A.7.4: Stage Frequency at Repaupo Creek Based Upon High Rate

Event	ACE	RM 82	RM 82.5	RM 83	RM 83.5	RM 84	RM 84.5	RM 85
2-year	50%	7.71	7.71	7.72	7.72	7.73	7.74	7.74
5-year	20%	8.29	8.29	8.30	8.30	8.31	8.32	8.32
10-year	10%	8.65	8.66	8.66	8.67	8.68	8.68	8.69
25-year	4%	9.13	9.13	9.14	9.15	9.15	9.16	9.16
50-year	2%	9.47	9.48	9.48	9.49	9.49	9.50	9.50
100-year	1%	10.08	10.09	10.09	10.10	10.11	10.11	10.12
250-year	0.40%	11.57	11.57	11.58	11.58	11.59	11.59	11.60
500-year	0.20%	12.71	12.71	12.72	12.72	12.73	12.73	12.74

Datum: feet NAVD 88
RM = River Mile

Table A.7.4 (Continued): Stage Frequency at Repaupo Creek Based Upon High Rate

Event	ACE	RM 85.5	RM 86	RM 86.5	RM 87	RM 87.5	RM 88	RM 88.5
2-year	50%	7.75	7.76	7.76	7.77	7.77	7.78	7.79
5-year	20%	8.33	8.34	8.34	8.35	8.36	8.36	8.37
10-year	10%	8.70	8.70	8.71	8.71	8.72	8.73	8.73
25-year	4%	9.17	9.17	9.18	9.19	9.19	9.20	9.20
50-year	2%	9.51	9.52	9.52	9.53	9.53	9.54	9.54
100-year	1%	10.12	10.13	10.13	10.14	10.14	10.15	10.15
250-year	0.40%	11.60	11.61	11.61	11.62	11.63	11.63	11.64
500-year	0.20%	12.74	12.75	12.75	12.76	12.76	12.77	12.78

Datum: feet NAVD 88
RM = River Mile

A.7.4. Frequency Precipitation

It was assumed that precipitation duration frequency estimates for the maximum observed rainfall intervals using the Precipitation Frequency Data Server (NOAA Atlas 14) for Gloucester County was appropriate for the future "without" project conditions. No changes were made to the base year frequency estimates.

A.7.5. Future Watershed Percent Imperviousness

Values used for future percent imperviousness within the Repaupo Creek Watershed were based upon calculated watershed changes between the years 1995 and 2007 using land cover and land use datasets from NJDEP. Annual changes in percent imperviousness values between those years were calculated by region and applied to the base year values in order to come up with future percent imperviousness values. It was assumed that no further increases in percent imperviousness would occur after 20 years due to Local and/or State intervention to limit development in the watershed. Table A.7.5 summarizes by region the calculated percent imperviousness changes between the years 1995 and 2007.

Table A.7.5: Percent Imperviousness Changes Between 1995 and 2007

Region	Region Description	Percent Imperviousness Change btw 1995 and 2007	Percent Imperviousness Annual Change
1	South of NJ Turnpike	2.90%	0.24%
2	Between NJ Turnpike and I-295	1.73%	0.14%
3	North of I-295	0.00%	0.00%

A.7.6. Hydrologic Modeling Methods and Alternatives

To correctly depict future flood risk for the "without" project conditions and to objectively evaluate the reduction of flood risk for alternatives screened in the "with" project analysis, the expected inundation areas that would result from a future flood in year 2065 from the Delaware River or from interior drainage behind the Gibbstown Levee must be fully understood. The base year “without” project hydrologic model of the Repaupo Creek Watershed was modified for future conditions as necessary as outlined in the previous sections.

The same alternatives which portrayed varied hydrologic conditions for the base year “without” project hydrologic model were simulated for the future “without” project model, as well. One additional alternative was added, and that was for the three future projections of sea-level rise of the Delaware River.

A.7.7. Future Without Project Hydrologic Model Results

Table A.7.6 summarizes peak discharges at several locations computed by the HEC-HMS hydrologic model against the corresponding peak discharges for the same locations for the base year “without” project conditions. As the table show, the increases in percent imperviousness for the subbasins in Regions 1 and 2 increased the peak discharges throughout the watershed as would be expected.

Table A.7.6: "Without" Project Hydrologic Model Peak Discharge Comparison

			2% ACE (50-yr)	1% ACE (100-yr)	0.2% ACE (500-yr)
Location	Source	Drainage Area (sq. mi.)	Peak Discharge (cfs)	Peak Discharge (cfs)	Peak Discharge (cfs)
Repaupo Creek @ Delaware River	**Base**	**8.27**	1,580	1,815	2,617
	Future	**8.27**	1,633	1,884	2,713
Pargey Creek @ I-295	**Base**	**6.49**	1,611	2,124	2,927
	Future	**6.49**	2,072	2,414	3,267
White Sluice Race @ Deb's Ditch	**Base**	**11.84**	2,714	3,159	4,357
	Future	**11.84**	2,796	3,253	4,466
White Sluice Race @ Nehonsey Brook	**Base**	**7.36**	2,056	2,436	3,382
	Future	**7.36**	2,139	972	3,480
Nehonsey Brook @ White Sluice Race	**Base**	**4.07**	817	958	1,358
	Future	**4.07**	829	972	1,374
Nehonsey Brook @ Tomlin Station Rd.	**Base**	**1.96**	861	997	1,333
	Future	**1.96**	878	1,072	1,354
Still Run @ London Branch	**Base**	**5.38**	1,351	1,756	3,753
	Future	**5.38**	1,405	1,827	3,902
London Branch @ Still Run	**Base**	**1.64**	640	728	992
	Future	**1.64**	654	750	1,012
Clonmell Creek @ Delaware River	**Base**	**3.98**	1,001	1,176	2,176
	Future	**3.98**	1,347	1,580	2,655
Clonmell Creek @ I-295	**Base**	**1.10**	380	438	974
	Future	**1.10**	644	736	1,256

The same "without" project hydrologic alternatives as outlined in Section A.6.9 were also done for the future "without" project hydrologic analysis. Also, the same output tables pulled from the base year "without" project HEC-HMS hydrologic model were generated for the future "without" project hydrologic model for the interior areas leeward of the Gibbstown Levee. Three separate sets of simulations were done for the future "without" project conditions; each representing a different sea-level rise projection for the Delaware River tailwater condition, as previously discussed.

As with the base year "without" project conditions analysis, pumping over the levee was not assumed for any of these alternatives, and outflows from the interior areas were based upon the capacities of the floodgates themselves. It was also assumed that the floodgate openings in the future were the same as they are for existing conditions.

A.7.8. Final Future Without Project Hydrologic Model Results

A final stage frequency of interior ponding elevations for the future "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 for the future conditions. The assumptions and factors used were similar to the ones developed for the base year "without" project hydrologic model. Inflow from Cedar Swamp and Klondike Ditch was incorporated into the future interior pond stage frequency. It was assumed that no floodgate would be constructed on Klondike Ditch, and it was also assumed the system of interior levees on the Godwin Pump property were in disrepair and not effective in "blocking" flow from Cedar Swamp from entering the watershed. It was also assumed that the precipitation event over the watershed could happen on any random day of the year, and that precipitation over the watershed is independent of Delaware River tidal conditions. The precipitation event could happen when the tides on the Delaware River are normal or when the tides are elevated due to storm conditions. The simulations for the future "without" project conditions showed the same trends as the base year "without" project model showed, in terms of the difference in ponding elevations when the Delaware River is experiencing normal tidal conditions versus when a rare, low probability event occurs. A conservative estimate of tailwater conditions corresponding to a 1% annual chance of exceedance storm (100-yr) conditions was adopted for the final future interior ponding elevation stage frequency. Tables A.7.7 summarize the final future "without" project interior pond stage frequency curves for the three estimates of future sea-level rise.

The effects on the interior pond stage while increasing the percent imperviousness within the watershed and incorporating sea-level rise (SLR) can be seen in Tables A.7.8 for Repaupo Creek/White Sluice Race and Clonmell Creek areas, respectively. The tables compares the base year "without" project against the future "without" project interior stage frequency curves for the two areas. The difference for the Clonmell Creek interior area is not as pronounced as it is for the Repaupo Creek / White Sluice Race area because of the effects of inflow from Cedar Swamp. Since the inflow is increasing because of sea-level rise, it would be expected that the Repaupo / White Sluice area would be impacted accordingly. Since it was assumed that the Clonmell Creek interior area receives no inflow from Cedar Swamp; the impact of sea-level rise would be less there.

Table A.7.7: Future "Without" Project Interior Stage Frequency

Peak Interior Pond Elevation (feet NAVD 88)		Repaupo Creek / White Sluice Race			Clonmell Creek		
		Low	Intermediate	High	Low	Intermediate	High
Repaupo Watershed Precipitation Event	99% ACE (1-yr)	1.07	1.36	2.18	1.24	1.30	1.38
	50% ACE (2-yr)	1.30	1.57	2.41	1.61	1.71	1.85
	20% ACE (5-yr)	2.15	2.45	3.28	2.31	2.44	2.73
	10% ACE (10-yr)	2.47	2.79	3.63	2.81	2.95	3.29
	4% ACE (25-yr)	2.86	3.18	4.05	3.39	3.51	3.92
	2% ACE (50-yr)	3.15	3.46	4.34	3.81	3.94	4.34
	1% ACE (100-yr)	3.43	3.75	4.65	4.22	4.35	4.76
	0.4% ACE (250-yr)	3.84	4.15	5.04	4.79	4.92	5.32
	0.2% ACE (500-yr)	4.13	4.44	5.30	5.21	5.35	5.75

Table A.7.8: Comparison of Interior Stage Frequency for the Repaupo Creek / White Sluice Race Area & Clonmell Creek

Interior Stages (feet NAVD 88)		Repaupo Creek / White Sluice Race			Clonmell Creek		
		Base Year "Without" Project	Future "Without" Project Using Low SLR Rate	Difference in Stage (feet)	Base Year "Without" Project	Future "Without" Project Using Low SLR Rate	Difference in Stage (feet)
Repaupo Watershed Precipitation Event	99% ACE (1-yr)	0.35	1.07	0.72	1.00	1.24	0.24
	50% ACE (2-yr)	0.58	1.30	0.72	1.47	1.61	0.14
	20% ACE (5-yr)	1.10	2.15	1.05	2.18	2.31	0.13
	10% ACE (10-yr)	1.44	2.47	1.03	2.65	2.81	0.16
	4% ACE (25-yr)	1.86	2.86	1.00	3.23	3.39	0.16
	2% ACE (50-yr)	2.16	3.15	0.99	3.64	3.81	0.17
	1% ACE (100-yr)	2.46	3.43	0.97	4.06	4.22	0.16
	0.4% ACE (250-yr)	2.87	3.84	0.97	4.61	4.79	0.18
	0.2% ACE (500-yr)	3.16	4.13	0.97	5.04	5.21	0.17

A.8.0. REFERENCES

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