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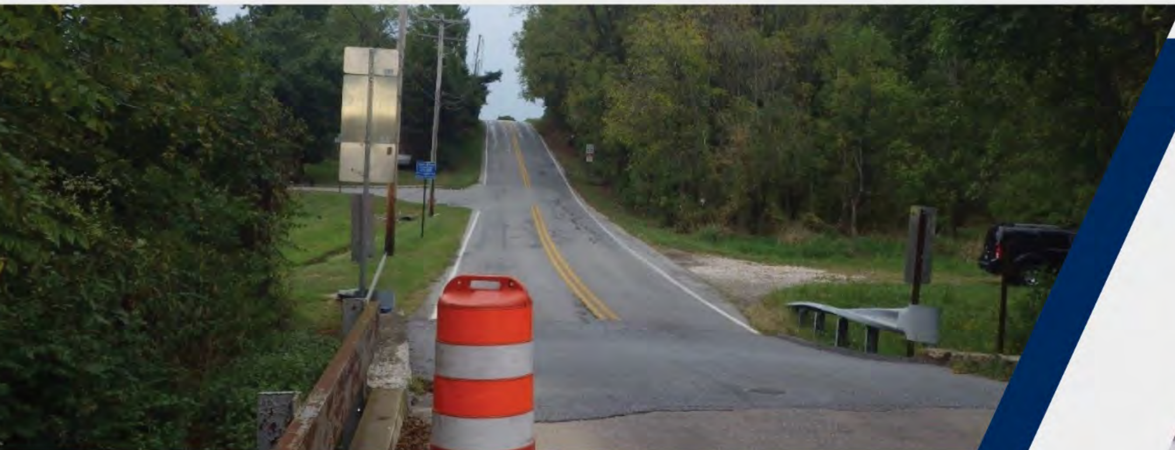

Development of Site-Specific Hydrologic and Hydraulic Analyses for Assessing Transportation Infrastructure Vulnerability & Risks to Climate Change

SEPTEMBER 2020

Notice

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16. Abstract

This pilot study supports PennDOT's continued efforts to develop processes and procedures to assess potential flooding vulnerabilities and risks related to transportation infrastructure. The study builds upon PennDOT's Extreme Weather Vulnerability Study and existing hydrologic and hydraulic (H&H) assessments that are typically conducted as part of the design process and used to obtain Pennsylvania Department of Environmental Protection (DEP) and U.S. Army Corps of Engineers environmental permits. The existing H&H assessments follow the procedures and requirements in PennDOT's Design Manuals. The pilot study includes three site-specific H&H assessments that have been expanded (as compared to current PennDOT procedures) to address the impacts of more intense and frequent precipitation events under future climate scenarios. The analyses and procedures conducted for the pilot study highlight the extraction and interpretation of global climate model precipitation data, the application of that data to the H&H process, and methods to evaluate adaptation strategies to improve infrastructure resiliency.

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Project Study Overview and Purpose

This pilot study supports the Pennsylvania Department of Transportation's (PennDOT's) continued efforts to develop processes and procedures to assess potential flooding vulnerabilities and risks related to transportation infrastructure. The study builds upon PennDOT's 2017 Extreme Weather Vulnerability Study and existing hydrologic and hydraulic (H&H) assessments that are typically conducted to obtain Pennsylvania Department of Environmental Protection (DEP) and U.S. Army Corps of Engineers environmental permits. The existing H&H assessments follow the procedures and requirements in PennDOT's Design Manuals.

The pilot study includes three site-specific H&H assessments that have been expanded (as compared to current PennDOT procedures) to address the impacts of more intense and frequent precipitation events under future climate scenarios. The analyses and procedures conducted for the pilot study serve several study goals as shown in **Figure 1**.

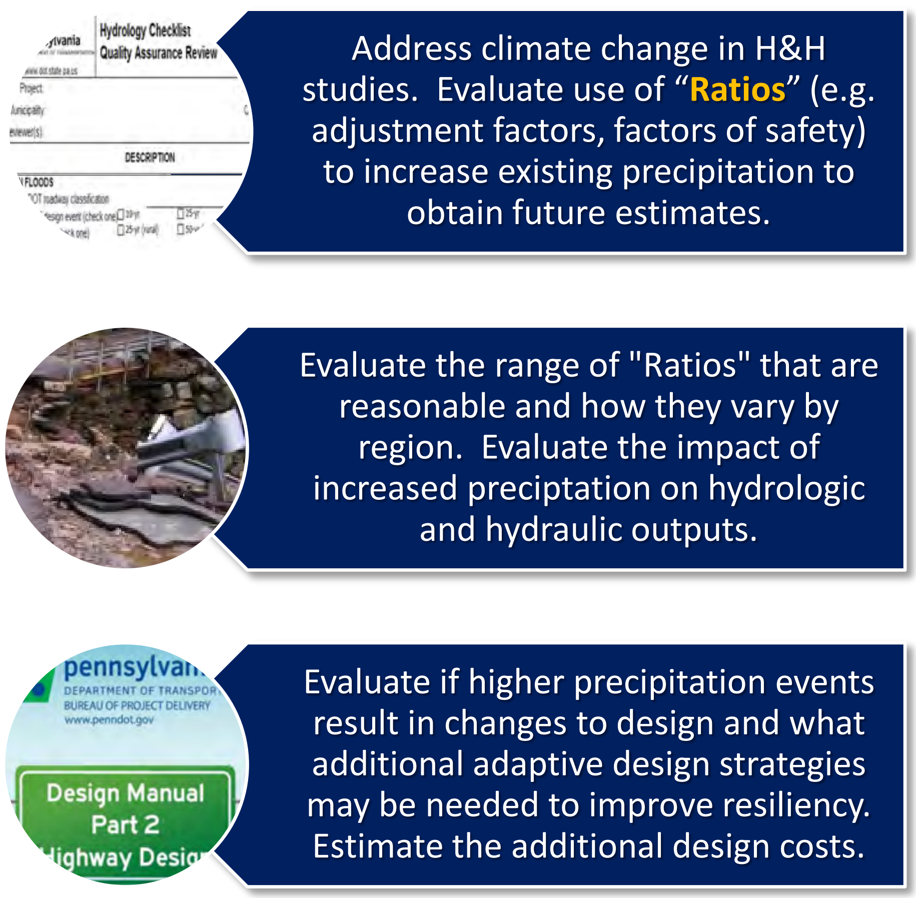


Figure 1: Study Goals

This pilot study includes site-specific hydrologic and hydraulic analyses at three locations within the state to evaluate the resiliency of the transportation system to future climate and land use change. The major steps included in this study analysis are shown in **Figure 2**.

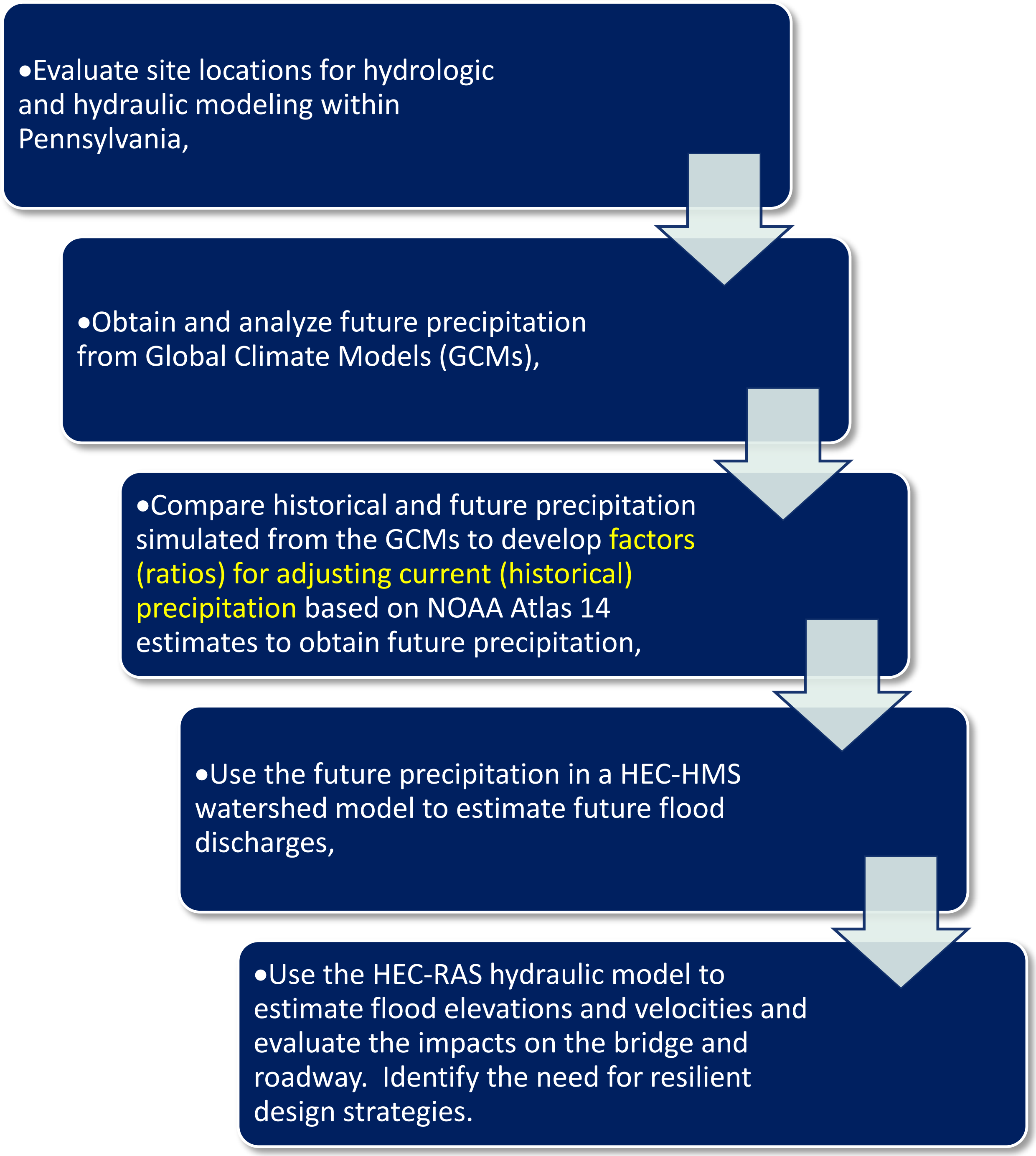


Figure 2: Key Study Analysis Steps

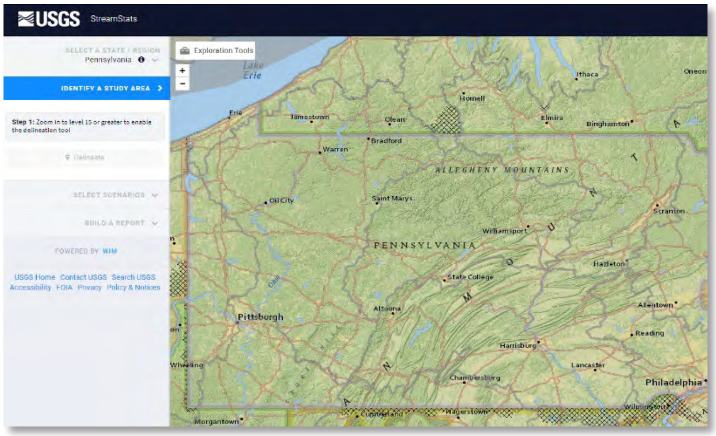
Site Selection Process

Analysis sites were selected in three separate counties within Pennsylvania (York, Allegheny, and Delaware counties), providing insights into the geographic differences of projected precipitation changes from global climate models. Each site consisted of a bridge and roadway structure.

Identifying Candidate Sites

The selection of potential sites was supported by regional Metropolitan Planning Organizations (MPOs) and PennDOT District Offices. The MPOs that provided support included the Southwestern Pennsylvania Commission (SPC), the Delaware Valley Regional Planning Commission (DVRPC) and the York County Planning Commission (YCPC). The sites selected include bridge and roadway locations that are generally seen as high-risk for flooding as determined from PennDOT's Extreme Weather Vulnerability Study and local knowledge. Data was collected on all candidate sites to assist with site prioritization and selection. The collected data included:

Stream Stats:
https://water.usgs.gov/osw/streamstats/



- A StreamStats basin delineation and report was created for each location using the USGS StreamStats application. This resource provided drainage area, basin characteristics (including land use characteristics), and flow statistics for each identified location.

- Available flood data for each location was obtained from the FEMA Flood Map Service Center. Flood Insurance Rate Map (FIRM) Panels and Flood Insurance Study (FIS) Reports were obtained to accompany the StreamStats data.

- If the FEMA information obtained for a location indicated there was a past detailed study at the bridge, a FEMA Data Request was made to obtain the hydraulic model used to produce the profiles and Water Surface Elevations (WSELs) in the FIS Report.

Methods for Prioritizing Sites for Study

The data collected was then reviewed to select one study site location in each county. The following factors were considered when reviewing each site, with the most desirable characteristics listed first:

1) **Availability of a FEMA HEC-RAS model**. Locations that are in a FEMA detailed study area received highest priority, especially if there was a usable HEC-RAS model received from the FEMA data request. Having an existing model with which to work greatly reduces the required effort when modelling the bridge.

2) **Bridge Overtopping**. Bridges with existing flooding issues were considered good candidates for analysis because they would best illustrate the negative effects of increased precipitation and allow for more opportunities for flood mitigation.

3) **Drainage Area.** A moderately sized drainage area (between 5 and 100 sq. mi) is desirable for each selected site. A small drainage area would likely not flow to a bridge opening large enough to accurately study the effects of precipitation fluctuations, while large drainage areas would introduce too many factors (such as a complex structure, or multiple drainage sources) to allow for simple comparisons. In addition, hydrologic modeling of large watersheds is more time consuming and complicated by variable land use conditions and variability of precipitation over the watershed.

4) **Proximity to other surface waters**. Bridges that were close to any confluences downstream were avoided because backwater from other streams may affect flood waters at the project bridge. This is especially true if the downstream confluence is with a larger stream. Needing to account for backwater from a confluence would make hydraulic modeling unnecessarily complex. Additionally, if there are other streams parallel to the project stream, it may be possible that both watersheds would combine in greater flood events and would make analysis more complicated.

5) **Proximity to other upstream/downstream structures**. Similar to above (but less of a factor), possible backwater from other structures could affect flood waters at the project bridge.

6) **Straight stream alignment.** A simple stream alignment is desirable to simplify hydraulic modeling and make varying factors within the model (like ineffective areas) simpler and better for comparison purposes.

Site Selection

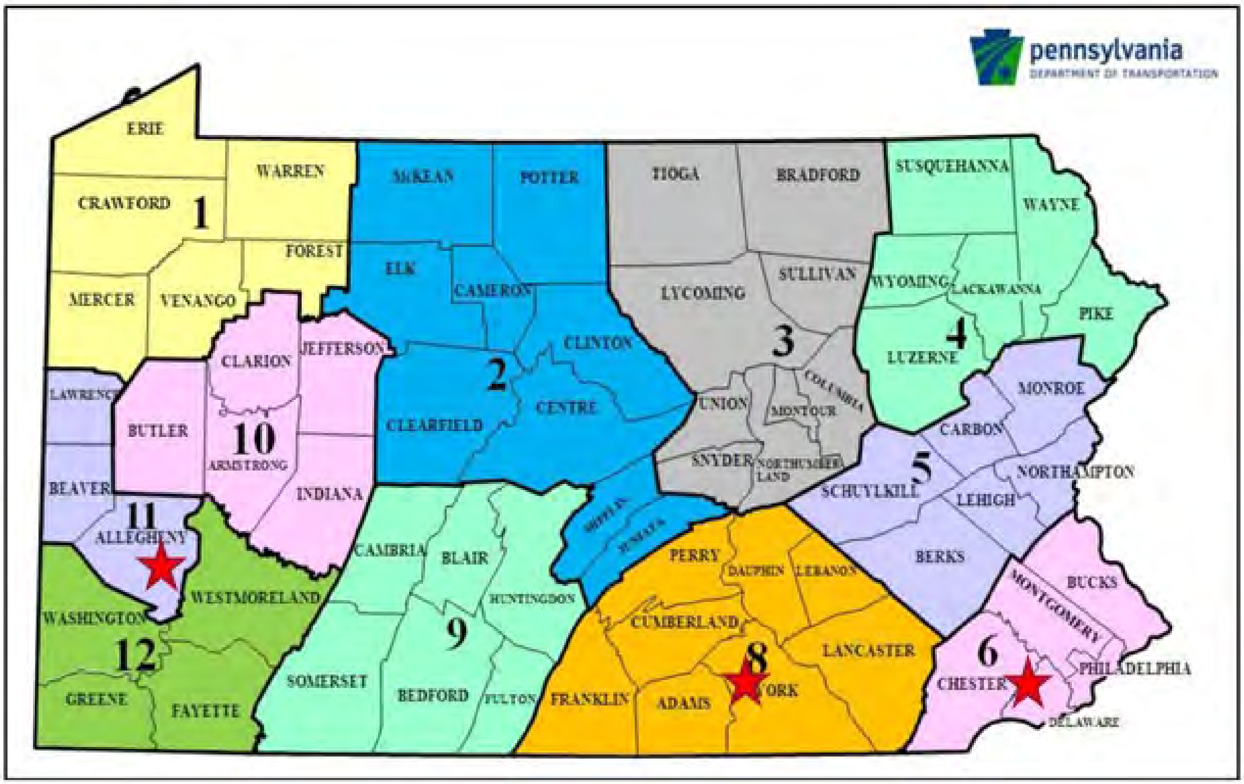


Figure 3: Locations Selected for Site-Specific H&H Studies

York County: Baker Road over Little Conewago Creek

The York County site has a 20.8 square mile watershed with an available HEC-RAS model. The site has historically flooded resulting in road closures. A design study is underway at this site location. The pilot study can be used to inform the bridge design and evaluate the potential need of additional protection measures to increase resiliency of the site.

Baker Road in York County. Credit: PennDOT

Allegheny County: Streets Run Road / Baldwin Road along Streets Run

The Allegheny County site has an 8.1 square mile watershed, which is a reasonable size for performing hydrologic calculations. The site has known flooding issues and is located within the study area of the Streets Run Flood Protection Alternatives Analysis currently being conducted by the Pennsylvania Department of General Services (DGS). As such, there is current stream survey and an existing conditions HEC-RAS model. As the alternatives study for the flood protection project is on-going, results of the pilot study could be beneficial to DGS to inform their evaluation of potential alternatives to reduce flooding.

Streets Run Road in Allegheny County
Credit: PennDOT

Delaware County: Station Road Bridge over East Branch of Chester Creek

The Delaware County site has a 22.9 square mile watershed with a stream confluence immediately upstream of the bridge opening. The bridge has overtopped historically resulting in roadway closures. The roadway is locally owned but is under evaluation for future improvements. An existing HEC-RAS model was available for study application.

Station Road Bridge in Delaware County
Credit: PennDOT

Assessing Future Precipitation Changes

Global Climate Models (GCMs) simulate the response of global climate to interactions between land-surface processes, ocean and atmospheric circulation, biogeochemical processes, and human influences such as the release of greenhouse gasses into the atmosphere, land-use changes, and more. The Intergovernmental Panel on Climate Change (IPCC), through the Coupled Model Intercomparison Project (CMIP), periodically collects and archives historical and future GCM simulations from the latest generation of models that are developed and maintained by climate modeling groups around the world. The previous 2000 archive was CMIP3; the most recent (2013) archive is CMIP5; and simulations for the CMIP6 archive are in progress and the archive is expected to be completed by 2020 or shortly thereafter.

There are several sources of future precipitation data from GCMs and guidance was obtained from Kilgore and others (2019a; 2019b). For any study, choices are required regarding the use of:

- greenhouse gas emissions,
- selection of GCMs,
- period of historical and future simulated data, and
- the appropriate climate data set.

Greenhouse Gas Emissions and Representative Pathway Concentrations

The climatic outputs (temperature, precipitation, etc.) from GCMs are a function of greenhouse gas emissions and, for the CMIP5 project in 2013, the IPCC adopted Representative Concentration Pathways (RCPs) to categorize a range of future greenhouse gas emissions. The increase or decrease in future emissions of greenhouse gases is related to what actions are taken by society to mitigate these gas emissions in the future (Kilgore and others, 2016). Greenhouse gas emissions accumulate in the atmosphere and increase global temperatures that in turn increase moisture in the atmosphere and result in increased precipitation. Greenhouse gas emissions include different gases like methane, nitrous oxide and fluorinated gases but about 80 percent of the emissions is carbon dioxide (CO_2). The relation between CO_2 equivalent concentrations (the primary greenhouse gas) and the various RCPs is illustrated in **Figure 4**.

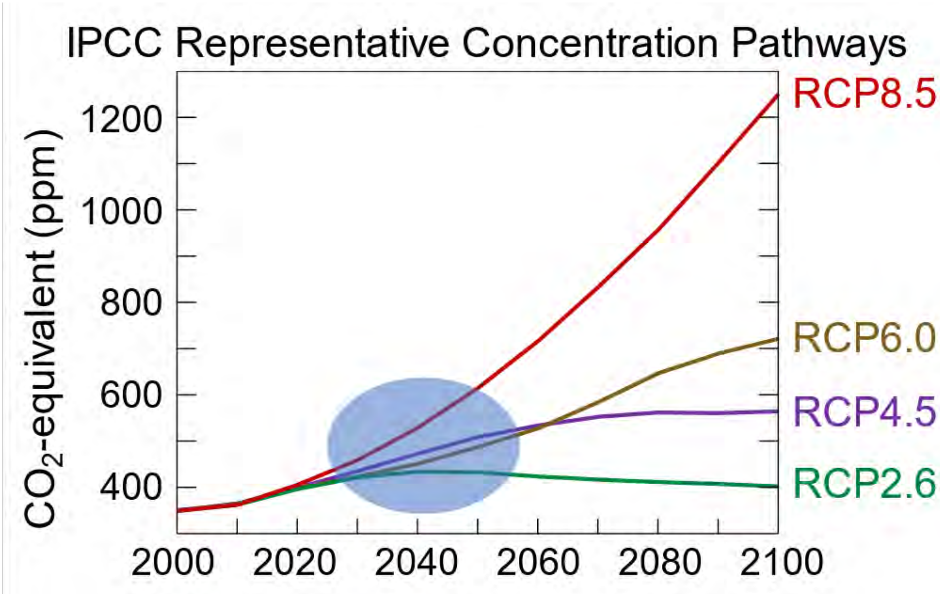


Figure 4: Relationship Between CO2 equivalents and RCP Scenarios
IPCC AR5 Greenhouse Gas Concentration Pathways
(Figure by Efbrazil - Own work, CC BY-SA 4.0,
https://commons.wikimedia.org/w/index.php?curid=87801257 modified by PennDOT)

The RCPs are indicative of increases in global temperature and range from low (RCP 2.6) to high (RCP 8.5) CO_2 atmospheric concentrations where:

- RCP 2.6 is indicative of stringent gas emission mitigation practices with peak CO_2 concentrations in about 2040 with CO_2 concentrations declining to about 430 ppm by 2100 that is similar to current (2020) concentrations,
- RCP 4.5 is indicative of a moderate increase in CO_2 until about 2080 when the CO_2 concentrations become rather stable at about 570 ppm,
- RCP 6.0 is indicative of a moderate increase in CO_2 concentrations to about 730 ppm by 2100, and
- RCP 8.5 is indicative of continued use of fossil fuels with a large increase in CO_2 concentrations to about 1,240 ppm by 2100.

The blue area in **Figure 4** is shown to emphasize that the scenarios do not differ significantly over the next 30 years or so. Therefore, scenario selection is less critical for design lifetimes less than about 30 years.

RCP 4.5 and RCP 8.5 were used in the PennDOT study to provide a range of future precipitation and because these scenarios are available in many GCMs.

Global Climate Models (GCMs)

GCMs are regularly evaluated by climate scientists on their ability to reproduce the large-scale physical characteristics and patterns of natural and human-forced variability in the climate system. GCMs have been grouped according to the reliability and performance (Infrastructure and Climate Network (http://theicenet.org)). Group 1 GCMs are defined as multi-generational versions (typically 3rd to 5th) of long-established global climate models from modeling groups with decades of experience, whose performance is well-documented in the literature (Kilgore and others, 2019a). The Group 1 GCMs are further grouped by high, medium, and low climate sensitivity as defined by the response of global temperature to increasing levels of carbon dioxide in the atmosphere. Using the above guidance, the eight Group 1 GCMs shown in **Figure 5** were used in the PennDOT study. These models provide a range of climate sensitivity.

BCC-CSM 1.1-m: developed by the Beijing Climate Center, China
CCSM4: developed by the National Center for Atmospheric Research, United States
CSIRO-Mk.6.0: developed by the Commonwealth Scientific and Industrial Research Organization in collaboration with the Queensland Climate Change Centre of Excellence, Australia
GFDL-CM3: developed by the Geophysical Fluid Dynamics Laboratory, United States
GISS-E2-R: developed by the National Aeronautical Space Administration, United States
HadGEM2-AO: developed by the Met Office Hadley Centre, United Kingdom
MIROC5: developed by the Atmosphere and Ocean Research Institute (The University of Tokyo), National Institute for Environmental Studies, and Japan Agency for Marine-Earth Science and Technology, Japan.
MRI-CGCM3: developed by the Meteorological Research Institute, Japan.

Figure 5: GCMs Used in PennDOT Study

Global Climate Data Sets

GCMs provide temperature and precipitation data at very large grid sizes that are not useful for hydrological modeling. There are two approaches for downscaling the GCM output to grid sizes that are more useful for engineering and hydrologic analyses: empirical-statistical downscaled model (ESDM) climate projections and dynamically downscaled climate projections from regional climate models (RCM). Kilgore and others (2019a) compared four climate data sets that were suitable for hydrologic design and analysis of transportation infrastructure. The data sets are compared in **Table 1** and a brief description of each follows:

- Asynchronous Regional Regression Model Version 1 (ARRM v1) – statistical downscaling (ESDM)
- Localized Constructed Analogs (LOCA) – statistical downscaling (ESDM)
- North American Regional Climate Change Assessment Program (NARCCAP) – dynamic downscaling (RCM)
- North American Coordinated Regional Climate Downscaling Experiment (NA-CORDEX) – dynamic downscaling (RCM)

Table 1: Comparison of Climate Data Sets
(Kilgore and others, 2019a)

Characteristics	ESDM Datasets		RCM Datasets	
	ARRM	LOCA	NARCCAP	NA-CORDEX
CMIP Generation	CMIP3	CMIP5	CMIP3	CMIP5
Future Scenarios	A1FI, A2, A1B, B1	RCP4.5, RCP8.5	A2	RCP4.5, RCP8.5
Time Period of Output	1960-2099	1950-2099	1968-2000, 2038-2070	1950-2100
Time Frequency	Daily	Daily	3-hourly	Daily
Spatial Resolution	1/8th degree (~12 km)	1/16th degree (~6 km)	50 km	25-50 km
Obs. Training Dataset	Maurer[1]	Livneh[2]	*not applicable*	*not applicable*
Number of GCMs	16	30	4	6
Number of Group 1 GCMs	13	14	3	0
Number of RCMs	*not applicable*	*not applicable*	8	6

[1] USBR. (2013).

[2] Livneh and others (2013).

Kilgore and others (2019a) recommended the LOCA data set (http://loca.ucsd.edu) for studies requiring quantitative precipitation estimates with the provision that this choice may not be appropriate for all applications. The LOCA data set has several desirable features such as data for two CMIP5 emission scenarios (RCP 4.5 and RCP 8.5), historical and future GCM simulations for the period 1950-2099, data at ~6 km grid sizes and future projections for 14 Group 1 GCMs. The GCMs used in the LOCA data were calibrated to historical data for 1950 to 2005 as described by Livneh and others (2013). Data for the historical period of 1950 to 2005 are simulated by each GCM so this period can be compared to simulated data for future periods.

All the climate data sets in **Table 1** are for daily precipitation except NARCCAP that has data at 3-hour intervals. However, the NARCCAP data are limited to one CMIP3 emission scenario (A2), are available at 50 km grids sizes, the future period ends in 2070 and the data are only available for three Group 1 GCMs. Sub-daily (n-hour or n-minute) precipitation data are not available for CMIP5 emission scenarios for a large sample of Group 1 GCMs, hence the need to use daily precipitation from the GCMs.

Based on the above data assessment, the LOCA data set based on ESDM (http://loca.ucsd.edu) was used for the PennDOT study.

Future Precipitation Analyses

The data and analyses used for estimating future precipitation for the PennDOT study followed guidance in Kilgore and others (2019a; 2019b). The watersheds selected for study ranged in size from 8.19 to 22.9 square miles so four to six GCM grids (~ 6 km x 6 km) cover the watershed area. The data used for analysis included:

- The LOCA data set (http://loca.ucsd.edu) that is available from the Lawrence Livermore National Laboratory Green Data Oasis. This website is easily accessible with good guidance on obtaining precipitation data.
- Eight GCMs listed previously that have simulated data for the historical period 1950 to 2005 and future simulated data from 2006 to 2099. Two future periods of 2006 to 2050 and 2051 to 2099 were defined so that future precipitation estimates could be made for 2050 and 2100. Emission scenarios RCP 4.5 and RCP 8.5 were used to provide a broader range of future projections.

The data processing steps included those summarized in **Figure 6**.

1	Assemble raw daily precipitation, in millimeters, for each grid covering each watershed for the period 1950 to 2099,
2	Calculate the weighted average daily precipitation by weighting each grid by the percentage area within the watershed (four to six grids depending on the watershed),
3	Estimate the maximum daily precipitation for each year through 2099,
4	Transform the annual maximum daily precipitation to logarithms to achieve more normally distributed data (skews closer to zero),
5	Group annual maximum data by the following time periods: 1950 to 2005 (historical), 2006 to 2050, and 2051 to 2099 to obtain future estimates in 2050 and 2100,
6	Fit a Pearson Type III distribution to the logarithms of the annual maximum daily precipitation for each time period to get estimates for selected annual exceedance probabilities (AEPs) ranging from 0.5 (2-year event) to 0.002 (500-year event). The Pearson Type III distribution was used because it is a flexible 3-parameter distribution that can be applied in a spreadsheet. Most 3-parameter distributions would give similar estimates.
7	Estimate the ratio of future to historical period precipitation for each AEP and GCM for 2050 and 2100. Ratio for "t" AEP = future precipitation for "t" AEP / historical precipitation for "t" AEP, where "t" reflects the event exceedance probability.
8	Graph and summarize the results.

Figure 6: Precipitation Data Processing Steps

The objective of the analyses is to obtain a ratio of future to historical daily precipitation for each GCM and AEP that can be used to adjust the existing conditions precipitation based on NOAA Atlas 14, Volume 2 to future conditions (Bonnin and others, 2006; PennDOT, 2010). Note the ratio is based on GCM simulated data for the future to GCM simulated data for the historical period 1950 to 2005 in order to get an estimate of the percentage increase based only on model simulated data. The future GCM simulated data are not compared to existing historical data because of model biases in replicating historical data.

Ratios were estimated for both 2050 and 2100 time periods and for RCP 4.5 and RCP 8.5. The results for RCP 8.5 and the 2100 time period were considered more reasonable and relevant and are discussed below. Results for 2050 and RCP 4.5 are discussed later.

The ratios for the future (2051-2099) to historical (1950-2005) precipitation for all three study sites for RCP 8.5 and 2100 are summarized in **Table 2** for annual exceedance probabilities (AEPs) ranging from 0.5 to 0.002. As shown in **Table 2,** there is significant variation in the ratios across the three sites with the Baker Road (York County) site differing the most from the other two sites. Some observations based on the results include:

- The ratios are quite large for the Baker Road (York County) site for some GCMs for the 0.04 to 0.002 AEP events with the ratios increasing with severity of the event.
- The ratios for the Baldwin Road (Allegheny County) site are nearly constant across the AEPs and generally less than the York County ratios with the average ratios decreasing slightly with severity of the event.
- The ratios for the Station Road (Delaware County) site are also nearly constant across AEPs and generally less than the York County ratios with the average ratios increasing slightly with severity of the event.
- There is more variability in the ratios for the extreme events with 0.01 and 0.002 AEP with ratios varying from 4.55 to less than 1.0 implying decreased future precipitation.
- For the 0.10 AEP event and less, the GCM ratios are greater than 1.0 for all GCMs implying increased future precipitation for the more frequent events.

The large variation in ratios across GCMs, locations and AEPs is attributed to uncertainty in the global modeling process and may or may not be related to local meteorological conditions.

Table 2: Future to Historical Precipitation Ratios at Study Sites
RCP 8.5 2100 Scenario for 0.5 (2-year event) to 0.002 (500-year event)

Site Location	GCMs	Exceedance Probabilities							
Baker Road, York		**0.5**	**0.2**	**0.1**	**0.04**	**0.02**	**0.01**	**0.005**	**0.002**
	BCC-CSM 1.1-m	1.28	1.45	1.60	1.82	2.00	2.21	2.43	2.76
	CCSM4	1.09	1.27	1.50	1.90	2.30	2.81	3.45	4.55
	CSIRO-k3.6.0	1.07	1.22	1.39	1.68	1.95	2.28	2.67	3.31
	GFDL-CM3	1.19	1.11	1.03	0.94	0.87	0.80	0.74	0.67
	GISS-E2-R	1.16	1.13	1.13	1.14	1.16	1.18	1.20	1.24
	HadGEM2-AO	1.03	1.02	1.01	1.01	1.01	1.01	1.02	1.02
	MIROC5	1.22	1.23	1.21	1.18	1.16	1.13	1.10	1.05
	MRI-CGCM3	1.04	1.07	1.12	1.22	1.30	1.39	1.50	1.65
	Average	1.13	1.19	1.25	1.36	1.47	1.60	1.76	2.03
Street Road, Allegheny		**0.5**	**0.2**	**0.1**	**0.04**	**0.02**	**0.01**	**0.005**	**0.002**
	BCC-CSM 1.1-m	1.31	1.37	1.37	1.36	1.35	1.32	1.30	1.26
	CCSM4	1.19	1.19	1.18	1.17	1.17	1.17	1.16	1.16
	CSIRO-Mk3.6.0	1.23	1.26	1.27	1.29	1.30	1.31	1.32	1.33
	GFDL-CM3	1.25	1.21	1.18	1.13	1.09	1.05	1.01	0.97
	GISS-E2-R	1.21	1.24	1.25	1.26	1.26	1.26	1.26	1.26
	HadGEM2-AO	1.06	1.09	1.11	1.14	1.16	1.19	1.22	1.25
	MIROC5	1.43	1.49	1.51	1.51	1.50	1.49	1.47	1.44
	MRI-CGCM3	1.18	1.13	1.08	1.01	0.96	0.92	0.87	0.81
	Average	1.23	1.25	1.24	1.23	1.22	1.21	1.20	1.18
Station Road, Delaware		**0.5**	**0.2**	**0.1**	**0.04**	**0.02**	**0.01**	**0.005**	**0.002**
	BCC-CSM 1.1-m	1.18	1.10	1.04	0.95	0.90	0.84	0.79	0.72
	CCSM4	1.04	1.05	1.10	1.20	1.30	1.42	1.55	1.76
	CSIRO-Mk3.6.0	1.24	1.30	1.33	1.36	1.37	1.39	1.40	1.41
	GFDL-CM3	1.33	1.23	1.16	1.08	1.03	0.97	0.92	0.86
	GISS-E2-R	1.16	1.13	1.11	1.08	1.06	1.04	1.03	1.00
	HadGEM2-AO	1.20	1.27	1.34	1.42	1.49	1.56	1.63	1.73
	MIROC5	1.16	1.18	1.18	1.17	1.17	1.16	1.15	1.14
	MRI-CGCM3	1.08	1.06	1.05	1.04	1.03	1.03	1.03	1.03
	Average	1.17	1.16	1.16	1.16	1.17	1.18	1.19	1.21

Analysis of the Precipitation Ratios for Individual GCMs

The precipitation ratios from **Table 2** for the York County site are plotted in **Figure 7**. The graph illustrates the variation in the precipitation ratios across the eight GCMs where ratios greater than 1 indicate increases in future precipitation over historical precipitation. The first three GCMs (BCC-CSM, CCSM4, CSIRO) show very large ratios for future precipitation particularly for the 0.01 to 0.002 AEP events (ratios greater than 2.2). The last five GCMs show minimal increases in future precipitation with the GFDL model showing a decrease in future precipitation for the extreme events. The ratios for the 0.5 to 0.10 AEP events are either increasing slightly or staying the same. The variability of the data illustrates the uncertainty in the global modeling process.

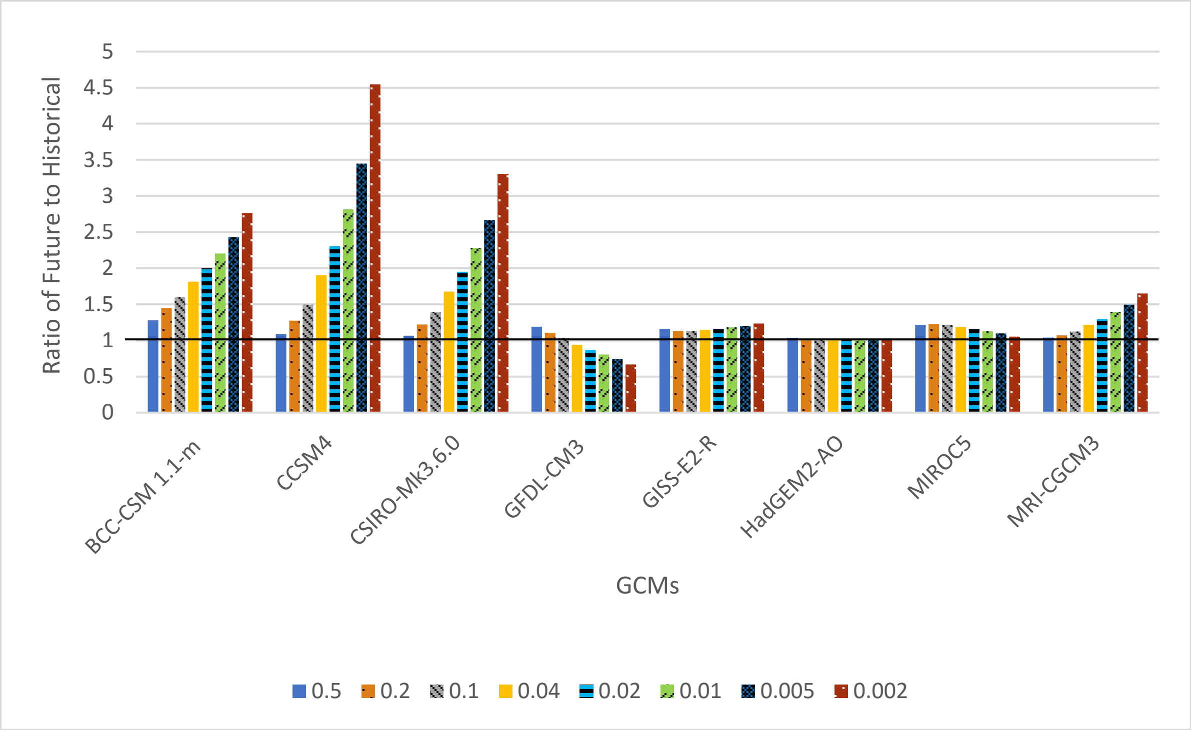


Figure 7: York County Site GCM Precipitation Ratios (Future to Historical)
RCP 8.5 for 2100

The precipitation ratios from **Table 2** for the Allegheny County site are plotted in **Figure 8**. The graph illustrates much less variation in the precipitation ratios across the eight GCMs than for York County. The same vertical scale is used in Figure 8 as in Figure 7 to illustrate the difference in GCM variability as compared to the York County site. In general, there is not much variation in the ratios across AEPs. Two models (GFDL and MRI-CGCM3) show a decrease in the ratios as the severity of the event increases. The ratios for the 0.5 to 0.10 AEP events are greater than 1 and indicate an increase in future precipitation across all GCMs for the more frequent events.

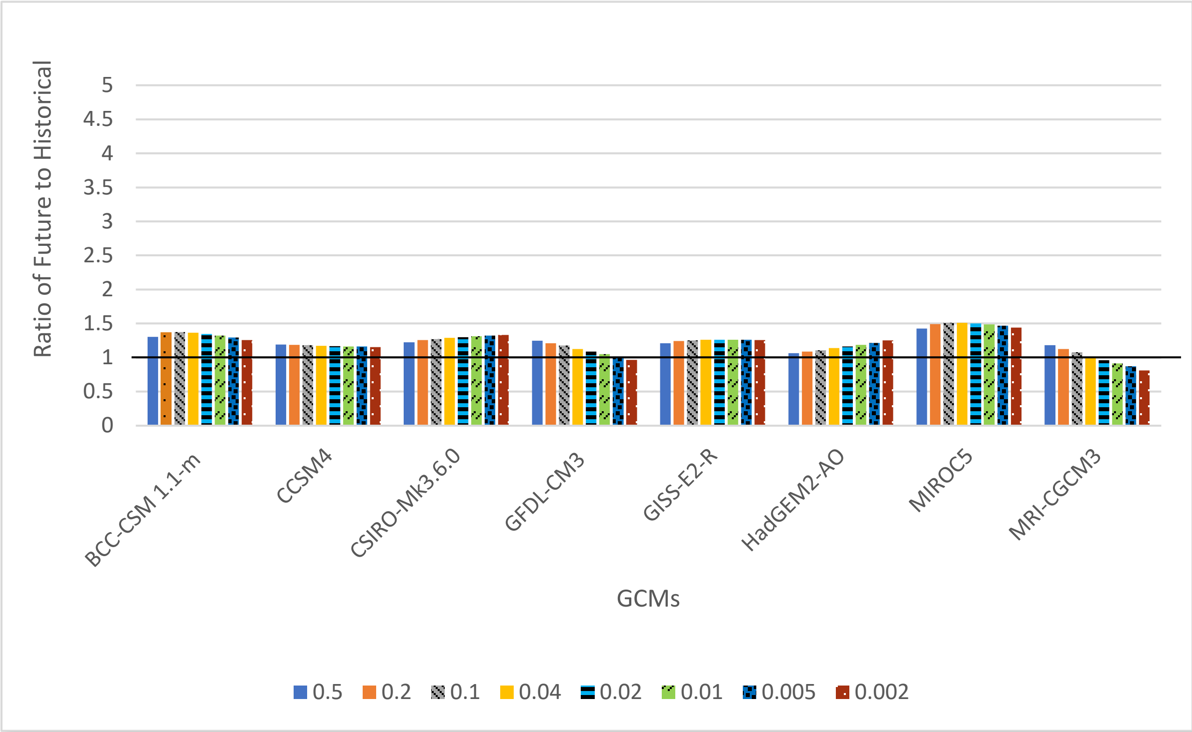


Figure 8: Allegheny County Site GCM Precipitation Ratios (Future to Historical)
RCP 8.5 for 2100

The precipitation ratios from **Table 2** for the Delaware County site are plotted in **Figure 9** using the same vertical scale as Figures 7 and 8. The graph illustrates significant variation in precipitation ratios across GCMs with roughly half the GCMs showing increased ratios and half showing decreased ratios for the extreme events (0.01 to 0.002 AEP). Two GCMs (BCC-CSM and GFDL) show decreases in the future precipitation for the extreme events. The ratios for the 0.5 to 0.10 AEP events are greater than 1 and indicate an increase in future precipitation across all GCMs. Note the BCC-CSM model is showing a decrease in future precipitation for the extreme events for the Delaware County site while the same model is showing large increases in future precipitation for the extreme events for the York County site. The HadGEM2 model is not showing an increase in future precipitation across the AEPs for York County but the same model is showing large increases in extreme precipitation for the Delaware County site. Delaware and York County are both in southeastern Pennsylvania in similar meteorological regions. These comparisons further illustrate the uncertainty in the global modeling process.

The variation across the GCMs is much less for the Allegheny County site than the other two sites. Allegheny County is in southwestern Pennsylvania where precipitation is lower than in York and Delaware Counties. The consistency in GCMs in Allegheny County may be related to the lower precipitation occurring across this region. This hypothesis can be evaluated by performing more regional analyses in western Pennsylvania.

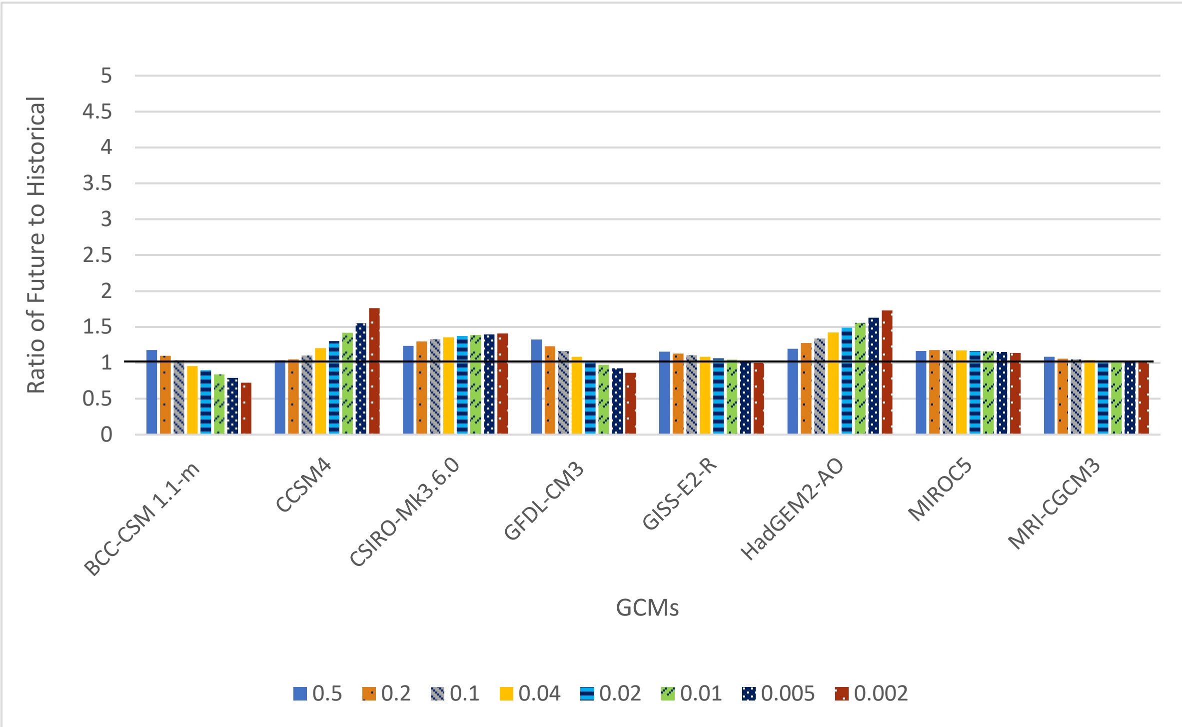


Figure 9: Delaware County Site GCM Precipitation Ratios (Future to Historical)
RCP 8.5 for 2100

Analysis of Average Precipitation Ratios Across All GCMs

The average ratios across all eight GCMs are shown in **Figure 10** for the York County site for 2050 and 2100 and for the RCP 4.5 and RCP 8.5 scenarios. Some pertinent observations are:

- For RCP 8.5 and 2100, the average ratio for the 0.01 AEP event is 1.60 and 2.03 for the 0.002 AEP. Based on engineering judgement, it is unrealistic that the future precipitation will increase, on average, 60 and 100 percent, respectively, by 2100. These estimates are inconsistent with results at the Allegheny and Delaware sites and the Fourth National Climate Assessment (Easterling and others, 2017) as discussed later.
- For RCP 8.5 and 2100, the average ratio for the 0.10 AEP event is 1.25 and more reasonable than the ratios for the extreme events like 0.01 and 0.002 AEP.
- For 2050, the ratios are higher for all AEP extreme events for RCP 4.5 than for RCP 8.5.
- For RCP 4.5, the 2050 ratios are larger than 2100 for all AEP events from 0.20 to 0.002.

It is unrealistic that precipitation, on average across all GCMs, is higher for 2050 than 2100 and higher for RCP 4.5 than RCP 8.5. This variation is attributed to the fact that the emission scenarios do not differ much at 2050 as shown in **Figure 4**, and these results are likely due to model uncertainty. For this reason, the precipitation data for 2050 and RCP 4.5 were not used in evaluating adaptive design options.

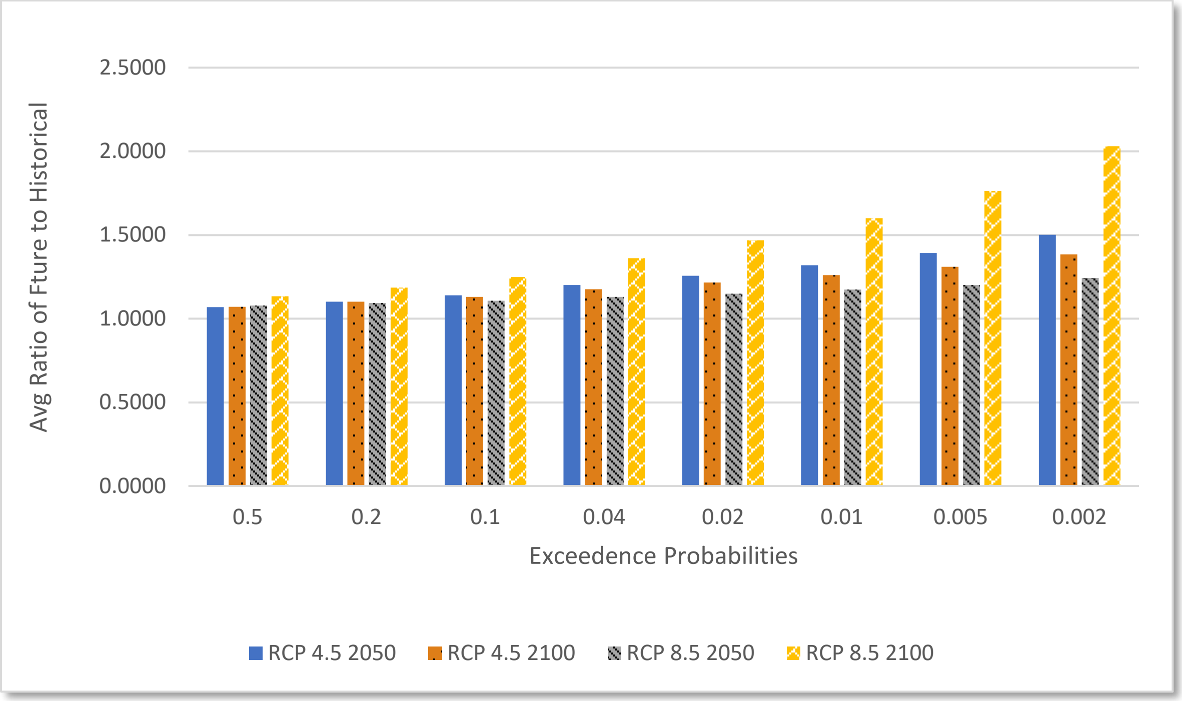


Figure 10: Average Precipitation Ratios Across All Eight GCMs for York County Site

The average ratios across all eight GCMs are shown in **Figure 11** for the Allegheny County site for 2050 and 2100 and for the RCP 4.5 and RCP 8.5 scenarios. Some pertinent observations are:

- For RCP 8.5 and 2100, the average ratios are essentially the same decreasing from 1.23 for the 0.5 AEP event to 1.18 for the 0.002 AEP event.
- For 2050, the ratios are larger for RCP 4.5 than RCP 8.5 for all AEP events of 0.02 to 0.002.
- For RCP 4.5, the ratios are larger for 2050 than 2100 for AEP events from 0.01 to 0.002.

It is unrealistic that precipitation, on average across all GCMs, is higher for 2050 than 2100 and higher for RCP 4.5 than RCP 8.5. Similar to the York County site, this variation is attributed to the fact that the scenarios do not differ much at 2050 as shown in **Figure 4** and these results are likely due to model uncertainty. For this reason, the precipitation data for 2050 and RCP 4.5 were not used in evaluating the alternative design options. Data for 2100 and RCP 8.5 were used to evaluate the alternative design options because they were more consistent and reasonable across the eight GCMs.

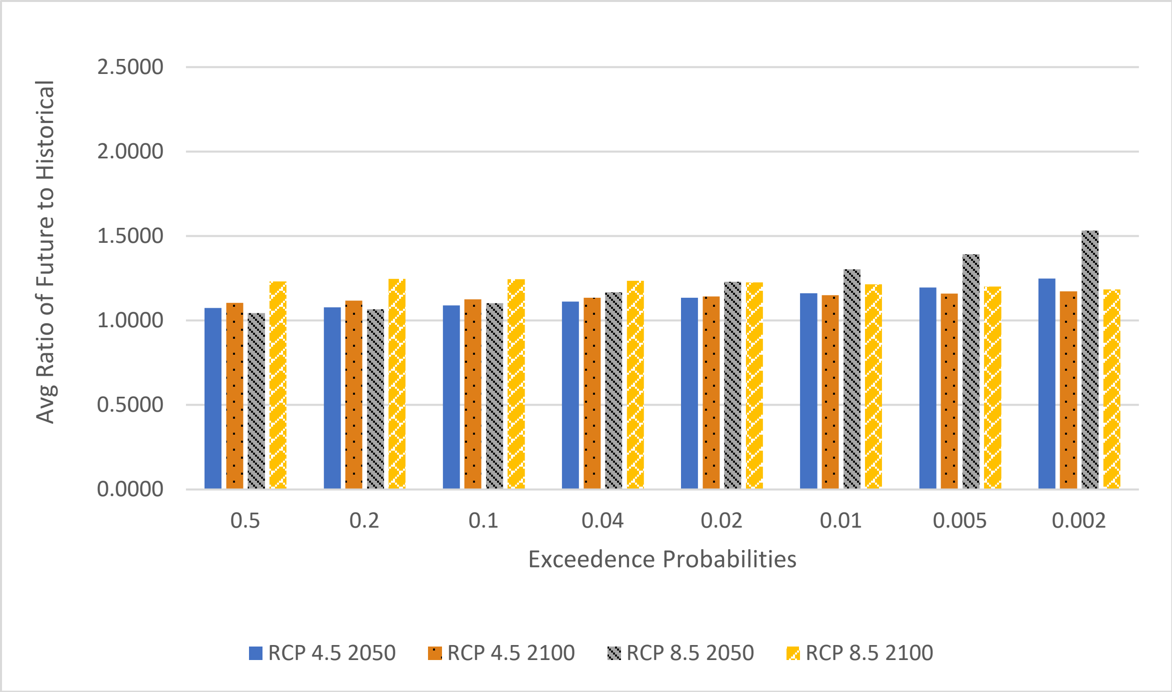


Figure 11: Average Precipitation Ratios Across All Eight GCMs for Allegheny County Site

The average ratios across all eight GCMs are shown in **Figure 12** for the Delaware County site for 2050 and 2100 and for the RCP 4.5 and RCP 8.5 scenarios. Some pertinent observations are:

- For RCP 8.5 and 2100, the average ratios are increasing slightly from 1.17 at the 0.5 AEP event to 1.21 at the 0.002 AEP event.
- For 2050, the average ratios are larger for RCP 4.5 than RCP 8.5 for all AEP events.
- For 2100, the average ratios for RCP 4.5 are just slightly less than RCP 8.5.

It is unrealistic that precipitation, on average across all GCMs, is higher for 2050 than 2100. For this site, the results for RCP 4.5 are just slightly less than RCP 8.5. Similar to the other sites, this variation is attributed to the fact that the scenarios do not differ much at 2050 as shown in **Figure 4** and these results are likely due to model uncertainty. For this reason, the precipitation data for 2050 and RCP 4.5 were not used in evaluating the alternative design options. Data for 2100 and RCP 8.5 were used to evaluate the alternative design options because they were more consistent and reasonable across the eight GCMs.

For this reason, the precipitation data for 2050 and RCP 4.5 will not be used in evaluating the alternative design options. The results for 2100 and RCP 8.5 are more consistent and reasonable and will be used to evaluate the alternative design options.

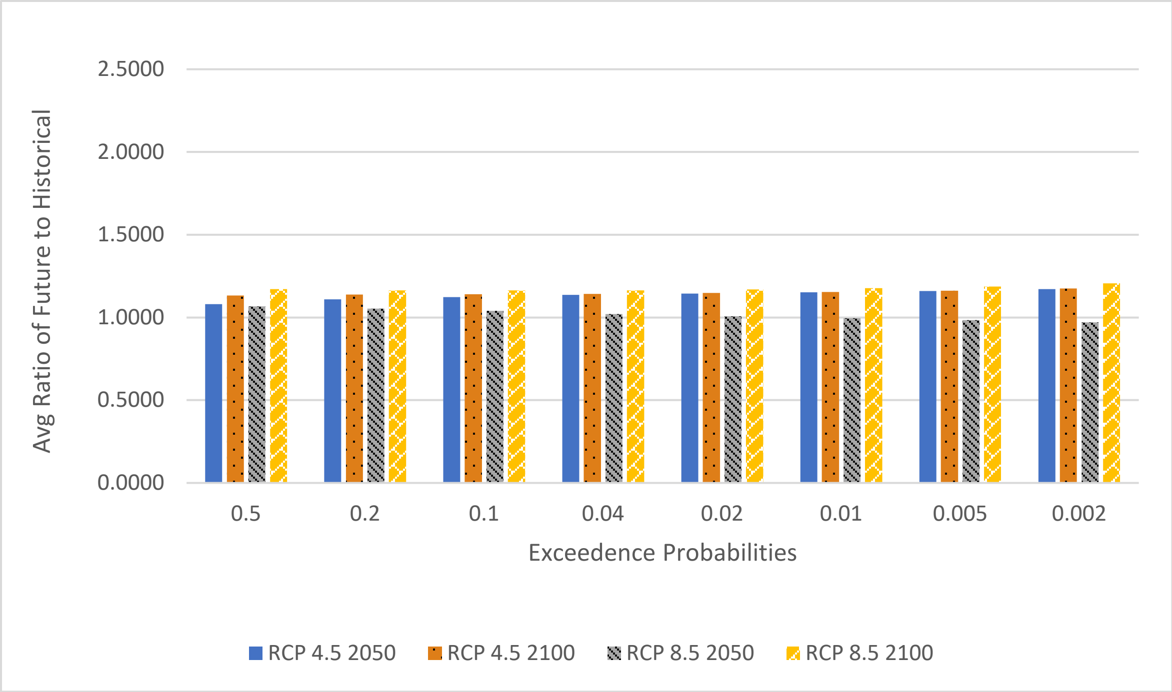


Figure 12: Average Precipitation Ratios Across All Eight GCMs for Delaware County Site

Comparison to the Fourth National Climate Assessment

Figure 13 shows the projected change in the daily 20-year extreme precipitation from the Fourth National Climate Assessment (NCA4) (Easterling and others, 2017). For higher emissions (RCP 8.5) for late-century, the increase in the 20-year daily precipitation was estimated to be 22 percent for the northeastern US. As a reality check, the increase in the 25-year daily precipitation from this pilot study for the three project sites ranged from 16 to 36 percent for the 0.04 AEP (25-year) event (see **Table 2**). The results from the National Climate Assessment are provided to illustrate that the results from the three pilot watersheds are reasonably similar. For the mid-century time frame and the lower emissions (RCP 4.5) for NCA4, the increase in the 20-year extreme daily precipitation was 14 percent (Figure 12), that is, less than increases for late-century and higher emissions (RCP 8.5) of 22 percent. These results illustrate that for our three pilot watersheds, the larger increases in precipitation for 2050 and RCP 4.5 than 2100 and RCP 8.5 are not reasonable.

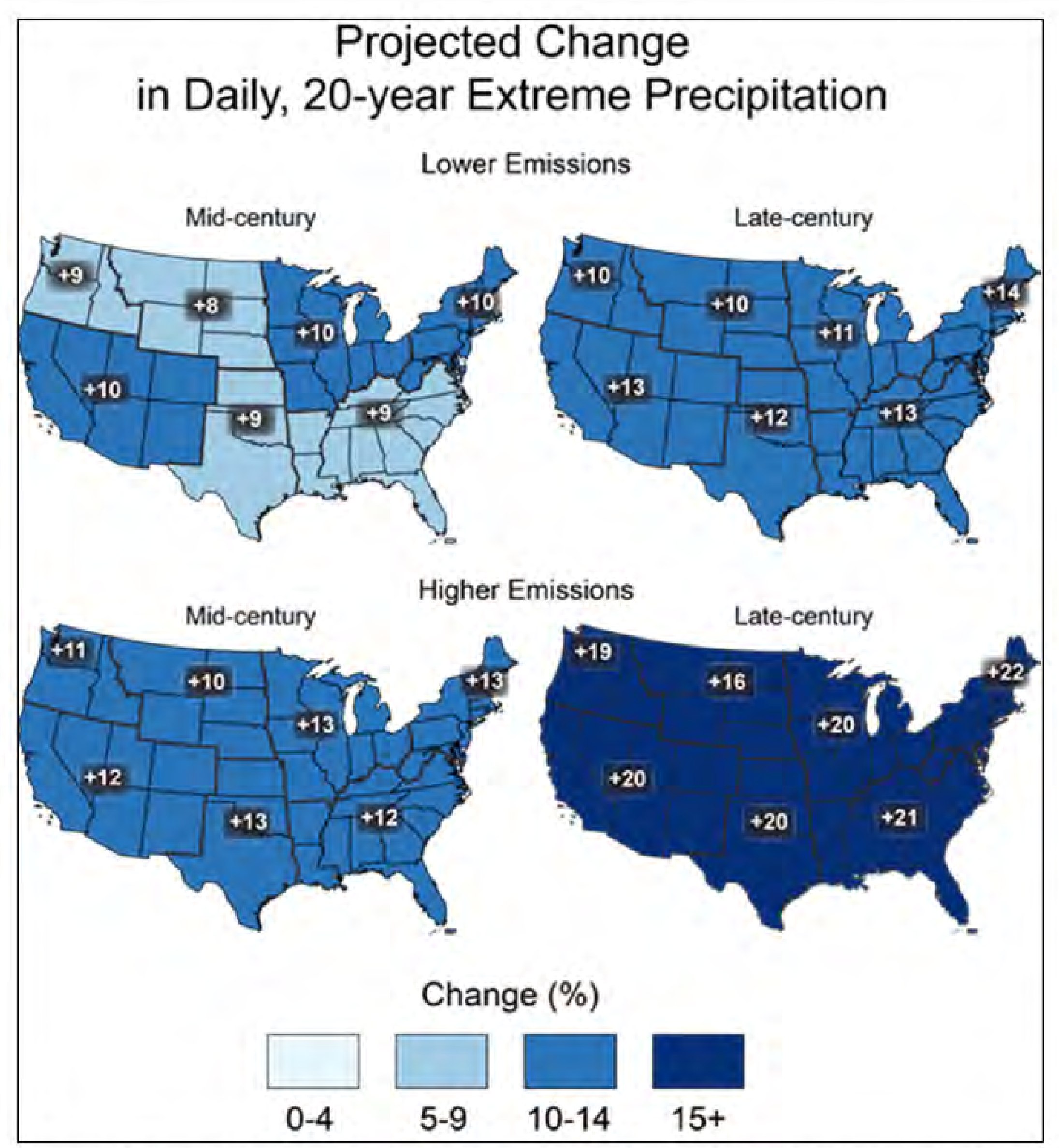


Figure 4.1. Projected change in the 20-year return period amount for daily precipitation for the mid- and late 21st century for a lower scenario (top, RCP4.5) and a higher scenario (bottom, RCP8.5), calculated from the LOCA dataset (Source: NCA4, Easterling et al. 2017).

Figure 13: National Climate Assessment Projected Changes

Projected changes in the daily 20-year extreme daily precipitation from the Fourth National Climate Assessment (Easterling and others, 2017)

Statewide Assessment of Future Precipitation Variation

The pilot study scope of precipitation analyses focused on three sites in Allegheny, Delaware and York counties. The results presented in the previous sections highlight the variability of the GCM results in those areas. During the project development process, PennDOT expressed interest in gaining a better understanding of how GCM results vary statewide. As a result, the precipitation analysis was expanded to include more areas within the state.

In order to further investigate the variability in GCM results, future precipitation data were obtained for an additional seven counties in different regions of Pennsylvania. The average precipitation ratios across the eight GCMs were calculated using the same procedures discussed for the three pilot county locations and represent an average across the 8 GCMs used for this study. A summary of the future to historical precipitation ratios by AEP are provided in **Table 3**. The geographical distribution of the sites is shown in **Figure 14.**

Table 3: Future to Historical Precipitation Ratios for 10 Counties in Pennsylvania
RCP 8.5 2100 Scenario for 0.5 (2-year event) to 0.002 (500-year event)

County*	Watershed Size (sq.-mile)	# of Grids	0.50 AEP Ratio	0.10 AEP Ratio	0.01 AEP Ratio	0.002 AEP Ratio
York	21.2	6	1.13	1.25	1.60	2.03
Allegheny	8.9	4	1.23	1.24	1.21	1.18
Delaware	22.9	6	1.17	1.16	1.18	1.21
Potter	68.4	10	1.18	1.17	1.19	1.22
Erie	56.5	11	1.18	1.20	1.25	1.30
Butler	43	8	1.16	1.12	1.05	1.01
Wayne	47.4	8	1.19	1.30	1.53	1.76
Cambria	47.2	9	1.10	1.20	1.50	1.82
Clearfield	57.6	13	1.15	1.25	1.47	1.67
Northumberland	61.8	12	1.20	1.25	1.33	1.40

* *The three pilot locations discussed in previous sections are highlighted*

Assessment of Regional Precipitation Variation

As shown in **Table 3**, the average precipitation ratios vary from 1.12 to 1.30 for the 0.10 AEP event. In contrast, the average precipitation ratios vary from 1.05 to 1.60 for the 0.01 AEP event and 1.01 to 2.03 for the 0.002 AEP event. The increased range in ratios is consistent with the increased uncertainty in predicting future extreme precipitation. Analyses in the additional seven counties indicated there are other areas in addition to York County where the average precipitation ratios increased by more than 50 percent for the extreme events like 0.01 and 0.002 AEP.

As discussed in Kilgore and others (2019a), the ability of GCMs to accurately estimate the extreme precipitation events used in hydrologic analyses is limited. For the pilot studies, the historical and future periods are based on approximately 50 years of daily precipitation data. The uncertainty in the ratios are increased when only 50 years of record are used to estimate extreme events with 0.01 AEP and 0.002 AEP (100- and 500-year events, respectively).

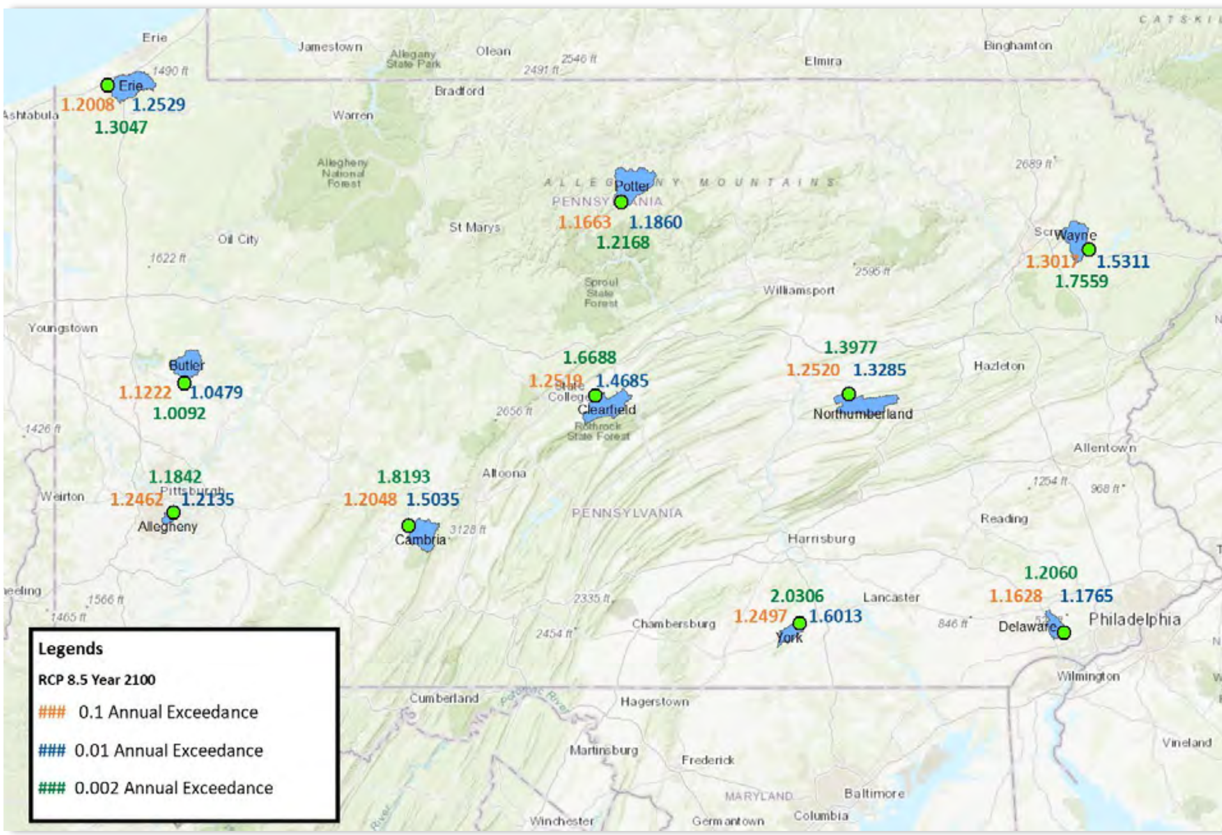


Figure 14: Map of Future to Historical Average Precipitation Ratios for 10 precipitation sites

The expanded precipitation analysis aimed to identify how GCM forecasts vary within different areas of the state. The average precipitation ratios across Allegheny and Delaware Counties were reasonably consistent across a large range of annual exceedance probabilities (0.5 to 0.002 AEP). However, the average ratios for the York County site varied significantly by GCM and AEP. The average precipitation ratios shown in **Figure 14** are not sufficient to define a regional trend. However, the average precipitation ratios for the more extreme events (0.01 and 0.002 AEP) appear to be highest in the central and northeastern parts of the state that includes York, Cambria, Clearfield, Northumberland and Wayne Counties. This trend could be related to topographic and orographic characteristics but we do not have enough data to determine that correlation. Additional data and further analyses are recommended to determine if the precipitation ratios vary with topographic or climatic characteristics.

Analyses and research on future precipitation are recommended for at least 15 additional sites (for a total of 25 sites) in the state to determine if a regional trend is more obvious with data for more sites. The objective would be to define regional values of precipitation ratios that could be used in future hydrologic analyses by PennDOT. Relating the precipitation ratios to topographic and climatic characteristics may be an approach for estimating these ratios at any site in the state.

Application to PennDOT Design Process

This pilot study has evaluated possible methods and procedures for incorporating future precipitation scenarios into hydrologic and hydraulic procedures as part of the PennDOT design process. In specific, the current research has aimed to address the questions shown in **Figure 15**. However, more research and evaluation are needed before the climate prediction methods are formally integrated into the design manuals and process. Several of the checklists or resiliency considerations related to the hydraulic analysis discussed later could also be considered for implementation into the PennDOT design process. The three site analyses conducted for this pilot study serve as a potential template on how the precipitation ratios may be integrated into analyses that utilize hydrologic and hydraulic models.

- How should those precipitation ratios be applied within the H&H process?
- What future precipitation ratios are appropriate for application?
- What hydraulic parameters should be evaluated under existing and future precipitation scenarios to help identify resilient design options?
- What resilient design options should be considered?

Figure 15: Research Questions for Resilient Design

Applying Future Precipitation Values to the H&H Process

Integrating the results of GCM precipitation analyses will require additional coordination and discussions within PennDOT. As summarized in **Figure 15**, several key elements have been identified through this pilot study that affect how future precipitation may potentially be used for design purposes.

There are many available GCMs that provide data under a multitude of scenarios and time periods. As shown within the pilot study assessments, the results can vary significantly depending on the choice of models and outputs. The determination of future precipitation ratios should be conducted carefully through a separate study conducted by persons with an understanding of the GCM models and parameters. The additional study would define the precipitation ratios that could be used for individual site H&H analyses using the approach described in **Figure 16**.

Estimate Precipitaton Ratios Outside of Specific H&H Studies

Evaluate Use of Future Ratios for Study Based on Site Flooding Risks

Use Ratios Based on Daily Data to Adjust Sub-daily Data

Develop Future Discharges Using Projected Precipitation Scenarios

Evaluate Application to Regresssion Equations (More Research Needed)

Figure 16: Using Precipitation Ratios in the H&H Process

The application or consideration of future precipitation may not be appropriate for all site locations under study (Kilgore and others, 2016). The use of future precipitation should be reserved for those structures that have a significant risk to flooding or have experienced historic impacts. A risk-based approach is recommended where the life span of the structure, the critical nature of the structure, the cost of potential damage to the structure and potential loss of life are factors considered in determining the extent of the H&H analyses. Other PennDOT risk factors including the results of PennDOT's Extreme Weather Vulnerability Study can support the assessment of locations where additional H&H scenario analyses may be warranted.

The GCM models and LOCA data base used for this study's precipitation analyses only produce daily precipitation estimates. The precipitation ratios based on the daily data are used to adjust sub-daily (n-hour and n-minute) NOAA Atlas 14 existing conditions precipitation values. The Atlas 14 data for all durations will be scaled using the daily precipitation data. This is necessary because there are currently no long term simulated sub-daily data for the RCP scenarios. In the future, projected sub-daily data should be available for several RCP scenarios.

The precipitation projections are used to estimate discharges that can inform the values of key hydraulic parameters needed for assessment of resiliency design options. For studies where hydrologic models are available, these projected precipitation values can be used as input to the models. The three pilot analyses conducted for this study illustrate the integration of precipitation ratios in HEC-HMS hydrologic modeling. The application of future precipitation ratios to regression-based approaches will require additional research to better understand the relationships between precipitation and discharge increases for different locations within Pennsylvania.

Defining Future Precipitation Values for Application

The pilot study precipitation analyses have highlighted the important variability in future precipitation estimates from GCMs. As discussed in Kilgore and others (2019a; 2019b), the ability of GCMs to accurately estimate the extreme precipitation events used in hydrologic analyses is limited. The pilot analyses for York County have shown an average GCM ratio for extreme events that is two times that of historical values. Those values are not reasonable for application to the design process and they would result in significant design changes and high infrastructure costs. **Figure 17** summarizes the pilot study recommendations for defining precipitation ratios for PennDOT design purposes.

Use 0.10 AEP Ratio for All Events For Evaluating Resiliency Design Options

Use Established Independent GCMs

Focus on 2100 and RCP 8.5 Scenarios using LOCA data set

Figure 17: Defining Appropriate Precipitation Ratios for PennDOT Application

For application to hydrologic and hydraulic modeling, the three pilot study analyses use the precipitation ratio for the 0.10 AEP for all events (0.50 and 0.04 to 0.002 AEP) for evaluating alternative adaptive options. The 0.10 AEP ratio is more stable and reasonable across all GCMs as illustrated in the precipitation analyses and assessments conducted for this study. The use of the ratio for the 0.10 AEP provides more reasonable estimates of future flood discharges particularly for the York County site. Kilgore and others (2019a) recommend using the 0.10 AEP ratio for the more extreme events because the current ability of high-resolution datasets to represent precipitation extremes (in the engineering hydrologic sense) is limited.

For the pilot studies, the precipitation ratios for 2050 and RCP 4.5 conditions were quite variable and not always reasonable. As shown previously in **Figure 4**, there is not much differences in the emission scenarios out to 2050 and, therefore, data for 2050 may not provide useful data for evaluating alternative designs for future studies. For the pilot studies, data for 2050 and RCP 4.5 were not used in evaluating the alternative design options. Precipitation data based on 2100 and RCP 8.5 were used in evaluating the alternative design options because the data were more reasonable and consistent across the eight GCMs. As discussed previously, this study recommends the use of the LOCA data set for determining precipitation estimates for these scenarios and these recommendations pertain to this data set.

Identifying Parameters for Resilient Design Assessment

A resilient design checklist is recommended to assist in the evaluation of the need for various resilient design options for each site-specific analysis. The checklist consists of interdisciplinary parameters, including hydraulics, traffic, safety, and others, that are compared between existing and future conditions to determine if the site will be more vulnerable to issues such as scour, stability, and roadway overtopping. The parameters evaluated consist of the typical metrics used in hydraulic design of bridges and roadways. Future conditions are defined as a hydrologic conditions in 2100 based on the RCP 8.5 scenario where existing precipitation depths are modified by applying the appropriate precipitation factor. For the purposes of this study, precipitation factors were determined using site-specific climate information discussed in preceding sections.

Each of the three pilot site analyses conducted for this study included a resilient design checklist. Two different types of checklists were developed. The first checklist in **Table 4** is applicable to typical bridge projects. The second checklist in **Table 5** is applicable to roadways that parallel streams or rivers. The example of the bridge checklist for the York site is shown in **Table 4** and the example for the lateral flooding checklist for the Allegheny site is shown in **Table 5**. The "Potential for Resilient Design" column in both checklists were completed with the following codes to indicate the level of potential for resilient design considerations:

- **Low:** minor or no special designs for resiliency anticipated
- **Medium:** considerations for resiliency related designs may be beneficial
- **High:** consideration for resiliency design modifications is highly recommended

Because every site is different, establishing ranges or thresholds for checklist parameters is not practical. When determining the level of potential for resilient design, engineering judgment, interdisciplinary coordination, and knowledge of existing site conditions should be used to evaluate the following:

- Risk posed to physical infrastructure and the traveling public,
- Comparison of existing and future parameters, and
- Magnitudes of future condition parameters and their effects on infrastructure performance.

These sample checklists were developed for the sites evaluated for this study. Additional coordination with PennDOT, FHWA, and other state agencies may result in revisions to these checklists prior to implementation as part of the PennDOT design process.

Table 4: Sample Resilient Design Checklist for Bridges (York County site)

	Parameter	Existing Condition[a]	Future Condition (RCP 8.5 for 2100)	Indicates Potential for Resilient Design
Site Data	**Hydrology Method**	HEC-HMS	HEC-HMS using 1.25*P[b,c]	N/A
	Embankment Instability	No issues noted	Minimal risk	Low
	Overtopping Frequency	0.5 AEP	0.5 AEP	Low
	Design Event Frequency	0.10 AEP		N/A
	Provides Access to Critical Services	N/A – short detour routes easily available	N/A – short detour routes easily available	Low
Design Event	**Discharge (cfs)**	2,850	4,500	N/A
	% Q Bridge	56	34	Low
	Pressure Flow	Yes	Yes	Low
	Bridge Velocity (fps)	5.6	5.5	Low
	Overtopping Velocity (fps)	2.7	3.2	Medium
	Overtopping Depth (Roadway) (ft)	2.6	4.3	Medium
	Overtopping Depth (Structure) (ft)	N/A	Below top of barrier	Medium
	Adjacent Roadway(s) Impacted	No	Yes	Medium
0.01 AEP Event	**Discharge (cfs)**	6,600	9,300	N/A
	% Q Bridge	23	17	Low
	Pressure Flow	Yes	Yes	Low
	Bridge Velocity (fps)	5.5	5.6	Low
	Scour Depth (ft)	2.9 ft	3.1 ft	Low
	Riprap Size	R-7	R-7	Low
	Overtopping Velocity (fps)	3.6 fps	3.9 fps	Medium
	Overtopping Depth (Roadway) (ft)	6 ft	7.9 ft	Medium
	Overtopping Depth (Structure) (ft)	Less than 0.2 ft	Up to 2 ft over barrier	High
	Adjacent Roadway(s) Affected	Yes	Yes	Medium

[a]Utilizes the geometry for the proposed bridge replacement project for consistent comparison to future conditions
[b]1.25 is the average 0.1 AEP ratio computed from 8 GCMs for the year 2100 and was selected for use in resilient design for the pilot study for the York County site analysis
[c]Future condition also includes projected land use changes within the watershed

Table 5: Sample Resilient Design Checklist for Roadways (Allegheny County site)

	Parameter	Existing Condition	Future Condition (RCP 8.5 for 2100)	Indicates Potential for Resilient Design
Site Data	**Hydrology Method**	HEC-HMS	HEC-HMS using 1.24*P[a]	N/A
Site Data	**Embankment Instability**	Yes	Yes	High
Site Data	**Overtopping Frequency**	0.20 AEP	0.5 AEP	Medium
Site Data	**Design Event Frequency**	0.04 AEP		N/A
Site Data	**Provides Access to Critical Services**	Yes - Pittsburgh Fire Bureau Station 20 located along Baldwin Road; Public Bus Route 56 (McKeesport to Downtown)		High
Bank full Flow	**Discharge (cfs)**	1,750 cfs		N/A
Bank full Flow	**Channel Velocity (fps)**	7.6		Medium
Bank full Flow	**Scour Depth (ft)**	20.5		High
Bank full Flow	**Event Frequency**	0.20 AEP	0.5 AEP	High
Design Event	**Discharge (cfs)**	3,400	4,800	N/A
Design Event	**Channel Velocity (fps)**	7.7	6.5	Low
Design Event	**Overtopping Velocity (fps)**	7.5	8.8	High
Design Event	**Overtopping Depth (Roadway) (ft)**	1.4	2.7	Medium
0.01 AEP Event	**Discharge (cfs)**	4,800	7,350	N/A
0.01 AEP Event	**Channel Velocity (fps)**	6.5	5.7	Low
0.01 AEP Event	**Overtopping Velocity (fps)**	8.8	9.5	High
0.01 AEP Event	**Overtopping Depth (Roadway) (ft)**	2.7	4.6	Medium

[a]1.24 is the average 0.1 AEP ratio computed from 8 GCMs for the year 2100 and was selected for use in resilient design for the pilot study for the Allegheny County site analysis

[b]Overtopping for the 0.5 AEP event is limited to certain areas in the existing condition. Under future conditions, 0.5 AEP roadway overtopping is more widespread

[c]Bankfull discharge and frequency vary across the full reach. For the purposes of this checklist, bankfull parameters are being evaluated where Streets Run makes a sharp turn along Baldwin Road (Cross Section 5085)

In addition to hydraulic parameters obtained from typical HEC-RAS output tables, velocity distribution plots for existing and future conditions can also be compared to determine potential areas of isolated velocity increases. Velocity distribution plots are a valuable tool for pinpointing areas of greater increase. For projects where site conditions warrant a 2-dimensional hydraulic model, more refined flow distribution and velocity output may provide further insights. Higher velocity increases could indicate locations within the bridge or along the roadway embankment that would benefit from resilient design improvements. The pilot case study analyses highlight the use of the velocity distribution plots.

Resilient Design Strategies

Where future changes in hydraulic conditions indicate the potential need for resilient design, possible design alternatives that should be considered. **Figure 18** highlights potential resilient design alternatives. The goal of implementing resilient design strategies are to protect critical infrastructure from failure and minimize risk to the public and the environment, while considering future climate scenarios. This list is not all-inclusive, and any design recommendations should be determined using an interdisciplinary approach. PennDOT will continue to work towards developing a resilient strategy toolbox highlighting successful approaches used in the past within Pennsylvania. Refer to the *Summary of Site-Specific Analyses* in the sections that follow, as well as **Appendices A-C**, for detailed information on the resilient design recommendations specific to each site.

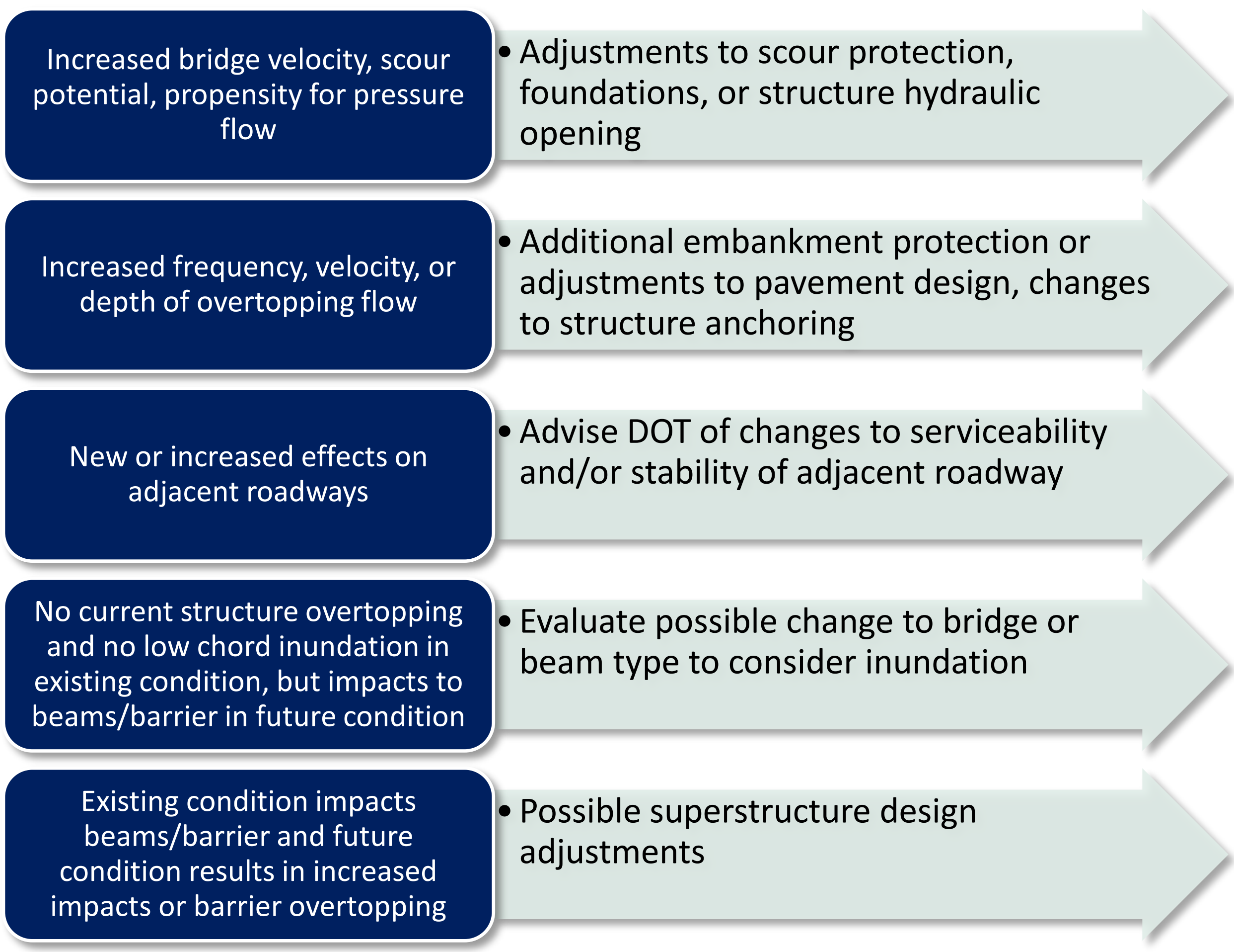


Figure 18: Example of Relating Resilient Design Alternatives to Hydraulic Conditions

Summary of Study Site Location Analysis Results

Three pilot H&H analyses have been completed in Pennsylvania to illustrate the application of projected precipitation ratios and to evaluate adaptation options. The pilot studies apply the concepts and results provided in the previous sections using the HEC-HMS and HEC-RAS modeling software. A brief summary of each of the pilot sites is provided below and a detailed analysis for each site is documented as separate appendices to this report as follows:

- Appendix A – York County (Baker Road) Pilot Analysis
- Appendix B – Allegheny County (Baldwin Road) Pilot Analysis
- Appendix C – Delaware County (Station Road) Pilot Analysis

York County (Baker Road) Pilot Analysis

The York County Bridge #177 along T-500 (locally known as Baker Road) over Little Conewago Creek is located in southcentral Pennsylvania adjacent to the Maryland Border. Baker Road is a rural collector roadway that provides access for residential property owners. Per PennDOT criteria the design event for the roadway is the 0.1 AEP event. The existing bridge is located on a sag curve with the left approach roadway experiencing significant overtopping during rain events. The existing Baker Road bridge is a single-span concrete adjacent box beam bridge with a normal clear span of 46.4 feet, out-to-out structure width of 18.5 feet, and average under clearance of 6.3 feet.

The drainage area at the Baker Road crossing is approximately 20.8 mi^2. A summary of the existing and future condition hydrologic and hydraulic results for the Baker Road Bridge for the 0.1 AEP roadway design event, and 0.01 AEP regulatory event are shown in **Table 6**.

Table 6: Baker Road over Little Conewago Creek Existing and Future Hydraulic Results

	Parameter	Existing Condition	Future Condition (RCP 8.5 for 2100)	Difference
0.10 AEP Event	**Discharge (cfs)**	2,850	4,500	+1,650
	Upstream WSE (ft)	389.21	390.89	1.68
	Upstream Channel Velocity (fps)	4.8	5.2	+0.4
0.01 AEP Event	**Discharge (cfs)**	6,600	9,300	+2,700
	Upstream WSE (ft)	392.65	394.49	1.84
	Upstream Channel Velocity (fps)	5.3	5.4	+0.1

In the future condition, water surface elevations are projected to increase 1.68 feet for the 0.1 AEP event and 1.84 feet for the 0.01 AEP event. Channel velocities are projected to increase 0.4 ft/s for the 0.1 AEP event and 0.1 ft/s for the 0.01 AEP event. In the existing condition, the 0.1 AEP event overtops the left approach roadway. In the future condition, the 0.1 AEP event is projected to overtop the bridge deck and both approach roadways. The 0.01 AEP event overtops the bridge deck and both approach roadways in the existing condition; increased water surface elevations are projected to result in the bridge barrier being overtopped in the future condition. Several options were considered for resilient design as outlined below:

- An increased hydraulic opening/bridge structure requires significant increases in the roadway profile which offsets any benefit from the increased opening. It is not feasible at this site to span the entire floodplain, so the increased elevation of the roadway profile associated with the larger span bridge causes increases in velocity and water surface elevations and is not effective for resilient design at this site.

- An evaluation of the velocity distribution across the approach roadways and through the bridge structure indicates that although the flows may increase significantly in the future, the velocities through the bridge and over the approach roadways increase minimally and may not necessitate any resilient design changes.

- The most significant finding is related to more frequent inundation of the bridge with future flows. An evaluation of several cases of deck and bridge barrier submergence was performed and determined that the force of water in these highly submerged cases could result in changes to a typical bridge design. If a submerged bridge is evaluated and determined to be insufficient for uplift and overturning, the following resilient design options could be considered:

    - Mechanically restrain bearings, such as with bolted steel plates
    - Extend smooth dowel bars so that they can better develop in the concrete or replace smooth dowels with deformed bars.

For the Baker Road bridge, it is recommended that the increased force of water on the bridge in the future condition be evaluated in more detail to determine if the above resilient design options may be needed. Considering that bridge superstructure submergence could become more common or occur more frequently under future climate scenarios, additional research is needed to determine standard factors and design methodologies for determining the force of water as a component of the typical bridge design procedure. Additional information regarding the resilient design measures, assumptions, and economic evaluation are included in **Appendix A**.

Allegheny County (Baldwin Road) Pilot Analysis

SR 2046 (locally known as Baldwin Road) is located in western Pennsylvania within the Hays neighborhood of the City of Pittsburgh. For much of its length SR 2046 is a minor arterial roadway with a 0.04 AEP design event. To the north of the SR 885 underpass, SR 2046 is a local roadway with a 0.1 AEP design event. Streets Run flows parallel to SR 2046 along much of the approximately 5,000-foot study reach. For approximately 3,500 feet, Streets Run flows directly adjacent to the roadway. For the remaining 1,500 feet, Streets Run flows parallel but approximately 300 feet to the east along the railroad embankment, before making a sharp bend back towards SR 2046. This reach of Streets Run has a history of frequent and significant flooding along Baldwin Road, the adjacent local roadways, and nearby properties.

The drainage area at SR 2046 along Streets Run is approximately 8.1 mi^2. A summary of the existing and future condition hydrologic and hydraulic results at four locations along the reach for the 0.04 AEP roadway design event, and 0.01 AEP regulatory event are shown in **Table 7**.

Table 7: SR 2046 Along Streets Run Existing and Future Hydraulic Results

Event	Location	Existing WSE (ft) Q = 3,400 cfs	Future WSE (ft) Q = 4,800 cfs	Difference	Existing Velocity (fps)	Future Velocity (fps)	Difference
0.04 AEP Event	**7430**	781.60	782.03	+0.43	15.6	16.8	+1.2
	6000	767.79	768.22	+0.43	8.9	9.3	+0.4
	5085	760.76	762.11	+1.35	7.7	6.5	-1.2
	4160[a]	750.83	752.14	+1.31	6.9	6.3	-0.6
Event	**Location**	**Existing WSE (ft) Q = 4,800 cfs**	**Future WSE (ft) Q = 7,350 cfs**	**Difference**	**Existing Velocity (fps)**	**Future Velocity (fps)**	**Difference**
0.01 AEP Event	**7430**	782.03	782.70	+0.67	16.8	17.3	+0.5
	6000	768.22	769.10	+0.88	9.3	8.9	-0.4
	5085	762.11	764.03	+1.92	6.5	5.7	-0.8
	4160	752.14	753.54	+1.40	6.3	6.5	+0.2

[a]At this location, roadway classification changes to "local roadway" (0.1 AEP design event). Results are provided for 0.04 AEP event for consistency and comparison with other locations along the reach.

In the future condition, water surface elevations are projected to increase up to 1.35 feet for the 0.04 AEP event and up to 1.9 feet for the 0.01 AEP event. Channel velocities are projected to increase up to 1.2 ft/s for the 0.04 AEP event and 0.5 ft/s for the 0.01 AEP event. Changes in velocity along the reach are variable (alternating between minor increases and decreases) due to changes in flow distribution throughout the wide floodplain and mixed flow regime along the reach. In the existing condition, the 0.04 AEP event overtops the adjacent roadway by approximately 1.4 feet, with overtopping beginning between a 0.5 AEP and 0.1 AEP event. In the future condition, increased depths and frequency of overtopping along the roadway are expected.

Resilient design alternatives were considered to address stability issues where SR 2046 is adjacent to Streets Run and existing scour and stability issues may be present. It is anticipated that under future climate scenarios, bankfull flow, which is anticipated to cause the worst-case scour condition, will occur more frequently. This will result in a higher possibility of damage to the roadway, serviceability issues due to roadway damage, and reduced design life of the roadway. Resilient design recommendations are provided for two different conditions within the study reach, as described below.

- Sections of the existing concrete wall may need modifications to improve the level of protection of the roadway for resistance to erosion and scour.
- Areas of existing roadway that are visibly damaged but no concrete wall or other embankment protection exists.

For each of these conditions, the following resilient designs are possible options to provide increased resiliency:

1. **Existing Wall Improvements:**
   a. Evaluate the level of protection needed at the toe of the existing wall.
      1) If needed, underpin the wall with micropiles and replace scour-prone soil below to an assumed depth of 6 feet below stream bed elevation with concrete filled bags.
      2) As an alternate to underpinning the existing wall, consider lining the stream channel with Fabriform to reduce risk of scour and eliminate the need to underpin the existing wall. Fabriform lining is placed from top-of-bank to top-of-bank and buried in anchor trenches beyond the top-of-bank on each side.

   b. Replace existing guide rail and pavement at edge of road with a reinforced concrete moment slab and single-face concrete barrier, to prevent frequent flood events from damaging the roadway (not intended to provide flood control for less frequent events but may reduce inundation of the roadway for more frequent storms).

2. **Add New Protection for Section Without Existing Wall:**
   a. Construct a new flood wall at edge of road with soldier pile and precast concrete lagging panels; place structure backfill to fill the void on the roadway side of panels; and repave one lane of roadway.

   b. As an alternate, reconstruct the stream bank with durable AASHTO #1 stone with 1.5H:1V side slope; line the stream channel with Fabriform from top-of-bank to top-of-bank to enhance scour protection; and repave one lane of roadway.

Additional details, including inherent assumptions with these recommendations and a preliminary economic evaluation are provided in **Appendix B**.

Delaware County (Station Road) Pilot Analysis

The Delaware County Bridge along T-314 (locally known as Station Road) over East Branch Chester Creek is located in southeastern Pennsylvania adjacent to the Delaware/Chester County Border. Station Road is a rural collector roadway that provides access for residential property owners. Per PennDOT criteria the design event for Station Road is the 0.1 AEP event. The existing bridge is located on a sag curve with the north approach roadway experiencing overtopping during 0.1 AEP event. The existing Station Road bridge is a single-span composite concrete beam bridge with a normal clear span of 50.0 feet, out-to-out structure width of 22.0 feet, and under-clearance ranging from 2.8 to 8.1 feet. The bridge has a concrete parapet with stone masonry wingwalls.

The drainage area at the Baker Road crossing is approximately 22.9 mi^2. A summary of the existing and future condition hydrologic and hydraulic results for the Station Road Bridge for the 0.1 AEP roadway design event, and 0.01 AEP regulatory event are shown in **Table 8**.

Table 8: Station Road over East Branch Chester Creek Existing and Future Hydraulic Results

	Parameter	Existing Condition	Future Conditions	Difference
0.10 AEP Event	Discharge (cfs)	3,950	6,150	+2,200
	Upstream WSE (ft)	242.64	243.61	0.97
	Upstream Channel Velocity (fps)	3.3	4.0	+0.7
0.01 AEP Event	Discharge (cfs)	9,950	14,400	+4,450
	Upstream WSE (ft)	244.65	245.61	0.96
	Upstream Channel Velocity (fps)	5.1	6.2	+1.1

Water surface elevations would increase up to 1.0 feet and average channel velocities would increase up to 1.1 ft/s in the future condition. In the existing condition, the 0.1 AEP event overtops the both approaches and the structure. In the future condition, the 0.1 AEP event would overtop the bridge deck and both approach roadways and the structure. The 0.01 AEP event overtops the bridge deck and both approach roadways in the existing condition. For that same AEP increased water surface elevations would result in the bridge barrier being overtopped in the future condition.

Because of the significant roadway overtopping at this site, which would only be exacerbated under future conditions, design options were explored with the goal of reducing overtopping depth and frequency. To demonstrate how changes to the structure and roadway will affect hydraulic parameters such as velocity and water surface elevations, a sensitivity analysis was performed for the site that explored two alternatives:

1. An approximately 25% increase in the proposed bridge span, and
2. An approximately 90% increase in the proposed bridge span with left bank grading through bridge to further increase the hydraulic opening, and an approximate 3.5-foot roadway profile increase.

Alternative 2 was designed to remove the overtopping for the 0.10 AEP event, while Alternative 1 shows a reasonable design that might be selected for a standard bridge replacement. Results indicate that only a significantly larger span and higher profile (Alternative 2) would remove overtopping for the 0.10 AEP event. However, that design alternative would result in increased bridge velocities and an approximately 3-foot increase to the upstream water surface elevation for the 0.01 AEP event. Additionally, calculated scour depths doubled over the existing condition, and larger rock size would be required at the abutments due to increased velocities. Alternative 2 was not recommended for practical reasons. In addition, large increases to the 0.01 AEP event would also remove this option from consideration because the bridge is in a FEMA Zone AE, which would require no more than a 1-foot increase to 0.01 AEP WSELs. Alternative 1 only slightly affects flooding conditions, but patterns generally remain the same as the no resilient design.

Additional information regarding the resilient design measures, assumptions, and economic evaluation are included in **Appendix C**.

Relationship between Precipitation Increases and Flow Increases

The hydrologic analyses for the three pilot studies (York, Allegheny and Delaware Counties) were based on Natural Resources Conservation Service (NRCS) hydrologic procedures namely the NRCS direct runoff equation and curve number. For the three pilot studies, the HEC-HMS modeling indicated that the large flood discharges increased approximately twice as much as the increase in future precipitation. If this trend is reasonable, then this could be used to increase estimates from USGS regression equations where hydrologic modeling results are not available. The larger increase in discharge as compared to precipitation is consistent with modeling results for the three pilot locations:

- York County site where there was an increase of 60 percent for the 0.5 AEP discharge and 40 percent increase for the 0.01 and 0.002 AEP discharges when the precipitation increased 25 percent,
- Delaware County site where there was an increase of 40 percent for the 0.5 AEP discharge and 30 percent increase for the 0.01 and 0.002 AEP discharges when the precipitation increased 16 percent, and
- Allegheny County site where there was an increase of 80 percent for the 0.5 AEP discharge and 50 percent increase for the 0.01 and 0.002 AEP discharges when the precipitation increased 24 percent

Further research is needed but this may be a reasonable approach for estimating future peak discharges from USGS regression equations when modeling results are not available.

Conclusions and Lessons Learned

This pilot study has provided initial research into the potential application of future precipitation to the hydrologic and hydraulic analyses conducted as part of the PennDOT design process. Conclusions and "lessons learned" garnered through this study are provided below:

- This study has highlighted the significant variability of GCM models across individual models, areas within the state, RCP scenarios and analysis years. In Pennsylvania, this study found the results from the RCP 4.5 and 2050 scenarios not useful in the design process.
- Additional research is needed to help select appropriate GCMs for projected precipitation assessments. This pilot study has selected well-tested and evaluated models based on recent research reviewed by the study team. Further guidance and research may be needed to ensure that agencies apply consistent and reasonable methods in selecting and assessing GCMs.
- The expanded precipitation analysis conducted for this study aimed to identify how GCM forecasts vary within different areas of the state. Analyses and research on future precipitation are recommended for additional sites in the state to further assess regional data variances. The objective would be to define regional values of precipitation ratios that could be used in future hydrologic analyses by PennDOT.
- Applying the results of GCM precipitation forecasts to the design process requires careful consideration. Directly applying GCM results for low probability precipitation events could result in very high design stream discharges requiring expensive or unreasonable resiliency improvement actions to address. The three pilot study analyses use the precipitation ratio for the 0.10 AEP for all events (0.04 to 0.002 AEP) for evaluating alternative adaptive options. The 0.10 AEP ratio is more stable and reasonable across all GCMs as illustrated in the precipitation analyses and assessments conducted for this study.
- A resilient design checklist is recommended to assist in the evaluation of the need for various resilient design options for each site-specific analysis. The checklist consists of hydraulic parameters that are compared between existing and future conditions to determine if the site will be more vulnerable to issues such as scour, stability, and roadway overtopping.
- Identifying cost-effective adaptation strategies for individual sites is a difficult process requiring expertise from technical staff across multiple disciplines. This study has highlighted some potential strategies, but it may be beneficial for PennDOT and other DOTs to provide their designers with an adaptation strategy toolbox. The toolbox could provide

strategies that have been successfully implemented in other areas and illustrate potential costs and considerations.

- A key factor that is apparent is that each project site will have its own specific issues and must be assessed individually. It is recommended that a risk-based approach be considered. The risk factors could consider other non-H&H factors in the analysis that impact the priority of resilient design options such as the intended life span of the structure. Risk factors could include potential loss of life, length of detour if road is closed, average daily traffic, emergency services priority route, potential for overtopping to cause significant erosion, and others.

Future Research and Assessments

This pilot study has provided a framework for integrating resiliency into the transportation infrastructure design process. Building on this pilot study, future research and assessments are needed to better understand GCM precipitation projections across the state, evaluate how precipitation projections can be integrated into regression-based hydrology processes, and to assemble an adaptation strategy toolbox supported by state and local agencies. These future research needs are highlighted below.

Additional Data Assessments Across State

The three pilot locations highlight the variances of precipitation for different regions within the state and the relationships between precipitation and discharges at individual sites. Precipitation ratios were also estimated at seven sites around the State in addition to the three pilot sites. However, additional analyses that expand upon those presented in this study are needed to gain a better understanding of appropriate precipitation ratios and/or discharges for application within the PennDOT design process.

It is recommended the future precipitation analyses be conducted for at least 15 more sites around the state in addition to the 10 sites analyzed in this study (for a total of 25 sites statewide) in order to better define any regional trends. Further evaluation of NRCS hydrologic procedures should be conducted to demonstrate that this study's modeling results of larger increases in discharge (i.e., approximately twice as much for the larger discharges) than precipitation is logical. These assessments could be supported by compiling existing HEC-HMS and HEC-1 modeling results at other watersheds throughout the state where the runoff curve number was used. Some of these watersheds are close to the locations where future precipitation has been determined. The HEC-HMS and HEC-1 modeling results at sites throughout the state can be used to further evaluate the hypothesis that flood discharges increase approximately twice as much as the precipitation increases. The objective is to define a relation between discharge and precipitation that can be used to increase the USGS regression estimates where modeling results are not available or too time consuming to pursue.

Evaluate Application to Regression Analyses

The USGS regression equations in StreamStats for Pennsylvania are those documented in Scientific Information Report 2008-5102 by Roland and Stuckey (2008). This study divided the state into four hydrologic regions, but no region included a precipitation variable. USGS has updated those equations with Scientific Information Report 2019-5094 by Roland and Stuckey (2019) but these equations are not in StreamStats yet. The 2019 study divided the state into five hydrologic regions but again no precipitation variable was statistically significant in any region. Therefore, neither the previous nor recently published USGS regression equations for Pennsylvania can be used to predict future discharges based on future precipitation. This is typical of most statewide regression equations developed by USGS because the state is often divided into four or more hydrologic regions like Pennsylvania where the variation in precipitation is not sufficient to be statistically significant in the regression equations. An alternative approach as discussed earlier is to define the relation between discharge and precipitation and then use increases in future precipitation to increase flood discharges from USGS regression equations. The increases in precipitation would be based on regional estimates of precipitation ratios determined from additional analyses of future precipitation for at least 25 sites statewide.

Some USGS statewide regression equations do include a statistically significant precipitation variable. A cursory review of most USGS statewide regression equations revealed there are at least 12 states where the mean annual precipitation is statistically significant in at least one hydrologic region within the state. Of those 12 states, there were eight states where the exponent on mean annual precipitation was greater than 1 indicating the discharges increase more percentage wise than precipitation. An additional four states were found where some statistical estimate of precipitation like the x-year 24-hour precipitation was used in the regression equations. For all four states, the exponents on the precipitation frequency variable was greater than 1 again implying a larger percent increase in discharge than in precipitation. Therefore, there is limited evidence in 12 of 16 states that discharges increase more percentage wise than precipitation in the USGS regression equations.

The magnitude of the exponents on any precipitation variable in a regression equation is dependent on the variability of precipitation across the region and the correlation with other explanatory variables. Regression equations are statistical relations, not deterministic relations, so the exponent on precipitation can vary depending on regional conditions. Most USGS regression equations tend to use mean annual precipitation as an explanatory variable because it is easy to determine. This is different from the x-year 24-hour precipitation statistic used in hydrologic modeling so the comparisons between regression equations using mean annual precipitation and hydrologic modeling are not totally comparable.

Another alternative is to develop regression equations using gaging station data in or near Pennsylvania and use a precipitation frequency statistic from the LOCA data base. This approach is analogous to the development of a regression equation for the Extreme Weather Vulnerability study previously completed by PennDOT (2017). For that study, an equation was developed for predicting the 1-percent annual chance (0.01 AEP) flood based on watershed characteristics, a measure of land use (impervious area) and a precipitation variable from the Downscaled CMIP3 and CMIP5 Climate and Hydrology Projections (DCHP) data set (USBR, 2013). Future discharges were estimated by using future impervious area and future precipitation. The LOCA data set as used in the pilot study is now considered more applicable than the DCHP data set for engineering and hydrologic studies as described earlier. The regression equation developed for the 2017 PennDOT Extreme Weather Vulnerability study for the 1-percent annual chance flood ($Q_{1\%}$) is shown below.

$$Q_{1\%} = 4.442\, DA^{0.898} SL^{0.307} (Stor + 1)^{-0.580} (IA + 1)^{0.217} Pmeanrain^{0.809}$$

Where DA = Drainage area, in square miles;
SL = channel slope, in feet per mile,
Stor = surface area of lakes and ponds, in percent of the watershed area,
IA = impervious area, in percent of the watershed area, and
Pmeanrain = the mean of the annual maximum daily precipitation for the period 1950-99.

The Pmeanrain variable in the above equation could be replaced by a precipitation frequency statistic from LOCA data such as the mean of the annual maximum daily precipitation or perhaps the 0.5 AEP or 0.10 AEP event. If this research effort were pursued, regression equations for other AEP events other than 0.01 would be developed to provide a range of future design discharges. LOCA data would need to be downloaded for all gaging stations used to develop the regression equations consistent with the approach used for the three pilot study watersheds. About 100 gaging stations are needed to develop regression equations that are applicable statewide.

Resilient Strategy Toolbox

Improving the resiliency of Pennsylvania's transportation system ultimately requires the application and integration of cost-effective adaptation strategies. For PennDOT's Extreme Weather Vulnerability Study completed in 2017, example strategies were highlighted from stakeholder outreach comments, current practices within Pennsylvania, and a national literature review. These strategies and the associated toolbox were intended as a starting point for future discussions and activities in determining what strategies may be most viable for PennDOT and its planning partners. As PennDOT works to evaluate the role of resiliency in the design process, additional steps may be needed to review and assess viable strategies. Specific strategies that address erosion and scour potential as well as the resilience of bridges and roadways to more frequent inundation should be prioritized for future research.

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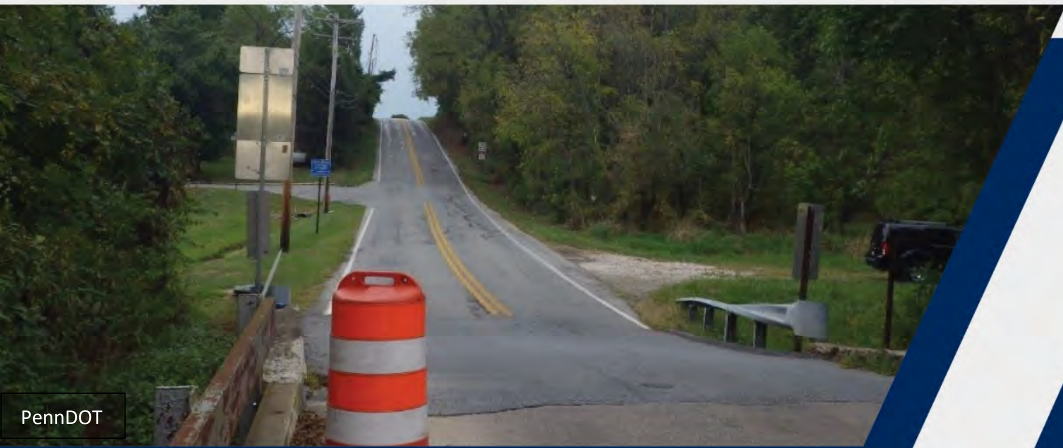

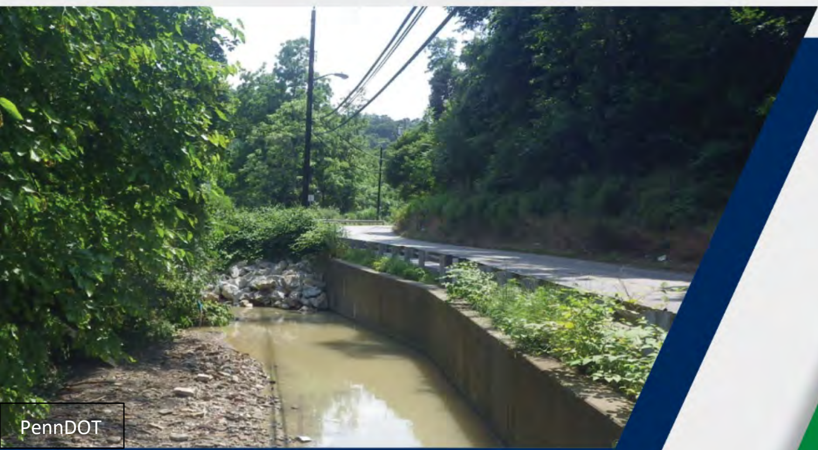

APPENDIX A

York County (Baker Road) Pilot Analysis

SEPTEMBER 2020

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Introduction

The hydrologic and hydraulic (H&H) analysis for York County Bridge #177 along T-500 (locally known as Baker Road) over Little Conewago Creek was performed as a part of PennDOT's Pilot Study for Resilience and Durability to Extreme Weather. The project involves computing existing and future peak flows using physically based hydrologic methods and developing a one-dimensional hydraulic model that considers existing and future conditions for three sites in Pennsylvania. The York County site was selected because of the history of roadway overtopping at the site and the possibility of developing resilient design strategies for an upcoming project to replace the existing bridge. York County is located in southcentral Pennsylvania adjacent to the Maryland Border.

Site Description

York County Bridge #177 is located west of the City of York, at the border between Dover Township and West Manchester Township, in York County, Pennsylvania. Its location on the USGS quadrangle map entitled West York, PA is approximately 39° 57' 55" N latitude and 76° 49' 29" W longitude. The project location is shown in **Figure A-1**.

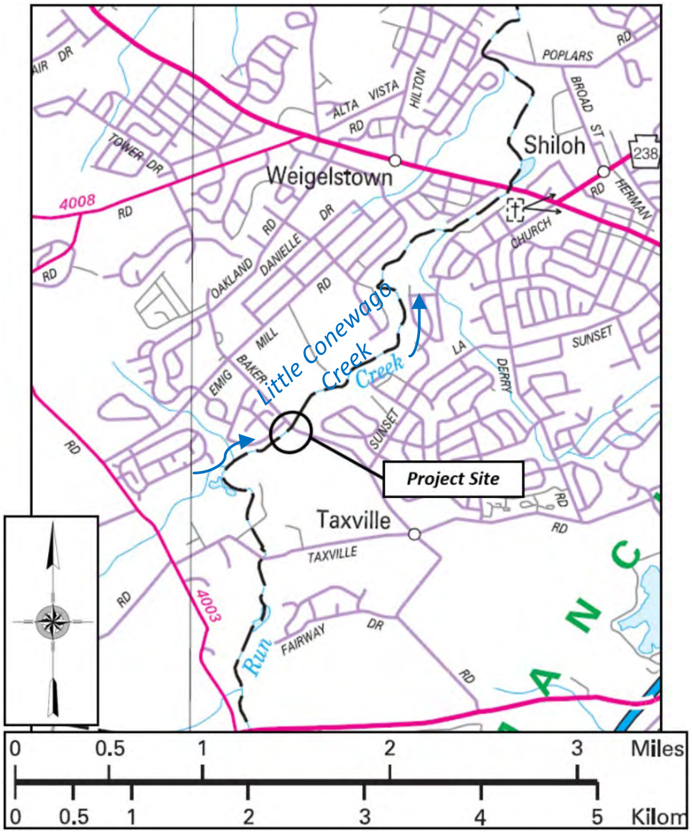

Figure A-1: Project Location

Baker Road is a rural collector roadway that provides access for residential property owners. Per PennDOT DM-2 Chapter 10.6.E the design event for the roadway is the 0.1 AEP event. The existing bridge is located on a sag curve with the left (north) approach roadway experiencing significant overtopping during rain events. Photographs of the approach roadway and stream channel are in **Figure A-2** and **Figure A-3**. Additional location maps and site photographs are included in **Attachment A-1**.

Figure A-2: Baker Road Left Approach. Credit: PennDOT

Figure A-3: Baker Road Bridge – Looking Downstream. Credit: PennDOT

Existing and Proposed Structures

The existing Baker Road bridge is a single-span concrete adjacent box beam bridge with a normal clear span of 46.4 feet, out-to-out structure width of 18.5 feet, and average underclearance of 6.3 feet. The existing bridge has concrete curb and metal railing over the structure. The existing bridge is planned to be replaced with a single-span pre-stressed concrete adjacent box beam bridge on the same horizontal alignment and will have a slightly larger span (47.8 feet) and an underclearance of 6.1 feet. Bridge barrier is proposed across the structure. Since H&H analyses and environmental permitting of the proposed bridge are complete, the proposed structure will be incorporated into the hydraulic analysis for the resiliency study.

Watershed Characteristics

The drainage area at the Baker Road crossing is approximately 20.8 mi^2 as delineated in Watershed Modeling System (WMS) 11.0 (Aquaveo, 2019) using a 10-meter Digital Elevation Model (DEM) as shown in **Figure A-4**. SSURGO soils, obtained from NRCS, and 2011 National Land Cover Database (USGS, 2011) land use data were used to compute curve numbers for each subbasin, which range from 64 to 76. Note that the curve numbers presented in Figure A-4 were calculated in WMS; adjustments were made in the HEC-HMS model as discussed below. The basin generally consists of agricultural land with residential development in the lower watershed and some forested area in the headwaters. There are two large quarries located within the basin, requiring adjustments to the DEM and subbasin curve numbers to accurately represent the hydrologic effects. The quarry on the eastern edge of the watershed was removed from the delineation as this area does not contribute to Little Conewago Creek peak flows. To represent flow lost from the 1 mi^2 quarry in the southcentral portion of the watershed, the curve number for subbasin 11B was manually adjusted to 58 in HEC-HMS. The basin has 23% carbonate area which was considered by applying a reduction factor to the flows computed using the physical hydrologic model. The carbonate factors were determined from the USGS SIR 2008-5102 equations for Region 2.

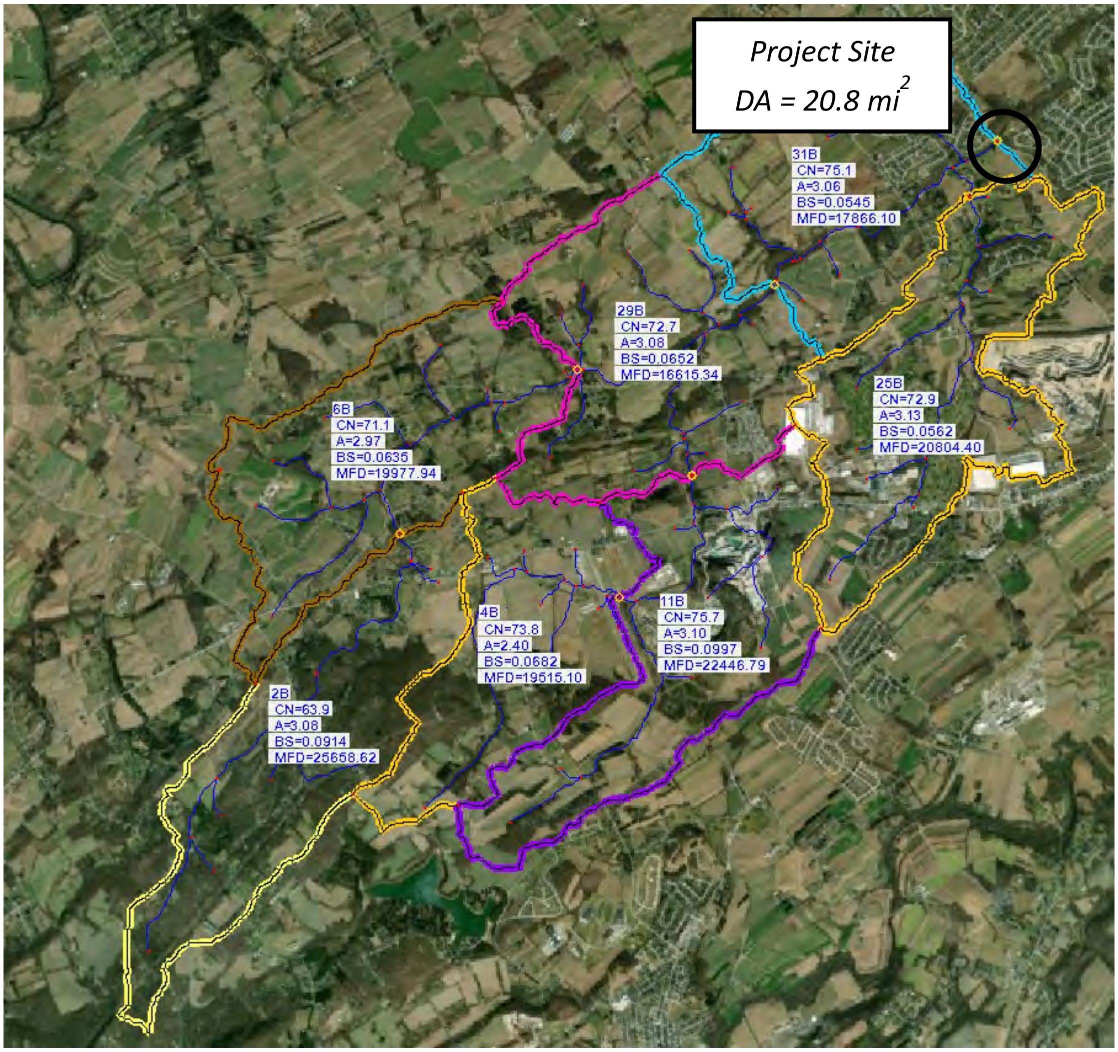

Figure A-4: Little Conewago Creek Watershed

Hydrologic Method

Peak flows for the current study were computed using the Army Corps of Engineers' Hydrologic Modeling System (HEC-HMS) 4.2.1. HEC-HMS is applicable for basins of almost any size and complexity. The subbasins were delineated and geometric parameters for each subbasin were calculated using WMS. At all subbasins, the curve number method was used as the loss rate method. The time of concentration (T_c) was calculated in the WMS program using the NRCS segmental method. The Muskingum-Cunge routing method was used for all reaches and was calculated using a trapezoidal channel with bottom widths of 20-30 feet, floodplain widths of 250-600 feet, side slopes of 2H:1V, channel Manning's n of 0.055, and overbank Manning's n of 0.2. Routing parameters were estimated using aerial imagery and topographic data. The subbasin data was then exported from WMS to HMS. The Lag Time (T_L) for each subbasin was manually adjusted to be 0.6* T_c. Subbasin lag times range from 62 minutes to 122 minutes. All events were modeled using the NRCS Type C storm distribution that provides precipitation depths at intervals ranging from 5 minutes to 24 hours, with existing precipitation depths determined using the PDT-IDF curves from PennDOT Publication 584 Appendix 7A, which are based on data from NOAA Atlas 14, Volume 2. Existing 24-hour precipitation depths are shown in **Table A-1**. Additional details regarding watershed characteristics and hydrologic methods and results are in **Attachment A-2**.

Table A-1: Existing Precipitation Depths

Annual Exceedance Probability	0.5	0.1	0.04	0.02	0.01	0.002
Precipitation Depth (in)	3.16	4.57	5.60	6.53	7.63	10.98

Existing Flows and Validation

Existing flows computed using HEC-HMS were validated using local flood history and compared to other hydrologic methods as shown in **Table A-2**. Comparison flows for Little Conewago Creek at Baker Road were computed using two USGS regression methods: USGS WRIR 2000-4189 (Stuckey and Reed, 2000) and USGS SIR 2008-5102 (Roland and Stuckey, 2008). Flows were compared to the FEMA published flows for the reach, which were also computed using the USGS 5102 method. The HEC-HMS flows computed for the current study are similar to the USGS regression results for most events, producing more conservative flows for the 0.01 and 0.002 AEP events. The HEC-HMS flows also produce hydraulic results that are consistent with reports of frequent overtopping obtained from local news sources. Therefore, the HEC-HMS flows are considered validated for use in the hydraulic analysis for the current study.

Table A-2: Existing Flow Comparison

Method	Peak Flows (cfs)					
	0.5	0.1	0.04	0.02	0.01	0.002
HEC-HMS*	1,700	2,850	3,950	5,000	6,600	10,950
FEMA	-	2,985	-	5,056	6,117	9,107
USGS 5102*	1,400	3,050	-	5,000	5,950	8,600
USGS 4189*	-	2,550	3,650	4,650	5,800	9,450

*Calculated flows rounded to nearest 50 cfs.

Existing Hydraulic Performance

A hydraulic model for the existing and proposed structures was developed and run in HEC-RAS 5.0.7. Topographic information consists of site-specific field survey data in the channel and immediate overbank areas. The hydraulic model results indicate that both the existing and proposed structures have frequent, significant overtopping of the left approach roadway. This result is validated by documented flooding history at the site. As shown in **Figure A-5**, a photograph from WTMP Fox43's broadcasting of the heavy precipitation event that occurred on October 11, 2013 show overtopping of the left approach roadway and the upstream left overbank from the project site. According to the “Community Collaborative Rain Hail and Snow Network”, there was 4.97 inches of rain in 24 hours. This precipitation amount for a 24-hour duration indicates the storm was between a 0.1 and 0.04 AEP event.

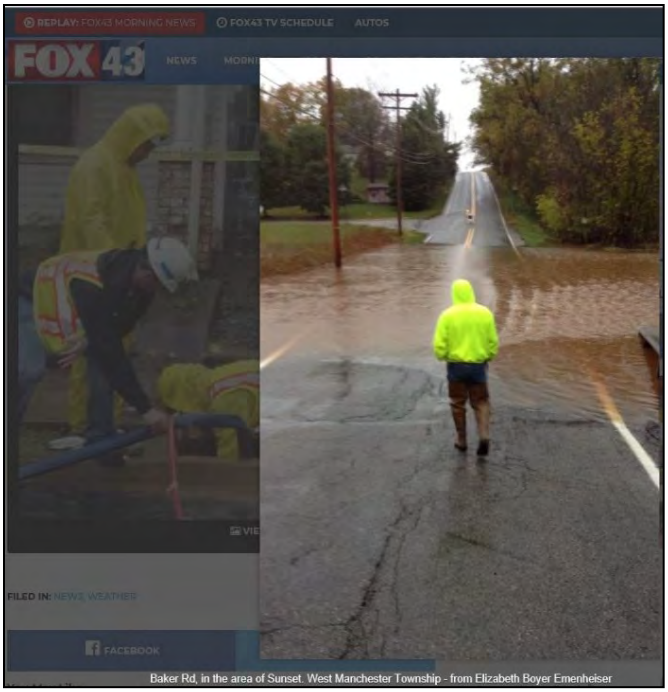

Figure A-5: October 2013 Flooding (Left Roadway Approach)
Photo Source: Fox 43 News; taken by Elizabeth Boyer Emenheiser

Global Climate Model Summary

The Global Climate Model (GCM) evaluation for the current study looked at eight distinct GCMs for Representative Concentration Pathway (RCP) 8.5 for year 2100, which predicts the highest future CO_2 equivalent of the various RCPs adopted by the Intergovernmental Panel on Climate Change (IPCC). RCP 8.5 was the trajectory adopted in the PA Climate Impacts Assessment Update (Shortle and others, 2015). The GCMs selected for this study were determined using guidance from the Infrastructure and Climate Network (ICNET) and Transportation Research Board reports developed for the NCHRP Project 15-61 (Kilgore and others, 2019a; 2019b). Future (year 2100) 24-hour precipitation data for the eight GCMs were compared against GCM estimates of historical precipitation to develop ratios for various annual exceedance probabilities (AEPs). The calculated ratios for the Little Conewago Creek watershed are shown in **Table A-3** and compared in **Figure A-6**. Precipitation depths and flows computed using the ratios for each AEP across all GCMs are shown in **Table A-4** and **Table A-5**, respectively. Flows computed using the future precipitation depths are in **Table A-5**. The calculated ratios, precipitation depths, and flows for each GCM are provided to demonstrate the significant variability across the selected climate models and are not recommended for design purposes.

Table A-3: Future Precipitation Ratios

GCM	Precipitation Ratios					
	Annual Exceedance Probability					
	0.5	0.1	0.04	0.02	0.01	0.002
BCC-CSM 1.1-m	1.28	**1.60**	1.82	2.00	2.21	2.76
CCSM4	1.09	**1.50**	1.90	2.30	2.81	4.55
CSIRO-Mk3.6.0	1.07	**1.39**	1.68	1.95	2.28	3.31
GFDL-CM3	1.19	**1.03**	0.94	0.87	0.80	0.67
GISS-E2-R	1.16	**1.13**	1.14	1.16	1.18	1.24
HadGEM2-AO	1.03	**1.01**	1.01	1.01	1.01	1.02
MIROC5	1.22	**1.21**	1.18	1.16	1.13	1.05
MRI-CGCM3	1.04	**1.12**	1.22	1.30	1.39	1.65
Average Ratios	**1.13**	**1.25**	**1.36**	**1.47**	**1.60**	**2.03**

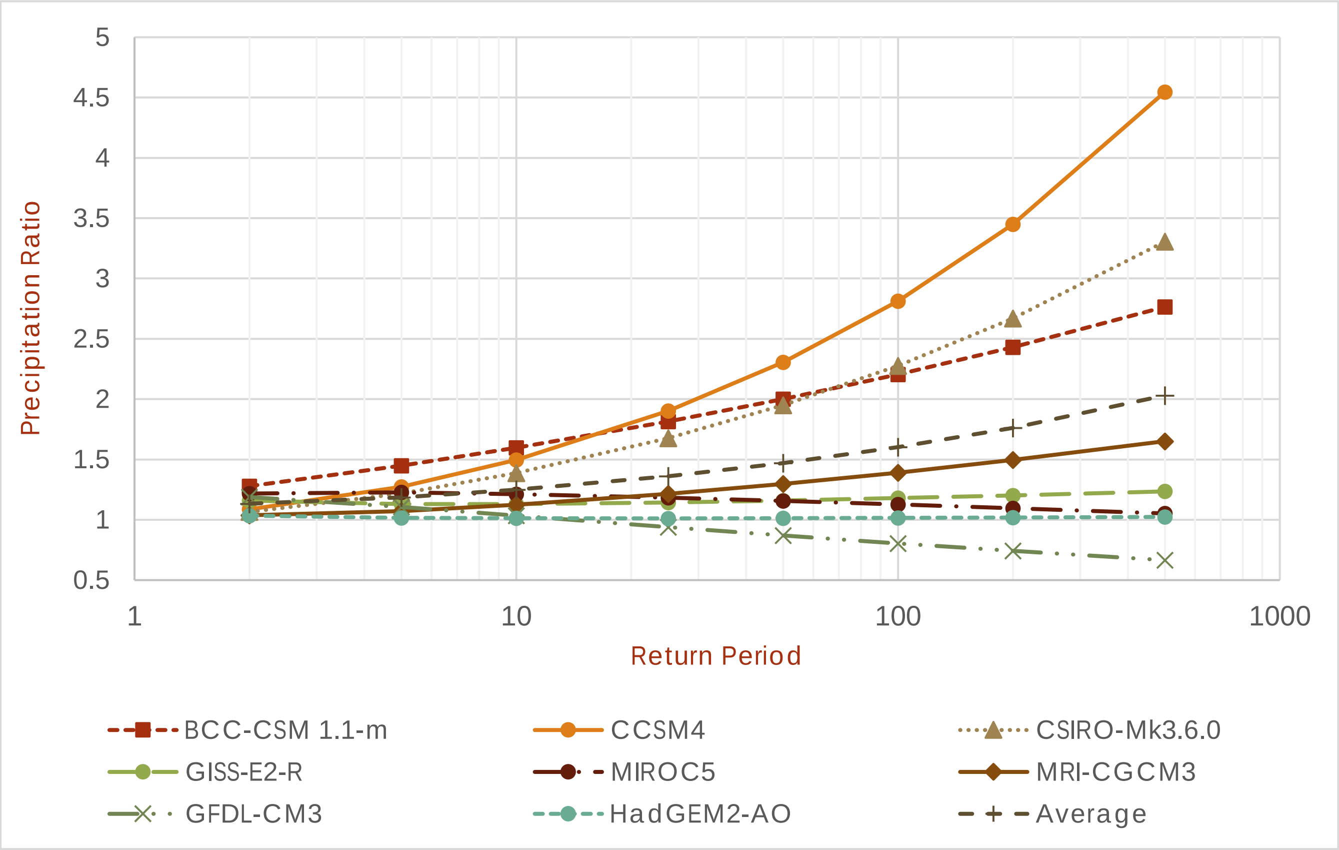

Figure A-6: Future (Year 2100) Precipitation Ratios for Selected Climate Models

Table A-4: Precipitation Depths – GCM Variability

Existing and Future Precipitation Depths (in)						
GCM	Annual Exceedance Probability					
	0.5	0.1	0.04	0.02	0.01	0.002
Existing	**3.16**	**4.57**	**5.60**	**6.53**	**7.63**	**10.98**
BCC-CSM 1.1-m	4.04	7.30	10.17	13.07	16.83	30.36
CCSM4	3.43	6.84	10.65	15.05	21.46	49.92
CSIRO-Mk3.6.0	3.37	6.34	9.39	12.73	17.37	36.30
GFDL-CM3	3.76	4.73	5.26	5.68	6.13	7.30
GISS-E2-R	3.66	5.17	6.41	7.57	9.00	13.56
HadGEM2-AO	3.27	4.63	5.66	6.61	7.74	11.25
MIROC5	3.84	5.55	6.63	7.55	8.60	11.56
MRI-CGCM3	3.28	5.14	6.81	8.47	10.61	18.13

Table A-5: Flows – GCM Variability

Existing and Future Flows (cfs)						
GCM	Annual Exceedance Probability					
	0.5	0.1	0.04	0.02	0.01	0.002
Existing	1,700	2,850	3,950	5,000	6,600	10,950
BCC-CSM 1.1-m	2,650	6,750	10,500	15,350	22,850	52,300
CCSM4	2,200	5,900	11,250	19,350	32,900	97,200
CSIRO-Mk3.6.0	2,100	5,300	9,400	14,700	24,000	65,950
GFDL-CM3	2,550	3,400	3,900	4,300	4,800	6,100
GISS-E2-R	2,350	3,700	5,300	6,850	8,650	15,800
HadGEM2-AO	2,050	3,250	4,350	5,400	6,800	11,700
MIROC5	2,600	4,300	5,500	6,600	7,950	12,150
MRI-CGCM3	2,000	3,650	5,950	8,050	10,900	25,050

Table A-6 lists ratios of future to existing discharge for each GCM for each return period, which range from less than 1.0 to 8.9. The relationship between precipitation and discharge is a function of the specific characteristics of the watershed, such as land use, soils, and available storage. For the Little Conewago Creek watershed, the increase in precipitation for more extreme events result in a future discharge ratio that is nearly double to precipitation ratio (i.e. a factor of 2.8 applied to the precipitation depth results in future flows that are approximately 5 times the existing flows).

Table A-6: Ratio of Future Flows to Existing Flows

Ratio of Future to Existing Flows						
GCM	**Annual Exceedance Probability**					
	0.5	**0.1**	**0.04**	**0.02**	**0.01**	**0.002**
BCC-CSM 1.1-m	1.6	2.4	2.7	3.1	3.5	4.8
CCSM4	1.3	2.1	2.8	3.9	5.0	8.9
CSIRO-Mk3.6.0	1.2	1.9	2.4	2.9	3.6	6.0
GFDL-CM3	1.5	1.2	1.0	0.9	0.7	0.6
GISS-E2-R	1.4	1.3	1.3	1.4	1.3	1.4
HadGEM2-AO	1.2	1.1	1.1	1.1	1.0	1.1
MIROC5	1.5	1.5	1.4	1.3	1.2	1.1
MRI-CGCM3	1.2	1.3	1.5	1.6	1.7	2.3

Future Land Use Changes

Information provided by the York County Planning Commission indicated that there is little planned development within the watershed. However, a golf course located within the watershed is rezoning 18 of their 36 holes to industrial. Based on aerial imagery, the area to be rezoned was estimated to be 0.27 mi^2. The assumed development area has B soils and a Developed – Open Space existing land use, resulting in a curve number of 61. Under future conditions, the land use was assumed to be Developed – High Intensity with a curve number of 92. The future conditions hydrologic model conservatively assumes that any proposed stormwater management would not affect peak flows on Little Conewago Creek at Baker Road. The curve number adjustment was incorporated into hydrologic analysis for future conditions.

Future Hydrology

Future flows for each GCM were computed using precipitation depths adjusted with the 0.1 AEP ratio for the following events (AEPs): 0.5, 0.1, 0.04, 0.02, 0.01, 0.002. Use of the 0.1 AEP ratio for the more extreme events was recommended in a Transportation Research Board Design Practices report, completed under NCHRP Project 15-61 (Kilgore and others, 2019), due to the current limitations of high-resolution datasets to represent precipitation extremes. For the purposes of the current study, the 0.1 AEP ratio was used for all evaluated events, including the 0.5 AEP event, for simplicity. Use of the actual ratios for more frequent events (less than 0.1 AEP) may be recommended in instances where such events are pertinent for design. Flows computed by applying the average ratio for the 0.1 AEP across GCMs (1.25) were used to develop resilient design strategies. Existing and future precipitation depths computed using the 0.1 AEP ratios for each GCM, as well as the final precipitation depths for resilient design, are shown in **Table A-7**. The range of ratios and precipitation depths in **Table A-7** shows that there is still significant variability between climate models, even when utilizing ratios from the less extreme 0.1 AEP event.

Table A-7: Precipitation Depths Computed Using 0.1 AEP Ratios

Existing and Future Precipitation Depths (in)							
GCM	0.1 AEP Ratio[a]	Annual Exceedance Probability					
		0.5	0.1	0.04	0.02	0.01	0.002
Existing	-	***3.16***	***4.57***	***5.60***	***6.53***	***7.63***	***10.98***
BCC-CSM 1.1-m	1.60	5.05	7.30	8.94	10.43	12.18	17.53
CCSM4	1.50	4.73	6.84	8.38	9.77	11.42	16.44
CSIRO-Mk3.6.0	1.39	4.39	6.34	7.77	9.07	10.59	15.24
GFDL-CM3	1.03	3.27	4.73	5.79	6.75	7.89	11.36
GISS-E2-R	1.13	3.57	5.17	6.33	7.39	8.63	12.42
HadGEM2-AO	1.01	3.20	4.63	5.67	6.61	7.73	11.12
MIROC5	1.21	3.83	5.55	6.80	7.92	9.26	13.32
MRI-CGCM3	1.12	3.55	5.14	6.30	7.34	8.58	12.34
Resilient Design Precipitation[b]	**1.25 (average)**	**3.95**	**5.71**	**7.00**	**8.16**	**9.54**	**13.72**

[a]Originally presented in Table A-3

[b]Computed using 1.25 times the existing precipitation depth (average 0.1 AEP ratio across GCMs)

Existing and future flows using the precipitation depths in **Table A-7** are summarized in **Table A-8**. Future flows range from approximately 1.1 to 2.4 times the existing discharge, with an average future to existing flow ratio of 1.5. Although the average precipitation ratio was 1.25, the average flow ratio is 1.5 implying the discharge increases twice as much as the precipitation.

Table A-8: Flows Computed Using 0.1 AEP Ratios

Existing and Future Flows (cfs)						
GCM	Annual Exceedance Probability					
	0.5	0.1	0.04	0.02	0.01	0.002
Existing	1,700	2,850	3,950	5,000	6,600	10,950
BCC-CSM 1.1-m	3,950	6,750	8,650	10,750	13,500	22,850
CCSM4	3,600	5,900	7,850	9,750	12,250	20,750
CSIRO-Mk3.6.0	3,200	5,300	7,000	8,700	10,900	18,500
GFDL-CM3	2,050	3,400	4,500	5,600	7,000	11,850
GISS-E2-R	2,350	3,700	5,150	6,400	8,000	13,600
HadGEM2-AO	2,000	3,250	4,350	5,400	6,800	11,500
MIROC5	2,600	4,300	5,700	7,100	8,900	15,100
MRI-CGCM3	2,350	3,650	5,100	6,350	7,950	13,450
Resilient Design Flows	**2,750**	**4,500**	**5,950**	**7,450**	**9,300**	**15,800**
Ratio of Resilient Design Flows to Existing	**1.6**	**1.6**	**1.5**	**1.5**	**1.4**	**1.4**

Future Hydraulic Model Results

The future flows for resilient design in **Table A-8** were run in HEC-RAS and results were compared to existing. **Table A-9** includes existing and future water surface elevations and velocities upstream of the Baker Road Bridge for the 0.1 AEP, roadway design event, and 0.01 AEP regulatory event. The HEC-RAS output table for all of the AEP events analyzed is in **Attachment A-3**. The 0.01 AEP floodplain map for existing and future conditions is also provided in **Attachment A-3**.

Table A-9: Existing and Future Hydraulic Results

	Parameter	Existing Condition	Future Condition	Difference
0.10 AEP Event	Discharge (cfs)	2,850	4,500	+1,650
	Upstream WSE (ft)	389.21	390.89	1.68
	Upstream Channel Velocity (fps)	4.8	5.2	+0.4
0.01 AEP Event	Discharge (cfs)	6,600	9,300	+2,700
	Upstream WSE (ft)	392.65	394.49	1.84
	Upstream Channel Velocity (fps)	5.3	5.4	+0.1

In the future condition, water surface elevations are projected to increase 1.68 feet for the 0.1 AEP event and 1.84 feet for the 0.01 AEP event. Channel velocities are projected to increase 0.4 ft/s for the 0.1 AEP event and 0.1 ft/s for the 0.01 AEP event. In the existing condition, the 0.1 AEP event overtops the left approach roadway. In the future condition, the 0.1 AEP event is projected to overtop the bridge deck and both approach roadways. The 0.01 AEP event overtops the bridge deck and both approach roadways in the existing condition; increased water surface elevations are projected to result in the bridge barrier being overtopped in the future condition. Existing and future results are compared further in the Resilient Design Options section.

Resilient Design Options

A resilient design checklist is recommended to assist in the evaluation of the need for various resilient design options for each site-specific analysis. The checklist consists of interdisciplinary parameters, including hydraulics, traffic, safety, and others, that are compared between existing and future conditions to determine if the site will be more vulnerable to issues such as scour, stability, and roadway overtopping. The parameters evaluated consist of the typical metrics used in hydraulic design of bridges and roadways. Future conditions are defined as a hydrologic conditions for 2100 for the RCP 8.5 scenario that modifies existing precipitation by applying the appropriate precipitation factor. For the purposes of this study, precipitation factors have been determined using site-specific climate information discussed in preceding sections.

Table A-10 includes existing and future site and hydraulic characteristics for determining resilient design needs for the Baker Road project area. Scour calculations are included in **Attachment A-4**. The "Potential for Resilient Design" column was completed using the following codes to indicate level of potential for resilient design considerations:

- **Low:** minor or no special designs for resiliency anticipated
- **Medium:** considerations for resiliency related designs may be beneficial
- **High:** consideration for resiliency design modifications is highly recommended

Because every site is different, establishing ranges or thresholds for checklist parameters is not practical. When determining the level of potential for resilient design, engineering judgment, interdisciplinary coordination, and knowledge of existing site conditions should be used to evaluate the following:

- Risk posed to physical infrastructure and/or the traveling public
- Comparison of existing and future parameters
- Magnitudes of future condition parameters and their effects on infrastructure performance

Where future changes in hydraulic conditions indicate the potential need for resilient design, the following are possible design alternatives that should be considered. This list is not all-inclusive, and any design recommendations should be determined using an interdisciplinary approach.

- Increased bridge velocity, scour potential, propensity for pressure flow → adjustments to scour protection, foundations, or structure hydraulic opening
- Increased frequency, velocity, or depth of overtopping flow → additional embankment protection or adjustments to pavement design, changes to structure anchoring
- New or increased effects on adjacent roadways → advise DOT of changes to serviceability and/or stability of adjacent roadway
- No current structure overtopping and no low chord inundation in existing, but impacts to beams/barrier in future condition → evaluate possible change to bridge or beam type to consider inundation
- Existing conditions impacts beams/barrier and future condition results in increased impacts or barrier overtopping → possible superstructure design adjustments

Table A-10: Resilient Design Checklist

	Parameter	Existing Condition[a]	Future Condition	Indicates Potential for Resilient Design
Site Data	**Hydrology Method**	HEC-HMS	HEC-HMS using 1.25*P[b,c]	N/A
	Embankment Instability	No issues noted	Minimal risk	Low
	Overtopping Frequency	0.5 AEP	0.5 AEP	Low
	Design Event Frequency	0.10 AEP		N/A
	Provides Access to Critical Services	N/A – short detour routes easily available	N/A – short detour routes easily available	Low
Design Event	**Discharge (cfs)**	2,850	4,500	N/A
	% Q Bridge	56	34	Low
	Pressure Flow	Yes	Yes	Low
	Bridge Velocity (fps)	5.6	5.5	Low
	Overtopping Velocity (fps)	2.7	3.2	Medium
	Overtopping Depth (Roadway) (ft)	2.6	4.3	Medium
	Overtopping Depth (Structure) (ft)	N/A	Below top of barrier	Medium
	Adjacent Roadway(s) Impacted	No	Yes	Medium
0.01 AEP Event	**Discharge (cfs)**	6,600	9,300	N/A
	% Q Bridge	23	17	Low
	Pressure Flow	Yes	Yes	Low
	Bridge Velocity (fps)	5.5	5.6	Low
	Scour Depth (ft)	2.9 ft	3.1 ft	Low
	Riprap Size	R-7	R-7	Low
	Overtopping Velocity (fps)	3.6 fps	3.9 fps	Medium
	Overtopping Depth (Roadway) (ft)	6 ft	7.9 ft	Medium
	Overtopping Depth (Structure) (ft)	Less than 0.2 ft	Up to 2 ft over barrier	High
	Adjacent Roadway(s) Affected	Yes	Yes	Medium

[a]Utilizes the geometry for the proposed bridge replacement project for consistent comparison to future conditions
[b]1.25 is the average 0.1 AEP ratio computed from 8 GCMs for the year 2100 and was selected for use in resilient design for the pilot study
[c]Future condition also includes projected land use changes within the watershed

Velocity Distributions

In addition to hydraulic parameters obtained from typical HEC-RAS output tables, velocity distribution plots for existing and future conditions were compared to determine potential areas of isolated velocity increases. Comparison plots for the 0.01 AEP event at the cross section immediately upstream of the Baker Road Bridge are shown in **Figure A-7**. Velocity distribution plots are a valuable tool for pinpointing areas of greater increase. For projects where site conditions warrant a 2-dimensional hydraulic model, more refined flow distribution and velocity output may provide further insight. Higher velocity increases could indicate locations within the bridge or along the roadway embankment that would benefit from resilient design improvements.

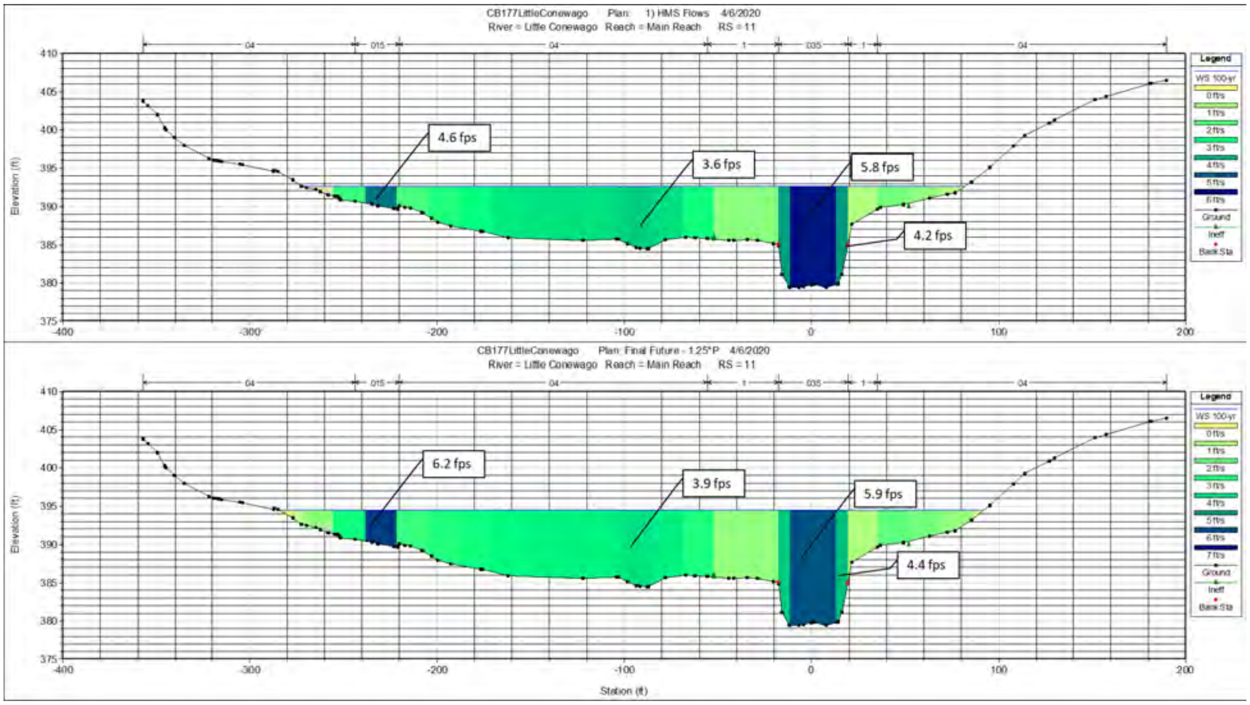

Figure A-7: Existing (top) and Future (bottom) 0.01 AEP velocity distribution plots

The overtopping event for Baker Road over Little Conewago Creek is the 0.5 AEP event for both existing and future conditions. However, under future conditions overtopping depths are projected to increase. Due to the significant amount of weir flow present with the low-lying left approach roadway, the velocities through the bridge are not high and bridge scour is not a significant concern at this site. The existing bridge does not have a history of scour concerns and no evidence of structure scour was observed during the site visit. Because the roadway is essentially at-grade, embankment scour and stability are not likely to be major concerns under future conditions where overtopping frequency and depths may increase. Additionally, Hadley Drive in the upstream left overbank is likely to see increased flooding depths, frequency, and velocities under future conditions and alternative pavement designs may be considered by the facility owner to avoid pavement stability concerns in the future.

Structure Size Sensitivity Analysis

Because of the significant roadway overtopping at this site, which will only be exacerbated under future conditions, it may initially seem appropriate to explore design options to reduce overtopping depth and frequency. However, at this site, due to the perched nature of the structure, resilient design alternatives that include increasing the structure size or raising the roadway profile are unlikely to mitigate overtopping concerns. To demonstrate how changes to the structure and roadway will affect hydraulic parameters such as velocity and water surface elevations, a sensitivity analysis was performed for the site that explored two alternatives:

1. An approximately 25% increase in the proposed bridge span, and
2. An approximately 90% increase in the proposed Bridge span with left bank grading through bridge to further increase hydraulic opening.

Both Alternatives 1 and 2 include raising the roadway profile by approximately 2 feet and raising the low chords while assuming a similar superstructure depth. A selection of pertinent hydraulic results from the sensitivity analysis are provided in **Table A-11**. Results indicate that even a significantly larger span will only minimally improve overtopping depths, will result in increased bridge velocities and greater scour depths, and cause nearly 0.5 foot increase to the upstream water surface elevation. Increases in water surface elevation for these scenarios are anticipated because only 17% of the 0.01 AEP flow goes through the bridge when future flows are paired with the proposed bridge, with the remaining flow being conveyed unobstructed in the overbanks and over the approach roadways. For the future flows with the larger bridges (Alternates 1 and 2), 31 to 39% of the flow is conveyed through the bridge, with less of the total flow being conveyed unobstructed over the approach roadways.

Table A-11: Resilient Design Checklist for the 0.01 AEP Event

0.01 AEP Event	Parameter	Existing Condition	Future Condition (No Resilient Design)	Future Condition (Resilient Design Alt 1)	Future Condition (Resilient Design Alt 2)
	Discharge (cfs)	6,600	9,300	9,300	9,300
	Bridge Span (ft)	47.8	47.8	60	90
	Bridge Hydraulic Open Area (ft^2)	293	193	422	549
	% Q Bridge	23	17	31%	39%
	Bridge Velocity (fps)	5.5	5.6	7.4	7.2
	Scour Depth (ft)	2.9	3.1	6.2	6.2
	Riprap Size	R-7	R-7	R-7	R-7
	Upstream WSE (ft)	392.65	394.49	394.96	394.89
	Overtopping Depth (Roadway) (ft)	6 ft	7.9 ft	6.3 ft	6.3 ft

Sensitivity analysis is a powerful tool for evaluating the hydraulic effects of estimated changes to the proposed design without the significant effort required to develop detailed resilient design options for a proposed bridge and roadway. In this case, a conceptual analysis was able to determine that the significant changes to the roadway profile to accommodate a larger structure are not hydraulically feasible and are not effective for resilient design at this site.

Bridge Design for Highly Inundated Superstructures

In the existing condition, the Baker Road bridge deck begins to overtop for flood events greater than the 0.10 AEP event. For the future flow scenario, deck overtopping begins for the 0.50 AEP event, with full deck submergence for the 0.10 AEP event. For the future 0.02 AEP event, the bridge barrier is fully inundated. As these future flow scenarios indicate more frequent inundation of the bridge, an evaluation of several cases of deck and bridge barrier submergence was performed to determine if the force of water in these highly submerged cases could result in changes to a typical bridge design:

Case 1: WSE approximately 3 inches above Low Chord
Case 2: WSE up to Top of Beam
Case 3: WSE overtopping Deck
Case 4: WSE to Top of Barrier
Case 5: WSE approximately 1.5 feet above Barrier

These case studies were performed for several sample bridges (adjacent box beams) with spans of approximately 50 feet, 72 feet, and 100 feet. The results of the study determined that typical bridge designs may be insufficient when the beams begin to uplift off their bearings. This can occur because the concrete beams displace water (buoyancy), the water flowing under the angled superstructure lifts it up, and water impacting the side tends to try and overturn it. Dowels, which connect the bridge superstructure to the substructure (abutments), resist the overturning by pulling up on the dowels at the upstream end and pushing down on those at the downstream end. Note that entrapped air was not considered for this study as it was considered highly unlikely that a typically constructed bridge would be airtight and that flow would fully inundate the superstructure.

For all three span lengths evaluated, uplift began to occur between Cases 2 and 3. For the 72-foot and 100-foot spans, once WSEs were between Cases 3 and 5, the uplift at the fixed end of the beam would exceed the dowel's capacity to hold it down, and beams could lift off the bearings on both ends. Bearings are designed to be centered under the beams; if the uplift allowed them to shift to one side, the beams would see unequal support, which would cause cracking over time or result in other serviceability issues.

If a submerged bridge is evaluated and determined to be insufficient for uplift and overturning, the following resilient design options could be considered:

- Mechanically restrain bearings, such as with bolted steel plates
- Extend smooth dowel bars so that they can better develop in the concrete, or replace smooth dowels with deformed bars

For the Baker Road bridge, it is recommended that the increased force of water on the bridge in the future condition be evaluated in more detail to determine if the above resilient design options may be needed. In general, considering that bridge superstructure submergence could become more common or occur more frequently under future climate scenarios, additional research is needed to determine standard factors and design methodologies for determining the force of water as a component of the typical bridge design procedure. A brief, initial review of AASHTO Guide Specifications for Bridges Vulnerable to Coastal Storms was performed for this study. Applicability to inland waterways appears to be limited as the procedures used are dependent on wave height and wave period; additional research could be performed outside of the current study to evaluate the application for design.

Stream Stability

In addition to the quantitative assessment of the resilient design parameters discussed throughout this section, engineering judgment should be used when evaluating existing concerns and their potential to worsen as a result of predicted changes in precipitation and discharge. For example, the potential for lateral and vertical changes to the waterway in the vicinity of the bridge or roadway being studied should be considered. If a site visit indicates existing concerns due to channel migration, future changes are likely to exacerbate the problem, and resilient design is highly recommended. For the Baker Road Bridge, no issues with lateral or vertical stream migration were noted. More research is needed to support predictions about stream changes and stream stability in the future in cases where existing issues are not present.

Economic Analysis

Preliminary cost information for mechanically restraining bearings was estimated based on discussions with a prestressed beam supplier. The beam could be restrained with a sole plate assembly, which would increase the cost by approximately $3,700 per beam. This preliminary evaluation considers order-of-magnitude material costs and does not include engineering costs or other site-specific considerations.

- Total costs for mechanically restraining bearings using this method for a short to medium span spread box beam bridge with a 5-beam cross section would be approximately $20,000.
- Total costs of similar retrofits to a similarly-sized adjacent box beam bridge would be $40,000-$70,000. Note that adjustments would be required for the adjacent box beam sole plate assembly due to the lack of space between beams.

Assuming a rough total bridge cost of about $1,000,000, mechanically restraining bearings would increase the cost by approximately 2-7%.

Conclusion

The existing Baker Road Bridge over Little Conewago Creek is characterized by frequent flooding of the low-lying left approach roadway. The results of this study indicate that the frequency of roadway overtopping, as well as the frequency of inundation of the bridge superstructure, will increase under future climate scenarios. As a result, this study recommends evaluating the force of the flow from Little Conewago Creek on the bridge to determine if mechanically restraining the bridge bearings and extending or adjusting the design of dowel bars may be needed to increase resiliency of the proposed bridge. Some of the procedures used to perform the evaluation of inundated bridges for this study should be incorporated into future research to determine standard design methodologies for typical bridge replacement projects where overtopping of the bridge deck or barrier occurs.

References

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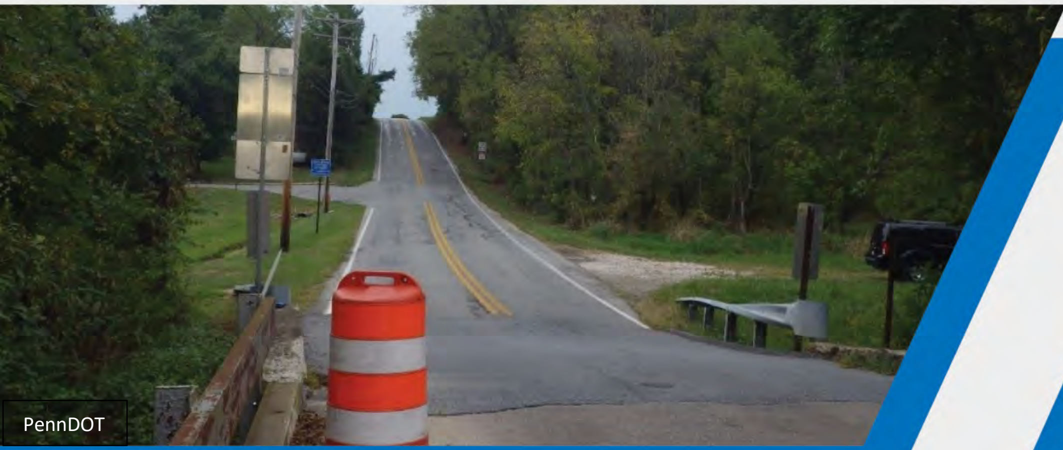

APPENDIX B

Allegheny County (Baldwin Road) Pilot Analysis

SEPTEMBER 2020

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Introduction

The hydrologic and hydraulic (H&H) analysis for SR 2046 along Streets Run in Allegheny County was performed as a part of PennDOT's Pilot Study for Resilience and Durability to Extreme Weather. The project involves computing existing and future peak flows using physically based hydrologic methods and developing a one-dimensional hydraulic model that considers existing and future conditions for three sites in Pennsylvania. The Allegheny County site was selected because of the history of lateral flooding along SR 2046 as well as landslide susceptibility in a varied, relatively developed watershed. Allegheny County is located in southwestern Pennsylvania and contains the city of Pittsburgh.

Site Description

SR 2046 along Streets Run is located in the southeast portion of the City of Pittsburgh, to the east of the Borough of Baldwin in Allegheny County, Pennsylvania. Its location on the USGS quadrangle map entitled Pittsburgh East, PA is approximately 40° 22' 52" N latitude and 79° 55' 59" W longitude. The project location is shown in **Figure B-1**.

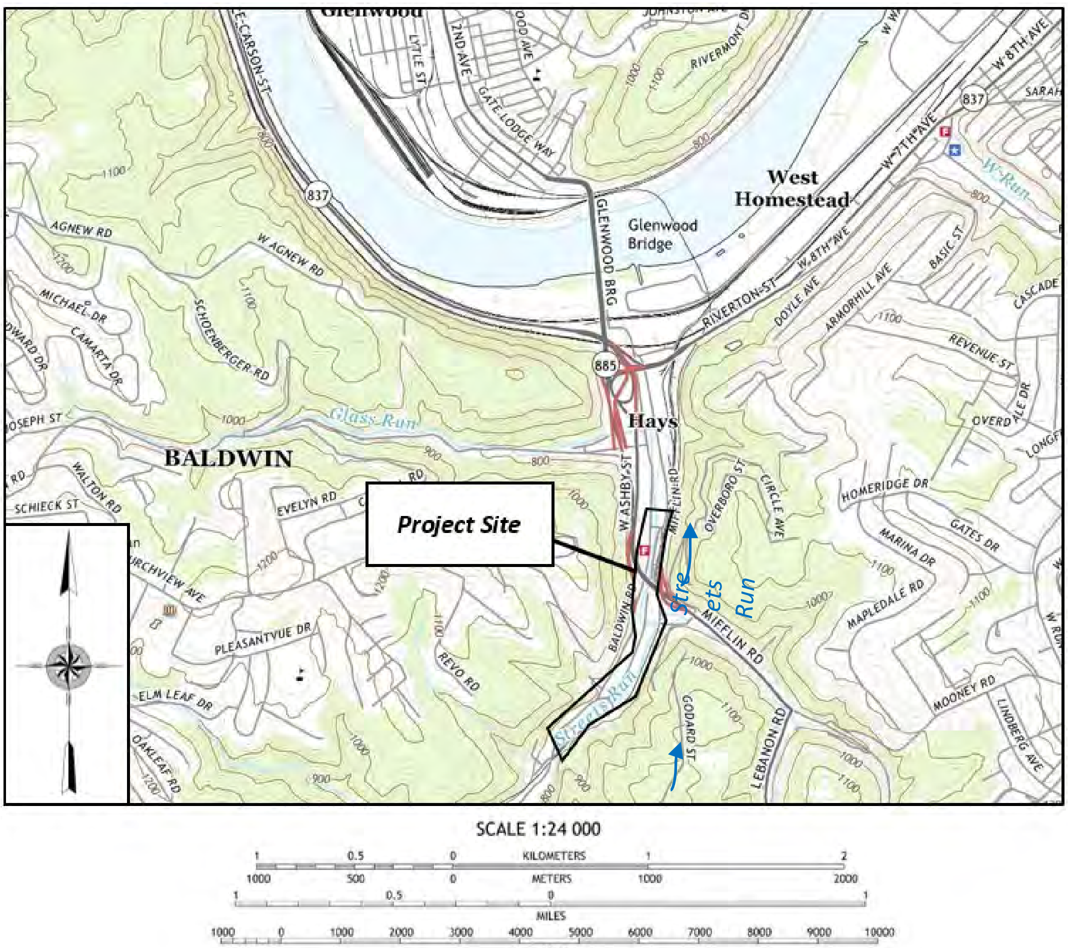

Figure B-1: Project Location

SR 2046 (Baldwin Road) is a minor arterial roadway that connects the southern Pittsburgh suburbs to the City of Pittsburgh. Per PennDOT DM-2 Chapter 10.6.E the design event for the roadway is the 0.04 AEP event. Note that a portion of Baldwin Road, to the north of the SR 885 underpass, is a local roadway with a 0.1 AEP design event. For the purposes of this study, results will be compared for the 0.04 AEP design event throughout the reach. The study reach is located in a watershed that has steep, forested areas as well as significant commercial and residential development. There is a history of flooding in the area, particularly from lateral flow from the left banks of Streets Run onto SR 2046. Photographs of the roadway and stream channel are in **Figure B-2** and **Figure B-3**. Additional site photographs are included in **Attachment B-1**.

Figure B-2: Looking downstream along Streets Run with SR 2046 beyond the left banks. PennDOT photo

Figure B-3: Looking downstream along Streets Run from Corley Street Bridge. PennDOT photo

Existing Roadway and Structures

Streets Run flows parallel to SR 2046 along much of the approximately 5,000-foot study reach. For approximately 3,500 feet, Streets Run flows directly adjacent to the roadway. For the remaining 1,500 feet, Streets Run flows parallel but approximately 300 feet to the east along the railroad embankment, before making a sharp bend back towards SR 2046. Throughout the study reach, there are also five existing bridges. The characteristics of the existing structures are shown in **Table B-1.**

Table B-1: Existing Structures along Streets Run Study Reach

Crossing Name	Structure Type	Clear Span (feet)	Piers	Out-to-Out Structure Width (feet)	Average Underclearance (feet)	Barrier Type/ Average Height
Railroad Bridge (adjacent to Baldwin Road)	Concrete Bridge	37.2	1	53	4.5	None
Corley Street Bridge	Concrete Bridge	33.3	0	25	6.7	Concrete/4’
Calera Street Bridge	Steel Stringer Bridge	28	0	27	5.0	Concrete/4’
Ramp Street	Single Span Box Beam Bridge	30.6	0	36	5.7	Concrete/3’
Footbridge (immediately upstream of 300 Mifflin Road)	Concrete Bridge	28	0	5	6.3	Metal Railing/3’

The existing structures along the Streets Run study reach will be incorporated into the hydraulic analysis for the resiliency study.

Watershed Characteristics

The drainage area at SR 2046 along Streets Run is approximately 8.1 mi^2 as delineated in Watershed Modeling System (WMS) 11.0 (Aquaveo, 2019) using a 10-meter Digital Elevation Model (DEM) and as shown in **Figure B-4**. SSURGO soils, obtained from NRCS, and 2011 National Land Cover Database (USGS, 2011) land use data were used to compute curve numbers. The basin generally consists of residential and commercial land uses with some steep, forested areas concentrated along Streets Run and its tributaries. The basin contains no carbonate area.

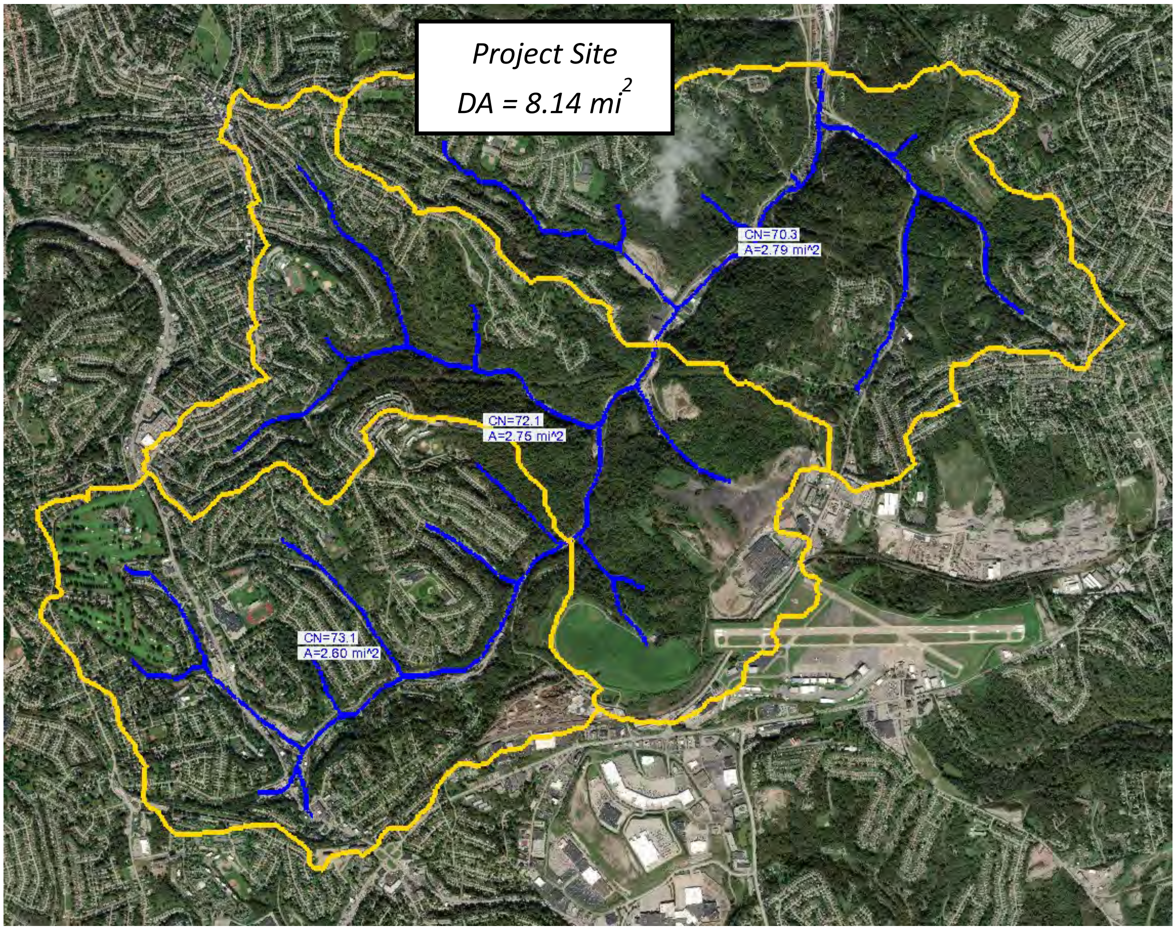

Figure B-4: Streets Run Watershed

Hydrologic Method

Peak flows for the current study were computed using the Army Corps of Engineers' Hydrologic Modeling System (HEC-HMS) 4.2. HEC-HMS is applicable for basins of almost any size and complexity. The subbasins were delineated and geometric parameters for each subbasin were calculated using WMS. At all subbasins, the curve number method was used as the loss rate method. The time of concentration (T_c) was calculated in the WMS program using the NRCS segmental method. The Muskingum-Cunge routing method was used for all reaches and was calculated using a trapezoidal channel with bottom widths of 25-40 feet, floodplain widths of 125-240 feet, side slopes of 0.5H:1V, channel Manning's n of 0.06, and overbank Manning's n of 0.2. Routing parameters were estimated using aerial imagery and topographic data. The subbasin data was then exported from WMS to HMS. The Lag Time (T_L) for each subbasin was manually adjusted to be 0.6* T_c. All events were modeled using the NRCS Type A storm distribution that provided precipitation depths from 5 minutes to 24 hours, with existing precipitation depths determined using the PDT-IDF curves from PennDOT Publication 584 Appendix 7A, which are based on data from NOAA Atlas 14, Volume 2. Existing 24-hour precipitation depths are shown in **Table B-2**.

Table B-2: Creek Existing Precipitation Depths

Annual Exceedance Probability	0.5	0.1	0.04	0.02	0.01	0.002
Precipitation Depth (in)	2.44	3.44	4.09	4.65	5.24	6.74

Existing Flows and Validation

Existing flows computed using HEC-HMS were validated using local flood history and compared to other hydrologic methods as shown in **Table B-3**. Comparison flows for Streets Run along SR 2046 were computed using two USGS regression methods: USGS WRIR 2000-4189 (Stuckey and Reed, 2000) and USGS SIR 2008-5102 (Roland and Stuckey, 2008). Flows were compared to the flows published as part of a 1990 DEP Feasibility Study, which were computed using PSU-IV methodology. The HEC-HMS flows computed for the current study are the most conservative flows for all of the events. The USGS 4189 results are similar to the DEP flows, producing flows that are less conservative than the HEC-HMS flows. The USGS 5102 results are the least conservative of all of the considered flows, by a significant amount. The HEC-HMS flows produce hydraulic results that are consistent with reports of frequent flooding of Baldwin Road and Streets Run Road obtained from local news sources and local residents. Excerpts from news reports citing road closures and depicting flooding from several events in 2017 and 2019 are provided in **Attachment A-2**. Since the HEC-HMS flows are consistent with historical flooding and are the most conservative of the flows, the HEC-HMS flows are considered validated for use in the hydraulic analysis for the current study.

Table B-3: Existing Flow Comparison

Method	Peak Flows (cfs)					
	0.5	0.1	0.04	0.02	0.01	0.002
HEC-HMS	1,000	2,450	3,400	4,000	4,800	7,800
DEP (PSU-IV)	-	1,950	2,500	3,000	3,550	5,500
USGS 5102	400	900	-	1,600	1,950	2,950
USGS 4189	-	2,150	2,750	3,250	3,750	5,150

Existing Hydraulic Performance

A hydraulic model for the existing structures was developed and run in HEC-RAS 5.0.7. Topographic information consists of site-specific field survey data in the channel and immediate overbank areas. The hydraulic model results indicate that the existing structures have frequent and significant overtopping, particularly laterally from the left banks onto SR 2046. This result is validated by documented flooding history at the site. As shown in **Figure B-5** and **Figure B-6**, photographs from the heavy precipitation event that occurred on July 6, 2019 show lateral overtopping along the left banks of Streets Run, approximately 400 feet upstream of the Calera Street Bridge crossing. According to daily precipitation data for the nearby Allegheny County

Airport from the Pennsylvania State Climatologist website, there were 1.32 inches of rain on July 6, 2019. This precipitation amount for a 24-hour duration indicates the storm was smaller than a 0.5 AEP event. While it is likely that this precipitation occurred over a shorter duration than 24 hours, which would result in a more extreme AEP event estimate (larger storm), duration information for the storm was not available. In general, design flow validation with specific storm events is limited by the lack of availability of hourly precipitation data, particularly for localized events. For example, if 1.32 inches of rainfall occurred over 1 hour, this would correlate with a 0.2 AEP event. It is also possible that the available data does not capture localized precipitation depths within the Streets Run watershed. While an exact return period is difficult to determine for this event, the results of the hydraulic model indicate Streets Run may flow out-of-bank for events as frequent as the 0.5 AEP event, which is consistent with the reports of significant flooding for this event and other recent events in the watershed.

Figure B-5: July 2019 Flooding (Left Overbanks)

Photo Source: Eileen and Frank Halsaver, 609 Calera Street

Figure B-6: July 2019 Flooding (Left Overbanks)

Photo Source: Eileen and Frank Halsaver, 609 Calera Street

Global Climate Model Summary

The Global Climate Model (GCM) evaluation for the current study looked at eight distinct GCMs for Representative Concentration Pathway (RCP) 8.5, which predicts the highest future CO_2 equivalent of the various RCPs adopted by the International Panel on Climate Change (IPCC). RCP 8.5 was the trajectory adopted in the PA Climate Impacts Assessment Update (Shortle and others, 2015). The GCMs selected for this study were determined using guidance from the Infrastructure and Climate Network (ICNET) and Transportation Research Board reports developed for the NCHRP Project 15-61 (Kilgore and others, 2019a; 2019b). Future (year 2100) 24-hour precipitation data for the eight GCMs were compared against GCM estimates of historical precipitation to develop ratios for various annual exceedance probabilities (AEPs). The calculated ratios for the Streets Run watershed are shown in **Table B-4** and compared in **Figure B-7**. Precipitation depths and flows computed using the ratios for each AEP across all GCMs are shown in **Table B-5** and **Table B-6**, respectively. Flows computed using the future precipitation depths are in **Table B-6**. The calculated ratios, precipitation depths, and flows for each GCM are provided to demonstrate the significant variability across the selected climate models and are not recommended for design purposes.

Table B-4: Future Precipitation Ratios

Precipitation Ratios						
GCM	**Annual Exceedance Probability**					
	0.5	**0.1**	**0.04**	**0.02**	**0.01**	**0.002**
BCC-CSM 1.1-m	1.31	**1.37**	1.36	1.35	1.32	1.26
CCSM4	1.19	**1.18**	1.17	1.17	1.17	1.16
CSIRO-Mk3.6.0	1.23	**1.27**	1.29	1.30	1.31	1.33
GFDL-CM3	1.25	**1.18**	1.13	1.09	1.05	0.97
GISS-E2-R	1.21	**1.25**	1.26	1.26	1.26	1.26
HadGEM2-AO	1.06	**1.11**	1.14	1.16	1.19	1.25
MIROC5	1.43	**1.51**	1.51	1.50	1.49	1.44
MRI-CGCM3	1.18	**1.08**	1.01	0.96	0.92	0.81
Average Ratios	**1.23**	**1.24**	**1.23**	**1.22**	**1.21**	**1.18**

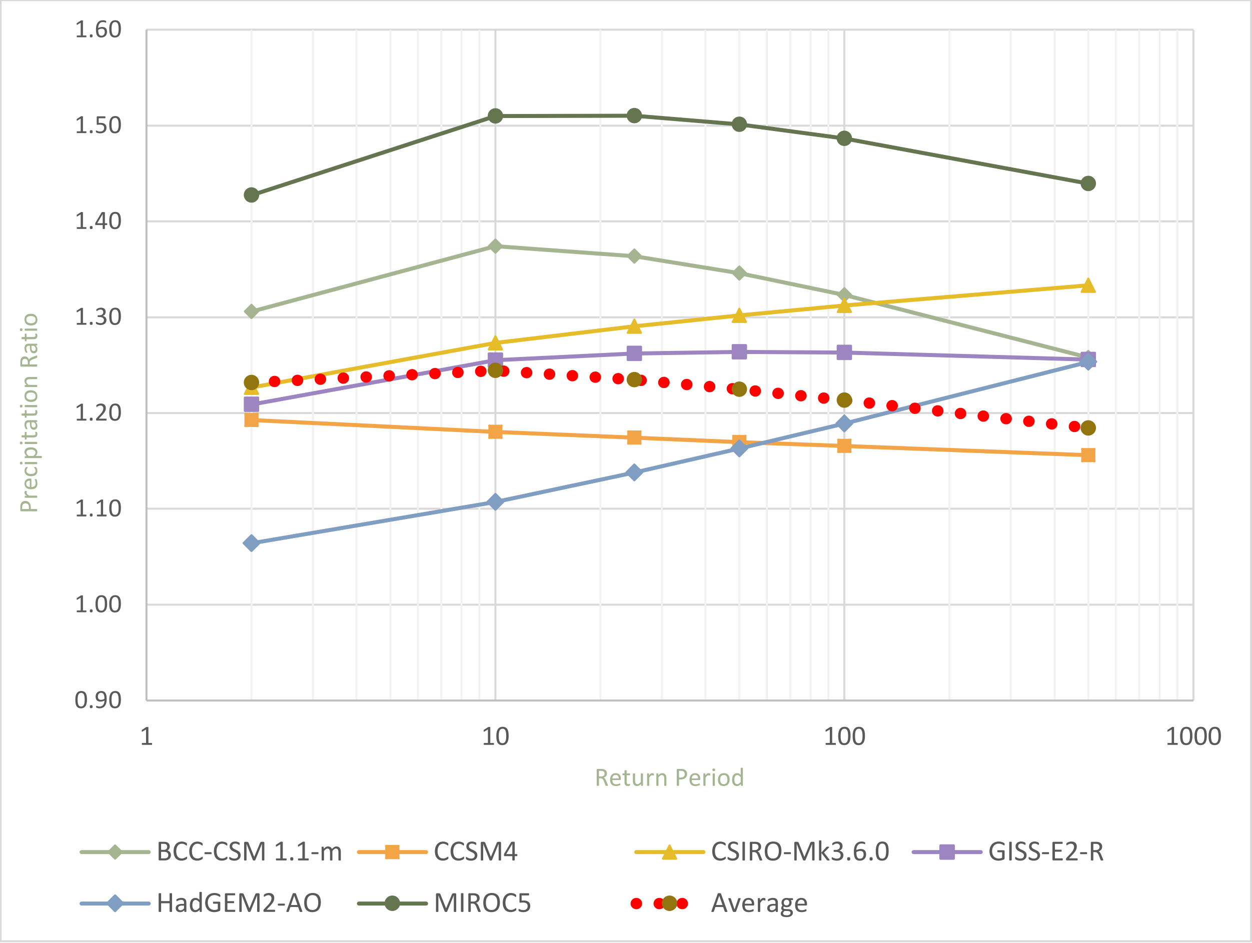

Figure B-7: Future (Year 2100) Precipitation Ratios for Selected Climate Models

Table B-5: Precipitation Depths – GCM Variability

Existing and Future Precipitation Depths (in)						
GCM	Annual Exceedance Probability					
	0.5	0.1	0.04	0.02	0.01	0.002
Existing	**2.44**	**3.44**	**4.09**	**4.65**	**5.24**	**6.74**
BCC-CSM 1.1-m	3.19	4.73	5.58	6.26	6.93	8.48
CCSM4	2.91	4.06	4.80	5.44	6.11	7.79
CSIRO-Mk3.6.0	2.99	4.38	5.28	6.05	6.88	8.99
GFDL-CM3	3.04	4.05	4.61	5.07	5.51	6.51
GISS-E2-R	2.95	4.32	5.16	5.88	6.62	8.46
HadGEM2-AO	2.60	3.81	4.65	5.41	6.23	8.45
MIROC5	3.48	5.19	6.18	6.98	7.79	9.70
MRI-CGCM3	2.88	3.71	4.14	4.48	4.80	5.48

Table B-6: Flows – GCM Variability

Existing and Future Flows (cfs)						
GCM	Annual Exceedance Probability					
	0.5	0.1	0.04	0.02	0.01	0.002
Existing	1,000	2,450	3,400	4,000	4,800	7,800
BCC-CSM 1.1-m	1,950	4,200	5,600	6,850	8,150	11,400
CCSM4	1,600	3,200	4,350	5,400	6,550	9,900
CSIRO-Mk3.6.0	1,700	3,650	5,100	6,450	8,050	12,550
GFDL-CM3	1,750	3,150	4,050	4,750	5,500	7,300
GISS-E2-R	1,650	3,550	4,900	6,150	7,550	11,350
HadGEM2-AO	1,200	2,800	4,100	5,350	6,800	11,300
MIROC5	2,350	4,950	6,700	8,250	9,900	14,250
MRI-CGCM3	1,550	2,650	3,300	3,800	4,300	5,450

Table B-7 lists ratios of future to existing discharge for each GCM for each annual exceedance probability, which range from less than 1.0 to 2.4. The relationship between precipitation and discharge is a function of the specific characteristics of the watershed, such as land use, soils, and available storage. For the Streets Run watershed, the increase in precipitation for more extreme events results in a future discharge ratio that is slightly larger than the precipitation ratio (i.e. a factor of 1.5 applied to the precipitation depth results in future flows that are approximately 2 times the existing flows).

Table B-7: Ratio of Future Flows to Existing Flows

Ratio of Future to Existing Flows						
GCM	Annual Exceedance Probability					
	0.5	0.1	0.04	0.02	0.01	0.002
BCC-CSM 1.1-m	2.0	1.7	1.6	1.7	1.7	1.5
CCSM4	1.6	1.3	1.3	1.4	1.4	1.3
CSIRO-Mk3.6.0	1.7	1.5	1.5	1.6	1.7	1.6
GFDL-CM3	1.8	1.3	1.2	1.2	1.1	0.9
GISS-E2-R	1.7	1.4	1.4	1.5	1.6	1.5
HadGEM2-AO	1.2	1.1	1.2	1.3	1.4	1.4
MIROC5	2.4	2.0	2.0	2.1	2.1	1.8
MRI-CGCM3	1.6	1.1	1.0	1.0	0.9	0.7

Future Land Use Changes

Information provided by the Southwest Pennsylvania Commission indicated that there is little anticipated development within the already significantly developed watershed, with a conservative possible estimate of 10% shift in developed and undeveloped land. The undeveloped areas within the watershed are likely to remain undeveloped since they are generally forested and steep with high risk for landslides. Since the likelihood and magnitude of land use change are fairly low and the existing conditions are quite developed, the future conditions hydrology model assumes that the increase in precipitation will govern the future hydrology flows rather than any changes to land use.

Future Hydrology

Future flows for each GCM were computed using precipitation depths adjusted with the 0.1 AEP ratio for the following events (AEPs): 0.5, 0.1, 0.04, 0.02, 0.01, 0.002. Use of the 0.1 AEP ratio for the more extreme events was recommended in a Transportation Research Board Design Practices report, completed under NCHRP Project 15-61 (Kilgore and others, 2019), due to the current limitations of high-resolution datasets to represent precipitation extremes. For the purposes of the current study, the 0.1 AEP ratio was used for all evaluated events, including the 0.5 AEP event, for simplicity. Use of the actual ratios for more frequent events (less than 0.1 AEP) may be recommended in instances where such events are pertinent for design. Flows computed by applying the average ratio for the 0.1 AEP across GCMs (1.24) were used to develop resilient design strategies. Existing and future precipitation depths computed using the 0.1 AEP ratios for each GCM, as well as the final precipitation depths for resilient design, are shown in **Table B-8**. The range of ratios and precipitation depths in **Table B-8** shows that there is still significant variability between climate models, even when utilizing ratios from the less extreme 0.1 AEP event.

Table B-8: Precipitation Depths Computed Using 0.1 AEP Ratios

Existing and Future Precipitation Depths (in)								
GCM	0.1 AEP Ratio[a]	Annual Exceedance Probability					
		0.5	0.1	0.04	0.02	0.01	0.002
Existing	**-**	***2.44***	***3.44***	***4.09***	***4.65***	***5.24***	***6.74***
BCC-CSM 1.1-m	1.37	3.35	4.73	5.62	6.39	7.20	9.26
CCSM4	1.18	2.88	4.06	4.83	5.49	6.18	7.95
CSIRO-Mk3.6.0	1.27	3.11	4.38	5.21	5.92	6.67	8.58
GFDL-CM3	1.18	2.87	4.05	4.82	5.48	6.17	7.94
GISS-E2-R	1.25	3.06	4.32	5.13	5.84	6.58	8.46
HadGEM2-AO	1.11	2.70	3.81	4.53	5.15	5.80	7.46
MIROC5	1.51	3.68	5.19	6.18	7.02	7.91	10.18
MRI-CGCM3	1.08	2.63	3.71	4.41	5.01	5.64	7.26
Resilient Design Precipitation[b]	**1.24 (average)**	**3.04**	**4.28**	**5.09**	**5.79**	**6.52**	**8.39**

[a]Originally presented in Table 4

[b]Computed using 1.24 times the existing precipitation depth (average 0.1 AEP ratio across GCMs)

Existing and future flows using the precipitation depths in **Table B-8** are summarized in **Table B-9**. Future flows range from approximately 1.1 to 2.7 times the existing discharge, with an average future to existing flow ratio of 1.5. Although the average precipitation ratio was 1.24, the average flow ratio is 1.5 implying the discharge increases twice as much as precipitation.

Table B-9: Flows Computed Using 0.1 AEP Ratios

Existing and Future Flows (cfs)						
GCM	Annual Exceedance Probability					
	0.5	0.1	0.04	0.02	0.01	0.002
Existing	1,000	2,450	3,400	4,000	4,800	7,800
BCC-CSM 1.1-m	2,200	4,200	5,700	7,100	8,650	13,200
CCSM4	1,550	3,200	4,350	5,450	6,700	10,250
CSIRO-Mk3.6.0	1,850	3,650	5,000	6,250	7,650	11,600
GFDL-CM3	1,550	3,150	4,350	5,450	6,700	10,200
GISS-E2-R	1,800	3,550	4,850	6,100	7,450	11,350
HadGEM2-AO	1,350	2,800	3,900	4,900	6,000	9,200
MIROC5	2,650	4,950	6,700	8,300	10,150	15,450
MRI-CGCM3	1,250	2,650	3,700	4,650	5,750	8,800
Resilient Design Flows	**1,750**	**3,500**	**4,800**	**6,000**	**7,350**	**11,200**
Ratio of Resilient Design Flows to Existing	**1.8**	**1.4**	**1.4**	**1.5**	**1.5**	**1.5**

Future Hydraulic Model Results

The future flows for resilient design in **Table B-9** were run in HEC-RAS and results were compared to existing. Future water surface elevations and velocities are compared to existing in **Table B-10** for the 0.04 AEP, roadway design event, and 0.01 AEP regulatory event at three characteristic locations along the reach as shown in **Figure B-8**. The HEC-RAS output table throughout the reach for all of the AEP events analyzed is in **Attachment B-3**. The 0.01 AEP floodplain map for existing and future conditions is also provided in **Attachment B-3**.

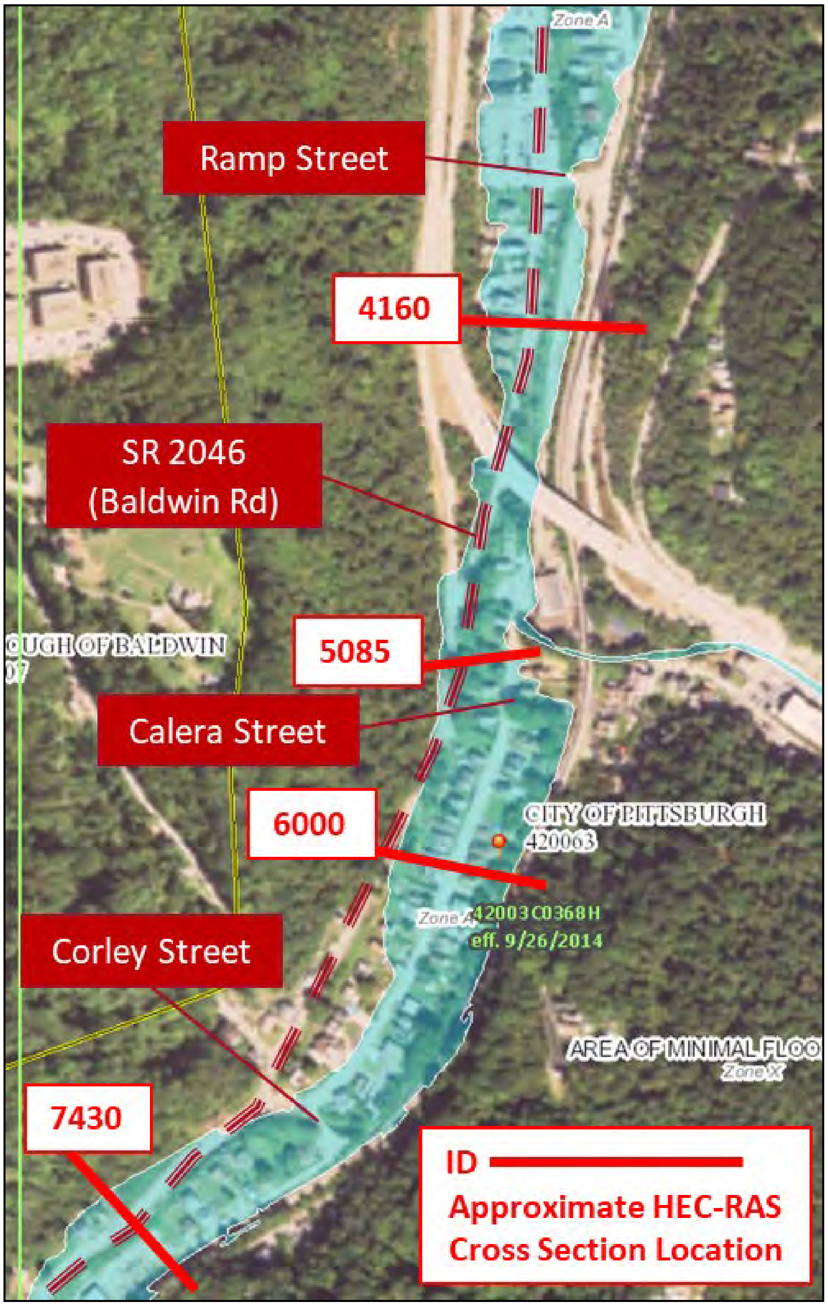

Figure B-8: Locations for Summary Hydraulic Results in Table B-10

Table B-10: Existing and Future Hydraulic Results

	Location	Existing WSE (ft) Q = 3,400 cfs	Future WSE (ft) Q = 4,800 cfs	Difference	Existing Velocity (fps)	Future Velocity (fps)	Difference
0.04 AEP Event	**7430**	781.60	782.03	+0.43	15.6	16.8	+1.2
	6000	767.79	768.22	+0.43	8.9	9.3	+0.4
	5085	760.76	762.11	+1.35	7.7	6.5	-1.2
	4160[a]	750.83	752.14	+1.31	6.9	6.3	-0.6
0.01 AEP Event	**Location**	**Existing WSE (ft) Q = 4,800 cfs**	**Future WSE (ft) Q = 7,350 cfs**	**Difference**	**Existing Velocity (fps)**	**Future Velocity (fps)**	**Difference**
	7430	782.03	782.70	+0.67	16.8	17.3	+0.5
	6000	768.22	769.10	+0.88	9.3	8.9	-0.4
	5085	762.11	764.03	+1.92	6.5	5.7	-0.8
	4160	752.14	753.54	+1.40	6.3	6.5	+0.2

[a]At this location, roadway classification changes to "local roadway" (0.1 AEP design event). Results are provided for 0.04 AEP event for consistency and comparison with other locations along the reach.

In the future condition, water surface elevations are projected to increase up to 1.35 feet for the 0.04 AEP event and up to 1.9 feet for the 0.01 AEP event. Channel velocities are projected to increase up to 1.2 ft/s for the 0.04 AEP event and 0.5 ft/s for the 0.01 AEP event. Changes in velocity along the reach are variable (alternating between minor increases and decreases) due to changes in flow distribution throughout the wide floodplain and mixed flow regime along the reach. In the existing condition, the 0.04 AEP event overtops the adjacent roadway by approximately 1.4 feet, with overtopping beginning between a 0.5 AEP and 0.1 AEP event. In the future condition, increased depths and frequency of overtopping along the roadway are expected. Existing and future results are compared further in the Resilient Design Options section.

Resilient Design Options

A resilient design checklist is recommended to assist in the evaluation of the need for various resilient design options for each site-specific analysis. The checklist consists of interdisciplinary parameters, including hydraulics, traffic, safety, and others, that are compared between existing and future conditions to determine if the site will be more vulnerable to issues such as scour, stability, and roadway overtopping. The parameters evaluated consist of the typical metrics used in hydraulic design of bridges and roadways. Future conditions are defined as a hydrologic conditions in 2100 based on the RCP 8.5 scenario that modifies existing flows by applying the appropriate precipitation factor. For the purposes of this study, precipitation factors have been determined using site-specific climate information discussed in preceding sections.

Table B-11 includes existing and future site and hydraulic characteristics for determining resilient design needs for the Streets Run project area. Scour calculations are included in **Attachment B-4**. The "Potential for Resilient Design" column was completed using the following codes to indicate level of potential for resilient design considerations:

- **Low:** minor or no special designs for resiliency anticipated,
- **Medium:** considerations for resiliency related designs may be beneficial, and
- **High:** consideration for resiliency design modifications is highly recommended.

Because every site is different, establishing ranges or thresholds for checklist parameters is not practical. When determining the level of potential for resilient design, engineering judgment, interdisciplinary coordination, and knowledge of existing site conditions should be used to evaluate the following:

- Risk posed to physical infrastructure and/or the traveling public
- Comparison of existing and future parameters

Magnitudes of future condition parameters and their effects on infrastructure performance where future changes in hydraulic conditions indicate the potential need for resilient design, the following are possible design alternatives that should be considered for roads that parallel a roadway or floodplain. This list is not all-inclusive, and any design recommendations should be determined using an interdisciplinary approach.

- Increased frequency of bankfull flow, increased channel velocity, higher scour potential, → adjustments to scour and bank protection or changes to wall foundations along roadway
- Increased frequency, velocity, or depth of overtopping flow → additional embankment protection or adjustments to pavement design

- New or increased effects on adjacent roadways → advise DOT of changes to serviceability and/or stability of adjacent roadway

Table B-11: Resilient Design Checklist (Cross Section 5085)

	Parameter	Existing Condition	Future Condition	Indicates Potential for Resilient Design
Site Data	**Hydrology Method**	HEC-HMS	HEC-HMS using 1.24*P[a]	N/A
Site Data	**Embankment Instability**	Yes	Yes	High
Site Data	**Overtopping Frequency**	0.2 AEP	0.5 AEP	Medium
Site Data	**Design Event Frequency**	0.04 AEP		N/A
Site Data	**Provides Access to Critical Services**	Yes - Pittsburgh Fire Bureau Station 20 located along Baldwin Road; Public Bus Route 56 (McKeesport to Downtown)		High
Bankfull Flow[c]	**Discharge (cfs)**	1,750 cfs		N/A
Bankfull Flow[c]	**Channel Velocity (fps)**	7.6		Medium
Bankfull Flow[c]	**Scour Depth (ft)**	20.5		High
Bankfull Flow[c]	**Event Frequency**	0.20 AEP	0.50 AEP	High
Design Event	**Discharge (cfs)**	3,400	4,800	N/A
Design Event	**Channel Velocity (fps)**	7.7	6.5	Low
Design Event	**Overtopping Velocity (fps)**	7.5	8.8	High
Design Event	**Overtopping Depth (Roadway) (ft)**	1.4	2.7	Medium
0.01 AEP Event	**Discharge (cfs)**	4,800	7,350	N/A
0.01 AEP Event	**Channel Velocity (fps)**	6.5	5.7	Low
0.01 AEP Event	**Overtopping Velocity (fps)**	8.8	9.5	High
0.01 AEP Event	**Overtopping Depth (Roadway) (ft)**	2.7	4.6	Medium

[a]1.24 is the average 0.1 AEP ratio computed from 8 GCMs for the year 2100 and was selected for use in resilient design for the pilot study

[b]Overtopping for the 0.50AEP event is limited to certain areas in the existing condition. Under future conditions, 0.50 AEP roadway overtopping is more widespread

[c]Bankfull discharge and frequency vary across the full reach. For the purposes of this checklist, bankfull parameters are being evaluated where Streets Run makes a sharp turn along Baldwin Road (Cross Section 5085)

Velocity Distributions

In addition to hydraulic parameters obtained from typical HEC-RAS output tables, velocity distribution plots for existing and future conditions were compared to determine potential areas of isolated velocity increases. Comparison plots for the 0.01 AEP event at cross section 5085 are shown in **Figure B-9**. Velocity distribution plots are a valuable tool for pinpointing areas of greater increase. For projects where site conditions warrant a two-dimensional (2D) hydraulic model, more refined flow distribution and velocity output may provide further insight. Higher velocity increases could indicate locations within the bridge or along the roadway embankment that would benefit from resilient design improvements.

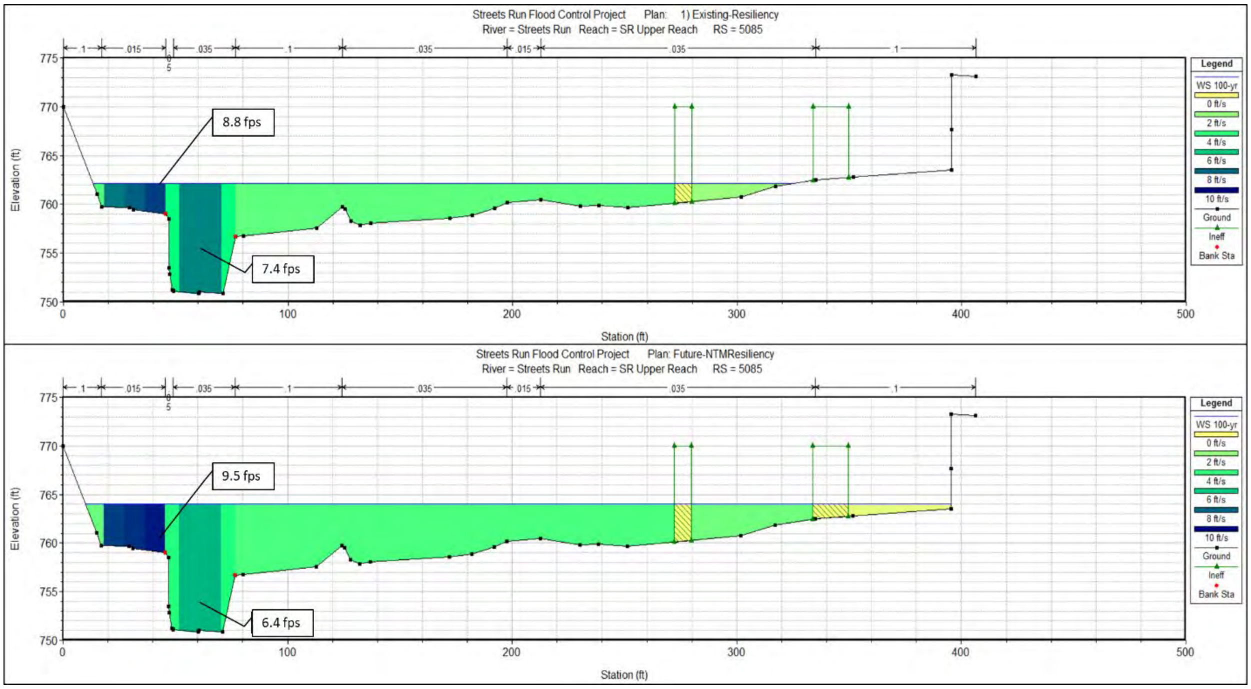

Figure B-9: Existing (top) and Future (bottom) 0.01 AEP velocity distribution plots

2D Hydraulic Modeling

The hydraulics throughout this reach of Streets Run are complicated by the 2D flow characteristics in the floodplain, specifically:

- the low-flow stream channel makes several sharp bends, which will result in different characteristics between flow in the channel and flow in the wide floodplain;
- there are four main bridges and several smaller foot bridges that influence water stages along the reach;
- complex overtopping flow patterns at each of the bridge crossings and the adjacent SR 2046 roadway; and
- there are many buildings in the floodplain that are affected by flooding and act as obstructions for overbank flow.

The use of 2D hydraulic modeling for studying this reach of Streets Run was not a part of the current pilot study. However, the use of 2D modeling tools for this reach and other waterways with similar characteristics and levels of risk could be highly beneficial for understanding existing floodplain dynamics and refining areas of high velocities that cause erosion and damage to roadways. This would also help facilitate interdisciplinary discussions on resilient design strategies by identifying more isolated areas of future concern and support more detailed risk evaluations and more cost-effective solutions.

Conceptual Evaluation of Stream Stability and Migration

In addition to the quantitative assessment of the resilient design parameters discussed throughout this section, engineering judgment should be used when evaluating existing concerns and their potential to worsen as a result of predicted changes in precipitation and discharge. For example, the potential for lateral and vertical changes to the waterway in the vicinity of the bridge or roadway being studied should be considered. If a site visit indicates existing concerns due to channel migration, future changes are likely to exacerbate the problem, and resilient design is highly recommended. For this study reach of Streets Run, the lateral migration of the stream is constrained by widespread property development, adjacent roadways (including SR 2046), and an adjacent railroad embankment. Issues stemming from this significant urbanization throughout the reach are exemplified by visible erosion of embankments, damage to pavement, and scour along existing walls and bridge abutments. Therefore, given the outlook for increased precipitation and discharge in the future, resilient design to protect existing roadway facilities is highly recommended.

Embankment Stability for Roadways Adjacent to Waterways and Floodplains

Resilient design alternatives were considered to address stability issues where SR 2046 is adjacent to Streets Run and existing scour and stability issues may be present. It is anticipated that under future climate scenarios, bankfull flow, which is anticipated to cause the worst-case scour condition, will occur more frequently. This will result in a higher possibility of damage to the roadway, serviceability issues due to roadway damage, and reduced design life of the roadway. Resilient design recommendations are provided for two different conditions within the study reach:

1. Sections of the existing concrete wall may need modifications to improve the level of protection of the roadway for resistance to erosion and scour.
2. Areas of existing roadway that are visibly damaged but no concrete wall or other embankment protection exists.

For each of these scenarios, the following resilient designs are possible options to provide increased resiliency:

1. Existing Wall Improvements:
2. Evaluate the level of protection needed at the toe of the existing wall.
    a. If needed, underpin the wall with micropiles and replace scour-prone soil below to an assumed depth of 6 feet below stream bed elevation with concrete filled bags.
    b. As an alternate to underpinning the existing wall, consider lining the stream channel with Fabriform to reduce risk of scour and eliminate the need to underpin the existing wall. Fabriform Lining is placed from top-of-bank to top-of-bank and buried in anchor trenches beyond the top-of-bank on each side.

3. Replace existing guide rail and pavement at edge of road with a reinforced concrete moment slab and single-face concrete barrier, to prevent high-frequency rain events from damaging roadway (not intended to provide flood control for less frequent events but may reduce inundation of the roadway for more frequent storms).

   The above solutions for the existing wall required the following assumptions as additional investigation was outside the scope of the pilot study:
   - The existing flood wall (assumed to be a gravity/semi-gravity dry-stack flood wall) along the edge of road is stable and in reasonably good structural condition, with no evidence of excessive distortion or undermining.
   - Predicted remaining service life is good for the existing wall.
   - The top of existing wall is at about the same elevation as the edge of pavement.
   - Existing standard guide rail is present at edge of road.

4. Add New Protection for Section Without Existing Wall:
   a. Construct a new flood wall at edge of road with soldier pile and precast concrete lagging panels; place structure backfill to fill the void on the roadway side of panels; and repave one lane of roadway.
   b. As an alternate, reconstruct the stream bank with durable AASHTO #1 stone with 1.5H:1V side slope; line the stream channel with Fabriform from top-of-bank to top-of-bank to enhance scour protection; and repave one lane of roadway.

   The above solutions for the existing roadway section without protection required the following assumptions as additional investigation was outside the scope of the pilot study:
   - The existing stream bank is eroded at edge of road with no apparent embankment protection.
   - Visible damage to edge of roadway has resulted in closure of the existing pavement shoulder.
   - No existing guiderail or barrier exists.

Economic Analysis

Preliminary construction costs for the embankment and roadway stability design solutions were estimated based on similar solutions implemented for previous projects. These costs present an order of magnitude estimate for preliminary budgeting purposes, only. Cost estimates assume that excavation spoil can remain on site, that minimal waste is require, and that waste can be disposed of locally off-site with no tipping fee. Other assumptions include: work will be completed when, and if, in-stream access is required under normal flow with shallow depth of stream water with temporary Portadam (or equal); and that a hydraulic excavator is permitted to operate within the protected side of stream channel under normal flow without out-ofordinary environmental, permitting and/or operational restriction (e.g. turbidity curtain required with no other special E&SC measures).

1. Existing Wall Improvements:
    a. Micropiles – average unit cost of $750/linear foot; assumes 16 feet long micropiles with brackets for underpinning to a maximum depth of 6 feet and 1 feet x 3 feet x 5 feet concrete filled bags stacked 6 feet high between each micropile installed.
    b. Stream lining using Fabriform – average unit cost of $500/linear foot; assumes 40 feet long x 6 inches (nominal thickness) Fabriform channel lining to span entire width of stream channel from top-of-bank to top-of-bank (including portion that is buried in anchor trenches).
    c. Moment slab – average unit cost of $1,000/linear foot

Existing wall improvements could be prioritized for specific areas of the reach after additional evaluations of existing conditions is performed. For example, downstream of Calera Street where Streets Run makes a sharp turn towards SR 2046 should be evaluated. Using the unit costs described above for this 75-foot section of wall, the total preliminary construction cost would be approximately $131,250 for the micropile alterative and $112,500 for the Fabriform stream-lining alternative. An additional 1,000 feet of existing wall between the Allegheny Railroad bridge and Corley Street bridge should be similarly evaluated for continuous or partial resilient design solutions.

2. New Protection for Section Without Existing Wall:
    a. New concrete wall – average unit cost of approximately $3,200/linear foot; assumes cantilevered steel soldier pile at 12 feet c/c, which is embedded in rock sockets.
    b. Reconstruct bank and Fabriform channel lining – average unit cost of $900/linear foot; assumes 7 CY stone / LF to reconstruct eroded stream bank and 40 feet long x 6 inches (nominal thickness) Fabriform channel lining to span entire width of stream channel from top-of-bank to top-of-bank (including portion that is buried in anchor trenches).

Using the above unit costs and an identified 200-foot section of roadway with existing damage or high propensity for future damage, the total preliminary construction cost would be approximately $640,000 for the new wall alternative and $180,000 for bank reconstruction alternative. Note that additional assumptions for preliminary costs for these alternatives include:

- The edge of pavement is situated approximately 10 feet above the elevation of the existing streambed.

- The top of scour-resistant bedrock is located within a depth of about 5 feet below the elevation of the existing streambed.

Conclusions

The existing Streets Run reach along SR 2046 is characterized by frequent and significant flooding of the roadway and surrounding properties. Portions of the existing channel are lined with concrete walls, while other areas have sloped embankments that are prone to erosion resulting in visible damage to the roadway. The results of this study indicate that the frequency of flooding events will increase under future precipitation scenarios. More frequent flooding will increase both the risk of scour and the risk of damage to the roadway. In order to increase the resiliency of the roadway under future conditions, this study recommends that the existing roadway embankment and concrete channel walls be further evaluated to determine if new concrete walls, Fabriform stream-lining, or underpinning of existing wall foundations are needed.

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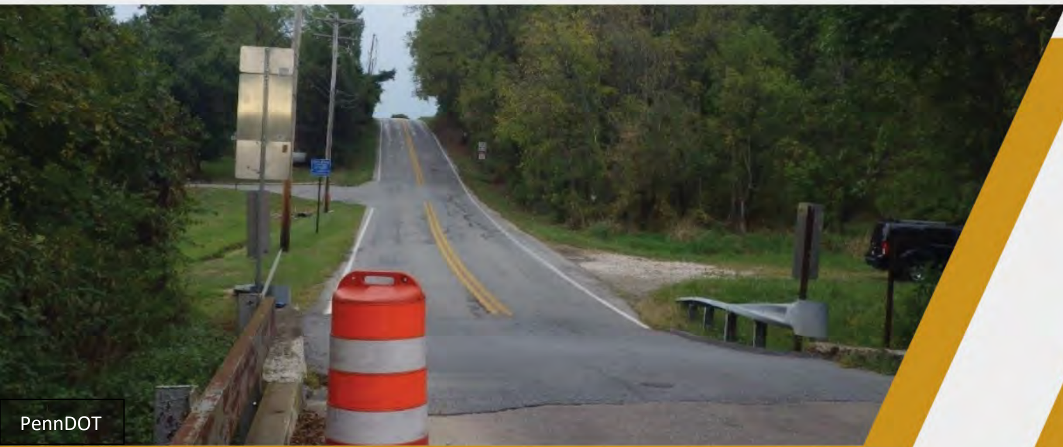

APPENDIX C

Delaware County (Station Road) Pilot Analysis

SEPTEMBER 2020

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Introduction

The hydrologic and hydraulic analysis for Delaware County Bridge along T-314 (locally known as Station Road) over East Branch Chester Creek was performed as a part of PennDOT's Pilot Study for Resilience and Durability to Extreme Weather. The project involves computing existing and future peak flows using physically based hydrologic methods and developing a one-dimensional hydraulic model that considers existing and future conditions for three sites in Pennsylvania. The Delaware County site was selected because of the history of roadway overtopping and the possibility of developing resilient design strategies for an upcoming project to replace the existing bridge. Delaware County in located in southeastern Pennsylvania.

Site Description

The point of interest is a structure located along T-314 (Station Road) [also known as North Station Road per PennDOT Type 5 Map] crossing over East Branch Chester Creek in Thornbury Township, Delaware County, Pennsylvania. Its location on the USGS quadrangle map entitled Delaware, PA is approximately 39° 55' 57"N latitude and 75° 31' 01" W longitude. The overreaching area of the drainage watershed is primarily within Chester County, Pennsylvania. See Figure C-1 below for the project location:

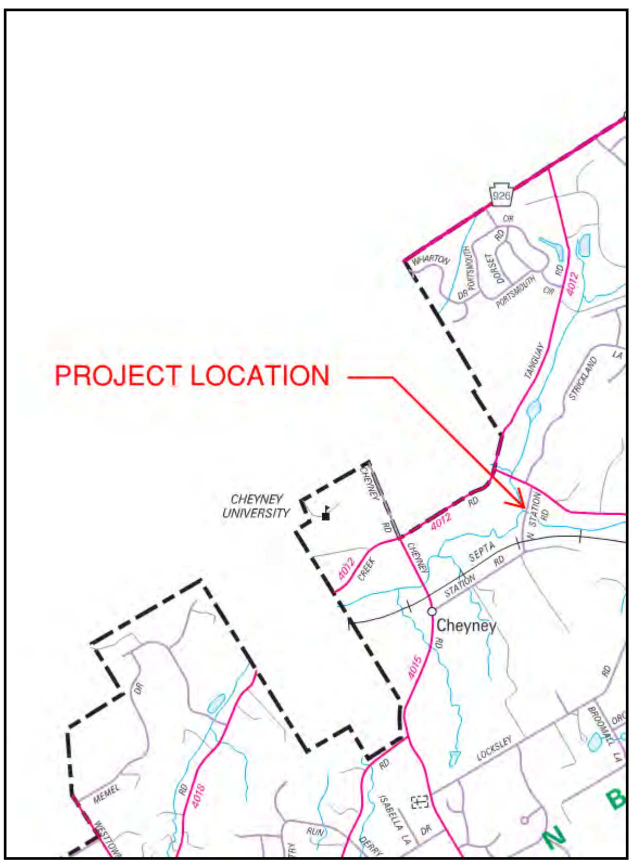

Figure C-1: Project Location

The existing topography is mostly a mix of open space, wood lots, and subdivisions (1 acre or less lot sizes.) The upper reach of the watershed is urbanized with city blocks, an airport, and commercial/industrial lots. The terrain of the area is within the Piedmont Upland Section of the Piedmont Provence. The dominant topographic form is broad, rounded to flat-topped hills and shallow valleys with low to moderate local relief. The underlying rock is mainly schist, gnesiss, and quartzite.

Station Road is a rural collector roadway that provides access for residential property owners. Per PennDOT DM-2 Chapter 10.6.E the design event for the roadway is the 0.1 AEP event. The existing bridge is located on a sag curve with the north approach experiencing overtopping during the 10-year rainfall event. Photographs of the approach roadway and structure inlet are in **Figure C-2** and **Figure C-3**. Additional site photographs are included in **Attachment C-1**. There currently are no planned large-scale developments within Chester County. Chester County only anticipates future growth as laid out by current zoning and organic development.

Figure C-2: View of North approach to structure. PennDOT photo.

Figure C-3: Inlet of structure. PennDOT photo.

Existing Structure

The existing Station Road bridge is a single-span composite concrete beam bridge with a normal clear span of 50.0 feet, out-to-out structure width of 22.0 feet, and under-clearance range of 2.8 to 8.1 feet. The existing bridge has a concrete parapet with stone masonry wingwalls.

Watershed Characteristics

The drainage area at the Station Road crossing is approximately 22.9 mi^2 as delineated using StreamStats web application and verified with HEC-GeoHMS addon software with ArcGIS. The GIS HECGeoHMS model used a 10-meter Digital Elevation Model (DEM), SSURGO soils, obtained from NRCS, and 2011 National Land Cover Database (USGS, 2011) land use data to compute curve numbers. The basin generally consists of agricultural land with residential development in the lower watershed and some forested area in the headwaters. There is an airport and various urban areas within the basin, requiring adjustments to the DEM and subbasin curve numbers to accurately represent the hydrologic effects. There is no carbonate geology within the watershed. See **Figure C-4** below for the delineated basin to the project area.

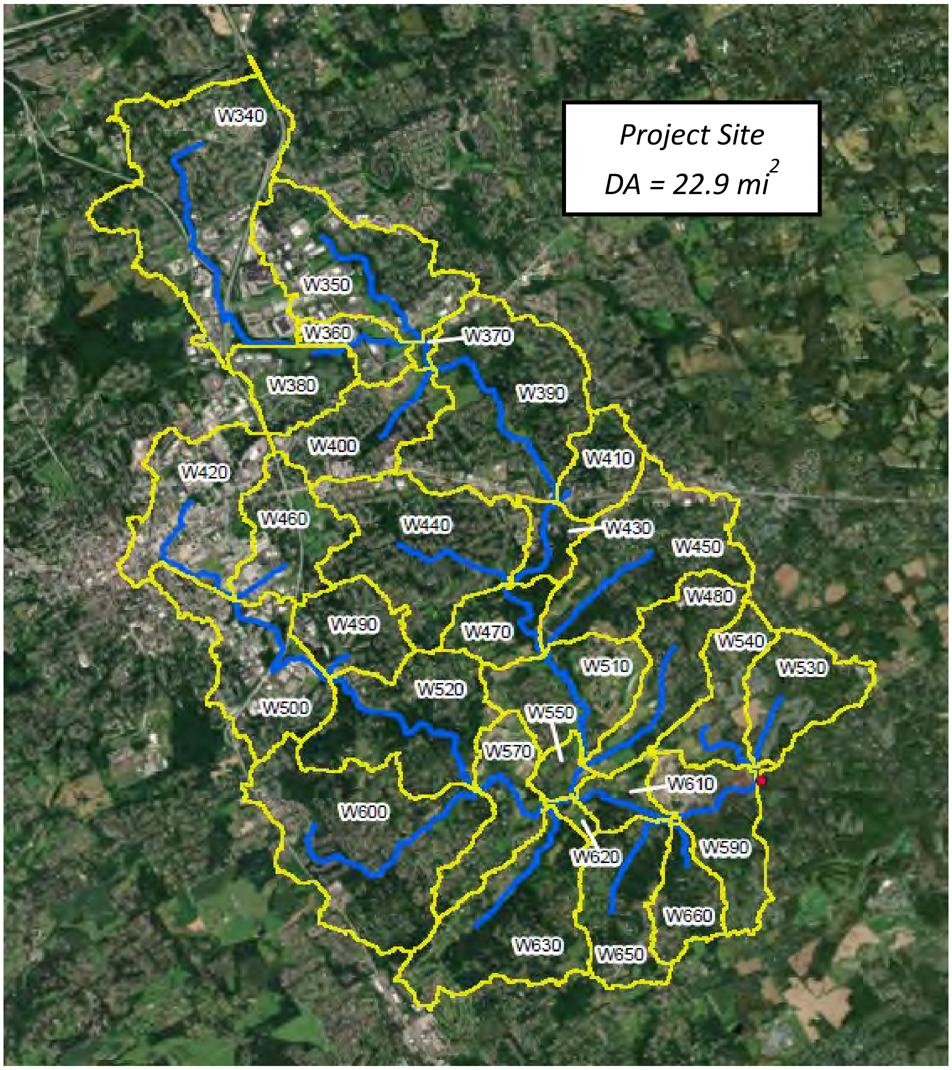

Figure C-4: East Branch Chester Creek

Hydrologic Method

Peak flows for the current study were computed using the Army Corps of Engineers' Hydrologic Modeling System (HEC-HMS) 4.2.1. HEC-HMS is applicable for basins of almost any size and complexity. The subbasins were delineated and geometric parameters for each subbasin were calculated using WMS. At all subbasins, the curve number method was used as the loss rate method. The time of concentration (T_c) was calculated in the WMS program using the NRCS segmental method. Additional watershed characteristics are included in **Attachment C-2**. The Muskingum-Cunge routing method was used for all reaches and was calculated using a trapezoidal channel with bottom widths of 20-30 feet, floodplain widths of 200-500 feet, side slopes of 2H:1V, channel Manning's n of 0.035, and overbank Manning's n of 0.04 to 0.08. Routing parameters were estimated using aerial imagery and topographic data. The subbasin data was then exported from WMS to HMS. All events were modeled using the NRCS Type C storm distribution that provided precipitation depths from 5 minutes to 24 hours, with existing precipitation depths determined using the PDT-IDF curves from PennDOT Publication 584 Appendix 7A, which are based on data from NOAA Atlas 14, Volume 2. Existing 24-hour precipitation depths are shown in **Table C-1**.

Rainfall totals in **Table C-1** are based on Region 5 rainfall totals in PennDOT Publication 584. The site is located within Region 4; however, the majority of Chester County is located in Region 5. Region 5 has more conservative values compared to Region 4. Utilizing Region 5 data provides an opportunity to study a worst-case scenario for developing resilient design analysis.

Table C-1: Existing Precipitation Depths

Annual Exceedance Probability	0.5	0.1	0.04	0.02	0.01	0.002
Precipitation Depth (in)	3.40	4.95	6.10	7.16	8.43	12.40

Existing Flows and Validation

Existing flows computed using HEC-HMS were validated comparing to other hydrologic methods as shown in **Table C-2**. Comparison flows for East Branch Chester Creek at Station Road were gathered from USGS regression equations (SIR 2008-5102 StreamStats) and historical flood events recorded by USGS stream gage 01476853 and frequency analysis using Bulletin 17C and USGS Program PeakFQ V7.2. USGS stream gage 01476853 was previously located at the point of interest being studied and since been removed, having only collected nine years of data, which produced much lower flows. A neighboring stream gage (Gage No. 01476480) on Ridley Creek was also analyzed for comparison, which has similar drainage area and watershed characteristics, and has 31 years of continuous data. These Ridley Creek Gage flows were translated to the same drainage area as the study site and are more comparable to the other Chester Creek flows. The HEC-HMS flows computed for the current study are similar to the USGS regression results for most events, producing more conservative flows for the 0.01 and 0.002 AEP events. Therefore, the HEC-HMS flows are considered validated for use in the hydraulic analysis for the current study.

Table C-2: Existing Flow Comparison

Method	Peak Flows (cfs)					
	0.5	0.1	0.04	0.02	0.01	0.002
HEC-HMS	1,850	3,950	5,800	7,600	9,950	14,350
StreamStats	2,010	4,110	-----	6,550	7,740	10,900
USGS 01476853	972	1,707	2,141	2,494	2,872	3,871
USGS 01476480	1,253	3,493	5,316	7,076	9,234	16,260

Existing Hydraulic Performance

A hydraulic model for the existing and proposed structures was developed and run in HEC-RAS 5.0.7. Topographic information consists of site-specific field survey data in the channel and immediate overbank areas. The hydraulic model results indicate that both the existing structure and Station Road has frequent overtopping of the roadway. Additionally, Creek Road also has some overtopping due to backwater created by the Station Road crossing.

Global Climate Model Summary

The Global Climate Model (GCM) evaluation for the current study looked at eight distinct GCMs for Representative Concentration Pathway (RCP) 8.5, which predicts the highest future CO_2 equivalent of the various RCPs adopted by the International Panel on Climate Change (IPCC). RCP 8.5 was the trajectory adopted in the PA Climate Impacts Assessment Update (Shortle and others, 2015). The GCMs selected for this study were determined using guidance from the Infrastructure and Climate Network (ICNET) and Transportation Research Board reports developed for the NCHRP Project 15-61 (Kilgore and others, 2019a; 2019b). Future (year 2100) 24-hour precipitation data for the eight GCMs were compared against GCM estimates of historical precipitation to develop ratios for various annual exceedance probabilities (AEPs). The calculated ratios for the East Branch Chester Creek watershed are shown in **Table C-3** and compared in **Figure C-6**. Precipitation depths and flows computed using the ratios for each AEP across all GCMs are shown in **Table C-4** and **Table C-5**, respectively. Flows computed using the future precipitation depths are in **Table C-5**. The calculated ratios, precipitation depths, and flows for each GCM are provided to demonstrate the significant variability across the selected climate models and are not recommended for design purposes.

Table C-3: Future Precipitation Ratios

Precipitation Ratios						
GCM	Annual Exceedance Probability					
	0.5	0.1	0.04	0.02	0.01	0.002
BCC-CSM 1.1-m	1.18	1.04	0.95	0.90	0.84	0.72
CCSM4	1.04	1.10	1.20	1.30	1.42	1.76
CSIRO-Mk3.6.0	1.24	1.33	1.36	1.37	1.39	1.41
GFDL-CM3	1.33	1.16	1.08	1.03	0.97	0.86
GISS-E2-R	1.16	1.11	1.08	1.06	1.04	1.00
HadGEM2-AO	1.20	1.34	1.42	1.49	1.56	1.73
MIROC5	1.16	1.18	1.17	1.17	1.16	1.14
MRI-CGCM3	1.08	1.05	1.04	1.03	1.03	1.03
Average Ratios	**1.17**	**1.16**	**1.16**	**1.16**	**1.17**	**1.20**

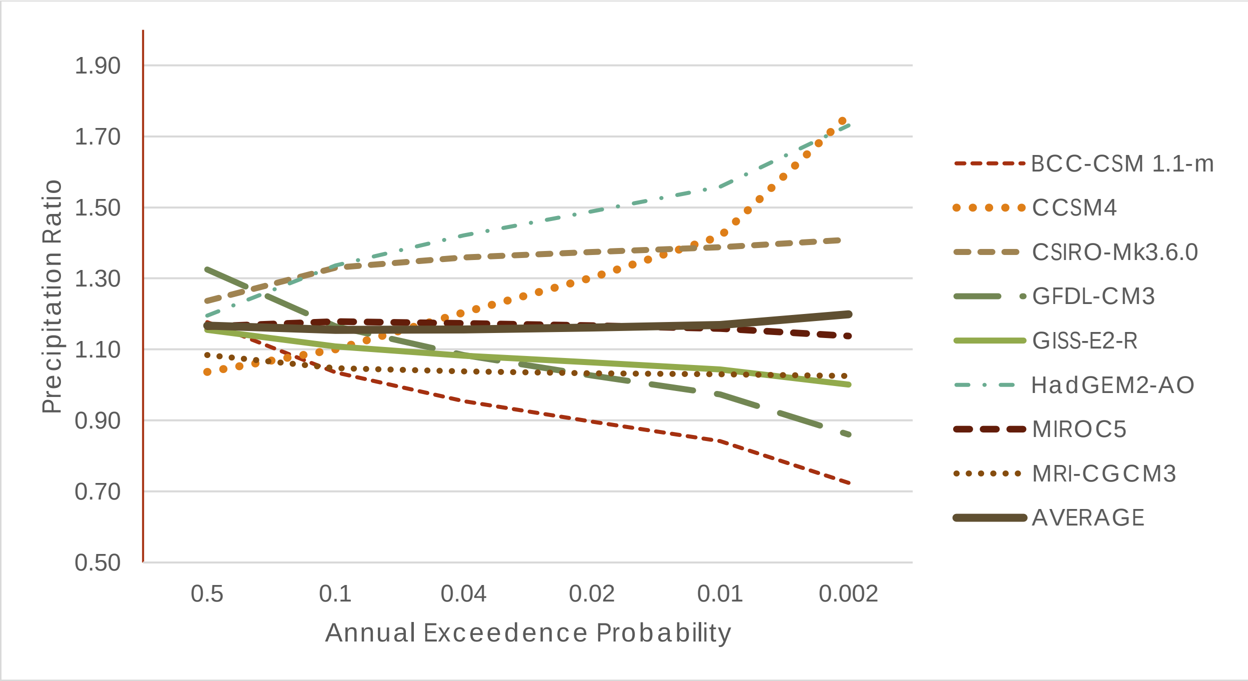

Figure C-5: Future (Year 2100) Precipitation Ratios for Selected Climate Models

Table C-4: Precipitation Depths – GCM Variability

Existing and Future Precipitation Depths (in)						
GCM	Annual Exceedance Probability					
	0.5	0.1	0.04	0.02	0.01	0.002
Existing	***3.4***	***4.95***	***6.1***	***7.16***	***8.43***	***12.4***
BCC-CSM 1.1-m	4.00	5.12	5.82	6.42	7.09	8.98
CCSM4	3.52	5.45	7.34	9.33	11.96	21.83
CSIRO-Mk3.6.0	4.20	6.59	8.29	9.84	11.70	17.47
GFDL-CM3	4.51	5.77	6.61	7.35	8.21	10.67
GISS-E2-R	3.93	5.49	6.60	7.61	8.80	12.41
HadGEM2-AO	4.06	6.62	8.67	10.66	13.14	21.46
MIROC5	3.96	5.83	7.16	8.36	9.77	14.11
MRI-CGCM3	3.69	5.18	6.33	7.40	8.68	12.71

Table C-5: Flows – GCM Variability

Existing and Future Flows (cfs)						
GCM	Annual Exceedance Probability					
	0.5	0.1	0.04	0.02	0.01	0.002
Existing	***1,850***	***3,950***	***5,800***	***7,600***	***9,950***	***14,350***
BCC-CSM 1.1-m	2,550	4,250	5,300	6,200	7,350	8,400
CCSM4	1,950	4,800	8,200	12,000	17,500	32,550
MIROC5	2,500	5,500	7,800	10,000	12,750	17,500
HadGEM2-AO	2,650	6,950	10,950	14,900	20,150	31,850
CSIRO-Mk3.6.0	2,850	6,900	10,150	13,150	16,900	23,950
GFDL-CM3	3,250	5,350	6,750	8,000	9,500	14,350
GISS-E2-R	2,450	4,850	6,750	8,500	10,700	14,350
MRI-CGCM3	2,150	4,350	6,250	8,050	10,450	14,900

Future Land Use Changes

Information provided by the Chester County Planning Commission indicated that there is little planned development within the watershed. The existing land use within the watershed is largely developed.

Future Hydrology

Future flows for each GCM were computed using precipitation depths adjusted with the 0.1 AEP ratio for the following events (AEPs): 0.5, 0.1, 0.04, 0.02, 0.01, 0.002. Use of the 0.1 AEP ratio for the more extreme events was recommended in a Transportation Research Board reports completed under NCHRP Project 15-61 (Kilgore and others, 2019a; 2019b), due to the current limitations of high-resolution datasets to represent precipitation extremes. For the purposes of the current study, the 0.1 AEP ratio was used for all evaluated events, including the 0.5 AEP event, for simplicity. Use of the actual ratios for more frequent events (less than 0.1 AEP) may be recommended in instances where such events are pertinent for design.

Table C-6 lists ratios of future to existing discharge for each GCM for each return period, which range from less than 1.0 to 2.3. The relationship between precipitation and discharge is a function of the specific characteristics of the watershed, such as land use, soils, and available storage.

Table C-6: Ratio of Future Flows to Existing Flows

Ratio of Future to Existing Flows						
GCM	Annual Exceedance Probability					
	0.5	0.1	0.04	0.02	0.01	0.002
BCC-CSM 1.1-m	1.4	1.1	0.9	0.8	0.7	0.6
CCSM4	1.1	1.2	1.4	1.6	1.8	2.3
MIROC5	1.4	1.4	1.3	1.3	1.3	1.2
HadGEM2-AO	1.5	1.8	1.9	2.0	2.0	2.2
CSIRO-Mk3.6.0	1.6	1.7	1.7	1.7	1.7	1.7
GFDL-CM3	1.8	1.4	1.2	1.0	1.0	1.0
GISS-E2-R	1.4	1.2	1.2	1.1	1.1	1.0
MRI-CGCM3	1.2	1.1	1.1	1.1	1.1	1.0

Flows computed by applying the average ratio (1.16) for the 0.1 AEP across all GCMs were used to develop resilient design strategies. Existing and future precipitation depths computed using the 0.1 AEP ratios for each GCM, as well as the final precipitation depths for resilient design, are shown in **Table C-7**. The range of ratios and precipitation depths in **Table C-7** shows that there is still significant variability between climate models, even when utilizing ratios from the less extreme 0.1 AEP event.

Table C-7: Precipitation Depths

Computed Using 0.1 AEP Ratios

Existing and Future Precipitation Depths (in)							
GCM	0.1 AEP Ratio[a]	Annual Exceedance Probability					
		0.5	0.1	0.04	0.02	0.01	0.002
Existing	-	**3.40**	**4.22**	**4.95**	**6.10**	**7.16**	**8.43**
BCC-CSM 1.1-m	1.04	3.52	4.37	5.12	6.32	7.41	8.73
CCSM4	1.10	3.74	4.64	5.45	6.71	7.88	9.28
CSIRO-Mk3.6.0	1.33	4.52	5.61	6.59	8.12	9.53	11.22
GFDL-CM3	1.16	3.96	4.92	5.77	7.11	8.34	9.82
GISS-E2-R	1.11	3.77	4.68	5.49	6.76	7.94	9.35
HadGEM2-AO	1.34	4.54	5.64	6.62	8.15	9.57	11.27
MIROC5	1.18	4.01	4.97	5.83	7.19	8.44	9.94
MRI-CGCM3	1.05	3.56	4.42	5.18	6.39	7.50	8.83
Resilient Design Precipitation[b]	**1.16 (average)**	**3.94**	**4.90**	**5.74**	**7.08**	**8.31**	**9.78**

[a]Originally presented in Table 3

[b]Computed using 1.16 times the existing precipitation depth (average 0.1 AEP ratio across GCMs)

Existing and future flows using the precipitation depths in **Table C-7** are summarized in **Table C-8**. Future flows range from approximately 1.3 to 1.4 times the existing discharge, with an average future to existing flow ratio of 1.3. Although the average precipitation ratio was 1.16, the average flow ratio is 1.4 implying the discharge increases more than twice as much as the precipitation.

Table C-8: Flows Computed Using 0.1 AEP Ratios

Existing and Future Flows (cfs)						
GCM	**Annual Exceedance Probability**					
	0.5	**0.1**	**0.04**	**0.02**	**0.01**	**0.002**
Existing Flows	1,850	3,950	5,800	7,600	9,950	14,350
BCC-CSM 1.1-m	2550	4250	5300	6200	7350	8400
CCSM4	1,950	4,800	8,200	12,000	17,500	32,550
MIROC5	2500	5500	7800	10000	12750	17500
Calculated Average 1.16x	2,500	5,300	7,650	9,900	12,900	18,950
HadGEM2-AO	2650	6950	10950	14900	20150	31850
CSIRO-Mk3.6.0	2,850	6,900	10,150	13,150	16,900	23,950
GFDL-CM3	3300	5350	6750	8000	9500	14350
Resilient Design Flows	**2,500**	**5,300**	**7,650**	**9,900**	**12,900**	**18,950**
Ratio of Resilient Design Flows to Existing	**1.4**	**1.3**	**1.3**	**1.3**	**1.3**	**1.3**

Future Hydraulic Model Results

The future flows for resilient design in **Table C-8** were run in HEC-RAS and results were compared to existing. **Table C-9** includes existing and future water surface elevations and velocities upstream of the Baker Road Bridge for the 0.1 AEP, roadway design event, and 0.01 AEP regulatory event. The HEC-RAS output table for all of the AEP events analyzed is in **Attachment C-3**. The 0.01 AEP floodplain map for existing and future conditions is also provided in **Attachment C-3**.

Table C-9: Existing and Future Hydraulic Results

	Parameter	**Existing Condition**	**Future Condition**	**Difference**
0.10 AEP Event	**Discharge (cfs)**	3,950	6,150	+2,200
	Upstream WSE (ft)	242.64	243.61	0.97
	Upstream Channel Velocity (fps)	3.3	4.0	+0.7
0.01 AEP Event	**Discharge (cfs)**	9,950	14,400	+4,450
	Upstream WSE (ft)	244.65	245.61	0.96
	Upstream Channel Velocity (fps)	5.1	6.2	+1.1

Water surface elevations would increase up to 1.0 feet and average channel velocities would increase up to 1.1 ft/s in the future condition. In the existing condition, the 0.1 AEP event overtops the both approaches and the structure. In the future condition, the 0.1 AEP event would overtop the bridge deck and both approach roadways and the structure. The 0.01 AEP event overtops the bridge deck and both approach roadways in the existing condition; increased water surface elevations would result in the bridge barrier being overtopped in the future condition. Existing and future results are compared further in the Resilient Design Options section.

Resilient Design Options

A resilient design checklist is recommended to assist in the evaluation of the need for various resilient design options for each site-specific analysis. The checklist consists of interdisciplinary parameters, including hydraulics, traffic, safety, and others, that are compared between existing and future conditions to determine if the site would be more vulnerable to issues such as scour, stability, and roadway overtopping. The parameters evaluated consist of the typical metrics used in hydraulic design of bridges and roadways. Future conditions are defined as a hydrologic conditions in 2100 for the RCP 8.5 scenario that modifies existing precipitation by applying the appropriate precipitation or factor. For the purposes of this study, precipitation factors have been determined using site-specific climate information discussed in preceding sections.

Table C-10 includes existing and future site and hydraulic characteristics for determining resilient design needs for the Station Road project area. Scour calculations are included in **Attachment A-4**. The "Potential for Resilient Design" column was completed using the following codes to indicate level of potential for resilient design considerations:

- **Low:** minor or no special designs for resiliency anticipated,
- **Medium:** considerations for resiliency related designs may be beneficial, and
- **High:** consideration for resiliency design modifications is highly recommended.

Because every site is different, establishing ranges or thresholds for checklist parameters is not practical. When determining the level of potential for resilient design, engineering judgment, interdisciplinary coordination, and knowledge of existing site conditions should be used to evaluate the following:

- Risk posed to physical infrastructure and/or the traveling public
- Comparison of existing and future parameters
- Magnitudes of future condition parameters and their effects on infrastructure performance

Where future changes in hydraulic conditions indicate the potential need for resilient design, the following are possible design alternatives that should be considered. This list is not all-inclusive, and any design recommendations should be determined using an interdisciplinary approach.

- Increased bridge velocity, scour potential, propensity for pressure flow → adjustments to scour protection, foundations, or structure hydraulic opening
- Increased frequency, velocity, or depth of overtopping flow → additional embankment protection or adjustments to pavement design, changes to structure anchoring
- New or increased effects on adjacent roadways → advise DOT of changes to serviceability and/or stability of adjacent roadway
- No current structure overtopping and no low chord inundation in existing, but impacts to beams/barrier in future condition → evaluate possible change to bridge or beam type to consider inundation
- Existing conditions impacts beams/barrier and future condition results in increased impacts or barrier overtopping → possible superstructure design adjustments

Table C-10: Resilient Design Checklist

	Parameter	Existing Condition[a]	Future Condition	Indicates Potential for Resilient Design
Site Data	**Hydrology Method**	HEC-HMS	HEC-HMS using 1.16*P[b]	N/A
	Embankment Instability	No issues noted	Minimal risk	Low
	Overtopping Frequency	0.10 AEP	0.50 AEP	Medium
	Provides Access to Critical Services	N/A – short detour routes easily available	N/A – short detour routes easily available	Low
	Design Event Frequency	0.10 AEP		N/A
Design Event	**Discharge (cfs)**	3,950	5,300	N/A
	% Q Bridge	74	54	Low
	Pressure Flow	Yes	Yes	Low
	Bridge Velocity (fps)	9.0	6.5	Low
	Overtopping Velocity (fps)	5.6	8.9	Medium
	Overtopping Depth (Roadway) (ft)	2.3	2.3	Medium
	Overtopping Depth (Structure) (ft)	Below top of barrier	Below top of barrier	Medium
	Adjacent Roadway(s) Impacted	Yes	Yes	Medium
0.01 AEP Event	**Discharge (cfs)**	9,950	12,900	N/A
	% Q Bridge	24	15	Low
	Pressure Flow	Yes	Yes	Low
	Scour Depth (ft)	5.52	5.41	Low
	Riprap Size	R-7	R-6	Low
	Bridge Velocity (fps)	7.3	6.5	Low
	Overtopping Velocity (fps)	4.7	8.8	Medium
	Overtopping Depth (Roadway) (ft)	4.4	4.4	High
	Overtopping Depth (Structure) (ft)	Up to 1 ft over barrier	Up to 1 ft over barrier	High
	Adjacent Roadway(s) Affected	Yes	Yes	High

[a]Utilizes the geometry for the proposed bridge replacement project for consistent comparison to future conditions
[b]1.16 is the average 0.1 AEP ratio computed from 8 GCMs for the year 2100 and was selected for use in resilient design for the pilot study

In addition to hydraulic parameters obtained from typical HEC-RAS output tables, velocity distribution plots for existing and future conditions were compared to determine potential areas of isolated velocity increases. Comparison plots for the 100-year (0.01 AEP) event at the cross section immediately upstream of the Baker Road Bridge are shown in **Figure C-7**. Velocity distribution plots are a valuable tool for pinpointing areas of greater increase. For projects where site conditions warrant a 2-dimensional hydraulic model, more refined flow distribution and velocity output may provide further insight. Higher velocity increases could indicate locations within the bridge or along the roadway embankment that would benefit from resilient design improvements.

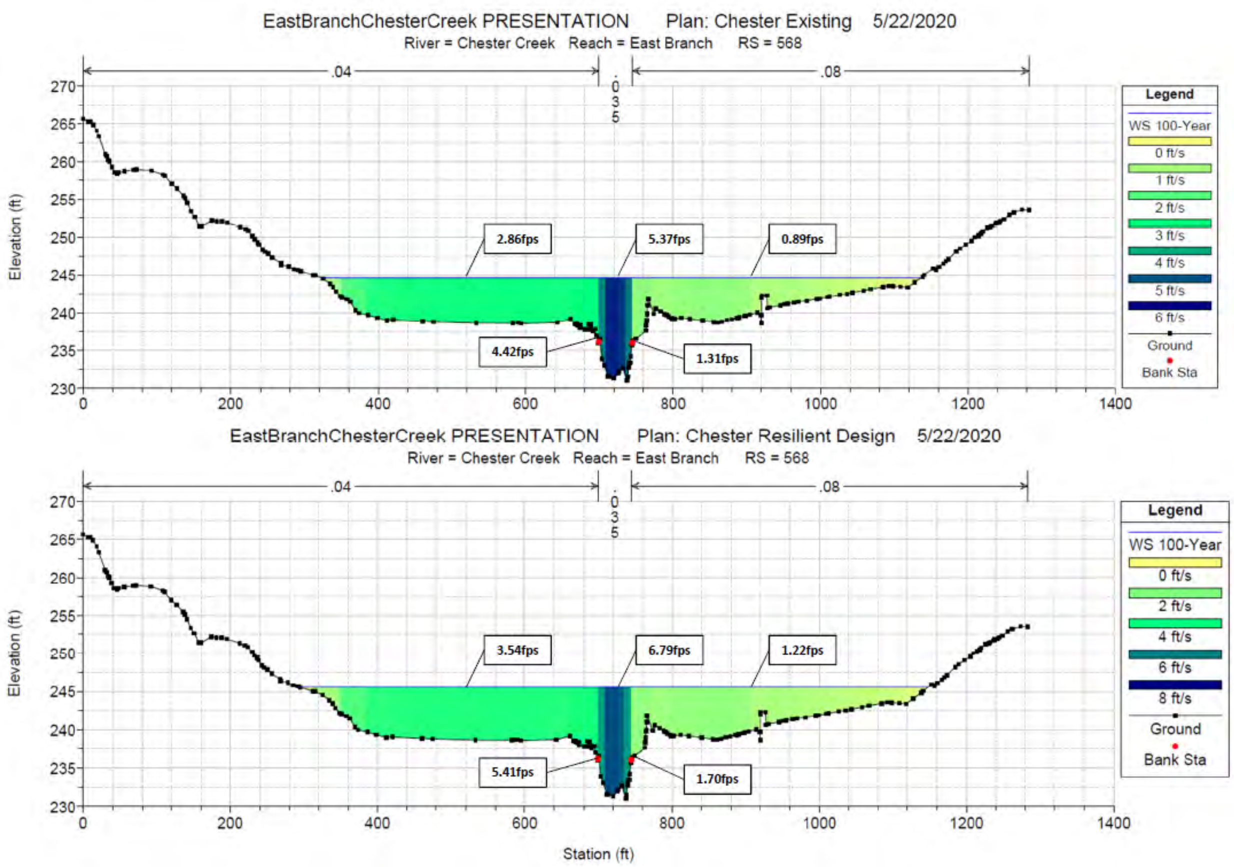

Figure C-6: Existing (top) and Future (bottom) 0.01 AEP velocity distribution plots

The overtopping event for Station Road over East Branch Chester Creek is the 10-year (0.1 AEP) event for both existing and future conditions. However, under future conditions overtopping depths would increase. Due to the significant amount of weir flow present over the roadway, the velocities through the bridge are not high. However, there is some evidence at the existing structure of channel scour due to pressure flow conditions. In addition, Creek Road in the upstream left overbank is likely to see increased flooding depths and frequency under future conditions. **Figure C-7** demonstrates the increase in water surface elevation from Existing vs Future flow conditions.

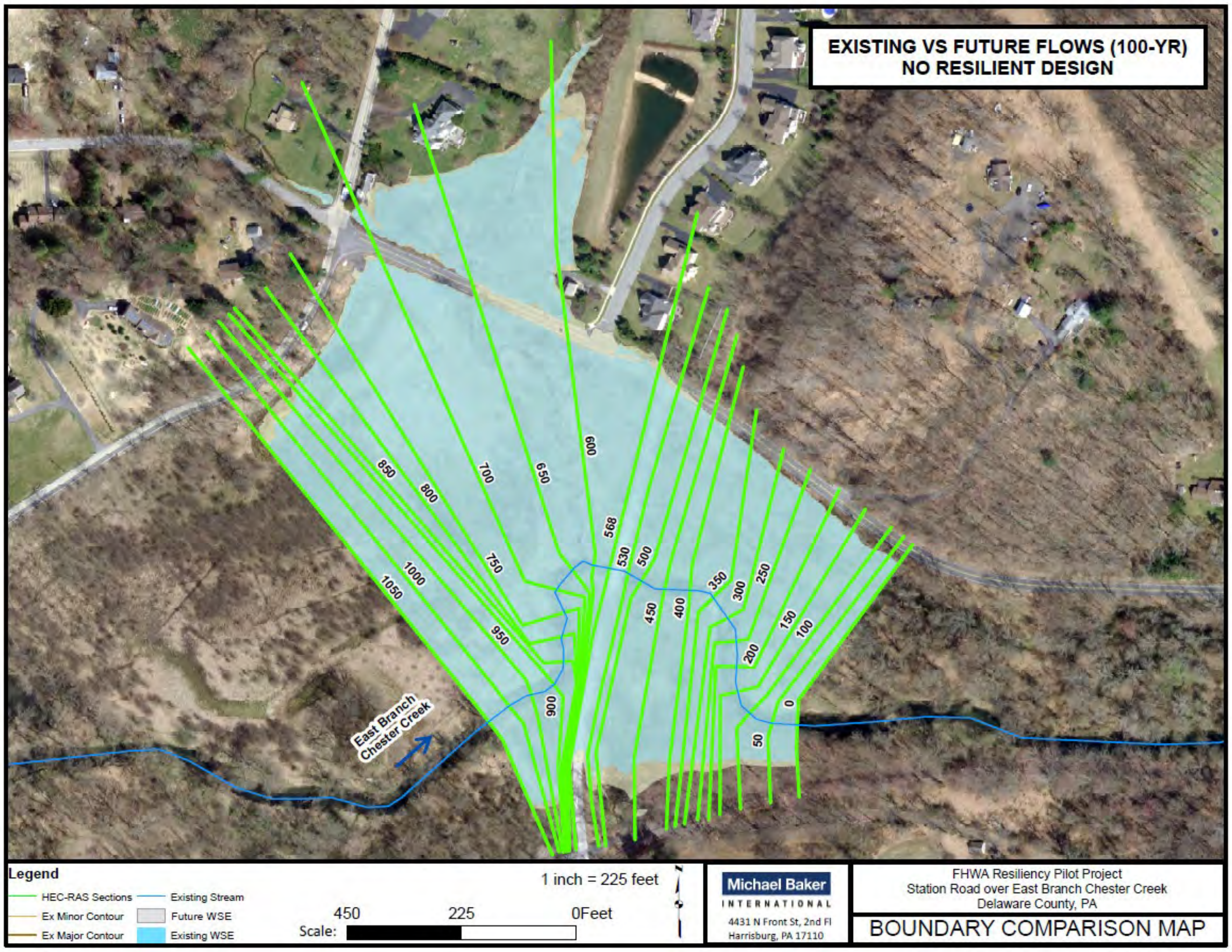

Figure C-7: Existing vs Future Water Surface Elevations

Because of the significant roadway overtopping at this site, which would only be exacerbated under future conditions, design options were explored with the goal of reducing overtopping depth and frequency. To demonstrate how changes to the structure and roadway will affect hydraulic parameters such as velocity and water surface elevations, a sensitivity analysis was performed for the site that explored two alternatives:

3. An approximately 25% increase in the proposed bridge span, and
4. An approximately 90% increase in the proposed Bridge span with left bank grading through bridge to further increase hydraulic opening, and an approximate 3.5-foot roadway profile increase.

Alternative 2 was designed to remove the overtopping for the 0.10 AEP event, while Alternative 1 shows a reasonable design that might be selected for a standard bridge replacement. A selection of pertinent hydraulic results from the sensitivity analysis are provided in **Table C-11**. Results indicate that only a significantly larger span and higher profile remove overtopping for the 0.10 AEP event but will result in increased bridge velocities and approximately 3-foot increase to the upstream water surface elevation for the 0.01 AEP event. Additionally, calculated scour depths doubled over the existing condition, and larger rock size would be required at the abutments due to increased velocities. This Alternate 2 is not only impractical and not recommended for practical reasons, but large increases to the 0.01 AEP event would also remove this option from consideration because the bridge is in a FEMA Zone AE, which would require no more than a 1-foot increase to 0.01 AEP WSELs. Alternative 1 only slightly affects flooding conditions, but patterns generally remain the same as no resilient design.

Table C-11: Resilient Design Checklist for the 0.01 AEP Event

	Parameter	Existing Condition	Future Condition (No Resilient Design)	Future Condition (Resilient Design Alt 1)	Future Condition (Resilient Design Alt 2)
0.01 AEP Event	Discharge (cfs)	9,950	12,900	12,900	12,900
	Bridge Span (ft)	50	50	62.5	95
	Bridge Hydraulic Open Area (ft^2)	324.96	324.96	396.84	637.12
	% Q Bridge	23.8	15.3	18.0	56.7
	Bridge Velocity (fps)	7.29	6.09	5.86	11.49
	Scour Depth (ft)	4.04	2.92	2.65	8.02
	Riprap Size	--	--	R-6	R-8
	Upstream WSE (ft)	244.65	245.29	245.21	247.74
	Overtopping Depth (Roadway) (ft)	3.57	4.21	4.13	3.16

Sensitivity analysis is a powerful tool for evaluating the hydraulic effects of estimated changes to the proposed design without the significant effort required to develop detailed resilient design options for a proposed bridge and roadway. In this case, a conceptual analysis was able to determine that the significant changes to the roadway profile to accommodate a larger structure are not hydraulically feasible and are not effective for resilient design at this site.

Economic Analysis

Based on the alternative design analysis performed for the Station Road structure, there is not a practical or cost effective structural or roadway solution that fixes the existing flooding issues at the site. Alternative 1 provided a reasonable replacement design option that increases the hydraulic opening of the structure and would be a typical design for a District bridge replacement. This option slightly improved WSELs, but general flooding issues remained, including the overtopping of the 0.10 AEP flood. Alternative 2 explored a scope of design required to pass the 0.10 AEP flood, and required a large increase in roadway profile, along with drastic widening of the opening. The results showed the passing of the 0.10 AEP flood, but greatly increased WSELs for all higher flood events. This option would be extremely expensive, not only by replacing the structure with a much larger one, but increasing the roadway profile, which affects the adjacent roadways, and expands fill limits with likely environmental impacts as well. The 0.01 AEP flood increases present in Alternative 2 would also disqualify the project due to FEMA requirements, which do not allow increases of more than one foot.

In addition to WSEL impacts, velocities through the opening are also increased for Alternative 2, with approximately a 100% increase in scour depth for the 0.01 AEP event. Additional rip-rap scour protection would be needed at a minimum, using an R-8 rock size based on District requirements. From **Table C-11**, the future condition (no resilient design) and Alternative 1, velocities are decreased over existing due to increased flood depths, which results in decreased scour depths.

If an analysis shows that structure changes will not be feasible to mitigate severe existing or future flooding, it may be worth discussing structure or roadway abandonment in very limited cases. For the Station Road structure, this option was briefly explored due to the infeasibility of a structure design that would mitigate the design flood. Station road itself looks to be a somewhat redundant roadway, servicing only a few residences for access to Creek Road to the north. All residences south of Station Road have other means of outward travel from their respective driveways and would only need to detour approximately 0.5 miles to the west for access to Creek Road. Instead of maintaining or replacing the Station Road structure, the roadway could be abandoned and ultimately removed to fix the damming effect the existing road has on the floodplain. There is not much impact to the surrounding area, and the funds that might have been used on this structure could be rerouted to a more critical structure on the same reach.

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