PECKMAN RIVER BASIN, NEW JERSEY

FLOOD RISK MANAGEMENT FEASIBILITY STUDY

HYDROLOGY APPENDIX

US Army Corps
of Engineers®
New York District

February 2020

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PECKMAN RIVER BASIN
ESSEX AND PASSAIC COUNTIES, NEW JERSEY
FLOOD RISK MANAGEMENT STUDY

1.0 OBJECTIVE OF STUDY

The Peckman River basin experiences frequent flash flooding during heavy rainfall events. The objective of this study is to fully evaluate all reasonable solutions to the problems identified and determine whether there is Federal interest in providing flood risk management measures for the Peckman River basin. This appendix documents basic hydrologic data, analysis, and interpretation in conjunction with the formulation of a plan of improved conditions for flood risk management in the Peckman River basin.

2.0 WATERSHED DESCRIPTION

The Peckman River basin is located in Passaic and Essex Counties, New Jersey. Figure 1 shows the location of the basin. The drainage area is approximately 10 square miles and is one of the sub-watersheds of the Passaic River basin. Peckman River originates in the Town of West Orange, New Jersey and flows northeast through the Borough of Verona, the Township of Cedar Grove, the Township of Little Falls, and the Borough of Woodland Park to its confluence with the Passaic River. The elevation change along the river is approximately 260 feet with the majority of the fall occurring within Cedar Grove.

Great Notch Brook is a major tributary to the Peckman River, its confluence with the Peckman River being just downstream of New Jersey State Highway 46. Great Notch Brook is subject to flash flooding from higher elevations in the eastern side of the watershed. Two other small tributaries enter the Peckman River in Cedar Grove.

The majority of the Peckman River watershed is heavily urbanized. Residential housing developments make up the largest sub-category of development. Undeveloped areas consist of forested areas, reservoirs, and wetlands along the river corridor.

The downstream portion of the Peckman River in Woodland Park is within close proximity to Dowling Brook, which is also a tributary to the Passaic River (see Figure 1). During extreme flooding events, diversion of flow from the Peckman River across Woodland Park to Dowling Brook has been reported.

3.0 PRIOR STUDIES AND REPORTS

- Detailed Project Report for the Peckman River, Township of Little Falls, under Section 205 of the 1948 Flood Control Act (September 1981)
- Peckman River Basin, New Jersey, 905(b) (January 2002)

4.0 PROJECT AREA

The project area is located in the Borough of Woodland Park, between the north side of State Highway 46 and the mouth of the Peckman River, and in the Township of Little Falls, along a reach south of Route 46, including a section adjacent to the Jackson Park residential area. Another point included in the project area is the confluence of Great Notch Brook. The discharge from Great Notch Brook is primarily from surface runoff and enters the Peckman River in the area of the primary damage center. The distance along the Peckman River within the project area is about 11,000 feet or 2.1 miles. The lower reach of Great Notch Brook, from Browertown Road to its mouth, a distance of 2,800 feet or 0.53 miles, is also included within the project area. The project area within the Peckman River watershed is shown in Figure 2.

5.0 CLIMATOLOGY

5.1 Climate

The climate of the Peckman River basin is characteristic of the entire Middle Atlantic seaboard. Marked changes of weather are frequent, particularly during the spring and fall. The winters are moderate in temperature and snowfall. The summers are moderate with hot sultry weather and the potential for frequent thunderstorms. Rainfall is moderate and well distributed throughout the year. The relative humidity is high. The average annual temperature is 52 degrees Fahrenheit at Little Falls, with extremes from 11 degrees Fahrenheit below zero to 105 degrees Fahrenheit above zero. The growing season averages 167 days. The mean annual relative humidity is 65 percent. Prevailing winds are from the northwest, with an annual average velocity of 10 miles per hour at Newark, N.J. The number of days with precipitation average about 122 per year. The mean annual precipitation in the Peckman River watershed is approximately 51.5 inches, as derived from the records of the Little Falls, New Jersey station. The observed highest daily value at this station was 12.79 inches (17 September 1999). The monthly extremes were 17.85 inches in September 1999 and 0.36 inches in November 1976. The distribution of precipitation throughout the year is fairly uniform with higher amounts occurring during the summer months. The mean annual snowfall is 21.4 inches as recorded at Little Falls, New Jersey precipitation station.

5.2 Precipitation Station

The Peckman River basin is best represented by the precipitation station at Little Falls, New Jersey. The location of the station is at Latitude 40 degrees 53 minutes north and Longitude 74 degrees 14 minutes west (Figure 1). The datum of the station elevation is 150 feet. This station was used for obtaining historic precipitation records for this study.

5.3 Storm Types

The storms which occur over the northeastern states have their origins in or near the Pacific and the South Atlantic oceans and may be classified as: extratropical storms, which include thunderstorms, cyclonic (transcontinental) storms and tropical storms, which include the West Indies hurricanes. The extratropical storms which due to rapid convective circulation that occurs when a tropical marine air mass is lifted suddenly on contact with hills and mountainous terrain, cause heavy rains, usually in the summer and fall seasons. The thunderstorms, due to rapid convective circulation, usually occurring in July, are limited in extent and cause local flooding on flashy streams. The cyclonic storms, containing transcontinental air mass movements with attendant "highs" and “lows," usually occur in the winter or early spring and are potential flood producers over large areas because of their widespread extent. The West Indies hurricanes, of tropical origin, proceed northward along the coastal areas, and are accompanied by extremely violent winds and torrential rains of several days duration.

5.4 Past Storms/Historical Floods

5.4.1 *Storm of 15-23 July 1945*

This extratropical type storm was widespread over most of the northeastern states when moisture laden marine air masses traveled inland and suddenly rose over a stationary cold front along the Atlantic coast. It consisted of six days of moderate rain followed by about 15 hours of heavy showers on 22 and 23 July 1945. The heavy showers were localized and spotty, resulting in flash floods on many small streams in and near the Passaic River basin. The maximum daily rainfall recorded in the vicinity was 7.60 inches at Little Falls, N.J. An average total of 8.5 inches of rain fell on the Passaic River Basin. At the Ozone Avenue at Verona, New Jersey USGS stream gage this storm produced an estimated historic flood peak of 3800 cfs.

5.4.2 *Storm of 13 September 1987*

A slow moving weather system dropped over 6 inches of rain across parts of northern New Jersey. Other rainfall total amounts ranged between 3 to 5 inches. This storm produced a flood peak at the Ozone Avenue at Verona, New Jersey USGS stream gage of 2400 cfs, third highest to the estimated July 1945 historic flood peak of record of 3800 cfs.

5.4.3 *Storm of 15-16 September 1999 (Tropical Storm Floyd)*

Hurricane Floyd began as a Cape Verde type hurricane east of the Lesser Antilles. Floyd made landfall on September 16th near Cape Fear, North Carolina with Category Two winds of 105 mph. After crossing eastern North Carolina and Virginia, Floyd weakened to a tropical storm. Its center moved offshore along the coasts of the Delmarva Peninsula and New Jersey. On September 17th Floyd moved over Long Island, NY, making landfall again roughly at the Queens-Nassau county border, and headed towards New England, where it became extratropical.

Rainfall totals from Tropical Storm Floyd were as high as 12 to 16 inches over portions of New Jersey, 4 to 8 inches over southeastern New York, and up to 11 inches over portions of New England.

The total storm rainfall observed at the Little Falls precipitation gage for Floyd was 14.13 inches, resulting in a new flood peak of record in the systematic record of the Peckman River at Ozone Avenue at Verona, New Jersey USGS stream gage. At the Ozone Avenue gage, the peak discharge of 2770 cfs was second only to the estimated July 1945 historic flood peak of record of 3800 cfs.

5.5 Climate Change

Consistent with the objective of ECB 2018-14 (Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs, and Projects), to enhance the climate preparedness and resilience of USACE projects by incorporating relevant information about observed and expected climate change impacts in hydrologic analysis for planned, new, and existing USACE projects, a qualitative analysis for inland hydrology was conducted using the best available data for the Peckman River basin. The qualitative analysis was conducted in three phases as specified by ECB 2018-14: Initial Scoping, Vulnerability Assessment, and a Risk Assessment.

5.5.1 Phase I Initial Scoping

The objective of this study is to fully evaluate all reasonable solutions to provide flood risk management measures for the Peckman River basin. For this study, it was determined that precipitation and streamflow are the most relevant climate variables that may affect the objective to develop flood risk management measures for the basin. Since the Peckman River basin is within a fluvial environment only, it was not necessary to evaluate the issue of sea level rise.

5.5.2 Phase II Vulnerability Assessment

For the vulnerability assessment phase, information was collected and analyzed to determine whether changes are presently occurring and whether expected changes in future hydrologic conditions will result in performance requirements significantly different from the present.

The vulnerability assessment includes a literature review of current, observed and projected climate trends, and the application of climate tools used to provide information on observed and projected climate trends relevant to the project area.

5.5.2.1 Literature Review

A synthesis of the USACE peer-reviewed climate literature is available for the Mid-Atlantic Region and was one of the primary sources of information referenced in this literature review. Additionally the Fourth National Climate Assessment produced by the US Global Change

Research Program was used as a source for understanding observed and projected climate trends in the northeast. The USACE report summarizes observed and projected climate and hydrological patterns cited in reputable peer-reviewed literature and authoritative national and regional reports, and characterizes climate threats to the USACE business lines (USACE, 2015a). The project watershed falls within the Mid-Atlantic Region, which is also referred to as Water Resources Region 02 (2-digit hydrologic unit code, or HUC02); see Figure 3(A) below.

5.5.2.2 Observed Climate Trends

Based on the observations made by the Fourth National Climate Assessment for the Northeast region, river flooding will pose a growing challenge to the Northeast region's systems and infrastructure will be increasingly compromised by future intense precipitation events. The Northeast has experienced a greater recent increase in extreme precipitation than any other region in the United States; between 1958 and 2010, the Northeast saw more than a 70 percent increase in the amount of precipitation falling in very heavy events (defined as the heaviest 1 percent of all daily events). Winter and spring precipitation is projected to increase; winter precipitation by about 5 to 20 percent by the end of the century.

In the Climate Change and Hydrology Literature Synthesis for the US Army Corps of Engineers Missions in the United States for the Mid-Atlantic Region 2, the USACE Institute of Water Resources cites Burns et al. (2007) identified statistically significant ($p<0.05$) increasing trends in annual precipitation for half of their climate stations in the Catskill Mountains in Southern New York. These authors used data from the period 1952-2005, and quantified average rates of increase in annual precipitation in the range of 79-263 mm per fifty years of record. However, no such trend was found by Warrach et al. (2006) for a climate station also in southern New York State. These authors analyzed annual precipitation totals for the period 1900-2000. While no significant annual trends were detected, seasonal trends were detected: including decreasing winter and summer monthly precipitation totals. The overall summary of observed climate trends indicates "there is also a good consensus in the literature that precipitation, and the occurrence of extreme storm events, has increased over the past century in the study region. However, despite the increased precipitation in the region, there is no evidence of significant increases in streamflow over the same period".

Projected climate trends in this report indicate "the majority of the studies reviewed here project increases in precipitation and streamflow through the 21st century. Extreme high events (storms and floods), in particular, are projected to increase in the future. Low flows, however, have been projected to decrease in the future as a result of the projected temperature (and ET) increases." A summary of the observed and projected climate variables are shown in Figure 3(B).

5.5.2.3 Projected Climate Trends

In the Climate Change and Hydrology Literature Synthesis for the US Army Corps of Engineers Missions in the United States for the Mid-Atlantic Region 2, the USACE Institute of Water Resources cites Najjar et al. (2009). This data quantifies an ensemble mean increase in annual precipitation for three major Mid-Atlantic watersheds. Mid and end of century projections show an average 2-5% increase in annual precipitation for the study region compared to the historical baseline (1971-2000). However, the uncertainty in these projections is reflected with relatively high standard deviations (3-12%) associated with these values.

Future projections of extreme events, including storm events and droughts forecasts increases in the occurrence and intensity of storm events by the end of the 21st century for the general study region. Wang and Zhang (2008) used downscaled GCMs to look at potential future changes in precipitation events across North America. They used an ensemble of GCMs and a single high emissions scenario (A2) to quantify a significant increase (20-50%) in the recurrence of the current 20-year 24-hour storm event for their future planning horizon (2075) and the General Mid-Atlantic Region. Additional uncertainty is introduced by the use of hydrologic models, there is moderate consensus that flows, particularly peak flows, will increase in the region through the 21st century as a result of increased precipitation. Low flows, however, are generally projected to decrease in the future. However, the frequency of heavy downpours is projected to continue to increase as the century progresses. Figure 3(C) summarizes the projected climate trends and impacts on each of the USACE business lines.

5.5.2.4 Climate Hydrology Assessment Tool

The Climate Hydrology Assessment Tool (CHAT) assesses trends in both observed and projected hydrometeorological data to project future changes in streamflow using GCMs at the watershed scale (HUC 04) seen on Figure 3(D). The USGS maintains a gage on the Peckman River at Little Falls (station number: 1389550) but was not used due to its extremely short period record of 7 years. However, a USGS gage adjacent to the Peckman River at the Passaic River at Little Falls (station number: 01389500) was used instead.

Using the Climate Hydrology Assessment Tool with annual Peak Instantaneous Streamflow, the p-value obtained from this analysis was 0.53 which is not statistically significant, considering a p-value < 0.05 is typically used as a threshold for significance. The overall projected trend in annual peak instantaneous streamflow shows a slight downward trend over the period of 1988-2014 seen on Figure 3(E).

The CHAT displayed a range of 93 projections that projected annual maximum monthly streamflow ranged from 1952-2099 modeled in the HUC 0203 of Lower Hudson-Long Island. This trend seems to steadily increase from 15,987 CFS in WY 1952 to 19,897 CFS in WY 2099 seen on Figure 3(F). The projected trends of annual maximum monthly streamflow ranged from 1952-2099 was also used to determine projected p-values. After 2000, the trajectory of the blue line over the HUC 04 basin indicates that the p-value is less than 0.0001, whereas the grey line steadily increases with a p-value of 0.15 which is observed on Figure 3(F).

5.5.2.5 Vulnerability Assessment Tool

The USACE Vulnerability Assessment tool is necessary to help guide adaptation planning and implementation so that USACE can successfully perform its missions, operations, programs, and projects in an increasingly dynamic physical, socioeconomic, and political environment. This tool provides indicators to develop vulnerability scores specific to each of the watersheds located within the contiguous United States.

A Vulnerability Assessment was conducted in the USACE North Atlantic Division (NAD), and within the New York District (NAN). Table 1(A) lists the vulnerability scores for the Flood Risk Reduction Business Line for HUC 0203, as well as the ranges of scores nationally, and within NAD and NAN for scenario changes in Table 1(A). As shown in the table, this watershed vulnerability of the Flood Risk Reduction business line is ranked the highest within the ranges NAN and NAD for all scenarios (wet and dry). When comparing these scores nationally, the HUC 0203 watershed falls within the middle for dry scenarios and below average for wet scenarios. Further analysis using the VA tool characterizes the HUC 0203 watershed as vulnerable for all scenarios for the Flood Risk Reduction Business Line when compared to the rest of the nation (top 20%).

The VA tool analyzed changes that were centered on two epochs, 2050 (2035-2065) and 2085 (2070-2099) grouping those epochs in “wet” and “dry” scenarios. Projections with total runoff values above the median value for the set are grouped as "wet", and ones with total runoff values below the median are grouped as "dry". All results were then given in scenario-epochs; Dry-2050, Dry-2085, Wet-2050, and Wet-2085. Several indicators localized within NAN were used to determine the overall climate risk score. These indicators include: Acres of Urban Area within 500-Year Floodplain (590), Flood Magnification Factor (568C/568L), and Percent Change in Runoff divided by Percent Change in Precipitation (277), and Annual Coefficient of Variant (CV) of Unregulated Runoff (175C).

The indicator that dominates vulnerability in both scenarios is Indicator #568C (flood magnification factor) which contributes approximately 41% for both dry epochs, and 43% for both wet epochs with indicator values greater than 1 (1.124 and 1.14 for Dry-2050 and Dry-2085 respectively; and 1.2311 and 1.3381 for Wet-2050 and Wet-2085 respectively) which indicates positive increases in future flood flows for both dry and wet scenarios. Meanwhile, Indicator #590 (area of the 500-year flood plain) has the second highest contribution with roughly 26% for both dry and wet epochs which suggests higher vulnerability relative to other watersheds. The use of this tool suggests that “dry” scenario-epochs are vulnerable and considerations should be given to projects located within the urbanized 500-year flood plain area. Table 1(B) provides absolute values of all relative indicators for both scenarios and epochs indicating the percent contribution to the overall vulnerability score.

The results of the VA tool analysis indicate that the HUC 0203 watershed is vulnerable to impacts to the Flood Risk Reduction Business Line and should be taken in consideration during the planning process and in communication with the local sponsor.

5.5.2.6 Nonstationarity Detection Tool

Stationarity, or the assumption that the statistical characteristics of hydrologic time series data are constant through time, enables the use of well-accepted statistical methods in water resources planning and design in which future conditions rely primarily on the observed record. However, recent scientific evidence shows that—in some places, and for some impacts relevant to the operations of the U.S. Army Corps of Engineers (USACE)—climate change and human modifications of the watersheds are undermining this fundamental assumption, resulting in nonstationarity.

The Nonstationarity Detection Tool enables the user to apply a series of statistical tests to assess the stationarity of annual instantaneous peak streamflow data series at any United States Geological Survey (USGS) streamflow gage site with more than 30 years of annual instantaneous peak streamflow records through Water Year 2014. The tool aids in identifying continuous periods of statistically homogenous (stationary) annual instantaneous peak streamflow datasets that can be adopted for further hydrologic analysis. The tool also allows users to conduct monotonic trend analyses on the identified subsets of stationary flow records. The tool facilitates access to USGS annual instantaneous peak streamflow records; does not require the user to have either specialized software or a background in advanced statistical analysis; provides consistent, repeatable analytical results that support peer review processes; and allows for consistent updates over time.

Since the Peckman River does not have stream gages with a record length suitable for this analysis, the Nonstationarity Detection Tool was applied to the adjacent USGS gage 1389500 Passaic River at Little Falls, New Jersey. The tool was applied to the period of 1898 through 2014 which detected nonstationarities using maximum annual flow/height, and provided graphical statistical results in addition to the mean and variance of detected nonstationarities which are shown on Figure 3(G). Using default sensitivity settings, nonstationarities were detected in the following water years: 1905 (mean), 1924 (distributional), 1927 (variance), 1928 (distributional), 1968 (distributional) and 1969 (distributional). However, only 1924 and 1969 indicate consensus that a statistically significant nonstationarity exists since both the Cramer-Von-Mises and Kolmogorov-Smirnov test agree for these years. The nonstationarities are not considered to be strong since the test results do not indicate consensus among 3 or more statistical tests for any given year detected.

It is unclear what the nonstationarities detected on the Passaic River indicate for the Peckman River watershed, since there are significant physical differences in the two watersheds. Furthermore, it is difficult to say if the nonstationarities detected on the Passaic River gage are indicative of trends in the region, and in-turn the Peckman River, since the attribution of the nonstationarity is unknown. Monotonic trend analysis does not detect any statistically significant trends in the data as seen on Figure 3(H).

5.5.3 Phase III Risk Assessment

The Phase II vulnerability assessment conducted on the Lower Hudson – Long Island basin indicates that the project area is located in a 2-digit HUC watershed that is vulnerable to climate change. Though, nonstationarities were detected within the Passaic River streamflow gage records, no direct evidence of nonstationarities were found on the Peckman River due to the lack of long-term gage records available for analysis. The best available scientific evidence based on climate literature and the Vulnerability Assessment tool indicates projected moderate increases precipitation and peak streamflow, as well as increases in storm frequency and intensity in the future. However, due to lack of quantitative information the impact of climate change to the project hydrology is inconclusive. Increases and storm frequency and intensity in the future may lead to increases in stream flow and instances of elevated river stages in the Peckman River, which may lead to more frequent overtopping instances of the levee feature in the future. The design of the proposed diversion culvert is robust enough to handle larger storm events and is expected to perform as designed in the future. Based on the results of this analysis it is recommended the potential risks of climate change to the region be communicated to the local sponsor for consideration for future watershed planning and adaptation of the levee feature in the future. Currently the typical levee cross-section specifies 3:1 side slopes with a 12 foot top width. It is possible the levee may be raised in the future by re-grading the slide slopes to 2.5:1, reducing the top width, and adding additional height. Modifying this structure would require engineering analysis and coordination with the USACE. Recognizing structural solutions are only one solution for reducing flood risk, non-structural measures such as flood proofing or raising structures above the flood elevation may be considered in the future.

6.0 HYPOTHETICAL RAINFALL

A 48-hour duration hypothetical storm was modeled so that the basin-wide HEC-HMS model would be accurate for watersheds with times of concentration between 24 and 48 hours.

Point precipitation frequency estimates in inches were obtained for the Peckman River basin from "Precipitation-Frequency Atlas of the United States" NOAA Atlas 14, volume 2. The precipitation frequency estimates are based on a partial duration series. The data was determined at the location of the Little Falls Water Company (40.8858N, 74.2261W). Adjustments were subsequently made to the 2-year values during initial calibration procedures as an additional parameter to use in achieving a calibrated model within a challenging scenario.

Point rainfall depths were part of the HEC-HMS model input and were converted to finite area rainfall depths with transposition storm areas and procedures contained in program HEC-HMS. The hypothetical point rainfall data is given in Table 1. The resultant hypothetical storm time series data is given in Table 2. A storm area of 10 square miles was used to reduce point rainfall values to finite drainage area values, because it is the drainage area of the Peckman River at its mouth. The hypothetical storm rainfall totals shown at the bottom of table 2 and 10 square mile rainfall values.

7.0 STREAMFLOW

The Little Falls gage has only been in operation since July 2007 and was not used for this study except to obtain average discharge values. Only peak discharges are available from the Ozone Avenue gage. The annual peak discharges recorded at the Ozone Avenue gage through WY 2003 are given in Table 3. The average annual runoff in the Peckman River basin at the Little Falls gage is 20 cfs over the 7.82 square mile drainage area, or 2.56 cfs per square mile. This runoff is equivalent to 34.72 inches per year at the Little Falls precipitation station, equivalent to 67 percent of the average yearly total rainfall of 51.5 inches at that station.

8.0 FLOOD FREQUENCY

A statistical analysis of the period of record annual peak discharges for the Peckman River at Ozone Avenue, Verona, New Jersey, USGS gage # 01389534, was made following the procedures presented in Bulletin 17B of the Water Resources Council. The period of record used for the Peckman River at Ozone Avenue USGS gage annual peak discharge vs. frequency curve: systematic record = 25 years (WY 1979-2003), historic period = 58 years (historic peak: July 1945). A plot of this analysis, an annual peak discharge versus frequency curve, with Langbein partial duration adjustment, is shown in Figure 3.

Due to the length of time that has passed between the alternative analysis (2008) and the selection of the NED plan (2019), there has been 18 years of record added to this gage and this could change the hydrology results. To determine the change in hydrology due the additional 18 years of data was to develop another discharge versus frequency curve with the additional 18 years of observed peak flow and compared the two data sets (WY2003 and WY2018). Table 5(A) shows both data set. The results shows, for all the return periods, that the WY2003 peak discharge is higher than the WY2018 peak discharge. We can conclude that the WY2003 peak discharges that was used for the hydrology analysis is on the conservative end pertaining to peak flow. In addition, since the TSP was first initially established in 2011, it would be a significant impact to schedule and costs if the PDT decided to redo the hydrology and hydraulics for this study based upon adding observed peak discharge data into the hydrology analysis.

9.0 HYDROLOGIC MODEL

9.1 Modeling Technique

The HEC-HMS computer program developed by The Hydrologic Engineering Center (HEC), Davis, California was used to perform the hydrologic analysis of the basin and to determine its rainfall/runoff/streamflow interrelationships.

Points of interest (POI) exist within the Peckman River basin. These are both natural, and those referred to as "map points". The natural points usually occur at stream junctions and highway crossings. The basin was divided into sub-basins, chosen so that their outlets were at least one of the following: a) primary stream confluences; b) damage areas; c) stream gage areas; d) potential sites for flood detention or diversion and e) potential location for beginning or end of potential hydraulic features (i.e., levee/floodwall) in alternative analysis. The 12 sub-basins are shown on the watershed map, Figure 5. A network diagram of the HEC-HMS model is shown on Figure 6. Table 5 lists the physical parameters of the sub-basins.

9.2 Simulation Process/Model Calibrations

The HEC HMS model was calibrated by adjusting its parameters so as to reproduce the recorded September 16, 1999 (Floyd) flood peak discharge at the Ozone Avenue USGS gage. This was the only flood event for which flood marks were available to use in calibration of the hydraulic model. The USGS gage is at the headwaters of this watershed and most of the floodmarks is approximately 3.0 miles downstream of this USGS gage. This was a challenge to be able to get acceptable calibrations for the hydrology and hydraulic models. For the hydrology, the emphasis in the calibration was on reproducing the flood peak at the Ozone Avenue gage, because no hydrographs were recorded at this site and this gage was the only one within the Peckman River basin at the time of this flood. The hypothetical flood HEC-HMS models were calibrated so that their peak flows matched the statistically computed peak discharge vs. frequency relation at the Ozone Avenue gage. See Table 6. Subsequent to the calibration process, model parameters were further adjusted to yield higher discharges in an attempt to match available flood marks in the hydraulic model. These adjusted values are reflected in the final discharges presented in this report.

For the high peak discharges used as input for the steady state hydraulic model, below is the rationale to justify this approach. Tropical Storm Floyd was the event chose for calibration and the recorded peak flow was 2,770 cfs at the Ozone Avenue Gage. This flow was originally used to calibrate the HMS at this gage. Then adjustments were made to come with discharge values that was acceptable with the calibration of the hydraulic model. The adjusted peak discharge value at the Ozone Avenue gage is 3,520 cfs. What was done to justify this value was to go to the peak discharge versus frequency curve to determine if the value of 3,520 cfs was within the allowable confidence band if we assume the value of 2,770 cfs is the expected value (between a 25-year and 50-year event). By using the values of 2,770 cfs as the expected values, we extracted the values for the 5% and 95% confidence bands. The 5% value is 2,222 cfs and the 95% value is 3, 850 cfs. So we are 90% confidence that the value of 3,520 cfs is within the allowed threshold of what the peak discharge value can be. This result gave flexibility to increase the flows within the hydrology model to get better peak flow to successfully calibrate the hydraulic model.

Portions of the Peckman River and its tributaries were also modeled with unsteady HEC-RAS. For the hypothetical floods, the upstream boundary locations from the HMS model were identified as input points to the unsteady HEC-RAS model. The discharge hydrographs from the HMS model at these selected points were provided as input to the unsteady HEC-RAS model. These input locations are shown in Table 5(A). Please see the Hydraulics Appendix for details of this modeling effort.

9.2.1 *Rainfall*

The total rainfall of 14.13 inches for the September 15-16 1999 (Floyd) storm was determined for the Peckman River basin using the Little Falls rain gage. The Newark Airport rain gage provided the temporal distribution. This resultant time-series data is shown in Table 7.

9.2.2 *Infiltration Loss*

The infiltration losses used in the HEC-HMS model for the Peckman River basin were computed by the NRCS Curve Number method. NRCS Curve Numbers were developed for each sub-basin through an extensive analysis of land use and soils information by GIS procedures. A Hydrologic Soil Group (HSG) was determined for each soil type within the basin. Relevant land uses were also determined for the entire basin. An integration of the land uses and HSG on a composite basis, related to lists of associated NRCS Curve Numbers, for unique pairings of HSG's and land uses, resulted in area-averaged Curve Number assignments for each sub-basin. Adopted existing conditions values of the sub-basin NRCS Curve Numbers, and corresponding initial abstraction values, are shown in Table 8.

9.2.3 *Unit Hydrographs*

The unit hydrograph parameters for the HEC-HMS model sub-basins were determined using the NRCS unit graph lag method. This method gives an empirical equation for NRCS sub-basin lag as a function of sub-basin length, slope, and Curve Number. The lag was developed through calculations based on map measurements of physical parameters via GIS methods. Sub-basin physical parameters are provided in Table 5. The adopted existing conditions unit graph parameters are provided in Table 9.

9.2.4 *Base flow*

Average monthly base flow was used for each month at a rate of 2 cfs per square mile. The baseflow rate of 2 cfs per square mile is an average rate deemed acceptable for this purpose and is based on examination of the average flow rates at numerous stream gaging sites in the Passaic River basin. This rate was applied to each sub-basin and is shown in Table 10.

9.2.5 *Channel Routing*

The Muskingum and the Modified Puls methods are the channel routing procedures used in this study within program HEC-HMS. For the Modified Puls method the storage versus outflow data is taken from a comprehensive set of water surface profile program (HEC-RAS) runs which capture the full channel and overbank storage along the Peckman River. Muskingum routing is used in the reaches where HEC-RAS data is not available. The value of Muskingum weighting factor X is between 0.0 and 0.5. A value of 0.0 gives maximum attenuation from the procedure and 0.5 provides the minimum attenuation. Muskingum parameters are used for reach RTE3, below Cedar Grove Reservoir. They are: Travel time K =1 hour, weighting factor X = 0.3 and the number of sub reaches = 8. Existing conditions storage-outflow relationships for Modified Puls routing reaches are given in Table 11.

10.0 MODIFICATIONS TO EXISTING CONDITIONS MODEL

10.1 Diversion to Passaic River

There is a naturally occurring overland diversion of flow along the west bank of the Peckman River to the Passaic River at certain water surface elevations. This is different than the natural diversion from the Peckman River to Dowling Brook previously mentioned. This flow (diversion) of Peckman River flood water occurs between RS (river cross section) 4365 and RS 4850 in the hydraulic model. Values of the diversion are shown in Table 12. The diversion takes place immediately upstream of the Rt. 46 Bridge and is also the proposed location of the Diversion Culvert alternative (at RS 4850) where flow from the Peckman River would be conveyed to the Passaic River within a constructed concrete culvert.

10.2 Sub-division of Great Notch Brook

The alternatives considered along Great Notch Brook occur from Browertown Road to the confluence with the Peckman River. Peak discharges were calculated at the Browertown Road location independently from the HEC-HMS model by multiplying the peak flows at the mouth of Great Notch Brook by the ratio of the drainage area at Browertown Road to the drainage area at the mouth, raised to the exponent of 0.75. Its peak discharge values are shown in Table 13.

11.0 RESULTS OF EXISTING CONDITIONS HISTORIC AND HYPOTHETICAL FLOODS

A summary of the final basin-wide peak discharges, adjusted to account for the natural diversion, developed with HEC-HMS for existing conditions at important locations within the Peckman River watershed is shown in Table 12. This table includes results for both the September 15-16, 1999 (Floyd) flood plus a full range of hypothetical floods. Similar data is presented on Table 13 for the tributary of Great Notch Brook that enters the Peckman River immediately downstream of Route 46. Existing conditions peak and coincidental flows for the Peckman and Passaic Rivers are shown on Table 14 and for the Peckman River and Great Notch Brook on Table 15. Appendix C-2 “Hydraulics” contains more information on this subject. A plot of existing conditions peak discharges versus basin drainage area is shown in Figure 6.

12.0 FUTURE WITHOUT PROJECT CONDITIONS

Future development of the Peckman River watershed will increase the amount of impervious surfaces within it. This will result in an increase in storm runoff from any given rainfall. The increase in impervious surfaces will also affect the rainfall-runoff response of the watershed by increasing the peak flows of the sub-basin unit hydrographs and decreasing their lag times. In other words, as impervious surfaces increase in a watershed, for any given rainfall, more runoff will occur, its peak flow will increase, and will occur more rapidly. This is because sheet and gutter flow of runoff over impervious surfaces is greater than for undeveloped pervious surfaces.

NRCS runoff curve numbers were developed for future without project (FWOP) conditions for the sub-basins of the Peckman River basin to model and account for the hydrologic effect of anticipated future development. The increase in sub-basin NRCS runoff curve number from present to future conditions results in a future sub-basin unit hydrograph that is more peaked and rapid (higher peak discharge and shorter lag time) than the present condition sub-basin unit hydrograph. Table 18 shows present and future conditions curve numbers and unit hydrograph lag times for the HEC-HMS model sub-basins of the Peckman River watershed.

After a thorough examination of the currently available zoning maps, aerial photography, Land Use/Master Plans for Cedar Grove, West Orange, Verona, Little Falls, and Woodland Park an assessment of impervious surfaces and changes in river discharge was determined. The local community provided some of the input that was incorporated into the FWOP computations. The most likely parcels, within the watershed boundaries, to undergo future land use changes were identified. Estimates were made of revised CNs and their resulting NRCS unit hydrograph parameters. These new values, as shown in Table 16, were input into the existing conditions

HEC-HMS models and the results of the subsequent model runs yielded FWOP conditions discharges as shown in Table 17. Great Notch Brook peak discharges for FWOP conditions are shown in Table 13. Also shown in Table 13 are FWOP conditions peak discharges for Great Notch Brook at Browertown Road (drainage area = 0.519 square miles). These values were transferred from those at the mouth of Great Notch Brook and are required for the design of certain improvements along Great Notch Brook. A comparison of Existing and FWOP Conditions hydrologic data is presented in Table 18. See Figure 8 for comparison plots of 50-year existing conditions and FWOP conditions hydrographs for the Peckman River downstream of Great Notch Brook.

13.0 RISK AND UNCERTAINTY

An equivalent record length in years was developed for the peak discharge vs. frequency relations used and analyzed in this study. This equivalent record length was then applied to all economic reaches in the study. This equivalent record length was also used in the hydraulic analysis of this study to determine confidence bands of stage-frequency curves for defined economic reaches. This is discussed in further detail in the hydraulics support document of this study.

The stream gage used for this analysis was USGS 01389534: Peckman River at Ozone Avenue, Verona, NJ (drainage area=4.5 square miles). An annual peak discharge vs frequency analysis was performed at the gage using WRC Bulletin 17B. The systematic record was 25 years (WY 1979 - 2003); historic period = 58 years (historic peak: July 1945). The drainage area at the project location is about 9.3 square miles.

An equivalent record length for the project was determined to be 10 years utilizing the Equivalent Record Length Guidelines shown in Table 4-5 of EM1110-2-1619. The selection was made based on judgment to account for the quality of the data used in the analysis and for the degree of confidence in the HMS model.

For risk assessment analysis that was done following ER 1105-2-101, see the Economic Appendix, Section 7.2.3 for more information.

14.0 CONSIDERED PLANS OF IMPROVEMENTS

14.1 Introduction of Alternatives

Various types of improvements were analyzed to lessen the risk of flooding from the Peckman River at the basin's primary damage centers. These improvements included, separately or in combination, such structural measures as: channel modification, diversions, levees and floodwalls, detention storage and non-structural measures: floodproofing, raising, etc.

14.2 Plans of Improvement Considered

Alternatives include:

- **Alternative #1** – Without Project Future Conditions
- **Alternative #2** – Non-Structural Alternatives
- **Alternative #3** – Diversion Culvert
- **Alternative #4** – Channel Improvements providing flood damage reduction upstream and downstream of Rt. 46
- **Alternative #5** – Levee/Floodwall providing flood damage reduction upstream and downstream of Rt. 46
- **Alternative #6** – Levee/Floodwall providing flood damage reduction downstream of Rt. 46
- **Alternative #7** – Channel Improvements providing flood damage reduction downstream of Rt. 46
- **Alternative #8** – Combined Channel Improvements with Diversion Culvert providing flood damage reduction upstream of Rt. 46
- **Alternative #9** – Combined Levee/Floodwall with Diversion Culvert providing flood damage reduction upstream of Rt. 46
- **Alternative #10a** – Diversion Culvert plus 0.02 AEP Non-structural Measures Upstream of Rt. 46
- **Alternative #10b** – Diversion Culvert plus 0.1 AEP Non-structural Measures Upstream of Rt. 46

Further details of the plans of improvement considered can be found in the hydraulics appendix.

PECKMAN RIVER BASIN
ESSEX AND PASSAIC COUNTIES, NEW JERSEY
FLOOD RISK MANAGEMENT PROJECT

APPENDIX – TABLES & FIGURES

HYDROLOGY

TABLE 1(A): VULNERABILITY SCORES FOR HUC 0203 FOR THE FLOOD RISK REDUCTION BUSINESS LINE FOR EACH SCENARIO-EPOCH COMBINATION NATIONALLY, NAD AND NAN.

Business Line	Scenario-Epoch	WOWA Score	Range Nationally	Range in NAD	Range in NAN
Flood Risk Reduction	Dry – 2050	52.48	35.15-70.08	40.04-52.58	44.36-52.48
	Dry - 2085	53.37	35.15-70.08	40.01-53.37	45.32-53.37
	Wet - 2050	54.42	39.80-92.85	43.13-54.82	48.14-54.42
	Wet - 2085	56.91	39.80-92.85	43.12-56.91	49.69-56.91

TABLE 1(B): VALUES/PERCENT CONTRIBUTION TO VULNERABILITY OF EACH INDICATOR ASSOCIATED WITH THE FLOOD RISK REDUCTION BUSINESS LINE FOR ALL SCENARIO-EPOCH COMBINATIONS ALONG WITH PERCENT CHANGES BETWEEN EPOCHS FOR EACH SCENARIO

Number	Dry-2050	Dry-2085	Percent Change	Wet-2050	Wet-2085	Percent Change
590	25.75/20.95	26.25/20.74	1.95	25.75/19.71	26.25/19.06	1.95
568C	14.046/41.26	14.274/41.61	1.62	15.38/43.06	16.72/43.82	8.69
568L	7.239/18.11	7.340/18.32	2.64	7.952/21.88	8.674/22.29	9.08
277	4.121/16.15	4.165/15.85	1.07	4.098/12.12	3.977/11.64	-2.94
175C	1.326/3.53	1.252/3.47	-5.57	1.240/3.23	1.294/3.20	4.36

TABLE 2: HYPOTHETICAL POINT RAINFALL (ANNUAL MAXIMA SERIES)

Precipitation in inches									
Duration	**5 min**	**15 min**	**1 hour**	**2 hours**	**3 hours**	**6 hours**	**12 hours**	**24 hours**	**48 hours**
1-year	0.36	0.72	1.22	1.46	1.63	1.90	2.30	2.70	3.19
2-year	0.44	0.89	1.51	1.86	2.08	2.69	3.35	3.78	4.46
5-year	0.47	0.93	1.68	2.08	2.33	3.01	3.77	4.29	5.04
10-year	0.53	1.05	1.95	2.46	2.75	3.57	4.51	5.16	6.04
25-year	0.60	1.19	2.30	2.96	3.00	4.34	5.56	6.43	7.45
50-year	0.65	1.28	2.55	3.35	3.76	4.97	6.44	7.51	8.62
100-year	0.69	1.37	2.81	3.76	4.22	5.64	7.40	8.70	9.89
250-year	0.71	1.40	3.10	4.35	4.95	6.70	8.90	10.50	11.80
500-year	0.78	1.54	3.41	4.79	5.37	7.40	10.02	12.05	13.35

TABLE 3: 10 SQUARE MILE TIME SERIES HYPOTHETICAL RAINFALL

		Precipitation in Inches								
Day	Time	1-yr	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	250-yr	500-yr
Day 1	0 to 8:15	0	0	0	0	0	0	0	0	0
Day 1	8:20 to 8:25	0	0	0	0	0	0	0	0	0.01
Day 1	8:30 to 10:05	0	0	0	0	0	0	0	0.01	0.01
Day 1	10:10 to 11:20	0	0	0	0	0	0	0.01	0.01	0.01
Day 1	11:25 to 12:40	0	0	0	0	0	0.01	0.01	0.01	0.01
Day 1	12:45 to 15:55	0	0	0	0	0.01	0.01	0.01	0.01	0.01
Day 1	16:00 to 16:15	0	0	0	0	0.01	0.01	0.01	0.01	0.02
Day 1	16:20 to 18:00	0	0	0	0.01	0.01	0.01	0.01	0.01	0.02
Day 1	18:05 to 18:40	0	0.01	0.01	0.01	0.01	0.02	0.02	0.03	0.03
Day 1	18:45 to 19:20	0	0.01	0.01	0.01	0.02	0.02	0.02	0.03	0.03
Day 1	19:25 to 19:40	0	0.01	0.01	0.01	0.02	0.02	0.02	0.03	0.04
Day 1	19:45	0.01	0.01	0.01	0.01	0.02	0.02	0.02	0.03	0.04
Day 1	19:50 to 20:30	0.01	0.01	0.01	0.01	0.02	0.02	0.03	0.03	0.04
Day 1	20:35	0.01	0.01	0.01	0.02	0.02	0.02	0.03	0.04	0.04
Day 1	20:40 to 20:50	0.01	0.01	0.01	0.02	0.02	0.02	0.03	0.04	0.04
Day 1	20:55 to 21:00	0.01	0.01	0.01	0.02	0.02	0.03	0.03	0.04	0.05
Day 1	21:05	0.01	0.01	0.01	0.02	0.03	0.03	0.03	0.04	0.05
Day 1	21:10 to 21:20	0.01	0.01	0.02	0.02	0.03	0.03	0.03	0.04	0.05
Day 1	21:25 to 21:30	0.01	0.01	0.02	0.02	0.03	0.03	0.04	0.04	0.05
Day 1	21:35	0.01	0.01	0.02	0.02	0.03	0.03	0.04	0.05	0.05
Day 1	21:40 to 21:45	0.01	0.01	0.02	0.02	0.04	0.03	0.04	0.05	0.05
Day 1	21:50	0.01	0.01	0.02	0.02	0.04	0.03	0.04	0.05	0.06
Day 1	21:55 to 22:00	0.01	0.02	0.02	0.02	0.04	0.03	0.04	0.05	0.06
Day 1	22:05 to 22:10	0.01	0.02	0.02	0.02	0.04	0.04	0.04	0.05	0.06
Day 1	22:15	0.01	0.02	0.02	0.03	0.04	0.04	0.04	0.06	0.06
Day 1	22:20	0.01	0.02	0.02	0.03	0.04	0.04	0.04	0.06	0.06
Day 1	22:25 to 22:30	0.01	0.02	0.02	0.03	0.04	0.04	0.05	0.06	0.07
Day 1	22:35	0.01	0.01	0.02	0.02	0	0.03	0.03	0.04	0.04
Day 1	22:40 to 22:50	0.01	0.02	0.02	0.02	0	0.03	0.04	0.05	0.05
Day 1	22:55	0.01	0.02	0.02	0.03	0	0.04	0.04	0.05	0.05
Day 1	23:00	0.01	0.02	0.02	0.03	0.01	0.04	0.04	0.06	0.05
Day 1	23:05	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.09	0.1
Day 1	23:10	0.02	0.02	0.03	0.04	0.05	0.06	0.07	0.09	0.1
Day 1	23:15	0.02	0.02	0.03	0.04	0.05	0.06	0.07	0.1	0.11
Day 1	23:20	0.02	0.03	0.03	0.04	0.06	0.07	0.08	0.1	0.11
Day 1	23:25	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.11	0.12
Day 1	23:30	0.02	0.03	0.04	0.05	0.06	0.08	0.09	0.12	0.13
Day 1	23:35	0.03	0.04	0.05	0.07	0.08	0.09	0.11	0.13	0.14
Day 1	23:40	0.04	0.04	0.06	0.07	0.09	0.1	0.12	0.14	0.15
Day 1	23:45	0.04	0.05	0.07	0.08	0.1	0.12	0.13	0.16	0.17
Day 1	23:50	0.06	0.07	0.1	0.12	0.15	0.18	0.2	0.24	0.26
Day 1	23:55	0.08	0.09	0.12	0.15	0.19	0.21	0.24	0.28	0.31

TABLE 3: 10 SQUARE MILE TIME SERIES HYPOTHETICAL RAINFALL (CONT.)

		Precipitation in Inches								
Day	**Time**	**1-yr**	**2-yr**	**5-yr**	**10-yr**	**25-yr**	**50-yr**	**100-yr**	**250-yr**	**500-yr**
Day 2	0:00	0.17	0.22	0.24	0.28	0.32	0.35	0.37	0.39	0.41
Day 2	0:05	**0.3**	**0.36**	**0.42**	**0.49**	**0.56**	**0.61**	**0.64**	**0.66**	**0.72**
Day 2	0:10	0.12	0.15	0.17	0.2	0.23	0.24	0.26	0.27	0.29
Day 2	0:15	0.07	0.08	0.11	0.13	0.17	0.19	0.22	0.26	0.28
Day 2	0:20	0.05	0.06	0.07	0.09	0.11	0.13	0.14	0.17	0.18
Day 2	0:25	0.04	0.05	0.06	0.08	0.1	0.11	0.12	0.15	0.16
Day 2	0:30	0.03	0.04	0.06	0.07	0.09	0.1	0.11	0.13	0.14
Day 2	0:35	0.02	0.03	0.04	0.05	0.07	0.08	0.1	0.12	0.13
Day 2	0:40	0.02	0.03	0.04	0.05	0.06	0.07	0.09	0.11	0.13
Day 2	0:45	0.02	0.03	0.03	0.04	0.06	0.07	0.08	0.11	0.12
Day 2	0:50	0.02	0.03	0.03	0.04	0.05	0.06	0.08	0.1	0.11
Day 2	0:55	0.02	0.02	0.03	0.04	0.05	0.06	0.07	0.1	0.11
Day 2	1:00	0.02	0.02	0.03	0.04	0.05	0.06	0.07	0.09	0.1
Day 2	1:05	0.01	0.02	0.02	0.03	0.01	0.04	0.04	0.06	0.06
Day 2	1:10	0.01	0.02	0.02	0.03	0.01	0.04	0.04	0.05	0.05
Day 2	1:15	0.01	0.02	0.02	0.03	0	0.04	0.04	0.05	0.05
Day 2	1:20 to 1:25	0.01	0.02	0.02	0.02	0	0.03	0.04	0.05	0.05
Day 2	1:30	0.01	0.01	0.02	0.02	0	0.03	0.04	0.05	0.04
Day 2	1:35 to 1:40	0.01	0.02	0.02	0.03	0.04	0.04	0.05	0.06	0.07
Day 2	1:45	0.01	0.02	0.02	0.03	0.04	0.04	0.05	0.06	0.06
Day 2	1:50	0.01	0.02	0.02	0.03	0.04	0.04	0.04	0.05	0.06
Day 2	1:55 to 2:05	0.01	0.02	0.02	0.02	0.04	0.04	0.04	0.05	0.06
Day 2	2:10	0.01	0.02	0.02	0.02	0.04	0.03	0.04	0.05	0.06
Day 2	2:15	0.01	0.01	0.02	0.02	0.04	0.03	0.04	0.05	0.06
Day 2	2:20 to 2:30	0.01	0.01	0.02	0.02	0.04	0.03	0.04	0.05	0.05
Day 2	2:35 to 2:40	0.01	0.01	0.02	0.02	0.03	0.03	0.04	0.04	0.05
Day 2	2:45 to 3:00	0.01	0.01	0.02	0.02	0.03	0.03	0.03	0.04	0.05
Day 2	3:05 to 3:10	0.01	0.01	0.01	0.02	0.02	0.03	0.03	0.04	0.04
Day 2	3:15 to 3:30	0.01	0.01	0.01	0.02	0.02	0.02	0.03	0.04	0.04
Day 2	3:35 to 4:15	0.01	0.01	0.01	0.01	0.02	0.02	0.03	0.03	0.04
Day 2	4:20	0.01	0.01	0.01	0.01	0.02	0.02	0.02	0.03	0.04
Day 2	4:25 to 4:45	0	0.01	0.01	0.01	0.02	0.02	0.02	0.03	0.04
Day 2	4:50 to 5:20	0	0.01	0.01	0.01	0.02	0.02	0.02	0.03	0.03
Day 2	5:25 to 6:00	0	0.01	0.01	0.01	0.01	0.02	0.02	0.03	0.03
Day 2	6:05 to 7:45	0	0	0	0.01	0.01	0.01	0.01	0.01	0.02
Day 2	7:50 to 8:10	0	0	0	0	0.01	0.01	0.01	0.01	0.02
Day 2	8:15 to 11:20	0	0	0	0	0.01	0.01	0.01	0.01	0.02
Day 2	11:25 to 12:45	0	0	0	0	0	0.01	0.01	0.01	0.02
Day 2	12:50 to 14:00	0	0	0	0	0	0	0.01	0.01	0.01
Day 2	14:05 to 15:45	0	0	0	0	0	0	0	0.01	0.01
Day 2	15:50	0	0	0	0	0	0	0	0	0.01
Day 2/3/4	15:55 to 22:00	0	0	0	0	0	0	0	0	0
	Totals:	**3.14**	**4.39**	**4.96**	**5.95**	**7.34**	**8.49**	**9.74**	**11.62**	**13.15**

TABLE 4: ANNUAL PEAK FLOWS AT PECKMAN RIVER AT OZONE AVENUE AT VERONA, NJ (USGS GAGE #01389534)

Water Year	Date of Annual Peak Flow	Annual Peak Flow (cfs)
1945	7/23/1945	3800
1979	9/6/1979	1610
1980	4/28/1980	820
1981	7/21/1981	1250
1982	1/4/1982	1070
1983	4/16/1983	920
1984	4/5/1984	780
1985	9/27/1985	993
1986	8/3/1986	705
1987	9/13/1987	2400
1988	7/26/1988	730
1989	9/20/1989	1520
1990	5/16/1990	1350
1991	3/3/1991	1040
1992	6/5/1992	1020
1993	9/26/1993	590
1994	11/28/1993	1640
1995	7/17/1995	1020
1996	6/3/1996	834
1997	10/19/1996	1550
1998	4/2/1998	754
1999	9/16/1999	2770
2000	5/18/2000	660
2001	6/17/2001	1350
2002	7/19/2002	1420
2003	8/4/2003	520
2004	7/27/2004	803
2005	3/28/2005	1220
2006	7/18/2006	1110
2007	8/8/2007	1030

TABLE 5: PHYSICAL PARAMETERS OF WATERSHED

GIS Name	HMS Name	Description	Drainage Area (sq. mi)	Length[1] (ft.)	Slope (ft./ft.)
Pkwest1	Subarea E (W1)	Subarea E	0.2025	3,700	0.005
Pkwest2	Subarea D (W2)	Subarea D	0.8832	5,989	0.030
Pkwest3	Tributary C (W3)	Little Falls Tributary	0.3667	5,271	0.034
Pkwest4	Tributary B (W4)	Cedar Grove Tributary	0.3532	5,215	0.026
Pkwest5	Subarea A (W5)	Subarea A	0.5696	4,736	0.015
Pkwest6	Ozone Ave USGS Gage (W6)	Ozone Ave USGS Gage	1.8311	9,132	0.026
Pkwest7	Verona Lake (W7)	Verona Lake and Verona Tributary	2.6314	11,235	0.022
Pkeast1	Great Notch Brook (E1)	Great Notch Brook	0.5618	9,120	0.019
Pkeast2	Cedar Grove Reservoir (E2)	Cedar Grove Reservoir	0.4220	4,166	0.008
Pkeast3	Tributary A (E3)	Taylor Brook	0.9386	7,867	0.027
Pkcenter1	Subarea C (C1)	Subarea C	0.7827	4,715	0.029
Pkcenter2	Subarea B (C2)	Subarea B	0.4841	5,633	0.028

Note: (1) Hydrologic length is 75% of longest length from sub-basin boundary to outlet, measured between points 10 and 85% upstream of the outlet to the boundary. Sub-basin slope is defined between these points as well.

TABLE 5(A): INPUT OF DISCHARGE HYDROGRAPHS FOR UNSTEADY HEC-RAS MODEL

Location of discharge hydrographs for unsteady HEC-RAS inputs	Drainage Area (mi^2)
Upstream Little Falls Tributary (Map Point 7)	8.01
Little Falls Tributary (Tributary C (W3))	0.37
(Subarea D (W2))	0.88
(Subarea E (W1))	0.20
Great Notch Brook at Mouth (Great Notch Brook (E1))	0.56

TABLE 6: HYPOTHETICAL FLOOD CALIBRATION : USGS GAGE NO. 01389534 PECKMAN RIVER AT OZONE AVENUE IN VERONA NEW JERSEY

Return Period	Computed Flow[1] (cfs)	Flow Based on Gage Data[2] (cfs)	Percent Difference[3]
2-year	1,190	1,185	-0.5%
5-year	1,620	1,625	0.5%
10-year	2,020	2,020	0%
25-year	2,600	2,600	0%
50-year	3,130	3,175	2%
100-year	3,740	3,775	1%
250-year	4,710	4,725	0.5%
500-year	5,480	5,500	0.5%

(1) From HEC-HMS model
(2) Annual peak discharge vs. frequency relations via WRC Bulletin 17B with partial duration adjustments.
(3) Positive value represent gage data is higher than computed flow; negative value represents gage data is lower than computed flow.

TABLE 6 (A): HYPOTHETICAL FLOOD CALIBRATION : USGS GAGE NO. 01389534 PECKMAN RIVER AT OZONE AVENUE IN VERONA NEW JERSEY (WY 2003 VERSUS WY2018)

Return Period	WY2003[1] (cfs)	WY2018[1] (cfs)	Percent Difference
2-year	1,185	1,150	-3.0%
5-year	1,625	1,480	-9.0%
10-year	2,020	1,790	-11.0%
25-year	2,600	2,230	-14.0%
50-year	3,175	2,600	-18.0%
100-year	3,775	3,020	-20.0%
250-year	4,725	3,650	-23.0%
500-year	5,500	4,140	-25.0%

(1) Annual peak discharge vs. frequency relations via WRC Bulletin 17B with partial duration adjustments.

TABLE 7: TIME-SERIES RAIN DATA FOR TROPICAL STORM FLOYD

Date	Time interval	Precipitation in Inches
9/15/1999	8:05 to 11:00	0.00
9/15/1999	11:05 to 12:00	0.20
9/15/1999	12:05 to 17:00	0.00
9/15/1999	17:05 to 21:00	0.86
9/15/1999 to 9/16/1999	21:05 to 2:00	0.00
9/16/1999	2:05 to 3:00	0.20
9/16/1999	3:05 to 4:00	0.00
9/16/1999	4:05 to 6:00	0.42
9/16/1999	6:00 to 7:00	0.88
9/16/1999	7:05 to 8:00	1.00
9/16/1999	8:05 to 9:00	0.80
9/16/1999	**9:05 to 11:00**	**3.35**
9/16/1999	11:05 to 12:00	0.80
9/16/1999	12:05 to 13:00	0.20
9/16/1999	13:05 to 14:00	0.00
9/16/1999	14:05 to 15:00	0.60
9/16/1999	15:05 to 16:00	0.20
9/16/1999	16:05 to 17:00	1.20
9/16/1999	17:05 to 18:00	2.61
9/16/1999	18:05 to 19:00	0.80
9/16/1999 to 9/19/1999	19:05 to 24:00	0.00

Total: **14.13 inches**

TABLE 8: SCS CURVE NUMBERS AND INITIAL ABSTRACTIONS FOR EXISTING CONDITIONS

Sub-basin	SCS Curve Number	Initial Abstraction (inches)								
Return Periods		1-year	2-year	5-year	10-year	25-year	50-year	100-year	250-year	500-year
Verona Lake	74	0.50	0.50	0.55	0.95	1.25	1.43	1.45	1.10	0.50
Ozone Avenue	80	0.30	0.30	0.35	0.75	1.05	1.23	1.25	0.90	0.30
Sub-basin A	78	0.36	0.36	0.41	0.81	1.11	1.29	1.31	0.96	0.36
Cedar Grove	85	0.15	0.15	0.20	0.60	0.90	1.08	1.10	0.75	0.15
Tributary A	79	0.33	0.33	0.38	0.78	1.08	1.26	1.28	0.93	0.33
Tributary B	77	0.40	0.40	0.45	0.85	1.15	1.33	1.35	1.00	0.40
Subarea B	78	0.36	0.36	0.41	0.81	1.11	1.29	1.31	0.96	0.36
Tributary C	72	0.58	0.58	0.63	1.03	1.33	1.51	1.53	1.18	0.58
Subarea C	78	0.36	0.36	0.41	0.81	1.11	1.29	1.31	0.96	0.36
Subarea D	86	0.13	0.13	0.18	0.58	0.88	1.06	1.08	0.73	0.13
Great Notch Bk	85	0.15	0.15	0.20	0.60	0.90	1.08	1.10	0.75	0.15
Subarea E	92	0.00	0.00	0.55	0.42	0.72	1.43	0.92	0.57	0.00

TABLE 9: SCS LAG

Sub-basin	Present Conditions (minutes)	Future Without Project Conditions (minutes)
Verona Lake	64	60
Ozone Avenue	36	33
Sub-basin A	8	7
Cedar Grove	14	14
Tributary A	35	33
Tributary B	10	9
Subarea B	15	14
Tributary C	30	24
Subarea C	30	25
Subarea D	63	54
Great Notch Brook	45	39
Subarea E	90	90

TABLE 10: BASE FLOW FOR EACH SUB-BASIN

Subbasin	Baseflow (cfs)
Verona Lake	0.26
Ozone Avenue	0.18
Sub-basin A	0.06
Cedar Grove	0.04
Tributary A	0.09
Tributary B	0.04
Subarea B	0.05
Tributary C	0.04
Sub-area C	0.08
Sub-area D	0.09
Great Notch Brook	0.06
Sub-area E	0.02

TABLE 11: MODIFIED PULS (STORAGE VS. DISCHARGE RELATIONSHIP) FOR EXISTING CONDITIONS

Reach RT-C1		Reach RT-C2		Reach RT-W1	
Storage (acre-ft.)	Flow (cfs)	Storage (acre-ft.)	Flow (cfs)	Storage (acre-ft.)	Flow (cfs)
0	0	0	0	0	0
31.91	2,080.00	21.12	1,970.00	91.17	2,370.00
40.50	2,790.00	31.74	2,650.00	145.02	3,180.00
49.18	3,460.00	39.16	3,280.00	195.00	3,950.00
54.13	3,790.00	49.80	3,770.00	198.29	4,000.00
60.56	4,360.00	53.28	4,180.00	254.62	4,850.00
72.81	5,290.00	64.49	5,000.00	301.33	6,040.00
83.32	6,280.00	79.47	5,390.00	332.96	7,170.00
118.44	7,860.00	113.55	7,400.00	387.49	9,040.00
141.34	9,060.00	134.87	8,550.00	449.67	10,420.00

Reach RT-W2		Reach RT-W5		Reach RT-W6	
Storage (acre-ft.)	Flow (cfs)	Storage (acre-ft.)	Flow (cfs)	Storage (acre-ft.)	Flow (cfs)
0	0	0	0	0	0
117.12	2,350.00	22.59	1,250.00	28.8	1,190.00
177.08	3,170.00	25.63	2,390.00	39.53	1,620.00
234.35	3,930.00	42.77	2,960.00	51.18	2,020.00
238.74	3,990.00	43.98	3,770.00	67.36	2,600.00
282.73	4,840.00	55.28	4,500.00	81.11	2,770.00
350.28	6,030.00	67.74	5,340.00	86.70	3,130.00
433.42	7,150.00	85.64	6,650.00	110.28	3,740.00
515.43	9,010.00	98.72	7,690.00	148.47	4,710.00
575.43	10,400.00			179.74	5,480.00

TABLE 12: EXISTING CONDITIONS – PEAK DISCHARGE

Peckman River Basin													
Existing Conditions Peak Discharges (in cfs)													
Map Point	**Location Description**	**Drainage Area (square miles)**	**Floyd 16-Sep-99**	**1-yr flood**	**2-yr flood**	**5-yr flood**	**10-yr flood**	**25-yr flood**	**50-yr flood**	**100-yr flood**	**250-yr flood**	**500-yr flood**
16	Verona Lake	2.631	2060	430	750	1040	1300	1700	2050	2460	3090	3620
15	USGS Gage at Ozone Avenue	4.463	3520	910	1490	2020	2500	3140	3660	4280	5310	6100
14	US Taylor Brook	5.032	3810	970	1580	2140	2650	3370	3930	4600	5720	6570
11	DS Taylor Brook	6.393	4960	1330	2150	2900	3590	4560	5320	6220	7670	8800
10	US Cedar Grove Tributary	6.877	5330	1390	2230	3010	3710	4720	5500	6540	7990	9180
8	DS Cedar Grove Tributary	7.230	5620	1430	2290	3080	3810	4860	5670	6750	8260	9470
7	US Little Falls Tributary	8.013	6210	1580	2560	3380	4140	5320	6300	7390	9010	10250
5	DS Little Falls Tributary	8.379	6560	1650	2680	3580	4330	5580	6640	7810	9500	10770
4	US Great Notch Brook (US Natural Diversion)	9.263	6240	1670	2680	3690	4200	5400	6370	7400	9170	10640
4	Diverted Flow	9.263	1840	0	0	860	1170	1500	1890	2260	3280	3610
4	US Great Notch Brook (DS Natural Diversion)	9.263	4400	1670	2680	2820	3030	3900	4480	5140	5900	7030
2	DS Great Notch Brook	9.824	4860	1820	2900	3170	3420	4300	4970	5690	6530	7720
1	Peckman River at Mouth	10.027	4440	1670	2600	2900	3140	3730	4460	5190	6180	7310

TABLE 13: GREAT NOTCH BROOK PEAK DISCHARGE

Peckman River Basin												
Peak Discharges (in cfs)												
Map Point	Location Description	Drainage Area (square miles)	Floyd 16-Sep-99	1-yr flood	2-yr flood	5-yr flood	10-yr flood	25-yr flood	50-yr flood	100-yr flood	250-yr flood	500-yr flood
Existing Conditions												
3	Great Notch Brook at Mouth	0.56	540	210	310	390	480	590	680	780	900	1020
	Great Notch Brook at Browertown Road	0.519	510	198	293	369	454	558	643	737	851	964
Future Without Project Conditions												
3	Great Notch Brook at Mouth	0.56	570	270	370	470	560	690	780	880	1000	1110
	Great Notch Brook at Browertown Road	0.519	539	255	350	444	529	652	737	832	945	1049

TABLE 14: PECKMAN AND PASSAIC RIVER PEAK & COINCIDENTAL FLOWS FOR THE PECKMAN RIVER BASIN

Peckman River Basin										
Existing Conditions Discharges (in cfs) *										
Location Description	**Floyd 16-Sep-99**	**1-yr flood**	**2-yr flood**	**5-yr flood**	**10-yr flood**	**25-yr flood**	**50-yr flood**	**100-yr flood**	**250-yr flood**	**500-yr flood**
Peckman River at mouth peak	6150	1670	2600	3390	4020	4890	6170	7370	9270	10680
Passaic R. below Peckman R. coincidental	8680	1640	1640	1640	1640	1650	1680	2110	1870	1780
Passaic R. below Peckman R. peak	11600	5360	7780	11550	14330	18440	21980	26100	31700	36330
Peckman River at mouth coincidental	1	1	1	1	1	1	1	1	1	1

* Peckman River discharges do not include natural diversions

TABLE 15: PECKMAN R. AND GREAT NOTCH BROOK PEAK & COINCIDENTAL FLOWS FOR THE PECKMAN RIVER BASIN

Peckman River Basin										
Existing Conditions Discharges (in cfs) *										
Location Description	**Floyd 16-Sep-99**	**1-yr flood**	**2-yr flood**	**5-yr flood**	**10-yr flood**	**25-yr flood**	**50-yr flood**	**100-yr flood**	**250-yr flood**	**500-yr flood**
Peckman River below Great Notch Brook peak	6680	1820	2900	3990	4550	5790	6840	7930	9780	11330
Great Notch Brook at mouth coincidental	440	160	230	330	370	410	490	560	630	700
Great Notch Brook at mouth peak	540	210	310	390	480	590	680	780	900	1020
Peckman River below Great Notch Brook coincidental	6290	1590	2580	3550	4280	4960	5800	7080	8540	9740

* Peckman River discharges do not include natural diversion

Table 16: SCS Curve Numbers and Initial Abstractions for Future Without Project Conditions

Sub-basin	SCS Curve Number	Initial Abstraction (inches)								
Return Periods		1-year	2-year	5-year	10-year	25-year	50-year	100-year	250-year	500-year
Verona Lake	76	0.50	0.50	0.55	0.95	1.25	1.43	1.45	1.10	0.50
Ozone Avenue	83	0.30	0.30	0.35	0.75	1.05	1.23	1.25	0.90	0.30
Sub-basin A	81	0.36	0.36	0.41	0.81	1.11	1.29	1.31	0.96	0.36
Cedar Grove	85	0.15	0.15	0.20	0.60	0.90	1.08	1.10	0.75	0.15
Tributary A	81	0.33	0.33	0.38	0.78	1.08	1.26	1.28	0.93	0.33
Tributary B	79	0.40	0.40	0.45	0.85	1.15	1.33	1.35	1.00	0.40
Subarea B	81	0.36	0.36	0.41	0.81	1.11	1.29	1.31	0.96	0.36
Tributary C	79.4	0.58	0.58	0.63	1.03	1.33	1.51	1.53	1.18	0.58
Subarea C	84	0.36	0.36	0.41	0.81	1.11	1.29	1.31	0.96	0.36
Subarea D	90	0.13	0.13	0.18	0.58	0.88	1.06	1.08	0.73	0.13
Great Notch Bk	89	0.15	0.15	0.20	0.60	0.90	1.08	1.10	0.75	0.15
Subarea E	92	0.00	0.00	0.02	0.42	0.72	0.90	0.92	0.57	0.00

TABLE 17: FUTURE WITHOUT PROJECT CONDITIONS – PEAK DISCHARGE

Peckman River Basin

Future Without Project Conditions - Peak Discharges (in cfs)

Map Point	Location Description	Drainage Area (square miles)	Floyd 16-Sep-99	1-yr flood	2-yr flood	5-yr flood	10-yr flood	25-yr flood	50-yr flood	100-yr flood	250-yr flood	500-yr flood
16	Verona Lake	2.631	2150	490	830	1150	1430	1860	2220	2640	3280	3820
15	USGS Gage at Ozone Avenue	4.463	3670	1050	1670	2240	2760	3410	3950	4580	5600	6430
14	US Taylor Brook	5.032	3990	1110	1770	2370	2920	3660	4230	4920	6040	6900
11	DS Taylor Brook	6.393	5170	1500	2370	3160	3890	4890	5670	6590	8010	9140
10	US Cedar Grove Tributary	6.877	5520	1580	2460	3290	4010	5080	5920	6940	8320	9580
8	DS Cedar Grove Tributary	7.230	5860	1620	2530	3370	4120	5230	6100	7180	8610	9910
7	US Little Falls Tributary	8.013	6440	1810	2850	3680	4530	5740	6710	7810	9350	10690
5	DS Little Falls Tributary	8.379	6800	1910	3030	3930	4770	6050	7140	8300	9930	11200
4	US Great Notch Brook (US Natural Diversion)	9.263	6420	1900	3020	3890	4480	5830	6750	7750	9560	11040
4	Diverted Flow	9.263	1910	0	290	980	1250	1670	2030	2460	3370	3840
4	US Great Notch Brook (DS Natural Diversion)	9.263	4510	1900	2730	2910	3240	4160	4720	5290	6190	7200
2	DS Great Notch Brook	9.824	4980	2080	3040	3310	3640	46500	5250	5920	6820	7940
1	Peckman River at Mouth	10.027	4530	1880	2720	2970	3280	4000	4690	5420	6430	7540

TABLE 18: COMPARISON OF EXISTING AND FUTURE WITHOUT PROJECT CONDITIONS HYDROLOGIC DATA

Subbasin Name	Drainage Area (mi^2)	Existing 100-yr (cfs)	FWOP 100-yr (cfs)	Existing Conditions CN	FWOP Conditions CN	Existing Lag (min.)	FWOP Lag (min.)
W7	2.6314	2460	2640	74	76	64	60
W6	1.8311	2630	2870	80	83	36	33
W5	0.5696	1470	1600	78	81	8	7
E2	0.4220	990	990	85	85	14	14
E3	0.9386	1350	1420	79	81	35	33
W4	0.3532	810	880	77	79	10	9
C2	0.4841	990	1060	78	81	15	14
W3	0.3667	490	620	72	79.4	30	24
C1	0.7827	1190	1410	78	84	30	25
W2	0.8832	1030	1180	86	90	63	54
E1	0.5618	780	880	85	89	45	39
W1	0.2025	210	210	92	92	90	90

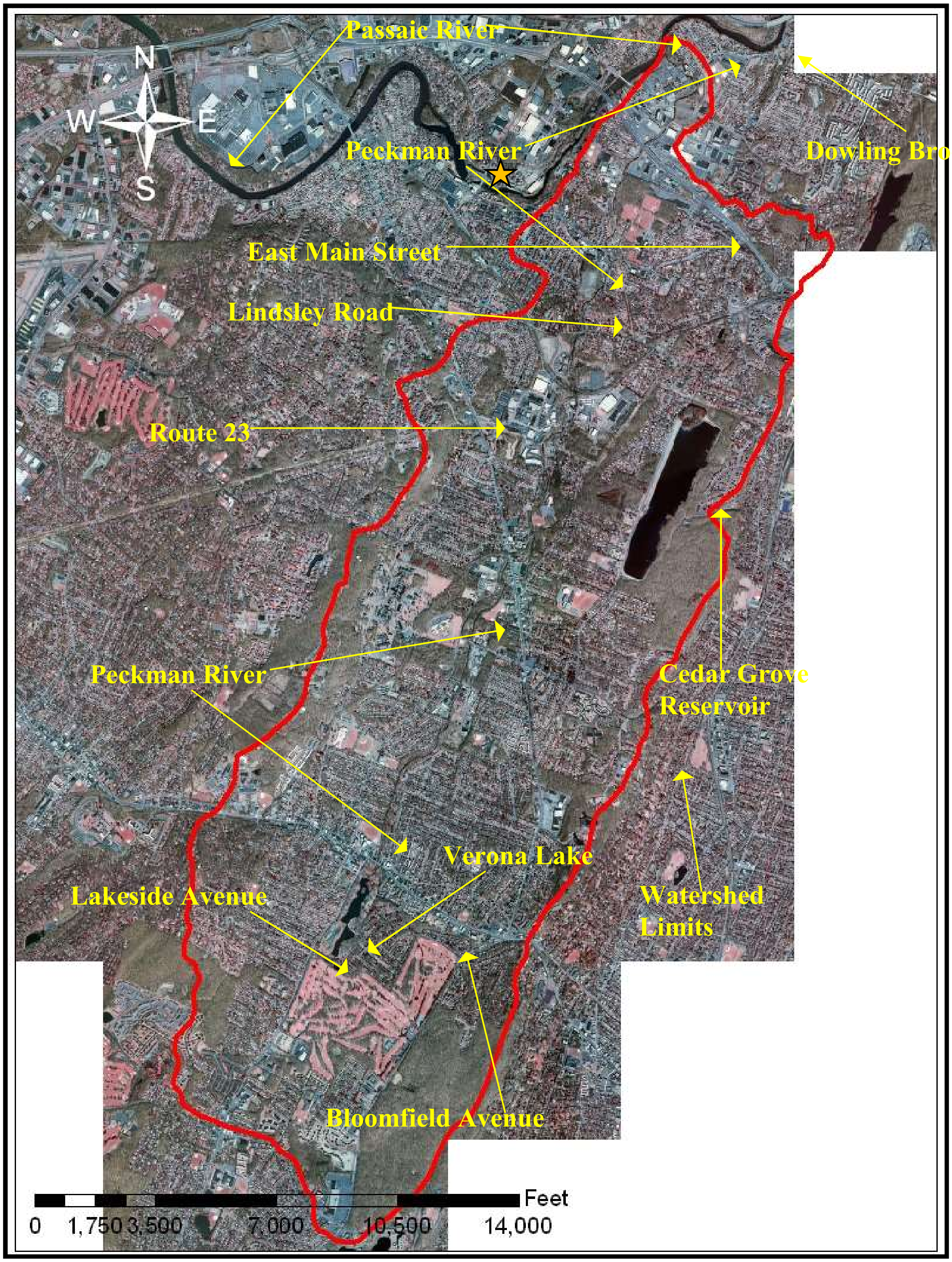


FIGURE 1: PECKMAN RIVER BASIN LOCATION MAP

*Location of precipitation station shown by orange star.

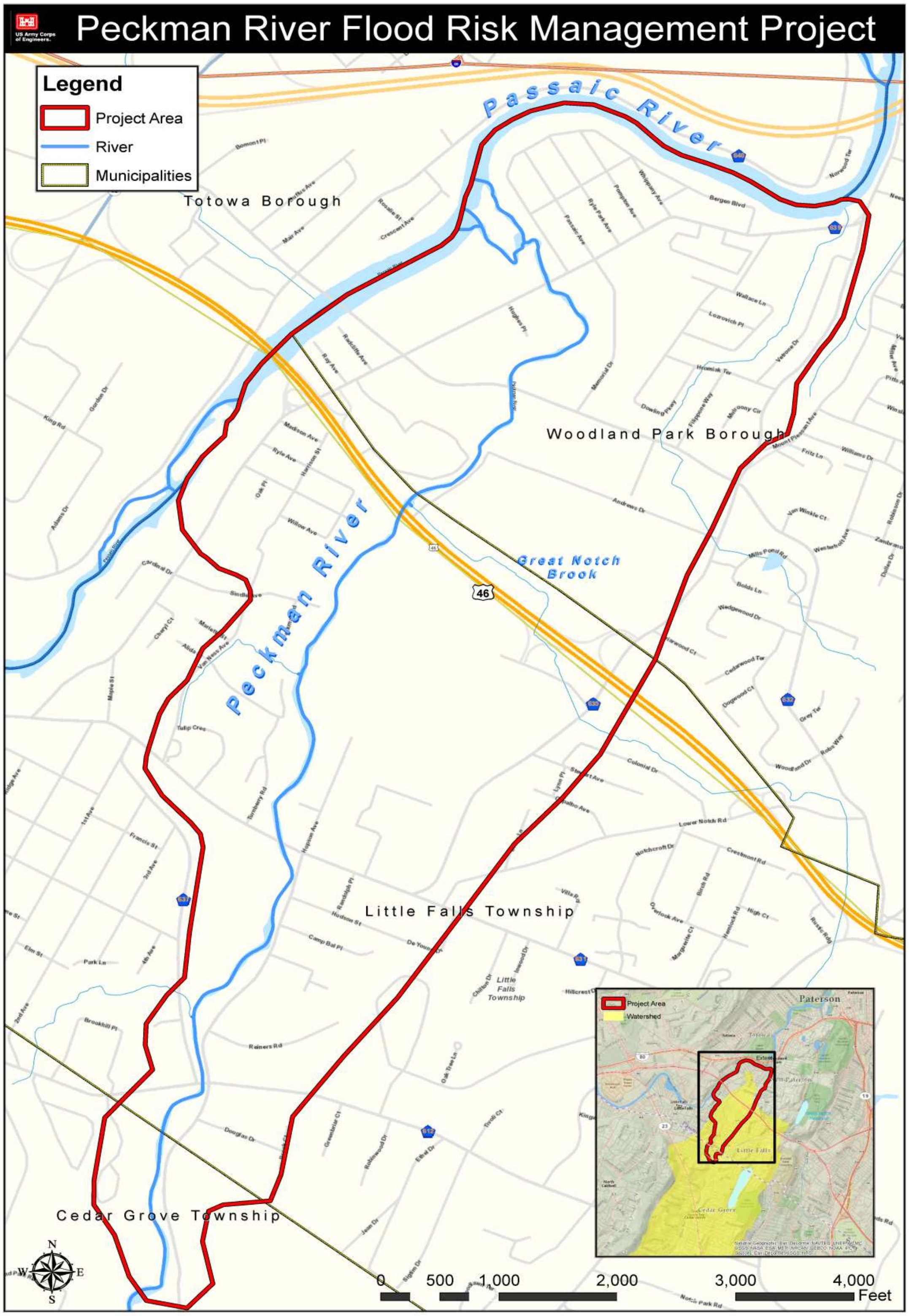


FIGURE 2: STUDY AREA WITHIN PECKMAN RIVER BASIN

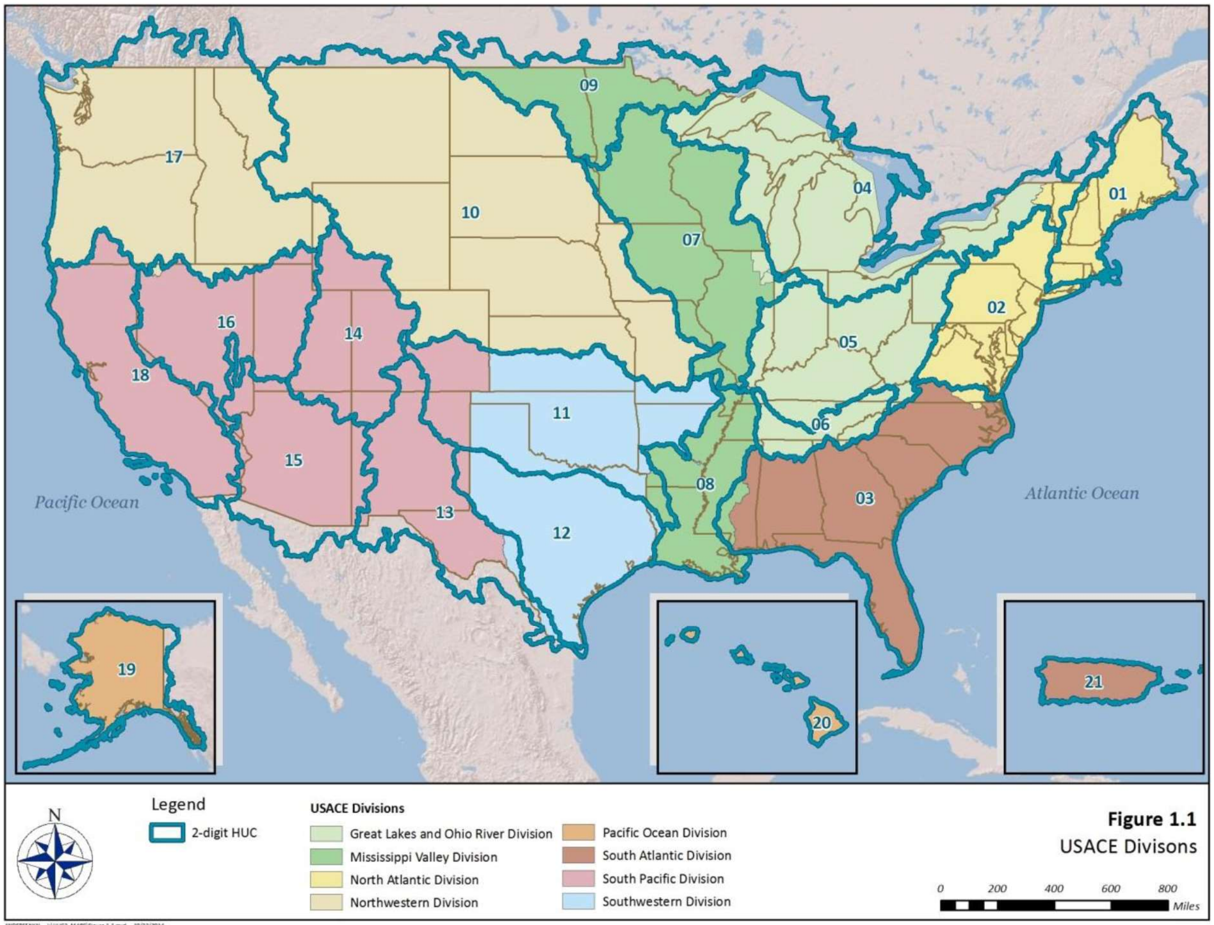


FIGURE 3(A): TWO-DIGIT WATER RESOURCES REGOION BOUNDARIES FOR CONUS ALASKA, HAWAII AND PUERTO RICO

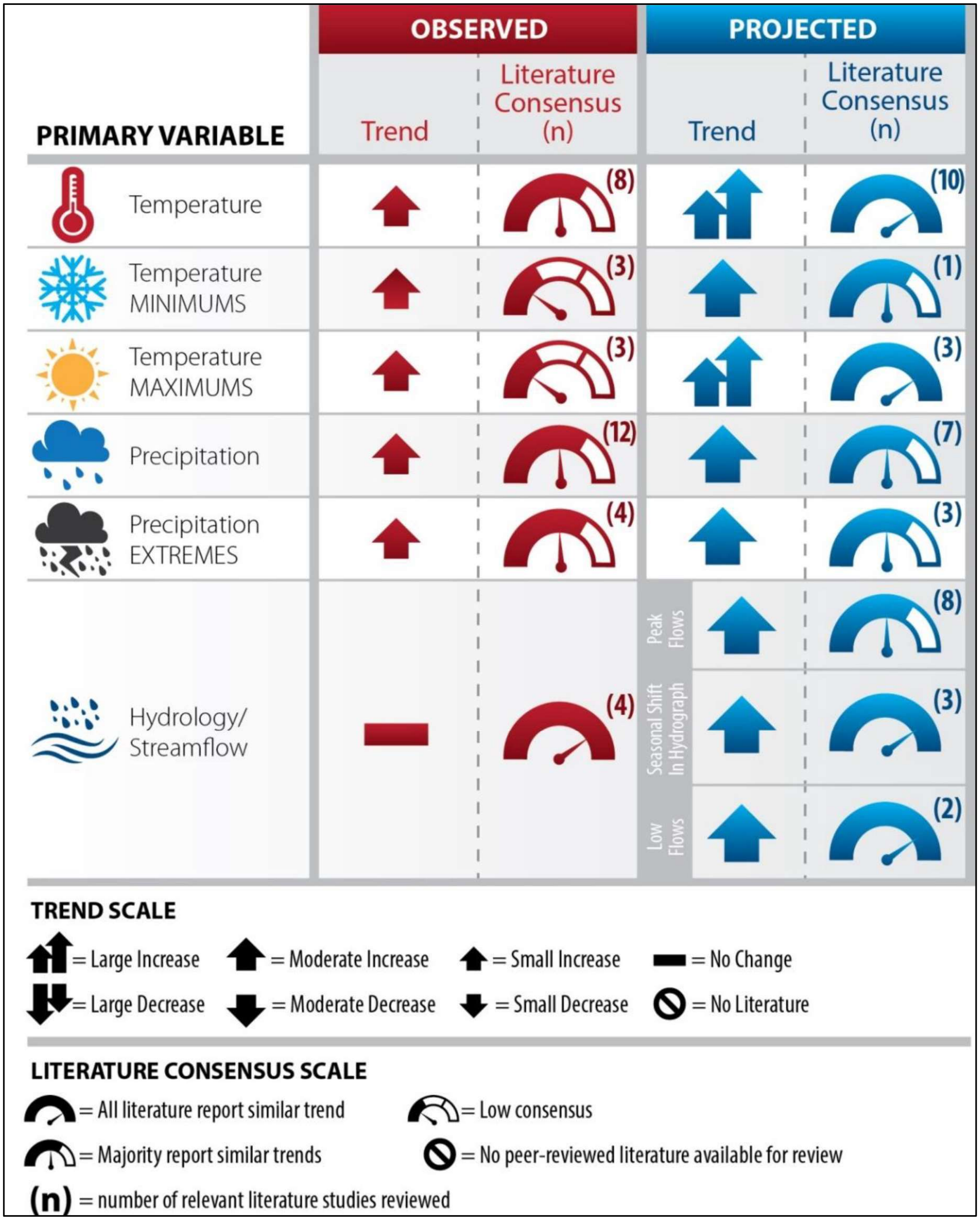
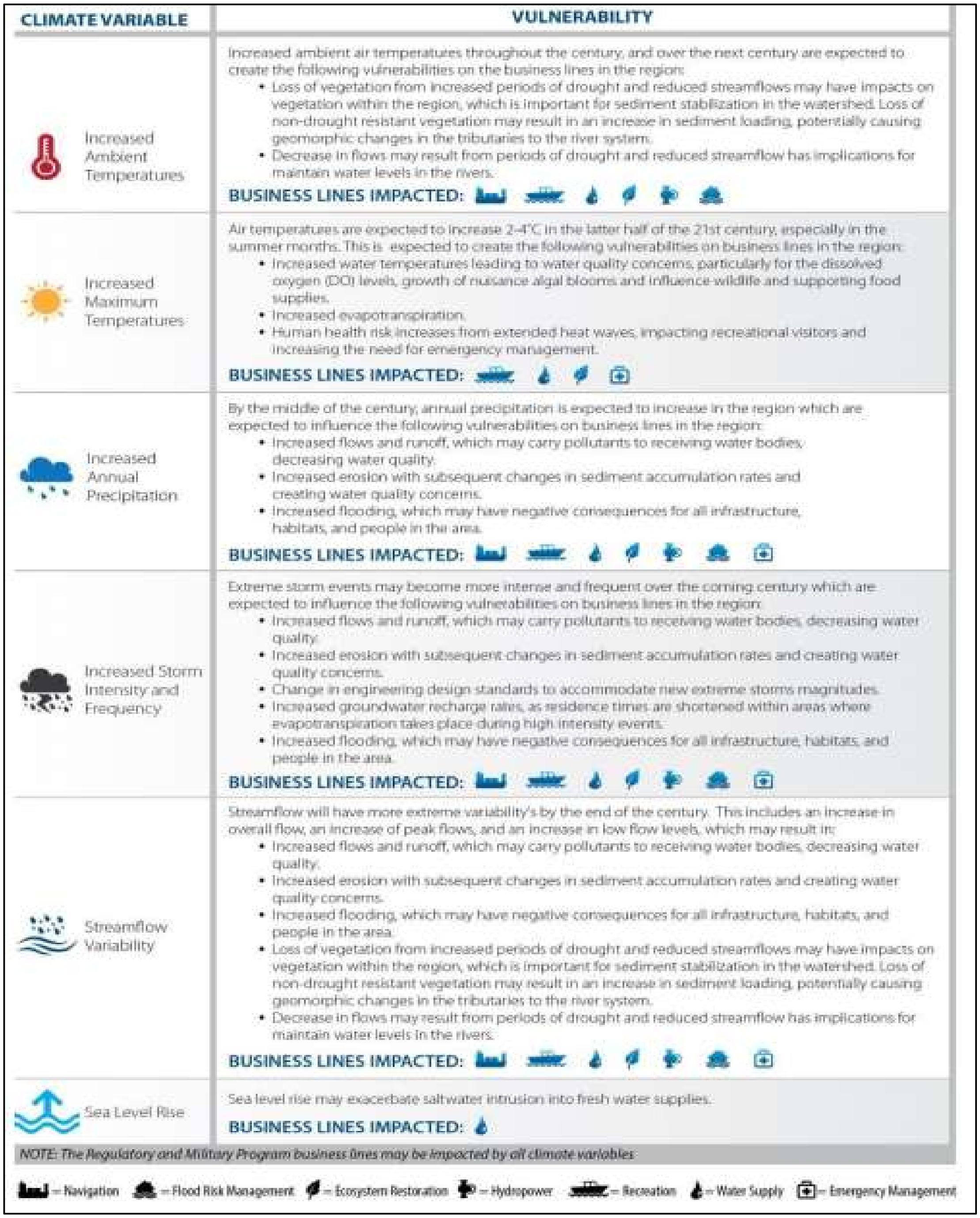


FIGURE 3(B): SUMMARY MATRIX OF OBSERVED AND PROJECTED CLIMATE TRENDS AND LITERATURE SYNTHESIS



CLIMATE VARIABLE	VULNERABILITY
Increased Ambient Temperatures	Increased ambient air temperatures throughout the century, and over the next century are expected to create the following vulnerabilities on the business lines in the region: • Loss of vegetation from increased periods of drought and reduced streamflows may have impacts on vegetation within the region, which is important for sediment stabilization in the watershed. Loss of non-drought resistant vegetation may result in an increase in sediment loading, potentially causing geomorphic changes in the tributaries to the river system. • Decrease in flows may result from periods of drought and reduced streamflow has implications for maintain water levels in the rivers. **BUSINESS LINES IMPACTED:**
Increased Maximum Temperatures	Air temperatures are expected to increase 2-4°C in the latter half of the 21st century, especially in the summer months. This is expected to create the following vulnerabilities on business lines in the region: • Increased water temperatures leading to water quality concerns, particularly for the dissolved oxygen (DO) levels, growth of nuisance algal blooms and influence wildlife and supporting food supplies. • Increased evapotranspiration. • Human health risk increases from extended heat waves, impacting recreational visitors and increasing the need for emergency management. **BUSINESS LINES IMPACTED:**
Increased Annual Precipitation	By the middle of the century, annual precipitation is expected to increase in the region which are expected to influence the following vulnerabilities on business lines in the region: • Increased flows and runoff, which may carry pollutants to receiving water bodies, decreasing water quality. • Increased erosion with subsequent changes in sediment accumulation rates and creating water quality concerns. • Increased flooding, which may have negative consequences for all infrastructure, habitats, and people in the area. **BUSINESS LINES IMPACTED:**
Increased Storm Intensity and Frequency	Extreme storm events may become more intense and frequent over the coming century which are expected to influence the following vulnerabilities on business lines in the region: • Increased flows and runoff, which may carry pollutants to receiving water bodies, decreasing water quality. • Increased erosion with subsequent changes in sediment accumulation rates and creating water quality concerns. • Change in engineering design standards to accommodate new extreme storms magnitudes. • Increased groundwater recharge rates, as residence times are shortened within areas where evapotranspiration takes place during high intensity events. • Increased flooding, which may have negative consequences for all infrastructure, habitats, and people in the area. **BUSINESS LINES IMPACTED:**
Streamflow Variability	Streamflow will have more extreme variability's by the end of the century. This includes an increase in overall flow, an increase of peak flows, and an increase in low flow levels, which may result in: • Increased flows and runoff, which may carry pollutants to receiving water bodies, decreasing water quality. • Increased erosion with subsequent changes in sediment accumulation rates and creating water quality concerns. • Increased flooding, which may have negative consequences for all infrastructure, habitats, and people in the area. • Loss of vegetation from increased periods of drought and reduced streamflows may have impacts on vegetation within the region, which is important for sediment stabilization in the watershed. Loss of non-drought resistant vegetation may result in an increase in sediment loading, potentially causing geomorphic changes in the tributaries to the river system. • Decrease in flows may result from periods of drought and reduced streamflow has implications for maintain water levels in the rivers. **BUSINESS LINES IMPACTED:**
Sea Level Rise	Sea level rise may exacerbate saltwater intrusion into fresh water supplies. **BUSINESS LINES IMPACTED:**

NOTE: The Regulatory and Military Program business lines may be impacted by all climate variables

= Navigation = Flood Risk Management = Ecosystem Restoration = Hydropower = Recreation = Water Supply = Emergency Management

FIGURE 3(C): SUMMARY OF PROJECTED CLIMATE TRENDS AND IMPACTS ON USACE BUSINESS LINES

FIGURE 3(D): WATER RESOURCES REGION O2-MID-ATLANTIC REGION BOUND

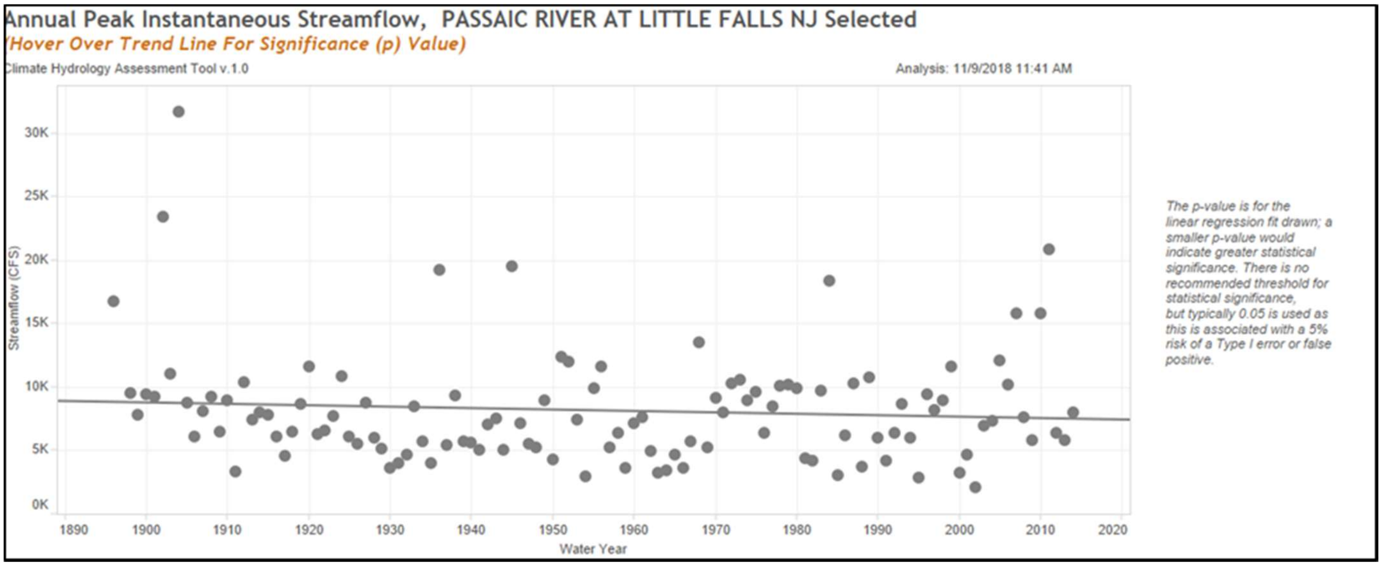


FIGURE 3(E): CLIMATE HYDROLOGY ASSESSMENT TOOL OUTPUT USING ANNUAL PEAK INSTANTANEOUS STREAMFLOW OF PASSAIC RIVER AT LITTLE FALLS

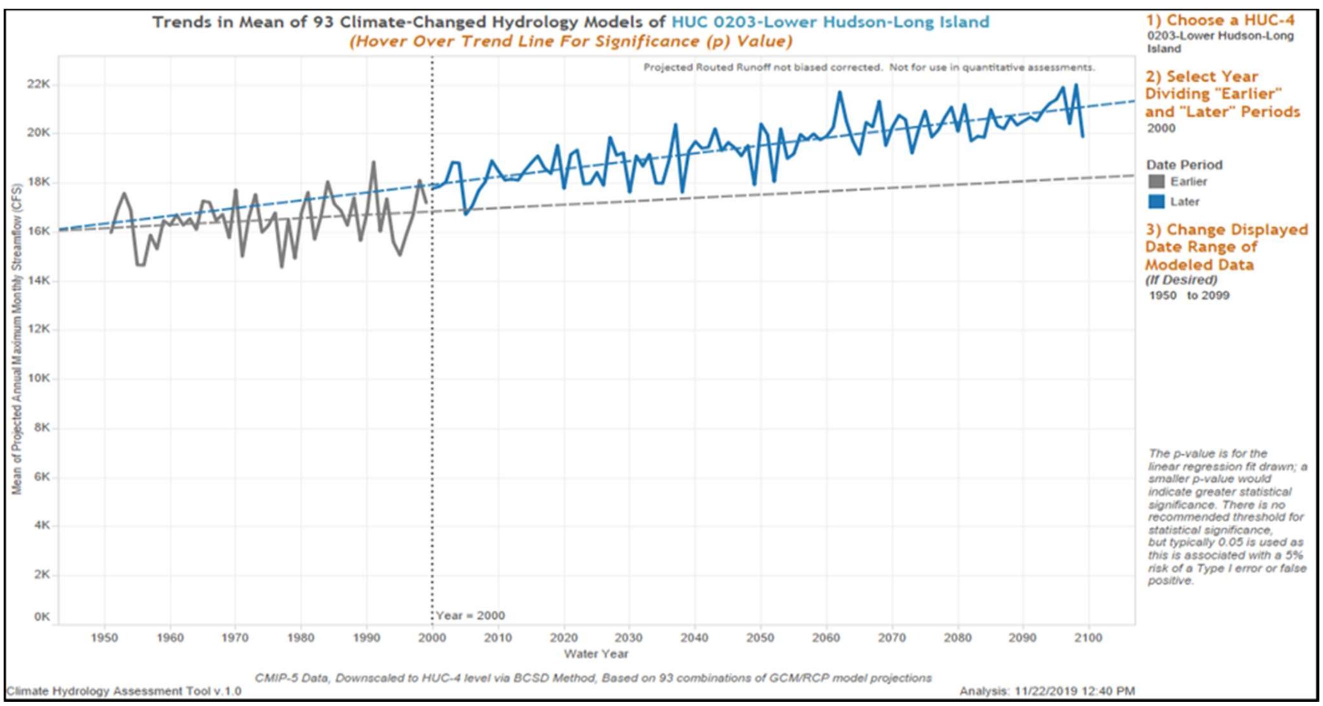


FIGURE 3(F): TRENDS IN PROJECTED MEAN ANNUAL MAXIMUM MONTHLY STREAMFLOW IN HUC 0203 – LOWER HUDSON – LONG ISLAND BASIN

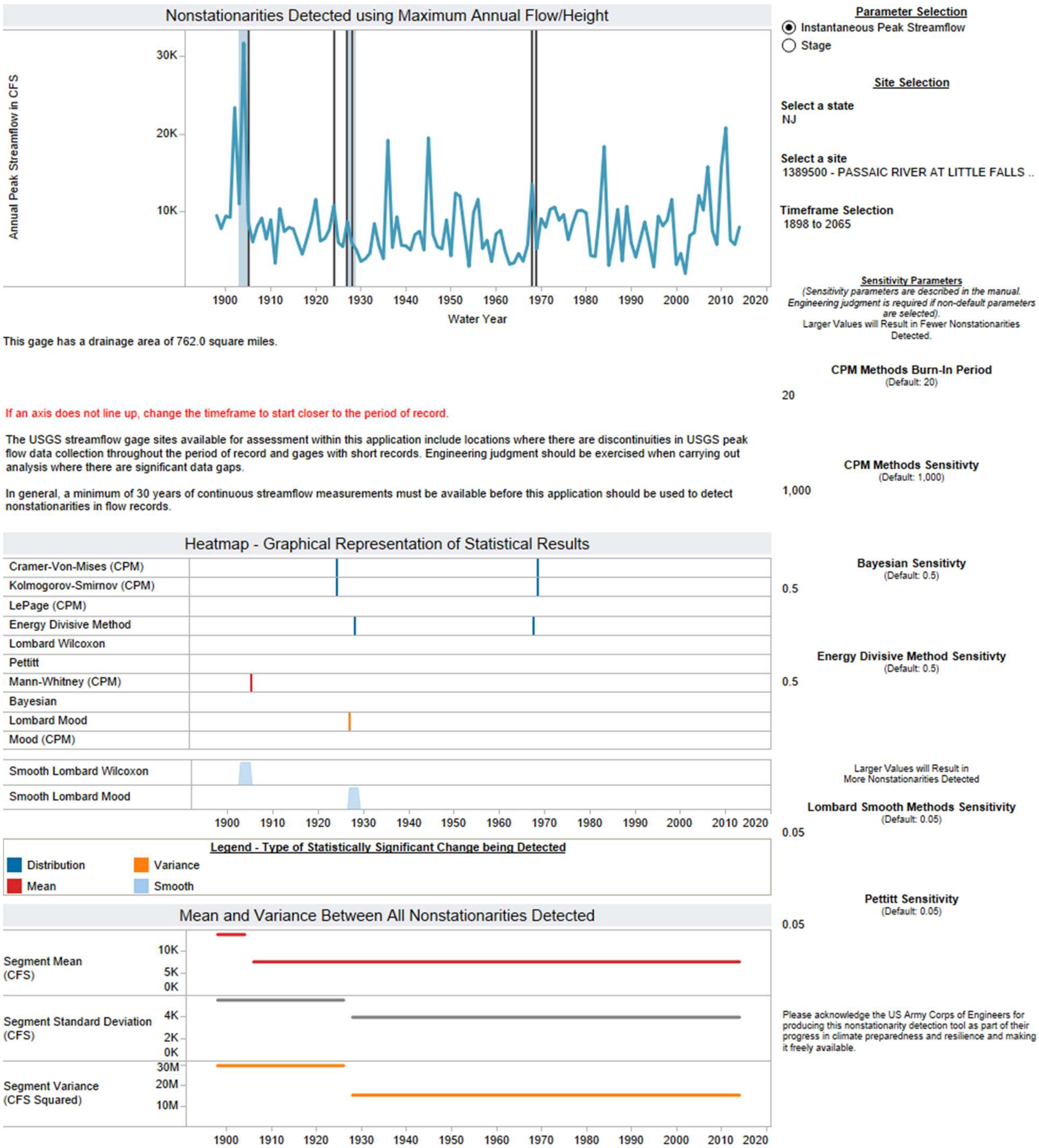
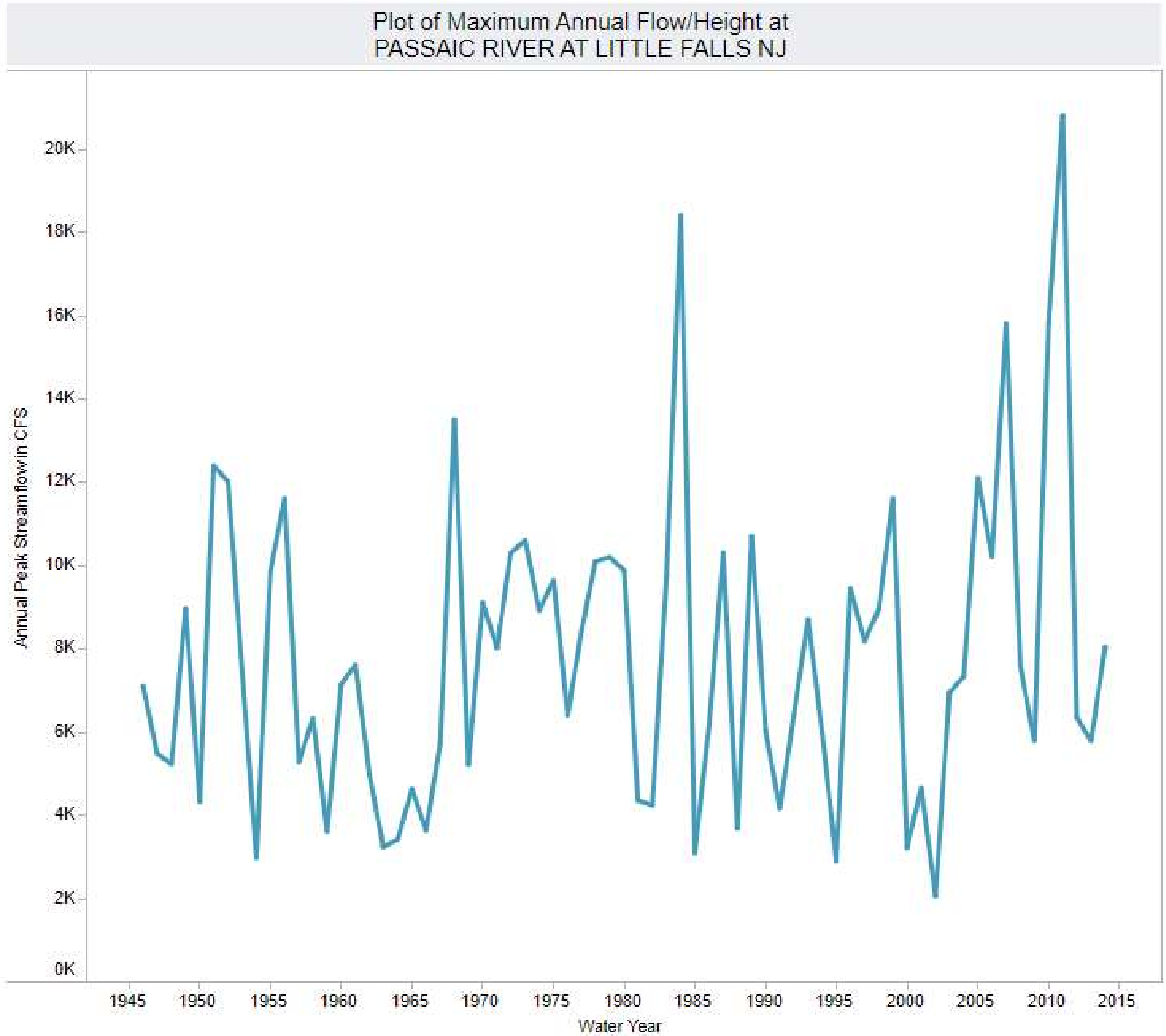


FIGURE 3(G): OUTPUT FROM NONSTATIONARITY DETECTION TOOL-PASSAIC RIVER @ LITTLE FALLS



Monotonic Trend Analysis

Is there a statistically significant trend?
No, using the Mann-Kendall Test at the .05 level of significance. The exact p-value for this test was 0.295.
No, using the Spearman Rank Order Test at the .05 level of significance. The exact p-value for this test was 0.293.

What type of trend was detected?
Using parametric statistical methods, **no trend** was detected.
Using robust parametric statistical methods (Sen's Slope), **no trend** was detected.

FIGURE 3(H): MONOTONIC TREND ANALYSIS RESULTS

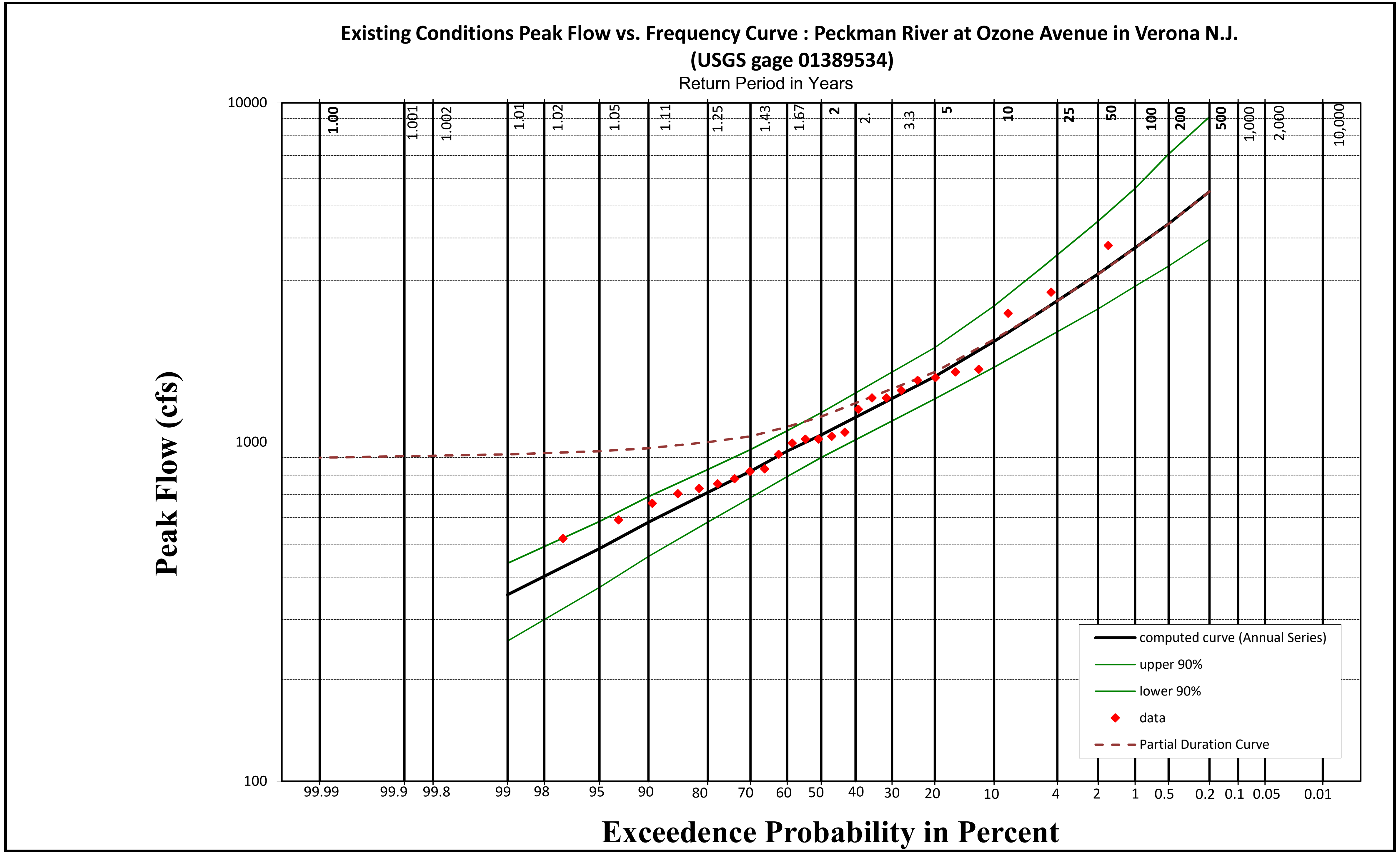


FIGURE 4: EXISTING CONDITIONS ANNUAL PEAK DISCHARGE VS. FREQUENCY: PECKMAN RIVER AT OZONE AVENUE

FIGURE 5: SUB-BASIN MAP OF PECKMAN RIVER BASIN

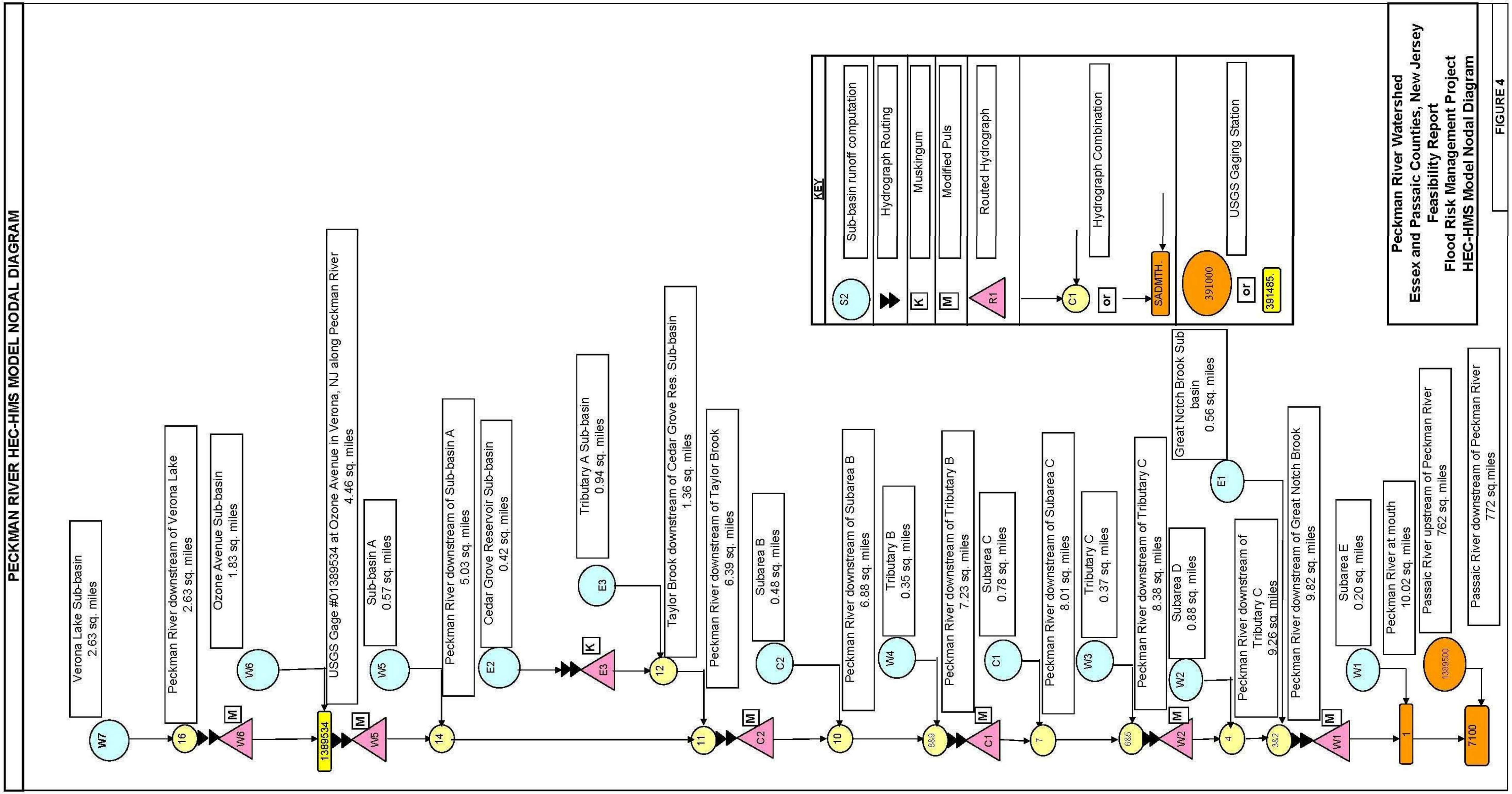


FIGURE 6: PECKMAN RIVER HEC-HMS MODEL—NODAL DIAGRAM

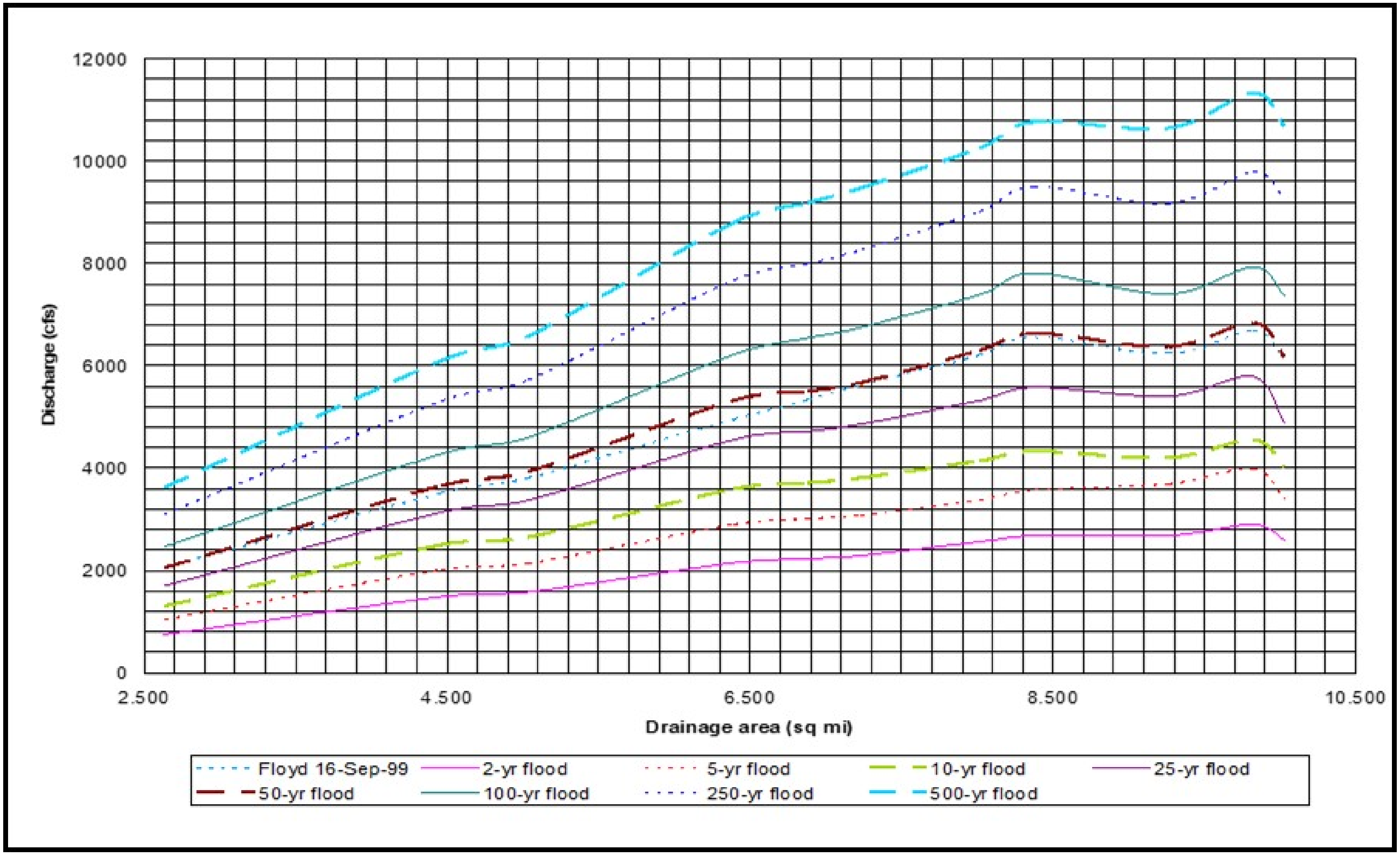


FIGURE 7: PEAK DISCHARGES VS. DRAINAGE AREA – EXISTING CONDITIONS

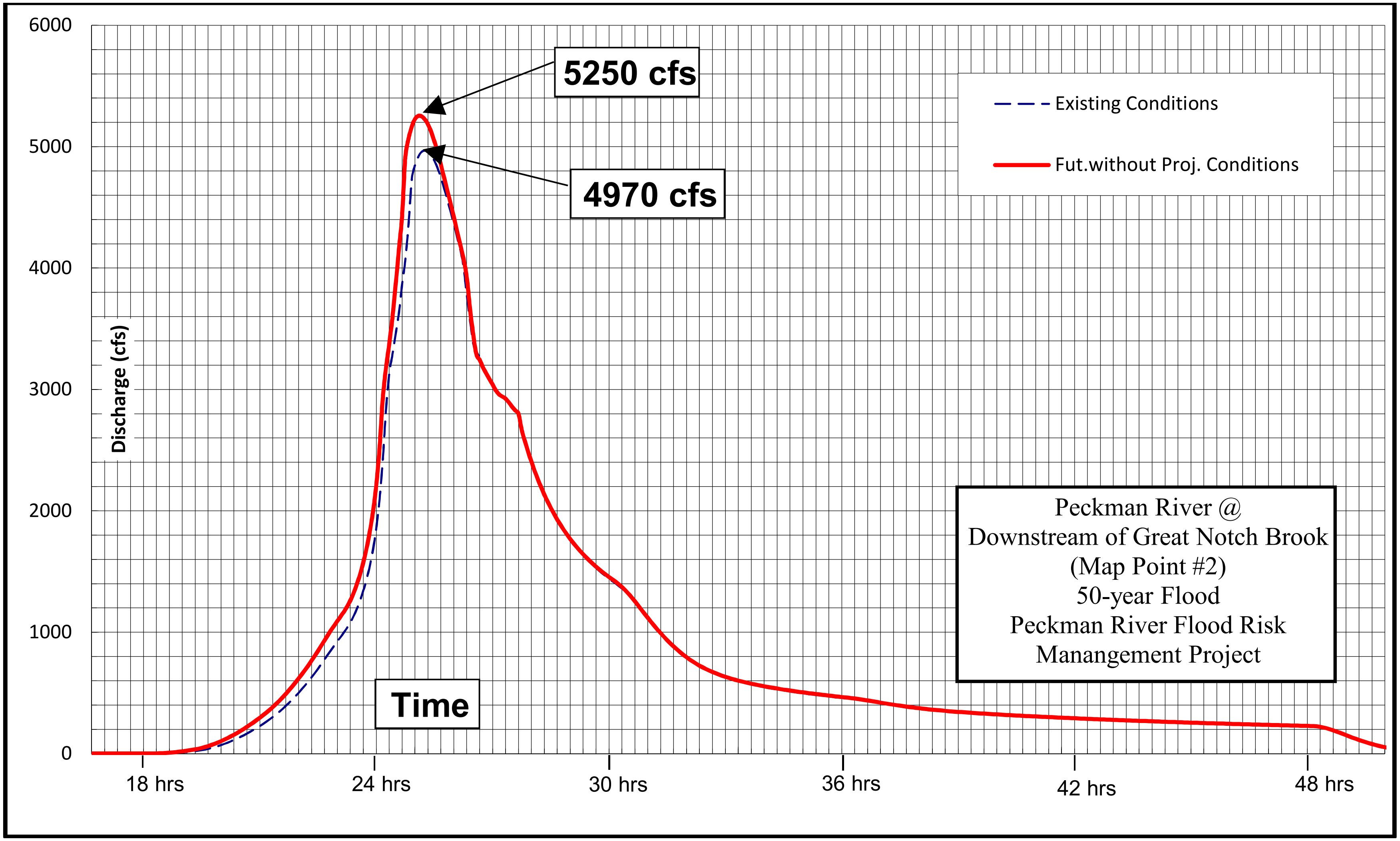


FIGURE 8: EXISTING VS. FUTURE WITHOUT PROJECT CONDITIONS HYDROGRAPHS – 50-YEAR RETURN PERIOD