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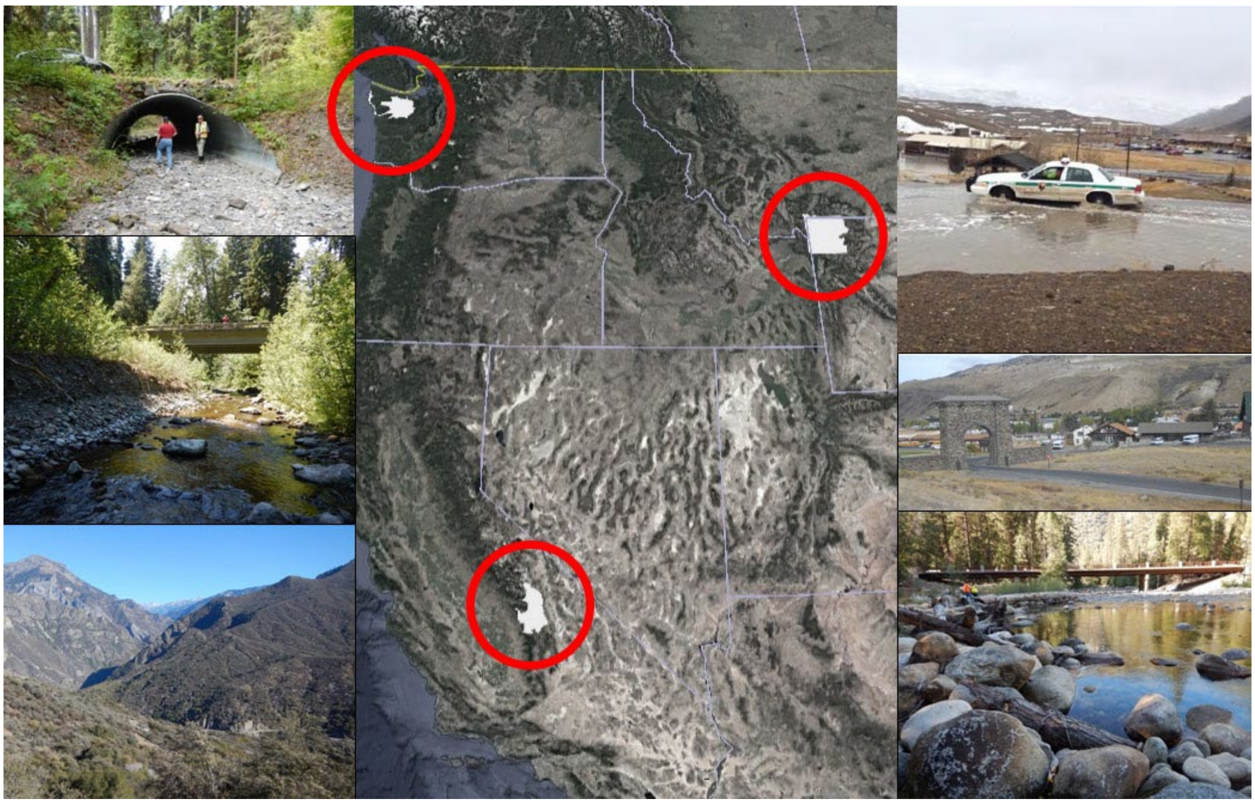

Infrastructure Resilience to Extreme Events & Climate Change

Federal Lands Sensitivity Case Studies

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16. Abstract The FHWA has provided technical resources such as HEC-17 *Highways in the River Environment*, to help analyze the vulnerability of transportation infrastructure in relation to extreme events and climate change. Following this philosophy, this document collects a series of Case Studies that examine the resilience of infrastructure against different extreme events representing potential climate change scenarios. The location of these Case Studies was within four National Parks. The rationale for focusing on National Parks was an attempt to disaggregate direct changes in precipitation or streamflow characteristics from those associated with land use or urbanization. The research investigated the sensitivity of extreme events (i.e., extreme weather events and climate changes), manifested by precipitation and streamflow, upon the resilience of transportation infrastructure hydraulic appurtenances (e.g., roadside drainage, storm drains, culverts, or bridges).			
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Acknowledgments

The cover image is a mosaic showing (from the top) photos of: (A) looking upstream at the West Twin Creek Bridge located in the Olympic National Park, Washington (FHWA WFLHD 2015), (B) looking upstream at Snider Creek Culvert located in the Olympic National Park, Washington (FHWA WFLHD 2015), (C) the North Entrance to Yellowstone National Park and the Roosevelt Arch. The town of Gardiner, Montana in the background (WFLHD 2015), (D). runoff ponding at road profile sag just west of the North Entrance to Yellowstone National Park and the Roosevelt Arch, located in Gardiner, Montana (WFLHD 2015), (E) looking downstream at the Cedar Grove Village Bridge spanning the South Fork Kings River in Kings Canyon National Park, California (FHWA CFLHD 2012), (F) photo of Sequoia and Kings Canyon National Parks (CFLHD 2012). The central image depicting locations of the Parks is copyright property of Google® Earth™ and can be accessed from https://www.google.com/earth (Google 2021). The authors developed the map overlays for that image.

Unless noted otherwise, the FHWA developed all photographs and graphics in this document. The US National Park Service (NPS) photographs came from the https://npgallery.nps.gov website, with each photo being public domain with the NPS fully granting rights for use in this document. The US Geologic Survey (USGS) Topographic map images are public domain with the USGS fully granting rights for use in this document.

In Figure 2, Figure 9, Figure 11, Figure 21, Figure 23, Figure 27, Figure 29, Figure 30, Figure 31, Figure 32, Figure 35, Figure 37, Figure 45, Figure 47, Figure 53, Figure 55, Figure 62, Figure 64, Figure 69, Figure 71, Figure 75, and Figure 76, the original maps are the copyright property of Google® Earth™ and can be accessed from https://www.google.com/earth (Google 2021). The authors developed the map overlays for this document.

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Supporting Purpose

The FHWA developed this manual, in part, to support 23 U.S.C. § 503(b)(3)(B)(viii), that directs the Department of Transportation to *"… carry out research and development activities … to study vulnerabilities of the transportation system to … extreme events and methods to reduce those vulnerabilities"*. The manual also supports Executive Order 14008 directives to that seeks to *"increase resilience to the impacts of climate change"* [§ 203(b)] and *"bolster adaptation and increase resilience to the impacts of climate change."* [§ 211] by providing a potential framework to achieve such objectives.

List of Acronyms

AASHTO	American Association of State Highway and Transportation Officials
AEP	Annual Exceedance Probability
CFR	Code of Federal Regulations
CFLHD	Central Federal Lands Highway Division
CMIP	Coupled Model Intercomparison Project
CU	US Customary Units (English units)
DOT	Department of Transportation
EGL	Energy Grade Line
EO	Executive Order
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
FIRM	Flood Insurance Rate Map (FEMA acronym)
FIS	Flood Insurance Study (FEMA acronym)
FLHD	Federal Lands Highways Division
FLMA	Federal Land Management Agency
GIS	Geographical Information System
HDS	Hydraulic Design Series
HEC	Hydraulic Engineering Circular (FHWA)
HEC	Hydrologic Engineering Center (USACE)
HEC-RAS	Hydrologic Engineering Center's River Analysis System
HGL	Hydraulic Grade Line
IPCC	Intergovernmental Panel on Climate Change
LIDAR	Light Detection and Ranging
NAVD 88	North American Vertical Datum of 1988
NCA	National Climate Assessment
NED	National Elevation Dataset
NOAA	National Oceanic and Atmospheric Administration
NPS	National Park Service
NRCS	Natural Resources Conservation Service
NWS	National Weather Service
PDDM	FLHD Project Development and Design Manual
RAS	River Analysis System (HEC-RAS)
SCS	Soil Conservation Service (now NRCS)
SDOT	State Department of Transportation
SEKI	Sequoia and Kings Canyon National Parks
SH	State Highway
SI	System International (metric system of units)
SMS	Surface-water Modeling System
US	United States of America

USACE	US Army Corps of Engineers
USBR	US Bureau of Reclamation
USDA	US Department of Agriculture
USDOT	US Department of Transportation
USGS	US Geological Survey
WCRP	World Climate Research Programme
WFLHD	Western Federal Lands Highway Division
WSEL	Water Surface Elevation

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Part 1. Background Information

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Executive Summary

The Federal Government owns or controls approximately one-third of the total land area of the United States. Several Federal Land Management Agencies (FLMAs) are responsible for managing these lands; including the vast roadway infrastructure used to access them.

The Federal Highway Administration's (FHWA) Office of Federal Lands Highway (FLH) provides technical services to FLMAs, States, Territorial partners, and Tribal governments (and peoples) to improve transportation to and within Federal and Tribal lands.

For more than 100 years, FLH and its predecessor offices have provided technical expertise to other Government agencies for the planning, location, design, and construction of Federal lands transportation facilities.

The investigation described in this document was the result of two separate but aligned concerns. The FLMA partners raised a concern on the potential impacts of climate change on the transportation projects that FLH helps to deliver. Like other engineering fields, the design of hydraulic structures follows practices that provide a measure of safety and resiliency. However, the amount of resiliency realized from these practices has never been fully quantified, especially in the context that these FLH projects exist.

During this same period, the FHWA Office of Bridges and Structures hydraulics staff had pondered means to better understand sensitivity of climate induced changes to transportation projects. The FHWA recognized the FLMA had access to areas, such as National Parks, relatively unaffected by development changes.

To align these insights and concerns, the FHWA Office of Bridges and Structures enlisted the FLH hydraulics group to: (1) investigate the potential impacts of climate change on recently constructed transportation projects in these areas; (2) use FLH's specified approaches (FHWA 2014); and (3) better quantify resiliency of designed hydraulic structures using those approaches.

The investigations (or studies) chose several recently completed projects from three general regions: Sequoia and Kings Canyon National Parks (SEKI), Hoh River Watershed as part of the Olympic National Park, and Yellowstone National Park. For research purposes, an advantage for using these sites is that the land uses are relatively stable; meaning that hydrologic and hydraulic analyses could more readily consider potential changes from climate in a comparable manner.

The projects also exemplify a range of typical FLH hydraulic designs. The project list includes: two bridge structures, a small culvert, a curb and paved ditch, an aquatic organism passage (AOP) structure, a large culvert, bank stabilization, and a storm water drainage system. For each project site, the study performed a sensitivity analysis, increasing the design flows by 10%, 20%, and 50% from the original, baseline values. The resulting impacts of these increased flows were then evaluated using the original FLH designs.

In general, these studies indicate that the FLH hydraulic designs are robust and provide a significant level of resilience for increased precipitation or flows. Each of the project sites held up under the potential flow increases of 20% and many sites are expected to function with little modification under a 50% increase in flow. It is difficult to quantify an exact future flow increase due to climate change over the life of a structure. However, wherever a study provided an estimate of such climate change, the level of flow increase was anticipated to be below 50% (i.e., the highest flow analyzed in these studies). This effort finds that the example projects used for this research can be expected to align to increases in flows similar to those that could be expected from a changing climate, over the service lives of the hydraulic structures.

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Chapter 1. Introduction

This document presents a group of Case Studies in National Parks in the western half of the United States that examine the sensitivity of roadway infrastructure to extreme events representing potential climate change scenarios. The Federal Highway Administration (FHWA) has long promoted designs accounting for changes in future conditions potentially experienced during the service life of roadway infrastructure. These changes can come in many forms including: changes in land use of the watershed, which can reduce infiltration and increase runoff; changes resulting from stream morphology, which can modify flood storage in the system or change flood paths; and climate change, which can increase the intensity of precipitation. Accounting for these potential future changes in project planning and/or design, can greatly improve the resilience of the structures (Federal Register 2021).

1.1 Sensitivity Analyses

A ***sensitivity analysis*** is one such method engineers use for testing the resilience of an asset against these changing conditions (or independent variables, etc.). A normal process for a design project would involve using existing data to determine what the current hydrologic input for a model should be. The process uses that input to determine how the infrastructure reacts to different flows within hydraulic models, scour equations, etc. Practitioners generally describe this normal process as a hydrologic and hydraulic (H&H) design or analysis.

1.1.1 Contrasts between H&H and Sensitivity Analyses

In contrast, a sensitivity analysis may change some independent H&H variable by some range of percentages, for example, increasing them by 1%, 5%, 10%, 20%, or 50%, etc. within an equation or model or process. An outcome might be to observe whether, compared to the baseline (i.e., normal H&H) results, that model has some corresponding increase as well. A change may not become apparent at the 1% to 5% change but may exhibit a non-linear increase once reaching the 10% or 20% level.

The sensitivity analysis for these hydraulic modeling examples tests how "sensitive" the structure would be from changes to the (usual) hydrologic inputs.[1] In such scenarios, the engineer performs the sensitivity analysis by inputting higher flow values into hydraulic models and examining if and how the interactions with the infrastructure change. Questions to look for in a sensitivity analysis after inputting higher flows generally relate to two main categories: structure capacity and structure stability.

To illustrate, using a bridge example, the practitioner may monitor the sensitivity of ***structure capacity*** by checking the potential decreases in freeboard for a given design event. Additionally, the practitioner may monitor the sensitivity of ***structure stability*** by analyzing potential increases in scour. For hydraulic structures, capacity and stability are not mutually exclusive. In this bridge example, overtopping of an under-capacity bridge which can create hydraulics that can, in turn, result in pressure flow, increasing vertical contraction scour, and ultimately impacting the stability.

1.1.2 Contrasts with Other Approaches

The sensitivity analyses are also different from analyses using approaches to determine projected future changes in precipitation or streamflow. Using a five-level framework, the FHWA Hydraulic

[1] Alternatively, the engineer may consider other variables, such as loss of land cover (perhaps from a wildfire), increased debris affecting channel roughness conditions, or drought conditions on receiving streams, etc.

Engineering Circular (HEC) number 17 (HEC-17) *Highways in the River Environment - Floodplains, Extreme Events, Risk, and Resilience* (FHWA 2016) describes H&H approaches (e.g., HEC-17 Level 1), sensitivity based approaches (e.g., HEC-17 Level 2), as well as some approaches for determining and applying such projected future conditions (HEC-17 Levels 3, 4 and 5). Essentially, the three highest levels (i.e., Levels 3, 4, and 5) incorporate climate change projections that affect the depth/intensity of precipitation or the quantity/duration of flow, and these effects vary geographically. HEC-17 also describes models and tools, such as the Coupled Model Intercomparison Project (CMIP) Climate Data Processing Tool (FHWA 2021a), that provides the ability to obtain certain types of projected future information.

The case studies in this document do not focus on projections of future precipitation or flows. Rather it adopts HEC-17's Level 2 analyses approaches.

1.2 Federal Lands Highway Division

The US Department of Transportation (USDOT) established the FHWA Office of Federal Lands Highway (FLH) to promote effective, efficient, and reliable administration for a coordinated program of Federal public roads and bridges; to protect and enhance our Nation's natural resources; and to provide needed transportation access for Native Americans. The FLH primary purpose is to provide financial resources and transportation engineering assistance for public roads that service the transportation needs of Federal and Indian lands. (FHWA, 2021) The FLH provides these services in all 50 states, the District of Columbia, Puerto Rico, and US Territories through our Headquarters, Eastern, Central, and Western Federal Lands Highway Division (FLHD) offices (Figure 1). Since the early 1900's, these services include the original planning, design and construction of many roads within our National Parks and National Forests.

Figure 1. FHWA Federal Lands Highway Division geographic locations.

Two FLHD Offices – Central FLHD (CFLHD) and Western FLHD (WFLHD) – independently performed a group of sensitivity analyses to explore potential effects of climate change to a variety of infrastructure projects. The selection of projects within National Parks assumed that many have stable and protected land use. Therefore, the research could neglect changes in those aspects of potential hydrological driving factors. Although stream morphology is still a factor, based on cursory site studies, the Case Studies assumed these factors were negligible as a hydrologic driving force. By doing so, the Case Studies assumed selection of these sites within the National Parks allowed for isolating climate change as the driving factor for future potential change.

1.3 Scope of Research

The purpose of this research was to perform a group of sensitivity analyses and determine how sensitive the different infrastructure projects within the National Parks were to future risks from climate change. Infrastructure projects are vastly diverse in their vulnerabilities and challenges. Even with large sample sizes of projects, it is rare if not impossible to find "one size fits all" rules. Thus, the objective of this document is not to extrapolate generalizations for all projects from just a handful of cases. Rather, it is to explore the sensitivity of various types of projects and help designers and other practitioners think through their own sensitivity analyses and learn what to pay attention to.

The research selected four National Parks (see Figure 2) as candidates for these sensitivity analyses. These candidates are the adjacent Sequoia/Kings Canyon National Parks (California); Yellowstone National Park (Wyoming, Idaho, and Montana); and Olympia National Park (Washington State).

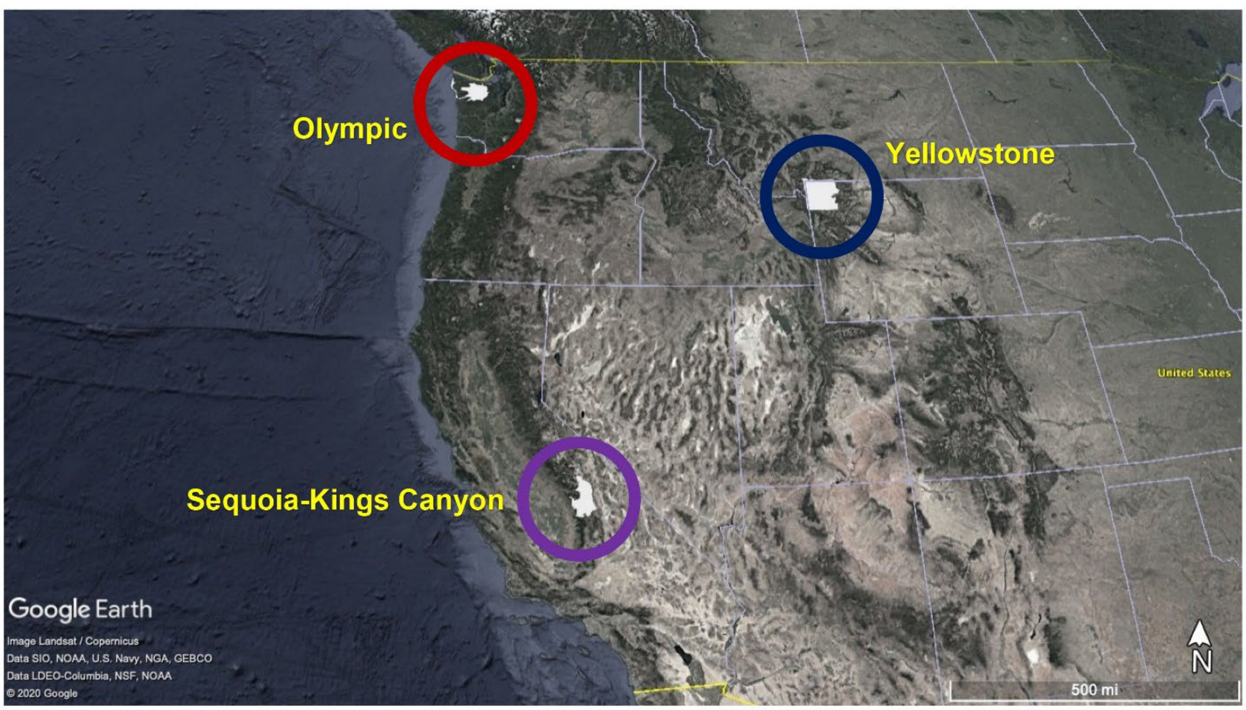

Figure 2. Location of Four National Parks (Google 2021).

Given the range of potential projects within the four National Parks, the methodologies employed in the research may vary from site to site. This document attempts to provide some structure and consistent formatting as much as possible. However, readers may notice that different members of the FLHD team applied slightly different approaches and presentation of information within their research investigations. Again, the FHWA suggests a focus on the sensitivity of the Case Study, rather than, for example, one approach using the Rational method versus another the curve number; or the detail of the hydraulic output of 2D modeling efforts; or the number of photos taken for a given site. Additionally, maps, photos, and images may vary from park to park, indicating available resources.

This document reflects work over the period from 2016 to 2021. Some of the technical sources and citations reflect the period (2016 to 2017) when research for this document was actively underway. Additionally, to take advantage of even earlier (previous) efforts and information, this 2016 to 2017 work obtained and applied information from older FHWA design studies conducted for the US National Park Service (NPS) (as noted in specific case studies).The complexity of merging and collating the different research and investigations into a cogent and meaningful information resource and other factors contributed to the time needed to publish this document. Meanwhile, other research relating to the sensitivity of roadway infrastructure to extreme events and climate change impacts has continued to update earlier work (FHWA 2021). Instead of updating all sources cited in this document and inserting further analysis to reflect those updates, the FHWA has generally retained the document and sources as originally drafted to focus on the Case Studies, updating only as needed to ensure it is understood by readers.

1.4 Document Organization

This document has Parts and Chapters. The Parts collect more substantial information, for example, the Background Information, National Park Case Studies or Summary and References. Within each Part are Chapters, containing information on, for example, a site location or H&H approach or type of hydraulic appurtenance (e.g., bridge versus culvert, versus drainage etc.).

Part 1 – Background Information, discusses the background and context of these Case Studies, including the following Sections and Chapters:

- Executive Summary
- Chapter 1. Introduction
- Chapter 2. Overview of Parks
- Chapter 3. Overview of Case Studies

Part 2 – Case Studies, collects provides information relating to the Case Studies:

- Chapter 4. Cedar Grove Village Bridge
- Chapter 5. Cedar Grove Wildfire Scenario
- Chapter 6. Culvert at the Wye
- Chapter 7. Curb & Gutter
- Chapter 8. Gardiner Stormwater Management
- Chapter 9. West Twin Creek Bridge
- Chapter 10. Snider Creek Culvert
- Chapter 11. East Twin Creek AOP Culvert
- Chapter 12. Hoh River Bank Stabilization

Part 3 – Findings and supporting information for the document:

- Chapter 13. Findings
- References
- Appendix A. Unit Conversions

1.5 Target Audience

This document is written for a wide cross-section of users with varying backgrounds and expertise. The target audience is civil engineers, hydraulic engineers, roadway designers, planners, environmental staff and scientists. Part 1 and Part 3 are for those interested in the findings of these Case Studies and reading them does not need a hydraulics modelling background. Part 2, however, assumes the reader has some background knowledge in hydrologic, hydraulic, and scour engineering and modelling as the FLHD investigations go into detail on their modeling approaches and methods.

For the sake of brevity, this document does not detail specific equations or theory behind methods used in typical hydrologic, hydraulic, and scour transportation practice. For example, the document may describe obtaining FHWA pier scour depth estimates or using the Natural Resources Conservation Service (NRCS)[2] curve number method to determine peak discharge, but without also providing the equations from the FHWA HEC-18 *Evaluating Scour at Bridges* or NRCS literature (respectively).

Likewise, the document assumes the reader knows the differences between precipitation and streamflow; or why there is not usually a "one-to-one" relationship between rainfall and runoff. The document assumes that interested readers will consult such FHWA and NRCS resources to improve their understanding of various hydrologic and hydraulic concepts and terms.

1.6 Units in this Manual

With a few exceptions, and consistent with FHWA policy, this manual uses US Customary (English) units. In limited situations, the manual may use both customary (English) units and SI units or use only SI units because these are the predominant metrics nationwide and globally for these topics. In such situations, the manual provides the rationale for the use of units. Appendix A provides some information, including a conversions table.

1.7 Related Materials

Highway Project Development and Design Manual (PDDM) (2014 version)[3] provides "current policies and guidance for the interdisciplinary project development and design related activities performed by FLH Divisions and their consultants" (FHWA 2021). PDDM Chapter 7 "Hydrology & Hydraulics" serves as the source of majority of the criteria and technical approaches within this document.

HDS-2 Highway Hydrology, 2nd Edition collects background on various hydrologic theory and methods applied in this document. That includes methods developed by other agencies and sources (such as NRCS Curve Number) used in Hydrologic practice. HDS-2 also has a comprehensive glossary of terms that related to hydrology.

HEC-17 Highways in the River Environment: Floodplains, Extreme Events, Risk, and Resilience 2nd Edition provides valuable background information for this document. The Case Studies explored in this document are just a small application of the substantial content within HEC-17.

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[2] Formerly the Soil Conservation Service (SCS). This document may use these two acronyms interchangeably, depending on the date of the cited reference.

[3] In 2018, there was an update to the 2014 version of the PDDM. However, unless stated otherwise, analyses in this document reflect the 2014 version and associated content and processes.

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Chapter 2. Overview of Parks

The research investigated thirteen Case Studies located within four US National Parks – Sequoia, Kings Canyon, Yellowstone, and Olympic. This Chapter provides a brief overview of these parks.

2.1 Sequoia & Kings Canyon National Parks

The US government established the Sequoia National Park in September 1890[4] and Kings Canyon National Park in October 1890[5] (NPS 2021). These parks are geographically adjacent. Since 1943, the National Park Service has jointly administered them as the "Sequoia and Kings Canyon National Parks" or SEKI (SEKI 2014, NPS 2021).

The parks locations are in the southern Sierra Nevada range, east of the San Joaquin Valley. Found within Tulare County, California, the area of these parks is 1,351 square miles (mi^2). Weather varies by season and elevation, which ranges from 1,370 feet to 14,494 feet (NPS 2021). The SEKI are known for their forests and mountainous venues, supporting a large and diverse ecology and environment (see Figure 3).

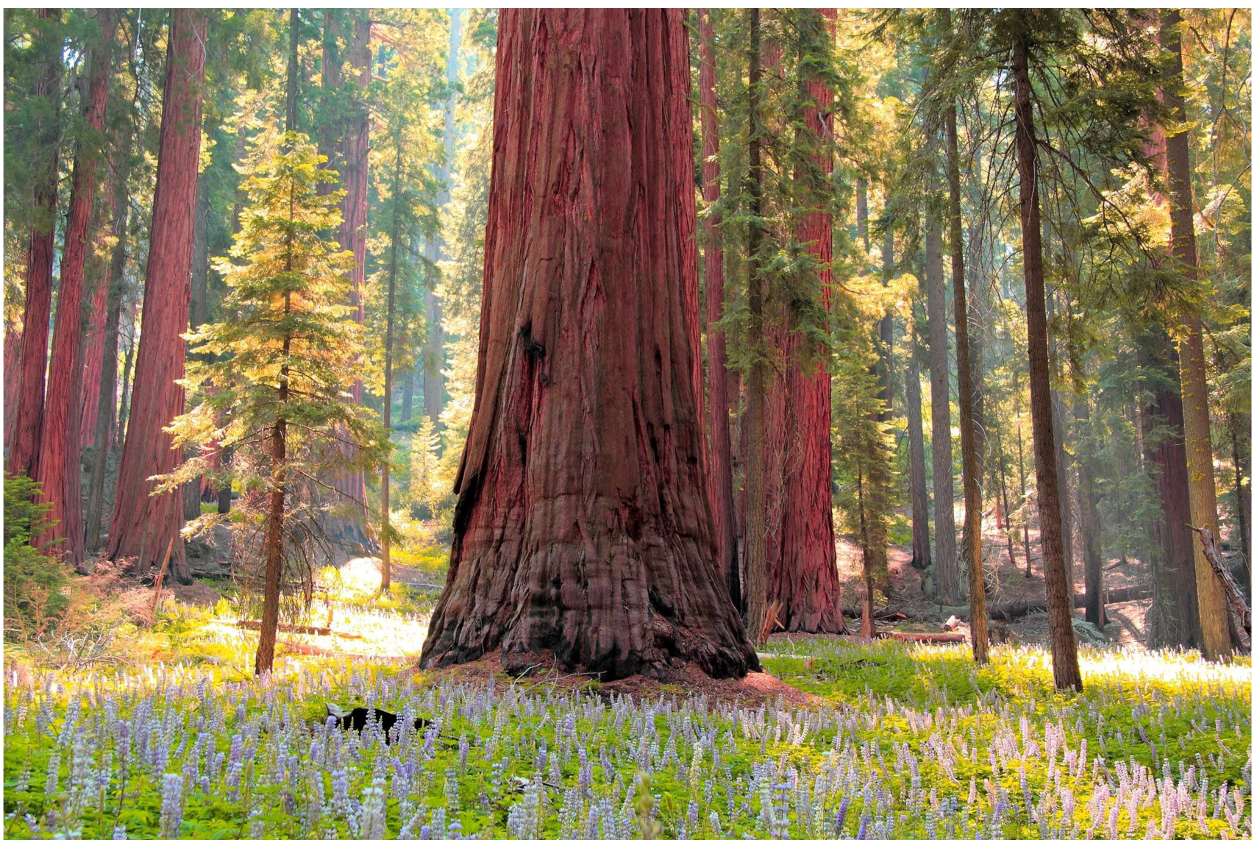

Figure 3. Sequoias found in SEKI National Parks (NPS 2021).

Kings Canyon (Figure 4) is one of the deepest canyons in North America -- even deeper than the Grand Canyon! The Kings Canyon Scenic Byway – California State Highway (SH) 180 – provides access to the Parks. Another highway, California SH 198, also serves portions of the Parks.

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[4] 26 Stat. 478, 16 USC 41 (September 25, 1890).

[5] 26 Stat. 650 (October 1, 1890), 54 Stat. 41, 16 USC 80a (March 4, 1940)

Figure 4. View of Kings Canyon (NPS 2021).

The adjacency of Sequoia and Kings Canyon National Parks supported formulating joint Case Studies. The investigation performed Case Studies at the Cedar Grove Road Village Bridge, Culvert at the Wye, and a Curb and Gutter analysis along California State Highway (SH) 180. The Cedar Grove Bridge site also allowed a watershed level sensitivity analysis focused on increased flooding from changes to land use because of hypothetical wildfires. The FHWA Central Federal Lands Highway Division (CFLHD) provides the US National Park Service (NPS) SEKI engineering services and conducted the SEKI Case Studies.

2.2 Yellowstone National Park

Established March 1, 1872, Yellowstone is the world's first National Park. Yellowstone National Park spans 3,471 square miles in area across the States of Wyoming, Montana, and Idaho (NPS 2021). The elevations in the Park range from 5,282 feet to 11,358 feet (Eagle Peak) (ibid.). Sitting on top of an active volcano, Yellowstone is known for its geysers and other hydrothermal features, diverse wildlife and biota, and wonderful views (ibid.) (see Figure 5).The Park hosts approximately 4 million visitors each year (ibid.).

Gardiner, Montana, located at the north entrance of Yellowstone, is a major access to the interior of the Park. Gardiner is an important community for the Park. The Yellowstone River splits Gardiner; running east to west through the middle of the town, while US Highway 89 is the major road entering Yellowstone, leading through the iconic "Roosevelt Arch" (see Figure 6), a 52-foot high, rusticated triumphal entrance, built around 1903.

Figure 5. Bison grazing near geysers in Yellowstone Park (NPS 2021).

Figure 6. Looking northwest towards Gardiner and Roosevelt Arch on US 89.

US 89 leads from the Arch into Yellowstone National Park (i.e., the "Gateway"). Northward flowing runoff from the Park causes flooding in Gardiner and the Gateway. The Case Study focused on five storm drains and stormwater management facilities in this area.

The FHWA Western Federal Lands Highway Division (WFLHD) provided NPS engineering services and conducted research associated with these Yellowstone Case Studies.

2.3 Olympic National Park

Located in northwest Washington State, the Olympic National Park is a 1,442 square mile venue on the Olympic Peninsula. Established on June 29, 1938, this National Park contains glaciers, rain forests (140 to 170 inches of annual precipitation) and other diverse (and unique) ecological, wildlife and plant species and environmental elements. The topography ranges from mountainous to coastal terrain; with elevations from sea level to 7,965 feet (Mount Olympus). The Park protects one of the largest remaining blocks of old growth forest and temperate rain forest in the lower 48 states (NPS 2021). Congress designated 95% of the Park area as the Olympic Wilderness.

The Hoh River, with a large portion (58%) within the Park, has a 299 square mile drainage area. The river name comes from the Hoh tribal peoples (Hoh Tribe 2021). The Hoh River is approximately 56 miles long. The river originates from Hoh Glacier (see Figure 7) and flows into the Pacific Ocean. However, during the journey to the Pacific, Hoh River passes through rain forests and obtains flow from other glaciers (NPS 2021).

Figure 7. Hoh Glacier in Olympic Peninsula (NPS 2021)

The Hoh River supports a variety of salmonid fish species, managed by the Hoh tribe in cooperation with the Washington Department of Fish and Wildlife. The Park (and Hoh River) is also the habitat of the Roosevelt Elk, a species only found on the Peninsula (see Figure 8).

Figure 8. Roosevelt Elk within the braided channels of the Hoh River (NPS 2021).

One of the major roads leading into Olympic National Park is the Upper Hoh Road located off US Highway 101 on the far western side of Olympic National Park. The road is the only entryway into the Hoh Rain Forest and the Olympia Park Rain Forest Visitor Center. The Upper Hoh Road is approximately 18 miles in length. Jefferson County owns and maintains the portion of the road from the junction with US 101 to the Park boundary, approximately 12 miles. The Park owns and maintains the remaining 6 miles.

The FHWA Western Federal Lands Highway Division (WFLHD) provided the NPSengineering services and conducted research in these Olympic Park Case Studies.

On the Hoh River and tributaries, the research focused on four structures: West Twin Creek Bridge, Snider Creek Culvert, East Twin Creek AOP Culvert and Hoh River Bank Stabilization. East Twin Creek Culvert also includes analyses on the sensitivity of AOP if low flows reduce the discharge.

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Chapter 3. Overview of Case Studies

3.1 Rationale for Case Study Selection

The research selected the Case Studies to help provide a broad range of potential hydraulic transportation appurtenances, i.e., bridges, culverts, storm drains, scour, stabilization, aquatic organism passage, etc. The intent was to demonstrate some of the approaches encountered during typical transportation practice. The other basis for selection was the availability of (relatively) recent project and design information. This allowed FLHD engineers to include more Case Studies in the research while meeting the constraints of the research budget.

3.2 Case Study Outline & Format

The format for each Case Study initially provides some description and background information related to the site. The next sections describe additional project descriptions, including topography, habitat, and other features. Subsequent sections describe characteristics of any existing structures or appurtenance. As appropriate, these sections also described the design criteria[6] that might be applicable to the Case Study during the period of the original research. Each Case Study notes application of different design criteria.

The next section of each Case Study typically describes the baseline analyses. These include hydrologic, hydraulic, and other analyses of interest. In such sections, the Case Study will describe the applicable methods and tools. Examples include use of US Geological Survey (USGS) regression equations to determine peak flow or application of a type of hydraulic model. The baseline section will also provide sources of these methods to provide the interested reader a means to gain additional background and understanding.

The subsequent sensitivity analyses section parallels the same topics (e.g., hydrologic, hydraulics, etc.) covered in the baseline, but attempts to investigate how they may react to changes in those conditions. These changes nearly always are tied to increased hydrologic conditions, but could describe other changes (i.e., effects of a wildfire). The sensitivity analyses may provide some useful approaches in helping others to apply to their own sensitivity studies.

Finally, each Case Study provides some summary of findings or notable insights.

3.3 Some Caveats

The FHWA cautions the reader against drawing any inferences from any Case Study to reach more general conclusions about causation as applied in different cases. For example, the outcomes of any Case Study may be specific to that given site and its unique characteristics. The FHWA does not recommend the use of any Case Study in a manner that would not align with good science and engineering.

Additionally, for sake of brevity, this document does not contain all information contained in the original (unpublished) FLHD studies. Additionally, for the sake of brevity and organizational clarity, this document omits some Case Studies or Case Study materials in those studies.

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[6] In nearly all cases, this design criteria resided in the 2014 version of the FHWA FLHD *"Highway Project Development and Design Manual"* (PDDM) (FLHD 2014). The PDDM included Standards that the Office of Federal Lands Highway imposes to guide the content of FLHD products. Standards in the PDDM constitute design standards for these types of FLHD projects (see PDDM, Chapter 1), similar to how a Bridge Manual or Drainage Manual may govern a State DOT's project delivery efforts. Variances to FLH Standards are not uncommon, but they need always be justified in writing (Id.).

3.4 Synopsis of Each Case Study

Table 1 lists the Case Studies, including the coordinates, associated National Park, transportation focus or appurtenance of interest, and a short description of the area or site. Note that the location of Case Study sites may not necessarily be within the park proper. For example, the project location might be in adjacent Communities or Counties but have some nexus to the Park.

Table 1. Summary of Case Study sites.

	Case Study	Coordinates	National Park			Appurtenance or Focus	Description
			SEKI	Yellowstone	Olympic		
A	Cedar Grove Bridge	N 36.789962° W 118.669755°	X			Bridge	This is a 2012 bridge replacement over South Fork Kings River (a Wild and Scenic River); part of larger river restoration project and included streambank re-alignment.
B	Cedar Grove Wildfires	N 36.789962° W 118.669755°	X			Watershed	Building upon the Cedar Grove Bridge Case Study, this additional Case Study attempts to determine potential impacts of a wildfire on the hydrology and hydraulics.
C	Culvert at the Wye	N 36.726033° W 118.95499°	X			Culvert	A 30-inch culvert with a 10-foot section of 24-inch culvert attached to outlet. Culvert is under approximately 15-feet of cover. Roadway overtopping occurs during high flow events.
D	Curb & Gutter	36.69989444° W 118.871331°	X			Roadside Drainage	Sheet flow across roadway through a section of Generals Highway results in icy conditions during the winter months. Roadside drainage would mitigate this issue and improve drainage.
E	Main Street Storm Drain	N 45.03054° W 110.70553°		X		Storm Drain	An approximately 1,500-foot-long storm drain line, most of which is 24 inches in diameter, and ranges in slope from 0.17 to 9.59 percent.
F	Fourth Street Storm Drain	N 45.03054° W 110.70866°		X		Storm Drain	An approximately 950-foot-long storm drain line, 30 inches in diameter, and ranges in slope from 0.25 to 11.71 percent.
G	Yellowstone Storm Drain	N 45.02955° W 110.70884°		X		Storm Drain	An approximately 915-foot-long storm drain line, diameter ranges from 24 to 30 inches, and slope ranges from 0.90 to 5.62 percent.
H	East Park Street Storm Drain	N 45.02976° W 110.70552°		X		Storm Drain	An approximately 900-foot-long storm drain line, 18 inches in diameter, and ranges in slope from 0.49 to 22.16 percent.
I	Park Street to Arch Infiltration Swale	N 45.02942° W 110.70671°		X		Infiltration Swale	A large infiltration swale with bottom area of 18,000 square-feet. Ten feet in width and 1,800 feet in length, designed to infiltrate the 100-year design storm with a maximum water depth of 6 inches.
J	West Twin Creek Bridge	N 47.83292° W 123.989832°			X	Bridge	Bridge over West Twin/Twin Creek, a tributary of the Hoh River. The original design for this structure was completed in 2002. A major 2006 flooding event provided incentive for reconstruction in 2007.
K	Snider Creek Culvert	N 47.844146° W 123.96668°			X	Culvert	Snider Creek is a tributary of the Hoh River. During a winter 2006 flood event, a large debris jam formed on the channel upstream of the crossing. The channel rerouted to its current location, approximately 700 feet east of the old crossing, at a historic drainageway. At this location, there was a 36-inch culvert. It was replaced to convey the rerouted channel with heavy debris potential in 2007
L	East Twin Creek Aquatic Organism Passage (AOP) Culvert	N 47.834513° W 123.98929°			X	Culvert & AOP	Culvert replacement to include AOP characteristics for East Twin Creek, a small tributary of Twin Creek. The confluence is immediately upstream of the Twin Creek and Hoh River confluence. This crossing was initially evaluated as a culvert in 2002.
M	Hoh River Bank Stabilization Section	N 47.823° W -124.187°			X	Stabilize Banks	Bank stabilization project to address a section of the Upper Hoh River Road currently within the Historic Channel Migration Zone. Using combination of traditional and nature based solutions.

Part 2. Case Studies

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Chapter 4. Cedar Grove Village Bridge

Cedar Grove Village Bridge is a 280-foot bridge spanning the South Fork Kings River in Kings Canyon National Park, California. A 2012 replacement bridge accommodates the free flow conditions prescribed by the 1968 Wild and Scenic Rivers Act (WSRA). This Case Study will investigate the sensitivity of hydrology, hydraulics and scour at the bridge, contrasting the baseline condition versus 10%, 20%, and 50% increases of streamflow. Figure 9 provides a 3D visualization of Kings Canyon looking eastward up the river and valley.

Figure 9. 3D visualization of Cedar Grove Bridge and Kings Canyon (Google 2021).

4.1 Project Area Description

The Kings River through Kings Canyon is a designated Wild and Scenic River (National Wild and Scenic River System 2021). This includes the tributary South Fork Kings River that flows in a roughly southeast to northwest direction toward the confluence of the Kings River, approximately 13.4 miles downstream. The location of the Cedar Grove Village Bridge (Figure 10) is along California State Highway (SH) 180 (coordinates: N 36.789962°, W 118.669755°). Figure 10 and Figure 11 provide a topographic map and aerial photo of the bridge and surrounding project area.

The project area has plantings of willows and other native plant species on the backfilled abutment slopes, up and downstream of the bridge, to help restore the Wild and Scenic River characteristics. Additionally, there has been restoration and protection of the west bank of the river, which had been impacted by the old bridge constriction. Bed material in the main channel appears to consist of coarse sand and gravel.

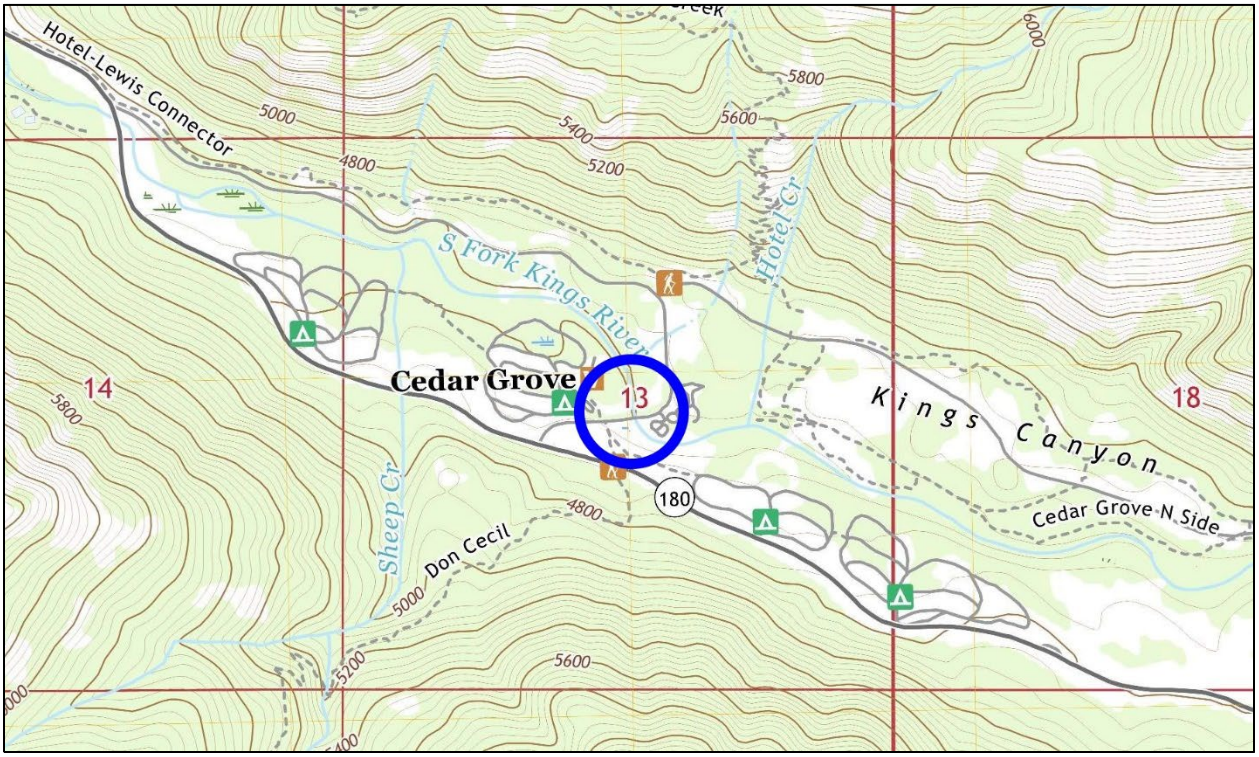

Figure 10. South Fork Kings River Cedar Grove Village Bridge topographic map (USGS 2021).

Figure 11. Aerial view of Cedar Grove Village Bridge (Google 2021).

4.1.1 Bridge Description

Replaced in 2012, the Cedar Grove Bridge (Figure 12) is 280-feet wide and consists of three spans with two 3-foot wide piers; each consisting of three 3-foot diameter columns connected with a debris wall and 2H:1V riprap spill through abutments. The 6-foot thick bridge superstructure slopes at 0.5% downward to the west. Low chord elevations at the left and right abutments are 4,618.4 feet and 4,617.1 feet, respectively.

Figure 12. South Fork Kings River, Cedar Grove Bridge, view upstream of bridge.

The hydraulic design event for the bridge was the 50-year streamflow. The design also used the 50-year event for abutment riprap sizing. The bridge passes the 50- and 100-year flows with approximately 5.3 feet and 3.3 feet of freeboard, respectively, at the upstream face of the bridge. The bridge has had a scour evaluation using the 100- and 500-year events.

4.2 Baseline Analyses

The Case Study began by conducting or recreating the hydrologic, hydraulic, and scour analyses for the bridge. In this and other Case Study Chapters, these analyses will yield the "baseline" values for use in the sensitivity analyses.

4.2.1 Hydrologic Analyses

The Case Study applied a hydrologic/GIS software product to delineate the tributary drainage basin and determine the contributing watershed area (357.6 square miles). The Case Study estimated peak discharges for several return periods using the USGS regression equations for the Sierra Mountain Region (USGS 1993), using the following variable parameters:

- Drainage basin area: 357.6 square miles
- Mean annual precipitation: 32 inches
- Altitude Index: 10 (thousand feet)

The Case Study did not identify any USGS (or other agency) stream gages along this reach of the South Fork Kings River for potential use to validate the computed flows.

Table 2 summarizes the results from the USGS regression equations for the 2- through 500-year peak flows. Table 2 also presents values for peak flow amplified by 10%, 20%, and 50%, as reference for the sensitivity analysis presented later in this section. Of interest in subsequent analyses will be the 50-year, 100-year, and 500-year peaks flows for the four scenarios.

Table 2. Estimated Peak Flows for the South Fork of Kings River at Cedar Grove Bridge.

Flood Event	Peak Discharge (cfs)	Increased Discharge by 10%	Increased Discharge by 20%	Increase Discharge by 50%
2-year	1,600	1,760	1,920	2,400
10-year	5,800	6,380	6,960	8,700
25-year	9,980	10,978	11,976	14,970
50-year	13,300	14,630	15,960	19,950
100-year	18,500	20,350	22,200	27,750
500-year	33,300	36,630	39,960	49,950

4.2.2 Bridge Hydraulics

The analysis used the US Army Corps of Engineers (USACE) Hydrologic Engineering Center’s River Analysis System (HEC-RAS) model to evaluate the bridge hydraulics. The modeled reach extends approximately 500 feet downstream and 1,000 feet upstream of the bridge. The Case Study extracted the cross sections used to compile the model from a digital terrain model (DTM) of the reach. The data that derived the DTM consisted of overbank Light Detection and Ranging (LIDAR) mapping and limited in-channel cross section surveys. The Case Study used 44 cross sections to represent the reach.

The Manning’s n values used to represent roughness in the model were 0.05 for the main channel and 0.08 for the overbank areas. The Case Study determined these values based on calibrated roughness coefficients for similar channels evaluated by the USGS in Water Supply Paper 1849 (USGS 1967). Given the steep slope of the channel (approximately 1.6%), much of the flow is supercritical, which neccesitated use of the HEC-RAS mixed flow regime computations.

The bridge low chord elevation is 4,617.1 feet and the channel thalweg elevation at the bridge upstream cross section is 4,602.1 feet.

These yielded the water surface elevations and associated mean velocities at each of the 44 cross sections, including the bridge. For the 50-year design event, the water surface elevation at the upstream face of the bridge was 4,610.8 feet with an associated freeboard of 6.3 feet. The average cross section velocities in the reach ranges between 6.89 to 12.86 ft/s, with the velocities at cross sections near the bridge averaging approximately 10.16 ft/s.

4.2.3 Scour Evaluations

The Case Study performed scour evaluations using the FHWA HEC-18 *Evaluating Scour at Bridges* methodology and 1D modeling results. Since the 280-foot bridge spans the floodplain, the study assumed there will be no contraction scour. Additionally, since the buried abutments do not constrict the channel, the study assumed there will be no abutment scour. Based on field information provided by others, the study assumed a median diameter (D_{50}) of 9 inches and D_{95} of 2 feet, are reasonable estimates of the bed material gradation near the bridge. The pier nose shape was assumed as rounded, and the angle of attack for the approach flow at the piers was estimated at 10 degrees for Pier 1 and 0 degrees (parallel) for Pier 2.

The scour depths for the 100-year flow was 5.6 feet at Pier 1 and 3.8 feet at Pier 2. Assuming the 500-year flow event yields the maximum scour depths, the evaluation yielded predictions of scour depths of 6.5 feet for Pier 1 and 4.5 feet for Pier 2. The differences between Pier 1 and Pier 2 are a result of the 10-degree angle of attack at Pier 1 effectively increasing the relative pier width (and thus, exposure to scour inducing hydrodynamics).

4.3 Sensitivity Analyses

With these baseline values, the study developed hydrologic “forcings” and performed some sensitivity analyses on several hydraulic and scour related results.

4.3.1 Hydrologic Forcings

A “forcing” (aka forcing functions) describes boundary conditions that "force" the model, usually describing phenomena that cause the movement of water in the model domain. For this Case Study, the forcing functions are the different flow increases (i.e., 10%, 20%, 50%) associated with the design flow event. Table 2 already summarizes the hydrologic increases used as the forcing functions for the sensitivity analyses. Specifically, for the 50-year design event, focusing on four (4) flow scenarios:[7]

- 50-year baseline flow (Q), equal to 13,300 cfs
- Q+10% increase, equal to 14,630 cfs
- Q+20% increase, equal to 15,960 cfs
- Q+50% increase, equal to 19,950 cfs

4.3.2 Hydraulic Sensitivity

The hydraulic modeling determined the one dimensional (1D) derived water surface elevations (WSEL) within the reach (and including the bridge) for the four flow scenarios. At the bridge, the hydraulic modeling also determined the freeboard (i.e., minimum clearance between the water surface and the low chord of the bridge) for each of the scenarios. Recall that the bridge low chord elevation is 4617.1 feet and the channel thalweg elevation at this cross section is 4602.1 feet.

Table 3 presents the hydraulic conditions (i.e., water surface elevation and freeboard) at the bridge for increases in flow of 10%, 20%, and 50%. The Case Study measured these hydraulic parameters at the bridge upstream cross section. The percent increase for the water surface elevation used the increase in depth (i.e., subtracting WSEL from thalweg elevation) to determine the value.

[7] For the sake of brevity, this document will adopt the convention of “Baseline Q,” “Q+10%,” “Q+20%,” and “Q+50%” to describe such scenarios.

Table 3. Hydraulic sensitivity results at Cedar Grove Bridge.

Hydraulic Parameters	Baseline Q	Q + 10%	Q + 20%	Q + 50%
Water Surface Elevation: 50-year				
1D Model Result (feet)	4610.8	4611.3	4611.8	4613.1
% Increase	-	5.7%	11.5%	26.4%
Freeboard: 50-year				
Available Freeboard (feet)	6.3	5.8	5.3	4.0
% Decrease	-	7.9%	15.9%	36.5%

Figure 13 provides a visual example of how these sensitivity analyses function. The three lines represent (1) percent flow increase versus percent flow increase (i.e., a linear line as the values are the same); (2) water surface elevation (WSE) percentage versus percent flow increase; and (3) percent freeboard decrease versus percent flow increase. The three lines appear relatively linear (there appears to be some slight downward trend for both WSEL and Freeboard between 20% and 50%). If either WSEL or Freeboard were sensitive to flow increase, we might expect to see some non-linearity. If either of the two lines exceeded the slope of the line of flow, then again, that may indicate sensitivity.

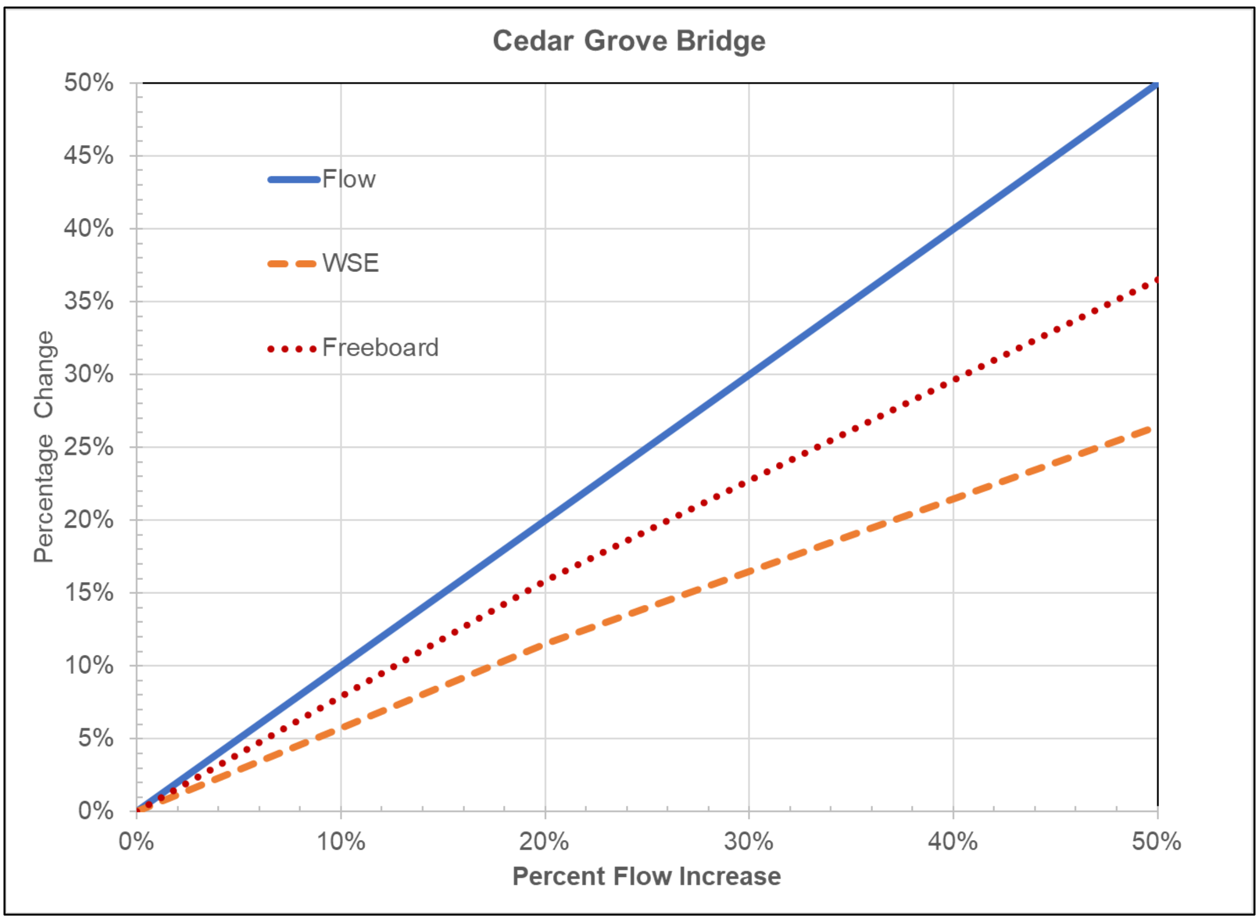

Figure 13. Hydraulic Sensitivity of Cedar Grove Bridge Variables.

The hydraulic analyses reveal that, at this bridge, the percentage of water surface elevations and freeboard changes for those flow scenarios are relatively linear and not expressing any sensitivity compared to the changes in flow. These variables also seem to indicate that increases in flow have some correlation with their increases, but not to the same rate.

Therefore, at this bridge, with the assumptions in the methodology, there appears to be some resilience to increases in flow quantities to water surface elevation and freeboard sufficiency.

However, it is inappropriate to assume that this behavior extends to other bridges (i.e., exhibits causality) or that there are not some other hydraulic constraints having sensitivity to flow increases. For example, the loss of freeboard (increase in water surface elevation) may adversely affect a bridge subject to "no-rise" permit requirements under FEMA's regulations.[8]

4.3.3 Scour Sensitivity

For the four (4) flow scenarios, the study applied the HEC-18 methodology and HEC-RAS modeling results to perform scour evaluations. The Case Study completed local pier scour computations for all flow scenarios.

As described earlier, under several of the flow scenarios, the 280-foot bridge spans the floodplain. Thus, the study did not need to calculate contraction scour or abutment scour. However, for those scenarios where the flow depth was greater than the bridge height, the study calculated pressure flow scour. In such cases of pressure flow scour, the HEC-18 methodology assumes vertical contraction scour occurs and adds that value to the local (i.e., pier and abutment) scour values. Table 4 presents a summary of these scour estimates.

Table 4. Cedar Grove Bridge – Scour analysis results (units in feet).

Scour Variable	Baseline Q		Q + 10%		Q + 20%		Q + 50%	
	100-yr	500-yr	100-yr	500-yr	100-yr	500-yr	100-yr	500-yr
Clearwater (CW) or Live Bed (LB)	CW	CW	CW	CW	CW	CW	CW	CW
Pressure Flow Scour	n/a	n/a	n/a	n/a	n/a	1.3	n/a	4.6
Pier 1								
Scour Depth	5.6	6.5	5.9	6.7	6.0	8.1	6.5	11.5
Total Scour Depth	5.6	6.5	5.9	6.7	6.0	9.5	6.5	16.1
Pier 2								
Scour Depth	3.8	4.5	4.3	4.6	4.4	6.0	4.7	9.1
Total Scour Depth	3.8	4.5	4.3	4.6	4.4	7.3	4.7	13.7
Abutments								
Abutment 1 Total Scour Depth	--	--	--	--	--	1.3	--	4.6
Abutment 2 Total Scour Depth	--	--	--	--	--	1.3	--	4.6

The Case Study predicted maximum scour depths of 16.1 feet (Piers 1) and 13.7 feet (Piers 2), during a 500-year event with a 50% increase in the baseline flow. These scour depths are roughly

[8] See 44 CFR Parts 59, 60, 65, and 70.

2.5 times greater for Pier 1 and three times greater for Pier 2 than the scour depths calculated for the baseline flow conditions. Table 5 depicts the percentage increase for the pier scour for the various scenarios and associated 100-year and 500-year scour evaluations. Figure 14 provides a sensitivity analysis plot of these variables.

Table 5. Sensitivity of Pier Scour at Cedar Grove Bridge.

Scour Variable	Baseline Q		Q + 10%		Q + 20%		Q + 50%	
	100-yr	500-yr	100-yr	500-yr	100-yr	500-yr	100-yr	500-yr
Pier 1								
Scour Depth	0.0%	0.0%	5.4%	3.1%	7.1%	24.6%	16.1%	76.9%
Total Scour Depth	0.0%	0.0%	5.4%	3.1%	7.1%	44.6%	16.1%	147.7%
Pier 2								
Scour Depth	0.0%	0.0%	13.2%	2.2%	15.8%	33.3%	23.7%	102.2%
Total Scour Depth	0.0%	0.0%	13.2%	2.2%	15.8%	62.2%	23.7%	204.4%

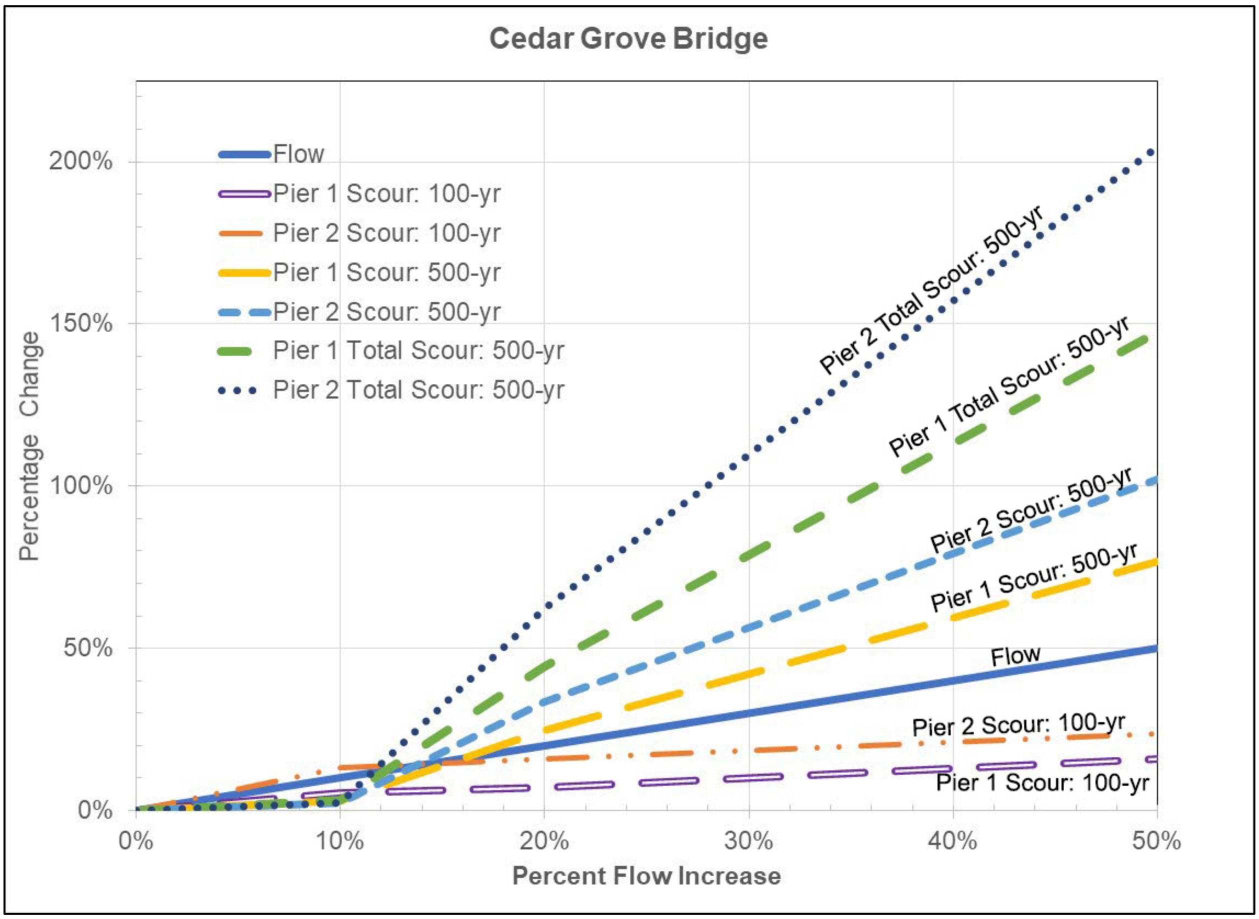

Figure 14. Scour Sensitivity of Cedar Grove Piers.

Figure 14 does not depict the Pier 1 and Pier 2 100-year total scour depths as they do not change from the pier scour values alone (i.e., there is no pressure flow scour). The Pier 1 scour does not appear to be sensitive to the 100-year scenario flows.[9] Both the Table and Figure depict that Pier 2 is sensitive to scour at the $Q_{10\%,100}$ where a 10% increase in flow results in a 13.2% increase in scour depth. However, after that $Q_{10\%,100}$, subsequent increases (i.e., $Q_{20\%,100}$, $Q_{50\%,100}$) fall below the flow line.

This trend seemingly reverses for the $Q_{20\%,500}$ and $Q_{50\%,500}$ scenarios. The sensitivity analyses reveal that once considering the 500-year events and scenarios, the scour depth appears very sensitive to changing flow conditions, e.g., the Pier 2 $Q_{50\%,500}$ scour depth increasing 102.2% from baseline scour (4.5 feet to 9.1 feet). Additionally, after the pressure scour becomes added in, the sensitivity to those changing flows has a dramatic effect on pier scour.

4.3.3.1 Foundation Vulnerability

The results of the scour sensitivity led to some further study of the foundation elements. To evaluate the vulnerability of the structure to the increased flow scenarios, the study compared depth of the foundations for the abutments and the piers to the scour estimates. At the Cedar Grove Bridge, each abutment has nine (9), 51-foot long micropiles and each pier has 27, 43-foot long micropiles. Table 6 summarizes the percentage of the micropiles that would be exposed for each scour condition.

Table 6. Percentage of micropile exposed by scour at Cedar Grove Bridge.

Foundation Element	Baseline Q		Q + 10%		Q + 20%		Q + 50%	
	100-yr	500-yr	100-yr	500-yr	100-yr	500-yr	100-yr	500-yr
Abutment 1 - % Exposed	--	--	--	--	--	28%	--	34%
Abutment 2 - % Exposed	--	--	--	--	--	30%	--	37%
Pier 1 – % Exposed	43%	46%	44%	46%	44%	49%	46%	57%
Pier 2 – % Exposed	41%	42%	42%	43%	42%	46%	43%	53%

A geotechnical engineer would provide an assessment whether exposure of those micropile foundation elements still maintains overall stability of the substructure. If so, then the bridge will remain stable for the potential scour conditions. If not, proper design and additions of scour countermeasures, such as armoring of the abutments and piers with riprap, allow retention of overall scour stability.

4.4 Summary

The Cedar Grove Bridge sensitivity analyses reveals that the structure appears resilient for changes in water surface elevation and freeboard hydraulic scenarios. However, they reveal that increases in flows have an impact on the resulting scour depths. For this reason, the study conducted a further analysis of the foundation elements. The original CFLHD investigation

[9] For the sake of brevity, this document will adopt the convention “$Q_{x\%,rp}$“ to represent these value, where “x” is the percent change and “rp” is the return period. Thus, the 100-year, 10% flow increase is $Q_{10\%,100}$. The document will apply this convention thoughout the Case Studies.

conducted additional analyses and designs to address information raised by the sensitivity analyses.

Chapter 5. Cedar Grove Wildfire Scenario

The previous Cedar Grove Bridge Case Study could assume National Park lands remain undeveloped, and therefore make sensitivity analyses using that assumption. However, impacts of other conditions, such as wildfires, challenge those assumptions. For this reason, the study added an additional Case Study to investigate the sensitivity of the area to wildfire threats.

This Wildfire Case Study differs from the format of others in this document in that it does not vary the flow by certain percentages. Rather this Case Study seeks to contrast the effects of a wildfire by comparing pre- and post-wildfire hydrologic conditions, including peak flows. The original CFLHD investigation went even further in also applying these pre- and post-hydrologic flows to hydraulic modeling runs. For the sake of brevity, this document does not include that hydraulic modeling material.

5.1 Context & Concerns

Sequoia and Kings Canyon National Park maintenance and natural resource staff have expressed concerns regarding the bridge and surrounding riverine area at Cedar Grove Village. Published works by Park natural resource staff indicated this watershed is particularly susceptible to wildfire and will likely increase in susceptibility with climate change (Nydick and Sydoriak, 2011). Forest fires in this wooded, steep sloped watershed could lead to flashier, higher peaked flows and a high potential for debris clogging. Park personnel have put a lot of work into revegetation efforts at this site.

The Cedar Grove watershed upstream of Cedar Grove Bridge is, and will be in the future, highly susceptible to wildfires. The ecoregion containing the Cedar Grove watershed is dependent on periodic fire for health and persistence. A study by Nydick and Sydoriak (2011) notes that because of a century of fire exclusion, the Park landscape has developed a forest structure with heavy fuel accumulations. Their study indicates that recent warming temperatures and a shift toward earlier snowmelt, combined with the heavy fuel accumulation, will result in more frequent lightning ignitions, more area burned, more frequent large wildfires, greater extent of stand-replacing high-severity fire, longer wildfire durations, and longer wildfire seasons. Four climate change scenarios presented by Nydick and Sydoriak forecast an increased probability of large, high severity wildfires from 100% to 400% by the years 2070 to 2099.

The effects of moderate to high-severity fire wildfire include burned soils, hydrophobic soils, removal of vegetative surface cover, and other effects. This Case Study created a hydrologic model to account for a plausible future environmental condition where an extreme wildfire has burned an identifiable portion of the watershed, leading to an increased runoff response.

5.2 Hydrology & Wildfires

The Case Study approach modified land use codes in the Natural Resources Conservation Service (NRCS), Curve Number (CN) method to reflect severity of supposed burn areas and to develop a projected post-fire hydrology.

5.2.1 Curve Number Method

The CN method is an empirical hydrologic approach that simulates a rainfall/runoff relationship. The NRCS presents the CN method in TR-55 (USDA, 1986). The CN method (aka the Graphical Peak Discharge Method) relates the 24-hour rainfall depth (P) (associated with some annual exceedance probability or return period) to depth of direct runoff (Q). The Curve Number itself is an index (between 1 and 100)[10] that represents the combination of a hydrologic soil group (identified by letters A, B, C, D), land use (e.g., plant cover, impervious surface, etc.), hydrologic condition (Good or Poor) and treatment class. The CN index allows derivation of maximum potential retention (S) and initial abstraction (I_a). Given those parameters, the ratio of I_a over P describes the fraction of rainfall that occurs before runoff begins. That ratio also relates to behavior the 24-hour rainfall distribution for regions of the United States (there are four: I, IA, II, and III). After deriving the value of Q (direct runoff depth), the method uses drainage area and unit peak discharge (derived from I_a/P and rainfall distribution type) to yield peak flow.

5.2.1.1 Applicability to Wildfires

Yochum and Norman (2015) found that when regarding rainfall-runoff modeling for wildfire areas, the reliability of the CN method for predicting peak flow from forested, mountainous watersheds is debatable and CN values of such watersheds are not well established for burned conditions. Despite these potential shortcomings, Yochum and Norman (2015) determined that the CN method is a favored approach for predicting the flow responses of wildfire areas due to its relative simplicity, achievable data requirements on large scales, the relatively-short timeframe needed to develop a model, as well as the most qualitatively-reasonable results when compared to actual post-fire runoff events. Further, they conclude the most useful application of CN modeling results is through comparison of different catchment's flood response to identical rainfall events. For peak flow ratios (i.e. post-fire peak flow/pre-fire peak flow), the authors of the 2015 study concluded that CN can be the most effective tool for such comparisons. Given this, the CFLHD developed CN models for the existing (2012) conditions for relative comparison with a modified base model to represent realistic high severity burn impacts on estimated peak discharges.

5.2.2 Hydrology Data & Approaches

The contributing watershed to the Cedar Grove Bridge is 358 square miles (see Figure 15). For the Cedar Grove watershed, the study applied the NRCS rainfall distribution type IA.

The Case Study obtained the rainfall depth (P) using the Precipitation Frequency Data Server of the NOAA ATLAS 14 (NOAA 2021). Using the coordinates of the Cedar Grove Bridge, N 36.789881° and W 118.688928° yielded the annual maximum series, 50-year, 24-hours rainfall depth of 10.6 inches.

When using the CN method, generally, watershed areas should be limited to 3.1 square miles or smaller (USDA, 1986). For larger drainage areas, the method allows delineation of more than one catchment (sub-area) and runoff hydrographs estimated for each. Since the area of the Cedar Grove Watershed is 358 square miles, this meant dividing it into 219 sub-basins with an average area of 1.6 square miles per sub-basin. The Case Study based these divisions on stream density and hydrologic distribution across the entire watershed. Figure 16 presents the sub-basins of the Cedar Grove watershed.

However, the hydrologic model needs to transfer these individual hydrographs downstream until reaching the Cedar Grove Bridge. Capturing and attenuating these flows necessitated using the Muskingum equation as the routing method. In this routing method, equation variables (e.g., inflow (I), channel storage (S), reach distance, wave travel time (K), and empirical weight of the importance of inflow versus outflow (X), etc.) describe the attenuation of the hydrograph as it moves downstream, yielding the outflow hydrograph (O).

[10] Essentially, the higher the number, the more potential runoff. See HDS-2 (FHWA 2002) for these and other hydrologic terms and concepts.

5.3 Baseline Analyses

The baseline analyses describe the hydrologic conditions applicable to the watershed.

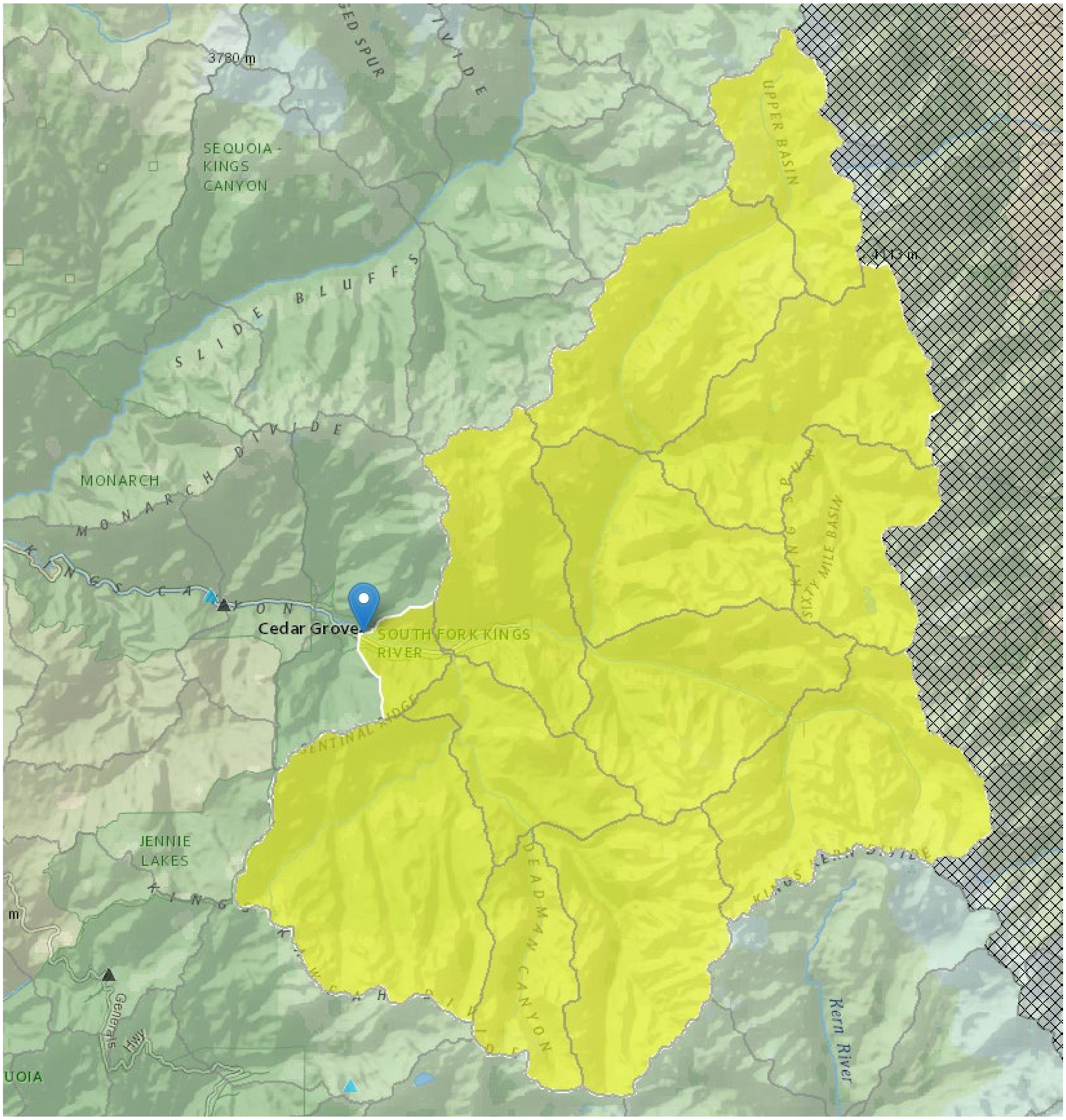

Figure 15. Cedar Grove Bridge watershed (USGS 2021).

As opposed to describing these as hydrological and hydraulic analyses (as is the format in other Case Studies), this Case Study describes them as “Pre-Wildfire Analyses.”

5.3.1 Pre-Wildfire Analysis

The Cedar Grove Bridge sensitivity analysis includes estimates of peak discharges using USGS regressions. However, the relative comparison between pre- and post-fire conditions is not possible using the local USGS regression equation estimates as they lack the ability to reflect changing land and soil conditions occurring during and after these events.

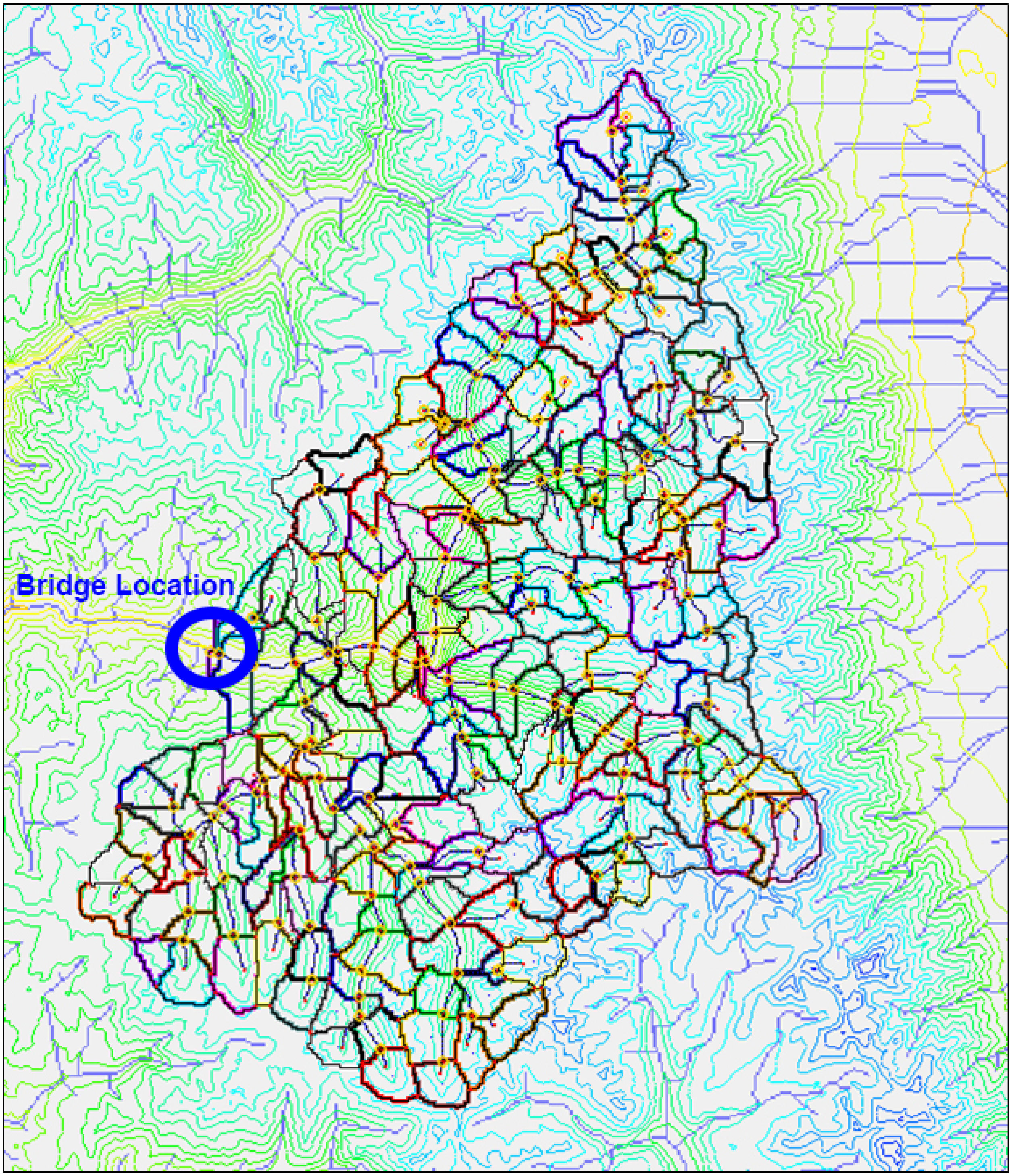

Figure 16. Cedar Grove Watershed Sub-basins.

Therefore, the study developed an existing condition and a post-wildfire wildfire model for the Cedar Grove Bridge watershed. The analysis estimated peak discharges using the CN method, as implemented within the USACE Hydraulic Engineering Center, Hydrologic Modeling System

(HEC-HMS). The Case Study describes the procedure in the following sections (for brevity, this study omits actual theory and equations).

The Case Study assigned the CN for the pre-fire base model based on ground cover conditions, or land use (LU), and the hydrologic soil group (HSG). The Case Study obtained soil data from the Geospatial Data Gateway from the NRCS dataset for the State of California. The Case Study assigned “All Cemented Type D” soils to “Loamy Coarse Sand Type B” based on geotechnical and soils information found in the CFLHD 2012 *Geotechnical Report for the Cedar Grove Bridge Replacement* (CFLHD 2012). Table 7 and Figure 17 present the soil descriptions and spatial distributions, respectively.

Table 7. Soils Descriptions for Cedar Grove Watershed.

Map ID	Description	HSG	Map ID	Description	HSG
s1063	Sandy Loam	B	s1074	Gravelly Sandy Loam	A
s1068	Loamy Coarse Sand	B	s1117	Loamy Coarse Sand	B
s1069	Loamy Coarse Sand	B	s1118	Sandy Loam	B

The Case Study downloaded LU data from NRCS website based on the California Counties of Fresno and Mariposa. Most of the watershed has a cover classification of either Evergreen Forest (ID = 42) or Pasture/Hay (ID = 81). Since the watershed is in a National Park, vegetation was assumed to be in “good” condition. Table 8 presents a key to the LU codes shown in Figure 18 along with the CN for each HSG and each LU code’s description from TR-55. Figure 18 shows the spatial distribution of the LU in Cedar Grove Watershed.

Table 8. Curve Numbers for the Cedar Grove Watershed based on LU and HSG.

Land Use Code	Description	Curve Number for HSG	
		A	B
32	Barren Land (Rock/Sand/Clay) - Barren areas of bedrock, desert pavement, scarps, talus, slides, volcanic material, glacial debris, sand dunes, strip mines, gravel pits and other accumulations of earthen material. Good Hydrologic Condition.	49	68
33	Desert Shrub - major plants include saltbush, greasewood, creosote bush, black brush, bursage, palo verde, mesquite, and cactus.	55	72
42	Evergreen Forest - Areas dominated by trees generally greater than 5 meters tall, and greater than 20% of total vegetation cover. More than 75% of the tree species maintain their leaves all year. Canopy is never without green foliage.	32	58
43	Mixed Forest - Areas dominated by trees generally greater than 5 meters tall, and greater than 20% of total vegetation cover. Neither deciduous nor evergreen species are greater than 75% of total tree cover.	30	48
52	Shrub/Scrub - Areas dominated by shrubs; less than 5 meters tall with shrub canopy typically greater than 20% of total vegetation. This class includes true shrubs, young trees in an early successional stage or trees stunted from environmental conditions.	30	48
81	Pasture/Hay - Areas of grasses, legumes, or grass-legume mixtures planted for livestock grazing or the production of seed or hay crops, typically on a perennial cycle. Pasture/hay vegetation accounts for greater than 20% of total vegetation.	55	69

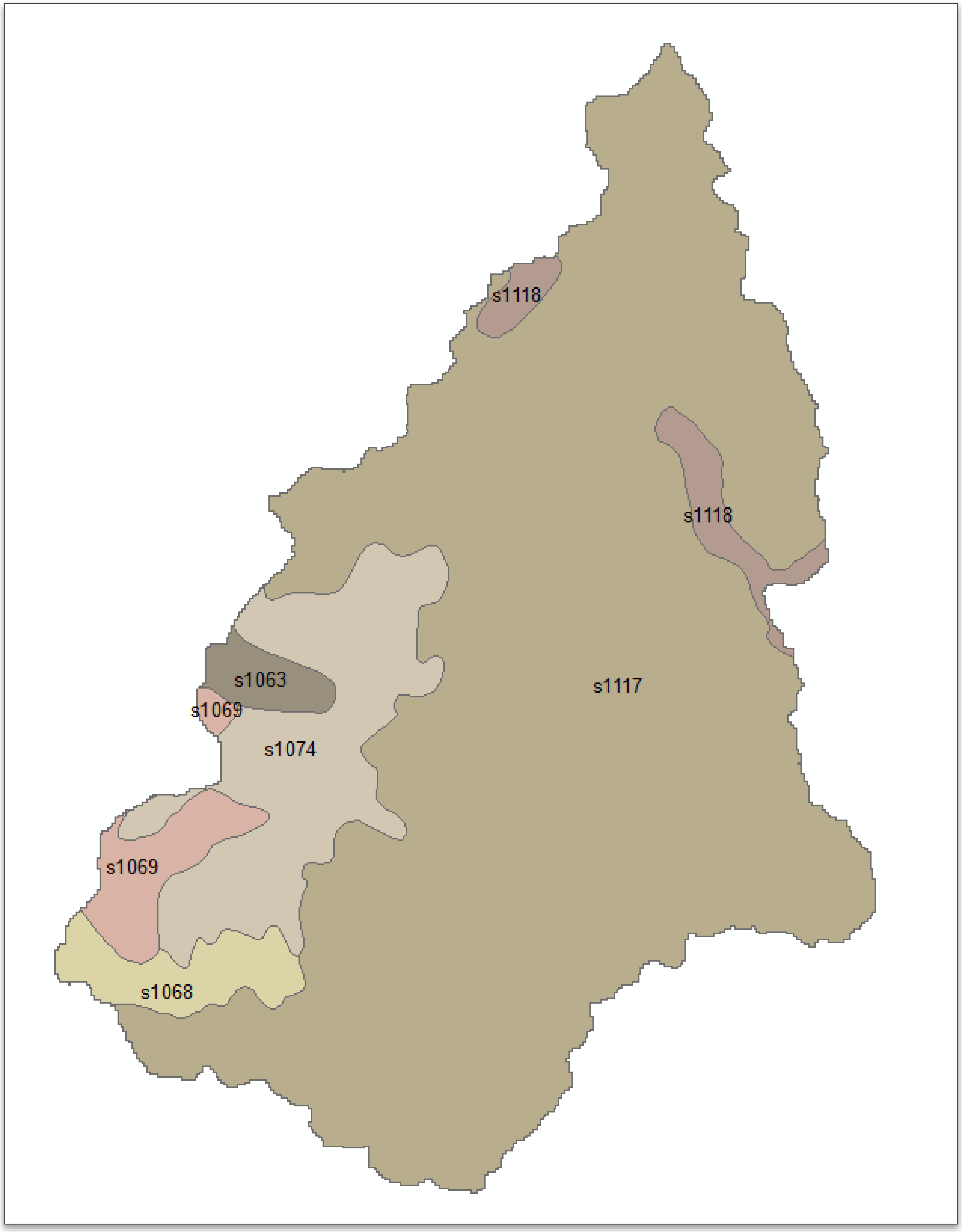

Figure 17. Soils Map of Cedar Grove Watershed.

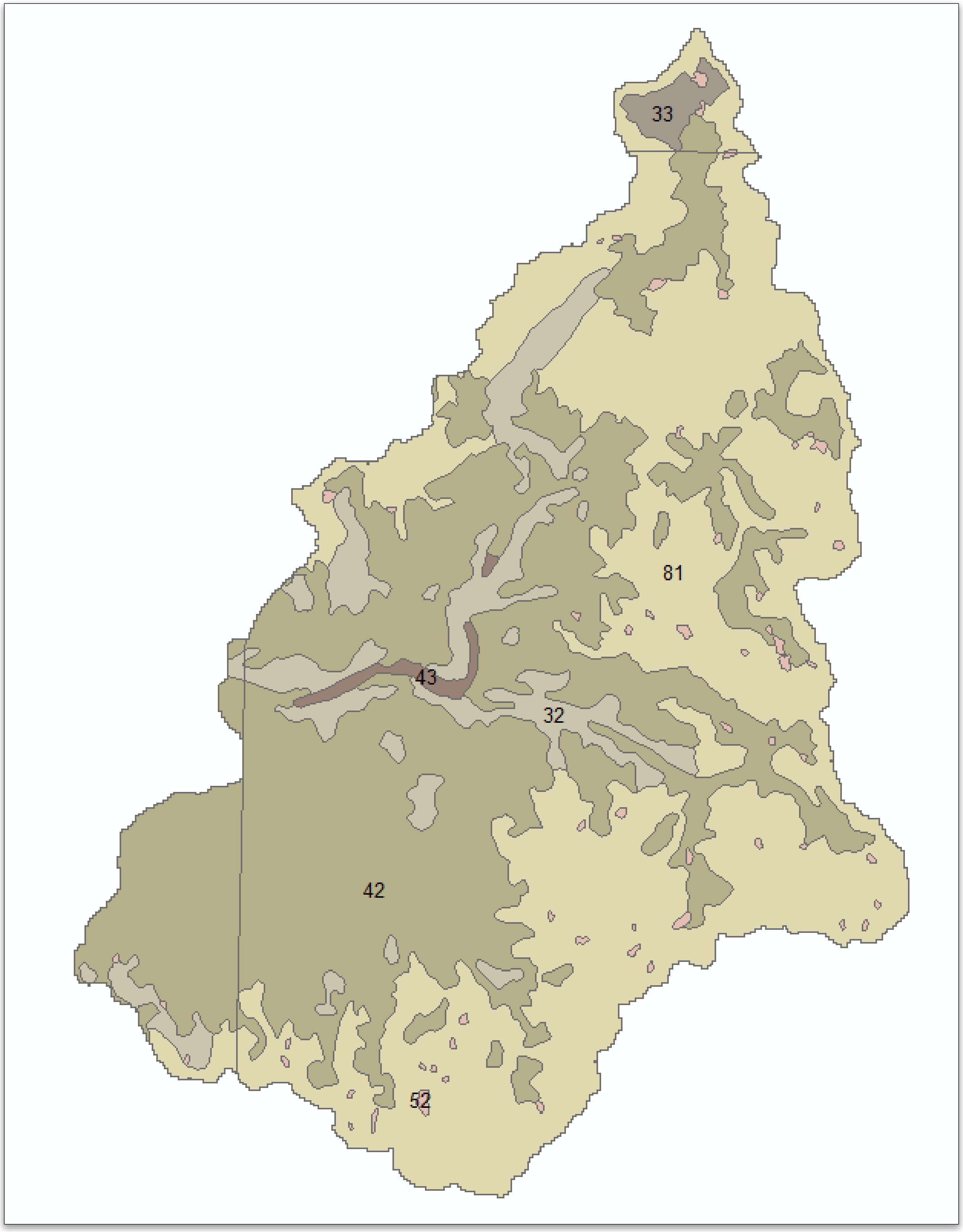

Figure 18. Land Uses of the Cedar Grove Watershed.

The analysis used a hydrologic modeling/geographical information system (GIS) software product to assign a weighted CN to each of the sub-basins. With those values, that software produced a composite (i.e., area averaged) curve number. Table 9 presents the calculations for the composite curve number of Cedar Grove watershed. The total area equals 357.9 square miles and the composite curve number equals 61.

Table 9. Composite Curve Numbers for Cedar Grove Watershed Sub-basins.

Map ID	LU	Area (mi^2)	HSG	CN
s1063	32	2.5	B	68
s1063	42	1.9	B	58
s1063	43	0.5	B	48
s1068	32	1.2	B	68
s1068	42	9.2	B	58
s1068	52	0.1	B	48
s1068	81	0.3	B	69
s1069	42	10.5	B	58
s1074	32	4.2	A	49
s1074	42	36.0	A	32
s1074	43	1.0	A	30
s1074	81	0.0	A	55
s1117	32	21.7	B	68
s1117	33	3.0	B	72
s1117	42	101.6	B	58
s1117	43	0.7	B	48
s1117	52	2.6	B	48
s1117	81	151.9	B	69
s1118	42	1.9	B	58
s1118	52	0.0	B	48
s1118	81	7.0	B	69

5.3.2 Pre-Wildfire Peak Flows

Table 10 presents the peak flows at Cedar Grove Bridge for the existing conditions, pre-wildfire hydrologic base model.

Table 10. Peak Flows for the Cedar Grove Village Bridge Watershed.

Flood Events and Peak CN method discharges (in cfs)						
2-year	5-year	10-year	25-year	50-year	100-year	500-year
2,540	7,500	13,980	24,550	34,410	45,530	74,380

5.4 Sensitivity Analyses

Given the baseline hydrology for the entire watershed, the Case Study sought to understand the behavior after some hypothetical wildfire event (i.e., "Post-Wildfire"). For these analyses, the Case Study drew upon literature made available by SEKI Park staff. Some of the literature was specific to SEKI, others provided more general information.

5.4.1 Post-Wildfire Analysis

Per Parsons et al. (2010), accounting for changes in runoff hydrology due to wildfire is generally a matter of determining the extent to which runoff has been accelerated due to the loss of vegetation or the lack of infiltration due to soil hydrophobicity. Forested watersheds in unburned conditions may be dominated by saturation-excess overland flow when rainfall depths exceed the soil capacity to retain water, where runoff is produced from relatively small and variable portions of a catchment (Yochum and Norman, 2015). Newly burned catchments, on the other hand, may be dominated by infiltration-excess overland flow, where surface runoff is generated when rainfall intensity is greater than soil infiltration capacity, and flow runs down the hillslope surface (Yochum and Norman, 2015). Yochum and Norman (2015) concluded that, in general post-wildfire, saturation-excess overland flow may likely be the dominant source of streamflow for storms of lower intensity while infiltration-excessive overland flow may be dominant during high-intensity storms.

The Case Study research recognizes that potential of increased debris loads is also associated with post-wildfire conditions. However, application of such considerations was beyond the scope of this Case Study. The interested reader may look at the cited literature for more information and resources on that topic.

5.4.1.1 Application of CN method

The CN method lends itself to adjustment for the effects of wildfire through selection of a CN that accounts for burned areas and by an adjustment in the time of concentration (USDA, 2013). Time of concentration is the time for runoff to drain from the most hydraulically remote point in the watershed to its outlet. The analysis applied the NRCS method for watershed lag equation in which the only wildfire-affected variable is CN (USDA, 2010).

Increases in CN result in decreasing time of concentration, in other words, runoff travels faster through a watershed as CN increases. Further, post-fire conditions affect the infiltration capacity of the soils leading to increased runoff in a catchment. The infiltration capacity can be directly related to the CN and the initial abstraction term considers infiltration and interceptions during a storm event.

5.4.1.2 Burn Severity

Although wildfire events vary in intensity and burn-severity, the general effect is reduced vegetation and ground cover, resulting in increased runoff and erosion. Fire can also cause the soil to become hydrophobic, reducing infiltration and thereby increasing runoff. For many past wildfire events, researches attempting to quantify post-fire hydrologic effects have studied the combination of vegetation type and fire burn-severity (USDA, 2013). Burn severity, a qualitative assessment of the heat pulse directed toward the ground during a fire, relates to soil heating, large fuel and duff consumption, consumption of the litter and organic layer beneath trees and isolated shrubs, and mortality of buried plant parts (NWCG, 2006).

The basis of on-the-ground evaluations of soil burn severity were observations of the following changes in soil characteristics (Parsons et al., 2010):

- Formation of water repellent layers that reduce infiltration
- Loss of effective ground cover due to consumption of litter and duff
- Surface color change due to char, ash cover, or soil oxidation
- Loss of soil structure due to consumption of soil organic matter
- Consumption of fine roots in the surface soil horizon.

The burn severity is usually classified in three categories (Parsons et al., 2010):

- Low Soil Burn Severity – Surface organic layers are not completely consumed and roots are generally unchanged, due to minimal heat penetration of the soil. While exposed mineral soil may appear lightly charred, the canopy and understory vegetation generally appears unchanged.
- Moderate Soil Burn Severity – Up to 80% of the pre-fire ground cover may be consumed. Roots may be scorched but generally not completely consumed, and soil structure is unchanged.
- High Soil Burn Severity – All or nearly all the pre-fire ground cover is generally consumed, along with roots up to 0.1 inches (0.25 cm) in diameter. Charring may be visible on larger roots. Significant bare or ash-covered soil is exposed and soil structure is less stable due to loss of root mass.

Creating a post-fire runoff estimate at Cedar Grove Bridge involved creating a theoretical map of burn severity within the watershed. This was done by overlaying the GIS-based Fire Management Planning Units (FMPU) maps provided by Park on the Cedar Grove Bridge watershed. The basis of the FMPU were the National Park's specific Fire Return Interval Departure (FRID) datasets developed for fire management planning. FRID analyses use historic fire data to determine a land unit's departure from historic fire cycles and ultimately represent a land units' fire susceptibility (Safford and Van de Water, 2014). Within the FMPU maps there were 5 classes of FRID data: extreme, high, moderate, low/none, and rock or water.

5.4.1.3 Analyses Approach

This Case Study proposed to examine the scenario where a land unit varied considerably from historic fire cycles. Therefore, the Case Study selected land units rated extreme or high for departure from the previously described CNs in the hydrologic base model. The assumptions were (1) not all the watershed would burn at the same time and (2) in the areas with low to moderate predicted severity of the burn, the cover would not suffer significant changes or if wildfire removed vegetation it would reestablish quickly.

The approach was to vary the CNs based on the type of cover and vegetation in areas overlaid by land units classified as extreme or high departure in the FMPU maps. There are 72 sub-basins that encounter FRID departures of high and extreme. Figure 19 presents the extreme and high departure FRID land units transposed over the sub-basins in the Cedar Grove watershed.

As mentioned, the basis of CN in the hydrologic base model were a classification of vegetation in good condition and the specific LU and HSG. In the post-fire model, when fire susceptibility was introduced, modification to vegetation condition varied. For example, land use code ID 32 is for rock/sand/clay combination and remained good, assuming post fire conditions would not change significantly. Land use code ID 42 is for evergreen forest, which burn at high intensity and recover slowly. Consequently, the Case Study changed the condition from good to poor when the LU intersected land units with an extreme to high departure rating.

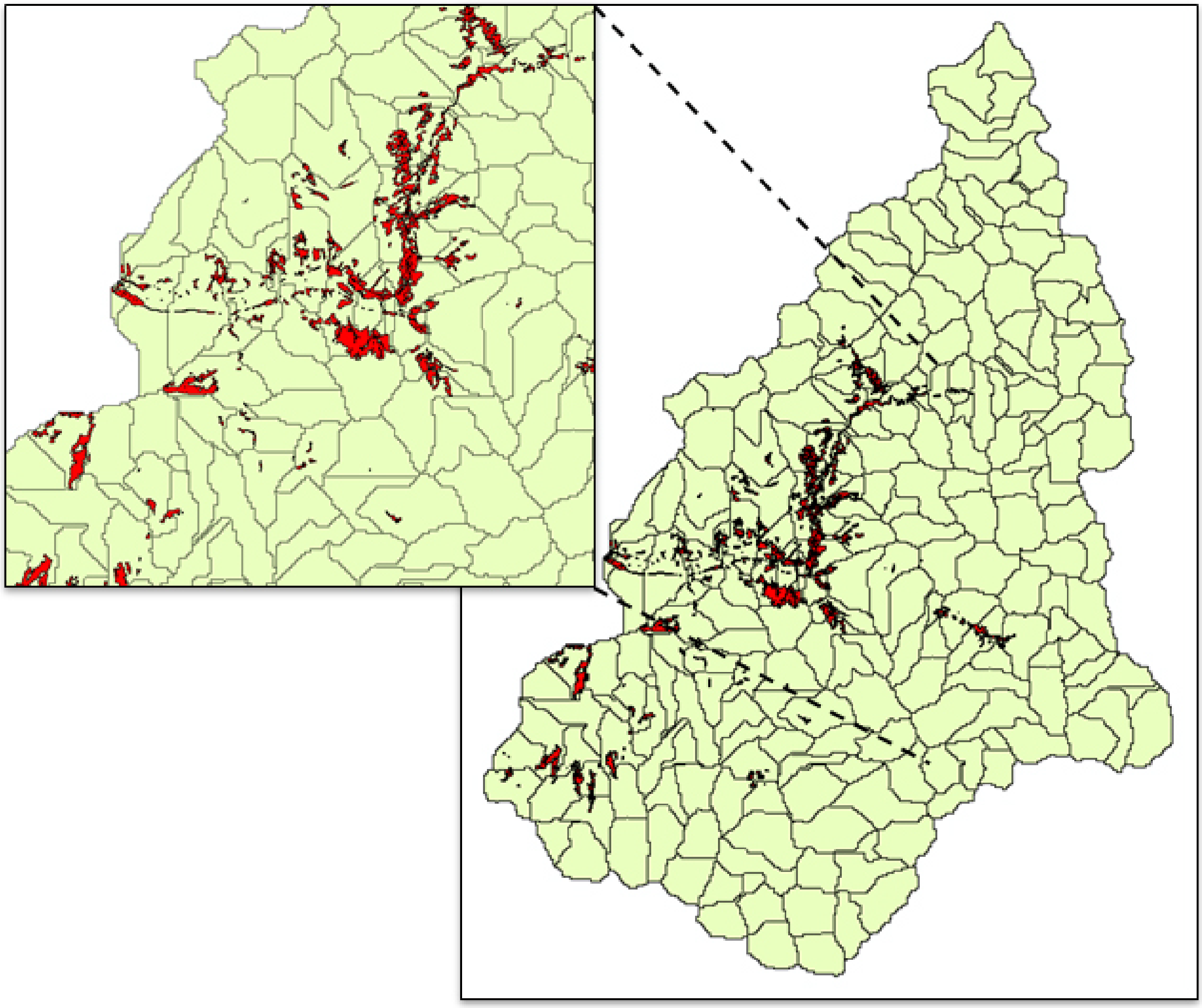

Figure 19. Areas of High and Extreme Departure FRID Data for the Cedar Grove Watershed.

5.5 Summary

Table 11 presents the changes in conditions for the land use codes from the hydrologic base model along with the resulting curve number. The table also provides the percent change in the pre- and post-wildfire curve numbers for each land use (denoted in Table 11 as "% CN"). The basin wide composite (e.g., average) CN increased from a baseline (pre-wildfire) value of 61 to a post-wildfire value of 71 (a 16% change). However, the CN is only one variable that influences the peak flows.

As expected, the Case Study found that the peak flows obtained for the various recurrences were higher. On average, the increase in peak flows is 25% of the pre-fire condition. Table 12 presents the modified peak flows and the percent of increase of flow. Figure 20 shows a comparison between predicted pre- and post-fire flows. The 50-year had a 22% increase and the 100-year had a 20% increase.

This illustrates the vulnerability of such watershed and hydraulic transportation features that go beyond the actual wildfire event. Recall the sensitivity of the Cedar Grove bridge to certain scour events. These vulnerabilities could be exacerbated should the bridge asset face future conditions where extreme events of rainfall and temperature coincide.

Table 11. Comparison of Curve Numbers between Base and Fire Hydrologic Models.

				Curve Number		Changes	
Map ID	LU	Area (mi^2)	HSG	Baseline	Post-Fire	Condition	% CN
s1063	32	2.5	B	68	68	No Change	0%
s1063	42	1.9	B	58	73	Good → Poor	26%
s1063	43	0.5	B	58	73	Good → Poor	26%
s1068	32	1.2	B	68	68	No Change	0%
s1068	42	9.2	B	58	73	Good → Poor	26%
s1068	52	0.1	B	48	56	Good → Fair	17%
s1068	81	0.3	B	69	75	Good → Poor	9%
s1069	42	10.5	B	58	73	Good → Poor	26%
s1074	32	4.2	A	49	68	No Change	39%
s1074	42	36.0	A	32	57	Good → Poor	78%
s1074	43	1.0	A	30	57	Good → Poor	90%
s1074	81	0.0	A	55	64	Good → Poor	16%
s1117	32	21.7	B	68	68	No Change	0%
s1117	33	3.0	B	68	77	Good → Fair	13%
s1117	42	101.6	B	58	73	Good → Poor	26%
s1117	43	0.7	B	58	73	Good → Poor	26%
s1117	52	2.6	B	48	56	Good → Fair	17%
s1117	81	151.9	B	69	73	Good → Fair	6%
s1118	42	1.9	B	58	73	Good → Poor	26%
s1118	52	0.0	B	48	56	Good → Fair	17%
s1118	81	7.0	B	69	75	Good → Poor	9%

Table 12. Peak Flows for the Pre- and Post-Fire Condition.

Peak Flows (in cfs)							
Scenario	2-year	5-year	10-year	25-year	50-year	100-year	500-year
Baseline Model	2,540	7,500	13,980	24,550	34,410	45,530	74,380
Fire Model	3,380	9,940	17,980	30,480	41,860	54,480	86,460
% Change	33.1%	32.5%	28.6%	24.2%	21.7%	19.7%	16.2%

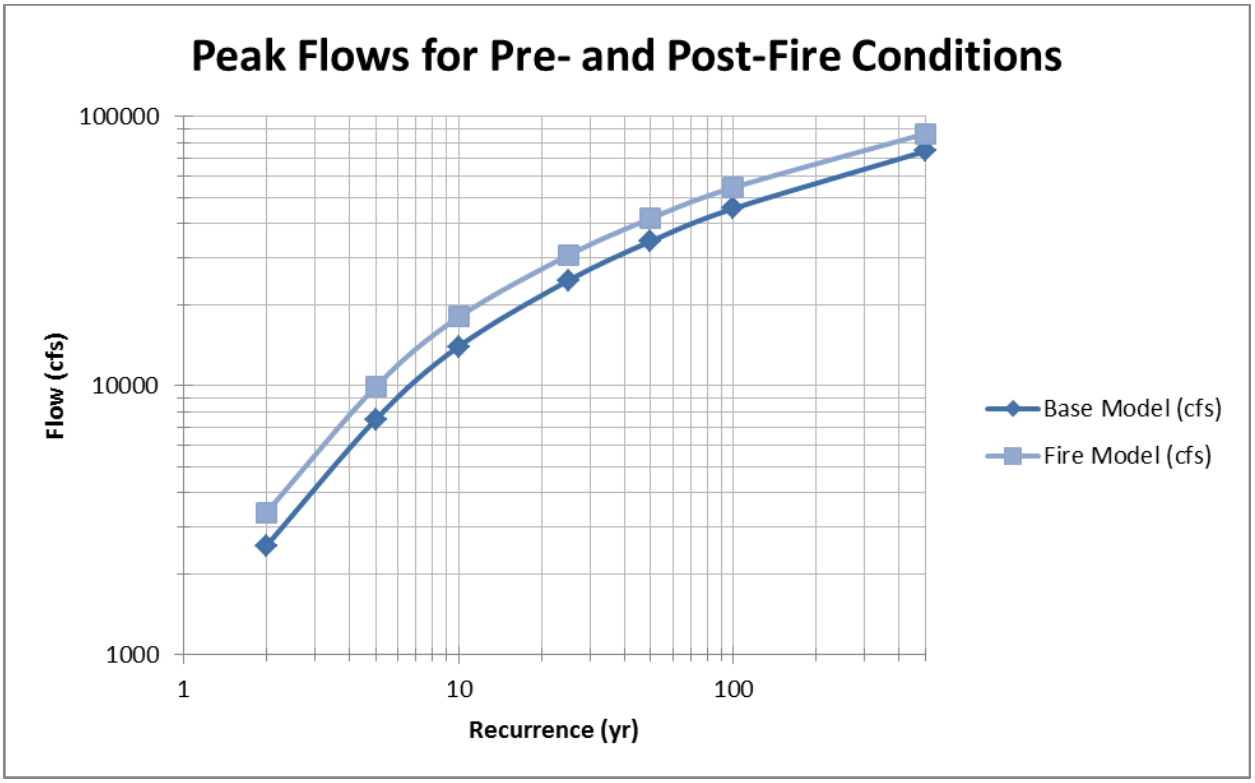

Figure 20. Peak Flows of Pre- and Post-Fire Conditions.

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Chapter 6. Culvert at the Wye

A culvert near the intersection of California SH 180 and SH 198 (referred to as the Wye) is a combination of a 30-inch culvert coupled to a downstream 10-foot section of a 24-inch culvert at the outlet; sited under a deep embankment (see Figure 21). The existing culvert does not provide sufficient capacity. Thus, upstream of the roadway embankment, a large, deep pond frequently forms after rain events, often resulting in road overtopping. Because the existing culvert is under a deep embankment, replacement of the existing culvert is not feasible.

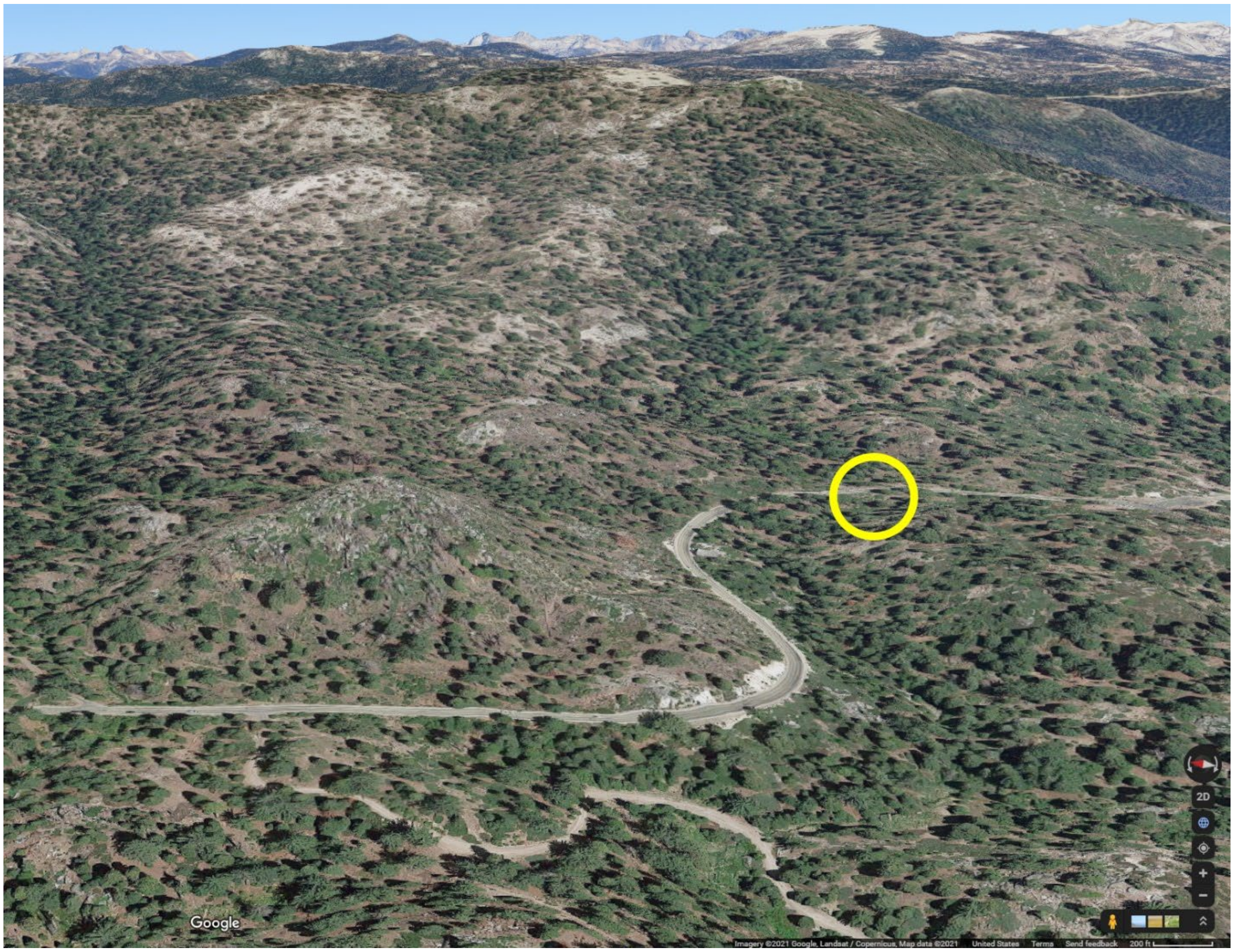

Figure 21. 3D visualization of Culvert at Wye area (looking eastward) (Google 2021).

6.1 Project Area Description

The location of the culvert is at coordinates: N 36.726033°, W 118.95499°. Figure 22 and Figure 23 illustrate a topographic map and aerial view for the Culvert at the Wye area.

The project watershed is heavily forested with mountainous terrain. The elevations ranged from 6,536 feet at the outlet to nearly 7,600 feet, with a mean basin elevation of 7,014 feet. The USGS classifies the waterway as an intermittent stream.

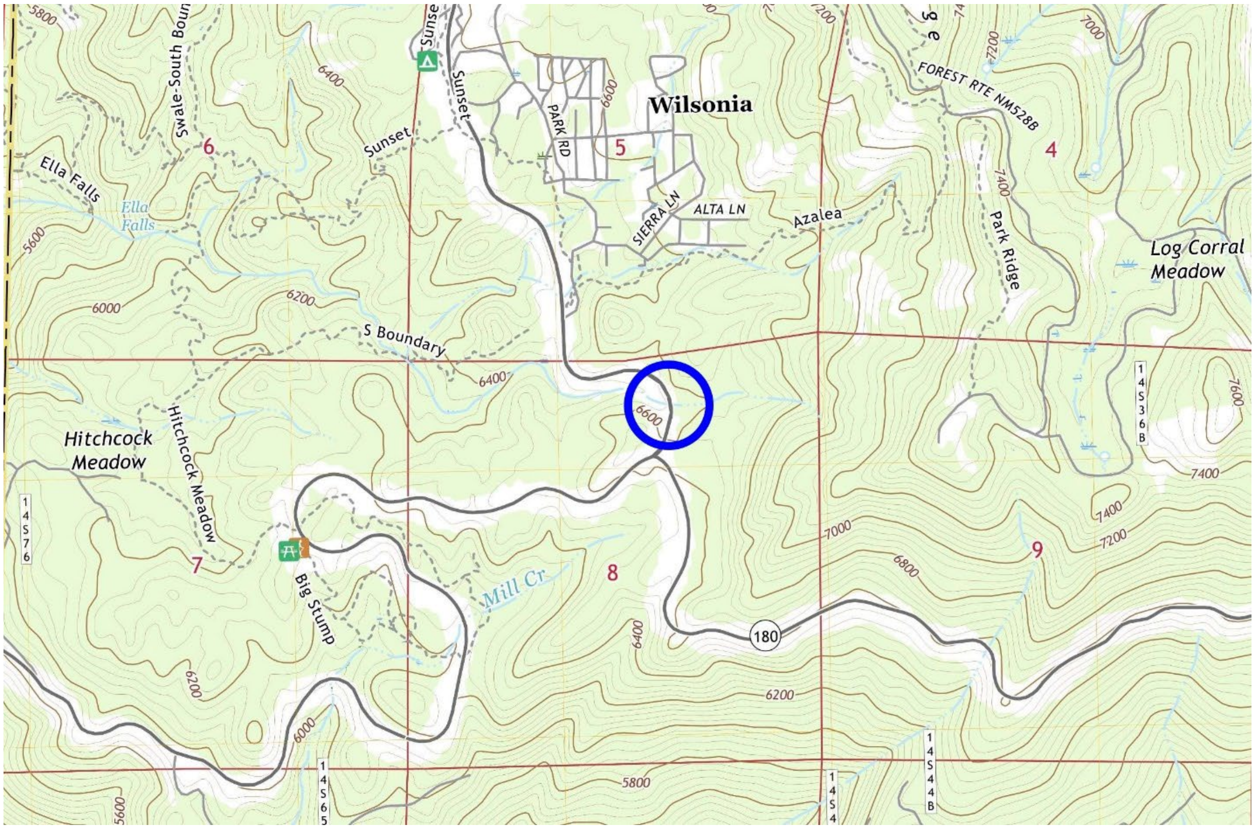

Figure 22. Culvert at the Wye topographic map (USGS 2021).

Figure 23. Culvert at the Wye aerial view (Google 2021).

As depicted by Figure 24, the culvert outlet is at the bottom of a steep roadway embankment. The site conditions would preclude use of traditional cut and cover approaches to culvert repair or replacement. However, the culvert may be a candidate for slip lining approaches.

Figure 24. Outlet of Existing Culvert at the Wye.

From the original design materials, the culvert is 88-feet long, with the upstream invert at elevation at 6,540 feet and downstream invert at 6,536 feet elevation. The culvert material is a corrugated metal pipe of the two diameters (30 inches and 24 inches) described earlier.

6.1.1 Design Criteria

The design criteria applied the FLHD PDDM for such culverts. These criteria are:

- Capacity: 50-year flood
- Stability: 50-year flood
- Headwater/Diameter Ratio: 1.5 (< 48 inch culverts), 1.2 (≥ 48 inch culverts)

6.2 Baseline Analyses

The Case Study performed a baseline analyses for the hydrologic and hydraulic conditions. The methodologies roughly aligned with the approaches described earlier. In this Case Study, the hydrology applied the USGS regression equations and the hydraulics applied a culvert specific model. Since the diameter is less than 20 feet, this culvert is not subject to FHWA's regulatory requirements regarding scour (i.e., 23 CFR 650 subpart C). This Case Study did not perform those scour analyses.

6.2.1 Hydrologic Analyses

The Case Study applied the USGS StreamStats on-line tool to obtain regression equations from USGS report, *Methods for Determining Magnitude and Frequency of Flood in California, Based on Data through Water Year 2006* (USGS 2006). The watershed location for the Culvert at the Wye is within the USGS California regression equation, Sierra-Nevada region (Region 3).

Regression equations for this region are applicable when the watershed area is within 0.07 and 2,000 square miles, the mean annual precipitation is between 15 and 100 inches, and the watershed elevation is between 90 and 11,000 feet.

From USGS StreamStats, the drainage area for the watershed is 0.3 square miles, mean annual precipitation is 42.7 inches and a mean drainage basin elevation of 7,014 feet. The hydrologic analyses used watershed area-averaged peak discharges, based on hydrologic region for design. Figure 25 illustrates the delineated watershed for the project site.

Figure 25. Watershed delineation for Culvert at the Wye (USGS 2021).

Table 13 summarizes the results from the equations for the 2- through 100-year peak flows. The Table also includes the 500-year flood estimate. In this area of California, StreamStats determines this flood frequency by extrapolating the frequency curve, not regression equations.

Table 13. Peak Flows for Culvert at the Wye (units in cfs).

2-year	10-year	25-year	50-year	100-year	500-year*
6	26	43	61	83	150

6.2.2 Hydraulic Analysis

The Case Study modeled existing conditions using the FHWA HY-8 culvert software. Recall that the existing Culvert at the Wye is a combination of a 30-inch culvert with a 10-foot section of 24-inch culvert coupled to the 30-inch culvert at the outlet. For simplifying the hydraulics, the HY-8 analyses assumed that the entire length of the barrel was 30 inches. Figure 26 presents the resulting profile for the 50-year discharge from the HY-8 model analyses.

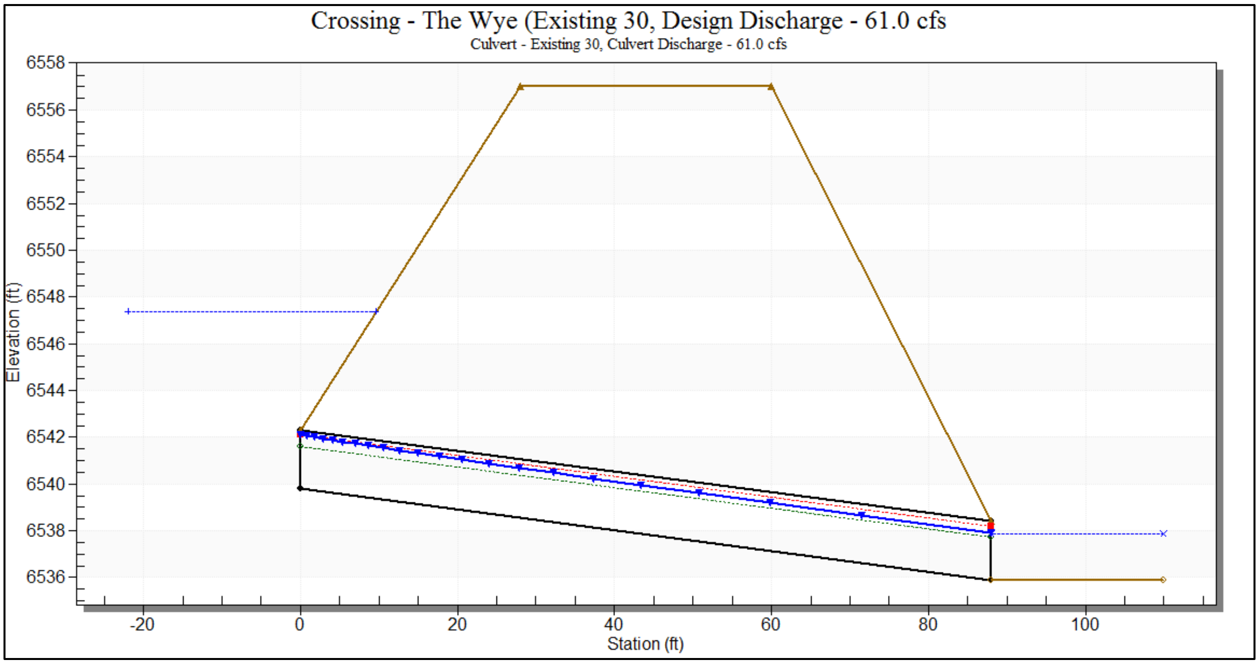

Figure 26 . HY-8 design discharge profile for Culvert at the Wye.

The culvert exhibits supercritical (i.e., S2), inlet control flow for the discharges up to the 50-year design flow. The 50-year headwater elevation is 6,547.38 feet with the inlet control depth of 7.58 feet. The outlet velocity was 13.19 ft/s. Given that the inlet diameter is 30 inches, the Hw/D of 7.58 feet / 2.5 feet = 3.032 is twice the 1.5 design criteria. Therefore, the existing culvert system does not have the necessary capacity to meet the design criteria. This also reflects the National Park personnel observations regarding issues with ponding at the inlet and overtopping of the roadway on occasion.

6.3 Sensitivity Analyses

The Case Study conducted sensitivity analyses of the hydrologic and hydraulic characteristics of the culvert. Given that the baseline hydraulic condition already fails to meet design criteria, the sensitivity analyses explored potential design improvements for this site.

6.3.1 Hydrologic Forcings

Table 14 presents values amplifying the baseline peak flows by 10%, 20%, and 50% to reflect potential increases in future mean precipitation. The Case Study will use the 50-year baseline and flow increase scenarios for hydraulic sensitivity analyses.

Table 14. Peak Flows for Culvert at the Wye (units in cfs).

Flood Event	Baseline Q	Q + 10%	Q + 20%	Q + 50%
2-year	6	7	7	9
10-year	26	28	31	38
25-year	43	47	52	65
50-year	61	67	73	91
100-year	83	91	99	124
500-year*	150	165	180	225

6.3.2 Hydraulic Analyses

The hydraulic sensitivity analyses determined that the hydrologic discharges (i.e., $Q_{10\%,50}$, $Q_{20\%,50}$, $Q_{50\%,50}$) would increasingly fail to meet the design criteria. The analyses also reveal that just providing baseline "status quo" culvert performance correlates with a reduction of resilience of the current design to the point that the $Q_{50\%,25}$ discharge (i.e., Q + 50%, 25 year = 65 cfs) would not satisfy the existing design criteria. Achieving an acceptable Hw/D ratio would correlate with flood events between approximately 2-year to 10-year frequencies for those increased flow scenarios.

6.3.2.1 Potential Design Improvements

The Case Study also investigated potential design features to alleviate some of the flooding and overtopping concerns. To improve drainage through the existing culvert, the study investigated (1) replacing the 10-foot section of the 24-inch CMP with 30-inch CMP and (2) lining the entire length of the culvert (approximately 88 feet). As per the PDDM, the lining method needs to be either close-fit lining or cured-in-place lining. The lining would reduce the diameter to 28 inches.

Sliplined 28-inch culvert

The HY-8 analyses indicate the sliplined, 28-inch culvert would have a Q_{50} headwater depth of 7.93 cfs (resulting from a reduction in barrel diameter). As depicted in Table 15, the sensitivity scenario flows would result in headwaters for $Q_{10\%,50}$ (67 cfs) equal to 8.45 feet, $Q_{20\%,50}$ (73 cfs) equal to 9.04 feet and $Q_{50\%,50}$ (91 cfs) equal to 10.83 feet. Obviously, none of these conditions would satisfy the design criteria.

Table 15. Hydraulic Conditions for Culvert at the Wye.

Scenario	Baseline Q	Q + 10%	Q + 20%	Q + 50%
Design Flow (cfs)	61	67	73	91
Headwater Depth (feet)	7.93	8.45	9.04	10.83
Hw/D ratio	3.4	3.6	3.9	4.6

Overflow culvert

To compensate, the analyses investigated installation of an additional 24-inch overflow culvert with upstream invert elevation at 6,546.28 feet and downstream elevation of 6,544.5 feet. To limit disturbance of the site, the study anticipated to tunnel or jack the 24-inch pipe through the embankment.

The analysis evaluated design alternatives using HY-8 and applying the four flow scenarios (i.e., Q_{50}, $Q_{10\%,50}$, $Q_{20\%,50}$, $Q_{50\%,50}$). The analyses determined the headwater to diameter ratio using the inlet invert elevation of the overflow culvert and the calculated headwater. The roadway begins overtopping around 84 cfs, indicating that there is some overtopping resilience up to that flow event (the $Q_{50\%,50}$ event would overtop the road). Table 16 provides the hydraulic results for each flow scenario for these improvement approaches.

Table 16. Hydraulic Conditions for Culvert at the Wye.

Scenario	Baseline Q	Q + 10%	Q + 20%	Q + 50%
Design Flow (cfs)	61	67	73	91
Headwater Depth (feet)	1.2	1.7	2.2	4.5
Hw/D ratio	0.6	0.9	1.1	2.3

6.4 Summary

The results indicate that the potential design improvements meet the Hw/D design criteria up to the 20% increase in the design flow. Further sensitivity analyses could better focus on the percentage of flow increase that coincides with the Hw/D design constraint. Such an analysis could also provide some comparison to some future hydrologic projections; for example, a FHWA HEC-17 Level 3, 4, or 5 approach (FHWA 2016).

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Chapter 7. Curb & Gutter

One section of General's Highway (California SH 180/198)[11] consists of a rock cut. This rock cut confines available drainage, resulting in little to no existing ditch capacity on either side of the roadway (see Figure 27).

During the winter months, sheet flow across the roadway freezes that results in dangerous driving conditions. This Case Study describes proposed work through this section including adding a paved gutter and curb system on either side to contain the runoff within ditches. The intended design outcome would be to reduce the amount of sheet flow across the roadway.

Figure 27. 3D visualization of the Curb & Gutter project (looking north) (Google 2021).

7.1 Project Area Description

Located within mountainous terrain, a topographic map (Figure 28) and aerial view (Figure 29) provide geographical context for this Case Study. The proposed project begins at coordinate N 36.69989444°, W 118.8713306° (station 2024+83) and proceeds to coordinate N 36.69861111°, W 118.8737139° (station 2011+16). The roadway is on a steady 5% incline from southwest to northeast. There are existing 24-inch culvert crossings at the start and end of the section.

[11] This roadway is variously designated as California SH 180 or SH 198 as both routes share the road.

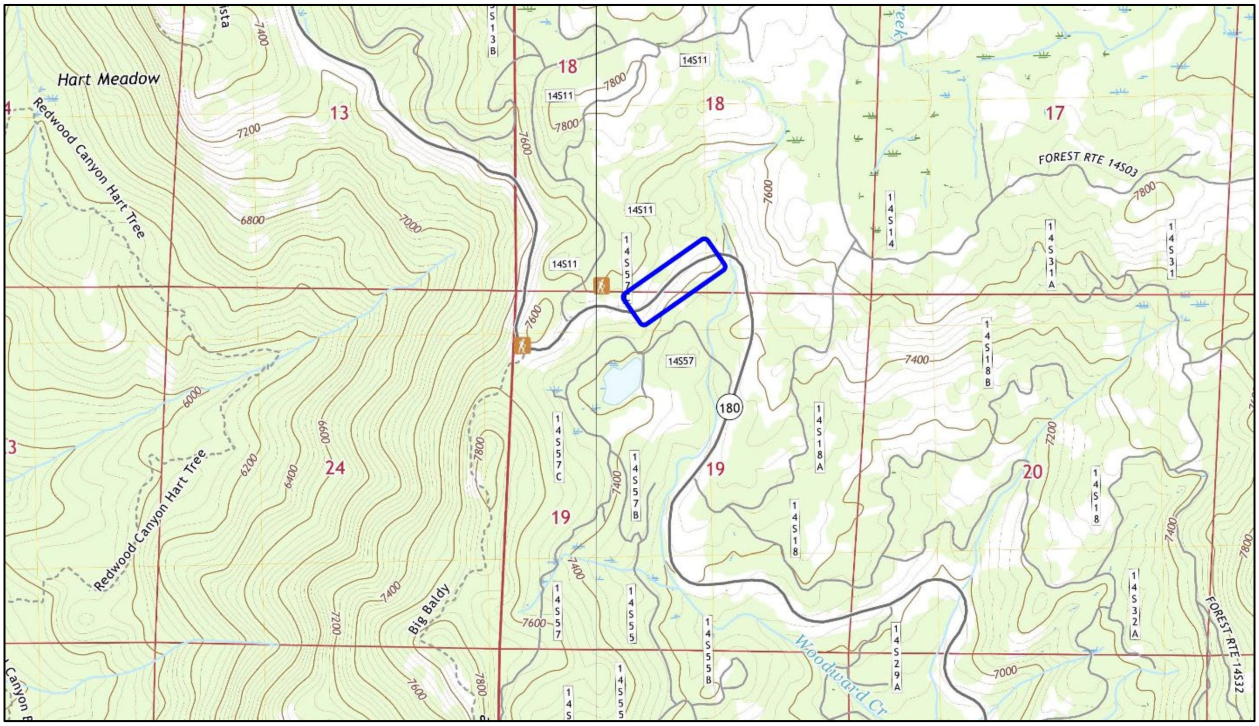

Figure 28. Curb and Gutter topographic map (USGS 2021).

Figure 29. Curb and Gutter aerial view (Google 2021).

As depicted in Figure 29 and Figure 30, the project area in a mountainous region with land cover in the watershed is entirely forested with slopes over 10% and with rock outcrops that total less than 5% of the watershed area (Figure 30). The predominant soil type in the area is sandy loam (Hydrologic Soil Group B). The contributing watershed is approximately 5.7 acres.

Figure 30. Looking southwest to depict slopes and terrain (Google 2021).

7.1.1 Proposed Project

To alleviate the drainage issues, the study proposed a curb and gutter design. From the north side of the roadway (Figure 31), proposed curb and gutter will discharge runoff into an existing ditch that is conveyed through an existing culvert crossing approximately 500 feet down station.

From the south side of the roadway (Figure 32), the curb and gutter will discharge runoff down the existing roadway embankment just beyond the rock outcrop.

Figure 33 provides a plan view of the proposed curb and gutter section of the roadway (note that the north arrow is “pointing” to the lower left quadrant). The plan view also depicts areas where the proposed curb and gutter section would tie into other hydraulic appurtenances.

7.1.2 Design Criteria

The design criteria for the curb and gutter follows the Federal Lands Highway *Project Development and Design Manual* (PDDM) published in December 2014.

- Design flow event was the 10-year.
- Maximum spread was 3 feet into one travel lane
- Maximum depth at the curb shall allow for the maximum spread identified above and shall not exceed the curb height.

Figure 31. Looking northeast along north side curb and gutter (Google 2021).

Figure 32. Looking northeast along south side curb and gutter (Google 2021).

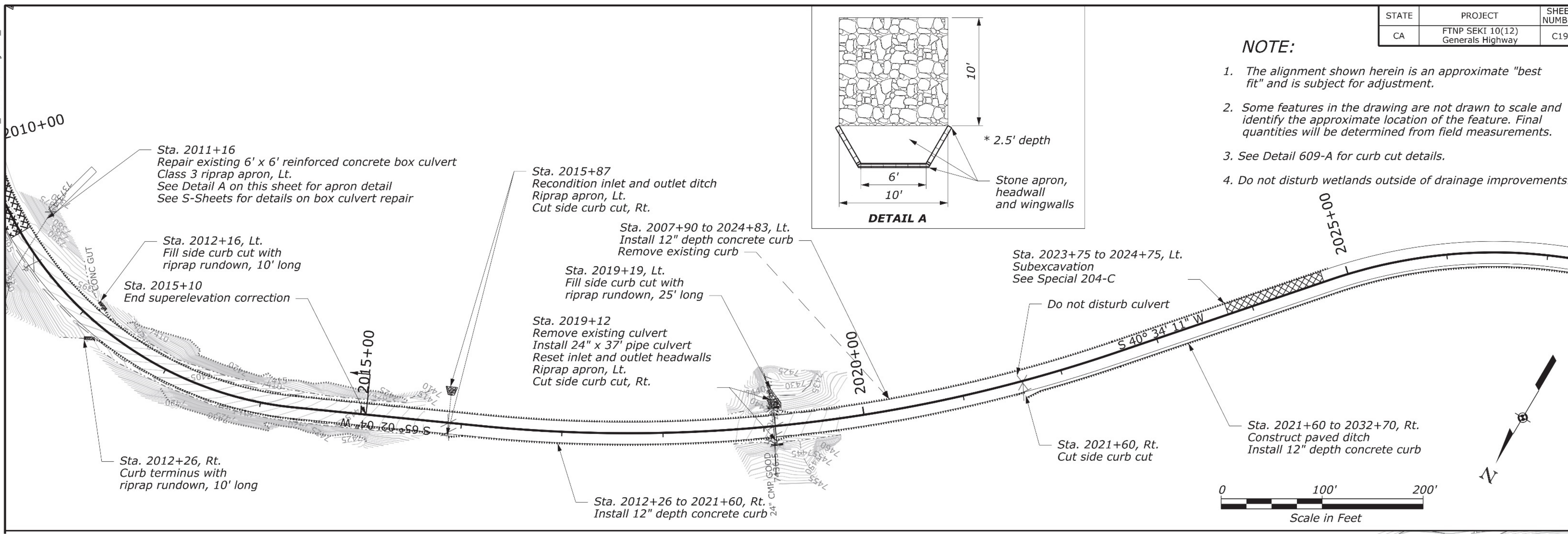

Figure 33. Proposed Plan View of Curb and Gutter.

7.2 Baseline Analyses

The baseline analyses conducted both hydrologic and hydraulic analyses of the project area. Rather than the existing roadway configuration, the baseline approach used the proposed design and construction of the curb and gutter system as the basis of the initial analyses.

The proposed curb and gutter system consists of a 3-foot wide asphalt-paved section with a 4-inch curb face. Because of the confined nature of the roadway cross section, the critical capacity consideration is the maximum spread into the traveled lane. When the runoff reaches the curb depth, the abutting vertical rock face behind the curb will serve as an additional confinement for the flow to remain within the roadway prism.

7.2.1 Hydrologic Analyses

The hydrologic analysis used a hydrologic/GIS software product to delineate the watershed and calculate the basin drainage area. The analysis also used the NRCS Web Soil Survey application to gather soil and land cover data. As the contributing watershed was less than 200 acres, the analysis used the Rational method to calculate peak discharges (FHWA 2002, FHWA 2014). The data and sources for the hydrologic analyses were:

- Drainage basin area: 5.7 acres (0.0089 square miles)
- Land Use: 100% Evergreen Forest
- Soil Type: Sandy loam (Group B)
- Runoff Coefficient: 0.2 (Hilly Woodlands, Forested)

The NOAA Atlas 14 on-line product provided tables of the annual maximum series rainfall intensities for the site (NOAA 2021). However, to apply information from these tables, the hydrologic process needs an associated rainfall duration. The Case Study calculated a time of concentration using the TR 55 sheet flow and shallow concentrated flow equations, yielding a value of 61 minutes. Using this time of concentration, and assuming it equals the duration (as per Rational method), yielded a 10-year rainfall intensity value of 1.4 inches/hour. Applying the Rational method equation yields the 10-year runoff value equal to 1.6 cfs.

The analyses selected rainfall intensities for other frequencies of potential interest. Table 17 summarizes the results the Rational method for the 2- through 100-year peak.

Table 17. Peak Flows for Curb and Gutter (units in cfs).

2-year	10-year	25-year	50-year	100-year
1.0	1.6	2.0	2.3	2.7

7.2.2 Hydraulic Analyses

The roadway slope along this section is an average of 5% from west to east. The cross-slope exhibits typical 2% slopes. The proposed gutter is an asphalt-paved ditch with a 1(v):10(h) gutter cross section slope. The Manning’s roughness coefficient used for the asphalt-paved gutter was 0.015 representative of rough finish asphalt.

Therefore, the hydraulic analyses assumed the longitudinal slope of the road equaled 0.050 ft/ft, cross-slope of pavement equaled 0.020 ft/ft, cross-slope of the gutter equaled 0.100 ft/ft, gutter width of 3 feet (from design criteria), and these appurtenances all had a Manning’s n of 0.015.

The Case Study conducted the Curb and Gutter hydraulic analysis using the FHWA Hydraulic Toolbox software program (FHWA 2014). Given design flow and the variables described above, the Toolbox allows determination of spread (feet), efficiency (i.e., E_o or the ratio of gutter flow to total flow), and the water depth at the curb (inches). As shown in Figure 34, entering the physical parameters (i.e., longitudinal slope, cross-slopes of pavement and gutter, gutter width and Manning's n) and the 10-year baseline flow (e.g., 1.6 cfs), yields the width of spread of 2.270 feet, E_o equal to one (i.e., gutter conveys entire flow), and depth of water at the curb equaling 3.425 inches.[12]

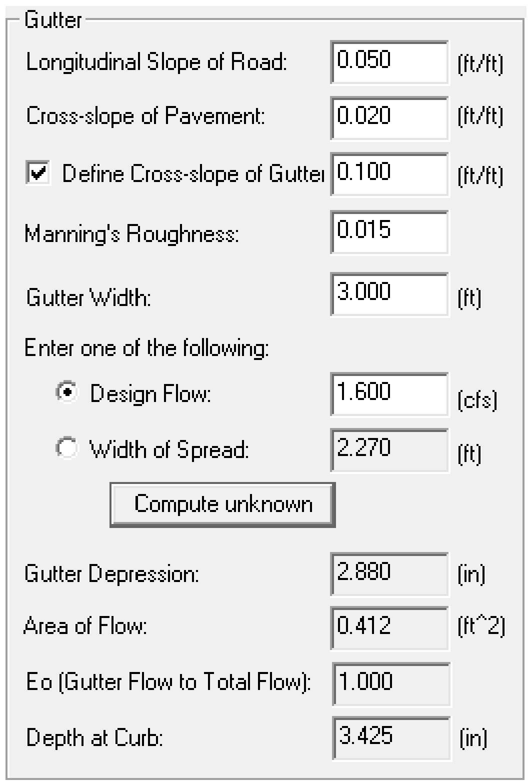

Figure 34. Toolbox screenshot of Curb and Gutter Baseline flow window.

Using the Hydraulic Toolbox software, the hydraulic analyses determined the gutter section can contain the flow up to 3.6 cfs. At this flow, the depth at the curb is 3.6 inches. To reach the maximum depth of 4 inches at the curb face, the discharge would be 4.5 cfs and the resulting spread would be 4.7 feet, or 1.7 feet into the traveled lane. To reach the maximum allowable spread of 6 feet, the discharge would have to reach 5.7 cfs, with the depth of flow at the curb of 4.3 inches. This indicates that the curb depth design criteria are more limiting than the spread design criteria.

7.3 Sensitivity Analyses

The sensitivity analyses focused on the effects of hydrologic forcings upon the desired hydraulic design criteria associated with curb and gutter sections: spread, relationship of gutter flow to total

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[12] While this is a Case Study analysis, the expectation is any actual FLHD design should meet associated PDDM criteria. An existing drainage feature might not do so, justifying the need for such design effort.

flow (i.e., an indicator of whether the gutter conveys entire runoff or if some bypass flow could occur), and curb depth.

7.3.1 Hydrologic Forcings

As depicted in Table 18, the Case Study conducted sensitivity analyses using hydrologic forcings of a 10%, 20%, and 50% increase of runoff (i.e., multiplying baseline peak flow by 110%, 120% and 150%).

Table 18. Peak Flow Scenarios for Curb and Gutter (units in cfs).

Flood Event	Baseline Q	Q + 10%	Q + 20%	Q + 50%
2-year	1.0	1.1	1.2	1.5
10-year	1.6	1.8	1.9	2.4
25-year	2.0	2.2	2.4	3.0
50-year	2.3	2.5	2.8	3.5
100-year	2.7	3.0	3.2	4.1

Design criteria for curb and gutter projects typically use the 10-year peak flow for sites such as the one found in this Case Study. However, other return periods may serve as criteria or be more appropriate given the site conditions. For example, a depressed roadway section or tunnel may use the 50-year runoff event. Likewise, the tabulation of other return periods allows some initial assessment of the amplified flow scenarios vis-à-vis the baseline conditions. For example, from Table 18, the $Q_{50\%,10}$ flow value (2.4 cfs) is slightly (4.3%) higher than the baseline Q_{50} event (2.3 cfs).

7.3.2 Hydraulic Sensitivity

For the 10-year design event, the study used the Hydraulic Toolbox software (FHWA 2014) to evaluate hydraulic behavior in the curb and gutter for the four (4) flow scenarios. Table 19 provides the hydraulic conditions in the gutter for each flow scenario.

Table 19. Hydraulic results for the Curb and Gutter.

Outcome	Baseline Q	Q + 10%	Q + 20%	Q + 50%
Design Flow (cfs)	1.6	1.8	1.9	2.4
Width of Spread (feet)	2.3	2.4	2.4	2.6
Gutter Flow to Total Flow	1	1	1	1
Depth at Curb (in)	3.4	3.4	3.5	3.5

7.4 Summary

The hydraulic analyses revealed that exceeding the design criteria for curb depth and spread corresponds to runoff discharges of 4.5 cfs and 5.7 cfs, respectively. These values are well above the highest modeled flows. The proposed curb and gutter are within the design criteria for the four flow scenarios, both at the 10-year event, but also events and scenarios up to the $Q_{50\%,100}$. Other curb and gutter projects may experience different degrees of sensitivity.

Chapter 8. Gardiner Stormwater Management

Gardiner, Montana is within Park County and located at the North Entrance of Yellowstone National Park. The North Entrance is the only entrance that is open year-round and serves as the location of the 52-foot high “Roosevelt Arch” – built around 1903 as a “rusticated triumphal entrance” to the north entrance (NPS 2021). As such, the town of Gardiner serves as an important gateway for visitors to the Park.

Gardiner is within the floodplain of the Yellowstone River, that essentially divides the town. As such, Gardiner receives northward flowing runoff from the National Park (see Figure 35). Figure 36 and Figure 37 provide a topographic map and aerial view of the Case Study area.

Figure 35. 3D visualization looking south across Gardiner into Yellowstone (Google 2021).

8.1 Stormwater Improvements

Gardiner, Park County, and Yellowstone National Park constructed infrastructure improvements in Gardiner (Coordinates: N 45.0310°, W 110.7057°) and the North Entrance to the Park as part of a five-year design and construction project.

Stormwater management improvements included a series of ditches, swales, culverts, trench drains, catch basins, inlets, and storm drains, which discharged at two primary outfalls, as well as four infiltration facilities. The project designed and constructed new storm drain lines along Main Street, Yellowstone Trail Road, North Entrance Road, and East Park Street. The 50-year event was the design event for the storm drain systems. The design assumes the infiltration basins and swale are not infiltrating water (i.e., basins providing additional capacity). Figure 38 depicts the various road, parking, and sidewalk improvements.

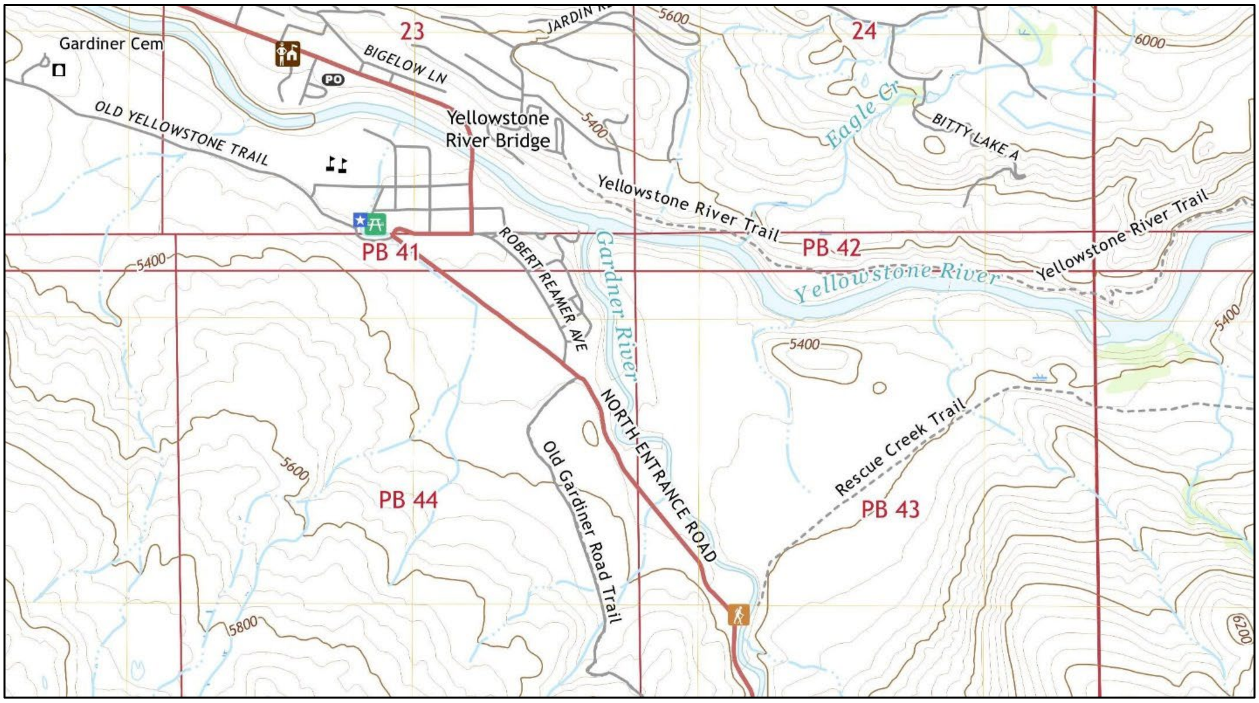

Figure 36. Topographic map of Yellowstone and Gardiner areas of interest (USGS 2021).

Figure 37. Aerial photo of Yellowstone and Gardiner areas of interest (Google 2021).

When flooding occurs, stormwater management systems may experience multiple outcomes. For example, pressurized (surcharged) flows or flows more than the storm drain capacity that can cause noticeable flooding "upstream." Pressure (surcharged) flow may occur when flow exceeds the capacity of the storm drain. If the capacity of the storm drain is exceeded by a large enough amount, the system can overflow at upstream storm drain junctions[13] and inlets, overflowing into the street, ditch, or swale. This Case Study evaluated water depths within the access holes and junctions, capacity within the storm drains, and velocities at the outfalls.

8.2 Project Area Description

Stormwater and snow-melt runoff is generally conveyed northward through the City following the slope of the ground surface. The hillside drainage, south of the City and within the Park, contributes runoff to roadside ditches along the North Entrance Road.

Prior to the Gardiner Gateway project, the stormwater management system consisted of undersized, deteriorated, and non-functioning components. Also contributing were the finite service life of storm drain systems. The result was runoff flooding occurring after events less than the system should convey (see Figure 39 and Figure 40).

8.3 Description of Project

The Gardiner-Gateway stormwater improvement project collects storm and snow melt runoff by a series of ditches, swales, culverts, trench drains, catch basins, inlets, and storm drains and discharged at two primary outfalls; north end of 4th Street and east end of East Park Street. The project constructed three infiltration basins and one infiltration swale for reducing the amount of stormwater discharged and improving water quality. New storm drain lines service Main Street, Yellowstone Trail Road, North Entrance Road, and East Park Street. The project conveys runoff from West Park Street through sidewalk trench drains to a new infiltration swale located south of West Park Street.

To reduce the flooding near the Arch, proposed system of ditches and culverts along the North Entrance Road would collect runoff from the hillside drainage and disperse the runoff over the flat meadow area north of the road. Existing and proposed ditches and swales eventually convey the dispersed water to the North Entrance Road storm drain. The project did not anticipate dispersing the runoff would significantly reduce the volume conveyed to the storm sewer. Dispersing was expected to delay the flood peak enough so that the proposed storm drain system can convey the runoff from both the improved City areas and the hillside drainage.

8.3.1 Main Street Storm Drain

An approximately 1,500-foot-long storm drain line, much of which is 24 inches in diameter, and ranges in slope from 0.17 to 9.59 percent.

8.3.2 Fourth Street Storm Drain

An approximately 950-foot-long storm drain line, 30 inches in diameter, and ranges in slope from 0.25 to 11.71 percent.

8.3.3 Yellowstone Storm Drain

An approximately 915-foot-long storm drain line, diameter ranges from 24 to 30 inches, and slope ranges from 0.90 to 5.62 percent.

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[13] Junctions may consist of access holes or other appurtenances where pipes converge, turn, or provide entry for maintenance. Additionally, they may have connections for grates and inlets.

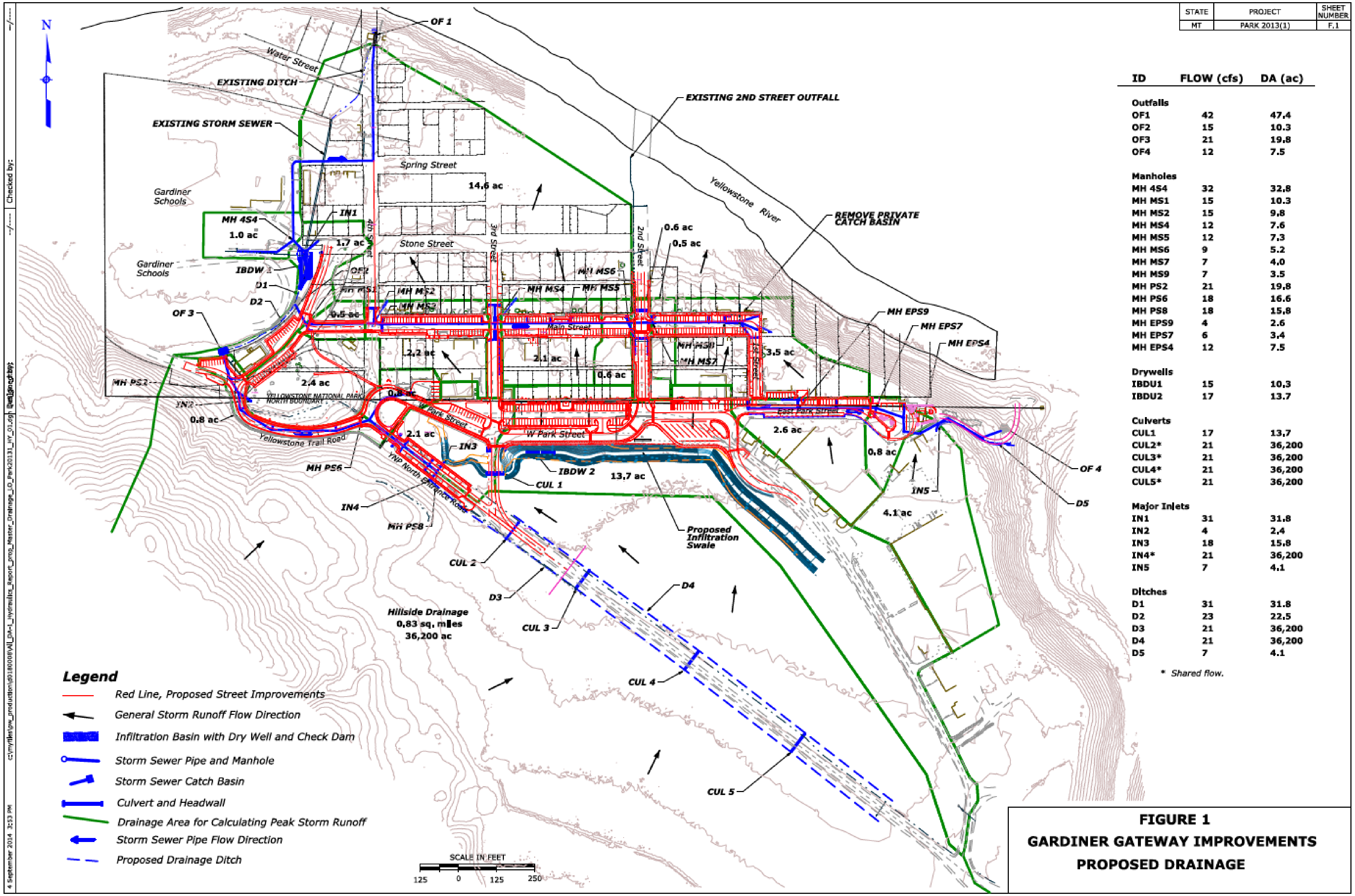

Figure 38. Gardiner Gateway Stormwater Management – Improvements.

Figure 39. Runoff ponding at road profile sag west of the Arch.

Figure 40. Runoff overtopping existing system and impacting Arch Park.

8.3.4 East Park Street Storm Drain

An approximately 900-foot-long storm drain line, 18 inches in diameter, and ranges in slope from 0.49 to 22.16 percent.

8.3.5 Park Street to Arch Infiltration Swale

A large infiltration swale with bottom area of 18,000 square-feet. Ten feet in width and 1,800 feet in length, designed to infiltrate the 100-year design storm with a maximum water depth of 6 inches.

8.4 Baseline Analyses

The Case Study began by conducting or recreating the hydrologic and hydraulic analyses for the stormwater management systems. These will yield the “baseline” values for use in the sensitivity analyses.

8.4.1 Rainfall, flow & elevation data

The improvement project used the 50-year storm event as the design frequency. Practice suggests using a minimum of 20 years of local rainfall data provides sufficient statistical information for conveying the 50-year runoff within a municipal storm drain system; particularly when there is a real possibility of flooding private property. If enough data is not available, then practice applies an appropriate alternative method. The NOAA Atlas 2 approach, based on area rainfall data with typically longer record periods, is one such alternative. The approach averages rainfall data from locations having both higher and lower annual precipitation. The location did not have appropriate local rainfall data. Thus, the study used the NOAA, Atlas 2 approach for developing the RIDF curves.

The Case Study obtained precipitation frequency data for Gardiner, Montana from the Hydrometeorological Design Studies Center, NOAA/National Weather Service. The hillside south of Gardiner contributes stormwater runoff to the drainage system. The Case Study estimated peak discharges with regression equations from the USGS report *Peak-flow Characteristics of Wyoming Streams,* WRIR 03-4107 (USGS 2003). The Case Study obtained elevation and longitude from topographic mapping. The design assumes the hillside drainage flood peak occurs after the street runoff flood peak has passed through the drainage system. To accomplish this, the hillside drainage runoff would need to be dispersed by the proposed system of culverts, ditches, and swales across the flat meadow area.

8.4.2 Hydrologic Analyses

The Case Study used the Rational method for estimating peak runoff for the urbanized areas within Gardiner and the Park. The Rational method uses drainage area, runoff coefficient, time of concentration, and rainfall intensity. The Case Study estimated drainage areas to the storm sewers, culverts, ditches, catch basins, infiltration basins, and infiltration swale from ground reconnaissance, aerial satellite imagery and LIDAR topographic mapping. The Case Study selected runoff coefficients based on observed and measured impervious areas using HDS-2 Table 5.7 (FHWA 2002) as presented in Table 7-5 of Montana Department of Transportation’s (MDT) *Hydrology Design Manual* (MDT 2017). Drainage areas and runoff coefficients represent the expected future drainage conditions. The Case Study estimated time of concentrations using the approaches in FHWA, HEC-22 *Urban Drainage Design Manual* (FHWA 2013).

8.5 Sensitivity Analyses

The Case Study conducted a hydrologic sensitivity – increasing the baseline hydrology by 10%, 20%, and 50%. The Case Study uses these hydrologic forcings as inputs into the storm drain and stormwater hydraulic facilities and appurtenances.

8.5.1 Hydrologic Forcings

The Case Study estimated the hillside drainage area using topographic mapping to be 0.83 square miles (531 acres). Figure 41 provides delineation for the hillside drainage area, outlined with a blue dotted line and a red dashed line indicating the delineation of impervious area within Gardiner.

Transportation hydrologic practices describe maximum suggested drainage area for the Rational method to range between 50 to 200 acres (FHWA 2002, FHWA 2012, FHWA 2014, AASHTO 2014). The Case Study estimated the time of concentration to be 63 minutes. Suggested maximum time of concentration for the Rational method is 30 minutes. The subdivision of areas allows for resolving these criteria.

Runoff coefficients are 0.3 for unpaved areas, 0.9 for paved areas, and 0.6 as an average. A 0.6 average runoff coefficient appropriately represents the expected rain on frozen ground or rapid snow-melt runoff conditions.

The Case Study used the 50-year storm to size the conveyance storm drains. The Case Study modeled each storm drain system in smaller drainage basins flowing into manholes or nodes. The Case Study sized the Park Street to Arch infiltration swale to completely infiltrate the 100-year storm event. To simulate increased precipitation associated with climate change, the study increased baseline peak design discharges for each storm drain run and infiltration swale by 10%, 20%, and 50% for 50 and 100-year events, respectively.

Table 20 through Table 23 provides the resulting flows for the storm drain systems.

Table 20. Main Street Storm Line design discharge increases (units in cfs).

Main Street Storm Drain	Baseline Q	Q + 10%	Q + 20%	Q + 50%
MH 0+98.40	15	16.5	18	22.5
MH 3+21.97	15	16.5	18	22.5
MH 3+47.62	15	16.5	18	22.5
MH 7+16.97	12	13.2	14.4	18
MH 9+57.78	12	13.2	14.4	18
MH 11+53.15	9	9.9	10.8	13.5
MH 14+68.19	7	7.7	8.4	10.5
MH 15+17.87	3.5	3.85	4.2	5.25

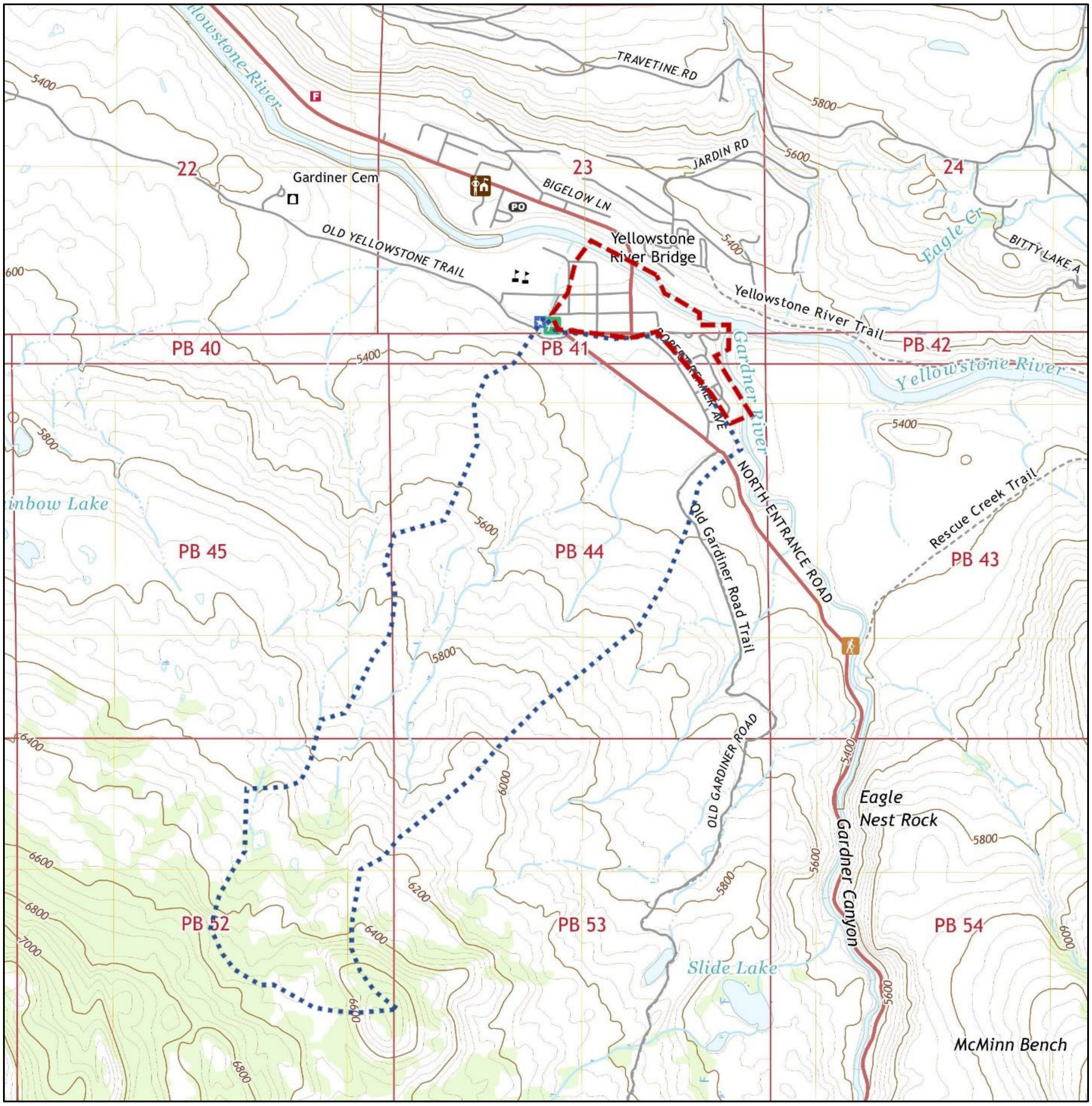

Figure 41. Gardiner Gateway Stormwater Management – drainage basins (USGS 2021).

Table 21. Fourth Street Storm Drain design discharge increases (units in cfs).

Fourth Street Storm Drain	Baseline Q	Q + 10%	Q + 20%	Q + 50%
MH 1+79.64	42	46.2	50.4	63
MH 2+77.30	42	46.2	50.4	63
MH 3+14.18	42	46.2	50.4	63
MH 4+33.10	39	42.9	46.8	58.5
MH 4+95.71	39	42.9	46.8	58.5
MH 7+50.54	32	35.2	38.4	48
MH 7+64.68	32	35.2	38.4	48
MH 9+29.94	32	35.2	38.4	48
MH 10+12.90	32	35.2	38.4	48
INLET 10+61.93	31	34.1	37.2	46.5

Table 22. Yellowstone Storm Drain design discharge increases (units in cfs).

Yellowstone Storm Drain	Baseline Q	Q + 10%	Q + 20%	Q + 50%
INLET 15+57.81	21	23.1	25.2	31.5
MH 201+16.00	21	23.1	25.2	31.5
MH 201+66.55	21	23.1	25.2	31.5
MH 202+21.50	18	19.8	21.6	27
MH 203+45.14	18	19.8	21.6	27
MH 205+57.00	18	19.8	21.6	27
MH 206+69.73	18	19.8	21.6	27
MH 400+03.22	18	19.8	21.6	27
MH 389+35.54	18	19.8	21.6	27

Table 23. East Park Street Storm Drain design discharge increases (units in cfs).

East Park Street Storm Drain	Baseline Q	Q + 10%	Q + 20%	Q + 50%
MH 201+59.39	12	13.2	14.4	18
MH 200+89.43	12	13.2	14.4	18
MH 200+29.48	12	13.2	14.4	18
MH 106+39.19	12	13.2	14.4	18
MH 105+88.61	6	6.6	7.2	9
MH 105+86.88	6	6.6	7.2	9
MH 104+97.33	6	6.6	7.2	9
MH 103+67.62	4	4.4	4.8	6
MH 102+42.50	4	4.4	4.8	6
MH 100+79.96	4	4.4	4.8	6

8.5.2 Infiltration Swale

The analysis developed a hydrograph to model design flows in the infiltration swale. The Case Study increased this hydrograph baseline flow by 10%, 20%, and 50%. Figure 42 provides the design hydrograph as well as increased hydrographs for the study.

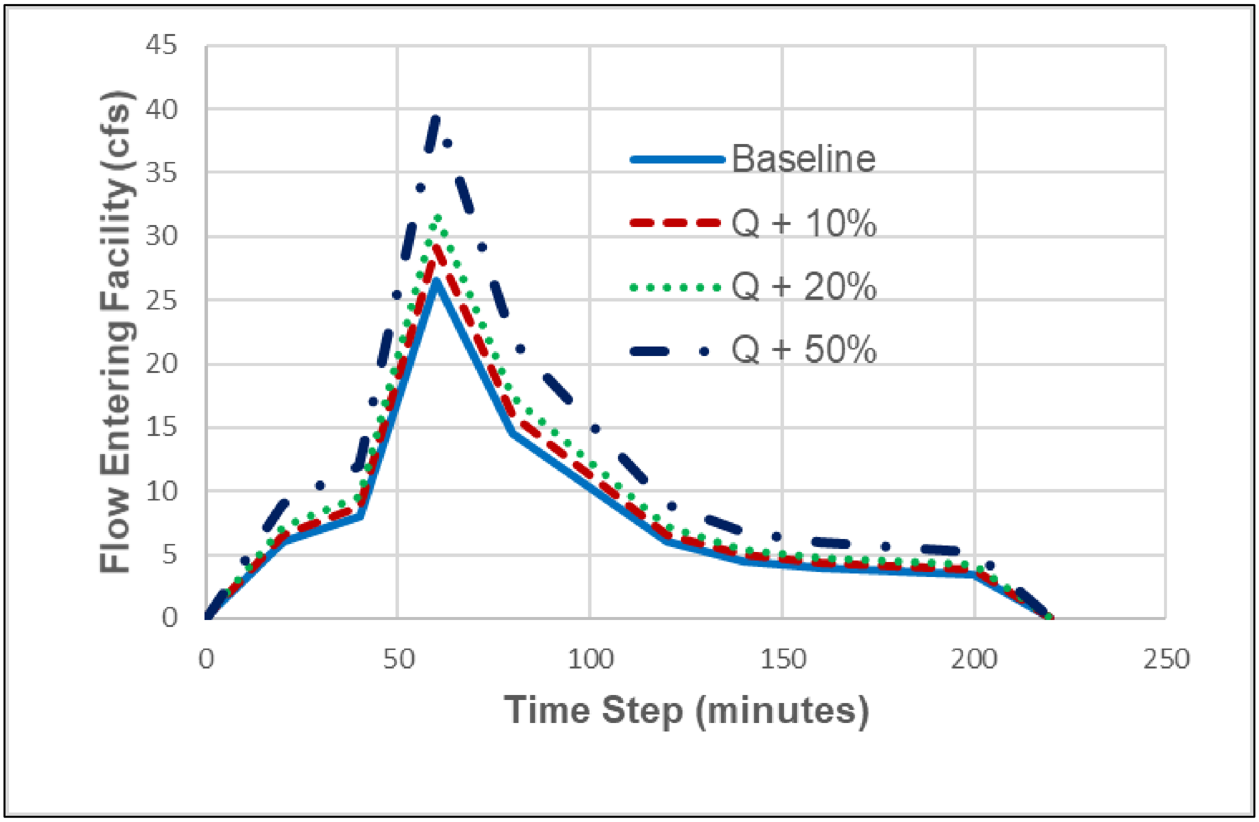

Figure 42. Infiltration swale design hydrograph with discharge increases.

8.5.3 Hydraulic Sensitivity

Storm drain characteristics determine the available capacity of each pipe along the length of a storm drain line. In this analysis, capacity describes the ability to convey water without entering flowing full or entering pressure flow. In addition, storm drain systems retain some capacity in manhole and inlet structures between pipe lengths.

Even if a pipe is flowing full, in pressure flow, a manhole or inlet may not surcharge, or overtop. The Case Study simulated increased flows through the storm drain model for the four storm drains examined. Table 24 through Table 31 provide capacity as it is compared to increased design flows. The following provides a discussion of each of the lines sensitivity to increases of flow. For each line, one table presents flows, the other as percentage of capacity.

8.5.3.1 Main Street Storm Drain

There are seven situations where increased flow exceeds capacity of the system at the nodes (i.e. junctions). The capacity of MH 7+16.97 and MH 9+57.78 is 13.16 cfs. The $Q_{10\%,50}$ is 13.2 cfs at both, indicating slight surcharging. The $Q_{20\%,50}$ is 14.4 cfs indicating additional surcharge at both. At $Q_{50\%,50}$ there are three nodes experiencing surcharge; MH 7+16.97 and MH 9+57.78, both experiencing 18 cfs and MH 11+53.15, experiencing 13.5 cfs over the capacity of 13.20 cfs.

Table 24. Main Street Storm Drain capacity comparison (units in cfs).

Main Street Node	Capacity	Baseline	Q + 10%	Q + 20%	Q + 50%
MH 0+98.40	49.02	15	16.5	18	22.5
MH 3+21.97	81.64	15	16.5	18	22.5
MH 3+47.62	60.70	15	16.5	18	22.5
MH 7+16.97	13.16	12	13.2	14.4	18
MH 9+57.78	13.16	12	13.2	14.4	18
MH 11+53.15	13.20	9	9.9	10.8	13.5
MH 14+68.19	10.81	7	7.7	8.4	10.5
MH 15+17.87	6.50	3.5	3.85	4.2	5.25

Table 25 expresses this relationship as percentage of capacity. Where the percentage exceeds 100%, some surcharge exists. The highest value is 136.8% of capacity at the two nodes.

Table 25. Main Street Storm Drain capacity percentage comparison.

Main Street Node	Capacity	Baseline	Q + 10%	Q + 20%	Q + 50%
MH 0+98.40	49.02	30.6%	33.7%	36.7%	45.9%
MH 3+21.97	81.64	18.4%	20.2%	22.0%	27.6%
MH 3+47.62	60.70	24.7%	27.2%	29.7%	37.1%
MH 7+16.97	13.16	91.2%	100.3%	109.4%	136.8%
MH 9+57.78	13.16	91.2%	100.3%	109.4%	136.8%
MH 11+53.15	13.20	68.2%	75.0%	81.8%	102.3%
MH 14+68.19	10.81	64.8%	71.2%	77.7%	97.1%
MH 15+17.87	6.50	53.8%	59.2%	64.6%	80.8%

8.5.3.2 Fourth Street Storm Drain

As depicted in Table 26 and Table 27, there are six situations where the Fourth Street nodes exceed capacity. In this case, the MH 4+95.71 node has a $Q_{50\%,50}$ nearly 150% of capacity.

Table 26. Fourth Street Storm Drain capacity comparison (units in cfs).

Fourth Street Node	Capacity	Baseline	Q + 10%	Q + 20%	Q + 50%
MH 1+79.64	124.23	42	46.2	50.4	63
MH 2+77.30	157.11	42	46.2	50.4	63
MH 3+14.18	126.05	42	46.2	50.4	63
MH 4+33.10	265.93	39	42.9	46.8	58.5
MH 4+95.71	39.29	39	42.9	46.8	58.5
MH 7+50.54	38.65	32	35.2	38.4	48
MH 7+64.68	41.34	32	35.2	38.4	48
MH 9+29.94	67.87	32	35.2	38.4	48
MH 10+12.90	39.19	32	35.2	38.4	48
INLET 10+61.93	135.84	31	34.1	37.2	46.5

Table 27. Fourth Street Storm Drain capacity percentage comparison.

Fourth Street Node	Capacity	Baseline	Q + 10%	Q + 20%	Q + 50%
MH 1+79.64	124.23	33.8%	37.2%	40.6%	50.7%
MH 2+77.30	157.11	26.7%	29.4%	32.1%	40.1%
MH 3+14.18	126.05	33.3%	36.7%	40.0%	50.0%
MH 4+33.10	265.93	14.7%	16.1%	17.6%	22.0%
MH 4+95.71	39.29	99.3%	109.2%	119.1%	148.9%
MH 7+50.54	38.65	82.8%	91.1%	99.4%	124.2%
MH 7+64.68	41.34	77.4%	85.1%	92.9%	116.1%
MH 9+29.94	67.87	47.1%	51.9%	56.6%	70.7%
MH 10+12.90	39.19	81.7%	89.8%	98.0%	122.5%
INLET 10+61.93	135.84	22.8%	25.1%	27.4%	34.2%

At the furthest upstream access hole for the Fourth Street storm drain, the existing ground drops off slightly. At this location, water would flow out of the access hole in the baseline plus 50% model, where pipes in pressure flow backwatered upstream all the way to the first access hole. Figure 43 depicts the drainage profile.

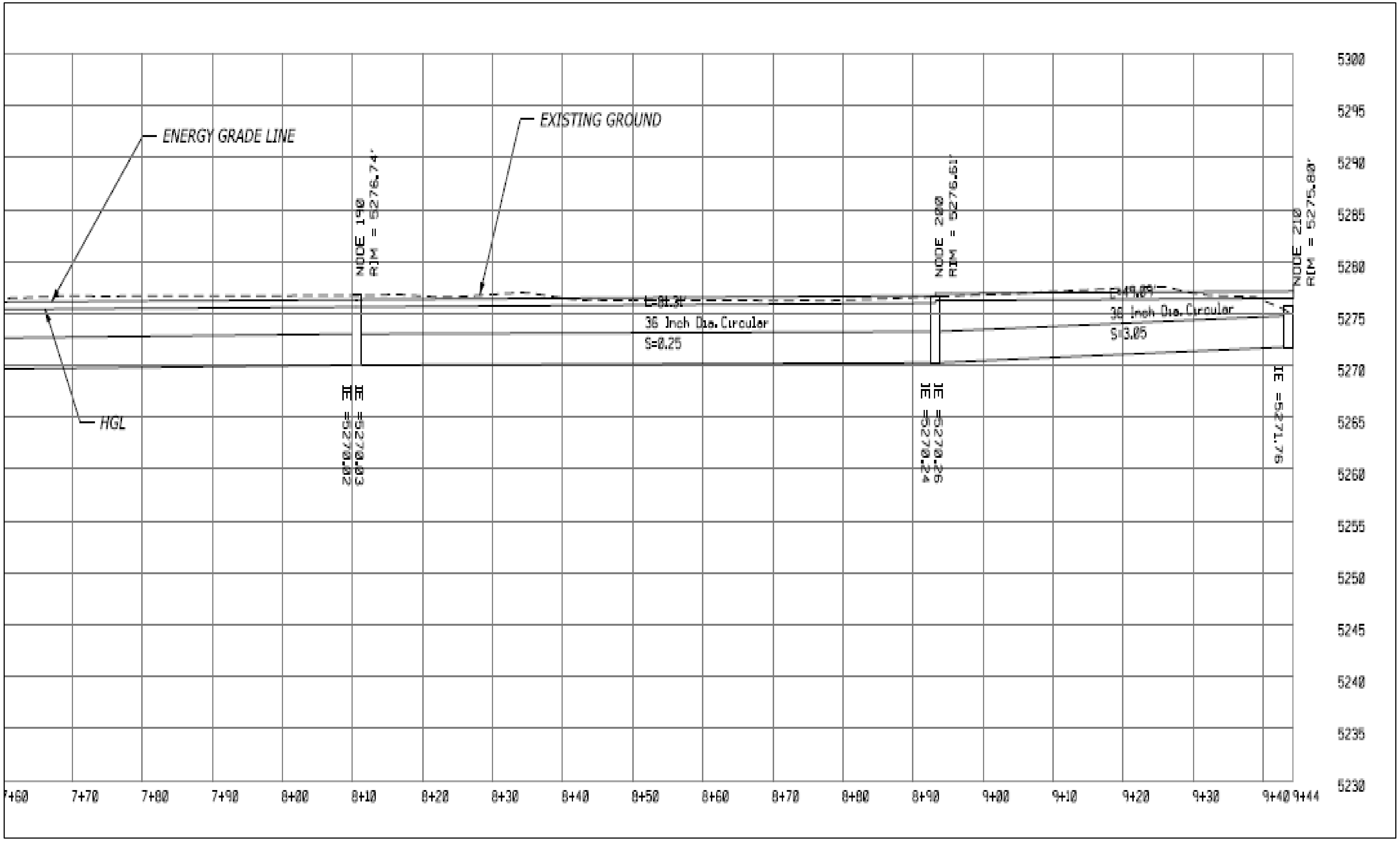

Figure 43. Fourth street upstream profile for baseline + 50% design discharge.

8.5.3.3 Yellowstone Storm Drain

The analyses indicate that the Yellowstone Storm Drain system has resilience for increasing flows (see Table 28 and Table 29). The "worst" node (MH 201+16.00) using 69.6% of potential capacity at $Q_{50\%,50}$.

Table 28. Yellowstone Storm Drain capacity comparison (units in cfs).

Yellowstone Node	Capacity	Baseline	Q + 10%	Q + 20%	Q + 50%
INLET 15+57.81	80.34	21	23.1	25.2	31.5
MH 201+16.00	45.26	21	23.1	25.2	31.5
MH 201+66.55	63.89	21	23.1	25.2	31.5
MH 202+21.50	79.88	18	19.8	21.6	27
MH 203+45.14	90.68	18	19.8	21.6	27
MH 205+57.00	112.72	18	19.8	21.6	27
MH 206+69.73	91.90	18	19.8	21.6	27
MH 400+03.22	62.48	18	19.8	21.6	27
MH 389+35.54	52.30	18	19.8	21.6	27

Table 29. Yellowstone Storm Drain capacity percentage comparison.

Yellowstone Node	Capacity	Baseline	Q + 10%	Q + 20%	Q + 50%
INLET 15+57.81	80.34	26.1%	28.8%	31.4%	39.2%
MH 201+16.00	45.26	46.4%	51.0%	55.7%	69.6%
MH 201+66.55	63.89	32.9%	36.2%	39.4%	49.3%
MH 202+21.50	79.88	22.5%	24.8%	27.0%	33.8%
MH 203+45.14	90.68	19.9%	21.8%	23.8%	29.8%
MH 205+57.00	112.72	16.0%	17.6%	19.2%	24.0%
MH 206+69.73	91.90	19.6%	21.5%	23.5%	29.4%
MH 400+03.22	62.48	28.8%	31.7%	34.6%	43.2%
MH 389+35.54	52.30	34.4%	37.9%	41.3%	51.6%

8.5.3.4 East Park Street Storm Drain

As shown in Table 30 and Table 31, the East Park storm drain system only exhibits potential surcharging at two nodes (MH 105+88.61 and MH 105+86.66) at the $Q_{50\%,50}$ condition. The percentage of capacity are 105.2% and 103.9% respectively. This seems to indicate some resilience of the system.

Table 30. East Park Storm Drain capacity comparison (units in cfs).

East Park Node	Capacity	Baseline	Q + 10%	Q + 20%	Q + 50%
MH 201+59.39	57.626	12	13.2	14.4	18
MH 200+89.43	45.024	12	13.2	14.4	18
MH 200+29.48	56.03	12	13.2	14.4	18
MH 106+39.19	24.92	12	13.2	14.4	18
MH 105+88.61	8.557	6	6.6	7.2	9
MH 105+86.88	8.664	6	6.6	7.2	9
MH 104+97.33	9.728	6	6.6	7.2	9
MH 103+67.62	8.665	4	4.4	4.8	6
MH 102+42.50	8.686	4	4.4	4.8	6
MH 100+79.96	8.641	4	4.4	4.8	6

Table 31. East Park Street Storm Drain capacity percentage comparison.

East Park Node	Capacity	Baseline	Q + 10%	Q + 20%	Q + 50%
MH 201+59.39	57.626	20.8%	22.9%	25.0%	31.2%
MH 200+89.43	45.024	26.7%	29.3%	32.0%	40.0%
MH 200+29.48	56.03	21.4%	23.6%	25.7%	32.1%
MH 106+39.19	24.92	48.2%	53.0%	57.8%	72.2%
MH 105+88.61	8.557	70.1%	77.1%	84.1%	105.2%
MH 105+86.88	8.664	69.3%	76.2%	83.1%	103.9%
MH 104+97.33	9.728	61.7%	67.8%	74.0%	92.5%
MH 103+67.62	8.665	46.2%	50.8%	55.4%	69.2%
MH 102+42.50	8.686	46.1%	50.7%	55.3%	69.1%
MH 100+79.96	8.641	46.3%	50.9%	55.5%	69.4%

8.5.3.5 Park Street to Arch Infiltration Swale

The infiltration swale outlet invert was set six inches above the facility bottom. This allows six inches of depth to accumulate prior to overflow into the stormwater sewer system. The project used the 100-year design storm to size the basin to allow all runoff to be stored below the outlet invert. The hydrograph increases the depth necessary to retain the all runoff. Figure 44 provides infiltration swale details. Table 32 provides variation in depths to retain all increases in runoff volume.

Table 32. Infiltration Swale capacity comparison.

Infiltration Swale	Baseline Q	Q + 10%	Q + 20%	Q + 50%
Maximum water depth (inches)	6	7	7	9

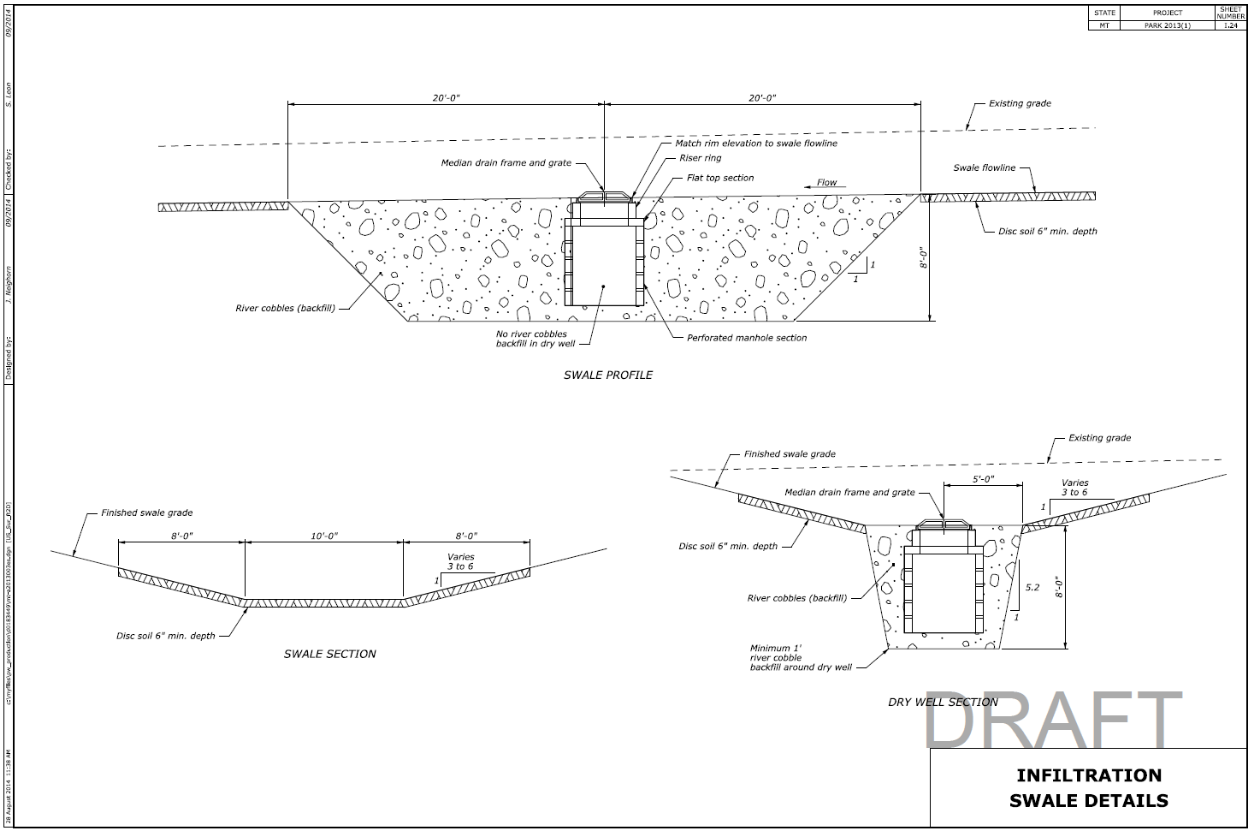

Figure 44. Infiltration swale details.

8.6 Summary

The Case Study examined recently constructed storm drain systems and infiltration swale at the North Entrance of Yellowstone National Park, in Gardiner, Montana, for capacity for the baseline design storm, and increased baseline flows by 10%, 20%, and 50%. Even as baseline flows increased by 50% the system still functions, though some pipes are in pressure flow. The one manhole where the lid flooded could be retrofitted with locking cap to allow more storage during larger storm events.

The design of the infiltration facility seeks to infiltrate 100% of the 100-year design flood event. The 36-inch diameter outlet, or overflow structure, is more than adequately sized to convey any flow above this design flow. Other retrofit options could be to increase the bottom size of the facility to allow additional infiltration or raise the outlet pipe to be able to have the maximum depth for increased runoff volume storage.

Chapter 9. West Twin Creek Bridge

The West Twin Creek Bridge (coordinates: N 47.83292°, W 123.989832°) is approximately 14.4 miles east of United States Highway 101 (US 101) on the Upper Hoh Road. Figure 45 provides a 3D visualization of the Case Study site (looking north toward the portions of the Olympic Mountain range). Figure 46 and Figure 47 provide topographic and aerial views of the site location.

Figure 45. 3D visualization of West Twin Creek area (Google 2021)

9.1 Project Area Description

The West Twin Creek watershed is a tributary to the Hoh River basin in northwestern Washington State, within the Olympic National Park. The West Twin Creek Bridge[14] is not significantly skewed relative to the stream. The confluence with the Hoh River is approximately 750 feet downstream.

9.1.1 Watershed & Stream Characteristics

The watershed is prone to flooding, with some events apparently having caused failure of previous structures (see Figure 48). The West Twin Creek is moderately sloped (2 to 4%) both upstream and downstream of the existing bridge (see Figure 49).

[14] The labeling of the bridge may be slightly confusing. Various maps depict the bridge actually crossing Twin Creek, with West Twin Creek being an upstream tributary. Additionally, East Twin Creek (encountered later) has a confluence with Twin Creek downstream of the bridge. This document will honor the naming designation by the NPS and use "West Twin Creek Bridge."

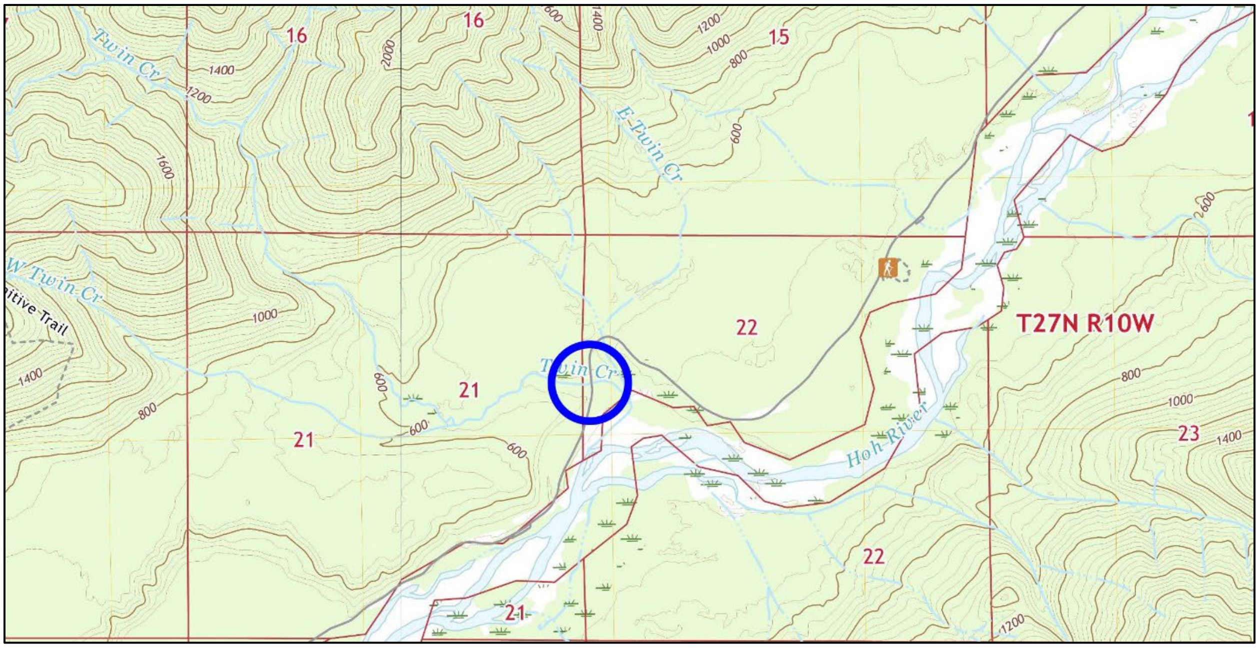

Figure 46. Topographic map of West Twin Creek area (USGS 2021)

Figure 47. Aerial view of West Twin Creek Bridge area (Google 2021).

The creek is moderately entrenched (entrenchment ratio 1.4) and has moderate sinuosity. Entrenchment is the vertical containment of a river and the degree to which it is incised in the valley floor. Lateral channel migration is largely controlled by valley wall colluvium. The stream is perennial. Bankfull width is approximately 42 feet. Bankfull depth is approximately 2 feet. The stream lies in a narrow colluvial valley with flood prone area widths between 50 and 60 feet. The floodplains appear to exhibit poorly developed functional characteristics and values.

Figure 48. Washout of earlier bridge crossings (NPS 2021).

Figure 49. Looking upstream toward current West Twin Creek Bridge.

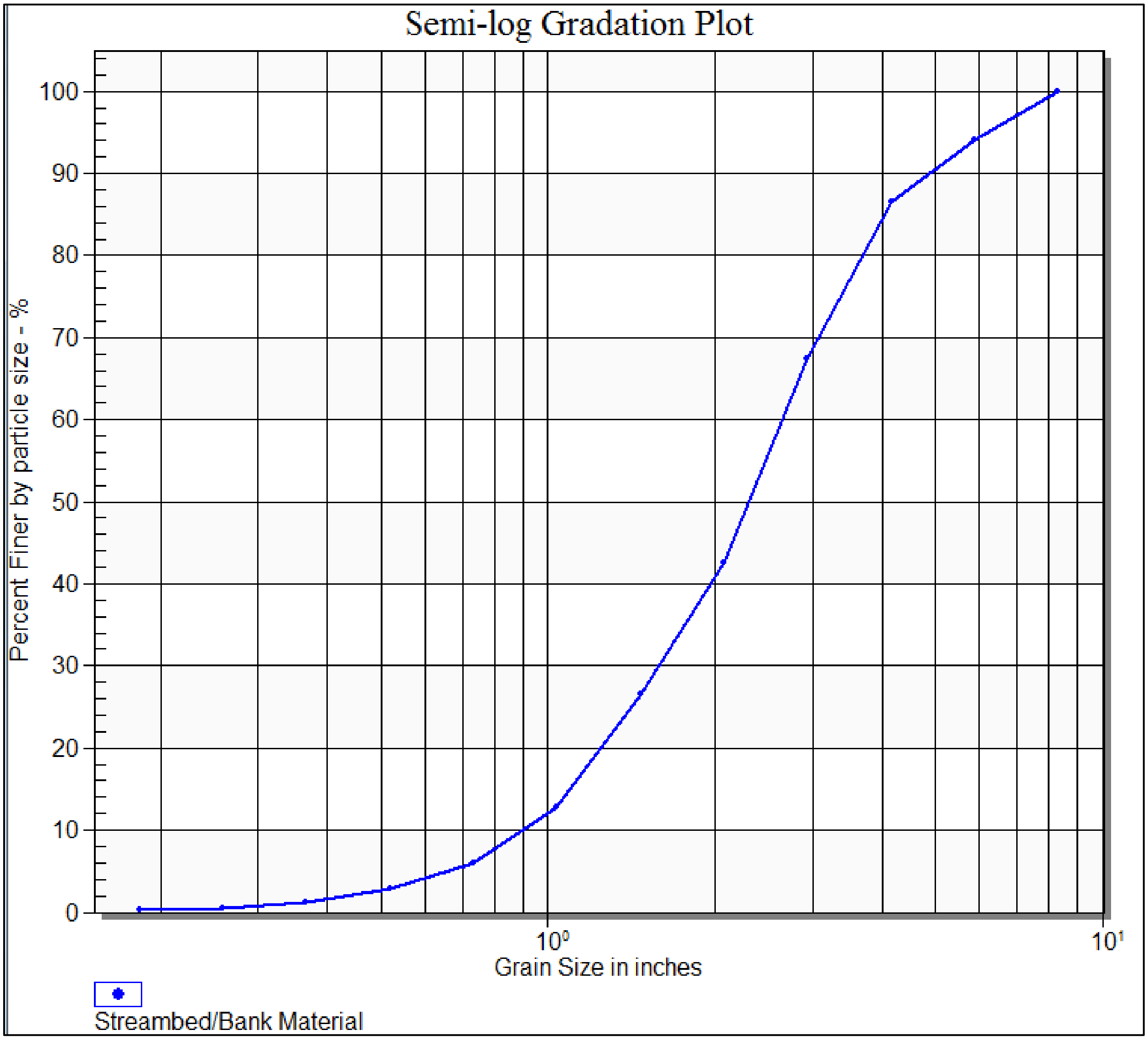

Figure 50. Gradation plot of streambed/bank material using digital gradation analysis.

Riparian vegetation is predominantly large conifer trees and dense underbrush. The Case Study did not observe bedrock in the stream banks or bottom near the crossing. The stream banks are vegetated and not generally undercut and appear moderately stable.

The watershed is mountainous, with elevations ranging from 500 to 3,300 feet, and over 85% having slopes greater than 30 percent. The mean elevation is 1,800 feet. The entire drainage area is heavily forested, with over 90% covered by canopy. The mean annual precipitation is approximately 140 inches. The mean December/January/February (DJF) temperature is between 32 degrees and 36 degrees Fahrenheit. Rainfall is the dominant stream-flow control.

9.1.2 Bridge Description

As described earlier, the current West Twin Creek Bridge is a 115-foot single-span pre-stressed concrete beam bridge on steel pilings with spill-through abutments. The bridge low chord elevation is 524 feet. The minimum abutment pile tip elevation was 472 feet.[15] The channel thalweg elevation at this cross section is 510 feet. This gives a total initial embedment depth below the current channel thalweg of 38 feet.

9.1.3 Bridge Abutment Protection

Riprap has been installed at the bridge abutments to provide protection from scour and debris. Figure 51 presents the original riprap design for West Twin Creek Bridge.

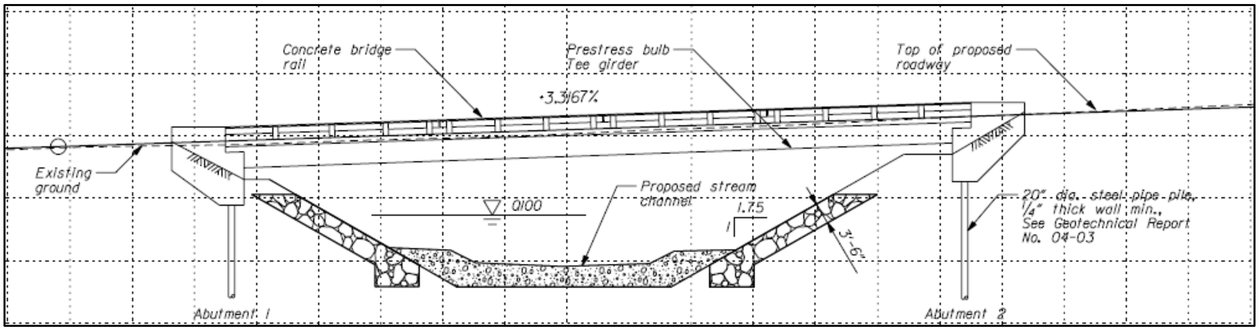

Figure 51. West Twin Creek Bridge – Cross section view of riprap design.

The design practice for sizing riprap has evolved over time. Additionally, sometimes the “designed riprap” is not the actual riprap used during construction.

To aid in later discussions, Table 33 provides a comparison between (1) Class V riprap[16], specified in the original plan, (2) Class III riprap[17] and (3) existing riprap currently in place.

Table 33. Riprap sizing comparison (size in inches).

Riprap Type	D_{15}	D_{50}	D_{85}	D_{100}
FHWA Standard Specification, FP-03 – Class V	0-8	8-20	20-26	26-28
FHWA Standard Specification, FP-14 – Class III	8-10	11-14	15-19	21-27
Existing at Bridge	4.3	10.5	19.1	29.9

9.1.4 Design Criteria

The Case Study evaluated the design parameters of clearance (i.e., freeboard), scour, and riprap size. The FLHD *Project Development and Design Manual* (PDDM) specifies a bridged waterway use a 50-year flood for capacity and a 100-year flood for scour (FHWA 2014). The PDDM specifies a minimum of 2-feet freeboard (FHWA 2014). The Case Study determined capacity, or available freeboard, using the USACE HEC-RAS model. The analysis estimated scour using the methodology from HEC-18, *Evaluating Scour at Bridges* (FHWA 2012). The analysis determined riprap sizing using the FHWA Hydraulic Toolbox software program (FHWA 2014).

Scour calculations are typically performed for the 100-year flood event (FHWA 2014, AASHTO 2020). As riprap is typically sized for protection up to the 50-year storm event (FHWA 2012, FHWA 2014), a primary design assumption is that there is no longer riprap abutment protection in place.

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[15] From National Park Service ERFO 2007(1)-40(3) plan set.
[16] From FHWA Standard Specification for Construction of Roads and Bridges on Federal Highway Projects, FP-03 (FHWA 2003).
[17] From FHWA Standard Specification for Construction of Roads and Bridges on Federal Highway Projects, FP-14 (FHWA 2014). FP-14 superseded FP-03.

Scour depths and elevations are used to ensure design of the bridge structure itself can withstand large flooding events, even with all abutment protection having been washed away, including riprap and approach embankments.

When the available area for a stream channel is reduced by some sort of obstruction or constriction, scour will occur at the contraction location. This could be an abutment face in the case of a bridge construction. Pier scour should be examined in two different scenarios. One scenario is when the location of a pier is in the stream channel, the other is during the 100-year or greater storm event where piles are a deep foundation.

9.2 Baseline Analyses

The Case Study began by conducting or recreating the hydrologic, hydraulic, and scour analyses for the bridge. These will yield the "baseline" values for use in the sensitivity analyses.

9.2.1 Hydrologic Analyses

The Case Study used the USGS StreamStats online tool to determine a drainage area of 2.9 square miles (see Figure 52). The reach did not have any recording stream gages.

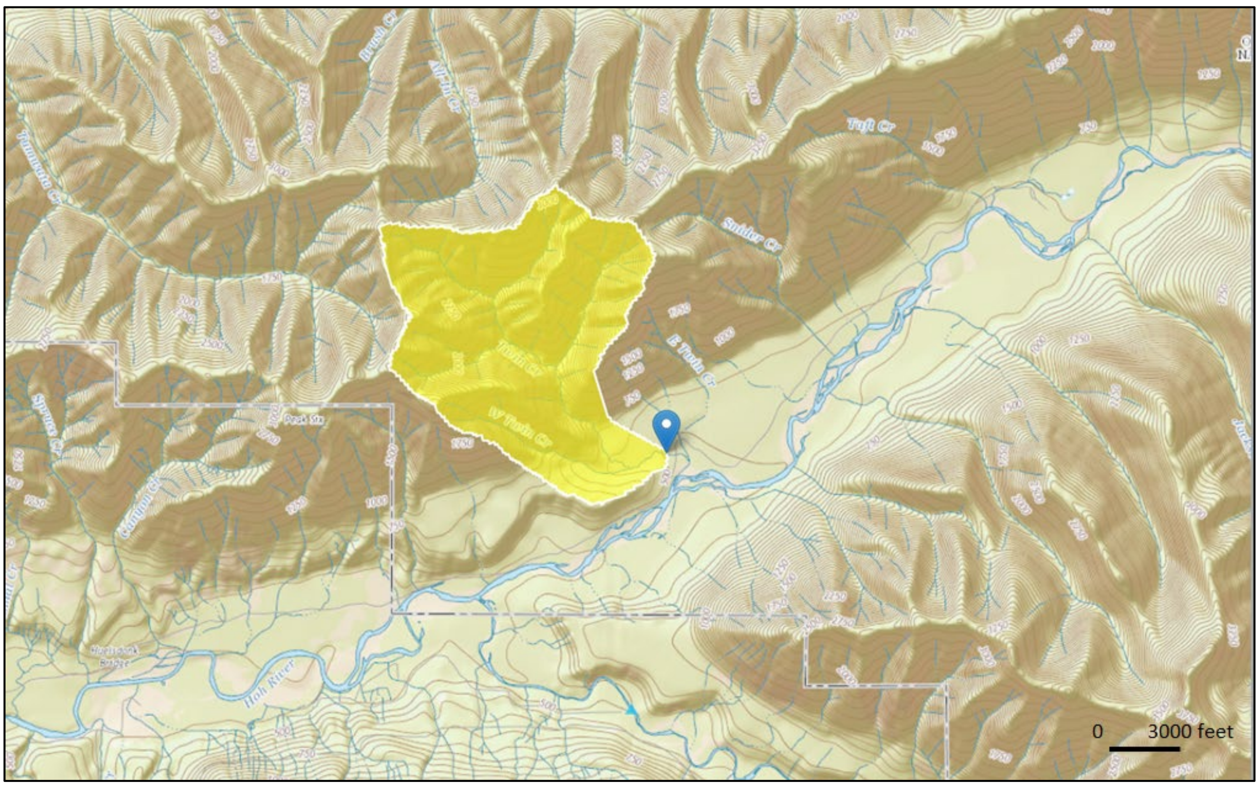

Figure 52. West Twin Creek Bridge – Drainage basin 2.9 square miles (USGS 2021).

The Case Study also used StreamStats to determine peak flood discharges at the crossing site. The regression equation used for this basin is from USGS *Report 97-4277*, *Magnitude and Frequency of Floods in Washington* (USGS 1998), with independent variables being area and mean annual precipitation. Table 34 presents the results of the hydrologic peak flow values.

Table 34. Estimated peak flows for West Twin Creek at Bridge site (units in cfs).

2-year	10-year	25-year	50-year	100-year	500-year
427	675	793	896	1,000	1,250

Of interest is that the 500-year flow equals 1,250 cfs. Later analyses will compare this value with the maximum hydraulic capacity of the bridge.

9.2.2 Hydraulic Analyses

The Case Study applied the USACE HEC-RAS model to simulate the hydraulic conditions of the stream and bridge. The HEC-RAS model developed extends approximately 600 feet downstream and 600 feet upstream of the bridge. The cross sections used to compile the model were from channel cross section ground survey. The Case Study used a total of 17 cross sections to represent the reach.

The Manning's n values used to represent the channel roughness in the model were 0.10 for the main channel and 0.07 for the overbank areas. Given the steep slope of the channel (approximately 2.0%), much of the flow is supercritical, which neccesitated use of the HEC-RAS mixed flow regime computations.

Table 35 provides resulting model outputs of water surface elevation, average velocities, and the energy gradeline for the 50-year flow event.

Maximum flood capacity was also of interest. The Case Study determined the maximum flood the structure could pass without inducing pressure flow or overtopping by performing an iterative run; increasing flow until the resulting water surface elevation touched the low chord of the bridge (i.e., having a freeboard of zero). This yielded a "maximum" flow of 7,000 cfs.

Ideally, the maximum flow would be that which puts the bridge into a pressure flow situation. Because the maximum flow was so much larger than the baseline flows, the study did not examine a pressure flow scenario.

9.2.3 Bridge Abutment Protection

The Case Study calculated riprap size using HEC-23 (FHWA 2012) and the Hydraulic Toolbox software (FHWA 2014). The analyses calculate riprap size and majority particle size (i.e., D_{50}) with maximum velocity at the bridge (from the hydraulic analyses) as the primary variable.

9.2.4 Scour Evaluations

The Case Study performed scour evaluations using the HEC-18 (FHWA 2012) methodology and HEC-RAS modeling results. The analyses calculated pier scour for the piles supporting the abutment structure. Because the sloped abutments are not expected to be present in the scour event, the abutment scour was not included in the total scour depth. The pier nose shape was assumed as rounded, and the angle of attack for the approach flow at the piers was estimated at 20 degrees. The bridge is subject to clearwater (as opposed to live bed) contraction scour. There is 6.7 feet of local scour, acting on the piles supporting the abutment.

Table 35. Baseline Hydraulic results for the West Twin Creek Bridge.

Cross Section Station	Water Surface Elevation (WSEL) (feet)	Average Cross Section Velocity (ft/s)	Energy Grade Line (EGL) Elevation (feet)
1235	527.11	7.18	527.96
1105	525.96	2.96	526.10
994	524.44	5.62	524.98
877	522.81	3.67	523.03
775	519.88	6.79	520.65
748	519.32	4.17	519.61
710	518.72	5.08	519.13
684	518.09	5.22	518.52
655	517.51	4.89	517.89
Bridge			
567	511.79	6.78	512.59
555	510.06	4.56	510.42
539	509.25	6.04	509.88
457	507.55	4.01	507.81
375	506.38	4.15	506.66
283	504.60	5.09	505.02
182	502.41	5.01	502.85
90	499.76	5.70	500.30

The Case Study assumed long term degradation to be controlled by equilibrium slope. For this analysis, long term degradation is a parameter that is expected to occur over time and not necessarily before events that cause contraction or local scour. The Case Study calculated long term degradation to be 8.5 feet per the Meyer-Peter, Muller Method in the Hydraulic Toolbox (FHWA 2014). Given the bridge construction in 2007 and assuming a 75-year service life, this structure has a remaining 68 years left of structural integrity. The summation of the local and long-term scour equaled 15.7 feet of scour depth, with a total scour elevation equal to 494.3 feet.

9.3 Sensitivity Analyses

The Case Study applied a series of sensitivity analyses to the hydrologic, hydraulic, riprap, and scour characteristics at the bridge.

9.3.1 Hydrologic Forcings

To simulate increased precipitation associated with climate change, the study increased the 50-year (hydraulic) and 100-year (scour) baseline flows by 10%, 20%, and 50%. Table 36 presents these flows.

Table 36. West Twin Creek Bridge – Design discharge increases (units in cfs).

Storm Event	Baseline Q	Q + 10%	Q + 20%	Q + 50%
50	896	986	1,075	1,344
100	1,000	1,100	1,200	1,500

9.3.2 Hydraulic Sensitivity

The Case Study placed the three 50-year flow scenarios (i.e., $Q_{10\%,50}$, $Q_{20\%,50}$, $Q_{50\%,50}$) into the HEC-RAS model developed in the baseline analyses. Table 37 provides design parameters and the results of the analysis for the 50-year design event just upstream of the bridge location. The table also includes that “maximum” (zero freeboard) flood as well.

Table 37. West Twin Creek Bridge – Hydraulic parameters (50-year event).

Hydraulic Parameters	Baseline Q	Q + 10%	Q + 20%	Q + 50%	Maximum
Water Surface Elevation (feet)	517.5	517.7	517.8	518.3	524.0
Clearance/ Freeboard (feet)	6.5	6.3	6.2	5.7	0.0

Recall that the FLHD PDDM specifies a minimum allowable freeboard of 2 feet. Given the bridge low chord elevation is 524 feet, all increased scenarios continue to meet the freeboard criteria. That indicates hydraulic resilience of the current bridge to these flood events.

9.3.3 Bridge Abutment Protection Sensitivity

The Case Study calculated riprap size D_{50}, with maximum velocity as the primary variable. As depicted in Table 38, as velocities are not expected to change more than by about 1 ft/s between baseline and $Q_{50\%,50}$ (i.e., 5.17 to 6.05 ft/s), the riprap size only changes by a few inches. The riprap gradation is such that these few inches keep Class III as the minimum for the baseline storm event, and increased percentage flood events.

Table 38. Abutment protection parameters (50-year) for West Twin Creek Bridge.

Abutment Protection Parameters	Baseline Q	Q + 10%	Q + 20%	Q + 50%	Maximum
Maximum Channel Velocity (ft/s)	5.17	5.36	5.55	6.05	10.51
Riprap D_{50} Size (in)	9.77	10.28	10.54	11.22	17.80
Riprap Class	III	III	III	III	V

9.3.4 Scour Sensitivity

The Case Study performed scour evaluations for the baseline, three increased flood events, and maximum (zero freeboard) events. Table 39 presents a summary of these scour evaluations.

Table 39. Scour analysis resultant data (values in feet).

Scour Variable	Baseline Q	Q + 10%	Q + 20%	Q + 50%	Maximum
Clearwater (CW) or Live Bed (LB)	CW	CW	CW	CW	LB
Contraction Scour	0.0	0.0	0.0	0.1	6.8
Long term degradation	8.5	8.5	8.5	8.5	8.5
Local Scour	6.7	6.9	7.2	7.7	19.6
Total Scour Depth	15.7	15.9	616.2	16.8	535.4
Total Scour Elevation	494.3	494.1	3493.8	493.2	4474.6
Local Scour Components					
Abutment Scour	0	0	0	0	0
Pier Scour	6.7	6.9	7.2	7.7	19.6

Recall the minimum pile tip elevation was 472 feet and thalweg elevation 510 feet. This gives a total initial pile embedment depth below the current channel thalweg of 38 feet. The predicted $Q_{50\%,100}$ total scour is only 1.1 feet deeper than the baseline storm event. Therefore, the study concludes there would be no expected effect on structure stability from these scenarios. However, for the maximum flood conditions (i.e., 7,000 cfs) it is unlikely the bridge would be able to remain stable with less than 3 feet of bury depth on the pile. Recall that such a flood event is far more than the 500-year event (1,250 cfs).

9.4 Summary

The Case Study examined the existing West Twin Creek Bridge for capacity, scour, and riprap protection for the baseline design storms, increased baseline flows by 10%, 20%, and 50%, and at the maximum capacity flow. The Case Study also examined climate change predictions for the Hoh River Watershed, of which the West Twin Creek Bridge is a part, to validate baseline flow increase percentages. Because the range of flows analyzed were greater than the climate change predictions, the structure can be considered adequate for capacity, scour, and riprap protection.

Chapter 10. Snider Creek Culvert

The Snider Creek Culvert (coordinates: N 47.844146, W 123.966680) is approximately 16.1 miles east of United States Highway 101 (US 101) on the Upper Hoh Road. The culvert allows Upper Hoh Road to cross the creek. The confluence with the Hoh River is approximately 550 feet downstream of the culvert. Figure 53 provides a 3D visualization of the Case Study site (looking north towards the portions of the Olympic Mountain range). Figure 54 and Figure 55 provide topographic and aerial views of the site location.

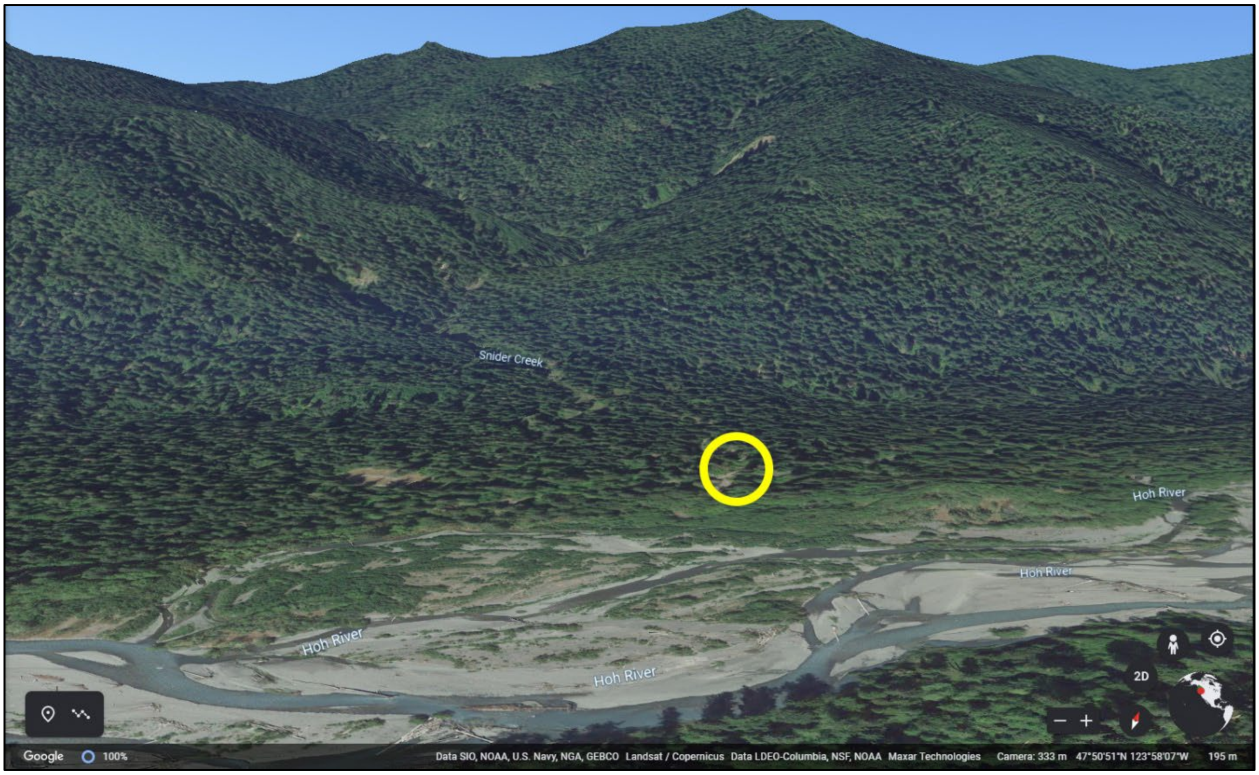

Figure 53. 3D visualization of Snider Creek area (Google 2021).

10.1 Project Area Description

The Snider Creek watershed is a tributary to the Hoh River basin in northwestern Washington State, within the Olympic National Park. The road crosses an active alluvial fan. Snider Creek Culvert is not significantly skewed relative to the stream. The structure is skewed approximately 12 degrees to the roadway centerline. Upstream of the culvert is steeply sloped (5%), entrenched (entrenchment ratio 1.1), and has low sinuosity. Downstream is moderately sloped (3%), slightly entrenched (entrenchment ratio greater than 2.2), and has low sinuosity. Upstream lateral channel migration is largely controlled by valley wall colluvium. The stream is perennial. Active channel width is 16 feet both upstream and downstream (see Figure 56). Bankfull depth is approximately 3 feet.

The Case Study applied the USGS StreamStats on-line tool to determine the 1.1 square mile drainage area (see Figure 57 for the watershed delineation). The Case Study did not identify any recording stream gages along this reach.

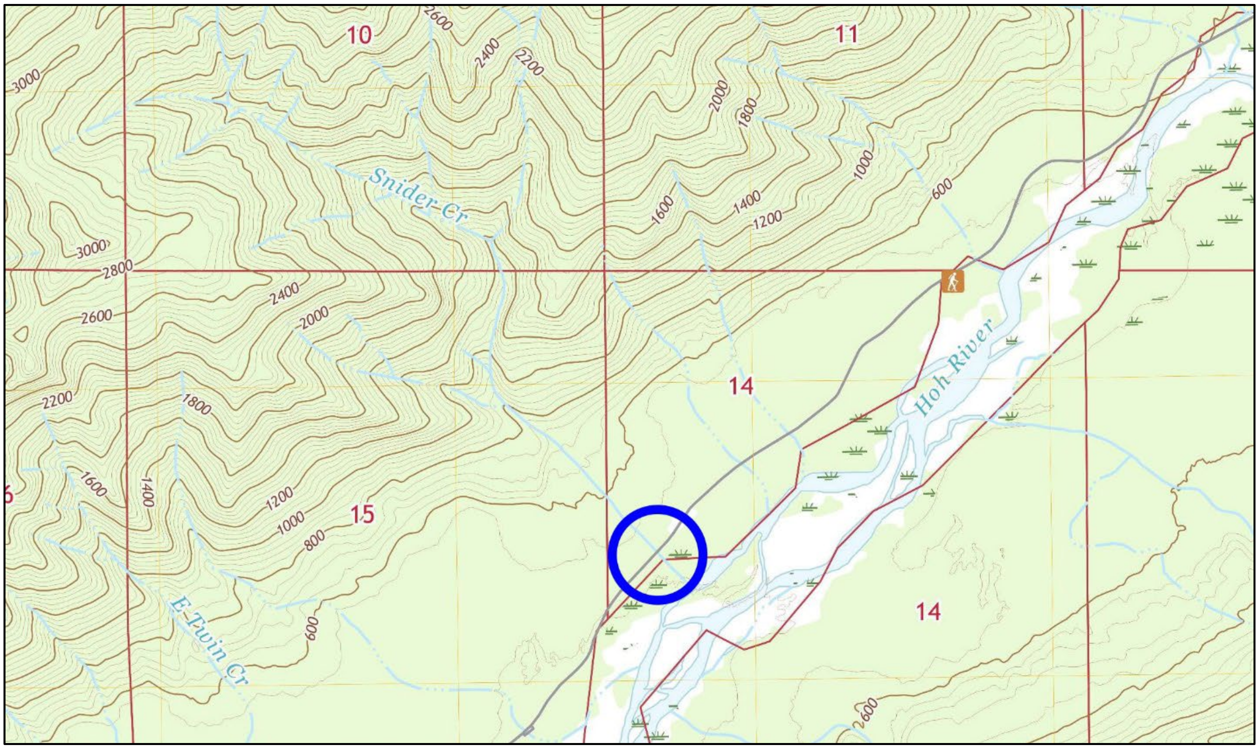

Figure 54. Snider Creek topographic map (USGS 2021).

Figure 55. Aerial view of Snider Creek Culvert (Google 2021)

Figure 56. Looking upstream towards the Snider Creek Culvert.

Figure 57. Snider Creek Culvert drainage area (USGS 2021).

The watershed is mountainous, with elevations from 500 to 3,500 feet, and over 88% having slopes greater than 30 percent. The mean elevation is 2,090 feet. The entire drainage area is heavily forested, with almost 90% covered by canopy. The mean annual precipitation is approximately 141 inches. The mean December/January/February (DJF) temperature is between 32- and 36-degrees Fahrenheit. Rainfall is the dominant stream-flow control.

10.1.1 Culvert Description

The Snider Creek Culvert is an 82-foot long structural plate arch structure (see Figure 56 and Figure 58). The span is 16.5-feet and rise 11-feet. Both inlet and outlet inverts are at the 1,995.6-foot elevation. The inlet and outlet have been beveled to a 2(h) to 1(v) slope. The barrel has been placed into an excavation of the stream bed (about 2 to 4 feet of embedment). The bottom of the culvert was then filled with stream material to match with the existing stream grade (which is at about the 2,000-foot elevation). The roadway crown is at approximately the 2,010-foot elevation.

10.1.2 Design Criteria

The FLHD Project Development and Design Manual (PDDM) specifies use of the 50-year flood for hydraulic capacity. The Case Study determined such hydraulic capacity, measured by headwater elevations and headwater-to-diameter (Hw/D) ratios, using the FHWA HY-8 software program.

The FLHD PDDM specifies that, for new culverts, the minimum allowable headwater elevation is below the subgrade elevation (FHWA 2014). Additionally, the PDDM specifies a Hw/D ratio of between 0.8 and 1.0 for culverts located in drainages with a high potential for sediments or debris.

10.2 Baseline Analyses

The Case Study conducted a hydrologic, hydraulic, sediment/debris and stream stability analyses for the baseline conditions. These provide some understanding of the situation found at the site for the designed conditions. This section also provides some discussion relevant to understanding the analyses.

10.2.1 Hydrologic Analyses

The Case Study applied the USGS StreamStats regression equations to calculate peak flood discharges at the crossing site (see Table 40). The regression equation used for this basin is from USGS *Report 97-4277, Magnitude and Frequency of Floods in Washington* (USGS 1998) that uses area and mean annual precipitation as independent variables.

Table 40. Estimated peak flows for Snider Creek at Culvert (cfs).

2-year	10-year	25-year	50-year	100-year	500-year
117	280	329	371	415	516

10.2.2 Hydraulic Analysis

When flooding occurs, multiple outcomes are seen at the location of a hydraulic structure, such as a bridge or culvert. The structure can pass the increased streamflow and associated excess debris and sediment, or it can become inundated and overtopped without causing harm to the roadway, or it can be undermined and the roadway destroyed at the crossing. The structure needs to have the capacity to pass flows as well as associated debris and sediment. The analyses examined headwater elevation (headwater-to-diameter ratio) to determine adequate hydraulic capacity.

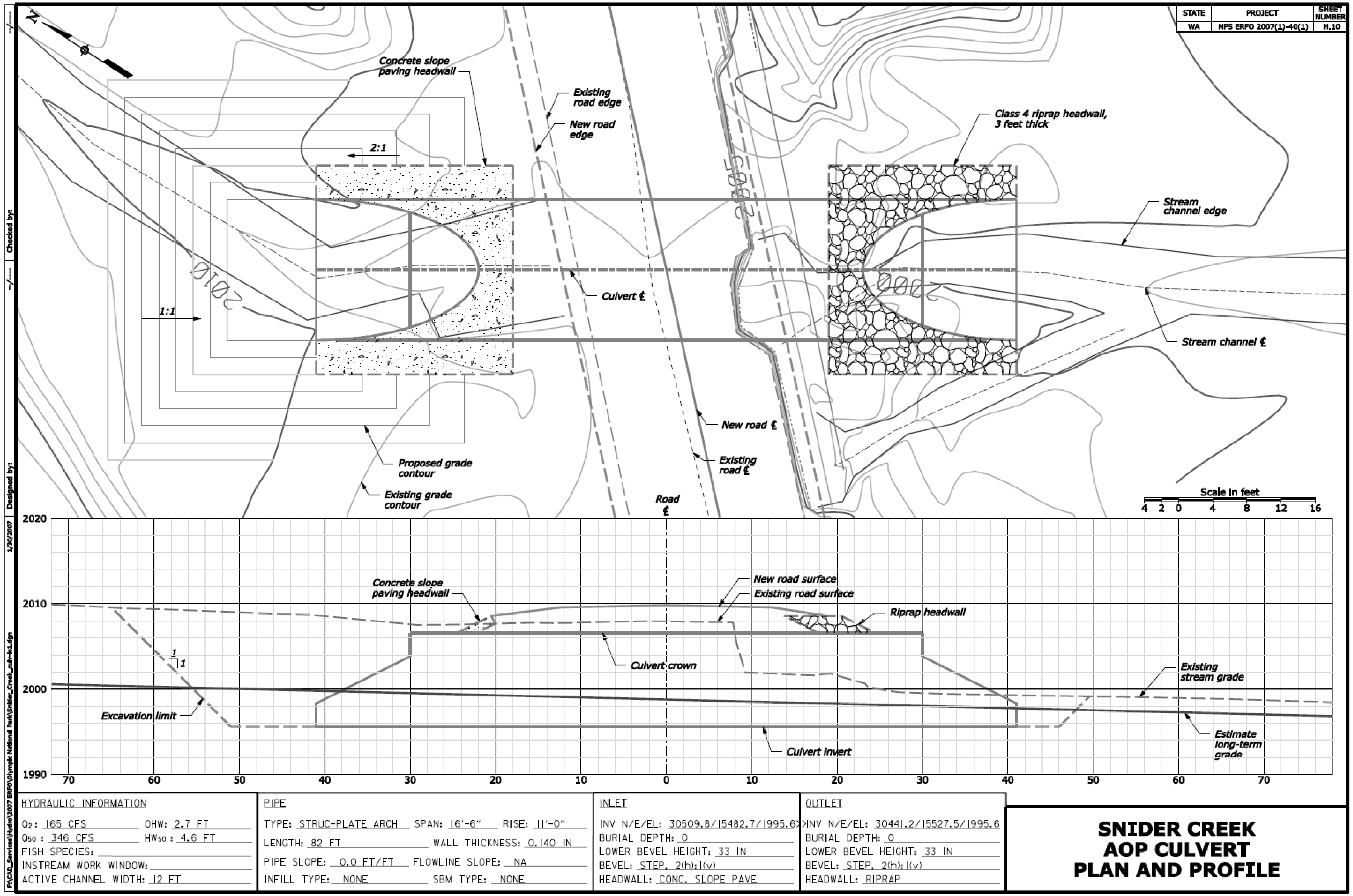

Figure 58. Snider Creek Culvert – Plan and profile sheet.

The Case Study entered site geometry and design flows into the FHWA HY-8 model. Maximum capacity was also of interest. To determine the maximum flow the culvert structure could pass, the Case Study performed an iterative analysis increasing flow until headwater elevation met the top of subgrade. The Case Study estimated subgrade elevation as the top of the concrete headwall at the culvert inlet.

For the 50-year design event, the resulting hydraulic parameters were a headwater elevation of 2,004.9 feet, headwater depth of 5.1 feet, freeboard value of 1.8 feet, and the resulting Hw/D of 0.74.

Overtopping of the road occurs at 860 cfs (beyond the 500-year event. Figure 59 presents the culvert performance curve for this culvert.

10.2.3 Debris & Sediment Passage

Plane-bed is the dominant channel bed morphology. Bed load material observed both upstream and downstream of the crossing appears to be predominantly gravel and cobbles, though upstream observation yielded some small to medium boulders. A digital gradation analysis from photos taken on site yielded the gradation distribution in Table 41 for upstream and downstream streambeds.

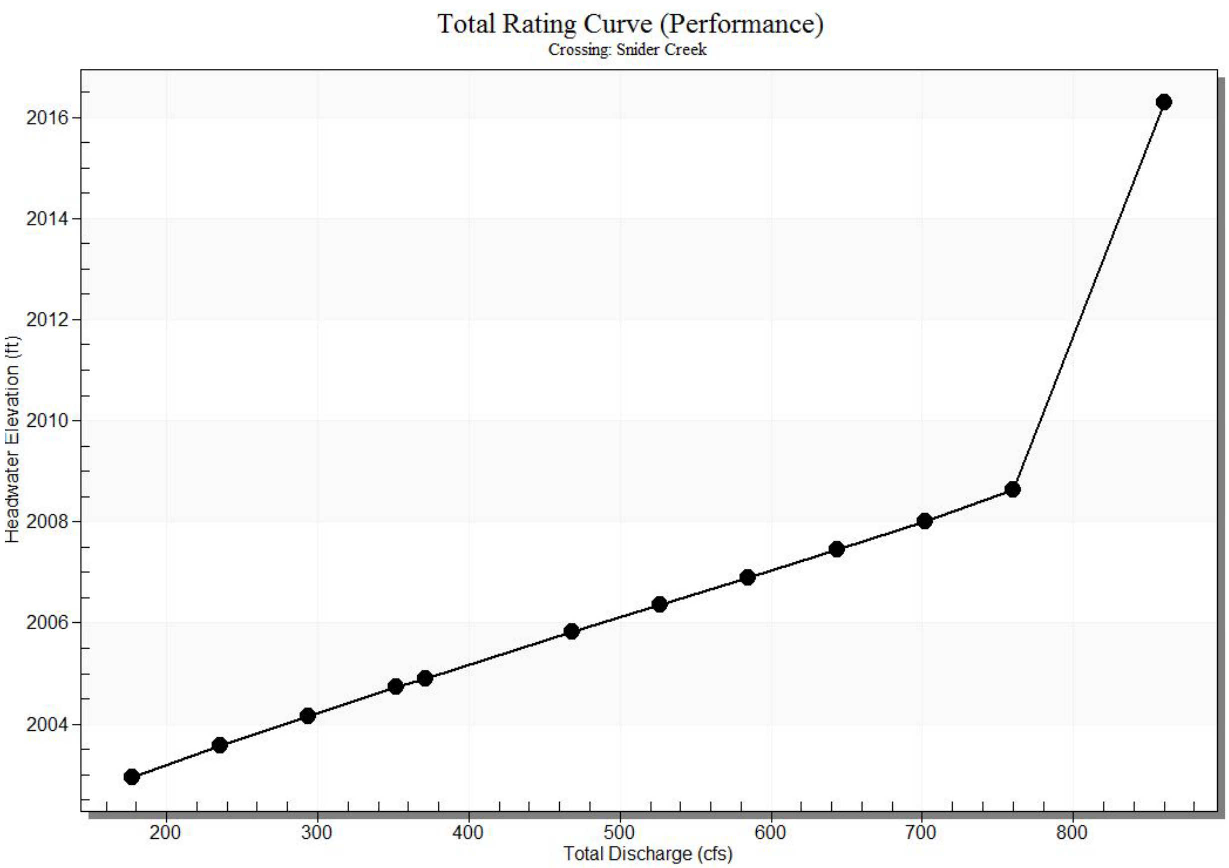

Figure 59. Snider Creek Culvert performance curve.

Table 41. Gradation table for upstream and downstream streambeds (in inches).

Location	D_5	D_{15}	D_{50}	D_{85}	D_{100}
Upstream	1.7	3.1	7.6	18	24
Downstream	0.83	1.5	3.1	6.6	9.5

Figure 60 provides a gradation plot of Snider Creek upstream and downstream streambed material (a digital gradation analysis produced the values). Large woody debris in the channel and floodway indicate a moderate debris supply. Abundant, mid-coarse sediment deposits indicate the stream is transport-limited. Snider Creek is aggrading. Riparian vegetation is predominantly large conifer trees and dense underbrush, with willows dominating the floodplain of the creek near its confluence with the Hoh River. No bedrock was observed in the stream banks or bottom near the crossing. The stream banks appear stable where vegetated.

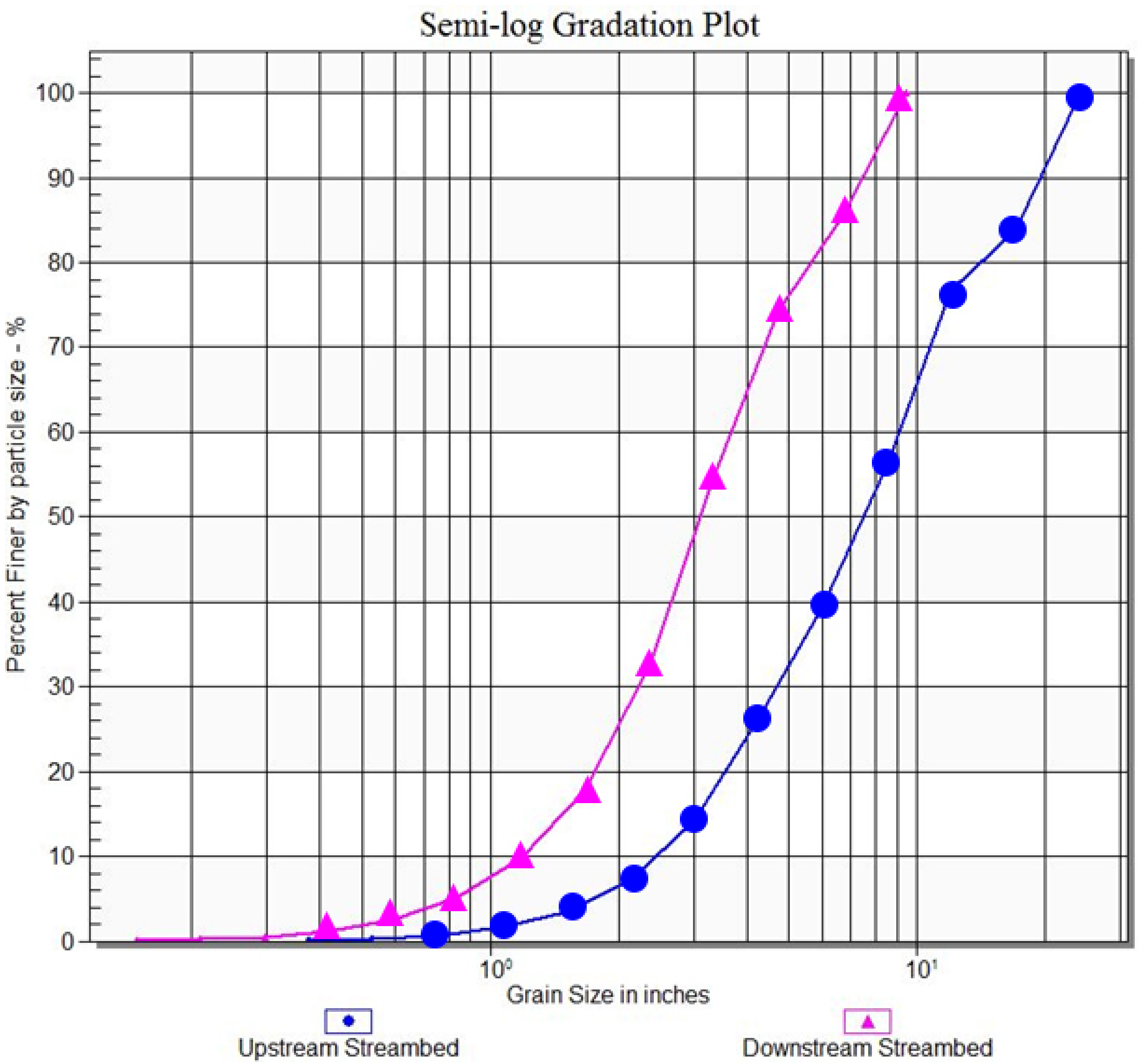

Figure 60. Upstream and downstream gradation plot of streambed material.

10.2.4 Vertical Channel Stability

Channel aggradation or degradation can occur with a change in discharge or a change in sediment supply. A stable channel will neither aggrade (deposit sediment) nor degrade (remove sediment). Unstable channels, those that are transitioning to either steepened or flattened slope, can be a result of some change in environment.

Snider Creek crossing is over an unstable channel. As the profile in Figure 58 indicates, the reach upstream of the crossing has aggraded due to extreme grade transition at the road crossing (see Figure 61 for a photo that depicts a point bar upstream of the crossing). Aggradation is expected to continue.

Figure 61. Point bar upstream of Snider Creek culvert crossing.

10.3 Sensitivity Analyses

The Case Study applied sensitivity analyses to these baseline elements.

10.3.1 Hydrologic Forcings

To simulate increased precipitation associated with climate change, the baseline peak design discharges were increased by 10%, 20%, and 50% for 50 and 100-year events. Table 42 presents these flows. The “maximum” flow is 760 cfs, more than either the $Q_{50\%,50}$ and $Q_{50\%,100}$ events.

Table 42. Snider Creek Culvert - Design discharge increases (units in cfs).

Storm Event	Baseline Q	Q + 10%	Q + 20%)	Q + 50%
50-year	371	408	445	557
100-year	415	457	498	623

10.3.2 Hydraulic Sensitivity

Once again, the study applied the HY-8 culvert model to compute the hydraulic parameters using the baseline and sensitivity flow events. Table 43 presents the results of the modeling efforts. Overtopping of the roadway occurs at 860 cfs.

Table 43. Snider Creek Culvert hydraulic analyses results.

Hydraulic Parameters	Baseline Q	Q + 10%	Q + 20%	Q + 50%	Maximum
Headwater Elevation (feet)	2004.9	2005.3	2005.6	2006.6	2008.6
Headwater Depth (feet)	5.1	5.5	5.8	6.8	8.8
Clearance/ Freeboard (feet)	1.8	1.4	1.1	0.1	0.0
Headwater-to-Diameter Ratio, HW/D	0.74	0.80	0.84	0.99	1.28

10.4 Summary

The Case Study examined the existing Snider Creek Culvert for capacity and vertical stability for the baseline design storms, increased baseline flows by 10%, 20%, and 50%, and at the maximum capacity flow. The Hoh River Watershed, of which the Snider Creek Culvert is a part, climate change predictions for streamflow was compared with increases examined in the study. For the predicted maximum increase of 20% above baseline flows the structure is adequate for capacity and vertical stability.

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Chapter 11. East Twin Creek AOP Culvert

Culvert installations have the potential to restrict the movement of aquatic organisms, including, but not limited to, fish. Lengths of stream habitat that have been historically accessed by fish will often become inaccessible or disjointed due to a roadway crossing. A culvert which is installed because of the crossing may create a barrier initially, or it may also have one develop over time. To mitigate these impediments, highway crossings, referred to as aquatic organism passage (AOP) structures seek to enable natural streamflow performance.

The US National Park Service (NPS) identified the culvert at East Twin Creek (coordinates N 47.834513°, W -123.98929°) on the Upper Hoh River Road as a barrier to salmon migration and asked WFLHD to provide conceptual level design and AOP culvert recommendations. Figure 62 provides a 3D visualization of the Case Study site (looking north towards the portions of the Olympic Mountain range). Figure 63 and Figure 64 provide topographic and aerial views of the site location.

Figure 62. 3D visualization of ETCA Culvert vicinity (Google 2021)

The WFLHD designed the East Twin Creek Culvert to a conceptual level in 2002. That conceptional design called for construction of a 20-foot diameter, multi-plate structure. The location of the resulting East Twin Creek AOP (ETCA) culvert is approximately 14.5 miles east of United States Highway 101 (US 101) on the Upper Hoh River Road. The ETCA culvert is not significantly skewed relative to the stream. The Case Study evaluated hydraulic capacity, structure stability, and functionality of the site.

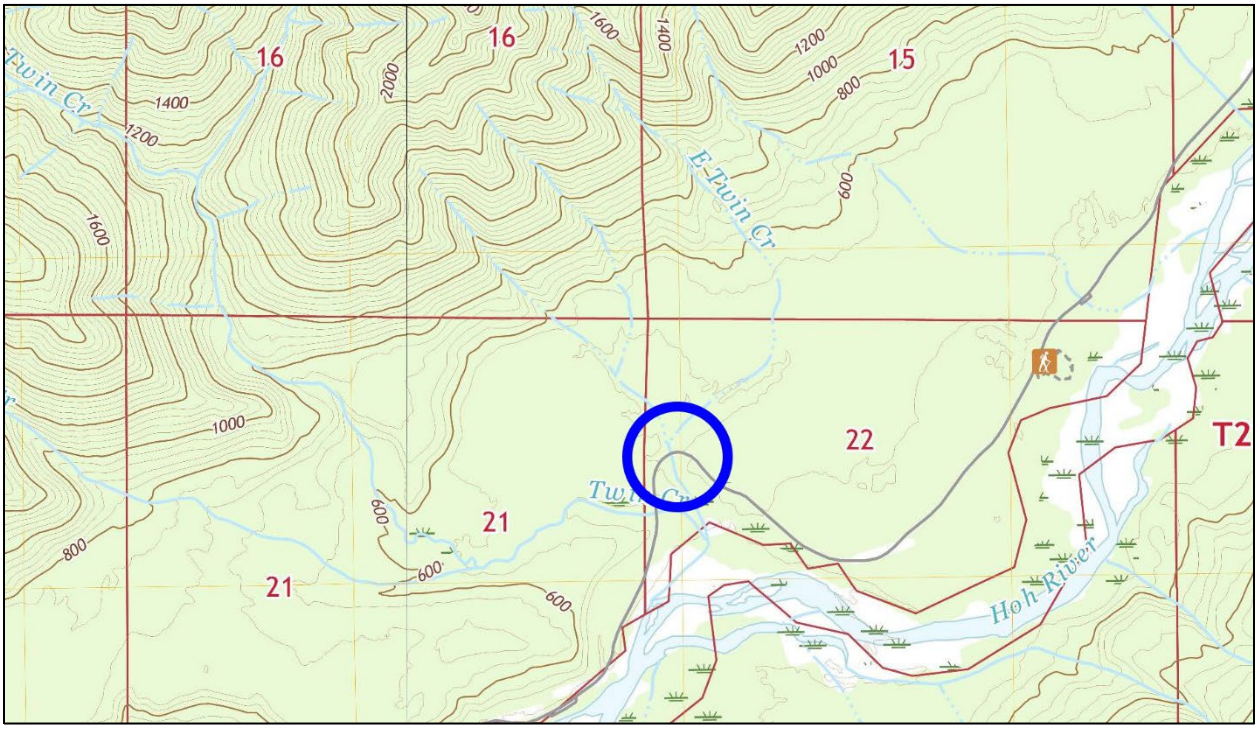

Figure 63. Topographic map of ETCA Culvert vicinity (USGS 2021)

Figure 64. Aerial photo of ETCA Culvert project area (Google 2021).

11.1 Project Area Description

The East Twin Creek watershed is a tributary to the Hoh River basin in northwestern Washington State, within the Olympic National Park. The Case Study site was not accessible during a June 2015 visit. Rather, the study used observations of characteristics during previous site visits (primarily after flood events in December of 2001) to provide the following descriptions.

The watershed is mountainous, with elevations from 500 to 2,800 feet, and over 70% having slopes greater than 30 percent. The mean elevation is 1,070 feet. The entire drainage area is heavily forested, with over 87% covered by canopy. The mean annual precipitation is approximately 140 inches. Rainfall is the dominant streamflow control.

Immediately upstream of the crossing, two equally sized stream channels merge. Both streams are moderately sloped (approximately 2%), entrenched (entrenchment ratio < 1.4), and have low sinuosity. The stream downstream is moderately sloped, slightly more entrenched, and less sinuous. Bankfull width is approximately 14 feet. Bankfull depth is approximately 2 feet. The stream lies in a narrow colluvial valley with flood prone area widths between 14 and 20 feet. Floodplains are poorly developed. The crossing is approximately 700 feet upstream from the confluence with West Twin Creek. The stream is perennial. Spring and storm flow are the dominant stream-flow control.

Steps and pools are the dominant channel bed morphology. Steps in the channel develop at large woody debris sills. Bed load material observed both upstream and downstream of the crossing appears to be predominantly sand and gravel. A grain size analysis completed on a bedload sample collected immediately downstream of the crossing site indicated a D_{50} of 0.4 inches. The largest particle size measured was 1.2 inches. Bank materials appear predominantly silt and sand. Riparian vegetation is predominantly large conifer trees and dense underbrush. No bedrock was observed in the stream banks or bottom near the crossing.

Large woody debris in the channel and floodway, noted in Figure 65, indicate an abundant debris supply. The large woody debris is more abundant upstream of the crossing. Abundant vegetation, moss, and organic soil established on the woody debris indicate it may remain stationary for long periods of time. Eroding banks, sediment stored upstream of woody debris sills, and pools refilled with bedload suggest a high sediment supply.

Downstream of the culvert stream banks are generally undercut and thinly vegetated and appear unstable. Upstream the banks are vegetated and generally not undercut and appear moderately stable. Based on the surveyed stream profile, the deeper entrenchment, and the undercut and unstable banks, the stream below the crossing appears to be degrading. The stream above appears moderately stable.

11.1.1 Design & Other Criteria

The design of AOP culvert crossings involves ensuring passage of two main components, (a) streamflow with associated sediment and debris in a downstream direction, and (b) aquatic organisms in both upstream and downstream direction. Changing precipitation and weather patterns can affect an AOP culvert in multiple ways. Higher flows, as with any hydraulic structure under a roadway, can pass the increased streamflow and associated excess debris and sediment, can become inundated and overtopped without causing harm to the roadway, or can be undermined and the roadway destroyed at the crossing.

Figure 65. East Twin Creek AOP Culvert – Looking downstream.

A key element of AOP culverts is to provide a natural stream bottom, or streambed material. Higher flows, in addition to impacting the roadway, can also impact the integrity of the AOP streambed material by blowing out the associated embedment material. Low flows, on the other hand, can create subsurface flows and cause organism stranding.

Fish need some minimum depth of water in which to swim. Such needs and associated criteria vary by species, size, and age. Individual State agencies often determine their own sets of such criteria based on the local species. The Washington Department of Fish and Wildlife (WDFW) *Water Crossing Design Guidelines* (2013) has specific requirements for minimum water depths based on species type and age. The most conservative approach would be to consider the adult trout greater than 6 inches. For this type of fish, 0.8 feet is the minimum allowable water depth for passage.

The FLHD PDDM specifies that, for new culverts, the minimum allowable headwater elevation is below the subgrade elevation (FLHD 2014). The PDDM specifies a headwater-to-diameter ratio of between 0.8 and 1.0.

11.1.2 Culvert Characteristics

The design used Washington Department of Fish and Wildlife guidelines in the sizing, slope, and embedment depth determination. Figure 66 provides a conceptual plan and profile sheet. The proposed culvert would be a 124-foot structural plate arch structure, embedded into the channel to allow a natural stream bed profile.

11.1.3 Other Parameters Considered

In addition to hydraulic parameters to be considered, the design of an AOP structure also neccesitates examination of bed material characteristics, including gradation and permissible shear stress. Natural streambed material embedment in the culvert is advantageous for aquatic habitat. Lower boundary layer velocities at the culvert walls are associated with roughening caused by this streambed material and can provide areas for fish to hold and rest, or swim upstream through culverts. It is important that the streambed material provided in a culvert does not get scoured out. Shear stresses and streambed gradation are the key components to be compared for increased predicted climate flows.

Determining suitable streambed material necessitates determination of appropriate sizing and gradation to simulate natural stream processes. Site conditions may indicate an expectation of continued downstream degradation conditions. Accommodating such conditions may use a closed bottom structure and providing adequate streambed material and grade control structures.

11.2 Baseline Analyses

The Case Study conducted hydrologic, hydraulic, and structure stability analyses as the baseline conditions. These provide some understanding of the situations found at the site. This section also provides some discussion relevant to understanding some analyses.

11.2.1 Hydrologic Analysis

The Case Study determined a drainage area of 0.58 square miles using the USGS StreamStats on-line tool (Figure 67 depicts the basin delineation). The Case Study did not identify any recording stream gages along this reach.

The Case Study determined peak flood discharges at the crossing site with StreamStats. The regression equation used for this basin is from USGS *Report 97-4277, Magnitude and Frequency of Floods in Washington* (1998). Table 44 presents the resulting peak flows for various return periods.

This table also includes the 2-year flow that will serve as a baseline for subsequent low-flow sensitivity analyses.

Table 44. Estimated peak flows at ETCA Culvert (units in cfs).

2-year	10-year	25-year	50-year	100-year	500-year
97	154	181	204	228	283

11.2.2 Hydraulic Analysis

When flooding occurs, multiple outcomes are seen at the location of the culvert. The structure can pass the increased streamflow and associated excess debris and sediment, or it can become inundated and overtopped without causing harm to the roadway, or it can be undermined, and the roadway destroyed at the crossing. The structure should have the capacity to pass flows as well as associated debris and sediment. The analyses examined headwater elevation (headwater-to-diameter ratio) to determine adequate hydraulic capacity.

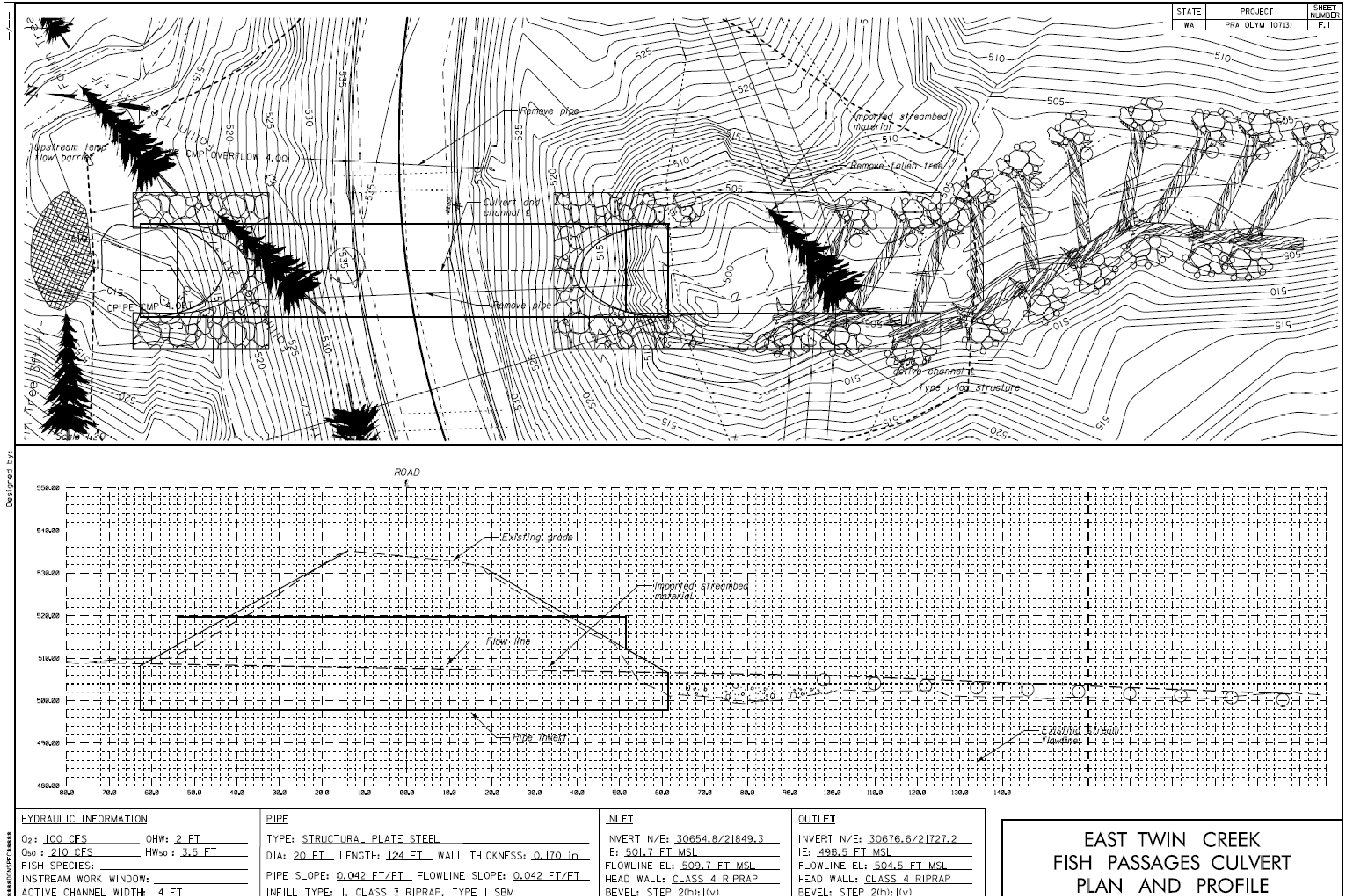

Figure 66. East Twin Creek AOP Culvert – Plan and profile sheet.

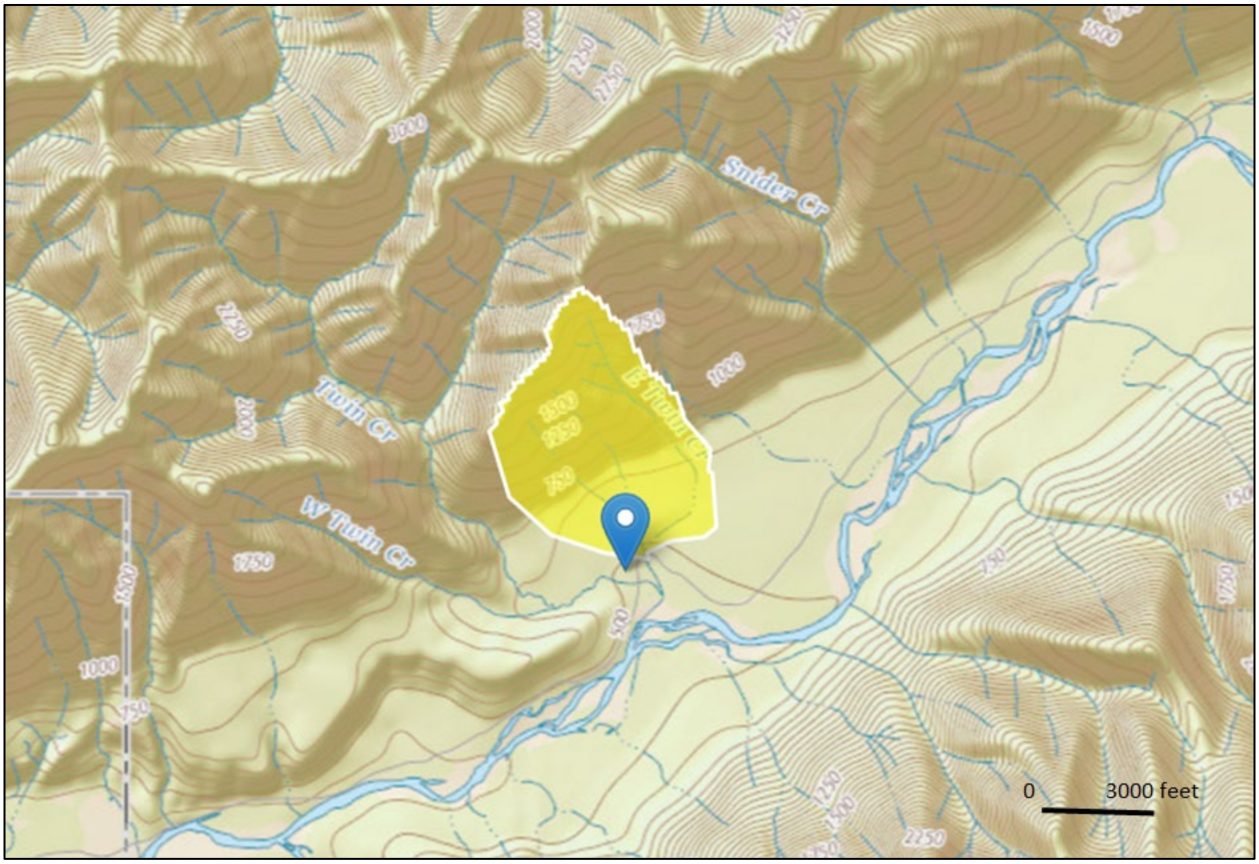

Figure 67. Drainage basin for ETCA Culvert (USGS 2021).

The Case Study entered site geometry and design flows into the FHWA HY-8 culvert model. Maximum capacity was also of interest. To determine the maximum flood the culvert structure could pass, the study performed an iterative analysis increasing flow until headwater elevation met the top of subgrade. The Case Study estimated subgrade elevation as the top of the concrete headwall at the culvert inlet.

For the 50-year design event, the resulting hydraulic parameters were the headwater elevation of 509.6 feet, headwater depth of 1.6 feet, freeboard value of 6.9 feet, and the resulting headwater to diameter ratio of 0.17. Overtopping of the road occurs at 4,464 cfs (beyond the 500-year event). Figure 68 presents the performance curve for this culvert (the upper point represents overtopping).

11.2.3 Stream Stability

The Case Study selected and conserved culvert streambed material embedment from the existing streambed material immediately upstream and downstream of the crossing. The armoring which has occurred historically has sufficiently armored the streambed upstream and downstream, resulting in a stable streambed. However, the study will characterize the sensitivity to potential increases and decreases of flow in subsequent sections.

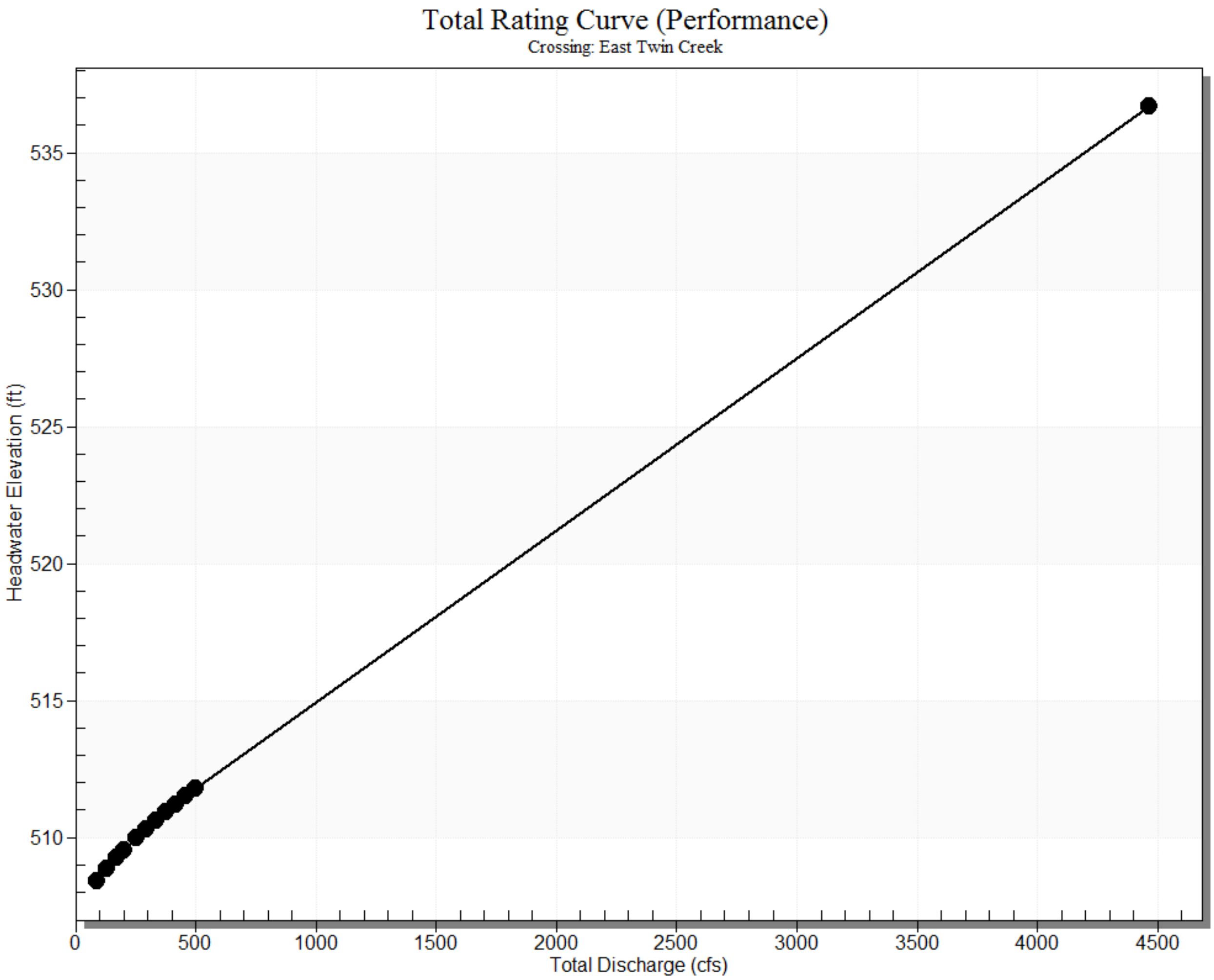

Figure 68. Culvert performance curve for ETCA.

11.2.4 AOP Functionality

As described earlier, fish need some minimum depth of water in which to swim. Specific conditions vary by species, size, and age. The most conservative approach would be to consider the adult trout greater than 6 inches. Using the Washington State materials (2013), this species of fish needs 0.8 feet as the minimum allowable water depth for passage. The depth from the hydraulic analysis (1.6 feet) appears to be sufficient to allow passage at design event. However, low flow events also occur and influence functionality as well. In the next sections, the study will seek to understand how such changes may affect the AOP functionality.

11.3 Sensitivity Analyses

The sensitivity analyses include hydrologic, hydraulic, stability, and functionality analyses. As the culvert serves as an AOP, the sensitivity will investigate both increasing certain design events by 10%, 20%, and 50%, but also decreasing other events by a similar range of percentages.

11.3.1 Hydrologic Forcings

To simulate increased precipitation for 50-year event, the study increased the baseline peak design discharge (i.e., 204 cfs) by 10%, 20%, and 50%. Table 45 presents these values; also, providing the maximum flow (as described above).

Table 45. Design discharge increases for ETCA Culvert (units in cfs).

Storm Event	Baseline Q	Q + 10%	Q + 20%	Q + 50%	Maximum
50-year	204	224	245	306	4,335

11.3.2 Hydraulic Sensitivity

The Case Study took those three flow scenarios and once again, used the HY-8 culvert model to determine resulting hydraulic parameters. Table 46 presents the hydraulic parameters. Table 47 represents the percent change for these parameters. For example, headwater depth for the $Q_{10\%,50}$ appears not to be as sensitive to change as the $Q_{50\%,50}$, that appears to have a correlation.

Table 46. HY-8 hydraulic results for ETCA Culvert.

Hydraulic Parameters	Baseline Q	Q + 10%	Q + 20%	Q + 50%	Maximum
Headwater Elevation (ft)	509.6	509.7	509.9	510.4	534
Headwater Depth (ft)	1.6	1.7	1.9	2.4	26
Clearance/ Freeboard (ft)	6.9	6.8	6.6	6.1	0.0
Headwater-to-Diameter Ratio, HW/D	0.17	0.18	0.20	0.25	2.74

Table 47. Hydraulic sensitivity for ETCA Culvert (% difference).

Hydraulic Parameters	Baseline Q	Q + 10%	Q + 20%	Q + 50%	Maximum
Headwater Elevation	0%	0.02%	0.06%	0.16%	4.8%
Headwater Depth	0%	6%	19%	50%	1525%
Clearance/ Freeboard	0%	1%	4%	12%	100%
Headwater-to-Diameter Ratio	0%	6%	18%	47%	1512%

If desired, further analyses could introduce additional flow change scenarios to these parameters to discern any insightful trends or outcomes applicable to this culvert. However, as always, the site-specific nature of hydrology, hydraulics, and site conditions do not allow making assumptions of causality of any outcomes.

11.3.3 Stream Stability Sensitivity

As described above, at this site, historic armoring has resulted in a stable streambed. However, for streamflow increases, additional armoring with channel incision could be a result. The Case Study analyzed bed stability at increased streamflow using the Permissible Shear Stress method in HEC-26 *Culvert Design for Aquatic Organism Passage* (FHWA 2010).

As this site contains a natural bed channel, the study used the Modified Shield's Equation to calculate permissible shear stress, given specific bed characteristics.

Next, the analysis calculated applied shear stress for the baseline, as well as increased flows. Comparing these shear stresses yielded the results found in Table 48.

Table 48. Stability parameters for ETCA Culvert.

Stability Parameters	Baseline Q	Q + 10%	Q + 20%	Q + 50%	Maximum
Water Depth (ft)	1.6	1.7	1.9	2.4	26
Maximum Velocity (ft/s)	1.47	1.52	1.60	1.80	5.93
Permissible Shear Stress (lb/ft2)	0.21				
Resultant Parameter					
Actual Shear Stress (lb/ft2)	4.2	4.5	5.0	6.3	30
Permissible > Actual?	No	No	No	No	No

11.3.4 AOP Functionality

The Case Study selected the 2-year storm event as the baseline for low flows. To simulate decreased precipitation associated with climate change, the study decreased this flow by 10%, 20%, and 50%. The Case Study then hydraulically computed the resulting water depths associated with these flow scenarios. If the 0.8-foot depth represents the minimum for the species of interest (i.e., trout), then the sensitivity analyses allows determination of AOP functionality. The Case Study also iteratively calculated the minimum low flow for functionality. Table 49 presents the outcomes from these analyses.

Table 49. East Twin Creek AOP Culvert – Functionality parameters.

Functionality Parameters	Baseline Q	Q - 10%	Q - 20%	Q - 50%	Minimum
Flows (cfs) 2-year Baseline	97	87	78	49	63
Water Depth (feet)	1.0	0.98	0.91	0.69	0.80
Water Depth > 0.8 feet?	Yes	Yes	Yes	No	-

Performing such analyses is inherently full of assumptions and prone to misrepresent the complexity of actual AOP functionality. However, the analyses provide a framework for more developed and nuanced approaches seeking to investigate the topic.

11.4 Summary

The Case Study examined the preliminary design for the East Twin Creek AOP Culvert for capacity and stability for the 50-year baseline design storm, increased baseline flows by 10%, 20%, and 50%, and at the maximum capacity flow.

The Case Study also examined the culvert for functionality for the 2-year baseline design storm, decreased flows by 10%, 20%, and 50%, and at the minimum functionality flow. The analysis compared climate change predictions for streamflow in the Hoh River Watershed, of which the East Twin Creek AOP Culvert is a part, with increases examined in the study. For the predicted maximum increase of 20% above baseline flows, the structure is adequate for capacity.

The existing bed material proposed for the culvert has a higher applied shear stress than the permissible shear stress for baseline as well as increased flows. It is likely that the bed material placed within the culvert could become scoured out. The Case Study suggests adjusting the bed material gradation to incorporate some larger material to resist increases in shear stress resulting from higher flows.

The Case Study also compared climate change predictions for streamflow in the Hoh River Watershed, of which the East Twin Creek AOP Culvert is a part, with decreases examined in the study. For the predicted maximum decrease of 60% below baseline flows, the structure would likely need to be adjusted for low flow functionality.

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Chapter 12. Hoh River Bank Stabilization

Many roadways were built, and continue to run, parallel to river channels, along the edge of a valley wall and within a natural floodplain. Thus, these roadways can be at risk of inundation, overtopping, and embankment destabilization. When threatened, protecting these roadways from the adjacent waterway often becomes a primary objective. Multiple reports and studies identified a bank stabilization site along the Upper Hoh River Road (coordinates: N 47.823°, W -124.187°). The NPS asked WFLHD to investigate means to accomplish bank stabilization at this site.

Figure 69 provides a 3D visualization of the Case Study site (looking north towards the portions of the Olympic Mountain range). Figure 70 and Figure 71 provide topographic and aerial views of the site location.

Figure 69. 3D visualization of Upper Hoh River stabilization site (Google 2021).

The site has 2,570 lineal feet of proposed bank stabilization. The project evaluated two bank stabilization design options. These included stream barbs with mitigation logs and wood buffer with dolosse ballast. Based on hydraulics analyses and cost, the NPS selected an installation of wood buffer with dolosse ballast option. The Case Study also evaluated overtopping, structure stability, and scour potential.

12.1 Project Area Description

The Hoh River drains the western slope of the Olympic Mountains. The river originates on the slopes surrounding Mount Olympus and adjacent mountain peaks at an elevation of 7,800 feet (NAVD 88) and flows approximately 50 miles through relatively wide, moderately high-relief, glacial valleys before discharging to the Pacific Ocean. The site is at river milepost 20 to 20.4 and at elevation 245 feet.

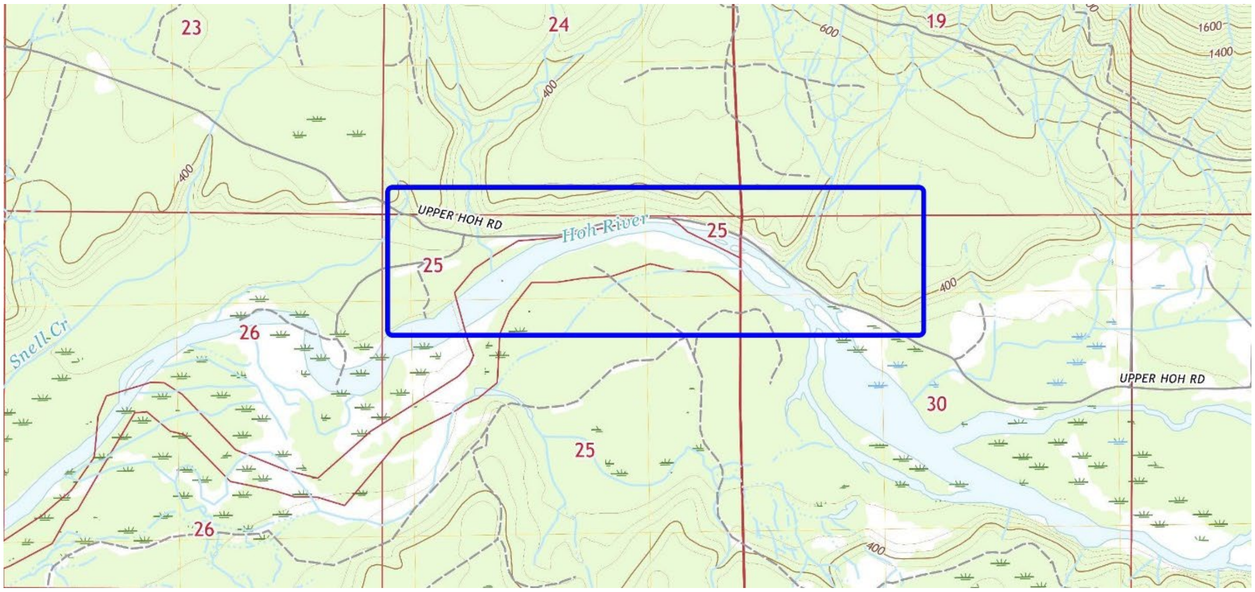

Figure 70. Topographic map of Upper Hoh River stabilization site (USGS 2021).

Figure 71. Aerial photo of Hoh River Stabilization area (Google 2021).

The Hoh River is braided with dramatically shifting active flow channels (see Figure 72). Bank erosion is observed at all bank areas not protected by riprap revetments, heavy vegetation, or boulder lag deposits (see Figure 73). The bank erosion is caused by mid-channel sediment deposits and woody debris shifting across the braid plain and redirecting flood flows at unstable bank areas. Erosion is severest where flow is directed at sharp angles against an erodible bank. Large woody debris appears to play a significant role in deflecting and redirecting flood flows. Cobbles and small boulders naturally armor the toe and large trees growing in the stream bank inhibit the bank erosion.

Figure 72. Hoh River braiding and debris (NPS 2021).

Figure 73. Embankment and roadway erosion along banks of Hoh River (NPS 2021).

The site parallels the outside bank of a river bend (recall Figure 70 and Figure 71). Approximately 3,900 linear feet of riprap revetment along the apex of the river bend appears to be effectively controlling road embankment erosion. The 2 to 4-foot diameter riprap comprising the revetment is properly graded and placed. Revetments are in two segments. The upstream segment is approximately 1,350 feet long. The downstream segment is approximately 1,150 feet long. Both segments are densely planted with willow and alder and appear stable. Riprap revetment segments nearly devoid of alder and willows, with 1.5(h):1(v) or steeper finished surface slopes appear less stable. At these steeper sections, riprap has been dislodged from toe and mid slope areas. The damaged revetment segments generally appear at maximum point of stream bank curvature and likely experiences high shear stress when floods occur.

The Case Study observed toe erosion and undermining of the stream bank between the existing revetment segments and immediately downstream of the downstream revetment segment. The channel edge is approximately 10 to 20 feet away and 10 to 18 feet below the road pavement edge. Mid-channel sediment deposits and large woody debris jams entrapped next to the banks, deflect stream flow towards the stream banks, exacerbating the erosion. Continued stream bank erosion could undermine the road (see Figure 74).

Upstream the active channel width is 400 to 1,200 feet. Downstream width is 400 to 1,600 feet. At the site the width is 250 to 400 feet. Based on historical satellite imagery, the active channel has not changed significantly in width and location from 1994 (Figure 75) to 2013 (Figure 76).

Sand, gravel, and small boulders comprise the stream bed material. Gradation analysis indicates the bed material ranges from sands to 10-inch cobbles with a D_{50} of 3 inches.

Figure 74. Looking downstream from the Hoh River stream bank (north to right).

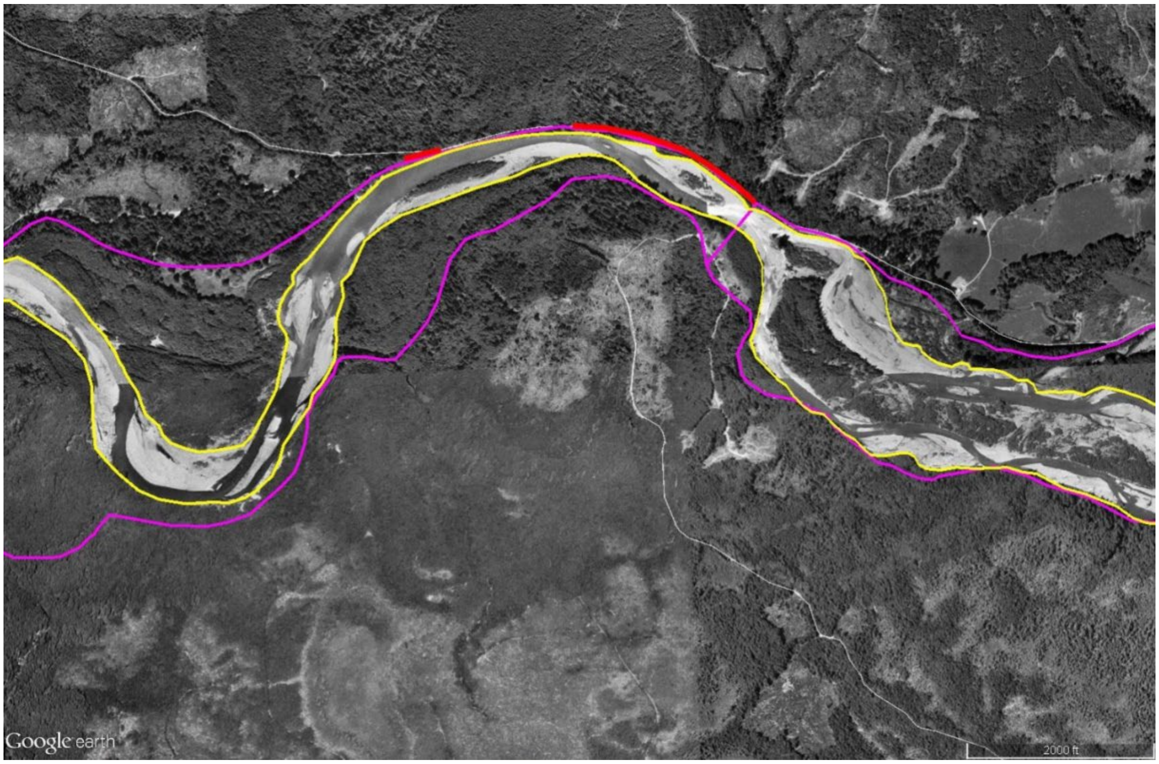

Figure 75. Hoh River historic satellite imagery from 1994 (Google 2021).

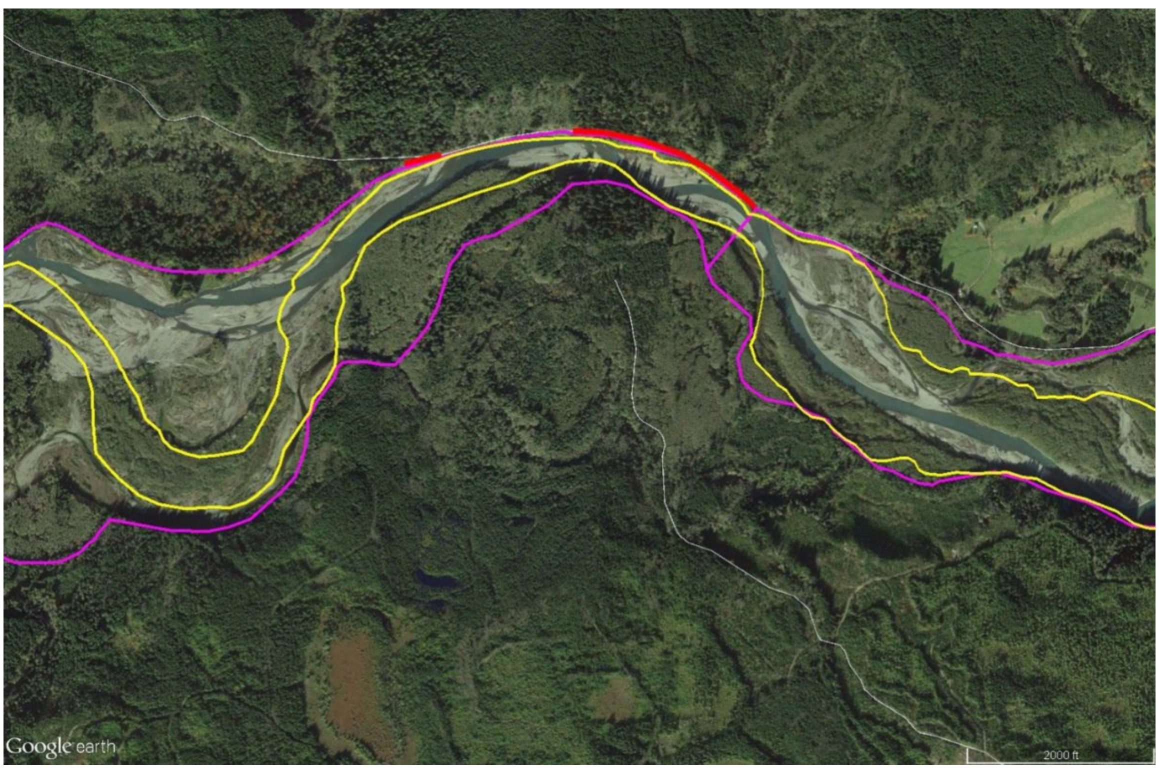

Figure 76. Hoh River historic satellite imagery from 2013 (Google 2021).

12.1.1 Design Criteria

The bank stabilization approach involves placing a wood buffer in a series of engineered-log-jams (ELJs) along the unstable, eroding banks. The ELJs deflect river flow away from the bank area, reducing the risk of scour and channel incision undermining the bank. The ELJs also reduce flow velocities and shear stress along the bank area upstream and between each ELJ. This promotes sediment deposition and retention along the bank toe. In turn, this encourages riparian vegetation establishment. The large woody debris, deposition between the ELJs, and scour along the ELJ streamside face creates channel complexity.

The spacing of the ELJs are approximately 30-foot intervals. Each is 75 feet long, 20 feet wide, and aligned along the bank toe. The site has 25 proposed ELJs (Figure 77).

To accommodate different channel conditions than currently mapped and future channel migration, ELJ elevations are not set relative to actual streambed elevations at time of construction. ELJ elevations are set relative to the modeled 50-year flood design water surface elevation. Scour will induce some settlement of the ELJ. The ELJ top is set approximately 3 feet above the 50-year flood design water surface elevation for accommodating expected settlement.

Each ELJ should be anchored for resisting floating away and being pushed downstream by flood flow. The anchor system should consider additional forces imposed by woody debris carried by the river entangling on the ELJ. The ELJ should be flexible enough to allow settlement when undermined by scour. To be easy to construct and be successful in controlling bank erosion, each ELJ is constructed of a repeatable sequence of log bundles and logs with root wads. Anchoring is provided by chaining the log bundles to a precast concrete dolos ballast. Based on expected scour, flood flow velocities, and depths, chaining facilitates achievement of long-term ELJ stability. To increase log bundle stability, the process locates the dolos towards the middle of the bundle length. Orientation is critical for deflecting flow away from bank toe and achieving log jam stability. The log bundles and logs with root wads should be placed in a random manner above the bottom layer. The approach carefully packs bundles as densely as possible and to place key members along the bank line for effectively controlling bank erosion.

12.2 Baseline Analyses

The baseline analyses investigated the site hydrology, hydraulics, stability and functionality of the Hoh River banks stabilization site.

12.2.1 Hydrologic Analysis

The Case Study used the USGS StreamStats on-line tool to determine the drainage area, including Willoughby Creek, equaled 223 square miles. Figure 78 depicts the location of the analysis, including the delineated contributing drainage area.

Approximately 70% of the watershed has heavy timber and 20% has exposed bedrock. The watershed has four small glaciers, White, Blue, Hoh, and Hubert, in the higher elevations; occupying approximately 7 square miles (3 percent) of the drainage area. Only small lakes are present. The USGS StreamStats on-line tool reports 168 inches of mean annual precipitation. The watershed lies mostly within the Olympic National Park and Olympic National Forest. Development is sparse, primarily light rural residential. No diversions for irrigation occur upstream.

Maritime weather dominates. Storms and moderate to heavy precipitation occurs year-round. Storms are more frequent, and precipitation is heavier September through January. September through November have the heaviest recorded rainfall. Snow occurs frequently during winter months, but melts after a few days. Lowest flows occur in February, March, April, July, and August.

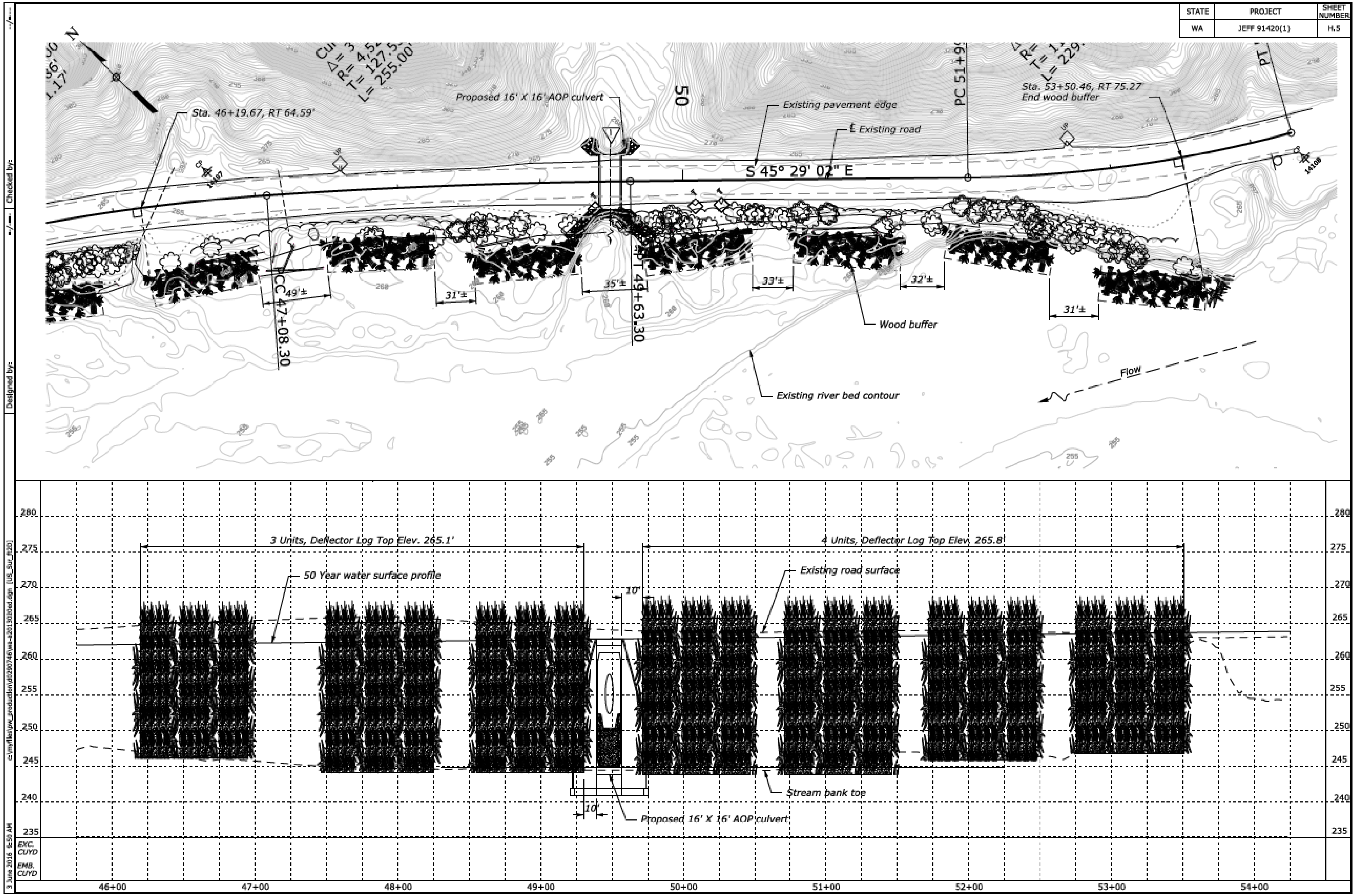

Figure 77. Hoh River Bank Stabilization – Plan sheet for site.

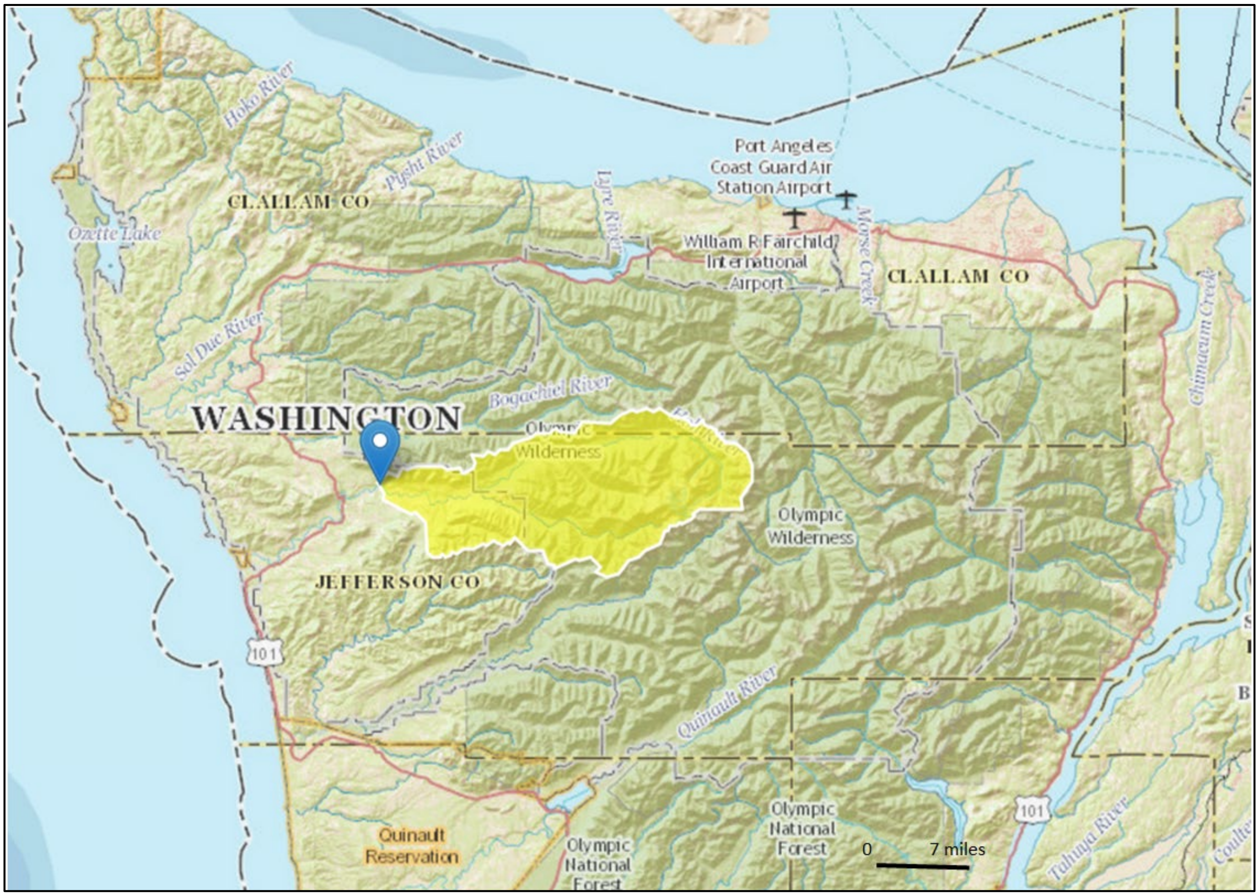

Figure 78. Delineation of the study drainage area (USGS 2021).

Winter season snowfall ranges from 10 to 30 inches in the lower elevations and between 250 to 500 inches in the higher mountains. In the lower elevations, snow melts rather quickly and depths seldom exceed 6 to 15 inches. In midwinter, the snowline is between 1,500 and 3,000 feet above sea level. The higher ridges are covered with snow from November until June.

12.2.2 USGS gage station

The USGS maintains a stream gage station (12041200) on Hoh River, near the State Highway 101 Bridge. The location of the station is river mile 15.4 and has a drainage area of 253 square miles. The gage has 59 years of record, beginning in 1961. Figure 79 depicts annual peak stream flow for the gage station.

The gaging station has not experienced floods greater than the 50-year event. The largest floods of record were 62,100 cfs occurring on USGS water year 2004[18] and 60,700 cfs in water year 2007. Both were approximately equal to the 25-year flood event for that gaging station.

The analysis estimated peak flood discharges with the weighting equation in USGS WRIR 97-4277 for ungaged sites on gaged streams as follows: (1) applying USGS StreamStats regression equations to estimate peak discharges and (2) improving the regression estimates with the

[18] Specifically, October 17, 2003. USGS “water years” extend from October to September each year.

weighted equation for the USGS 12041200 gage station.[19] Table 50 provides peak discharge estimates used for the baseline analyses.

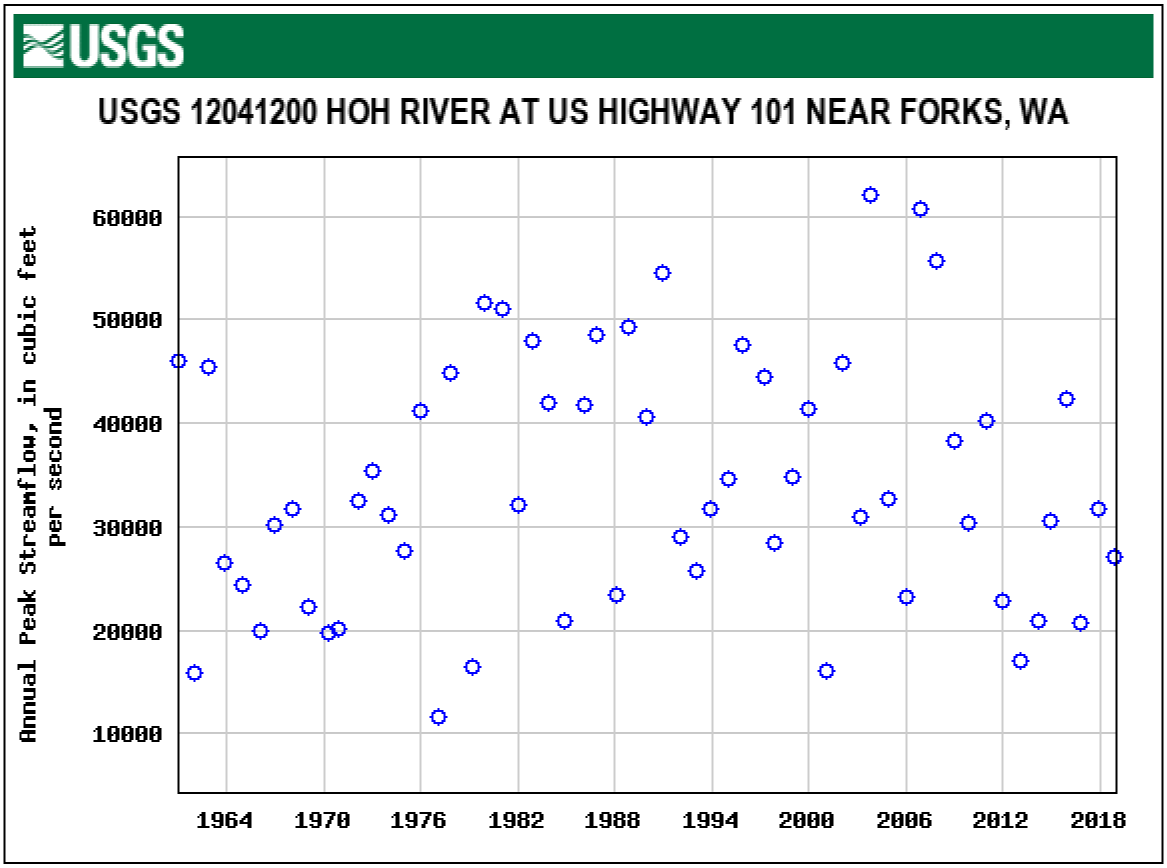

Figure 79. Annual peak stream flow for USGS gage station 12041200 (USGS 2021).

Table 50. Estimated peak flows for Hoh River at Study site (units in cfs).

2-year	10-year	25-year	50-year	100-year
28,492	45,409	53,066	58,497	63,394

12.2.3 Hydraulic Analyses

The Case Study estimated water surface elevations and flow velocities using the USACE HEC-RAS model able to perform two-dimensional (2D) unsteady steady flow calculations.

To represent worst case flow conditions, the analyses aligned the active flow channel along the revetment toe. The analyses placed each wood buffer in the models at design location and elevation.

12.2.3.1 Mesh Generation & Model Inputs

Such 2D models use a grid or mesh to represent the topographic and bathymetric features (e.g., terrain data). The Case Study obtained LIDAR terrain data from Puget Sound LIDAR Consortium. To supplement these data, the WFLHD surveyed topography and cross sections of the river

[19] See Equation 5 and Table 2 of the USGS WRIR 97-4277 report (USGS 1998).

channel at the bank stabilization site. Next, the analyses developed terrain data for the existing condition models by merging the LIDAR terrain data with the surveyed river cross sections and ground topography data.

The analysis generated meshes with 10 feet by 10 feet grid spacing encompassing the flow areas for each model. Floodplains and areas with higher flow roughness were delineated on the meshes from aerial imagery. Floods occurring 2004 and 2006 approximately equaled the 25-year event. The analyses validated existing condition models for sites by adjusting the Manning's Roughness Coefficients until the 25-year flood flow water surfaces approximately equaled observed high-water marks and debris limits. The analyses selected a Manning's Roughness Coefficient of 0.045 for the main channel 2D flow areas and Manning's Roughness Coefficient of 0.09 for the floodplain areas. Normal flow depth with 0.01 feet per feet friction slope was set for the downstream boundary condition. The analyses used an 8-hour duration, 1-minute interval hydrograph for the upstream boundary condition. The model uses a 4 second computation interval.

12.2.3.2 Hydraulic Baseline Results

The baseline hydraulics model results determined the 50-year water surface elevation equaled 253.7 feet. The roadway elevation at this location is 259.8 feet. The top of the wood buffer is located three feet above the 50-year storm water surface elevation (i.e., 256.7 feet) and considered the freeboard or clearance.

12.2.4 Scour Evaluation

Total scour for a wood buffer design option is a combination of contraction scour and bend scour. Long term degradation is not expected to occur. The analyses estimated contraction scour using HEC-18 *Evaluating Scour at Bridges* (FHWA 2012). The analyses estimated bend scour using the NRCS's *National Engineering Handbook, Technical Supplement 14B* (USDA 2007). The 2D hydraulic modeling produced the water depths and flow velocities for the scour analysis. As this was not a bridge or culvert, the 50-year flow was the scour evaluation discharge. The scour analyses results reveal that the baseline scenario was clear-water conditions, with zero (0) feet of contraction scour and 8.7-feet of bend scour. This means the total scour depth equaled 8.7 feet. This study does not provide the scour elevations to the channel bed as they varied along the length of the buffer system.

12.2.5 Stability Analyses

The Case Study completed a wood buoyancy and sliding analysis[20] for the ELJ's. The analysis assumes single log-dolose bundles. The 2D modeling produced the water depths and flow velocities for the design. The analysis uses an average 50-year flood flow velocity (e.g. 12 ft/sec) along the ELJ sides. Satellite imagery allowed estimation of an active channel width of 330 feet and radius of 400 feet. The Case Study selected a vertical velocity correction factor of 1.3 for representing high flow impingement angles and flow contracted or deflected around debris and mid-channel sediment deposits. The analysis investigated 18, 24, and 36-inch average log diameters. The analysis assumed the unit weight of concrete to be 150 lbs/ft^3. Each dolose weighs 8 tons. The analysis assumed the fluid drag coefficient to be 1.2 and friction angle to be 70 degrees. The design assumes the log mass will settle into scour holes as scour occurs. ELJ heights were set to accommodate the design water depth plus displacement from scour.

Based on HEC-RAS 2D modeling, the ELJ's "push" the high flow velocity line away from the bank, maintaining low velocity along the bank and between the ELJ's. Flow velocity increases along the

[20] As these approaches are relatively new within the US, the study used information from the Australian Government's *Design Guidelines for Reintroducing Wood in Australian Streams* (Brooks et al. 2006).

base of the ELJ's. Flow velocities do not appear to increase above background level for bank areas downstream of the ELJ's. The analysis reveals creation of refugia habitat and channel complexity along the entire length of ELJ.

12.3 Sensitivity Analyses

Given the baseline conditions for hydrology, hydraulics, scour, and stability, the sensitivity analyses allow consideration of effects of increased discharges on design events of interest.

12.3.1 Hydrologic Forcings

The analyses increased baseline peak design discharges by 10%, 20%, and 50% for 50 and 100-year events (see Table 51).

Table 51. Hoh River Bank Stabilization – Design discharge increases (cfs).

Storm Event	Baseline Q	Q + 10%	Q + 20%	Q + 50%
50-year	58,497	64,346	77,216	115,824
100-year	63,394	69,733	83,680	125,520

12.3.2 Hydraulic Sensitivities

The hydraulic analyses used the same 2D HEC-RAS model, substituting the baseline hydrology (e.g., peaks and hydrographs) with the comparable values of the three flow scenarios. Table 52 provides design parameters and the results of the analysis for the 50-year flow events. The table also presents the percent change from the baseline value.[21] The sensitivity seems to indicate that the changes to flow have the greatest impact with freeboard; with the $Q_{50\%,50}$ having nearly a 57% decrease from the baseline.

Table 52. Hydraulic sensitivity results for Hoh River Bank Stabilization.

	Baseline Q		Q + 10%		Q + 20%		Q + 50%	
Hydraulic Parameters	feet	%	feet	%	feet	%	feet	%
Water Surface Elevation	253.7	0	253.8	0.04	254.2	0.20	255.4	0.67
Water Surface Comparison	0.0	n/a	0.1	n/a	0.5	n/a	1.7	n/a
Clearance/ Freeboard	3.0	0	2.9	3.3	2.5	16.7	1.3	56.7

12.3.3 Scour Sensitivities

The results indicate that the system will exhibit clearwater scour for the baseline, $Q_{10\%,50}$, and $Q_{20\%,50}$ scenarios, changing to live bed scour at the $Q_{50\%,50}$ scenario. Table 53 provides results of the scour analyses for contraction, bend, and total scour depths. The table does not provide scour elevations, as they vary along the length of the buffer system. Where possible, Table 53 also provides the percentage change from the baseline value.

[21] Note: the percentage of the comparison value is left blank as the WSEL reflects the increase.

Table 53. Scour sensitivity results for Hoh River Bank Stabilization.

Scour Variables	Baseline Q		Q + 10%		Q + 20%		Q + 50%	
	feet	%	feet	%	Feet	%	feet	%
Contraction Scour	0.0	n/a	0.0	n/a	0.0	n/a	2.0	n/a
Bend Scour	8.6	0	9.2	7.0	10.2	18.6	12.6	46.5
Total Scour Depth	8.6	0	9.2	7.0	10.2	18.6	14.6	69.8

Of note is that the scour depths appear less sensitive to increases to the $Q_{10\%,100}$, and $Q_{20\%,100}$. However, the scour sensitivity increases with the $Q_{50\%,100}$ scenario; and when considering the live-bed, contraction and bend (i.e., total scour), demonstrates notable sensitivity (almost 70% for an increase of 50% of flow) to that $Q_{50\%,100}$ scenario.

12.3.4 Structure Stability & Functionality

Table 54 provides the various structure stability and functionality values for the various scenarios. In this case, the forcing hydrology was the 50-year event. The analyses used these 50-year flood derived flow velocities and water surface elevations for designing the bank stabilization features (i.e., force resistance drag, drag, factor of safety for sliding) and thus, evaluating potential effect on stream processes.

Table 54. Stability parameters for Hoh River Bank Stabilization.

Stability Parameters	Baseline Q	Q + 10%	Q + 20%	Q + 50%
Design Velocity (ft/s)	10.0	10.1	10.3	11.0
Force Resistance Drag, Ffs (lbs)	13,402	13,402	13,402	13,402
Drag, Fd (lbs)	23,545	24,019	24,979	28,490
Factor of Safety for Sliding (Ffs/Fd)	0.6	0.6	0.5	0.5

The sensitivities did not appear to significantly vary with the increases in flow. For example, there was a 10% change between baseline and $Q_{50\%,50}$ velocities; drag 21% and a 16.7% reduction in the sliding factor of safety. This appears to support a hypothesis that the design has resilience for increased flows.

12.4 Summary

The Case Study examined the Hoh River Bank Stabilization design for hydraulic clearance, scour and stability for the 50-year baseline design storm and increased baseline flows by 10%, 20%, and 50%. Neither the designed wood buffer, nor the roadway, was overtopped for any increase in streamflow, meaning the structure is adequate for capacity.

For scour, at higher flows the bed stability condition changes from clear water to live bed. Contraction scour, not occurring at baseline or up to $Q_{20\%,50}$, but does occur when flows are at 50% above baseline. Bend scour increases by about 50% at $Q_{50\%,50}$. Overall, the total scour depth is six feet greater at $Q_{50\%,50}$. If climate studies expect flows to increase by 50% or greater, consider increasing the structure height to accommodate expected scour and associated settling of the wood buffer.

For baseline and increased flow scenarios, the stability of the wood buffer does not change appreciably. Velocity does not change by more than one foot per second, even at $Q_{50\%,50}$. Factor of safety for sliding goes down by only 0.1, about 17% of baseline factor of safety, at higher flows. The structure will likely retain the same structural stability as design flows at higher flows.

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Part 3. Findings & Supporting Information

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Chapter 13. Findings

The research performed sensitivity analyses with assumed increases (or decreases) in design flows of 10%, 20%, or 50%. For each Case Study, analysis determined design flow and then modeled the hydraulics with each of these potential flow increases. Following this, the research performed a brief analysis to determine which of the flow increases might show sensitivity perhaps applicable to future climate change scenarios.

13.1.1 Discussions on Case Studies

Although the Case Studies mostly followed the same process, each scenario is different, with different project concerns and different results from the sensitivity analyses. Some of the projects including the Curb and Gutter on General's Highway, the West Twin Creek Bridge, the Snider Creek Culvert, and the East Twin Creek AOP Culvert appeared extremely resilient in regards to flow and headwater criteria and were still within their safety limits even at the "worst-case" scenario flow increases of 50%. These project sites would likely be largely unaffected by future climate change scenarios.

Other Case Study project sites did show some sensitivity to increasing flow. These projects include parts of the storm drain system in the Gardiner Gateway Stormwater Management Project, the Culvert at the Wye, and increases in scour depth at the Hoh River Bank Stabilization and Cedar Grove Village Bridge.

For the East Twin Creek Case Study, where aquatic organism passage needed consideration, the research also examined the culvert in an alternative way by assuming decreases in flow. The thought process was to find out if climate change might lead to lower regular recurring events such as the 2-year flow. If so, how sensitive were these twin culverts? The East Twin Culverts could still provide adequate passage at 10% and 20% decreases in flow but failed at 50% decreases in flow.

Most of these Case Studies also examined which sensitivity scenario was most likely to represent potential climate change inflections (or where indications of climate change might begin to become apparent). One interesting technique included analyzing wildfire scenarios for land use changes leading to increased flows. These assessments provided a rough estimate of what the most appropriate scenario could be. For the most part, the assumed future climate change scenario was the 20% increase to flow. This insinuates that all structures analyzed would remain resilient for the probable variations resulting from climate change.

Recall that the research conducted the original Case Study analyses during the period of 2015 to 2016. It is worth noting, climate change is a very complex field with new and improved data constantly emerging. These likely sensitivities may change as more data and improved processes becomes available. However, that is part of the benefit of performing a sensitivity test with a wider range in possibilities. It allows for easier adaptation in the future moving forward if certain assumptions made are to change.

13.1.2 Some Caveats

The research recognizes several elements that might differentiate the performance of these Case Studies from other locations, even those in National Parks elsewhere in the United States.

13.1.2.1 Rural Area Caveats

An important point for these Case Study projects is that they are in rural areas. The investigation intentionally selected these sites as it allowed analyses to disregard land use change. However, it also leads to an unintended consequence – most of the floodplains and flow within these Case Studies sites are in areas that are uninfringed by heavy development. This allows for two observations.

First, flows may be slightly lower than more highly developed areas with more impervious land. The resulting flows are probably smaller than those that would be encountered in metropolitan areas of similar sized watersheds.

The second important factor related to more metropolitan projects is floodplain encroachment. Historically, infrastructure has been built close to rivers, sometimes within their natural floodplains. Levees and other techniques help to protect infrastructure but can lead to further confining floodplains. When the flows from the river have less room to spread out, the river can more quickly rise in water surface elevation (WSEL) within a confined channel. This has a possibility to result in more risk of bridge or culvert inundation, greater velocities within the river, and greater risks for scouring. These National Parks have minimal infrastructure built within the floodplains of rivers allowing for more access to the floodplains. The greater floodplain access reduces the rate at which the water rises and increases in velocity.

13.1.2.2 Mountainous Areas Caveats

Some final key observations are that most of these Case Study projects are in more mountainous regions. The associated greater slopes allow flows to move more quickly through the river system, which can lead to increased flows having lesser impacts on WSEL. Flows in flatter areas move slower and build upon themselves which can lead to increased WSELs. Constraints with the floodplain can also lead to building bridges that more readily span the floodplain. In flat areas, the larger horizontal extent of a floodplain results in more expense and complexity when building a bridge that spans that floodplain.

The purpose of this discussion is to say, if this study were to be performed in areas with flatter terrain, more unnatural floodplain encroachments, and more impervious land area, the results would probably differ. In some cases, even a 10% increase in flow would lead to unacceptable risks to the infrastructure. But as stated in the Introduction, it all depends on the context and characteristics of the site and project.

13.1.3 Summary

As described throughout, the evolution of climate science data, practices and tools continues to evolve and improve. For example, the FHWA provides a tool to allow practitioners to access the World Climate Research Programme's (WCRP) Coupled Model Intercomparison Project (CMIP) phase 5 products (WCRP 2021, FHWA 2021). A concern could be that information from 2015 to 2016 no longer represents the consensus on such approaches described in more recent literature. A response is that such concerns focus on the *causes* that produce values, rather than the *effects* of those values within actual projects. Sensitivity tests are excellent tools to help gain insights on risks to infrastructure projects. Change occurs and preparedness helps better understand potential vulnerabilities and resilience from events.

These Case Studies will help with inspiration and direction in future examination of infrastructure projects. Designers and planners should feel free to take the aspects of these Case Studies that they believe worked best and apply them to their own projects.

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Appendix A - Units

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised March 2003)

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