Project Report: Criteria for Predicting Scour of Erodible Rock in West Virginia

RP-273

Revised Version Submitted: September 25, 2013

Criteria for Predicting Scour of Erodible Rock in West Virginia

WVDOT / DOH Research Project Number 273

by

Administrative Principal Investigator:
Wael Zatar (Ph.D.), Marshall University, Huntington, WV

Principal Investigator - Hydraulics:
Dr. Isaac Wait (Ph.D., P.E.), Marshall University, Huntington, WV

Co-Principal Investigator – Geotechnical / Geology:
William Niemann (Ph.D., P.G.), Marshall University, Huntington, WV

Subcontractor:
Terry Downs (P.G.), Gannett Fleming, Inc., Pittsburgh, PA
Brian Greene (Ph.D., P.G.), Gannett Fleming, Inc., Pittsburgh, PA

Prepared for the West Virginia Department of Transportation, Division of Highways, in cooperation with the U.S. Department of Transportation, Federal Highway Administration.

Technical Report Documentation Page

1. Report No. RP-273	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Criteria for Predicting Scour of Erodible Rock in West Virginia		5. Report Date September 25, 2013 6. Performing Organization Code
7. Author(s) Dr. Wael Zatar, Dr. Isaac Wait, Dr. William Niemann		8. Performing Organization Report No.
9. Performing Organization Name and Address Nick J. Rahall II, Appalachian Transportation Institute at Marshall University, PO Box 5425, Huntington, WV, 25703		10. Work Unit No. (TRAIS) 11. Contract or Grant No. MURC: 211222
12. Sponsoring Agency Name and Address West Virginia Department of Transportation, Research and Special Studies		13. Type of Report and Period Covered Final. March 31, 2011 – September 30, 2013 14. Sponsoring Agency Code

15. Supplementary Notes

16. Abstract

The research project "Criteria for Predicting Scour of Erodible Rock in West Virginia (RP-273)" was conducted to characterize the hydraulic scour of rock at 15 selected bridge sites in West Virginia (at least one site in each of WVDOH's ten districts). The study assessed the applicability of a recently-developed rock scour prediction technique to the types of rock and scour conditions found in West Virginia, and identify techniques that can be used to better characterize scour potential at existing and proposed bridge locations. Foundation inspection, rock coring and sample collection, and measurement of scour depth at all sites led to a determination of each site's mode of scour and enabled modified slake durability testing to develop a median Geotechnical Scour Number (GSN) for rock encountered at each bridge site. Flow conditions over time were assessed for each site using stream gage data where such data was available (three sites), and watershed models, coupled with probabilistic characterizations of precipitation, where stream gage data was not available (12 sites). Scour depth observed at each site, coupled with estimates of Cumulative Excess Stream Power, enabled the calculation of a Scour Number at each site.

The scour prediction method applied for this project is designed specifically for sites characterized by abrasion of degradable rock. Ideally, this mode of scour can be confirmed before the method is applied, but in cases where the mode of scour is not clear from field data, or where more than one mode of scour is operating at the same site, this may not be possible or practical. For sites affected only by abrasion, Scour Number and GSN should be consistent, allowing GSN to be used as a predictive measure of scour potential. In this project, however, Scour Number and GSN deviated considerably. The Scour Number/GSN (which, if GSN is to be predictive should have been near 1.0) varied from 3.9 – 17,000. The quarrying mode of scour, where durable, fractured rock is plucked away in discrete scour events from its original location, was evident to varying degrees at 14 of the 15 project sites, and explains why the GSN-centered approach of predicting rock scour, which is contingent on the progressive wear (i.e., abrasion) of non-durable degradable rock, did not apply to the data collected. In those cases where the mode of scour was unclear from field evidence, the lack of agreement between Scour Number and GSN served as indirect confirmation that scour was significantly or wholly attributable to quarrying and not abrasion. Though methodologies have been proposed to attempt scour depth prediction for quarrying , such methods require a comprehensive characterization of the distribution of block shape and dimensions, joint angle, block density, block protrusion, and flow field conditions, and applying such an approach was beyond the scope of this project. Other likely contributors to differences between Scour Number and GSN include uncertainties inherent in probabilistic characterizations of flow, and variations in the flow field along abutments (in contrast to the relative simplicity of the flow field at a pier).

Results lead to the conclusion that GSN may not be a universally-appropriate tool for predicting rock scour across the range of rock types and flow conditions found in West Virginia, particularly in view of widespread quarrying of durable rock at the project sites. A Scour Mode Decision Tree was developed to aid in identifying scour mode at future sites where scour assessments are performed, and a method was developed for generating a probabilistic estimation of Average Annual Cumulative Effective Stream Power.

17. Key Words Rock scour; stream power; bridge stability.		18. Distribution Statement	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 335	22. Price

DOT Form F 1700.7 Reproduction of completed page authorized

Acknowledgements

The authors of this report gratefully acknowledge the technical assistance, support, and insight provided by the project Technical Advisory Committee at the West Virginia Department of Transportation, including Douglas Kirk, P.E.,C.F.M., George Chappell, M.S., and Joe Carte, P.E.. The project coordination and monitoring provided by Donald Williams, P.E. and Michael Pumphrey, P.E. is also gratefully acknowledged.

The authors also express appreciation for the valuable project contributions provided by Jeffrey Keaton, Ph.D., P.E., P.G. of AMEC, Kimberley Baker of the Rahall Transportation Institute, and Nicholas Koutsunis.

Table of Contents

Additional Report Components

List of Figures

List of Tables

Executive Summary

Scour of rock that underlies bridge foundations has the potential to reduce their service life or threaten their viability. Scour behavior varies widely from site to site within West Virginia, and in response to uncertainty about how resistant to scour the rock at a site will be, the present design approach is generally one of conservatism and overdesign. At sites where rock does not scour, this conservative design process is inefficient, and may lead to foundations that are deeper – and more costly – than are necessary. Likewise, uncertainty about rock resistance to scour means the remaining service life of existing bridges cannot be reliably predicted.

In November 2011 the West Virginia Department of Transportation's Office of Research and Special Studies issued a request for proposals (RFP) entitled "Criteria for Predicting Scour of Erodible Rock in West Virginia", outlining a research project to characterize scour behavior at 15 bridge sites across WVDOT's 10 districts and evaluate the applicability of a newly-proposed scour prediction technique. The RFP specified that the research approach would include implementation of the methodologies developed in the National Cooperative Highway Research Program (NCHRP) Project 24-29, "Scour at Bridge Foundations on Rock". NCHRP Project 24-29 was conducted at five sites across the US, and in July of 2012 was summarized in NCHRP Report 717, which presents a methodology for predicting scour at bridge piers that incorporates several key elements, including: 1) characterization of actual scour depth at field sites; 2) quantification of stream energy exerted over time at the field site (i.e., Cumulative Effective Stream Power); 3) development of a Scour Number that equates a rock's actual scour depth and Cumulative Effective Stream Power; and 4) and determination of a Geotechnical Scour Number (GSN) that quantifies a rock's abrasion as a function of applied energy in a lab environment (i.e., modified slake durability testing). By characterizing a bridge site by GSN and stream energy (i.e., Average Annual Cumulative Stream Power), it would be theoretically possible to predict future Average Annual Scour Depth, such that the service life of a bridge can be estimated. This method is specifically designed to assess scour of degradable (i.e., non-durable) rock.

In cases where a rock's behavior in the field (i.e., Scour Number) is similar to its behavior in the lab (i.e., GSN), the predictive power implicit in the NCHRP Project 24-29 methodology arises from the idea that once a degradable rock's resistance to scour is defined, it then becomes possible to compute an average annual scour depth utilizing probabilistic representations of how much energy is likely to be delivered by a stream at a bridge site over time. In a best-case scenario, the observationally-based Scour Number at a site would agree with the laboratory-based GSN. Such a case would support extrapolation of GSN data to predict future scour at a site before any scour was actually observed in the field. This approach cannot, as demonstrated by this research project, be reliably applied to durable rock where the scour mode is hydraulic plucking of fragmented rock. However, application of GSN-based prediction of scour at sites with degradable rock could liberate designers from their previous conservatism, and enable more efficient, economical, and defensible design. Thus, a key motivation for this research project was to determine the reliability and justifiability of scour prediction using GSN at the types of sites that are characteristic of the rock and hydraulic conditions in West Virginia.

In RP-273, hydraulic and hydrologic conditions at each of the 15 sites were assessed to develop each site's Cumulative Effective Stream Power for the period since a bridge was constructed. Field inspections were conducted to characterize the mode and extent of scour, Scour Number was defined at

each site, and core sampling and laboratory testing was performed to enable computation of GSN. As noted in the RFP, the majority of bridges in WV are single-span, meaning that an additional element of this project was adaptation of the pier-focused NCHRP Project 24-29 methodology to locations where scour was observed on abutments.

The results of this research indicate that scour at all but one of the investigated bridge sites is occurring partially or completely in the form of quarrying and plucking of durable, jointed rock. Therefore, the GSN, which is specifically designed to predict scour for abrasion and grain-scale plucking of degradable rock, is not an appropriate or useful predictor at these bridges. This assessment was made by evaluating mode of scour and also by comparing site Scour Numbers to GSNs for each site; for GSN to be a reliable indicator of Scour Number, the two should have generally been found to be in agreement. For the sites evaluated in this research, the ratio of Scour Number / GSN varied from 3.9 to more than 17,000, meaning that if implemented for prediction, GSN would consistently under-predict the actual scour at sites where the scour was likewise of a quarrying and plucking mode.

Chief among the likely reasons for the observed discrepancies between the Scour Number and GSN is scour mode. GSN is meant to assess a rock's response to the abrasion mode of scour. Determination of scour mode at a site is non-trivial, and in many cases there are indicators of both abrasion and quarrying. Pure abrasion was interpreted to be the mode of scour at only one project site (Coon Creek), and interpreted to be the predominant scour mode at only one other site (Caldwell Run). At other locations, scour appeared to be either dominated or influenced at least in part by the quarrying mode of scour, wherein fractured rock blocks are carried away (or plucked) from a foundation during high-velocity flood events. Rather than the slow, long-term, grain-by-grain removal of rock during abrasion, quarrying results in discrete losses of material, with the magnitude of scour determined by the geometry of blocks defined by jointing and bedding. This effect is evident in the elevated Scour Numbers that were observed. Since the modified slake durability test used to determine GSN does not account for the quarrying mode of scour, the response of fractured rock to hydraulic scour cannot be predicted with GSN. Lack of agreement between Scour Number and GSN serves as indirect confirmation that most all of the scour observed at the studied bridges is not attributable to abrasion.

Another challenge in applying the NCHRP Project 24-29 methodology to the investigated bridge sites was the relative lack of stream gage data. Though a common situation in small watersheds found in rural areas, to conclusively calculate the Cumulative Stream Power that has occurred at a site, flow conditions at the site should be known for the entire period since construction. However, stream gage data was available at only three of 15 sites (Audra, Beverly, and Clear Fork), meaning that a watershed model based probabilistic flow synthesis approach had to be developed and applied at the remaining sites. Uncertainty inherent in hydrologic modeling, and in the application of probabilistic methods, resulted in Cumulative Stream Power estimates that may differ significantly from conditions that were actually present in the field or might be experienced in the future, thus compromising the reliability of scour predictions.

Variation in flow conditions, and thus stream power, at abutments compared to conditions at piers is another complicating factor. Whereas the energy encountered at piers may be directly related to upstream flow conditions, variations in flow conditions along the length of an abutment likely arise due to abutment orientation relative to flow direction, flow eddies, turbulence-inducing vortices, and other variables. Such phenomena can be accounted for in sophisticated hydraulic models, but simplicity of implementation was a stated preference of WVDOH, and increased hydraulic model complexity decreases the accessibility of the overall scour prediction approach.

Rather than classifying rock scour characteristics on a continuous scale (e.g., the GSN based approach) in an attempt to predict a specific future scour depth, identification and management of scour prone bridge sites in West Virginia may be better accomplished through a tiered approach that classifies bridges by their suspected scour mode and by the relative stream power encountered at a site. A mode of scour determination flow chart enables categorization of a site into Tier I (no scour observed), Tier II (abrasion), or Tier III (significant quarrying probable). Likewise, a site's annual cumulative stream power can be classified into three groupings defined by energy level thresholds corresponding to low (Group A), medium (Group B), and high (Group C) stream power. This approach, which is outlined in the Best Management Practices (BMP) document that accompanies this project report, will enable WVDOH to identify bridge sites that are at an elevated risk of experiencing rock scour, and build a database that over time will allow for further refinement of the definitions and thresholds used to classify sites into their respective scour mode Tier and energy level Group.

In conclusion, the findings and outcomes of this research project include: 1) characterization of the rock types and scour modes found at the 15 project sites (which were selected in collaboration with WVDOH to be a representative sample of the inventory of bridges prone to scour); 2) the finding that scour likely occurs partially or completely by quarrying at the vast majority of these bridges, meaning that GSN is not an appropriate or reliable indicator of future scour for the rock types and potential scour locations (i.e., predominantly abutments) that are most common in West Virginia; 3) development of a Cumulative Stream Power calculation methodology that can be utilized to assess the energy state at a site in the absence of gage data, and can also be used in classifying sites according to annual stream power Groups; 4) development of a Scour Number approach that could be used to extrapolate observed scour at individual sites into a quantification of average annual scour, and potentially service life remaining; and 5) development of a scour mode diagnosis procedure that can be incorporated into bridge site scour mode Tiers.

Abstract

The research project "Criteria for Predicting Scour of Erodible Rock in West Virginia (RP-273)" was conducted to characterize the hydraulic scour of rock at 15 selected bridge sites in West Virginia (at least one site in each of WVDOH's ten districts). The study assessed the applicability of a recently-developed rock scour prediction technique to the types of rock and scour conditions found in West Virginia, and identify techniques that can be used to better characterize scour potential at existing and proposed bridge locations. Foundation inspection, rock coring and sample collection, and measurement of scour depth at all sites led to a determination of each site's mode of scour and enabled modified slake durability testing to develop a median Geotechnical Scour Number (GSN) for rock encountered at each bridge site. Flow conditions over time were assessed for each site using stream gage data where such data was available (three sites), and watershed models, coupled with probabilistic characterizations of precipitation, where stream gage data was not available (12 sites). Scour depth observed at each site, coupled with estimates of Cumulative Excess Stream Power, enabled the calculation of a Scour Number at each site.

The scour prediction method applied for this project is designed specifically for sites characterized by abrasion of degradable rock. Ideally, this mode of scour can be confirmed before the method is applied, but in cases where the mode of scour is not clear from field data, or where more than one mode of scour is operating at the same site, this may not be possible or practical. For sites affected only by abrasion, Scour Number and GSN should be consistent, allowing GSN to be used as a predictive measure of scour potential. In this project, however, Scour Number and GSN deviated considerably. The Scour Number/GSN (which, if GSN is to be predictive should have been near 1.0) varied from 3.9 – 17,000. The quarrying mode of scour, where durable, fractured rock is plucked away in discrete scour events from its original location, was evident to varying degrees at 14 of the 15 project sites, and explains why the GSN-centered approach of predicting rock scour, which is contingent on the progressive wear (i.e., abrasion) of non-durable degradable rock, did not apply to the data collected. In those cases where the mode of scour was unclear from field evidence, the lack of agreement between Scour Number and GSN served as indirect confirmation that scour was significantly or wholly attributable to quarrying and not abrasion. Though methodologies have been proposed to attempt scour depth prediction for quarrying , such methods require a comprehensive characterization of the distribution of block shape and dimensions, joint angle, block density, block protrusion, and flow field conditions, and applying such an approach was beyond the scope of this project. Other likely contributors to differences between Scour Number and GSN include uncertainties inherent in probabilistic characterizations of flow, and variations in the flow field along abutments (in contrast to the relative simplicity of the flow field at a pier).

Despite the conclusion that GSN may not be a universally-appropriate tool for predicting rock scour across the range of rock types and flow conditions found in West Virginia, the project's methods and findings have been incorporated into a classification system that will enable WVDOH to differentiate between sites by Scour Mode and Energy Level, yielding a matrix that allows for subsequent monitoring and, over time, relative predictions of potential scour performance by comparisons to sites of similar scour characteristics and hydraulic conditions.

Background

Project Tasks

The federal Highway Administration (FHWA) has encouraged state transportation agencies to improve existing bridge scour prediction methods (FWHA, 2001). Accordingly, in 2011 the West Virginia Department of Highways issued a Request for Proposal RP-273: Criteria for Predicting Scour of Erodible Rock in West Virginia. The research team consisting of members from Marshall University, Rahall Transportation Institute, and Gannett Fleming, Inc. partnered together in response to the Request for Proposal, and ultimately received approval to conduct the research summarized in this report.

Bridge foundations in West Virginia are founded in sedimentary rock, and stream scour of erodible foundation rocks creates the potential for undermining of the bridges. However, resistance of sedimentary rocks in West Virginia to scour, and the power of streams to cause scour, varies widely in the state. A study was proposed to apply a NCHRP Research Project developed methodology to conditions found in West Virginia in order to develop methods to predict the extent of bridge foundation scour based on characterization of rock parameters and hydraulic conditions.

The following were identified for the project:

Task 1: Literature Review, including a review of NCHRP Project 24-29, other state DOT projects, WVDOH-provided scour location data, and related technical reports.

Task 2: Study Site Selection; the identification of sites best meeting a set of selection criteria that included the presence of scour, the availability of stream gage data, presence of a single rock type, the availability of bridge surface elevation data, the availability of channel cross-section profiles over time, the availability of channel cross-sectional surveys, the absence of stream sediment covering bedrock surfaces, and accessibility by a drill rig to take core samples.

Task 3: Field and Lab Testing, including inspection of surface bedrock, core logging and rock sampling for laboratory testing at 15 bridge locations, preservation of core samples, modified slake durability testing, and unconfined compressive strength testing.

Task 4: Data Analysis, including derivation of depth of scour and equivalent stream power for samples tested by modified slake durability testing, derivation of Geotechnical Scour Number, and utilization of site-specific evidence to identify quarrying/plucking mode of scour. Hydraulic and hydrologic characterization of conditions at each site, including an assessment of gage station data, the development of hydrologic models, development of channel geometry for utilization in hydraulic models, preparation of hydraulic models of channels, and finally the computation of Stream Power and Cumulative Effective Stream Power.

Task 5: Reporting, including periodic informal contact with the Technical Advisory Committee (TAC), quarterly progress reports, a formalized annual progress meeting, a final report, and a separate Best Practices document to detail how to best utilize the results of the research, targeting design engineers, bridge evaluators, and engineering consultants. Reporting also includes a scheduled presentation to selected DOH personnel in Charleston WV upon completion to summarize findings and answer questions.

Supplemental: Core Drilling; during the project 'kick-off' meeting held in March 2011, the idea of utilizing Gannett Fleming, Inc. affiliate company Hetager Drilling for core drilling was discussed in the context of finding a way to mobilize drilling for the following summer. Ultimately a supplemental agreement was executed between WVDOH and RTI / Marshall University to enable Hetager to conduct core drilling for the 15 project sites.

Related Projects and Literature Review

The major information sources for this project were the National Cooperative Highway Research Program Project 24-29 document and subsequent report, NCHRP 717 (2012): "Scour at Bridge Foundations on Rock." Per WVDOH RP 273: "Criteria for Predicting Erodible Rock in West Virginia," NCHRP 24-29 and NCHRP 717 (2012) dictated the research approach for the current project. Documents directly related to that project that were reviewed included several conference abstracts and presentations published prior to issuance of NCHRP (2012); these sources presented preliminary results from that project: e.g., Mishra et al. (2010) and Keaton et al. (2010a, 2010b).

As the definitive statement on rock scour to date, NCHHRP 717 (2012) included a thorough review of related literature; Appendix A of that report consists of 144 citations. Given the scope and currency of that report, the literature review conducted for the West Virginia study did not attempt to duplicate the review from NCHRP 717 (2012) but instead used that document as a basis for focusing on research with potential for direct application to the West Virginia project. For example, publications cited in NCHRP 717 (2012) related to the historical emergence of rock scour as an engineering issue and previous test methods that were evaluated during development of the NCHRP (2012) methodology generally were not included in the current literature review. An exception is Dickenson and Baillie (1999) which describes development of the continuous abrasion (i.e., slake durability) test for evaluation of sedimentary rock samples from bridge sites in the Oregon Coast Range. This laboratory test is a centerpiece of the NCHRP 717 (2012) methodology for determining the geotechnical scour number (GSN), which forms the basis for predicting abrasion scour at bridges founded on the same rock. The GSN* parameter, developed during the current project as a modification to the GSN, is related to the "abrasion number" of Dickenson and Baillie (1999). In addition, a number of the geologic, geotechnical, and hydraulic factors identified by Dickenson and Baillie (1999) as contributing to rock scour have been incorporated into the best practices document accompanying the current report.

The Federal Highways Administration's (FHWA) "Stream Stability at Highways Structures, Fourth Edition" (HEC-20) introduces methods of identifying and classifying the stream instability issues that often contribute to scour. Through stream classification and stability evaluation, the geomorphic assessment outlined in HEC-20 enables evaluators to assess whether streams are susceptible to scour, and if so, whether a detailed hydrologic and hydraulic assessment of scour should be pursued. Hydraulic

Engineering Circular No. 18 (HEC-18), “Evaluating Scour at Bridges, Fourth Edition” provides information related to scour resistant design, identifying vulnerability to scour, and means of estimating scour at bridges. HEC-18 presents a formula for determining stream power as a function of energy grade line slope, unit discharge, and water unit weight. Combined with the Erodibility Index Method that is also presented, HEC-18 suggests a procedure of relating resistance to scour with the energy exerted by a stream at a bridge site to quantify “the relative ability of non-uniform earth material to resist erosion.” This approach is analogous to that inherent to the Scour Number approach presented in NCHRP Report 717, namely relating an observed scour depth to stream power. Among the ways in which NCHRP Report 717 builds on the HEC-18 approach is to differentiate between stream power for a given discharge, and the *effective* (i.e., greater than some threshold below which scour may not be expected) stream power that accumulates at a site over time.

In cases where hydrologic and hydraulic analysis indicates that scour issues at bridges can be addressed through countermeasures, rather than bridge replacement, HEC-23, “Bridge Scour and Stream Instability Countermeasures” enables designers to evaluate a variety of countermeasures and develop bridge monitoring plans that can help mitigate the risk associated with scour.

The hydrologic and hydraulic methods described in NCHRP Report 717 make use of stream gage data that is reported on the basis of average daily flow. For large watersheds, and the correspondingly large streams and rivers they feed, the average daily flow is closer, on a relative basis, to the peak flow for a day than for small watersheds and streams. In other words, for a large watershed, the ratio of daily peak flow / average daily flow will be closer to 1 than for small watersheds. As shown in Table 2, the watersheds studied in this project are drastically smaller than the watersheds present in NCHRP Report 717. In view of this, and coupled with the fact that stream gage data was generally not available for the watersheds under study, alternative hydrologic and hydraulic methodologies had to be developed and implemented in this research project. The USDA’s Natural Resources Conservation Service National Engineering Handbook – Part 630 Hydrology (2007) was a primary basis for development of the hydrographs that were ultimately used to compute cumulative effective stream power. Estimation of time of concentration was guided by information presented in the USDA’s Soil Conservation Service Revised Technical Note Revised Hydrology No. N4 (2003).

Selection of Manning’s n values for utilization in hydraulic calculations performed in HEC-RAS was guided by values presented in the US Army Corps of Engineers’ HEC-RAS River Analysis System Hydraulic Reference Manual (2008). Identification of values to use for Curve Number assignment associated with land use and soil type combinations was guided by recommended values presented in the documentation that accompanies the USDA’s Technical Release 55 (1986), along with curve number calibration equations presented by Ponce and Hawkins (1996). Dr. Steven McCutcheon’s 2003 report to the West Virginia Division of Forestry entitled, “Hydrologic Evaluation of the Curve Number Method for Forest Management in West Virginia” was used for comparison purposes against watersheds modeled in this project, particularly with respect to lag time estimation.

In view of significant mining activities within West Virginia, and the potential for mine drainage to acidify streams, the effect of stream pH levels and water quality on rock scour may be of some concern. Baedecker and Reddy (1993) identify limestone and marble as two rock types that are particularly sensitive to acidic deposition, given that both are largely composed of calcite ($CaCO_3$). Citing the combined effects of chemical erosion and the mechanical loss of grains from stone surfaces, Baedecker and Reddy identify a pH of 4.2 as contributing to 10% of limestone weathering for the flow rate conditions present over a range of field experiment sites. In a separate study of limestone and marble sensitivity to acidic deposition due to acidic rain, Baedecker et al. (1992) measure an average in situ physical recession of 25 to 45 μm/yr for limestone in an acidic environment. None of the literature consulted with respect to chemical participation in rock scour indicated an enhanced erosion or scour effect in the basic pH range that was observed at sites in this project.

Other literature particularly relevant to the current project includes Hopkins and Beckham (1999), who developed a risk-based scoring system for rock scour potential based partly on rock quality designation (RQD) for bridges in Kentucky. Based on this system Froehlich et al. (1999) determined that the scour hazard was high for 8.5% of bridges founded on rock, moderate for 12.1%, and low for 79.4%.

Additional literature reviewed for the current project but not cited in NCHRP 717 (2012) includes Holnbeck and Parrett (1987), which provides a useful, if dated, summary of bridge scour as an engineering issue and describes the magnitude of the problem. For example, Holnbeck and Parrett (1987) estimated that 485,000, or about 84%, of bridges in the U.S. are over waterways, and that annual scour-related damage to bridges costs $30 million. Holnbeck and Parrett (1987) proposed a method for rapid estimation of scour; the paper, however, is focused entirely on scour of unconsolidated materials and does not even mention rock scour. Zhang et al. (2013), stating that scour of bridge foundations is the most common cause of bridge failure, attempted to improve on estimates of scour rate and amount. Their research involved study of seven bridges founded on sands, silts and clay; the results are not applicable to rock scour.

The scour identification and management principles employed by the State of Maryland's State Highway Administration (SHA) are described in the Office of Structures Manual on Hydrologic and Hydraulic Design (2011), Chapter 11 – Evaluating Scour at Bridges. This document, along with related documents published by other state agencies, were reviewed for purposes of defining the scour tier and stream energy classification approach that was adopted in lieu of scour prediction with GSN. The Commonwealth of Pennsylvania's Bridge Management System 2 (BMS2) Coding Manual – Publication 100A (2009) contains information related to scour threshold tier levels for categories such as potential for trapping debris, stream bed materials, channel alignment, as well as detailed procedures for characterizing observed scour depth. Scour Critical Category classification information, and a description of Pennsylvania's unique scour incident bridge inspection program, wherein local municipal employees and first responders are used to monitor scour critical bridges during storm events, is found in the undated, publically-accessible presentation entitled "Monitoring Scour Critical Bridges During Floods for Local Bridge Owners".

Several documents outline the procedures implemented by the State of Ohio Department of Transportation to mitigate the threat of scour to highway bridges. Ohio's Scour Plan of Action is outlined in a 2008 memo by Bridge Inspection Engineer Mike Brokaw, making reference to a program of (1) office assessment of bridge scour vulnerability, (2) field review of scour vulnerability and prioritization, and (3) detailed scour analysis and additional monitoring of bridges that are deemed very susceptible to scour. The Ohio DOT's Manual of Bridge Inspection (2010), Item 40 – Scour, includes definitions of how to measure scour depth, length of undermining, and a multi-tiered condition rating matrix that ranges from "Good" (i.e., Condition 1) to "Failed" (i.e., Condition 4). Ohio's Scour Critical Susceptibility coding is defined in the Office of Structural Engineering's Bridge Inventory Coding Guide (2012).

State Survey

A survey of states adjacent to West Virginia (i.e., Ohio, Kentucky, Virginia, North Carolina, Maryland, and Pennsylvania) was conducted in order to assess the methods utilized to identify and manage scour at state agency owned highway bridges. For each state, the primary engineer in charge of bridge inspection was identified, contacted by telephone, and interviewed to determine:

- To what extent rock scour at bridge piers and abutments is perceived to be a significant problem in their state.
- Whether their organization addresses rock scour separately from sediment scour.
- Whether formal guidelines are in place to identify / diagnose rock scour.
- Whether their state distinguishes between different modes of scour (e.g., abrasion vs. quarrying)
- Whether the methods described in NCHRP Report 717 are utilized to identify, manage, or address rock scour.

Among the findings of this survey is that only one state (Kentucky) has specifically distinguished between rock scour and scour of other materials at bridge sites. Other states maintain databases of which bridge sites have experienced problematic scour, and also have separate information about the foundation materials present at bridges, but these two traits have not been combined to perform analyses of what percentage of rock-founded bridges experience rock scour.

A range of scour identification and analysis techniques are employed among the states. In Maryland, for example, scour classification includes an analytical component, drawing on the Erodibility Index Method outlined in HEC-18. Though the Erodibility Index Method was not developed exclusively with rock scour in mind, and does not distinguish between different scour modes that may arise in view of different rock types, it is used in Maryland to identify sites that are potentially score prone, and compute predictions of scour depth. In Ohio, Pennsylvania, and North Carolina, identification of scour prone bridges depends primarily on periodic bridge inspections, rather than analytical techniques, and minimal or no efforts are made to predict future scour that may be encountered at existing bridges that have exhibited a potential for scour. Periodic inspections in Kentucky include detailed surveys of stream cross section, in order to monitor erosion over time, and enable tracking of the severity of scour as it occurs.

Rock Quality Designation (RQD) was cited as a primary means of identifying scour resistant material by Virginia and Kentucky, but in other states limited emphasis was cited for efforts to relate rock properties at bridge sites with propensity to scour.

A summary of survey responses is provided in Table 1.

Table 1 – Summary of State Survey Responses

Point-of-Contact	**Ohio**. Mike Brokaw, Bridge Inspection Engineer, Ohio DOT, 614-387-6210, michael.brokaw@dot.state.oh.us	**Pennsylvania**. James Long, Chief Bridge Insp. & Mgmt Section, PADOT, 717-783-7616, jamelong@pa.gov	**Kentucky**. David Steele, Bridge Maintenance/Preservation Branch Manager, Kentucky Transportation Cabinet, 502-564-4556, david.steele@ky.gov
1 - Is rock scour at bridge piers and abutments a significant problem in your state?	Don't know the % of bridges where it is a problem and where the foundation is rock.	Less than 1% of state-owned bridges (61 of appx. 15,500) are Scour Critical Class A. Including Class B (1573) and Class C (193), the total number of Scour Critical Bridges is 11%.	No. 400 rock-founded bridges were inspected, and 2% were found to have scour. Only 3 of 400 had scour depth > 10 in.
2 - Does your organization address rock scour separate from sediment scour?	Yes. Different criteria are developed for assigning Scour Condition	Yes. The "Procedures for Bridge Scour Assessment" document distinguishes between different foundation material types.	Yes. An RQD-based approach is used. Furthermore, a point-based Rock Scour Hazard Rating System was developed, including scour proximity, depth, penetration, and average annual daily traffic. However, this system is not actually utilized.
3 - If yes to #'s 1 and 2, are there formal guidelines for identifying/diagnosing rock scour?	Yes, see the "Scour Critical Susceptibility" checklist.	Yes - Three Scour Category Tiers (A,B,C) exist. Additional guidelines are found in "Procedures for Bridge Scour Assessment" pg 61	Yes. Cross section profiles are taken during periodic bridge inspections, and monitored for change.
4 - If yes to #2, does your state include distinguishing different modes of rock scour?	No.	No.	"Scour mode" is not distinguished, but rock type (erodible vs non-erodible) is assessed during bridge inspection.
5 - Is your state utilizing NCHRP 717, "Scour at Bridge Foundations on Rock" in managing or addressing rock scour?	No.	No.	No
Notes:	See Ohio Bridge Inventory Manual. Also, Item 74, Scour Critical Susceptibility	Locally-owned Scour Critical Bridges monitored during storm - see presentation. Also, Pub100A (Bridge Inspection Coding Manual)	See report KTC-99-57, Correlation of Rock Quality Designation and Rock Scour Around Bridge Piers and Abutments Founded on Rock

Table 1 cont. – Summary of State Survey Responses

Point-of-Contact	**Virginia**. John Matthews, PE Assistant State Hydraulic Engineer, VDOT, 804-786-4031, john.matthews@vdot.virginia.gov	**Maryland**. Glenn Vaughan, Deputy Director Office of Structures, 888-375-1084, gvaughan@sha.state.md.us	**North Carolina.** Henry Black, Asst. State Structures Engineer - Inspection, 919-707-6479, hblack@ncdot.gov
1 - Is rock scour at bridge piers and abutments a significant problem in your state?	No	Precise statistics not available	Will attempt to acess database for this information, and will make future contact if available.
2 - Does your organization address rock scour separate from sediment scour?	Yes. For existing bridges the determination is made by the bridge maintenance staff what the appropriate repair may be needed based upon the conditions encountered. In design, VDOT follows the prior FHWA Guidance that Rock Cores with a RQD >50% may be considered to be scour resistant material. No special consideration has been given to the scour resistance of lesser quality material.	Yes. The "Erodibility Index" method is used in computations to predict scour depth, and this method distinguishes between rock and other materials.	No.
3 - If yes to #'s 1 and 2, are there formal guidelines for identifying/diagnosing rock scour?	Not to date. VDOT is hosting the NHI 135046 HEC-18 Scour Class for Bridge, Geotechnical and Hydraulics Staff and may incorporate a more formal process in the future.	Yes. Three scour classification states are utilized: 1 scour exists, but little concern, 2 scour exists and left unchecked could adversely impact, 3 scour is significant and analysis of the bridge is needed.	No. The state follows an inspection-based, rather than numerical analysis based, approach to identifying scour.
4 - If yes to #2, does your state include distinguishing different modes of rock scour?	N/A	The Erodibility Index Method ends up incorporating different scour modes, but is not explicit in identifying them.	No. However, the state is sensitive to different scour modes (e.g., mountain stream vs hurricane-affected shoreline areas)
5 - Is your state utilizing NCHRP 717, "Scour at Bridge Foundations on Rock" in managing or addressing rock scour?	No	No.	No.
Notes:	-	Scour analysis is performed for all bridges where rock is involved. See HEC-18, 4.37	A "scour committee" was convened a few years ago to identify and address bridges where scour was identified as problematic or where the foundation type was unknown. As a result, all unknown data was gathered.

Research Approach

The methods applied in this research project were, as directed in WVDOH's original Request for Proposal and as outlined in the proposal for this project, largely based on the techniques developed in NCHRP Project 24-29. Included among these are core drilling and application of the modified slake durability test to define a sample's GSN, the use of a one-dimensional hydraulic model (e.g., HEC-RAS) to define flow velocity and depth at a location as a function of flow rate, characterization of flow conditions at a site over a period of years, calculation of Cumulative Effective Stream Power exerted by the flow onto rock surfaces, and incorporating observed scour depths into determinations of Scour Number.

A number of differences in watershed characteristics, scour locations, and data availability between the five project sites included in the NCHRP project and the 15 sites from this WVDOH project meant that some of the methods taken from NCHRP Project 24-29 procedures must be adapted for use in WVDOH RP-273. Among these differences are:

- **Watershed size.** The average watershed size for NCHRP sites is 1801 mi^2, compared to 40 mi^2 for the WVDOH project sites. This smaller watershed size is related to several of the other differences, such as the fact that the majority of bridges in WVDOH's inventory are single-span (i.e., cross relatively small streams with relatively steep banks). Small watershed size is also related to the fact that the scour location of interest was abutments, rather than piers, at most of the project sites. Likewise, since small watersheds are less likely to have their streams gaged than large watersheds, the relatively small size of the watersheds studied in this project correlated with the scarcity of stream gage data for sites under study.

  Smaller watersheds also yield peak flows that exceed average daily flows by a greater margin. For a large watershed, where storm events may take several days to fully drain, the peak flow is closer to the average daily flow than for a small watershed, which may have a relatively brief time to peak. This affect, as described later, made it important to consider flow rates on a basis other than a simple average of daily flow.

- **Stream gage data availability**. At the NCHRP project, a complete stream gage data set was available at four of five sites, and a partial stream gage data set was available at the fifth. By comparison, only three of 15 sites in RP-273 have stream gage data available, with partial coverage for a portion of the watershed at a forth.

  Lacking historical data about the actual daily flow conditions at most of the project sites, watershed models were utilized to synthesize flow rates for sites where gage measurements were not available. A watershed model was utilized in the process of synthesizing flow data where it was missing at the NCHRP Mill Creek, Oregon project site, and a similar approach was used in this project, as described below.

- **Cross section data availability**. The scour depth at project sites in the NCHRP project were defined by historical survey data of the stream channel at the upstream face of bridges, such that the abrasion of rock over time could be characterized by comparison of past channel depths to present channel depths. Having this historical cross section data also enabled NCHRP project researchers to select a period of study other than that period since the bridge was constructed (i.e., scour could be examined in the time interval between cross sections, and not only since the time of construction).

  Since cross sectional data was not available at WVDOH project sites, scour depth was defined by inspection; the original location of the rock foundation was inferred from the presence of surrounding rock, bridge geometry, and other information. Since only a single estimation of scour depth was available, corresponding to scour occurring during the period of bridge construction until the date of inspection, this meant less flexibility in the period of study for determining Scour Numbers and greater uncertainty and potential error.

- **Scour location**. The methodologies developed during NCHRP Project 24-29 focus on scour at bridge piers, and a key element of this project was to investigate scour at abutments, including a directive to attempt normalization of pier scour behavior to abutment scour locations.

Table 2 – Comparison of NCHRP and WVDOH rock scour project sites.

Project	Site	Watershed Area (mi^2)	Stream Gage Data Available	Historical Cross Sections Available	Potential Scour Location Studied
NCHRP	Schoharie Creek, NY	886	Yes	Yes	Pier
	Chipola River, FL	464	Yes	No scour	Pier
	Mill Creek, OR	32.7	Partial	Yes	Pier
	Sacramento River, CA	6468	Yes	Yes	Pier
	Montezuma Creek, UT	1154	Yes	Yes	Pier
	Average	*1801*			
WVDOH	Leatherwood Rd, Left Fork of Leatherwood Creek	3.34	No	No	Abutment
	Fifth Street, Fourpole Creek	13.24	Partial	No	Abutment & Pier
	Little Sandy Creek	8.40	No	No	Abutment
	Grassy Run, Prickett Creek	2.52	No	No	Abutment
	Caldwell Run	1.46	No	No	Abutment
	Paden Fork	1.01	No	No	Abutment
	Audra Park, Middle Fork River	149.31	Yes	No	Abutment & Pier
	Laurel Fork	11.74	No	No	Abutment
	Roaring Creek	14.01	No	No	Abutment & Pier
	Beverly, Tygart Valley River	219.02	Yes	No	Abutment & Pier
	Coon Creek	3.4	No	No	Abutment
	Bridge Fork	4.25	No	No	Abutment
	Cucumber, Jacob's Fork	30.63	No	No	Abutment
	Clear Fork, Cedar Creek	123.42	Yes	No	Abutment & Pier
	Mish Road, Mill Creek	14.21	No	No	Abutment
	Average	*40*			

Field Drilling of Rock Samples

Drilling for this project was conducted in June and July 2011 by L.G. Hetager, an affiliate company of Gannett Fleming. Inc. Rock coring into the 10 feet of bedrock immediately below the bridge abutments was performed at two locations per site, one location adjacent to each abutment. Upon extraction from the borehole, the rock core was immediately logged, photographed, and evaluated for rock quality designation (RQD) according to ASTM 6032.

Each 5-foot length of rock core was then double-wrapped, first in a 5-mil wax film (Parafilm or equal) followed by a heavy gauge sheath of flexible plastic. The package was secured with duct tape and labeled, then placed in a protective core box for transport. Prior to transport the core was kept in a locked vehicle and/or within site of the project personnel. The core was transported within a maximum of 72 hours of collection to the engineering laboratory at Marshall University. During transport the core boxes were wrapped in moving blankets (or equivalent padding) to minimize the impact of potential vibration. Mode of transport was by personal or rental vehicle driven by the P.I. or his student assistants or by personnel from Gannett Fleming. A chain-of custody form was used for each core or group of cores to document transfer of samples from one site to another or from one person to another. The above procedures were consistent with ASTM D 2113 Section 5.0.

One or two additional rock samples were collected and preserved from surface exposures at or immediately adjacent to the bridges. Stream pH was measured and recorded at each bridge location (see Table 15). The drilling schedule is presented in Table 3.

Table 3 – Project Drilling Schedule

WVDOH Scour Boring Locations							
ROTATION	DATE**	INSPECTOR	BRIDGE NAME	COUNTY	BORING NAME	Lat_DMS	Long_DMS
1	6/20/2011	NJW	Caldwell Run	Ohio	CAL-01	40d02'37.56"N	80d42'41.69"W
	6/20/2011	NJW			CAL-02	40d02'38.44"N	80d42'41.92"W
	6/21/2011	NJW	Paden Fork	Wetzel	PAD-02	39d35'33.45"N	80d54'34.16"W
	6/21/2011	NJW			PAD-01	39d35'33.39"N	80d54'33.26"W
	6/22/2011	NJW	Little Sandy Creek	Jackson	SAN-01	38d59'01.7"N	81d42'53.41"W
	6/22/2011	NJW			SAN-02	38d59'01.02"N	81d42'53.45"W
	6/23/2011	NJW	5th St. Ritter Park	Cabell	RIT-01	38d24'22.95"N	82d26'49.67"W
	6/23/2011	NJW			RIT-02	38d24'21.59"N	82d26'49.63"W
	6/24/2011	NJW	Leatherwood	Kanawha	LEA-02	38d28'08.17"N	81d21'52.69"W
	6/24/2011	NJW			LEA-01	38d28'08.61"N	81d21'53.35"W
	6/25/2011	NJW	Bridge Fork	Fayette	BRF-01	38d12'24.03"N	81d08'24.33"W
	6/25/2011	NJW			BRF-02	38d12'24.22"N	81d08'23.67"W
	6/26/2011	NJW	Coon Creek	Summers	COO-01	37d50'46.79"N	80d51'14.65"W
	6/26/2011	NJW			COO-02	37d50'46.39"N	80d51'14.94"W
	6/27/2011	NJW	Laurel Fork	Upshur	LAU-01	38d52'30.42"N	80d15'50.58"W
	6/27/2011	NJW			LAU-02	38d52'31.40"N	80d15'51.16"W
	6/28/2011	NJW	Audra Park	Barbour	AUD-01	39d02'28.35"N	80d04'02.81"W
	6/28/2011	NJW			AUD-02	39d02'29.57"N	80d04'03.57"W
	6/29/2011	NJW	Grassy Run	Marion	GRA-01	39d27'12.99"N	80d03'53.93"W
	6/29/2011	NJW			GRA-02	39d27'12.55"N	80d03'54.67"W
2	7/6/2011	JMG	Cucumber	McDowell	CUC-01	37d16'16.71"N	81d37'14.86"W
	7/6/2011	JMG			CUC-02	37d16'15.55"N	81d37'14.81"W
	7/7/2011	JMG	Clear Fork	Wyoming	CLE-01	37d37'26.61"N	81d42'13.40"W
	7/7/2011	JMG			CLE-02	37d37'25.81"N	81d42'11.61"W
	7/8/2011	JMG	Beverly	Randolph	BEV-01A	38d50'33.12"N	79d52'35.67"W
	7/9/2011	JMG			BEV-02A	38d50'32.28"N	79d52'33.68"W
	7/10/2011	JMG	Roaring Creek	Pendeleton	ROA-01	38d50'48.79"N	79d25'18.93"W
	7/11/2011	JMG			ROA-02	38d50'49.68"N	79d25'18.61"W
	7/12/2011	JMG	Mish Road	Berkeley	MIS-01	39d20'43"N	78d04'02"W
	7/12/2011	JMG			MIS-02	39d20'43"N	78d04'01"W

Lab Analysis of Rock Sampling

Rock core and surface exposure samples were subjected to testing for specific gravity at natural (field) moisture conditions and modified slake durability (continuous abrasion). Cores samples were also tested for unconfined compressive strength (UCS). All tests were conducted in accordance with applicable ASTM standards and NCHRP Project 24-29.

The slake durability tests were performed by the P.I. and undergraduate geology students supervised by the P.I. and trained in the method. Slake durability testing was conducted using standard lab equipment owned by the Marshall Department of Geology and housed in the Weisberg Engineering Laboratories at Marshall (see Figure 1). The slake durability apparatus used for this project consisted of two drums, so two tests could be run at a time. The testing methodology was based on the modified procedure as described in the NCHRP Project 24-29, which is based on Dickenson and Baillie (1999).   Rock pieces taken from both surface exposure and core samples met the test method requirements for mass (10 pieces at 40-60 grams each). However, pieces taken from the surface exposure samples more nearly met the requirement that pieces be equidimensional than those from the core samples which, by necessity, were disc-shaped, with the long dimension equal to the core diameter (see Figure 2). The effect of these shape differences is evaluated later in this report when discussing test results.

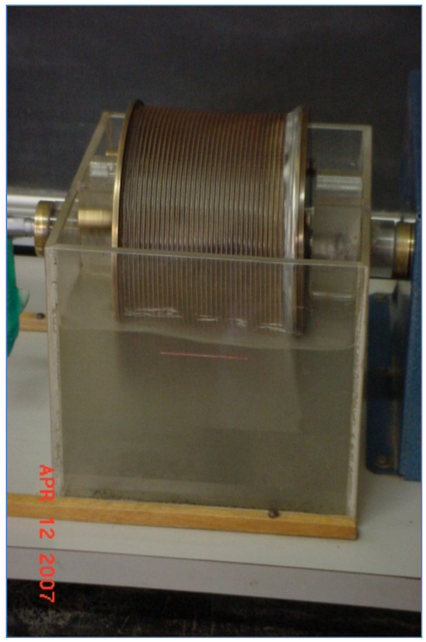

Figure 1 – Rotating drum used in the continuous slake durability tests.

The fundamental goal of the testing was to define the linear portion of the curve relating scour depth to stream power. This required three or more points on this portion of the curve, with each point corresponding to a multiple of a 60-minute continuous abrasion cycle. For example, depending on the rock type, the linear portion of the curve was sufficiently delineated by the weight loss at 180, 240, and 300 minutes; 300, 360 and 420 minutes; or 360, 420 and 480 minutes of continuous abrasion. Typically two tests were run per day.   Figure 2 shows an example of rock samples before and after continuous slake durability testing.

Figure 2 – Example of rock samples before (left) and after (right) continuous slake durability testing.

Core samples for UCS testing were trimmed to shape and size specifications (ASTM D-4543) by Triad Engineering, Inc. in St. Albans, WV. The UCS tests were performed on the prepared samples at Triad by the P.I. and a student assistant per ASTM D2938.

Computation of Geotechnical Scour Number

Derivation of Depth of Scour and Equivalent Stream Power from Lab Testing

The modified slake durability testing requires that mass lost during each 60-minute cycle be recorded and then converted to a volume using the natural (field) saturated specific gravity of the rock being tested:

$$V_i = M_i / \gamma_{sat}$$

V_i = incremental volume lost during 60-minute test interval (L^3),

M_i = incremental mass lost during 60-minute test interval (M),

γ_{sat} = natural (field) saturated specific gravity of the rock being tested (M/L^3).

Multiple measurements at successive test intervals up to 9 hours result in a series of V_i which are then normalized by a unit area to derive a linear dimension equivalent to a depth of scour for each interval:

$$D_{(X*60\ min)} = V_{i\ (X*60\ min)} / \text{ unit area}$$

$D_{(X*60\ min)}$ = equivalent depth of scour corresponding to test interval x (= 1,2,3 …9) minutes (L),

$V_{i\ (X*60\ min)}$ = incremental volume lost during test interval x (= 1,2,3 …9) minutes, (L^3),

unit area = area in appropriate units (L^2).

Equivalent stream power for each test interval can be calculated based on the average weight of sample per test interval, the distance (equivalent length) traveled by the sample in the submerged rotating drum during the test interval, and the time of the test interval:

$$\omega = L\,t\;[\,W_{(X)} + W_{(X+1)}\,]\,/\,[2\,A_{(1/8)}]$$

ω = equivalent stream power [force* L/T/L^2],

L = equivalent length (L),

t = incremental time of test interval (T).

$[\,W_{(X)} + W_{(X+1)}\,]\,/\,2$ = average weight of sample during test interval (force).

$A_{(1/8)}$ = area of submerged portion of drum (1/8 of total area).

The final result of each modified slake durability test, then, allows a plot of ω versus D for multiple test intervals. An example from Keaton and Mishra (2010) is shown in Figure 3:

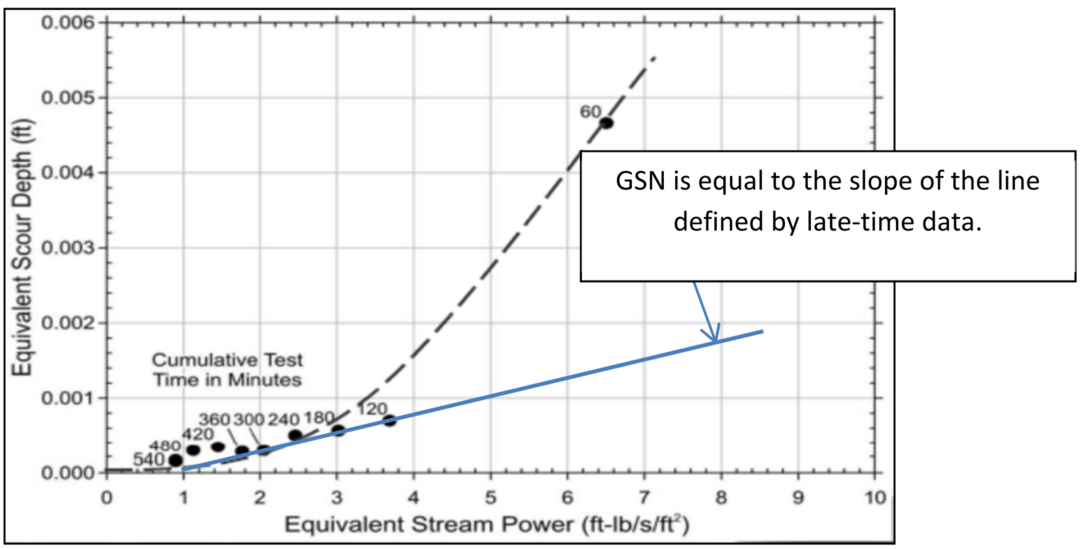

Figure 3 – Plot of Equivalent Scour Depth vs. Equivalent Stream Power

It should be noted that as the test proceeds (increasing time, above), data points define a trend that approaches lower values of both stream power and scour. This is because stream power is a function of sample weight which, as the sample progressively abrades, is decreased in later test intervals.

Derivation of Geotechnical Scour Number

When the linear portion of the equivalent scour versus equivalent stream power plot is considered, a slope can be defined for each test (Figure 3). This slope can then be expressed as scour per unit stream power (e.g., foot of scour per unit of power). A higher GSN represents a higher slope; a lower GSN represents a lower slope. In other words, higher values reflect a greater amount of scour, or abrasion, per unit stream power, while lower values reflect a lesser amount of scour, or abrasion, for the same expenditure of stream power. This parameter, the geotechnical scour number (GSN), becomes the basis by which the scour potential of different rock types can be compared when scour is occurring by abrasion of degradable rock. In addition, for sites affected by abrasion, the GSN becomes the starting point for relating calculated values of cumulative stream power at bridge sites to predicted scour depth. Since the mode of scour was not always initially evident from field inspection, GSN was calculated for all bridge sites evaluated for this project. Regardless of scour mode, the GSN and GSN* serve as a useful means of comparing the abrasion resistance of different rock types.

The GSN was calculated two ways for this project: 1) by forcing a zero-intercept for the best-fit line, and 2) by not forcing a zero-intercept. The former method follows the procedures specified in NCHRP 717, which is meant for degradable (i.e., less abrasion-resistant) rocks that trend toward near-zero values of equivalent steam power and equivalent scour depth. In such cases, forcing a zero-intercept both honors the data and produces a relatively strong linear correlation. However, for continuous slake durability data from the 15 West Virginia bridges, the median coefficient of determination (R^2) was only 0.13 when the GSN was calculated in this manner. In contrast, for GSN calculated without forcing a zero-intercept, the median R^2 value was 0.79. These differences are primarily due to the fact that scour of rock at the majority of West Virginia bridges studied occurs by some degree of quarrying of durable, jointed rock, rather than by abrasion of degradable rock. Durable rock, by definition, tends to be resistant to abrasion. When subjected to a modified slake durability procedure, durable rocks retain significant mass, and thus stream power, throughout the test, and a linear regression on the data intersects the x-axis (stream power axis) at a significant positive value. The physical significance of this x-axis is known as the “threshold value” and represents the minimum stream power needed to initiate abrasion scour (NCHRP 717).

Figure 4 shows a representative example from the project data of GSN calculated both with and without forcing a zero-intercept and the resulting change in GSN and R^2. For the remainder of this report, GSN will be used to refer to the slope of the best-fit line determined from the zero-intercept method (per NCHRP 717), and GSN* will be used to refer to the slope of the best-fit line determined from the non-zero-intercept method.

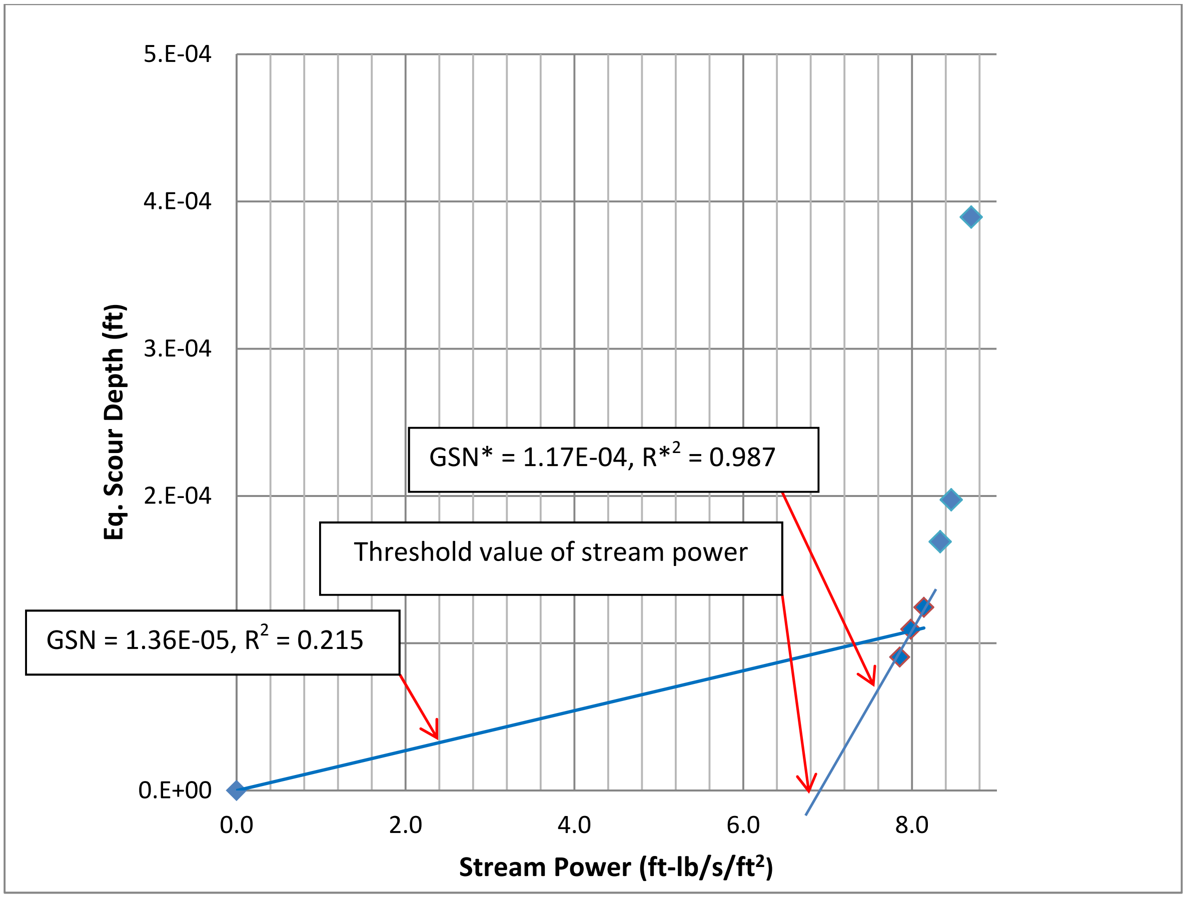

Figure 4 -- Representative plot of continuous slake durability test results with linear trend of late-time data determined with and without forcing a zero-intercept.

Hydrology Methods and Flow Rate Characterization

Stream Gage Data

Initial assessment of flow data availability at each of the project sites was performed by a comparison of project locations to the inventory of stream gage sites that is maintained at the USGS National Water Information System Mapper (USGS, National Water information System Mapper, 2013), and cross-referencing the stream gages identified in the USGS publication "Estimation of Flood-Frequency Discharges for Rural, Unregulated Streams in West Virginia (USGS, SIR 2010-5033, 2010). Nearby stream gages were identified for each project site, including data type (e.g., annual peak vs. daily average flow), the dates of available data, coverage percentage during the availability range, distance from the gage to its respective project site, and characteristics of the watershed gaged such as area, mean elevation, and the presence of any flow-regulating structures within the watershed. For each project site, a table was prepared summarizing the available stream gage data, an example of which for the 5th Street Ritter Park project site is shown in Table 4. Corresponding tables for all project sites are provided in Appendix A.

Table 4 – Stream gage stations near the 5th St. Ritter Park project site.

Project / Stream Gage Location	USGS Station ID	Period	Covg. (%)	Lat.	Long.	Dist. (mi.)	Elev. (ft.)	Drainage Area (mi^2)	Type
5th St. Ritter Park, Fourpole Creek	-	*1921-Present*	-	*38.4063*	*-82.447*	-	*547*	*13.24*	-
Mud River Near Milton, WV	03204500	1939-1980	100	38.388	-82.113	18.1	572.6	256	DD
Sandusky Creek near Burlington OH	03205995	1978-2011	61	38.418	-82.51	3.5	553	0.73	AP
FOURPOLE CREEK NEAR HUNTINGTON, WV	03206450	1999-2011	100	38.3625	-82.392	4.3	599.4	4.02	AP
TWELVEPOLE CREEK BELOW WAYNE, WV	03207020	1915-1982	63	38.249	-82.434	10.9	559.3	300	DD

At four of 15 project locations – 5th Street Ritter Park, Audra Park, Beverly, and Clear Fork – the stream flowing under the bridge in question was gaged. At 5th Street Ritter Park, however, the coverage range and gage location (i.e., upstream area) relative to project location limited the applicability of gage data. For the rest, stream gage data was downloaded and a flood frequency analysis was performed based on the guidelines identified in USGS Bulletin 17B (Geological Survey Office of Water Data Coordination, 1982), using the USGS-sponsored PeakFQ version 5.2 software (Flynn, Kirby, & Hummel, 2006). The output of this flood frequency analysis was a characterization of discharge as a function of recurrence interval, including an estimate of flow rates associated with the 95% confidence interval. An example of this output is provided in Table 5 for the Audra Park project location, and tables for Beverly and Clear Fork are found in Appendix A.

Confidence intervals are identified throughout this report for two primary reasons. First is to acknowledge the relative uncertainty that is inherent in hydrologic modeling and estimation, and specifically with respect to the data sources utilized in this project. Since a modeling-based approach inevitably yields more uncertainty about time-series flow rates than the stream gage based approach that served as the foundation of NCHRP Project 24-29, it is important to characterize the extent of this uncertainty, where possible. The second reason why confidence intervals are provided is in case additional threshold analyses are conducted at the project sites. Since the quarrying mode of scour is a threshold, rather than gradual, phenomenon, it is important to understand the entire probabilistic range of flow rates, rather than just the 50 percentile probability flow rate.

Table 5 – Flood frequency analysis for the Middle Fork River at Audra Park.

Recurrence Interval (yr)	Discharge (ft^3/s)	95% Confidence Interval	
		Lower	Upper
2	6035	5547	6564
5	8495	7719	9394
10	10239	9140	11510
25	12601	10950	14423
50	14483	12301	16765
100	16474	13682	19247
200	18596	15093	21869
500	21649	17015	25612

An analysis of daily average flow data at Audra Park, Beverly, and Clear Fork was conducted to determine the typical duration of flow events that contribute to Cumulative Effective Stream Power. By comparing flow rates during the day prior to and day after days where the highest flow rates at each site were reported, it was concluded that flow durations above the 2-yr peak flow rate are not multi-day in nature. That means, for example, that although it may require more than one day to fully drain the Audra Park watershed following the 10-year storm, that portion of the 10-year storm hydrograph that is above the peak flow rate of the 2-year storm is less than one day in duration. This finding guided subsequent calculations of stream power.

Instantaneous peak flows from the flood frequency analysis were incorporated into runoff hydrographs that characterized flow as a function of time. Since computations of Cumulative Effective Stream Power integrate stream powers over time that exceed the peak stream power for the two year storm event, and since the watersheds studied in this research exhibited relatively short times of concentration, computing stream power on the basis of Average Daily Flow data would neglect important high energy, peak flow times during storm events. These peak flows are particularly relevant for the quarrying and plucking scour mode. Additional information related to watershed modeling and the approach utilized for hydrograph generation is provided in the Watershed Modeling section of the Research Approach.

Watershed Modeling

Since stream gage data was unavailable for 12 of 15 project sites, a flow synthesis approach was used based on watershed characteristics. A watershed model was created for each site; Figure 5 represents the delineated watershed at Leatherwood Road, and Table 6 summarizes several relevant watershed parameters at Leatherwood Road. Corresponding figures and tables for each project site are included in Appendix A.

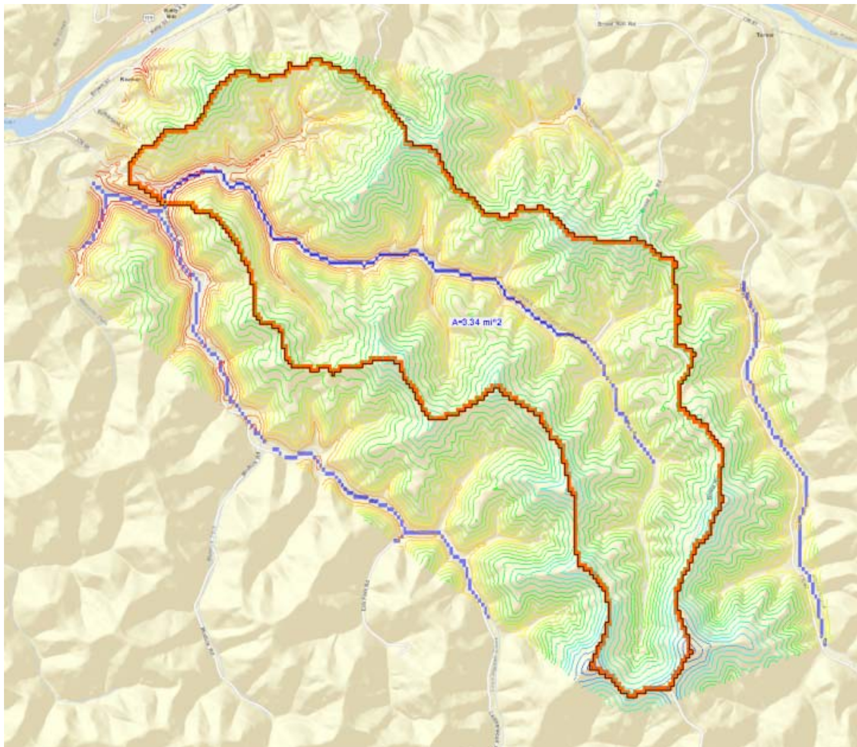

Figure 5 - Image of delineated Leatherwood watershed.

Elevation data for each watershed was taken from Digital Elevation Map (DEM), National Elevation Dataset (NED) data that is available at the USGS National Map Viewer website (USGS, 2013). Watershed boundary delineation was performed using the TOPAZ digital landscape analysis tool (Garbrecht & Martz, 1999) that is embedded into Aquaveo's Watershed Modeling System (WMS) software.

Table 6 - Watershed characteristics for Leatherwood Road.

Parameter	**Value**
Location	Lat: 38.468833 Long:-81.365000
Region – USGS SIR2010-5033	Western Plateaus
Basin Area (mi^2)	3.34
Average basin elevation (ft)	3202
Average basin overland slope (%)	97.5
Maximum flow distance (ft)	24565
Slope along maximum flow distance (%)	10.5
Shape factor (basin length / basin width)	3.44
Sinuosity factor of stream (max. stream length / basin length)	0.80
Runoff Curve Number	65.3
Time of Concentration (hr.)	1.049

The National Land Cover Database 2006 data incorporated into calculations of runoff Curve Number is also available at the National Map Viewer website. Soil Survey Geographic (SSURGO) soil type data utilized in watershed modeling was taken from the Natural Resources Conservation Service (NRCS) Soil Data Mart website (NRCS, 2013). Curve Number calculations incorporated overlapping land cover and soil type polygons, according to the classifications described in Chapter 9 (Hydrologic Soil-Cover Complexes) of the Natural Resources Conservation Service's Part 630 Hydrology – National Engineering Handbook, and as summarized in Table 7. A table summarizing Curve Number calculations for each site's watershed is provided in Appendix A.

Table 7 – Land Cover, Soil Group, and Curve Number.

Class \ Value	NLCD (2006) Description	USDA Hydrologic Soil Group			
		A	B	C	D
Water					
11	Open Water	98	98	98	98
12	Perennial Ice/Snow	98	98	98	98
Developed					
21	Developed, Open Space	39	61	74	80
22	Developed, Low Intensity	51	68	79	84
23	Developed, Medium Intensity	54	70	80	85
24	Developed High Intensity	89	92	94	95
Barren					
31	Barren Land (Rock/Sand/Clay)	77	86	91	94
Forest					
41	Deciduous Forest	30	55	70	77
42	Evergreen Forest	30	55	70	77
43	Mixed Forest	30	55	70	77
Shrubland					
52	Shrub/Scrub	30	58	71	78
Herbaceous					
71	Grassland/Herbaceous	30	58	71	78
Planted/Cultivated					
81	Pasture/Hay	39	61	74	78
82	Cultivated Crops	65	75	82	86
Wetlands					
90	Woody Wetlands	30	55	70	77
95	Emergent Herbaceous Wetlands	98	98	98	98

Watershed time of concentration was determined according to the Soil Conservation Service (SCS) method, which incorporates the maximum flow distance within a watershed, average watershed slope, and curve number into an estimate of lag time.

A review of available precipitation data in the vicinity of each project site was conducted to assess the viability of incorporating historical rainfall data into watershed models. An example of the weather data station summary table prepared for each site, and available in Appendix A, is provided in Table 8 for 5th

St. Ritter Park. Among the factors that constrained utilization of historical rainfall data was the period of record (i.e., was precipitation data available in the years since the bridge at each site was constructed), the consistency of data availability within the period of record, and the distance from the project site. Review of precipitation data from stations as close together as 5 miles revealed considerable differences in precipitation depths for the same days, reflecting spatial distribution of precipitation patterns that ultimately limited the incorporation of historical precipitation data into watershed models. Where precipitation gages were outside of watershed boundaries, it was not possible to know with confidence what the actual precipitation patterns inside the watershed were over time. This limited the ability of watershed models to be used to predict runoff flows on specific dates, and use this data to compute stream powers associated with actual flood events. An analysis of each site's precipitation data, and relative quality with respect to coverage and proximity, is summarized in the corresponding sections of Appendix A.

Table 8 – Summary of weather stations and data availability for 5th St. Ritter Park.

Station Name	Station ID	Period of Record	Coverage (%)	Lat.	Long.	Elev. (ft.)	Dist. (mi.)
Huntington Federal Bldg.	USC00464388	1948-1967	92	38.416	-82.45	565	0.7
Huntington 2	USC00464383	1943-1951	94	38.416	-82.433	600	1.0
Huntington Sewage Plant	USC00464397	1967-Present	85	38.401	-82.526	520	4.3
Huntington 1	USC00464378	1891-1957	90	38.416	-82.366	679	4.5

In the absence of historical precipitation data, probabilistic precipitation data was obtained for use in watershed modeling. The National Oceanic and Atmospheric Administration's (NOAA) Hydrometeorological Design Studies Center Precipitation Frequency Data Server (NOAA, 2013) was utilized to obtain precipitation depths, including the 90% confidence interval, for storms associated with average recurrence intervals of 2-, 5-, 10-, 25-, 50-, 100-, 200- and 500-years. An example of this data for the 5th St. Ritter Park project site is shown in Table 9, and corresponding tables for each project site are provided in Appendix A.

Table 9 – Precipitation depth at 5th St. Ritter Park for a 24-hour duration storm, including 90% confidence interval.

Average Recurrence Interval (yr.)	Precipitation Depth (in.)	90% Confidence Interval	
		Precip. Depth, Lower (in.)	Precip. Depth, Upper (in.)
2	2.64	2.48	2.82
5	3.20	2.99	3.41
10	3.65	3.41	3.89
25	4.28	3.99	4.55
50	4.79	4.45	5.09
100	5.32	4.93	5.64
200	5.87	5.42	6.22
500	6.63	6.10	7.01

Peak flow rates for each of the 12 watersheds lacking stream gage data were developed using probabilistic precipitation depth data (24-hour storm, Type II temporal distribution) according to three different methods, for sake of comparison and also to illustrate the relative uncertainties inherent in application of watershed models utilizing probabilistic rainfall depths. In increasing order of complexity these methods are: the regression equations presented in the USGS publication "Estimation of Flood-Frequency Discharges for Rural, Unregulated Streams in West Virginia" (USGS, SIR 2010-5033, 2010); the NRCS TR-55 model (NRCS Conservation Engineering Division, 2009); and the US Army Corps of Engineers HEC-1 flood hydrograph model (US Army Corps of Engineers, 1998). Both TR-55 and HEC-1 were executed using WMS as a pre- and post-processor. Where necessary to avoid drainage area limitations, watersheds were broken up into contributing sub-basins.

Examples of the output of these three methods for the Leatherwood Road project site are shown in Table 10 (USGS regression), Table 11 (TR-55), and Table 12 (HEC-1). Corresponding tables for each of the project sites are provided in Appendix A.

Table 10 – USGS SIR-2010-5033 derived peak discharges, average prediction error, and corresponding 90% confidence interval for the Leatherwood Road watershed.

Return Period (yr)	Peak Discharge, Q (cfs)	Avg. Prediction Error (%)	90% Confidence Interval	
			Q, Lower (cfs)	Q, Upper (cfs)
2	311	32.2	146	476
5	520	30.0	264	777
10	678	29.7	347	1010
25	896	30.3	450	1343
50	1073	31.3	521	1626
100	1256	32.5	584	1927
200	1448	33.9	641	2255
500	1720	36.1	699	2741

Table 11 – Peak discharges derived from TR-55 watershed model of Leatherwood Road.

Return Period (yr)	Peak Discharge, Q (cfs)	90% Confidence Interval	
		Q, Lower (cfs)	Q, Upper (cfs)
2	225	172	298
5	470	375	589
10	724	589	868
25	1122	940	1314
50	1483	1249	1711
100	1874	1596	2138
200	2309	1969	2600
500	2931	2508	3260

Table 12 – Peak discharges derived from HEC-1 watershed model of Leatherwood Road.

Return Period (yr)	Peak Discharge, Q (cfs)	90% Confidence Interval	
		Q, Lower (cfs)	Q, Upper (cfs)
2	211	162	277
5	427	345	529
10	649	529	781
25	1017	848	1196
50	1354	1136	1568
100	1722	1460	1970
200	2132	1811	2408
500	2722	2320	3036

In addition to providing a peak flow rate associated with each return period storm, the HEC-1 model output includes a runoff hydrograph (using the SCS dimensionless unit hydrograph approach) that defines flow rates over user-defined time increments, in this case being 15-minute intervals. With this characterization of flow rate over time, it was possible to calculate stream power at each interval during the storm duration. While details related to the computation of stream power are provided later in this report, an illustration of the runoff hydrograph for the 2-, 5-, and 10-year storms at the Leatherwood Rd project site are shown in Figure 6. In total there were 24 HEC-1 models executed for each of the 12 ungaged watersheds, yielding 288 runoff hydrographs, each of which had 200 time / flow rate data-points.

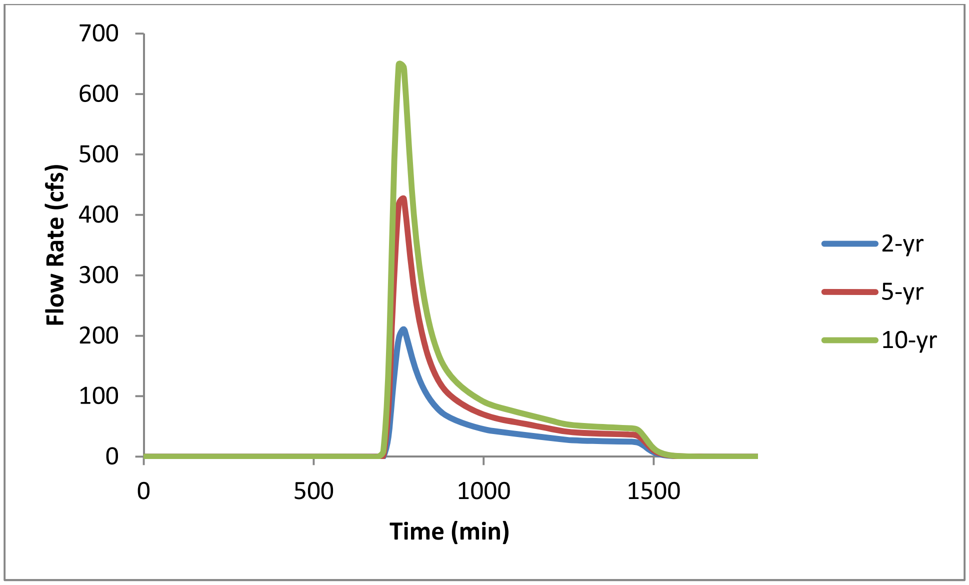

Figure 6- Runoff hydrographs (2-, 5-, and 10-yr flow) for Leatherwood Rd watershed.

For Audra Park, Beverly, and Clear Fork watersheds, a watershed model was prepared for the sole purpose of defining the shape of runoff hydrographs. Since PeakFQ analysis output defined only the peak flow rate for a variety of recurrence interval storms, some method was needed to translate peak flows into a runoff hydrograph. Land cover, soil type, and other watershed data was utilized to estimate times of concentration for watershed sub-basins, and these were coupled with estimates of hydraulic routing of flow from sub-basins within each watershed to determine the flow rate as a function of time. Runoff flows predicted by HEC-1 models were normalized to the peak flow rates defined by PeakFQ, and subsequent hydrographs were ultimately utilized in stream power computations.

Hydraulic Analysis

Although it was initially anticipated that characterization of the stream cross section for use in hydraulic modeling would be based on WVDOH-provided streambed surveys, such data ended up not being available for the project sites that were ultimately selected. Likewise, the possibility of using high-definition Digital Elevation Mapping to define channel geometry was inhibited by the small size of project site channels relative to the resolution of DEM data. Thus it was necessary to conduct a field survey of each site, including channel cross section and bridge geometry.

A minimum of three upstream and three downstream cross-sections were surveyed for each bridge, at approximately 50 ft spacing, defining both the channel geometry and slope for subsequent translation into HEC-RAS models. A survey of the bridge structure was also conducted in order to include bridge elements in the hydraulic model. Channel roughness (Manning's n) values were determined according to material descriptions found in the HEC-RAS user's manual (US Army Corps of Engineers, 2008).

Additional information about each of the site hydraulic models can be found in the HEC-RAS project files, previously provided to WVDOH.

To ensure that geometric data was available over a wide range of likely flows, data points were collected to characterize each of the stream banks (i.e., left and right) in addition to the main channel. An example of the results of survey data translation into model geometry is shown for the 5th St. Ritter Park project site in Figure 7, and similar depictions are provided for each project site in Appendix A.

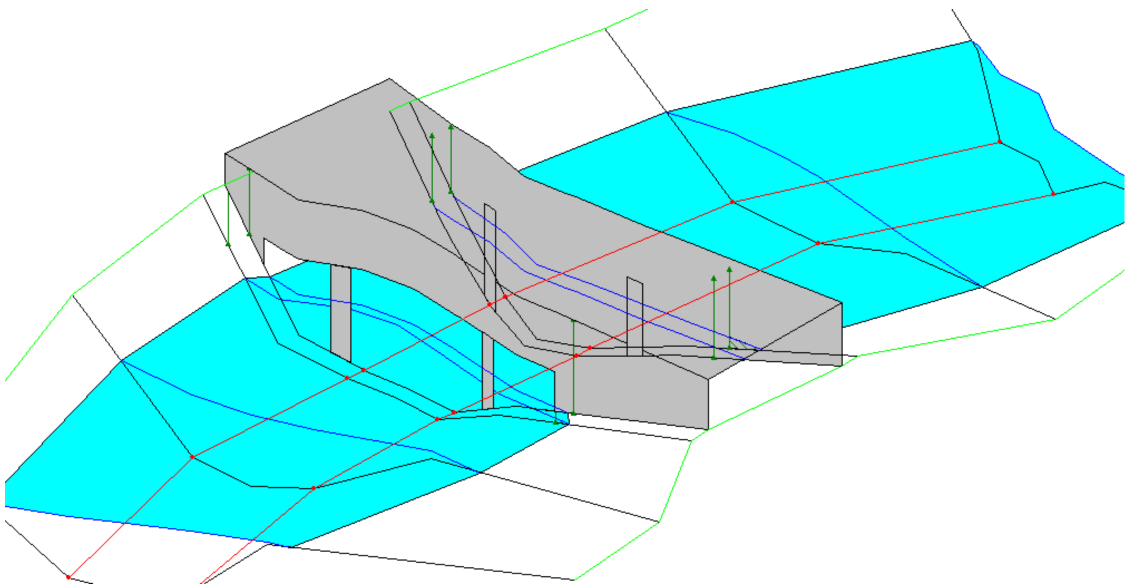

Figure 7- HEC-RAS X-Y-Z Perspective Plot for 5th Street Ritter Park.

The HEC-RAS numerical model developed for each site was used to define flow velocity and depth over the range of flow rates between the 2- and 500-year recurrence interval.

Since runoff hydrographs at each site would define flow rate over the course of a storm event, where flow rate would vary flow baseline conditions (e.g., zero flow) to the peak flow rate, it was not practical to execute the HEC-RAS model at each flow rate value for each site. A 24-hour duration storm that is modeled at 15 minute increments will have 96 data points within the 24 hour period, each of which has a unique flow rate. Each project site was modeled for baseline condition, 90% high, and 90% low of the 2-, 5-, 10-, 25-, 50-, 100-, 200- and 500-year storms: totaling 24 conditions for HEC-1, and 24 conditions for TR-55. Thus, in total, more than 4600 instantaneous flow rates were computed for each site. To avoid running HEC-RAS at each of these, HEC-RAS was instead executed at a variety of flow rates over the range of values experienced at each site, and then regression models were used to predict flow velocity and depth as a function of flow rate.

An example of such a model for the Paden Fork site is shown in Figure 8, and similar depictions for each of the project locations is provided in Appendix A. In most cases, the best-fit was obtained with a power relationship, although at some sites linear or polynomial relationships were employed.

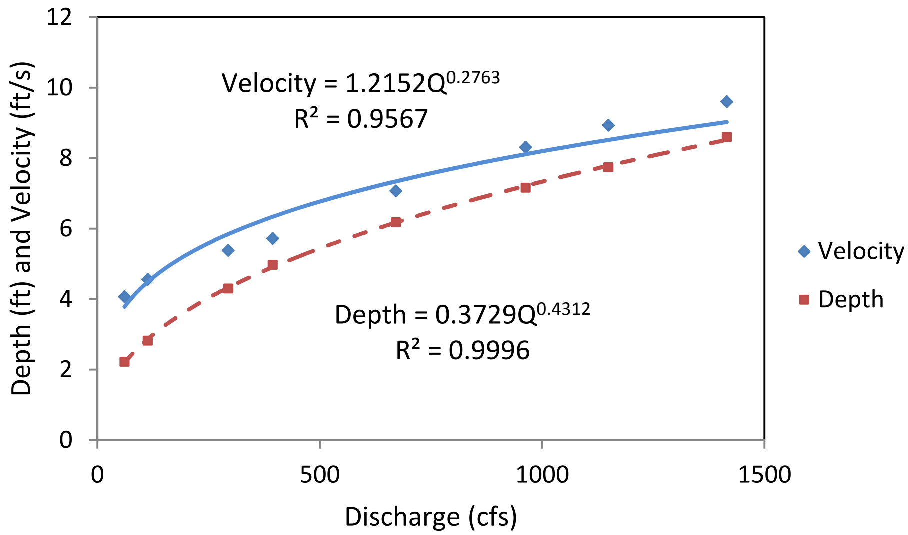

Figure 8- Flow depth and velocity rating curve for Paden Fork.

Cumulative Effective Stream Power

As defined in NCHRP Report 717 (NCHRP, 2012), stream power is a function of shear stress and flow velocity, and can be computed according to Equation 1:

$$P = \left(\frac{nK_pV}{1.486}\right)^2 \cdot \left(\frac{\gamma}{y_0^{1/3}}\right) \cdot V \qquad \text{Equation 1}$$

where n = Manning's roughness coefficient

K_p = Turbulence-related velocity enhancement factor; 1.0 for approach flow, 1.5 for round-nosed piers, 1.7 for square-nosed piers

V = flow velocity, (ft/s)

1.486 = factor for U.S. customary units (1.0 for metric units)

γ = unit weight of water, 62.4 lb/ft^3

y_0 = depth of approach flow, ft

Instantaneous power can be computed in a channel at any point in time by first defining the flow velocity and depth at that time. Runoff hydrographs are developed for each project site, so that site's Cumulative Effective Stream Power can be computed through a process of inputting flow rate data into hydraulic models, and determining the instantaneous stream power for each time interval. Since “effective” stream power is desired, the next step is screening out stream powers below the threshold of the peak flow during the 2-yr event, and integrating power over the duration of an event. Finally, the relative importance of any particular storm event can be accounted for by combining the probability of encountering each event in a given year with the Cumulative Effective Stream Power for that event (see Equation 2).

Figure 9 illustrates the concept of applying a threshold, to account for a minimum stream power below which rock scour does not occur. This illustration, presented conceptually in terms of flow rate (although the threshold is actually applied on a stream power basis when calculations are performed), shows that none of the flows during a 2-year event (and thus none of the stream power associated with the 2-year event) will contribute to the Cumulative Effective Stream Power, because those flows (and stream powers) are less than the peak value of the 2-year event. By definition, Cumulative “Effective” Stream Power only includes that stream power exerted beyond the peak of the 2 year event. This screen is applied to all flows for each site.

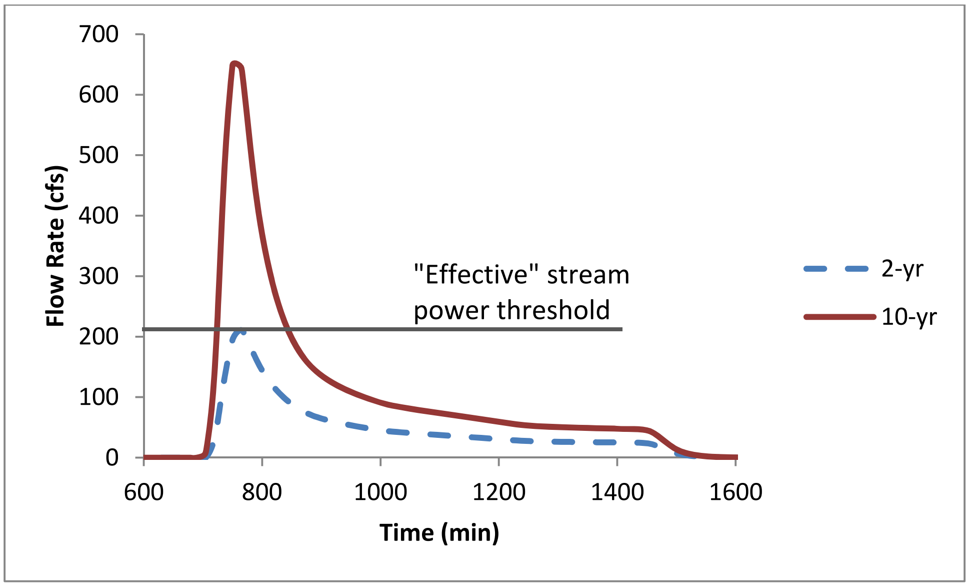

Figure 9- Illustration of the “effective” threshold, as applied to runoff hydrograph (Leatherwood Rd).

Finding the Cumulative Effective Stream Power for a given event is a matter of finding the area under the Excess Stream Power curve, as illustrated for the 10-year event at Leatherwood Rd in Figure 10.

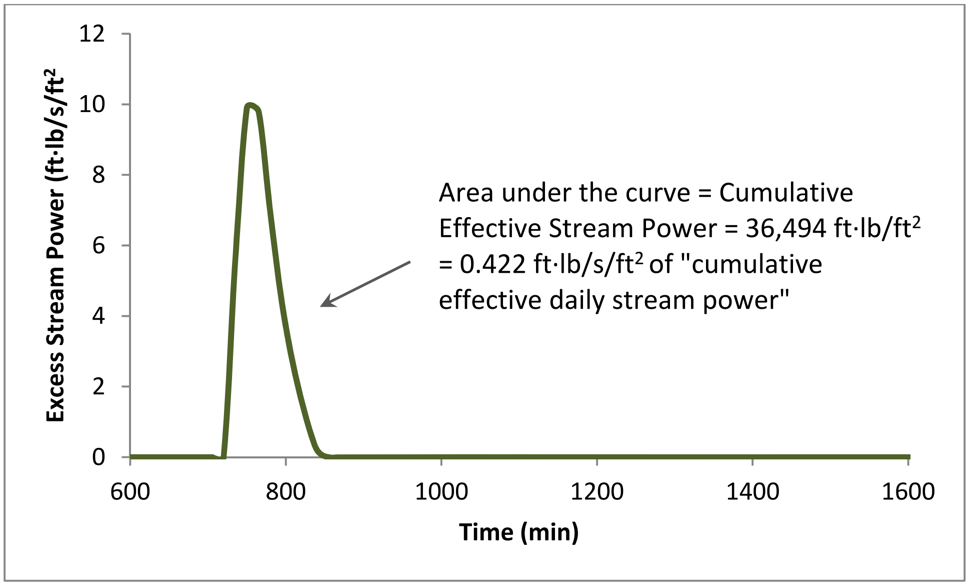

Figure 10- Illustration of the Effective Stream Power for the 10-yr event at Leatherwood Rd.

Table 13 – Scour event Cumulative Effective Stream Power Ω (approach flow) for Leatherwood Road – based on HEC-1 generated flow rates and hydrograph.

Return Period (yr)	Cumulative Effective Stream Power, Ω (ft·lb/ft^2)	90% Confidence Interval	
		Lower (ft·lb/ft^2)	Upper (ft·lb/ft^2)
2	0	0	0
5	14,957	13,415	16,323
10	36,494	32,797	38,460
25	73,339	67,249	76,882
50	108,963	99,835	113,612
100	148,335	137,087	153,744
200	192,546	177,744	197,523
500	257,652	238,383	261,442

An example of the scour event Cumulative Effective Stream Power tables that are provided for each site in Appendix A is shown for the Leatherwood Rd project site in Table 13. Once Cumulative Effective Stream Powers are known for each of a site's recurrence interval events, the cumulative impact of each event, on an annual basis, can be estimated by developing an Annual Average Cumulative Effective Stream power, $\Omega_{annual\ average}$, using each event's annual probability λ and Cumulative Effective Stream Power Ω in a probability weighting approach, summarized in Equation 2. Since Cumulative "Effective"

Stream Power has been defined as the energy exerted above the two year flow, the term corresponding to the interval between 0 and Ω_2 is omitted from Equation 2.

$$\Omega_{\text{annual average}} = \left(\frac{\Omega_2 + \Omega_5}{2}\right)(\lambda_2 - \lambda_5) + \left(\frac{\Omega_5 + \Omega_{10}}{2}\right)(\lambda_5 - \lambda_{10}) + \left(\frac{\Omega_{10} + \Omega_{25}}{2}\right)(\lambda_{10} - \lambda_{25})$$
$$+ \left(\frac{\Omega_{25} + \Omega_{50}}{2}\right)(\lambda_{25} - \lambda_{50}) + \left(\frac{\Omega_{50} + \Omega_{100}}{2}\right)(\lambda_{50} - \lambda_{100}) + \left(\frac{\Omega_{100} + \Omega_{200}}{2}\right)(\lambda_{100} - \lambda_{200}) \qquad \text{Equation 2}$$
$$+ \left(\frac{\Omega_{200} + \Omega_{500}}{2}\right)(\lambda_{200} - \lambda_{500}) + \Omega_{500}\lambda_{500}$$

The result of this process is illustrated in Table 14 for Leatherwood Road, and corresponding tables are provided in Appendix A for each project site.

Table 14 – Average Annual Cumulative Effective Stream Power $\Omega_{\text{annual average}}$ at Leatherwood Road.

Increment (return period range)	λ size	Interval Cumulative Effective Stream Power, Ω (ft·lb/ft^2)		
		Baseline Value	Lower 90% Conf.	Upper 90% Conf.
2-5	0.300	7,479	6,707	8,161
5-10	0.100	25,726	23,106	27,391
10-25	0.060	54,917	50,023	57,671
25-50	0.020	91,151	83,542	95,247
50-100	0.010	128,649	118,461	133,678
100-200	0.005	170,441	157,416	175,633
200-500	0.003	225,099	208,064	229,482
500-	0.002	257,652	238,383	261,442
Avg. Annual Cum. Eff. Stream Power, Ω (ft·lb/ft^2):		13,263	12,068	13,979

Once the Average Annual Cumulative Effective Stream Power has been determined for a site, this is multiplied by the number of years since the bridge at that site was constructed, resulting in an estimate of the total Cumulative Effective Stream Power at that site. Thus, as applied in this project, the Cumulative Effective Stream Power calculation workflow for each site consisted of:

- Input precipitation data into HEC-1 model to obtain hydrograph (i.e., flow rate as a function of time in 15-minute increments during the storm event)
- Input hydrograph into hydraulic models to obtain flow velocity and depth at each of the time points included in the hydrograph

- Input flow velocity and depth values into the stream power equation (i.e., Equation 1), yielding a characterization of instantaneous stream power at each of the recurrence intervals of interest (i.e., 2-, 5-, 10-, 25-, 50-, 100-, 200- and 500-years)
- Determine the peak stream power associated with the 2-year event – this is the threshold above which stream power is "effective"
- Subtract the 2-year peak stream power from stream powers of all other recurrence intervals, yielding a temporal distribution of effective stream power for each recurrence interval event at each site
- Multiply the effective stream power [ft·lb/s/ft^2] during each 15-minute time increment by the duration of that increment (i.e., 900 seconds) to yield the Cumulative Effective Stream Power [ft·lb/ft^2] for that increment.
    - Note: Cumulative Effective Stream Power can also be expressed in terms of 'Cumulative Effective Daily Stream Power' – an expression used in NCHRP Report 717 – by dividing Cumulative Effective Stream Power, ft·lb/ft^2, by the number of seconds in a day (i.e., 86,400), yielding units of ft·lb/s/ft^2
- Sum all of the incremental Cumulative Effective Stream Powers for an event to find the Cumulative Effective Stream Power associated with that event.
- Apply the probability-weighting approach, multiplying the average Cumulative Effective Stream Power for two events by the incremental probability between those events, and summing the product of these over the entire range of probabilities. This approach is a stepwise integration that approximates the Annual Cumulative Effective Stream Power.
- Multiply the Annual Cumulative Effective Stream Power by the number of years since each site was constructed, resulting in a probabilistically-based estimate of how much Cumulative Effective Stream Power has been exerted at the site since it was built.

Finally, computing the Scour Number is a matter of dividing the scour depth (measured in field inspection of the project site) by the Cumulative Effective Stream Power.

Although the design life of WVDOH bridges is 75 years, storms with a return period greater than this may occur. For example, in any given year, there is a 0.5% chance of a 200-year storm occurring, regardless of what the bridge design life is. For this reason, storms of up to a 500 year return period are included in the probabilistic computations summarized in Equation 2.

Findings

Geotechnical Findings

Characterization of Scour modes

Two of the scour modes defined in NCHRP 24-29 were recognized at bridge sites evaluated during this project: abrasion and grain-scale plucking (heretofore referred to simply as “abrasion”) that affects degradable (i.e., non-durable rocks), whereas the quarrying and plucking (heretofore referred to simply as “quarrying”) mode affects durable rocks that are fractured into blocks small enough to be mobilized by the available stream power. According to NCHRP 717 (2012), more than one scour mode may operate over the life of a bridge. The majority of West Virginia sites showed some evidence for both abrasion and quarrying. Five scour categories of scour were recognized, three of them hybrid: 1) pure abrasion (PAB); 2) dominantly abrasion (DAB), 3) sub-equal abrasion and quarrying (AB/QP); 3) dominantly quarrying (DQP); and 5) pure quarrying (PQP). Only one site (Coon Creek) was judged to be affected solely by abrasion (PAB); four sites were judged to be affected solely by quarrying (PQP). The remainder of sites fell into one of the three hybrid categories. As summarized in Table 15, pH was measured to be greater than 7.0 at each of the sites where measurements were taken. In view of this, chemical dissolution is not believed to have been a participating factor in the rock scour that was observed.

Table 15 – Stream pH measurements at project sites.

Site	Stream pH
1 - Leatherwood	8.4
2 - Fifth Street	-
3 - Little Sandy	8.3
4 - Grassy Run	8.9
5 - Caldwell Run	8.6
6 - Paden Fork	8.3
7 - Audra Park	8.8
8 - Laurel Fork	8.3
9 - Roaring Creek	8.3
10 - Beverly	8.3
11 - Coon Creek	8.9
12 - Bridge Fork	8.8
13 - Cucumber	8.3
14 - Clear Fork	7.6
15 - Mish Road	8.2

Figure 11 shows an example of pure abrasion, where the scalloped, contoured appearance of the rock indicates gradual, grain-by-grain wear. Figure 12 shows an example of pure quarrying. Scour by quarrying occurs in discrete events as blocks are worked loose; the vertical extent of scour is a function of joint spacing, bed thickness, and hydraulic loading. Figure 13 shows an example of sub-equal abrasion / quarrying.

The information provided in Table 16 is a summary of scour mode and rock type by site, illustrating the range of conditions represented at the 15 bridges selected for this project. As would be expected, degradable rocks such as shales, claystones, and weathered sandstones scour primarily by abrasion and durable rocks such as pure sandstones and limestones scour predominantly by quarrying.

Figure 11 – Abrasion of highly weathered sandstone at Coon Creek bridge site. Note scalloped appearance of sandstone surface. Depth of scour, here as undercutting of protruding sandstone ledge, is indicated by arrow.

Figure 12 – Quarrying of jointed and bedded sandstone at Roaring Creek bridge site. Depth of scour (white arrow) measured as vertical thickness of block removed at base of pier.

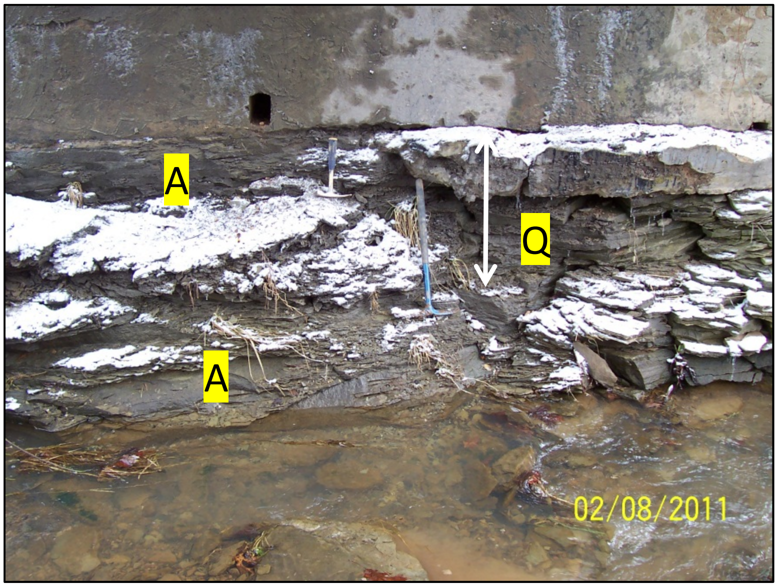

Figure 13 – Scour by sub-equal abrasion and quarrying at Paden Fork bridge site. Areas of abrasion (A) and quarrying (Q) are indicated in yellow letters. The depth of scour (white arrow) is measured as the distance from base of scoured interval to base of abutment.

Table 16 – Summary of scour mode by bridge site and rock type.

		Durable	<<<	Rock Type		>>>	Non-durable
		Sandstone	Limestone	Siltstone	Sandstone - impure	Shale / Claystone	Sandstone - weathered
Scour Mode	PAB						Coon Creek
	DAB					Caldwell Run	
	AB/QP				Bridge Fork; Clear Fork; Laurel Fork; Paden Fork; Ritter Park; Leatherwood	Leatherwood	
	DQP				Cucumber; Grassy Run		
	PQP	Audra Park; Roaring Creek	Mish	Beverly			
No Scour					Little Sandy	Little Sandy	

* PAB – Pure Abrasion; DAB – Dominantly Abrasion; AB/QP – Abrasion and Quarrying & Plucking; DQP – Dominantly Quarrying & Plucking; PQP – Pure Quarrying & Plucking

Characterization of Scour Extent

Scour was measured at all of the bridge sites in three dimensions: 1) vertical scour (in inches) below bridge abutments or piers; 2) horizontal extent (in feet) along piers or abutments parallel to steam flow; and 3) depth of scour (in inches), the extent of scour beneath abutments/piers in a horizontal plane perpendicular to the abutment/pier wall. Figure 14 illustrates the vertical and depth measurements. Table 17 lists the measurements for all sites. Field sketches in Appendix B show specific locations and measurements of scour for all bridges.

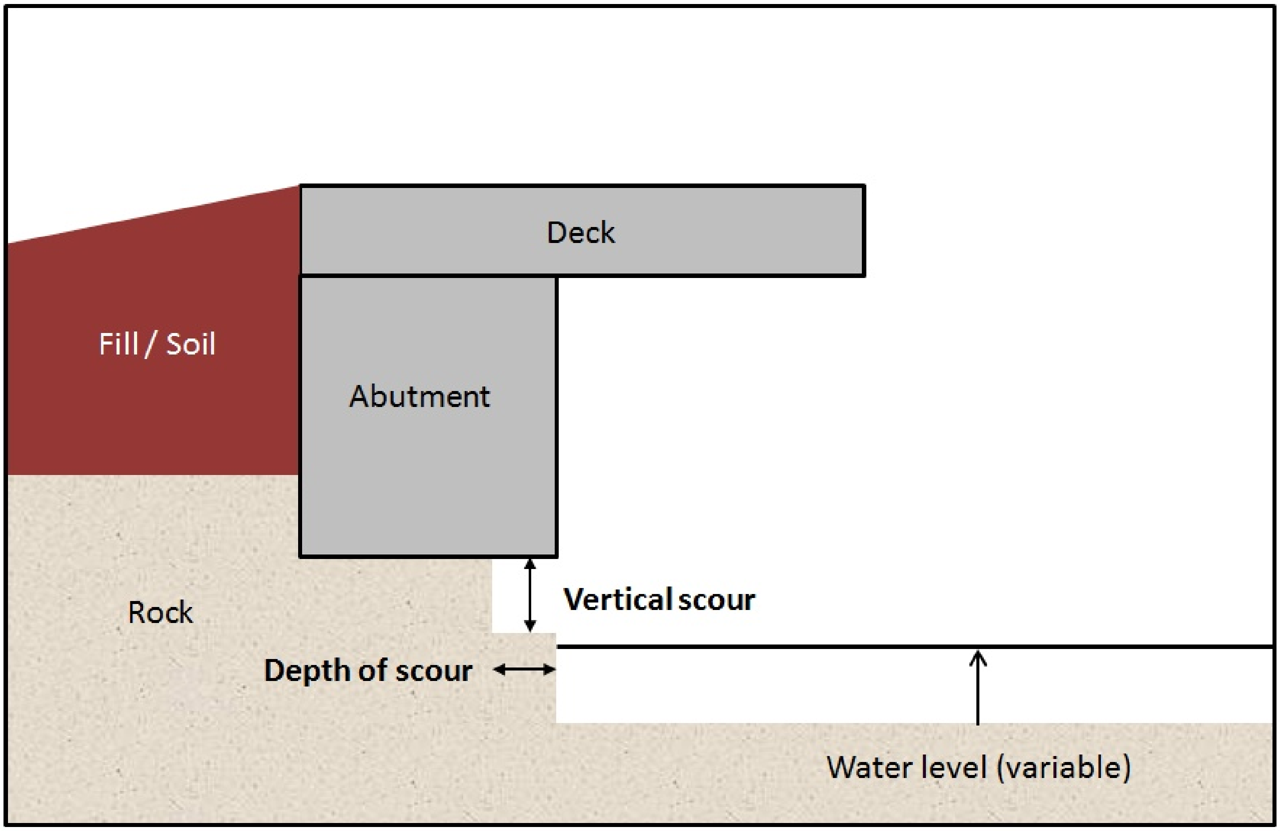

Figure 14 – Measurements of vertical scour and depth of scour. Horizontal extent of scour, not shown, was measured parallel to the abutment or pier (perpendicular to the figure plane).

Table 17 – Scour extent and mode at each bridge site. "NM" indicates locations where measurements could not be obtained due to inaccessibility or safety concens, e.g., high water.

Site	Mode of Scour (MOS)	Scour Extent						
		Location	Vertical (in)		Horizontal (ft)		Depth (in)	
			min	max	min	max	min	max
Audra	**PQP**							
		abutment	30	40	40	40	114	138
		pier	1	2	2	2	4	5
Beverly	**PQP**							
		abutment	4	6	10.25	10.25	4	12
		pier	NM	NM	NM	NM	NM	NM
Bridge Fork	**AB/QP**	abutment	2.5	4.5	15	15	1	6
Caldwell Run	**DAB**	abutment	12	20	12.5	24	9	30
Clear Fork	**AB/QP**							
		abutment	3	3	3	3	2.5	2.5
		pier	4	4	2	2	1	8
Coon Creek	**PAB**	abutment	1	5	1	5.5	1	10
Cucumber	**DQP**	abutment	NM	NM	NM	NM	NM	NM
Grassy Run	**DQP**	abutment	12	12	12	12	5	18
Laurel Fork	**AB/QP**	abutment	2	4	24	24	2	10
Leatherwood -- Ss	**AB/QP**	abutment	5	12	18	18	?	?
Leatherwood --Sh	**AB/QP**	abutment	7	14	5	5	?	?

Table 16 (continued)

Site	Mode of Scour (MOS)*	Scour Extent						
		Location	Vertical (in)		Horizontal (ft)		Depth (in)	
			min	max	min	max	min	max
Little Sandy	**NA**	abutment	NA	NA	NA	NA	NA	NA
Mish	**PQP**	abutment	2	4	15	NM	NM	NM
Paden Fork	**AB/QP**	abutment	15	15	?	6.7	12	12
Ritter Park / 5th St	**AB/QP**							
		abutment	NA	NA	NA	NA	NA	NA
		pier	8	10	10	12	?	?
Roaring Creek	**PQP**							
		abutment	6	7	2	2.5	3	10
		pier	1.5	2.5	3.5	12.5	1.5	2.5

Geotechnical Scour Numbers

As discussed in the Research Approach, the geotechnical scour number was calculated by two different methods: GSN refers to the geotechnical scour number calculated per NCHRP 717 (2012) and GSN* refers to this parameter calculated by an alternative method. The GSN and GSN* values calculated from the slake durability data are shown in Table 18. The median GSN* value, 1.43×10^{-4}, is approximately 18 times the median GSN of 1.11×10^{-5}. The median coefficient of determination for GSN* (R^{*2}=0.79) is approximately 6 times that of GSN (R^2=0.13). The GSN has the advantage of being linked to the method described in NCHRP report 717 for predicting scour of degradable rock from abrasion. The GSN*, however, appears to better represent the durable rocks found in West Virginia.

Table 18 – Geotechnical Scour Numbers calculated from continuous slake durability test data.

Bridge Site	GSN	R^2	GSN*	R^{*2}	GSN*/GSN	R^{*2}/R^2
Paden Fork	4.42E-05	0.31	7.84E-05	0.45	1.8	1.5
Coon Creek	8.99E-06	0.03	5.32E-04	0.88	59.2	29.3
Roaring Creek	1.04E-05	0.12	1.38E-04	0.80	13.3	6.4
Caldwell Run	1.04E-05	0.17	1.86E-04	0.92	17.9	5.4
Mish	2.89E-06	0.00	2.07E-04	0.82	71.6	189.4
Beverly	1.40E-05	0.15	1.32E-04	0.79	9.4	5.4
Laurel	1.18E-05	0.14	2.06E-04	0.63	17.5	4.5
Bridge Fork	5.60E-06	0.09	1.39E-04	0.71	24.8	7.7
Cucumber	1.79E-05	0.34	1.08E-04	0.87	6.1	2.6
Grassy Run	6.47E-06	0.11	1.17E-04	0.63	18.1	6.0
Ritter Park	7.46E-05	0.05	8.68E-05	0.70	1.2	13.8
Little Sandy	5.74E-05	0.84	8.04E-05	0.94	1.4	1.1
Audra Park	5.55E-06	0.03	2.41E-04	0.85	43.4	25.9
Leatherwood (Sh)	4.69E-05	0.34	8.97E-04	0.81	19.1	2.4
Leatherwood (Ss)	1.70E-05	0.30	1.48E-04	0.73	8.7	2.4
Clear Fork	4.05E-06	0.04	2.25E-04	0.77	55.6	18.6
MEDIAN VALUES	**1.11E-05**	**0.13**	**1.43E-04**	**0.79**	**17.7**	**5.7**

Note: GSN and GSN* denote the values calculated with a zero-intercept (NCHRP 717 method) and without a zero-intercept (alternative method), respectively. Units of GSN and GSN* are feet of equivalent scour depth per foot-pounds per second per square-foot of equivalent stream power. R^2 and R^{*2} denote the coefficient of determination associated with GSN and GSN*, respectively.

The range in GSN values obtained for this project range from 2.98×10^{-6} to 7.46×10^{-5} and a median of 1.15×10^{-5} (units of feet per ft-lb/s/ft^2). These compare to GSN values ranging from 2×10^{-5} to 1×10^{-3} and a median of 2×10^{-4} from NCHRP 717 (2012) expressed in the same units. The significantly lower values and more limited range in GSN from the current study reflect the widespread presence of durable rock across West Virginia, primarily relatively unweathered, quartz-rich Pennsylvanian- and Mississippian-aged sandstone.

The GSN* were evaluated to determine if there was a significant difference between values obtained for core samples and those from surface samples. In Figure 15, each data point represents an individual bridge site, with the average GSN* of two core samples plotted against the GSN* for the one surface sample. This analysis indicated that for most sites there was no significant difference. Exceptions to this general trend are the Roaring Creek (RCK) and Mish Road (MSH) sites, which both plot below the line in Figure 15, indicating a significantly higher GSN* for the surface sample compared to the average for the core samples. In contrast, data for the Laurel Fork (LFK) and Coon Creek (CCK) sites plot above the line in Figure 15, indicating a significantly higher average GSN* for the core surface sample compared to the surface sample.   The largest deviation between core and surface GSN* is Coon Creek, which can best be explained by near-surface fracturing and weathering in bedrock adjacent to the left abutment. The general consistency between core and surface GSN* suggests that the shape differences of rock pieces from the different sample types (see earlier discussion regarding lab methods) did not affect the results from the continuous slake durability tests.

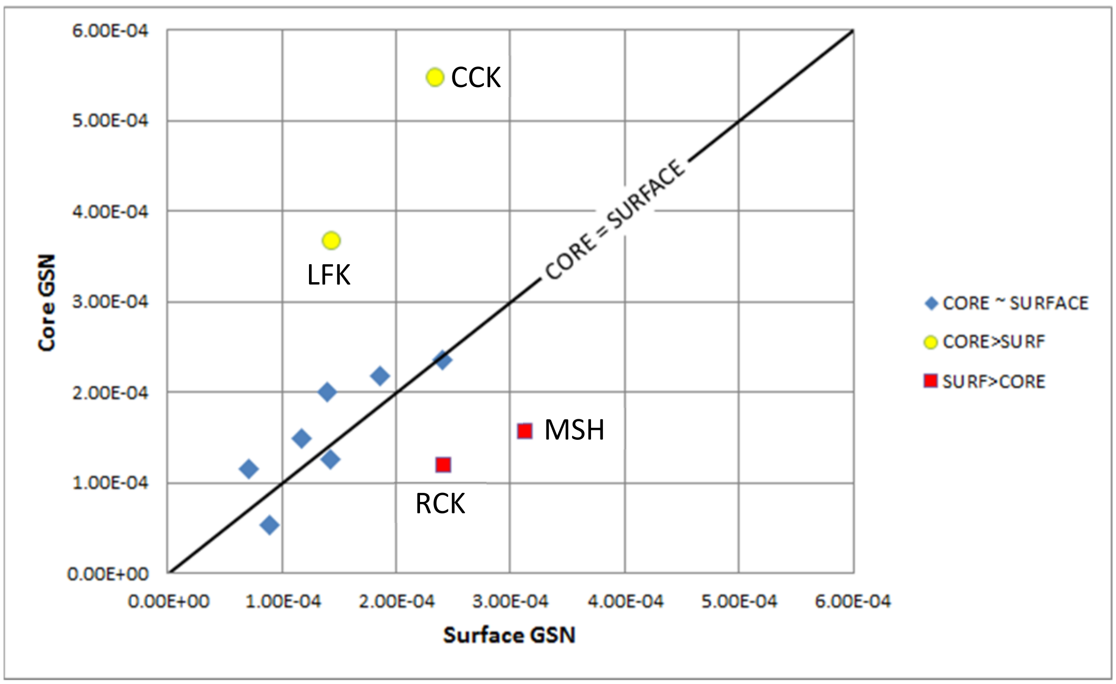

Figure 15 - Comparison of GSN* values obtained from core and surface samples. GSN* is in units of feet per foot-pounds per second per square feet.

<u>Geographic Distribution of Scour mode</u>

Figure 16 shows the distribution of observed mode of scour (MOS) for the 15 bridge sites used in this study.   Five of the six sites where quarrying was judged to be the sole or dominant mode of scour (PQP and DQP, respectively) are located in the northeastern quarter of the state along the Allegheny Front and within the Valley and Ridge Province. In contrast, excepting the Laurel Fork bridge in Upshur County, abrasion is the sub-equal, dominant, or exclusive scour mode in the western and southwestern parts of the state in the Appalachian Plateau (AB/QP, DAB, PAB, respectively).

The map of scour mode in Figure 16 is overlain on a geologic map of West Virginia, in which different rock types appear as specific colors. The distribution of scour mode clearly is not controlled solely by rock type. A more reasonable explanation is that tectonic-related rock deformation characteristic of the Valley and Ridge and Allegheny Front included recrystallization and fracturing. These characteristics make the rocks more durable and susceptible to quarrying. The steeper topography in these areas also provides stream velocities capable of quarrying rock blocks formed by the intersection of fractures and bedding planes.

Since rock scour is a phenomenon that is dependent upon both rock type and the highly variable stream energy conditions encountered at a site, it is not possible to meaningfully depict 'Potential for Scour' in a map. Although rock type can be readily presented on a map, stream energy conditions can vary drastically within even a small watershed. A relatively flat, grassy area will experience extremely low overland flow runoff velocities, and yet it may be in the immediate proximity to steeply inclined channel sections where runoff velocity is high, and stream power accumulates quickly. Therefore, a statewide, or even regional, combined depiction of stream power and rock type is not feasible. Where scour potential needs to be determined, a detailed field investigation of rock conditions should be initiated to look for signs of past rock scour, and site-specific assessment of hydrologic and hydraulic conditions should be conducted to assess the relative quantity of stream power available for scour.

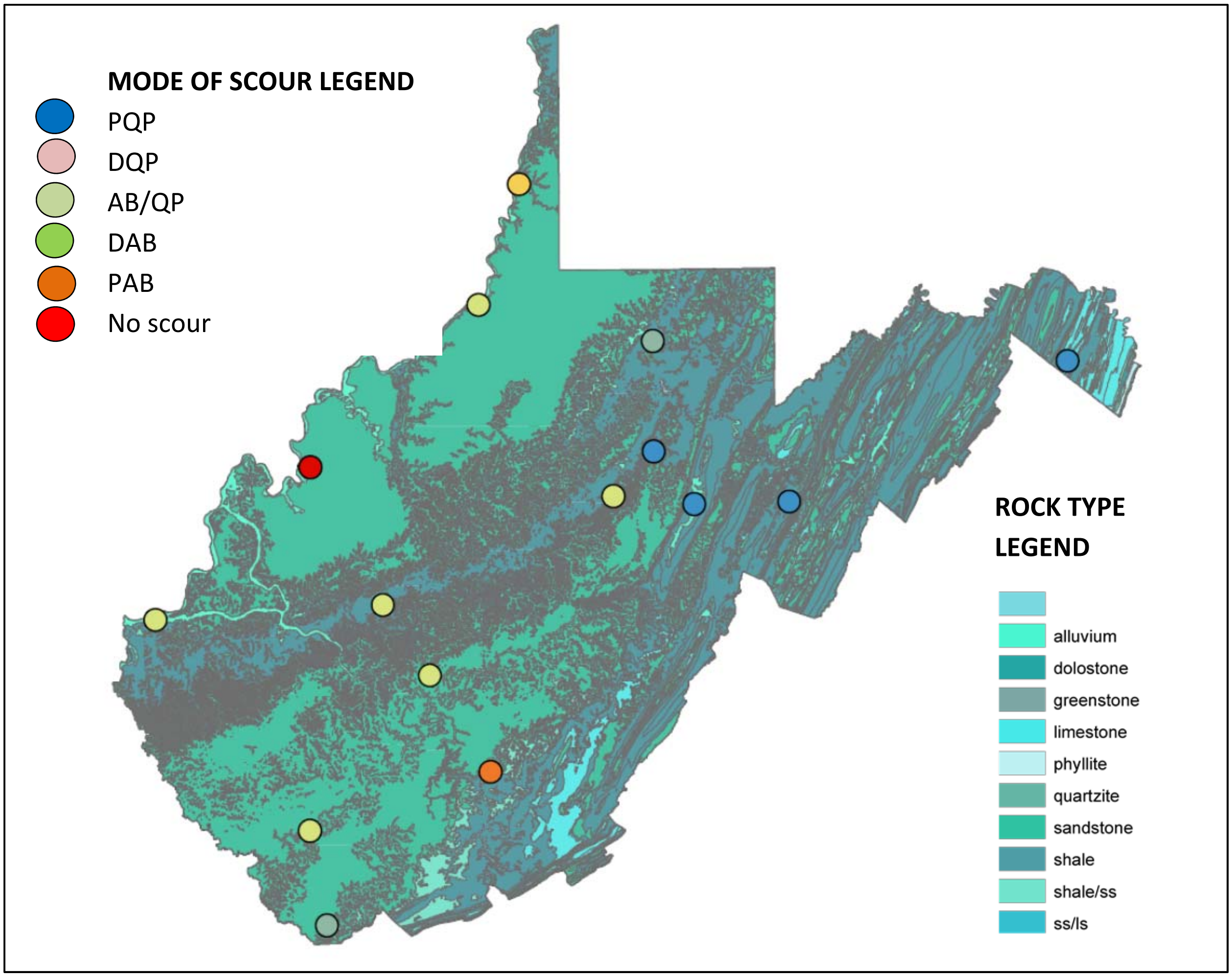

Figure 16 – Geographic distribution of scour mode overlain on a simplified geologic map of West Virginia. Circles indicate location of bridge sites. Color of circles indicates mode of scour.

<u>Geographic Distribution of GSN</u>

Figure 17 shows the distribution of GSN* values for the 15 bridge sites used in this study. No clear pattern in geographic or geologic distribution of GSN* is apparent, although the three smallest GSN* values were measured in rocks from Permian and uppermost-Pennsylvanian non-marine strata located adjacent to the Ohio River in the far western part of the state. For the most part, however, the presence of rocks with either low or high GSN* values (i.e., more or less resistant to abrasion) cannot readily be explained in terms of regional distribution of rock type, age of strata, or geologic province. Even if the GSN* hinted at some geographic or geologic control, it should be emphasized that GSN (or GSN*) is not appropriate for predicting the magnitude of vertical scour at bridge sites where abrasion is the mode of scour. As seen in Figure 16, only one site of the 15 in this study is characterized by pure abrasion.

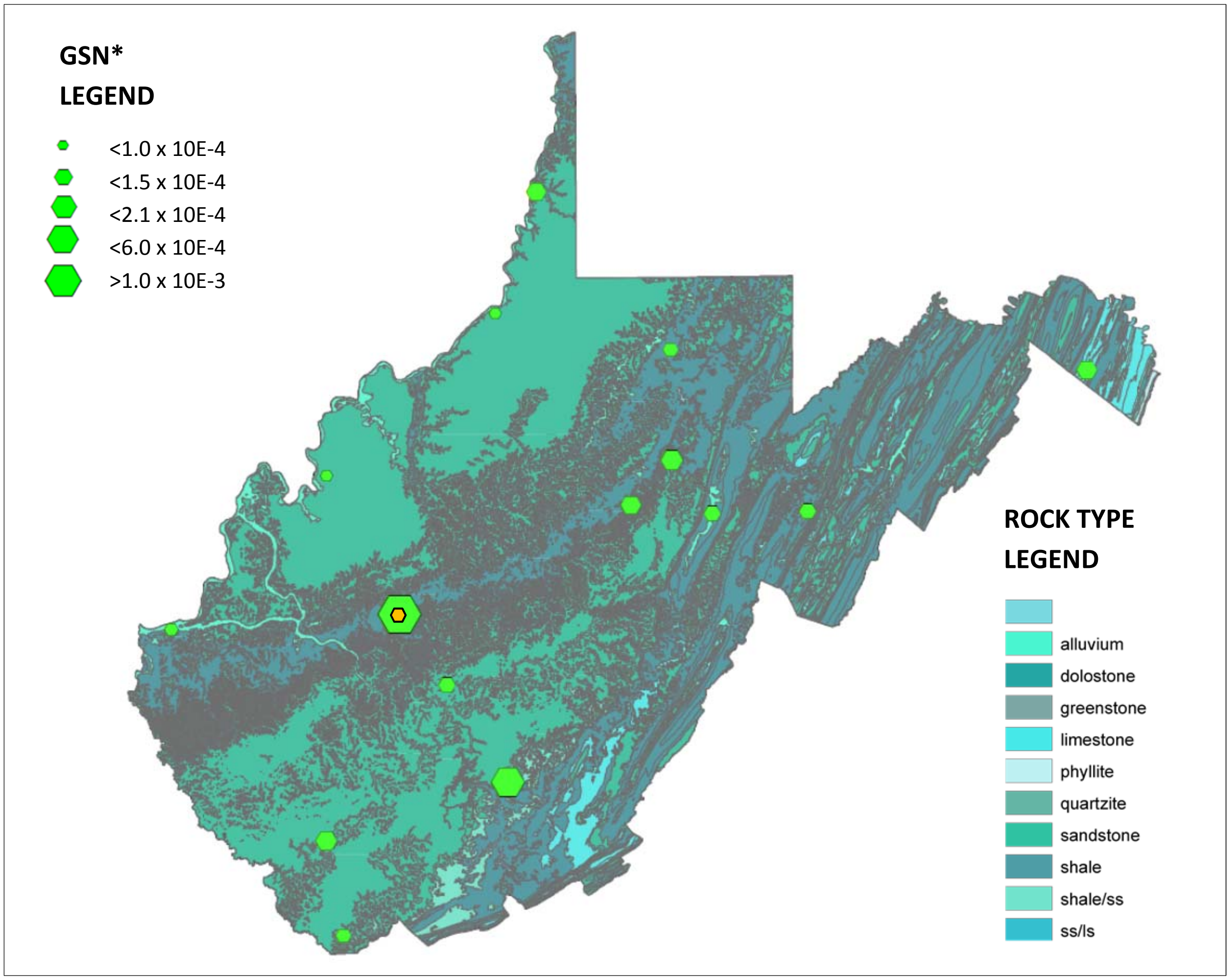

Figure 17 - Geographic distribution of GSN* overlain on geologic map of state. Ranges in GSN* are shown in units of feet per foot-pounds per second per square feet. The two symbols superimposed for the Leatherwood Bridge correspond to different GSN* for shale and sandstone samples.

Relationship between GSN and Mode of Scour

Figure 18 illustrates a plot of median GSN* for each mode of scour. There is a general trend toward higher GSN* with increasing dominance of abrasion. This result is not unexpected: after all, GSN and GSN* both measure abrasion resistance, and GSN was designed specifically as a predictor of scour at sites affected by abrasion. Figure 18 indeed suggests that sites with higher GSN* (i.e., less resistant to abrasion) are more likely to be affected by this type of scour. However, the data also reveals a notable discrepancy to this general trend: the median GSN is higher for sites where pure quarrying (PQP) is the mode of scour than for sites where quarrying is accompanied by an equal degree of abrasion (AB/QP) or lesser degree of abrasion (DQP). This apparent discrepancy can be explained by recognition that rock scour results from rock-water interaction, meaning that both rock and hydraulic characteristics must be considered (NCHRP 717, 2012). For example, if fractures are closely spaced, the size of rock blocks is relatively small, and the stream energy necessary for quarrying to occur is small compared to sites where fractures are more widely spaced and blocks are larger. If rocks with a smaller, but still significant, abrasion resistance (higher relative GSN*) happen to have a high fracture density, quarrying could occur at relatively low stream velocities and not necessarily be accompanied by abrasion. If on the other hand, more durable rocks (lower relative GSN*) happen to have a low fracture density, the significant velocities necessary for quarrying large blocks may be attained only on an irregular basis but stream energy would also likely be sufficient to perform abrasion.

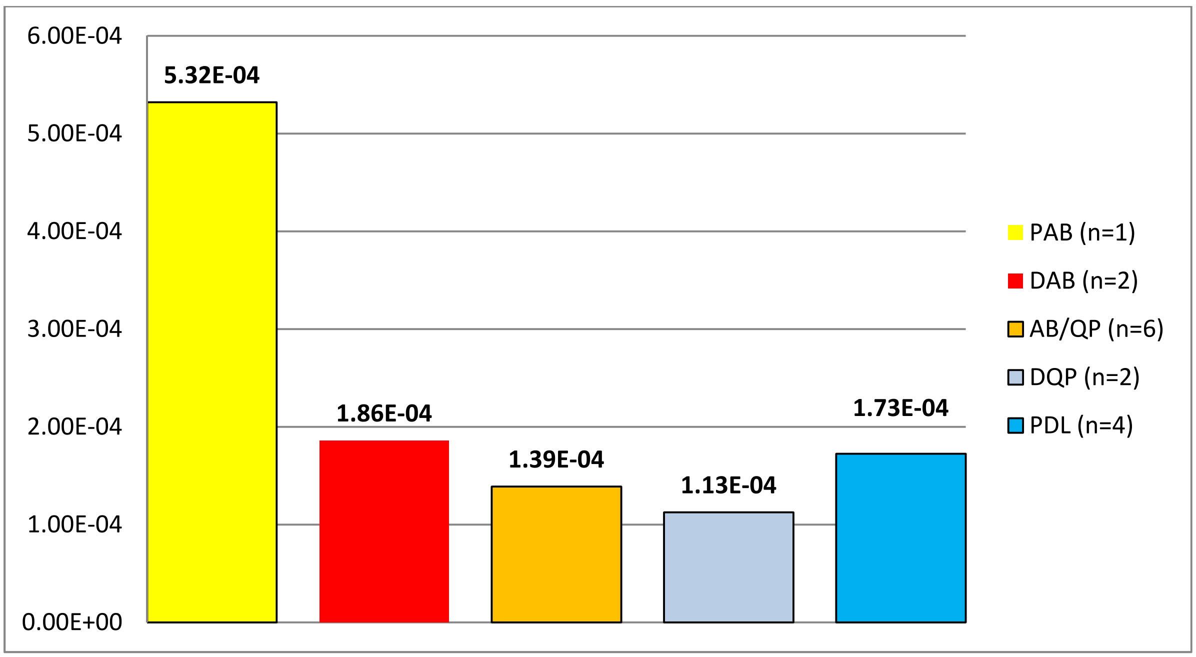

Figure 18 -- Comparison of median GSN* according to mode of scour. (GSN* is in units of feet per foot-pounds per second per square feet).

Stream Power and Scour Number

The flow velocity and depth values required for computing Stream Power were derived using watershed and hydraulic models, detailed summaries of which are provided in Appendix A for each site. Additional discussion of hydraulic and hydrologic modeling and results is found in Appendix A, rather than in the body text, because of length limitations. Using the 90% precipitation depth confidence interval for the 12 ungaged watersheds, and 95% peak flow confidence interval for the 3 gaged watersheds, Cumulative Daily Effective Stream Power was computed for the baseline flow condition, and both low and high range confidence interval values.

Since additional uncertainty exists within the chain of hydrologic and hydraulic models besides precipitation depths (e.g., within GIS data characterizing soil types within watershed boundaries, in the computation of catchment time of concentration, in the Manning's roughness coefficient estimates made for channels, etc.), the actual Cumulative Daily Effective Stream Power encountered at each project site may differ considerably from the estimates provided in Table 19. Since these estimates are probabilistically-derived, deviation of actual flow conditions from those that might be expected on an average basis may be particularly pronounced for relatively young sites, where 'reversion to the mean' has not yet occurred. Additionally, since eddies, vortices, and other turbulent effects likely arise along abutment surfaces during the range of flows encountered during unsteady conditions, a generalized assessment of average energy may differ considerably from the energy applied at any single location over time. Therefore, the relative confidence interval size reported in Table 19 is merely a relative indication of the impact of uncertainty built into precipitation depth and peak flow estimates, and not as an absolute indication of the range of Cumulative Daily Effective Stream Powers occurring at each site.

Scour number, derived from the scour depth identified during field inspection and estimated Cumulative Daily Effective Stream Power, is summarized in Table 19, along with the confidence interval range that is implied by the range of Cumulative Daily Effective Stream Powers.

Table 19 – Cumulative daily effective stream power at each bridge site since it was constructed.

Site	**Cumulative Daily Effective Stream Power Since Construction (ft·lb/s/ft^2)**	**Size of confidence interval, as % of baseline value shown**
1 - Leatherwood	3.7	14%
2 - Fifth Street	33.2	25%
2 - Fifth Street (Pier)	95.9	25%
3 - Little Sandy	3.0	70%
4 - Grassy Run	0.5	120%
5 - Caldwell Run	11.5	64%
6 - Paden Fork	4.6	96%
7 - Audra Park	159.0	66%
7 - Audra Park (Pier)	459.4	66%
8 - Laurel Fork	0.5	67%
9 - Roaring Creek	184.6	80%
9 - Roaring Creek (Pier)	533.5	80%
10 - Beverly	15.2	39%
10 - Beverly (Pier)	44.0	39%
11 - Coon Creek	53.6	69%
12 - Bridge Fork	5.9	36%
13 - Cucumber	3.1	60%
14 - Clear Fork	323.1	73%
14 - Clear Fork (Pier)	933.7	73%
15 - Mish Road	6.9	27%

Table 20 – Scour number (vertical scour depth / cumulative daily effective stream power) at each site.

Site	**Scour Number** (ft of vertical scour / daily ft·lb/s/ft^2)	**Confidence Interval Range (ft of vertical scour / daily ft·lb/s/ft^2)**
1 - Leatherwood - Sandst.	1.9E-01	1.8E-01 - 2.1E-01
1 - Leatherwood - Shale	2.4E-01	2.3E-01 - 2.6E-01
2 - Fifth Street	N/A	N/A
2 - Fifth Street (Pier)	7.8E-03	5.2E-03 – Note A
3 - Little Sandy	N/A	N/A
4 - Grassy Run	2.0E+00	1.1E+00 - 3.1E+00
5 - Caldwell Run	1.2E-01	8.3E-02 – Note A
6 - Paden Fork	2.7E-01	1.4E-01 - 2.9E-01
7 - Audra Park	1.8E-02	1.3E-02 - 2.6E-02
7 - Audra Park (Pier)	2.7E-04	2.0E-04 - 3.8E-04
8 - Laurel Fork	5.5E-01	3.5E-01 - 6.2E-01
9 - Roaring Creek	2.9E-03	1.9E-03 - 4.0E-03
9 - Roaring Creek (Pier)	3.1E-04	2.0E-04 - 4.2E-04
10 - Beverly	2.7E-02	2.3E-02 - 3.4E-02
10 - Beverly (Pier)	N/A	N/A
11 - Coon Creek	4.7E-03	3.0E-03 - 5.5E-03
12 - Bridge Fork	5.0E-02	4.0E-02 - 5.6E-02
13 - Cucumber	N/A	N/A
14 - Clear Fork	7.7E-04	5.5E-04 - 1.1E-03
14 - Clear Fork (Pier)	3.6E-04	2.5E-04 - 5.3E-04
15 - Mish Road	3.6E-02	2.7E-02 – Note A

Note A – At the three sites noted, the upper limit of the confidence interval is not meaningful. This is due to the 2-yr flow screening technique implicit in determining "effective" stream power, and a particularly wide range of flow rates at these locations between the 2-yr and 500-yr storms.

In the absence of scour modes besides abrasion , and if Cumulative Excess Stream Power could be definitively calculated, rather than estimated probabilistically, then one might expect a relatively linear relationship between Cumulative Excess Stream Power and scour depth. Instead, as shown in Figure 19, there appears to be additional variance among the project data that such a model does not account for. As outlined previously, the likely contributors to this variance include additional scour modes (i.e., quarrying), and the multitude of uncertainties that accumulate along the path from precipitation depth – duration – frequency estimates at a site to eventual estimation of Cumulative Daily Excess Stream Power by way of hydrologic and hydraulic models.

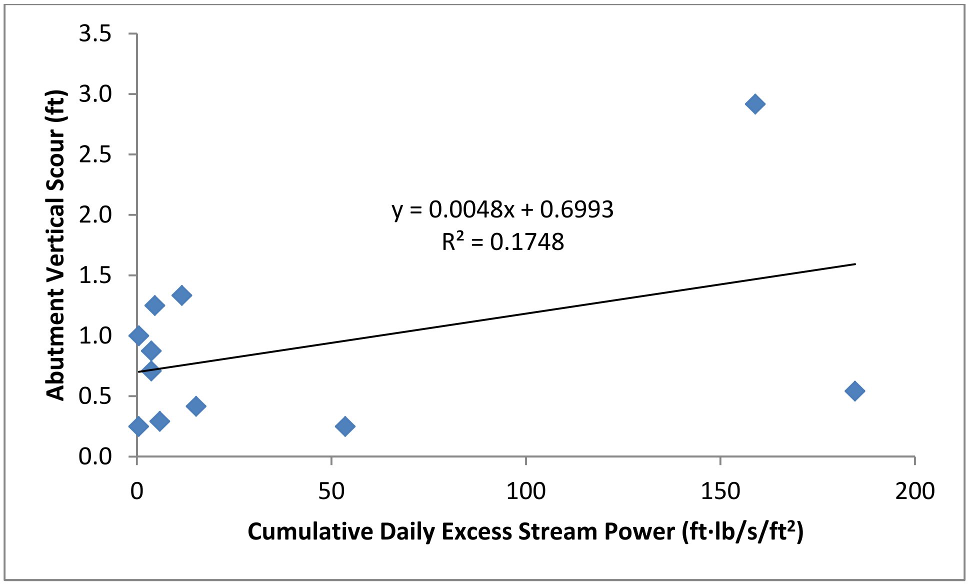

Figure 19 - Vertical scour (abutment) versus cumulative daily effective stream power for each site.

Comparison of Geotechnical and Empirical Scour Numbers

A comparison of Geotechnical Scour Number (GSN) and Scour Number for each of the project sites is provided in Table 21.

Also included is a computed ratio of Scour Number / GSN, which, if GSN is to be a suitable predictive surrogate for Scour Number, should be near a value of 1.0. If the Scour Numbers and GSNs at each project site were equal, then a linear response with a slope of 1.0 would be observed. Such was not observed to be the case, as shown in Figure 20. A linear regression on the data indicates a very poor fit.

In a ratio of Scour Number / GSN, a value less than 1.0 would mean that the equivalent scour depth implied by GSN for a given cumulative effective stream power is too large compared to the actual scour depth observed for the same cumulative effective stream power. A ratio value greater than 1.0, on the other hand, would mean GSN-derived equivalent scour depth is too small compared to the actual scour depth observed. With the exception of the Leatherwood Bridge, comparisons are also made for the Scour Number and the Geotechnical Scour Number calculated by the alternative method (GSN*). The GSN* agrees much better with the Scour Number than does GSN (median ratio of 189). Like the GSN, using the GSN* to predict actual scour would underestimate the amount of scour at all sites except the Leatherwood Bridge, but the prediction would be much better than using the GSN, and in the case of six bridges would be off by less than two orders of magnitude. As with GSN, Figure 20 shows a linear regression on the GSN* data indicating a very poor fit and all of the data excepting Leatherwood falling above the green reference line equal to a ratio of 1.0.

Although the results from using the GSN* may appear more promising than for GSN, there are some important qualifications that apply to these results. First, the GSN is part of the NCHRP 717 methodology for predicting scour by pure abrasion (PAB). As a modified version of the Geotechnical Scour Number, this should also be true of GSN*. As such, it is encouraging that three of the bridges where the ratio of Scour Number to GSN* is less than 100 include one bridge characterized by pure abrasion (Coon Creek) and three others characterized by sub-equal abrasion and quarrying (Ritter Park/5th Street, Leatherwood shale, and Clear Fork). However, two of the six sites where the ratio is less than 100 are bridges where the mode of scour was determined to be pure quarrying (Audra Park and Roaring Creek). Because there is no known theoretical basis for claiming that GSN* would apply to quarrying, it would be irresponsible at this time to imply that the current results suggest that GSN* could be useful as a reliable predictor of scour at such sites.

Table 21 shows that the ratio is much greater than 1.0 for each project site (median value of over 4,600), indicating that the equivalent scour depth implied by GSN is much less than the actual depth that was observed, in most cases by orders of magnitude. Incorporating GSN into prediction in such case would lead to non-conservative results (i.e., more scour would occur than expected). The same phenomenon can be viewed graphically in Figure 20 where all of the data points excepting Leatherwood fall above the green reference line equal to a ratio of 1.0.

Table 20 and Figure 20 also show comparisons of the Scour Number and the Geotechnical Scour Number calculated by the alternative method (GSN*). The GSN* agrees much better with the Scour Number than does GSN (median ratio of 189). Like the GSN, using the GSN* to predict actual scour would underestimate the amount of scour at all sites except the Leatherwood Bridge, but the prediction would be much better than using the GSN, and in the case of six bridges would be off by less than two orders of magnitude. As with GSN, Figure 20 shows a linear regression on the GSN* data indicating a very poor fit and all of the data excepting Leatherwood falling above the green reference line equal to a ratio of 1.0.

Although the results from using the GSN* may appear more promising than for GSN, there are some important qualifications that apply to these results. First, the GSN is part of the NCHRP 717 methodology for predicting scour by pure abrasion (PAB). As a modified version of the Geotechnical Scour Number, this should also be true of GSN*. As such, it is encouraging that three of the bridges where the ratio of Scour Number to GSN* is less than 100 include one bridge characterized by pure abrasion (Coon Creek) and three others characterized by sub-equal abrasion and quarrying (Ritter Park/5th Street, Leatherwood shale, and Clear Fork). However, two of the six sites where the ratio is less than 100 are bridges where the mode of scour was determined to be pure quarrying (Audra Park and Roaring Creek). Because there is no known theoretical basis for claiming that GSN* would apply to quarrying, it would be irresponsible at this time to imply that the current results suggest that GSN* could be useful as a reliable predictor of scour at such sites.

Table 21 – Rock type, scour mode, scour number, and scour number to GSN ratios.

Site	Dominant Rock Type	Scour Mode	Scour Number (ft/daily ft·lb/s/ft^2)	GSN (ft / daily ft·lb/s^2/ft^2)	Scour Number / GSN	GSN* (ft / daily ft·lb/s^2/ft^2)	Scour Number / GSN*
Audra	Sandstone	PQP	1.80E-02	5.55E-06	3,243	2.41E-04	75
Beverly	Siltstone	PQP	2.70E-02	1.40E-05	1,929	1.32E-04	205
Bridge Fork	Sandstone	AB/QP	5.00E-02	5.60E-06	8,929	1.39E-04	360
Caldwell Run	Shale	DAB	1.20E-01	5.60E-06	21,429	1.86E-04	645
Clear Fork	Sandstone	AB/QP	7.70E-04	4.05E-06	190	2.25E-04	3.4
Coon Creek	Sandstone (weathered)	PAB	4.70E-03	8.99E-06	523	5.32E-04	8.8
Grassy Run	Sandstone	DQP	2.00E+00	1.79E-05	111,732	1.08E-04	18,467
Laurel Fork	Sandstone	AB/QP	5.50E-01	6.47E-06	85,008	1.17E-04	4,701
Leatherwood	Sandstone (Shaley)	AB/QP	1.90E-01	1.18E-05	16,102	2.06E-04	922
Leatherwood	Shale	AB/QP	3.35E-05	4.69E-05	0.7	8.97E-04	0.04
Mish	Limestone	PQP	3.60E-02	2.89E-06	12,457	2.07E-04	174
Paden Fork	Sandstone	AB/QP	2.70E-01	4.42E-05	6,109	7.84E-05	3,444
Ritter Park / 5th St	Sandstone	AB/QP	7.80E-03	7.46E-05	105	8.68E-05	90
Roaring Creek	Sandstone	PQP	2.90E-03	1.04E-05	279	1.38E-04	21
		Median Values	**3.15E-02**	**9.70E-06**	**4,676**	**1.63E-04**	**189**

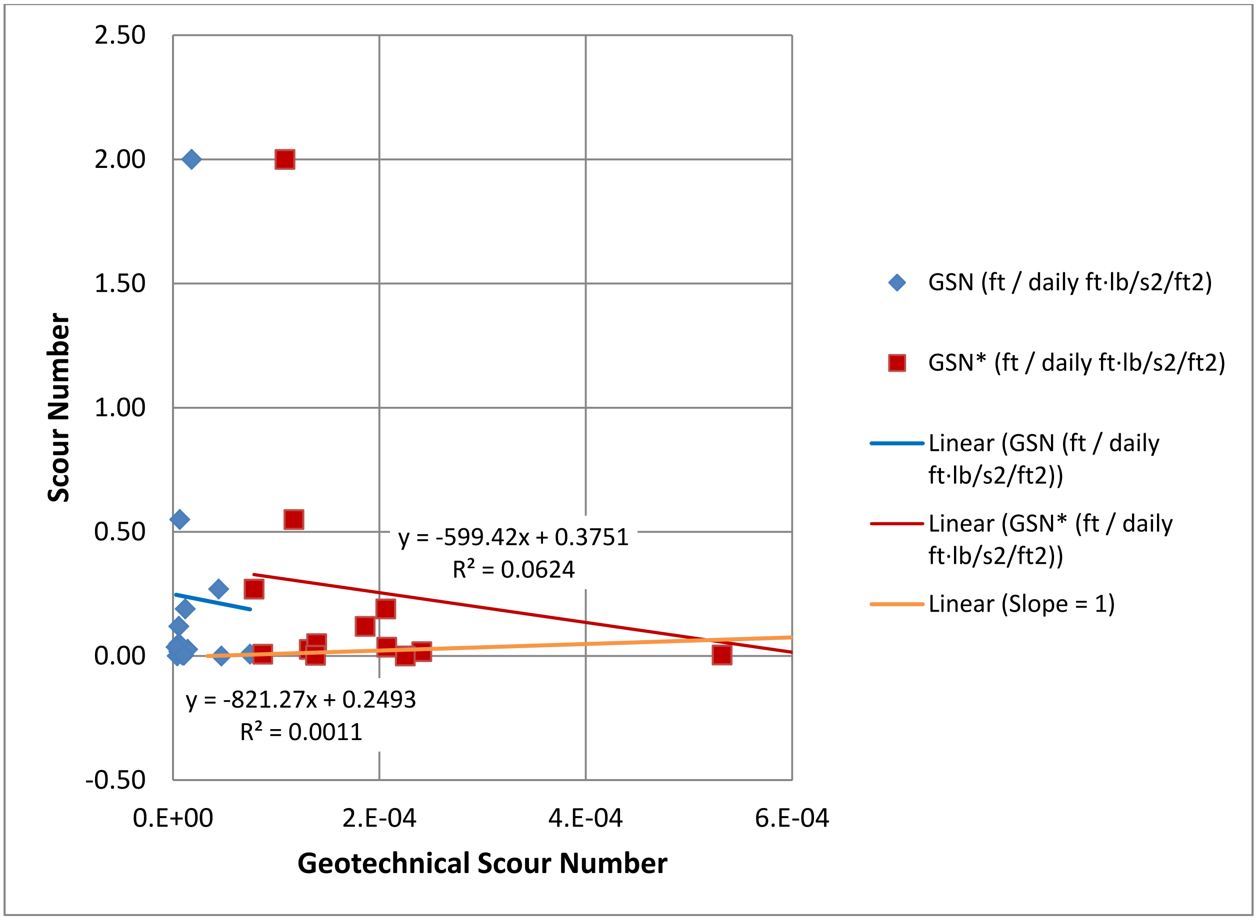

Figure 20 - Project-wide comparison of Geotechnical Scour Number (GSN) and GSN* vs. Scour Number.

Conclusions

The following are the primary conclusions of RP-273, Criteria for Predicting Scour of Erodible Rock in West Virginia:

Evaluation of the feasibility of using Geotechnical Scour Number (GSN) as a predictive tool for the rock types and hydraulic conditions found in West Virginia.

The GSN centered methods described in NCHRP report 717 (2012) are intended for predicting scour only at sites affected by abrasion, and not quarrying. When the GSN based approach was selected by WVDOH for evaluation, the distribution of scour modes at bridge sites across the state was unknown. The influence of quarrying was observed in field assessments as a contributor to scour at 14 of 15 project sites. As a consequence, minimal correlation was observed between Scour Number and GSN. For this reason, the results from this project do not support the generalized implementation of the NCHRP 717 (2012) approach in West Virginia. The differences between GSN and Scour Number observed in this project are likely attributable in large part to multi-modal scour. Since GSN consistently under-predicted Scour Number, that would mean under-prediction of scour depths (or over-prediction of service life remaining) if the predictive application of GSN was implemented.

The Best Management Practices (BMP) document associated with this project seeks to overcome these challenges by presenting an approach to classify rock by scour mode and classify sites by stream power, and includes guidance and methodology for assessing mode(s) of scour at bridge sites in West Virginia and recommended actions to be taken based on the results.

An additional benefit of having applied the GSN based analyses at the 15 project sites is that having a dataset of Scour Numbers, representing a wide range of geographic, hydraulic, and geologic conditions, may be useful to WVDOH and others in interpreting the cause of hydraulic scour of rock at other sites. Should a Cumulative Effective Stream Power and corresponding Scour Number be determined for an existing bridge site where scour has been observed, future performance could be estimating using a probability weighting approach. Thus, the Scour Number data generated in this project will provide a context for any attempt to predict bridge service life through with estimates of allowable additional scour depth and Average Annual Cumulative Effective Stream Power.

Development and implementation of an approach that enables estimation of Stream Power at sites where neither stream gage data nor historical precipitation data is available.

12 of 15 project sites from the project lacked stream gage data, and it is anticipated that the majority of WVDOH's bridge inventory will similarly lack stream gage data, or even direct historical precipitation data. The capacity to determine Stream Power is requisite to computing Scour Number, and thus to applying predictive techniques that depend on projecting future energy conditions at a site.

The Best Management Practices document associated with this project includes detailed instructions on how to estimate stream power based on watershed models, probabilistic representations of precipitation, and the computational steps required to determine average annual cumulative effective stream power. Process validation should be undertaken prior to the widespread implementation of this approach, including an effort to field-validate watershed models for the unique geographic conditions found in West Virginia. Further investigation is also recommended to characterize the range of relationships that can exist between generalized approach flow conditions and specific hydraulic conditions that can arise at abutments as a function of abutment geometry, orientation, and location in the flow field.

Works Cited

Baedecker, P.A., Reddy, M. M., Reimann, K. J., Sciammarella, C. A. (1992) *Effects of acidic deposition on the erosion of carbonate stone – experimental results from the U.S. National Acid Precipitation Asessment Program (NAPAP)*, Atmospheric Environment Part B Urban Atmosphere, 26(2), 147-158.

Baedecker, P.A., and Reddy, M. M. (1993). *The Erosion of Carbonate Stone by Acid Rain: Laboratory and Field Investigations*, J. Chem. Educ., 70(2), p. 104.

Brokaw, M. (2008). Ohio Department of Transportation, Scour Plan of Action – memo to Ohio Bridge Inspectors, Program Managers, Team Leaders, and Hydraulic Engineers

Commonwealth of Pennsylvania, Department of Transportation (2009). *Bridge Management System 2 (BMS2) Coding Manual*, Publication 100A, July 2009 Edition.

Commonwealth of Pennsylvania, Department of Transportation (undated presentation, posted online), Monitoring Scour Critical Bridges During Floods For Local Bridge Owners.

Dickenson, S. E., and Baillie, M. W. (1999), Predicting Scour in Weak Rock of the Oregon Coast Range: unpublished research report, Department of Civil, Construction, and Environmental Engineering, Oregon State University, Corvallis, OR, Final Report SPR 382, Oregon Department of Transportation and Report No. FHWA-OR-RD-00-04.

Federal Highways Administration, NHI 01-001, Hydraulic Engineering Circular No. 18. (2001). Evaluating Scour at Bridges, Fourth Edition.

Federal Highways Administration, HIF 12-004. Hydraulic Engineering Circular No. 20. (2012). Stream Stability at Highway Structures, Fourth Edition.

Federal Highways Administration, NHI 01-003, Hydraulic Engineering Circular No. 23, (2001). Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance, Second Edition.

Flynn, K. M., Kirby, W. H., & Hummel, P. R. (2006). *User's manual for program PeakFQ.* U.S. Geological Survey Techniques and Methods Book 4.

Froehlich, D. C., Hopkins, T. C., and Beckham, T. L. (1999) Preliminary Assessment of Local Scour Potential at Bridge Piers Founded on Rock, in Richardson, E. V., and Lagasse, P. F. eds., Stream Stability and Scour at Highway Bridges: Water Resources Engineering Division, ASCE, Reston, VA, 976–980.

Garbrecht, J., & Martz, L. W. (1999). *TOPAZ: An Automated Digital Landscape Analysis Tool for Topographic Evaluation, Drainage Identification, Watershed Segmentation, and Subcatchment Parameterization.* U.S. Department of Agriculture, Agricultural Research Service.

Geological Survey Office of Water Data Coordination. (1982). *Bulletin #17B of the Hydrology Subcommittee, Guidelines for Determining Flood Flow Frequency.* Interagency Advisory Committee on Water Data.

Holnbeck, S.R, and Parrett, C. (1987) *Method for Rapid Estimation of Scour at Highway Bridges Based on Limited Site Data*, U.S. Geological Survey Water Resources Investigation 96-4310.

Hopkins, T. C., and Beckham, T. L. (1999) *Correlation of Rock Quality Designation and Rock Scour Around Bridge Piers and Abutments Founded on Rock,* Kentucky Transportation Center, College of Engineering, University of Kentucky, Report No. KTC-99-57.

Keaton, J. R., and Mishra, S. K., (2010a) *Modified Slake Durability Test for Erodible Rock Material*, in Burns, S. E., Bhatia, S. K., Avila, C. M. C., and Hunt, B. E., eds., Scour and Erosion: Proceedings of the Fifth International Conference on Scour and Erosion, November 7–10, 2010, San Francisco, American Society of Civil Engineers Geotechnical Special Publication No. 210, 743–748.

Keaton, J. R., Mishra, S. K., and Clopper, P. E. (2010b), *Scour at Bridge Foundations on Rock: Overview of NCHRP Project No. 24-29*, in Burns, S. E., Bhatia, S. K., Avila, C. M. C., and Hunt, B. E., eds., Scour and Erosion: Proceedings of the Fifth International Conference on Scour and Erosion, November 7–10, 2010, San Francisco, American Society of Civil Engineers Geotechnical Special Publication No. 210, 749–756.

McCutcheon, S. C. (2003). *Hydrologic Evaluation of the Curve Number Method for Forest Management in West Virginia.* Report Prepared for the West Virginia Division of Forestry, Charleston, WV.

Mishra, S. K., Keaton, J. R., Clopper, P. E., and Lagasse, P. F. (2010) *Hydraulic Loading for Bridges Founded on Erodible Rock*, in Burns, S. E., Bhatia, S. K., Avila, C. M. C., and Hunt, B. E., eds., Scour and Erosion: Proceedings of the Fifth International Conference on Scour and Erosion, November 7–10, 2010, San Francisco, American Society of Civil Engineers Geotechnical Special Publication No. 210, 734–742.

Multi-Resolution Land Characteristics Consortium, (2012). *National Land Cover Database 2006 (NLCD2006) - Product Legend*. Retrieved December 31, 2012, from http://www.mrlc.gov/nlcd06_leg.php

National Cooperative Highways Research Program. (2012). *Scour at Bridge Foundations on Rock - NCHRP Report 717.*

National Oceanic and Atmospheric Administration (NOAA). (2013). *Precipitation Frequency Data Server*. Retrieved 2013, from http://dipper.nws.noaa.gov/hdsc/pfds/

National Resource Conservation Service (NRCS). (2010). Part 630 Hydrology, National Engineering Handbook. United States Department of Agriculture, Natural Resources Conservation Service.

National Resource Conservation Service (NRCS). (2013). *Natural Resources Conservation Service Soil Data Mart*. Retrieved 2013, from http://soildatamart.nrcs.usda.gov/

National Resource Conservation Service (NRCS), Conservation Engineering Division. (2009). *Small Watershed Hydrology, WinTR-55 User Guide.*

Ohio Department of Transportation (2010). *Manual of Bridge Inspection.*

Ohio Department of Transportation, Office of Structural Engineering (2012). *Bridge Inventory Coding Guide.*

Ponce, V.M. and Hawkins, R. H. (1996). *Runoff Curve Number: Has It Reached Maturity?*, Journal of Hydrologic Engineering, Vol. 19.

State of Maryland, State Highway Administration (SHA). (2011). *Office of Structures Manual on Hydrologic and Hydraulic Design, Chapter 11 – Evaluating Scour at Bridges.*

US Army Corps of Engineers. (1998). HEC-1 Flood Hydrograph Package User's Manual.

US Army Corps of Engineers. (2008). HEC-RAS River Analysis System, Hydraulic Reverence Manual, Version 4.0, CPD-69.

United States Department of Agriculture (USDA), Soil Conservation Service. (2003). Techical Note Revised Hydrology No. N4 – Time of Consentration, Chester Pennsylvania.

United States Department of Agraculture (USDA), Natural Resource Conservation Service. (1986). Technical Resease – 55, *Urban Hydrology for Small Watersheds*.

United States Geological Service (USGS). (2006). *Estimating Magnitude and Frequency of Floods Using the PeakFQ Program*. FS-2006-3143.

United States Geological Service (USGS). (2013). Retrieved 2013, from National Water information System Mapper: http://maps.waterdata.usgs.gov/mapper/index.html

United States Geological Service (USGS). (2013). *National Map Viewer*. Retrieved 2013, from http://viewer.nationalmap.gov/viewer/

United States Geological Service (USGS), SIR 2010-5033. (2010). *Estimation of Flood-Frequency Discharges for Rural, Unregulated Streams in West Virginia (Scientific Investigations Report 2010-5033).* U.S. Geological Survey.

Appendix A – Project Site Hydrologic and Hydraulic Information

In this appendix, information is provided for each of the fifteen project sites with respect to their watershed characteristics, methods utilized for characterizing flow rates over time at each site, development of relationships that describe flow velocity and depth, and computation of cumulative effective stream power.

Appendix A Figures

1 – Leatherwood Road

Watershed characteristics

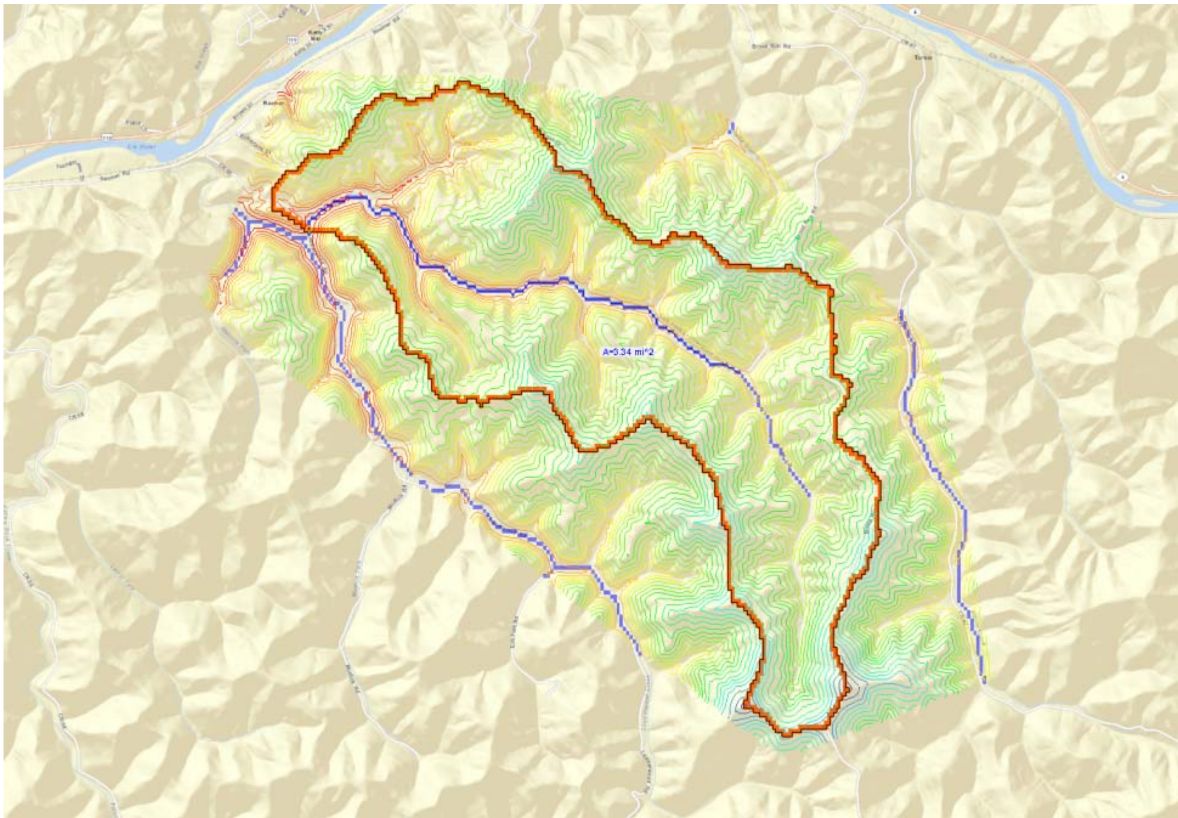

Figure 1- Image of delineated Leatherwood watershed.

Several relevant hydrologic parameters for the Leatherwood Road watershed are provided below in Table 1.

Table 1 - Watershed characteristics for Leatherwood Road.

Parameter	**Value**
Location	Lat: 38.468833 Long:-81.365000
Region – USGS SIR2010-5033	Western Plateaus
Basin Area (mi^2)	3.34
Average basin elevation (ft)	3202
Average basin overland slope (%)	97.5
Maximum flow distance (ft)	24565
Slope along maximum flow distance (%)	10.5
Shape factor (basin length / basin width)	3.44
Sinuosity factor of stream (max. stream length / basin length)	0.80
Runoff Curve Number	65.3
Time of Concentration (hr.)	1.049

A summary of the parameters utilized to compute the weighted watershed Curve Number is provided in Table 2 below, including Natural Resources Conservation Service Soil Survey Geographic Database (SSURGO) derived hydrologic soil group, National Land Cover Database 2006 derived land cover classification, corresponding Curve Number, and sub-element area.

Table 2 – Soil type, land cover, and curve number characteristics for Leatherwood Road.

Hydrologic Soil Group	**Land Cover Description**	**CN**	**Area (mi^2)**
B	Grassland/Herbaceous	58	0.003
B	Developed Open Space	61	0.02
B	Developed Low Intensity	68	0.002
B	Deciduous Forest	55	1.145
C	Deciduous Forest	70	1.862
B	Evergreen Forest	55	0.025
C	Developed Open Space	74	0.049
D	Deciduous Forest	77	0.176
D	Developed Open Space	80	0.046
C	Evergreen Forest	70	0.015
Weighted CN = 65.2			

Flow synthesis

Stream Gage Data. In Table 3, characteristics of the stream gage stations in proximity to the Leatherwood Road project site are summarized, including period of record and data coverage within that period, location, distance from the project area, elevation, drainage area upstream of the gage, and whether the available data consists of Daily Discharge (DD) or Annual Peak (AP).

The Left Fork of Leatherwood Creek (the water body which the Leatherwood Road bridge crosses) is an ungauged stream. Additionally, none of the gauged streams in the nearby area are well-suited for flow extrapolation to the Leatherwood Road project site, because of significant dissimilarities in watershed drainage area, time period coverage that does not overlap with the period since the bridge was constructed, and excessive distances from the project site location. Therefore, flow synthesis at the Leatherwood Road project site utilizes two methods (as outlined below): the applicable USGS peak flow regression equations, and creation of a watershed model that utilizes probabilistic rainfall data.

Table 3 – Stream gage stations near Leatherwood Road.

Project / Stream Gage Location	USGS Station ID	Period	Covg. (%)	Lat.	Long.	Dist. (mi.)	Elev. (ft.)	Drainage Area (mi^2)	Type
Leatherwood Rd – Left Fork of Leatherwood Creek	-	*1987-Present*	-	*38.4688*	*-81.365*	-	*675*	*3.34*	-
Elk River at Queen Shoals, WV	03197000	1928 - Present	99	38.471	-81.284	4.4	603.5	1145	DD
Elk Twomile C Trib NR Charleston, WV	03197900	1964-1974	100	38.354	-81.523	11.7	796	0.49	AP
Pocatalico River at Sissonville, WV	03201000	1908-1998	60	38.526	-81.631	14.9	594.6	238	DD
Ashleycamp Run Near Lefthand, WV	03197150	1966-2006	50	38.626	-81.234	13.0	810	2.01	AP

<u>Rainfall Data</u>. A summary of weather stations near the project location are provided in Table **4**, including Station ID, Period of Record, data coverage during the Period of Record, location, elevation, and distance from the project site. Although the Clendenin weather station is near enough to the Leatherwood Road project site that it could be considered for direct integration into watershed models that characterize flow conditions since the bridge was constructed, the period of record at this weather station ends in 2007, and does not enjoy 100% data coverage during the period when readings were collected. Therefore, the Clendenin weather station is not a suitable data source for direct integration into the Leatherwood Road watershed model. The Yeager Airport weather station, which has 100% data coverage and includes all time periods since the Leatherwood Road bridge was constructed, is 13.7 miles away from the project site. In view of substantial spatial variations in precipitation depths over this spatial scale, the Yeager Airport weather station is likewise not a suitable source for direct watershed model integration.

Table 4 – Weather stations near Leatherwood Road.

Station Name	Station ID	Period of Record	Coverage (%)	Lat.	Long.	Elev. (ft.)	Dist. (mi.)
Clendenin	USC00461723	1951-2007	93	38.493	-81.348	700	1.9
Yeager Airport	USW00013866	1948-Present	100	38.3794	-81.59	910	13.7

Point precipitation frequency estimates for a 24-hour duration storm at the watershed, based on partial duration series NOAA Atlas 14 data available at the Hydrometeorological Design Studies Center's Precipitation Frequency Data Server are provided in Table 5.

Table 5 – Precipitation depth at Leatherwood Road for a 24-hour duration storm, including 90% confidence interval.

Average Recurrence Interval (yr.)	Precipitation Depth (in.)	90% Confidence Interval	
		Precip. Depth, Lower (in.)	Precip. Depth, Upper (in.)
2	2.57	2.42	2.75
5	3.11	2.92	3.33
10	3.56	3.33	3.79
25	4.17	3.90	4.44
50	4.67	4.35	4.97
100	5.18	4.82	5.51
200	5.72	5.30	6.07
500	6.46	5.96	6.84

Development of flow rates at Leatherwood Road for a variety of return period storms utilized three methods: USGS-provided regional regression equations (see Table 6), a TR-55 based watershed model that incorporated 24-hour storm duration precipitation depths (see

Table 7), and a HEC-1 based watershed model that incorporated 24-hour storm duration precipitation depths (see Table 8). Since calculation of cumulative effective stream power requires characterization of flow rate as a function of time – and not simply the peak flow rate associated with a storm event – the runoff hydrographs generated by the HEC-1 analysis were utilized for this purpose.

Table 6 – USGS SIR-2010-5033 derived peak discharges, average prediction error, and corresponding 90% confidence interval for the Leatherwood Road watershed.

Return Period (yr)	Peak Discharge, Q (cfs)	Avg. Prediction Error (%)	90% Confidence Interval	
			Q, Lower (cfs)	Q, Upper (cfs)
2	311	32.2	146	476
5	520	30.0	264	777
10	678	29.7	347	1010
25	896	30.3	450	1343
50	1073	31.3	521	1626
100	1256	32.5	584	1927
200	1448	33.9	641	2255
500	1720	36.1	699	2741

Table 7 – Peak discharges derived from TR-55 watershed model of Leatherwood Road.

Return Period (yr)	Peak Discharge, Q (cfs)	90% Confidence Interval	
		Q, Lower (cfs)	Q, Upper (cfs)
2	225	172	298
5	470	375	589
10	724	589	868
25	1122	940	1314
50	1483	1249	1711
100	1874	1596	2138
200	2309	1969	2600
500	2931	2508	3260

Table 8 – Peak discharges derived from HEC-1 watershed model of Leatherwood Road.

Return Period (yr)	Peak Discharge, Q (cfs)	90% Confidence Interval	
		Q, Lower (cfs)	Q, Upper (cfs)
2	211	162	277
5	427	345	529
10	649	529	781
25	1017	848	1196
50	1354	1136	1568
100	1722	1460	1970
200	2132	1811	2408
500	2722	2320	3036

Hydraulics

An isometric view of the HEC-RAS Perspective Plot for Leatherwood Road is shown in Figure 2, the underlying geometry for which is based on a field survey that was performed at the site. Manning's n-values for the site are summarized in Table 9. The result of HEC-RAS modeling at the site is a flow depth and velocity rating curve, presented in Figure 3.

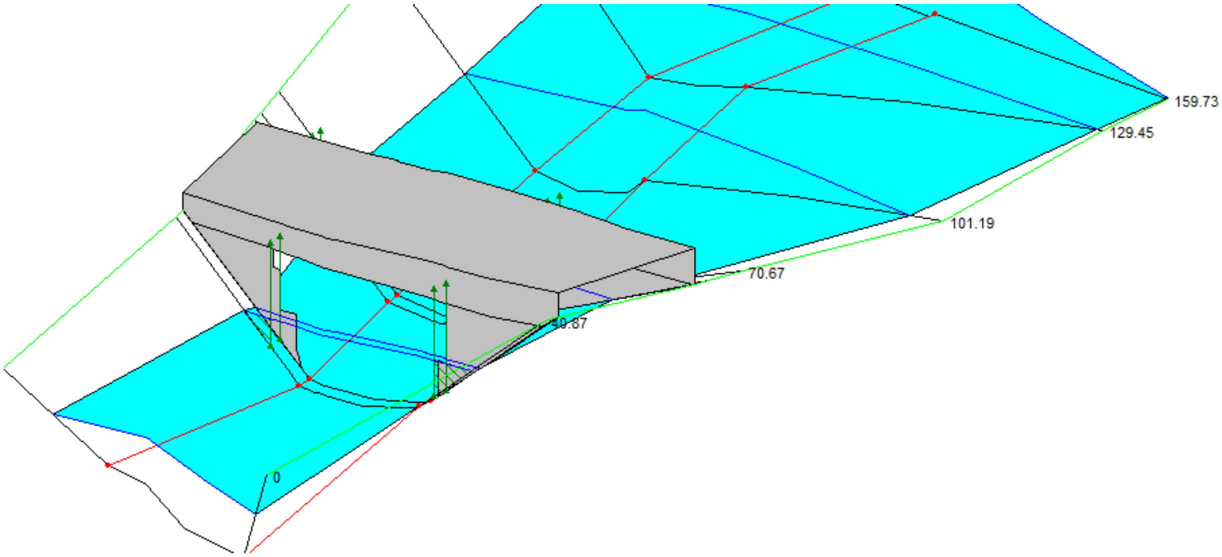

Figure 2- HEC-RAS X-Y-Z Perspective Plot for Leatherwood Road.

Table 9 – Manning's n-values utilized in HEC-RAS model for Leatherwood Road.

	Left Overbank	Channel	Right Overbank
U/S of Bridge	0.035	0.045	0.09
D/S of Bridge	0.07	0.045	0.09

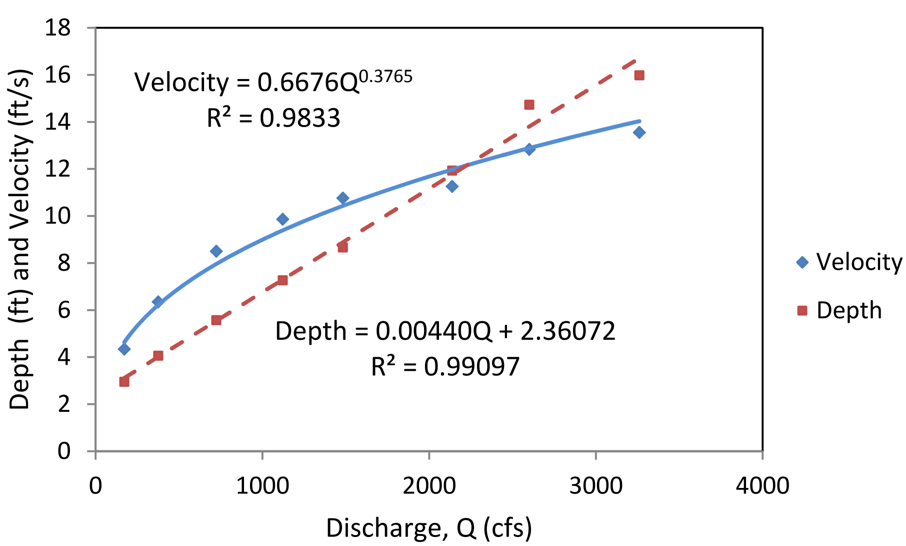

Figure 3- Flow depth and velocity rating curve for Leatherwood Road.

Stream Power

Cumulative Effective Stream Power at the Leatherwood Road project site incorporated HEC-1 generated flow rates and runoff hydrograph. Amounts of Cumulative Effective Stream Power associated with a variety of return period storms are summarized in Table 10, including amounts associated with flow rates (and corresponding flow velocities and depths) that were generated using the lower and upper 90% confidence interval precipitation depths. The Average Annual Cumulative Effective Stream Power, which accounts for the Cumulative Effective Stream Power associated with each return period storm and the annual probability of each of these storms, is shown in Table 11.

Table 10 – Scour event Cumulative Effective Stream Power Ω (approach flow) for Leatherwood Road – based on HEC-1 generated flow rates and hydrograph.

Return Period (yr)	Cumulative Effective Stream Power, Ω (ft·lb/ft^2)	90% Confidence Interval	
		Lower (ft·lb/ft^2)	Upper (ft·lb/ft^2)
2	0	0	0
5	14,957	13,415	16,323
10	36,494	32,797	38,460
25	73,339	67,249	76,882
50	108,963	99,835	113,612
100	148,335	137,087	153,744
200	192,546	177,744	197,523
500	257,652	238,383	261,442

Table 11 – Average Annual Cumulative Effective Stream Power Ω (approach flow) at Leatherwood Road.

Increment (return period range)	λ size	Interval Cumulative Effective Stream Power, Ω (ft·lb/ft^2)		
		Baseline Value	Lower 90% Conf.	Upper 90% Conf.
2-5	0.300	7,479	6,707	8,161
5-10	0.100	25,726	23,106	27,391
10-25	0.060	54,917	50,023	57,671
25-50	0.020	91,151	83,542	95,247
50-100	0.010	128,649	118,461	133,678
100-200	0.005	170,441	157,416	175,633
200-500	0.003	225,099	208,064	229,482
500-	0.002	257,652	238,383	261,442
Avg. Annual Cum. Eff. Stream Power, Ω (ft·lb/ft^2):		13,263	12,068	13,979

2 – 5th Street, Ritter Park

Watershed characteristics

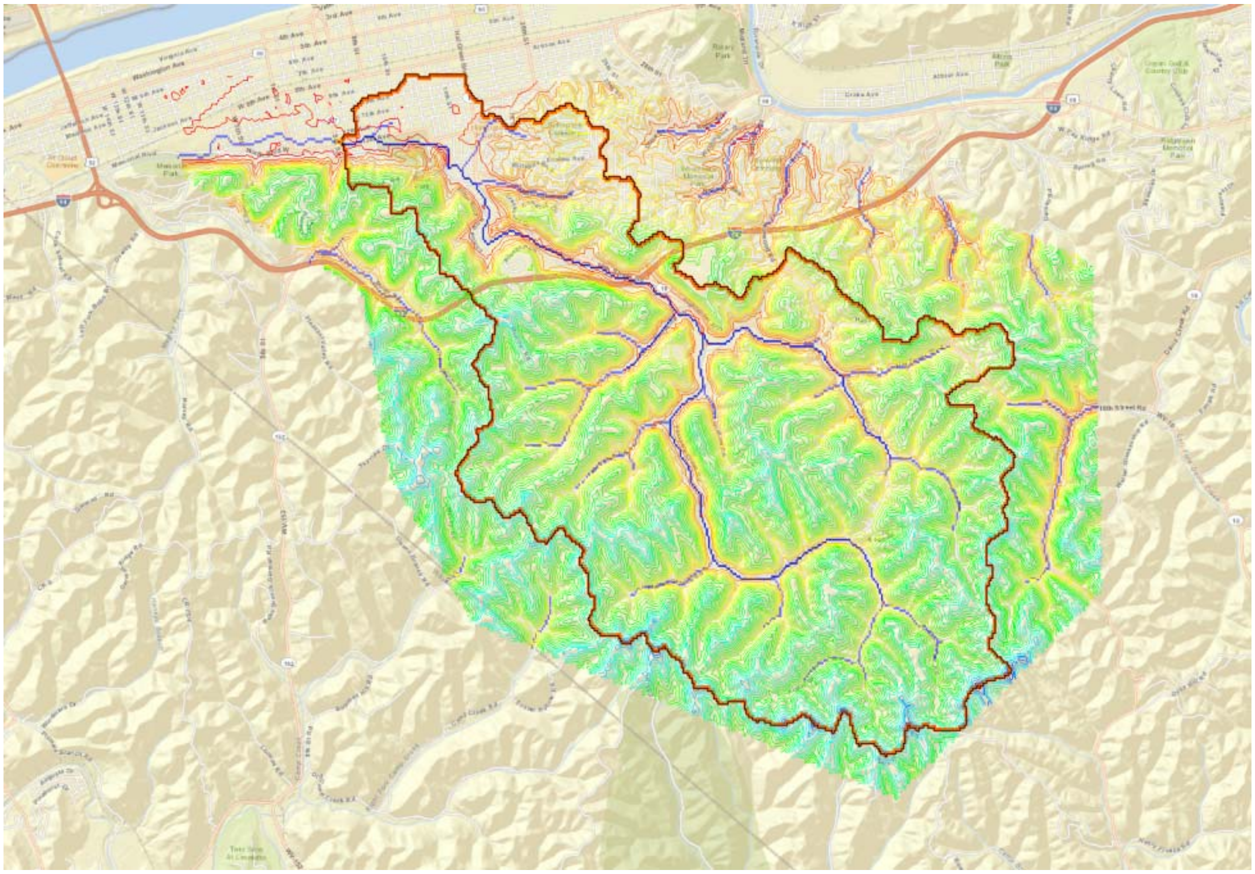

Figure 4- Image of delineated 5th Street Ritter park watershed.

Several relevant hydrologic parameters for the 5th Street Ritter Park watershed are provided below in Table 12.

Table 12 - Watershed characteristics for 5th Street Ritter Park.

Parameter	Value
Location	Lat: 38.406253 Long: -82.447272
Region – USGS SIR2010-5033	Western Plateaus
Basin Area (mi^2)	13.24
Average basin elevation (ft.)	766
Average basin overland slope (%)	23.5
Maximum flow distance (ft)	49132
Slope along maximum flow distance (%)	0.9
Shape factor (basin length / basin width)	3.10
Sinuosity factor of stream (max. stream length / basin length)	1.20
Runoff Curve Number	73.2
Time of Concentration (hr.)	3.007

A summary of the parameters utilized to compute the weighted watershed Curve Number is provided in the table below, including Natural Resources Conservation Service Soil Survey Geographic Database (SSURGO) derived hydrologic soil group, National Land Cover Database 2006 derived land cover classification, corresponding Curve Number, and sub-element area.

Table 13 – Soil type, land cover, and curve number characteristics for 5th Street Ritter Park.

Hydrologic Soil Group	Land Cover Description	CN	Area (mi^2)
D	Developed Low Intensity	84	1.093
C	Developed Open Space	74	0.798
D	Developed Open Space	80	0.629
D	Developed Medium Intensity	85	0.39
C	Developed Low Intensity	79	0.38
D	Deciduous Forest	77	0.58
C	Deciduous Forest	70	7.353
C	Developed Medium Intensity	80	0.092
D	Developed High Intensity	95	0.049
C	Grassland/Herbaceous	71	0.292
C	Pasture/Hay	74	0.636
C	Evergreen Forest	70	0.154
B	Developed Low Intensity	68	0.192
B	Developed Open Space	61	0.108
B	Developed Medium Intensity	70	0.031
D	Evergreen Forest	77	0.015
B	Deciduous Forest	55	0.064
B	Grassland/Herbaceous	58	0.023
D	Grassland/Herbaceous	78	0.062
D	Pasture/Hay	78	0.121
B	Pasture/Hay	61	0.077
C	Woody Wetlands	70	0.01
C	Developed High Intensity	94	0.013
B	Developed High Intensity	92	0.01
B	Cultivated Crops	75	0.003
C	Cultivated Crops	82	0.003
C	Open Water	98	0.003
C	Mixed Forest	70	0.049
D	Mixed Forest	77	0.005
C	Shrub/Scrub	71	0.005
Weighted CN = 73.2			

Flow synthesis

Stream Gage Data. In Table 14 below, characteristics of the stream gage stations in proximity to the project site are summarized, including period of record and data coverage within that period, location, distance from the project area, elevation, drainage area upstream of the gage, and whether the available data consists of Daily Discharge (DD) or Annual Peak (AP).

Fourpole Creek (the water body which the 5th Street Ritter Park bridge crosses) is a gauged stream. However, the stream gage that is available is a considerable distance upstream of the project location, gauging a watershed area of 4.02 mi^2 of the 13.24 mi^2 in question. Additionally, the Fourpole Creek stream gauging location only covers from 1999-2011, whereas the bridge at 5th Street was constructed in 1921. In view of these facts, the Fourpole Creek stream gauging station is not able to provide a clear picture of flow conditions at the project site since construction. Likewise, none of the other gauged streams in the nearby area are well-suited for flow extrapolation to the 5th Street Ritter Park project site, because of significant dissimilarities in watershed drainage area, time period coverage that does not overlap with the period since the bridge was constructed, and excessive distances from the project site location. Therefore, flow synthesis at the 5th Street Ritter Park site utilizes two methods (as outlined below): the applicable USGS peak flow regression equations, and creation of a watershed model that utilizes probabilistic rainfall data.

Table 14 – Stream gage stations near 5th Street Ritter Park.

Project / Stream Gage Location	USGS Station ID	Period	Covg. (%)	Lat.	Long.	Dist. (mi.)	Elev. (ft.)	Drainage Area (mi^2)	Type
5th St. Ritter Park, Fourpole Creek	-	*1921-Present*	-	*38.4063*	*-82.447*	-	*547*	*13.24*	-
Mud River Near Milton, WV	03204500	1939-1980	100	38.388	-82.113	18.1	572.6	256	DD
Sandusky Creek near Burlington OH	03205995	1978-2011	61	38.418	-82.51	3.5	553	0.73	AP
FOURPOLE CREEK NEAR HUNTINGTON, WV	03206450	1999-2011	100	38.3625	-82.392	4.3	599.4	4.02	AP
TWELVEPOLE CREEK BELOW WAYNE, WV	03207020	1915-1982	63	38.249	-82.434	10.9	559.3	300	DD

Rainfall Data. A summary of weather stations near the project location are provided in Table 15, including Station ID, Period of Record, data coverage during the Period of Record, location, elevation, and distance from the project site. None of the four nearby weather stations offer uninterrupted and complete data coverage since the 1921 date of bridge construction at the 5th Street Ritter Park site.

Table 15 – Weather stations near 5th Street Ritter Park.

Station Name	Station ID	Period of Record	Coverage (%)	Lat.	Long.	Elev. (ft.)	Dist. (mi.)
Huntington Federal Bldg.	USC00464388	1948-1967	92	38.416	-82.45	565	0.7
Huntington 2	USC00464383	1943-1951	94	38.416	-82.433	600	1.0
Huntington Sewage Plant	USC00464397	1967-Present	85	38.401	-82.526	520	4.3
Huntington 1	USC00464378	1891-1957	90	38.416	-82.366	679	4.5

Point precipitation frequency estimates for a 24-hour duration storm at the watershed, based on partial duration series NOAA Atlas 14 data available at the Hydrometeorological Design Studies Center's Precipitation Frequency Data Server are provided in Table 16.

Table 16 – Precipitation depth at 5th St. Ritter Park for a 24-hour duration storm, including 90% confidence interval.

Average Recurrence Interval (yr.)	Precipitation Depth (in.)	90% Confidence Interval	
		Precip. Depth, Lower (in.)	Precip. Depth, Upper (in.)
2	2.64	2.48	2.82
5	3.20	2.99	3.41
10	3.65	3.41	3.89
25	4.28	3.99	4.55
50	4.79	4.45	5.09
100	5.32	4.93	5.64
200	5.87	5.42	6.22
500	6.63	6.10	7.01

Development of flow rates at 5th St. Ritter Park for a variety of return period storms utilized three methods: USGS-provided regional regression equations (see Table 17), a TR-55 based watershed model that incorporated 24-hour storm duration precipitation depths (see Table 18), and a HEC-1 based watershed model that incorporated 24-hour storm duration precipitation depths (see Table 19). Since calculation of cumulative effective stream power requires characterization of flow rate as a function of time – and not simply the peak flow rate associated with a storm event – the runoff hydrographs generated by the HEC-1 analysis were utilized for this purpose.

Table 17 – USGS SIR-2010-5033 derived peak discharges, average prediction error, and corresponding 90% confidence interval for the 5th Street Ritter Park watershed.

Return Period (yr)	Peak Discharge, Q (cfs)	Avg. Prediction Error (%)	90% Confidence Interval	
			Q, Lower (cfs)	Q, Upper (cfs)
2	850	32.2	400	1301
5	1383	30.0	701	2066
10	1777	29.7	909	2645
25	2312	30.3	1160	3465
50	2741	31.3	1330	4153
100	3177	32.5	1478	4875
200	3634	33.9	1607	5660
500	4274	36.1	1736	6813

Table 18 – Peak discharges derived from TR-55 watershed model of 5th Street Ritter Park.

Return Period (yr)	Peak Discharge, Q (cfs)	90% Confidence Interval	
		Q, Lower (cfs)	Q, Upper (cfs)
2	1176	990	1397
5	1898	1616	2193
10	2543	2193	2905
25	3515	3059	3951
50	4347	3789	4852
100	5245	4581	5800
200	6205	5418	6828
500	7569	6614	8264

Table 19 – Peak discharges derived from HEC-1 watershed model of 5th Street Ritter Park.

Return Period (yr)	Peak Discharge, Q (cfs)	90% Confidence Interval	
		Q, Lower (cfs)	Q, Upper (cfs)
2	1121	945	1330
5	1806	1537	2086
10	2419	2086	2764
25	3346	2911	3766
50	4151	3608	4642
100	5025	4379	5566
200	5960	5193	6567
500	7287	6358	7964

Hydraulics

An isometric view of the HEC-RAS Perspective Plot for 5th Street Ritter Park is shown in Figure 5, the underlying geometry for which is based on a field survey that was performed at the site. Manning's n-values for the site are summarized in Table 20. The result of HEC-RAS modeling at the site is a flow depth and velocity rating curve, presented in Figure **6**.

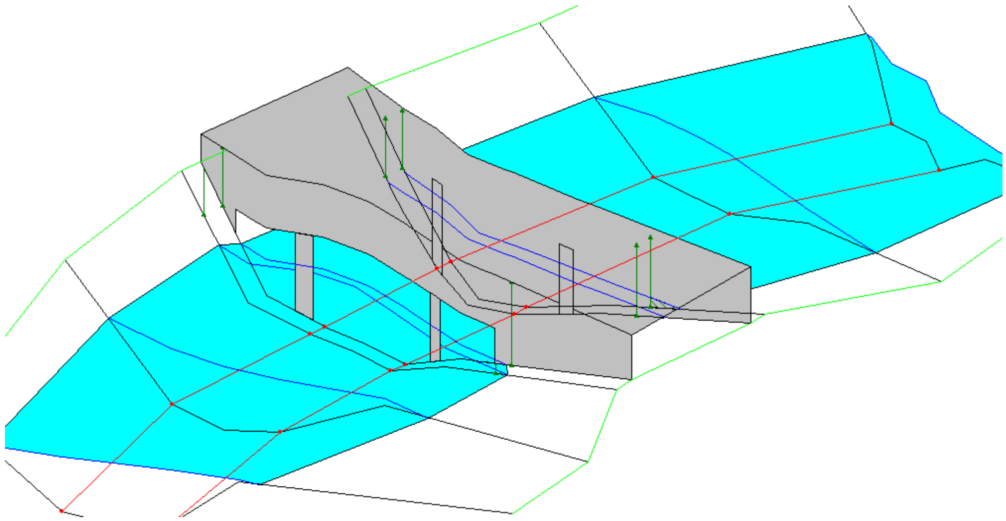

Figure 5- HEC-RAS X-Y-Z Perspective Plot for 5th Street Ritter Park.

Table 20 – Manning's n-values utilized in HEC-RAS model for 5th Street Ritter Park.

	Left Overbank	Channel	Right Overbank
U/S of Bridge	0.08	0.04	0.045
D/S of Bridge	0.08	0.04	0.045

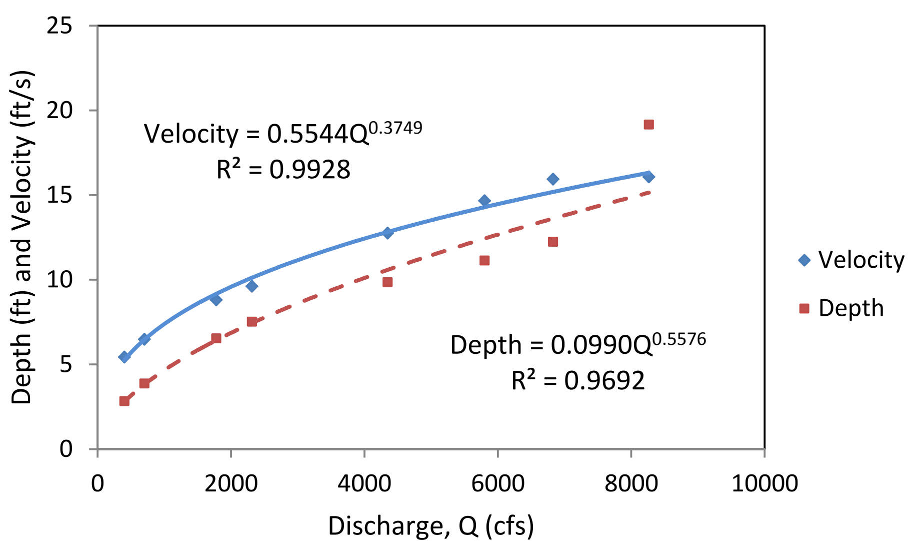

Figure 6- Flow depth and velocity rating curve for 5th Street Ritter Park.

Stream Power

Cumulative Effective Stream Power at the 5th Street Ritter Park project site incorporated HEC-1 generated flow rates and runoff hydrograph. Amounts of Cumulative Effective Stream Power associated with a variety of return period storms are summarized in Table 21 (approach flow) and Table 22 (at pier), including amounts associated with flow rates (and corresponding flow velocities and depths) that were generated using the lower and upper 90% confidence interval precipitation depths. The Average Annual Cumulative Effective Stream Power, which accounts for the Cumulative Effective Stream Power associated with each return period storm and the annual probability of each of these storms, is shown in Table 23 and Table 24.

Table 21 – Scour event Cumulative Effective Stream Power Ω (approach flow) for 5th Street Ritter Park – based on HEC-1 generated flow rates and hydrograph.

Return Period (yr)	Cumulative Effective Stream Power , Ω ($ft \cdot lb/ft^2$)	90% Confidence Interval	
		Lower ($ft \cdot lb/ft^2$)	Upper ($ft \cdot lb/ft^2$)
2	0	0	0
5	37,218	53,869	67,442
10	87,469	107,740	127,886
25	175,898	208,530	246,995
50	257,804	306,060	358,364
100	349,663	400,521	467,898
200	451,305	512,419	602,582
500	599,790	664,968	772,204

Table 22 – Scour event Cumulative Effective Stream Power Ω (pier) for 5th Street Ritter Park – based on HEC-1 generated flow rates and hydrograph.

Return Period (yr)	Cumulative Effective Stream Power , Ω ($ft \cdot lb/ft^2$)	90% Confidence Interval	
		Lower ($ft \cdot lb/ft^2$)	Upper ($ft \cdot lb/ft^2$)
2	0	0	0
5	107,561	155,681	194,906
10	252,786	311,368	369,590
25	508,347	602,651	713,816
50	745,054	884,514	1,035,671
100	1,010,526	1,157,506	1,352,225
200	1,304,271	1,480,890	1,741,462
500	1,733,393	1,921,757	2,231,668

Table 23 – Average Annual Cumulative Effective Stream Power Ω (approach flow) at 5th Street Ritter Park.

Increment (return period range)	λ size	Interval Cumulative Effective Stream Power, Ω ($ft \cdot lb/ft^2$)		
		Baseline Value	Lower 90% Conf.	Upper 90% Conf.
2-5	0.300	18,609	26,934	33,721
5-10	0.100	62,344	80,804	97,664
10-25	0.060	131,684	158,135	187,440
25-50	0.020	216,851	257,295	302,679
50-100	0.010	303,734	353,291	413,131
100-200	0.005	400,484	456,470	535,240
200-500	0.003	525,547	588,693	687,393
500-	0.002	599,790	664,968	772,204
Avg. Annual Cum. Eff. Stream Power, Ω ($ft \cdot lb/ft^2$):		31,871	39,706	47,597

Table 24 – Average Annual Cumulative Effective Stream Power Ω (pier) at 5th Street Ritter Park.

Increment (return period range)	λ size	Interval Cumulative Effective Stream Power, ($ft \cdot lb/ft^2$)		
		Baseline Value	Lower 90% Conf.	Upper 90% Conf.
2-5	0.300	53,780	77,841	97,453
5-10	0.100	180,173	233,524	282,248
10-25	0.060	380,566	457,009	541,703
25-50	0.020	626,700	743,583	874,743
50-100	0.010	877,790	1,021,010	1,193,948
100-200	0.005	1,157,398	1,319,198	1,546,844
200-500	0.003	1,518,832	1,701,323	1,986,565
500-	0.002	1,733,393	1,921,757	2,231,668
Avg. Annual Cum. Eff. Stream Power, Ω ($ft \cdot lb/ft^2$):		92,108	114,750	137,555

3 – Little Sandy Creek

Watershed characteristics

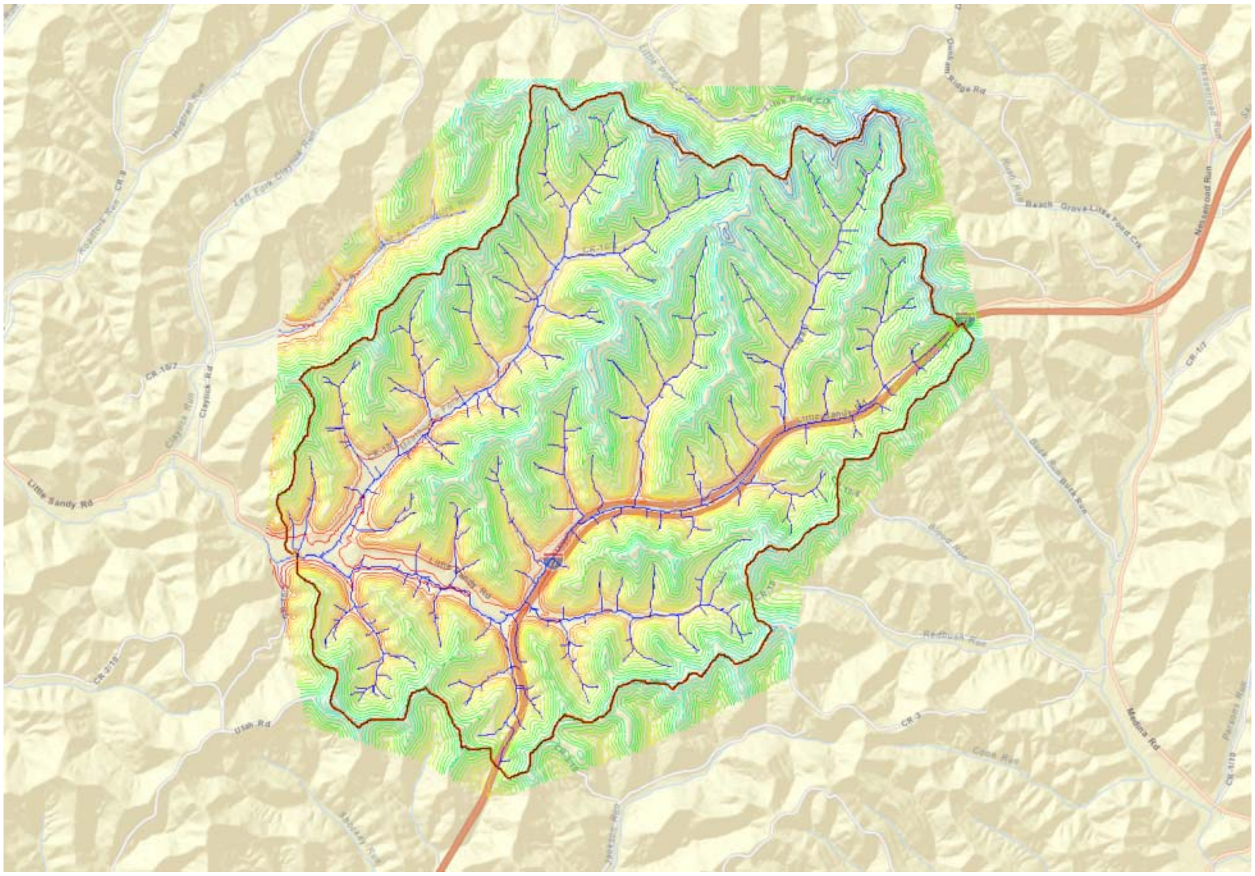

Figure 7- Image of delineated Little Sandy watershed.

Several relevant hydrologic parameters for the Little Sandy Creek watershed are provided below in Table 25.

Table 25 - Watershed characteristics for Little Sandy Creek.

Parameter	**Value**
Location	Lat: 38.987800 Long: -81.719661
Region – USGS SIR2010-5033	Western Plateaus
Basin Area (mi^2)	8.40
Average basin elevation (ft)	835
Average basin overland slope (%)	27.5
Maximum flow distance (ft)	29260
Slope along maximum flow distance (%)	1.9
Shape factor (basin length / basin width)	1.92
Sinuosity factor of stream (max. stream length / basin length)	1.35
Runoff Curve Number	75.5
Time of Concentration (hr.)	1.722

A summary of the parameters utilized to compute the weighted watershed Curve Number is provided in Table 26, including Natural Resources Conservation Service Soil Survey Geographic Database (SSURGO) derived hydrologic soil group, National Land Cover Database 2006 derived land cover classification, corresponding Curve Number, and sub-element area.

Table 26 – Soil type, land cover, and curve number characteristics for Little Sandy Creek.

Hydrologic Soil Group	Land Cover Description	CN	Area (mi^2)
D	Deciduous Forest	77	6.605
D	Pasture/Hay	78	0.262
B	Pasture/Hay	61	0.248
B	Developed Open Space	61	0.087
B	Deciduous Forest	55	0.406
D	Developed Open Space	80	0.483
C	Pasture/Hay	74	0.018
D	Grassland/Herbaceous	78	0.018
D	Shrub/Scrub	78	0.003
C	Deciduous Forest	70	0.09
B	Open Water	98	0.001
C	Developed Open Space	74	0.003
D	Developed Low Intensity	84	0.104
D	Developed Medium Intensity	85	0.028
D	Evergreen Forest	77	0.014
B	Grassland/Herbaceous	58	0.011
D	Cultivated Crops	86	0.018
B	Cultivated Crops	75	0.003
Weighted CN = 75.5			

Flow synthesis

Stream Gage Data. In Table 27, characteristics of the stream gage stations in proximity to the Little Sandy Creek project site are summarized, including period of record and data coverage within that period, location, distance from the project area, elevation, drainage area upstream of the gage, and whether the available data consists of Daily Discharge (DD) or Annual Peak (AP).

Little Sandy Creek (the water body which the Little Sandy Creek bridge crosses) is an ungauged stream. Additionally, none of the gauged streams in the nearby area are well-suited for direct flow extrapolation to the project site, due to excessive distance, significant differences in watershed drainage area, and dissimilarities in the time period in question. Therefore, flow synthesis at the Little Sandy Creek site utilizes two methods (as outlined below): the applicable USGS peak flow regression equations, and creation of a watershed model that utilizes probabilistic rainfall data.

Table 27 – Stream gage stations near Little Sandy Creek.

Project / Stream Gage Location	USGS Station ID	Period	Covg. (%)	Lat.	Long.	Dist. (mi.)	Elev. (ft.)	Drainage Area (mi^2)	Type
Little Sandy Creek	-	*1977-Present*	-	*38.9878*	*-81.72*	-	*667*	*8.40*	-
Shade River near Chester OH	03159540	1965-Present	100	39.064	-81.882	10.2	576.9	1.56	DD
GRASSLICK RN NR RIPLEY, WV	03159700	1965-1976	100	38.765	-81.678	15.6	835	0.70	AP
REEDY CREEK NEAR REEDY, WV	03154500	1951-1978	100	38.961	-81.39	17.8	669	79.4	DD
LITTLE KANAWHA RIVER AT PALESTINE, WV	03155000	1939-Present	100	39.059	-81.39	18.4	584.9	1,516	DD

Rainfall Data. A summary of weather stations near the project location are provided in Table 28, including Station ID, Period of Record, data coverage during the Period of Record, location, elevation, and distance from the project site. Since the nearest weather station is nearly 9 miles away from the project site, and in view of the precipitation variations that can exist over such a spatial range, the rainfall data from the Belleville Lock and Dam weather station is not suitable for direct integration into the Little Sandy Creek watershed model.

Table 28 – Weather stations near Little Sandy Creek.

Station Name	Station ID	Period of Record	Coverage (%)	Lat.	Long.	Elev. (ft.)	Dist. (mi.)
Belleville Lock and Dam	USC00330573	1968-Present	96	39.116	-81.743	560	8.9

Point precipitation frequency estimates for a 24-hour duration storm at the watershed, based on partial duration series NOAA Atlas 14 data available at the Hydrometeorological Design Studies Center's Precipitation Frequency Data Server are provided in Table 29.

Table 29 – Precipitation depth at Little Sandy Creek for a 24-hour duration storm, including 90% confidence interval.

Average Recurrence Interval (yr.)	Precipitation Depth (in.)	90% Confidence Interval	
		Precip. Depth, Lower (in.)	Precip. Depth, Upper (in.)
2	2.56	2.41	2.73
5	3.10	2.92	3.31
10	3.54	3.33	3.77
25	4.15	3.89	4.41
50	4.65	4.34	4.93
100	5.16	4.81	5.46
200	5.70	5.29	6.02
500	6.43	5.94	6.79

Development of flow rates at Little Sandy Creek for a variety of return period storms utilized three methods: USGS-provided regional regression equations (see Table 30), a TR-55 based watershed model that incorporated 24-hour storm duration precipitation depths (see Table 31), and a HEC-1 based watershed model that incorporated 24-hour storm duration precipitation depths (see Table 32). Since calculation of cumulative effective stream power requires characterization of flow rate as a function of time – and not simply the peak flow rate associated with a storm event – the runoff hydrographs generated by the HEC-1 analysis were utilized for this purpose.

Table 30 – USGS SIR-2010-5033 derived peak discharges, average prediction error, and corresponding 90% confidence interval for the Little Sandy Creek watershed.

Return Period (yr)	Peak Discharge, Q (cfs)	Avg. Prediction Error (%)	90% Confidence Interval	
			Q, Lower (cfs)	Q, Upper (cfs)
2	610	32.2	287	933
5	1001	30.0	507	1496
10	1293	29.7	661	1924
25	1691	30.3	848	2533
50	2011	31.3	975	3046
100	2338	32.5	1088	3588
200	2681	33.9	1186	4176
500	3164	36.1	1285	5043

Table 31 – Peak discharges derived from TR-55 watershed model of Little Sandy Creek.

Return Period (yr)	Peak Discharge, Q (cfs)	90% Confidence Interval	
		Q, Lower (cfs)	Q, Upper (cfs)
2	1253	1070	1469
5	1970	1722	2269
10	2607	2298	2955
25	3546	3139	3961
50	4350	3849	4812
100	5196	4613	5702
200	6111	5414	6662
500	7376	6524	8009

Table 32 – Peak discharges derived from HEC-1 watershed model of Little Sandy Creek.

Return Period (yr)	Peak Discharge, Q (cfs)	90% Confidence Interval	
		Q, Lower (cfs)	Q, Upper (cfs)
2	1171	999	1376
5	1851	1615	2136
10	2457	2163	2789
25	3353	2965	3750
50	4123	3643	4565
100	4933	4375	5419
200	5812	5143	6341
500	7025	6208	7632

Hydraulics

An isometric view of the HEC-RAS Perspective Plot for Little Sandy Creek is shown in Figure 8 the underlying geometry for which is based on a field survey that was performed at the site. Manning's n-values for the site are summarized in Table 33. The result of HEC-RAS modeling at the site is a flow depth and velocity rating curve, presented in Figure 9.

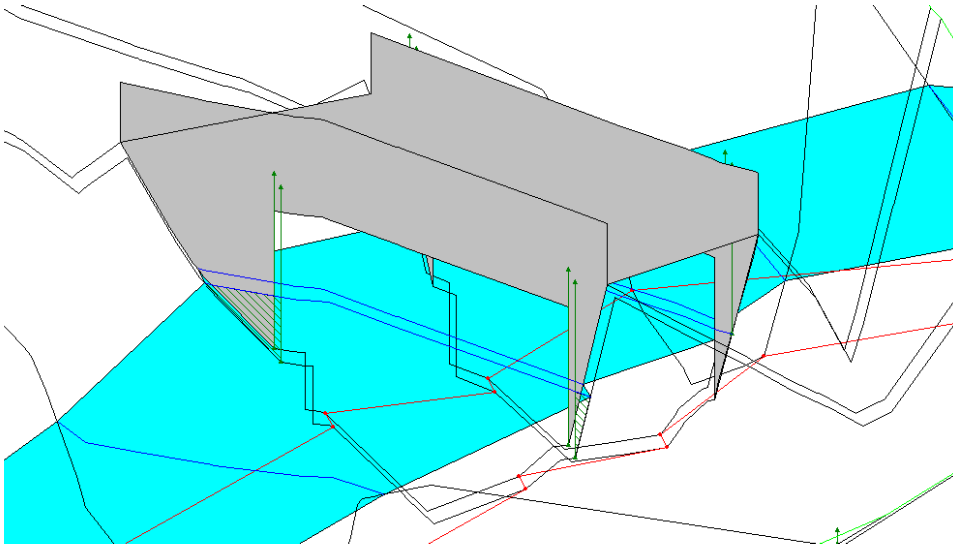

Figure 8- HEC-RAS X-Y-Z Perspective Plot for Little Sandy Creek.

Table 33 – Manning's n-values utilized in HEC-RAS model for Little Sandy Creek.

	Left Overbank	Channel	Right Overbank
U/S of Bridge	0.07	0.05	0.07
D/S of Bridge	0.07	0.05	0.07

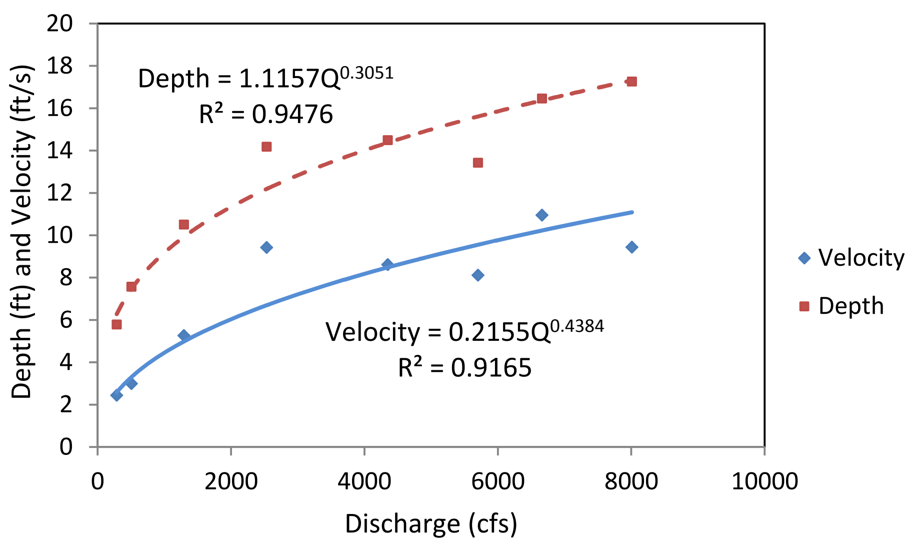

Figure 9- Flow depth and velocity rating curve for Little Sandy Creek.

Stream Power

Cumulative Effective Stream Power at the Little Sandy Creek project site incorporated HEC-1 generated flow rates and runoff hydrograph. Amounts of Cumulative Effective Stream Power associated with a variety of return period storms are summarized in Table 34, including amounts associated with flow rates (and corresponding flow velocities and depths) that were generated using the lower and upper 90% confidence interval precipitation depths. The Average Annual Cumulative Effective Stream Power, which accounts for the Cumulative Effective Stream Power associated with each return period storm and the annual probability of each of these storms, is shown in Table 35.

Table 34 – Scour event Cumulative Effective Stream Power Ω (approach flow) for Little Sandy Creek – based on HEC-1 generated flow rates and hydrograph.

Return Period (yr)	Cumulative Effective Stream Power , Ω ($ft \cdot lb/ft^2$)	90% Confidence Interval	
		Lower ($ft \cdot lb/ft^2$)	Upper ($ft \cdot lb/ft^2$)
2	0	0	0
5	8,294	7,512	17,938
10	20,209	18,164	31,653
25	41,822	37,323	62,022
50	63,096	55,757	86,980
100	87,140	77,186	115,034
200	115,375	101,519	151,345
500	157,254	137,607	199,113

Table 35 – Average Annual Cumulative Effective Stream Power Ω (approach flow) at Little Sandy Creek.

Increment (return period range)	λ size	Interval Cumulative Effective Stream Power, Ω ($ft \cdot lb/ft^2$)		
		Baseline Value	Lower 90% Conf.	Upper 90% Conf.
2-5	0.300	4,147	3,756	8,969
5-10	0.100	14,252	12,838	24,796
10-25	0.060	31,015	27,743	46,838
25-50	0.020	52,459	46,540	74,501
50-100	0.010	75,118	66,472	101,007
100-200	0.005	101,258	89,352	133,190
200-500	0.003	136,314	119,563	175,229
500-	0.002	157,254	137,607	199,113
Avg. Annual Cum. Eff. Stream Power, Ω ($ft \cdot lb/ft^2$):		7,560	6,751	12,070

4 – Grassy Run

Watershed characteristics

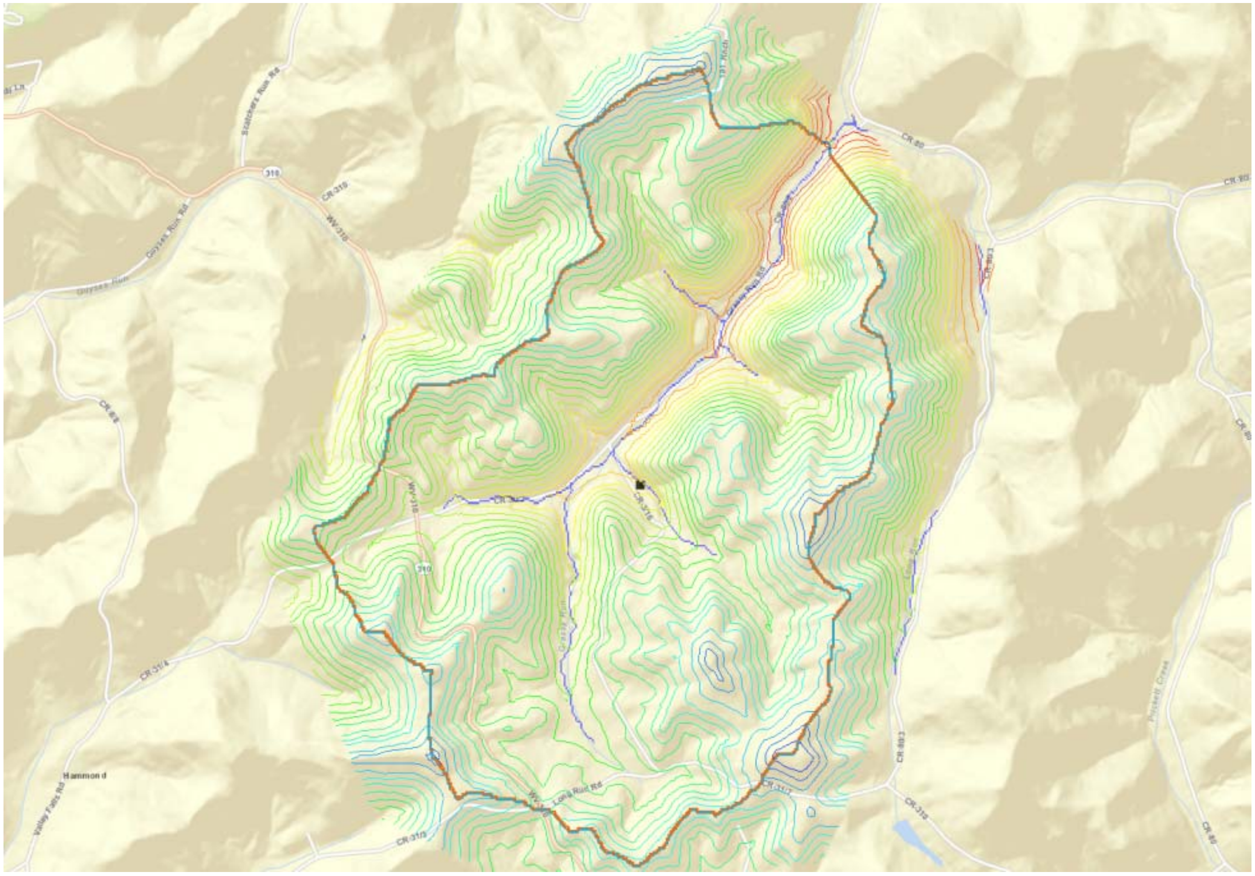

Figure 10- Image of delineated Grassy Run watershed.

Several relevant hydrologic parameters for the Grassy Run watershed are provided below in Table 36.

Table 36 - Watershed characteristics for Grassy Run.

Parameter	Value
Location	Lat: 39.453383 Long: -80.065397
Region – USGS SIR2010-5033	Western Plateaus
Basin Area (mi^2)	2.52
Average basin elevation (ft)	1394
Average basin overland slope (%)	23.7
Maximum flow distance (ft)	16229
Slope along maximum flow distance (%)	3.9
Shape factor (basin length / basin width)	2.07
Sinuosity factor of stream (max. stream length / basin length)	1.27
Runoff Curve Number	68.4
Time of Concentration (hr.)	1.407

A summary of the parameters utilized to compute the weighted watershed Curve Number is provided in Table 37, including Natural Resources Conservation Service Soil Survey Geographic Database (SSURGO) derived hydrologic soil group, National Land Cover Database 2006 derived land cover classification, corresponding Curve Number, and sub-element area.

Table 37 – Soil type, land cover, and curve number characteristics for Grassy Run.

Hydrologic Soil Group	Land Cover Description	CN	Area (mi^2)
C	Deciduous Forest	70	1.7
C	Developed Open Space	74	0.145
C	Pasture/Hay	74	0.175
C	Barren Land	91	0.005
B	Deciduous Forest	55	0.34
B	Developed Open Space	61	0.025
B	Pasture/Hay	61	0.063
C	Developed Low Intensity	79	0.018
C	Cultivated Crops	82	0.016
B	Cultivated Crops	75	0.014
B	Developed Low Intensity	68	0.001
D	Deciduous Forest	77	0.002
C	Mixed Forest	70	0.006
Weighted CN = 68.4			

Flow synthesis

Stream Gage Data. In Table 38, characteristics of the stream gage stations in proximity to the Grassy Run project site are summarized, including period of record and data coverage within that period, location, distance from the project area, elevation, drainage area upstream of the gage, and whether the available data consists of Daily Discharge (DD) or Annual Peak (AP).

Prickett Creek (the water body which the Grassy Run bridge crosses) is an ungauged stream. Additionally, none of the gauged streams in the nearby area are well-suited for direct flow extrapolation to the project site, due to excessive distance, significant differences in watershed drainage area, and data availability periods that do not overlap with the time since construction. Therefore, flow synthesis at the Grassy Run site utilizes two methods: the applicable USGS peak flow regression equations, and creation of a watershed model that utilizes probabilistic rainfall data.

Table 38 – Stream gage stations near Grassy Run.

Project / Stream Gage Location	USGS Station ID	Period	Covg. (%)	Lat.	Long.	Dist. (mi.)	Elev. (ft.)	Drainage Area (mi^2)	Type
Grassy Run, Prickett Creek	-	*1949-Present*	-	*39.4534*	*-80.065*	-	*1031*	*2.52*	-
BUFFALO CREEK AT BARRACKVILLE, WV	03061500	1907-Present	87	39.504	-80.172	6.7	883.6	116	DD
WEST FORK RIVER AT ENTERPRISE, WV	03061000	1907-Present	84	39.422	-80.276	11.4	869.0	759	DD
THREE FORK CREEK NR GRAFTON, WV	03056250	1984-2011	100	39.336	-79.994	9.0	999.4	96.8	DD
TYGART VALLEY R AT FETTERMAN, WV	03056500	1907-1939	100	39.35	-80.042	7.3	957.9	1,304	DD
R F WICKWIRE RUN ON US HWY 119 NR	03056600	1965-1976	100	39.379	-79.963	7.5	1310	2.33	AP
COBUN CREEK AT MORGANTOWN, WV	03062400	1965-2002	93	39.608	-79.955	12.2	892	11	DD
DECKERS CREEK AT MORGANTOWN, WV	03062500	1946-2011	50	39.629	-79.953	13.5	805	63.2	DD

Rainfall Data. A summary of weather stations near the project location are provided in Table 39, including Station ID, Period of Record, data coverage during the Period of Record, location, elevation, and distance from the project site. The Fairmont weather station is near enough to the Grassy Run project site that precipitation data from this station may be suitable for integration into the Grassy Run watershed model. Rather than deviate from the established methodology that is utilized for the remainder of the project sites, however, a probabilistic characterization of precipitation at the project site is utilized. Comparison of probabilistically-derived runoff flows to runoff flows generated from rainfall data near a project site should be considered in future research.

Table 39 – Weather stations near Grassy Run.

Station Name	**Station ID**	**Period of Record**	**Coverage (%)**	**Lat.**	**Long.**	**Elev. (ft.)**	**Dist. (mi.)**
Fairmont	USC00462920	1892-Present	99	39.466	-80.133	1300	3.7
Tygart Dam	USC00468986	1949-Present	23	39.313	-80.029	1200	9.9
Morgantown Lock and Dam	USC00466212	1921-Present	96	39.62	-79.969	825	12.6

Point precipitation frequency estimates for a 24-hour duration storm at the watershed, based on partial duration series NOAA Atlas 14 data available at the Hydrometeorological Design Studies Center's Precipitation Frequency Data Server are provided in Table 40.

Table 40 – Precipitation depth at Grassy Run for a 24-hour duration storm, including 90% confidence interval.

Average Recurrence Interval (yr.)	Precipitation Depth (in.)	90% Confidence Interval	
		Precip. Depth, Lower (in.)	Precip. Depth, Upper (in.)
2	2.53	2.38	2.71
5	3.07	2.88	3.28
10	3.51	3.29	3.75
25	4.12	3.85	4.39
50	4.61	4.30	4.91
100	5.13	4.76	5.45
200	5.67	5.24	6.01
500	6.41	5.90	6.79

Development of flow rates at Grassy Run for a variety of return period storms utilized three methods: USGS-provided regional regression equations (see Table 41), a TR-55 based watershed model that incorporated 24-hour storm duration precipitation depths (see Table 42), and a HEC-1 based watershed model that incorporated 24-hour storm duration precipitation depths (see Table 43). Since calculation of cumulative effective stream power requires characterization of flow rate as a function of time – and not simply the peak flow rate associated with a storm event – the runoff hydrographs generated by the HEC-1 analysis were utilized for this purpose.

Table 41 – USGS SIR-2010-5033 derived peak discharges, average prediction error, and corresponding 90% confidence interval for the Grassy Run watershed.

Return Period (yr)	Peak Discharge, Q (cfs)	Avg. Prediction Error (%)	90% Confidence Interval	
			Q, Lower (cfs)	Q, Upper (cfs)
2	253	32.2	119	387
5	426	30.0	216	636
10	557	29.7	285	829
25	738	30.3	370	1107
50	886	31.3	430	1342
100	1038	32.5	483	1594
200	1200	33.9	531	1869
500	1428	36.1	580	2275

Table 42 – Peak discharges derived from TR-55 watershed model of Grassy Run.

Return Period (yr)	Peak Discharge, Q (cfs)	90% Confidence Interval	
		Q, Lower (cfs)	Q, Upper (cfs)
2	202	160	257
5	381	313	462
10	555	466	658
25	825	702	952
50	1059	909	1209
100	1322	1134	1488
200	1605	1379	1789
500	2009	1729	2221

Table 43 – Peak discharges derived from HEC-1 watershed model of Grassy Run.

Return Period (yr)	Peak Discharge, Q (cfs)	90% Confidence Interval	
		Q, Lower (cfs)	Q, Upper (cfs)
2	189	151	238
5	347	287	420
10	507	424	603
25	760	645	880
50	981	840	1123
100	1230	1052	1388
200	1500	1284	1675
500	1885	1618	2088

Hydraulics

An isometric view of the HEC-RAS Perspective Plot for Grassy Run is shown in Figure 11, the underlying geometry for which is based on a field survey that was performed at the site. Manning's n-values for the site are summarized in Table 44. The result of HEC-RAS modeling at the site is a flow depth and velocity rating curve, presented in Figure 12.

Note that when the velocity rating curve for Grassy Run was utilized in computing stream power, that since the regression expression has a linear relationship, this would entail assigning a positive velocity when flow rates were very low (due to the y-axis intercept). Thus, a threshold condition of V = 0 was assigned for any flow rates below 20 cfs when computing stream power at this site.

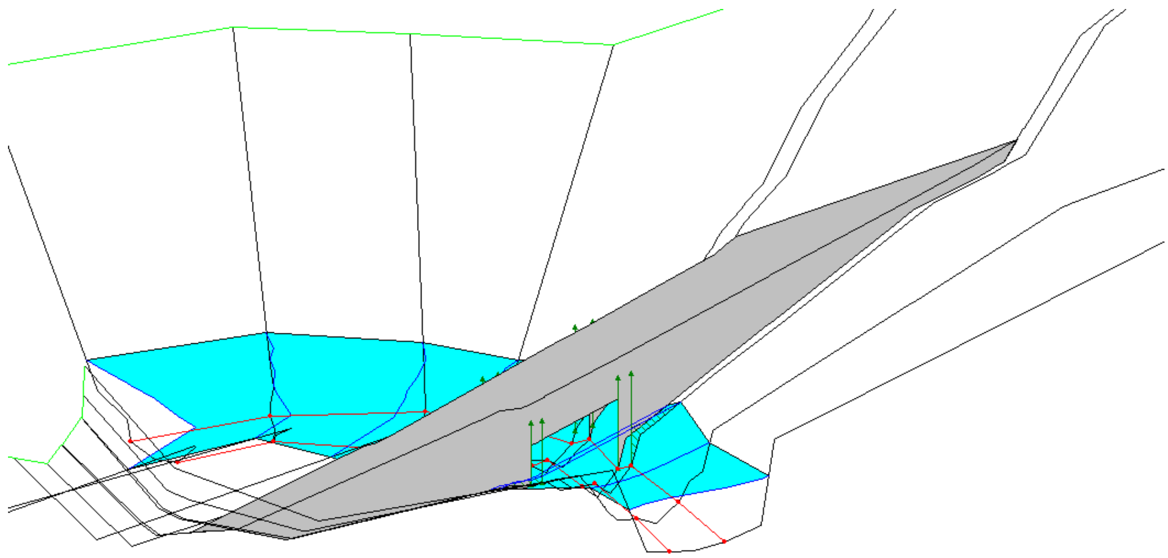

Figure 11- HEC-RAS X-Y-Z Perspective Plot for Grassy Run.

Table 44 – Manning's n-values utilized in HEC-RAS model for Grassy Run.

	Left Overbank	Channel	Right Overbank
U/S of Bridge	0.1	0.045	0.04
D/S of Bridge	0.1	0.045	0.1

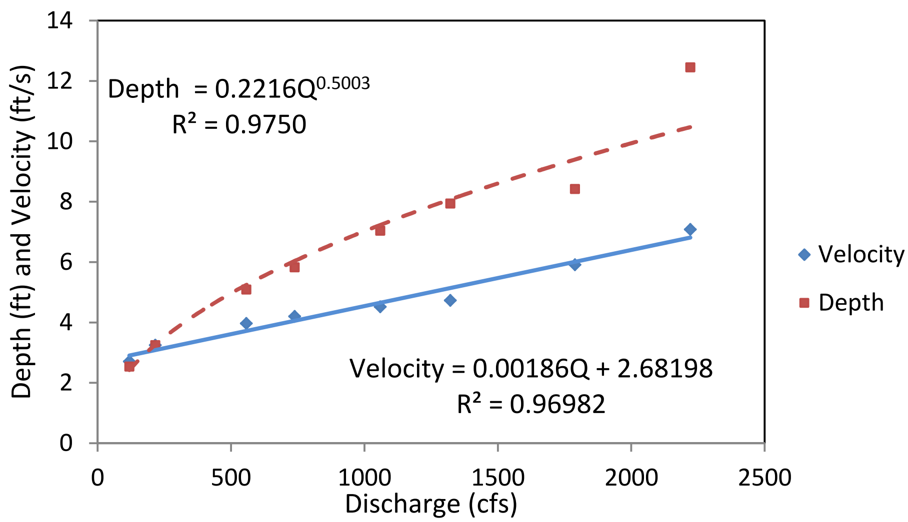

Figure 12- Flow depth and velocity rating curve for Grassy Run.

Stream Power

Cumulative Effective Stream Power at the Grassy Run project site incorporated HEC-1 generated flow rates and runoff hydrograph. Amounts of Cumulative Effective Stream Power associated with a variety of return period storms are summarized in Table 45, including amounts associated with flow rates (and corresponding flow velocities and depths) that were generated using the lower and upper 90% confidence interval precipitation depths. The Average Annual Cumulative Effective Stream Power, which accounts for the Cumulative Effective Stream Power associated with each return period storm and the annual probability of each of these storms, is shown in Table 46.

Table 45 – Scour event Cumulative Effective Stream Power Ω (approach flow) for Grassy Run – based on HEC-1 generated flow rates and hydrograph.

Return Period (yr)	Cumulative Effective Stream Power , Ω ($ft\cdot lb/ft^2$)	90% Confidence Interval	
		Lower ($ft\cdot lb/ft^2$)	Upper ($ft\cdot lb/ft^2$)
2	0	0	0
5	448	149	1,109
10	1,512	917	3,667
25	3,956	2,754	6,838
50	6,641	4,868	11,356
100	10,275	7,590	15,778
200	14,970	11,157	21,467
500	22,993	17,259	31,124

Table 46 – Average Annual Cumulative Effective Stream Power Ω (approach flow) at Grassy Run.

Increment (return period range)	λ size	Interval Cumulative Effective Stream Power, Ω (ft·lb/ft^2)		
		Baseline Value	Lower 90% Conf.	Upper 90% Conf.
2-5	0.300	224	74	554
5-10	0.100	980	533	2,388
10-25	0.060	2,734	1,835	5,253
25-50	0.020	5,298	3,811	9,097
50-100	0.010	8,458	6,229	13,567
100-200	0.005	12,622	9,373	18,622
200-500	0.003	18,982	14,208	26,295
500-	0.002	22,993	17,259	31,124
Avg. Annual Cum. Eff. Stream Power, Ω (ft·lb/ft^2):		686	448	1,272

5 – Caldwell Run

Watershed characteristics

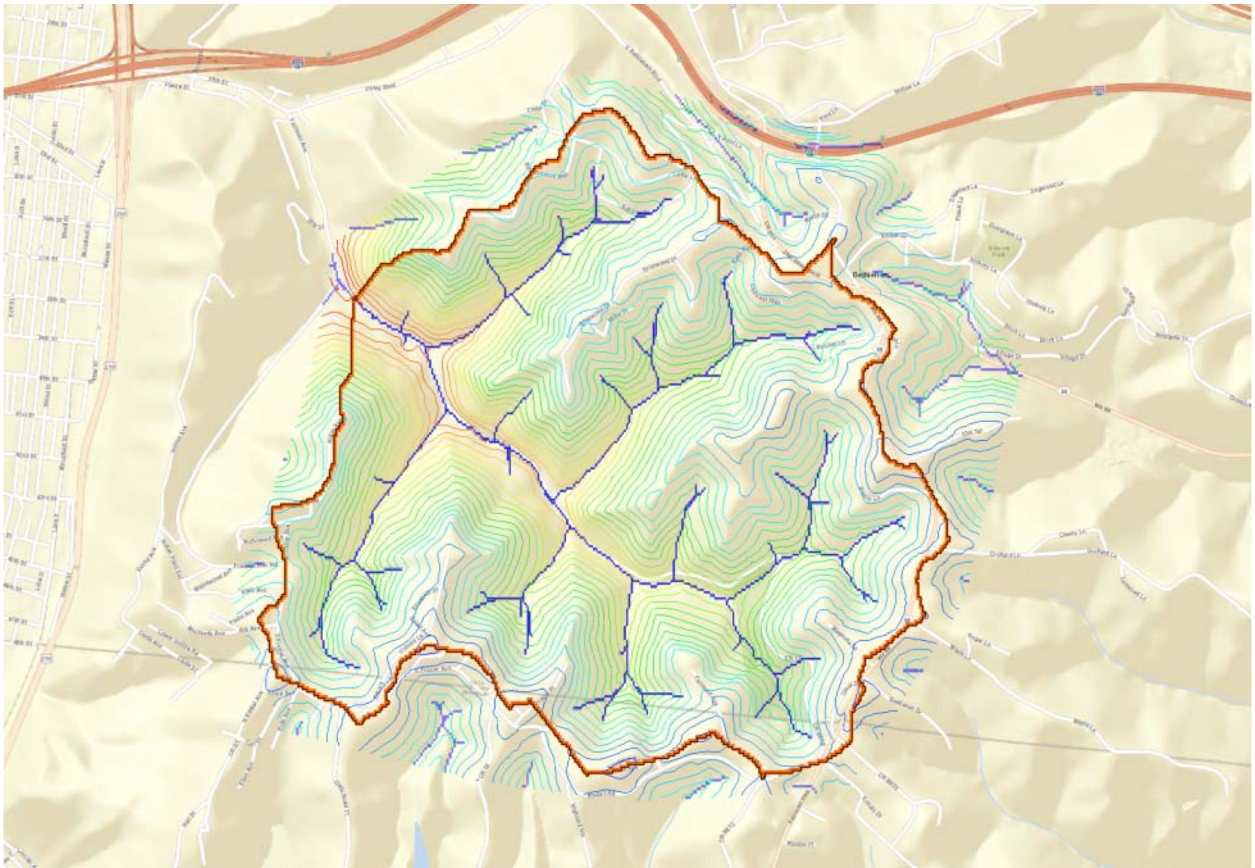

Figure 13- Image of delineated Caldwell Run watershed.

Several relevant hydrologic parameters for the Caldwell Run watershed are provided below in Table 47.

Table 47 - Watershed characteristics for Caldwell Run.

Parameter	**Value**
Location	Lat: 40.043708 Long: -80.711797
Region – USGS SIR2010-5033	Western Plateaus
Basin Area (mi^2)	1.46
Average basin elevation (ft)	1113
Average basin overland slope (%)	30.9
Maximum flow distance (ft)	9881
Slope along maximum flow distance (%)	5.7
Shape factor (basin length / basin width)	1.50
Sinuosity factor of stream (max. stream length / basin length)	1.18
Runoff Curve Number	58.0
Time of Concentration (hr.)	1.092

A summary of the parameters utilized to compute the weighted watershed Curve Number is provided in the table below, including Natural Resources Conservation Service Soil Survey Geographic Database (SSURGO) derived hydrologic soil group, National Land Cover Database 2006 derived land cover classification, corresponding Curve Number, and sub-element area.

Table 48 – Soil type, land cover, and curve number characteristics for Caldwell Run.

Hydrologic Soil Group	Land Cover Description	CN	Area (mi^2)
D	Developed Low Intensity	84	0.02
D	Developed Open Space	80	0.012
B	Developed Low Intensity	68	0.087
D	Deciduous Forest	77	0.012
D	Developed Medium Intensity	85	0.008
B	Deciduous Forest	55	1.107
B	Developed Open Space	61	0.144
C	Deciduous Forest	70	0.047
B	Developed Medium Intensity	70	0.005
B	Pasture/Hay	61	0
D	Evergreen Forest	77	0.001
B	Evergreen Forest	55	0.006
B	Grassland/Herbaceous	58	0.008
D	Grassland/Herbaceous	78	0.001
C	Developed Low Intensity	79	0.001
C	Developed Open Space	74	0.001
D	Developed High Intensity	95	0.001
Weighted CN = 58.0			

Flow synthesis

Stream Gage Data. In Table 49 below, characteristics of the stream gage stations in proximity to the Caldwell Run project site are summarized, including period of record and data coverage within that period, location, distance from the project area, elevation, drainage area upstream of the gage, and whether the available data consists of Daily Discharge (DD) or Annual Peak (AP).

Caldwell Run, with a drainage area of only 1.45 mi^2, is an ungauged stream. The nearest gauged stream, Wheeling Creek at Elm Grove, gauges a watershed that is 281 mi^2 and only since 1940. Additionally, none of the other gauged streams in the nearby area are well-suited for direct flow extrapolation to the project site, due to excessive distance, significant differences in watershed drainage area, and data availability periods that do not overlap with the time since construction. Therefore, flow synthesis at the Caldwell Run site utilizes two methods: the applicable USGS peak flow regression equations, and creation of a watershed model that utilizes probabilistic rainfall data.

Table 49 – Stream gage stations near Caldwell Run.

Project / Stream Gage Location	**USGS Station ID**	**Period**	**Covg. (%)**	**Lat.**	**Long.**	**Dist. (mi.)**	**Elev. (ft.)**	**Drainage Area (mi^2)**	**Type**
Caldwell Run	-	*1924-Present*	-	*40.0437*	*-80.712*	-	*753*	*1.46*	-
WHEELING CREEK AT ELM GROVE, WV	03112000	1940-2011	100	40.044	-80.661	2.7	667.1	281	DD
Short Creek near Dillonvale OH	03111500	1941-Present	100	40.19	-80.73	10.2	676.1	123	DD
Brush Run near Buffalo, PA	03111150	1960-1985	85	40.2	-80.41	19.3	954.2	10.3	DD

Rainfall Data. A summary of weather stations near the project location are provided in the table below, including Station ID, Period of Record, data coverage during the Period of Record, location, elevation, and distance from the project site. Neither of the available weather stations are suitable for direct integration into the Caldwell Run watershed model, Pike Island Lock and Dam because it is too far away, and McMechen Dam because the period of record does not cover the entire time since the bridge at Caldwell Run was constructed.

Table 50 – Weather stations near Caldwell Run.

Station Name	Station ID	Period of Record	Coverage (%)	Lat.	Long.	Elev. (ft.)	Dist. (mi.)
Pike Island Lock and Dam	USC00467018 USC00469484 USC00469492	1916-Present	99	40.147	-80.701	640	7.2
McMechen Dam	USC00465847	1933-1975	100	39.98333	-80.7333	659	4.3

Point precipitation frequency estimates for a 24-hour duration storm at the watershed, based on partial duration series NOAA Atlas 14 data available at the Hydrometeorological Design Studies Center's Precipitation Frequency Data Server are provided in Table 51.

Table 51 – Precipitation depth at Caldwell Run for a 24-hour duration storm, including 90% confidence interval.

Average Recurrence Interval (yr.)	Precipitation Depth (in.)	90% Confidence Interval	
		Precip. Depth, Lower (in.)	Precip. Depth, Upper (in.)
2	2.41	2.25	2.60
5	2.95	2.75	3.17
10	3.39	3.15	3.64
25	4.00	3.71	4.29
50	4.50	4.17	4.82
100	5.03	4.63	5.37
200	5.57	5.11	5.94
500	6.33	5.77	6.74

Development of flow rates at Caldwell Run for a variety of return period storms utilized three methods: USGS-provided regional regression equations (see Table 52), a TR-55 based watershed model that incorporated 24-hour storm duration precipitation depths (see Table 53), and a HEC-1 based watershed model that incorporated 24-hour storm duration precipitation depths (see Table 54). Since calculation of cumulative effective stream power requires characterization of flow rate as a function of time – and not simply the peak flow rate associated with a storm event – the runoff hydrographs generated by the HEC-1 analysis were utilized for this purpose.

Table 52 – USGS SIR-2010-5033 derived peak discharges, average prediction error, and corresponding 90% confidence interval for the Caldwell Run watershed.

Return Period (yr)	Peak Discharge, Q (cfs)	Avg. Prediction Error (%)	90% Confidence Interval	
			Q, Lower (cfs)	Q, Upper (cfs)
2	170	32.2	80	260
5	289	30.0	146	432
10	380	29.7	195	566
25	507	30.3	254	760
50	611	31.3	296	925
100	719	32.5	335	1103
200	833	33.9	369	1298
500	995	36.1	404	1586

Table 53 – Peak discharges derived from TR-55 watershed model of Caldwell Run

Return Period (yr)	Peak Discharge, Q (cfs)	90% Confidence Interval	
		Q, Lower (cfs)	Q, Upper (cfs)
2	15	9	26
5	57	38	83
10	114	80	154
25	222	167	285
50	333	258	413
100	468	365	561
200	619	489	729
500	850	678	982

Table 54 – Peak discharges derived from HEC-1 watershed model of Caldwell Run.

Return Period (yr)	Peak Discharge, Q (cfs)	90% Confidence Interval	
		Q, Lower (cfs)	Q, Upper (cfs)
2	15	9	25
5	53	36	78
10	107	76	145
25	206	156	261
50	304	238	373
100	421	331	502
200	553	440	656
500	768	608	892

Hydraulics

An isometric view of the HEC-RAS Perspective Plot for Caldwell Run is shown in Figure 14, the underlying geometry for which is based on a field survey that was performed at the site. Manning's n-values for the site are summarized in Table 55. The result of HEC-RAS modeling at the site is a flow depth and velocity rating curve, presented in Figure 15.

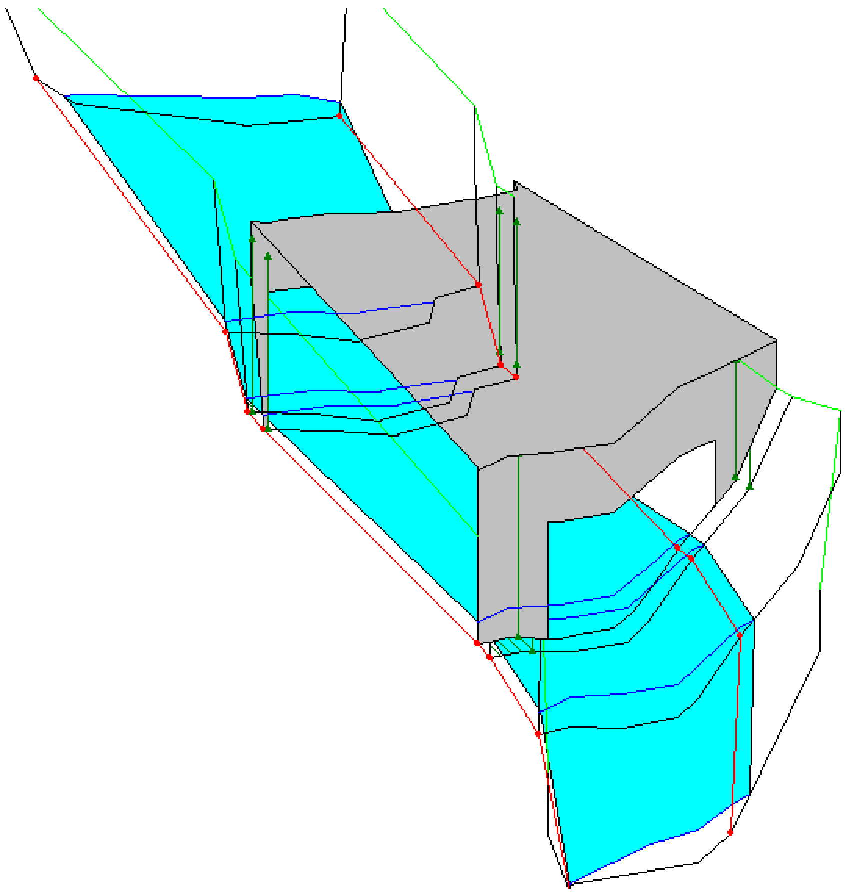

Figure 14- HEC-RAS X-Y-Z Perspective Plot for Caldwell Run.

Table 55 – Manning's n-values utilized in HEC-RAS model for Caldwell Run.

	Left Overbank	Channel	Right Overbank
U/S of Bridge	0.09	0.1	0.09
D/S of Bridge	0.09	0.1	0.02

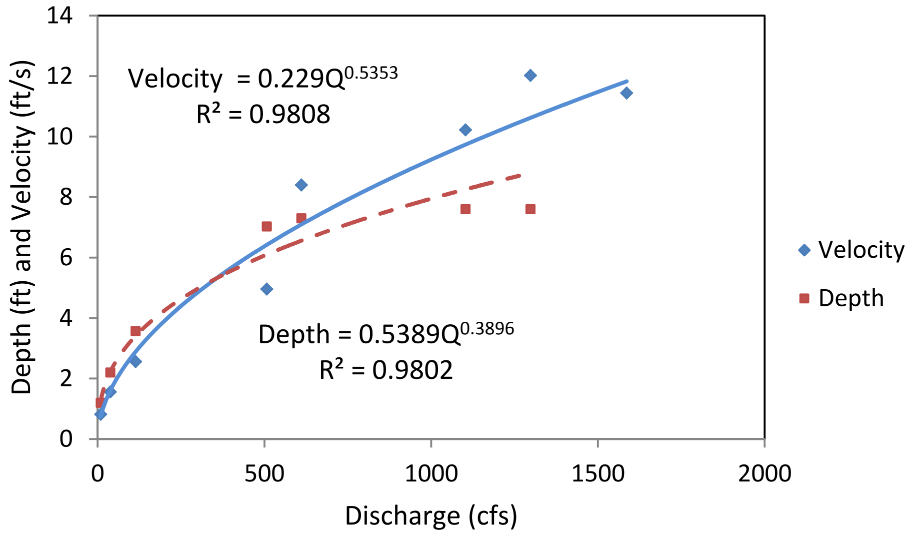

Figure 15- Flow depth and velocity rating curve for Caldwell Run

Stream Power

Cumulative Effective Stream Power at the Caldwell Run project site incorporated HEC-1 generated flow rates and runoff hydrograph. Amounts of Cumulative Effective Stream Power associated with a variety of return period storms are summarized in Table 56, including amounts associated with flow rates (and corresponding flow velocities and depths) that were generated using the lower and upper 90% confidence interval precipitation depths. The Average Annual Cumulative Effective Stream Power, which accounts for the Cumulative Effective Stream Power associated with each return period storm and the annual probability of each of these storms, is shown inTable 57.

Table 56 – Scour event Cumulative Effective Stream Power Ω (approach flow) for Caldwell Run – based on HEC-1 generated flow rates and hydrograph.

Return Period (yr)	Cumulative Effective Stream Power , Ω (ft·lb/ft^2)	90% Confidence Interval	
		Lower (ft·lb/ft^2)	Upper (ft·lb/ft^2)
2	0	0	0
5	7,849	6,011	12,802
10	24,130	18,432	35,586
25	63,749	47,920	87,822
50	110,757	83,736	150,773
100	175,550	130,742	228,473
200	257,850	192,299	326,532
500	401,395	298,260	495,220

Table 57 – Average Annual Cumulative Effective Stream Power Ω (approach flow) at Caldwell Run.

Increment (return period range)	λ size	Interval Cumulative Effective Stream Power, Ω (ft·lb/ft^2)		
		Baseline Value	Lower 90% Conf.	Upper 90% Conf.
2-5	0.300	3,924	3,005	6,401
5-10	0.100	15,989	12,221	24,194
10-25	0.060	43,939	33,176	61,704
25-50	0.020	87,253	65,828	119,297
50-100	0.010	143,154	107,239	189,623
100-200	0.005	216,700	161,520	277,502
200-500	0.003	329,623	245,279	410,876
500-	0.002	401,395	298,260	495,220
Avg. Annual Cum. Eff. Stream Power, Ω (ft·lb/ft^2):		11,464	8,643	15,935

6 – Paden Fork

Watershed characteristics

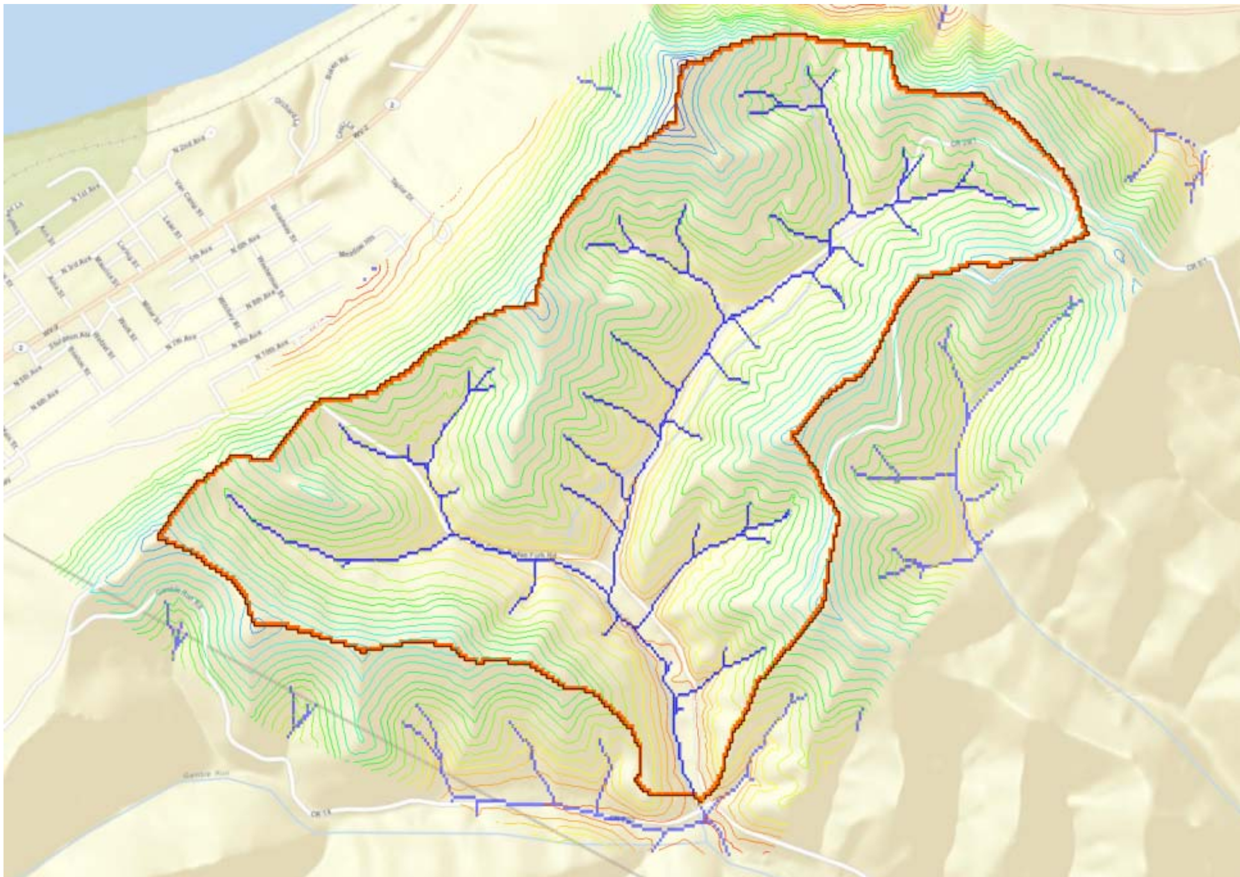

Figure 16- Image of delineated Paden Fork watershed.

Several relevant hydrologic parameters for the Paden Fork watershed are provided below in Table 58.

Table 58 - Watershed characteristics for Paden Fork.

Parameter	Value
Location	Lat: 39.592561 Long: -80.909456
Region – USGS SIR2010-5033	Western Plateaus
Basin Area (mi^2)	1.01
Average basin elevation (ft)	1019
Average basin overland slope (%)	27.1
Maximum flow distance (ft)	9866
Slope along maximum flow distance (%)	4.9
Shape factor (basin length / basin width)	1.92
Sinuosity factor of stream (max. stream length / basin length)	1.25
Runoff Curve Number	70.2
Time of Concentration (hr.)	0.835

A summary of the parameters utilized to compute the weighted watershed Curve Number is provided in the table below, including Natural Resources Conservation Service Soil Survey Geographic Database (SSURGO) derived hydrologic soil group, National Land Cover Database 2006 derived land cover classification, corresponding Curve Number, and sub-element area.

Table 59 – Soil type, land cover, and curve number characteristics for Paden Fork.

Hydrologic Soil Group	Land Cover Description	CN	Area (mi^2)
C	Pasture/Hay	74	0.057
C	Deciduous Forest	70	0.678
C	Developed Open Space	74	0.023
C	Evergreen Forest	70	0.003
C	Grassland/Herbaceous	71	0.002
D	Evergreen Forest	77	0.001
D	Pasture/Hay	78	0.012
D	Deciduous Forest	77	0.118
D	Developed Open Space	80	0.016
B	Deciduous Forest	55	0.041
B	Developed Open Space	61	0.05
B	Pasture/Hay	61	0.011
B	Grassland/Herbaceous	58	0.002
Weighted CN = 70.2			

Flow synthesis

Stream Gage Data. In the table below, characteristics of the stream gage stations in proximity to the Paden Fork project site are summarized, including period of record and data coverage within that period, location, distance from the project area, elevation, drainage area upstream of the gage, and whether the available data consists of Daily Discharge (DD) or Annual Peak (AP).

Paden Fork, with a drainage area of only 1.01 mi^2, is an ungauged stream. The nearest gauged stream, Train Run near Antioch, Ohio, gauges a watershed that is 5.45 mi^2 , which is an area that might be reasonably extrapolated to synthesize stream gage data at the project site. However, the period of data availability for the Trail Run stream gage station is only 1978-1987, less than one-quarter of the time since the Paden Fork bridge was constructed. Therefore, flow synthesis at the Paden Fork project site utilizes two methods: the applicable USGS peak flow regression equations, and creation of a watershed model that utilizes probabilistic rainfall data.

Table 60 – Stream gage stations near Paden Fork.

Project / Stream Gage Location	USGS Station ID	Period	Covg. (%)	Lat.	Long.	Dist. (mi.)	Elev. (ft.)	Drainage Area (mi^2)	Type
Paden Fork	-	*1973-Present*	-	*39.5926*	*-80.909*	-	*803*	*1.01*	-
Trail Run near Antioch OH	03115280	1978-1987	100	39.625	-81.048	7.7	727	5.45	AP

Rainfall Data. A summary of weather stations near the project location are provided in the table below, including Station ID, Period of Record, data coverage during the Period of Record, location, elevation, and distance from the project site. When considering distance and the range of dates during which data was collected at the weather stations, neither is suitable for direct watershed model integration.

Table 61 – Weather stations near Paden Fork.

Station Name	Station ID	Period of Record	Coverage (%)	Lat.	Long.	Elev. (ft.)	Dist. (mi.)
Hannibal Lock and Dam	USC00333500	1975-Present	99	39.666	-80.866	620	5.6
New Martinsville 4N	USC00466467	1892-1975	97	39.6833	-80.85	702	7.0

Point precipitation frequency estimates for a 24-hour duration storm at the watershed, based on partial duration series NOAA Atlas 14 data available at the Hydrometeorological Design Studies Center's Precipitation Frequency Data Server are provided in Table 62.

Table 62 – Precipitation depth at Paden Fork for a 24-hour duration storm, including 90% confidence interval.

Average Recurrence Interval (yr.)	Precipitation Depth (in.)	90% Confidence Interval	
		Precip. Depth, Lower (in.)	Precip. Depth, Upper (in.)
2	2.53	2.33	2.76
5	3.07	2.83	3.36
10	3.52	3.23	3.84
25	4.15	3.79	4.52
50	4.66	4.25	5.07
100	5.19	4.72	5.64
200	5.75	5.20	6.24
500	6.53	5.86	7.08

Development of flow rates at Paden Fork for a variety of return period storms utilized three methods: USGS-provided regional regression equations (see Table 63), a TR-55 based watershed model that incorporated 24-hour storm duration precipitation depths (see Table 64), and a HEC-1 based watershed model that incorporated 24-hour storm duration precipitation depths (see Table 65). Since calculation of cumulative effective stream power requires characterization of flow rate as a function of time – and not simply the peak flow rate associated with a storm event – the runoff hydrographs generated by the HEC-1 analysis were utilized for this purpose. Table 63 – USGS SIR-2010-5033 derived peak discharges, average prediction error, and corresponding 90% confidence interval for the Paden Fork watershed.

Return Period (yr)	Peak Discharge, Q (cfs)	Avg. Prediction Error (%)	90% Confidence Interval	
			Q, Lower (cfs)	Q, Upper (cfs)
2	130	32.2	61	199
5	223	30.0	113	332
10	294	29.7	150	438
25	394	30.3	197	590
50	475	31.3	231	720
100	561	32.5	261	861
200	651	33.9	288	1015
500	780	36.1	317	1243

Table 64 – Peak discharges derived from TR-55 watershed model of Paden Fork.

Return Period (yr)	Peak Discharge, Q (cfs)	90% Confidence Interval	
		Q, Lower (cfs)	Q, Upper (cfs)
2	139	104	185
5	252	199	321
10	360	289	443
25	527	430	631
50	671	555	791
100	827	688	963
200	997	830	1149
500	1240	1030	1415

Table 65 – Peak discharges derived from HEC-1 watershed model of Paden Fork.

Return Period (yr)	Peak Discharge, Q (cfs)	90% Confidence Interval	
		Q, Lower (cfs)	Q, Upper (cfs)
2	129	97	170
5	230	183	292
10	328	264	402
25	477	390	570
50	606	502	714
100	746	622	869
200	900	749	1038
500	1121	931	1281

Hydraulics

An isometric view of the HEC-RAS Perspective Plot for Paden Fork is shown in Figure 17, the underlying geometry for which is based on a field survey that was performed at the site. Manning's n-values for the site are summarized in Table 66. The result of HEC-RAS modeling at the site is a flow depth and velocity rating curve, presented in Figure 18.

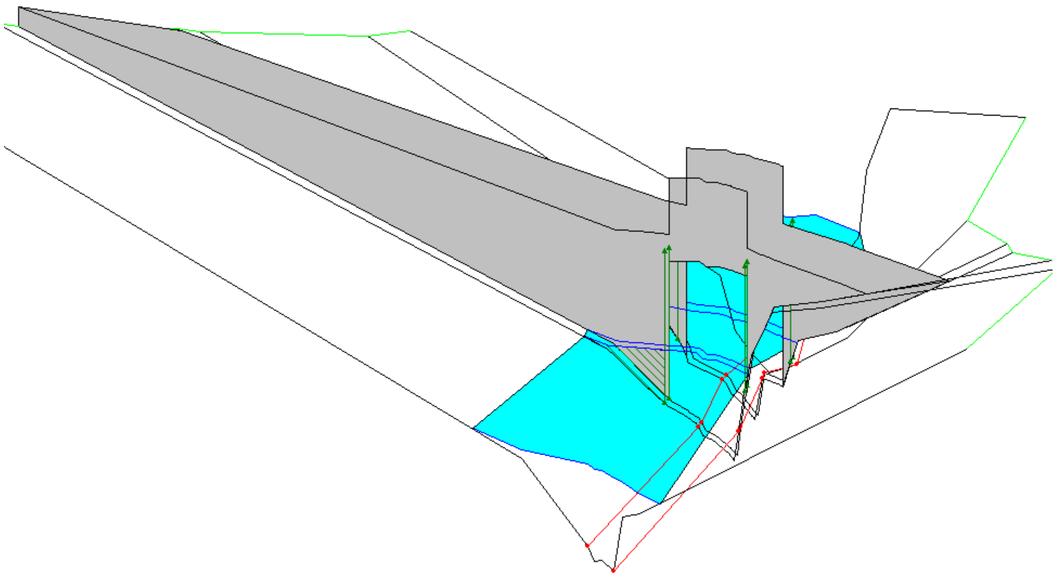

Figure 17- HEC-RAS X-Y-Z Perspective Plot for Paden Fork.

Table 66 – Manning's n-values utilized in HEC-RAS model for Paden Fork.

	Left Overbank	Channel	Right Overbank
U/S of Bridge	0.1	0.08	0.1
D/S of Bridge	0.1	0.08	0.1

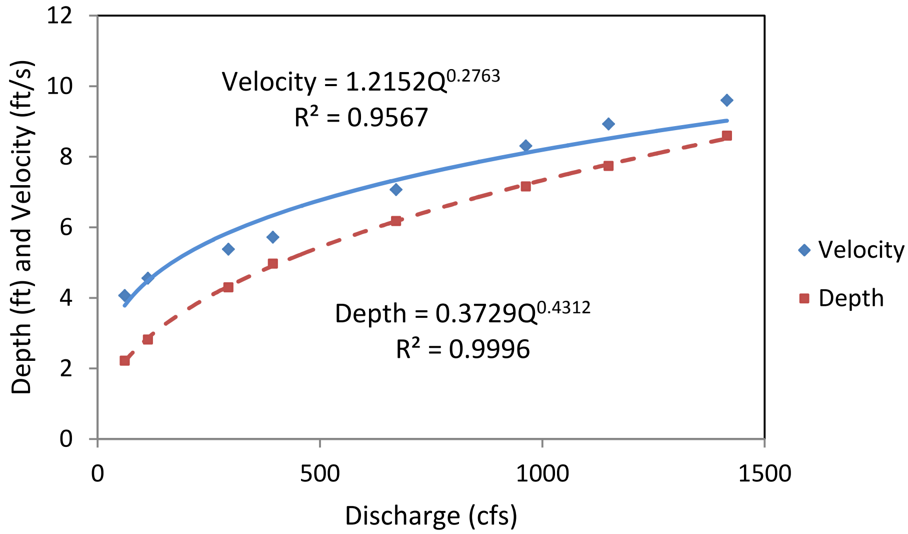

Figure 18- Flow depth and velocity rating curve for Paden Fork.

Stream Power

Cumulative Effective Stream Power at the Leatherwood Road project site incorporated HEC-1 generated flow rates and runoff hydrograph. Amounts of Cumulative Effective Stream Power associated with a variety of return period storms are summarized in Table 67, including amounts associated with flow rates (and corresponding flow velocities and depths) that were generated using the lower and upper 90% confidence interval precipitation depths. The Average Annual Cumulative Effective Stream Power, which accounts for the Cumulative Effective Stream Power associated with each return period storm and the annual probability of each of these storms, is shown in Table 68.

Table 67 – Scour event Cumulative Effective Stream Power Ω (approach flow) for Paden Fork – based on HEC-1 generated flow rates and hydrograph.

Return Period (yr)	Cumulative Effective Stream Power , Ω (ft·lb/ft^2)	90% Confidence Interval	
		Lower (ft·lb/ft^2)	Upper (ft·lb/ft^2)
2	0	0	0
5	12,860	12,570	35,866
10	28,780	27,464	51,942
25	57,792	54,381	96,758
50	82,800	78,369	126,404
100	109,535	103,633	159,160
200	138,317	130,003	194,888
500	180,856	168,767	258,512

Table 68 – Average Annual Cumulative Effective Stream Power Ω (approach flow) at Paden Fork.

Increment (return period range)	λ size	Interval Cumulative Effective Stream Power, Ω (ft·lb/ft^2)		
		Baseline Value	Lower 90% Conf.	Upper 90% Conf.
2-5	0.300	6,430	6,285	17,933
5-10	0.100	20,820	20,017	43,904
10-25	0.060	43,286	40,922	74,350
25-50	0.020	70,296	66,375	111,581
50-100	0.010	96,167	91,001	142,782
100-200	0.005	123,926	116,818	177,024
200-500	0.003	159,586	149,385	226,700
500-	0.002	180,856	168,767	258,512
Avg. Annual Cum. Eff. Stream Power, Ω (ft·lb/ft^2):		10,436	9,950	19,973

7 – Audra Park

Watershed characteristics

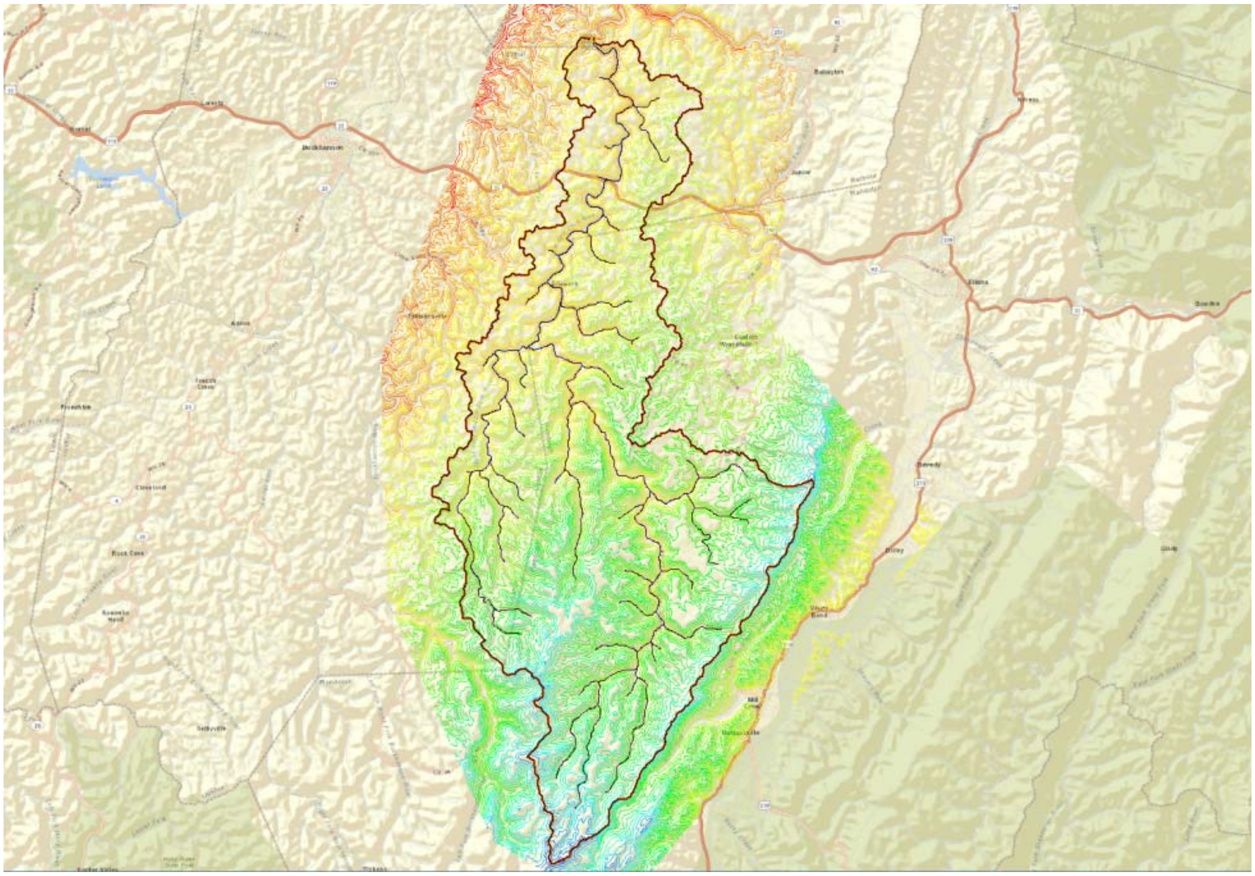

Figure 19- Image of delineated Audra Park watershed.

Although it was stream gauge data – and not watershed characteristics – that were utilized to define the flood frequency parameters at Audra Park, the same hydrologic parameters as have been summarized for other project sites are presented in

Table 69 for purposes of comparison. When generating a runoff hydrograph for purposes of scaling it against the PeakFQ generated peak flow rates, multiple sub-basins were created. Additional details about these sub-basins can be found in the digital watershed model files for this location.

Table 69 - Watershed characteristics for Audra Park.

Parameter	Value
Location	Lat: 39.041258 Long: -80.067811
Region – USGS SIR2010-5033	Central Mountains
Basin Area (mi^2)	149.3
Average basin elevation (ft)	2497
Average basin overland slope (%)	23.1
Maximum flow distance (ft)	217,934
Slope along maximum flow distance (%)	1.0
Shape factor (basin length / basin width)	4.78
Sinuosity factor of stream (max. stream length / basin length)	1.47
Runoff Curve Number	69.6

Flow synthesis

Stream Gage Data. In the table below, characteristics of the stream gage stations in proximity to the Audra Park project site are summarized, including period of record and data coverage within that period, location, distance from the project area, elevation, drainage area upstream of the gage, and whether the available data consists of Daily Discharge (DD) or Annual Peak (AP).

The Middle Fork River is a gauged stream, with a stream gauge location approximately 0.2 miles upstream from the Audra Park bridge site. Data at the stream gauge is available since 1942, with an overall coverage of approximately 88% during that period. The USGS flood frequency analysis software PKFQWin (USGS, Estimating Magnitude and Frequency of Floods Using the PeakFQ Program, 2006) was used to generate a probabilistic characterization of expected flow rates at the Audra Park project site location for various recurrence interval floods. From the 61 annual flow peaks of available data included in the analysis, and up-scaling the resulting flow rates by 0.9% to account for the additional drainage area at the project site compared to the stream gauge location, the resulting output is presented in Table 71.

Table 70 – Stream gage stations near Audra Park.

Project / Stream Gage Location	**USGS Station ID**	**Period**	**Covg. (%)**	**Lat.**	**Long.**	**Dist. (mi.)**	**Elev. (ft.)**	**Drainage Area (mi^2)**	**Type**
Audra Park, Middle Fork River	-	*1940-Present*	-	*39.0413*	*-80.068*	-	*1686*	*149.30*	-
MIDDLE FORK RIVER AT AUDRA, WV	03052000	1942-Present	88	39.039	-80.068	0.2	1669	148	DD
BUCKHANNON RIVER AT HALL, WV	03053500	1915-Present	100	39.051	-80.115	2.6	1369	277	DD

Table 71 – Flood frequency analysis for the Middle Fork River at Audra Park.

Recurrence Interval (yr)	Discharge (ft^3/s)	95% Confidence Interval	
		Lower	Upper
2	6035	5547	6564
5	8495	7719	9394
10	10239	9140	11510
25	12601	10950	14423
50	14483	12301	16765
100	16474	13682	19247
200	18596	15093	21869
500	21649	17015	25612

Rainfall Data. For reference purposes, a summary of weather stations near the project location are provided in Table 72, including Station ID, Period of Record, data coverage during the Period of Record, location, elevation, and distance from the project site. When considering distance and the range of dates during which data was collected at the weather stations, neither Belington nor Philippi is suitable for direct watershed model integration.

Table 72 – Weather stations near Audra Park.

Station Name	**Station ID**	**Period of Record**	**Coverage (%)**	**Lat.**	**Long.**	**Elev. (ft.)**	**Dist. (mi.)**
Belington	USC00460633	1938-Present	96	39.023	-79.933	1769	7.3
Philippi	USC00466982	1896-2010	94	39.146	-80.041	1300	7.4

Point precipitation frequency estimates for a 24-hour duration storm at the watershed, based on partial duration series NOAA Atlas 14 data available at the Hydrometeorological Design Studies Center's Precipitation Frequency Data Server are provided in Table 73.

Table 73 – Precipitation depth at Audra Park for a 24-hour duration storm, including 90% confidence interval.

Average Recurrence Interval (yr.)	Precipitation Depth (in.)	90% Confidence Interval	
		Precip. Depth, Lower (in.)	Precip. Depth, Upper (in.)
2	2.60	2.40	2.81
5	3.15	2.91	3.42
10	3.61	3.33	3.91
25	4.25	3.91	4.60
50	4.78	4.37	5.15
100	5.33	4.85	5.73
200	5.90	5.36	6.33
500	6.71	6.05	7.19

For purposes of comparison to the USGS PKFQWin derived flood frequency analysis presented in Table 73, the regression methods outlined in USGS SIR-2010-5033 were utilized to estimate flood-frequency discharge relationships for the Audra Park watershed. Peak flow rates, and corresponding average prediction error based on the underlying hydrologic data and statistical methods utilized, are identified in Table 74. Note that the baseline peak discharge flow rates are comparable between the two approaches, but the 95% confidence interval associated with the PKFQWin-derived flood frequency analysis is much narrower in range in comparison to the USGS regression approach, due to the long period of data availability very near the exact location of interest.

Table 74 – USGS SIR-2010-5033 derived peak discharges, average prediction error, and corresponding 90% confidence interval for the Audra Park watershed.

Return Period (yr)	Peak Discharge, Q (cfs)	Avg. Prediction Error (%)	90% Confidence Interval	
			Q, Lower (cfs)	Q, Upper (cfs)
2	5487	34.2	2400	8574
5	7972	35.1	3369	12575
10	9803	37.4	3772	15834
25	12244	41.2	3946	20542
50	14205	44.4	3830	24579
100	16294	47.9	3455	29133
200	18382	51.5	2809	33954
500	21375	56.3	1579	41171

Hydraulics

An isometric view of the HEC-RAS Perspective Plot for Audra Park is shown in Figure 20, the underlying geometry for which is based on a field survey that was performed at the site. Manning's n-values for the site are summarized in

Table 75. The result of HEC-RAS modeling at the site is a flow depth and velocity rating curve, presented in Figure 21.

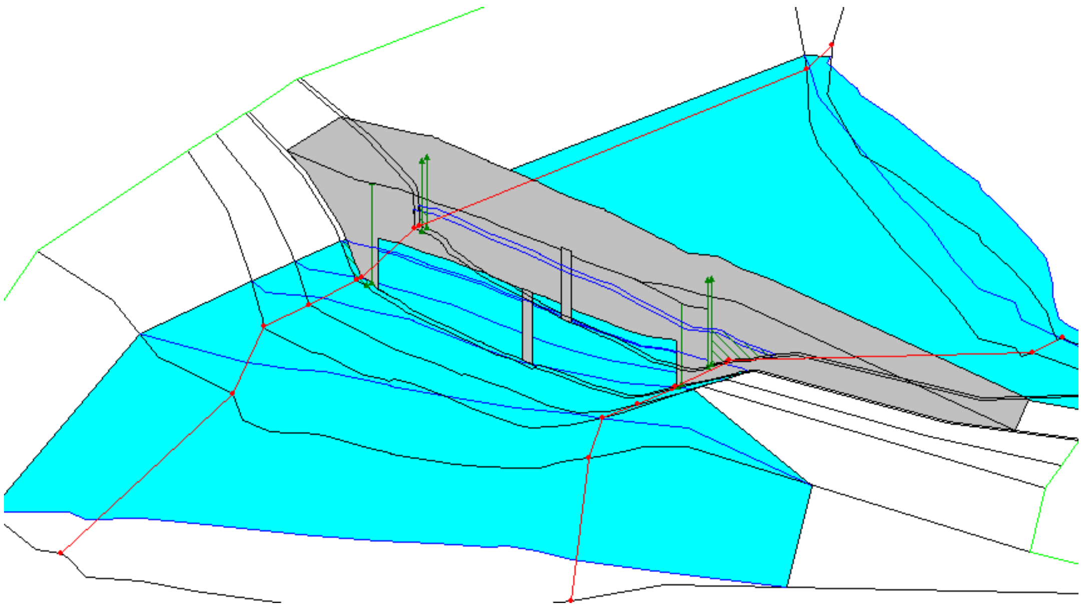

Figure 20- HEC-RAS X-Y-Z Perspective Plot for Audra Park.

Table 75 – Manning's n-values utilized in HEC-RAS model for Audra Park.

	Left Overbank	Channel	Right Overbank
U/S of Bridge	0.12	0.11	0.12
D/S of Bridge	0.12	0.11	0.12

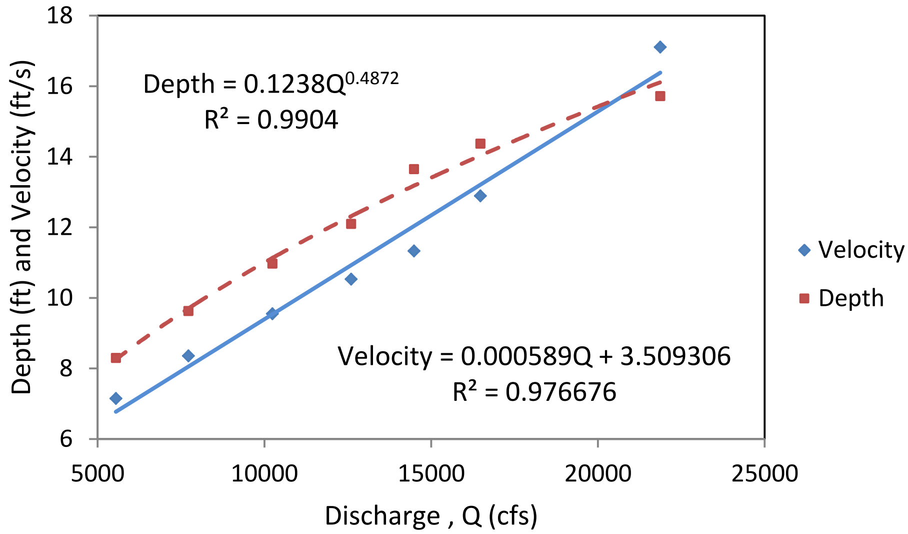

Figure 21- Flow depth and velocity rating curve for Audra Park.

Stream Power

Cumulative Effective Stream Power at the Audra Park project site incorporated PeakFQ generated peak flow rates and a runoff hydrograph shape that was determined through HEC-1 modeling. Amounts of Cumulative Effective Stream Power associated with a variety of return period storms are summarized in Table 76 (approach flow) and Table 77 (at pier), including amounts associated with flow rates (and corresponding flow velocities and depths) that were generated using the lower and upper 90% confidence interval precipitation depths. The Average Annual Cumulative Effective Stream Power, which accounts for the Cumulative Effective Stream Power associated with each return period storm and the annual probability of each of these storms, is shown in Table 78 and Table 79.

Table 76 – Scour event Cumulative Effective Stream Power Ω (approach flow) for Audra Park – based on PeakFQ generated flow rates and HEC-1 generated hydrograph shape.

Return Period (yr)	Cumulative Effective Stream Power , Ω (ft·lb/ft^2)	95% Confidence Interval	
		Lower (ft·lb/ft^2)	Upper (ft·lb/ft^2)
2	0	0	0
5	214,814	173,585	280,132
10	503,293	382,988	672,588
25	1,032,243	736,295	1,423,248
50	1,567,970	1,062,098	2,217,385
100	2,269,357	1,428,912	3,299,200
200	3,127,872	1,915,862	4,458,842
500	4,534,220	2,577,870	6,764,278

Table 77 – Scour event Cumulative Effective Stream Power Ω (pier) for Audra Park – based on PeakFQ generated flow rates and HEC-1 generated hydrograph shape.

Return Period (yr)	Cumulative Effective Stream Power , Ω (ft·lb/ft^2)	95% Confidence Interval	
		Lower (ft·lb/ft^2)	Upper (ft·lb/ft^2)
2	0	0	0
5	620,814	501,660	809,583
10	1,454,517	1,106,835	1,943,781
25	2,983,183	2,127,892	4,113,188
50	4,531,434	3,069,463	6,408,244
100	6,558,441	4,129,555	9,534,688
200	9,039,549	5,536,842	12,886,054
500	13,103,896	7,450,044	19,548,765

Table 78 – Average Annual Cumulative Effective Stream Power Ω (approach flow) at Audra Park.

Increment (return period range)	λ size	Interval Cumulative Effective Stream Power, Ω (ft·lb/ft^2)		
		Baseline Value	Lower 90% Conf.	Upper 90% Conf.
2-5	0.300	107,407	86,792	140,066
5-10	0.100	359,054	278,286	476,360
10-25	0.060	767,768	559,641	1,047,918
25-50	0.020	1,300,107	899,196	1,820,317
50-100	0.010	1,918,664	1,245,505	2,758,293
100-200	0.005	2,698,614	1,672,387	3,879,021
200-500	0.003	3,831,046	2,246,866	5,611,560
500	0.002	4,534,220	2,577,870	6,764,278
Avg. Annual Cum. Eff. Stream Power, Ω (ft·lb/ft^2):		193,437	138,142	266,279

Table 79 – Average Annual Cumulative Effective Stream Power Ω (pier) at Audra Park.

Increment (return period range)	λ size	Interval Cumulative Effective Stream Power, Ω ($ft \cdot lb/ft^2$)		
		Baseline Value	90% Lower	90% Upper
2-5	0.300	310,407	250,830	404,791
5-10	0.100	1,037,665	804,247	1,376,682
10-25	0.060	2,218,850	1,617,364	3,028,484
25-50	0.020	3,757,309	2,598,678	5,260,716
50-100	0.010	5,544,938	3,599,509	7,971,466
100-200	0.005	7,798,995	4,833,198	11,210,371
200-500	0.003	11,071,722	6,493,443	16,217,410
500	0.002	13,103,896	7,450,044	19,548,765
Avg. Annual Cum. Eff. Stream Power, Ω ($ft \cdot lb/ft^2$):		559,033	399,231	769,545

8 – Laurel Fork

Watershed characteristics

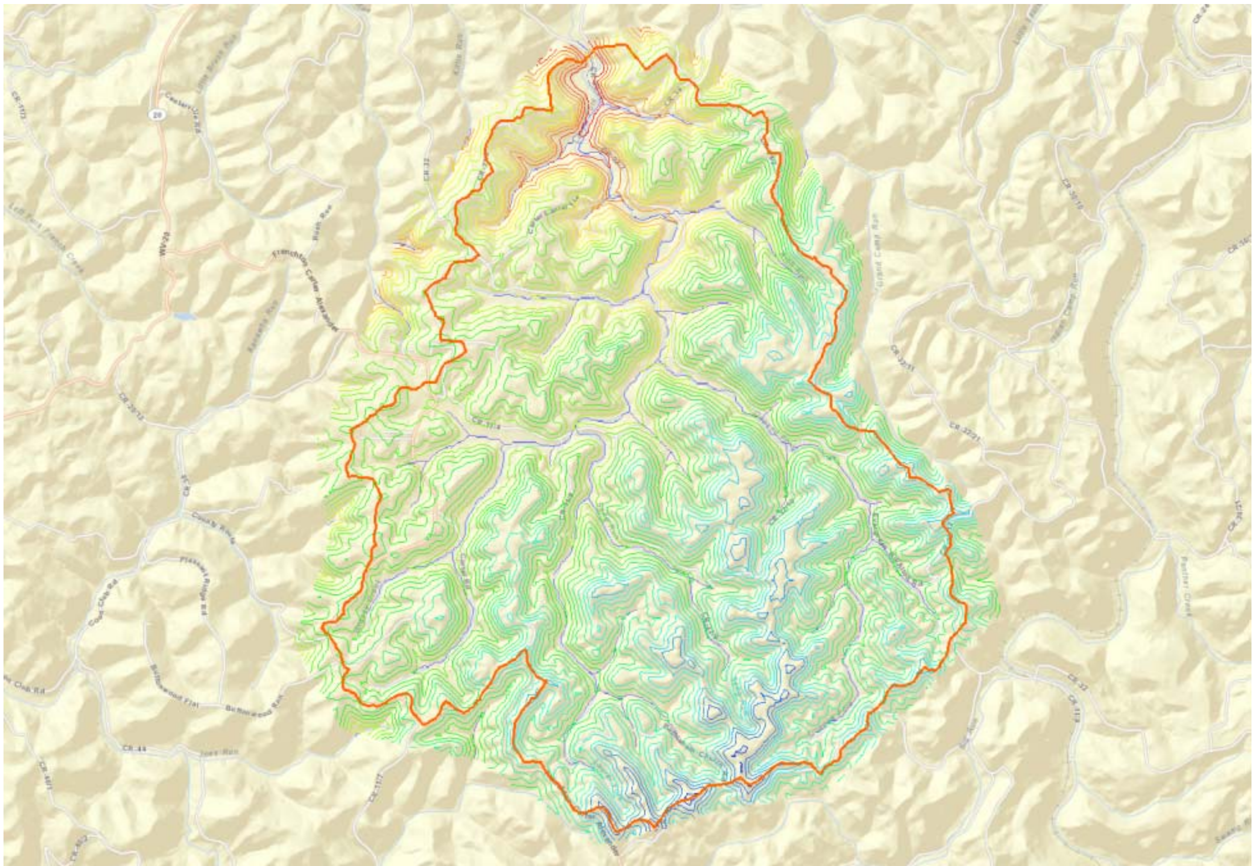

Figure 22- Image of delineated Laurel Fork watershed.

Several relevant hydrologic parameters for the Laurel Fork watershed are provided below in Table 80.

Table 80 - Watershed characteristics for Laurel Fork.

Parameter	**Value**
Location	Lat: 38.875336 Long: -80.264339
Region – USGS SIR2010-5033	Central Mountains
Basin Area (mi^2)	11.74
Average basin elevation (ft)	1949
Average basin overland slope (%)	25.2
Maximum flow distance (ft)	37,714
Slope along maximum flow distance (%)	2.3
Shape factor (basin length / basin width)	1.97
Sinuosity factor of stream (max. stream length / basin length)	1.46
Runoff Curve Number	70.1
Time of Concentration (hr.)	2.574

A summary of the parameters utilized to compute the weighted watershed Curve Number is provided in the table below, including Natural Resources Conservation Service Soil Survey Geographic Database (SSURGO) derived hydrologic soil group, National Land Cover Database 2006 derived land cover classification, corresponding Curve Number, and sub-element area.

Table 81 – Soil type, land cover, and curve number characteristics for Laurel Fork.

Hydrologic Soil Group	**Land Cover Description**	**CN**	**Area (mi^2)**
C	Pasture/Hay	74	0.393
C	Developed Open Space	74	0.629
C	Deciduous Forest	70	9.654
C	Cultivated Crops	82	0.294
D	Pasture/Hay	78	0.003
D	Deciduous Forest	77	0.013
D	Developed Open Space	80	0.002
B	Deciduous Forest	55	0.46
C	Barren Land	91	0.032
B	Developed Open Space	61	0.037
B	Pasture/Hay	61	0.043
B	Cultivated Crops	75	0.054
C	Mixed Forest	70	0.094
C	Evergreen Forest	70	0.035
B	Mixed Forest	55	0.01
B	Evergreen Forest	55	0.002
B	Barren Land	86	0.002
Weighted CN = 70.1			

Flow synthesis

Stream Gage Data. In Table 82, characteristics of the stream gage stations in proximity to the Laurel Fork project site are summarized, including period of record and data coverage within that period, location, distance from the project area, elevation, drainage area upstream of the gage, and whether the available data consists of Daily Discharge (DD) or Annual Peak (AP).

The Laurel Fork stream is ungauged. The nearest gauged stream, Mud Lick Run near Buckhannon, West Virginia, gauges a watershed that is nine miles away from the project site, approximately one-fifth the size, and for which only 51% data availability exists since the gauge has been in operation. To extrapolate gage data results to the characteristics of the Laurel Fork watershed, the flow rates from the Mud Lick Run stream gage have been scaled-up by a factor of 5.04. Since there is not a strict linear relationship between watershed area and runoff flows, this approach provides, at best, resulting data that is usable only for reference. In view of this, the PKFQWin flood frequency analysis results presented in Table 82 are presented only for purposes of comparison, and are not utilized in subsequent hydraulic calculations. Primary flow synthesis at the Laurel Fork site utilizes two methods, the results of which are presented hereafter: the applicable USGS peak flow regression equations, and creation of a watershed model that utilizes probabilistic rainfall data.

Table 82 – Stream gage stations near Laurel Fork.

Project / Stream Gage Location	**USGS Station ID**	**Period**	**Covg. (%)**	**Lat.**	**Long.**	**Dist. (mi.)**	**Elev. (ft.)**	**Drainage Area (mi^2)**	**Type**
Laurel Fork	-	*2004-Present*	-	*38.8753*	*-80.264*	-	*1433*	*11.74*	-
WEST FORK RIVER AT WALKERSVILLE, WV	03057300	1984-1992	100	38.869	-80.458	10.4	1070	28.8	DD
MUD LICK RUN NR BUCKHANNON, WV	03052340	1966-2011	51	39.005	-80.256	9.0	1408	2.33	AP

Table 83 – Flood frequency analysis for Laurel Fork (presented for purposes of comparison – not utilized in hydraulic calculations).

Recurrence Interval (yr)	Discharge (ft^3/s)	95% Confidence Interval	
		Lower	Upper
2	752	649	870
5	1099	932	1312
10	1371	1121	1687
25	1777	1366	2255
50	2136	1556	2751
100	2553	1754	3312
200	3044	1961	3948
500	3839	2251	4918

Rainfall Data. A summary of weather stations near the project location are provided in the table below, including Station ID, Period of Record, data coverage during the Period of Record, location, elevation, and distance from the project site. The Rock Cave 2 weather station is close enough to the Laurel Fork project site that its precipitation data may be justifiably integrated into a watershed model for flow synthesis. However, to maintain uniformity with the majority of the other project sites where such data is not available, probabilistic characterizations of precipitation at the project site are utilized instead. Future research investigations into direct weather station data integration, and the relative benefits in improved model accuracy, may be justified.

Table 84 – Weather stations near Laurel Fork.

Station Name	**Station ID**	**Period of Record**	**Coverage (%)**	**Lat.**	**Long.**	**Elev. (ft.)**	**Dist. (mi.)**
Buckhannon	USC00461220	1888-Present	89	38.98	-80.22	1455	7.6
Rock Cave 2	USC00467649	1953-Present	33	38.856	-80.307	1749	2.7

Point precipitation frequency estimates for a 24-hour duration storm at the watershed, based on partial duration series NOAA Atlas 14 data available at the Hydrometeorological Design Studies Center's Precipitation Frequency Data Server are provided in Table 85.

Table 85 – Precipitation depth at Laurel Fork for a 24-hour duration storm, including 90% confidence interval.

Average Recurrence Interval (yr.)	Precipitation Depth (in.)	90% Confidence Interval	
		Precip. Depth, Lower (in.)	Precip. Depth, Upper (in.)
2	2.64	2.45	2.87
5	3.20	2.97	3.47
10	3.66	3.39	3.96
25	4.29	3.96	4.64
50	4.81	4.43	5.18
100	5.35	4.91	5.75
200	5.91	5.40	6.34
500	6.69	6.08	7.16

Development of flow rates at Laurel Fork for a variety of return period storms utilized three methods: USGS-provided regional regression equations (see Table 86), a TR-55 based watershed model that incorporated 24-hour storm duration precipitation depths (see Table 87), and a HEC-1 based watershed model that incorporated 24-hour storm duration precipitation depths (see Table 88). Since calculation of cumulative effective stream power requires characterization of flow rate as a function of time – and not simply the peak flow rate associated with a storm event – the runoff hydrographs generated by the HEC-1 analysis were utilized for this purpose.

Table 86 – USGS SIR-2010-5033 derived peak discharges, average prediction error, and corresponding 90% confidence interval for the Laurel Fork watershed.

Return Period (yr)	Peak Discharge, Q (cfs)	Avg. Prediction Error (%)	90% Confidence Interval	
			Q, Lower (cfs)	Q, Upper (cfs)
2	596	34.2	261	931
5	930	35.1	393	1466
10	1185	37.4	456	1913
25	1537	41.2	495	2579
50	1825	44.4	492	3157
100	2131	47.9	452	3809
200	2447	51.5	374	4519
500	2904	56.3	214	5593

Table 87 – Peak discharges derived from TR-55 watershed model of Laurel Fork.

Return Period (yr)	Peak Discharge, Q (cfs)	90% Confidence Interval	
		Q, Lower (cfs)	Q, Upper (cfs)
2	887	698	1136
5	1528	1251	1874
10	2130	1770	2551
25	3035	2551	3570
50	3836	3247	4429
100	4706	3994	5372
200	5643	4789	6381
500	6992	5933	7825

Table 88 – Peak discharges derived from HEC-1 watershed model of Laurel Fork.

Return Period (yr)	Peak Discharge, Q (cfs)	90% Confidence Interval	
		Q, Lower (cfs)	Q, Upper (cfs)
2	963	760	1234
5	1647	1354	2015
10	2281	1904	2746
25	3273	2746	3861
50	4155	3516	4808
100	5114	4328	5860
200	6156	5204	6936
500	7602	6450	8539

Hydraulics

An isometric view of the HEC-RAS Perspective Plot for Laurel Fork is shown in Figure 23, the underlying geometry for which is based on a field survey that was performed at the site. Manning's n-values for the site are summarized in Table 89. The result of HEC-RAS modeling at the site is a flow depth and velocity rating curve, presented in Figure 24.

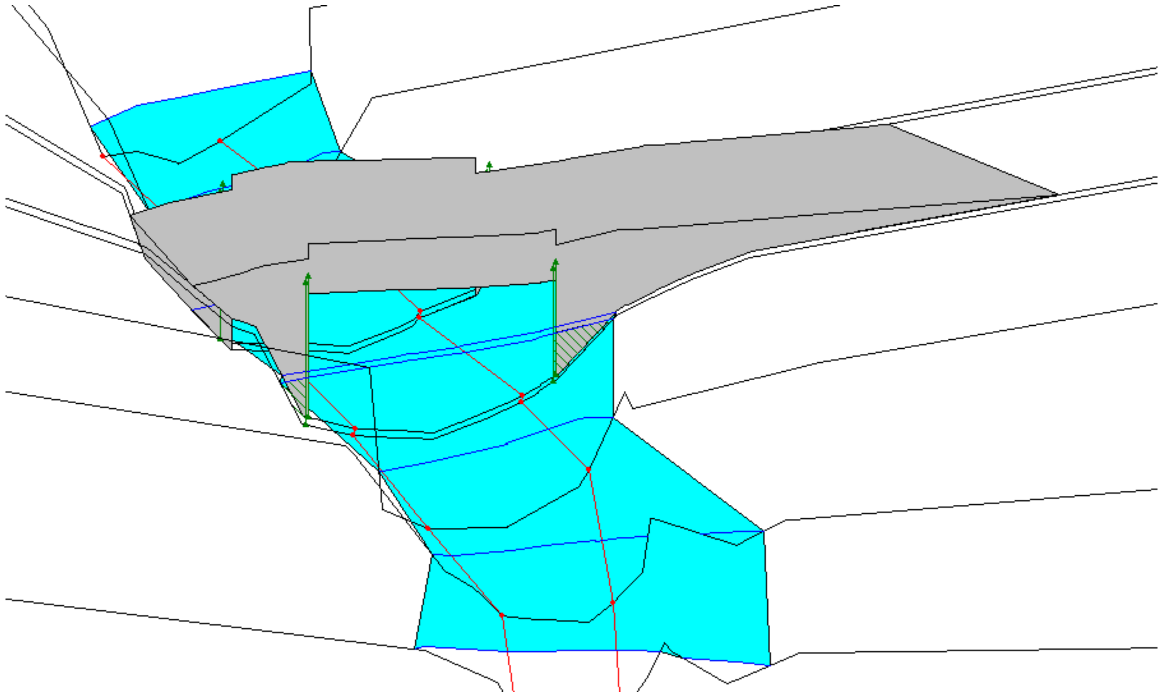

Figure 23- HEC-RAS X-Y-Z Perspective Plot for Laurel Fork.

Table 89 – Manning's n-values utilized in HEC-RAS model for Laurel Fork.

	Left Overbank	Channel	Right Overbank
U/S of Bridge	0.08	0.04	0.08
D/S of Bridge	0.1	0.04	0.1

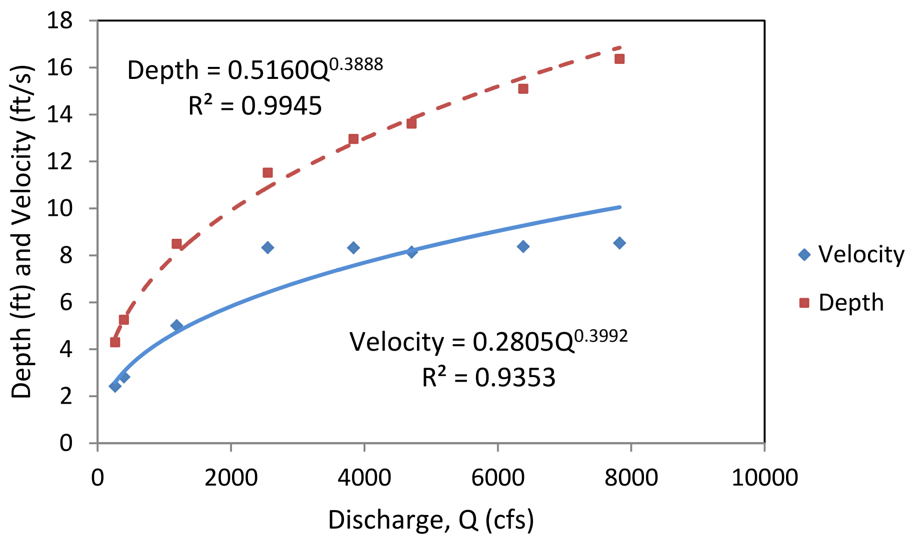

Figure 24- Flow depth and velocity rating curve for Laurel Fork.

Stream Power

Cumulative Effective Stream Power at the Leatherwood Road project site incorporated HEC-1 generated flow rates and runoff hydrograph. Amounts of Cumulative Effective Stream Power associated with a variety of return period storms are summarized in Table 90, including amounts associated with flow rates (and corresponding flow velocities and depths) that were generated using the lower and upper 90% confidence interval precipitation depths. The Average Annual Cumulative Effective Stream Power, which accounts for the Cumulative Effective Stream Power associated with each return period storm and the annual probability of each of these storms, is shown in Table 91.

Table 90 – Scour event Cumulative Effective Stream Power Ω (approach flow) for Laurel Fork – based on HEC-1 generated flow rates and hydrograph.

Return Period (yr)	Cumulative Effective Stream Power , Ω (ft·lb/ft^2)	90% Confidence Interval	
		Lower (ft·lb/ft^2)	Upper (ft·lb/ft^2)
2	0	0	0
5	6,111	5,493	11,370
10	15,218	13,466	24,801
25	31,092	27,348	46,623
50	47,247	41,472	65,973
100	65,518	57,565	88,337
200	86,441	75,216	114,459
500	118,042	102,775	152,398

Table 91 – Average Annual Cumulative Effective Stream Power Ω (approach flow) at Laurel Fork

Increment (return period range)	λ size	Interval Cumulative Effective Stream Power, Ω (ft·lb/ft^2)		
		Baseline Value	Lower 90% Conf.	Upper 90% Conf.
2-5	0.300	3,056	2,746	5,685
5-10	0.100	10,664	9,479	18,085
10-25	0.060	23,155	20,407	35,712
25-50	0.020	39,170	34,410	56,298
50-100	0.010	56,383	49,518	77,155
100-200	0.005	75,980	66,390	101,398
200-500	0.003	102,241	88,996	133,428
500-	0.002	118,042	102,775	152,398
Avg. Annual Cum. Eff. Stream Power, Ω (ft·lb/ft^2):		5,642	4,984	8,766

9 – Roaring Creek

Watershed characteristics

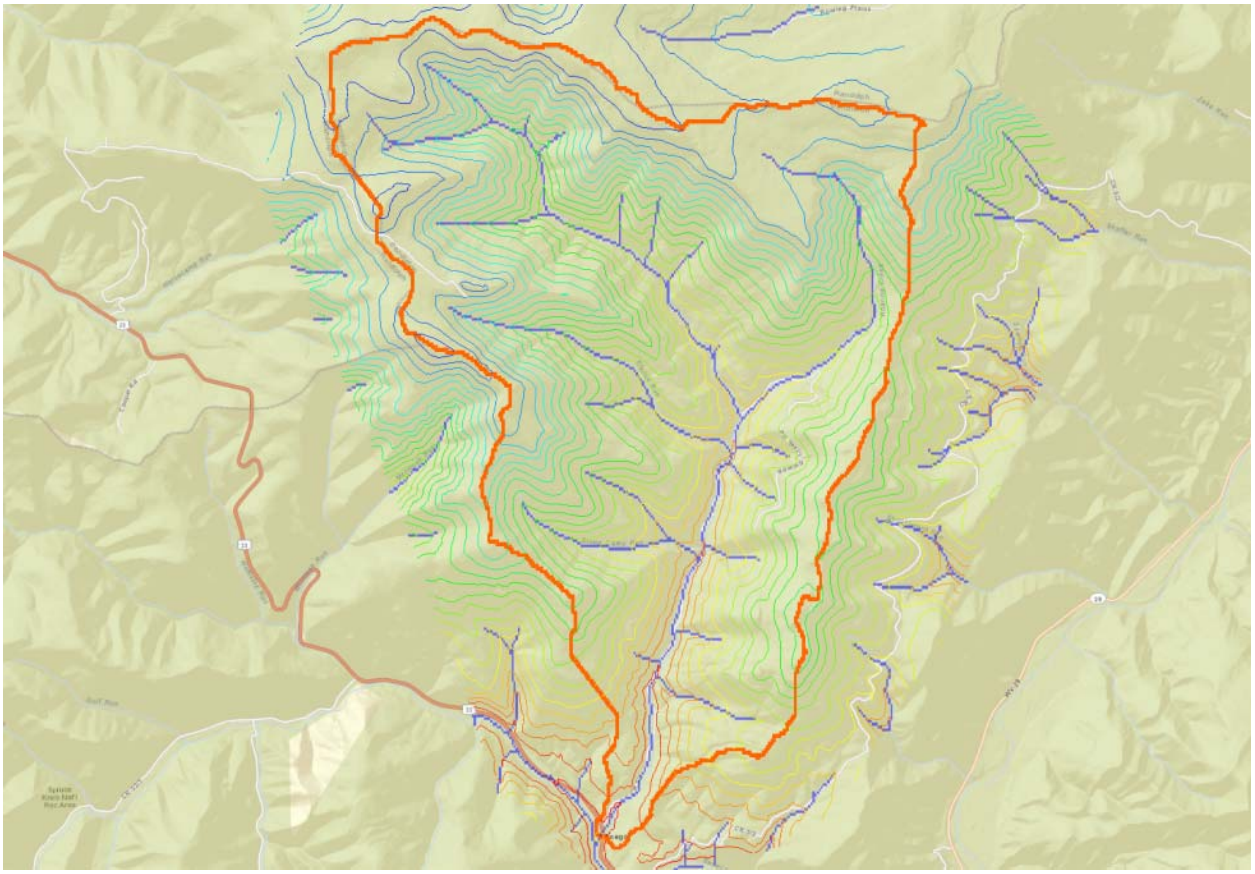

Figure 25- Image of delineated Roaring Creek watershed.

Several relevant hydrologic parameters for the Roaring Creek watershed are provided below in Table 92.

Table 92 - Watershed characteristics for Roaring Creek.

Parameter	Value
Location	Lat: 38.846847 Long: -79.422161
Region – USGS SIR2010-5033	Eastern Panhandle
Basin Area (mi^2)	14.01
Average basin elevation (ft)	3426
Average basin overland slope (%)	35.6
Maximum flow distance (ft)	43,941
Slope along maximum flow distance (%)	6.9
Shape factor (basin length / basin width)	2.58
Sinuosity factor of stream (max. stream length / basin length)	1.34
Runoff Curve Number	67.1
Time of Concentration (hr.)	2.636

A summary of the parameters utilized to compute the weighted watershed Curve Number is provided in the table below, including Natural Resources Conservation Service Soil Survey Geographic Database (SSURGO) derived hydrologic soil group, National Land Cover Database 2006 derived land cover classification, corresponding Curve Number, and sub-element area.

Table 93 – Soil type, land cover, and curve number characteristics for Roaring Creek.

Hydrologic Soil Group	Land Cover Description	CN	Area (mi^2)
C	Evergreen Forest	70	1.287
C	Deciduous Forest	70	9.311
C	Mixed Forest	70	0.203
B	Evergreen Forest	55	0.094
B	Deciduous Forest	55	2.257
D	Evergreen Forest	77	0.063
D	Mixed Forest	77	0.013
B	Mixed Forest	55	0.009
D	Deciduous Forest	77	0.144
C	Barren Land	91	0.024
C	Developed Open Space	74	0.052
D	Developed Open Space	80	0.002
D	Developed Low Intensity	84	0.002
C	Pasture/Hay	74	0.195
B	Pasture/Hay	61	0.1
B	Developed Open Space	61	0.079
A	Developed Open Space	39	0.063
A	Pasture/Hay	39	0.004
A	Developed Low Intensity	51	0.006
A	Deciduous Forest	30	0.1
Weighted CN = 67.1			

Flow synthesis

Stream Gage Data. In the table below, characteristics of the stream gage stations in proximity to the Roaring Creek project site are summarized, including period of record and data coverage within that period, location, distance from the project area, elevation, drainage area upstream of the gage, and whether the available data consists of Daily Discharge (DD) or Annual Peak (AP).

Roaring Creek is an ungauged stream, with an upstream contributing area of 14.01 mi^2. The nearest gauged stream, Job Run near Wymer, West Virginia, is nearly 10 miles away from the project site, gauges a watershed that is only 1.08 mi^2 , and has data for only a fraction of the 82 years since the Roaring Creek bridge was constructed. Other stream gauges in the relative proximity of the project site are likewise ill-suited to direct flow extrapolation to Roaring Creek, due to significant variations in drainage area, limited data availability, and significant distance. Therefore, flow synthesis at the Paden Fork project site utilizes two methods: the applicable USGS peak flow regression equations, and creation of a watershed model that utilizes probabilistic rainfall data.

Table 94 – Stream gage stations near Roaring Creek.

Project / Stream Gage Location	USGS Station ID	Period	Covg. (%)	Lat.	Long.	Dist. (mi.)	Elev. (ft.)	Drainage Area (mi^2)	Type
Roaring Creek	-	*1930-Present*	-	*38.8468*	*-79.422*	-	*1755*	*14.01*	-
JOB RUN NEAR WYMER, WV	03063950	1965-1976	100	38.882	-79.596	9.7	2825	1.08	AP
BRUSHY RUN NR PETERSBURG, WV	01606800	1965-1976	100	38.806	-79.214	11.6	1600	1.43	AP
REEDS CREEK TRIBUTARY NEAR FRANKLIN, WV	01605700	1965-1977	100	38.696	-79.388	10.6	3159	0.45	AP
N F SOUTH BRANCH POTOMAC RIVER AT CABINS, WV	01606000	1940-Present	53	38.985	-79.236	13.8	1045	335	DD
SOUTH BRANCH POTOMAC RIVER NEAR PETERSBURG, WV	01606500	1928-Present	100	38.991	-79.176	16.6	968	676	DD

Rainfall Data. A summary of weather stations near the project location are provided in the table below, including Station ID, Period of Record, data coverage during the Period of Record, location, elevation, and distance from the project site. Neither of the two available sites is close enough to the Roaring Creek watershed to merit direct inclusion into the Roaring Creek watershed model.

Table 95 – Weather stations near Roaring Creek.

Station Name	Station ID	Period of Record	Coverage (%)	Lat.	Long.	Elev. (ft.)	Dist. (mi.)
Alpena	USC00460143	1929-1973	81	38.916	-79.666	3022	14.0
Elkins Airport	USW00013729	1926-Present	99	38.885	-79.852	1979	23.3

Point precipitation frequency estimates for a 24-hour duration storm at the watershed, based on partial duration series NOAA Atlas 14 data available at the Hydrometeorological Design Studies Center's Precipitation Frequency Data Server are provided in Table 96.

Table 96 – Precipitation depth at Roaring Creek for a 24-hour duration storm, including 90% confidence interval.

Average Recurrence Interval (yr.)	Precipitation Depth (in.)	90% Confidence Interval	
		Precip. Depth, Lower (in.)	Precip. Depth, Upper (in.)
2	2.56	2.31	2.88
5	3.20	2.88	3.61
10	3.77	3.37	4.26
25	4.64	4.10	5.25
50	5.42	4.72	6.14
100	6.29	5.40	7.18
200	7.29	6.13	8.37
500	8.83	7.22	10.20

Development of flow rates at Roaring Creek for a variety of return period storms utilized three methods: USGS-provided regional regression equations (see Table 97), a TR-55 based watershed model that incorporated 24-hour storm duration precipitation depths (see Table 98), and a HEC-1 based watershed model that incorporated 24-hour storm duration precipitation depths (see Table 99). Since calculation of cumulative effective stream power requires characterization of flow rate as a function of time – and not simply the peak flow rate associated with a storm event – the runoff hydrographs generated by the HEC-1 analysis were utilized for this purpose.

Table 97 – USGS SIR-2010-5033 derived peak discharges, average prediction error, and corresponding 90% confidence interval for the Roaring Creek watershed.

Return Period (yr)	Peak Discharge, Q (cfs)	Avg. Prediction Error (%)	90% Confidence Interval	
			Q, Lower (cfs)	Q, Upper (cfs)
2	538	33.4	242	833
5	959	27.2	530	1388
10	1339	24.5	799	1878
25	1913	22.4	1208	2618
50	2408	21.7	1548	3267
100	2933	21.7	1886	3980
200	3524	22.4	2226	4823
500	4374	23.8	2661	6086

Table 98 – Peak discharges derived from TR-55 watershed model of Roaring Creek.

Return Period (yr)	Peak Discharge, Q (cfs)	90% Confidence Interval	
		Q, Lower (cfs)	Q, Upper (cfs)
2	667	447	1002
5	1388	1002	1943
10	2175	1610	2933
25	3561	2678	4628
50	4937	3697	6287
100	6576	4900	8338
200	8560	6268	10789
500	11760	8418	14710

Table 99 – Peak discharges derived from HEC-1 watershed model of Roaring Creek.

Return Period (yr)	Peak Discharge, Q (cfs)	90% Confidence Interval	
		Q, Lower (cfs)	Q, Upper (cfs)
2	652	446	963
5	1324	963	1845
10	2064	1533	2778
25	3371	2538	4379
50	4671	3500	5964
100	6243	4636	7948
200	8164	5945	10325
500	11265	8027	14121

Hydraulics

An isometric view of the HEC-RAS Perspective Plot for Roaring Creek is shown in Figure 26, the underlying geometry for which is based on a field survey that was performed at the site. Manning's n-values for the site are summarized in Table 100. The result of HEC-RAS modeling at the site is a flow depth and velocity rating curve, presented in Figure 27.

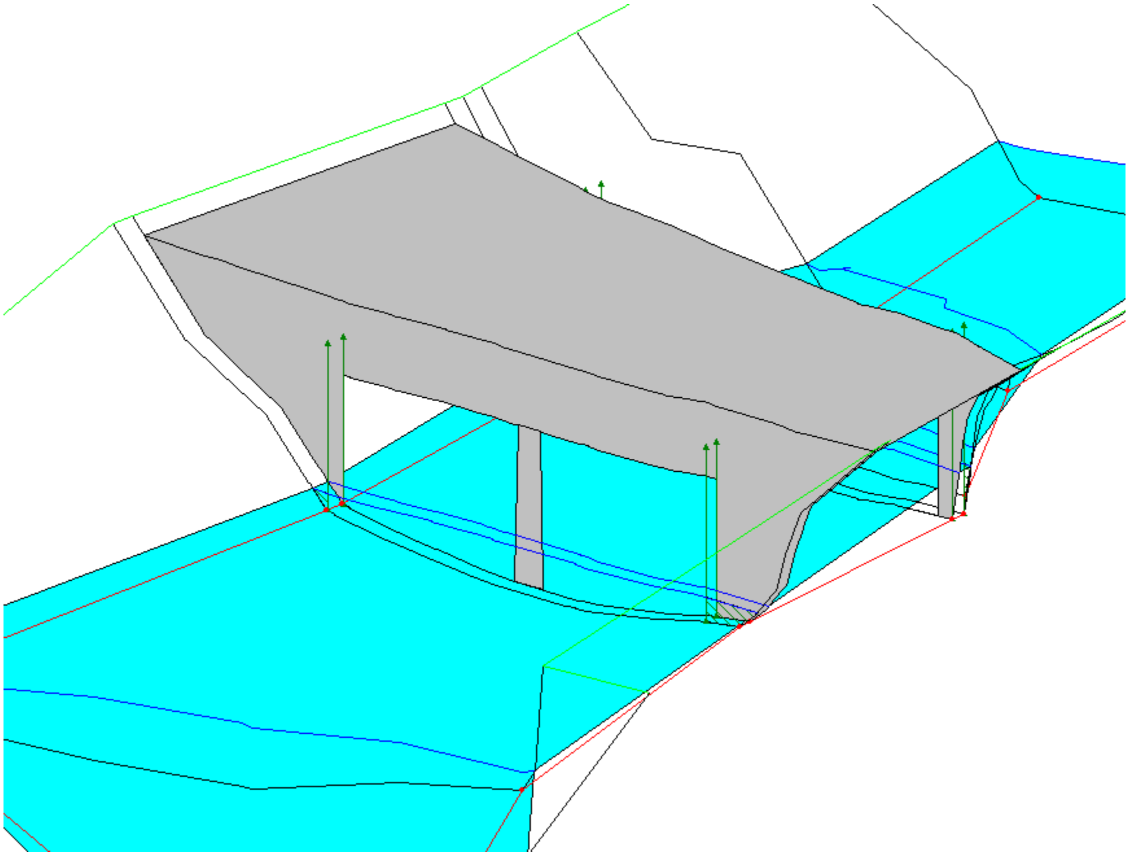

Figure 26- HEC-RAS X-Y-Z Perspective Plot for Roaring Creek.

Table 100 – Manning's n-values utilized in HEC-RAS model for Roaring Creek.

	Left Overbank	Channel	Right Overbank
U/S of Bridge	0.04	0.14	0.04
D/S of Bridge	0.06	0.14	0.06

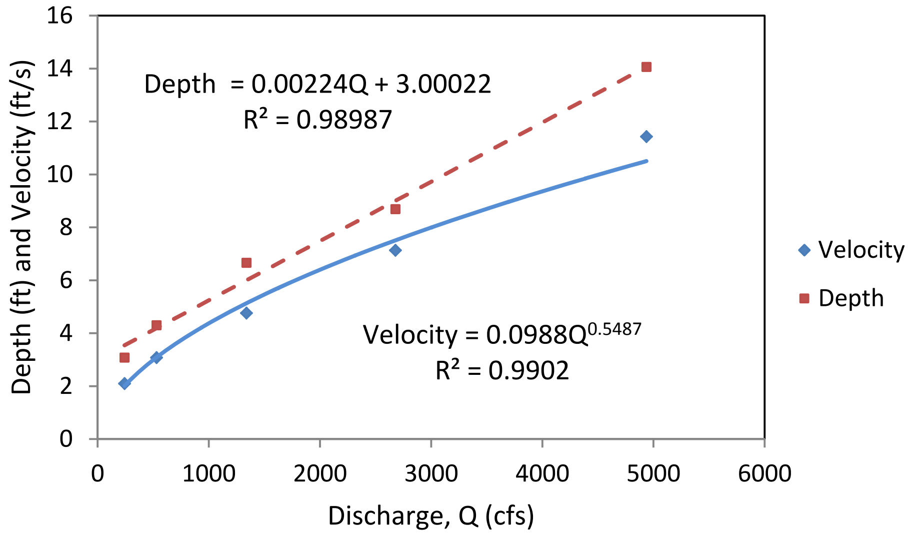

Figure 27- Flow depth and velocity rating curve for Roaring Creek.

Stream Power

Cumulative Effective Stream Power at the Leatherwood Road project site incorporated HEC-1 generated flow rates and runoff hydrograph. Amounts of Cumulative Effective Stream Power associated with a variety of return period storms are summarized in Table 101 (approach flow) and Table 102 (at pier), including amounts associated with flow rates (and corresponding flow velocities and depths) that were generated using the lower and upper 90% confidence interval precipitation depths. The Average Annual Cumulative Effective Stream Power, which accounts for the Cumulative Effective Stream Power associated with each return period storm and the annual probability of each of these storms, is shown in Table 103 and Table 104.

Table 101 – Scour event Cumulative Effective Stream Power Ω (approach flow) for Roaring Creek – based on HEC-1 generated flow rates and hydrograph.

Return Period (yr)	Cumulative Effective Stream Power , Ω (ft·lb/ft^2)	90% Confidence Interval	
		Lower (ft·lb/ft^2)	Upper (ft·lb/ft^2)
2	0	0	0
5	158,711	113,333	321,198
10	459,252	327,050	769,860
25	1,162,607	826,226	1,800,108
50	2,017,911	1,407,521	2,772,855
100	2,865,683	2,191,940	3,594,336
200	3,580,674	2,981,120	4,349,718
500	4,526,428	3,826,631	5,549,549

Table 102 – Scour event Cumulative Effective Stream Power Ω (pier) for Roaring Creek – based on HEC-1 generated flow rates and hydrograph.

Return Period (yr)	Cumulative Effective Stream Power , Ω (ft·lb/ft^2)	90% Confidence Interval	
		Lower (ft·lb/ft^2)	Upper (ft·lb/ft^2)
2	0	0	0
5	458,674	327,532	928,263
10	1,327,238	945,175	2,224,894
25	3,359,933	2,387,793	5,202,311
50	5,831,764	4,067,735	8,013,551
100	8,281,824	6,334,706	10,387,632
200	10,348,148	8,615,438	12,570,686
500	13,081,377	11,058,964	16,038,198

Table 103 – Average Annual Cumulative Effective Stream Power Ω (approach flow) at Roaring Creek.

Increment (return period range)	λ size	Interval Cumulative Effective Stream Power, Ω (ft·lb/ft^2)		
		Baseline Value	Lower 90% Conf.	Upper 90% Conf.
2-5	0.300	79,355	56,666	160,599
5-10	0.100	308,981	220,192	545,529
10-25	0.060	810,929	576,638	1,284,984
25-50	0.020	1,590,259	1,116,873	2,286,481
50-100	0.010	2,441,797	1,799,730	3,183,596
100-200	0.005	3,223,179	2,586,530	3,972,027
200-500	0.003	4,053,551	3,403,876	4,949,634
500-	0.002	4,526,428	3,826,631	5,549,549
Avg. Annual Cum. Eff. Stream Power, Ω (ft·lb/ft^2):		196,913	144,750	303,205

Table 104 – Average Annual Cumulative Effective Stream Power Ω (pier) at Roaring Creek.

Increment (return period range)	λ size	Interval Cumulative Effective Stream Power, (ft·lb/ft^2)		
		Baseline Value	Lower 90% Conf.	Upper 90% Conf.
2-5	0.300	229,337	163,766	464,132
5-10	0.100	892,956	636,353	1,576,579
10-25	0.060	2,343,586	1,666,484	3,713,603
25-50	0.020	4,595,849	3,227,764	6,607,931
50-100	0.010	7,056,794	5,201,220	9,200,592
100-200	0.005	9,314,986	7,475,072	11,479,159
200-500	0.003	11,714,762	9,837,201	14,304,442
500-	0.002	13,081,377	11,058,964	16,038,198
Avg. Annual Cum. Eff. Stream Power, Ω (ft·lb/ft^2):		569,079	418,327	876,264

10 – Beverly

Watershed characteristics

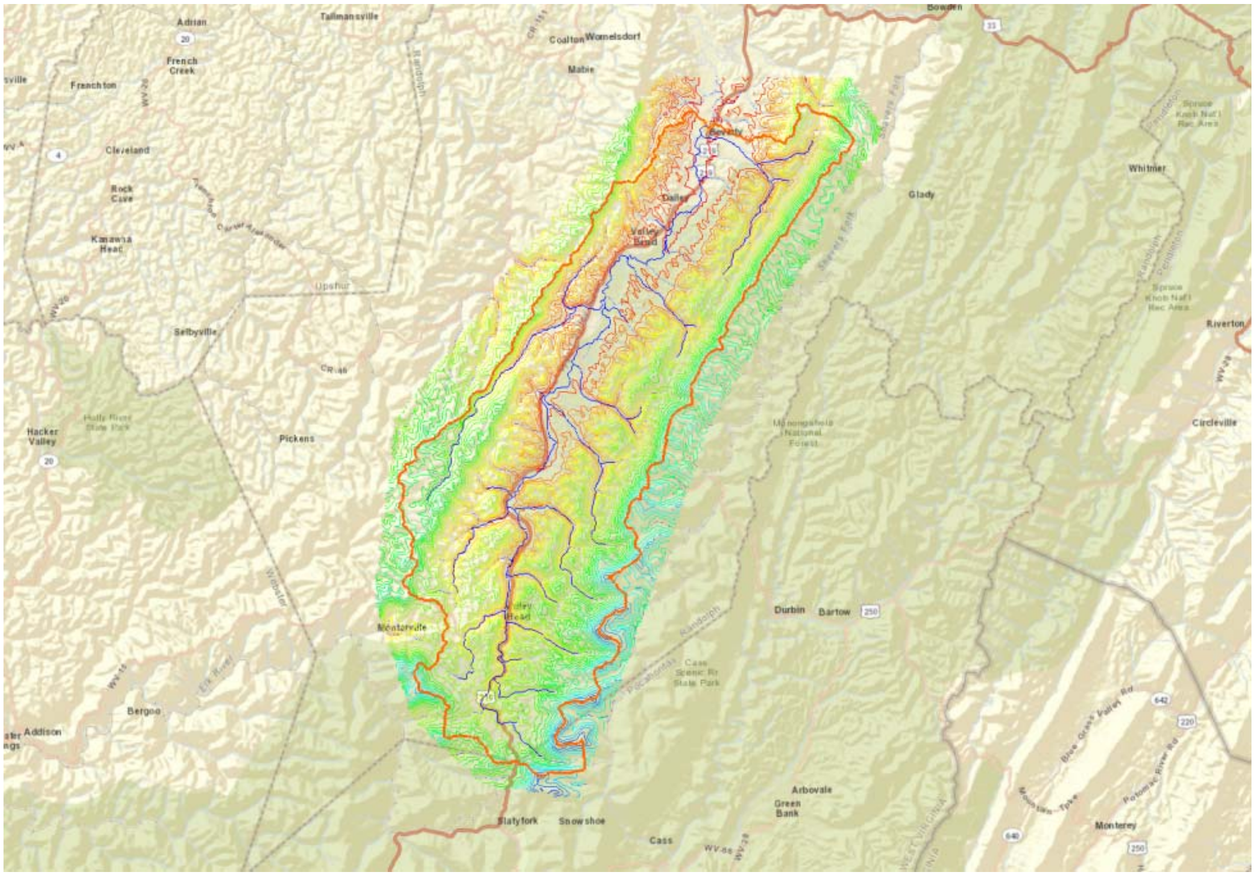

Figure 28- Image of delineated Beverly watershed.

Several relevant hydrologic parameters for the Beverly watershed are provided below in Table 105. Since stream gage data, and not watershed modeling, was utilized to characterize flow conditions at the Beverly project site, land cover and soil type analysis reports are not provided herein. When generating a runoff hydrograph for purposes of scaling it against the PeakFQ generated peak flow rates, multiple sub-basins were created. Additional details about these sub-basins can be found in the digital watershed model files for this location.

Table 105 - Watershed characteristics for Beverly.

Parameter	Value
Location	Lat: 38.842372 Long: -79.876425
Region – USGS SIR2010-5033	Central Mountains
Basin Area (mi^2)	219.0
Average basin elevation (ft)	2800
Average basin overland slope (%)	28.8
Maximum flow distance (ft)	225,857
Slope along maximum flow distance (%)	1.2
Shape factor (basin length / basin width)	3.62
Sinuosity factor of stream (max. stream length / basin length)	1.43

Flow synthesis

Stream Gage Data. In the table below, characteristics of the stream gage stations in proximity to the Beverly project site are summarized, including period of record and data coverage within that period, location, distance from the project area, elevation, drainage area upstream of the gage, and whether the available data consists of Daily Discharge (DD) or Annual Peak (AP).

The Tygart Valley River is a gauged stream, with stream gauges being found both upstream and downstream of the project site, and with substantial data coverage at each site. Stream gauge data was obtained from USGS and PKFQWin analysis was performed at each stream gauge site, and the resulting flood frequency analysis results were interpolated between to yield the flood frequency analysis presented in Table 107.

Table 106 – Stream gage stations near Beverly.

Project / Stream Gage Location	USGS Station ID	Period	Covg. (%)	Lat.	Long.	Dist. (mi.)	Elev. (ft.)	Drainage Area (mi^2)	Type
Beverly, Tygart Valley River	-	*1995-Present*	-	*38.8424*	*-79.876*	-	*1941*	*219.00*	-
TYGART VALLEY RIVER NEAR DAILEY, WV	03050000	1915-2011	88	38.809	-79.882	2.3	1940	185	DD
TYGART VALLEY RIVER NEAR ELKINS, WV	03050500	1944-2004	100	38.924	-79.879	5.6	1895	271	DD

Table 107 – Flood frequency analysis for the Tygart Valley River at the Beverly project location

Recurrence Interval (yr)	Discharge (ft^3/s)	95% Confidence Interval	
		Lower	Upper
2	7046	6565	7556
5	9725	8978	10564
10	11681	10615	12872
25	14386	12745	16125
50	16589	14397	18793
100	18966	16111	21677
200	21553	17894	24795
500	25337	20387	29333

Rainfall Data. A summary of weather stations near the project location are provided in Table 108, including Station ID, Period of Record, data coverage during the Period of Record, location, elevation, and distance from the project site. Since stream gage data are utilized for flow characterization at Beverly, it is not necessary to utilize characterizations of precipitation for purposes of watershed modeling. However, such data is provided for reference purposes.

Table 108 – Weather stations near Beverly.

Station Name	Station ID	Period of Record	Coverage (%)	Lat.	Long.	Elev. (ft.)	Dist. (mi.)
Elkins Airport	USW00013729	1926-Present	99	38.885	-79.852	1979	3.2
Valley Head	USC00469086	1938-Present	96	38.545	-80.036	2425	22.3

Point precipitation frequency estimates for a 24-hour duration storm at the watershed, based on partial duration series NOAA Atlas 14 data available at the Hydrometeorological Design Studies Center's Precipitation Frequency Data Server are provided in Table 109.

Table 109 – Precipitation depth at Beverly for a 24-hour duration storm, including 90% confidence interval.

Average Recurrence Interval (yr.)	Precipitation Depth (in.)	90% Confidence Interval	
		Precip. Depth, Lower (in.)	Precip. Depth, Upper (in.)
2	2.59	2.40	2.81
5	3.15	2.91	3.42
10	3.62	3.33	3.92
25	4.27	3.92	4.63
50	4.81	4.39	5.22
100	5.38	4.88	5.84
200	5.98	5.39	6.49
500	6.82	6.08	7.43

Development of flow rates for a variety of return period storms utilized three methods: USGS-provided regional regression equations , a TR-55 based watershed model that incorporated 24-hour storm duration precipitation depths, and a HEC-1 based watershed model that incorporated 24-hour storm duration precipitation depths. Since calculation of cumulative effective stream power requires characterization of flow rate as a function of time – and not simply the peak flow rate associated with a storm event – the runoff hydrographs generated by the HEC-1 analysis were utilized for this purpose.

Table 110 – USGS SIR-2010-5033 derived peak discharges, average prediction error, and corresponding 90% confidence interval for the Beverly watershed.

Return Period (yr)	Peak Discharge, Q (cfs)	Avg. Prediction Error (%)	90% Confidence Interval	
			Q, Lower (cfs)	Q, Upper (cfs)
2	7667	34.2	3353	11980
5	11020	35.1	4657	17382
10	13478	37.4	5186	21770
25	16738	41.2	5394	28082
50	19351	44.4	5218	33485
100	22138	47.9	4694	39582
200	24908	51.5	3807	46009
500	28875	56.3	2133	55618

Hydraulics

An isometric view of the HEC-RAS Perspective Plot for Beverly is shown in Figure 29, the underlying geometry for which is based on a field survey that was performed at the site. Manning's n-values for the site are summarized in Table 111. The result of HEC-RAS modeling at the site is a flow depth and velocity rating curve, presented in Figure 30.

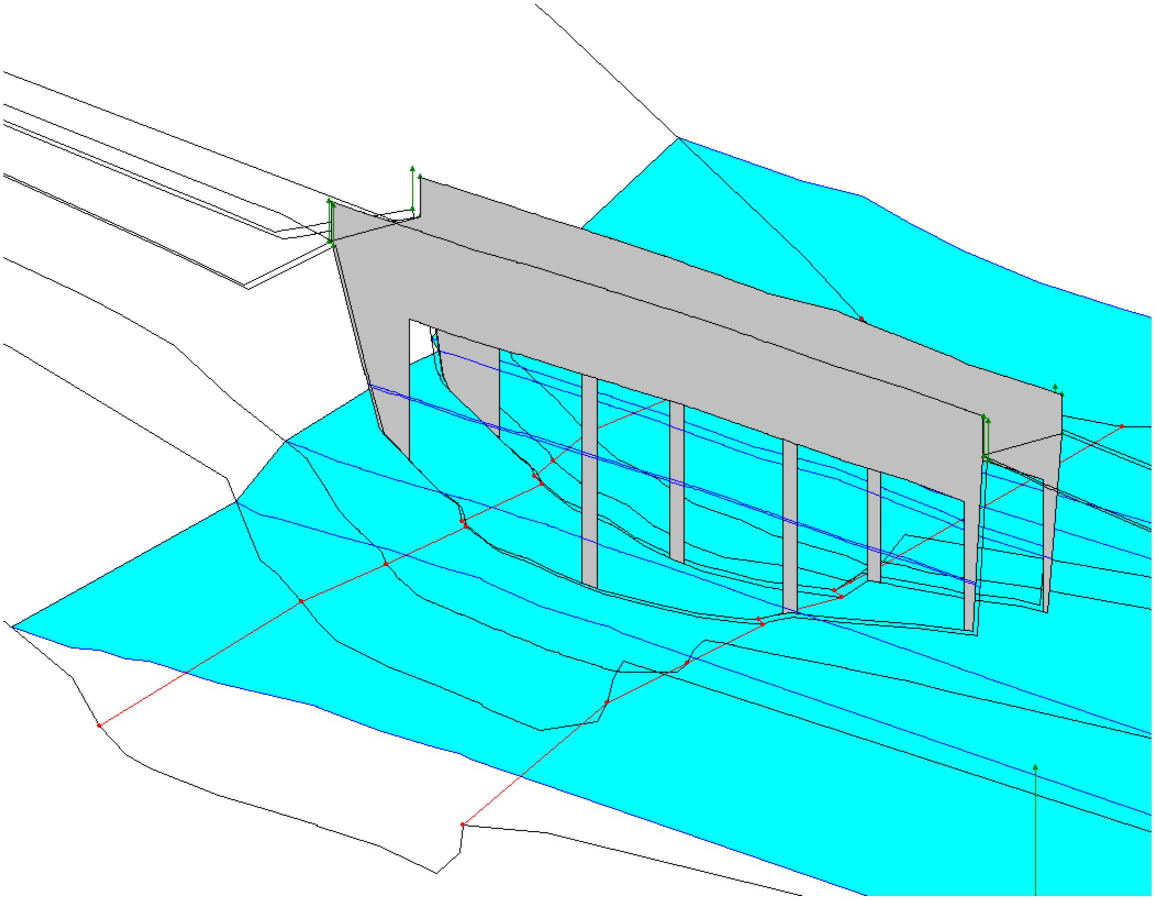

Figure 29- HEC-RAS X-Y-Z Perspective Plot for Beverly.

Table 111 – Manning's n-values utilized in HEC-RAS model for Beverly.

	Left Overbank	Channel	Right Overbank
U/S of Bridge	0.06	0.05	0.11
D/S of Bridge	0.06	0.05	0.11

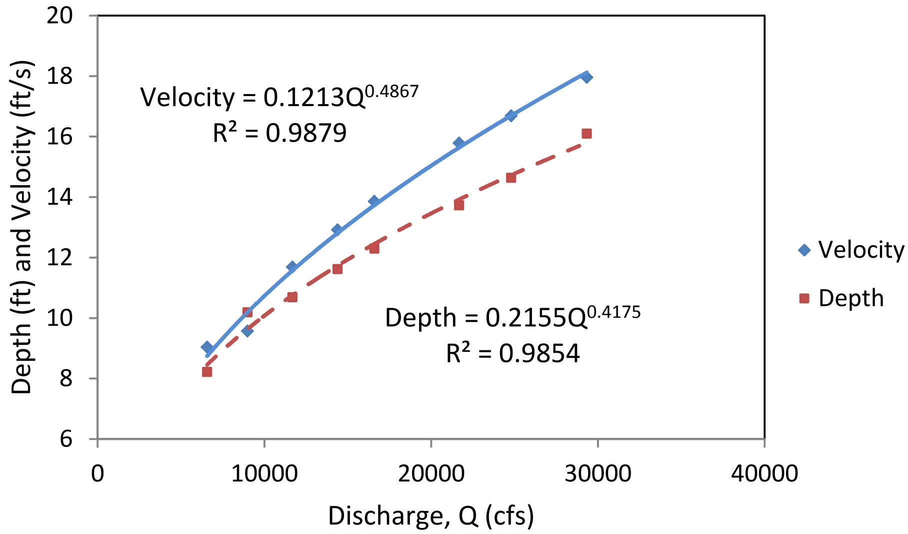

Figure 30- Flow depth and velocity rating curve for Beverly.

Stream Power

Cumulative Effective Stream Power at the Leatherwood Road project site incorporated PeakFQ generated peak flow rates and a runoff hydrograph shape that was determined through HEC-1 modeling. Amounts of Cumulative Effective Stream Power associated with a variety of return period storms are summarized in Table 112 (approach flow) and Table 113 (at pier), including amounts associated with flow rates (and corresponding flow velocities and depths) that were generated using the lower and upper 90% confidence interval precipitation depths. The Average Annual Cumulative Effective Stream Power, which accounts for the Cumulative Effective Stream Power associated with each return period storm and the annual probability of each of these storms, is shown in Table 114 and Table 115.

Table 112 – Scour event Cumulative Effective Stream Power Ω (approach flow) for Beverly – based on PeakFQ generated flow rates and HEC-1 generated hydrograph shape.

Return Period (yr)	Cumulative Effective Stream Power , Ω (ft·lb/ft^2)	95% Confidence Interval	
		Lower (ft·lb/ft^2)	Upper (ft·lb/ft^2)
2	0	0	0
5	102,205	90,678	119,030
10	228,155	191,527	268,684
25	437,149	350,776	535,824
50	630,897	489,078	778,422
100	861,897	649,109	1,084,581
200	1,147,452	823,074	1,441,704
500	1,598,898	1,088,867	1,988,779

Table 113 – Scour event Cumulative Effective Stream Power Ω (pier) for Beverly – based on PeakFQ generated flow rates and HEC-1 generated hydrograph shape.

Return Period (yr)	Cumulative Effective Stream Power , Ω ($ft \cdot lb/ft^2$)	95% Confidence Interval	
		Lower ($ft \cdot lb/ft^2$)	Upper ($ft \cdot lb/ft^2$)
2	0	0	0
5	295,372	262,060	343,996
10	659,367	553,513	776,498
25	1,263,360	1,013,744	1,548,532
50	1,823,293	1,413,437	2,249,639
100	2,490,882	1,875,926	3,134,440
200	3,316,137	2,378,684	4,166,525
500	4,620,815	3,146,826	5,747,571

Table 114 – Average Annual Cumulative Effective Stream Power Ω (approach flow) at Beverly.

Increment (return period range)	λ size	Interval Cumulative Effective Stream Power, Ω ($ft \cdot lb/ft^2$)		
		Baseline Value	Lower 90% Conf.	Upper 90% Conf.
2-5	0.300	51,102	45,339	59,515
5-10	0.100	165,180	141,103	193,857
10-25	0.060	332,652	271,152	402,254
25-50	0.020	534,023	419,927	657,123
50-100	0.010	746,397	569,094	931,501
100-200	0.005	1,004,674	736,092	1,263,143
200-500	0.003	1,373,175	955,971	1,715,242
500	0.002	1,598,898	1,088,867	1,988,779
Avg. Annual Cum. Eff. Stream Power, Ω ($ft \cdot lb/ft^2$):		82,293	66,797	99,272

Table 115 – Average Annual Cumulative Effective Stream Power Ω (pier) at Beverly.

Increment (return period range)	λ size	Interval Cumulative Effective Stream Power, Ω (ft·lb/ft^2)		
		Baseline Value	90% Lower	90% Upper
2-5	0.300	147,686	131,030	171,998
5-10	0.100	477,369	407,787	560,247
10-25	0.060	961,363	783,629	1,162,515
25-50	0.020	1,543,326	1,213,590	1,899,085
50-100	0.010	2,157,088	1,644,681	2,692,039
100-200	0.005	2,903,509	2,127,305	3,650,483
200-500	0.003	3,968,476	2,762,755	4,957,048
500	0.002	4,620,815	3,146,826	5,747,571
Avg. Annual Cum. Eff. Stream Power, Ω (ft·lb/ft^2):		237,826	193,042	286,896

11 – Coon Creek

Watershed characteristics

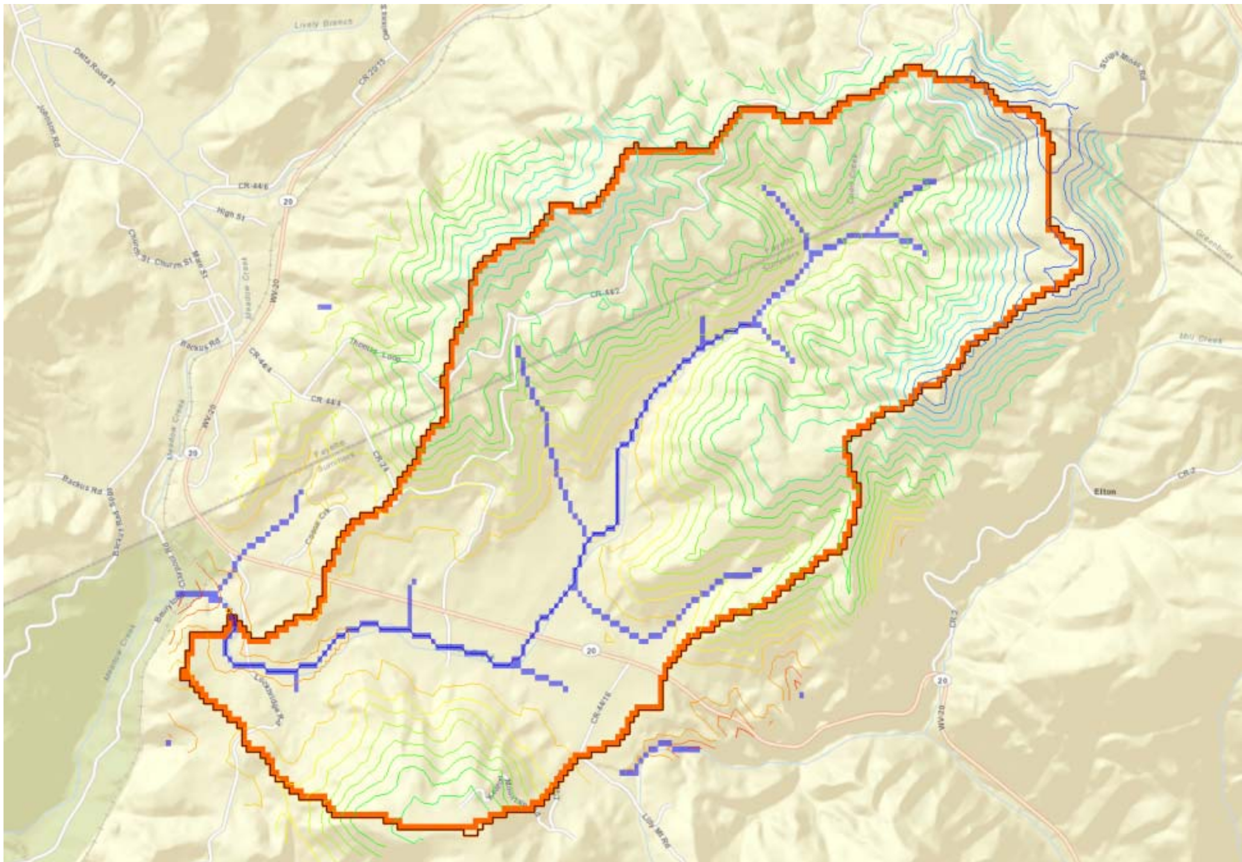

Figure 31- Image of delineated Coon Creek watershed.

Several relevant hydrologic parameters for the Coon Creek watershed are provided below in Table 116.

Table 116 - Watershed characteristics for Coon Creek.

Parameter	**Value**
Location	Lat: 37.846511 Long: -80.854742
Region – USGS SIR2010-5033	Central Mountains
Basin Area (mi^2)	3.40
Average basin elevation (ft)	2703
Average basin overland slope (%)	19.5
Maximum flow distance (ft)	21,260
Slope along maximum flow distance (%)	4.1
Shape factor (basin length / basin width)	2.74
Sinuosity factor of stream (max. stream length / basin length)	0.90
Runoff Curve Number	65.7
Time of Concentration (hr.)	2.066

A summary of the parameters utilized to compute the weighted watershed Curve Number is provided in the table below, including Natural Resources Conservation Service Soil Survey Geographic Database (SSURGO) derived hydrologic soil group, National Land Cover Database 2006 derived land cover classification, corresponding Curve Number, and sub-element area.

Table 117 – Soil type, land cover, and curve number characteristics for Coon Creek.

Hydrologic Soil Group	Land Cover Description	CN	Area (mi^2)
B	Developed Low Intensity	68	0.054
B	Pasture/Hay	61	0.06
C	Deciduous Forest	70	1.369
C	Pasture/Hay	74	0.172
C	Developed Open Space	74	0.096
C	Developed Low Intensity	79	0.074
C	Barren Land	91	0.1
B	Emergent Herbaceous Wetlands	98	0.099
B	Developed Open Space	61	0.042
A	Barren Land	77	0.007
C	Emergent Herbaceous Wetlands	98	0.025
A	Pasture/Hay	39	0.006
B	Barren Land	86	0.055
B	Deciduous Forest	55	1.039
A	Deciduous Forest	30	0.154
C	Mixed Forest	70	0.002
A	Developed Low Intensity	51	0.002
A	Developed Open Space	39	0.002
C	Evergreen Forest	70	0.001
B	Developed Medium Intensity	70	0.001
A	Developed Medium Intensity	54	0.002
B	Woody Wetlands	55	0.008
C	Woody Wetlands	70	0.004
C	Cultivated Crops	82	0.015
B	Cultivated Crops	75	0.01
Weighted CN = 65.7			

Flow synthesis

Stream Gage Data. In the table below, characteristics of the stream gage stations in proximity to the Coon Creek project site are summarized, including period of record and data coverage within that period, location, distance from the project area, elevation, drainage area upstream of the gage, and whether the available data consists of Daily Discharge (DD) or Annual Peak (AP).

Coon Creek is ungauged, with no stream gauges within ten miles of the project site. The stream gauge data that is available are for a river of a vastly larger scale than the Coon Creek watershed (i.e., the New River), or are for locations were stream gauge data ceases prior to the time period since the Coon Creek bridge was constructed. Therefore, primary flow synthesis at the Laurel Fork site utilizes two methods: the applicable USGS peak flow regression equations, and creation of a watershed model that utilizes probabilistic rainfall data.

Table 118 – Stream gage stations near Coon Creek.

Project / Stream Gage Location	USGS Station ID	Period	Covg. (%)	Lat.	Long.	Dist. (mi.)	Elev. (ft.)	Drainage Area (mi^2)	Type
Coon Creek	-	*1979-Present*	-	*37.8465*	*-80.855*	-	*2383*	*3.40*	-
NEW RIVER @ CAPERTON	03185500	1928-1958	100	38.022	-81.029	15.4	938.4	6,826	DD
NEW RIVER AT HINTON, WV	03184500	1936-2003	100	37.67	-80.893	12.4	1255	6,256	DD
L. BEAVER C. TRIB NR SHADY SPRINGS, WV	03185020	1966-1977	100	37.725	-81.101	15.9	2543	0.62	AP
GRIFFITH CREEK NR ALDERSON, WV	03183550	1966-1977	100	37.738	-80.71	10.9	1850	3.84	AP
BUGGAR LICK AT PENCE SPRINGS, WV	03183570	1966-1977	100	37.684	-80.717	13.5	1522	2.71	AP

Rainfall Data. A summary of weather stations near the project location are provided in the table below, including Station ID, Period of Record, data coverage during the Period of Record, location, elevation, and distance from the project site. While the Beckley Airport weather station has data over the entire range of dates since Coon Creek bridge was constructed, 15.3 miles is too far away from the project site to ensure that the precipitation data gathered at the weather station is a meaningful representation of conditions in the Coon Creek watershed.

Table 119 – Weather stations near Coon Creek.

Station Name	Station ID	Period of Record	Coverage (%)	Lat.	Long.	Elev. (ft.)	Dist. (mi.)
Beckley Airport	USW00003872	1963-Present	100	37.783	-81.123	2514	15.3

Point precipitation frequency estimates for a 24-hour duration storm at the watershed, based on partial duration series NOAA Atlas 14 data available at the Hydrometeorological Design Studies Center's Precipitation Frequency Data Server are provided in Table 120.

Table 120 – Precipitation depth at Coon Creek for a 24-hour duration storm, including 90% confidence interval.

Average Recurrence Interval (yr.)	Precipitation Depth (in.)	90% Confidence Interval	
		Precip. Depth, Lower (in.)	Precip. Depth, Upper (in.)
2	2.59	2.42	2.78
5	3.12	2.91	3.35
10	3.55	3.31	3.81
25	4.14	3.85	4.44
50	4.62	4.27	4.95
100	5.11	4.71	5.47
200	5.62	5.16	6.02
500	6.33	5.76	6.78

Development of flow rates at Coon Creek for a variety of return period storms utilized three methods: USGS-provided regional regression equations (see Table 121), a TR-55 based watershed model that incorporated 24-hour storm duration precipitation depths (see Table 122), and a HEC-1 based watershed model that incorporated 24-hour storm duration precipitation depths (see Table 123). Since calculation of cumulative effective stream power requires characterization of flow rate as a function of time – and not simply the peak flow rate associated with a storm event – the runoff hydrographs generated by the HEC-1 analysis were utilized for this purpose.

Table 121 – USGS SIR-2010-5033 derived peak discharges, average prediction error, and corresponding 90% confidence interval for the Coon Creek watershed.

Return Period (yr)	Peak Discharge, Q (cfs)	Avg. Prediction Error (%)	90% Confidence Interval	
			Q, Lower (cfs)	Q, Upper (cfs)
2	202	34.2	88	316
5	326	35.1	138	515
10	423	37.4	163	683
25	559	41.2	180	938
50	671	44.4	181	1161
100	791	47.9	168	1414
200	916	51.5	140	1692
500	1098	56.3	81	2114

Table 122 – Peak discharges derived from TR-55 watershed model of Coon Creek

Return Period (yr)	Peak Discharge, Q (cfs)	90% Confidence Interval	
		Q, Lower (cfs)	Q, Upper (cfs)
2	166	125	217
5	324	256	405
10	480	390	585
25	726	601	862
50	947	784	1107
100	1187	990	1371
200	1450	1212	1663
500	1833	1524	2084

Table 123 – Peak discharges derived from HEC-1 watershed model of Coon Creek.

Return Period (yr)	Peak Discharge, Q (cfs)	90% Confidence Interval	
		Q, Lower (cfs)	Q, Upper (cfs)
2	160	122	208
5	304	243	379
10	449	365	547
25	681	563	809
50	890	736	1042
100	1118	931	1293
200	1368	1142	1572
500	1734	1439	1974

Hydraulics

An isometric view of the HEC-RAS Perspective Plot for Coon Creek is shown in Figure 32, the underlying geometry for which is based on a field survey that was performed at the site. Manning's n-values for the site are summarized in Table 124. The result of HEC-RAS modeling at the site is a flow depth and velocity rating curve, presented in Figure 33.

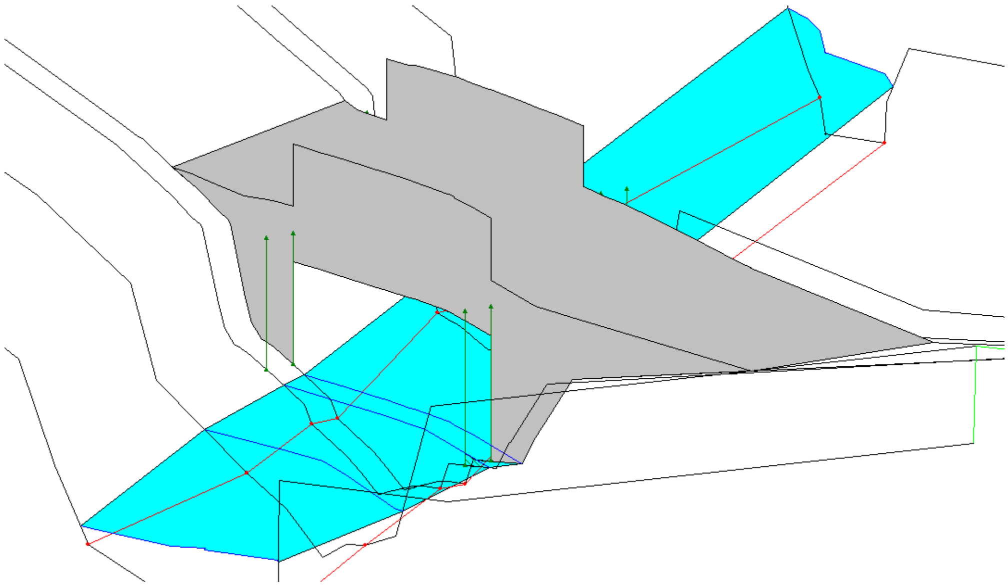

Figure 32- HEC-RAS X-Y-Z Perspective Plot for Coon Creek.

Table 124 – Manning's n-values utilized in HEC-RAS model for Coon Creek.

	Left Overbank	Channel	Right Overbank
U/S of Bridge	0.10	0.13	0.10
D/S of Bridge	0.10	0.13	0.10

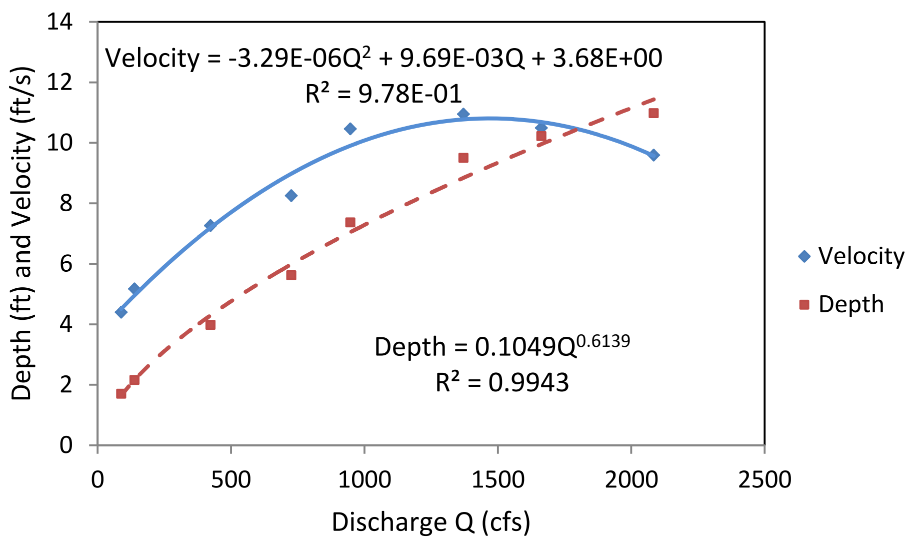

Figure 33- Flow depth and velocity rating curve for Coon Creek.

Stream Power

Cumulative Effective Stream Power at the Leatherwood Road project site incorporated HEC-1 generated flow rates and runoff hydrograph. Amounts of Cumulative Effective Stream Power associated with a variety of return period storms are summarized in Table 125, including amounts associated with flow rates (and corresponding flow velocities and depths) that were generated using the lower and upper 90% confidence interval precipitation depths. The Average Annual Cumulative Effective Stream Power, which accounts for the Cumulative Effective Stream Power associated with each return period storm and the annual probability of each of these storms, is shown in Table 126.

Table 125 – Scour event Cumulative Effective Stream Power Ω (approach flow) for Coon Creek – based on HEC-1 generated flow rates and hydrograph.

Return Period (yr)	Cumulative Effective Stream Power , Ω (ft·lb/ft^2)	90% Confidence Interval	
		Lower (ft·lb/ft^2)	Upper (ft·lb/ft^2)
2	0	0	0
5	145,946	117,578	282,668
10	384,904	316,405	640,990
25	845,392	706,712	1,203,491
50	1,296,769	1,090,724	1,782,598
100	1,767,773	1,532,436	2,290,284
200	2,210,064	1,986,207	2,776,159
500	2,656,424	2,522,880	3,167,889

Table 126 – Average Annual Cumulative Effective Stream Power Ω (approach flow) at Coon Creek.

Increment (return period range)	λ size	Interval Cumulative Effective Stream Power, Ω (ft·lb/ft^2)		
		Baseline Value	Lower 90% Conf.	Upper 90% Conf.
2-5	0.300	72,973	58,789	141,334
5-10	0.100	265,425	216,991	461,829
10-25	0.060	615,148	511,558	922,241
25-50	0.020	1,071,081	898,718	1,493,045
50-100	0.010	1,532,271	1,311,580	2,036,441
100-200	0.005	1,988,919	1,759,321	2,533,221
200-500	0.003	2,433,244	2,254,543	2,972,024
500-	0.002	2,656,424	2,522,880	3,167,889
Avg. Annual Cum. Eff. Stream Power, Ω (ft·lb/ft^2):		144,645	121,725	222,061

12 – Bridge Fork

Watershed characteristics

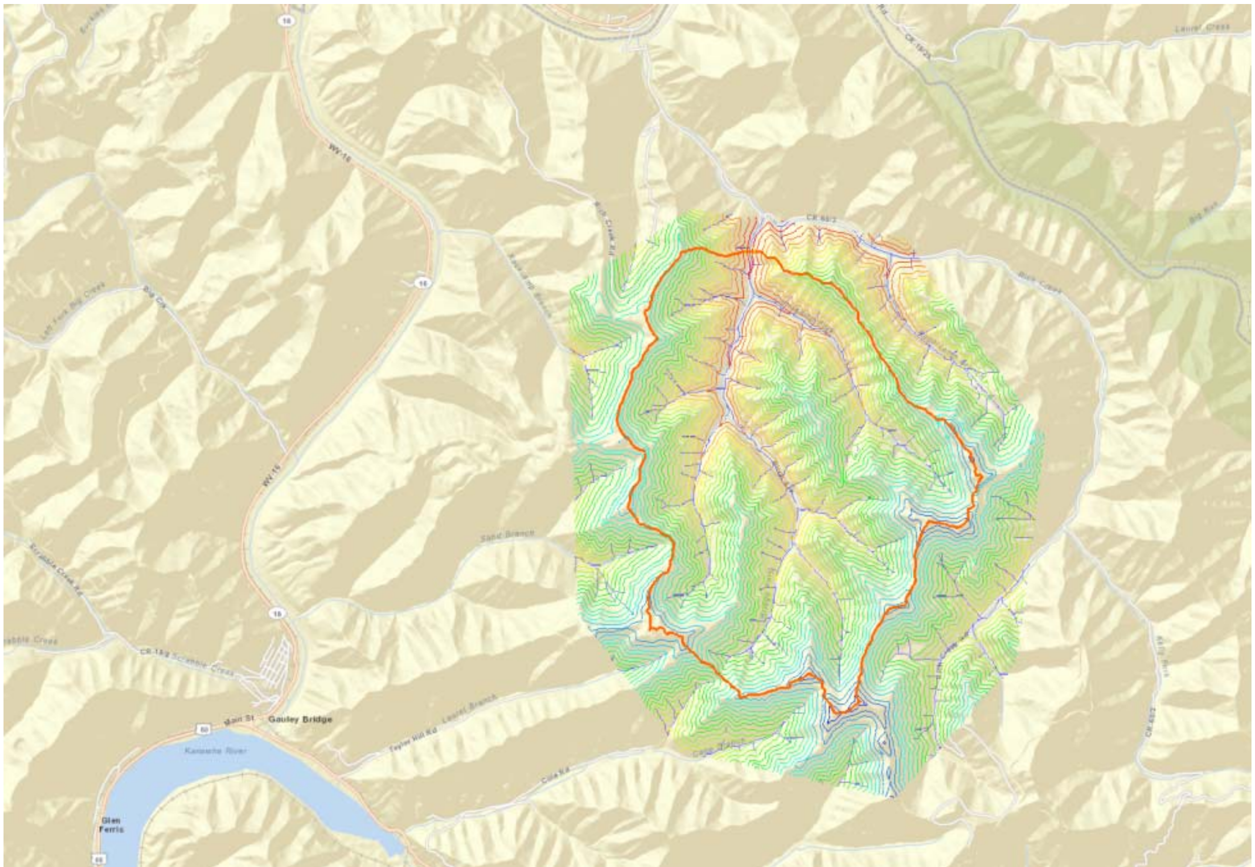

Figure 34- Image of delineated Bridge Fork watershed.

Several relevant hydrologic parameters for the Bridge Fork watershed are provided below in Table 127.

Table 127 - Watershed characteristics for Bridge Fork.

Parameter	Value
Location	Lat: 38.206508 Long: -81.140203
Region – USGS SIR2010-5033	Central Mountains
Basin Area (mi^2)	4.25
Average basin elevation (ft)	1596
Average basin overland slope (%)	54.4
Maximum flow distance (ft)	18,105
Slope along maximum flow distance (%)	8.1
Shape factor (basin length / basin width)	2.06
Sinuosity factor of stream (max. stream length / basin length)	1.09
Runoff Curve Number	53.1
Time of Concentration (hr.)	1.554

A summary of the parameters utilized to compute the weighted watershed Curve Number is provided in the table below, including Natural Resources Conservation Service Soil Survey Geographic Database (SSURGO) derived hydrologic soil group, National Land Cover Database 2006 derived land cover classification, corresponding Curve Number, and sub-element area.

Table 128 – Soil type, land cover, and curve number characteristics for Bridge Fork.

Hydrologic Soil Group	Land Cover Description	CN	Area (mi^2)
B	Deciduous Forest	55	3.458
A	Deciduous Forest	30	0.553
C	Deciduous Forest	70	0.028
B	Mixed Forest	55	0.01
A	Mixed Forest	30	0.004
B	Evergreen Forest	55	0.003
B	Barren Land	86	0.169
A	Developed Open Space	39	0.016
B	Developed Open Space	61	0.023
B	Grassland/Herbaceous	58	0.009
A	Barren Land	77	0.023
A	Grassland/Herbaceous	30	0.009
Weighted CN = 53.1			

Flow synthesis

Stream Gage Data. In the table below, characteristics of the stream gage stations in proximity to the Bridge Fork project site are summarized, including period of record and data coverage within that period, location, distance from the project area, elevation, drainage area upstream of the gage, and whether the available data consists of Daily Discharge (DD) or Annual Peak (AP).

The Bridge Fork stream itself is ungauged. Although there are several stream gauges within approximately 10 miles to the project site, they are of a vastly larger spatial scale than the Bridge Fork watershed, or do not include data coverage since the 2005 date that the Bridge Fork bridge was constructed. Thus, none of these stream gauges are suitable for direct flow history characterization at the Bridge Fork project site. Therefore, flow synthesis at the Bridge Fork site utilizes two methods: the applicable USGS peak flow regression equations, and creation of a watershed model that utilizes probabilistic rainfall data.

Table 129 – Stream gage stations near Bridge Fork.

Project / Stream Gage Location	USGS Station ID	Period	Covg. (%)	Lat.	Long.	Dist. (mi.)	Elev. (ft.)	Drainage Area (mi^2)	Type
Bridge Fork	-	*2005-Present*	-	*38.2065*	*-81.14*	-	*854*	*4.25*	-
GAULEY RIVER ABOVE BELVA, WV	03192000	1928-Present	100	38.233	-81.181	2.9	668.8	1,317	DD
GAULEY RIVER AT BELVA, WV	03192500	1909-1954	36	38.225	-81.192	3.1	663.5	1,402	AP
KANAWHA RIVER AT KANAWHA FALLS, WV	03193000	1877-Present	100	38.138	-81.214	6.2	620.6	8,371	DD
NEW RIVER @ FAYETTE	03186000	1895-1948	51	38.065	-81.078	10.3	838.4	6,850	AP
MEADOW RIVER NEAR MT. LOOKOUT, WV	03190400	1966-Present	96	38.19	-80.947	10.6	1199	365	DD
PETERS CREEK NEAR LOCKWOOD, WV	03191500	1945-Present	61	38.262	-81.023	7.4	1064	40.2	DD
LAUREL CREEK NR SUMMERSVILLE, WV	03191400	1966-1998	53	38.258	-80.99	8.9	1276	4.28	AP

Rainfall Data. A summary of weather stations near the project location are provided in the table below, including Station ID, Period of Record, data coverage during the Period of Record, location, elevation, and distance from the project site. Since the weather station that is available in the vicinity of the Bridge Fork watershed is 6.0 miles away, it is not suitable for direct inclusion into the watershed model.

Table 130 – Weather stations near Bridge Fork.

Station Name	**Station ID**	**Period of Record**	**Coverage (%)**	**Lat.**	**Long.**	**Elev. (ft.)**	**Dist. (mi.)**
Ansted Hawks Nest	USC00460202	1999-2012	97	38.121	-81.117	1296	6.0

Point precipitation frequency estimates for a 24-hour duration storm at the watershed, based on partial duration series NOAA Atlas 14 data available at the Hydrometeorological Design Studies Center's Precipitation Frequency Data Server are provided in Table 131.

Table 131 – Precipitation depth at Bridge Fork for a 24-hour duration storm, including 90% confidence interval.

Average Recurrence Interval (yr.)	Precipitation Depth (in.)	90% Confidence Interval	
		Precip. Depth, Lower (in.)	Precip. Depth, Upper (in.)
2	2.55	2.39	2.74
5	3.09	2.89	3.32
10	3.53	3.29	3.78
25	4.14	3.85	4.42
50	4.63	4.30	4.94
100	5.14	4.76	5.48
200	5.67	5.24	6.03
500	6.40	5.88	6.79

Development of flow rates at Bridge Fork for a variety of return period storms utilized three methods: USGS-provided regional regression equations (see Table 132), a TR-55 based watershed model that incorporated 24-hour storm duration precipitation depths (see Table 133), and a HEC-1 based watershed model that incorporated 24-hour storm duration precipitation depths (see Table 134). Since calculation of cumulative effective stream power requires characterization of flow rate as a function of time – and not simply the peak flow rate associated with a storm event – the runoff hydrographs generated by the HEC-1 analysis were utilized for this purpose.

Table 132 – USGS SIR-2010-5033 derived peak discharges, average prediction error, and corresponding 90% confidence interval for the Bridge Fork watershed.

Return Period (yr)	Peak Discharge, Q (cfs)	Avg. Prediction Error (%)	90% Confidence Interval	
			Q, Lower (cfs)	Q, Upper (cfs)
2	245	34.2	107	384
5	394	35.1	166	621
10	509	37.4	196	822
25	671	41.2	216	1126
50	804	44.4	217	1391
100	945	47.9	200	1690
200	1093	51.5	167	2019
500	1308	56.3	97	2519

Table 133 – Peak discharges derived from TR-55 watershed model of Bridge Fork

Return Period (yr)	Peak Discharge, Q (cfs)	90% Confidence Interval	
		Q, Lower (cfs)	Q, Upper (cfs)
2	14	7	27
5	67	41	106
10	151	101	216
25	330	236	435
50	521	388	662
100	759	579	937
200	1042	810	1252
500	1481	1163	1734

Table 134 – Peak discharges derived from HEC-1 watershed model of Bridge Fork.

Return Period (yr)	Peak Discharge, Q (cfs)	90% Confidence Interval	
		Q, Lower (cfs)	Q, Upper (cfs)
2	18	11	31
5	70	45	108
10	153	102	215
25	320	234	419
50	501	374	633
100	724	555	889
200	986	771	1181
500	1394	1098	1629

Hydraulics

An isometric view of the HEC-RAS Perspective Plot for Bridge Fork is shown in Figure 35, the underlying geometry for which is based on a field survey that was performed at the site. Manning's n-values for the site are summarized in Table 135. The result of HEC-RAS modeling at the site is a flow depth and velocity rating curve, presented in Figure 36.

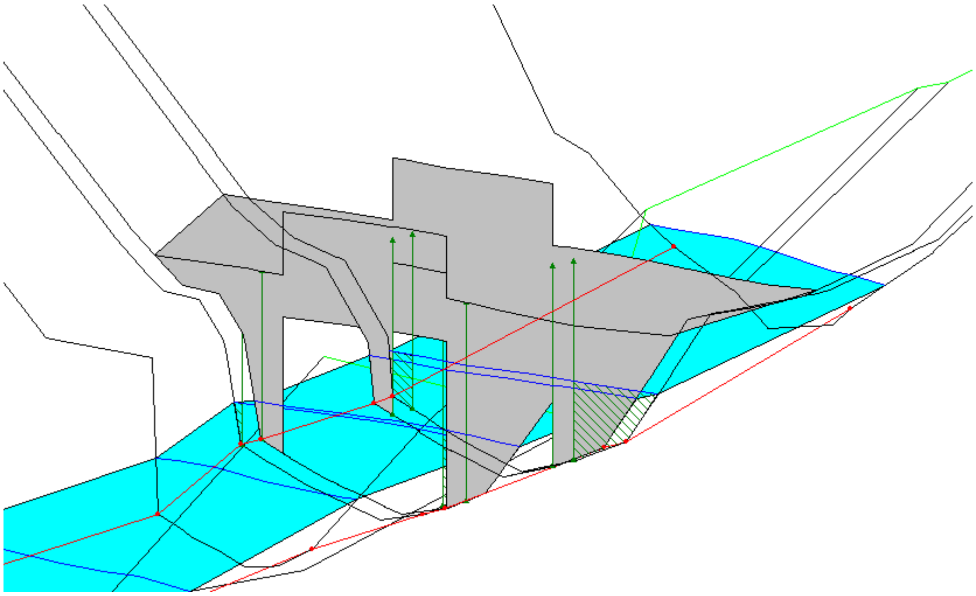

Figure 35- HEC-RAS X-Y-Z Perspective Plot for Bridge Fork.

Table 135 – Manning's n-values utilized in HEC-RAS model for Bridge Fork.

	Left Overbank	Channel	Right Overbank
U/S of Bridge	0.10	0.11	0.12
D/S of Bridge	0.10	0.11	0.12

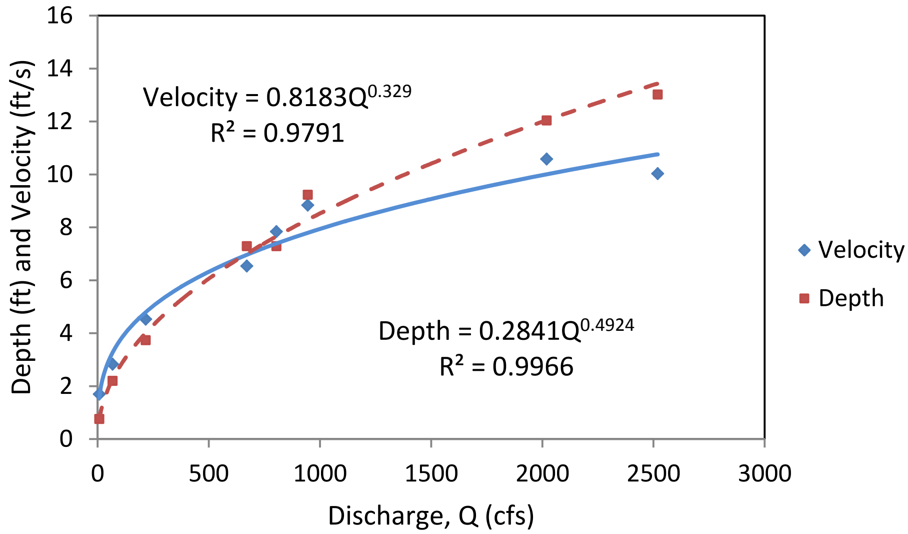

Figure 36- Flow depth and velocity rating curve for Bridge Fork.

Stream Power

Cumulative Effective Stream Power at the Leatherwood Road project site incorporated HEC-1 generated flow rates and runoff hydrograph. Amounts of Cumulative Effective Stream Power associated with a variety of return period storms are summarized in Table 136, including amounts associated with flow rates (and corresponding flow velocities and depths) that were generated using the lower and upper 90% confidence interval precipitation depths. The Average Annual Cumulative Effective Stream Power, which accounts for the Cumulative Effective Stream Power associated with each return period storm and the annual probability of each of these storms, is shown in Table 137.

Table 136 – Scour event Cumulative Effective Stream Power Ω (approach flow) for Bridge Fork – based on HEC-1 generated flow rates and hydrograph.

Return Period (yr)	Cumulative Effective Stream Power , Ω (ft·lb/ft^2)	90% Confidence Interval	
		Lower (ft·lb/ft^2)	Upper (ft·lb/ft^2)
2	0	0	0
5	108,824	99,871	145,836
10	243,148	214,900	316,388
25	454,735	400,245	554,197
50	640,446	564,351	757,125
100	844,483	742,228	980,792
200	1,065,519	936,702	1,214,669
500	1,381,581	1,206,898	1,551,284

Table 137 – Average Annual Cumulative Effective Stream Power Ω (approach flow) at Bridge Fork.

Increment (return period range)	λ size	Interval Cumulative Effective Stream Power, (ft·lb/ft^2)		
		Baseline Value	Lower 90% Conf.	Upper 90% Conf.
2-5	0.300	54,412	49,936	72,918
5-10	0.100	175,986	157,386	231,112
10-25	0.060	348,942	307,573	435,292
25-50	0.020	547,590	482,298	655,661
50-100	0.010	742,464	653,289	868,959
100-200	0.005	955,001	839,465	1,097,731
200-500	0.003	1,223,550	1,071,800	1,382,977
500-	0.002	1,381,581	1,206,898	1,551,284
Avg. Annual Cum. Eff. Stream Power, Ω (ft·lb/ft^2):		84,444	75,179	105,647

13 – Cucumber

Watershed characteristics

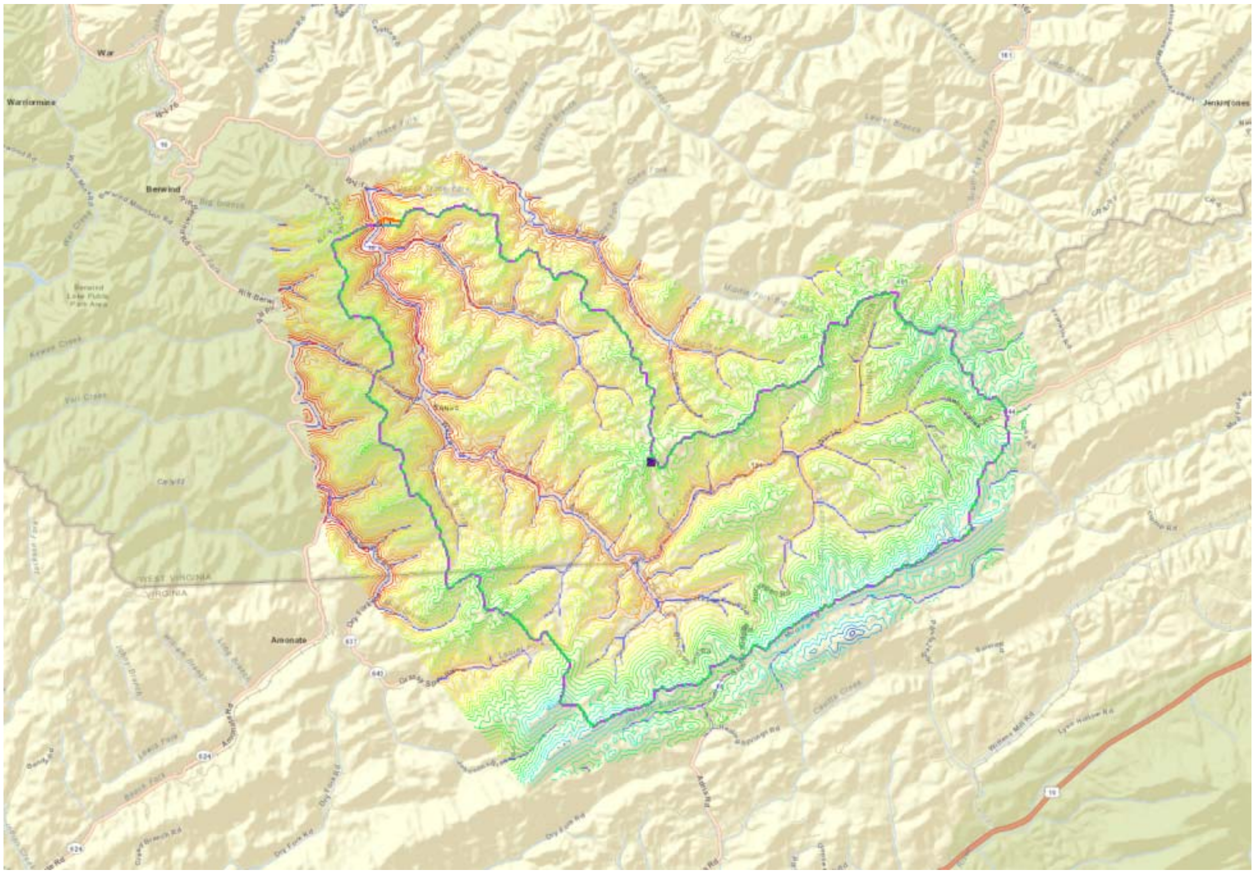

Figure 37- Image of delineated Cucumber watershed.

Since the Cucumber watershed exceeds the 19 mi^2 upper limit recommended for modeling with TR-55, it was broken into sub-basins, as shown in Figure 38.

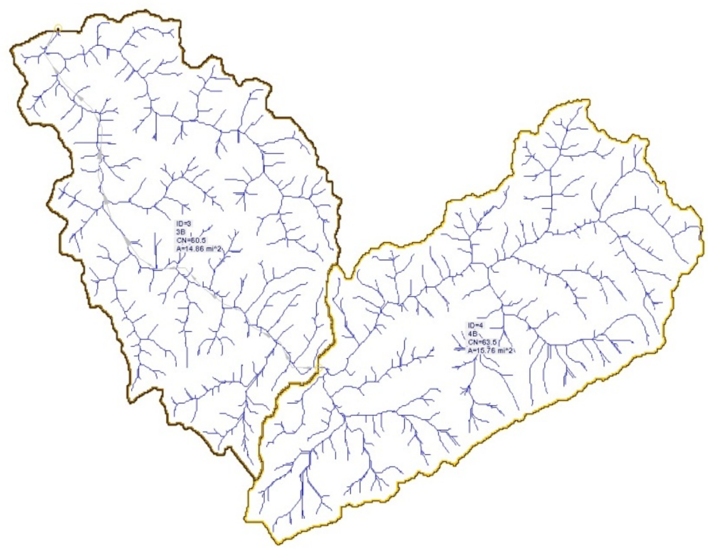

Figure 38- Cucumber watershed after subdivision.

Several relevant hydrologic parameters for the Cucumber watershed are provided below in Table 138.

Table 138 - Watershed characteristics for Cucumber.

Parameter	**Western Sub-Basin**	**Eastern Sub-Basin**
Main Watershed Outlet	Lat: 37.279844 Long: -81.621008	
Region – USGS SIR2010-5033	Western Plateaus	
Basin Area (mi^2)	14.86	15.76
Average basin elevation (ft)	2122.64	2435.21
Average basin overland slope (%)	39.7	35.4
Maximum flow distance (ft)	45128	35943
Slope along maximum flow distance (%)	2.7	3.6
Shape factor (basin length / basin width)	2.64	1.72
Sinuosity factor of stream (max. stream length / basin length)	1.34	1.27
Runoff Curve Number	60.5	63.5
Time of Concentration (hr.)	3.029	2.472

A summary of the parameters utilized to compute the weighted watershed Curve Number is provided in the table below, including Natural Resources Conservation Service Soil Survey Geographic Database

(SSURGO) derived hydrologic soil group, National Land Cover Database 2006 derived land cover classification, corresponding Curve Number, and sub-element area.

Table 139 – Soil type, land cover, and curve number characteristics for Cucumber, west sub-basin.

Hydrologic Soil Group	Land Cover Description	CN	Area (mi^2)
B	Deciduous Forest	55	10.111
C	Deciduous Forest	70	2.18
D	Grassland/Herbaceous	78	0.199
D	Developed Low Intensity	84	0.157
D	Developed Open Space	80	0.111
B	Developed Low Intensity	68	0.046
C	Mixed Forest	70	0.051
D	Deciduous Forest	77	0.18
B	Developed Open Space	61	0.148
D	Developed Medium Intensity	85	0.069
B	Grassland/Herbaceous	58	0.088
B	Mixed Forest	55	0.13
B	Pasture/Hay	61	0.102
C	Grassland/Herbaceous	71	0.083
B	Evergreen Forest	55	0.074
C	Evergreen Forest	70	0.074
C	Developed Open Space	74	0.185
A	Grassland/Herbaceous	30	0.014
A	Deciduous Forest	30	0.046
A	Developed Open Space	39	0.042
A	Evergreen Forest	30	0.005
A	Mixed Forest	30	0.005
C	Developed Low Intensity	79	0.028
B	Developed Medium Intensity	70	0.009
C	Pasture/Hay	74	0.042
D	Barren Land	94	0.046
C	Barren Land	91	0.162
D	Open Water	98	0.046
D	Shrub/Scrub	78	0.005
C	Developed Medium Intensity	80	0.005
B	Barren Land	86	0.398
D	Pasture/Hay	78	0.009
C	Open Water	98	0.005
B	Shrub/Scrub	58	0.009
Weighted CN = 60.5			

Table 140 – Soil type, land cover, and curve number characteristics for Cucumber, east sub-basin.

Hydrologic Soil Group	Land Cover Description	CN	Area (mi^2)
C	Deciduous Forest	70	5.654
C	Developed Open Space	74	0.286
B	Deciduous Forest	55	6.29
B	Developed Open Space	61	0.739
B	Pasture/Hay	61	0.335
C	Pasture/Hay	74	0.537
C	Developed Low Intensity	79	0.118
C	Developed Medium Intensity	80	0.035
B	Mixed Forest	55	0.118
B	Developed Low Intensity	68	0.148
C	Grassland/Herbaceous	71	0.242
C	Evergreen Forest	70	0.177
D	Deciduous Forest	77	0.039
D	Developed Medium Intensity	85	0.025
D	Developed Open Space	80	0.03
B	Evergreen Forest	55	0.217
B	Barren Land	86	0.158
D	Grassland/Herbaceous	78	0.005
B	Grassland/Herbaceous	58	0.212
C	Barren Land	91	0.212
D	Developed Low Intensity	84	0.01
D	Pasture/Hay	78	0.015
B	Developed Medium Intensity	70	0.01
A	Developed Low Intensity	51	0.01
C	Mixed Forest	70	0.039
A	Developed Open Space	39	0.079
A	Deciduous Forest	30	0.01
A	Developed Medium Intensity	54	0.01
D	Evergreen Forest	77	0.005
Weighted CN = 63.5			

Flow synthesis

Stream Gage Data. In the table below, characteristics of the stream gage stations in proximity to the Cucumber project site are summarized, including period of record and data coverage within that period, location, distance from the project area, elevation, drainage area upstream of the gage, and whether the available data consists of Daily Discharge (DD) or Annual Peak (AP).

The Jacob's Fork stream which the bridge at the Cucumber project location crosses is ungauged. The nearest stream gauge locations are for watersheds that include upstream flow regulation, are of significantly larger scale, or do not include data since the 1988 period since construction. Thus, none of the stream gauges identified are suitable for direct flow history characterization at the Cucumber project site. Therefore, flow synthesis at the Cucumber site utilizes two methods: the applicable USGS peak flow regression equations, and creation of a watershed model that utilizes probabilistic rainfall data.

Table 141 – Stream gage stations near Cucumber.

Project / Stream Gage Location	USGS Station ID	Period	Covg. (%)	Lat.	Long.	Dist. (mi.)	Elev. (ft.)	Drainage Area (mi^2)	Type
Cucumber, Jacob's Fork	-	*1988-Present*	-	*37.2798*	*-81.621*	-	*2070*	*30.63*	-
DRY FORK AT BEARTOWN, WV	03212980	1985-Present	92	37.395	-81.803	12.8	1055	209	DD
TUG FORK DOWNSTREAM OF ELKHORN CREEK AT WELCH, WV	03212750	1985-Present	92	37.441	-81.6	11.2	1267	174	DD
BLUESTONE RIVER AT BLUEFIELD, VA	03177700	1965-1980	100	37.26	-81.28	18.8	2350	39.8	DD

Rainfall Data. A summary of weather stations near the project location are provided in the table below, including Station ID, Period of Record, data coverage during the Period of Record, location, elevation, and distance from the project site. Neither of the weather stations is near enough to the Cucumber watershed to be meaningful for purposes of integration into watershed modeling.

Table 142 – Weather stations near Cucumber.

Station Name	Station ID	Period of Record	Coverage (%)	Lat.	Long.	Elev. (ft.)	Dist. (mi.)
Elkhorn	USC00462709	1892-2008	31	37.387	-81.405	1948	14.0
Bluefield	USC00460925	1998-2012	98	37.255	-81.226	2870	21.8

Point precipitation frequency estimates for a 24-hour duration storm at the watershed, based on partial duration series NOAA Atlas 14 data available at the Hydrometeorological Design Studies Center's Precipitation Frequency Data Server are provided in Table 143.

Table 143 – Precipitation depth at Cucumber for a 24-hour duration storm, including 90% confidence interval.

Average Recurrence Interval (yr.)	Precipitation Depth (in.)	90% Confidence Interval	
		Precip. Depth, Lower (in.)	Precip. Depth, Upper (in.)
2	2.49	2.33	2.67
5	3.01	2.81	3.23
10	3.43	3.20	3.68
25	4.03	3.74	4.31
50	4.51	4.17	4.82
100	5.00	4.61	5.36
200	5.53	5.06	5.91
500	6.24	5.66	6.67

Development of flow rates at Cucumber for a variety of return period storms utilized three methods: USGS-provided regional regression equations (see Table 144), a TR-55 based watershed model that incorporated 24-hour storm duration precipitation depths (see Table 145), and a HEC-1 based watershed model that incorporated 24-hour storm duration precipitation depths (see Table 146). Since calculation of cumulative effective stream power requires characterization of flow rate as a function of time – and not simply the peak flow rate associated with a storm event – the runoff hydrographs generated by the HEC-1 analysis were utilized for this purpose.

Table 144 – USGS SIR-2010-5033 derived peak discharges, average prediction error, and corresponding 90% confidence interval for the Cucumber watershed.

Return Period (yr)	Peak Discharge, Q (cfs)	Avg. Prediction Error (%)	90% Confidence Interval	
			Q, Lower (cfs)	Q, Upper (cfs)
2	1568	32.2	738	2399
5	2509	30.0	1271	3748
10	3193	29.7	1633	4753
25	4118	30.3	2065	6170
50	4853	31.3	2354	7352
100	5591	32.5	2602	8581
200	6363	33.9	2815	9911
500	7441	36.1	3022	11860

Table 145 – Peak discharges derived from TR-55 watershed model, two sub-basins, of Cucumber.

Return Period (yr)	Peak Discharge, Q (cfs)	90% Confidence Interval	
		Q, Lower (cfs)	Q, Upper (cfs)
2	743	538	1016
5	1643	1257	2121
10	2602	2053	3260
25	4287	3427	5191
50	5847	4731	6904
100	7545	6182	8881
200	9536	7763	11051
500	12418	10047	14267

Table 146 – Peak discharges derived from HEC-1 watershed model of Cucumber.

Return Period (yr)	Peak Discharge, Q (cfs)	90% Confidence Interval	
		Q, Lower (cfs)	Q, Upper (cfs)
2	649	479	907
5	1415	1070	1806
10	2203	1731	2770
25	3606	2914	4349
50	4923	3960	5856
100	6388	5228	7545
200	8093	6574	9354
500	10557	8497	12132

Hydraulics

An isometric view of the HEC-RAS Perspective Plot for Cucumber is shown in Figure 39, the underlying geometry for which is based on a field survey that was performed at the site. Manning's n-values for the site are summarized in Table 147. The result of HEC-RAS modeling at the site is a flow depth and velocity rating curve, presented in Figure 40.

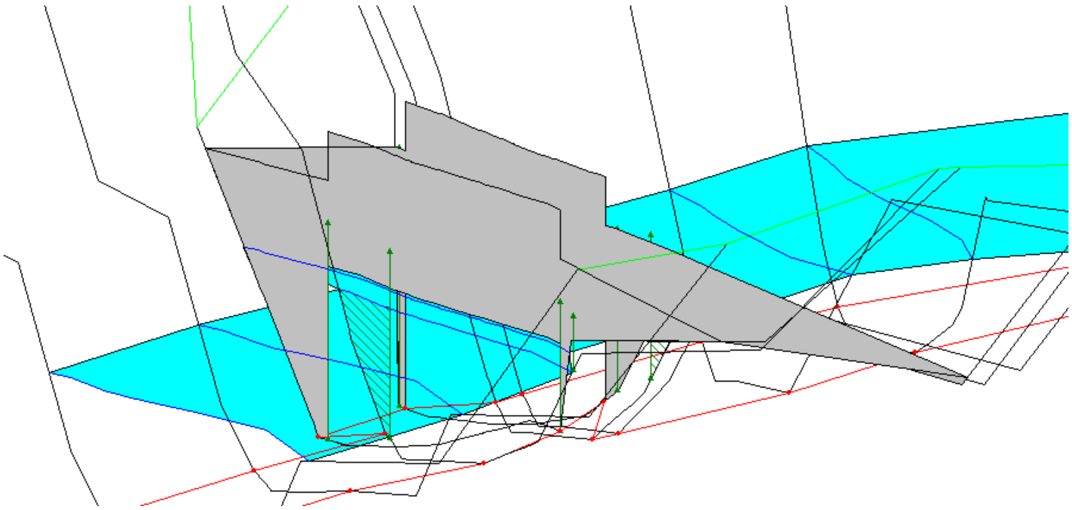

Figure 39- HEC-RAS X-Y-Z Perspective Plot for Cucumber.

Table 147 – Manning’s n-values utilized in HEC-RAS model for Cucumber.

	Left Overbank	Channel	Right Overbank
U/S of Bridge	0.07	0.04	0.10
D/S of Bridge	0.07	0.04	0.10

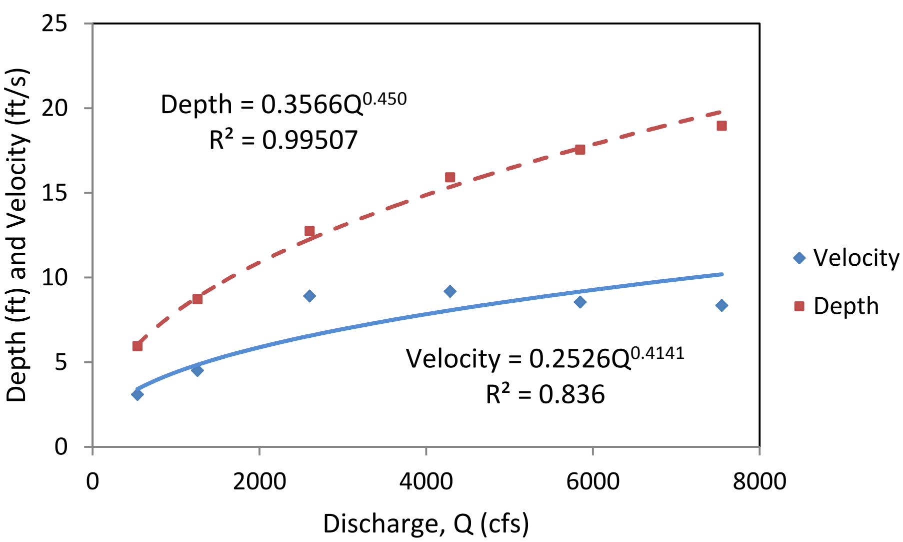

Figure 40- Flow depth and velocity rating curve for Cucumber.

Stream Power

Cumulative Effective Stream Power at the Leatherwood Road project site incorporated HEC-1 generated flow rates and runoff hydrograph. Amounts of Cumulative Effective Stream Power associated with a variety of return period storms are summarized in Table 148, including amounts associated with flow rates (and corresponding flow velocities and depths) that were generated using the lower and upper 90% confidence interval precipitation depths. The Average Annual Cumulative Effective Stream Power, which accounts for the Cumulative Effective Stream Power associated with each return period storm and the annual probability of each of these storms, is shown in Table 149.

Table 148 – Scour event Cumulative Effective Stream Power Ω (approach flow) for Cucumber – based on HEC-1 generated flow rates and hydrograph.

Return Period (yr)	Cumulative Effective Stream Power , Ω (ft·lb/ft^2)	90% Confidence Interval	
		Lower (ft·lb/ft^2)	Upper (ft·lb/ft^2)
2	0	0	0
5	10,742	9,451	18,196
10	29,132	25,561	45,507
25	67,687	58,764	96,058
50	106,957	92,251	145,104
100	152,545	131,177	204,678
200	206,739	175,563	258,208
500	259,089	234,579	369,081

Table 149 – Average Annual Cumulative Effective Stream Power Ω (approach flow) at Cucumber.

Increment (return period range)	λ size	Interval Cumulative Effective Stream Power, Ω (ft·lb/ft^2)		
		Baseline Value	Lower 90% Conf.	Upper 90% Conf.
2-5	0.300	5,371	4,726	9,098
5-10	0.100	19,937	17,506	31,852
10-25	0.060	48,410	42,162	70,782
25-50	0.020	87,322	75,508	120,581
50-100	0.010	129,751	111,714	174,891
100-200	0.005	179,642	153,370	231,443
200-500	0.003	232,914	205,071	313,644
500-	0.002	259,089	234,579	369,081
Avg. Annual Cum. Eff. Stream Power, Ω (ft·lb/ft^2):		11,669	10,177	17,158

14 – Clear Fork

Watershed characteristics

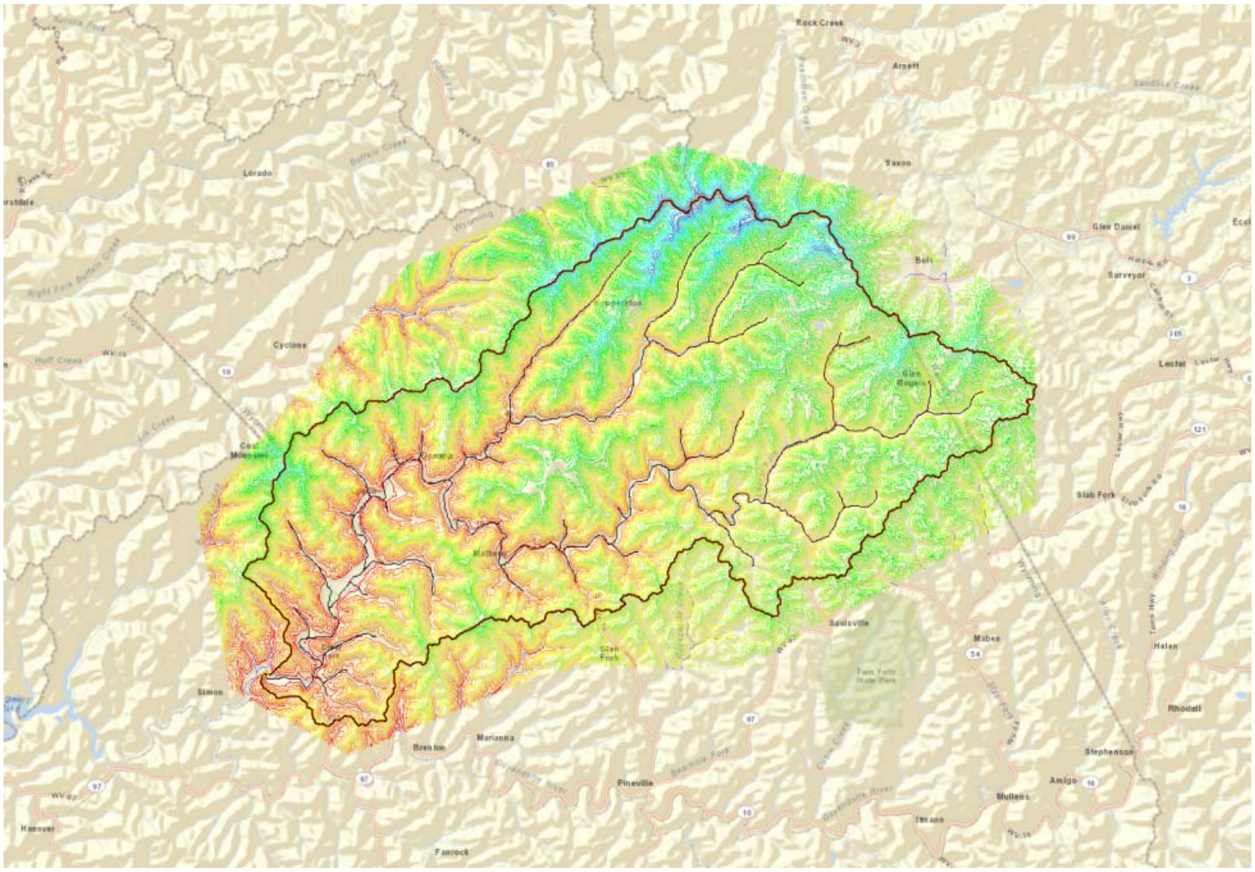

Figure 41- Image of delineated Clear Fork watershed.

Several relevant hydrologic parameters for the Clear Fork watershed are provided below in Table 150. Since flow conditions at the Clear Fork project site were characterized using stream gauge data, information related to watershed characteristics, precipitation patterns, and available weather stations is provided only for reference. When generating a runoff hydrograph for purposes of scaling it against the PeakFQ generated peak flow rates, multiple sub-basins were created. Additional details about these sub-basins can be found in the digital watershed model files for this location.

Table 150 - Watershed characteristics for Clear Fork.

Parameter	Value
Location	Lat: 37.623803 Long: -81.703639
Region – USGS SIR2010-5033	Western Plateaus
Basin Area (mi^2)	123.42
Average basin elevation (ft)	2017
Average basin overland slope (%)	45.2
Maximum flow distance (ft)	189,369
Slope along maximum flow distance (%)	0.8
Shape factor (basin length / basin width)	3.18
Sinuosity factor of stream (max. stream length / basin length)	1.73

Flow synthesis

Stream Gage Data. In the table below, characteristics of the stream gage stations in proximity to the Clear Fork project site are summarized, including period of record and data coverage within that period, location, distance from the project area, elevation, drainage area upstream of the gage, and whether the available data consists of Daily Discharge (DD) or Annual Peak (AP).

The Clear Fork river is a gauged stream, with a stream gauge being found approximately 0.2 miles downstream of the project site, where the drainage area is 126 mi^2, slightly larger than the 123.42 mi^2 contributing area at the project site. Stream gauge data was obtained from USGS and PKFQWin analysis was performed, and the resulting flood frequency analysis results were interpolated between to yield the flood frequency analysis presented in Table **152**.

Table 151 – Stream gage stations near Clear Fork.

Project / Stream Gage Location	USGS Station ID	Period	Covg. (%)	Lat.	Long.	Dist. (mi.)	Elev. (ft.)	Drainage Area (mi^2)	Type
Clear Fork	-	*1917-Present*	-	*37.6238*	*-81.704*	-	*1163*	*123.42*	-
CLEAR FORK AT CLEAR FORK, WV	03202750	1974-Present	100	37.623	-81.707	0.2	1149	126	DD

Table 152 – Flood Frequency Analysis for Clear Fork.

Recurrence Interval (yr)	Discharge (ft^3/s)	95% Confidence Interval	
		Lower	Upper
2	4137	3650	4681
5	6189	5375	7188
10	7766	6552	9243
25	10050	8104	12293
50	11989	9313	14899
100	14174	10569	17798
200	16642	11891	21021
500	20433	13753	25859

Rainfall Data. A summary of weather stations near the project location are provided in the table below, including Station ID, Period of Record, data coverage during the Period of Record, location, elevation, and distance from the project site. The single available weather station is not usable as a data source for integration into watershed modeling, both because of excessive distance from the Clear Fork watershed, and also because of gaps in data coverage during the period of record.

Table 153 – Weather stations near Clear Fork.

Station Name	**Station ID**	**Period of Record**	**Coverage (%)**	**Lat.**	**Long.**	**Elev. (ft.)**	**Dist. (mi.)**
Pineville	USC00467029	1908-Present	62	37.574	-81.535	1280	9.9

Point precipitation frequency estimates for a 24-hour duration storm at the watershed, based on partial duration series NOAA Atlas 14 data available at the Hydrometeorological Design Studies Center's Precipitation Frequency Data Server are provided in Table 154.

Table 154 – Precipitation depth at Clear Fork for a 24-hour duration storm, including 90% confidence interval.

Average Recurrence Interval (yr.)	Precipitation Depth (in.)	90% Confidence Interval	
		Precip. Depth, Lower (in.)	Precip. Depth, Upper (in.)
2	2.58	2.41	2.79
5	3.13	2.92	3.37
10	3.57	3.33	3.85
25	4.19	3.89	4.51
50	4.70	4.34	5.04
100	5.22	4.81	5.60
200	5.77	5.29	6.18
500	6.52	5.95	6.96

Table 155 – USGS SIR-2010-5033 derived peak discharges, average prediction error, and corresponding 90% confidence interval for the Clear Fork watershed.

Return Period (yr)	Peak Discharge, Q (cfs)	Avg. Prediction Error (%)	90% Confidence Interval	
			Q, Lower (cfs)	Q, Upper (cfs)
2	4338	32.2	2040	6636
5	6750	30.0	3419	10080
10	8458	29.7	4326	12590
25	10741	30.3	5387	16095
50	12536	31.3	6082	18991
100	14304	32.5	6657	21951
200	16142	33.9	7140	25143
500	18694	36.1	7593	29796

Hydraulics

An isometric view of the HEC-RAS Perspective Plot for Clear Fork is shown in Figure 42, the underlying geometry for which is based on a field survey that was performed at the site. Manning's n-values for the site are summarized in Table 156. The result of HEC-RAS modeling at the site is a flow depth and velocity rating curve, presented in Figure 43. Due to unique hydraulic characteristics at this site, the velocity rating curve at this location has been intentionally truncated.

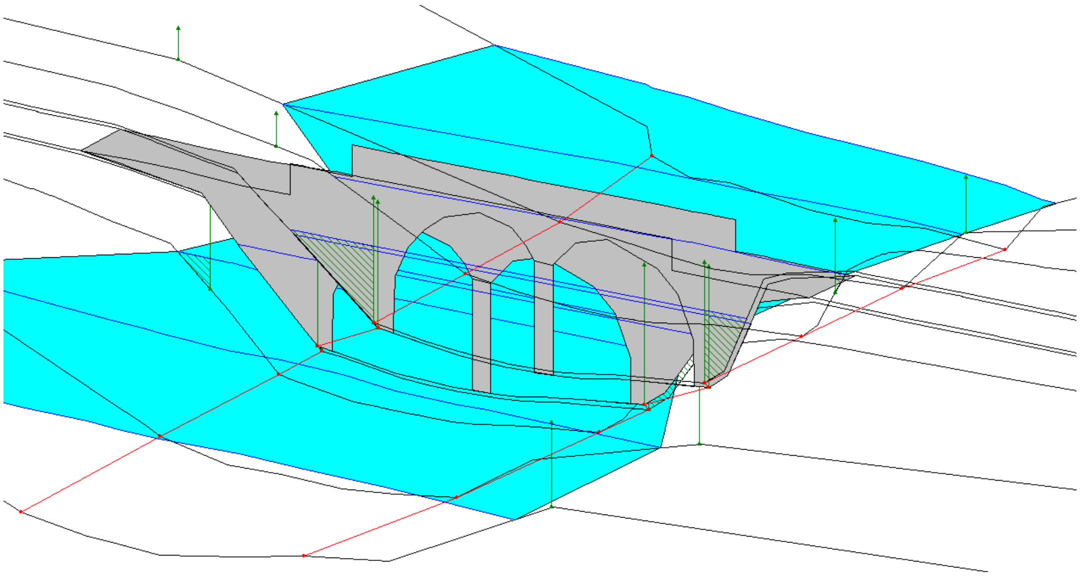

Figure 42- HEC-RAS X-Y-Z Perspective Plot for Clear Fork.

Table 156 – Manning's n-values utilized in HEC-RAS model for Clear Fork.

	Left Overbank	Channel	Right Overbank
U/S of Bridge	0.13	0.085	0.13
D/S of Bridge	0.13	0.085	0.13

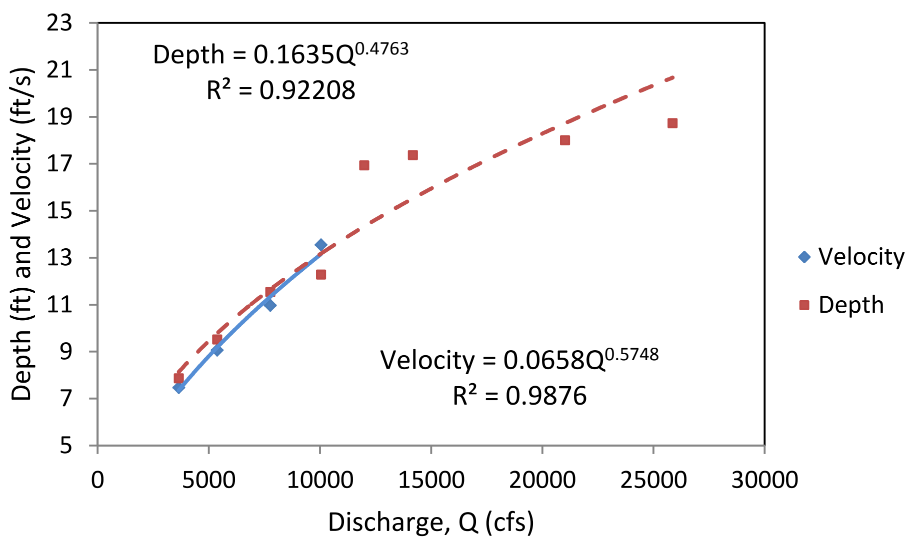

Figure 43- Flow depth and velocity rating curve for Clear Fork.

Stream Power

Cumulative Effective Stream Power at the Leatherwood Road project site incorporated PeakFQ generated peak flow rates and a runoff hydrograph shape that was determined through HEC-1 modeling. Amounts of Cumulative Effective Stream Power associated with a variety of return period storms are summarized in Table 157 (approach flow) and Table 158 (at pier), including amounts associated with flow rates (and corresponding flow velocities and depths) that were generated using the lower and upper 90% confidence interval precipitation depths. The Average Annual Cumulative Effective Stream Power, which accounts for the Cumulative Effective Stream Power associated with each return period storm and the annual probability of each of these storms, is shown in Table 159 and Table 160.

Table 157 – Scour event Cumulative Effective Stream Power Ω (approach flow) for Clear Fork – based on PeakFQ generated flow rates and HEC-1 generated hydrograph shape.

Return Period (yr)	Cumulative Effective Stream Power , Ω (ft·lb/ft^2)	95% Confidence Interval	
		Lower (ft·lb/ft^2)	Upper (ft·lb/ft^2)
2	0	0	0
5	328,519	258,174	459,809
10	799,401	572,061	1,088,940
25	1,619,349	1,066,702	2,250,527
50	2,359,916	1,503,952	3,425,617
100	3,383,652	2,036,361	4,790,608
200	4,602,354	2,538,973	6,716,977
500	6,670,029	3,428,959	9,374,418

Table 158 – Scour event Cumulative Effective Stream Power Ω (pier) for Clear Fork – based on PeakFQ generated flow rates and HEC-1 generated hydrograph shape.

Return Period (yr)	Cumulative Effective Stream Power , Ω (ft·lb/ft^2)	95% Confidence Interval	
		Lower (ft·lb/ft^2)	Upper (ft·lb/ft^2)
2	0	0	0
5	949,421	746,122	1,328,847
10	2,310,270	1,653,256	3,147,036
25	4,679,919	3,082,768	6,504,023
50	6,820,156	4,346,421	9,900,033
100	9,778,754	5,885,083	13,844,857
200	13,300,804	7,337,632	19,412,064
500	19,276,384	9,909,691	27,092,068

Table 159 – Average Annual Cumulative Effective Stream Power Ω (approach flow) at Clear Fork.

Increment (return period range)	λ size	Interval Cumulative Effective Stream Power, Ω (ft·lb/ft^2)		
		Baseline Value	Lower 90% Conf.	Upper 90% Conf.
2-5	0.300	164,260	129,087	229,904
5-10	0.100	563,960	415,117	774,374
10-25	0.060	1,209,375	819,381	1,669,733
25-50	0.020	1,989,632	1,285,327	2,838,072
50-100	0.010	2,871,784	1,770,156	4,108,112
100-200	0.005	3,993,003	2,287,667	5,753,793
200-500	0.003	5,636,192	2,983,966	8,045,698
500	0.002	6,670,029	3,428,959	9,374,418
Avg. Annual Cum. Eff. Stream Power, Ω (ft·lb/ft^2):		296,961	200,057	416,090

Table 160 – Average Annual Cumulative Effective Stream Power Ω (pier) at Clear Fork.

Increment (return period range)	λ size	Interval Cumulative Effective Stream Power, Ω (ft·lb/ft^2)		
		Baseline Value	90% Lower	90% Upper
2-5	0.300	474,710	373,061	664,424
5-10	0.100	1,629,845	1,199,689	2,237,942
10-25	0.060	3,495,095	2,368,012	4,825,530
25-50	0.020	5,750,038	3,714,594	8,202,028
50-100	0.010	8,299,455	5,115,752	11,872,445
100-200	0.005	11,539,779	6,611,357	16,628,460
200-500	0.003	16,288,594	8,623,662	23,252,066
500	0.002	19,276,384	9,909,691	27,092,068
Avg. Annual Cum. Eff. Stream Power, Ω (ft·lb/ft^2):		858,216	578,164	1,202,501

15 – Mish Road

Watershed characteristics

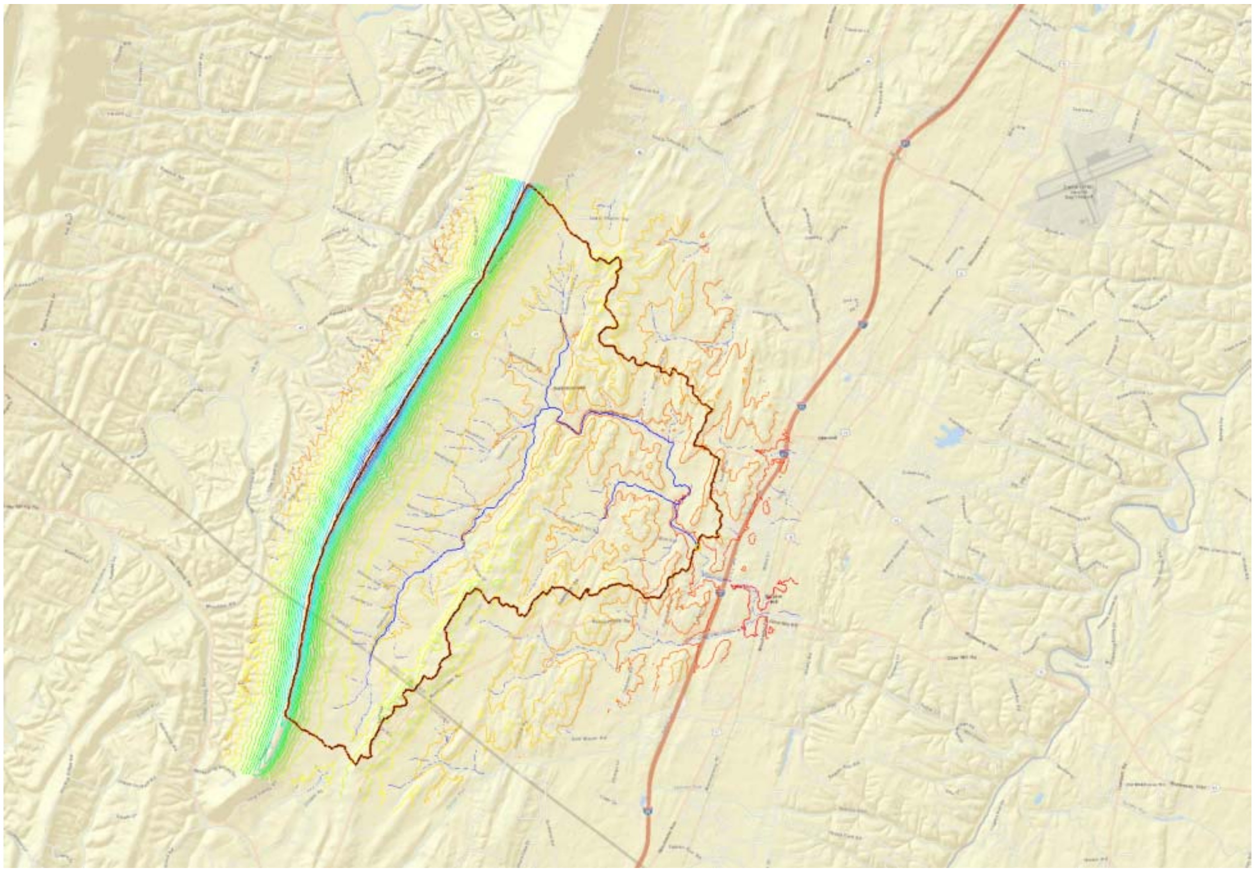

Figure 44- Image of delineated Mish Road watershed.

Several relevant hydrologic parameters for the Mish Road watershed are provided below in Table 161.

Table 161 - Watershed characteristics for Mish Road.

Parameter	**Value**
Location	Lat: 39.3800 Long: -78.066101
Region – USGS SIR2010-5033	Eastern Panhandle
Basin Area (mi^2)	14.21
Average basin elevation (ft)	772
Average basin overland slope (%)	9.8
Maximum flow distance (ft)	51,651
Slope along maximum flow distance (%)	1.3
Shape factor (basin length / basin width)	1.69
Sinuosity factor of stream (max. stream length / basin length)	1.59
Runoff Curve Number	68.0
Time of Concentration (hr.)	5.594

A summary of the parameters utilized to compute the weighted watershed Curve Number is provided in the table below, including Natural Resources Conservation Service Soil Survey Geographic Database (SSURGO) derived hydrologic soil group, National Land Cover Database 2006 derived land cover classification, corresponding Curve Number, and sub-element area.

Table 162 – Soil type, land cover, and curve number characteristics for Mish Road.

Hydrologic Soil Group	Land Cover Description	CN	Area (mi^2)
C	Deciduous Forest	70	2.1
C	Pasture/Hay	74	2.059
B	Deciduous Forest	55	2.273
D	Pasture/Hay	78	2.211
C	Developed Open Space	74	0.126
B	Pasture/Hay	61	2.643
D	Deciduous Forest	77	1.216
C	Mixed Forest	70	0.051
C	Developed Low Intensity	79	0.028
B	Cultivated Crops	75	0.034
D	Developed Open Space	80	0.099
B	Developed Open Space	61	0.237
D	Evergreen Forest	77	0.198
D	Developed Low Intensity	84	0.018
B	Evergreen Forest	55	0.354
C	Evergreen Forest	70	0.246
D	Barren Land	94	0.005
D	Mixed Forest	77	0.092
D	Cultivated Crops	86	0.007
B	Developed Low Intensity	68	0.039
B	Mixed Forest	55	0.064
B	Developed Medium Intensity	70	0.002
D	Open Water	98	0.005
D	Woody Wetlands	77	0.002
B	Woody Wetlands	55	0.002
B	Grassland/Herbaceous	58	0.039
C	Grassland/Herbaceous	71	0.021
D	Emergent Herbaceous Wetlands	98	0.005
A	Pasture/Hay	39	0.03
A	Deciduous Forest	30	0.009
A	Evergreen Forest	30	0.002
Weighted CN = 68.0			

Flow synthesis

Stream Gage Data. In the table below, characteristics of the stream gage stations in proximity to the Mish Road project site are summarized, including period of record and data coverage within that period, location, distance from the project area, elevation, drainage area upstream of the gage, and whether the available data consists of Daily Discharge (DD) or Annual Peak (AP).

Mill Creek is an ungauged stream. The nearest gauged stream, Opequon Creek near Martinsburg, West Virginia, gauges a watershed that is 273 mi^2 and only since 1947 (the bridge at Mish Road was constructed in 1926). Likewise, the other stream gauges in the vicinity of the Mish Road project site are all for much larger watersheds, do not include data during the entire period of interest, and/or are far enough away from the project site that precipitation patterns (and therefore stream flow conditions) are not likely to have been representative of conditions at the Mish Road project site. Therefore, flow synthesis at the Mish Road site utilizes two methods: the applicable USGS peak flow regression equations, and creation of a watershed model that utilizes probabilistic rainfall data.

Table 163 – Stream gage stations near Mish Road.

Project / Stream Gage Location	USGS Station ID	Period	Covg. (%)	Lat.	Long.	Dist. (mi.)	Elev. (ft.)	Drainage Area (mi^2)	Type
Mish Road, Mill Creek	-	*1926-Present*	-	*39.38*	*-78.066*	-	*666*	*14.21*	-
SHENANDOAH RIVER AT MILLVILLE, WV	01636500	1895-Present	84	39.282	-77.789	16.3	292.4	3,022	DD
OPEQUON CREEK NEAR MARTINSBURG, WV	01616500	1947-Present	100	39.424	-77.939	7.4	354.3	273	DD
OPEQUON CREEK NEAR BERRYVILLE, VA	01615000	1943-Present	93	39.18	-78.07	13.8	503.2	57.4	DD
ABRAMS CREEK NEAR WINCHESTER, VA	01616000	1949-1994	59	39.18	-78.09	13.9	526.4	16.5	DD
HOGUE CREEK NEAR HAYFIELD, VA	01613900	1960-2011	89	39.21	-78.29	16.8	668.6	15	DD

Rainfall Data. A summary of weather stations near the project location are provided in Table 164, including Station ID, Period of Record, data coverage during the Period of Record, location, elevation, and distance from the project site. The coverage and period of record at for the weather station at the Martinsburg Eastern WV Airport are acceptable, however the location of the weather station is 4.6 miles from the Mish Road watershed outlet, and is the opposite direction (east) from the area that contributes to flow at Mish Road (west). Thus, in view of the distance and spatial variations in rainfall conditions, data from this site are not suitable for direct integration into watershed modeling.

Table 164 – Weather stations near Mish Road.

Station Name	**Station ID**	**Period of Record**	**Coverage (%)**	**Lat.**	**Long.**	**Elev. (ft.)**	**Dist. (mi.)**
Martinsburg Eastern WV Airport	USW00013734	1926-2012	100	39.401	-77.984	534	4.6

Point precipitation frequency estimates for a 24-hour duration storm at the watershed, based on partial duration series NOAA Atlas 14 data available at the Hydrometeorological Design Studies Center's Precipitation Frequency Data Server are provided in Table 165.

Table 165 – Precipitation depth at Mish Road for a 24-hour duration storm, including 90% confidence interval.

Average Recurrence Interval (yr.)	Precipitation Depth (in.)	90% Confidence Interval	
		Precip. Depth, Lower (in.)	Precip. Depth, Upper (in.)
2	2.86	2.64	3.10
5	3.56	3.28	3.85
10	4.14	3.80	4.46
25	4.98	4.55	5.36
50	5.69	5.18	6.12
100	6.46	5.84	6.93
200	7.29	6.54	7.82
500	8.51	7.54	9.14

Development of flow rates at Mish Road for a variety of return period storms utilized three methods: USGS-provided regional regression equations (see Table 166), a TR-55 based watershed model that incorporated 24-hour storm duration precipitation depths (see Table 167), and a HEC-1 based watershed model that incorporated 24-hour storm duration precipitation depths (see Table 168). Since calculation of cumulative effective stream power requires characterization of flow rate as a function of time – and not simply the peak flow rate associated with a storm event – the runoff hydrographs generated by the HEC-1 analysis were utilized for this purpose.

Table 166 – USGS SIR-2010-5033 derived peak discharges, average prediction error, and corresponding 90% confidence interval for the Mish Road watershed.

Return Period (yr)	Peak Discharge, Q (cfs)	Avg. Prediction Error (%)	90% Confidence Interval	
			Q, Lower (cfs)	Q, Upper (cfs)
2	544	33.4	245	843
5	971	27.2	536	1405
10	1355	24.5	809	1901
25	1937	22.4	1223	2650
50	2437	21.7	1567	3307
100	2969	21.7	1909	4029
200	3568	22.4	2253	4882
500	4427	23.8	2694	6161

Table 167 – Peak discharges derived from TR-55 watershed model of Mish Road.

Return Period (yr)	Peak Discharge, Q (cfs)	90% Confidence Interval	
		Q, Lower (cfs)	Q, Upper (cfs)
2	626	490	787
5	1130	916	1365
10	1614	1324	1902
25	2396	1986	2774
50	3112	2593	3566
100	3933	3269	4451
200	4857	4020	5464
500	6271	5141	7022

Table 168 – Peak discharges derived from HEC-1 watershed model of Mish Road.

Return Period (yr)	Peak Discharge, Q (cfs)	90% Confidence Interval	
		Q, Lower (cfs)	Q, Upper (cfs)
2	629	496	788
5	1127	915	1363
10	1612	1321	1901
25	2398	1985	2778
50	3118	2596	3574
100	3947	3275	4473
200	4884	4036	5499
500	6315	5172	7073

Hydraulics

An isometric view of the HEC-RAS Perspective Plot for Mish Road is shown in Figure 45, the underlying geometry for which is based on a field survey that was performed at the site. Manning's n-values for the site are summarized in

Table 169. The result of HEC-RAS modeling at the site is a flow depth and velocity rating curve, presented in Figure 46.

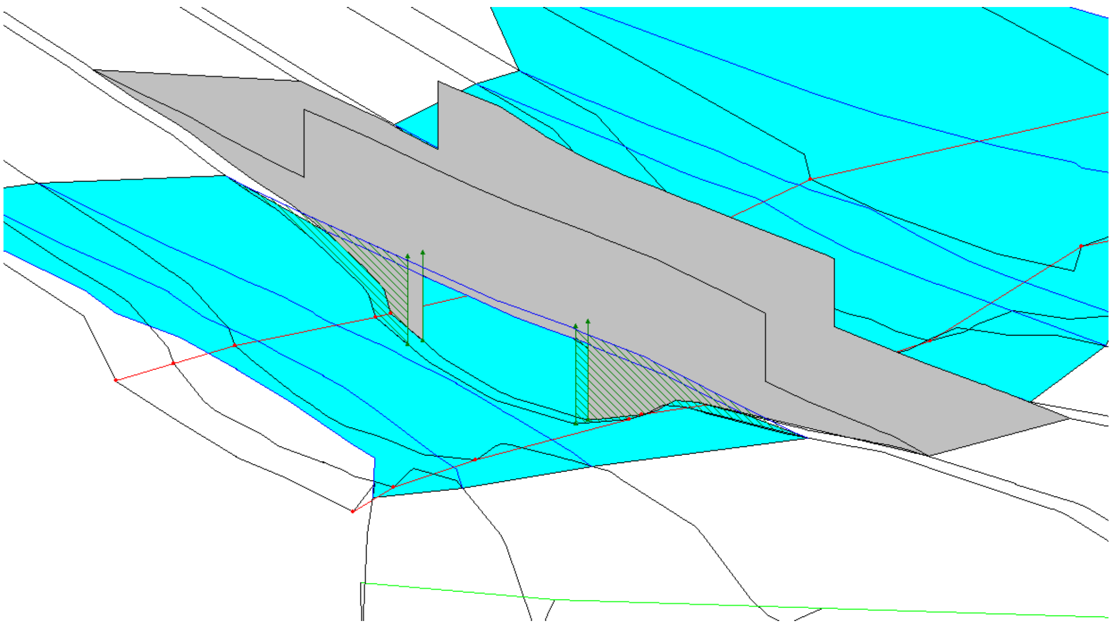

Figure 45- HEC-RAS X-Y-Z Perspective Plot for Mish Road.

Table 169 – Manning's n-values utilized in HEC-RAS model for Mish Road.

	Left Overbank	Channel	Right Overbank
U/S of Bridge	0.10	0.035	0.10
D/S of Bridge	0.10	0.035	0.10

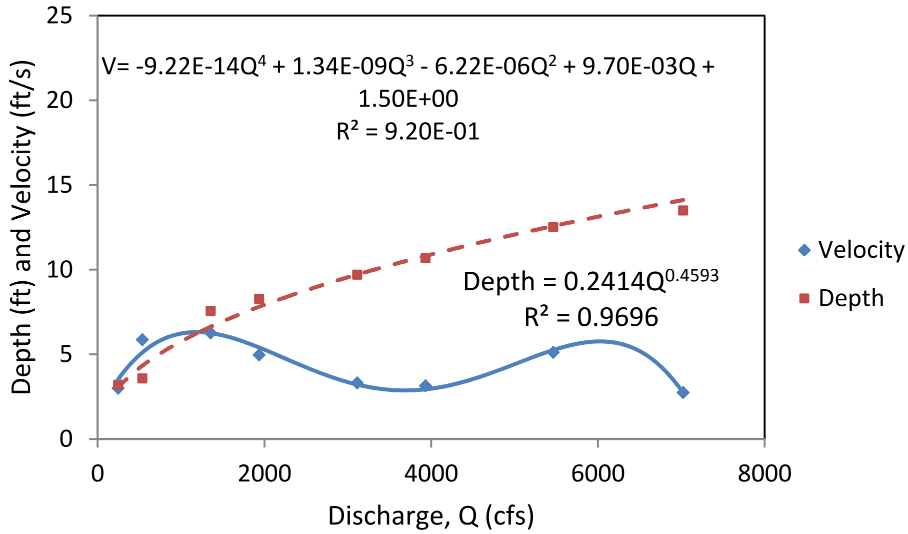

Figure 46- Flow depth and velocity rating curve for Mish Road.

Stream Power

Cumulative Effective Stream Power at the Leatherwood Road project site incorporated HEC-1 generated flow rates and runoff hydrograph. Amounts of Cumulative Effective Stream Power associated with a variety of return period storms are summarized in Table 170, including amounts associated with flow rates (and corresponding flow velocities and depths) that were generated using the lower and upper 90% confidence interval precipitation depths. The Average Annual Cumulative Effective Stream Power, which accounts for the Cumulative Effective Stream Power associated with each return period storm and the annual probability of each of these storms, is shown in Table 171.

Table 170 – Scour event Cumulative Effective Stream Power Ω (approach flow) for Mish Road – based on HEC-1 generated flow rates and hydrograph.

Return Period (yr)	Cumulative Effective Stream Power , Ω (ft·lb/ft^2)	90% Confidence Interval	
		Lower (ft·lb/ft^2)	Upper (ft·lb/ft^2)
2	0	0	0
5	19,107	25,057	21,558
10	22,637	42,653	26,012
25	19,116	41,442	39,209
50	19,782	40,622	44,260
100	21,029	42,142	53,253
200	21,955	43,263	57,043
500	21,534	43,302	55,627

Table 171 – Average Annual Cumulative Effective Stream Power Ω (approach flow) at Mish Road.

Increment (return period range)	λ size	Interval Cumulative Effective Stream Power, Ω (ft·lb/ft^2)		
		Baseline Value	Lower 90% Conf.	Upper 90% Conf.
2-5	0.300	9,553	12,529	10,779
5-10	0.100	20,872	33,855	23,785
10-25	0.060	20,876	42,048	32,610
25-50	0.020	19,449	41,032	41,734
50-100	0.010	20,406	41,382	48,757
100-200	0.005	21,492	42,702	55,148
200-500	0.003	21,744	43,282	56,335
500-	0.002	21,534	43,302	55,627
Avg. Annual Cum. Eff. Stream Power, Ω (ft·lb/ft^2):		7,015	11,331	9,447

Appendix B – Project Site Geologic and Geotechnical Information

Regional distribution, thickness, and stratigraphic classification of bedrock units encountered at the bridge sites are based on those from the digital version of the 1:250,000 state geologic map of West Virginia (Cardwell, D.H., Erwin, R.B., and Woodward, H.P., 1968 [slightly revised 1986], Geologic Map of West Virginia: West Virginia Geological and Economic Survey). These general properties of the rock units have been augmented with detailed site-specific descriptions based on the boring logs generated during rock coring and subsequent inspection of core and surface samples.

The following acronyms are used in this appendix:

RQD = rock quality designation, stated in percent (%).

GSN = geotechnical scour number obtained from modified slake durability tests following the method of NCHRP-717 (2012), stated in units of feet of equivalent scour per unit hourly ft-lb/s/ft^2 of stream power.

GSN*= geotechnical scour number obtained from modified slake durability tests based on a non-zero intercept of the linear regression line, an alternative method to NCHRP 717 (2012); stated in units of feet of equivalent scour per unit hourly ft-lb/s/ft^2 of stream power.

R^{*2} = coefficient of determination associated with linear regression used to determine GSN*.

UCS = unconfined compressive strength, stated in units of pounds per square inch (psi).

TZ = Target zone: the 10-foot vertical interval within bedrock immediately below the base of the adjacent bridge abutment.

A list of figures and respective pages for Appendix B appear below. The results of lab tests for all bridges are summarized in Table 1 on pages 111 and 112.

1 – Leatherwood Road

Field Results

<u>Geology</u>.

At the Leatherwood Bridge the bridge abutments are set in sandstone and claystone near the contact between the Pennsylvanian-age Allegheny and Glenshaw Formations. The Allegheny Formation covers in excess of 1,400 square miles over large strip of West Virginia extending from Wayne County in the southwestern portion of the state to Preston and Mineral Counties in the northeastern portion. The Allegheny is thickest in the western parts of its distribution, up to 325 feet, and thinner to the northeast, where it is typically in the range of 150-200 feet.

Together the Glenshaw and overlying Casselman Formation comprise the Conemaugh Group, which covers in excess of 3,100 square miles extending from Wayne, Cabell, and Mason Counties in the southwestern portion of the state to Monongalia, Preston and Mineral Counties in the northeastern portion. The Casselman/ Glenshaw Formations also cover part of Hancock County in the northern panhandle and Wirt and Pleasant Counties adjacent to the Ohio River in the north-central part of the state. In contrast to the Allegheny Formation, the Casselman/ Glenshaw Formations are thickest in the northeastern part of the state, up to 850 feet, and thinner, approximately 500 feet, in the western part of their distribution.

Based on surface exposure and the two boreholes drilled in June 2011, bedrock geology in the TZ at the Leatherwood bridge consists of gray, shaley sandstone below the right (east) abutment in borehole LEA-001-2011 and claystone and clayey shale in LEA-002-2011 below the left (west) abutment. The core from LEA-001-2011 is described as soft to average hardness, with very thin bedding and some fracturing, including one vertical, iron-stained fracture. The core from LEA-002-2011 is described as soft to very soft, with laminations and closely-spaced fracturing, including one vertical fracture. Boring logs and field sketches are shown below.

Surface exposure of bedrock at the base of the right (east) abutment shows shale on the downstream (north) side laterally adjacent to sandstone on the upstream (south) side. The sharp juxtaposition of these two dissimilar lithologies is interpreted as resulting from displacement along a near-vertical fault. The fault is assumed to be inactive, as it does not extend into the unconsolidated materials above bedrock or does not appear to have affected the abutment.

Two joint sets, oriented approximately 90° to each other and 45° to the stream axis, were observed in surface bedrock below and immediately downstream of the Leatherwood bridge. Spacing between individual joints in each set ranges from 2-3 meters.

Rock Quality Designation (RQD).

RQD values for the target depth intervals in LEA-001-2011 (shaley sandstone) and LEA-002-2011 (claystone) were 72% and 0%, respectively.

Scour Mode and Magnitude

Areas of scour at the Leatherwood bridge are shown on the field sketches below.

Figure 1 – Boring Log for Leatherwood Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **LEA-01-2011**
SHEET **1** OF **2**
DATE: START **6/24/11**
END **6/24/11**
O.G. ELEV. **642.0**

PROJECT NAME **WVDOH Scour** COUNTY **Kanawha**
PROJECT LOCATION **Leatherwood Bridge**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______
STATE RT. NO. **65** SECT. ______ SEGMENT ______ OFFSET ______
INSPECTOR **Nichole Wendlandt** DRILLERS NAME/COMPANY **Jim Hopkins/L.G. Hetager Drilling, Inc.**
EQUIPMENT USED **CME 45**
DRILLING METHODS **Hollow Stem Auger/SPT/NX Wireline Split Barrel**
CASING: SIZE: ______ ; DEPTH: ______ ; WATER: DEPTH: ______ TIME: ______ DATE: ______
CHECKED BY: **TLD** ; DATE: **8/1/11** DEPTH: **10.0'** TIME: **16:15** DATE: **6/24/11**
Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT		DESCRIPTION	REMARKS
0.0	A-1				-				UNSAMPLED	Started hole at 1400 hrs. 6/24/2011 Cloudy/70 degrees ~Bottom of Abutment at 626' Sand trap used for all soil sampling Stream pH=8.4
13.0	S-1	7 8 4	0.5'	33	-	sc - a-2-6	M		13.0 629.0 Clayey SAND (sc), brown, moist, medium dense, homogeneous, (residual)	
14.5	S-2	1 10 50/0.2	1.2'	100	-	sc - a-2-6	M			Started coring at 1448 hrs.
15.7	R-1		2.3'	100 - 22					15.7 626.3 Shaley SANDSTONE, gray, soft to average hard, slightly to freshly weathered, intensely to thinly bedded (RD= 0° to 25°), extremely closely to very widely spaced fractures (RD= 0° to 90°), RQD=72% 16.4'-16.6' stained vertical fracture	R-1: 1448-1451 hrs.
18.0	R-2		2.6'	87 - 80						R-2: 1520-1530 hrs.

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

PROJECT NAME **WVDOH Scour** COUNTY **Kanawha**
PROJECT LOCATION **Leatherwood Bridge**
STATION ________ OFFSET FROM CENTERLINE ________
NORTHING ________ EASTING ________

BORING NO. **LEA-01-2011**
SHEET **2** OF **2**
DATE: START **6/24/11**
END **6/24/11**
O.G. ELEV. **642.0**

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H2O CONTENT	DESCRIPTION	REMARKS
21.0								Shaley SANDSTONE, gray, soft to average hard, slightly to freshly weathered, intensely to thinly bedded (RD= 0° to 25°), extremely closely to very widely spaced fractures (RD= 0° to 90°), RQD=72% *(continued)*	
	R-3		5.0'	100 - 90					R-3: 1535-1545 hrs. Completed hole at 1360 hrs. 6/24/2011
26.0								26.0 616.0	
								Bottom of borehole at 26.00 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

Figure 2 – Boring Log 2 for Leatherwood Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. LEA-02-2011
SHEET 1 OF 2
DATE: START 6/24/11
END 6/24/11
O.G. ELEV. 636.0

PROJECT NAME WVDOH Scour COUNTY Kanawha
PROJECT LOCATION Leatherwood Bridge
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______
STATE RT. NO. 65 SECT. ______ SEGMENT ______ OFFSET ______
INSPECTOR Nichole Wendlandt DRILLERS NAME/COMPANY Jim Hopkins/L.G. Hetager Drilling, Inc.
EQUIPMENT USED CME 45
DRILLING METHODS Hollow Stem Auger/SPT/NX Wireline Split Barrel
CASING: SIZE: ______ ; DEPTH: ______ ; WATER: DEPTH: ______ TIME: ______ DATE: ______
CHECKED BY: TLD ; DATE: 8/1/11 DEPTH: 15.2' TIME: 13:00 DATE: 6/24/11
Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
0.0 – 5.0	A-1				-			UNSAMPLED	Started hole at 0925 hrs. 6/24/2011 Cloudy/68 degrees ~Bottom of Abutment at 626'
5.0 – 6.5	S-1	5, 3, 3	0.9'	60	-	cl - a-6	M	5.0 631.0 Sandy CLAY (cl), moist, medium stiff to stiff, homogeneous, (fill)	Sand trap used for all soil sampling
6.5 – 8.0	S-2	8, 9, 13	1.1'	73	-	sp - a-1-b	M	7.0 629.0 COARSE SAND (sp), moist, loose to medium dense, homogeneous, (alluvial)	
8.0 – 9.5	S-3	4, 3, 3	0.9'	60	-	sp - a-1-b	M		
9.5 – 11.0	S-4	8, 9, 12	0.9'	60	-	sp - a-1-b	M	S-4 through S-8 pieces of weathered sandstone	
11.0 – 12.5	S-5	5, 8, 6	1.0'	67	-	sp - a-1-b	M		
12.5 – 14.0	S-6	2, 3, 5	0.4'	27	-	sp - a-1-b	M		
14.0 – 15.5	S-7	5, 8, 10	1.3'	87	-	sp - a-1-b	W		
15.5 – 17.0	S-8	3, 4, 6	0.9'	60	-	sp - a-1-b	W		
17.0 – 18.5	S-9	4, 4, 8	0.2'	13	-	sp - a-1-b	M		
18.5 – 20.0	S-10	6, 7, 8	0.7'	47	-	sp - a-1-b	M	19.5 616.5	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **LEA-02-2011**
SHEET **2** OF **2**
DATE: START **6/24/11**
END **6/24/11**
O.G. ELEV. **636.0**

PROJECT NAME **WVDOH Scour** COUNTY **Kanawha**
PROJECT LOCATION **Leatherwood Bridge**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
								20.5 Sandy CLAY (cl), moist, stiff, homogeneous, (residual) *(continued)* 615.5	
21.5	S-11	6 8 22	1.5'	100	-	gc - a-2-6	M	Clayey GRAVEL (gc), moist, medium dense to very dense, homogeneous, (residual shale)	
22.2	S-12	8 50/0.2	0.7'	100	-	gc - a-2-6	M	22.2 613.8	Started coring at 1205 hrs.
24.5	R-1		2.3'	100 - 0				CLAYSTONE, gray, very soft, slightly to freshly weathered, intensely to thinly bedded (RD= 0° to 30°), extremely closely to closely spaced fractures (RD= 0° to 90°), RQD=0% 22.9'-23.2' vertical fracture 25.0 611.0	R-1: 1205-1215 hrs.
27.5	R-2		2.8'	93 - 50				Clayey SHALE, dark gray, soft, freshly weathered, intensely laminated (RD= 0° to 10°), very wide to closely spaced fractures (RD= 0° to 30°), RQD=60% 27.5 608.5	R-2: 1226-1236 hrs. Completed hole at 1335 hrs. 6/24/2011
								Bottom of borehole at 27.50 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

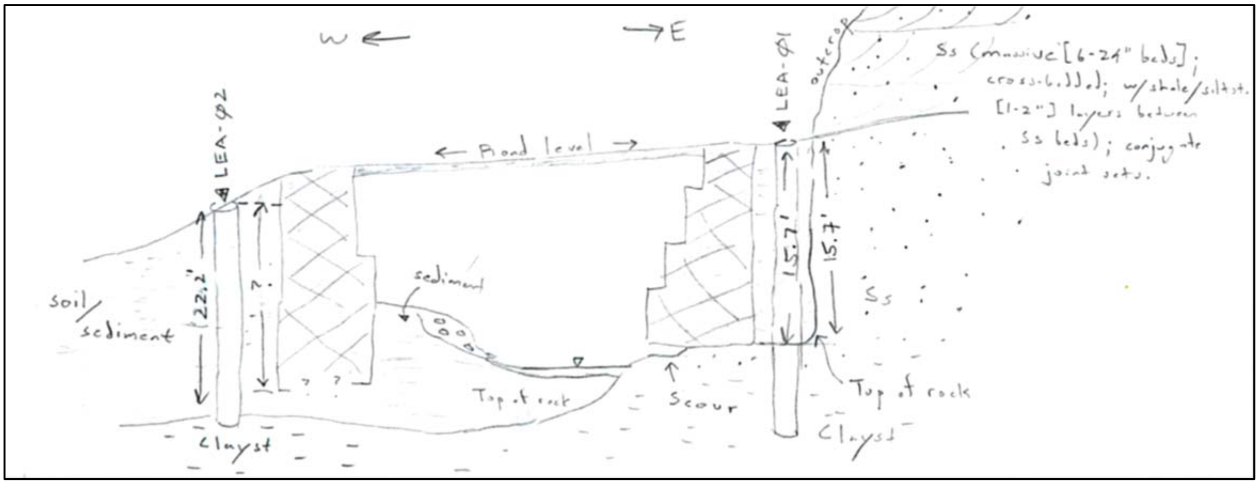

Figure 3 -- Field sketch of Leatherwood Bridge, looking downstream.

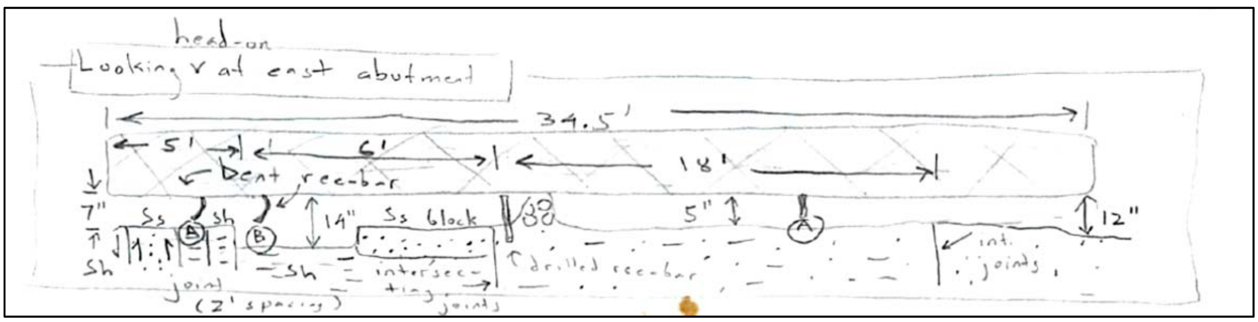

Figure 4 -- Field sketch of Leatherwood Bridge, looking at right abutment.

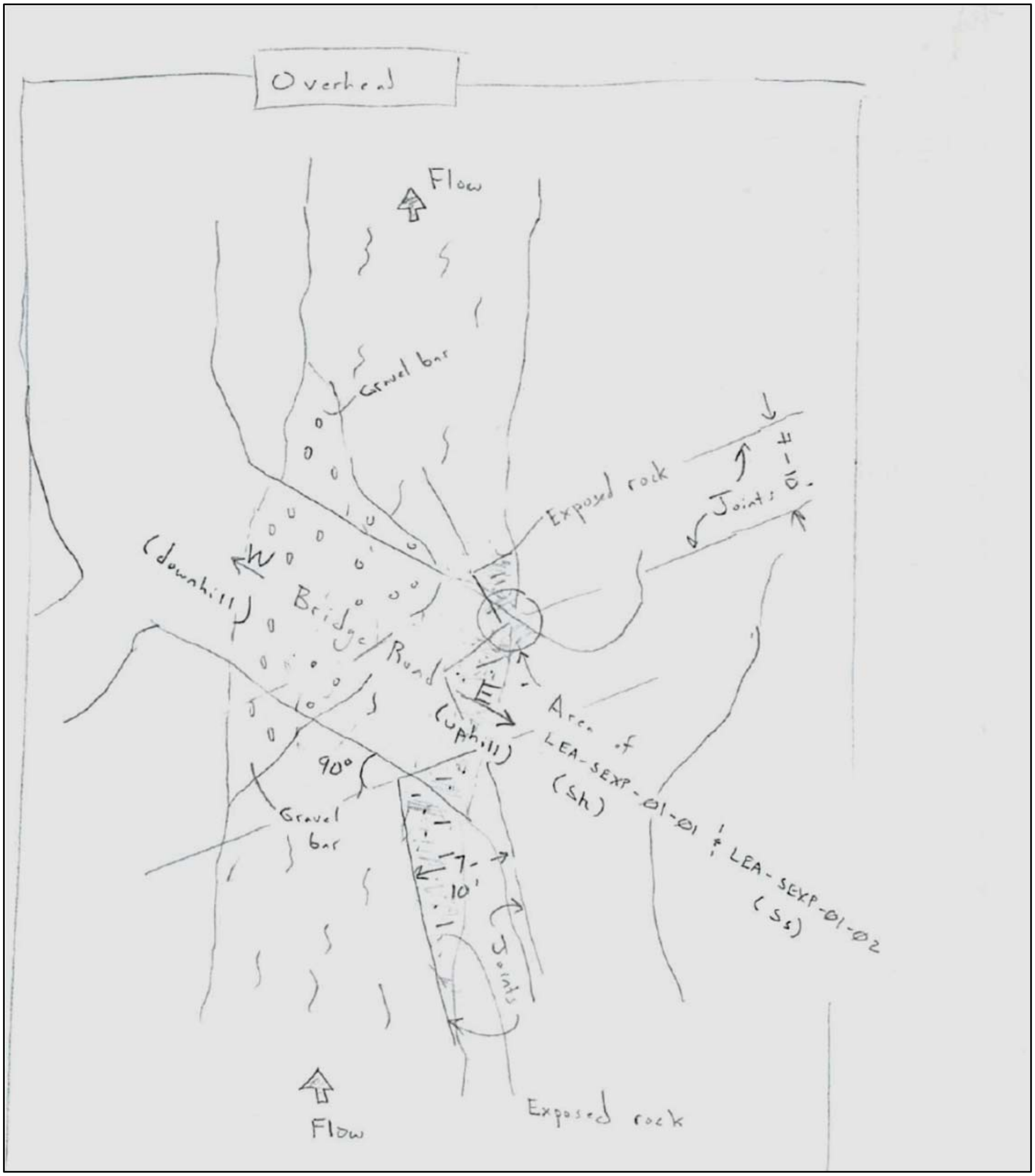

Figure 5 – Field sketch of Leatherwood Bridge, overhead view.

Lab Results

Modified Slake Durability and GSN*.

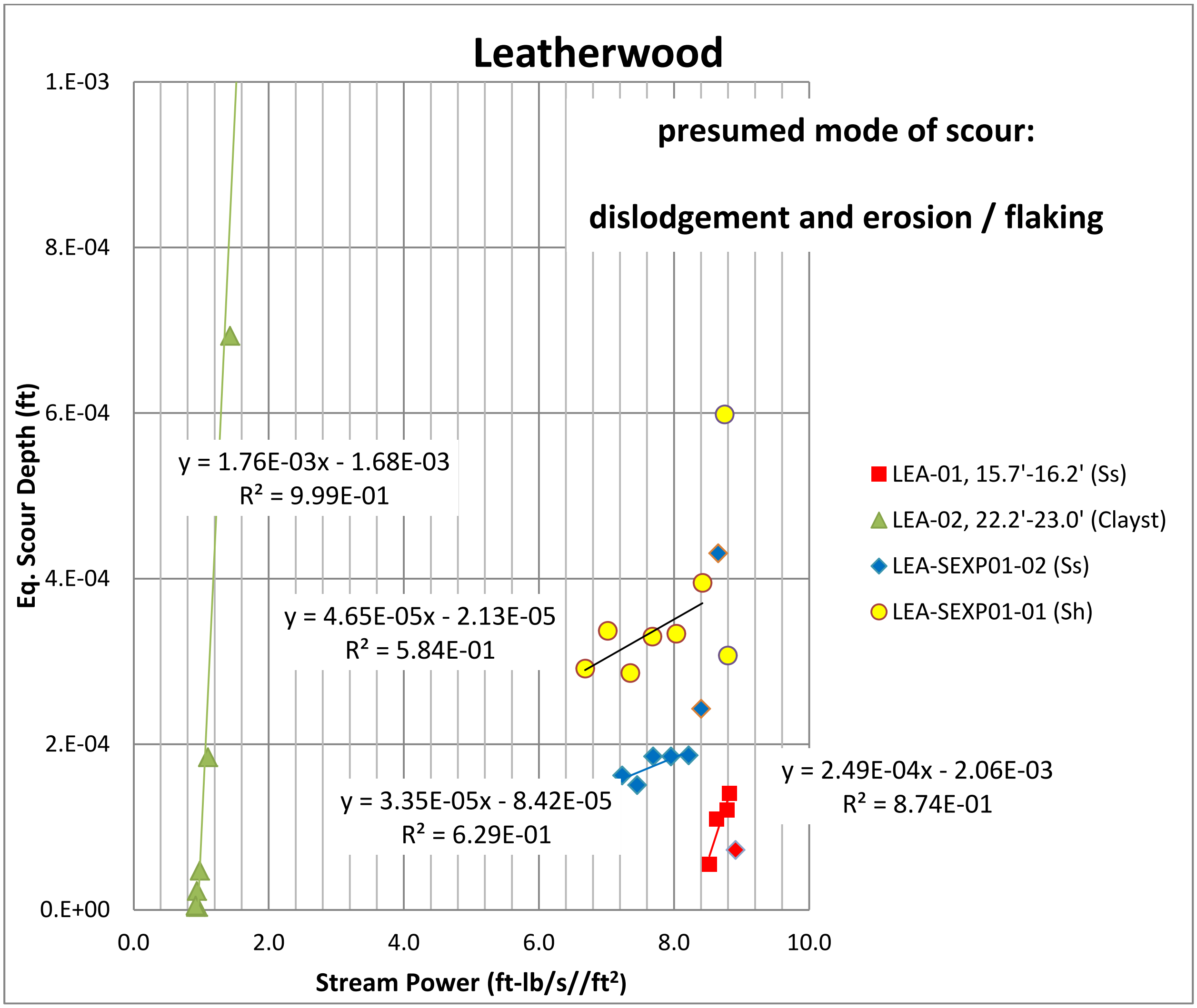

Figure 6 – Data from modified slake durability tests and calculated values of GSN* and R^{*2} for Leatherwood samples.

2 – Fifth Street Ritter Park

Field Results

Geology.

At the Fifth Street Ritter Park bridge site the bridge abutments are set in the Casselman/Glenshaw Formations which together comprise the Conemaugh Group and cover in excess of 3,100 square miles extending from Wayne, Cabell, and Mason Counties in the southwestern portion of the state to Monongalia, Preston and Mineral Counties in the northeastern portion. The Casselman/ Glenshaw Formations also cover part of Hancock County in the northern panhandle and Wirt and Pleasant Counties adjacent to the Ohio River in the north-central part of the state. The Casselman/ Glenshaw Formations are thickest in the northeastern part of the state, up to 850 feet, and thinner, approximately 500 feet, in the western part of their distribution.

Based on surface exposure and the two boreholes drilled in June 2011, bedrock geology at the Ritter Park bridge consists of gray, shaley sandstone in the TZ below both abutments. The cores from RIT-001-2011 and RIT-002-2011 are described as soft to average hardness, with very thin to medium bedding and some fracturing, including four vertical fractures in core from RTI-001-2011. Numerous micaceous stringers within the sandstone were noted in both cores. Boring logs and field sketches are shown below.

Rock Quality Designation (RQD).

RQD of core recovered from RIT-001-2011 ranged from 0 to 46%; RQD of core from RIT-002-2011 was 83%.

Scour Mode and Magnitude

Areas of scour at the Fifth Street (Ritter Park) bridge are shown on the field sketches below.

Figure 7 – Boring Log 1 for Ritter Park (5th Street) Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. RIT-01-2011
SHEET 1 OF 2
DATE: START 6/23/11
END 6/23/11
O.G. ELEV. 556.0

PROJECT NAME WVDOH Scour COUNTY Cabell
PROJECT LOCATION 5th St. Ritter Park
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______
STATE RT. NO. 527 SECT. ______ SEGMENT ______ OFFSET ______
INSPECTOR Nichole Wendlandt DRILLERS NAME/COMPANY Jim Hopkins/L.G. Hetager Drilling, Inc.
EQUIPMENT USED CME 45
DRILLING METHODS Hollow Stem Auger/SPT/NX Wireline Split Barrel
CASING: SIZE: ______ ; DEPTH: ______ ; WATER: DEPTH: ______ TIME: ______ DATE: ______
CHECKED BY: TLD ; DATE: 8/1/11 DEPTH: 4.0' TIME: 14:05 DATE: 6/23/11
Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
0.0–8.0	A-1				-			UNSAMPLED	Started hole at 1200 hrs. 6/23/2011 Sunny/82 degrees ~Bottom of Abutment at 543' Sand trap used for all soil sampling
8.0–9.5	S-1	2 4 8	1.5'	100	-	sc - a-2-6	M	8.0 548.0 8.5 547.5 Sandy CLAY (cl), brown, moist, medium stiff to medium stiff, homogeneous, (alluvial) Clayey SAND (sc), brown, moist, medium dense, homogeneous, (alluvial)	
9.5–11.0	S-2	9 4 6	1.5'	100	-	sc - a-2-6	M	10.5 545.5 Medium Grained SAND With Gravel (sp), reddish-brown, moist, medium dense, homogeneous, (alluvial)	
11.0–12.5	S-3	9 14 22	1.4'	93	-	gc - a-2-6	M	11.5 544.5 Clayey GRAVEL (gc), black, moist, dense to very dense, homogeneous, (residual sandstone)	
12.5–13.9	S-4	9 10 50/0.4	1.4'	100	-	gc - a-2-6	M		Started coring at 1256 hrs.
13.9–16.0	R-1		2.0'	95 - 0				13.9 542.1 Shaley SANDSTONE, gray, soft to average hard, slightly to freshly weathered, intensely to medium bedded (RD= 0° to 30°), extremely closely to very widely spaced fractures (RD= 0° to 90°)	R-1: 1256-1259 hrs.
16.0–19.0	R-2		3.0'	100 - 27				micaceous stringers throughout 15.5'-15.7' vertical fracture 17.1'-17.2' near vertical fracture 19.0'-19.7' very broken	R-2: 1303-1315 hrs.
19.0–				96 - 46					R-3: 1320-1330 hrs.

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

PROJECT NAME **WVDOH Scour** COUNTY **Cabell**
PROJECT LOCATION **5th St. Ritter Park**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______

BORING NO. **RIT-01-2011**
SHEET **2** OF **2**
DATE: START **6/23/11**
END **6/23/11**
O.G. ELEV. **556.0**

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
	R-3		4.8'	96 - 46				Shaley SANDSTONE, gray, soft to average hard, slightly to freshly weathered, intensely to medium bedded (RD= 0° to 30°), extremely closely to very widely spaced fractures (RD= 0° to 90°) *(continued)* 19.9'-20.1' vertical fracture 21.2'-21.3' vertical fracture	Completed hole at 1430 hrs. 6/23/2011
24.0								24.0 532.0	
								Bottom of borehole at 24.00 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

Figure 8 – Boring Log 2 for Ritter Park (5th Street) Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **RIT-02-2011**
SHEET **1** OF **2**
DATE: START **6/23/11**
END **6/23/11**
O.G. ELEV. **556.0**

PROJECT NAME **WVDOH Scour** COUNTY **Cabell**
PROJECT LOCATION **5th St. Ritter Park**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______
STATE RT. NO. **527** SECT. ______ SEGMENT ______ OFFSET ______
INSPECTOR **Nichole Wendlandt** DRILLERS NAME/COMPANY **Jim Hopkins/L.G. Hetager Drilling, Inc.**
EQUIPMENT USED **CME 45**
DRILLING METHODS **Hollow Stem Auger/SPT/NX Wireline Split Barrel**
CASING: SIZE: ______ ; DEPTH: ______ ; WATER: DEPTH: ______ TIME: ______ DATE: ______
CHECKED BY: **TLD** ; DATE: **8/1/11** DEPTH: **7.0'** TIME: **10:45** DATE: **6/23/11**
Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
0.0	A-1				-			UNSAMPLED	Started hole at 0820 hrs. 6/23/2011 Cloudy/75 degrees ~Bottom of Abutment at 543'
8.0	S-1	3 2 3	0.9'	60	PP 0.50	cl - a-6	M	8.0 548.0 Sandy CLAY With Trace Gravel (cl), brown, moist, medium stiff, homogeneous, (alluvial)	
9.5	S-2	1 50/0.4	0.6'	67	PP 2.00	cl - a-6	M		
10.4	R-1		1.4'	88 - 75				10.4 545.6 Shaley SANDSTONE, gray, soft to average hard, slightly to freshly weathered, v thinly to medium bedded (RD= 0° to 10°), very closely to very widely spaced fractures (RD= 0° to 45°), RQD=83%	Started coring at 0923 hrs. R-1: 0923-0928 hrs.
12.0	R-2		2.0'	100 - 85				micaceous stringers throughout	R-2: 0932-0938 hrs.
14.0	R-3		5.0'	100 - 86					R-3: 0945-1000 hrs.
19.0				100 - 83					R-4: 1010-1020 hrs.

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **RIT-02-2011**
SHEET **2** OF **2**
DATE: START **6/23/11**
END **6/23/11**
O.G. ELEV. **556.0**

PROJECT NAME **WVDOH Scour** COUNTY **Cabell**
PROJECT LOCATION **5th St. Ritter Park**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT		DESCRIPTION	REMARKS
23.0	R-4		4.0'	100 - 83					Shaley SANDSTONE, gray, soft to average hard, slightly to freshly weathered, v thinly to medium bedded (RD= 0° to 10°), very closely to very widely spaced fractures (RD= 0° to 45°), RQD=83% *(continued)* 23.0 533.0	Completed hole at 1100 hrs. 6/23/2011
									Bottom of borehole at 23.00 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

Lab Results

Modified Slake Durability and GSN*

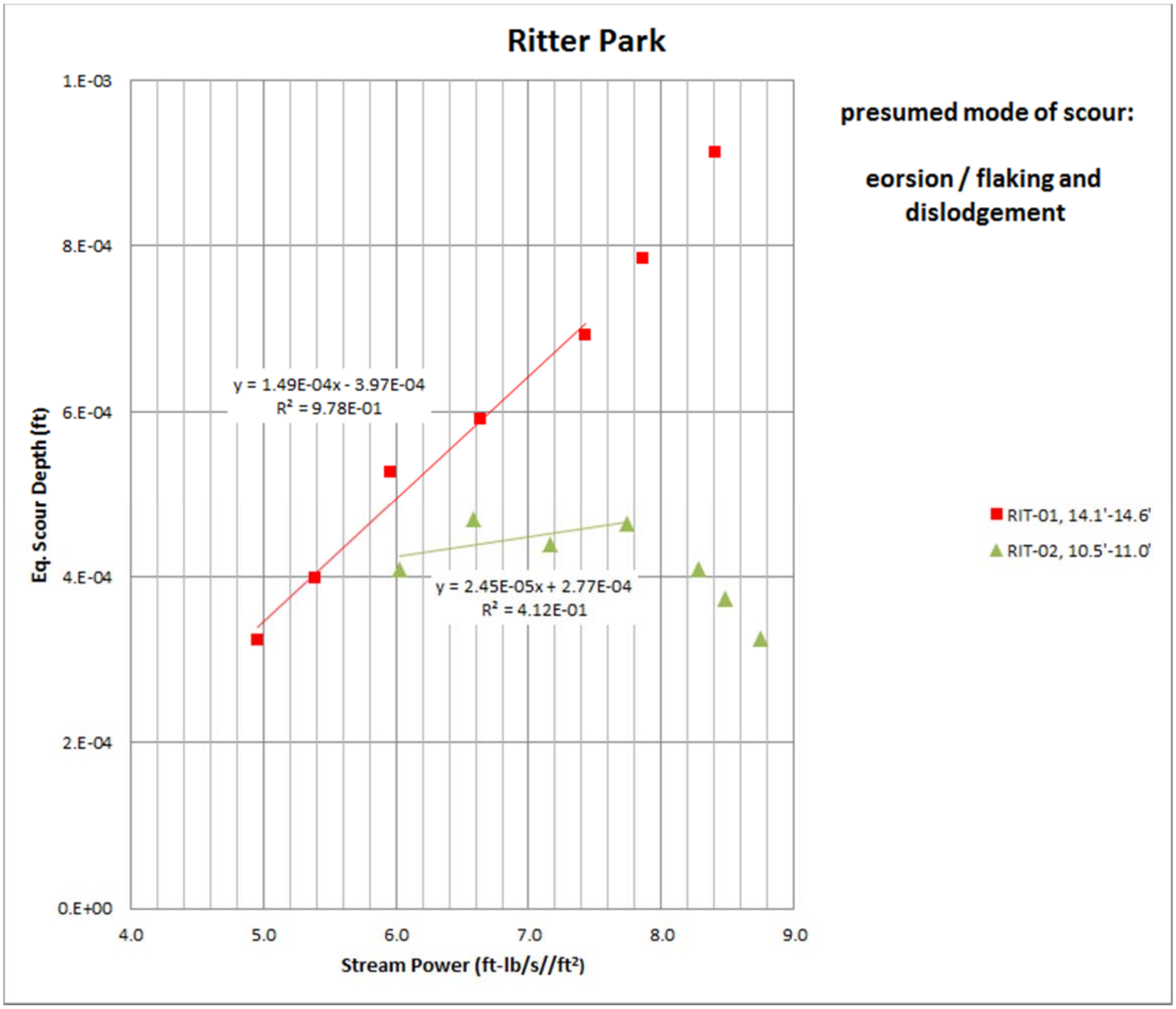

Figure 9 -- Data from modified slake durability tests and calculated values of GSN* and R^{*2} for Ritter Park (5^{th} Street) Bridge samples.

3 – Little Sandy Creek

Field Results

Geology.

At the Little Sandy Bridge the bridge abutments are set in sandstone belonging to the Pennsylvanian- and Permian-age Dunkard Group, a series of non-marine sedimentary strata. These rocks cover approximately 2,200 square miles over 14 counties in the northwestern part of West Virginia. The Dunkard is typically over 450 feet in thickness.

Based on rock core extracted from two boreholes drilled in June 2011, subsurface bedrock at the Little Sandy bridge consists of gray, fine- to medium-grained sandstone with micaceous, stringers in SAN-001-2011. In core from SAN-002-2011 approximately 2 feet of blue-gray, clayey shale overlies gray, fine- to medium-grained sandstone within the TZ. The clayey shale is described as very soft and heavily fractured with numerous laminations. The underlying sandstone is described as hard with very thin bedding, and one vertical fracture. Boring logs and field sketches are shown below.

Bedrock upstream of the bridge consists of silty, fine-grained sandstone with abundant micaceous and carbonaceous stringers. This rock is cut by single joint set oriented approximately 45° relative to the direction of the stream channel. Individual joints are widely-spaced (~3 meters) and filled with mica flakes.

Rock Quality Designation (RQD).

RQD for sandstone from SAN-001-2011 was 75%. RQD for clayey shale and sandstone from SAN-002-2011 was 17% and 99%, respectively.

Scour Mode and Magnitude

Areas of scour at the Little Sandy bridge are shown on the field sketches below.

Figure 10 – Boring Log 1 for Little Sandy Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **SAN-01-2011**
SHEET **1** OF **2**
DATE: START **6/22/11**
END **6/22/11**
O.G. ELEV. **592.0**

PROJECT NAME **WVDOH Scour** COUNTY **Jackson**
PROJECT LOCATION **Little Sandy Creek**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______
STATE RT. NO. **10/3** SECT. ______ SEGMENT ______ OFFSET ______
INSPECTOR **Nichole Wendlandt** DRILLERS NAME/COMPANY **Jim Hopkins/L.G. Hetager Drilling, Inc.**
EQUIPMENT USED **CME 45**
DRILLING METHODS **Hollow Stem Auger/SPT/NX Wireline Split Barrel**
CASING: SIZE: ______ ; DEPTH: ______ ; WATER: DEPTH: ______ TIME: ______ DATE: ______
CHECKED BY: **TLD** ; DATE: **8/1/11** DEPTH: **4.2'** TIME: **12:00** DATE: **6/22/11**

Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
0.0 – 4.0	A-1				-			UNSAMPLED	Started hole at 0825 hrs. 6/22/2011 Sunny/72 degrees ~Bottom of Abutment at 583'
4.0 – 5.5	S-1	33 30 9	1.5'	100	-	sp - a-3	D	4.0 — 588.0 Coarse SAND With Trace Gravel (sp), light brown, dry to moist, medium dense to dense, homogeneous, (alluvial)	Stream pH=8.3
5.5 – 7.0	S-2	9 10 10	1.3'	87	-	sp - a-3	M		
7.0 – 8.5	S-3	6 5 5	1.1'	73	-	sp - a-3	M	7.0 — 585.0 Coarse SAND With Trace Gravel (sp), light brown, moist, loose to medium dense, homogeneous, (alluvial)	
8.5 – 10.0	S-4	10 6 11	1.3'	87	-	sp - a-3	M	9.5 — 582.5 Sandy CLAY With Trace Gravel (cl), reddish-brown, moist, medium stiff to stiff, homogeneous, (alluvial)	
10.0 – 11.5	S-5	10 3 3	1.5'	100	-	cl - a-6	M	10.5 — 581.5 Lean CLAY (cl), dark brown, moist, medium stiff to stiff, homogeneous, (alluvial)	
11.5 – 13.0	S-6	4 4 6	1.5'	100	-	cl - a-6	M		
13.0 – 14.5	S-7	3 3 5	1.5'	100	-	cl - a-6	M		
14.5 – 16.0	S-8	8 10 12	1.5'	100	-	cl - a-6	M	15.0 — 577.0 Lean CLAY With Trace Gravel (cl), dark gray, moist, very stiff, homogeneous, (residual)	
16.0 – 16.8	S-9	4 50/0.3	0.8'	100	-	gp a-1-a	M	16.3 — 575.7 Sandy GRAVEL (gp), dark gray, moist, very dense, homogeneous, (residual)	Started coring at 1030 hrs.
16.8 – 19.0	R-1		2.2'	100 - 0				16.8 — 575.2 Fine to medium grained SANDSTONE, gray, average hard, slightly to freshly weathered, v inten to v thinly bedded (RD= 0° to 30°), extremely closely to very widely spaced fractures (RD= 0° to 60°), RQD=75%	R-1: 1030-1035 hrs.
19.0 –				100 - 84				micaceous stringers throughout	R-2: 1038-1042 hrs.

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **SAN-01-2011**
SHEET **2** OF **2**
DATE: START **6/22/11**
END **6/22/11**
O.G. ELEV. **592.0**

PROJECT NAME **WVDOH Scour** COUNTY **Jackson**
PROJECT LOCATION **Little Sandy Creek**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT		DESCRIPTION	REMARKS
21.5	R-2		2.5'	100 - 84				✓	Fine to medium grained SANDSTONE, gray, average hard, slightly to freshly weathered, v inten to v thinly bedded (RD= 0° to 30°), extremely closely to very widely spaced fractures (RD= 0° to 60°), RQD=75% *(continued)*	
24.0	R-3		2.5'	100 - 100				✓		R-3: 1046-1051 hrs.
27.0	R-4		3.0'	100 - 100				✓	27.0 565.0	R-4: 1100-1110 hrs. Completed hole at 1200 hrs. 6/22/2011
									Bottom of borehole at 27.00 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

Figure 11 – Boring Log 2 for Little Sandy Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **SAN-02-2011**
SHEET **1** OF **2**
DATE: START **6/22/11**
END **6/22/11**
O.G. ELEV. **591.0**

PROJECT NAME **WVDOH Scour** COUNTY **Jackson**
PROJECT LOCATION **Little Sandy Creek**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______
STATE RT. NO. **10/3** SECT. ______ SEGMENT ______ OFFSET ______
INSPECTOR **Nichole Wendlandt** DRILLERS NAME/COMPANY **Jim Hopkins/L.G. Hetager Drilling, Inc.**
EQUIPMENT USED **CME 45**
DRILLING METHODS **Hollow Stem Auger/SPT/NX Wireline Split Barrel**
CASING: SIZE: ______; DEPTH: ______; WATER: DEPTH: ______ TIME: ______ DATE: ______
CHECKED BY: **TLD**; DATE: **8/1/11** DEPTH: **3.0'** TIME: **14:15** DATE: **6/22/11**
Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT		DESCRIPTION	REMARKS
0.0	A-1				-				VOID	Started hole at 1220 hrs. 6/22/2011 Cloudy/78 degrees ~Bottom of Abutment at 581'
5.0	S-1	4 9 3	0.6'	40	PP 1.50	cl - a-6	M		5.0 — 586.0 Sandy CLAY (cl), brown, moist, medium stiff to stiff, homogeneous, (fill)	
6.5	S-2	4 4 4	0.5'	33	PP 1.00	cl - a-6	M			
8.0	S-3	2 2 2	1.4'	93	-	sc - a-2-6	M		8.2 — 582.8 Fine Clayey SAND (sc), brown, moist to wet, very loose to loose, homogeneous, (alluvial)	
9.5	S-4	1 2 4	1.2'	80	-	sc - a-2-6	M			
11.0	S-5	10 17 50/0.4	1.3'	93	-	gc - a-2-6	M		11.3 — 579.7 Clayey GRAVEL (gc), blue-gray, moist, very dense, homogeneous, (residual shale) 12.4 — 578.6	
12.4	R-1		1.3'	81 - 25					Clayey SHALE, blue-gray, very soft, freshly weathered, intensely laminated (RD= 0° to 10°), extremely closely to closely spaced fractures (RD= 0° to 30°), RQD=19%	Started coring at 1318 hrs. R-1: 1318-1322 hrs.
14.0	R-2		5.0'	100 - 88					14.5 — 576.5 Fine to medium grained SANDSTONE, gray, average hard to hard, freshly weathered, v inten to thinly bedded (RD= 0° to 30°), very close to very widely spaced fractures (RD= 0° to 90°), RQD=99% 18.8'-19.1' near vertical fracture	R-2: 1328-1338 hrs.
19.0				100 - 100						R-3: 1345-1355 hrs.

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **SAN-02-2011**
SHEET **2** OF **2**
DATE: START **6/22/11**
END **6/22/11**
O.G. ELEV. **591.0**

PROJECT NAME **WVDOH Scour** COUNTY **Jackson**
PROJECT LOCATION **Little Sandy Creek**
STATION OFFSET FROM CENTERLINE
NORTHING EASTING

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY (%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
23.0	R-3		4.0'	100 - 100				Fine to medium grained SANDSTONE, gray, average hard to hard, freshly weathered, v inten to thinly bedded (RD= 0° to 30°), very close to very widely spaced fractures (RD= 0° to 90°), RQD=99% *(continued)* 23.0 568.0	Completed hole at 1430 hrs. 6/22/2011
								Bottom of borehole at 23.00 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

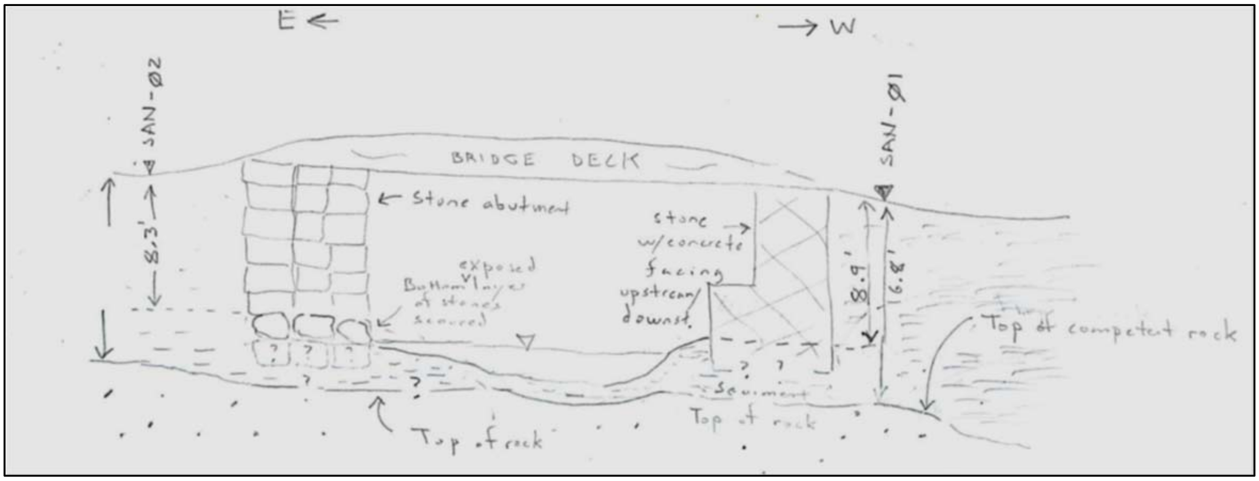

Figure 12 -- Field sketch of Little Sandy Bridge, looking downstream.

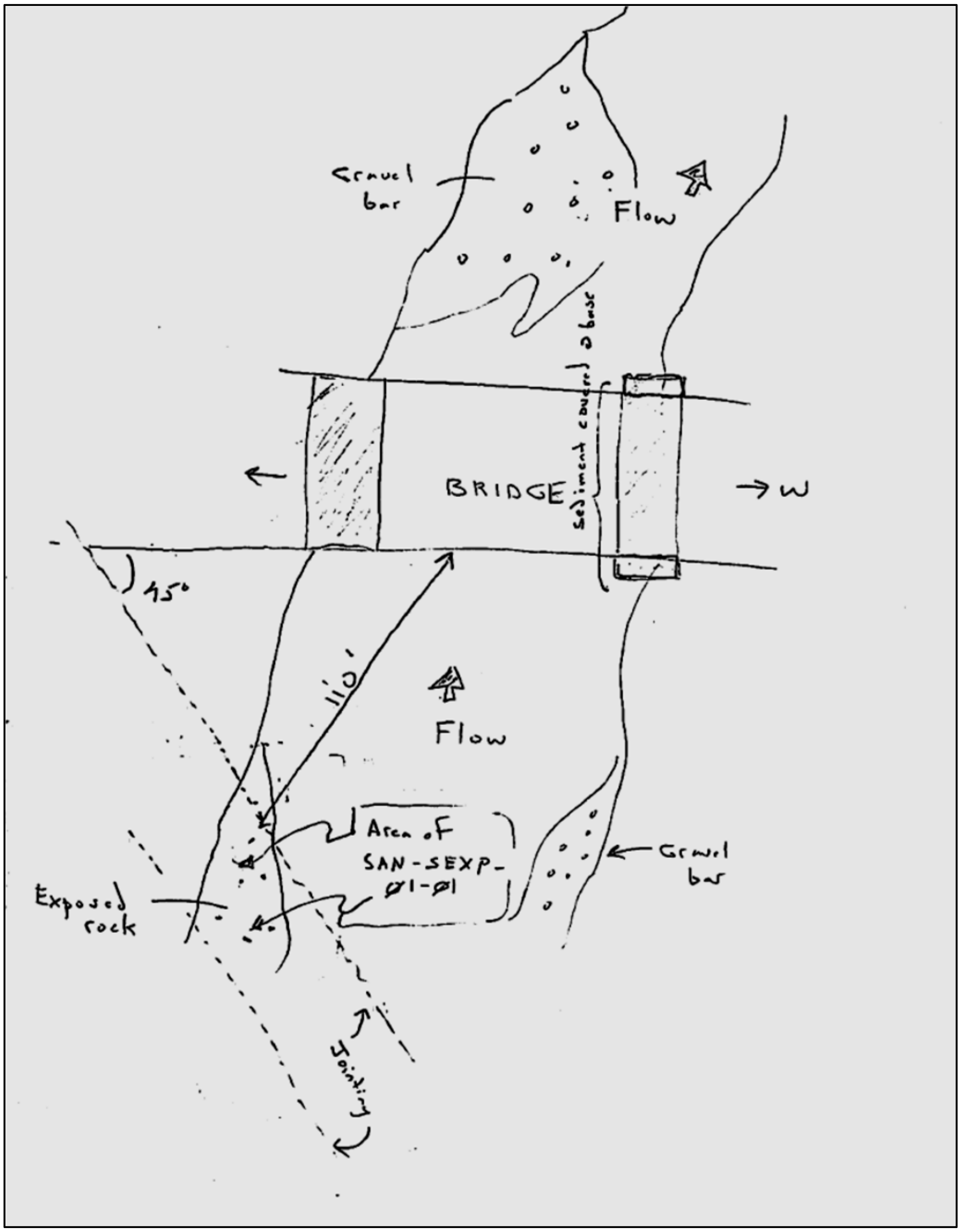

Figure 13 -- Field sketch of Little Sandy Bridge, overhead view.

Lab Results

Modified Slake Durability and GSN*

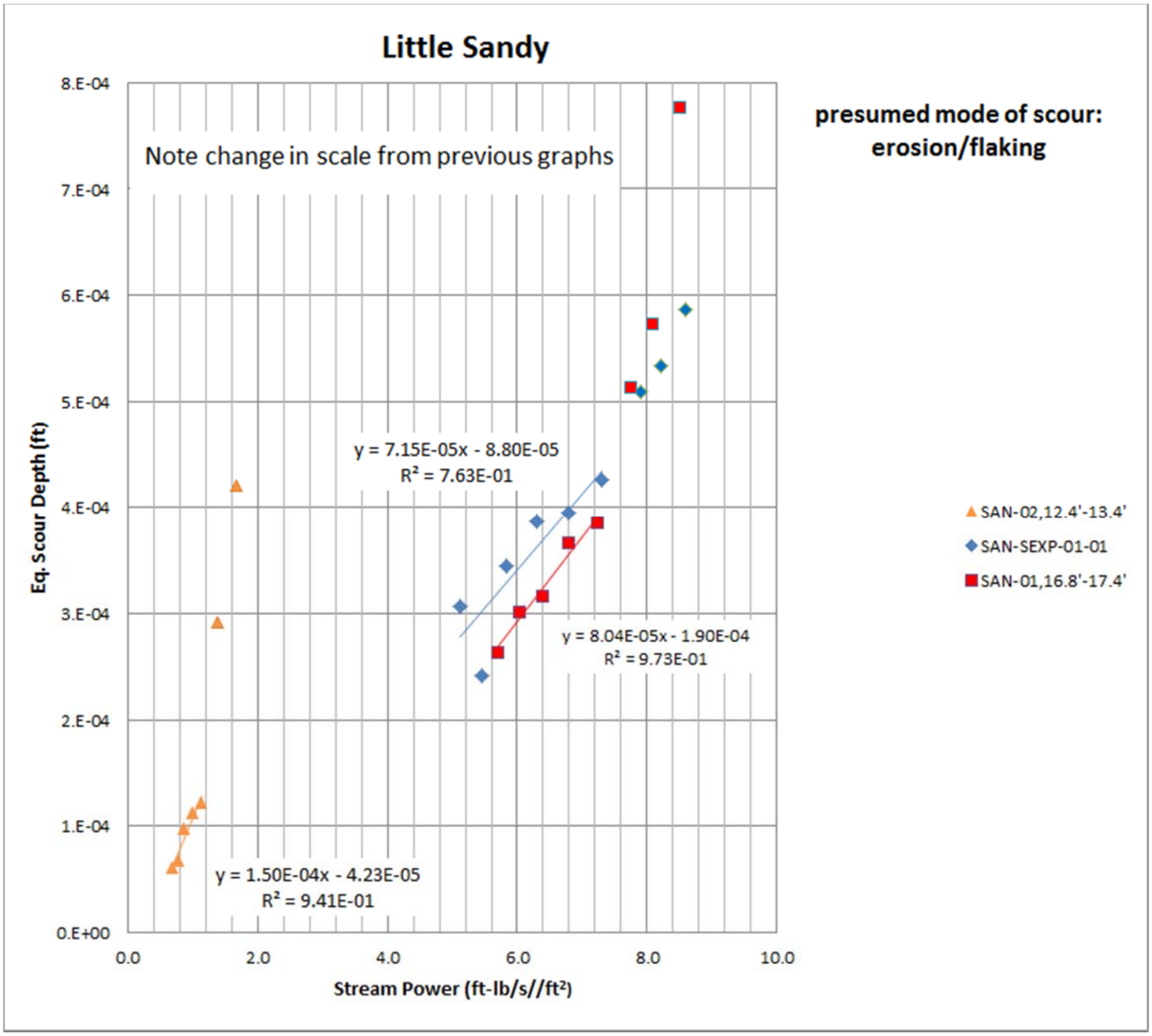

Figure 14 -- Data from modified slake durability tests and calculated values of GSN* and R^{*2} for Little Sandy samples.

4 – Grassy Run

Field Results

Geology.

At the Grassy Run bridge site the bridge abutments are set in sandstone belonging to the Pennsylvanian- age Allegheny Formation. The Allegheny Formation covers in excess of 1,400 square miles over a large strip of West Virginia extending from Wayne County in the southwestern portion of the state to Preston and Mineral Counties in the northeastern portion. The Allegheny is thickest in the western parts of its distribution, up to 325 feet, and thinner to the northeast, where it is typically in the range of 150-200 feet.

Based on core recovered from the TZ in the two boreholes drilled in June 2011, bedrock geology at the Grassy Run bridge consists of gray to tan, medium- to coarsely-grained sandstone of average to above-average hardness with numerous vertical fractures. The sandstone is thinly bedded with micaceous stringers throughout.

Bedrock exposed underneath and upstream of the bridge consists of durable to non-durable, thin- to medium-bedded (2-18 cm) sandstone. Regular jointing (0.8 to 1.2 m spacing) is evident immediately upstream of the bridge, with all joints oriented 30° to 45° relative to the stream channel.

Rock Quality Designation (RQD).

RQD of core from GRA-001-2011 and GRA-002-2011 is 48% and 63%, respectively.

Scour Mode and Magnitude

Areas of scour at the Grassy Run bridge are shown on the field sketches below.

Figure 15 – Boring Log 1 for Grassy Run Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **GRA-01-2011**
SHEET **1** OF **2**
DATE: START **6/29/11**
END **6/29/11**
O.G. ELEV. **1032.0**

PROJECT NAME **WVDOH Scour** COUNTY **Marion**
PROJECT LOCATION **Grassy Run Bridge**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______
STATE RT. NO. **80** SECT. ______ SEGMENT ______ OFFSET ______
INSPECTOR **Nichole Wendlandt** DRILLERS NAME/COMPANY **Jim Hopkins/L.G. Hetager Drilling, Inc.**
EQUIPMENT USED **CME 45**
DRILLING METHODS **Casing Advancer/SPT/NX Wireline Split Barrel**
CASING: SIZE: **3.25"** ; DEPTH: **14.2'** ; WATER: DEPTH: ______ TIME: ______ DATE: ______
CHECKED BY: **TLD** ; DATE: **8/1/11** DEPTH: **14.8'** TIME: **11:50** DATE: **6/29/11**

Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
0.0–5.0	A-1				-			UNSAMPLED	Started hole at 0939 hrs. 6/29/2011 Sunny/70 degrees ~Bottom of Abutment at 1022' Used casing due to nature of surrounding materials
5.0–6.5	S-1	7, 9, 9	0.8'	53	-	sc - a-2-6	M	5.0 … 1027.0 Clayey SAND (sc), brown, moist, loose to very dense, homogeneous, (alluvial)	Sand trap used for all soil sampling
6.5–8.0	S-2	9, 5, 5	1.2'	80	-	sc - a-2-6	M		
8.0–9.5	S-3	4, 5, 5	0.7'	47	-	sc - a-2-6	M		
9.5–11.0	S-4	8, 9, 14	0.3'	20	-	sc - a-2-6	M		
11.0–12.5	S-5	16, 40, 30	1.5'	100	-	sc - a-2-6	M	piece of sandstone in bottom of S-4 spoon	
12.5–14.0	S-6	11, 42, 32	0.9'	60	-	sc - a-2-6	M		
14.0–14.2	S-7	50/0.2	0.2'	100	-	sc - a-2-6	M	14.2 … 1017.8	Started coring at 1050 hrs.
14.2–16.2	R-1		2.0'	100 - 20				Medium to coarse grained SANDSTONE, gray tan, average hard to hard, slightly to freshly weathered, intensely to thinly bedded (RD= 0° to 30°), extremely closely to very widely spaced fractures (RD= 0° to 90°), RQD=48%	R-1: 1050-1100 hrs.
16.2–19.2	R-2		2.5'	83 - 37				micaceous stringers throughout 15.4'-15.7' vertical fracture 15.7'-15.9' pair of near vertical fractures 16.4'-16.6' vertical fracture 19.8'-19.9' near vertical fracture	R-2: 1108-1116 hrs.
19.2–				98 -					

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

PROJECT NAME **WVDOH Scour** COUNTY **Marion**
PROJECT LOCATION **Grassy Run Bridge**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______

BORING NO. **GRA-01-2011**
SHEET **2** OF **2**
DATE: START **6/29/11**
END **6/29/11**
O.G. ELEV. **1032.0**

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
	R-3		4.9'	66 98 - 66				Medium to coarse grained SANDSTONE, gray tan, average hard to hard, slightly to freshly weathered, intensely to thinly bedded (RD= 0° to 30°), extremely closely to very widely spaced fractures (RD= 0° to 90°), RQD=48% *(continued)* 22.9'-23.0' very broken	R-3: 1118-1125 hrs. Completed hole at 1210 hrs. 6/29/2011
24.2								24.2 1007.8	
								Bottom of borehole at 24.20 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

Figure 16 – Boring Log 2 for Grassy Run Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. GRA-02-2011
SHEET 1 OF 2
DATE: START 6/29/11
END 6/29/11
O.G. ELEV. 1035.0

PROJECT NAME WVDOH Scour COUNTY Marion
PROJECT LOCATION Grassy Run Bridge
STATION ____ OFFSET FROM CENTERLINE ____
NORTHING ____ EASTING ____
STATE RT. NO. 80 SECT. ____ SEGMENT ____ OFFSET ____
INSPECTOR Nichole Wendlandt DRILLERS NAME/COMPANY Jim Hopkins/L.G. Hetager Drilling, Inc.
EQUIPMENT USED CME 45
DRILLING METHODS Casing Advancer/SPT/NX Wireline Split Barrel
CASING: SIZE: 3.25" ; DEPTH: 10.6' ; WATER: DEPTH: ____ TIME: ____ DATE: ____
CHECKED BY: TLD ; DATE: 8/1/11 DEPTH: 12.0' TIME: 8:50 DATE: 6/29/11
Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT		DESCRIPTION	REMARKS
0.0 – 7.0	A-1				-				UNSAMPLED	Started hole at 0705 hrs. 6/29/2011 Cloudy/68 degrees ~Bottom of Abutment at 1023' Used casing due to nature of surrounding materials Sand trap used for all soil sampling
7.0 – 8.5	S-1	9 7 6	0.2'	13	-	sc - a-2-6	M		7.0 1028.0 Clayey SAND (sc), brown, moist, medium dense, homogeneous, (alluvial)	Stream pH=8.9
8.5 – 10.0	S-2	5 4 8	0.5'	33	-	sc - a-2-6	M			
10.0 – 10.6	S-3	30 50/0.1	0.6'	100	-	sc - a-2-6	M		pieces of weathered sandstone in S-3	Started coring at 0752 hrs.
10.6 – 12.0	R-1		1.3'	93 - 50					10.6 1024.4 Medium to coarse grained SANDSTONE, gray tan, average hard to hard, slightly to freshly weathered, intensely to thinly bedded (RD= 0° to 30°), extremely closely to very widely spaced fractures (RD= 0° to 90°), RQD=63% micaceous stringers throughout 11.4'-11.6' near vertical fracture 11.4'-11.5' very broken 13.0'-13.1' near vertical fracture	R-1: 0752-0759 hrs.
12.0 – 14.0			1.7'	85 - 50						R-2: 0803-0810 hrs.
14.0 – 19.0			4.7'	94 - 70					19.4'-19.6' stained near vertical fracture	R-3: 0812-0824 hrs.
19.0 –				100 - 67						R-4: 0826-0832 hrs.

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO.	GRA-02-2011
SHEET 2 OF 2	
DATE: START	6/29/11
END	6/29/11
O.G. ELEV.	1035.0

PROJECT NAME **WVDOH Scour** COUNTY **Marion**
PROJECT LOCATION **Grassy Run Bridge**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT		DESCRIPTION	REMARKS
			3.0'	100 - 67					20.0'-20.1' vertical fracture 20.7'-20.8' near vertical fracture	Completed hole at 0910 hrs. 6/29/2011
22.0									22.0 1013.0	
									Bottom of borehole at 22.00 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

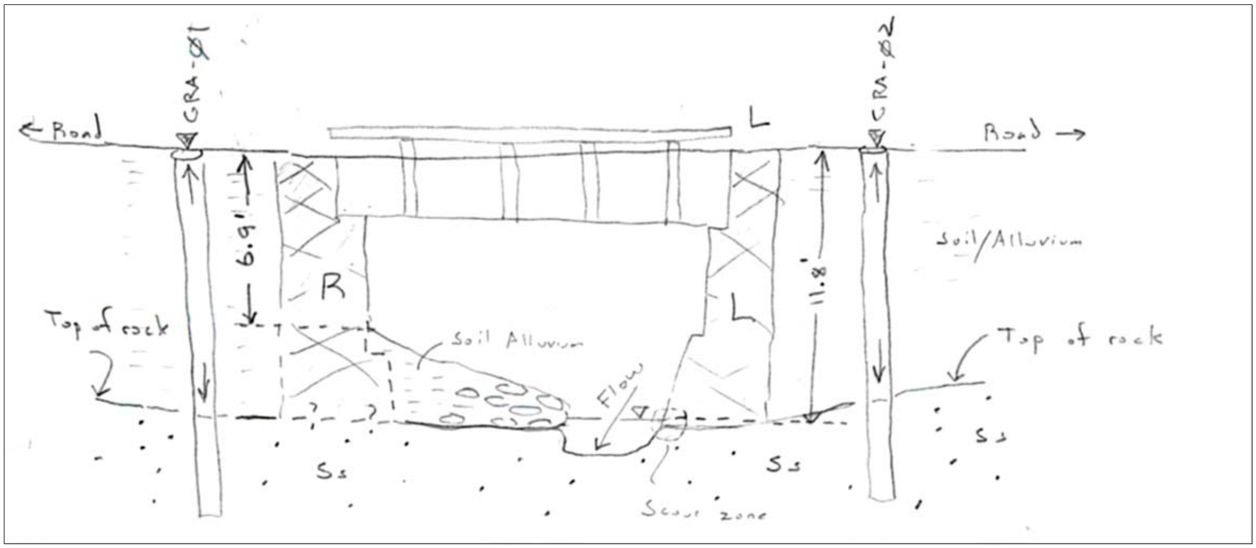

Figure 17 -- Field sketch of Little Grassy Run Bridge, looking upstream.

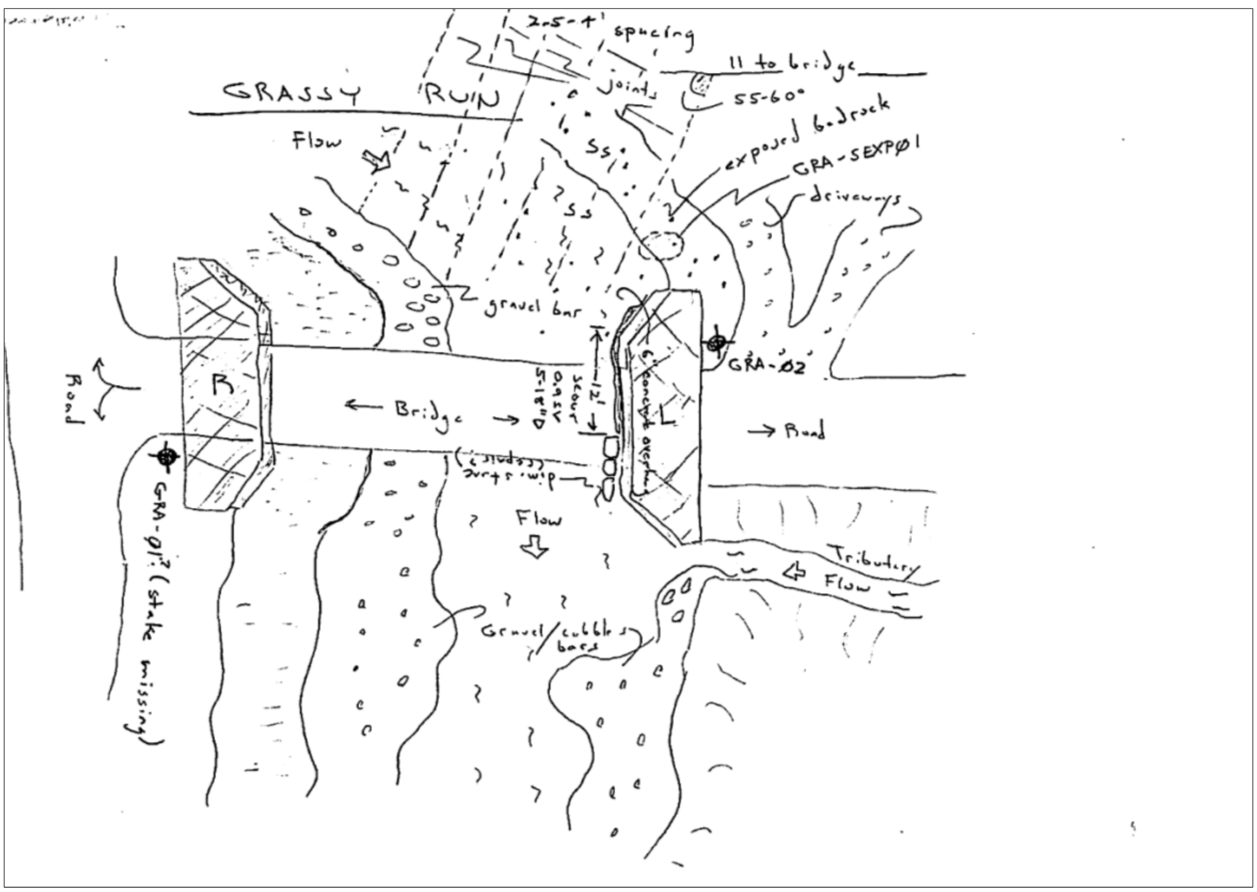

Figure 18 -- Field sketch of Little Grassy Run Bridge, overhead view.

Lab Results

Modified Slake Durability and GSN*

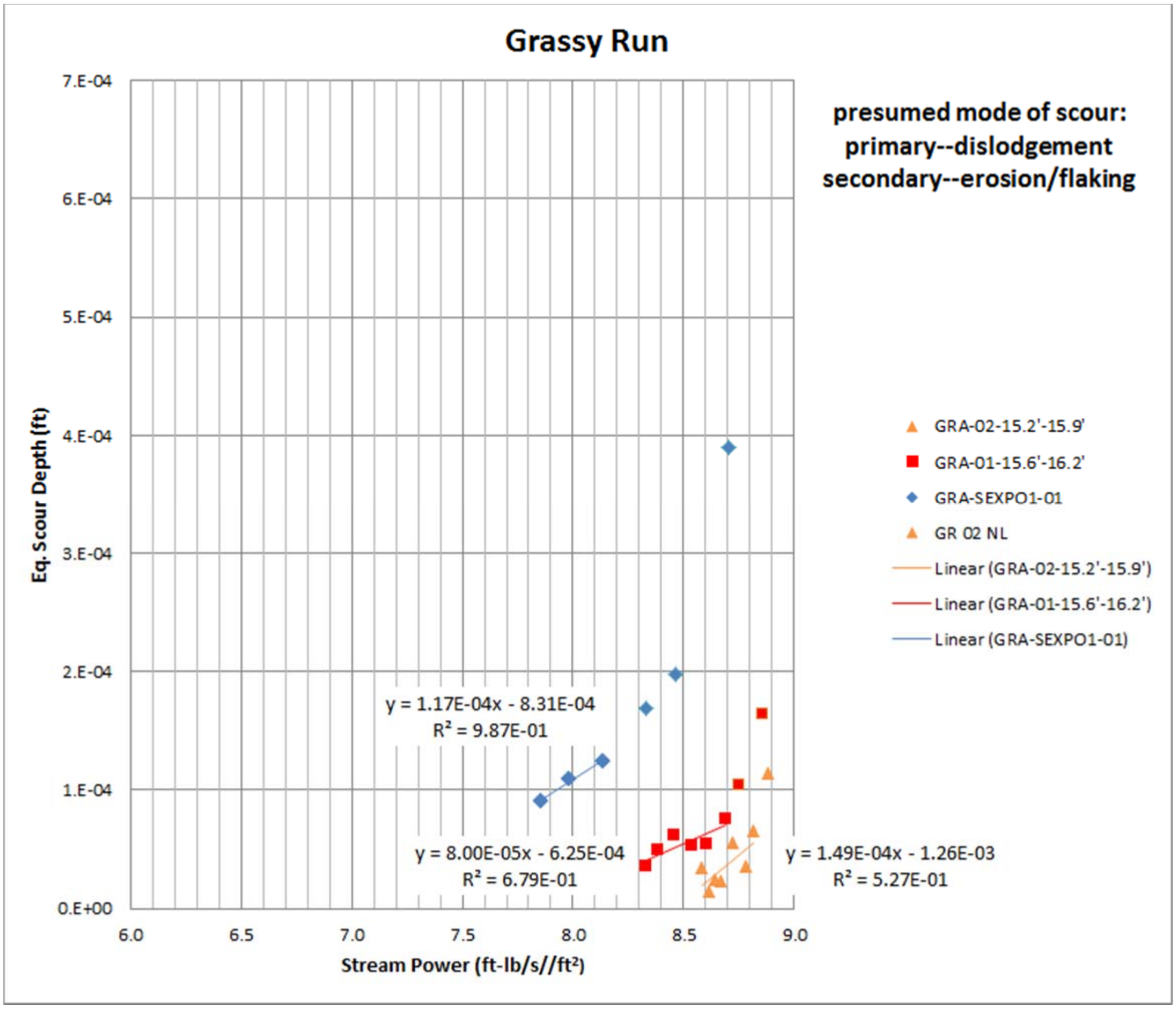

Figure 19 -- Data from modified slake durability tests and calculated values of GSN* and R^{*2} for Grassy Run samples.

5 – Caldwell Run

Field Results

Geology.

At the Caldwell Run bridge site the bridge abutments are set in shale and coal belonging to the Pennsylvanian- age Uniontown or Pittsburgh Formation, which together comprise the Monongahela Group. These formations represent cyclical, non-marine deposits of sandstone, red and gray shale, siltstone, limestone, and coal. Striking in a southwest-northwest direction, the Uniontown and Pittsburgh Formations cover in excess of 1,800 square miles over a large part of the northwestern and north-central parts of the state. These formations are found in Wayne and Cabell Counties in the southwestern part of the state and extend to the northeast as far as Monongalia County and as far north as Brooke County in the northern panhandle. The combined thickness of the Uniontown and Pittsburgh Formations ranges from 170 feet in the northeastern parts of their distribution to 300 feet in the western portions.

Based on core extracted from the two boreholes drilled in June 2011, bedrock geology in the top half of the TZ at the Caldwell Run bridge consists of gray, clayey shale. The shale is underlain by fractured coal and then gray, thin-bedded limestone. Both the shale and coal are described as soft and heavily fractured. The limestone is less severely fractured than the overlying material and of average hardness.

Surface exposure of bedrock at the base of both abutments is described as a non-durable, thinly bedded (0.3-1.3 cm), silty shale. Two joint sets, oriented at approximately 45° and 30° relative to the stream channel, intersect the surface bedrock. Individual joints are separated by 4 to 7 meters.

Rock Quality Designation (RQD).

RQD for the shale, coal, and limestone beds in CAL-01-2011 is 8%, 0%, and 72%, respectively. RQD for the shale, coal, and limestone beds in CAL-02-2011 is 34%, 0%, and 72%, respectively.

Scour Mode and Magnitude

Areas of scour at the Caldwell Run bridge are shown on the field sketches below.

Figure 20 – Boring Log 1 for Caldwell Run Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. CAL-01-2011
SHEET 1 OF 2
DATE: START 6/20/11
END 6/20/11
O.G. ELEV. 759.0

PROJECT NAME WVDOH Scour COUNTY Ohio
PROJECT LOCATION Caldwell Run Bridge
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______
STATE RT. NO. 19 SECT. ______ SEGMENT ______ OFFSET ______
INSPECTOR Nichole Wendlandt DRILLERS NAME/COMPANY Jim Hopkins/L.G. Hetager Drilling, Inc.
EQUIPMENT USED CME 45
DRILLING METHODS Hollow Stem Auger/SPT/NX Wireline Split Barrel
CASING: SIZE: ______ ; DEPTH: ______ ; WATER: DEPTH: ______ TIME: ______ DATE: ______
CHECKED BY: TLD ; DATE: 8/1/11 DEPTH: 2.0' TIME: 14:30 DATE: 6/20/11
Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
0.0 – 0.7	A-1				-			0.4 ASPHALT 758.6 0.7 BRICK 758.3 UNSAMPLED	Started hole at 1100 hrs. 6/20/2011
0.7 – 5.0	A-2				-				Cloudy/72 degrees ~Bottom of Abutment at 750'
5.0 – 6.5	S-1	5 6 7	0.5'	33	-	sm - a-1-b	D	5.0 754.0 Silty SAND (sm), dark brown, dry, medium dense, homogeneous, (fill)	
6.5 – 8.0	S-2	19 17 40	0.9'	60	-	gm - a-1-b	D	7.5 751.5 Silty GRAVEL (gm), brown gray, dry, very dense, homogeneous, (residual shale)	
8.0 – 8.7	S-3	25 50/0.2	0.7'	100	-	gm - a-1-b	D	8.7 750.3	Started coring rock at 1215 hrs.
8.7 – 9.0	R-1		0.3'	100 - 0				Clayey SHALE, gray, soft, freshly weathered, intensely laminated (RD= 0° to 10°), extremely closely to very closely spaced fractures (RD= 0° to 10°), RQD=8*	R-1: 1215-1220 hrs.
9.0 – 11.0	R-2		2.0'	100 - 20					R-2: 1228-1234 hrs.
11.0 – 14.0	R-3		2.7'	90 - 0				13.7'-14.0' highly weathered	R-3: 1245-1255 hrs.
14.0 – 19.0	R-4		5.0'	100 - 60				14.0 745.0 COAL, black, soft, freshly weathered, intensely bedded (RD= 0° to 10°), extremely closely to closely spaced fractures (RD= 0° to 90°), RQD=0% 15.7 743.3 LIMESTONE, gray, average hard, freshly weathered, v thinly to thinly bedded (RD= 0° to 10°), closely to very widely spaced fractures (RD= 0° to 90°), RQD=72%	R-4: 1310-1322 hrs.
19.0 –				100 - 40				17.5'-17.6' shale	R-5: 1335-1345 hrs.

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

PROJECT NAME WVDOH Scour   COUNTY Ohio
PROJECT LOCATION Caldwell Run Bridge
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______

BORING NO. CAL-01-2011
SHEET 2 OF 2
DATE: START 6/20/11
END 6/20/11
O.G. ELEV. 759.0

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
21.0	R-5		2.0'					20.0'-20.7' partially healed vertical fracture 21.0 738.0	Completed hole at 1430 hrs. 6/20/2011
								Bottom of borehole at 21.00 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

Figure 21 – Boring Log 2 for Caldwell Run Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **CAL-02-2011**
SHEET **1** OF **2**
DATE: START **6/20/11**
END **6/20/11**
O.G. ELEV. **760.0**

PROJECT NAME **WVDOH Scour** COUNTY **Ohio**
PROJECT LOCATION **Caldwell Run Bridge**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______
STATE RT. NO. **19** SECT. ______ SEGMENT ______ OFFSET ______
INSPECTOR **Nichole Wendlandt** DRILLERS NAME/COMPANY **Jim Hopkins/L.G. Hetager Drilling, Inc.**
EQUIPMENT USED **CME 45**
DRILLING METHODS **Hollow Stem Auger/SPT/NX Wireline Split Barrel**
CASING: SIZE: ______ ; DEPTH: ______ ; WATER: DEPTH: ______ TIME: ______ DATE: ______
CHECKED BY: **TLD** ; DATE: **8/1/11** DEPTH: **11.2'** TIME: **16:50** DATE: **6/20/11**

Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H₂O CONTENT	DESCRIPTION	REMARKS
0.0	A-1				-			UNSAMPLED	Started hole at 1500 hrs. 6/20/2011 Cloudy/80 degrees ~Bottom of Abutment at 748' Stream pH=8.6
8.0	S-1	23 41 50/0.3	1.3'	100	-	gm - a-1-b	D	8.0 — 752.0 Silty GRAVEL (sm), brown gray, dry, very dense, homogeneous, (residual shale)	
9.3	R-1		1.5'	88 - 0				9.3 — 750.7 Clayey SHALE, gray, soft, slightly to freshly weathered, intensely laminated (RD= 0° to 10°), extremely closely to closely spaced fractures (RD= 0° to 90°), RQD=34%	Started coring at 1545 hrs. R-1: 1545-1553 hrs.
11.0	R-2		3.0'	100 - 47				11.2'-11.4' stained vertical fracture 13.3'-13.6' stained vertical fracture 14.0'-14.4' stained vertical fracture 16.1'-16.4' highly weathered	R-2: 1602-1610 hrs.
14.0	R-3		4.7'	94 - 32				16.4 — 743.6 COAL, black, soft, freshly weathered, intensely bedded (RD= 0° to 10°), extremely closely to closely spaced fractures (RD= 0° to 90°), RQD=0% 18.4 — 741.6	R-3: 1615-1622 hrs.
19.0				100 - 67					R-4: 2625-1630 hrs.

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

PROJECT NAME **WVDOH Scour** COUNTY **Ohio**
PROJECT LOCATION **Caldwell Run Bridge**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______

BORING NO. **CAL-02-2011**
SHEET **2** OF **2**
DATE: START **6/20/11**
END **6/20/11**
O.G. ELEV. **760.0**

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
22.0	R-4		3.0'	100 - 67				LIMESTONE, gray, average hard, freshly weathered, v thinly to thinly bedded (RD= 0° to 10°), closely to medium spaced fractures (RD= 0° to 30°), RQD=72% *(continued)* 22.0 738.0	Completed hole at 1745 hrs. 6/20/2011
								Bottom of borehole at 22.00 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

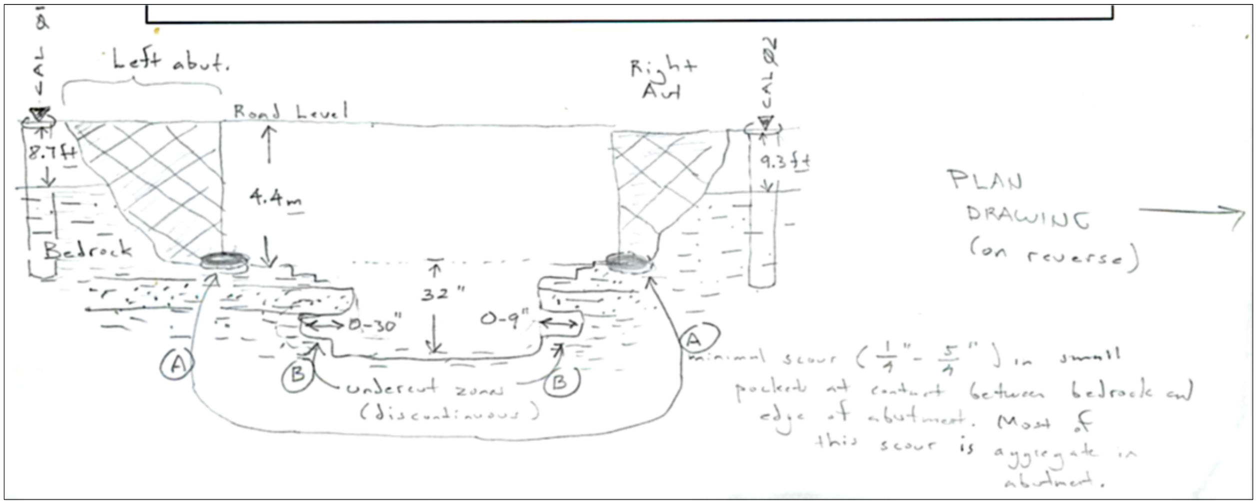

Figure 22 -- Field sketch of Caldwell Run Bridge, looking upstream.

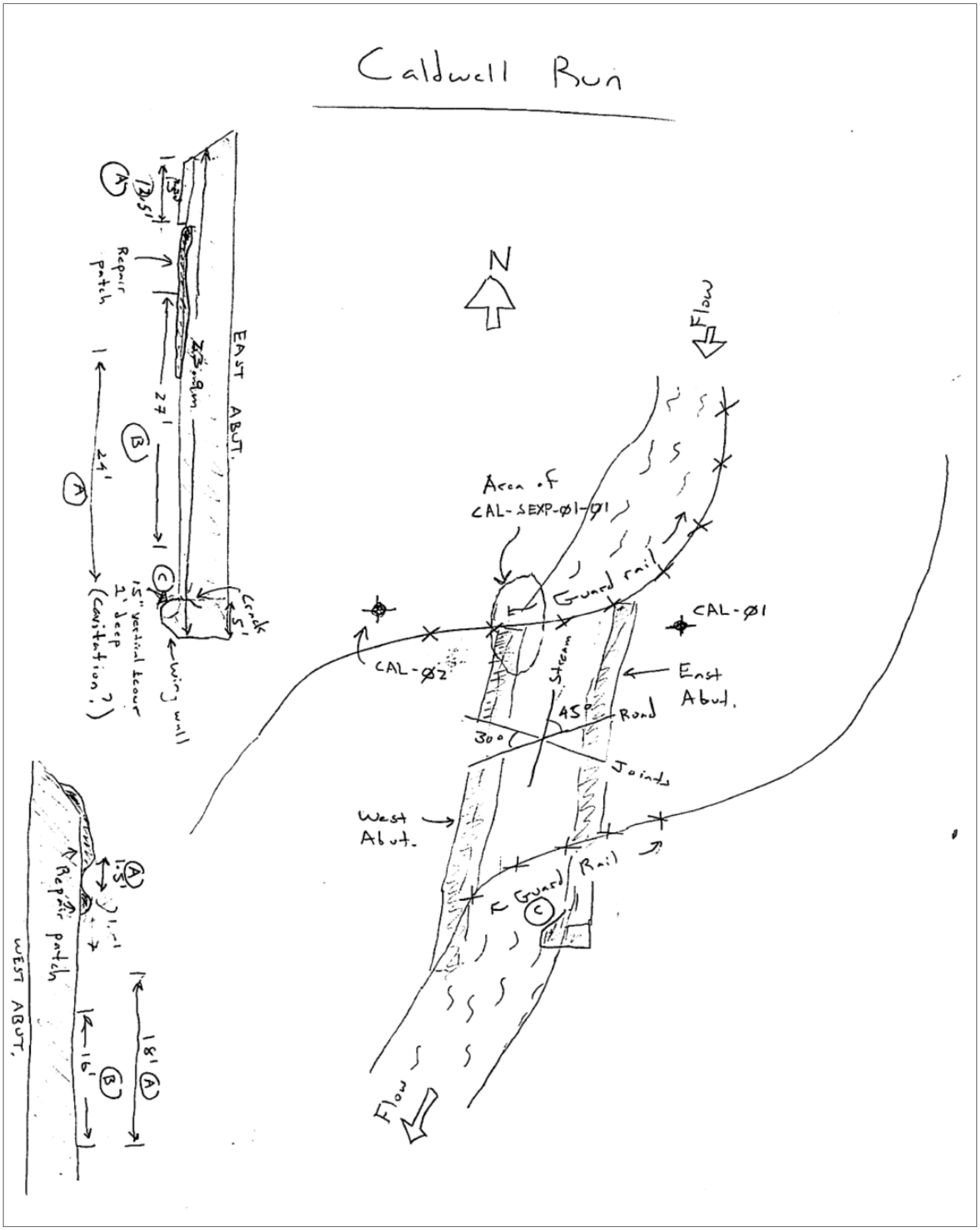

Figure 23 -- Field sketch of Caldwell Run Bridge: overhead view (right), east abutment (top left), and west abutment (bottom left).

Lab Results

Modified Slake Durability and GSN*

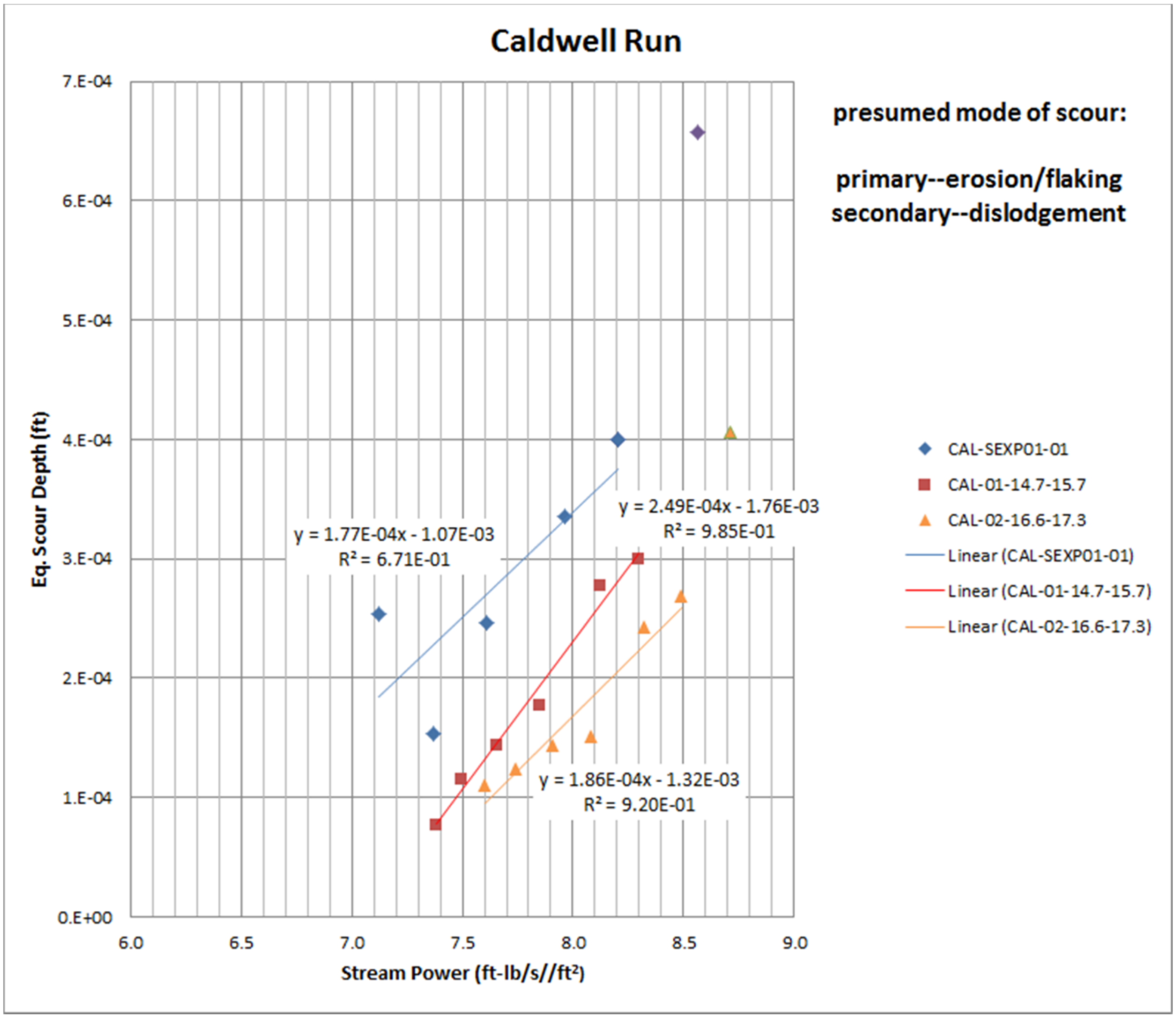

Figure 24 -- Data from modified slake durability tests and calculated values of GSN* and R^{*2} for Caldwell Run samples.

6 – Paden Fork

Field Results

Geology.

At the Paden Fork bridge site the bridge abutments are set in sandstone belonging to the Pennsylvanian- and Permian-age Dunkard Group, a series of non-marine sedimentary strata. The youngest rock formations in West Virginia, these strata cover approximately 2,200 square miles over 14 counties in the northwestern part of the state. The Dunkard is typically over 450 feet in thickness.

Based on the two boreholes drilled in June 2011, bedrock geology at the Paden Fork bridge consists of fine- to medium-grained sandstone of average hardness. A gray to red, clayey shale, approximately 1.5 meters thick, is present below the TZ in PAD-02-2011. Compared to PAD-02-2011, sandstone from the TZ in PAD-02-2011 (adjacent to the left abutment) was notably more fractured, more highly weathered, and contained shale interbeds.

Bedrock exposed both upstream and downstream of the left abutment consists of medium-bedded (1.25-2 m) sandstone with secondary stratification in the form of cross-bedding. This rock has weathered to a mostly non-durable condition, exposing thin lamina of 0.01 to 0.02 meters thickness. Joints, oriented sub-parallel to the stream channel and widely spaced (1.8-3.0 m), intersect the surface bedrock. A less prominent joint set, oriented 10° to 15° relative to the stream channel, is also present.

Rock Quality Designation (RQD).

RQD for the sandstone core recovered from PAD-01-2011 and PAD-02-2011 is 76% and 68%, respectively.

Scour Mode and Magnitude

Areas of scour at the Paden Fork bridge are shown on the field sketches below.

Figure 25 – Boring Log 1 for Paden Fork Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **PAD-01-2011**
SHEET **1** OF **2**
DATE: START **6/21/11**
END **6/21/11**
O.G. ELEV. **809.0**

PROJECT NAME **WVDOH Scour** COUNTY **Wetzel**
PROJECT LOCATION **Paden Fork Bridge**
STATION ____ OFFSET FROM CENTERLINE ____
NORTHING ____ EASTING ____
STATE RT. NO. **26/2** SECT. ____ SEGMENT ____ OFFSET ____
INSPECTOR **Nichole Wendlandt** DRILLERS NAME/COMPANY **Jim Hopkins/L.G. Hetager Drilling, Inc.**
EQUIPMENT USED **CME 45**
DRILLING METHODS **Hollow Stem Auger/SPT/NX Wireline Split Barrel**
CASING: SIZE: ____ ; DEPTH: ____ ; WATER: DEPTH: ____ TIME: ____ DATE: ____
CHECKED BY: **TLD** ; DATE: **8/1/11** DEPTH: **12.3'** TIME: **15:00** DATE: **6/21/11**
Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY (%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT		DESCRIPTION	REMARKS
0.0									UNSAMPLED	Started hole at 1245 hrs. 6/21/2011
										Cloudy/78 degrees
	A-1				-					
										~Bottom of Abutment at 799'
5.0									5.0 — 804.0	
5.6	S-1	18 / 50/0.1	0.6'	100	-	gc / a-2-6	D		5.6 — Sandy GRAVEL (gc), brown, dry, very dense, homogeneous, (residual sandstone) — 803.4	Started coring at 1330 hrs.
7.0	R-1		1.4'	100 / 0					Fine to medium grained SANDSTONE, gray, average hard, moderately to freshly weathered, v inten to v thinly bedded (RD= 0° to 10°), extremely closely to closely spaced fractures (RD= 0° to 90°), RQD=76%	R-1: 1330-1340 hrs.
9.5	R-2		2.5'	100 / 56					5.6'-11.5' interbedded shale 6.8'-6.9' stained vertical fracture 8.7'-8.8' healed vertical fracture 9.2'-9.5' stained vertical fracture 10.8'-11.2' stained vertical fracture 11.5'-23.0' micaceous stringers	
14.5	R-3		5.0'	100 / 76						
19.5	R-4		5.0'	100 / 96						

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **PAD-01-2011**
SHEET **2** OF **2**
DATE: START **6/21/11**
END **6/21/11**
O.G. ELEV. **809.0**

PROJECT NAME **WVDOH Scour** COUNTY **Wetzel**
PROJECT LOCATION **Paden Fork Bridge**
STATION ____ OFFSET FROM CENTERLINE ____
NORTHING ____ EASTING ____

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
22.0	R-5		2.5'	100 - 100				22.0 787.0	Completed hole at 1530 hrs. 5/21/2011
								Bottom of borehole at 22.00 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

Figure 26 – Boring Log 2 for Paden Fork Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **PAD-02-2011**
SHEET **1** OF **2**
DATE: START **6/21/11**
END **6/21/11**
O.G. ELEV. **806.0**

PROJECT NAME **WVDOH Scour** COUNTY **Wetzel**
PROJECT LOCATION **Paden Fork Bridge**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______
STATE RT. NO. **26/2** SECT. ______ SEGMENT ______ OFFSET ______
INSPECTOR **Nichole Wendlandt** DRILLERS NAME/COMPANY **Jim Hopkins/L.G. Hetager Drilling, Inc.**
EQUIPMENT USED **CME 45**
DRILLING METHODS **Hollow Stem Auger/SPT/NX Wireline Split Barrel**
CASING: SIZE: ______ ; DEPTH: ______ ; WATER: DEPTH: ______ TIME: ______ DATE: ______
CHECKED BY: **TLD** ; DATE: **8/1/11** DEPTH: **8.5'** TIME: **11:20** DATE: **6/21/11**
Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
0.0 – 5.0	A-1				-			UNSAMPLED	Started hole at 0900 hrs. 6/21/2011 Sunny/75 degrees ~Bottom of Abutment at 796'
5.0 – 6.5	S-1	2 2 2	0.3'	20	-	sc - a-2-6	D	5.0 — 801.0 Clayey SAND (sc), dark brown, dry, very loose, homogeneous, (fill)	Stream pH=8.3
6.5 – 8.0	S-2	2 4 6	0.3'	20	PP 0.50	cl - a-6	M	6.5 — 799.5 Silty CLAY (cl), brown, moist, medium stiff to stiff, homogeneous, (alluvial)	
8.0 – 9.5	S-3	4 4 4	1.5'	100	-	sc - a-2-6	M	8.5 — 797.5 Clayey SAND With Gravel (sc), brown, moist, loose, homogeneous, (alluvial)	
9.5 – 11.0	S-4	8 9 15	1.4'	93	-	gc - a-2-6	M	10.0 — 796.0 Sandy GRAVEL With Clay (gc), brown, wet, medium dense to very dense, homogeneous, (residual sandstone)	
11.0 – 11.8	S-5	50/0.3	0.7'	87	-	gc - a-2-6	M		Started coring at 1024 hrs.
11.8 – 14.0	R-1		2.2'	100 - 36				11.8 — 794.2 Fine to medium grained SANDSTONE, gray, average hard, slightly to freshly weathered, v inten to v thinly bedded (RD= 0° to 30°), very closely to closely spaced fractures (RD= 0° to 30°), RQD=68%	R-1: 1024-1030 hrs.
14.0 – 16.5	R-2		2.5'	100 - 64				micaceous crossbeds throughout	R-2: 1034-1041 hrs.
16.5 – 19.0	R-3		2.4'	96 - 80				18.3 — 787.7	R-3: 1045-1055 hrs.
19.0 –				75 - 45					R-4: 1105-1115 hrs.

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

PROJECT NAME **WVDOH Scour** COUNTY **Wetzel**
PROJECT LOCATION **Paden Fork Bridge**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______

BORING NO. **PAD-02-2011**
SHEET **2** OF **2**
DATE: START **6/21/11**
END **6/21/11**
O.G. ELEV. **806.0**

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
	R-4		3.0'	75 - 45				Clayey SHALE, red, soft, slightly to moderately weathered, intensely laminated (RD= 0° to 10°), extremely closely to very closely spaced fractures (RD= 0° to 10°), RQD=0% *(continued)* (21.2 / 784.8)	
23.0								Clayey SHALE, gray, soft, freshly weathered, intensely laminated (RD= 0° to 30°), widely to very widely spaced fractures (RD= 0° to 30°), RQD=100% (23.0 / 783.0)	Completed hole at 1130 hrs. 6/21/2011
								Bottom of borehole at 23.00 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

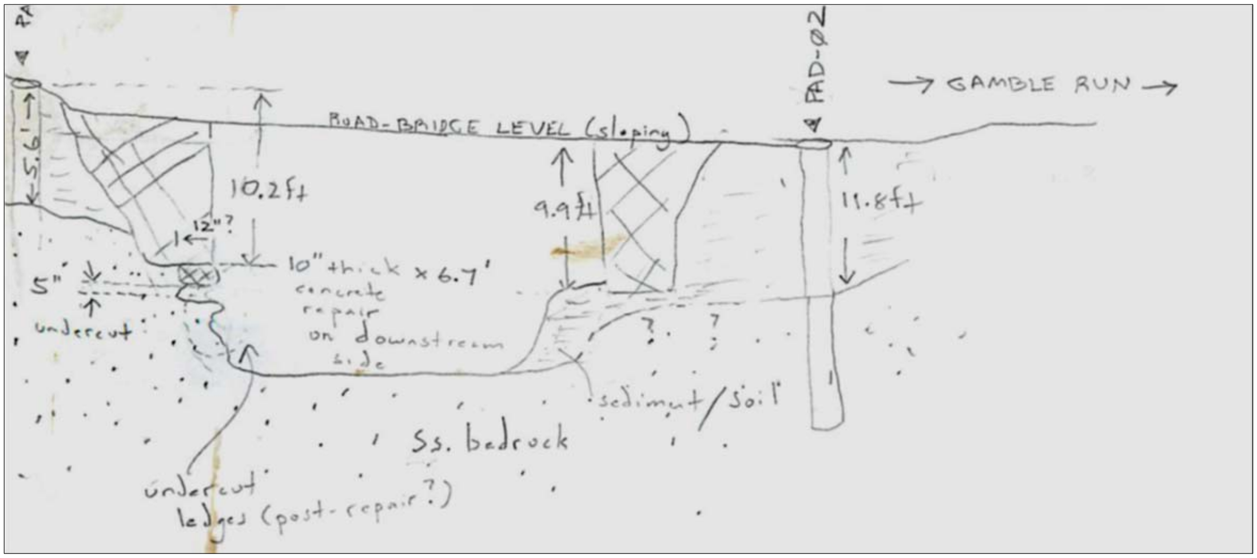

Figure 27 -- Field sketch of Paden Fork Bridge, looking downstream.

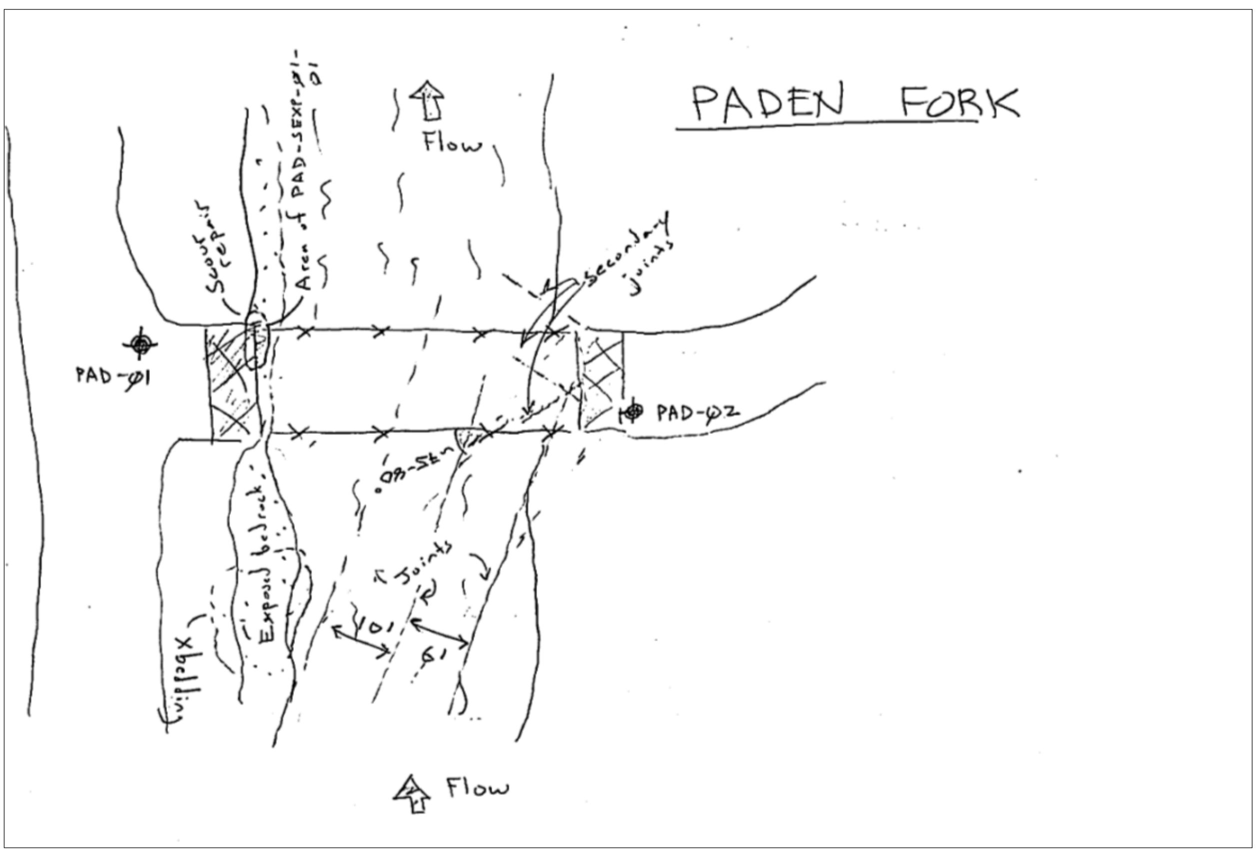

Figure 28 -- Field sketch of Paden Fork Bridge, overhead view.

Lab Results

Modified Slake Durability and GSN*

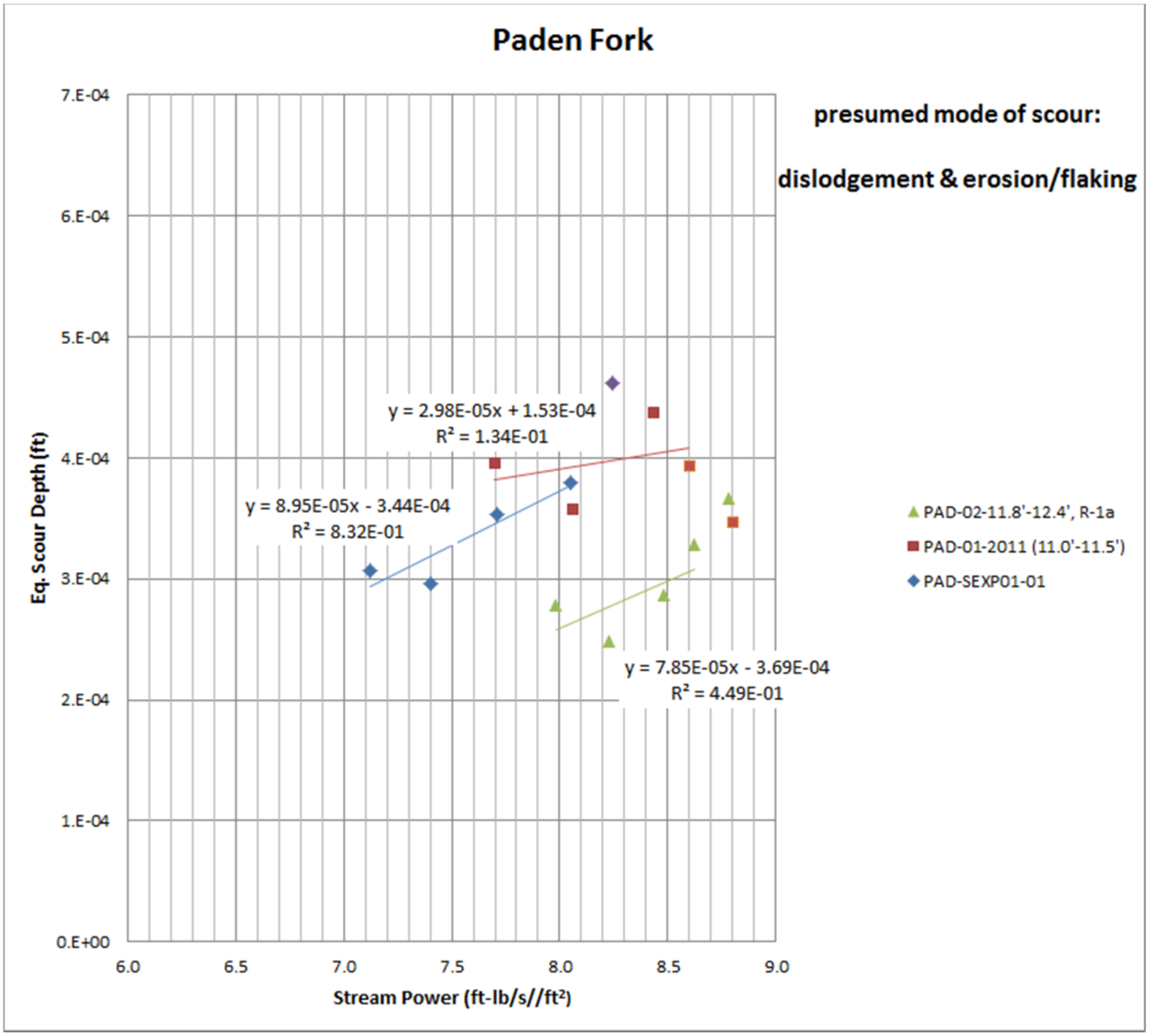

Figure 29 -- Data from modified slake durability tests and calculated values of GSN* and R^{*2} for Paden Fork samples.

7 – Audra Park

Field Results

Geology.

At the Audra Park bridge site the bridge abutments are set in sandstone belonging to the Pennsylvanian-aged Pottsville Group, which includes the Kanawha, New River, and Pocahontas Formations. Pottsville strata are predominantly sandstones, some of which are conglomeratic, along with some thin shale and coal beds. These rocks cover approximately 6,000 square miles over 12 counties, extending from Wayne and McDowell Counties in the southwestern and southern parts of West Virginia, respectively, to Preston and Tucker Counties in the north-central and northeastern portions of the state. The Pottsville Group is approximately 200 and 360 feet thick in the western and northeastern parts of its area, respectively, and thickens dramatically to over 3,800 feet in the southeastern part.

Based on surface exposure and the two boreholes drilled in June 2011, bedrock at the Audra bridge consists of a hard, gray, jointed sandstone of medium- to coarse-grained texture. Core recovered from the boreholes also revealed vugs, cross-bedding, and numerous fractures within the sandstone. A soft, fractured siltstone was encountered below the TZ in AUD-01-2011, which was drilled to a lower elevation than AUD-02-2011 due to location of the left abutment against the valley wall.

Bedrock exposed at the base of the left abutment is durable, coarse-grained sandstone with beds averaging 0.3 to 0.9 meters thick. More massive beds, up to 1 meter thick, are also present. Cross-bedding within surface exposure of the sandstone is prominent, with individual cross-beds approximately 0.15 meters thick.

Three joint sets were recognized, one approximately parallel to the stream channel, a second approximately perpendicular to the channel, and a third 40° to 50° to the channel. Spacing between individual joints is at least 0.2 meters.

Rock Quality Designation (RQD).

RQD for the sandstone core recovered from the target intervals in AUD-01-2011 and AUD-02-2011 is 44% and 79%, respectively.

Scour Mode and Magnitude

Areas of scour at the Audra Park bridge are shown on the field sketches below.

Figure 30 – Boring Log 1 for Audra Park Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. AUD-01-2011
SHEET 1 OF 2
DATE: START 6/28/11
END 6/28/11
O.G. ELEV. 1685.0

PROJECT NAME WVDOH Scour COUNTY Barbour
PROJECT LOCATION Audra Park Bridge
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______
STATE RT. NO. 11 SECT. ______ SEGMENT ______ OFFSET ______
INSPECTOR Nichole Wendlandt DRILLERS NAME/COMPANY Jim Hopkins/L.G. Hetager Drilling, Inc.
EQUIPMENT USED CME 45
DRILLING METHODS Hollow Stem Auger/Casing Advancer/SPT/NX Wireline Split Barrel
CASING: SIZE: 3.25" ; DEPTH: 23.1' ; WATER: DEPTH: ______ TIME: ______ DATE: ______
CHECKED BY: TLD ; DATE: 8/1/11 DEPTH: 22.0' TIME: 12:10 DATE: 6/28/11
Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT		DESCRIPTION	REMARKS
0.0	A-1				-				ASPHALT	Started hole at 0820 hrs. 6/28/2011 Cloudy/72 degrees
1.7									1.7 — 1683.3	
2.0	A-2				-				UNSAMPLED	~Bottom of Abutment @ 1662' Switched to casing at 2.0' Sand trap used for all soil sampling
	A-3				-					Stream pH=8.8
19.0									19.0 — 1666.0	
	S-1	2 10	0.6'	40	-	gp - a-1-a	W			

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **AUD-01-2011**
SHEET **2** OF **2**
DATE: START **6/28/11**
END **6/28/11**
O.G. ELEV. **1685.0**

PROJECT NAME **WVDOH Scour** COUNTY **Barbour**
PROJECT LOCATION **Audra Park Bridge**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H₂O CONTENT	DESCRIPTION	REMARKS
20.5		19		40			W	Sandy GRAVEL (gp), brown, wet, loose to medium dense, homogeneous, (alluvial) *(continued)*	
22.0	S-2	4 4 3	0.6'	40	-	gp - a-1-a	W		Started coring rock at 1043 hrs.
23.1	S-3	6 10 50/0.1	0.5'	45	-	gp - a-1-a	W	23.1 1661.9	
24.0	R-1		0.8'	89 - 0				Medium to coarse grained SANDSTONE, gray, hard, slightly to freshly weathered, intensely to thinly bedded (RD= 0° to 30°), extremely closely to very widely spaced fractures (RD= 0° to 90°), RQD=44%	R-1: 1043-1045 hrs.
26.0	R-2		1.9'	95 - 40					R-2: 1055-1105 hrs.
29.0	R-3		3.0'	100 - 60				23.4'-27.4' partially stained near vertical fracture 24.7'-24.8' vertical fracture 24.8'-24.9' very broken 24.7'-29.0' crossbedded and vuggy 27.3'-27.4' near vertical fracture 28.1' undulating fracture 28.6'-28.7' near vertical fractures 29.0 1656.0	R-3: 1113-1118 hrs.
34.0	R-4		5.0'	100 - 38				SILTSTONE, dark gray, soft, slightly to freshly weathered, intensely to v thinly bedded (RD= 0° to 10°), extremely closely to very widely spaced fractures (RD= 0° to 90°), RQD=38% 30.9'-31.0' very broken 32.9'-33.1' near vertical fractures 34.0 1651.0	R-4: 1124-1136 hrs. Completed hole at 1235 hrs. 6/28/2011
								Bottom of borehole at 34.00 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

Figure 31 – Boring Log 2 for Audra Park Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **AUD-02-2011**
SHEET **1** OF **2**
DATE: START **6/28/11**
END **6/28/11**
O.G. ELEV. **1693.0**

PROJECT NAME **WVDOH Scour** COUNTY **Barbour**
PROJECT LOCATION **Audra Park Bridge**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______
STATE RT. NO. **11** SECT. ______ SEGMENT ______ OFFSET ______
INSPECTOR **Nichole Wendlandt** DRILLERS NAME/COMPANY **Jim Hopkins/L.G. Hetager Drilling, Inc.**
EQUIPMENT USED **CME 45**
DRILLING METHODS **Hollow Stem Auger/Casing Advancer/SPT/NX Wireline Split Barrel**
CASING: SIZE: **3.25"** ; DEPTH: **16.0'** ; WATER: DEPTH: ______ TIME: ______ DATE: ______
CHECKED BY: **TLD** ; DATE: **8/1/11** DEPTH: **10.8'** TIME: **15:15** DATE: **6/28/11**

Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
0.0 – 0.6	A-1				-			0.6 ASPHALT 1692.4	Started hole at 1305 hrs. 6/28/2011
0.6 – 3.0	A-2				-			UNSAMPLED	Partly Sunny/78 degrees ~Bottom of Abutment @1683'
3.0 – 3.4	S-1	50/0.4	0.3'	75	-	gp	W	3.0 1690.0 3.4 Sandy GRAVEL (gp), dark gray, wet, very dense, homogeneous, (residual sandstone) 1689.6	Sand Trap used in all soil sampling
3.4 – 4.0	R-1		0.6'	100 - 0		a-1-a			Used casing due to nature of surrounding material
4.0 – 6.0	R-2		2.0'	100 - 50				Medium to coarse grained SANDSTONE, gray, hard, slightly to freshly weathered, intensely to thinly bedded (RD= 0° to 30°), extremely closely to widely spaced fractures (RD= 0° to 90°), RQD=79%	Started coring at 1335 hrs. R-1: 1335-1337 hrs. R-2: 1340-1348 hrs.
6.0 – 9.0	R-3		3.0'	100 - 80				3.5'-3.6' stained vertical fracture 4.0'-4.2' vertical fracture 4.9'-5.0' near vertical fracture 4.8'-5.1' healed vertical fracture 6.9'-7.0' very broken 7.0' to 16.0' crossbedded and vuggy	R-3: 1350-1358 hrs.
9.0 – 14.0	R-4		4.9'	98 - 90					R-4: 1412-1430 hrs.
14.0 – 19.0	R-5		4.9'	98 - 70				14.7'-14.8' healded undulating fracture 16.0 1677.0 Coarse grained SANDSTONE, gray, hard, freshly weathered, v thinly to medium bedded (RD= 0° to 30°), extremely closely to very widely spaced fractures (RD= 0° to 90°), RQD=56% 16.2' undulating fracture 16.0'-16.7' coal stringers 16.6'-16.7' vertical fracture	R-5: 1437-1446 hrs.
19.0 –				100 - 65					R-6: 1450-1456 hrs.

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO.	**AUD-02-2011**
SHEET **2** OF **2**	
DATE: START	**6/28/11**
END	**6/28/11**
O.G. ELEV.	**1693.0**

PROJECT NAME **WVDOH Scour** COUNTY **Barbour**
PROJECT LOCATION **Audra Park Bridge**
STATION OFFSET FROM CENTERLINE
NORTHING EASTING

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT		DESCRIPTION	REMARKS
	R-6		2.0'					✓	20.6'-20.7' near vertical fracture	Completed hole at 1535 hrs. 6/28/2011
21.0								✓	21.0 1672.0	
									Bottom of borehole at 21.00 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

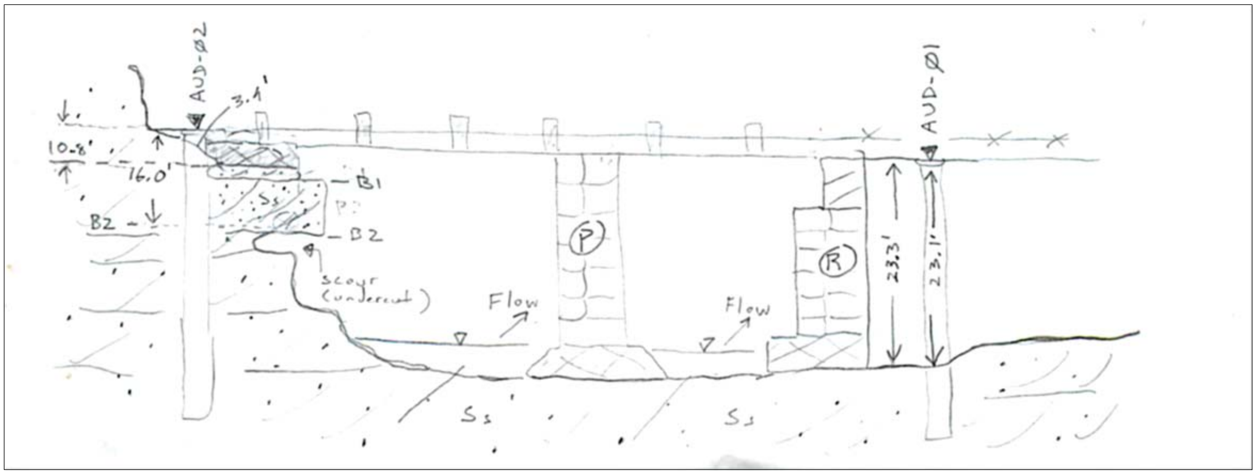

Figure 32 -- Field sketch of Audra Park Bridge, looking downstream.

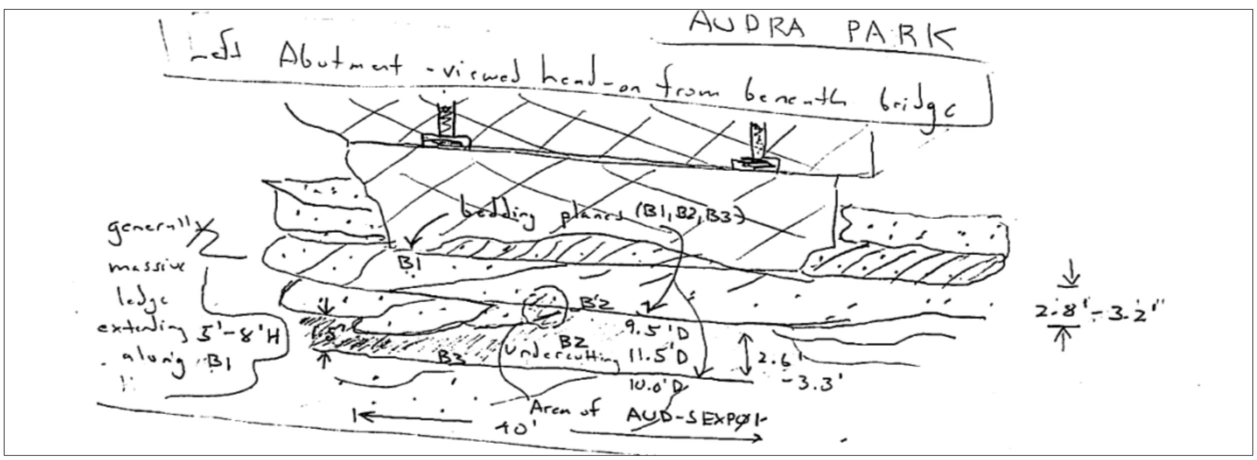

Figure 33 -- Field sketch of Audra Park Bridge, looking at left abutment.

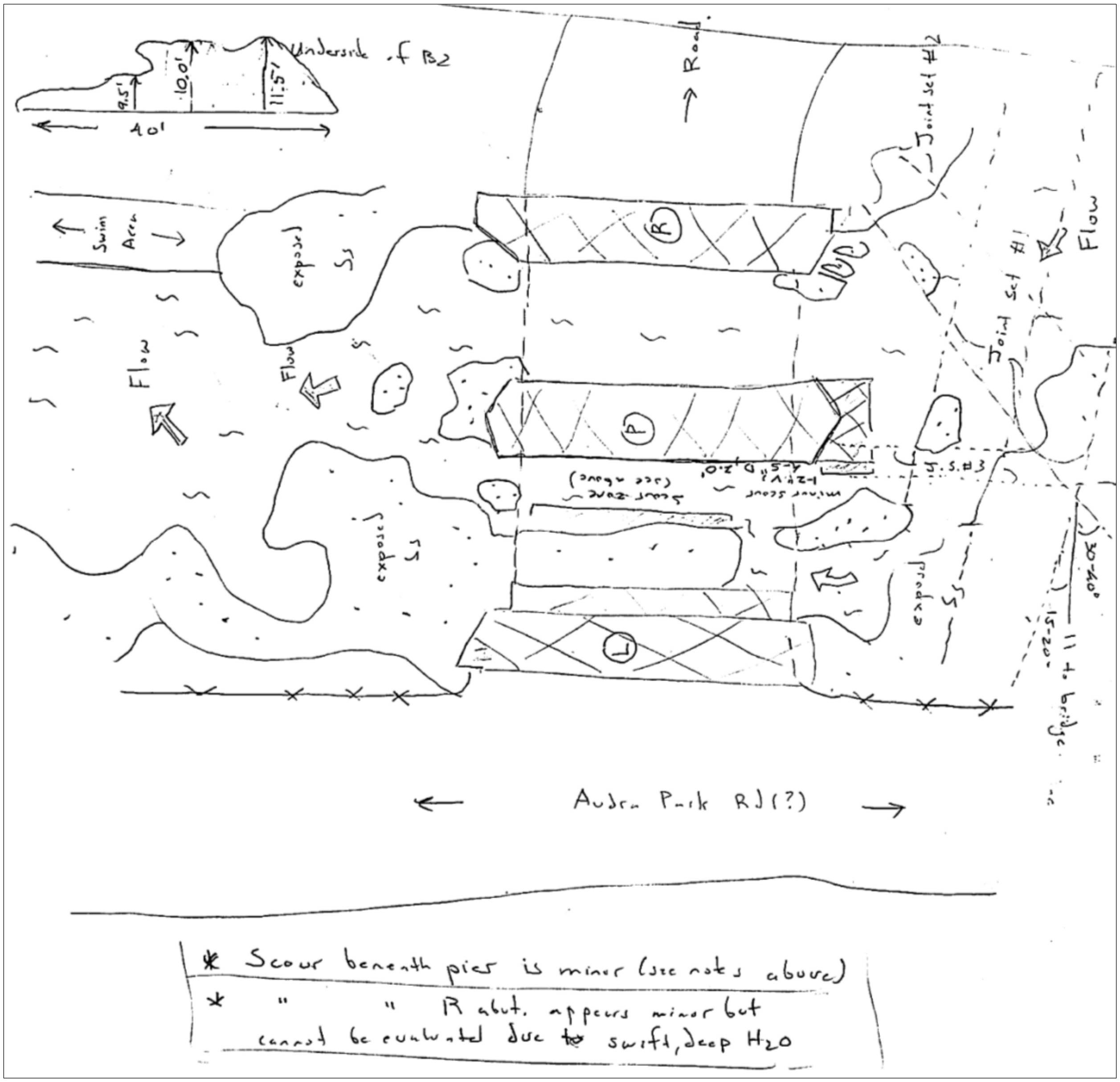

Figure 34 -- Field sketch of Audra Park Bridge, overhead view.

Lab Results

Modified Slake Durability and GSN*

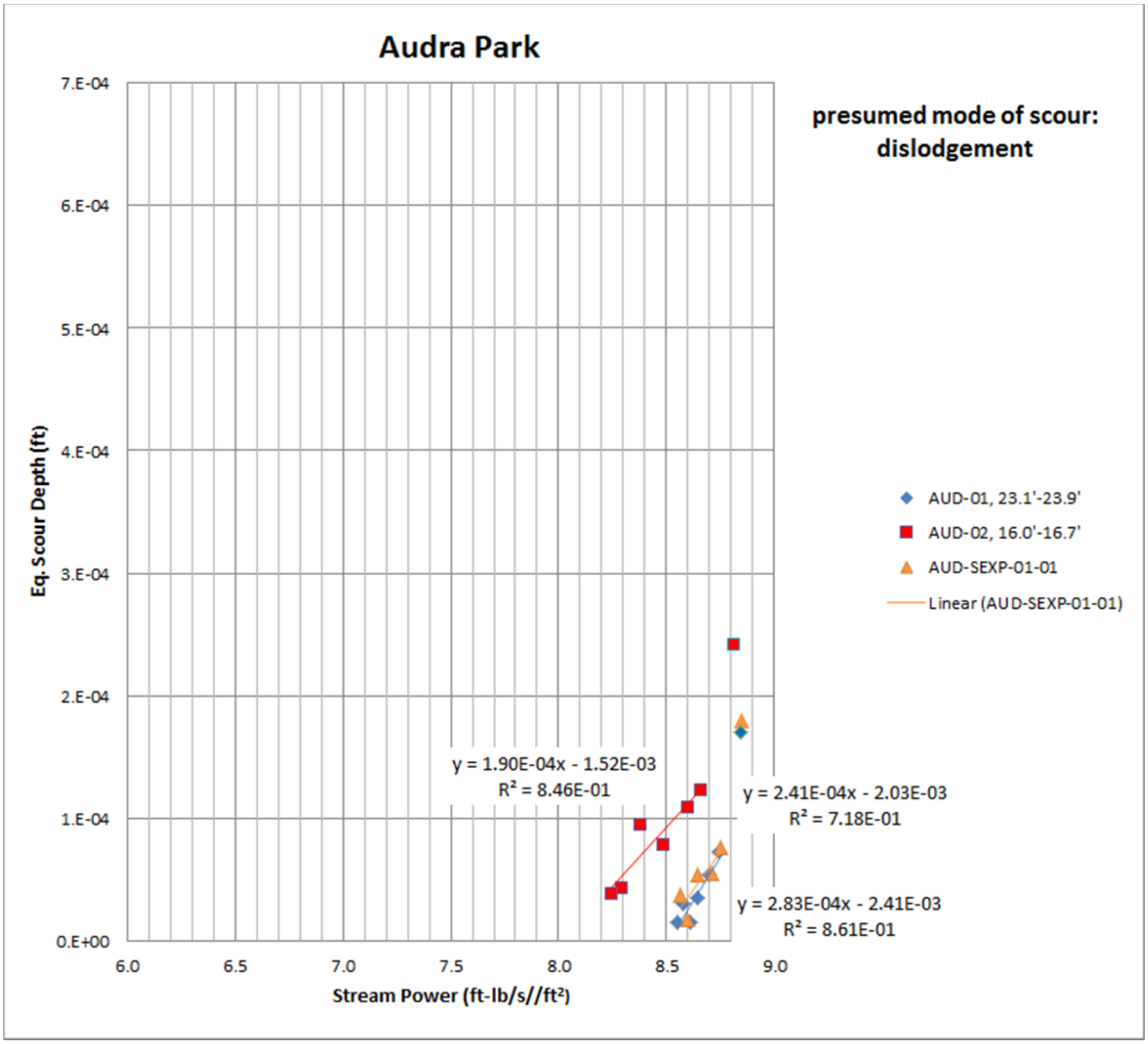

Figure 35 -- Data from modified slake durability tests and calculated values of GSN* and R^{*2} for Audra Park samples.

8 – Laurel Fork

Field Results

Geology.

At the Laurel Fork bridge site the bridge abutments are set in sandstone belonging to the Pennsylvanian-age Allegheny Formation, which covers in excess of 1,400 square miles over a large strip of West Virginia extending from Wayne County in the southwestern portion of the state to Preston and Mineral Counties in the northeastern portion. The Allegheny consists of cyclic sequences of sandstone, siltstone, shale, limestone, and coal. The formation is thickest in the western parts of its distribution, up to 325 feet, and thinner to the northeast, where it is typically in the range of 150-200 feet.

Based on surface exposure and the two boreholes drilled in June 2011, bedrock geology at the Laurel Fork bridge consists mostly of gray to tan, medium- to coarse-grained sandstone. A thin (0.2 m or less) carbonaceous shale occurs 0.2 meters and 0.8 meters into the target interval in LAU-01-2011 and LAU-02-2011, respectively. The sandstone from both boreholes is described as average to above-average hardness, thin-bedded, and heavily fractured.

Bedrock exposed upstream of the left bridge abutment is exceptionally hard, medium-bedded (0.2-0.6 m) sandstone. The rock is also cross-bedded. Jointing is evident but not prominent in the surface exposure. A single joint set is oriented at 40° to 50° relative to the stream channel. Based on only two available measurements, joint spacing is irregular: 0.6 and 2.1 meters.

Rock Quality Designation (RQD).

RQD for the sandstone, underlying shale, and underlying sandstone in LAU-01-2011 is 0%, 0% and 50%, respectively. RQD for the strata in LAU-02-2011 is 56%, 0% and 59%, respectively.

Scour Mode and Magnitude

Areas of scour at the Laurel Fork bridge are shown on the field sketches below.

Figure 36 – Boring Log 1 for Laurel Fork Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **LAU-01-2011**
SHEET **1** OF **1**
DATE: START **6/27/11** END **6/27/11**
O.G. ELEV. **1441.0**

PROJECT NAME **WVDOH Scour** COUNTY **Upshur**
PROJECT LOCATION **Laurel Fork Bridge**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______
STATE RT. NO. **20/10** SECT. ______ SEGMENT ______ OFFSET ______
INSPECTOR **Nichole Wendlandt** DRILLERS NAME/COMPANY **Jim Hopkins/L.G. Hetager Drilling, Inc.**
EQUIPMENT USED **CME 45**
DRILLING METHODS **Hollow Stem Auger/SPT/NX Wireline Split Barrel**
CASING: SIZE: ______ ; DEPTH: ______ ; WATER: DEPTH: ______ TIME: ______ DATE: ______
CHECKED BY: **TLD** ; DATE: **8/1/11** DEPTH: **8.4'** TIME: **10:35** DATE: **6/27/11**
Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
0.0 – 3.0	A-1				-			UNSAMPLED	Started hole at 0830 hrs. 6/27/2011 Partly Cloudy/70 degrees
3.0 – 4.5	S-1	18 / 9 / 9	1.2'	80	-	SC - a-2-6	M	3.0 1438.0 Clayey Medium To Coarse SAND (sc), brown, moist, medium dense, homogeneous, (alluvial)	~Bottom of Abutment at 1433'
4.5 – 6.0	S-2	8 / 8 / 9	1.3'	87	-	SC - a-2-6	M		Sand trap used for all soil sampling
6.0 – 7.5	S-3	7 / 10 / 7	1.4'	93	-	SC - a-2-6	M		Stream pH=8.3
7.5 – 8.6	S-4	9 / 10	0.7'	64	-	SC - a-2-6	M	8.6 1432.4	Started coring at 0930 hrs.
9.0	R-1	50/0.1	0.4'	100 - 0				Medium to coarse grained SANDSTONE, tan, average hard, slightly to freshly weathered, v thinly to thinly bedded (RD= 0° to 30°), very closely to closely spaced fractures (RD= 0° to 30°), RQD=0% 9.3 1431.7	R-1: 0930-0935 hrs.
9.0 – 11.5	R-2		2.5'	100 - 52				10.0 1431.0 Carbonaceous SHALE, black, soft, freshly weathered, intensely laminated (RD= 0° to 10°), extremely closely to very closely spaced fractures (RD= 0° to 30°), RQD=0%	R-2: 0938-0943 hrs.
11.5 – 14.0	R-3		2.4'	96 - 92				Medium to coarse grained SANDSTONE, gray, average hard to hard, slightly to freshly weathered, intensely to thinly bedded (RD= 0° to 30°), very closely to very widely spaced fractures (RD= 0° to 90°), RQD=50% some micaceous stringers	R-3: 0945-0952 hrs.
14.0 – 19.0	R-4		4.0'	80 - 18				13.1'-13.2' vertical fracture 15.7'-16.1' stained vertical fracture 16.6'-16.9' vertical fracture 16.9'-18.1' crossbedded (1/4"-1/2") 18.2'-18.4' vertical fracture 18.7'-18.8' vertical fracture 19.0 1422.0	Lost water at 14.5' (never returned) R-4: 0954-1004 hrs. Completed hole at 1055 hrs. 6/27/2011
19.0								Bottom of borehole at 19.00 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

Figure 37 – Boring Log 2 for Laurel Fork Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. LAU-02-2011
SHEET 1 OF 2
DATE: START 6/27/11
END 6/27/11
O.G. ELEV. 1443.0

PROJECT NAME WVDOH Scour — COUNTY Upshur
PROJECT LOCATION Laurel Fork Bridge
STATION ____ OFFSET FROM CENTERLINE ____
NORTHING ____ EASTING ____
STATE RT. NO. 20/10 SECT. ____ SEGMENT ____ OFFSET ____
INSPECTOR Nichole Wendlandt — DRILLERS NAME/COMPANY Jim Hopkins/L.G. Hetager Drilling, Inc.
EQUIPMENT USED CME 45
DRILLING METHODS Hollow Stem Auger/SPT/NX Wireline Split Barrel
CASING: SIZE: ____ ; DEPTH: ____ ; WATER: DEPTH: ____ TIME: ____ DATE: ____
CHECKED BY: TLD ; DATE: 8/1/11 DEPTH: 17.1' TIME: 13:54 DATE: 6/27/11

Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
0.0–0.8	A-1				-			ASPHALT 0.8 1442.2	Started hole at 1115 hr.s 6/27/2011
0.8–5.0	A-2				-			UNSAMPLED	Partly Cloudy/74 degrees ~Bottom of Abutment at 1433'
5.0–6.5	S-1	2, 3, 2	0.8'	53	-	sc - a-2-6	M	5.0 1438.0 Clayey SAND (sc), dark brown, moist, loose, homogeneous, (fill) 6.0 1437.0 Sandy CLAY (cl), brown, moist, medium dense to stiff, homogeneous, (alluvial)	Sand trap used for all soil sampling
6.5–8.0	S-2	2, 3, 3	0.9'	60	PP 1.50	cl - a-6	M		
8.0–9.5	S-3	2, 2, 2	0.9'	60	PP 1.00	cl - a-6	M		
9.5–11.0	S-4	6, 5, 5	1.0'	67	-	cl - a-6	M	10.3 1432.7 Clayey SAND (sc), brown, moist, medium dense, homogeneous, (residual)	
11.0–12.5	S-5	2, 7, 7	1.4'	93	-	sc - a-6	M	pieces of sandstone throughout 12.6 1430.4	Started coring at 1243 hrs.
12.5–12.6	S-6	50/0.1	0.1'	100	-	sc - a-6	M		
12.6–14.0	R-1		1.1'	79 - 36				Medium to coarse grained SANDSTONE, gray tan, average hard to hard, slightly to freshly weathered, v thinly to thinly bedded (RD= 0° to 30°), extremely closely to closely spaced fractures (RD= 0° to 90°), RQD=56% 15.1 micaceous stringers throughout 1427.9	R-1: 1243-1246 hrs.
14.0–16.0	R-2		2.0'	100 - 45				15.4 13.4'-13.5' stained near vertical fracture 1427.6 14.5'-14.6' stained near vertical fracture Carbonaceous SHALE, black, soft, slightly weathered, intensely laminated (RD= 0° to 10°), extremely closely spaced fractures (RD= 0° to 30°), RQD=0%	R-2: 1249-1253 hrs.
16.0–19.0	R-3		2.6'	87 - 53				18.1'-18.6' stained vertical fracture 20.3' to 20.4' near vertical fracture	R-3: 1258-1301 hrs.
19.0–				100 - 73					R-4: 1310-1316 hrs. Lost water at 19.0' (never

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

PROJECT NAME **WVDOH Scour** COUNTY **Upshur**
PROJECT LOCATION **Laurel Fork Bridge**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______

BORING NO. **LAU-02-2011**
SHEET **2** OF **2**
DATE: START **6/27/11**
END **6/27/11**
O.G. ELEV. **1443.0**

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT		DESCRIPTION	REMARKS
	R-4		4.0'	100 - 73				✓✓✓✓✓✓	Medium to coarse grained SANDSTONE, gray tan, average hard to hard, slightly to freshly weathered, intensely to thinly bedded (RD= 0° to 30°), extremely closely to very widely spaced fractures (RD= 0° to 90°), RQD=59% *(continued)*	returned) Completed hole at 1405 hrs. 6/27/2011
23.0									23.0 1420.0	
									Bottom of borehole at 23.00 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

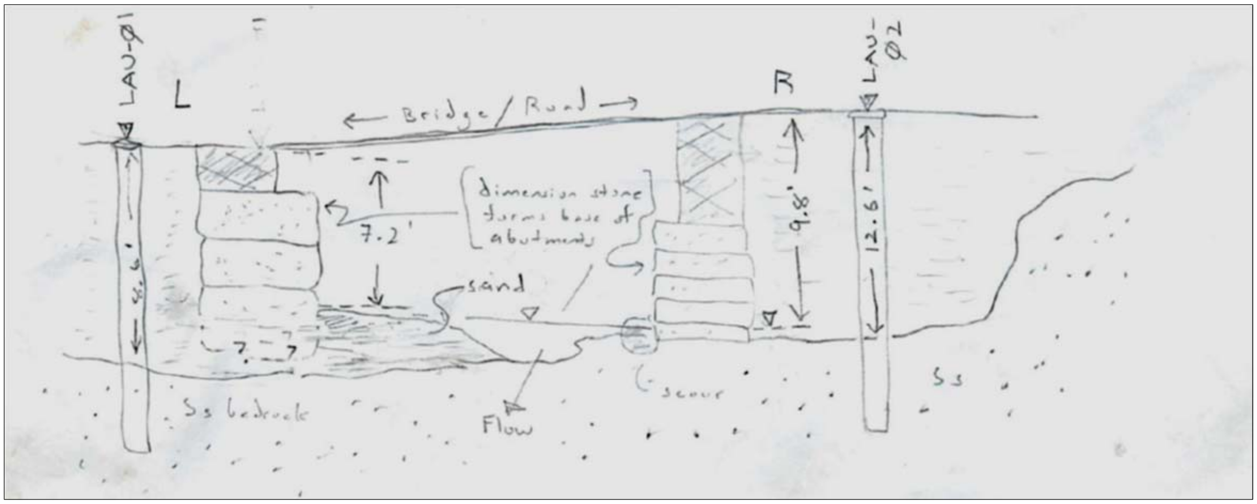

Figure 38 -- Field sketch of Laurel Fork Bridge, looking upstream.

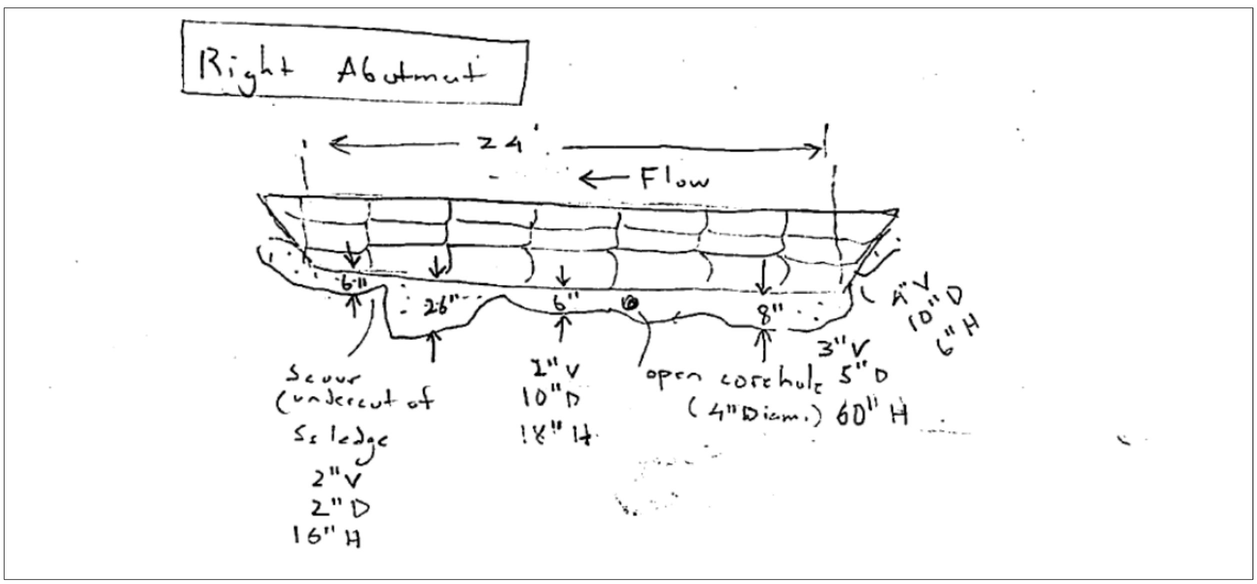

Figure 39 -- Field sketch of Laurel Fork Bridge, looking at right abutment.

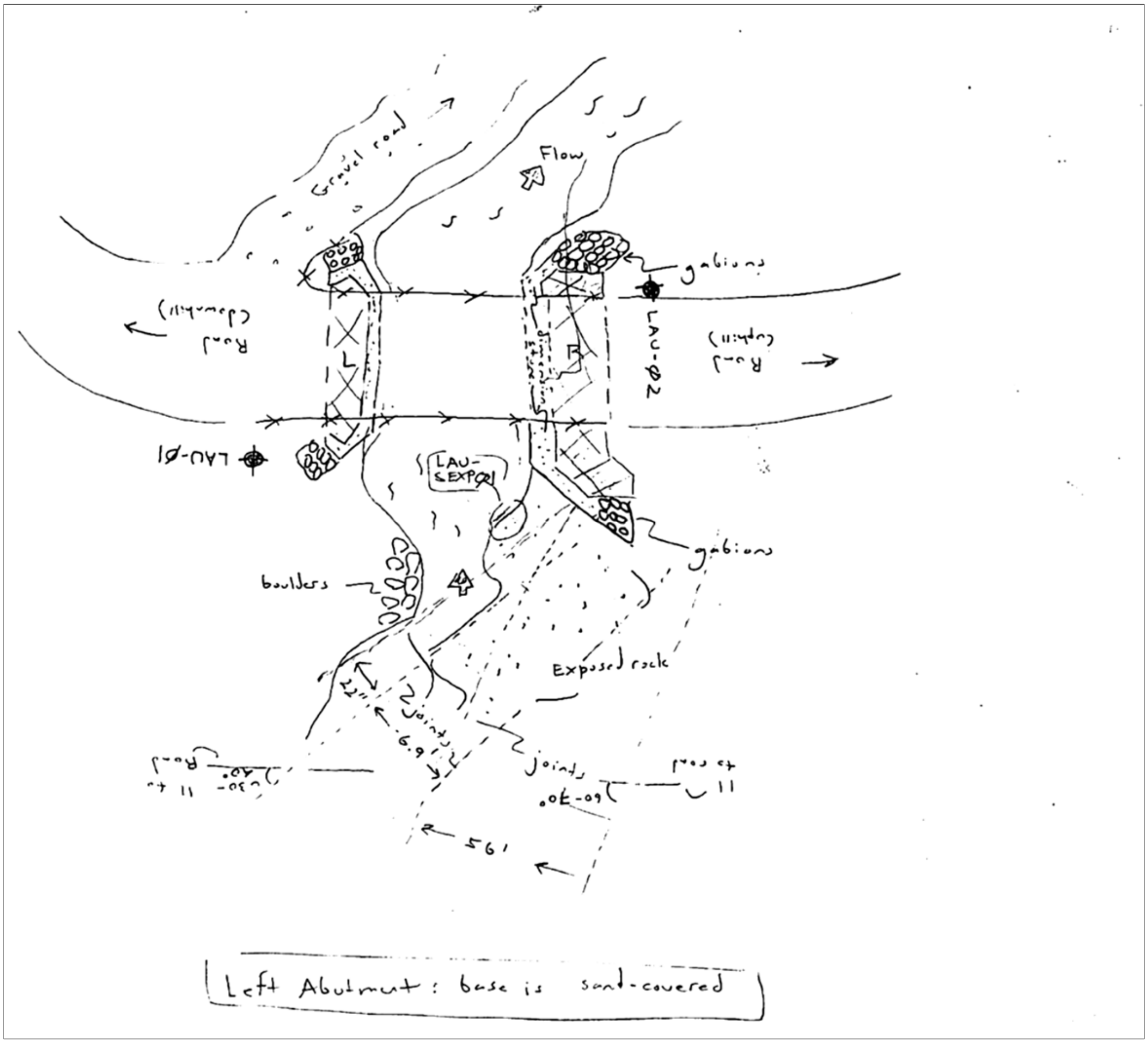

Figure 40 -- Field sketch of Laurel Fork Bridge, looking downstream.

Lab Results

Modified Slake Durability and GSN*

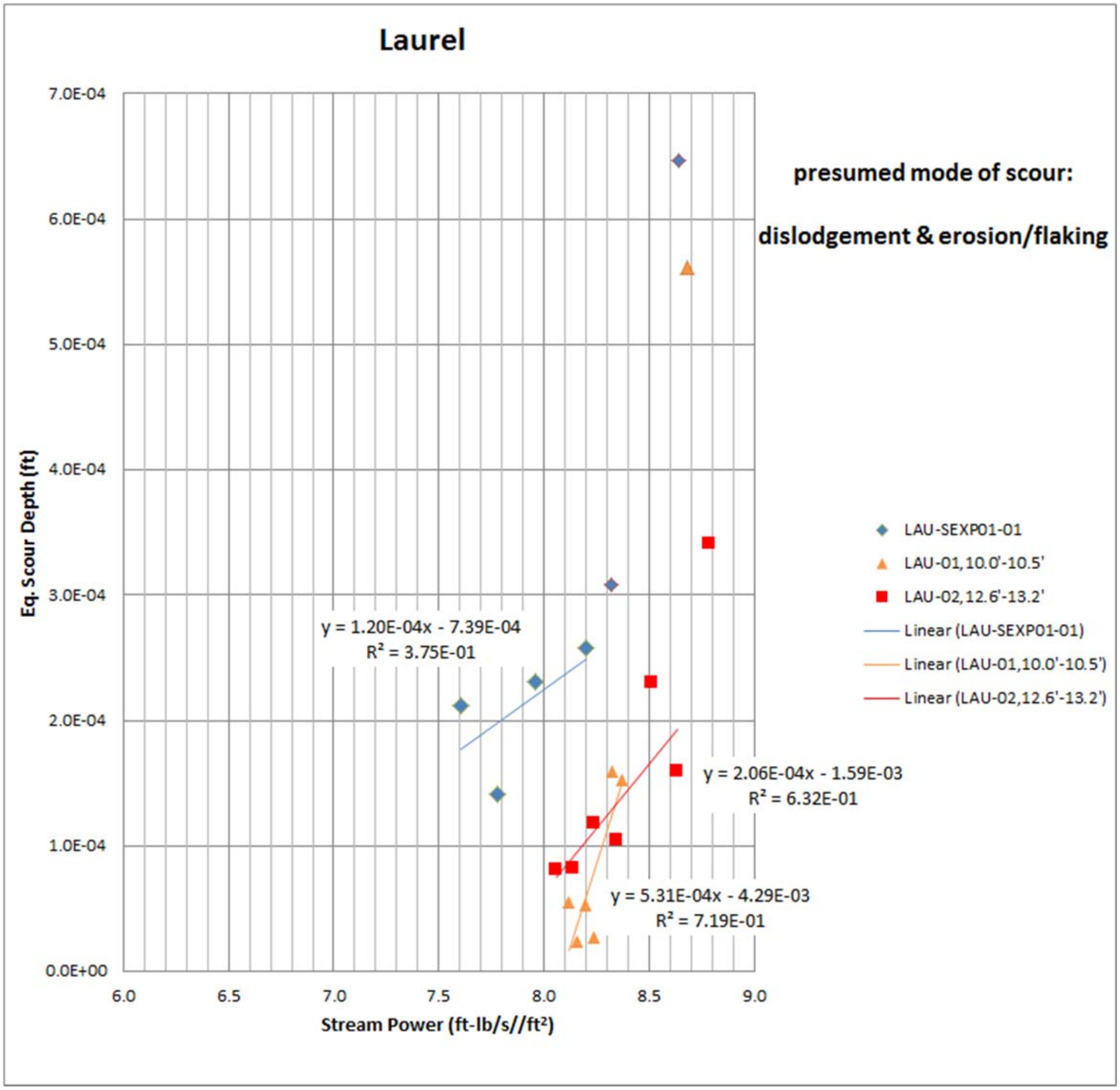

Figure 41 -- Data from modified slake durability tests and calculated values of GSN* and R^{*2} for Laurel Fork Bridge samples.

9 – Roaring Creek

Field Results

Geology.

The Roaring Creek bridge site is located in the Valley and Ridge Province. The bridge abutments and a single pier are set in sandstone belonging to the Mississippian-age Pocono Group, which is comprised of the Hedges, Purslane, and Rockwell Formations. These formations are dominantly massive, hard, gray sandstones with some shale. The Pocono Group occupies a thin, 400 square-mile band of outcrop in the eastern part of West Virginia. These strata extend over parts of 15 counties, from Mercer County in the extreme southeastern part of the state, to Preston County in the north-central part, to isolated outcroppings in the eastern panhandle. Thickness of the Pocono Group ranges from 570-1030 feet in the northeastern part of its area, to 525 feet in the west, to 350-900 feet in the southeast.

Based on the two boreholes drilled in June 2011, bedrock geology in the TZ at the Roaring Creek bridge consists of gray, hard to very hard, fine-grained sandstone with inconsistent fracturing. Bedding ranged from thick to very thick in ROA-01-2011 to very thin to medium in ROA-02-2011. A thin (0.3 m) claystone was sandwiched between sandstone beds within the TZ in ROA-01-2011.

Bedrock at Roaring Creek was exposed over an extensive area underneath, upstream and downstream of the bridge. This rock was durable sandstone with prominent jointing. Joint sets are well represented; both sets are oriented approximately 90 $^{\circ}$ to each other (i.e., "conjugate" sets) and 45° to the channel upstream of the bridge. Spacing in the first set ranges from 0.9 to 3 meters; spacing in the second set ranges from 0.22 to 1 meter.

Rock Quality Designation (RQD).

RQD in core from ROA-01-2011 and ROA-02-2011 is 75% and 50%, respectively.

Scour Mode and Magnitude

Areas of scour at the Roaring Creek bridge are shown on the field sketches below.

Figure 42 – Boring Log 1 for Roaring Creek Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **ROA-01-2011**
SHEET **1** OF **2**
DATE: START **7/10/11**
END **7/11/11**
O.G. ELEV. **1762.0**

PROJECT NAME **WVDOH Scour** COUNTY **Pendleton**
PROJECT LOCATION **Roaring Creek Bridge**
STATION ____ OFFSET FROM CENTERLINE ____
NORTHING ____ EASTING ____
STATE RT. NO. **33** SECT. ____ SEGMENT ____ OFFSET ____
INSPECTOR **Jared Govi** DRILLERS NAME/COMPANY **Jim Hopkins/L.G. Hetager Drilling, Inc.**
EQUIPMENT USED **HH 240**
DRILLING METHODS **Hollow Stem Auger/Casing Advancer/SPT/NX Wireline Split Barrel**
CASING: SIZE: **4.25"** ; DEPTH: **8.1'** ; WATER: DEPTH: ____ TIME: ____ DATE: ____
CHECKED BY: **NJW** ; DATE: **8/1/11** DEPTH: **10.0'** TIME: **11:00** DATE: **7/11/11**

Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
0.0								ASPHALT (0.6, 1761.4)	Started hole at 0700 hrs. 7/10/2011
	A-1				-			UNSAMPLED	Sunny/60 degrees Drilling stopped by WVDOH District 8-0 Bridge Engineer on 7/10/11 due to drilling restriction on weekends and holidays Resumed hole at 0700 hrs. 7/11/2011, Sunny/80 degrees Switched to casing at 2.0' due to boulders
6.0	S-1	41 39 50/0.2	0.7'	58	-	gp - a-1-a	W	(6.0, 1756.0) Sandy GRAVEL (gp), light tan and brown, wet, very dense, homogeneous, (alluvial)	Stream pH=8.3
7.2–7.5	A-2				-	gp - a-1-a			
7.5–8.1	S-2	50 50/0.1	0.5'	83	-	gp - a-1-a	W		
8.1	R-1		2.9'	100 - 93				(8.1, 1753.9) Fine grained SANDSTONE, gray, hard to very hard, freshly weathered, thickly to v thickly bedded (RD= 0° to 5°), widely to very widely spaced fractures (RD= 0° to 10°), RQD=75%	Started coring at 0840 hrs. R-1: 0840-0900 hrs.
11.0	R-2		2.1'	84 - 24				11.2'-12.2' claystone	R-2: 0900-0920 hrs.
13.5	R-3		2.3'	92 - 92				16.8'-18.5' broken	R-3: 0920-0935 hrs.
16.0	R-4		2.4'	96 - 68					R-4: 0935-1000 hrs.
18.5	R-5		2.5'	100 - 96					R-5: 1000-1020 hrs.

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **ROA-01-2011**
SHEET **2** OF **2**
DATE: START **7/10/11**
END **7/11/11**
O.G. ELEV. **1762.0**

PROJECT NAME **WVDOH Scour** COUNTY **Pendleton**
PROJECT LOCATION **Roaring Creek Bridge**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
21.0								21.0 1741.0 Bottom of borehole at 21.00 feet.	Completed hole at 1100 hrs. 7/11/2011

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

Figure 43 – Boring Log 2 for Roaring Creek Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. ROA-02-2011
SHEET 1 OF 2
DATE: START 7/11/11
END 7/11/11
O.G. ELEV. 1763.0

PROJECT NAME WVDOH Scour COUNTY Pendleton
PROJECT LOCATION Roaring Creek Bridge
STATION ____ OFFSET FROM CENTERLINE ____
NORTHING ____ EASTING ____
STATE RT. NO. 33 SECT. ____ SEGMENT ____ OFFSET ____
INSPECTOR Jared Govi DRILLERS NAME/COMPANY Jim Hopkins/L.G. Hetager Drilling, Inc.
EQUIPMENT USED HH 240
DRILLING METHODS Casing Advancer/SPT/NX Wireline Split Barrel
CASING: SIZE: 4.25" ; DEPTH: 6.1' ; WATER: DEPTH: ____ TIME: ____ DATE: ____
CHECKED BY: NJW ; DATE: 8/1/11 DEPTH: dry TIME: 16:00 DATE: 7/11/11
Checked = No Recorded Water Reading Measurements [X]

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H₂O CONTENT	DESCRIPTION	REMARKS
0.0 – 0.5	A-1				-			0.5 ASPHALT 1762.5	Started hole at 1145 hrs. 7/11/2011
0.5 – 6.0	A-2				-			UNSAMPLED	Sunny/85 degrees ~Bottom of Abutment at 1752'
6.0 – 6.1	S-1	50/0.1	0.1'	100	-	gp	D	6.0 1757.0 6.1 Poorly Graded GRAVEL (gp), gray, dry, 1756.9 very dense, homogeneous, (fill)	Started coring at 1300 hrs.
6.1 – 7.5	R-1		1.3'	93 - 0		a-1-a		Fine grained SANDSTONE, gray, hard to very hard, slightly to freshly weathered, v thinly to medium bedded (RD= 0° to 15°), very closely to very widely spaced fractures (RD= 0° to 10°), RQD=50%	R-1: 1300-1310 hrs.
7.5 – 10.5	R-2		2.8'	93 - 67					R-2: 1315-1335 hrs.
10.5 – 15.5	R-3		5.0'	100 - 72				11.8'-12.2' diagonal fracture	R-3: 1335-1400 hrs.
15.5 –	R-4		5.2'	95 - 35					R-4: 1400-1435 hrs.

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

PROJECT NAME **WVDOH Scour** COUNTY **Pendleton**
PROJECT LOCATION **Roaring Creek Bridge**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______

BORING NO. **ROA-02-2011**
SHEET **2** OF **2**
DATE: START **7/11/11**
END **7/11/11**
O.G. ELEV. **1763.0**

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT		DESCRIPTION	REMARKS
21.0									21.0 1742.0	Completed hole at 1500 hrs. 7/11/2011
									Bottom of borehole at 21.00 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

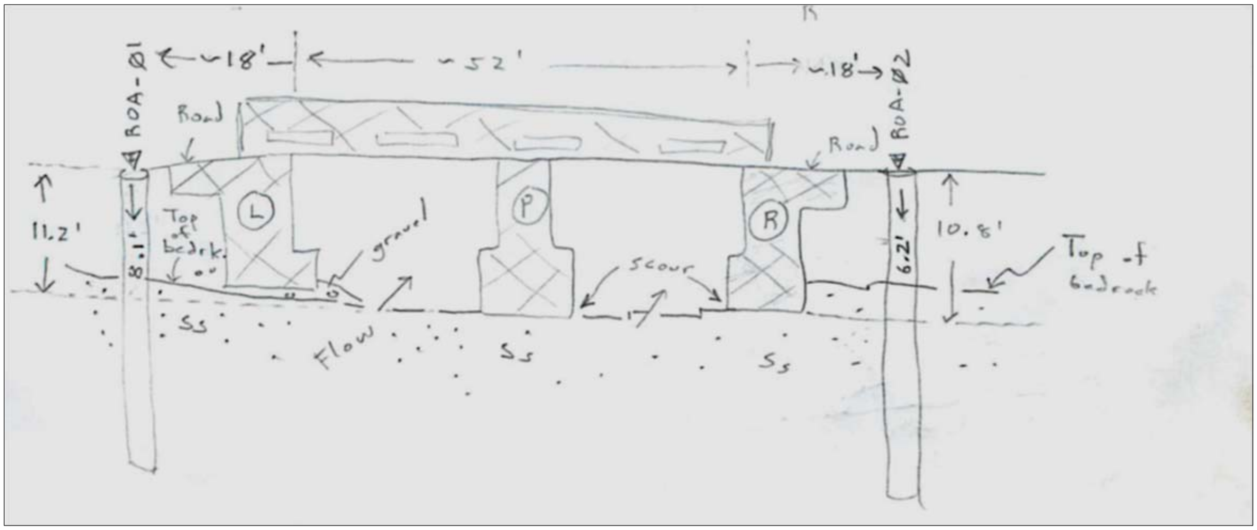

Figure 44 -- Field sketch of Roaring Creek Bridge, looking upstream.

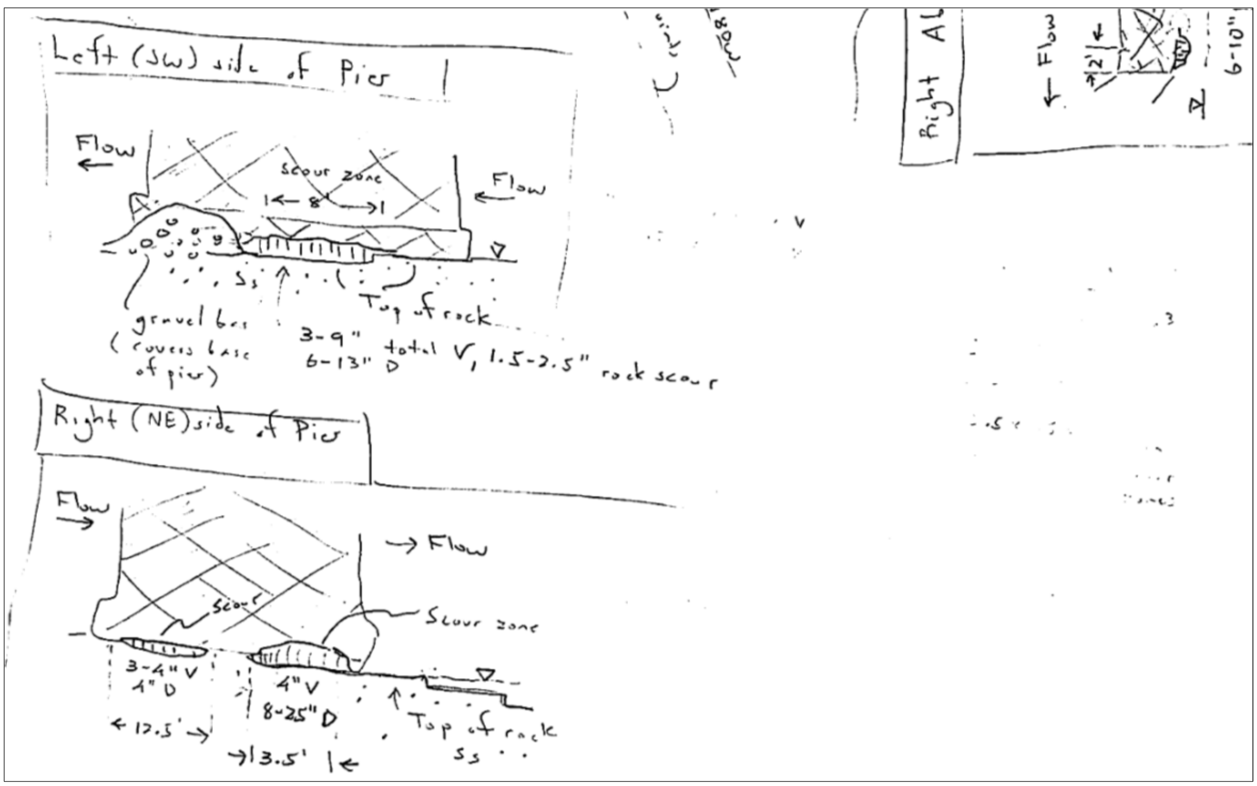

Figure 45 -- Field sketch of Roaring Creek Bridge, looking at insides of left pier (top sketch) and right pier (bottom sketch).

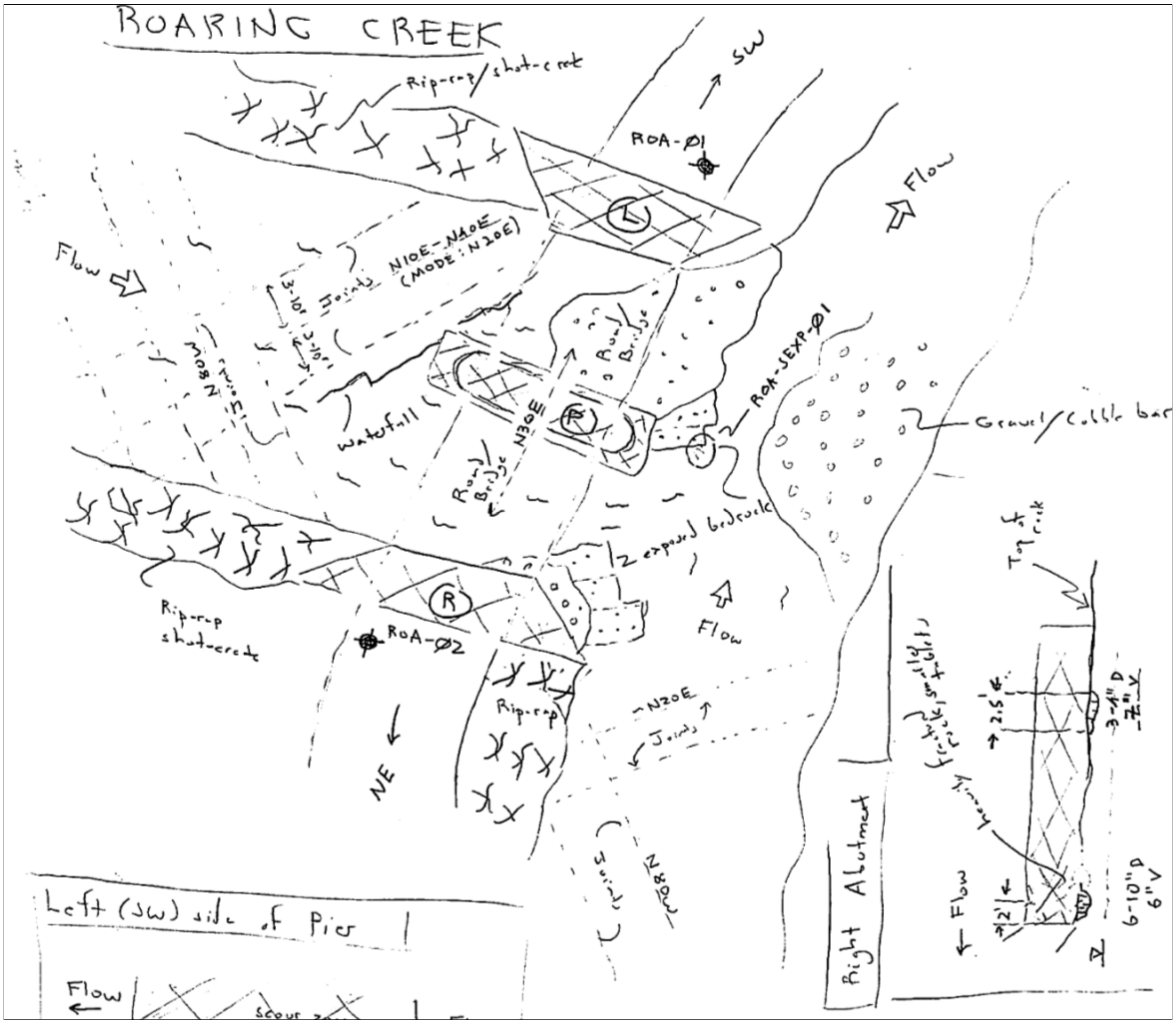

Figure 46 -- Field sketch of Roaring Creek Bridge, overhead view and right abutment (lower right).

Lab Results

<u>Modified Slake Durability and GSN*</u>

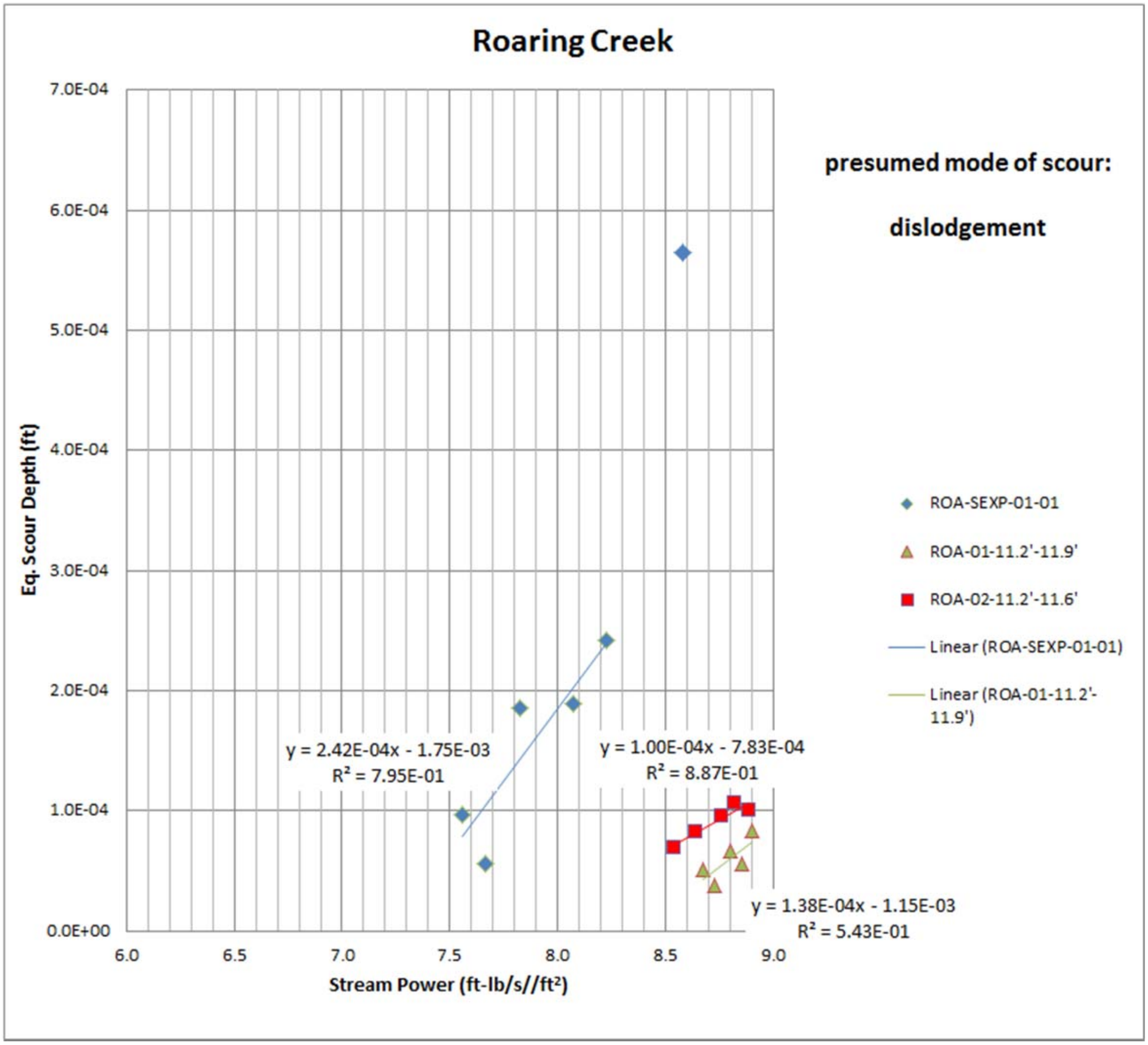

Figure 47 -- Data from modified slake durability tests and calculated values of GSN* and R^{*2} for Roaring Creek Bridge samples.

10 – Beverly

Field Results

Geology.

The Beverly bridge site is located in the Valley and Ridge Province. The bridge abutments and two piers are set in steeply-dipping sandstone and siltstone strata belonging to the Devonian-age Chemung Group. Regionally, the Chemung consists of sandstone and siltstone with conglomeratic and shale interbeds; these rocks are interpreted to have a marine origin. The Chemung Group occupies a thin, 1,100 square-mile band of outcrop in the extreme eastern part of West Virginia and the eastern panhandle, extending over parts of 14 counties. Thickness of these rocks ranges from 5oo feet in the southeastern part of its area (Monroe County) to 3,000 feet in the northeastern part (Randolph and Tucker Counties and the eastern panhandle).

Based on core recovered from the two boreholes drilled in June 2011, bedrock geology of the TZ at the Beverly bridge consists of sandy siltstone underlain by claystone in BEV-01A-2011 adjacent to the left (west) abutment and fine-grained siltstone underlain by claystone in BEV-02A-2011 adjacent to the right (east) abutment. It should be noted that due to the steep dip of rock strata at this bridge site, and the distance between boreholes, the siltstone and claystone encountered in the subsurface cannot be continuous from one borehole to another but instead represent different siltstone and claystone layers. The siltstone in both boreholes is described as hard and well fractured. The claystone in both boreholes is described as dark gray, thinly bedded, and highly fractured, with hardness ranging from soft to average.

Surface exposure of bedrock at the base of the right (east) abutment shows thin to medium (1.9-20 cm), layers of siltstone with interbedded shale. These strata are inclined to the northeast at an approximate angle of 75°. A single, high-angle joint set is oriented sub-parallel to the dip direction of bedding. Individual joints within this set are irregularly spaced (0.2 to 0.36m).

Rock Quality Designation (RQD).

RQD in BEV-01A-2011 for the siltstone and claystone is 8% and 0%, respectively. RQD in BEV-02A-2011 for the siltstone and claystone is 32% and 37%, respectively.

Scour Mode and Magnitude

Areas of scour at the Beverly bridge are shown on the field sketches below.

Figure 48 – Boring Log 1 for Beverly Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO BEV-01A-2011
SHEET 1 OF 2
DATE: START 7/9/11
END 7/9/11
O.G. ELEV. 1950.0

PROJECT NAME WVDOH Scour COUNTY Randolph
PROJECT LOCATION Beverly Bridge
STATION OFFSET FROM CENTERLINE
NORTHING EASTING
STATE RT. NO. 37/8 SECT. SEGMENT OFFSET
INSPECTOR Jared Govi DRILLERS NAME/COMPANY Jim Hopkins/L.G. Hetager Drilling, Inc.
EQUIPMENT USED HH 240
DRILLING METHODS Casing Advancer/SPT/NX Wireline Split Barrel
CASING: SIZE: 4.25" ; DEPTH: 4.0' ; WATER: DEPTH: TIME: DATE:
CHECKED BY: NJW ; DATE: 8/26/13 DEPTH: dry TIME: 12:00 DATE: 7/9/11
Checked = No Recorded Water Reading Measurements [X]

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
0.0–0.7	A-1				-			ASPHALT (0.7, 1949.3)	Started hole at 0730 hrs. 7/9/2011
0.7–4.0	A-2				-			UNSAMPLED (4.0, 1946.0)	Cloudy/65 degrees; ~Bottom of Abutment at 1934'
4.0–7.0	R-1		2.3'	77 - 23				Sandy SILTSTONE, gray, hard, moderately to freshly weathered, v thinly bedded (RD= 75°), very closely to widely spaced fractures (RD= 0° to 20°), RQD=8%	Used casing due to boulders and fill; Started coring at 0800 hrs.; R-1: 0800-0810 hrs.
7.0–9.5	R-2		1.6'	64 - 0					R-2: 0815-0830 hrs.
9.5–12.0	R-3		1.5'	60 - 0				11.5'-11.8' extremely broken	R-3: 0835-0850 hrs.
12.0–14.5	R-4		2.1'	84 - 12				12.2'-12.5' extremely broken	R-4: 0855-0915 hrs.
14.5–17.0	R-5		2.0'	80 - 0					R-5: 0920-0935 hrs.
17.0–19.5	R-6		1.2'	48 - 12					R-6: 0940-0950 hrs.

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO BEV-01A-2011
SHEET 2 OF 2
DATE: START 7/9/11
END 7/9/11
O.G. ELEV. 1950.0

PROJECT NAME WVDOH Scour
COUNTY Randolph
PROJECT LOCATION Beverly Bridge
STATION
OFFSET FROM CENTERLINE
NORTHING
EASTING

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
22.0	R-7		2.3'	92 - 0				20.1 CLAYSTONE, dark gray, soft to average hard, highly weathered, v inten to v thinly bedded (RD= 75°), very closely to widely spaced fractures (RD= 0° to 25°), RQD=0% 1929.9	R-7: 0950-1005 hrs.
24.5	R-8		2.5'	100 - 0				22.0'-24.5' diagonal fractures	R-8: 1005-1015 hrs.
27.0	R-9		2.5'	100 - 0				27.0 1923.0	R-9: 1015-1030 hrs. Completed drilling at 1030 hrs. 7/9/2011
								Bottom of borehole at 27.00 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

Figure 49 – Boring Log 2 for Beverly Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO **BEV-02A-2011**
SHEET **1** OF **2**
DATE: START **7/8/11**
END **7/8/11**
O.G. ELEV. **1944.0**

PROJECT NAME **WVDOH Scour** COUNTY **Randolph**
PROJECT LOCATION **Beverly Bridge**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______
STATE RT. NO. **37/8** SECT. ______ SEGMENT ______ OFFSET ______
INSPECTOR **Jared Govi** DRILLERS NAME/COMPANY **Jim Hopkins/L.G. Hetager Drilling, Inc.**
EQUIPMENT USED **HH 240**
DRILLING METHODS **Casing Advancer/SPT/NX Wireline Split Barrel**
CASING: SIZE: **4.25"** ; DEPTH: **9.7'** ; WATER: DEPTH: ______ TIME: ______ DATE: ______
CHECKED BY: **NJW** ; DATE: **8/26/13** DEPTH: **dry** TIME: **15:00** DATE: **7/8/11**

Checked = No Recorded Water Reading Measurements [X]

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
0.0–0.6	A-1				-			0.6 ASPHALT 1943.4	Started hole at 0905 hrs. 7/8/2011
0.6–9.7	A-2				-			UNSAMPLED	Cloudy/70 degrees Used casing due to surrounding materials ~Bottom of Abutment at 1926' Lost water at 3.5' (never returned) Stream pH=8.3
9.7–12.0	R-1		2.3'	100 - 26				9.7 Auger and Spoon refusal at 9.7' 1934.3 Fine grained SILTSTONE, dark gray, hard, slightly to freshly weathered, v thinly to thinly bedded (RD= 75°), indistinctly to widely spaced fractures (RD= 0° to 15°), RDQ=32%	Started coring at 1030 hrs. R-1: 1030-1100 hrs.
12.0–14.0	R-2		1.8'	90 - 60					R-2: 1100-1125 hrs.
14.0–17.0	R-3		2.2'	73 - 17					R-3: 1135-1200 hrs.
17.0–	R-4		3.9'	78 - 44				17.0 1927.0 CLAYSTONE, dark gray, soft to average hard, highly to slightly weathered, v inten to thinly bedded (RD= 75°), extremely closely to widely spaced fractures (RD= 0° to 15°), RQD=37% 17.9'-19.0' diagonal fracture	R-4: 1205-1225 hrs.

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

PROJECT NAME **WVDOH Scour** COUNTY **Randolph**
PROJECT LOCATION **Beverly Bridge**
STATION OFFSET FROM CENTERLINE
NORTHING EASTING

BORING NO **BEV-02A-2011**
SHEET **2** OF **2**
DATE: START **7/8/11**
END **7/8/11**
O.G. ELEV. **1944.0**

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
22.0				78 - 44				CLAYSTONE, dark gray, soft to average hard, highly to slightly weathered, v inten to thinly bedded (RD= 75°), extremely closely to widely spaced fractures (RD= 0° to 15°), RQD=37% *(continued)*	
27.0	R-5		3.8'	76 - 30				17.0'-17.4' very broken 20.2'-20.8' extremely broken 22.7'-22.9' diagonal fracture 23.3'-24.3' broken with diagonal fracture 24.3'-25.9' extremely broken 27.0 1917.0	R-5: 1230-1300 hrs. Completed drilling at 1300 hrs.
								Bottom of borehole at 27.00 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

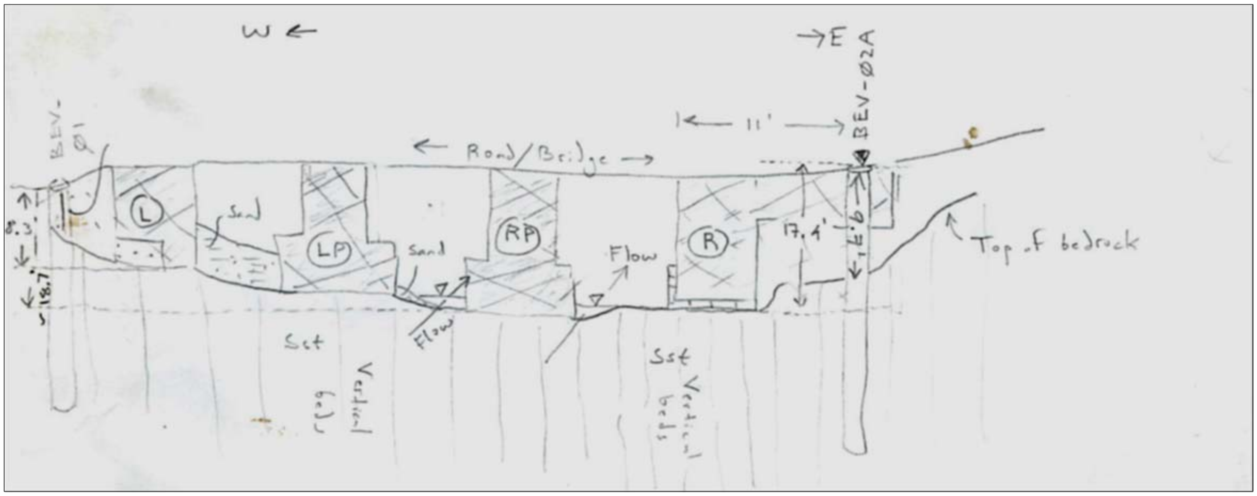

Figure 50 -- Field sketch of Beverly Bridge, looking downstream.

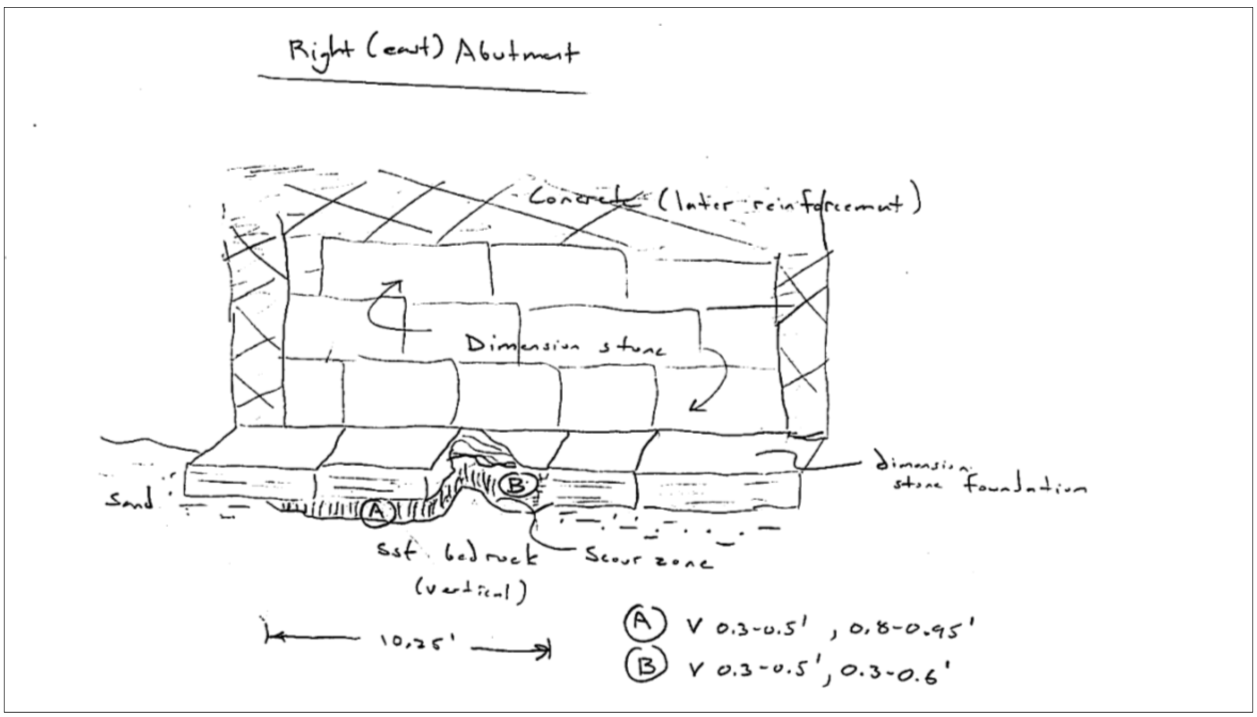

Figure 51 -- Field sketch of Beverly Bridge, looking at right abutment.

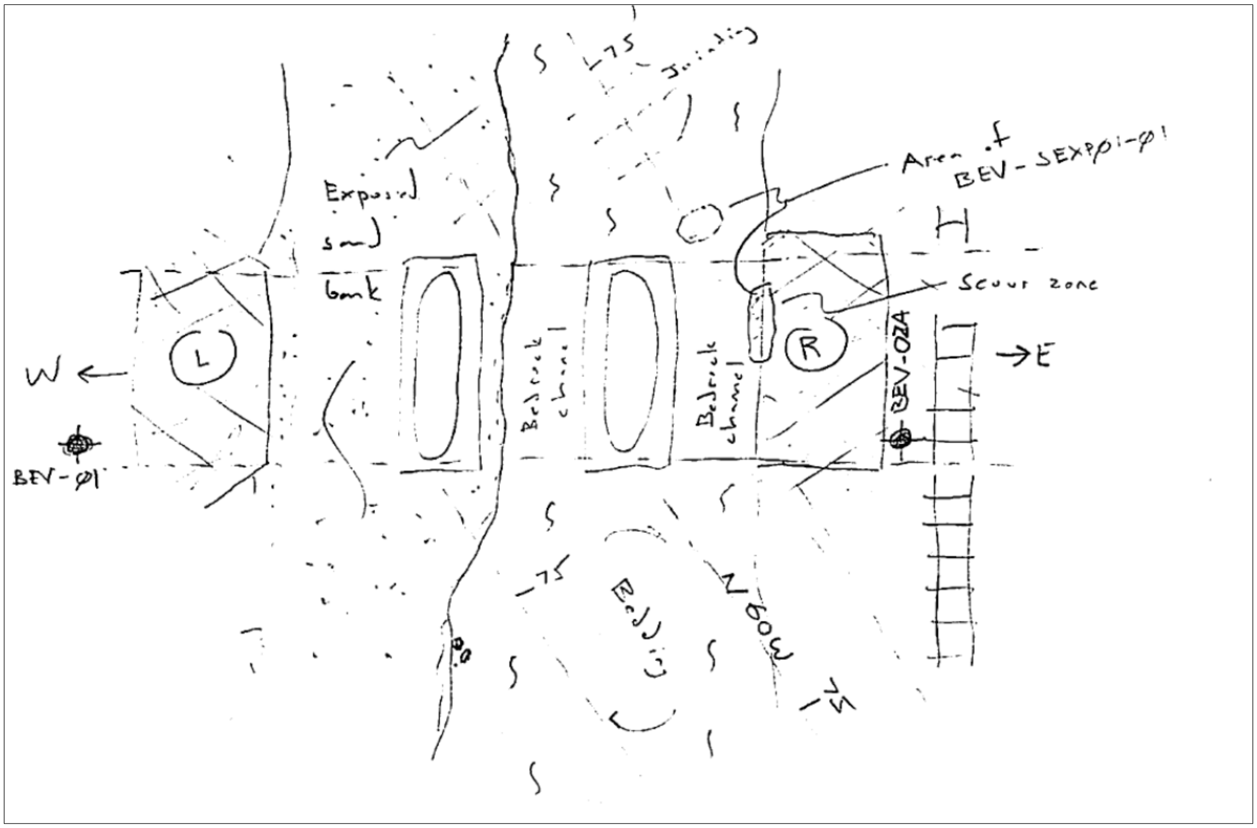

Figure 52 -- Field sketch of Beverly Bridge, overhead view.

Lab Results

Modified Slake Durability and GSN*

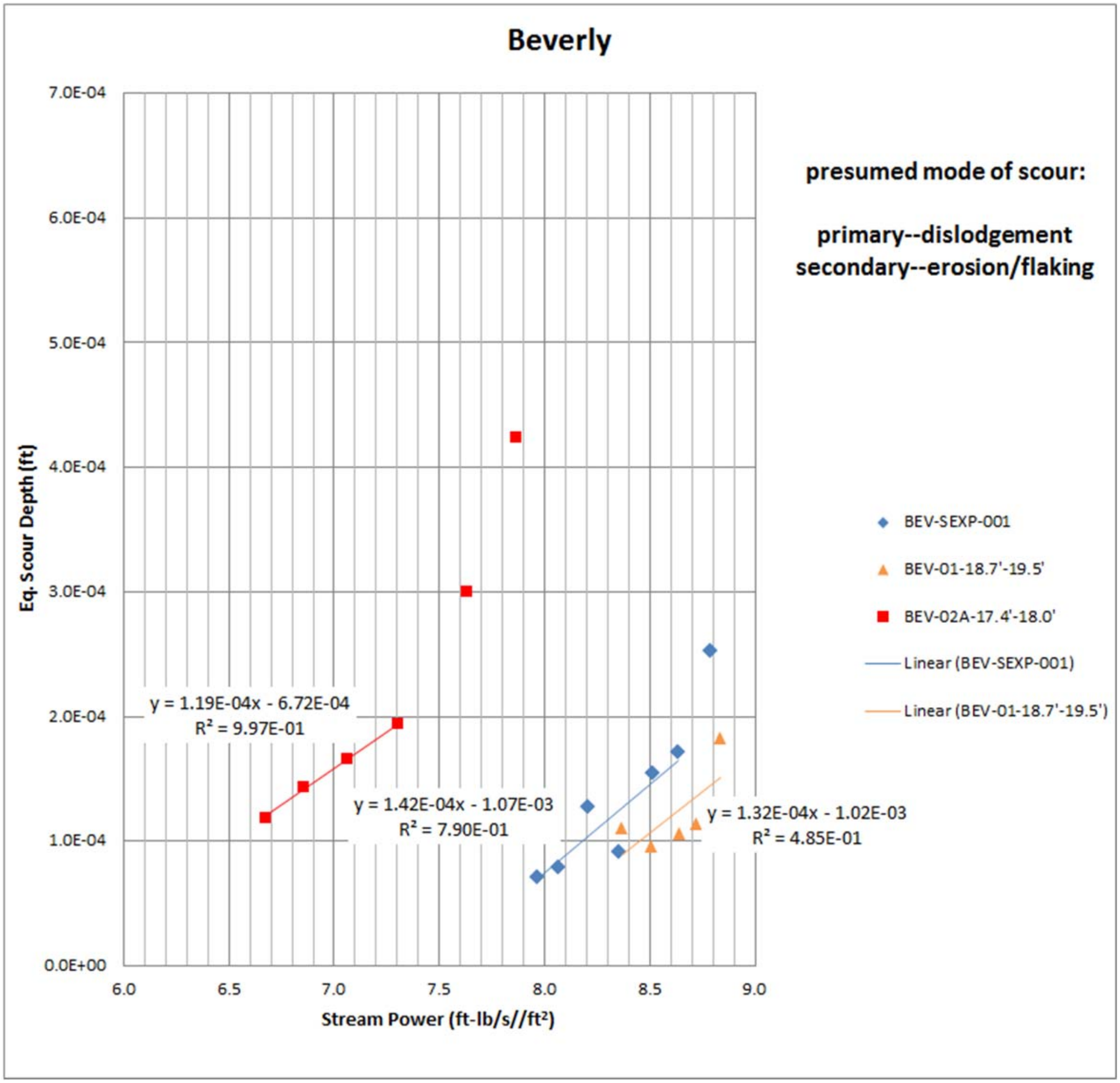

Figure 53 -- Data from modified slake durability tests and calculated values of GSN* and R^{*2} for Beverly Bridge samples.

11 – Coon Creek

Field Results

Geology.

At the Coon Creek Bridge the bridge abutments are set in sandstone belonging to the Mississippian-age Princeton Formation. On the state geologic map of West Virginia (Cardwell et al, 1968, 1986) the Princeton Formation is combined with the overlying Bluestone Formation. Both formations are part of the Mauch Chunk Group. Together these two formations range in thickness from 450-800 feet and occupy an approximate total area of 465 square miles over 12 counties in southeastern West Virginia (Cardwell et al, 1968, 1986).

At the bridge site surface exposure of the Princeton Formation in and directly adjacent to the channel consists of a moderately to extremely weathered (saprolitic), coarse- to very-coarse grained sandstone to vuggy, quartz pebble conglomerate. In the borehole COO-001-2011, adjacent to the right abutment, bedrock in the target interval was logged as medium- to coarse-grained sandstone, average hardness, slightly to freshly weathered, and very thinly to thinly bedded. RQD for this interval was 100%. In the borehole COO-002-2011, adjacent to the left abutment, bedrock at the target elevation was more similar to the bedrock exposed beneath the bridge: iron-stained, medium- to coarse-grained sandstone grading downward to very coarse-grained and conglomeratic. A vertical, clay-filled fracture bisected the core. RQD for this interval was 0%.

Rock Quality Designation (RQD).

RQD in COO-001-2011 for the target depth interval was 100%. In the borehole COO-002-2011, adjacent to the left abutment, RQD for this interval was 0%.

Scour Mode and Magnitude

Areas of scour at the Coon Creek bridge are shown on the field sketches below.

Figure 54 – Boring Log 1 for Coon Creek Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. COO-01-2011
SHEET 1 OF 1
DATE: START 6/26/11
END 6/26/11
O.G. ELEV. 2410.0

PROJECT NAME WVDOH Scour COUNTY Summers
PROJECT LOCATION Coon Creek Brigde
STATION ____ OFFSET FROM CENTERLINE ____
NORTHING ____ EASTING ____
STATE RT. NO. 1/4 SECT. ____ SEGMENT ____ OFFSET ____
INSPECTOR Nichole Wendlandt DRILLERS NAME/COMPANY Jim Hopkins/L.G. Hetager Drilling, Inc.
EQUIPMENT USED CME 45
DRILLING METHODS Hollow Stem Auger/SPT/NX Wireline Split Barrel
CASING: SIZE: ____ ; DEPTH: ____ ; WATER: DEPTH: ____ TIME: ____ DATE: ____
CHECKED BY: TLD ; DATE: 8/1/11 DEPTH: 5.0' TIME: 12:35 DATE: 6/26/11
Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
0.0–1.5	S-1	8 4 3	1.1'	73	-	gc - a-2-6	M	Clayey GRAVEL (gc), brown, moist, loose, homogeneous, (fill)	Started hole at 0955 hrs. 6/26/2011
1.5–3.0	S-2	4 6 11	1.5'	100	-	sc - a-2-6	M	2.0 … 2408.0 Clayey Medium To Coarse SAND (sc), brown, moist, medium dense, homogeneous, (alluvial)	Rain/70 degrees
3.0–4.5	S-3	2 5 11	1.5'	100	-	sc - a-2-6	M		~Bottom of Abutment at 2401' Sand trap used for all soil sampling
4.5–6.0	S-4	16 16 11	1.3'	87	-	sc - a-2-6	M	5.0 … 2405.0 Clayey SAND With Gravel (sc), brown, moist, dense to very dense, homogeneous, (residual sandstone)	Stream pH=8.9
6.0–6.6	S-5	10 50/0.1	0.6'	100	-	sc - a-2-6	M	6.6 … 2403.4	Started coring at 1108 hrs.
6.6–8.5	R-1		1.3'	68 / 26				Medium to coarse grained SANDSTONE, average hard to hard, slightly to freshly weathered, v thinly to thinly bedded (RD= 0° to 30°), extremely closely to widely spaced fractures (RD= 0° to 90°), RQD=63%	R-1: 1108-1114 hrs.
8.5–11.0	R-2		2.5'	100 / 100				6.9'-7.1' near vertical fracture 6.6'-7.4' vuggy 11.9'-21.1' healded vertical fracture 12.9'-13.5' near vertical fracture	R-2: 1118-1124 hrs.
11.0–13.5	R-3		2.3'	92 / 80				15.1'-15.4' gravel size quartz 17.4'-17.5' clay seam 17.7'-18.0' clay seam 18.8'-18.9' clay seam 19.3'-20.0' moderately weathered 19.3'-19.6' stained vertical fracture	R-3: 1143-1150 hrs.
13.5–18.5	R-4		4.9'	98 / 68					R-4: 1153-1203 hrs.
18.5–20.0	R-5		1.3'	87 / 0				20.0 … 2390.0	R-5: 1204-1210 hrs. Completed hole at 1245 hrs. 6/26/2011

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

Bottom of borehole at 20.00 feet.

Figure 55 – Boring Log 2 for Coon Creek Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **COO-02-2011**
SHEET **1** OF **1**
DATE: START **6/26/11**
END **6/26/11**
O.G. ELEV. **2405.0**

PROJECT NAME **WVDOH Scour** COUNTY **Summers**
PROJECT LOCATION **Coon Creek Bridge**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______
STATE RT. NO. **1/4** SECT. ______ SEGMENT ______ OFFSET ______
INSPECTOR **Nichole Wendlandt** DRILLERS NAME/COMPANY **Jim Hopkins/L.G. Hetager Drilling, Inc.**
EQUIPMENT USED **CME 45**
DRILLING METHODS **Hollow Stem Auger/SPT/NX Wireline Split Barrel**
CASING: SIZE: ______ ; DEPTH: ______ ; WATER: DEPTH: ______ TIME: ______ DATE: ______
CHECKED BY: **TLD** ; DATE: **8/1/11** DEPTH: **14.2'** TIME: **9:30** DATE: **6/26/11**

Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY (%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT
0.0–1.0	A-1				-		
1.0–1.1	S-1	50/0.1			-	gp - a-1-a	D
1.1–3.1	R-1		1.1'	55 - 55			
3.1–4.6	R-2		1.3'	87 - 87			
4.6–7.1	R-3		1.1'	44 - 16			
7.1–9.6	R-4		2.5'	100 - 0			
9.6–12.1	R-5		1.8'	72 - 40			
12.1–14.6	R-6		2.5'	100 - 80			
14.6–17.1	R-7		2.1'	84 - 76			
17.1–19.6	R-8		2.5'	100 - 68			

DESCRIPTION

UNSAMPLED

1.0 — 2404.0

1.1 Sandy GRAVEL (gp), brown, dry, very dense, homogeneous, (residual) — 2403.9

Medium to coarse grained SANDSTONE, reddish tan gray, average hard to hard, moderately to freshly weathered, v thinly to thinly bedded (RD= 0° to 30°), extremely closely to very widely spaced fractures (RD= 0° to 90°), RQD=51%

1.1'-17.2' poorly cemented
4.3'-4.6' healed vertical fracture
6.4'-6.9' vuggy with gravel sized quartz
7.3'-8.3' clay filled vertical fracture
8.6'-8.9' partially healed vertical fracture
8.6'-17.2' coarser grained, almost conglomerate

10.0'-10.2' very broken

14.2'-14.3' stained near vertical fracture

17.8'-18.1' vertical fracture

19.6 — 2385.4

Bottom of borehole at 19.60 feet.

REMARKS

Started hole at 0800 hrs. 6/26/2011

Cloudy/60 degrees

~Bottom of Abutment at 2398.4'

Lost water at 1.1', returned at 14.0'

Started coring at 0816 hrs.

R-1: 0816-0820 hrs.

R-2: 0823-0829 hrs.

R-3: 0831-0837 hrs.

R-4: 0840-0846 hrs.

R-5: 0859-0905 hrs.

R-6: 0907-0913 hrs.

R-7: 0913-0918 hrs.

R-8: 0920-0925 hrs.

Completed hole at 0940 hrs. 6/26/2011

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

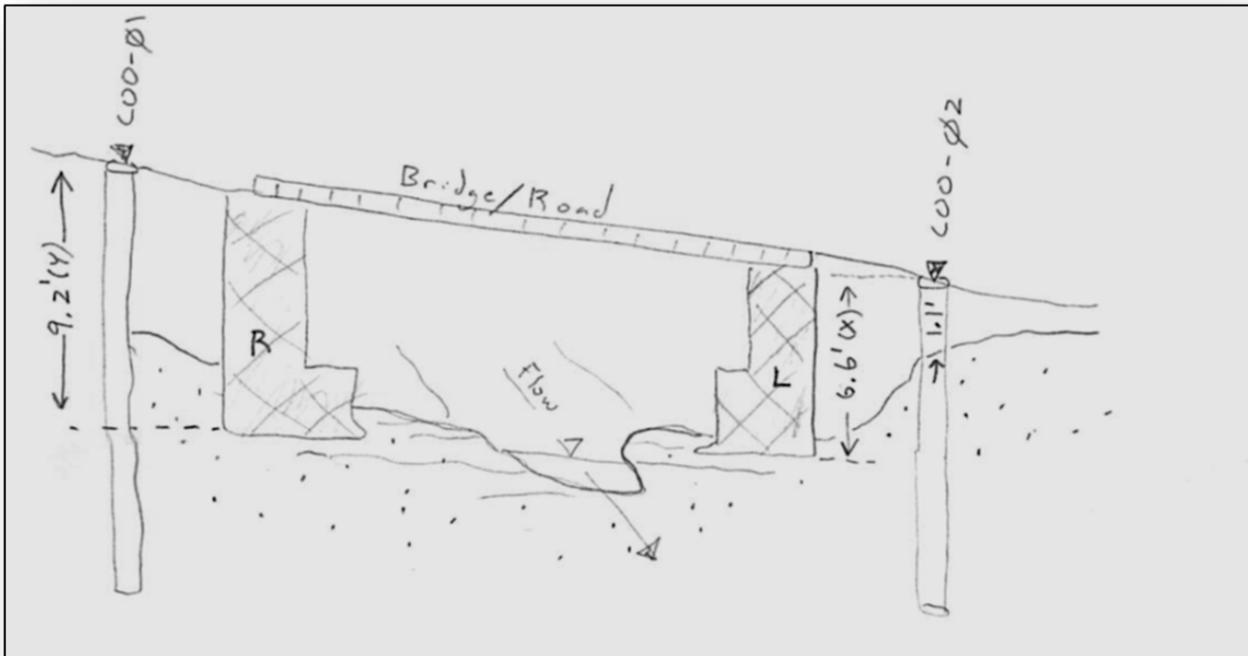

Figure 56 -- Field sketch of Coon Creek Bridge, looking upstream.

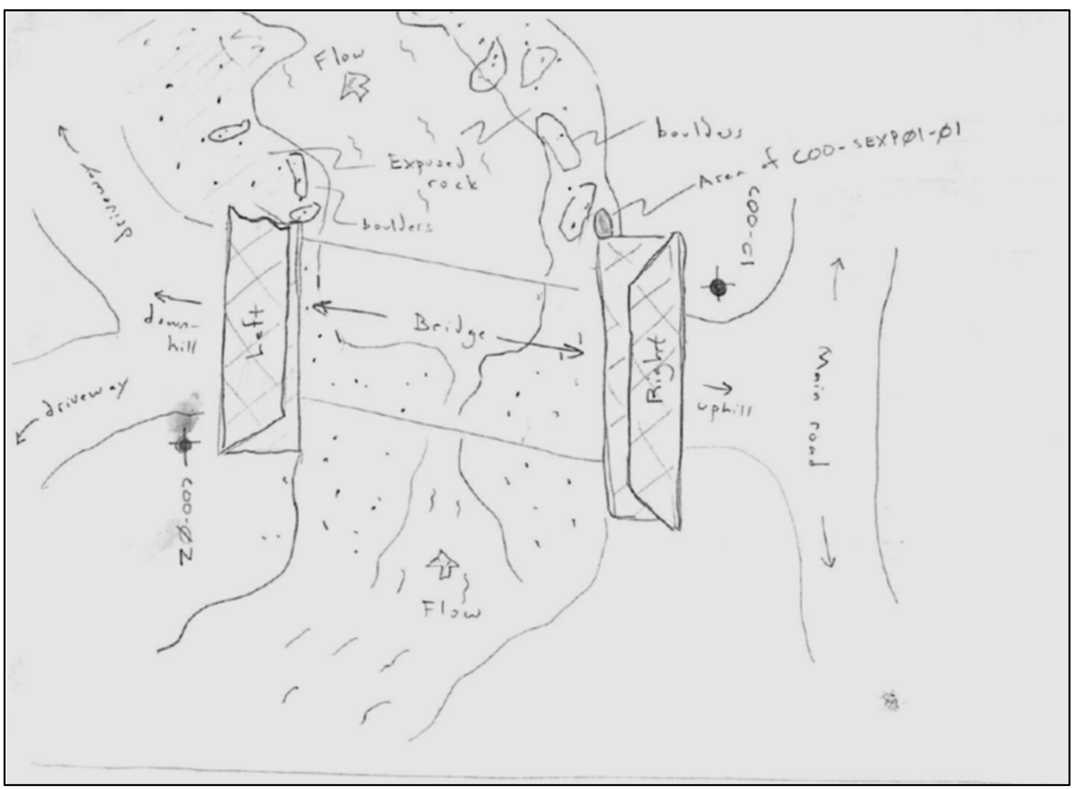

Figure 57 -- Field sketch of Coon Creek Bridge, overhead view.

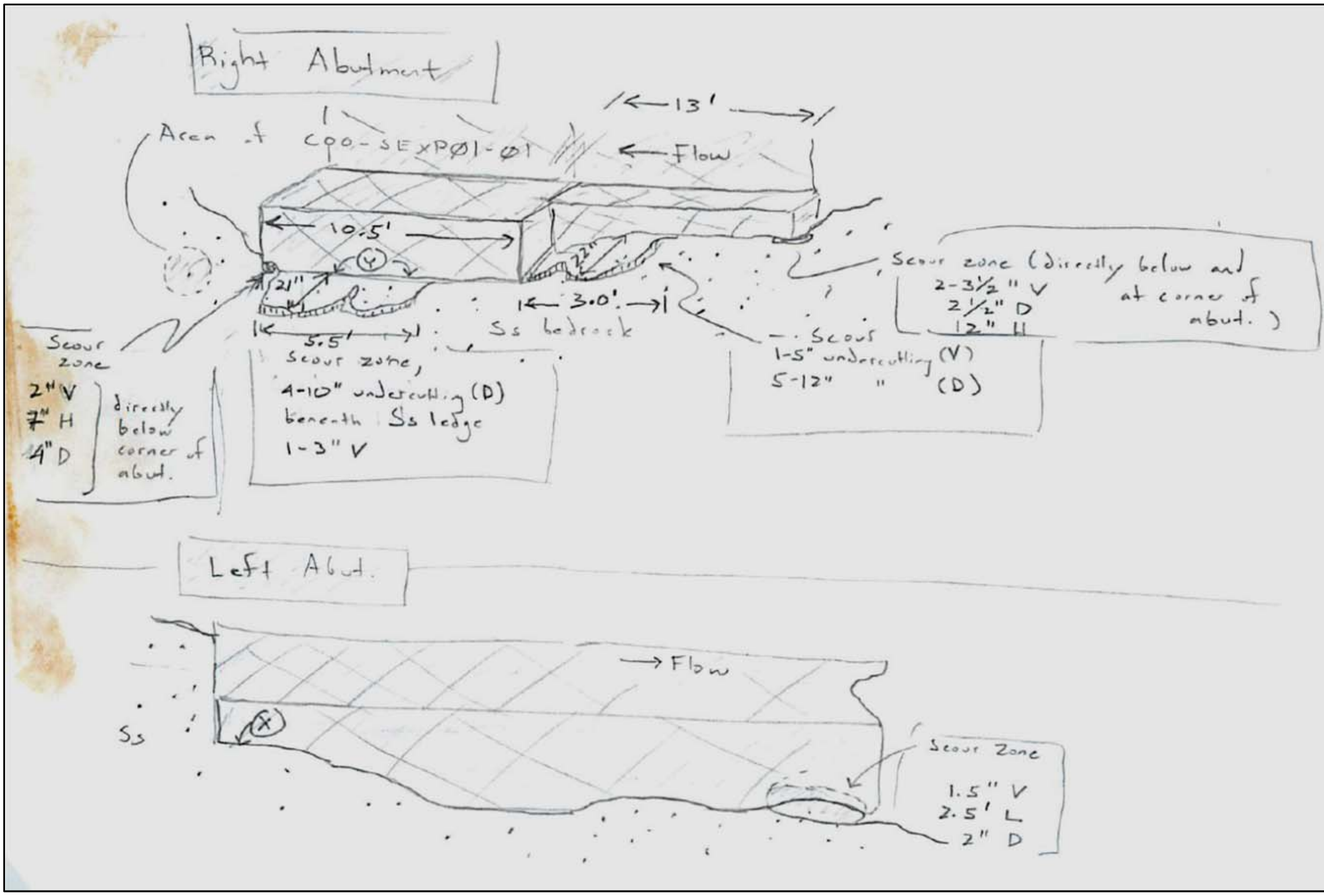

Figure 57 -- Field sketch of Coon Creek Bridge, looking at right abutment (top) and left abutment (bottom).

Lab Results

Modified Slake Durability and GSN*

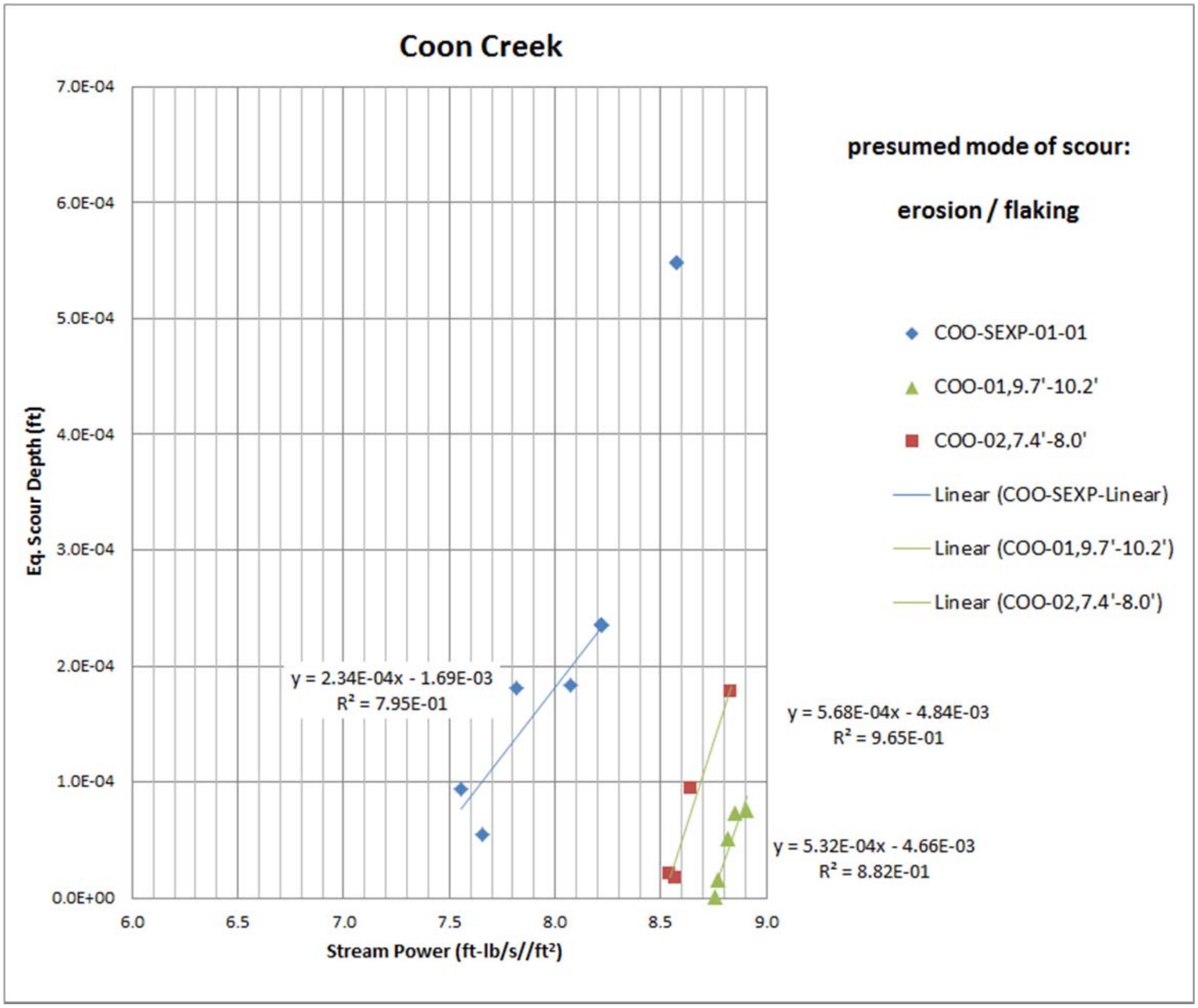

Figure 58 -- Data from modified slake durability tests and calculated values of GSN* and R^{*2} for Coon Creek Bridge samples.

12 – Bridge Fork

Field Results

Geology.

At the Bridge Fork bridge site the bridge abutments are set in sandstone belonging to the Pennsylvanian-aged Pottsville Group, which includes the Kanawha, New River, and Pocahontas Formations. Pottsville strata are predominantly sandstones, some of which are conglomeratic, along with some thin shale and coal beds. These rocks cover approximately 6,000 square miles over 12 counties, extending from Wayne and McDowell Counties in the southwestern and southern parts of West Virginia, respectively, to Preston and Tucker Counties in the north-central and northeastern portions of the state. The Pottsville Group is approximately 200 and 360 feet thick in the western and northeastern parts of its area, respectively, and thickens dramatically to over 3,800 feet in the southeastern part.

Based on surface exposure and the two boreholes drilled in June 2011, bedrock geology at the Bridge Fork bridge consists of fine- to medium-grained, micaceous and cross-bedded sandstone of average to above-average hardness. Bedding and fracturing are variable in core recovered from both BRF-01-2011, adjacent to the left abutment, and BRF-02-2011, adjacent to the right abutment. Bedrock exposed beneath both abutments is durable, thin- to medium-bedded (5-10 cm) sandstone. Cross-beds, approximately 3-cm thick, dip downstream at approximately 20° to 30°. Conjugate joint sets cross the stream channel upstream of the bridge but are not evident beneath the abutments.

Rock Quality Designation (RQD).

RQD of core recovered from BRF-01-2011 and BRF-02-2011 is 76% and 77%, respectively.

Scour Mode and Magnitude

Areas of scour at the Bridge Fork bridge are shown on the field sketches below.

Figure 59 – Boring Log 1 for Bridge Fork Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **BRF-01-2011**
SHEET **1** OF **1**
DATE: START **6/25/11**
END **6/25/11**
O.G. ELEV. **825.0**

PROJECT NAME **WVDOH Scour** COUNTY **Fayette**
PROJECT LOCATION **Bridge Fork Bridge**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______
STATE RT. NO. **60/21** SECT. ______ SEGMENT ______ OFFSET ______
INSPECTOR **Nichole Wendlandt** DRILLERS NAME/COMPANY **Jim Hopkins/L.G. Hetager Drilling, Inc.**
EQUIPMENT USED **CME 45**
DRILLING METHODS **Hollow Stem Auger/SPT/NX Wireline Split Barrel**
CASING: SIZE: ______ ; DEPTH: ______ ; WATER: DEPTH: ______ TIME: ______ DATE: ______
CHECKED BY: **TLD** ; DATE: **8/1/11** DEPTH: **5.0'** TIME: **10:00** DATE: **6/25/11**
Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT		DESCRIPTION	REMARKS
0.0 – 1.5	A-1				-				UNSAMPLED 1.5 Auger and Spoon refusal at 1.5' 823.5	Started hole at 0800 hrs. 6/25/2011 Cloudy/64 degrees
1.5 – 4.0	R-1		2.5'	100 - 92					Fine to medium grained SANDSTONE, gray, average hard to hard, moderately to freshly weathered, intensely to thinly bedded (RD= 0° to 30°), extremely closely to medium spaced fractures (RD= 0° to 60°), RQD=76%	~Bottom of Abutment at 817' Started coring at 0845 hrs.
4.0 – 6.5	R-2		2.4'	96 - 60						R-1: 0845-0849 hrs.
6.5 – 9.0	R-3		2.5'	100 - 80					micaceous stringers throughout	R-2: 0852-0856 hrs. R-3: 0859-0905 hrs.
9.0 – 14.0	R-4		4.9'	98 - 82					11.6' undulating fracture 13.1'-13.4' very broken and highly weathered 16.0'-18.0' moderately weathered	R-4: 0910-0920 hrs.
14.0 – 18.0	R-5		3.6'	90 - 65					18.0 807.0	R-5: 0925-0934 hrs. Completed hole at 1010 hrs. 6/25/2011
									Bottom of borehole at 18.00 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

Figure 60 – Boring Log 2 for Bridge Fork Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **BRF-02-2011**
SHEET **1** OF **1**
DATE: START **6/25/11**
END **6/25/11**
O.G. ELEV. **826.0**

PROJECT NAME **WVDOH Scour** COUNTY **Fayette**
PROJECT LOCATION **Bridge Fork Bridge**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______
STATE RT. NO. **60/21** SECT. ______ SEGMENT ______ OFFSET ______
INSPECTOR **Nichole Wendlandt** DRILLERS NAME/COMPANY **Jim Hopkins/L.G. Hetager Drilling, Inc.**
EQUIPMENT USED **CME 45**
DRILLING METHODS **Hollow Stem Auger/SPT/NX Wireline Split Barrel**
CASING: SIZE: ______ ; DEPTH: ______ ; WATER: DEPTH: ______ TIME: ______ DATE: ______
CHECKED BY: **TLD** ; DATE: **8/1/11** DEPTH: **1.0'** TIME: **12:05** DATE: **6/25/11**

Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
0.0 – 3.0	A-1				-			UNSAMPLED	Started hole at 1030 hrs. 6/25/2011 Cloudy/72 degrees ~Bottom of Abutment at 817'
3.0 – 3.1	S-1	50/0.1	0.1'	100	-	sc	M	3.0 (823.0) – 3.1 (822.9) Clayey SAND (sc), brown, moist, very dense, homogeneous, (residual)	Stream pH=8.8
3.1 – 4.0	R-1		0.9'	100 - 0		- a-2-6		Fine to medium grained SANDSTONE, gray, average hard to hard, slightly to freshly weathered, intensely to thinly bedded (RD= 0° to 30°), extremely closely to very widely spaced fractures (RD= 0° to 90°), RQD=77% 3.2'-3.3' undulating fracture 4.1'-4.3' stained vertical fracture 5.1'-5.2' very broken	Started coring at 1042 hrs. R-1: 1042-1045 hrs.
4.0 – 6.5	R-2		2.5'	100 - 40					R-2: 1047-1056 hrs.
6.5 – 9.0	R-3		2.5'	100 - 96					R-3: 1100-1108 hrs.
9.0 – 14.0	R-4		5.0'	100 - 86				9.2'-9.3' stained near vertical fracture 9.4'-9.5' very broken 13.2'-13.3' undulating fracture	R-4: 1120-1130 hrs.
14.0 – 19.0	R-5		5.0'	100 - 92				19.0 (807.0)	R-5: 1137-1150 hrs. Completed hole at 1240 hrs. 6/25/2011
								Bottom of borehole at 19.00 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

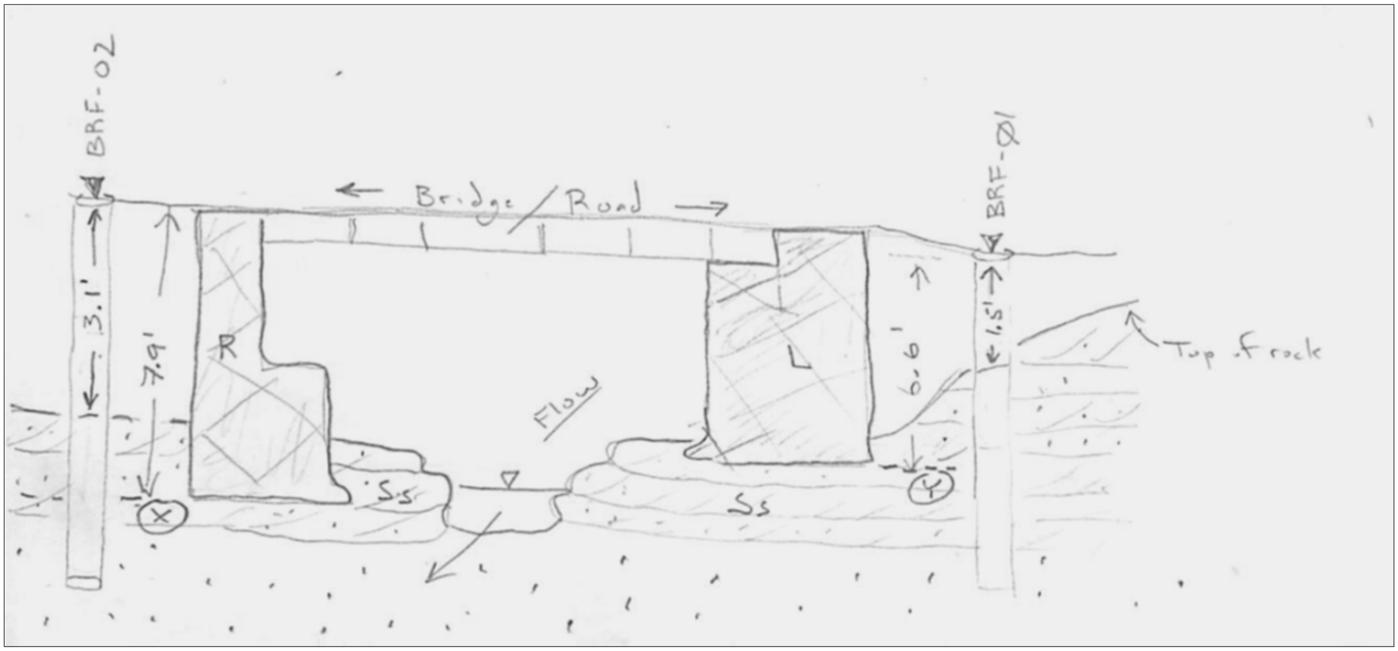

Figure 61 -- Field sketch of Bridge Fork Bridge, looking upstream.

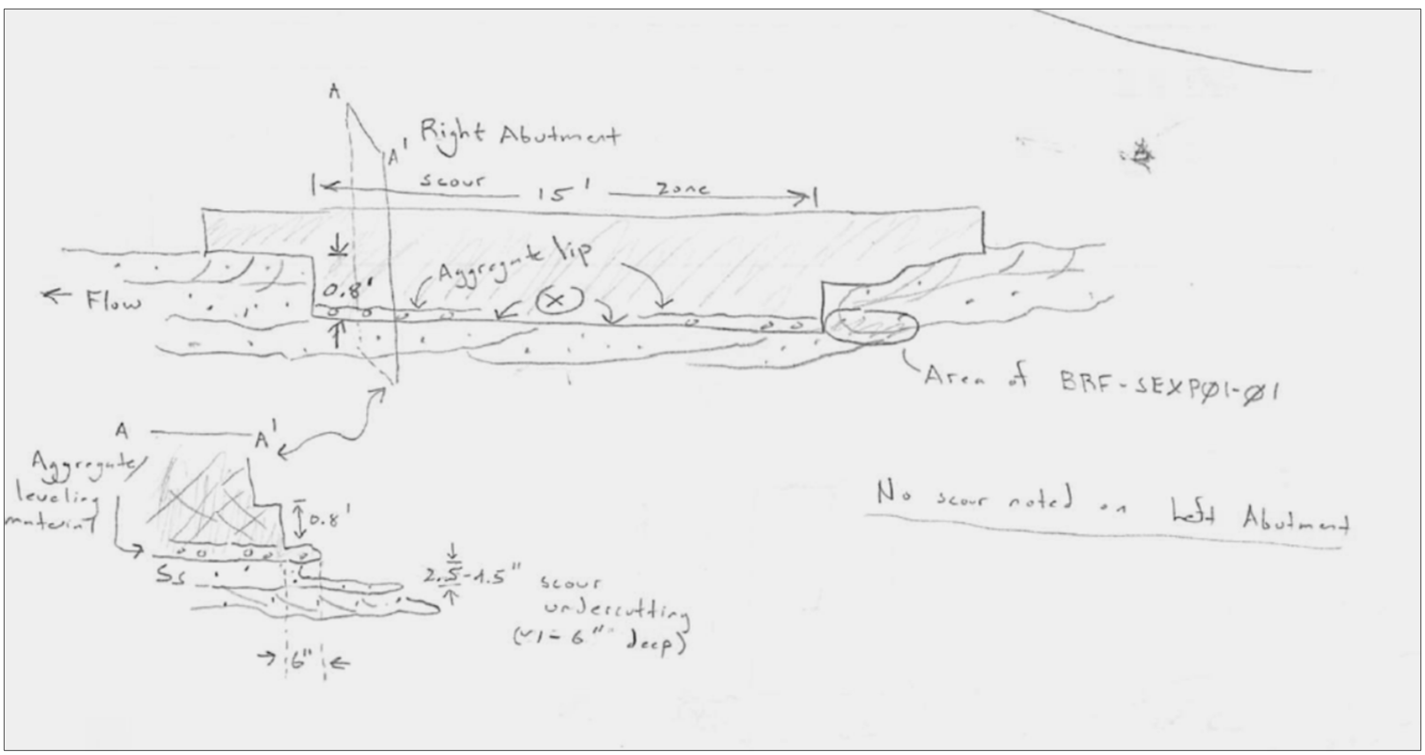

Figure 62 -- Field sketch of Bridge Fork Bridge, looking at right abutment head-on (top) and in cross-section (bottom left) .

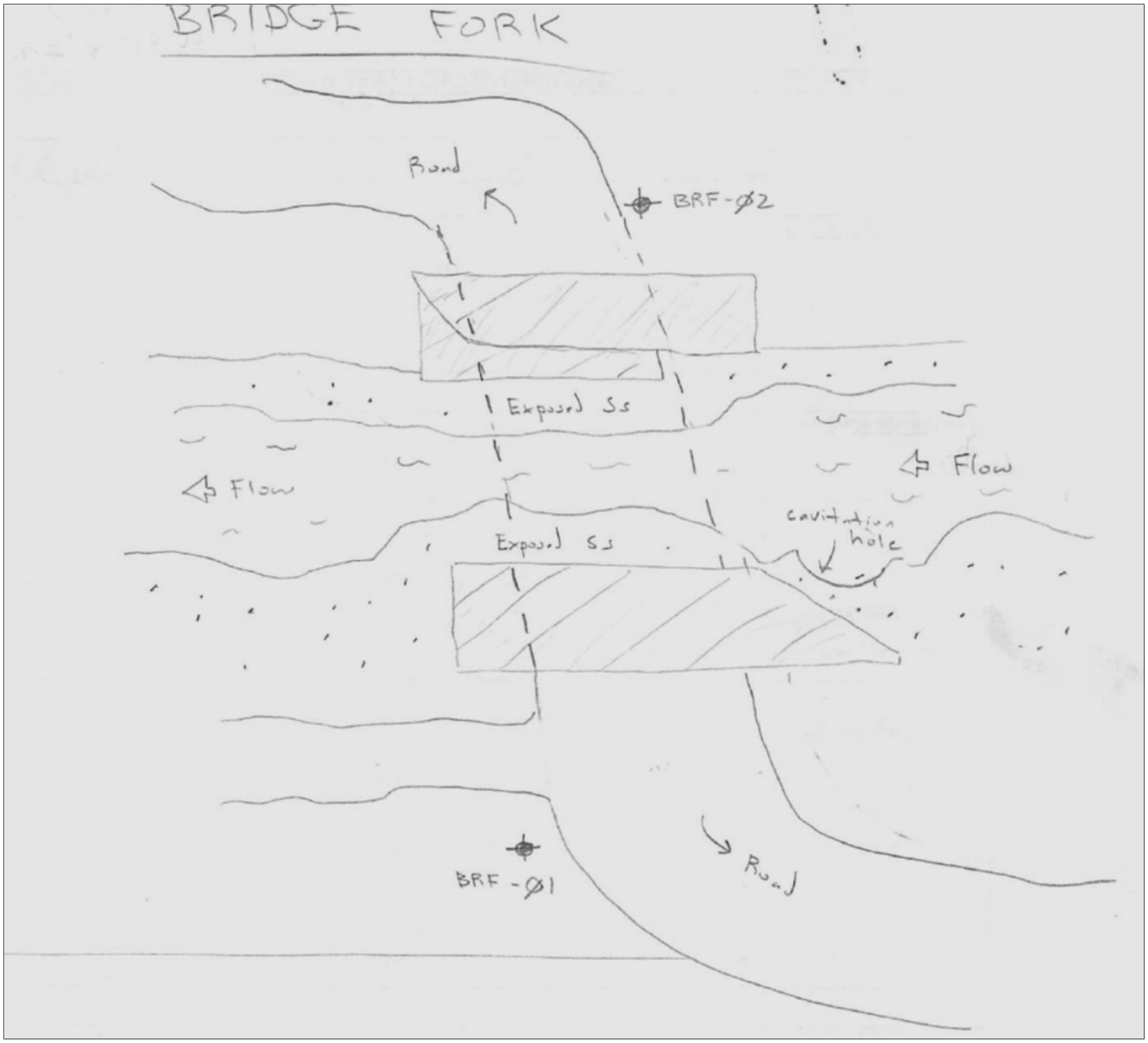

Figure 63 -- Field sketch of Bridge Fork Bridge, overhead view.

Lab Results

Modified Slake Durability and GSN*

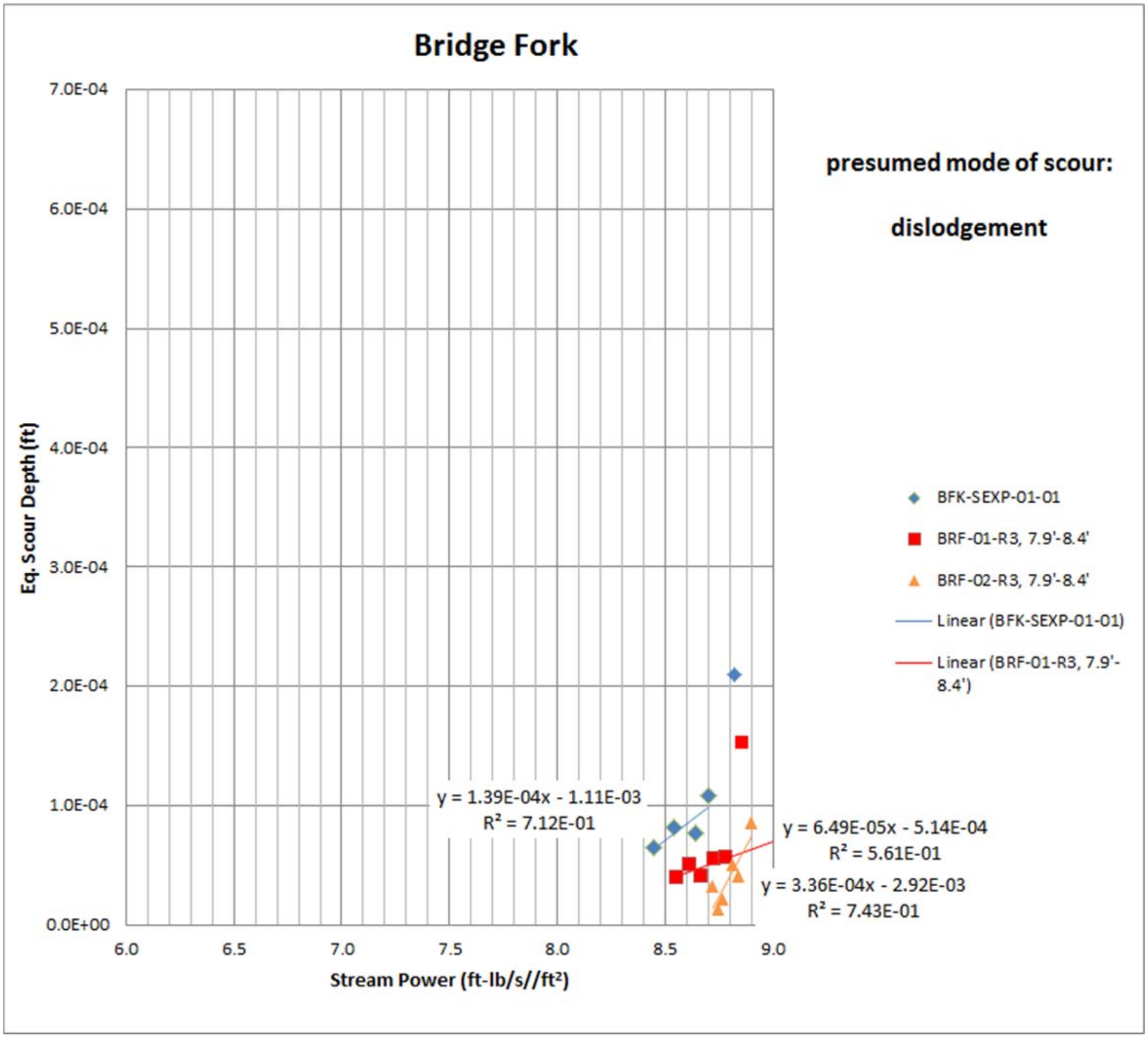

Figure 64 -- Data from modified slake durability tests and calculated values of GSN* and R^{*2} for Bridge Fork Bridge samples.

13 – Cucumber

Field Results

Geology.

At the Cucumber bridge site the bridge abutments are set in sandstone belonging to the Pennsylvanian-aged Pottsville Group, which includes the Kanawha, New River, and Pocahontas Formations. Pottsville strata are predominantly sandstones, some of which are conglomeratic, along with some thin shale and coal beds. These rocks cover approximately 6,000 square miles over 12 counties, extending from Wayne and McDowell Counties in the southwestern and southern parts of West Virginia, respectively, to Preston and Tucker Counties in the north-central and northeastern portions of the state. The Pottsville Group is approximately 200 and 360 feet thick in the western and northeastern parts of its area, respectively, and thickens dramatically to over 3,800 feet in the southeastern part.

Two boreholes were drilled at the Cucumber bridge in June 2011. Based on core extracted from CUC-01-2011, bedrock in the TZ consists of hard, gray, medium- to coarse-grained sandstone. Similar sandstone was encountered in CUC-02-2011 in the top 6 feet of the TZ; below this depth, clayey shale, siltstone and coal were present. High water at the bridge at the time of drilling covered the base of the bridge abutments. Soft, gray, thinly-bedded (0.6 cm) shale was present above stream level on the slope adjacent to the north abutment. It is not certain, however, if this rock represents in-place bedrock, as sandstone was encountered immediately below both abutments in the boreholes.

Rock Quality Designation (RQD).

RQD for the sandstone in CUC-01-2011 and CUC-02-2011 is 54% and 50%, respectively.

Scour Mode and Magnitude

Areas of scour at the Cucumber bridge are shown on the field sketches below. Table XX summarizes scour measurements at all of the bridge sites.

Figure 65 – Boring Log 1 for Cucumber Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. CUC-01-2011
SHEET 1 OF 2
DATE: START 7/6/11
END 7/6/11
O.G. ELEV. 1540.0

PROJECT NAME WVDOH Scour COUNTY McDowell
PROJECT LOCATION Cucumber Bridge
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______
STATE RT. NO. 16 SECT. ______ SEGMENT ______ OFFSET ______
INSPECTOR Jared Govi DRILLERS NAME/COMPANY Jim Hopkins/L.G. Hetager Drilling, Inc.
EQUIPMENT USED HH 240
DRILLING METHODS Hollow Stem Auger/SPT/NX Wireline Split Barrel
CASING: SIZE: ______ ; DEPTH: ______ ; WATER: DEPTH: ______ TIME: ______ DATE: ______
CHECKED BY: NJW ; DATE: 8/1/11 DEPTH: 15.0' TIME: 10:50 DATE: 7/6/11
Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT		DESCRIPTION	REMARKS
0.0–7.0	A-1				-				UNSAMPLED	Started hole at 0810 hrs. 7/6/2011 Sunny/70 degrees ~Bottom of Abutment at 1528' Stream pH=8.3
7.0–8.5	S-1	3 / 9 / 23	0.9'	60	-	gp - a-1-a	M		7.0 — 1533.0 Sandy GRAVEL (gp), orange-brown, moist to wet, dense to very dense, homogeneous, (alluvial)	
8.5–10.0	S-2	5 / 18 / 22	1.0'	67	-	gp - a-1-a	M			
10.0–11.5	S-3	25 / 32 / 30	1.0'	67	-	gp - a-1-a	M			
11.5–13.0	S-4	7 / 22 / 22	0.7'	47	-	gp - a-1-a	W		11.5'-13.0' dark brown	
13.0–14.5	S-5	18 / 22 / 20	0.9'	60	-	gp - a-1-a	W			
14.5–15.7	S-6	30 / 33 / 50/0.2	1.2'	100	-	gp - a-1-a	W			Started coring at 0945 hrs.
15.7–17.0	R-1		1.3'	100 - 0					15.7 — 1524.3 Medium to coarse grained SANDSTONE, gray, hard, slightly weathered, thinly bedded (RD= 0° to 15°), closely to very widely spaced fractures (RD= 0° to 30°), RQD=54%	R-1: 0945-0955 hrs.
17.0–19.5	R-3		2.3'	92 - 24						R-2: 1000-1008 hrs.

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **CUC-01-2011**
SHEET **2** OF **2**
DATE: START **7/6/11**
END **7/6/11**
O.G. ELEV. **1540.0**

PROJECT NAME **WVDOH Scour** COUNTY **McDowell**
PROJECT LOCATION **Cucumber Bridge**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
22.0	R-4		2.5'	100 - 56				Medium to coarse grained SANDSTONE, gray, hard, slightly weathered, thinly bedded (RD= 0° to 15°), closely to very widely spaced fractures (RD= 0° to 30°), RQD=54% *(continued)*	R-3: 1013-1016 hrs.
26.0	R-5		4.0'	100 - 90				26.0 1514.0	R-4: 1022-1030 hrs. Completed hole at 1100 hrs. 7/6/2011
								Bottom of borehole at 26.00 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

Figure 66 – Boring Log 2 for Cucumber Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **CUC-02-2011**
SHEET **1** OF **2**
DATE: START **7/6/11**
END **7/6/11**
O.G. ELEV. **1535.0**

PROJECT NAME **WVDOH Scour** COUNTY **McDowell**
PROJECT LOCATION **Cucumber Bridge**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______
STATE RT. NO. **16** SECT. ______ SEGMENT ______ OFFSET ______
INSPECTOR **Jared Govi** DRILLERS NAME/COMPANY **Jim Hopkins/L.G. Hetager Drilling, Inc.**
EQUIPMENT USED **HH 240**
DRILLING METHODS **Hollow Stem Auger/SPT/NX Wireline Split Barrel**
CASING: SIZE: ______ ; DEPTH: ______ ; WATER: DEPTH: ______ TIME: ______ DATE: ______
CHECKED BY: **NJW** ; DATE: **8/26/13** DEPTH: **15.5'** TIME: **15:00** DATE: **7/6/11**

Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
0.0 – 7.0	A-1				-			UNSAMPLED	Started hole at 1145 hrs. 7/6/2011 Partly Cloudy/80 degrees ~Bottom of Abutment at 1523'
7.0 – 8.5	S-1	3 5 6	1.0'	67	-	gp - a-1-a	D	7.0 — 1528.0 Sandy GRAVEL (gp), black, dry, medium dense, homogeneous, (fill)	
8.5 – 10.0	S-2	8 22 27	1.0'	67	-	gp - a-1-a	D	8.7 — 1526.3 Sandy GRAVEL (gp), gray-brown to light brown, dry to moist, dense to very dense, homogeneous, (alluvial)	
10.0 – 11.5	S-3	25 22 17	1.1'	73	-	gp - a-1-a	D		
11.5 – 13.0	S-4	23 23 38	0.6'	40	-	gp - a-1-a	M		
13.0 – 13.3	S-5	50/0.3	0.3'	100	-	gp - a-1-a	M		
13.3 – 16.0	R-1		2.7'	100 - 48				13.3 — 1521.7 Medium grained SANDSTONE, gray, hard, freshly weathered, thinly bedded (RD= 0° to 10°), closely to widely spaced fractures (RD= 0° to 45°), RQD=50%	Started coring at 1345 hrs. R-1: 1345-1355 hrs.
16.0 – 18.5	R-2		2.2'	88 - 72					R-2: 1355-1400 hrs.
18.5 –	R-3		2.0'	80 - 32				19.5 — 1515.5	R-3: 1400-1410 hrs.

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **CUC-02-2011**
SHEET **2** OF **2**
DATE: START **7/6/11**
END **7/6/11**
O.G. ELEV. **1535.0**

PROJECT NAME **WVDOH Scour** COUNTY **McDowell**
PROJECT LOCATION **Cucumber Bridge**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
21.0								20.1 Clayey SHALE, gray, very soft, completely weathered, v inten laminated to thinly bedded (RD= 0° to 5°), very closely to widely spaced fractures (RD= 0° to 15°), RQD=0% *(continued)* 1514.9	
	R-4		5.0'	100 - 74				SILTSTONE, gray, soft to average hard, slightly to freshly weathered, thinly to medium bedded (RD= 0° to 10°), widely to very widely spaced fractures (RD= 0° to 5°), RQD=87%	R-4: 1410-1420 hrs. Completed hole at 1530 hrs. 7/6/2011
26.0								25.3 1509.7 COAL, black, soft, freshly weathered, thinly bedded (RD= 0° to 5°), closely spaced fractures (RD= 0° to 5°), RQD=0% 26.0 1509.0	
								Bottom of borehole at 26.00 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

Lab Results

Modified Slake Durability and GSN*

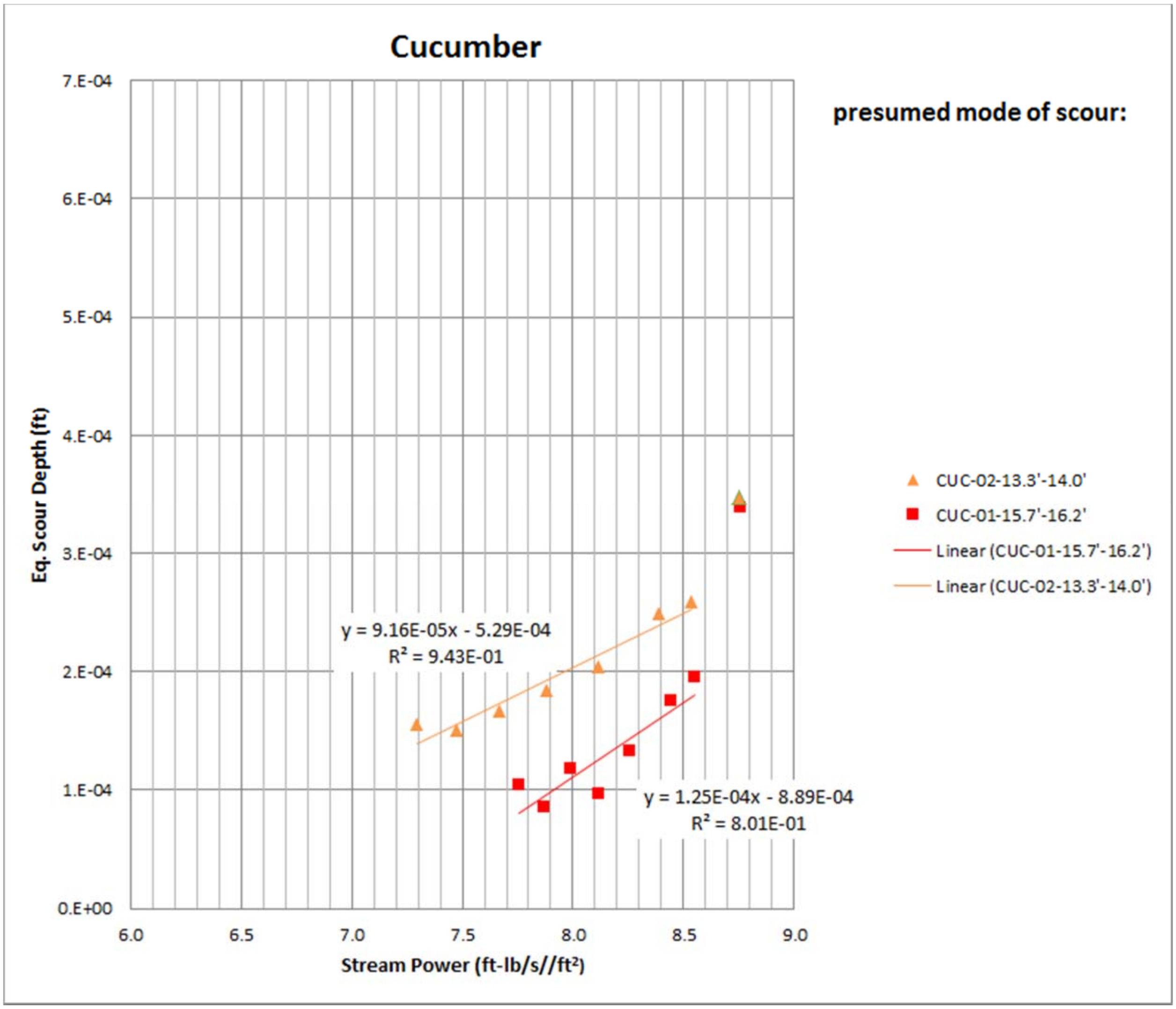

Figure 67 -- Data from modified slake durability tests and calculated values of GSN* and R^{*2} for Cucumber Bridge samples.

14 – Clear Fork

Field Results

Geology.

At the Clear Fork bridge site the bridge abutments are set in sandstone belonging to the Pennsylvanian-aged Pottsville Group, which includes the Kanawha, New River, and Pocahontas Formations. Pottsville strata are predominantly sandstones, some of which are conglomeratic, along with some thin shale and coal beds. These rocks cover approximately 6,000 square miles over 12 counties, extending from Wayne and McDowell Counties in the southwestern and southern parts of West Virginia, respectively, to Preston and Tucker Counties in the north-central and northeastern portions of the state. The Pottsville Group is approximately 200 and 360 feet thick in the western and northeastern parts of its area, respectively, and thickens dramatically to over 3,800 feet in the southeastern part.

Based on core extracted from the two boreholes drilled in June 2011, bedrock geology at the Clear Fork bridge consists of tan to gray, thinly-bedded, fine- to medium-grained sandstone of average to above average hardness.

Surface exposure of bedrock at this bridge site is limited to a small area below the right (southwest) abutment. This rock is durable, thin- to medium- bedded (3-15 cm) sandstone. Jointing in the surface rock, if present, is not obvious. However, numerous sub-polygonal shaped sandstone blocks present as stream bedload suggest some influence of jointing.

Rock Quality Designation (RQD).

RQD of core from CLE-01-2011 and CLE-02-2011 is 52% and 46%, respectively.

Scour Mode and Magnitude

Areas of scour at the Clear Fork bridge are shown on the field sketches below.

Figure 68 – Boring Log 1 for Clear Fork Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. CLE-01-2011
SHEET 1 OF 2
DATE: START 7/7/11
END 7/7/11
O.G. ELEV. 1160.0

PROJECT NAME WVDOH Scour COUNTY Wyoming
PROJECT LOCATION Clear Fork Bridge
STATION ____ OFFSET FROM CENTERLINE ____
NORTHING ____ EASTING ____
STATE RT. NO. 6 SECT. ____ SEGMENT ____ OFFSET ____
INSPECTOR Jared Govi DRILLERS NAME/COMPANY Jim Hopkins/L.G. Hetager Drilling, Inc.
EQUIPMENT USED HH 240
DRILLING METHODS Hollow Stem Auger/Casing Advancer/SPT/NX Wireline Split Barrel
CASING: SIZE: 4.25" ; DEPTH: 8.2' ; WATER: DEPTH: ____ TIME: ____ DATE: ____
CHECKED BY: NJW ; DATE: 8/1/11 DEPTH: 10.0' TIME: 15:30 DATE: 7/7/11

Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT		DESCRIPTION	REMARKS
0.0	A-1				-				UNSAMPLED	Started hole at 0730 hrs. 7/7/2011 Foggy/60 degrees ~Bottom of Abutment at 1146' Hydraulic line maintence from 0830-0940 hrs. Stream pH=7.6 Auger refusal at 6.0', switched to casing due to boulder
8.2									8.2 Auger and Spoon refusal at 8.2' 1151.8	
	R-1		1.8'	100 - 72				✓✓	Fine to medium grained SANDSTONE, tannish brown to gray, hard, freshly weathered, thinly bedded (RD= 0° to 15°), closely to widely spaced fractures (RD= 0° to 25°), RQD=52%	Started coring at 1030 hrs. R-1: 1030-1045 hrs.
10.0	R-2		1.4'	93 - 73				✓✓		R-2: 1050-1110 hrs.
11.5	R-3		2.5'	100 - 44				✓✓		R-3: 1120-1145 hrs.
14.0	R-4		2.5'	100 - 76				✓✓		R-4: 1150-1210 hrs.
16.5	R-5		5.0'	100 - 56				✓✓		R-5: 1215-1245 hrs.

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO.	**CLE-01-2011**
SHEET	**2** OF **2**
DATE: START	**7/7/11**
END	**7/7/11**
O.G. ELEV.	**1160.0**

PROJECT NAME **WVDOH Scour** COUNTY **Wyoming**
PROJECT LOCATION **Clear Fork Bridge**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H₂O CONTENT		DESCRIPTION	REMARKS
21.5				100 - 56					Fine to medium grained SANDSTONE, tannish brown to gray, hard, freshly weathered, thinly bedded (RD= 0° to 15°), closely to widely spaced fractures (RD= 0° to 25°), RQD=52% *(continued)* 20.8'-21.5' orange staining	
24.0	R-6		1.9'	76 - 0					24.0 1136.0	R-6: 1250-1310 hrs. Completed drilling at 1310 hrs.
									Bottom of borehole at 24.00 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

Figure 69 – Boring Log 2 for Clear Fork Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **CLE-02-2011**
SHEET **1** OF **2**
DATE: START **7/7/11**
END **7/7/11**
O.G. ELEV. **1160.0**

PROJECT NAME **WVDOH Scour** COUNTY **Wyoming**
PROJECT LOCATION **Clear Fork Bridge**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______
STATE RT. NO. **6** SECT. ______ SEGMENT ______ OFFSET ______
INSPECTOR **Jared Govi** DRILLERS NAME/COMPANY **Jim Hopkins/L.G. Hetager Drilling, Inc.**
EQUIPMENT USED **HH 240**
DRILLING METHODS **Casing Advancer/SPT/NX Wireline Split Barrel**
CASING: SIZE: **4.25"** ; DEPTH: **7.0'** ; WATER: DEPTH: ______ TIME: ______ DATE: ______
CHECKED BY: **NJW** ; DATE: **8/1/11** DEPTH: **10.1'** TIME: **16:30** DATE: **7/7/11**
Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT		DESCRIPTION	REMARKS
0.0 – 0.7	A-1				-				Asphalt (0.7 / 1159.3)	Started hole at 1330 hrs. 7/7/2011
0.7 – 7.0	A-2				-				UNSAMPLED (7.0 / 1153.0)	Sunny/80 degrees ~Bottom of Abutment at 1149'
7.0 – 8.5	S-1	1, 2, 2	0.3'	20	-	sc - a-2-6	W		Clayey SAND (sc), light brown, wet, very loose, homogeneous, (alluvial) (8.5 / 1151.5)	
8.5 – 10.0	S-2	5, 8, 12	1.3'	87	-	sp - a-1-b	W		Poorly Graded SAND (sp), light brown, wet, medium dense, homogeneous, (alluvial) (10.0 / 1150.0)	
10.0 – 11.5	S-3	5, 12, 32	1.5'	100	-	gp - a-1-a	W		Sandy GRAVEL (gp), light brown, wet, dense to very dense, homogeneous, (alluvial)	
11.5 – 13.0	S-4	15, 39, 40	1.1'	73	-	gp - a-1-a	W			
13.0 – 14.1	S-5	32, 42, 50/0.1	1.0'	91	-	gp - a-1-a	W		(14.1 / 1145.9)	
14.1 – 17.0	R-1		2.0'	69 - 14					Fine to medium grained SANDSTONE, light brown and orange to gray, average hard to very hard, slightly to freshly weathered, intensely to thinly bedded (RD= 0° to 10°), very closely to very widely spaced fractures (RD= 5° to 45°), RQD=46% 14.1'-14.6' very broken	Started coring at 1515 hrs. R-1: 1515-1527 hrs.
17.0 – 20.0	R-2		2.5'	83 - 60						R-2: 1530-1545 hrs.

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

PROJECT NAME WVDOH Scour | COUNTY Wyoming
PROJECT LOCATION Clear Fork Bridge
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______

BORING NO. CLE-02-2011
SHEET 2 OF 2
DATE: START 7/7/11
END 7/7/11
O.G. ELEV. 1160.0

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H₂O CONTENT		DESCRIPTION	REMARKS
22.0	R-3		2.0'	100 - 25					Fine to medium grained SANDSTONE, light brown and orange to gray, average hard to very hard, slightly to freshly weathered, intensely to thinly bedded (RD= 0° to 10°), very closely to very widely spaced fractures (RD= 5° to 45°), RQD=46% *(continued)*	R-3: 1545-1600 hrs.
25.0	R-4		2.8'	93 - 77					25.0 1135.0	R-4: 1600-1625 hrs. Completed hole at 1700 hrs.
									Bottom of borehole at 25.00 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

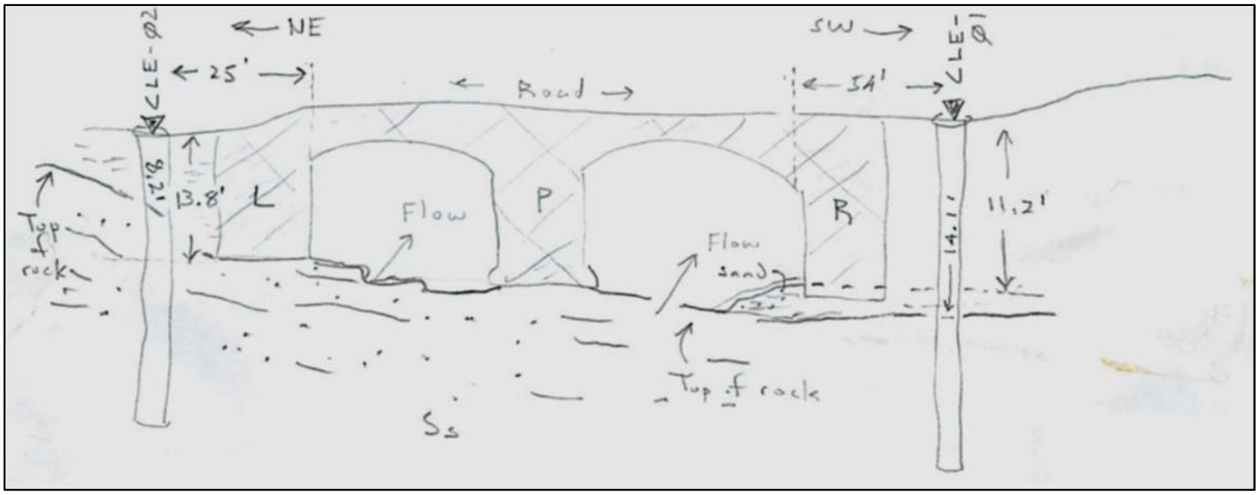

Figure 70 -- Field sketch of Clear Fork Bridge, looking downstream.

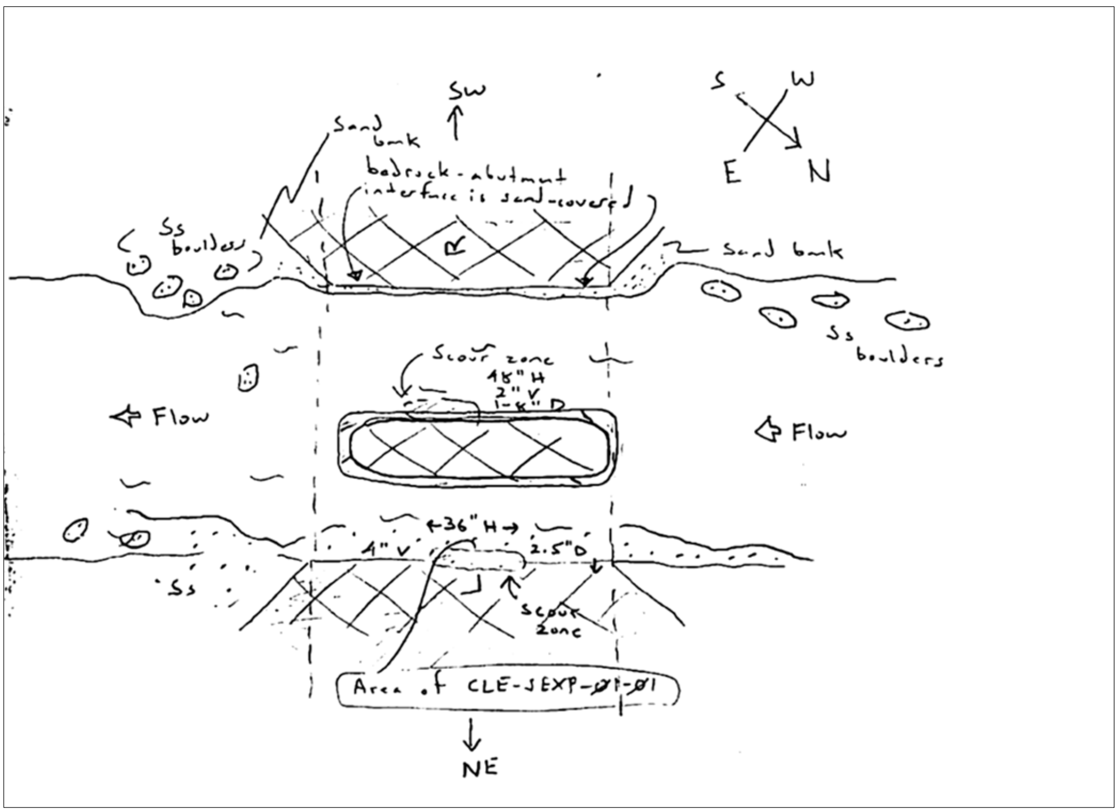

Figure 71 -- Field sketch of Clear Fork Bridge, overhead view.

Lab Results

Modified Slake Durability and GSN*

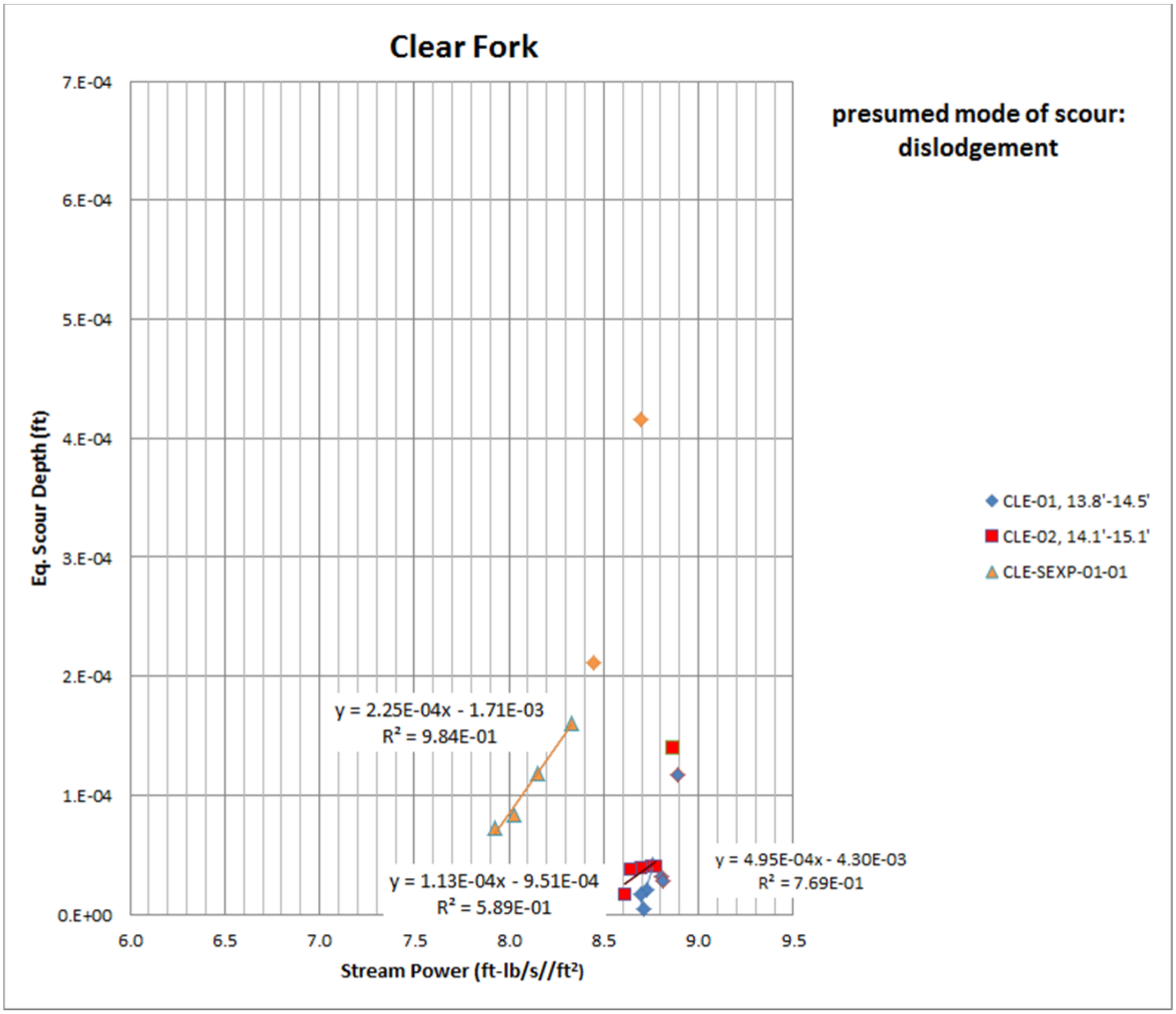

Figure 72 -- Data from modified slake durability tests and calculated values of GSN* and R^{*2} for Clear Fork Bridge samples.

15 – Mish Road

Field Results

Geology.

The Mish Road bridge site is located in the Valley and Ridge province. At this location the bridge abutments are set in limestone belonging to the Cambrian-age Conococheague Formation. This represents the only site in this investigation where carbonate rock (i.e., limestone or dolomite) was the foundation rock. Regionally, the Conococheague is a fine-grained limestone with some siliceous and dolomitic laminations. In West Virginia these rocks cover approximately 100 square miles and are restricted to Berkeley and Jefferson Counties in the eastern panhandle. The Conococheague is approximately 2,200 feet thick in this area.

Based on surface exposure and the two boreholes drilled in June 2011, bedrock geology at the Mish Road bridge consists of a tan to gray, thinly-bedded (1-3 cm), fine-grained (i.e., "micritic") limestone of hard to above average hardness. Bedrock exposed at the surface was very similar to bedrock in the TZ but varied in hardness from durable to non-durable. Fracturing in the rock core was variable, and jointing in the surface exposures was not evident. However, bedload in the stream included small (30 to 280 cm^3) limestone blocks suggesting the influence of joints.

Rock Quality Designation (RQD).

RQD in MIS-01-2011 and MIS-02-2011 is 39% and 35%, respectively.

Scour Mode and Magnitude

Areas of scour at the Mish Road bridge are shown on the field sketches below.

Figure 73 – Boring Log 1 for Mish Road Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **MIS-01-2001**
SHEET **1** OF **1**
DATE: START **7/12/11**
END **7/12/11**
O.G. ELEV. **580.0**

PROJECT NAME **WVDOH Scour** COUNTY **Berkeley**
PROJECT LOCATION **Mish Road**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______
STATE RT. NO. **24/3** SECT. ______ SEGMENT ______ OFFSET ______
INSPECTOR **Jared Govi** DRILLERS NAME/COMPANY **Jim Hopkins/L.G. Hetager Drilling, Inc.**
EQUIPMENT USED **HH 240**
DRILLING METHODS **Hollow Stem Auger/Casing Advancer/SPT/NX Wireline Split Barrel**
CASING: SIZE: **4.25"** ; DEPTH: **6.3'** ; WATER: DEPTH: ______ TIME: ______ DATE: ______
CHECKED BY: **NJW** ; DATE: **8/1/11** DEPTH: **4.0'** TIME: **9:05** DATE: **7/12/11**
Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
0.0 – 1.5	S-1	8 / 8 / 4	0.7'	47	-	gm - a-2-4	D	Silty GRAVEL With Sand (gm), brown, dry to wet, medium dense to very dense, homogeneous, (fill)	Started hole at 0700 hrs. 7/12/2011 Sunny/80 degrees
1.5 – 3.0	S-2	10 / 15 / 18	0.4'	27	-	gm - a-2-4	D		
3.0 – 4.5	S-3	8 / 11 / 6	0.0'	0	-	gm - a-2-4		3.0'-4.5' no recovery	Switched to casing at 1.0 due to boulders
4.5 – 6.0	S-4	6 / 5 / 12	0.3'	20	-	gm - a-2-4	W		Stream pH=8.2
6.0 – 6.3	S-5	50/0.3	0.2'	67	-	gm	W	6.3 573.7	Started coring at 0805 hrs.
6.3 – 7.0	R-1		0.7'	100 - 0		- a-2-4		LIMESTONE, light gray, average hard to hard, highly to slightly weathered, v thinly bedded (RD= 0° to 25°), very closely to widely spaced fractures (RD= 0° to 45°), RQD=39%	R-1: 0805-0813 hrs.
7.0 – 9.5	R-2		2.5'	100 - 16					R-2: 0813-0825 hrs.
9.5 – 12.0	R-3		2.5'	100 - 16					R-3: 0826-0835 hrs.
12.0 – 17.0	R-4		3.8'	76 - 68				17.0 563.0	R-4: 0836-0852 hrs. Completed hole at 0915 hrs. 7/12/2011
								Bottom of borehole at 17.00 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

Figure 74 – Boring Log 2 for Mish Road Bridge

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **MIS-02-2011**
SHEET **1** OF **2**
DATE: START **7/12/11**
END **7/12/11**
O.G. ELEV. **580.0**

PROJECT NAME **WVDOH Scour** COUNTY **Berkeley**
PROJECT LOCATION **Mish Road**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______
STATE RT. NO. **24/3** SECT. ______ SEGMENT ______ OFFSET ______
INSPECTOR **Jared Govi** DRILLERS NAME/COMPANY **Jim Hopkins/L.G. Hetager Drilling, Inc.**
EQUIPMENT USED **HH 240**
DRILLING METHODS **Hollow Stem Auger/Casing Advancer/SPT/NX Wireline Split Barrel**
CASING: SIZE: **4.25"** ; DEPTH: **6.1'** ; WATER: DEPTH: ______ TIME: ______ DATE: ______
CHECKED BY: **NJW** ; DATE: **8/1/11** DEPTH: **2.0'** TIME: **12:00** DATE: **7/12/11**
Checked = No Recorded Water Reading Measurements ☐

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY (%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
0.0–1.5	S-1	4 / 6 / 18	0.8'	53	-	gm - a-2-4	D	Silty GRAVEL With Sand (gm), brown, dry to moist, medium dense, homogeneous, (fill)	Started hole at 0940 hrs. 7/12/2011 Sunny/90 degrees
1.5–3.0	S-2	17 / 12 / 11	0.8'	53	-	gm - a-2-4	M		
3.0–4.5	S-3	12 / 18 / 33	1.4'	93	-	gp - a-1-a	W	3.5 576.5 Sandy GRAVEL (gp), light brown, wet, medium dense to very dense, homogeneous, (alluvial)	
4.5–6.0	S-4	20 / 10 / 6	1.1'	73	-	gp - a-1-a	W		
6.0–6.1	S-5	50/0.1	0.1'	100	-	gp	W	6.1 573.9 LIMESTONE, tannish gray to tan, average hard to hard, moderately to freshly weathered, thinly to medium bedded (RD= 0° to 15°), closely to very widely spaced fractures (RD= 0° to 25°), RQD=35%	Started coring at 1015 hrs.
6.1–7.0	R-1		0.9'	100 - 78		a-1-a			R-1: 1015-1025 hrs.
7.0–9.5	R-2		2.5'	100 - 76					R-2: 1030-1040 hrs.
9.5–12.0	R-3		0.9'	36 - 36					R-3: 1041-1050 hrs.
12.0–14.5	R-4		0.5'	20 - 0					R-4: 1050-1105 hrs.
14.5–17.0	R-5		0.0'	0 - 0				14.5'-17.0' void	R-5: 1105-1109 hrs.
17.0–19.5	R-6		0.0'	0 - 0				17.0'-19.5' void	R-6: 1109-1111 hrs.

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

FORM NO: D-481
(12/89)
REPRODUCE LOCALLY

ENGINEERS FIELD BORING LOG

BORING NO. **MIS-02-2011**
SHEET **2** OF **2**
DATE: START **7/12/11**
END **7/12/11**
O.G. ELEV. **580.0**

PROJECT NAME **WVDOH Scour** COUNTY **Berkeley**
PROJECT LOCATION **Mish Road**
STATION ______ OFFSET FROM CENTERLINE ______
NORTHING ______ EASTING ______

DEPTH (FT)	SAMPLE NO./ TYPE/CORE RUN	BLOWS/0.5 FT. ON SAMPLER	RECOVERY (Ft.)	RECOVERY(%) / RQD (%)	POCKET PENT/ TORVANE (TSF)	USCS / AASHTO	H_2O CONTENT	DESCRIPTION	REMARKS
	R-7		2.4'	96 / 84					R-7: 1111-1128 hrs.
22.0								22.0 558.0	
								Bottom of borehole at 22.00 feet.	

NOTE: DRAW STRATIFICATION LINES AT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES FOR THIS BORING LOCATION AND SHOW DEPTHS

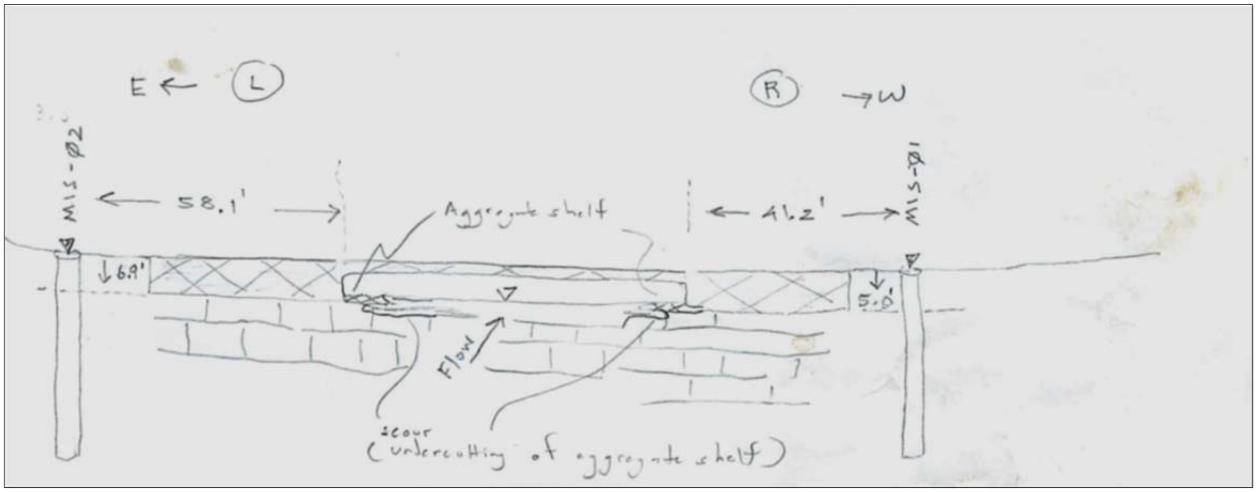

Figure 75 -- Field sketch of Mish Road Bridge, looking downstream.

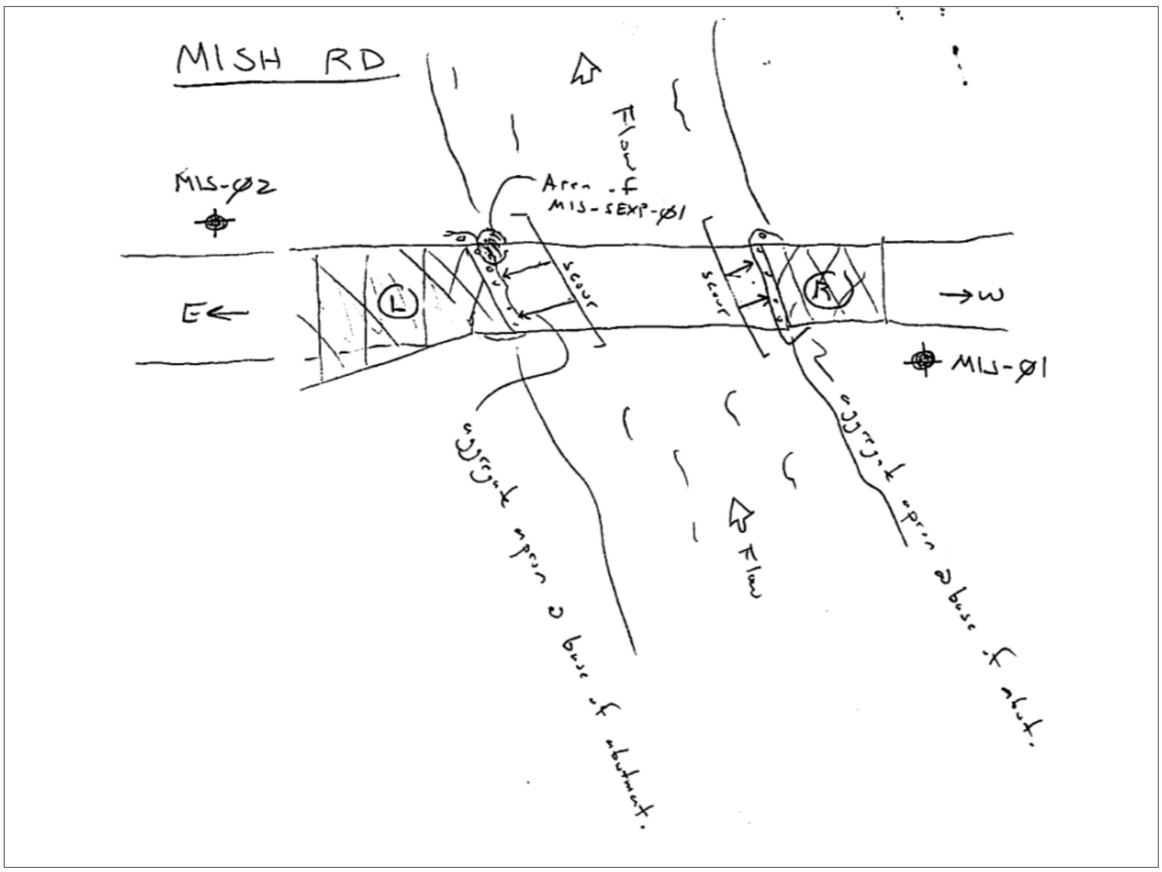

Figure 76 -- Field sketch of Bridge Fork Bridge, looking upstream.

Lab Results

Modified Slake Durability and GSN*

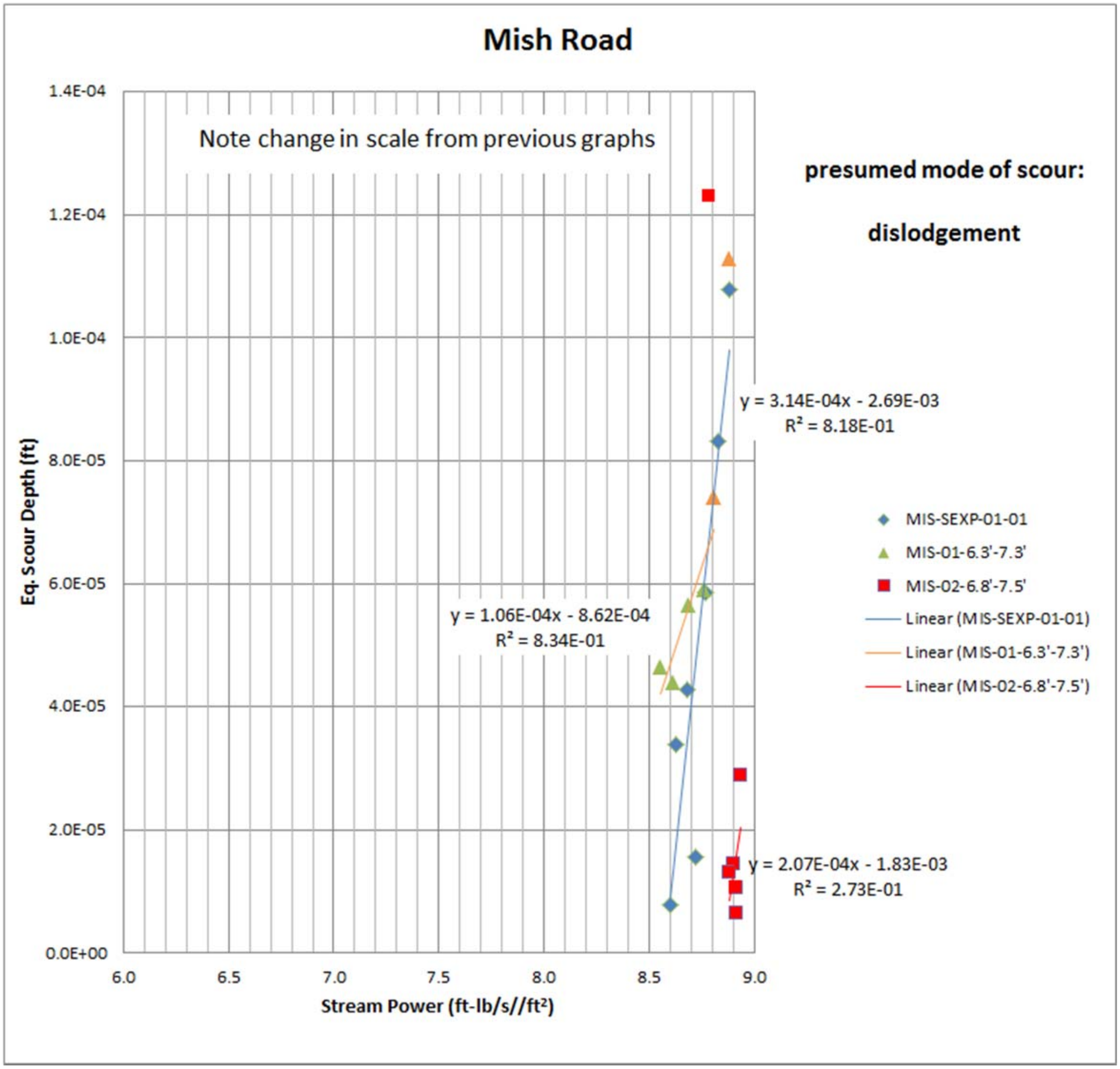

Figure 77 -- Data from modified slake durability tests and calculated values of GSN* and R^{*2} for Mish Road Bridge samples.

Summary of Lab Test Results

Table 1- Summary of Lab Test Results

Site	Dominant Rock Type(s)	Sample	GSN* (ft-lb/s^2/ft^2)	UCS (psi)
Audra	Sandstone	AUD-01-2011	2.83E-04	12,908
		AUD-02-2011	1.90E-04	11,086
		AUD-SEXP-001	2.41E-04	
		Median Value	**2.41E-04**	**11,997**
Beverly	Siltstone	BEV-01A-2011	1.32E-04	18,081
		BEV-02A-2011	1.19E-04	19,542
		BEV-SEXP-001	1.42E-04	
		Median Value	**1.32E-04**	**18,812**
Bridge Fork	Sandstone	BRF-01-2011	6.49E-05	17,472
		BRF-02-2011	3.36E-04	15,701
		BRF-SEXP-001	1.39E-04	
		Median Value	**1.39E-04**	**16,587**
Caldwell Run	Shale	CAL-01-2011	2.49E-04	6,752
		CAL-02-2011	1.86E-04	2,366
		CAL-SEXP-2011	1.77E-04	
		Median Value	**1.86E-04**	**4,559**
Clear Fork	Sandstone	CLE-01-2011	4.95E-04	11,086
		CLE-02-2011	1.00E-04	23,817
		CLE-SEXP-2011	2.00E-04	
		Median Value	**2.00E-04**	**17,451**
Coon Creek	Sandstone (weathered)	COO-01-2011	5.32E-04	2,081
		COO-02-2011	5.68E-04	11,723
		COO-SEXP-2011	2.34E-04	
		Median Value	**5.32E-04**	**6,902**
Cucumber	Sandstone	CUC-01-2011	1.25E-04	15,876
		CUC-02-2011	9.16E-05	12,095
		CUC-SEXP-2011	NA	
		Median Value	**1.08E-04**	**13,985**
Grassy Run	Sandstone	GRA-01-2011	8.00E-05	9,969
		GRA-02-2011	1.49E-04	12,195
		GRA-SEXP-2011	1.17E-04	
		Median Value	**1.17E-04**	**11,082**

Table 1 (cont.)

Site	Dominant Rock Type(s)	Sample	GSN* (ft-lb/s^2/ft^2)	UCS (psi)
Laurel Fork	Sandstone	LAU-01-2011	5.31E-04	11,231
		LAU-02-2011	2.06E-04	11,705
		LAU-SEXP-2011	1.42E-04	
		Median Value	**2.06E-04**	**11,468**
Leatherwood	Sandstone (shaley)	LEA-01-2011	2.49E-04	17,866
	Claystone	LEA-02-2011	1.76E-03	1,151
	Shale	LEA-SEXP01-01-2011	3.35E-05	
	Sandstone	LEA-SEXP01-02-2011	4.65E-05	
		Median Value (Shale/Claystone)	8.97E-04	**1,151**
		Median Value (Sandstone)	1.48E-04	**17,866**
Little Sandy	Sandstone	SAN-01-2011	8.04E-05	NA
	Shale	SAN-02-2011	1.50E-04	13,259
	Sandstone	SAN-SEXP-2011	7.15E-05	
		Median Value (Sandstone)	**7.60E-05**	**NA**
		Median Value (Shale)	**1.50E-04**	**13,259**
Mish	Limestone	MIS-01-2011	1.06E-04	4,071
		MIS-02-2011	2.07E-04	14,375
		MIS-SEXP-2011	3.14E-04	
		Median Value	**2.07E-04**	**9,223**
Paden Fork	Sandstone	PAD-01-2011	2.97E-05	9,744
		PAD-02-2011	7.84E-05	6,928
		PAD-SEXP-2011	8.94E-05	
		Median Value	**7.84E-05**	**8,336**
Ritter Park / 5th St	Sandstone	RIT-01-2011	1.49E-04	4,413
		RIT-02-2011	2.45E-05	6,910
		PAD-SEXP-2011	NA	
		Median Value	**8.68E-05**	**5,661**
Roaring Creek	Sandstone	ROA-01-2011	1.38E-04	10,430
		ROA-02-2011	1.00E-04	10,824
		ROA-SEXP-2011	2.42E-04	
		Median Value	**1.38E-04**	**10,627**

References cited in Appendix B

Cardwell, D.H., Erwin, R.B., and Woodward, H.P., 1968 [slightly revised 1986], Geologic Map of West Virginia: West Virginia Geological and Economic Survey).

Appendix C – Best Management Practices

Appendix C Contents

1. Calculating Cumulative Effective Stream Power

Characterizing the Cumulative Effective Stream Power, Ω, at a site can aid in understanding the flow conditions that may, over time, cause hydraulic scour, whether by gradual abrasion of degradable (i.e., non-durable) rock or by quarrying and plucking of durable, jointed rock. The determination of Cumulative Effective Stream Power has several prerequisites, including the identification of the threshold condition below which stream power is deemed not effective in promoting scour, characterizing flow conditions over time (i.e., obtaining historical stream gage data or synthesizing such data through watershed models), and defining the hydraulic conditions at the bridge site (i.e., preparing a model that describes the relationship between discharge and depth, and discharge and velocity). Detailed instructions on how to obtain stream flow data or generate a watershed runoff hydrograph, for example, are not provided in this document. Instead, this document focuses primarily on the computation of stream power once the aforementioned prerequisites are already complete.

Once the needed input data has been prepared, Cumulative Effective Stream Power can be computed for, (1) a past time period where actual flow conditions are known, (2) a time period (past or future) where probabilistic flow conditions are characterized, or (3) on an annualized basis using a probabilistic description of flow conditions, such that the Effective Stream Power associated with a certain intensity event is combined with the incremental probability of that event. In most cases, it is anticipated that Annual Cumulative Effective Stream Power (i.e., approach number 3) will be what is computed for sites and watersheds in West Virginia, due to the distribution of watershed sizes and corresponding stream gage data availability that was encountered in a 15 bridge survey of sites experiencing rock scour during a WVDOH-sponsored research project, RP-273 – "Criteria for Predicting Scour of Erodible Rock in West Virginia". Additional information related to the sites encountered in that project can be found in the corresponding final report document.

Threshold Condition

Section 3.6.22 of NCHRP Report 717, "Scour at Bridge Foundations on Rock," introduces the idea that a threshold condition, such as a critical velocity, shear stress, or stream power, or physical threshold such as bank-full or channel-forming discharge, be exceeded before scour can occur. Thus, the first decision to be made when computing Cumulative Effective Stream Power is what threshold condition to select. Since the threshold condition that is set defines the condition below which no work is expended in eroding the streambed, a detailed characterization of the channel cross section may be needed to identify the flow depth associated with water contact with rock that underlies piers and abutments. Likewise, field inspection of the site in question may aid in identifying a suitable exclusionary flow rate below which no scour is assumed to occur. As illustrated by Figure 1, flow depth may need to rise considerably above typical flow conditions before water is in contact with the rock beneath a particular bridge's abutment.

Figure 1- Photograph of abutment at Grassy Run project site. Water levels must rise before stream power begins to accumulate on the rocks under the abutment.

Where Cumulative Effective Stream Power is being estimated for a site that does not yet have a bridge, a more generalized approach may be necessary to define the threshold condition. For example, setting the threshold equal to the peak flow rate associated with the two year storm, coupled with iterative adjustments to the threshold as additional information becomes available.

Time-Series Flow Data

If the site under evaluation has historical flow data available as a time-series data set (i.e., stream gage records), then this will serve as a preferred and definitive characterization of what flows have occurred at the site over time. Stream gage data is typically available only for larger watersheds, although some smaller watersheds may occasionally have stream records available. The US Geological Survey makes stream gage data available online at the National Water Information System website (http://maps.waterdata.usgs.gov/mapper/index.html).

In cases where the actual flow rates over time are known, it will be unnecessary to undertake the procedures described later to synthesize flows through generating runoff hydrographs and probabilistically integrating the effect of hypothetical storms over time. Instead, the actual effect of a five year storm (for example) can be directly computed, and when prediction of future conditions is desired, the probability of such a storm in any given year can be accounted for.

The determination of stream power from long term data is described in NCHRP Report 717 section 3.6.2.3.

Runoff Hydrograph

For the types of watersheds and bridge sites generally present in West Virginia, historical flow data will usually not be present. Therefore, it is necessary to use alternate means to describe the temporal distribution of stream flows at a given site. This can be accomplished in a variety of ways, one of the simplest of which may be to determine the flood-frequency discharge using USGS Scientific Investigation Report 2010-5033 regression equations. The result of this method will be the *peak* discharge for a watershed during storms of a given return period frequency (e.g., 2-yr, 5-yr, 10-yr, etc.). The methods described in NCHRP Report 717 compute stream power on the basis of *average* daily flow, and so if regression equations are used as a basis for subsequent stream power computation, some estimation of the relationship between peak flow and average daily flow will need to be made.

Alternately, one could utilize a generalized unit hydrograph, such as the NRCS dimensionless unit hydrograph shown in Figure 2, in order to characterize the flow rate at any time during the storm event relative to the peak flow rate.

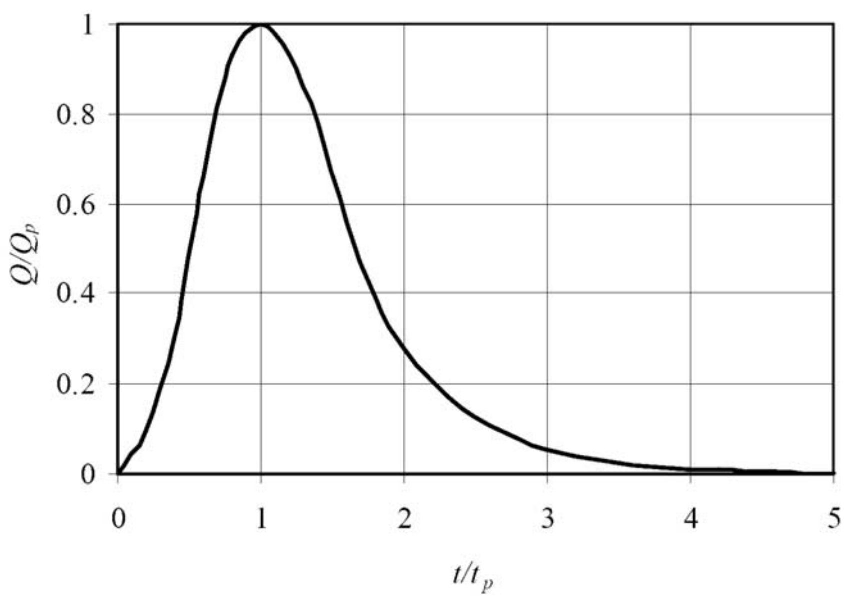

Figure 2- NRCS dimensionless unit hydrograph (USDA, 1985).

The approach used in RP-273 was to derive runoff hydrograph for the entire storm duration, rather than merely the average daily flow, and then compute the stream power that accumulated during each time increment during the storm, rather than for the entire day all at once. The reasoning for doing this was because in the small, relatively steep watersheds that are common in West Virginia, there is likely to be a significant difference in flow rate between the peak flow rate that is observed and the average flow rate during the entirety of the storm event. The hydrograph shown in Figure **3** illustrates a situation where the peak runoff flow rate for a small watershed is 82 cfs. For the 24 hour period in which this storm occurs, the average daily flow rate is only 5.6 cfs. For a watershed where flow rates vary rapidly due to small size, steep conditions, or other factors, computing stream power on the basis on an average daily flow rate could significantly under-represent that amount of energy actually expended by the stream onto the rock. This is particularly true given that stream power is a function of velocity cubed, meaning that there is a non-linear relationship between increasing stream velocity and increasing stream power.

Readily available GIS data (e.g., a digital elevation model of the watershed, land cover data, soil type, etc.), coupled with watershed model preprocessors (e.g., Aquaveo's WMS) enables the computation of parameters such as curve number, time of concentration, and runoff stream flow routing durations, and the subsequent utilization of these parameters in the HEC-1 hydrologic model. As illustrated in Figure 3, once a hydrograph is generated, tools exist to export time-series flow data as a text file, which can subsequently be imported into spreadsheet software (e.g., Microsoft Excel) for further calculations.

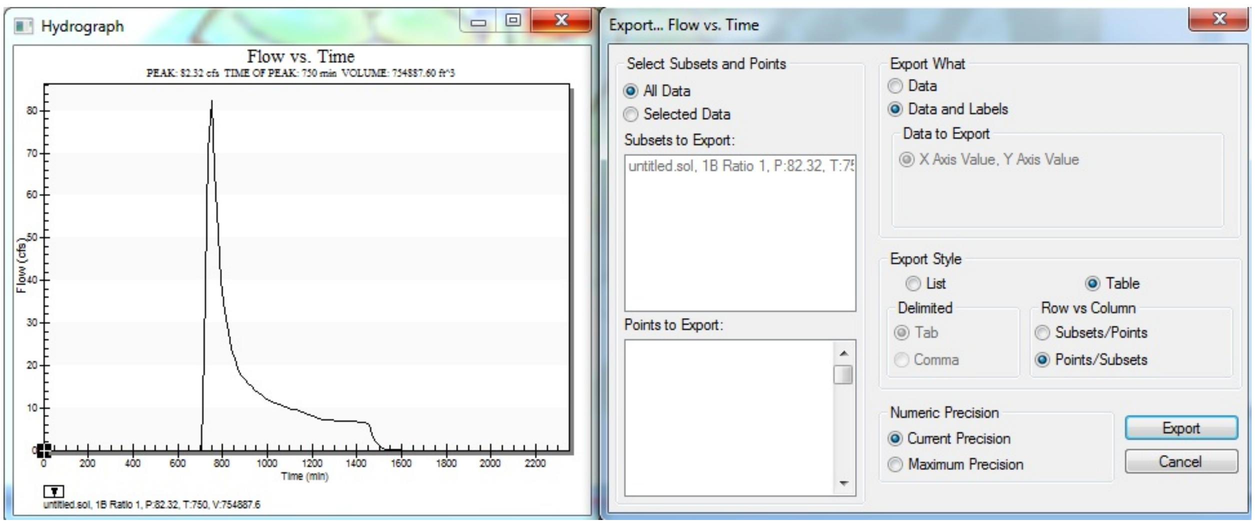

Figure 3- Example runoff hydrograph from WMS, along with dialog box to export hydrograph data points.

Discharge-Depth and Discharge-Velocity Relationships

Utilizing field surveys of channel cross sections, hydraulic models of a given site can be prepared using software such as HEC-RAS. Since stream power calculations will be performed over a wide variety of flow rates, in most cases it will be necessary to forego computation of flow velocity and depth for each individual flow rate encountered during a storm event, and instead develop equations that define the required relationships over the relevant flow range (see Figure 4). A 24 hour period that is modeled in 15 minute increments, for example, would include 96 different flow rates. Determining the corresponding flow depth and velocity using a regression equation, rather than going to the hydraulic model for each, will dramatically streamline the stream power computational process.

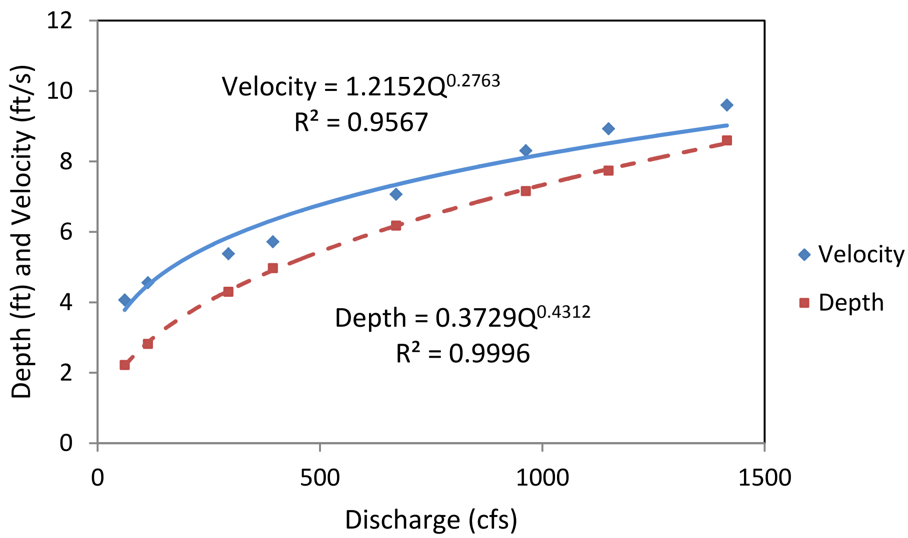

Figure 4 - Flow depth and velocity rating curve for Paden Fork.

Once the relationships between flow rate and velocity, and flow rate and depth, have been defined, a spreadsheet should be prepared that contains flow rate,

Instantaneous Stream Power and Cumulative Effective Stream Power, Ω

The approach of calculating instantaneous stream power from representative shear stress and velocity is outlined in NCHRP Report 717, section 3.6.2. As shown in Equation 1, stream power can be determined using approach flow velocity, depth, and characterizations related to localized stream conditions (e.g., whether stream power is being computed for approach flow or piers, the Manning's roughness coefficient being used, etc.).

$$P = \left(\frac{nK_p V}{1.486}\right)^2 \cdot \left(\frac{\gamma}{y_0^{1/3}}\right) \cdot V \qquad \text{Equation 1}$$

where P = Instantaneous stream power (ft·lb/s per square foot)

n = Manning's roughness coefficient

K_p = Turbulence-related velocity enhancement factor; 1.0 for approach flow, 1.5 for round-nosed piers, 1.7 for square-nosed piers

V = flow velocity, (ft/s)

1.486 = factor for U.S. customary units (1.0 for metric units)

γ = unit weight of water, 62.4 lb/ft^3

y_0 = depth of approach flow, ft

Cumulative Effective Stream Power, Ω, [ft·lb/ft^2] is determined by computing Instantaneous Stream Power [ft·lb/(s·ft^2)] at each time during a storm event, and then multiplying that Instantaneous Stream Power by the duration of the increment . A **spreadsheet example** [Stream Power Calculation

Example.xlsx] of such calculations accompanies this document, and a summary of the procedures utilized therein is as follows:

- Input flow velocity and depth values into the stream power equation (i.e., Equation 1), yielding a characterization of instantaneous stream power at each of the recurrence intervals of interest (i.e., 2-, 5-, 10-, 25-, 50-, 100-, 200- and 500-years).
- Determine the *peak* stream power associated with the 2-year event – this is the threshold above which stream power is “effective” .
- Subtract the 2-year peak stream power from stream powers of all other recurrence intervals, yielding a temporal distribution of effective stream power for each recurrence interval event at each site.
- Multiply the effective stream power [ft·lb/(s·ft^2)] during each 15-minute time increment by the duration of that increment (i.e., 900 seconds) to yield the Cumulative Effective Stream Power [ft·lb/ft^2] for that increment.
- Sum all of the incremental Cumulative Effective Stream Powers for an event (i.e., for each 15 minute increment) to find the Cumulative Effective Stream Power associated with that event.
    - Note: If desired, Cumulative Effective Stream Power can also be expressed in terms of ‘Cumulative Effective *Daily* Stream Power’ – an expression used in NCHRP Report 717 – by dividing Cumulative Effective Stream Power [ft·lb/ft^2], by the number of seconds in a day (i.e., 86,400), yielding units of [ft·lb/(s·ft^2)]. However, since it will likely be rare than daily stream flow data is the basis for stream power calculations in West Virginia, it is recommended that these counter-intuitive units of ‘daily stream power’ be avoided.

Where the Average Annual Cumulative Effective Stream Power ($\Omega_{averageannual}$) is desired, the following additional steps should be taken, which are also illustrated in the accompanying spreadsheet example.

- Apply the probability-weighting approach, multiplying the average Cumulative Effective Stream Power for two events by the incremental probability between those events, and summing the product of these over the entire range of probabilities. This approach, shown below in Equation 2, is a stepwise integration that approximates the Annual Cumulative Effective Stream Power.
- Multiply the Annual Cumulative Effective Stream Power by the number of years since each site was constructed, resulting in a probabilistically-based estimate of how much Cumulative Effective Stream Power has been exerted at the site since it was built.

$$\Omega_{\text{annual average}} = \left(\frac{\Omega_2 + \Omega_5}{2}\right)(\lambda_2 - \lambda_5) + \left(\frac{\Omega_5 + \Omega_{10}}{2}\right)(\lambda_5 - \lambda_{10}) + \left(\frac{\Omega_{10} + \Omega_{25}}{2}\right)(\lambda_{10} - \lambda_{25})$$
$$+ \left(\frac{\Omega_{25} + \Omega_{50}}{2}\right)(\lambda_{25} - \lambda_{50}) + \left(\frac{\Omega_{50} + \Omega_{100}}{2}\right)(\lambda_{50} - \lambda_{100}) + \left(\frac{\Omega_{100} + \Omega_{200}}{2}\right)(\lambda_{100} - \lambda_{200}) \qquad \text{Equation 2}$$
$$+ \left(\frac{\Omega_{200} + \Omega_{500}}{2}\right)(\lambda_{200} - \lambda_{500}) + \Omega_{500}\lambda_{500}$$

Where Ω = Cumulative Effective Stream Power
λ = Average annual probability for a certain event (e.g., 0.200 for a 5 year storm)

2. Scour Mode Decision Tree

Introduction

The scour evaluation methodology presented in NCHRP 717 (2012) is designed specifically for the mode of scour known as "abrasion and grain-scale plucking of degradable rock" (referred to hereafter simply as "abrasion"). The application of this methodology, therefore, is appropriate only when field and laboratory evidence support abrasion as the mode of scour. Fourteen (14) of the 15 bridge sites evaluated in West Virginia for the current study were characterized by a different mode of scour: quarrying and plucking of durable rock (referred to hereafter simply as "quarrying"), or some combination of abrasion and quarrying. For these sites application of the NCHRP 717 (2012) abrasion methodology vastly under-predicted the actual scour observed in the field. These results emphasize the importance of correctly identifying the mode of scour operating at bridge sites before attempting to predict scour for the purpose of bridge maintenance or design.

Presented in Figure 5 and described below is a process, here called a "mode of scour decision tree," for determining the mode(s) of scour occurring, or likely to occur, at bridges where piers and/or abutments are founded on bedrock. The end result from the decision tree places a site in either Tier II, in which case abrasion is determined to be the mode of scour and for which the NCHRP 717 (2012) methodology can be applied, or in Tier III, which means quarrying is operating as the exclusive mode of scour or in concert with abrasion. Tier III is the more serious condition, as quarrying can result in significant scour in short periods of time (e.g., during flood events). The decision tree is designed to be conservative, placing a site in Tier III if there is at least some evidence consistent with quarrying.

Development of Mode of Scour Decision Tree

According to NCHRP 717 (2012) concrete can be considered a practical benchmark for scour-resistant material, and therefore rocks that possess the characteristics of concrete can also be assumed to be resistant to scour. The similarities between these two materials include the ability to support significant loads, resistance to physical disintegration during wetting-drying (i.e., "slaking"), and heavy enough for large fragments to remain in place when subjected to significant current or wave action (NCHRP 717, 2012). These properties of scour-resistant material can be made more objective through the following:

- Unconfined compressive strength (UCS) ≥ 2,500 psi;
- Remains intact when immersed in water (e.g., jar slake test);
- Specific weight ≥ 150 lbs./ft^3;
- Absence of joints or joint spacing ≥ 4 ft. (1.2 m.)

If all of the above criteria are satisfied rock can assumed to be resistant to scour. The individual criteria, however, are relevant to specific modes of scour. For example, UCS and the water immersion test are relevant to the abrasion mode of scour, while jointing and specific weight are relevant to the quarrying mode of scour. Thus a rock mass that does not slake, and with UCS greater than 2,500 psi, should be resistant to abrasion but could still be scoured by quarrying if joints are closely spaced. To distinguish sites effected by abrasion from sites effected by quarrying (or hybrid sites where both modes occur), the decision tree developed from this project utilizes a set of criteria based on the above, as well as additional criteria from NCHRP 717 (2012). Most importantly the list of criteria incorporated into the decision tree are designed to reflect the geotechnical and geologic conditions encountered in the field or in the laboratory for rocks present at the 15 West Virginia bridges.   The decision tree begins with a set of field-based criteria:

- Identification of the site location with respect to the two major physiographic provinces in West Virginia: Appalachian Plateau or Valley and Ridge;
- Identification of dominant rock type(s);
- Inspection of degree of weathering of bedrock;
- Observations of longitudinal channel profile upstream and downstream of bridge;
- Inspection of bedrock surface in immediate vicinity of bridge;
- Identification of joints and joint spacing, if joints are present;
- General durability or hardness of rock, as measured by a rock hammer blow;

The decision tree assigns a score for each of the field–based criteria. Depending on the total score, the site is placed in either Tier III or samples from the site are recommended for a suite of laboratory tests:

- Continuous slake durability test;
- Unconfined compression test.

In order to validate the decision tree, the criteria were applied to the all 15 West Virginia bridge sites and scores assigned to each site. The decisions tree correctly identified the mode of scour at all 14 West Virginia bridge sites where scour was observed and thus recommended the appropriate action (Figure **6**).

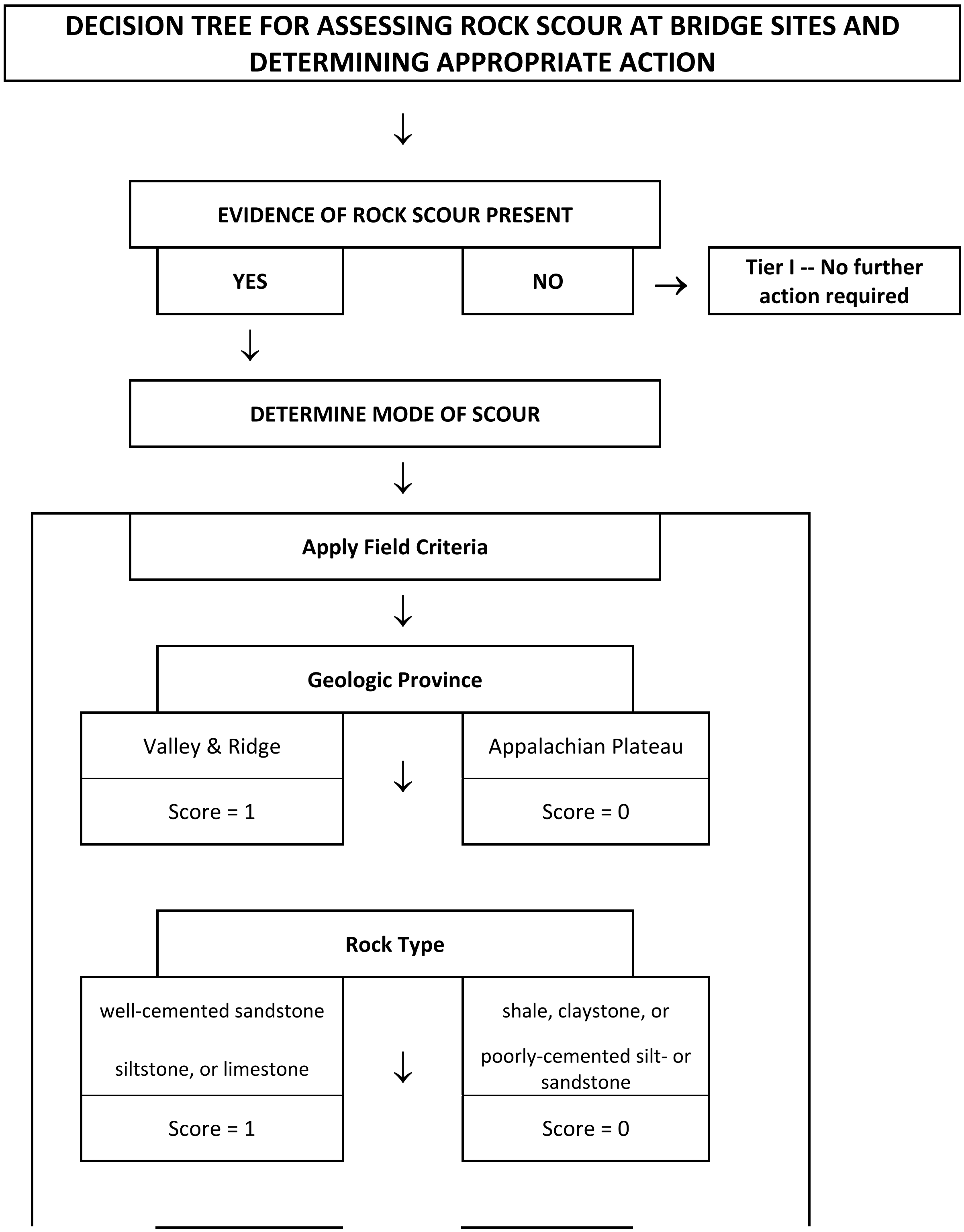

Figure 5 – Mode of Scour Decision Tree

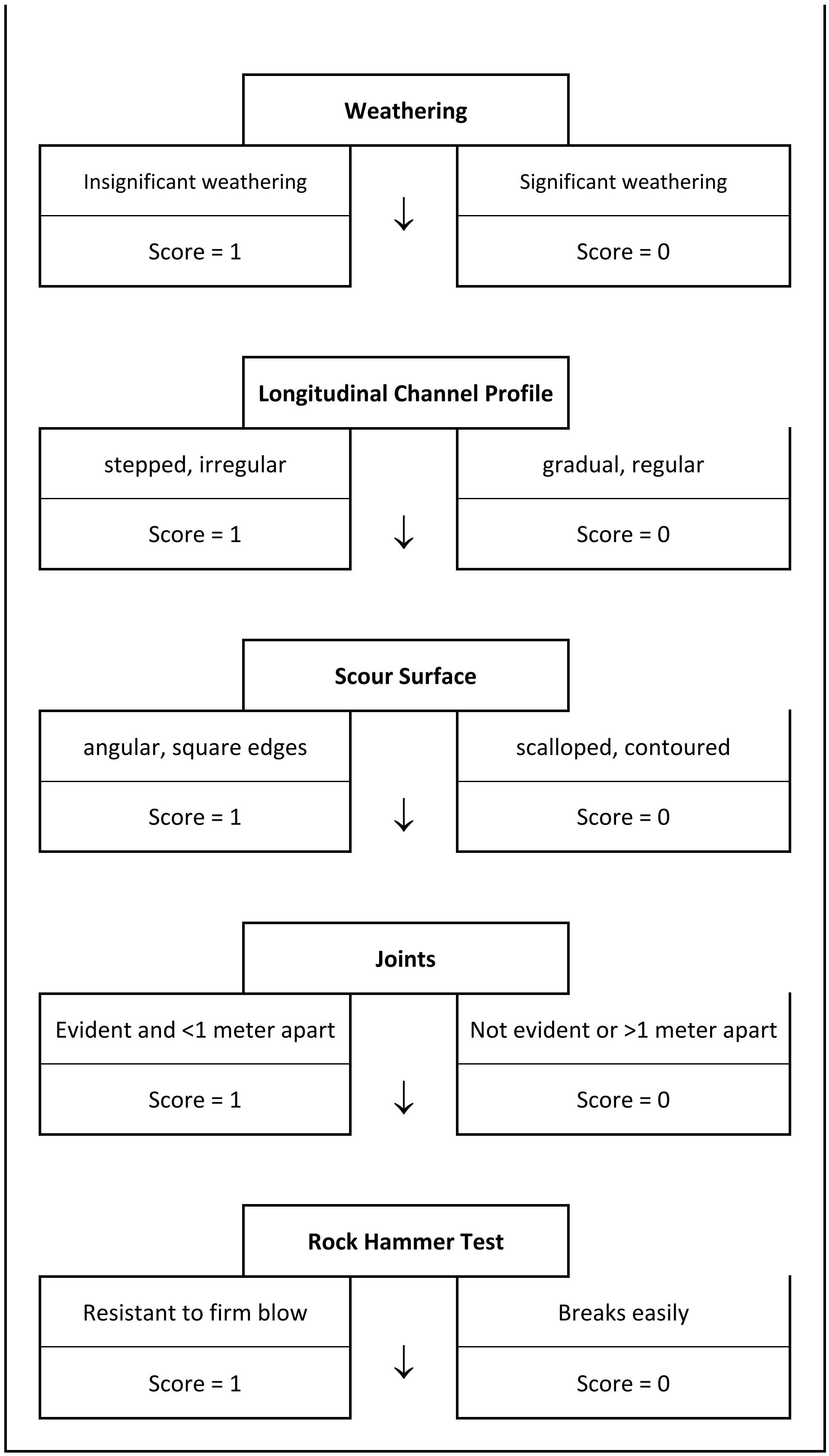

Figure 5 (cont.)

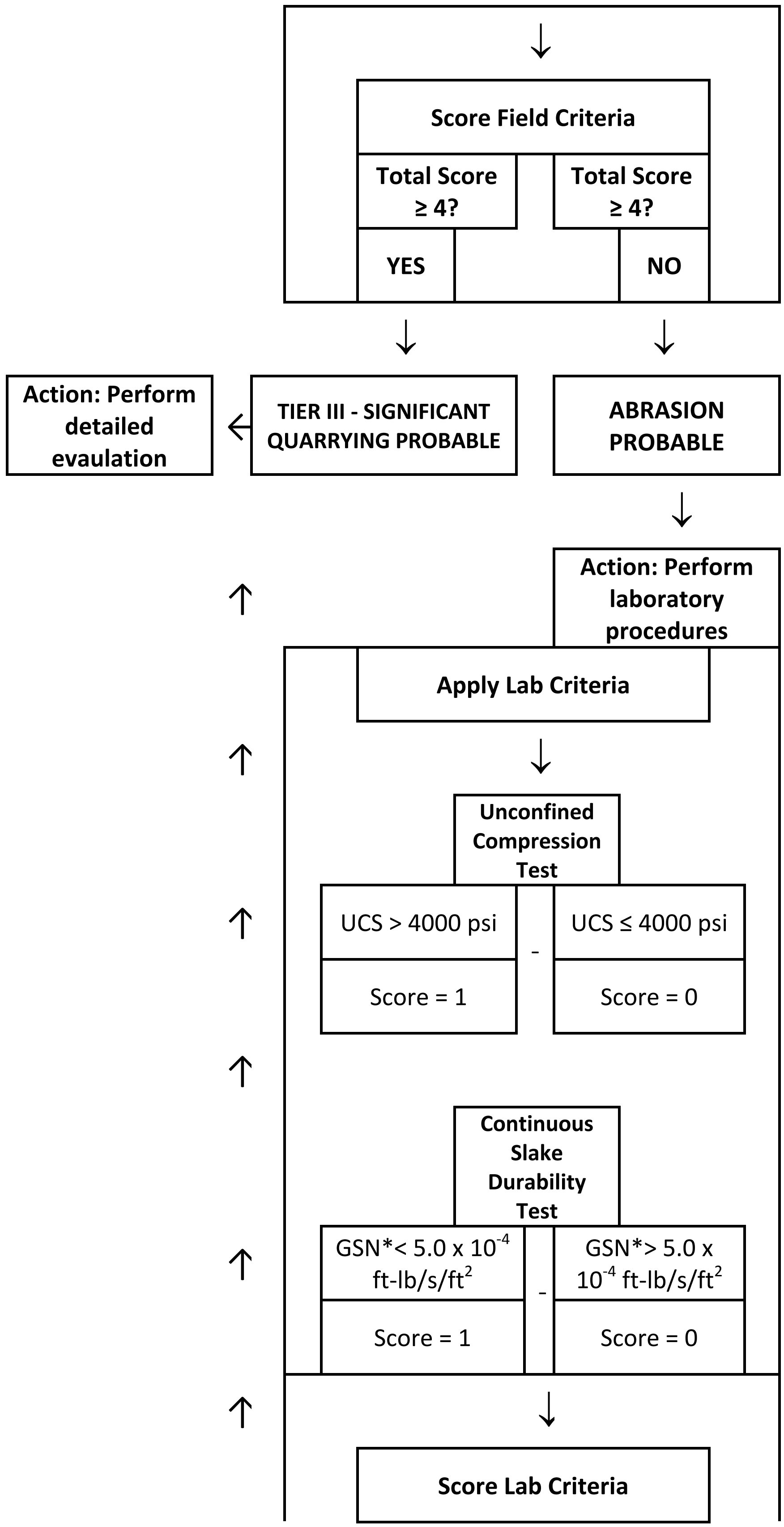

Figure 5 (cont.)

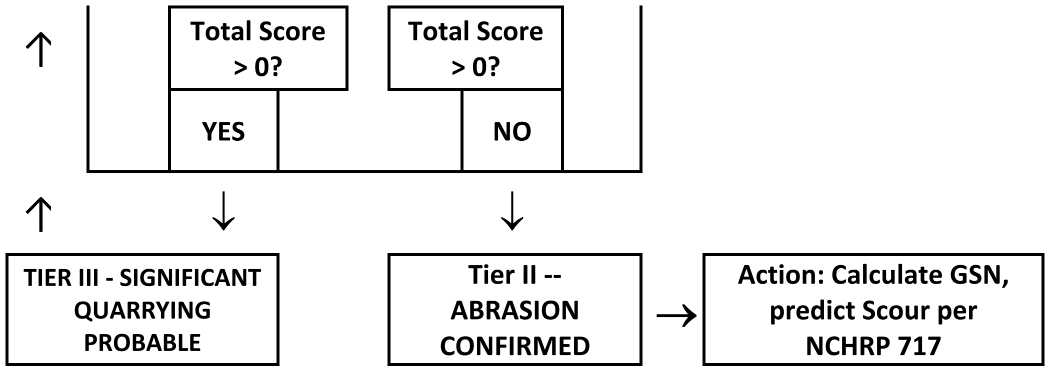

Figure 5 (cont.)

Site	FIELD CRITERIA									LAB CRITERIA			
	Geo Prov	Rock Type	Wxg	Profile	Surface	Joints	Hammer	**Total**	**Tier**	UCS	GSN	**Total**	**Tier**
Audra	0.5	1	1	1	1	1	1	**6.5**	**III**	NA	NA	**NA**	**NA**
Beverly	1	1	1	1	1	1	0.5	**6.5**	**III**	NA	NA	**NA**	**NA**
Bridge Fork	0	1	1	1	0.5	0	1	**4.5**	**III**	NA	NA	**NA**	**NA**
Caldwell Run	0	0	0.5	1	1	0	0	**2.5**	**Lab**	1	1	**2**	**III**
Clear Fork	0	1	1	1	0.5	0	1	**4.5**	**III**	1	0	**1**	**III**
Coon Creek	0.5	0	0	1	0.5	0	0.5	**2.5**	**Lab**	0	0	**0**	**II**
Cucumber	0	1	0	0.5	0.5	?	?	**2**	**Lab**	1	1	**2**	**III**
Grassy Run	0.5	1	0.5	1	1	1	0.5	**5.5**	**III**	NA	NA	**NA**	**NA**
Laurel Fork	0.5	1	1	1	1	0.5	1	**6**	**III**	NA	NA	**NA**	**NA**
Leatherwood -- Ss	0	1	0	1	1	0	1	**4**	**III**	NA	NA	**NA**	**III**
Leatherwood --Sh	0	0	0	1	0	1	0	**2**	**Lab**	0	0	**0**	**II**
Little Sandy	0	0	0.5	0	1	0	0	**1.5**	**Lab**	1	1	**2**	**III**
Mish	1	1	1	1	0.5	0	1	**5.5**	**III**	NA	NA	**NA**	**NA**
Paden Fork	0	0	0	1	1	1	0	**3**	**Lab**	1	0	**1**	**III**
Ritter Park / 5th St	0	1	1	1	0.5	1	1	**5.5**	**III**	NA	NA	**NA**	**NA**
Roaring Creek	1	1	1	1	1	1	1	**7**	**III**	NA	NA	**NA**	**NA**

Figure 6 – Scoring of 15 Bridge Sites using Decision Tree.

Guidelines for Users of the Mode of Scour Decision Tree

1. Determining the mode of rock scour requires that a field inspection of rock conditions be performed by a qualified geologist at the existing bridge or proposed bridge site in question. Tools required include a measuring tape or stick, camera, rock hammer, and logbook. The field inspection will include the following:

- Identification of the site location with respect to major physiographic province;
- Identification of dominant rock type(s);
- Inspection of degree of weathering of bedrock;
- Observations of longitudinal channel profile upstream and downstream of bridge;
- Inspection of bedrock surface in immediate vicinity of bridge;
- Identification of joints and joint spacing, if joints are present;
- General durability or hardness of rock, as measured by a rock hammer blow;

Each of these characteristics, along with guidance for scoring, is explained in more detail below. A score of 0 for a given parameter is consistent with scour by abrasion; a score of 1 is consistent with scour by quarrying.

Physiographic provinces. Most of West Virginia lies within the Appalachian Plateau. The eastern parts of the state lie within the Valley and Ridge (see Figure **7**). A transitional zone known as the Allegheny Front or Allegheny Mountain Section separates these two physiographic provinces. Rock strata in the Appalachian Plateau are typically flat-lying, while those in the Valley and Ridge have been deformed during ancient mountain-building events and often lie at an angle relative to horizontal. Sites within the Appalachian Plateau should be assigned a score of 0, sites in the Valley and Ridge should be scored as 1, and sites in the Allegheny Front or Mountain Section should be scored as 0.5.

Dominant rock type(s). Rocks in West Virginia tending to scour by abrasion consist of mudstones such as shale and claystone. Poorly indurated sandstone or siltstone may also scour by abrasion. These rock types should be assigned a score of 0. More durable rocks such as most sandstone, limestone and siltstone are more likely to resist scour altogether or, if jointed, to scour by quarrying. These latter rock types should be assigned a score of 1.

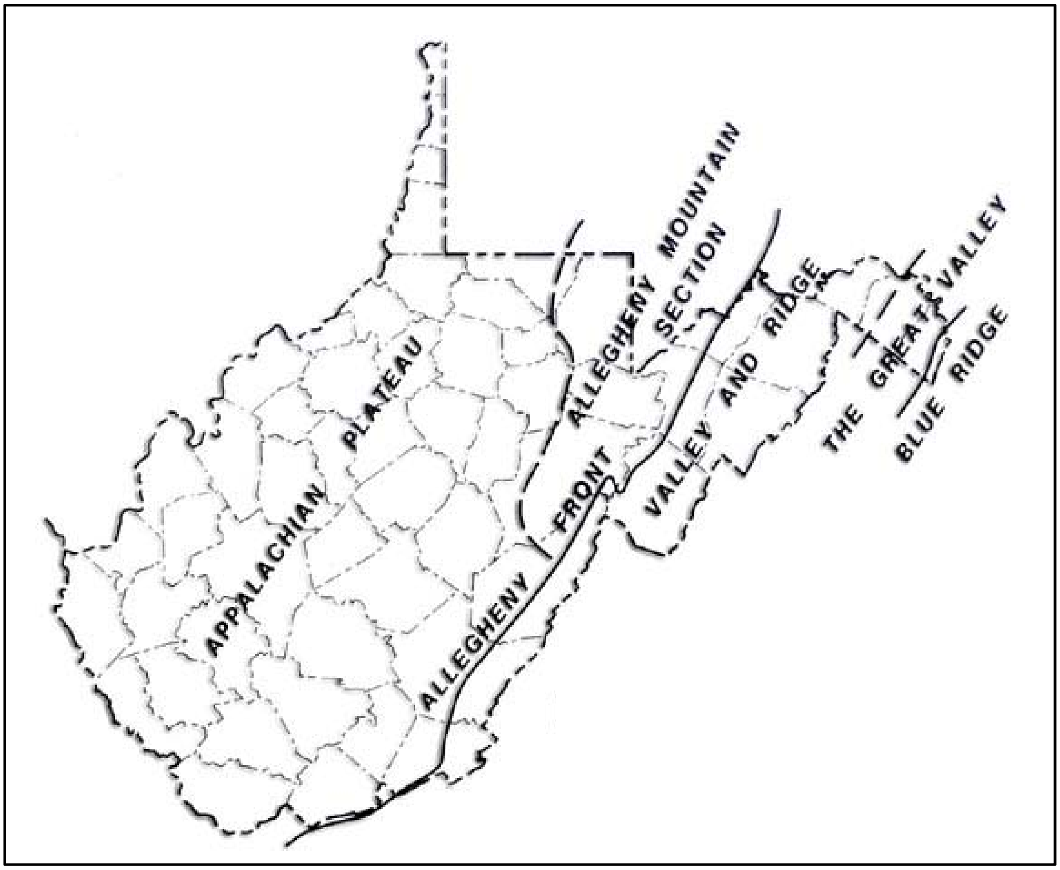

Figure 7 – Physiographic Provinces in West Virginia (from West Virginia Geological Survey).

Degree of weathering. Mechanical and chemical weathering occurs as air and water interact with the minerals that make up rocks. One effect of weathering is to reduce the strength of rocks. Generally the amount of weathering that would significantly weaken rock requires spans of geologic time, not the length of time that a bridge is in service. Those weathering processes that destroys bonding between mineral grains makes rocks more susceptible to scour by abrasion. These processes include slaking of shales, a type of mechanical weathering caused by repeated wetting and drying, and various types of chemical weathering such as dissolution, oxidation, and hydrolysis that can affect siltstone, sandstone, and limestone. Some indicators of chemical weathering include pitting, iron-staining, and formation of secondary clay minerals. Most rocks are weathered on their exterior surfaces but may not be weathered below this depth. The geologist should break open a rock sample during the field inspection to determine if weathering is pervasive within the interior of the rock and thus likely to compromise rock strength. A significantly weathered rock should be assigned a score of 0; a rock with insignificant weathering, appearing fresh on the inside, should be assigned a score of 1. If there is uncertainty in the result, or in the case of marginal weathering, a score of 0.5 should be assigned.

Longitudinal channel profile. This criterion involves the slope of the bedrock channel surface within a span extending approximately 300-500 feet upstream of a bridge to the same distance downstream. If the bedrock surface slopes gradually within this span it should be assigned a score of 0; if it slopes irregularly, or displays a stair-step like profile, it should be assigned a score of 1. In the latter case the channel floor is often littered with rock slabs or tablets that have been removed by quarrying upstream and are transported downstream during high-flow events. An intermediate case can be assigned a score of 0.5.

Bedrock surface. The surfaces of rocks below an abutment or pier should be inspected. If rock has been removed by scouring, the nature of the scoured surface will provide clues as to the mode of scour. Surfaces affected by abrasion have been variously described as smoothed, contoured, fluted, or scalloped, indicating gran-by-grain removal of material (Figure 8). This appearance should be assigned a score of 0. If on the other hand, the scoured surface appears to have had angular chunks or blocks of material removed, this is indicative of quarrying and a score of 1 should be assigned (Figure 9). If both types of surfaces are present a score of 0.5 can be assigned (Figure 10).

Figure 8 – Example of scalloped surface characteristic of abrasion scour (Coon Creek Bridge) .

Figure 9 – Example of blocky appearance characteristic of scour by quarrying (Roaring Creek Bridge).

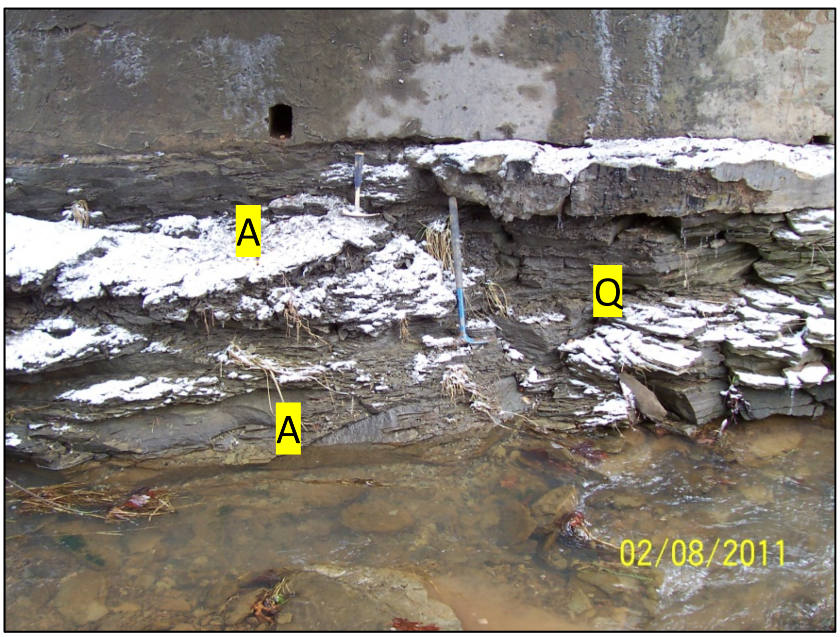

Figure 10 – Example of scour by both abrasion (A) and quarrying (Q) [Paden Fork Bridge].

Joints and joint spacing. If bedrock below a bridge is not jointed, or the average spacing between joints is more than approximately 3 feet, a score of 0 should be assigned. In contrast, if rock is jointed and the average spacing between joints is less than approximately 3 feet, the site should be assigned a score of 1 (Figure 11).

Hardness of Rock. A rock hammer should be used to strike a firm hammer blow against bedrock below bridge abutments and/or piers. If the hammer breaks the rock, the rock can be

Figure 11 – Example of jointing (Audra Park Bridge). Arrows show joint spacing

considered non-durable and should be assigned a score of 0; if the hammer does not break the rock, the rock should be considered durable and a score of 1 should be assigned. A marginal result can be assigned a score of 0.5. Additional considerations include sound (non-durable rocks will emit a dull thud when struck; durable rocks will emit a high-pitched ping) and rebound (a non-durable rock will absorb much of the energy from a hammer blow; a hammer striking a durable rock will often bounce back as a result of elastic rebound).

2. Scores from the field-based criteria should be totaled. A total score of 4 or more suggests a significant component of quarrying and places the bridge in Tier III (), requiring a more detailed evaluation. A total score of less than 4 suggests abrasion but requires confirmation from laboratory evaluation of rock samples.

3. For sites requiring laboratory analyses, the following tests should be performed:

- continuous slake durability test (CSDT);
- unconfined compression strength (UCS).

Figure 1 indicates that a geotechnical scour number (GSN) of greater than 5.0 x 10^{-4} ft-lbs/s/ft^2 from the CSDT should be assigned a score of 0, while a lesser GSN should be assigned a score of 1. In addition, a UCS of less than 4,000 psi should be assigned a score of 0, while a greater UCS should be assigned a score of 1.

4. Scores from the laboratory analyses should be totaled. A total score greater than 0 suggests a significant component of quarrying and places the bridge in Tier, requiring a more detailed evaluation. A total score of 0 confirms abrasion and places the site in Tier II.

3. Computing Geotechnical Scour Number (GSN)

Geotechnical Scour Number (GSN) is a computed parameter meant for application on degradable (i.e., non-durable) rock where the abrasion mode of scour is anticipated. GSN ultimately attempts to relate the scour behavior for a sample tested in a lab environment to the scour behavior of rock in the field. Specifically, Scour Number is the scour depth divided by cumulative stream power, and in the case of GSN, the procedure that has been developed equates the change in the sample during the experimental period to an ‘Equivalent Scour Depth’, and the energy imparted to the sample by a rotating experimental apparatus to an ‘Equivalent Stream Power’.

Application of the GSN approach was undertaken in RP-273, and the procedures described below summarize the approach that was used in that project. Similar methods can be used for any site where experimental derivation of GSN is desired.

Field Drilling of Rock Samples

During RP-273, rock coring into the 10 feet of bedrock immediately below the bridge abutments was performed at two locations per site, one location adjacent to each abutment. Upon extraction from the borehole, the rock core was immediately logged, photographed, and evaluated for rock quality designation (RQD) according to ASTM 6032.

Each 5-foot length of rock core was then double-wrapped, first in a 5-mil wax film (Parafilm or equal) followed by a heavy gauge sheath of flexible plastic. The package was secured with duct tape and labeled, then placed in a protective core box for transport. During transport the core boxes were wrapped in moving blankets (or equivalent padding) to minimize the impact of potential vibration. The above procedures were consistent with ASTM D 2113 Section 5.0. One or two additional rock samples were collected and preserved from surface exposures at or immediately adjacent to the bridges.

Lab Analysis of Rock Sampling

Rock core and surface exposure samples were subjected to testing for specific gravity at natural (field) moisture conditions and modified slake durability (continuous abrasion). Cores samples were also tested for unconfined compressive strength (UCS). All tests were conducted in accordance with applicable ASTM standards and NCHRP Project 24-29.

The slake durability apparatus used for this project consisted of two drums, so two tests could be run at a time. The testing methodology was based on the modified procedure as described in the NCHRP Project 24-29, which is based on Dickenson and Baillie (1999). Rock pieces taken from both surface exposure and core samples met the test method requirements for mass (10 pieces at 40-60 grams each). However, pieces taken from the surface exposure samples more nearly met the requirement that pieces be equidimensional than those from the core samples which, by necessity, were disc-shaped, with the long dimension equal to the core diameter (see Figure 13).

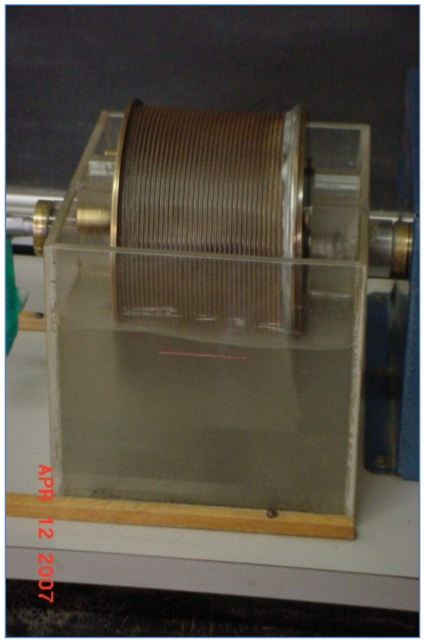

Figure 12 – Rotating drum used in the continuous slake durability tests.

The fundamental goal of the testing is to define the linear portion of the curve relating scour depth to stream power. This requires three or more points on this portion of the curve, with each point corresponding to a multiple of a 60-minute continuous abrasion cycle. For example, depending on the rock type, the linear portion of the curve may be sufficiently delineated by the weight loss at 180, 240, and 300 minutes; 300, 360 and 420 minutes; or 360, 420 and 480 minutes of continuous abrasion. Figure 13 shows an example of rock samples before and after continuous slake durability testing.

Figure 13 – Example of rock samples before (left) and after (right) continuous slake durability testing.

Derivation of Depth of Scour and Equivalent Stream Power from Lab Testing

The modified slake durability testing requires that mass lost during each 60-minute cycle be recorded and then converted to a volume using the natural (field) saturated specific gravity of the rock being tested:

$$V_i = M_i / \gamma_{sat}$$

V_i = incremental volume lost during 60-minute test interval (L^3),

M_i = incremental mass lost during 60-minute test interval (M),

γ_{sat} = natural (field) saturated specific gravity of the rock being tested (M/L^3).

Multiple measurements at successive test intervals up to 9 hours result in a series of V_i which are then normalized by a unit area to derive a linear dimension equivalent to a depth of scour for each interval:

$$D_{(X*60\ min)} = V_{i\ (X*60\ min)} / \text{ unit area}$$

$D_{(X*60\ min)}$ = equivalent depth of scour corresponding to test interval x (= 1,2,3 …9) minutes (L),

$V_{i\ (X*60\ min)}$ = incremental volume lost during test interval x (= 1,2,3 …9) minutes, (L^3),

unit area = area in appropriate units (L^2).

Equivalent stream power for each test interval can be calculated based on the average weight of sample per test interval, the distance (equivalent length) traveled by the sample in the submerged rotating drum during the test interval, and the time of the test interval:

$$\omega = L\ t\ [W_{(X)} + W_{(X+1)}] / [2\ A_{(1/8)}]$$

ω = equivalent stream power [force* $L/T/L^2$],

L = equivalent length (L),

t = incremental time of test interval (T).

$[W_{(X)} + W_{(X+1)}] / 2$ = average weight of sample during test interval (force).

$A_{(1/8)}$ = area of submerged portion of drum (1/8 of total area).

The final result of each modified slake durability test, then, allows a plot of ω versus D for multiple test intervals. An example from Keaton and Mishra (2010) is shown in Figure 14:

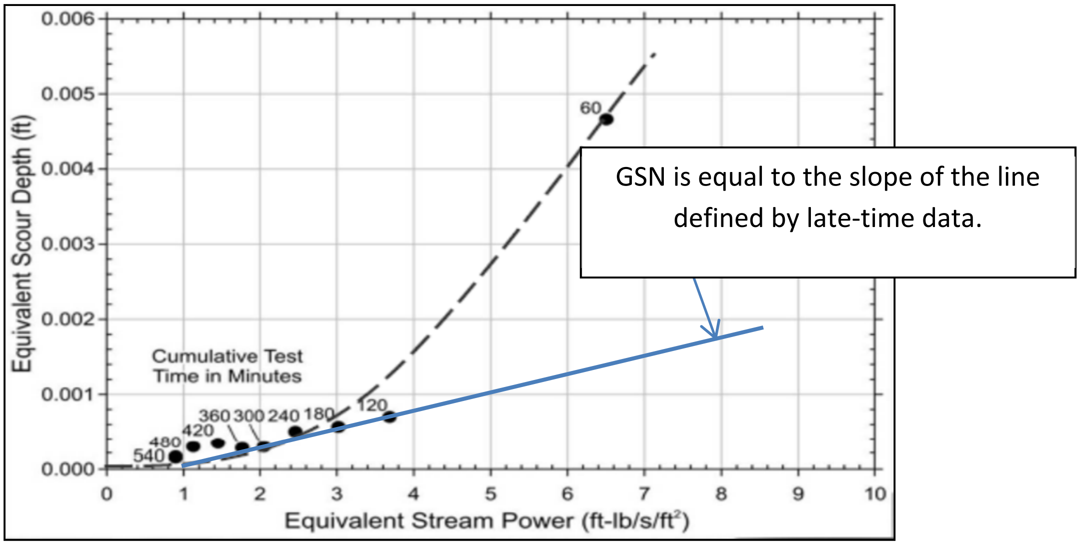

Figure 14 – Plot of Equivalent Scour Depth vs. Equivalent Stream Power

It should be noted that as the test proceeds (increasing time, above), data points define a trend that approaches lower values of both stream power and scour. This is because stream power is a function of sample weight which, as the sample progressively abrades, is decreased in later test intervals.

<u>Derivation of Geotechnical Scour Number</u>

When the linear portion of the equivalent scour versus equivalent stream power plot is considered, a slope can be defined for each test (Figure **15**). This slope can then be expressed as scour per unit stream power (e.g., foot of scour per unit of power). A higher GSN represents a higher slope; a lower GSN represents a lower slope. In other words, higher values reflect a greater amount of scour, or abrasion, per unit stream power, while lower values reflect a lesser amount of scour, or abrasion, for the same expenditure of stream power. This parameter, the geotechnical scour number (GSN), becomes the basis by which the scour potential of different rock types can be compared when scour is occurring by abrasion of degradable rock. In addition, for sites affected by abrasion, the GSN becomes the starting point for relating calculated values of cumulative stream power at bridge sites to predicted scour depth. Since the mode of scour was not always initially evident from field inspection, GSN was calculated for all bridge sites evaluated for this project. Regardless of scour mode, the GSN and GSN* serve as a useful means of comparing the abrasion resistance of different rock types.

The GSN can be calculated in two different ways: 1) by forcing a zero-intercept for the best-fit line, and 2) by not forcing a zero-intercept. The former method follows the procedures specified in NCHRP 717, which is meant for degradable (i.e., less abrasion-resistant) rocks that trend toward near-zero values of equivalent steam power and equivalent scour depth. In such cases, forcing a zero-intercept both honors the data and produces a relatively strong linear correlation. However, for continuous slake durability data from the 15 West Virginia bridges studied in RP-273, the median coefficient of determination (R^2) was only 0.13 when the GSN was calculated in this manner. In contrast, for GSN calculated without forcing a zero-intercept, the median R^2 value was 0.79. These differences are primarily due to the fact that scour of rock at the majority of West Virginia bridges studied occurs by some degree of quarrying of durable, jointed rock, rather than by abrasion of degradable rock. Durable rock, by definition, tends to be resistant to abrasion. When subjected to a modified slake durability procedure, durable rocks retain significant mass, and thus stream power, throughout the test, and a linear regression on the data intersects the x-axis (stream power axis) at a significant positive value. The physical significance of this x-axis is known as the “threshold value” and represents the minimum stream power needed to initiate abrasion scour (NCHRP 717).

Figure 15 shows a representative example from the project data of GSN calculated both with and without forcing a zero-intercept and the resulting change in GSN and R^2. For the remainder of this report, GSN will be used to refer to the slope of the best-fit line determined from the zero-intercept method (per NCHRP 717), and GSN* will be used to refer to the slope of the best-fit line determined from the non-zero-intercept method.

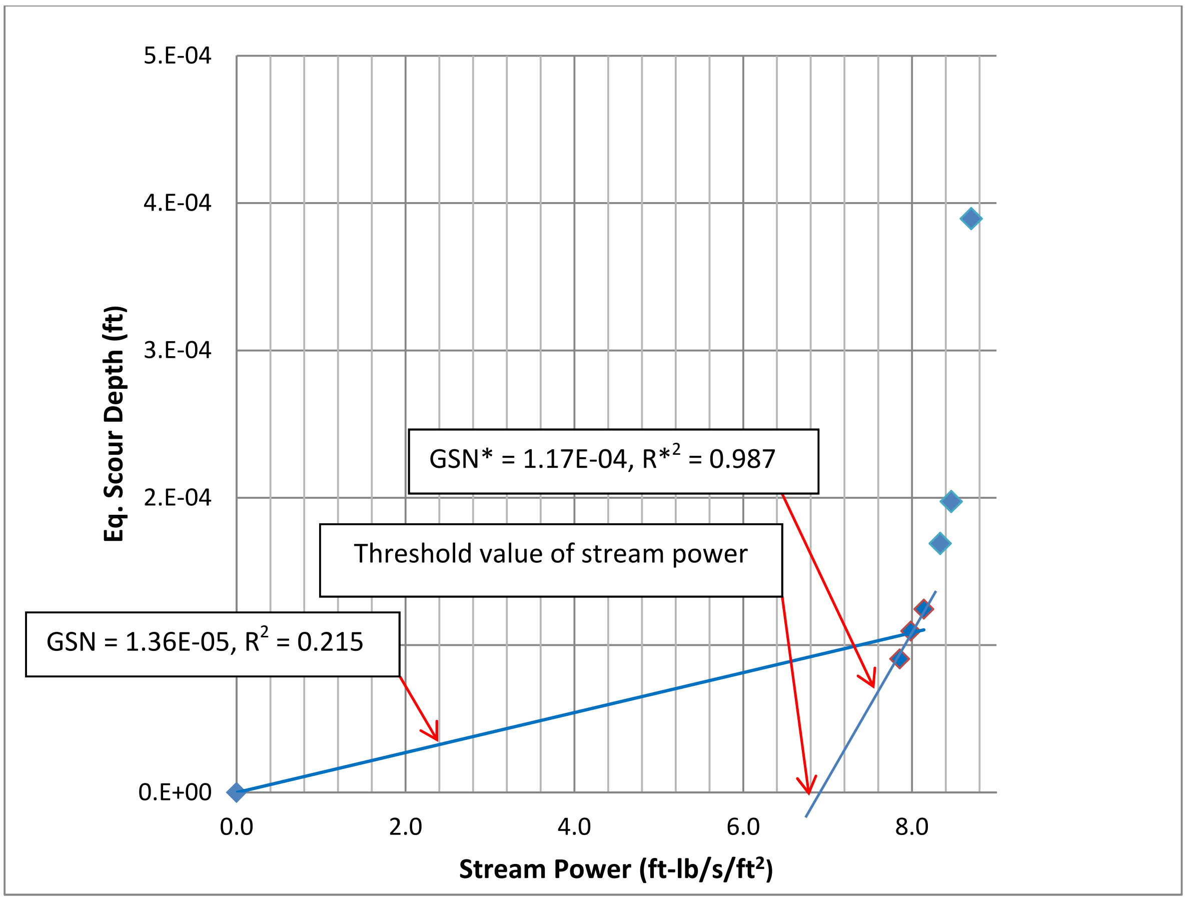

Figure 15 – Representative plot of continuous slake durability test results with linear trend of late-time data determined with and without forcing a zero-intercept.

4. Site Classification by Average Annual Cumulative Effective Stream Power

A study of bridge sites in West Virginia where rock scour was present demonstrated that quarrying of durable rock was a more common mode of scour than abrasion of degradable rock; quarrying was determined to be the primary or contributing mode of scour at 14 of 15 sites. Since the Geotechnical Scour Number approach is not applicable to scour caused by quarrying, an alternate means of classifying scour behavior must be undertaken.

Section 2 of this document introduced classification of sites by scour mode by using the Scour Mode Decision Tree. Implementing that approach at sites in West Virginia would add to the existing survey of scour mode at the 15 sites studied in RP-273. Coupled with estimates of any scour depth that is observed, this would enable WVDOH to develop a representation of how many sites fit into each scour mode category. The sites studied in RP-273 purposefully included those where scour was known to be a problem, and thus it is not known what fraction of WVDOH's inventory of bridges fit into each scour mode tier.

Similar to classifying sites by scour mode, it would also be possible for WVDOH to compute the Average Annual Cumulative Effective Stream Power at bridge sites, and use the resulting spectrum of values to categorize sites by the relative work imparted to rock by streams. Since stream power is a function of both flow depth and flow velocity, larger watersheds – with relatively deeper ongoing flow depths and higher flow velocities – accumulate stream power at a greater rate than smaller watersheds. There are exceptions to this general trend, however, and so watershed area cannot be used as a direct substitute for the computational process described in Section 1 of this document.

The sites included in RP-273 were investigated for breakpoints relative to Average Annual Cumulative Effective Stream Power, and were grouped according to relatively low, medium, and high values, as shown in Table 1.

Table 1 – Stream Power Groups and Sample Values

Average Annual Cumulative Effective Stream Power, $\Omega_{average\ annual}$ (ft·lb/ft^2)					
Group A (Low) ($\Omega_{average\ annual}$ < 1,000 ft·lb/ft^2)		Group B (Medium) (1,000 < $\Omega_{average\ annual}$ < 15,000 ft·lb/ft^2)		Group C (High) ($\Omega_{average\ annual}$ > 15,000 ft·lb/ft^2)	
Grassy Run	686	Laurel Fork	5,642	5th St Ritter Park	31,871
		Mish Rd.	7,015	Bridge Fork	54,444
		Little Sandy	7,560	Beverly	82,293
		Paden Fork	10,436	Coon Creek	144,645
		Caldwell Run	11,464	Audra Park	193,437
		Cucumber	11,669	Roaring Creek	196,913
		Leatherwood	13,263	Clear Fork	296,961
		Average	9,578	Average	142,938

If, over time, WVDOH expands the number of sites where analyses have been performed to determine Average Annual Cumulative Effective Stream power, then the distribution of bridge sites within each category would be better known. Additionally, adjustments to the threshold defining each group could be changed based on the number of sites observed to be in each category, and any trends related to simultaneous site classification by Scour Tier, and measurements of scour depth. Since Average Annual Cumulative Effective Stream power is expressed on the basis of power exerted per year, it may also be most meaningful to characterize observed scour depths in terms of scour depth per year, such that annualized work could be compared against an annualized result of that work (i.e., scour).