PECKMAN RIVER BASIN, NEW JERSEY

FLOOD RISK MANAGEMENT FEASIBILITY STUDY

HYDRAULICS APPENDIX

US Army Corps of Engineers®
New York District

February 2020

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ESSEX AND PASSAIC COUNTIES, NEW JERSEY
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1.0 INTRODUCTION

1.1 Description of Study Area and Vicinity

1.1.1 General

The Peckman River Basin is located in the Essex and Passaic Counties of New Jersey. A tributary to the Passaic River, the Peckman River originates in the Town of West Orange and flows northeasterly through the towns of Verona, Cedar Grove, and Little Falls to its confluence with the Passaic River in Woodland Park (formerly West Paterson). **Figure 1** illustrates the map of the Peckman River basin.

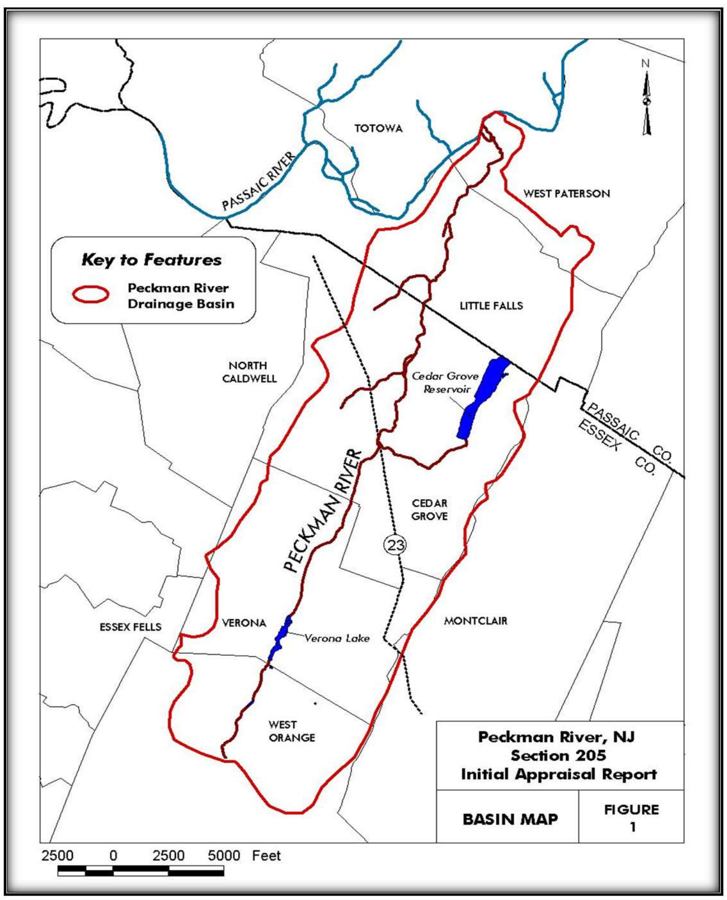

Figure 1: Schematic representation of the Peckman River Basin

Extensive development within the Basin has led to problems of flooding and ecosystem degradation. The majority of the watershed is heavily developed (71%). Residential housing comprises the largest subcategory at 50% and the undeveloped areas of remaining forested areas, reservoirs, and wetlands along the river corridor at 29%. **Figure 2** depicts an aerial representation of the Peckman River watershed.

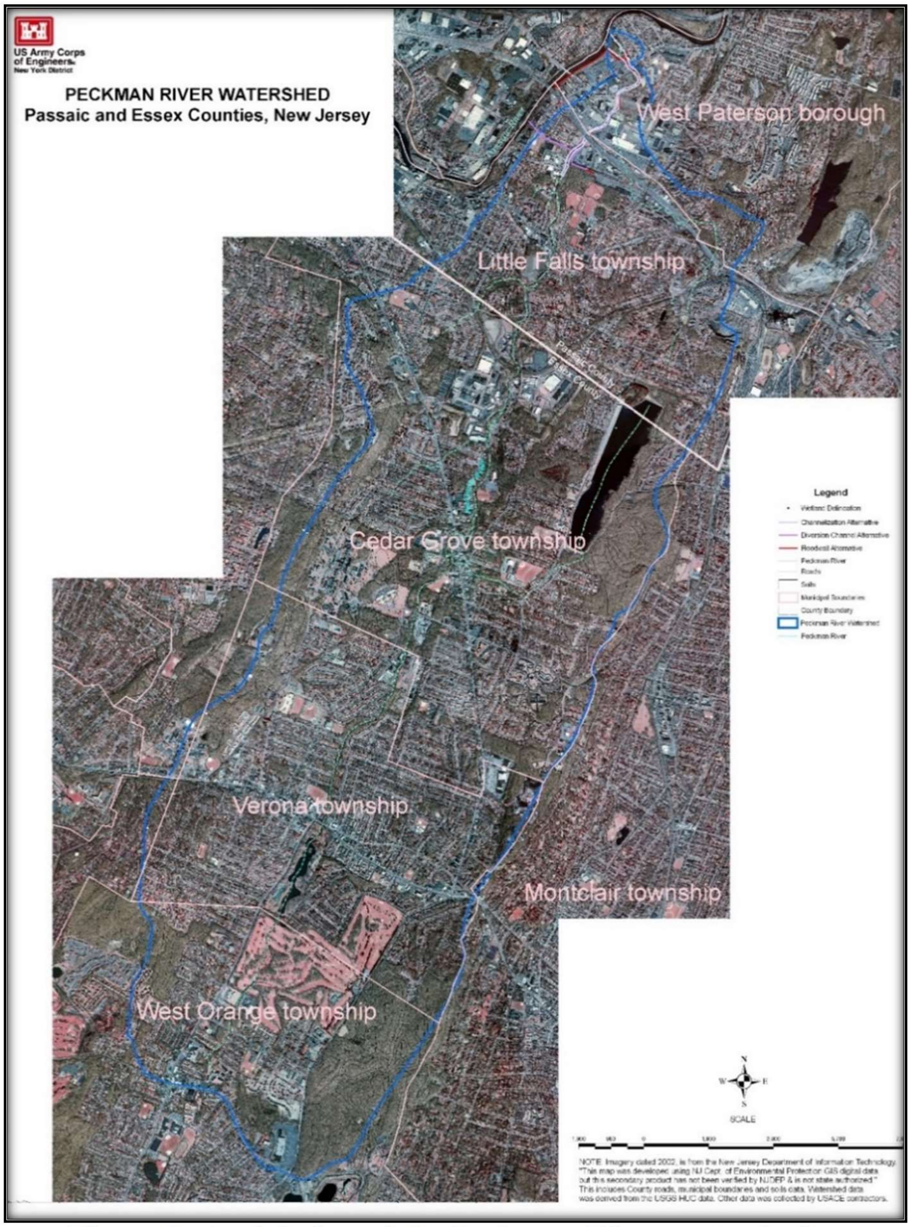

Figure 2: Aerial view of the Peckman River watershed

An Initial Appraisal Report (IAR) was prepared in July 2001 under the authority of Section 205 of the Flood Control Act of 1948 (PL 80-858) of the U.S. Army Corps of Engineers Continuing Authorities Program (CAP). This investigation demonstrated Federal interest in flood risk management in the Peckman River Basin and identified numerous opportunities for ecosystem restoration and/or enhancement. The drainage area is approximately 9.8 square miles and is one of the numerous major sub-watersheds of the Passaic River. As part of the Initial Appraisal Report, alternative structural plans considered included diversion of floodwaters from the Peckman River to the Passaic River, levees and floodwalls, and channel modifications of the Peckman River.

Commercial and residential development in the watershed has reduced the water holding capacity of the landscape and altered the natural dynamics of the river system. Storm events deposit large amounts of precipitation in the watershed, producing significant runoff. This quickly surpasses the capacity of the river channel, bridges, and culvert openings, resulting in overbank flooding which first begins to occur at approximatly the 0.2 annual exceedance probability (AEP) or the 5-year storm event frequency. The most intense flooding conditions occur in the Borough of Woodland Park and the Township of Little Falls. During Tropical Storm Floyd (1999), flood waters reportedly reached three to four feet of overbank flow causing an estimated $6.5 million in damages. Marked degradation of the river basin ecology has occurred with areas impacted by stream bank erosion, loss or riparian habitat, and the occurrence of invasive species (e.g., Knotweed).

1.1.2 Problem Identification

The Peckman River basin experiences flooding from several different sources: the Passaic River, the Peckman River, and the Great Notch Brook. The Great Notch Brook has a very small drainage area and is subject to flash flooding. The Peckman River is also a very flashy stream and typically peaks approximately an hour after Great Notch Brook. The Passaic River has a much larger drainage area and peaks approximately two to three days after the Peckman River. Flooding is caused by low channel capacity, in this urbanized area, and by backwater flooding from the Passaic. Generally about a 0.2 AEP causes flooding in the area. Near the mouth of the Peckman

River, the inundation mapping looks a bit like a three fingered glove with each river identifiable as a finger until they reach the palm of the glove where all the inundation limits merge into a single pool of water (excluding several high island like areas). Due to the significantly different sized drainage areas, there is not a good correlation between the Passaic and Peckman river flows.

1.2 Study Objective

The objective of this analysis is to evaluate the Federal interest and identify flood risk reduction measures along the Peckman River Basin in New Jersey. Federal interest (i.e., participation) in a project requires a demonstration of economic feasibility, which is established by determining whether the benefits to the national economy from the project exceed the annual economic costs.

As discussed below in Section 1.4 (Flood Prone Areas), there are portions of the Peckman River basin in Little Falls and Woodland Park that sustain periodic flooding. Other areas of minor flooding which may occur much further upstream will not be considered in this study. The damage reaches in this investigation are centered on Little Falls and Woodland Park. These two townships have various residential/commercial structures that are impacted from flood water. Flooding is further exacerbated as a result of the Passaic River backwater effect as described in Section 2.2.1 (Passaic River and Peckman River Historic Peak Flow Correlation).

1.3 Prior Studies

1.3.1 Detailed Project Report (DPR) for the Peckman River, Township of Little Falls, (September 1981)

The Detailed Project Report completed in 1981 for Peckman River covers the towns of Woodland Park, Little Falls, Cedar Grove, and Verona. Part of the evaluation included a determination as to whether a study of flood protection for Little Falls alone would be more favorable than pursuing a basin-wide solution. Nine flood control alternatives (6 structural; 3 non-structural) were considered for the basin, primarily in the Township of Little Falls.

It was determined that the flooding problem in Woodland Park is caused by both the Peckman River overbank flow and backwater from the Passaic River occuring separately at times and coincidentally at other times (see Section 2.2.1, Passaic River and Peckman River Historic Peak Flow Correlation for further details). In other words, if the Peckman River experiences a storm event but doesn't overtop it's banks, there is no guarantee that flooding wouldn't occur as a result of Passaic River backwater into the Peckman River during this same event. Since the formulation of flood control alternatives for the lower reaches of the Peckman River in Woodland Park would be affected by the formulation of plans for the Passaic River in Woodland Park, no further consideration was given to protecting this area as part of this study. At the time of this report, the flood problems in Woodland Park (West Paterson) caused by the Passaic River were scheduled to be studied under the Passaic River Basin GRR Study.

According to the results of this study, Cedar Grove's level of flood damages could not support a project. The Verona 0.1 AEP discharge was significantly below the 800 cfs requirement for a Corps of Engineers General Investigations study. Little Falls' focus was to protect the area between the State Highway, New Jersey's Route 46 upstream to Francisco Ave. For channel modifications to be effective in reducing flood levels in Little Falls, they would have to start downstream of Lackawanna Ave Bridge in Woodland Park. Nonstructural plans for 0.1, 0.02, and 0.01 AEP floodplains; 0.04 and 0.01 AEP channel improvements; 0.01 AEP levee for most significant damage reach, and 0.01 AEP SPF combination channel/levee plans were all economically infeasible alternatives.

Examination of the existing condition hydraulics analysis indicated that the area downstream of Route 46 is subject to fluvial flooding from the Peckman River. However, peak 0.01 AEP flows on the Passaic River cause extensive backwater flooding along the Peckman River from its confluence with the Passaic River to just downstream of Route 46. Therefore, any channel modification to the Peckman River in Woodland Park may only have a minimal effect in reducing flood damages from the Passaic River. For a channel improvement alternative to reduce the 0.01 AEP water surface elevation from the Peckman River to a non-damaging level upstream of the Route 46 Bridge, the channel modification would have to begin downstream at the Lackawanna

Avenue Bridge in Woodland Park. It was determined that optimal reduction in water surface elevations could be achieved upstream of Route 46 by modifying the channel with either a shallow cut channel having a bottom width of 100 feet or with a deeper cut channel having a bottom width of approximately 50 feet. However, for each modified channel size investigated, the cost versus benefit impact of lowering or relocating the 42" and 48" Pequannock Aqueducts and the 51" Passaic Valley Aqueduct which cross the Peckman River just downstream of the Route 46 Bridge was evaluated. In addition, a 24" sanitary sewer which crosses the river approximately 900 feet upstream of Route 46 had to be considered.

With a shallow cut channel in the Peckman River, the modified bottom slope would cross above the tops of the aqueducts and sanitary sewer. However, the large width of the channel would have encroached on private property necessitating the acquisition of land and structures. In addition, it would have been necessary to rebuild the Route 46 Bridge. Therefore, in order to obtain a 0.01 AEP level of performance, it was determined that it would be more economical to hold the channel bottom width to a maximum of 50 feet and further deepen it, thus requiring the relocation of the encasement of aqueducts and sanitary sewer. Although less expensive than buying out large tracts of private property and rebuilding the Route 46 Bridge, relocating these utilities is still a costly item. Without lowering the aqueducts or sanitary sewer or without encroaching on private property, it was determined that the maximum level of performance that could be provided by a channel modification would be against a 0.04 AEP flood event.

As indicated in the 1981 DPR, a report entitled, "Passaic River Report" was prepared by the Corps of Engineers, New York District in June 1972. This report provides background information on the Peckman River. The recommended plan in this report included protective structures along the reach of the Peckman River within the backwater influence of the Passaic River in Woodland Park.

It was concluded at that time that structural and non-structural alternatives for flood control on the Peckman River in Little Falls were not in the Federal interest based on benefit-cost ratios that ranged from a low of 0.10 to a high of 0.27. However, the basis of the hydrologic and hydraulic (H&H) analysis used in this report is unclear.

The reliability of available H&H (hydrology and hydraulic) data has been be enhanced for the current analysis since the installation of a United States Geological Survey (USGS) gage in Verona, which has recorded stream flow data from 1979 to the present. In 2007 an additional USGS gaging station (rain and stream flow) was installed in Little Falls at the Francisco Ave. Bridge and data continues to be collected. Both gaging stations will aid in the success of future modeling efforts.

1.3.2 General Design Memorandum (GDM), Passaic River Flood Damage Reduction Project (September 1995)

The purpose of the GDM was to refine the analysis and design of the Passaic River Flood Control Project Recommended Plan, which included the construction of a flood tunnel for diversion of Passaic River flood waters. Implementation of the Recommended Plan was expected to significantly reduce Passaic River flooding in areas of Woodland Park that are subject to inundation from flood waters from both the Passaic and Peckman Rivers.

In the GDM, Flood Insurance Study (FIS), and the Detailed Project Report of 1981, the Passaic River was assumed to be the primary source of flooding for Woodland Park; therefore, preliminary indications were that a reduction in flooding from the Passaic River would significantly reduce flooding in Woodland Park. No detailed analysis was performed on how the Passaic River Project would have affected Peckman River flooding within Woodland Park due to the reduction in backwater influence.

Although the specific dependence or independence of Passaic River and Peckman River flooding events have not been analyzed in this 1995 GDM, the Peckman River H&H data developed for this current report indicate that the Passaic River is a more significant source of flooding in Woodland Park and Little Falls than previously considered in the Passaic River GDM, the Flood Insurance Study, or the Detailed Project Report of 1981.

1.3.3 Peckman River Basin Initial Appraisal Report (July 2001)

The purpose of the Section 205 Initial Appraisal Report (July 2001) was to conduct an appraisal for flood protection opportunities and to evaluate the potential for Federal interest in flood damage reduction within the Peckman River Basin. Flooding in the Peckman River Basin results primarily from two sources: flash flooding from rapid runoff in the Peckman River watershed and backwater from the Passaic River. Despite the highly urbanized nature of the Peckman River basin, continued development is expected to increase the runoff and exacerbate the flooding in Woodland Park and Little Falls which have been identified as the predominant damage centers.

For the Peckman River Section 205 Initial Appraisal Report, structural alternatives providing flood damage reduction up to approximately the 0.02 AEP design level were evaluated. Alternative structural plans considered include diversion of flood waters from the Peckman River to the Passaic River, earthen levees and concrete floodwalls, and channel improvements to increase channel capacity. The estimated costs (2001 price level) of the structural alternatives considered in this analysis ranged from approximately $16 million for the diversion culvert, to $30 million for channel improvements, and $40 million for levees and floodwalls. It was expected that the annual benefits of one or more of these alternatives would exceed the estimated annual costs (Initial Appraisal Report July 2001). The diversion culvert alternative appears to be the most economically viable of the alternatives evaluated. The conclusion of the Peckman River Section 205 Initial Appraisal Report was that benefits of flood damage reduction measures would exceed the project costs resulting in positive contributions to the National Economic Development (NED) account.

The hydrologic and hydraulic data generated by the Federal Emergency Management Agency (FEMA) to develop the Flood Insurance Rate Map (FIRM) is sufficient for an Flood Insurance Study (FIS) level of analysis. However, a comparison of the USGS frequency analysis of peak flows on the Peckman River and the frequency-discharge curve presented in the Woodland Park FIS indicated that the results of the H&H analysis in the FIS may be obsolete (**Table 1**). The values for the USACE (New York District) data (2006) were obtained through the H&H effort that was conducted as part of the Existing conditions modeling effort.

Table 1: Comparison of discharges for the Peckman River at the confluence of the Passaic River.

Annual Excedance Probability	USGS[1] (cfs)	USACE[2] (cfs)	FEMA-FIS[3] (cfs)
0.10 (10-year)	3,580	4,020	1,220
0.02 (50-year)	5,780	6,170	1,800
0.01 (100-year)	6,980	7,370	2,200

1. Values obtained from Table 1, Peckman River frequency-discharge comparison (at the confluence with the Passaic River), Peckman River Basin Initial Appraisal Report (July 2001).
2. Values obtained from the current H&H effort, USACE (New York District, 2006).
3. Values obtained from the FEMA FIS for the Borough of Woodland Park (June 15, 1981).

There is appreciable difference between the hydrologic data developed by USGS, USACE, and FEMA. It is apparent that the FEMA-FIS discharges are much lower than the reported USGS data or the USACE. Although flood plain mapping was not created from the data in the Peckman River Section 205 Initial Appraisal Report (July 2001), the hydrology does indicate that a larger flood plain would be anticipated since the discharges are much higher. The larger flood area shown in the flood delineation maps for Peckman River produced in 2006 should therefore not be unexpected.

Based on the higher peak Peckman River flows identified in the Initial Appraisal Report, it may now be assumed that there are greater Peckman River influences on Woodland Park than originally thought. Flood protection alternatives along the Peckman River were assumed to protect against Peckman River flooding only. Unless a flood control alternative specifically blocked flooding from the Passaic River, it was assumed that all areas were still subject to damages from a Passaic River flooding event. Therefore flood control alternatives only provided benefits within the Passaic River backwater area if Peckman River stages were estimated to be higher than Passaic River flood stages. Only benefits upstream of the Passaic River flood stages were subsequently considered in the Initial Appraisal Report.

From the Initial Appraisal Report, structural alternatives providing flood damage reduction up to approximately the 0.02 AEP design level were evaluated. Structural plans considered included the diversion of flood waters from the Peckman River to the Passaic River. The diversion of flood waters upstream of the damage centers in Woodland Park and Little Falls were evaluated. Upstream of Route 46, flood water could be diverted from Peckman River to the Passaic River through a 1,450-foot long, 30-foot wide, by 10-foot high closed culvert located approximately 550 feet upstream of the Route 46 Bridge. The culvert would be constructed using a "cut-and-cover" approach. The diversion culvert would include a spillway (weir) and a channel constructed with additional levee sections to limit the flow downstream and create a pooling area near the inlet spillway. Nonstructural alternatives were not considered due to significant commercial and industrial development.

The current effort for a diversion culvert is based on the same river station as the prior location noted in the Initial Appraisal Report. A weir/spillway will be needed to convey flow into the diversion culvert. There will also need to be a stilling basin for the flow to enter the Passaic River since there will be a hydraulic jump due to a rapid energy change. The dimensions for the proposed diversion culvert necessary to convey the 0.02 AEP event includes a base of 41-foot and easements will need to be included on either side since 2/3rd of this culvert will be open channel. Of the 1,500 feet length of culvert, approximately 500 feet will need to be cut and cover (where the culvert passes under the road/parking lots).

1.4 Flood Prone Areas

There is a history of flooding and flash flooding in the Woodland Park and Little Falls sections of the Peckman River, e.g., as described in the 1995 GDM. These areas have various residential/commercial structures that are impacted from flood water until levels within the Peckman River recede. Flooding is further exacerbated as a result of the Passaic River backwater effect as described above. Flooding from the Peckman River is generally a much shorter duration than the flooding caused by the Passaic River backwater. The Passaic River has a greater time to rise (T_R) as a result of a much larger drainage area, e.g., 935 square miles, whereas the watershed for the Peckman River is approximately 9.8 square miles. Given this substantial difference in drainage area the time of rise for the Passaic River is on the order of days rather than hours as observed for the time of rise for the Peckman River. Consequently, flash flooding from the Peckman River is a greater concern upstream of Route 46 and downstream in Woodland Park.

Tropical Storm Floyd (Sept 1999) reportedly caused flooding on Hopson Avenue in Little Falls which was the result of Peckman River overbank flow. This flood event extended east and caused damage to the athletic field at the Little Falls High School. Overbank flow was also observed during Tropical Storm Floyd immediately upstream of the Route 46 Bridge. It was reported that debris might have created an obstruction of flow within the Route 46 culvert which caused higher water surface elevations upstream of Route 46. Portions of highway Route 46 were inundated during this weather event. Left overbank flooding occurred in the area of Willow Ave. and Jackson St. and the flow created a natural overland diversion that extended to the Passaic River.
Peckman River overbanking in Woodland Park typically causes flooding (frequencies above 0.2 AEP or 5-year return period) at the Memorial Middle School on Memorial Drive. The neighborhood east of the middle school, which includes Dowling Parkway and Wallace Lane, is inundated as well. Overbanking of Dowling Brook as a result of the Passaic River backwater effect will also add to flooding in this area.

The neighborhood in Woodland Park that is west of McBride Avenue is especially prone to flooding from the Passaic River backwater. Bergen Boulevard, Rockaway, Pompton, and Whippany Avenues are typically inundated by the Passaic River 0.1 AEP event while the Peckman River does not have an impact in this area until a much greater return period (i.e., frequency 500-year event).

1.5 Peckman River and Great Notch Brook Floodplain Hydraulics

The local residents and officials repeatedly told the Corps of Engineers about flooding along the Great Notch Brook and expressed concern that flooding in the retail area of Kohl's and Best Buy would not be addressed by this study. This area is prone to flash flooding from the both Great Notch Brook and the Peckman River. Great Notch Brook and Peckman River are not hydraulically separable and both contribute to flash flooding at this location.

The drainage basin for Great Notch Brook is approximately 0.6 square miles with its confluence on the Peckman River immediately downstream of the Route 46 Bridge. As noted above the drainage basin for Peckman River is approximately 9.8 square miles. The 1.0 AEP discharge on Great Notch Brook is 270 cfs while is 1880 cfs on the Peckman River at the confluence with Great Notch Brook. The 0.01 AEP discharges for Great Notch Brook and the Peckman River are 880 cfs and 5,420 cfs, respectively, for future unimproved conditions. **Figure 3**: Pedestrian Bridge over Great Notch Brook at the Commercial Retail Property, Woodlawn Park, NJ. and **Figure 4** show the retail parking lot on the right over bank of Great Notch Brook and a pedestrian bridge crossing the brook. **Figure 4**, facing downstream on Great Notch Brook shows the base flow conditions of approximately 5 cfs (photo taken ca. June 2004). **Figure 5** is a picture taken during the June 30th, 2009 weather event and illustrates the extent of flooding at the retail area (and parking lot) in the vicinity of the pedestrian bridge. As shown in the photo, the area is completely flooded and the water surface elevations are level indicating that this is not a local storm water drainage sewer issue but a greater flooding issue with either Peckman River and/or Great Notch Brook. The June 30th, 2009 weather event produced 1.25 inches in less than 2 hours in the Peckman River and Great Notch Brook basins. Flood hydrograph rose and fell very quickly suggesting short duration storm causing flash flooding which plagues this community. The corresponding discharge from this weather event was 944 cfs on Peckman River measured at the USGS 01389550 Gage, Peckman River at Little Falls, New Jersey. The discharge from Great Notch Brook is estimated to be less than 270 cfs and contributions for Peckman River backwater effect. **Figure 6** shows landmarks in the area mentioned in the text of this appendix.

Figure 3: Pedestrian Bridge over Great Notch Brook at the Commercial Retail Property, Woodlawn Park, NJ.

Figure 4: Looking Downstream on Great Notch Brook Facing the Pedestrian Bridge, Base Flow Conditions ca. June-2004.

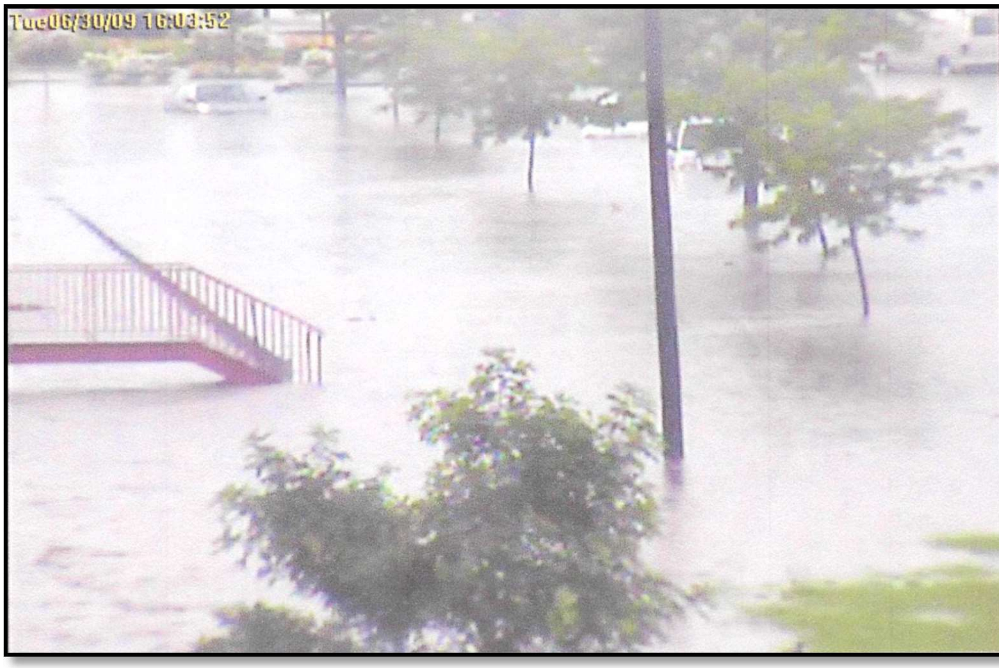

Figure 5: Pedestrian Bridge over Great Notch Brook during the 30-June-2009 Weather Event.

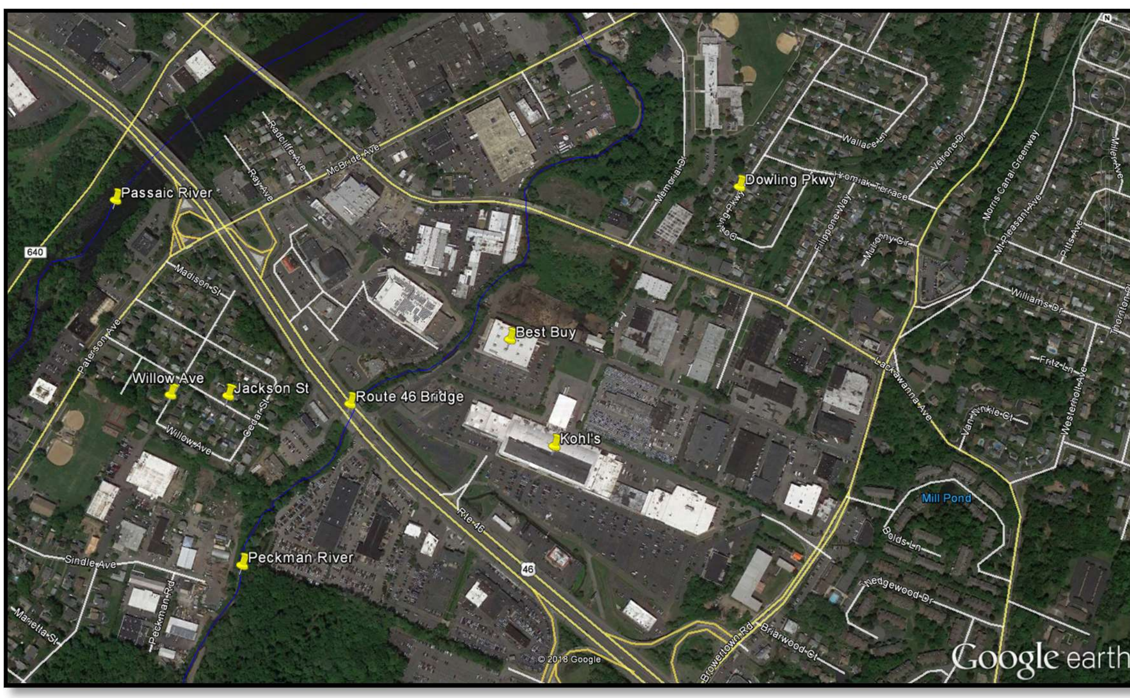

Figure 6: Landmarks mentioned in text

2.0 HYDRAULIC ANALYSIS

The approach to the study objectives described in Section 1.2, was achieved through hydraulic modeling of the Peckman River and using that model to evaluate a number of proposed alternatives, to obtain a viable soultion. Different combinations of flood control components were analyzed as various alternatives to obtain the most economical, but effective solution. These alternatives are described in Section 3.1. The approach begins with a steady state hydraulic model of the Peckman River. An existing conditions steady state hydraulic model was created and compared against all the formulated plan alternatives as with-project conditions. This comparison lead to the alternative with the highest net benefits. Alternative 10b (see Section 3.1 for a detail description of the alternative) is the Tentatively Selected Plan (TSP) with the highest net benefits.

On August 11th, 2018, the town of Little Falls experienced a flash flood, the worst flooding event since Tropical Storm Floyd. More than 3 inches of rain fell onto the already saturated ground in a short time span. The flood was so sudden that a Little Fall's car dealership adjacent to the Peckman River had 16 vehicles swept into the river. The vehicles obstructed the face of the Route 46 bridge further exacerbating the flooding in the area. The event triggered a closer look into the steady state hydraulic model representing the TSP. Due to the hydraulics characteristics of the area, it was decided that a 1D unsteady model with a 2D component would be a better modeling approach to represent the area. A steady flow model would not be able to accurately capture the full magnitude of the flood waters overtopping the river's right bank and traveling inland towards and onto Hopson Avenue and Great Notch Brook. Further advancements of HEC-RAS software at the time allowed USACE to reconstruct the hydraulic model using unsteady flow hydrographs and a 2D flow area. While steady flow models don't calculate changes in volume or use time in calculating a solution, unsteady models utilize both variables giving a greater degree of accuracy to such a challenging area to model. Having a 2D flow area along the right bank of the Peckman river allowed USACE to accurately model the flow path and movement of water towards Hopson avenue.

Past studies assumed a complete dependence of frequencies of flooding on both the Passaic and Peckman Rivers. It was assumed that, when a 0.04 AEP, or flood of any given frequency, occurred on the Peckman River, a 0.04 AEP flood, or flood of the same frequency, would also occur on the Passaic River upstream of the Peckman River. Due to the disparity of the drainage area sizes: 762 square miles in Passaic River at Little Falls NJ, USGS gage (near the confluence with the Peckman), and 10 square miles in Peckman River at mouth, and 4.45 square miles in Peckman River at Ozone Avenue in Verona NJ USGS gage, this occurrence of all the peaks occurring simultaneously is not feasible. A correlation analysis was conducted as part of the initial study to obtain a more accurate representation of the flows throuout the basin.

The extent of the backwater flooding from the Passaic River up the Peckman River and the degree of the coincidental upstream flows into the Peckman River was considered by a joint probability analysis using the unsteady modeling output data. Hydrologic Engineering Center's (HEC) Statistical Software Package (HEC-SSP) was used to perform this analysis. Refer to Section 2.3.4 for a detailed discussion of this topic.

2.1 Modeling Approach - Channel Cross-Sections and Geometry

Channel cross-sections for this study were obtained from TVGA Consultants, 1000 Maple Road, Elma, NY, under contract number DACW51-01-D-0008, date of survey: April through June 2004. The survey for the Peckman River basin included aerial photography, topographic mapping, stream cross-sections, utility survey, flood mark survey and geographic information system (GIS). Coordinates are expressed in U.S. survey feet and referenced to the North American Datum of 1983 (NAD83) and elevations are expressed in U.S. survey feet referenced to the National Geodetic Datum of 1929 (NGVD29). The datum was converted to North American Vertical Datum of 1988 (NAVD 88) for the HEC-RAS models.

The HEC-RAS model for Existing Conditions was completed in 2008 with 220 channel cross-sections for Peckman River. The average distance between surveyed cross-sections is approximately 200 feet and ranges from elevation 25 to 650 feet. Overbank cross-section data was obtained from the 2004 topographic mapping performed by TVGA. In 2018, the overbank terrain was updated with LIDAR data from the Passaic River FIRM model from 2006/2007.

Cross-sections at bridges were taken at their immediate downstream and upstream faces on bridge waterway openings and include piers, structural low steel, and tops of roadways. This hydraulic data was used as input to develop the existing conditions water surface profiles via a HEC-RAS model. Locations of the bridges, with their representative cross-sections, are shown in Table 3.

The existing conditions HEC-HMS model of the Peckman River Watershed includes rudimentary storage data for both the channel and overbanks. More accurate and comprehensive storage data was obtained from the HEC-RAS model and input to the HEC-HMS model to improve the results of the Modified Puls hydrograph routing procedure.

Peckman River originates in West Orange and flows a distance of approximately 40,300 feet where it discharges into the Passaic River in Woodland Park. The existing channel alignment of Peckman River exhibits relatively little meandering with some slight bends. A more appreciable bend in the alignment occurs in the lower reach of the river in Woodland Park in the area of Memorial Drive (refer to **Figure 1**). There is a 90° bend to the left in the channel alignment at this location and overbank flow may occur at the 0.1 AEP and higher. The elevation change along the river is approximately 260 feet with the most significant drop occurring within Cedar Grove (refer to **Figure 7**). Given a 260 foot drop in elevation over the total river reach of 40,300 feet the average channel slope is 0.0065 feet/foot. The slope in the project area with the focus in economic damages (lower reach) is 0.0055 feet/foot while the upper reach is 0.0111 feet/foot.

Much of the watershed is heavily urbanized. Residential housing developments comprise the largest sub-category. Undeveloped areas consist of forested areas, reservoirs and wetlands along the river corridor. **Figure 1** and **Figure 2** indicate the location of the Cedar Grove Reservoir (in Cedar Grove) and Verona Lake in Verona is also noted. Great Notch Brook is a tributary to the Peckman River, entering the river just downstream of Route 46. Great Notch Brook is subject to extremely rapid runoff from higher elevations in the lower eastern side of the watershed. Two other small tributaries enter the river in Cedar Grove. 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. During flooding events, diversion of flow from the Peckman River across Woodland Park to Dowling Brook has been reported as a result of overbank flow noted above.

The lower portion of Peckman River is where the economic damage areas are concentrated. Manning's n-values are given in **Table 2** for the designated river stations. **Table 3** indicates existing bridge section information for the 22 bridges that are part of the Peckman River hydraulic model. At river station RS 8670, there is a dam which has been scoured and subsequently breached with base flow going under the structure.

Table 2: Manning's n-value at designated river station.

River station	Left Bank	Channel	Right Bank
0 - 1000	0.050	0.060	0.080 – 0.090
1000 - 2300	0.065 – 0.080	0.040 – 0.060	0.060 – 0.090
2300 - 3500	0.060 – 0.080	0.050 – 0.060	0.060 – 0.090
3500 - 4550	0.060 – 0.080	0.050 – 0.060	0.070 – 0.080
4550 - 5480	0.060 – 0.065	0.040	0.060 – 0.080
5480 - 6450	0.065 – 0.080	0.045	0.060 – 0.070
6450 - 6850	0.065	0.045	0.080
6850 - 8300	0.060 – 0.090	0.045 – 0.050	0.070 – 0.085
8300 - 9400	0.060 – 0.080	0.040 – 0.045	0.070 – 0.080
9400 - 10750	0.060	0.040 – 0.045	0.070 – 0.080
10750 - 11550	0.070	0.045	0.070

Table 3: List of Bridge locations and stations.

Bridge	Station (RS)	Type/ Opening	Width of Opening Normal to Flow (ft)	Area Normal to Flow (ft^2)	Low Chord Elevation (ft)	Upstream Invert Elevation (ft)
McBride Ave.	1200	Single	69	283	127.14	117.30
Lackawanna Ave.	2662	Single	64	488	131.00	121.70
Rt. 46	4284	Single	142	295	135.90	128.40
E. Main Street	7737.5	Double	57	433	154.40	144.40
Francisco Ave	10279.5	Single	57	389	174.80	165.30
Commerce Rd	13405.5	Single	65	n/a	209.50	196.10
Little Falls Rd	14976	Single	52	n/a	216.50	209.80
Pompton Tpk	18253.5	Single	67	n/a	282.70	265.10
Bradford Ave	21155.5	Single	61	n/a	301.40	292.20
Ozone Ave	22312.5	Single	55	n/a	305.50	298.20
Linden Ave	26473.5	Single	51	n/a	329.10	321.40
Bloomfield Ave	28325.5	Double	75	n/a	340.70	332.90
Verona Lake Dam	28891	Single	12	n/a	343.00	337.70
Verona Lake	30388	Single	11	n/a	356.50	347.10
Verona Lake Bridge	31370	Triple	16	n/a	351.18	347.20
Club Access	34566	Single	16	n/a	363.60	357.30
Woodland Ave	25850	Single	49	n/a	370.70	365.40
Culvert at townhouses	36965	Single	48	n/a	382.85	379.89
Waldeck Ct	37450	Single	40	n/a	399.40	392.40
Forest Ave	37675	Single	48	n/a	405.30	399.30
Subdivision	39130	Single	49	n/a	453.20	447.60
Prospect Ave	40250	Single	50	n/a	495.50	485.90

Note: Designation of "n/a" indicates that the bridge is out of the project area and not of concern.

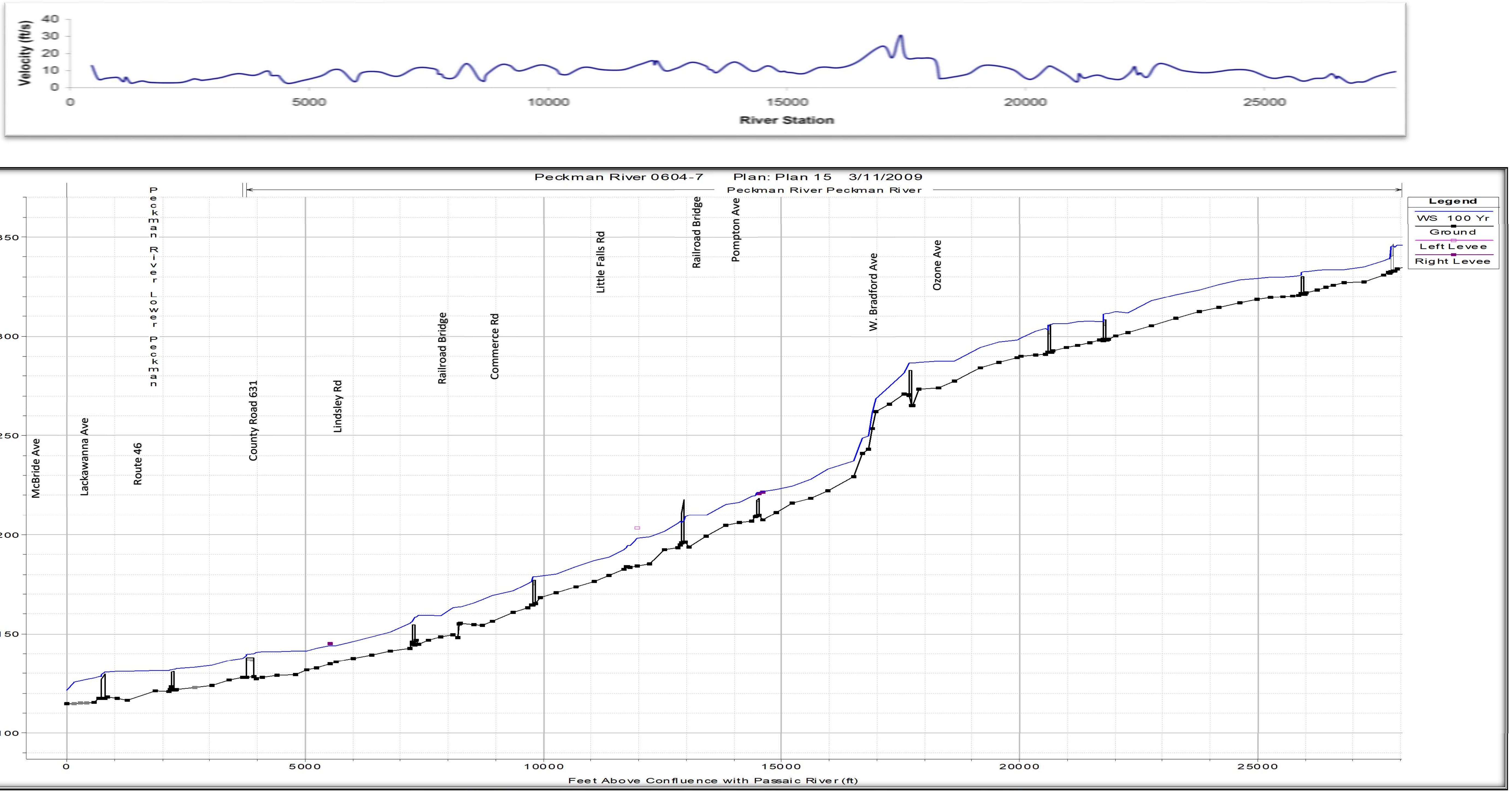

Figure 7: 100-year Water Surface Profile with corresponding water velocities.

2.2 Hydraulic Analysis

2.2.1 Passaic and Peckman Rivers Peak Flow Correlation Analysis

The historic record of annual peak flows of the Peckman River at Ozone Avenue in Verona NJ, USGS gage (water years 1945, and 1979 through 2007, 30 water years total) was obtained and the historic record of annual peak flows at the Passaic River at Little Falls NJ, USGS gage was also obtained for these same 30 water years, 1945 and 1979-2007. The storms that produced the annual peak flows at the Peckman River USGS gage were compared to peak flows produced at the Passaic River at Little Falls NJ, USGS gage. For only nine of the 30 common water years of record did the same storm produce the annual peak flows at both gages. In the remaining 21 common water years of record, the storms that produced the annual peak flows on the Peckman River at its gage produced something less than the annual peak flow at the Passaic River at Little Falls gage. These lower peak flows were estimated from three mean daily flows (the maximum, and preceding and succeeding days) at the Passaic River at Little Falls gage, using a USGS monograph. Use of a longer historic record could impact the values for flow frequency and uncertainty bands. There is a risk that the 30-year record used in the analysis doesn't fully represent the entire range of annual peak flows. Therefore, there is a risk that the feasibility-level design for the proposed project features may not be optimal. Based on a review of the gage data and known dynamics of the Peckman River, the PDT is willing to accept this risk during the feasibility study. Additional USGS gage data will be incorporated into the PED hydraulic analysis, as available.

A direct quantitative assessment of the hydrologic dependence of these two gaged watersheds would be difficult or impossible to develop. However, by working within a context of complete dependence (working with peak flows produced by the same storm at both gaged watersheds) and comparing the magnitude and frequency of flood peaks produced in both watersheds by the same historic storm, a usable approximate solution can be found, and a conclusion reached.

The peak flows produced by the same storm within any given water year at both gaged watersheds were tabulated, and a correlation analysis was done using the Corps HEC-supported program

MLRP (Multiple Linear Regression) which finds the best-fit relation to input data using the method of least squares. Peak flow data for both watersheds was transformed to common (base 10) logarithms to obtain the best possible correlation. The peak flow of the Peckman River at its gage was taken as the independent variable, because the annual peak flows at this gage were known for all 30 years of record, and did not need to be estimated from mean daily flows. The peak flow of the Passaic River at the Little Fall's USGS gage was taken as the dependent variable, because it did have to be estimated from mean daily flows for 21 of the 30 common water years of record. The data set was executed with program MLRP. The correlation was found to be positive but weak, with a simple correlation coefficient of 0.30, less than half. The resulting coefficient of determination (R bar squared) was found to be 0.0907, which is unacceptable for any predictive capability. The standard error of estimate (in log units) was 0.3750, working out to a multiplication (plus one Se) or division (minus one Se) by the antilog of 0.3750 or 2.3714. Results were similar for the untransformed data: a correlation coefficient of 0.352, and a standard error of estimate of 4727 cfs.

Next, partial duration exceedance frequencies in percent were determined for the peak flows in the data set, and an attempt was made to correlate these exceedance frequencies for the 30 water years of common storm and flood events at the two aforesaid USGS gages.

The exceedance frequencies data set was executed with program MLRP. Results were similar to those for the peak discharges of the common events. The correlation was found to be positive but weak, with a simple correlation coefficient of 0.33, less than half. The resulting coefficient of determination (R bar squared) was found to be 0.1097, unacceptable for any predictive capability. The standard error of estimate (in log units) was 0.6176, working out to a multiplication (plus one Se) or division (minus one Se) by the antilog of 0.6176 or 4.1457.

Results were even worse for the untransformed data: a correlation coefficient of 0.0 (no discernible correlation), and a standard error estimate of 305.594 percent exceedance frequency.

Partial duration annual exceedance frequencies in percent were assigned to the correlated peak discharges of both the Peckman River at Ozone Avenue in Verona NJ and Passaic River at Little Falls NJ USGS gages, using the current peak discharge vs. frequency curves for both gages.

The final results of the above analysis, in terms of exceedance frequency vs. exceedance frequency, are summarized in the **Table 4**.

All of the above work was done with the peak flows and exceedance frequencies of the Peckman River at Ozone Avenue in Verona NJ USGS gage as the independent variable. Note that the problem cannot be worked in the opposite direction, with the peak flows and exceedance frequencies of the Passaic River at Little Falls NJ USGS gage as the independent variable. This is because, with the exception of nine water years, in which the same storm produced the annual peak flows at both gages, the peak flows at the Peckman River at Ozone Avenue in Verona NJ USGS gage, corresponding to the annual peak flows at the Passaic River at Little Falls NJ USGS gage cannot be found. This is because the Peckman River USGS gage is a crest stage gage providing annual peak flow data only, and not a continuous recording gage, except for the past 31 days of real-time stage data, on any given day.

A correlation could be done of the nine data pairs of the Passaic and Peckman gages of the water years in which the annual peak flows were produced by the same storm, at both gages, but because it would be an overlooking and omission of data for the other 21 water years in the common period of data, it could be construed as a distortion of data. The only advantage would be that the nine annual peak flows of the Passaic River at Little Falls would be known, and not estimated from mean daily flows.

Table 4: Peckman (partial duration) Exceedance frequencies vs. two Predicted Passaic (partial duration) exceedance frequencies.

Peckman River (Ozone Avenue in Verona, NJ) return period & exceedance frequency	Predicted* Passaic River (Little Falls, NJ) return period & exceedance frequency	
	Predicted by Peak Flow	Predicted by Frequency
500 yr - 0.2%	5yr - 18.5%	10 yr - 10.5%
250 yr - 0.4%	4 yr - 26.0%	7 yr - 14.3%
100 yr - 1.0%	3 yr - 39.5%	5 yr - 21.6%
50 yr - 2.0%	2 yr - 52.0%	3 yr - 29.5%
25 yr - 4.0%	2 yr - 66.5%	2 yr - 40.3%
10 yr - 10.0%	1 yr - 92.0%	2 yr - 60.8%
5 yr - 20.0%	1 yr - 99.9%	1 yr - 82.9%
2 yr - 50.0%	1 yr - 116.1%	1 yr - 125.1%
1 yr - 100.0%	1 yr - 178.8%	1 yr - 170.8%

* = Developed using a weak correlation from 9 common storms in the 30 years of common record.

A visual examination of the plots of the above nine data pairs, in terms of both peak discharge vs. peak discharge, and exceedance frequency vs. exceedance frequency, suggests that a correlation analysis of them would be no better or worse than the correlation analyses that were done of all thirty data pairs available.

The results of the analysis indicate that there is no necessary correlation, in terms of either peak discharge or frequency, between historic flood peaks on the Peckman and Passaic Rivers. This is due to the weak (less than 0.500) correlation coefficients of both correlations. There is no reliable way to predict a flood stage on the Peckman River based on the Passaic River. The converse is also true that there is no reliable way to predict a flood stage on the Passaic River based on what is happening on the Peckman River.

There is no complete dependence of frequencies of flooding on both the Passaic and Peckman Rivers. We can assume that the larger area for the Passaic River watershed (762 square miles) will be much slower to respond to a weather event than the sub-watershed area of the Peckman River (4.45 square miles). For example, during the April 2007 storm the Passaic River crested more than 48 hours after the Peckman River peak. Furthermore, the Passaic River backwater caused flooding in the Woodland Park area of Peckman River well after the Peckman River returned to base flow.

As there is a low correlation between the Peckman and Passaic Rivers peak flows, an analysis that considers the chance of excedance between these two independent flooding sources was performed. Refer to Section 2.3.4 for further details about the joint probability analysis. This study and proposed structural and non-structural alternatives focuses on managing flood risk associated to the Peckman River peak flows, but to some extent will also manage flood risk from the Passaic River backwater.

2.2.2 Without Project Conditions

Flow line computations were made to develop the hydraulic gradient of the natural stream channel in its existing condition. The computations were generated in accordance with EM 1110-2-1409, "Backwater Curves in River Channels," using HEC-RAS version 4.0. The flow lines were used to develop the hydraulic gradients for Peckman River existing conditions. The computations are based on a starting point of known water surface at the confluence of the Peckman River with the Passaic River. As there is no correlation between events on the Peckman and Passaic Rivers, the the Passaic River Base flow and equivalent elevation was used as boundary condition for the Peckman River. These elevations were obtained from a Passaic River rating curve developed as part of the General Design Memorandum for the Passaic River Flood Damage Reduction Project, September 1995, Appendix C – Hydrology and Hydraulics. **Figure 8** represents the rating curves for the Passaic River at designated locations relative to the Peckman River.

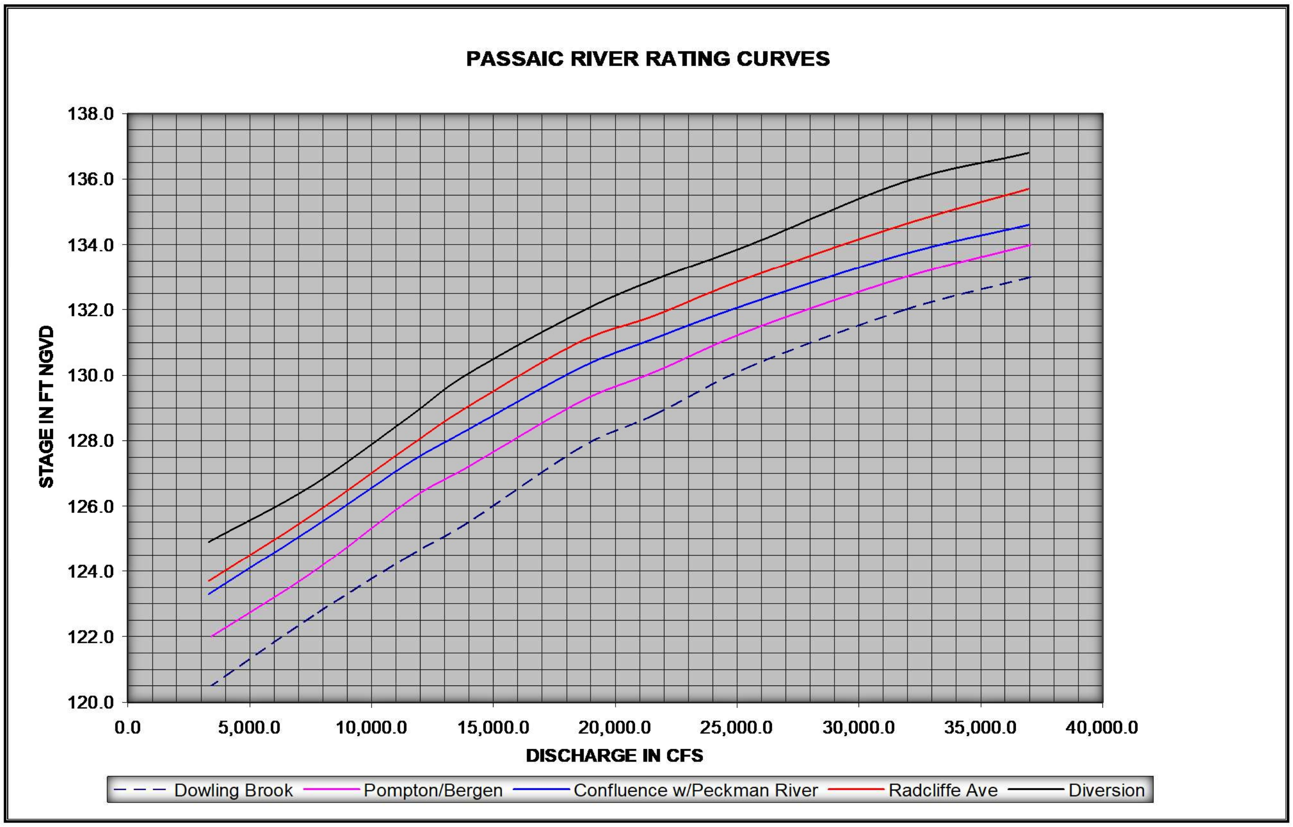

Figure 8: Passaic River rating curves at designated locations.

These river stations represent the various reaches that have been identified by Economics during this study. **Figure 9** depicts the steady state without project conditions water surface profile for the Peckman River, while **Figure 10** depicts the unsteady state without project conditions water surface profile for the Peckman River

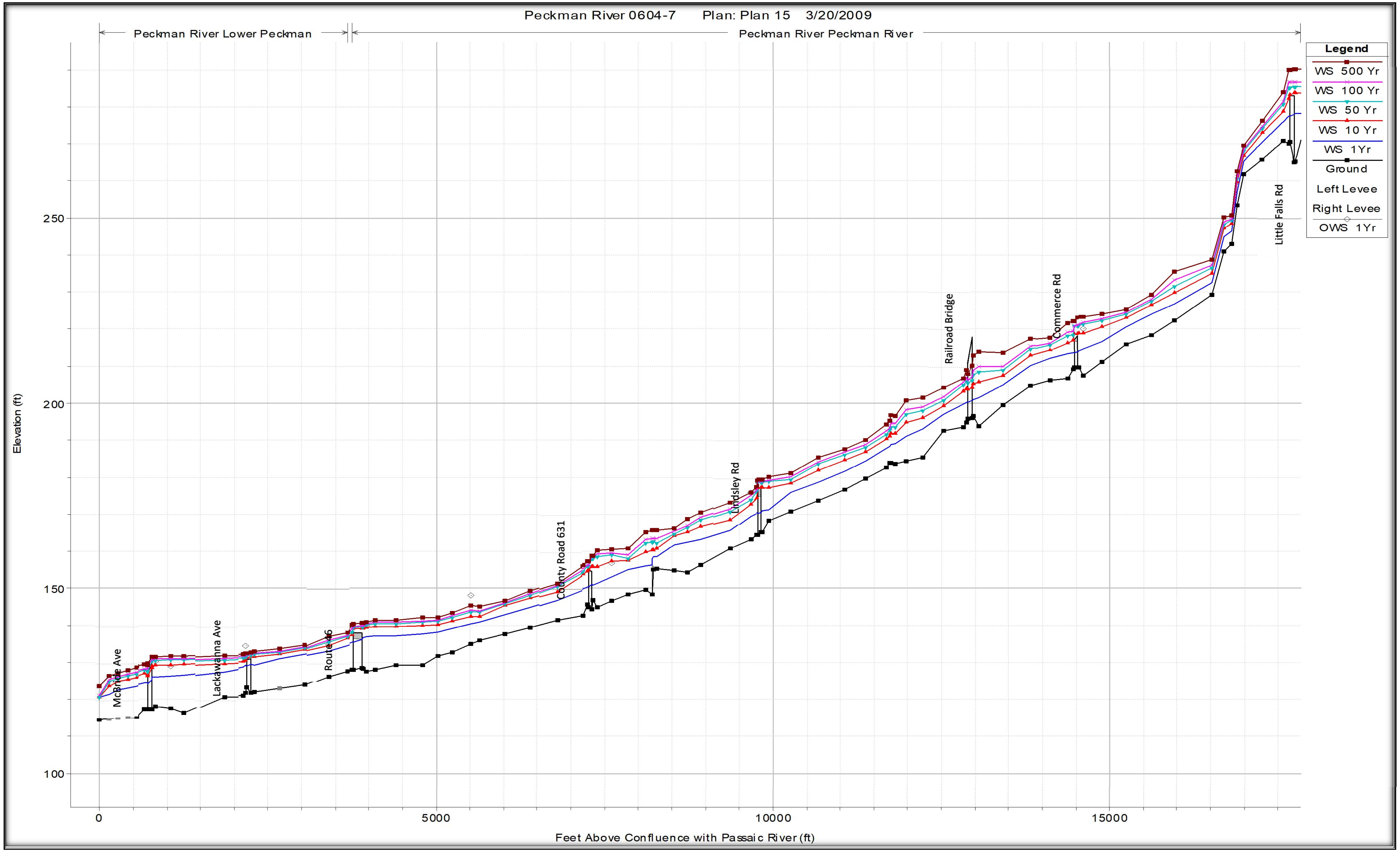

Figure 9: Steady State without project conditions water surface profile for the Peckman River

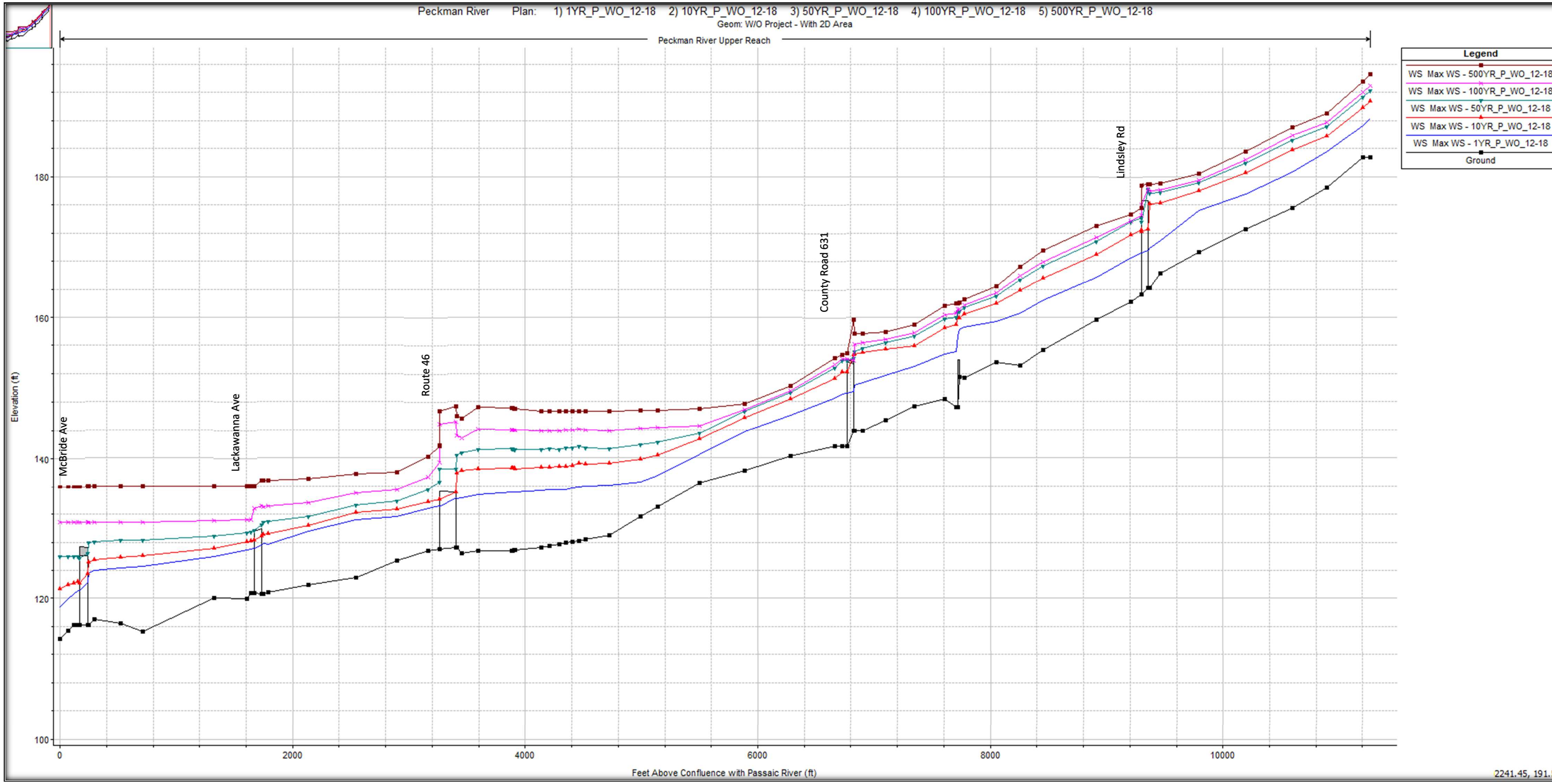

Figure 10: Unsteady State without project conditions water surface profile for the Peckman River.

2.3 Flow Line Computation

2.3.1 *Calibration and High Water Marks*

The steady and unsteady hydraulic HEC-RAS models of the Peckman River were calibrated using high water marks from the Tropical Storm Floyd (September 1999). The calibration process consists of computing water surface profiles using the HEC-RAS model with recorded flow data. Manning's n-values and other loss coefficients are also adjusted within reasonable limits until the computed water surface elevations are within reason of the observed flood marks. Apart from some questionable outlier high water marks, the HEC-RAS model calibration was successful as per the observed floodmarks from Tropical Storm Floyd. ` Tropical Storm Floyd corresponds to the 0.034 AEP (29.5-yr frequency). **Figure 11** displays a profile view of the computed water surface elevations and the observed floodmark for the steady state runs. **Table 6** shows the observed water surface elevation with the corresponding unsteady state hydraulic model output as computed from HEC-RAS.

Table 5: Tropical Storm Floyd - September 1999 High Water Marks - Steady State Modeling.

Flood Mark	Observed WS (ft) (NAVD88)	Floyd (computed) WS (ft) (NAVD88)	OWS - WSE Floyd (ft)
1	129.345	129.585	-0.24
2	130.445	129.765	0.68
3	134.245	134.965	-0.72
4	138.845	139.095	-0.25
5	147.645	147.155	0.49
6	219.045	220.395	-1.35
7	286.145	285.025	1.12
8	309.045	309.385	-0.34
9	310.045	309.385	0.66
10	330.245	328.915	1.33
11	335.745	336.605	-0.86

Figure 11 displays a plot of the predicted water surface elevation and the observed flood mark for the steady state model runs.

Table 6: Tropical Storm Floyd - September 1999 High Water Marks - Unsteady State Modeling.

Flood Mark	River Station	Observed WS (ft)	Floyd (computed) WS (ft)	OWS - WSE Floyd (ft)
1	8134.068	155.80	156.27	-0.47
2	6936.807	147.50	146.64	0.86
3	4528.103	138.70	138.17	0.53
4	3977.755	134.10	133.21	0.89
5	1627.2	127.90	127.63	0.27
6	1115.247	124.90	125.07	-0.17

Figure 12 displays a plot of the predicted water surface elevation and the observed flood mark for the unsteady state model runs.

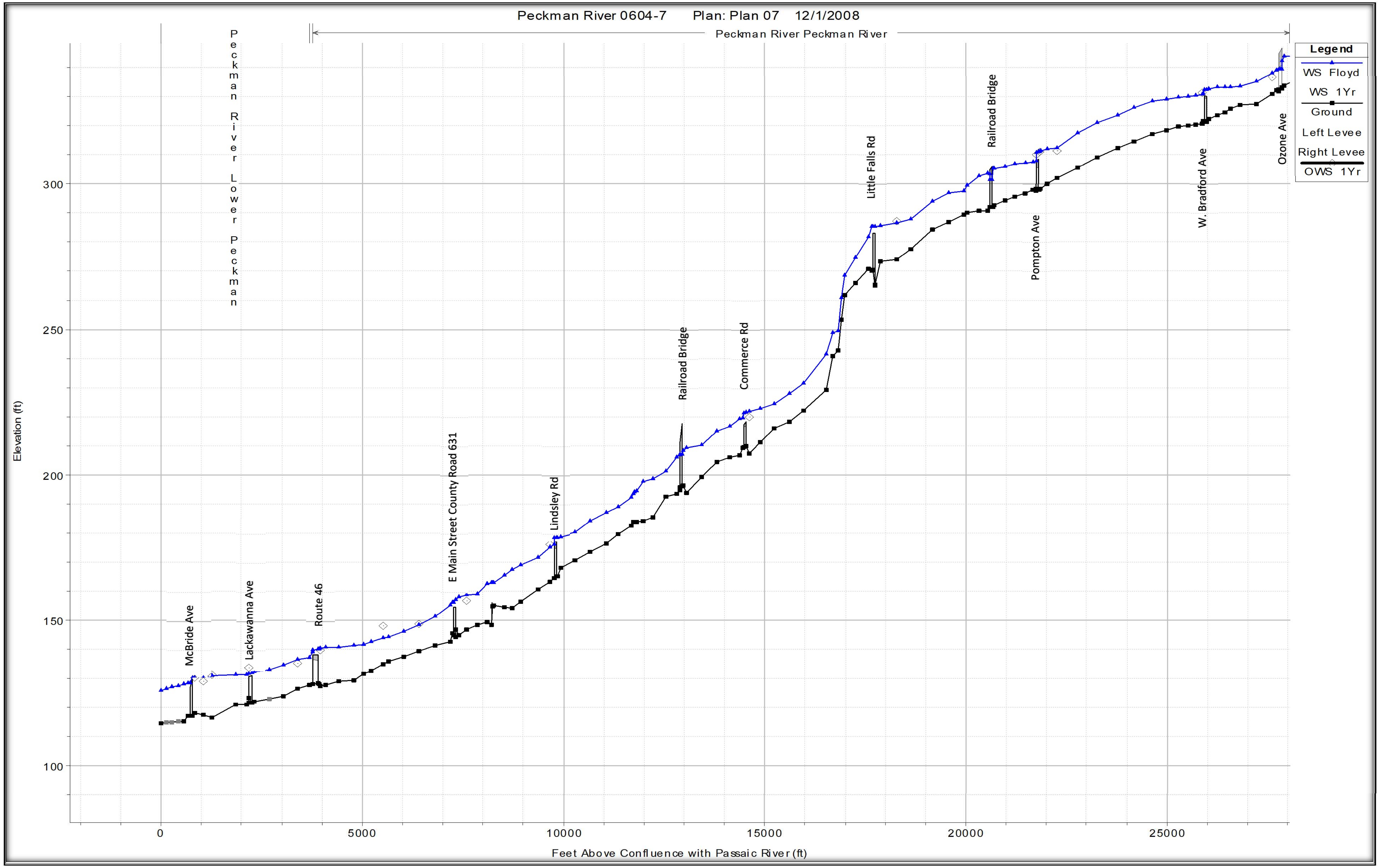

Figure 11: Steady state water surface profile with high water marks for Tropical Storm Floyd.

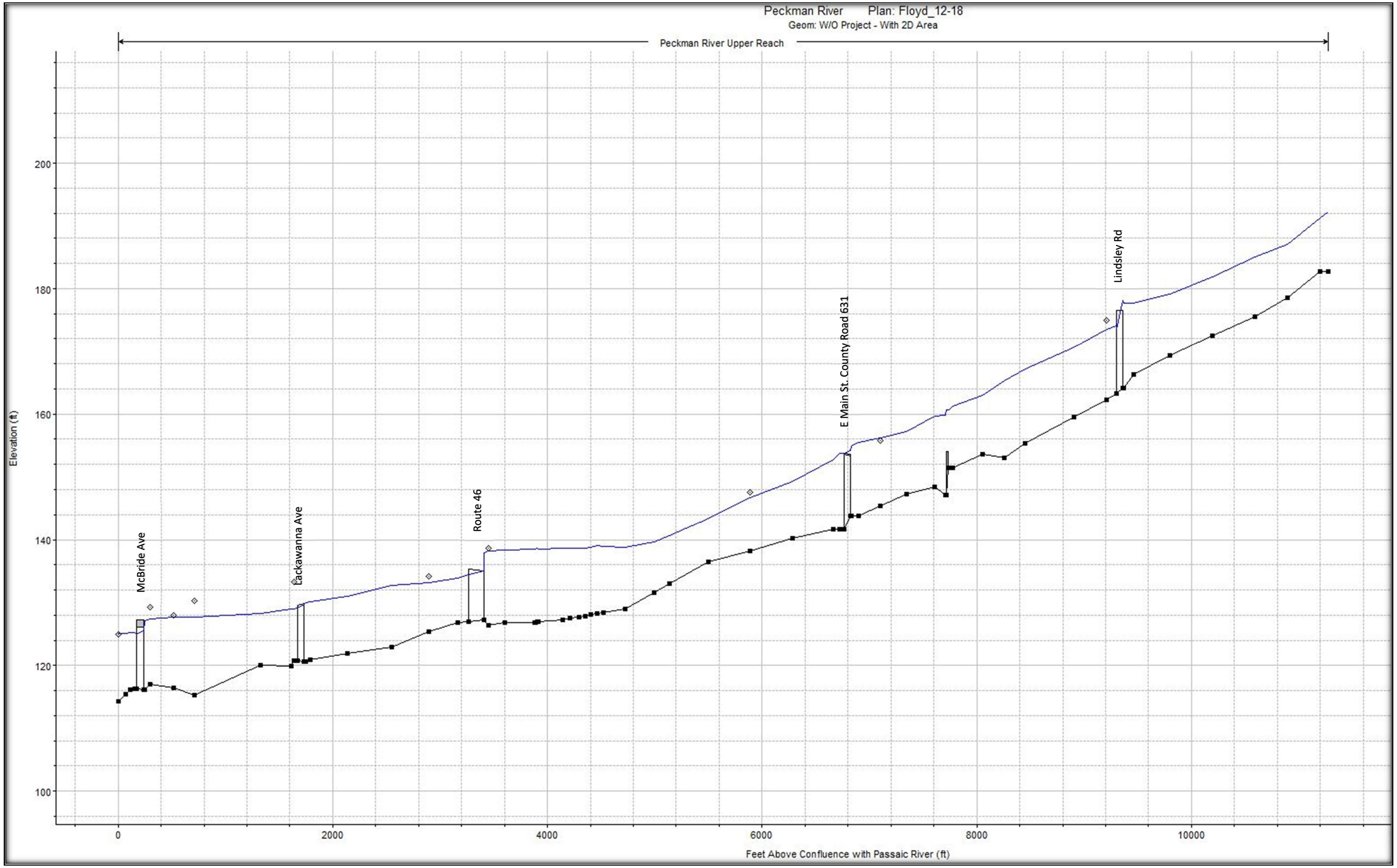

Figure 12: Unsteady state water surface profile with high water marks for Tropical Storm Floyd.

2.3.2 Steady State Flow Data

Present conditions peak flow data for Peckman River is presented in **Table 7** below. There are a total of 12 flow changes as indicated with the corresponding river station. The 1.0, 0.5, 0.2, 0.1, 0.04, 0.02, 0.01, 0.004, 0.002 AEP (1-, 2-, 5-, 10-, 25-, 50-, 100-, 250-, and 500-year) events are included in addition to the discharge for Tropical Storm Floyd (the calibration event).

Table 7: Peak Flow (cfs) data for Peckman River, Present Conditions.

River station	0.999	0.5	0.2	0.1	0.04	0.02	0.01	0.004	0.002	Floyd
40325	430	750	1040	1300	1700	2050	2460	3090	3620	2060
25520	910	1490	2020	2500	3140	3660	4280	5310	6100	3520
20550	970	1580	2140	2650	3370	3930	4600	5720	6570	3810
19150	1330	2150	2900	3590	4560	5320	6220	7670	8800	4960
17000	1390	2230	3010	3710	4720	5500	6540	7990	9180	5330
14850	1430	2290	3080	3810	4860	5670	6750	8260	9470	5620
10318	1580	2560	3380	4140	5320	6300	7390	9010	10250	6210
5950	1650	2680	3580	4330	5580	6640	7810	9500	10770	6560
5250	1670	2680	3690	4200	5400	6370	7400	9170	10640	6240
4550	1670	2680	2820	3030	3900	4480	5140	5900	7030	4400
4130	1820	2900	3170	3420	4300	4970	5690	6530	7720	4860
2300	1670	2600	2900	3140	3730	4460	5190	6180	7310	4440

Peak flow data for Peckman River, future unimproved conditions, is presented in **Table 8**. Future flows account for estimates of future land development. The time horizon is assumed to be 50 years (from a base year of 2013) and these discharges were computed to evaluate the conditions in the watershed without a flood damage reduction project. In the absence of a project there will be an increase in discharge over time.

Table 8: Peak Flow (cfs) data for Peckman River, Future unimproved conditions.

River station	0.999	0.5	0.2	0.1	0.04	0.02	0.01	0.004	0.002	Floyd
40325	490	830	1150	1430	1860	2220	2640	3280	3820	2150
25520	1050	1670	2240	2760	3410	3950	4580	5600	6430	3670
20550	1110	1770	2370	2920	3660	4230	4920	6040	6900	3990
19150	1500	2370	3160	3890	4890	5670	6590	8010	9140	5170
17000	1580	2460	3290	4010	5080	5920	6940	8320	9580	5520
14850	1620	2530	3370	4120	5230	6100	7180	8610	9910	5860
10318	1810	2850	3680	4530	5740	6710	7810	9350	10690	6440
5950	1910	3030	3930	4770	6050	7140	8300	9930	11200	6800
5250	1900	3020	3890	4480	5830	6750	7750	9560	11040	6420
4550	1900	2730	2910	3240	4160	4720	5290	6190	7200	4510
4130	2080	3040	3310	3640	4650	5250	5920	6820	7940	4980
2300	1880	2720	2970	3280	4000	4690	5420	6430	7540	4530

2.3.3 Unsteady State Flow Data

Approximately 45% of the storm water was exiting the Peckman River's banks. This water had such a tremoundous volume that the difference between the water surface elevations between the steady and unsteady models is approximately 3 ft in certain downstream areas. As explained in section 2.0, this new unsteady model with a 2D area was necessary and was calibrated to Tropical Storm Floyd. The steady flow model was functional, and was able to successfully determine the appropriate alternative used in this project, but ultimately the unsteady flow model superceded the steady flow model. The unsteady flow model was only used for the TSP and optimization purposes.

Without project condition, present and future, flow hydrographs for Peckman River were obtained from a Hydrologic Engineering Center, Hydrologic Modeling System (HEC-HMS) hydrologic model of the basin (refer to the Hydrology Appendix). The hydrographs were input into a Hydrologic Engineering Center, River Analysis System (HEC-RAS) unsteady flow model of the study area. There are a total of 8 boundary conditions inserted into the model. A description of these boundary conditions for the existing conditions are presented in **Table 9**. Refer to **Figure 13** to see the HEC-RAS geometry layout with the 2D boundary shown. The peak discharge of the 1.0, 0.5, 0.2, 0.1, 0.04, 0.02, 0.01, 0.004, 0.002 AEP (1-, 2-, 5-, 10-, 25-, 50-, 100-, 250-, and 500-year) events are included in addition to the peak discharge for Tropical Storm Floyd (the calibration event).

Flow hydrographs for the Peckman River, future unimproved conditions, were also obtained from a HEC-HMS model of the hydrology in the Peckman River watershed which accounted for future land development. The peak discharge for the future unimproved conditions are shown in **Table 9**. The time horizon is assumed to be 50 years. (from a base year of 2013) and these discharges were computed to evaluate the conditions in the watershed without a flood damage reduction project. In the absence of a project there will be an increase in discharge over time as a result of continued development and this data enables the hydraulic analysis to determine the corresponding stage frequencies. A description of these boundary conditions for the future unimproved conditions are presented in **Table 10**.

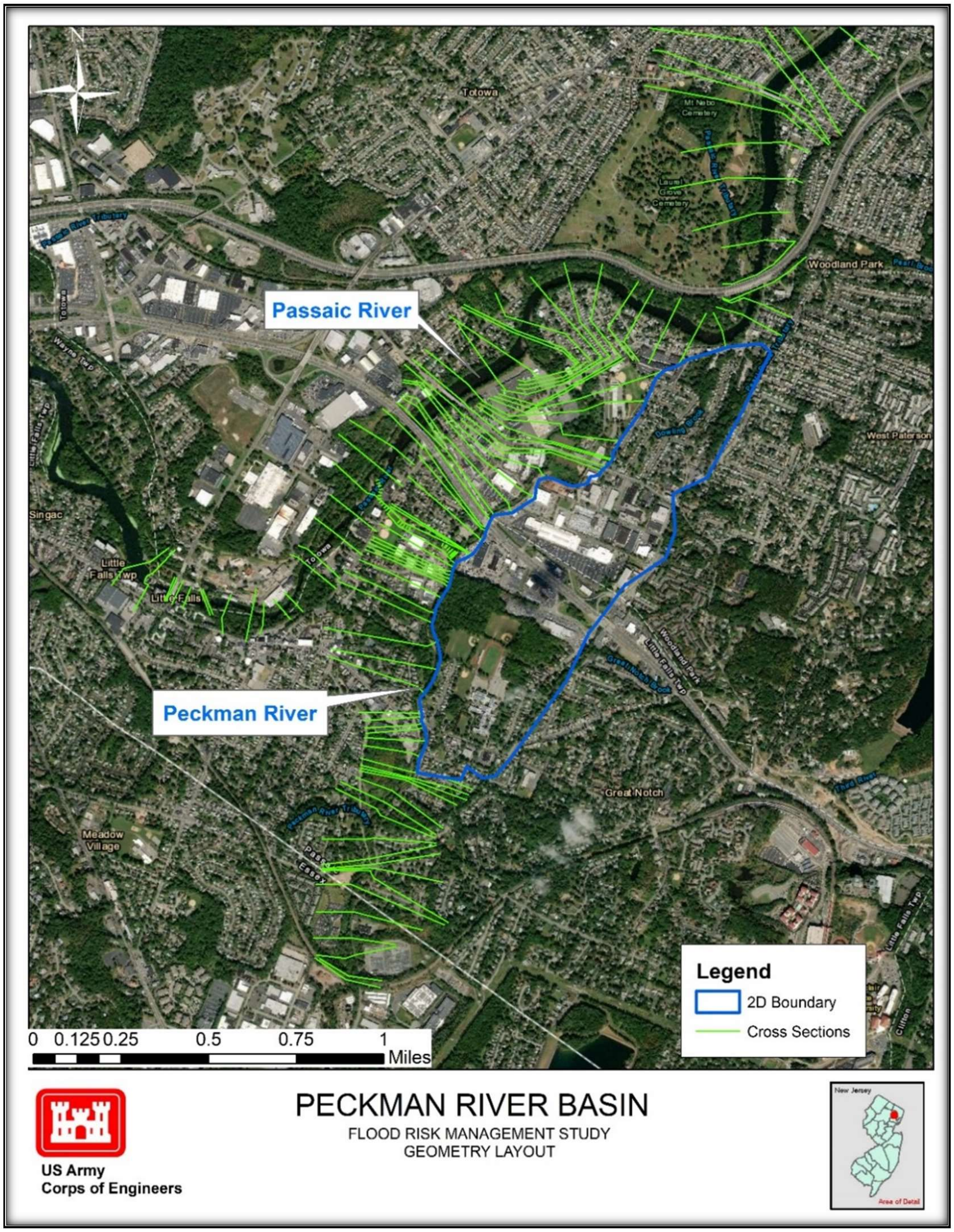

Figure 13: Peckman HEC-RAS Geometry Layout and 2D boundary

Table 9: Peak Flow (cfs) and Boundary Condition data for Peckman River, Present Conditions, Unsteady Flow Model.

River	Reach	River station	Type of Boundary Condition	0.999	0.5	0.2	0.1	0.04	0.02	0.01	0.004	0.002	Floyd
Passaic	**Ups Peckman**	161033.3	Flow Hydrograph	1000	1000	1000	1000	1000	1000	1000	1000	1000	5439
Passaic	**Ds Peckman**	136647.1	Normal Depth	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002
Peckman	**Culvert**	1795.165	Lateral Inflow Hydrograph	50	50	50	50	50	50	50	50	50	N/A
Peckman	**Upper Reach**	12254.05	Flow Hydrograph	1584	2559	3384	4146	5321	6300	7399	9011	10263	6216
Peckman	**Upper Reach**	9468.539	Lateral Inflow Hydrograph	85	153	213	266	347	412	491	601	707	353
Peckman	**Upper Reach**	6195.886	Lateral Inflow Hydrograph	279	403	517	632	781	905	1036	1218	1368	757
Peckman	**Lower Reach**	2422.488	Lateral Inflow Hydrograph	63	86	108	131	158	183	208	242	269	151
2D Flow Area	**Great Notch (2D)**	N/A	Flow Hydrograph	213	482	396	482	596	684	781	906	1020	540

Table 10: Peak Flow (cfs) and Boundary Condition data for Peckman River, Future Unimproved Conditions, Unsteady Flow Model.

River	Reach	River station	Type of Boundary Condition	0.999	0.5	0.2	0.1	0.04	0.02	0.01	0.004	0.002
Passaic	**Ups Peckman**	161033.3	Flow Hydrograph	1000	1000	1000	1000	1000	1000	1000	1000	1000
Passaic	**Ds Peckman**	136647.1	Normal Depth	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002
Peckman	**Culvert**	1795.165	Lateral Inflow Hydrograph	50	50	50	50	50	50	50	50	50
Peckman	**Upper Reach**	12254.05	Flow Hydrograph	1807	2850	3678	4538	5747	6719	7812	9356	10703
Peckman	**Upper Reach**	9468.539	Lateral Inflow Hydrograph	129	218	293	361	459	533	618	727	835
Peckman	**Upper Reach**	6195.886	Lateral Inflow Hydrograph	355	492	619	751	915	1045	1181	1356	1509
Peckman	**Lower Reach**	2422.488	Lateral Inflow Hydrograph	63	86	108	131	158	183	208	242	269
2D Flow Area	**Great Notch (2D)**	N/A	Flow Hydrograph	268	372	468	566	689	782	880	998	1113

2.3.4 Joint Probability

The extent of the backwater flooding from the Passaic River into the Peckman River was considered by a joint probability analysis. Hydrologic Engineering Center's (HEC) Statistical Software Package (HEC-SSP) was used to perform this analysis. The analysis was performed by running hypothetical storm events in the Passaic River against hypothetical storm events in the Peckman River. Refer to **Table 11** to see the different plans that were run through the hydraulic model. Both the present (Year 2013) and future conditions (Year 2063) were analyzed.

The HEC-RAS model's output were input into HEC-SSP and a coincident frequency analysis was performed. **Figure 14** through **Figure 19** are joint probability of excedance results at selected location for without project conditions. The graphs compare Peackman River stage-frequency curve without Passic River tailwater versus the Peckman River stages-frequency curve with Passaic backwater influence. The economic analysis was performed by using the joint probability outputs in HEC-FDA to produce a benefit to cost ratio.

Figure 20 shows the maximum upstream extent of elevated water surface elevations from the Passaic River backwater into the Peckman River for the present and future conditions. The different hypothetical storm events that were analyzed where the 1.0, 0.5, 0.2, 0.1, 0.04, 0.02, 0.01, 0.004, 0.002 AEP (1-year, 2-year, 5-year, 10-year, 25-year, 50-year, 100-year, 250-year, and 500-year storm events)

Table 11: Joint Probability Model Runs – Peckman vs. Passaic events

Model Plan #	Peckman River Storm Event - Fluvial - Return Interval	Passaic River Storm Event - Back Water - Return Interval
1	99.99% AEP (1 Year Event)	99.99% AEP (1 Year Event)
2	99.99% AEP (1 Year Event)	50% AEP (2 Year Event)
3	99.99% AEP (1 Year Event)	20% AEP (5 Year Event)
4	99.99% AEP (1 Year Event)	10% AEP (10 Year Event)
5	99.99% AEP (1 Year Event)	4% AEP (25 Year Event)
6	99.99% AEP (1 Year Event)	2% AEP (50 Year Event)
7	99.99% AEP (1 Year Event)	1% AEP (100 Year Event)
8	99.99% AEP (1 Year Event)	0.4% AEP (250 Year Event)
9	99.99% AEP (1 Year Event)	0.2% AEP (500 Year Event)
10	50% AEP (2 Year Event)	99.99% AEP (1 Year Event)
11	50% AEP (2 Year Event)	50% AEP (2 Year Event)
12	50% AEP (2 Year Event)	20% AEP (5 Year Event)
13	50% AEP (2 Year Event)	10% AEP (10 Year Event)
14	50% AEP (2 Year Event)	4% AEP (25 Year Event)
15	50% AEP (2 Year Event)	2% AEP (50 Year Event)
16	50% AEP (2 Year Event)	1% AEP (100 Year Event)
17	50% AEP (2 Year Event)	0.4% AEP (250 Year Event)
18	50% AEP (2 Year Event)	0.2% AEP (500 Year Event)
19	20% AEP (5 Year Event)	99.99% AEP (1 Year Event)
20	20% AEP (5 Year Event)	50% AEP (2 Year Event)
21	20% AEP (5 Year Event)	20% AEP (5 Year Event)
22	20% AEP (5 Year Event)	10% AEP (10 Year Event)
23	20% AEP (5 Year Event)	4% AEP (25 Year Event)
24	20% AEP (5 Year Event)	2% AEP (50 Year Event)
25	20% AEP (5 Year Event)	1% AEP (100 Year Event)
26	20% AEP (5 Year Event)	0.4% AEP (250 Year Event)
27	20% AEP (5 Year Event)	0.2% AEP (500 Year Event)
28	10% AEP (10 Year Event)	99.99% AEP (1 Year Event)
29	10% AEP (10 Year Event)	50% AEP (2 Year Event)
30	10% AEP (10 Year Event)	20% AEP (5 Year Event)
31	10% AEP (10 Year Event)	10% AEP (10 Year Event)
32	10% AEP (10 Year Event)	4% AEP (25 Year Event)
33	10% AEP (10 Year Event)	2% AEP (50 Year Event)
34	10% AEP (10 Year Event)	1% AEP (100 Year Event)
35	10% AEP (10 Year Event)	0.4% AEP (250 Year Event)
36	10% AEP (10 Year Event)	0.2% AEP (500 Year Event)
37	4% AEP (25 Year Event)	99.99% AEP (1 Year Event)
38	4% AEP (25 Year Event)	50% AEP (2 Year Event)
39	4% AEP (25 Year Event)	20% AEP (5 Year Event)

Model Plan #	Peckman River Storm Event - Fluvial - Return Interval	Passaic River Storm Event - Back Water - Return Interval
40	4% AEP (25 Year Event)	10% AEP (10 Year Event)
41	4% AEP (25 Year Event)	4% AEP (25 Year Event)
42	4% AEP (25 Year Event)	2% AEP (50 Year Event)
43	4% AEP (25 Year Event)	1% AEP (100 Year Event)
44	4% AEP (25 Year Event)	0.4% AEP (250 Year Event)
45	4% AEP (25 Year Event)	0.2% AEP (500 Year Event)
46	2% AEP (50 Year Event)	99.99% AEP (1 Year Event)
47	2% AEP (50 Year Event)	50% AEP (2 Year Event)
48	2% AEP (50 Year Event)	20% AEP (5 Year Event)
49	2% AEP (50 Year Event)	10% AEP (10 Year Event)
50	2% AEP (50 Year Event)	4% AEP (25 Year Event)
51	2% AEP (50 Year Event)	2% AEP (50 Year Event)
52	2% AEP (50 Year Event)	1% AEP (100 Year Event)
53	2% AEP (50 Year Event)	0.4% AEP (250 Year Event)
54	2% AEP (50 Year Event)	0.2% AEP (500 Year Event)
55	1% AEP (100 Year Event)	99.99% AEP (1 Year Event)
56	1% AEP (100 Year Event)	50% AEP (2 Year Event)
57	1% AEP (100 Year Event)	20% AEP (5 Year Event)
58	1% AEP (100 Year Event)	10% AEP (10 Year Event)
59	1% AEP (100 Year Event)	4% AEP (25 Year Event)
60	1% AEP (100 Year Event)	2% AEP (50 Year Event)
61	1% AEP (100 Year Event)	1% AEP (100 Year Event)
62	1% AEP (100 Year Event)	0.4% AEP (250 Year Event)
63	1% AEP (100 Year Event)	0.2% AEP (500 Year Event)
64	0.4% AEP (250 Year Event)	99.99% AEP (1 Year Event)
65	0.4% AEP (250 Year Event)	50% AEP (2 Year Event)
66	0.4% AEP (250 Year Event)	20% AEP (5 Year Event)
67	0.4% AEP (250 Year Event)	10% AEP (10 Year Event)
68	0.4% AEP (250 Year Event)	4% AEP (25 Year Event)
69	0.4% AEP (250 Year Event)	2% AEP (50 Year Event)
70	0.4% AEP (250 Year Event)	1% AEP (100 Year Event)
71	0.4% AEP (250 Year Event)	0.4% AEP (250 Year Event)
72	0.4% AEP (250 Year Event)	0.2% AEP (500 Year Event)
73	0.2% AEP (500 Year Event)	99.99% AEP (1 Year Event)
74	0.2% AEP (500 Year Event)	50% AEP (2 Year Event)
75	0.2% AEP (500 Year Event)	20% AEP (5 Year Event)
76	0.2% AEP (500 Year Event)	10% AEP (10 Year Event)
77	0.2% AEP (500 Year Event)	4% AEP (25 Year Event)
78	0.2% AEP (500 Year Event)	2% AEP (50 Year Event)
79	0.2% AEP (500 Year Event)	1% AEP (100 Year Event)
80	0.2% AEP (500 Year Event)	0.4% AEP (250 Year Event)
81	0.2% AEP (500 Year Event)	0.2% AEP (500 Year Event)

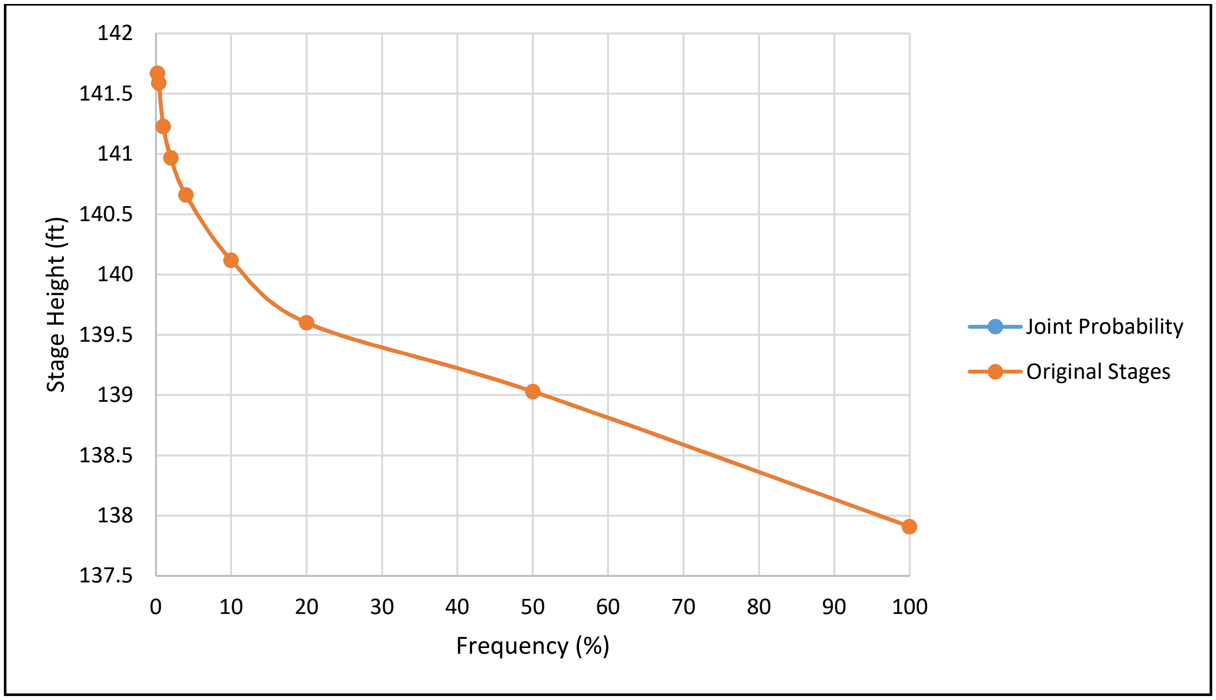

Figure 14: Cross section near Marietta St (XS 6195.886) – Future Flows

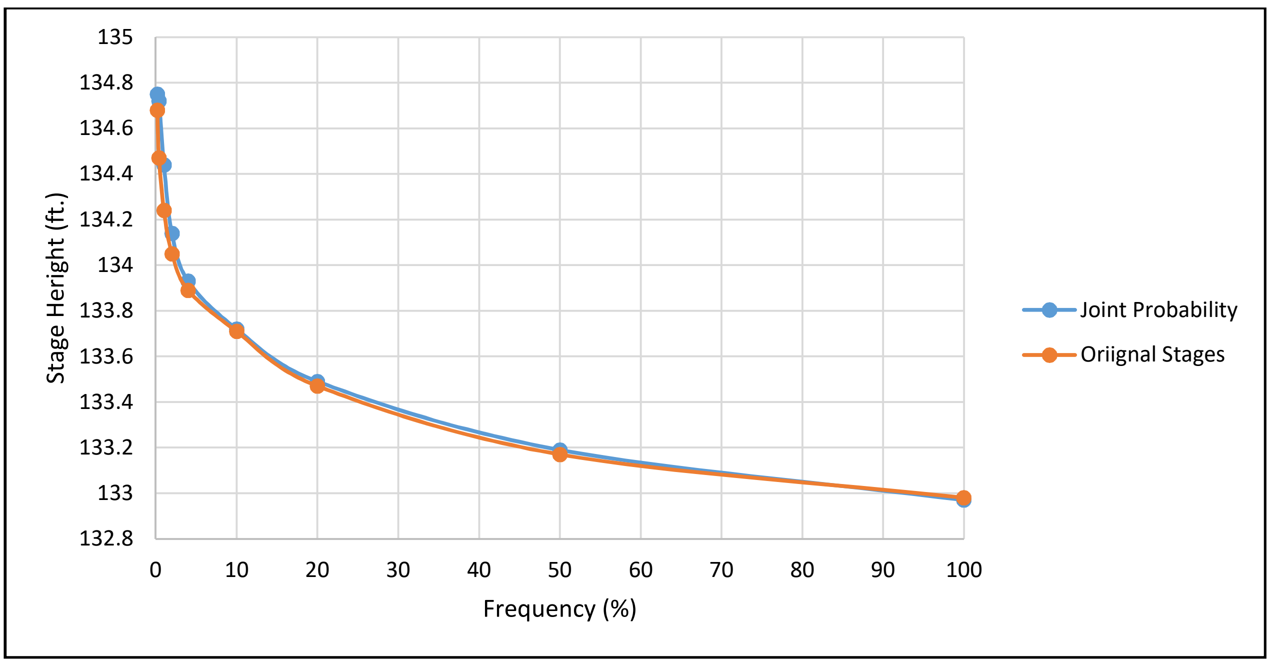

Figure 15: Cross section approximately 100 feet downstream of the Route 46 bridge (XS 4245.644) – Future Flows

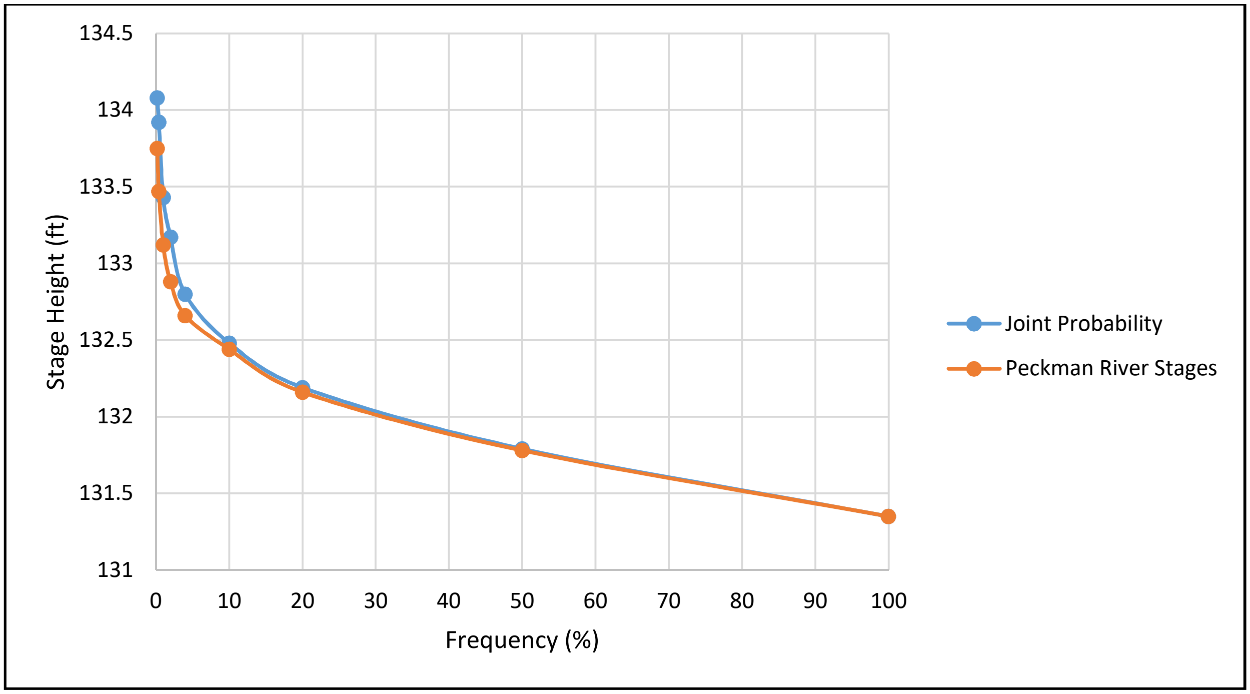

Figure 16: Cross section approximately 725 feet downstream of the Route 46 bridge (XS 3631.198) – Future Flows

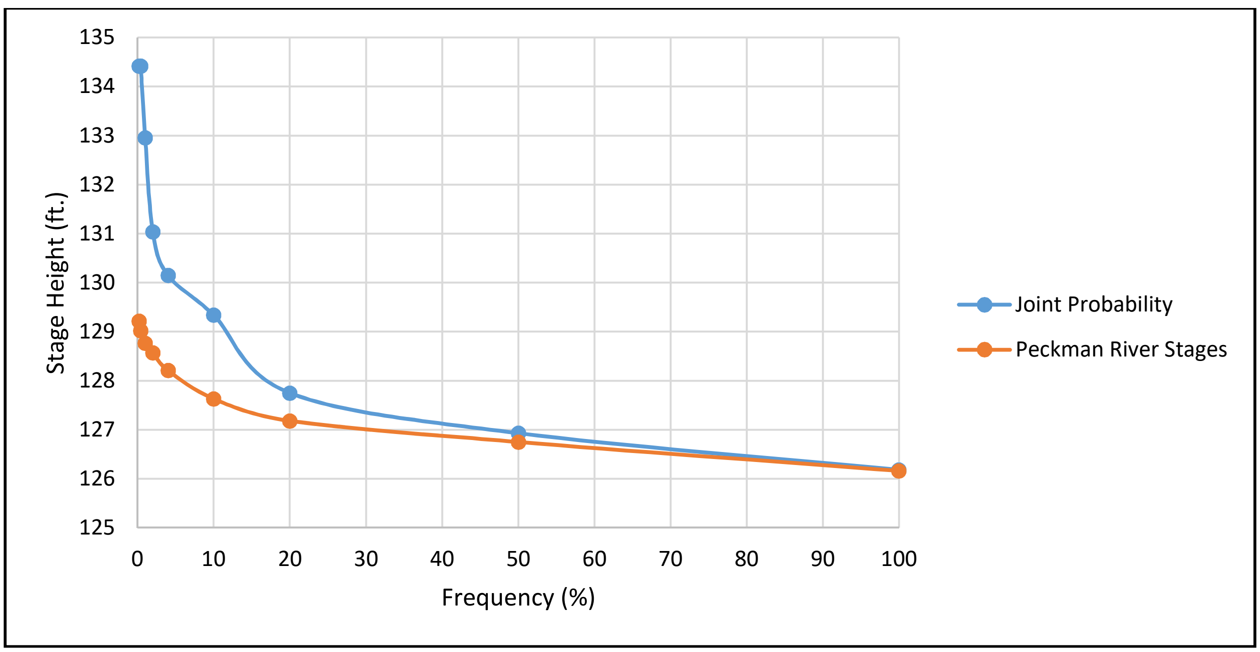

Figure 17: Cross section approximately 350 feet downstream of the Lackawanna Avenue bridge (XS 2422.488) – Future Flows

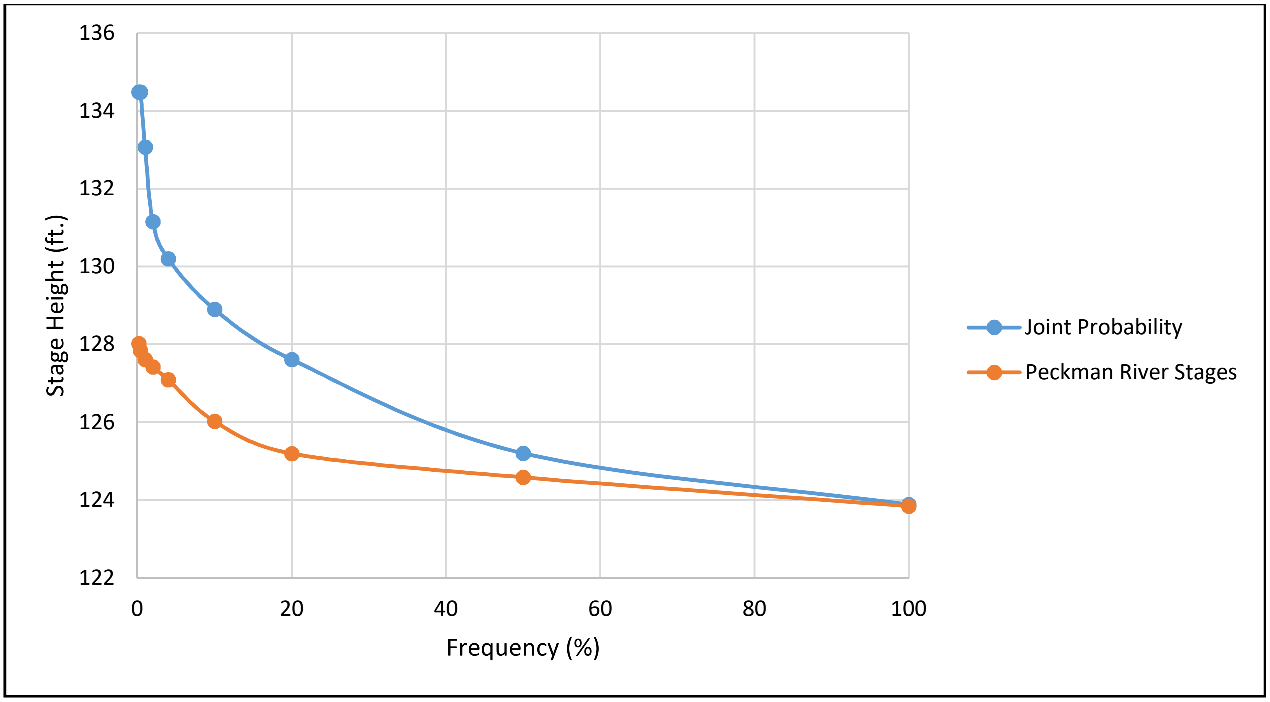

Figure 18: Cross section approximately 5 feet upstream of McBride Ave (XS 1356.862) – Future Flows

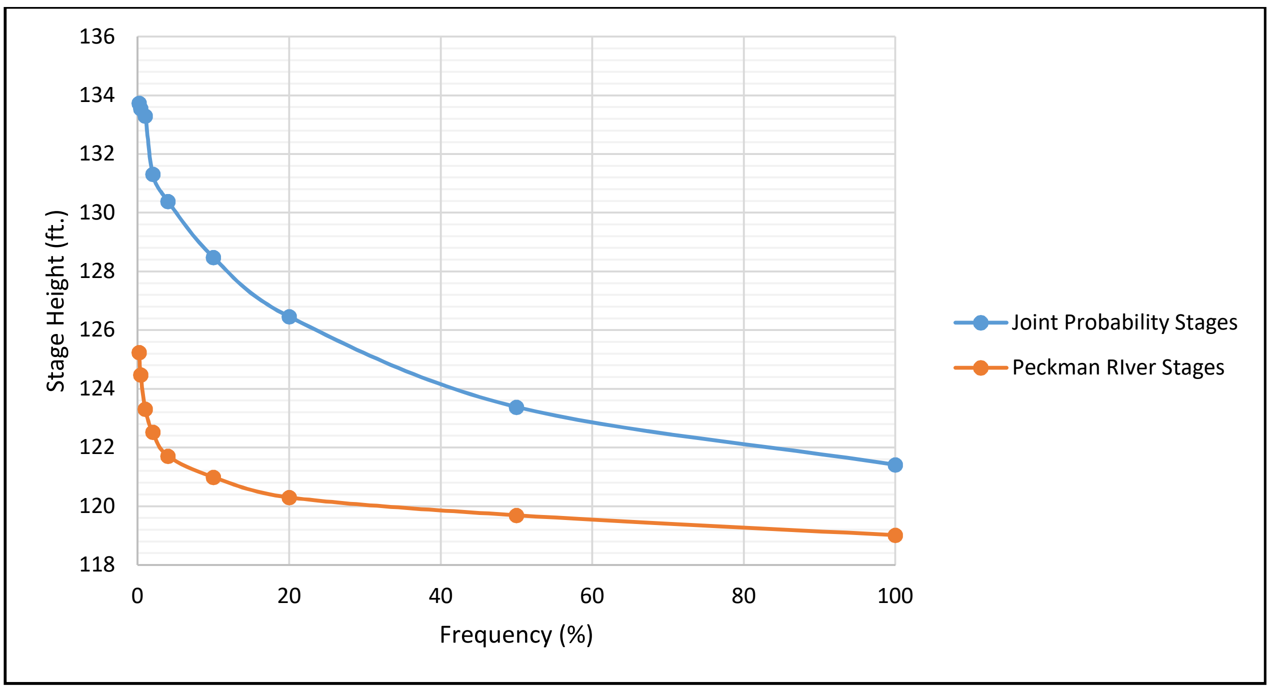

Figure 19: Cross section approximately 175 feet downstream of McBride Avenue (XS 1115.247) – Future Flows

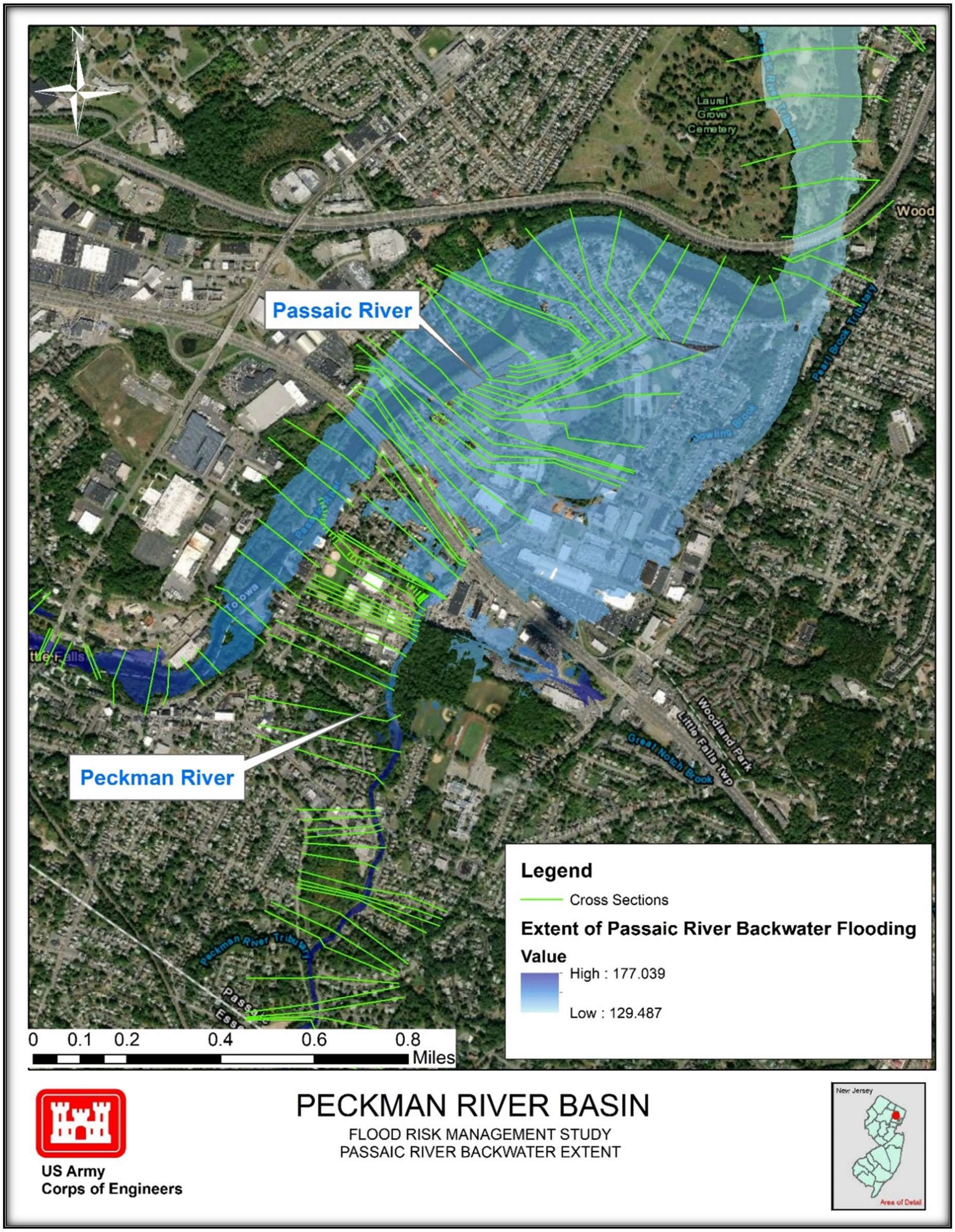

Figure 20: Extent of Passaic River 0.002 AEP (500-yr) backwater.

2.4 Hydraulic Model Uncertainty

2.4.1 Steady State Hydraulic Analysis

The uncertainty of the hydraulic model was developed based on varying appropriate input parameters. The parameters and percent varied were as follows: (1) Manning's n-values, +/- 25%, (2) flow (discharge) values, +/- 10%, and (3) expansion/contraction coefficients, +/- 50%. Two plans were generated with one plan with all the vairables at the higher values parameters (i.e., Manning's n-values +25%, flow data +10%, and expansion/contraction coefficients +50%) and another with the vairables at the lower values for the nine rainfall frequencies. Water surface elevations were obtained for the above two plans and standard deviation was calculated using the standard deviation equation as follows:

$$S = \sqrt{\frac{\sum_{i=1}^{N}(Xi - M)^2}{N - 1}}$$

S- Standard Deviation

N- Number of Observations

Xi- Difference Detween the Water Surface Elevations

M- Mean (average) of the Data

For both the increased and decreased plans, each location and each flood frequency, a standard devation value was calculated. The standard devations of all the frequencies were averaged to obtain one standard deviation value for that river station.

The results of this analysis are displayed in **Table 12**. Since the standard deviation between increasing and decreasing the input parameters were not equivalent, the larger of the two values was selected as the final uncertainty at a given river reach. For example, at river station 4550, the standard deviation for increasing the input parameters (i.e., Manning's n-values +25%, flow data +10%, and expansion/contraction coefficients +50%) was 0.13 while the standard deviation for decreasing the input parameters (e.g., Manning's n-value -25%) was 0.48. Therefore, at river station 4550 the standard deviation used for the HEC-FDA analysis was 0.48.

Table 12: Hydraulic Model Uncertainty Analysis.

River station	Increase		Decrease		Selected Standard Deviation
	Avg. (ft)	Std. Dev.	Avg. (ft)	Std. Dev.	
2300	0.47	0.48	-1.23	0.44	0.48
3500	0.72	0.16	-0.93	0.15	0.16
4550	0.44	0.13	-1.14	0.48	0.48
5480	0.91	0.10	-1.13	0.32	0.32
6450	0.64	0.23	-0.73	0.48	0.48
6850	0.62	0.14	-1.43	0.33	0.33
8300	0.89	0.40	-1.27	0.29	0.40
9400	0.98	0.06	-1.59	0.11	0.11
10750	1.14	0.26	-1.33	0.24	0.26
11550	1.07	0.20	-1.65	0.37	0.37

2.4.2 Unsteady State Hydraulic Analysis

An equivalent record length was developed for the peak discharge vs. frequency relations used and analyzed in this study. This equivalent record length was also used in the hydraulic analysis of this study to determine the confidence bands of the stage-frequency curves.

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 by utilizing the Equivalent Record Length Guidelines shown in Table 4-5 of EM 1110-2-1619. The selection was made based on engineering judgment to account for the quality of the data used in the analysis and for the degree of confidence in the HMS model.

2.5 Flood Delineations

For Existing Conditions, floodplain delineation maps were generated from the HEC-HMS/RAS model output. From this output, several flood delineations were created. **Figure 21** represents the floodplain delineations for the 0.1, 0.02, 0.01 AEP without project present condition. Delineations in **Figure 21** illustrate maximum extent of both Passaic River and Peckman River peak floods without project present 0.01 AEP peak flood event for the Peckman River and the the Passaic River.

Each is shown in a different color to illustrate the difference in flood elevations between Peckman River flood events and the Passaic River flood events. The aqua-colored delineation represents the Peckman River peaking with coincident flow on Passaic River. This means that the Peckman River is peaking (cresting) while the Passaic River is below its peak, i.e., coincident. The maroon (purple)-colored delineation represents the Passaic River peaking with coincident flow on Peckman River. This means that the Peckman River has fallen below its peak while the Passaic River has reached its peak (crest).

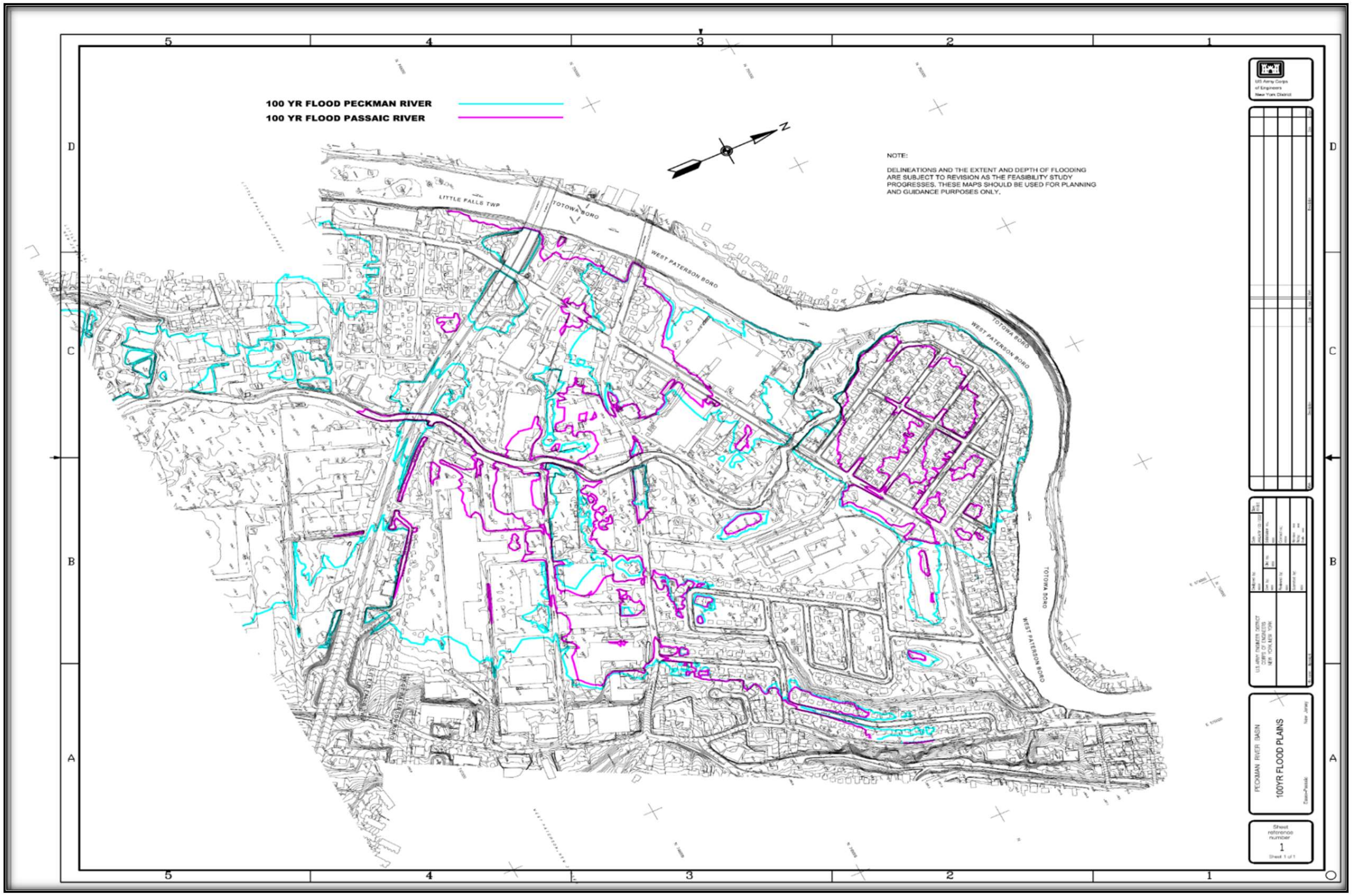

Figure 21: Floodplain Delineation for Peckma River and Passaic River

3.0 IMPROVED CONDITIONS HYDRAULICS

3.1 Formulated Plans

Structural alternatives providing flood damage reduction up to the 0.01 AEP design level were evaluated as well as the 0.1 and 0.02 AEP levels of performance. Structural plans under consideration include: diversion of flood water from the Peckman River, earthen levees and concrete floodwalls, channel improvement to increase channel capacity, and combinations of the aforementioned. Interior drainage structures, swales and/or pump stations were assumed to be necessary at all alternatives including levee/floodwalls to control interior drainage. In addition to structural alternatives, without project and non-structural alternatives were also evaluated. 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

3.1.1 Alternative #1 – No Action Plan (Without Project Future Conditions)

Under without project future conditions, the damage centers in Woodland Park and Little Falls will continue to be subject to flooding. The flood damage potential may be reduced with nonstructural measures, particularly the acquisition of flood-prone structures. However, due to the commercial nature of much of the flood prone areas in these two communities, acquisition is likely to be cost prohibitive and is not likely to be wide-spread. Development in central and upper portions of the Peckman River basin will increase the volume of runoff and increase the flooding in Woodland Park and Little Falls. Although much of the basin is highly urbanized, some development can be expected to continue, possibly in areas that may be subject to flooding. Therefore, without project future conditions is likely to experience an increase in flood inundation and damages.

3.1.2 Alternative #2 – Non-Structural Alternatives

Non-structural flood proofing techniques were identified and evaluated for structures in the Peckman River area. Three non-structural plans were developed to include structures within 0.1 AEP, 0.02 AEP, and 0.01 AEP floodplains. All three plans included non-structural features designed to withstand inundation up to and including a 0.01 AEP event. The target elevation for structure elevations is assumed to be one foot above the base flood elevation (BFE). The BFE varies in the project area from +130 feet to +190 feet NAVD88. Assumptions in screening these nonstructural measures are presented in **Table 13**.

Table 13: Assumptions Inherent to the Screening of Non-Structural Alternatives.

Structure type	Assumption
General	· Flood velocity is negligible.
	· Debris impacts will not be considered.
	· The area is considered non-coastal and thus not subject to wave and erosion impacts. No areas were designated as "V-zone" by FEMA, subject to 3-foot breaking waves.
	· Buildings elevated will be raised (finished floor elevation) to the 100-year water surface plus 1 foot to account for the uncertainty of wave effects
	· Flooding is gradual (no flash flooding).
Foundation Walls	· All basement foundation types are assumed to be unreinforced, 8" concrete masonry units (CMUs).
Raised Structures (Crawlspace)	· No utilities are located in the crawlspace.
	· Wet flood proofing of raised structures includes the elevation of utilities only.
Slab-On-Grade Structures	· Wet flood proofing is possible if the expected flood elevation is below the main floor (shallow flooding). This alternative includes the elevation of utilities only.
	· Consistent with Corps' flood proofing guidance, structures will not be dry flood proofed for flooding depths greater than 2 feet with a maximum 3 feet of dry flood proofing protection.
Structures With Basements	· All basements are unfinished and contain major utilities.
	· All basements are subgrade and none are walkout.
Bi-Levels	· The lower portion of the first floor walls are masonry construction.
	· The foundation is slab-on-grade.
	· The main floor can be raised separately from the lower level by lifting off the sill of the masonry wall.
Raised Ranches	· The first floor (lower) walls are masonry.
	· The foundation is slab-on-grade.
	· The main floor can be raised separately from the lower level (similar to a structure with a basement).
Split-Levels	· The lower level is slab-on-grade.
	· The lower portion of the lower level walls are masonry construction.
	· The main floor level is raised over a crawl space.
	· The main floor and upper level can be separated from the lower level by raising at the sill.

The non-structural measures considered in the Peckman River feasibility includes:

- *Dry Flood Proofing*. Dry flood proofing measures allow flood waters to reach the structure but diminish the flood threat by preventing the water from getting inside the structure. Dry flood proofing measures considered in this screening make the portion of a building that is below the flood level watertight through attaching watertight membranes and installing closure structures in doorway and window openings, referred to as sealants and closures.

- *Dry Flood Proofing with Liquid Storage Tank Modifications.* Liquid storage tanks are subject to floatation during flooding. The International Building Code Appendix G: Flood Resistant Construction specifies that tanks, if not located above the design flood elevation, are to be designed and anchored to prevent flotation, collapse, or lateral movement from hydrostatic loads (including the effect of buoyancy). All tank inlets and vents not above the design flood elevation are to be fitted with covers designed to prevent the inflow of floodwater and the outflow of tank contents, and that these inlets and vents be properly anchored. Anchoring involves installing anti-flotation measures, elevating sensitive equipment, and adding back-up power sources such as generators. Common operational measures include pre-filling the tanks prior to the high water storm event. If an above-ground tank is no longer in use, holes may be cut in the tank to allow the flow of water in and out preventing floatation. In this study, liquid storage tanks were found in conjunction with masonry buildings with slab foundations for which dry flood proofing was appropriate.

- *Wet Flood Proofing*. Wet flood proofing measures allow flood water to get inside lower, non-living space areas of the structure via vents and openings in order to reduce the effects of hydrostatic pressure and, in turn, reduce flood-related damages to the structure's foundation. When a basement is involved, it is filled with compacted earth for foundational stability. Wet flood proofing also involves elevating and/or protecting utilities.

- *Wet Flood Proofing by Pump Modification*. For storm water pump stations, continued operation during floods is desirable. Nonstructural measures involve replacing non-submersible pumps with submersible pumps, elevating sensitive equipment, and adding back up power sources such as generators. Pump controls and motors may be modified by replacing the pump shaft with a longer shaft and mounting the controls and motors at elevation above the design water surface elevation.

- *Elevation (Raise)*. Elevation involves raising the lowest finished floor of a building to a height that is above the flood level. In most cases, the structure is lifted in place and the foundation walls are extended up to the new level of the lowest floor. When a building is in poor condition, elevation is not feasible; in these cases demolition and rebuilding is recommended with the lowest finished floor above the flood levels. The elevation process differs for different foundation types: slab-on-grade, sub grade basement, walkout basement, raised (crawlspace) foundation, bi-levels/raised ranches, or split levels. In this study, no structures were assumed to be elevated on piers, posts, or piles. Elevation was assumed to be feasible for structures having footprint of less than 3,000 sf.

- *Acquisitions (Buyout)*. Relocating resident structures out of the inundation boundary and/or purchasing the structure. Buyouts are considered where the cost of the treatment exceeds the cost of the buyout. Relocations and acquisitions (buyouts) were not considered in this analysis. This evaluation occurs in the later design stages.

In addition to these nonstructural measures, ringwalls and ring levees were investigated as part of a last-added analysis, as decribed later in this document.

- *Barriers (Ringwalls or Ring Levees)*. Barriers usually surround the building but are not attached, such as in the case of ring walls, levees, or berms. It is used where nonstructural measures are not feasible.

3.1.3 Alternative #3 – Diversion Culvert

3.1.3.1 Culvert General Description

The diversion of flood water upstream of the damage center in Woodland Park was evaluated. Upstream of Route 46, flood water would be diverted from the Peckman River to the Passaic River through a 1,500-feet long, 35-foot wide culvert located approximately 550 feet upstream of the Route 46 Bridge. The dimensions of the culvert were analyzed in order to provide the 0.10, 0.02, 0.01 AEP level of performance in the Woodland Park area (for Peckamn River flows). For example, the 0.10 AEP would require a culvert with a width of 25 foot while the 0.01 AEP would be constructed with a 40-foot culvert width, nevertheless the dimension will be further optimized.

It is assumed that culvert would be constructed using a "cut-and-cover" approach, as presented in the Peckman River Basin Initial Appraisal Report (July 2001) and section 1.3.3 of the appendix.

The diversion culvert inlet also consists of an in-line weir, approximately 10-foot high and 130-feet long, that will help divert the flow from the Peckman River into the culvert discharging it into the Passaic River. The weir has a top elevation at 139 ft-NAVD88 with a 6-foot wide x 2-foot high low flow opening. The purpose of the weir and low level opening is to maintain low flow and create a pool near the diversion inlet spillway. This opening will allow daily water flow to pass through without impoundment and divert flows above the 1.0 AEP. Approximately 1,000 feet of channel modifications is required upstream and downstream of the diversion culvert and in-line weir. Although channel modification and excavation is intended to stay at a minimum, due to the high velocities along the river and unstable banks, streambank erosion measures are necessary. The streambank erosion measures includes riprap and articulated concrete blocks. Levees and/or floodwalls will be necessary along the banks upstream and downstream of the diversion culvert inlet and in-line weir. The levees and/or floodwalls will work in tandem with the in-line weir to divert the flow towards the diversion culvert. The levees/floodwall are approximately 2,500 feet long with and with height ranging from three to six foot. The performance of the in-line weir spillway, top elevation, culvert openings size and low flow opening will be optimized during future design stages.

The diversion culvert would significantly reduce downstream peak discharges (i.e., flash flooding), and subsequently, downstream flood elevations and flood damages. The diversion alternative would not protect against Passaic River backwater effects the lower reaches of the Peckman River basin in the Woodland Park damage areas. Nearly all flood risk management benefits from the diversion culvert would be in Woodland Park.

As a component of this alternative, it would also be necessary to construct approximately 3,000 feet of floodwalls and levees to contain the discharge in the lower reach of Great Notch Brook that would extend to the confluence with the Peckman River. The floodwall height ranges between five to 10 feet, with top elevation from approximately 139 feet-NAVD88 close to Route 46 to approximately 150 foot-NAVD88 close to Browertown Road.

The amount of flood water that would be diverted, even during large storm events, is relatively small in comparison to the Passaic River and its waterershed. The culvert is not expected to induce flooding in communities along the Passaic River that are located downriver of the project area. See **Figure 25** for the layout of this alternative.

The difference in elevation between the inlet and outlet of the diversion culvert is approximately 13.25 feet. If the Passaic River experience a 0.02 AEP storm event, there is the possibility that Passaic River flood waters would back up into the diversion culvert and enter the Peckman River. However, there would be no additional damages that would already be experienced by the Passaic River backwater backing up into the Peckman River at the confluence in Woodland Park.

3.1.3.2 Culvert Model Approach

The culvert was developed as an unsteady HEC-RAS model. The culvert is represented as a reach connecting the Peckman River to the Passaic River. The cross section is rectangular with a lid representing the top of the culvert. The mannings roughness coefficient varies between 0.015 to 0.020. Cross section are placed every 50 ft at areas with changes in slope and width. The upstream and downstream boundaries are computed with split flow and forcing equal water surface elevation at the confluence of the diversion from the Peckman and the Passaic Rivers. The culvert has three bends, two of 20 deg and one of 90 deg. The minimum curvature radius is 100 ft. The culvert entrance slope is 0.004 ft/ft and the main barrel is 0.008 ft/ft. The velocity at the culvert is variable and are expected to reach approximately 28 ft/s at the entrance of the diversion. See **Figure 22** through

Figure 24 for the preliminary profile and aligment of the diversion culvert.

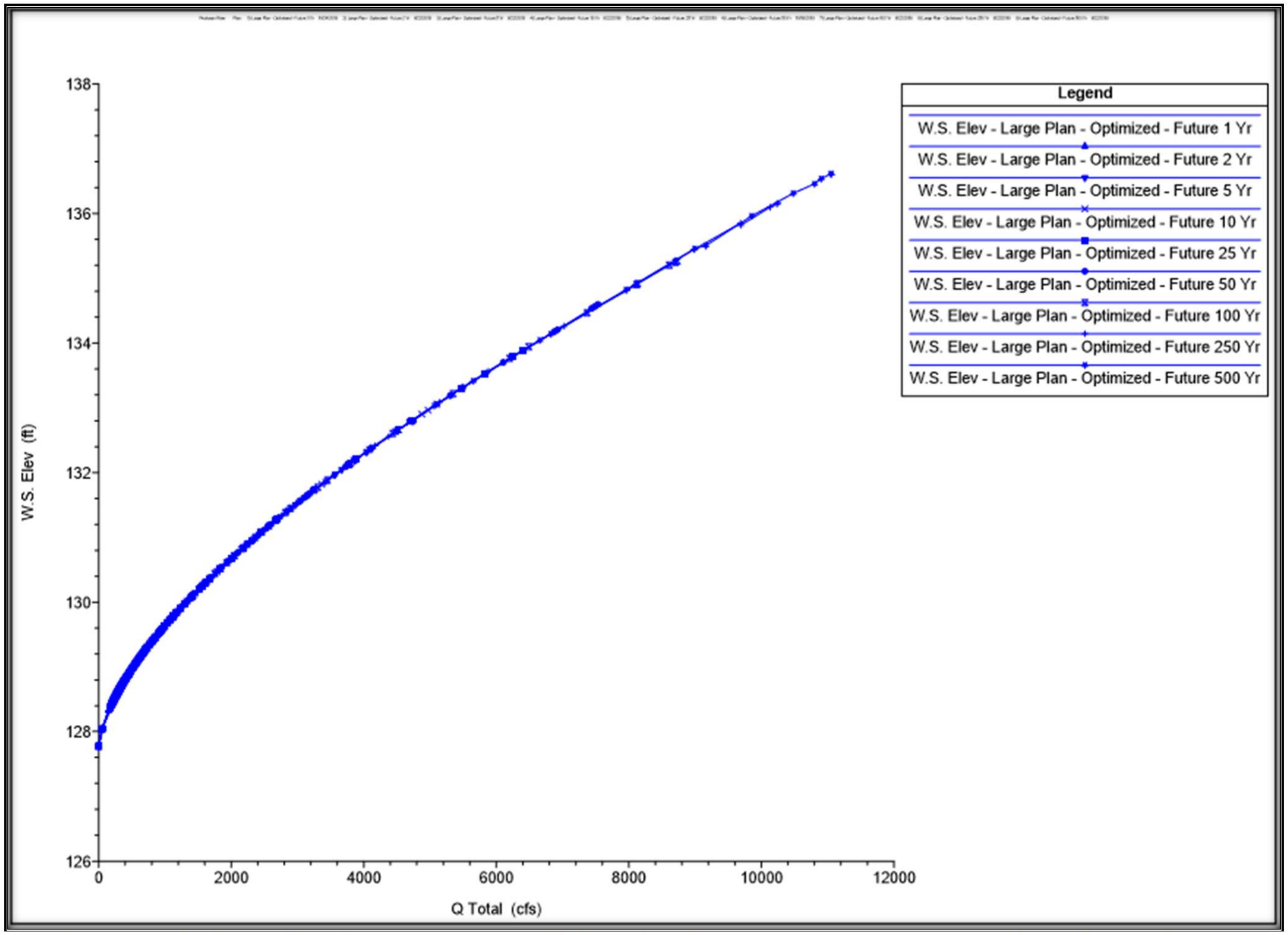

Figure 22: Stage Discharge curve for Inline Weir #1 at the entrance of diversion culvert.

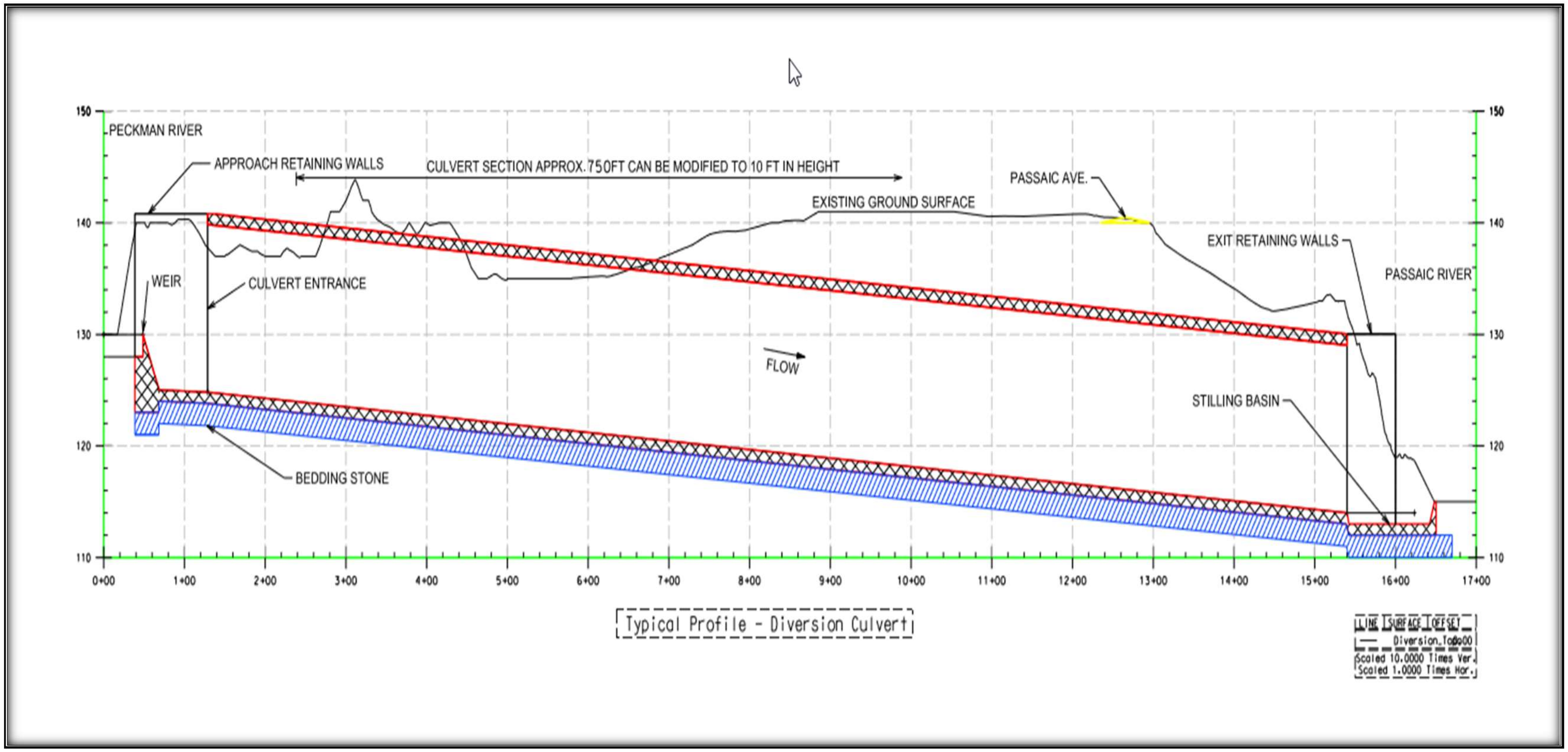

Figure 23: Alternative #3 – Diversion culvert profile.

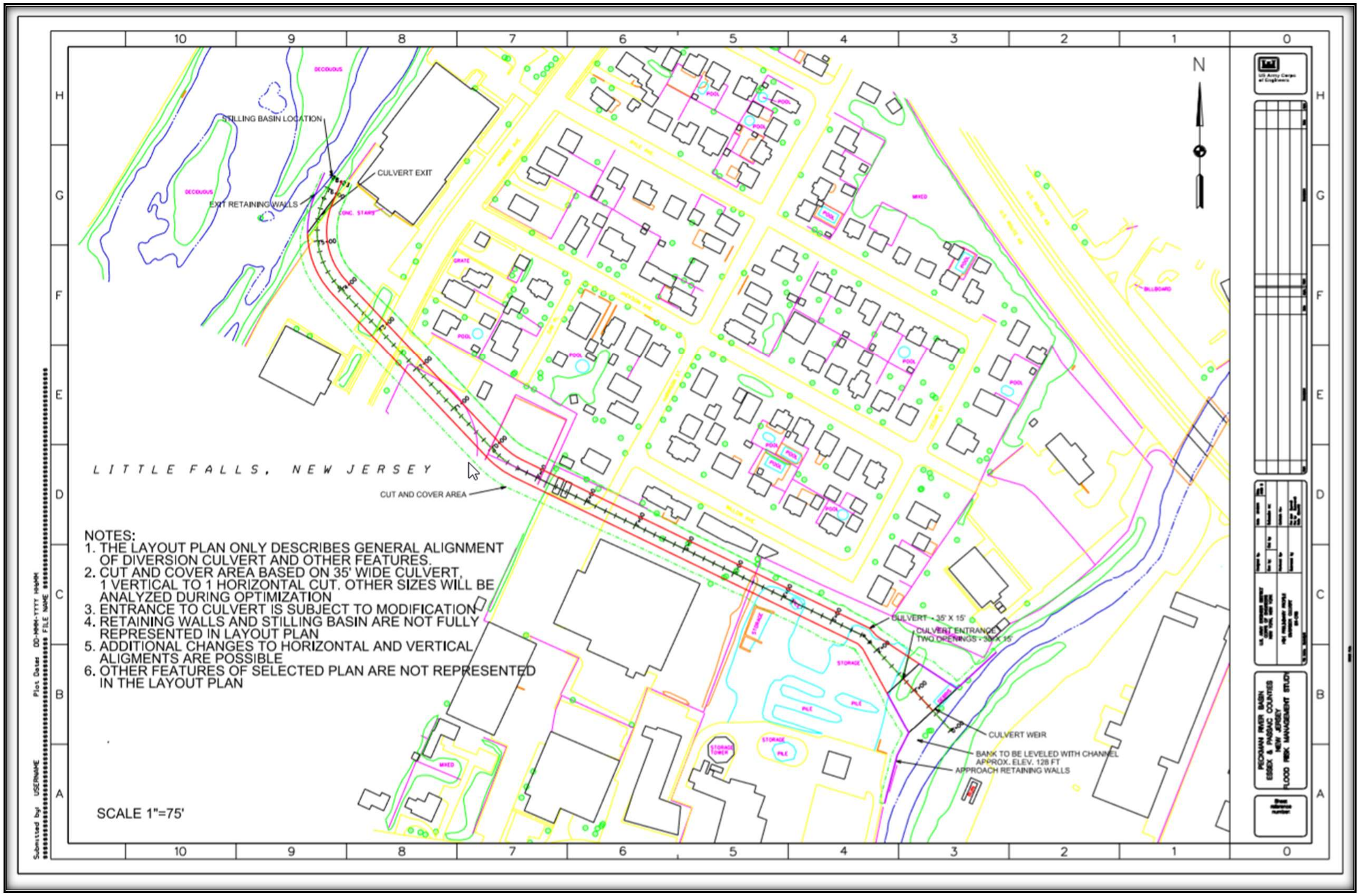

Figure 24: Alternative #3 – Diversion culvert plan view.

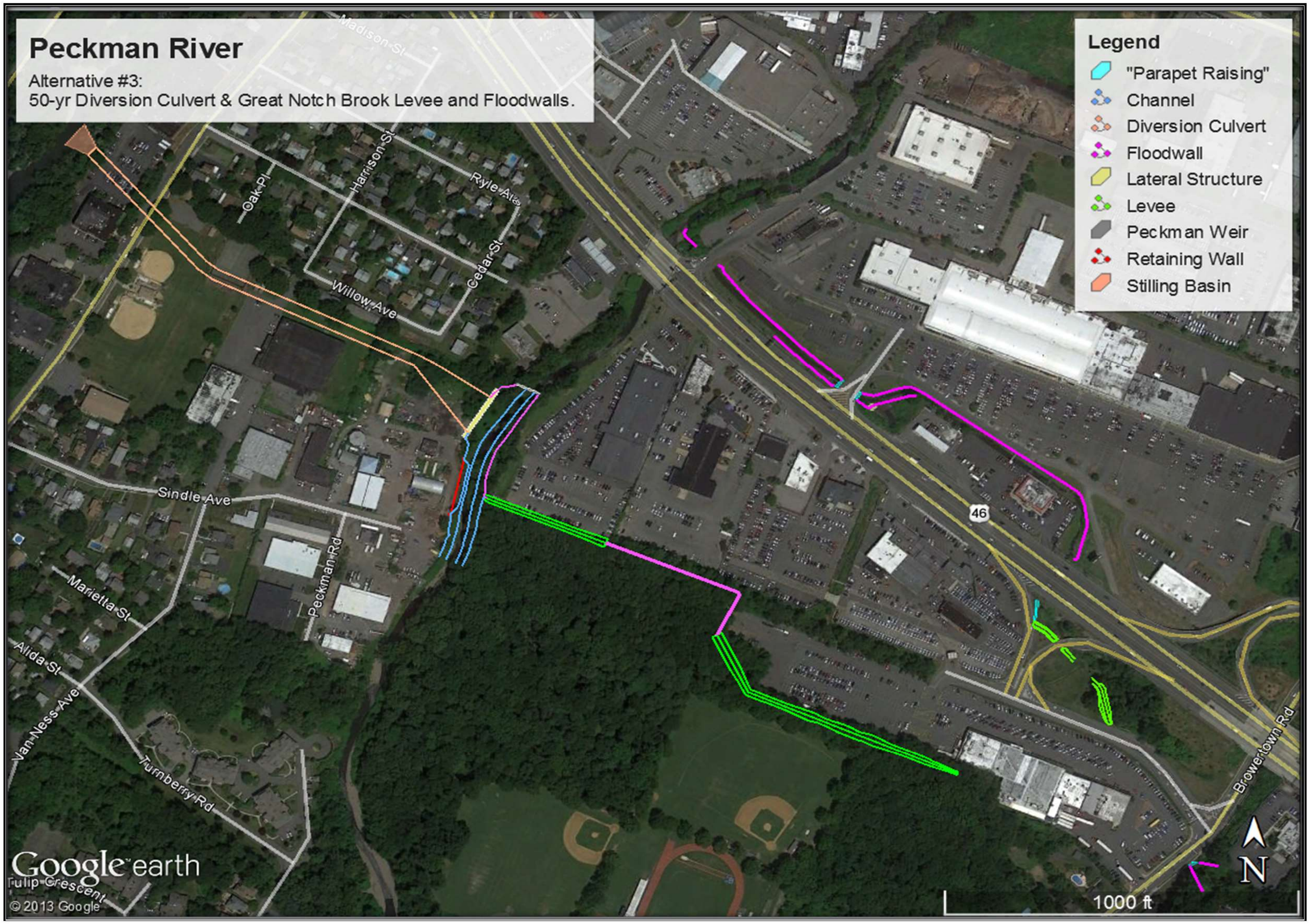

Figure 25: Alternative #3 – Diversion Culvert with Great Notch Brook Floodwalls.

3.1.4 Alternative #4 – Channel Improvements providing flood damage reduction up/downstream of Rt. 46

An extensive Peckman River channel modification was also considered in this analysis, see **Figure 26**. Due to the increased width of the Peckman River mainstem, reconstruction of the Route 46, Lackawanna Avenue, and McBride Avenue Bridges is required. To accommodate the discharge of the 0.10 AEP, a 30-foot (base) trapezoidal channel would be required with earthen side slopes. For the 0.02 AEP, a 60-foot rectangular channel with concrete sidewalls would be required to effectively convey the flood discharge downstream to the confluence of the Passaic River. The 0.01 AEP storm frequency would require a 70-foot rectangular channel cross section in combination with a 60-foot trapezoidal channel in the Peckman River downstream and upstream, respectively, of the Route 46 Bridge. As analysed the channel modification will require approximately 15,000 feet of retaining walls along the lower reach of the Peckman River.

As a component of this alternative, it would also be necessary to construct approximately 3,000 feet of floodwalls and levees to contain the discharge in the lower reach of Great Notch Brook that would extend to the confluence with the Peckman River. The floodwall height ranges between five to 10 foot, with top elevation from approximately 139 foot-NAVD88 close to Route 46 to approximately 150 foot-NAVD88 close to Browertown Road. See **Figure 26** for the layout of this alternative.

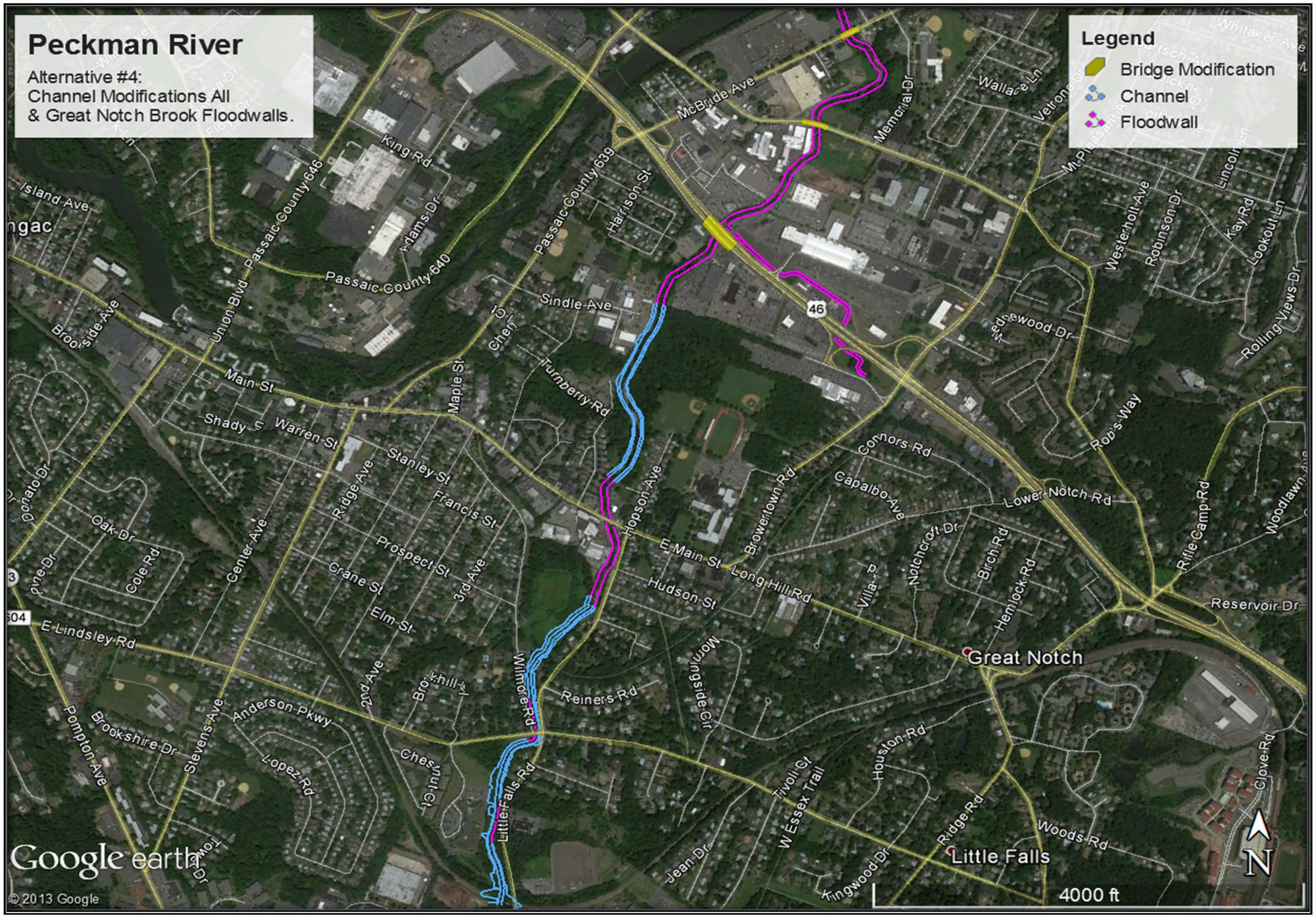

Figure 26: Alternative #4 – Channel Improvement Upstream and Downstream of Rt. 46 & Great Notch Brook Floodwalls.

3.1.5 Alternative #5 – Levee/Floodwall providing flood damage reduction up/downstream of Rt. 46

This alternative consists of approximately 12,000 feet of levees and floodwalls starting from the confluence of the Passaic River and extending upstream, with an average height of 8 feet along the Peckman River and four (4) bridge replacements. Refer to **Figure** 27 for a depiction of this alternative. Where adequate real estate is available, levees would be constructed, otherwise floodwalls would necessary. For this alternative approximately 20 percent of the design could be levees with the remainder being floodwalls. This would provide protection of the primary damage centers in Woodland Park and Little Falls up to the 0.01 AEP.

As a component of this alternative, it would also be necessary to construct approximately 3,000 feet of floodwalls and levees to contain the discharge in the lower reach of Great Notch Brook that would extend to the confluence with the Peckman River. The floodwall height ranges between five to 10 feet, with top elevation from approximately 139 foot-NAVD88 close to Route 46 to approximately 150 foot-NAVD88 close to Browertown Road. This plan would also require road closure gates and/or road raisings at Lackawanna Avenue and McBride Avenue Bridges. Pump stations would be needed with this plan to ensure interior drainage of the areas behind the levee/floodwall system. It should be noted that with this alternative there is a compounding effect with the Passaic River backwater. This design alternative only evaluated flood protection for the Peckman River and does not include measures to protect against Passaic River flooding or Passaic River backwater effects.

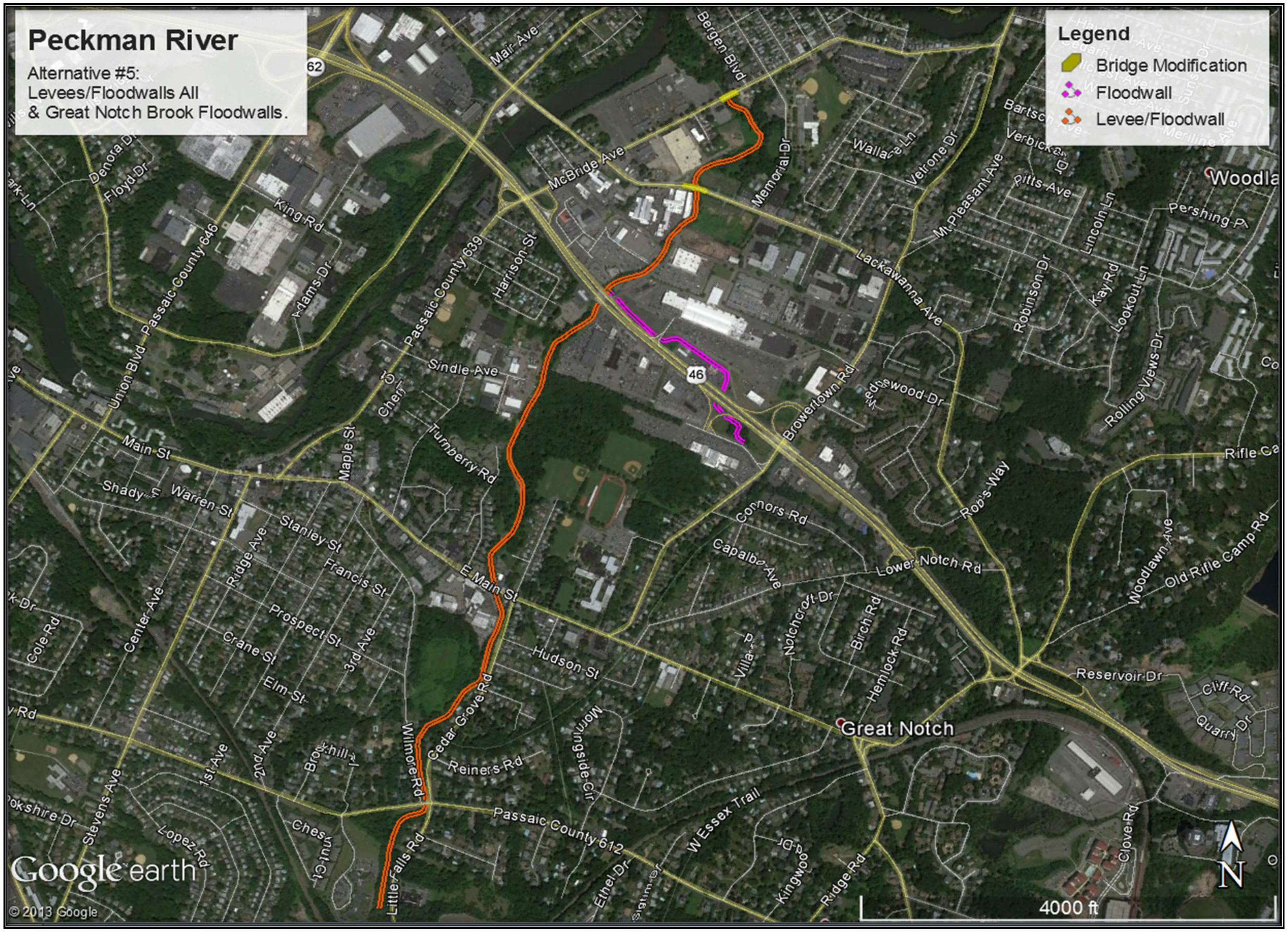

Figure 27: Alternative #5 – Levee/Floodwall Upstream and Downstream of Rt. 46 & Great Notch Brook Floodwalls.

3.1.6 Alternative #6 – Levee/Floodwall providing flood damage reduction downstream of Rt. 46

This alternative consists of approximately 12,000 feet of floodwalls starting from the confluence of the Passaic River and extending upstream to Route 46, with an average height of eight feet along the Peckman River. Refer to **Figure 28** for a depiction of this alternative.

As a component of this alternative, it would also be necessary to construct approximately 3,000 feet of floodwalls and levees to contain the discharge in the lower reach of Great Notch Brook that would extend to the confluence with the Peckman River. The floodwall height ranges between 5 to 10 feet, with top elevation from approximately 139 foot-NAVD88 close to Route 46 to approximately 150 foot-NAVD88 close to Browertown Road. This plan would also require road closure gates and/or road raisings at Lackawanna Avenue and McBride Avenue Bridges. Pump stations would be needed with this plan to ensure interior drainage of the areas behind the floodwall. It should be noted that with this alternative there is a compounding effect with the Passaic River backwater. This design alternative only evaluated flood protection for the Peckman River and does not include measures to protect against Passaic River flooding or Passaic River backwater effects.

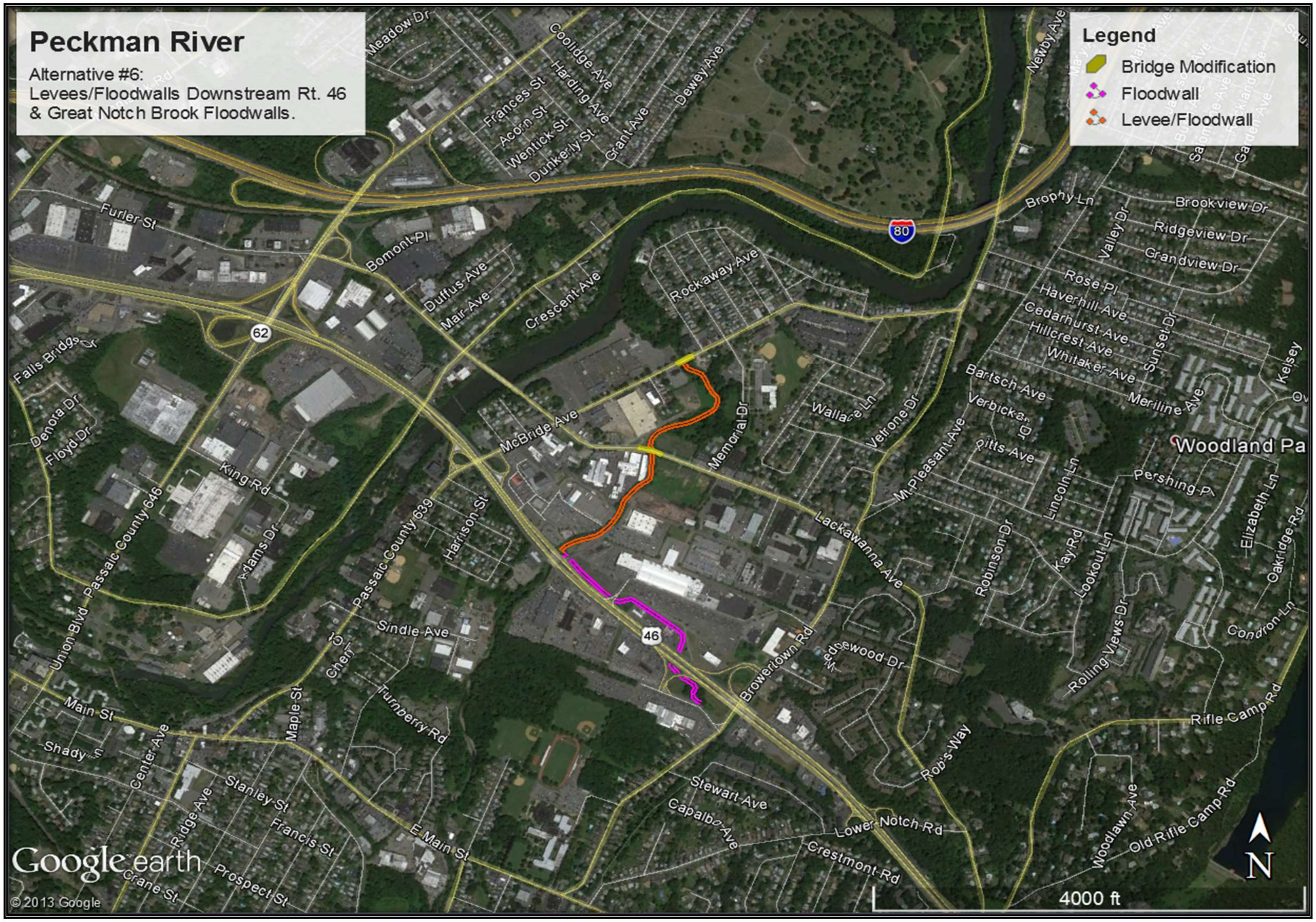

Figure 28: Alternative #6 – Levee/Floodwall Downstream of Rt.46 & Great Notch Brook Floodwalls.

3.1.7 Alternative #7 – Channel Improvements providing flood damage reduction downstream of Rt. 46

This alternative is as described above in section 3.1.4, channel improvement providing flood damage reduction upstream and downstream of Route 46, with the exception of the upstream of Route 46 component. This would reduce the amount of channel excavation by 4/5 in terms of volume and approximately 3,000 feet of retaining walls, but would not protect the upper reaches of the study area. See **Figure 29** for the layout of this alternative.

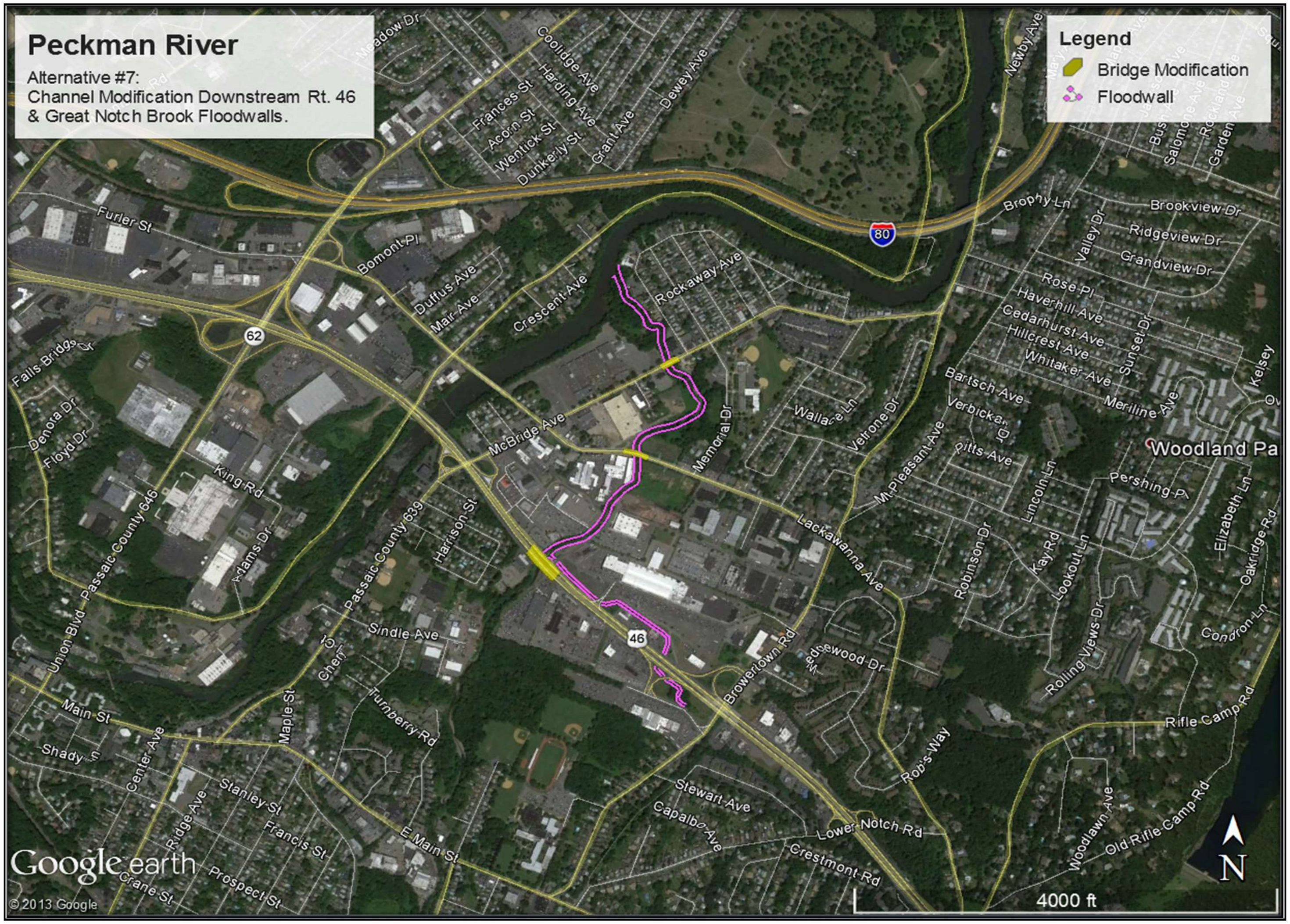

Figure 29: Alternative #7 – Channel Improvement Downstream of Rt. 46 & Great Notch Brook Floodwalls.

3.1.8 Alternative #8 – Combined Channel Improvements with Diversion Culvert providing flood damage reduction upstream of Rt. 46

This alternative is as described above in section 3.1.3, diversion culvert, plus channel improvement providing flood damage reduction upstream Route 46, section 3.1.4. However, unlike section 3.1.4, this alternative does not include channel improvement downstream of Route 46. See **Figure 30** for the layout of this alternative.

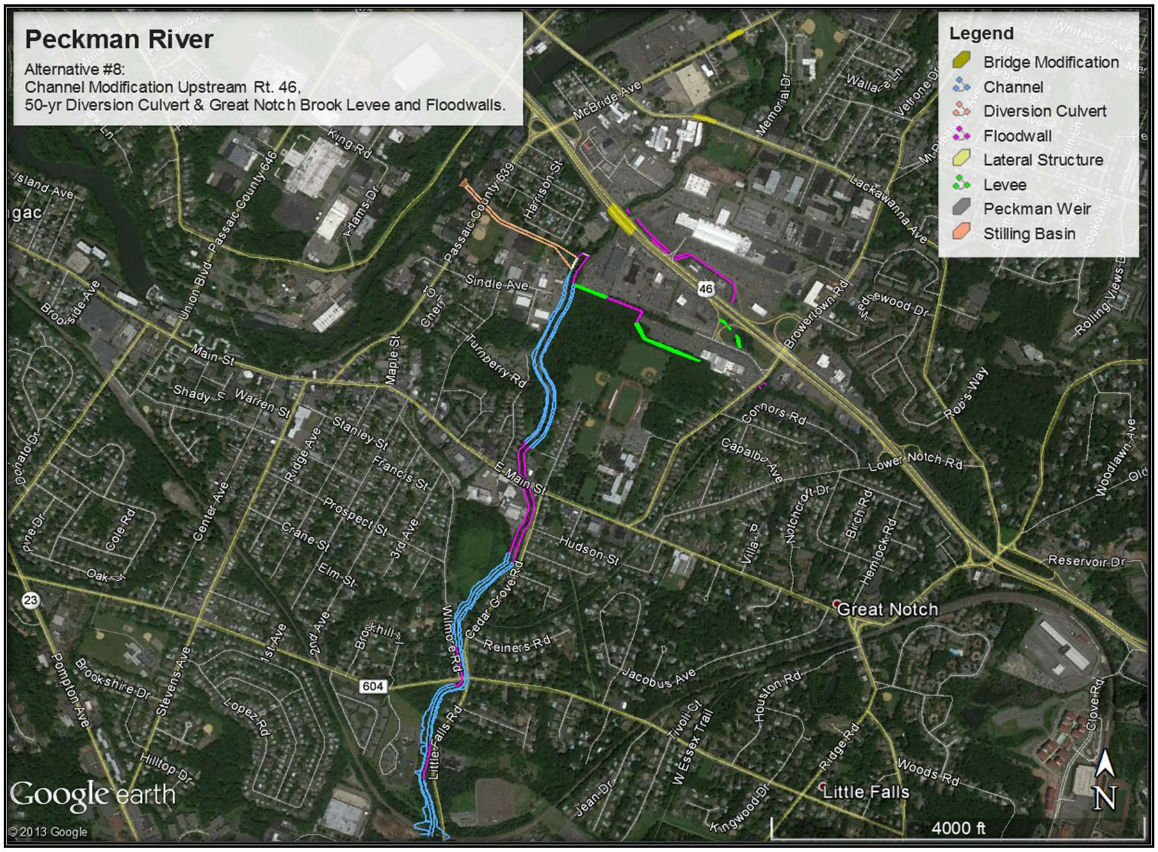

Figure 30: Alternative #8 – Combined Channel Improvement Upstream of Rt. 46 with Diversion Culvert & Great Notch Brook Floodwalls

3.1.9 Alternative #9 (Former LPP) – 0.01 AEP. Combined Levee/Floodwall with Diversion Culvert providing flood damage reduction upstream of Rt. 46

This alternative is as described above in section 3.1.3 and 3.1.5, a 35 foot diversion culvert plus levee/floodwall providing flood damage reduction upstream of Route 46. However, unlike section 3.1.5, this alternative does not include levee/floodwalls downstream of Route 46.

This alternative requires approximately 12,000 feet of levees and floodwall, 9,000 feet in Peckman River, and 3,000 feet in Great Notch Brook, with a average height of approximately 8 feet. It also requires approximately 18 interior drainage structures with at least three pump station for small tributaries and/or bigger drainage areas. To accommodate the levees and floodwalls the alternative includes approximately 6 structure buyouts near the bank of the river. The plan also includes two bridge replacements, Main Ave. E and Lindsley Road., and an automatic hydraulic gate structure at E. Main Street. This bridge would close to traffic during extreme storm events.

Although channel modification and excavation was intended to stay at a minimum, due to the high velocities along the river and unstable banks, streambank erosion measures are necessary along river sections with levees and floodwalls in the overbanks. Channel modification with riprap and articulated concrete blocks are required to eliminate the erosion and possible undermining of the proposed levee/floodwall.

As with any of the other alternatives, this study only evaluated flood protection for the Peckman River and does not include measures to protect against Passaic River flooding or Passaic River backwater effects. See **Figure 31** for the layout of this alternative.

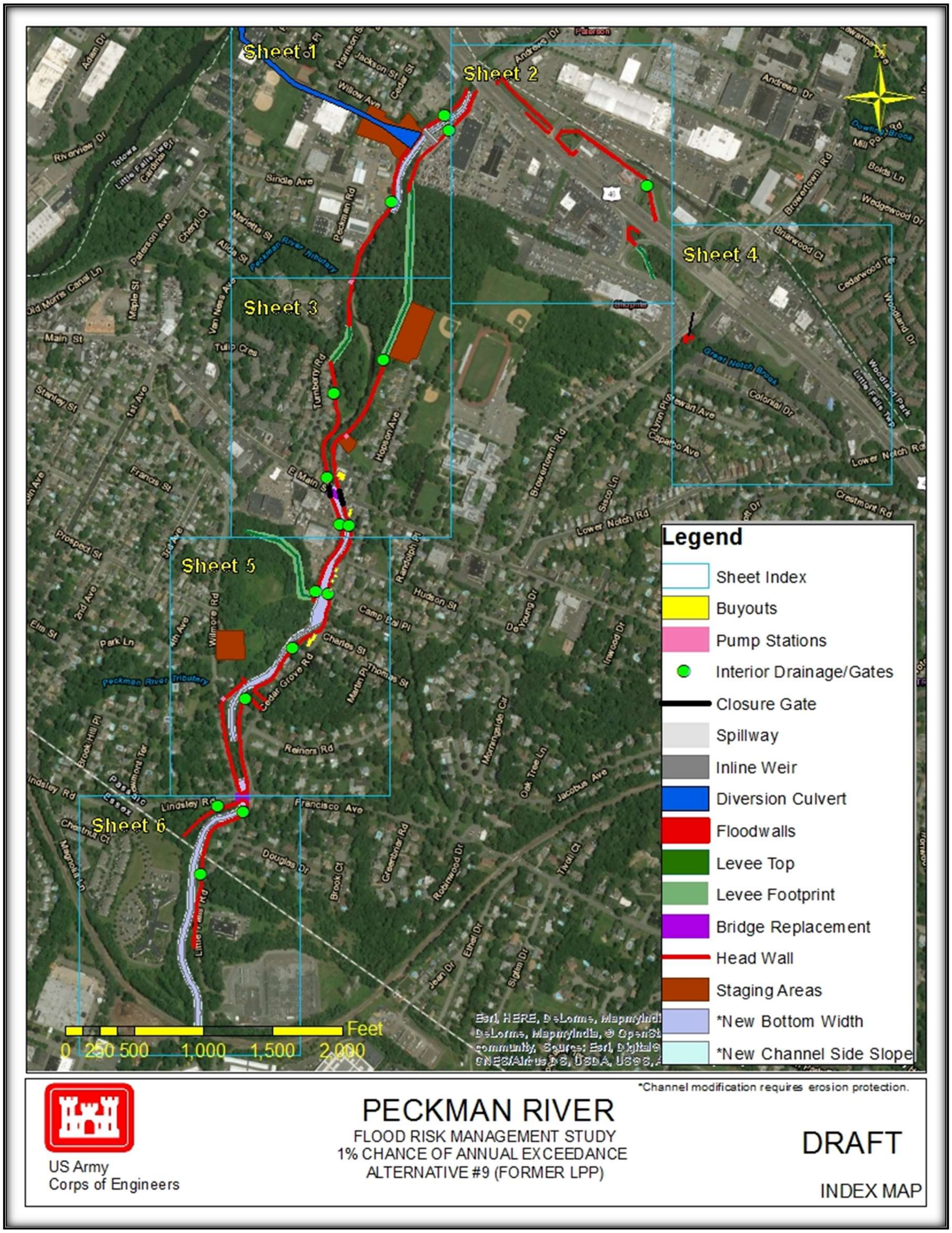

Figure 31: Alternative #9 – 100 yr. Combined Levee/Floodwall Upstream of Rt. 46 with Diversion Culvert & Great Notch Brook Floodwalls.

3.1.10 Alternative #10a – Diversion Culvert plus 0.02 AEP Non-structural Measures Upstream of Route 46

This alternative is a combination of a diversion culvert as described above in section 3.1.3, and non-structural measures upstream of US Route 46 within the 0. 02 AEP floodplain (**Table 14**).

Table 14: summary of the non-structural component of alternative 10a.

Treatment	Residential	Non-Residential	Sub-Total
Raise (Elevated)	71	0	71
Wet Proofing	27	2	29
Dry Proofing	17	12	29
*Barrier (Ringwall)	3	48	51
Buyout	N/A	N/A	N/A
Total	118	62	180

*Number of strctures within barrier.

Non-structural measures were identified and evaluated for structures in Little Falls near the Peckman River. The procedure to determine the non-structural component is described in section 3.1.2. These measures, within the 0.02 AEP inundation area, were designed to withstand inundation stage up to and including a 0.01 AEP event plus one foot to account for the uncertainty of wave effects. Measures evaluated included raising buildings (elevation), wet (protect utilities) and dry (sealants and closures) flood proofing, and buyouts (acquisition). Barriers (ring walls/ring levees) were considered during a final analysis.

3.1.11 Alternative #10b (TSP) – Diversion Culvert plus 0.10 AEP Non-structural Measures Upstream of Rt. 46

This alternative is a combination of a diversion culvert as described above in section 3.1.3, and non-structural measures upstream of US Route 46 within the 0.1 AEP floodplain (**Table 15**). See **Figure 32** for the layout of this alternative

.

Table 15: Summary of the non-structural and ringwall components of Alternative 10B.

Treatment	Residential	Non-Residential	Sub-Total
Raise (Elevated)	16	0	16
Wet Proofing	29	9	38
Dry Proofing	4	0	4
*Barrier (Ringwall)	0	0	0
Buyout	N/A	N/A	N/A
Total	49	9	58

*Number of strctures within barrier.

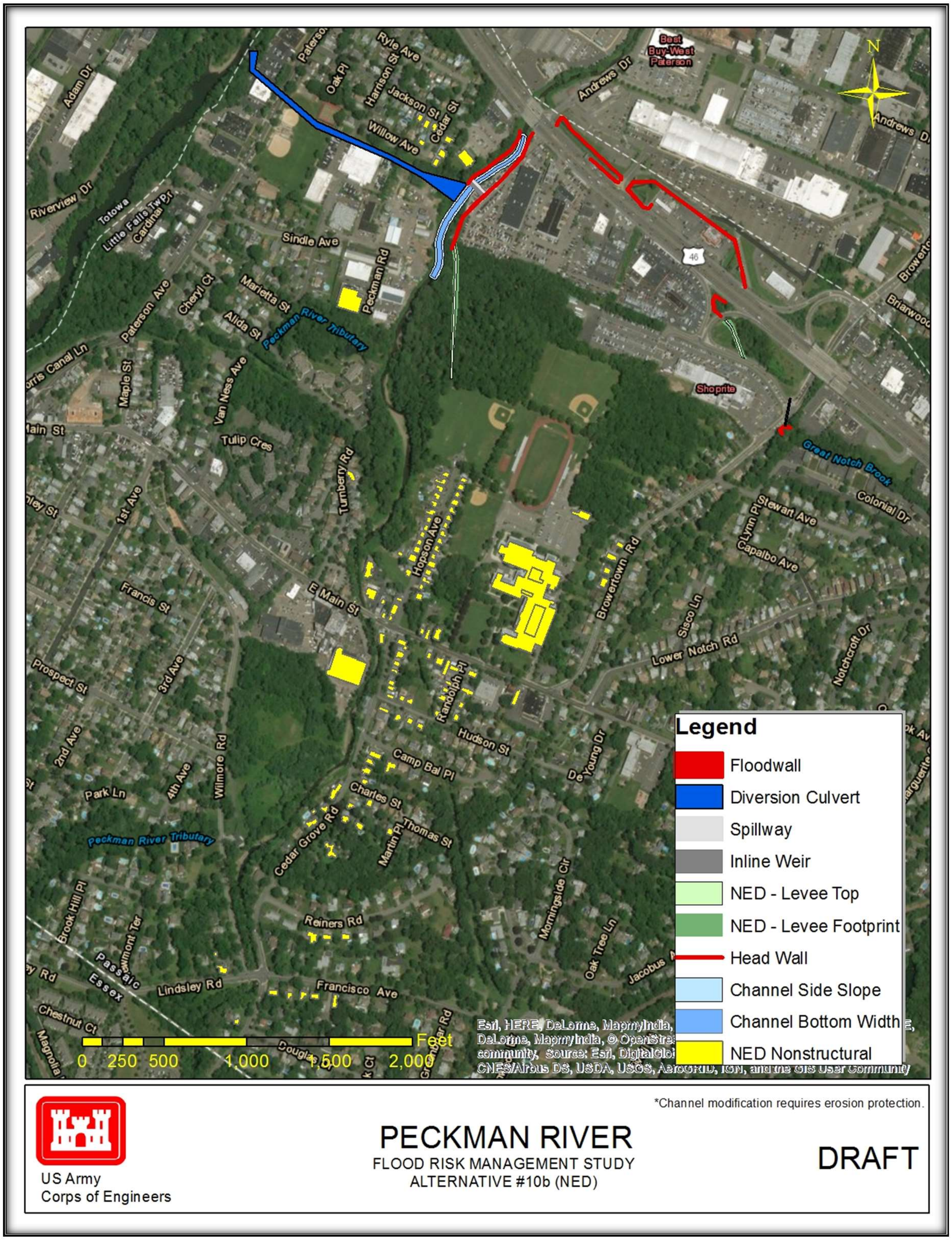

Figure 32: Alternative #10b –Nonstructural Combined with Diversion Culvert.

*Note Alternative #10a is similar to this alternative; the locations of nonstructural measures vary from Alternative 10b.

Non-structural measures were identified and evaluated for structures in Little Falls near the Peckman River. The procedure to determine the non-structural component is described in section 3.1.2. These measures, within the 0.1 AEP inundation area, were designed to withstand inundation stage up to and including a 0.01 AEP return period event plus one foot to account for the uncertainty of wave effects. Measures evaluated included raising buildings (elevation), wet (protect utilities) and dry (sealants and closures) flood proofing, and buyouts (acquisition). Barriers (ring walls/ring levees) were considered during a final analysis. The main objective for most of the non-structural measures is to reduce flood damages through modifications of the existing structures. The total amount of measures and the detailed location, type of structure and measure are presented in **Table 16**. In the table, for the type of strcture, “R” represents a residential stucturel, “NR”, a nonresidential structure.

Table 16: Structures and treatment for the 0.1 AEP non-structural plan.

Residential or Nonresidential	Structure ID	Description of Structure	Recommended Treatment
NR	643.1	School Structure	NR Raise A/C
NR	643.2	School Structure	NR Raise A/C
NR	644	School Structure	NR Raise A/C
NR	645	School Structure	NR Raise A/C
NR	648	School Structure	NR Raise A/C
NR	923	Medical Office	NR Fill Basement 2000 sf
NR	934	Auto Service	NR Raise A/C
NR	951	Medical Office	NR Raise A/C
NR	1055	Cape Cod converted to Office	NR Raise A/C

Residential or Nonresidential	Structure ID	Description of Structure	Recommended Treatment
R	649	Colonial SFH	R Elevate Basement Foundation 3000 sf
R	650	Two-Family	R Raise A/C
R	651	Colonial SFH	R Elevate Basement Foundation 1000 sf
R	654	Duplex	R Fill Basement 2000 sf
R	655	Ranch SFH	R Fill Basement 1000 sf
R	656	Ranch SFH	R Elevate Slab Foundation 2000 sf
R	657	Cape Cod SFH	R Elevate Slab Foundation 2000 sf
R	660	Cape Cod SFH	R Dry Floodproofing 1000 sf
R	661	Cape Cod SFH	R Dry Floodproofing 2000 sf
R	662	Colonial SFH	R Dry Floodproofing 2000 sf
R	663	Cape Cod SFH	R Elevate Slab Foundation 2000 sf
R	664	Ranch SFH	R Elevate Slab Foundation 1000 sf
R	667	Ranch SFH	R Dry Floodproofing 2000 sf
R	668	Ranch SFH	R Elevate Slab Foundation 1000 sf
R	669	Cape Cod SFH	R Elevate Slab Foundation 2000 sf
R	670	Cape Cod SFH	R Elevate Slab Foundation 2000 sf
R	671	Ranch SFH	R Elevate Slab Foundation 1000 sf
R	672	Ranch SFH	R Fill Basement 2000 sf

Residential or Nonresidential	Structure ID	Description of Structure	Recommended Treatment
R	674	Ranch SFH	R Fill Basement 2000 sf
R	675	Colonial SFH	R Fill Basement 2000 sf
R	678	Ranch SFH	R Fill Basement 2000 sf
R	679	Garden Apartment	R Elevate Slab Foundation 4000 sf
R	682	Garden Apartment	R Fill Basement 3000 sf
R	918	Colonial SFH	R Elevate Slab Foundation 1000 sf
R	924	Cape Cod SFH	R Fill Basement 3000 sf
R	925	Ranch SFH	R Fill Basement 2000 sf
R	927	Colonial SFH	R Fill Basement 2000 sf
R	929	Colonial SFH	R Fill Basement 1000 sf
R	931	Colonial SFH	R Fill Basement 2000 sf
R	933	Colonial SFH	R Fill Basement 1000 sf
R	935	Chiropractor w/Residential	R Elevate Slab Foundation 3000 sf
R	936	Duplex	R Fill Basement 2000 sf
R	937	Colonial SFH	R Fill Basement 1000 sf
R	938	Multi-Family	R Fill Basement 2000 sf
R	939	Colonial SFH	R Fill Basement 2000 sf
R	944	Ranch SFH	R Fill Basement 1000 sf

Residential or Nonresidential	Structure ID	Description of Structure	Recommended Treatment
R	945	Colonial SFH	R Fill Basement 2000 sf
R	946	Ranch SFH	R Fill Basement 2000 sf
R	982	Cape Cod SFH	R Fill Basement 2000 sf
R	983	Cape Cod SFH	R Fill Basement 2000 sf
R	984	Ranch SFH	R Fill Basement 2000 sf
R	1014	Ranch SFH	R Fill Basement 4000 sf
R	1028	Split Level SFH	R Fill Basement 2000 sf
R	1029	Split Level SFH	R Fill Basement 2000 sf
R	1030	Ranch SFH	R Fill Basement 4000 sf
R	1054	Cape Cod SFH	R Fill Basement 2000 sf
R	6720	Cape Cod SFH	R Elevate Slab Foundation 1000 sf
R	6730	Cape Cod SFH	R Elevate Slab Foundation 2000 sf
R	6750	Cape Cod SFH	R Elevate Slab Foundation 2000 sf

3.2 Interior Drainage

Since the TSP includes floodwalls and levees, interior drainage features are needed. However, a detailed interior drainage will be done in PED phase. Currently, interior drainage outfalls (spacing approximately 400 ft.) and size between 18’’ to 24’’ until be installed within levee/floodwalls to allow water to convey to the Peckman River.

4.0 OPTIMIZATION OF THE TSP

4.1 The Tentatively Selected Plan (TSP)

As a result of the plan formulation process, it was concluded that Alternative 10b was the plan with the highest net benefits and was identified as the Tentatively Selected Plan (TSP). The TSP was modified from the alternative 10b seen above in section 3.1.11, and featured changes such as removal of the Great Notch Brook flood walls and removal of the Hopson Avenue ringwall. Refer to the main report to see the full list of changes from the TSP to the renamed "Modified TSP" and all of the justifications associated with this change.

4.2 Optimization of the TSP (also known as Plan 10b-40)

Alternative 10b featured 3 options for sizing of the diversion culvert. The small plan featured a culvert width of 25'(also known as Plan 10b-25), the standard plan featured a culvert width of 35' (also known as Plan 10b-35), and the large plan featured a culvert width of 40' (also known as 10b-40). Additional optimization consideratings were the heights of the weirs, height of levess, and the invert elevation of the channel modifications. During the optimization process, it was apparent that there was an inverse relationship between the culvert size and the height of the levees. A smaller culvert opening would mean taller levees, therefore the small plan was not further analyzed.

The plan was simultaneously optimized in size and combined with non-structural features to determine the final size of the culvert with the maximum net benefits. The plan and size with the maximum net benefits would become the National Economic Development (NED) plan. See **Figure 33** for the future 50 year storm event inundation with the NED plan in place. All structural details and layouts including the levees, flood walls, culvert, stilling basin, weirs, and retaining walls can be seen in the Civil Appendix.

The optimization was a comparison between the 35 ft. wide culvert and the 40 ft. wide culvert. This plan featured one in-line weir (weir #2) and one small weir at the inlet of the culvert to control sedimentation and the amount of flow entering the culvert (weir #1). Opening sizes in weir #2 were altered to maintain a baseflow down the Peckman river. With the baseflow established, the plan selected was the option that passed the most flood water through the diversion culvert leading to the Passaic River. Additionally, the height of weir #1 was altered to allow additional flow to convey into the culvert. Refer to **Table 17** to see many of the different optimization options considered. Refer to **Table 18** to see the results.

After running the different options and analyzing the results, the optimization was complete. The final plan would feature:

-40ft. wide culvert

-Weir #1 upstream height of 0.5 ft.

-Weir #2 upstream height of 8.18 ft.

-Weir #2 opening size of 8 ft. wide by 3 ft. high

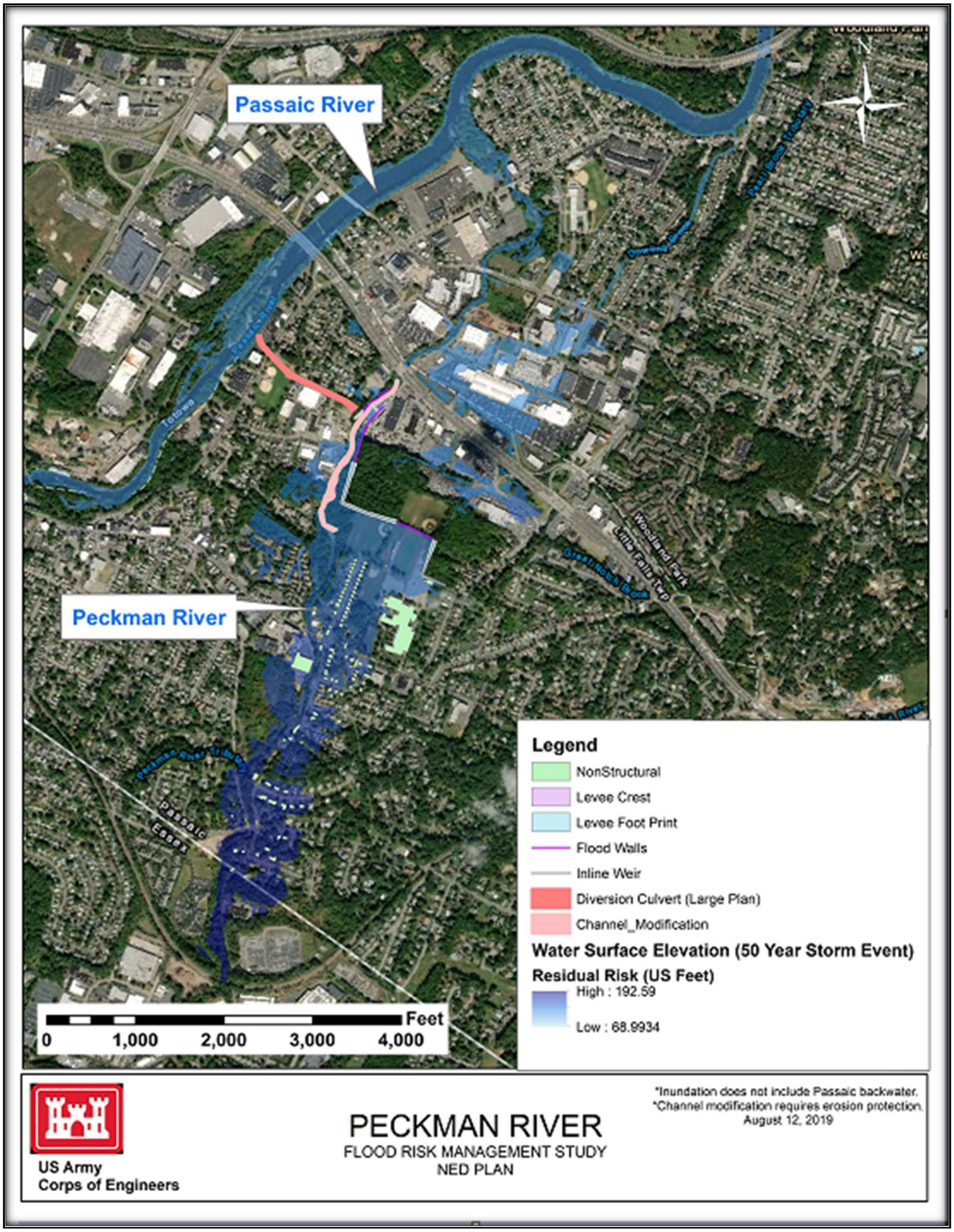

Figure 33: NED Plan with 50 Year Future Inundation map (Alternative 10b – Modified TSP).

Table 17: Optimization Options Tested.

Name	Weir #1 Size	Weir #2 Size	Weir #2 Openings Size (height x width)
Plan 10b-35	1.5 ft	Top of crest: 135 ft Width: 221.27 ft	6’x4’
Plan 10b-35 - 1 ft weir - 6'x4' Opening	1 ft	Top of crest: 135 ft Width: 221.27 ft	6’x4’
Plan 10b-35 – 0.5 ft weir - 6'x4' Opening	0.5 ft	Top of crest: 135 ft Width: 221.27 ft	6’x4’
Plan 10b-40	1.5 ft	Top of crest: 135 ft Width: 221.27 ft	6’x4’
Plan 10b-40 – 1 ft weir – 6’x4’	1 ft	Top of crest: 135 ft Width: 221.27 ft	6’x4’
Plan 10b-40 – 0.5 ft weir – 6’x4’	0.5 ft	Top of crest: 135 ft Width: 221.27 ft	6’x4’
Plan 10b-40 – 1.5 weir - 10x2.5	1.5 ft	Top of crest: 135 ft Width: 221.27 ft	10’x2.5’
Plan 10b-40 – 1 weir - 10x2.5	1 ft	Top of crest: 135 ft Width: 221.27 ft	10’x2.5’
Plan 10b-40 – 0.5 weir - 10x2.5	0.5 ft	Top of crest: 135 ft Width: 221.27 ft	10’x2.5’
Plan 10b-40 – 1.5 ft weir - 8x3	1.5 ft	Top of crest: 135 ft Width: 221.27 ft	8’x3’
Plan 10b-40 – 1 ft weir - 8x3	1.0 ft	Top of crest: 135 ft Width: 221.27 ft	8’x3’
Plan 10b-40 – 0.5 ft weir - 8x3	0.5 ft	Top of crest: 135 ft Width: 221.27 ft	8’x3’
Plan 10b-40 – 1.5 ft weir - 12x2	1.5 ft	Top of crest: 135 ft Width: 221.27 ft	12’x2’
Plan 10b-40 – 1.0 ft weir - 12x2	1.0 ft	Top of crest: 135 ft Width: 221.27 ft	12’x2’
Plan 10b-40 – 0.5 ft weir - 12x2	0.5 ft	Top of crest: 135 ft Width: 221.27 ft	12’x2’

Table 18: Optimization Results.

Name	Flow at XS 800 (in culvert)	WSE at XS 800 (in culvert)	Flow at XS 4671.792 (downstream of weir #2)	WSE at XS 4671.792 (downstream of weir #2)	Flow at XS 5275.938 (Upstream of weir #2)	WSE at XS 5275.938 (Upstream of weir #2)
Plan 10b-35	7356.09 cfs	127.24 ft	459.12 cfs	132.10 ft	7424.43 cfs	136.57 ft
Plan 10b-35 – 1 ft weir - 6'x4' Opening	7516.32 cfs	127.35 ft	299.52 cfs	131.57 ft	7426.34 cfs	136.22 ft
Plan 10b-35 – 0.5 ft weir - 6'x4' Opening	7575.68 cfs	127.39 ft	238.57 cfs	131.35 ft	7426.93 cfs	135.89 ft
Plan 10b-40	7390.47 cfs	126.90 ft	466.72 cfs	132.14 ft	7539.89 cfs	136.59 ft
Plan 10b-40 – 1 ft weir – 6’x4’	7686.17 cfs	126.97 ft	321.93 cfs	131.62 ft	7662.34 cfs	136.31 ft
Plan 10b-40 – 0.5 ft weir – 6’x4’	7603.84 cfs	127.02 ft	236.27 cfs	131.34 ft	7525.48 cfs	135.87 ft
Plan 10b-40 – 1.5 weir - 10x2.5	7349.94 cfs	126.87 ft	508.39 cfs	132.25 ft	7540.80 cfs	136.58 ft
Plan 10b-40 - 1.0 weir - 10x2.5	7485.42 cfs	126.95 ft	347.06 cfs	131.75 ft	7512.60 cfs	136.21 ft
Plan 10b-40 – 0.5 weir - 10x2.5	7551.47 cfs	126.99 ft	288.63 cfs	131.55 ft	7525.75 cfs	135.85 ft
Plan 10b-40 – 1.5 ft weir - 8x3	7370.15 cfs	126.88 ft	487.64 cfs	132.20 ft	7540.35 cfs	136.58 ft
Plan 10b-40 – 1.0 ft weir - 8x3	7607.42 cfs	126.96 ft	354.28 cfs	131.71 ft	7749.33 cfs	136.34 ft
Plan 10b-40 – 0.5 ft weir - 8x3	7575.88 cfs	127.00 ft	264.30 cfs	131.45 ft	7525.64 cfs	135.86 ft
Plan 10b-40 – 1.5 ft weir - 12x2	7353.71 cfs	126.87 ft	504.55 cfs	132.24 ft	7540.77 cfs	136.58 ft
Plan 10b-40 – 1.0 ft weir - 12x2	7485.78 cfs	126.95 ft	346.65 cfs	131.74 ft	7512.56 cfs	136.21 ft
Plan 10b-40 – 0.5 ft weir - 12x2	7550.42 cfs	126.99 ft	289.67 cfs	131.56 ft	7525.74 cfs	135.85 ft

5.0 RESULTS AND CONCLUSIONS

The Peckman River Project has been thoroughly analyzed using a one-dimensional steady state model to determine the TSP, and then further analyzed and optimized with an unsteady state HEC-RAS model, with a two-dimensional overbank area to analyze the overland flow from the Peckman River to the area in and around Hopson Avenue.

After careful analyses were performed by the PDT, a positive benefit to cost ratio was realized and the TSP was selected to be alternative 10b-40. The project will provide approximately a 2% AEP (50-year storm event) level of protection. **Figure 34** shows the water surface profiles of the NED plan while **Figure 35** shows an inundation map of the NED Plan with and without project for 0.02 AEP and **Figure 36** shows shows an inundation map of the NED Plan with and without project for 0.01 AEP.

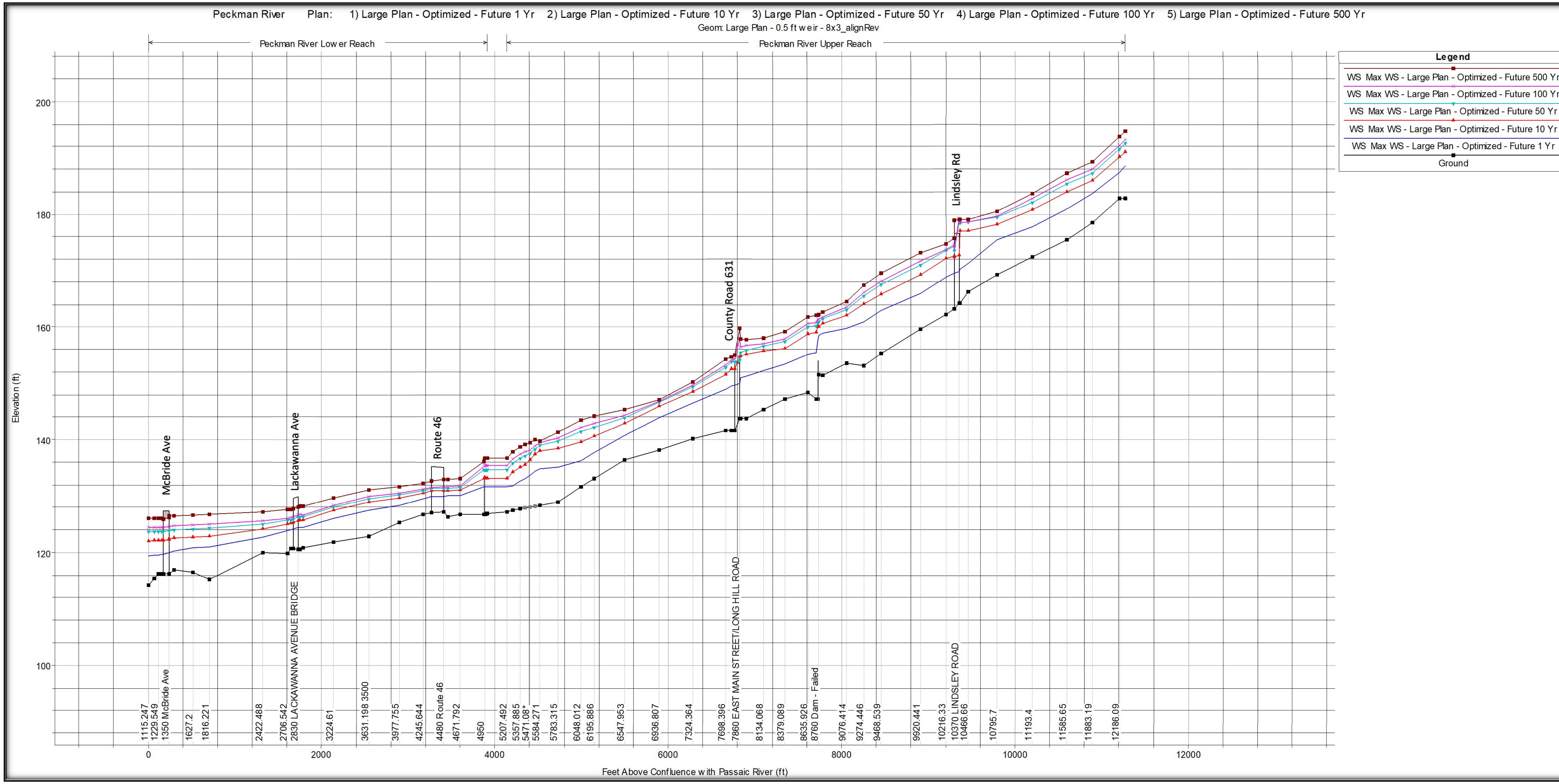

Figure 34: Alternative #10b – NED Plan with Diversion Culvert, Levee, Flood walls, and Weirs (Future Conditions)

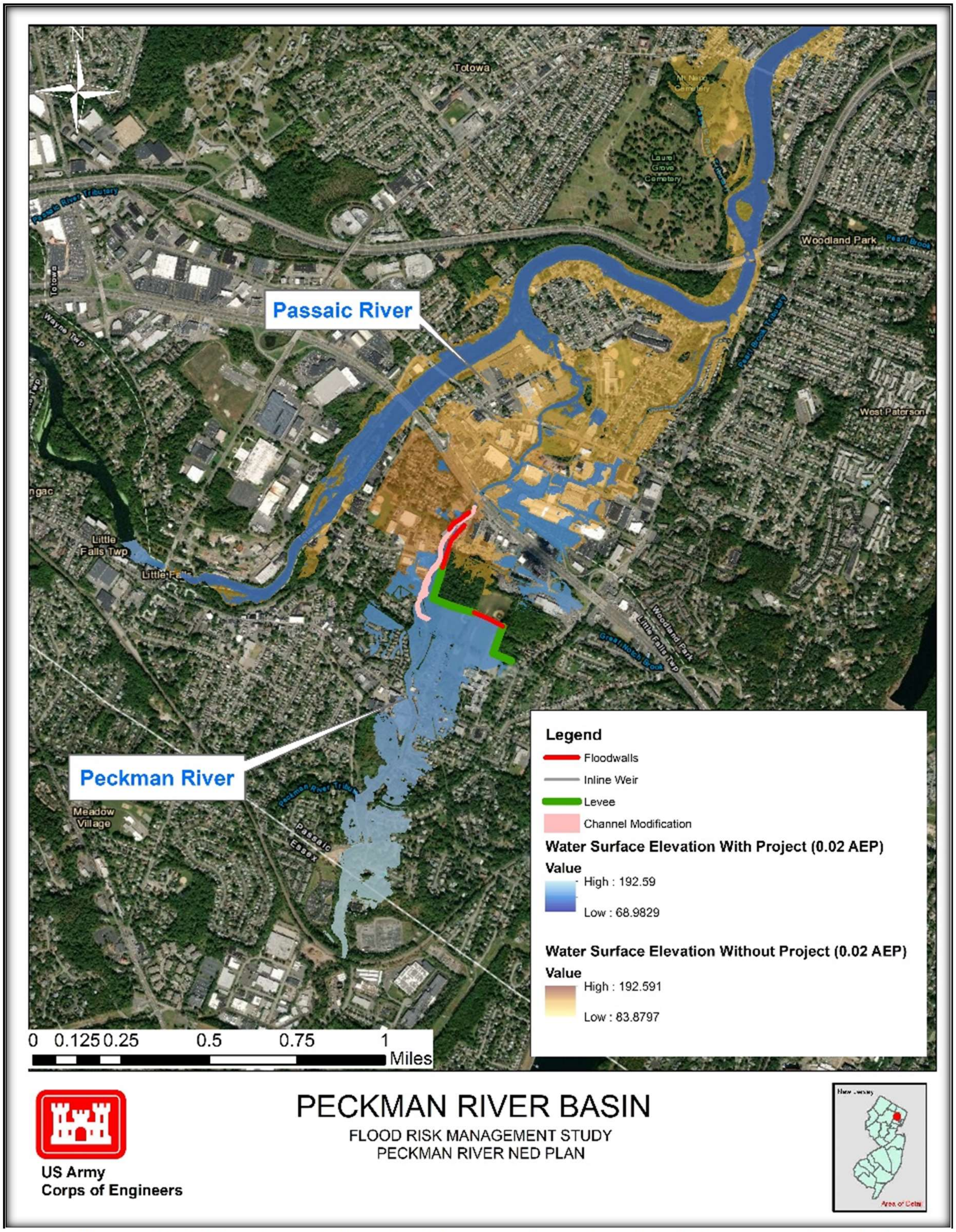

Figure 35: NED Plan with and without project for 0.02 AEP (50-yr) for Peckman Peaking.

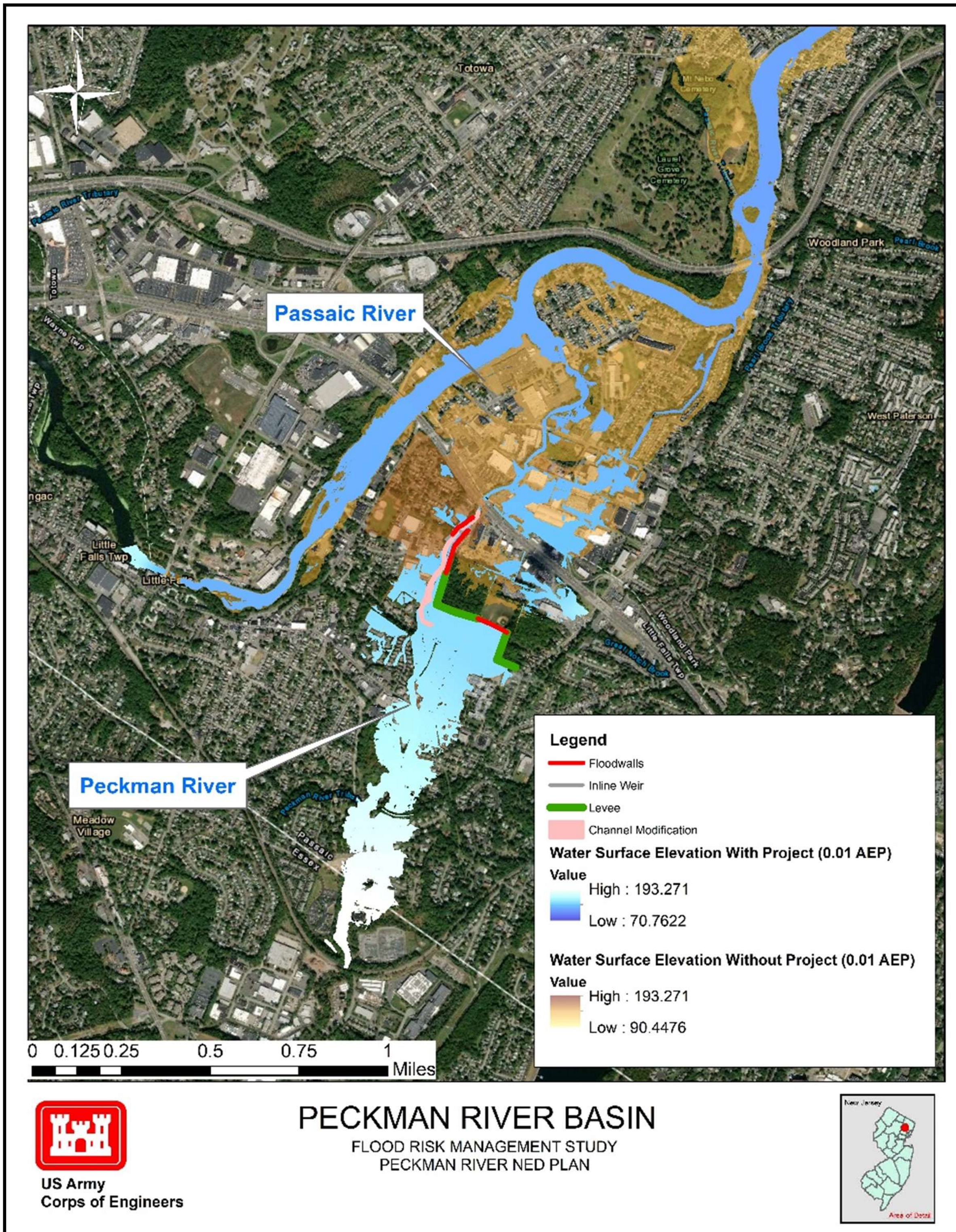

Figure 36: NED Plan with and without project for 0.01 AEP (100-yr) for Peckman Peaking.