Appalachian Trail - Brown Mountain Creek Pedestrian Bridge Replacement

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Abstract

In Amherst County, Virginia, a pedestrian bridge on the Appalachian Trail that spanned over Brown Mountain Creek was destroyed in 2019 by a fallen tree. The goal of this project is to design a new pedestrian bridge with a natural and rustic appearance that meets the design specifications of the Appalachian Trail Conservancy (ATC) and the United States Forest Service (USFS). A multiple log-stringer bridge design was selected to replace the old bridge, based on the design constraints: bridge aesthetics, constructability, resilience, and use of the existing abutments. These constraints were established by the stakeholders of this project, ATC and the USFS. The design must follow many standards from the USFS Standard Specifications for Construction Trails and Trail Bridges on Forest Service Projects, USFS Handbook, and USFS Manual. A 30-year design life and 3 feet of freeboard above the 100-year design storm were used as criteria for a hydraulic analysis performed with the Hydraulic Engineering Center River Analysis System (HEC-RAS). Additionally, we considered how climate change may increase the 100-year design flow, by 20%. The design can pass both the 100-year flow and the 100-year flow considering a 20% increase due to climate change both with at least 3 feet of freeboard. A final design plan set and design report, including a cost estimation, have been created for constructing this bridge.

Acknowledgements

The Brown Mountain Pedestrian Bridge Team would like to thank their senior design professor, Dr. Cully Hession (VT BSE), academic advisor Dr. Jonathan Czuba (VT BSE), and our external advisor Ben Bradley (VT), for their contributions and guidance throughout this project. We would also like to give a special thank you to our stakeholder representatives David Rasmussen (USFS), Jessie Howard (USFS), and Kathryn Herndon-Powell (ATC) for their support and assistance with this project. Additionally we want to thank Josh Thomas at High Country Conservation, LLC, for their cooperation in providing us detailed information for our cost estimate for this project.

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Problem Statement

The Brown Mountain Creek wooden stringer pedestrian bridge was built in the 1980s where the Appalachian National Scenic Trail (AT) passes through Amherst County, Virginia (Figure 1). The integrity of the bridge had severely diminished by 2017, and the stringers were snapped by a fallen tree in 2019, rendering the bridge unusable (Figure 2). The Appalachian Trail Conservancy (ATC) has asked the Biological Systems Engineering (BSE) department at Virginia Tech and the United States Forest Service (USFS) to collaborate to produce a design package for a new pedestrian bridge.

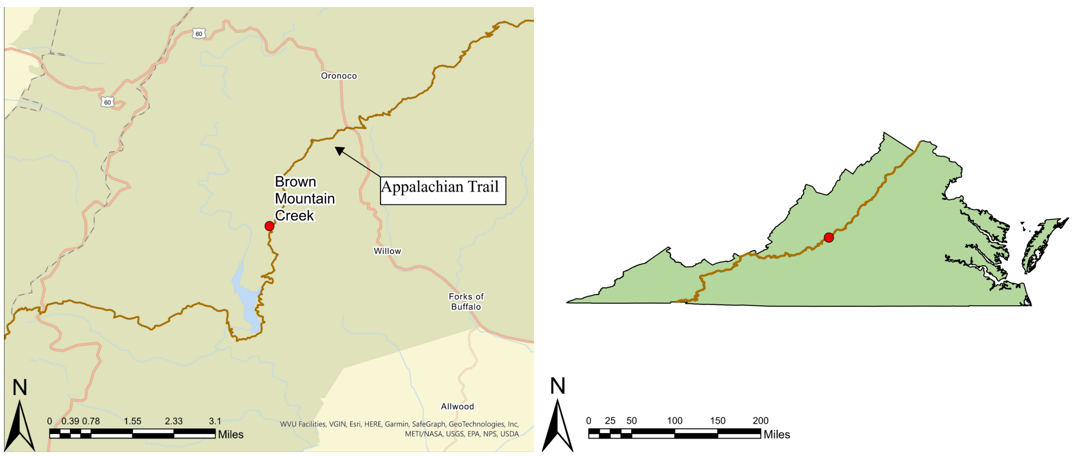

Figure 1. Site map and location of the Brown Mountain Creek site in Amherst County, Virginia, along the Appalachian Trail.

Figure 2. Pictures of the existing bridge looking upstream, damaged in 2019. Picture taken on October 14, 2022.

Background

The AT is a hiking trail spanning over 2,190 miles from Maine to Georgia. Hikers encounter hundreds of streams over the course of their 5 to 7 month journey, and it is important that they have safe access to cross (ATC, n.d.). The ATC is in charge of the "protect[ion], manage[ment], and advocat[ion] for the Appalachian National Scenic Trail" (ATC, n.d.), meaning they upkeep the trail and its bridges. The ATC is working in collaboration with the USFS in constructing this bridge because they are the agency within the U.S. Department of Agriculture in charge of managing the national forests in the United States (USFS, n.d.).

Building bridges requires following many standards and specifications. The USFS has standards regarding pedestrian bridges. A document called the "Standard Trail Plans and Specifications for Construction of Trail and Trail Bridges on Forest Service Projects" includes valuable information on construction, quality assurance, maintenance, and general specifications on bridge designs (USFS, 2014). Chapters 81 through 85 in section 7709.56b of the "Transportation Structures Handbook" (USFS, 2014) provides detailed information on the design requirements, materials involved, design plans, ideal structure life, and the allowance of including previously used materials in pedestrian bridges along forested trails. Finally the document "Standard Specifications for Construction Trails and Trail Bridges on Forest Service Projects" (USFS, 2014) contains requirements on trail bridges for site surveying, cost assessments, as well as quality control and assurance methods.

Goals and Objectives

The goal of this project is to design a new pedestrian bridge with a natural and rustic appearance that meets the design specifications of the ATC and the USFS. This goal will be reached through a site survey, geotechnical assessment, hydrologic analysis, hydraulic analysis in HEC-RAS, cost estimate, design report, and plan set created in AutoCAD. The design plans will be submitted in May of 2023 to the ATC and USFS so construction of the bridge can begin shortly after.

Criteria and Constraints

In the process of designing our bridge replacement, we took into account multiple different criteria: the bridge must include 3 ft of freeboard during the peak flow of a 100-year flood event and have a design life of 30 years. The constraints that were considered throughout the decision process were as follows: the bridge must have a rustic appearance, be accessible to transport materials and equipment, be resilient to environmental hazards, and tie into the existing abutments. These design criteria and constraints were communicated by our stakeholders and determined from standards.

Several bridge designs were considered. The material options were as follows: steel, concrete, and timber. Timber was chosen in our first design matrix (Appendix A, Table A-1) because it achieved a natural and rustic appearance, and it proved to be a cost-effective solution. The timber bridge design options were as follows: nail-laminated trail bridge, glulam trail bridge, log stringer trail bridge, and sawn timber trail bridge (USFS, 2014). Based on our assessment from our second design matrix (Appendix A, Table A-2), we decided to move forward with a multiple log stringer trail bridge. In order to make sure the design complied with necessary USFS standards, analyses were conducted as described in the Methods section of this report.

Methods

Total Station Survey and Raw Data Processing

Three site visits were conducted prior to analysis for the design. Our first site visit on October 14th, 2022 was conducted for reconnaissance to study the design of a similar bridge still standing farther upstream, which we refer to as the northern bridge, and visually assess the condition of the destroyed bridge, or the southern bridge, that our design will replace. We returned to the site on November 9th, 2022 to conduct a total station survey of four cross sections of the stream (two immediately upstream of the bridge and two immediately downstream of the bridge), existing abutments, other surrounding landmarks such as trees and pathways, and local benchmarks that we established in the field using a Topcon GTS-105N. A third site visit was made on January 25th, 2023 to conduct a GPS survey using a Trimble R12. We located the local benchmarks we had established during our total station survey. After our site visits, we corrected the GPS points using the Online Positioning User Service (NOAA's OPUS). With the corrected points, we were able to locate our total station points based on the OPUS corrected local benchmark locations.

Next, we used lidar data (VGIN, 2022) to add an additional four cross sections upstream and three cross sections downstream of our surveyed cross sections. We confirmed that the lidar cross sections were providing elevation data consistent with our surveyed cross sections by comparing our surveyed cross sections to the lidar data. Finally, a topographic surface was created from the survey points and lidar data, and we are showing this surface at 0.5 ft contour intervals.

Peak Flow Analysis

A design requirement for the bridge is to have at least 3 feet of freeboard during a 100-year flood event. In order to design for the correct height of the bridge, a hydrologic analysis was first conducted to determine the design discharge of the 100-year peak flow followed by a hydraulic analysis to determine how that flow would fill the channel.

The first step in this analysis was to estimate a 100-year peak flow value for the stream. A 100-year peak flow value of 3,020 cfs was calculated from the United States Geological Survey (USGS) StreamStats tool (USGS). This data presented by this tool is based on a relationship of "annual peak flow per square mile based on basin percent urban area and basin drainage area " (Austin, 2014). However, due to the given drainage area of 3.15 mi^2, this parameter was suspected to be an overestimate of the site at Brown Mountain Creek. This was justified by the fact that the majority of USGS stream gage equations used by this tool pertain to watersheds of much larger drainage areas. Furthermore, the overestimate could also be due to variations in land cover across the geographic region in this study. To analyze the accuracy of this estimate and to potentially determine a more accurate estimate of the 100-year peak flow value, a stream gage analysis was performed. Currently, there are no USGS stream gages on Brown Mountain Creek near the bridge site. Instead, the hydrologic analysis involved recording the 100-year peak flow values (cfs) from 14 stream gages within 80 miles of the location of Brown Mountain Creek. Their geographic locations are displayed in Figure 3. See Appendix B-1 for the detailed information on the 100-year peak flow records of each gage that were determined and analyzed using HEC-SSP software (HEC-SSP v2.3). All sites resided in the same physiographic province to mimic the geologic and geomorphic conditions of our site. Including stream gages that were within 80 miles of Brown Mountain Creek were important for maintaining consistent hydrologic conditions. It was also ideal that the drainage areas selected in this study had a similar drainage area (mi^2) as possible to Brown Mountain Creek. The values from this observation ranged from 0.37 mi^2 to 116 mi^2, where our site had a drainage area of 3.15mi^2.

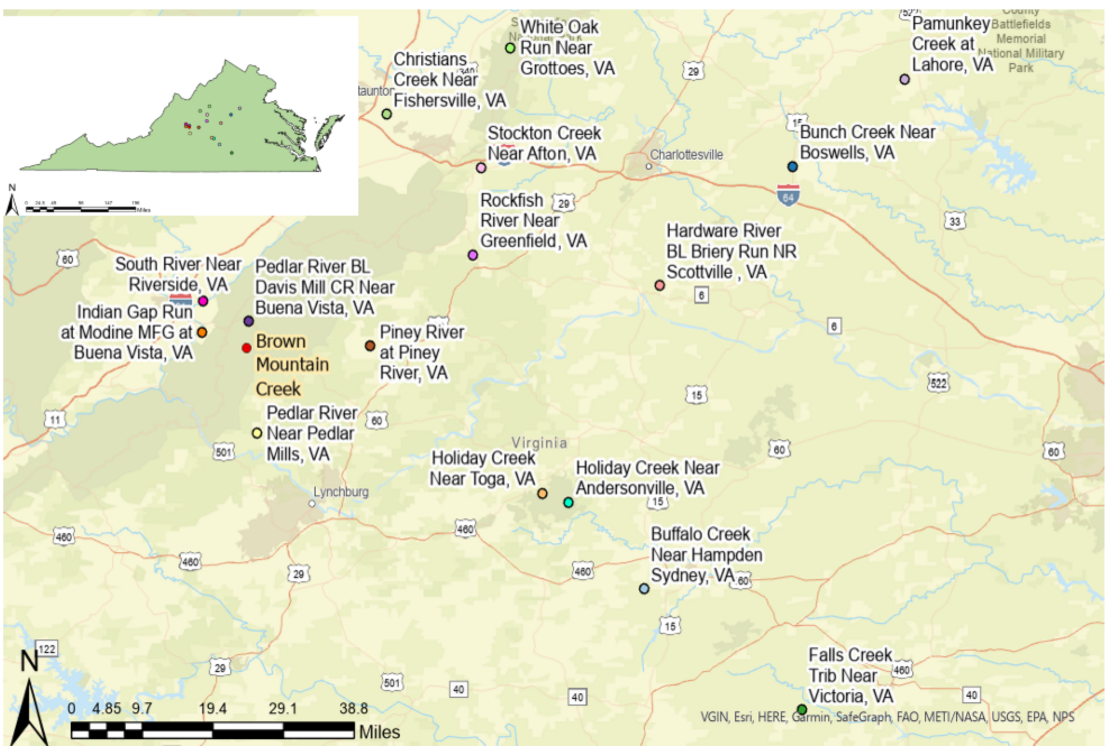

Figure 3. Stream gages involved in the peak flow analysis for Brown Mountain Creek, where all gages were within 80 miles of Brown Mountain Creek.

Once the peak flow values were calculated, a best-fit power-law relationship (linear relationship in log-log space) was established relating a USGS gage's drainage area to its respective 100-year peak flow (Figure 4). Based on this best-fit relationship, we calculated a 100-year peak flow value of 1,885 cfs. Conservatively, we rounded this value to 1,900 cfs as the 100-year peak flow value we simulated as our design flow in HEC-RAS.

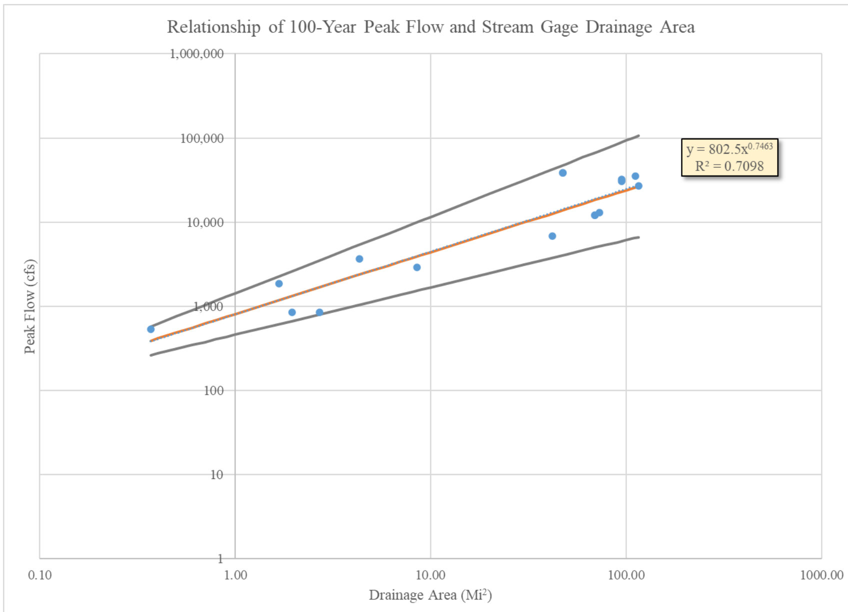

Figure 4. Relationship between drainage area and 100-year peak flow for selected USGS gages near the study site. Best fit line is shown in orange, and 95% confidence intervals on this relationship are shown in black.

The 95% confidence interval of the best-fit relationship to the stream gage data was also determined and shown in Figure 4. The upper bound of the 95% confidence interval was found to be 4,047 cfs, which is much higher than the StreamStats value of 3,020 cfs.

Geotechnical Assessment

A complete geotechnical analysis was not in the scope of this project, but both abutments were visually assessed and appeared to meet FSH 7709.56b sec. 31. The left bank abutment is a crib wall composed of wooden poles surrounding rocks that has been in place for 40 years with no visible signs of deterioration. The right bank abutment is bedrock. We believe that neither abutment is at risk of failing or scouring due to their good condition after 40 years of supporting the existing bridge. However, during construction if either abutment appears to be compromised then a more thorough geotechnical investigation should be completed to confirm load bearing capacity.

Hydraulic Analysis

The hydraulic analysis for this project was performed using the Hydrologic Engineering Center's River Analysis System (HEC-RAS) (USACE, n.d.). Within this software, we ran multiple model simulations using a one dimensional (1D) steady flow analysis. The steady flow model proceeds to calculate stages, or water levels, throughout the cross sections, while keeping the discharge constant (Ponce, 2011). Since we were only interested in a singular 100 yr peak flow event, a steady flow analysis was sufficient for our project.

The inputs needed for the hydraulic analysis included multiple cross sections upstream and downstream of our bridge. The cross sections included data for the station and elevation of the ground surface. Additionally, the Manning's roughness coefficient (n) values selected for the right-of-bank (ROB), left-of-bank (LOB), and main channel were kept the same for each cross section. The ROB and LOB Manning's n value was set as 0.2 for deciduous forest and high vegetation cover based on our observations during our field visits (USACE, 2023). We estimated Manning's n using two separate methods. The first method used a composite method based on channel characteristics: base of material, degree of irregularity, variation in channel cross section, effect of obstruction, and amount of vegetation (Arcement & Schneider, 1989). Based on the conditions of our study site, we calculated individual contributions of these channel characteristics to a composite roughness, provided in more detail by Equation B-1 in Appendix B, as 0.076 (Table 2).

Table 2. The composite Manning's roughness coefficient, where the values were chosen based on descriptive ranges from Arcement & Schneider (1989).

n_b	n_1	n_2	n_3	n_4	m
Cobbles and boulders	Minor	Alternating Occasionally	Minor	Small	318.41 m/266.94 m
0.04	0.004	0.003	0.015	0.01	1.06
				Sum = 0.072	Sum*m = **0.076**

The second method used a back-calculation method to estimate Manning's n with USGS field measurement data. The data was from the National Water Information System (USGS, n.d.), which included measured: discharge, velocity, cross-sectional area, and top width. Slope was estimated in GIS using the lidar data. Based on these USGS field measurements and the estimated channel slope, we used Manning's equation to back-calculate the Manning's roughness at each of the flows (full details in Appendix B). Because most of the streamflow measurements were made at low to moderate flows, we plotted the Manning's roughness data versus discharge to select the roughness value at high flows (at 1,900 cfs). For the three sites that we considered, we estimated the roughness value to be between 0.05

and 0.06. These two methods for determining Manning's n together give us confidence in using a conservative channel roughness value of 0.07 for our study site.

Design Storm Uncertainty Due to Climate Change

The lower cord elevation was designed to be at a height with an additional 3 feet of freeboard from the 100 year flow event. Morsy et al. (2019) mentions that climate change has caused design storms to become increasingly unpredictable, stating that climate change is impacting mid-range storms the most, with an estimated increase in design storm magnitudes. They predicted that percent increases in peak flow values are greater in smaller (<9.7 mi2) watersheds. Their study focused on Norfolk, VA, where they compared historical rainfall data to Global Climate Models (GCM) from the World Climate Research Programme's (WCRP's) Coupled Model Intercomparison Project Phase 5 (CMIP5). They modeled two emission scenarios (Representative Concentration Pathways): the intermediate scenario, RCP 4.5, and the extreme scenario, RCP 8.5. Based on this, they were able to verify a study conducted by Smirnov et al. (2018), where they suggested a 20% increase in peak flow to design storms when designing bridges and culverts.

Another study conducted by Modi et al. (2021) investigated how hydrological modeling frameworks can be used to directly assess regional flood inundation due to climate change. Their study focused on the Susquehanna River in Harrisburg, PA, where they evaluated the changes in flood inundation depth and extent between historical and future 30-year and 100-year flood events. For the extreme RCP 8.5 scenario between 2061-2090, flood volumes were predicted to increase by 57%, thus increasing flood extent by 18% and maximum depth of the floodplain by 28%. The paper used a downscaling approach of watersheds, which is why it doesn't explicitly state how peak flow values would change. The study did note that an increase in precipitation alone does not necessarily increase flood risk for future climate scenarios, as the interactions between soil moisture, evapotranspiration, temperature, and precipitation are the driving factors for changes in runoff volume and river discharge.

These studies reviewed for our design purposes were in Southeast Virginia and Pennsylvania, which show that increases in precipitation, runoff, and discharge volumes should be expected across the region and at our study site. Since increases in design storm magnitudes may be affecting the discharge volumes in smaller watersheds more than in larger watersheds (Morsy et al., 2019), we made a conservative estimate that due to climate change over the 30-year design life of our bridge, our estimated 100-year peak flow may be 20% higher, increasing from 1,900 cfs to 2,280 cfs. In addition to our design storm, HEC-RAS simulations also included the effect of climate change by including this higher flow, to verify that it would pass within the 3 feet of freeboard.

Steady Flow Simulations

Once all the inputs that were described in the previous section were prepared, two primary one-dimensional HEC-RAS simulations were run under steady flow conditions. The first model had the geometry file for the existing conditions of Brown Mountain Creek, while the second contained the existing geometry with the addition of the pedestrian bridge. The HEC-RAS model containing an overview of the existing conditions as well as the bridge design over aerial imagery is shown in Figure 5.

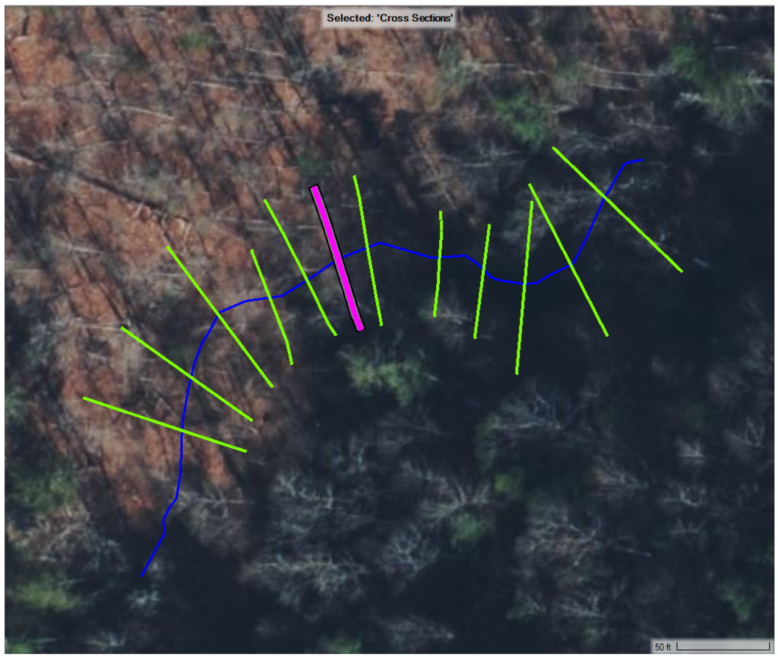

Figure 5. Aerial imagery of existing cross sections (green), the Brown Mountain Creek reach (blue) and the inputted bridge design (pink).

The hydraulic analysis primarily involved two different flow profiles: 1,900 cfs obtained from the USGS stream gage analysis, and the anticipated future flow of 2,280 cfs due to climate change. A steady flow analysis was justified for this project as it was assumed that the river reach in this study was short enough (nearly 397 feet) for the 100-year peak flow event to move throughout the model simultaneously. The model was run using a subcritical flow regime, which was confirmed by reviewing the Froude number at each cross-section and confirming it is less than one.

Results

Hydraulic Analysis Results

Shown in Figure 6 and Figure 7 are the cross-sectional and longitudinal profiles of the Brown Mountain Creek reach with the inputted pedestrian bridge design. Both profiles include the 1,900 cfs peak flow for the 100-year design storm as well as the climate-change-adjusted stream discharge of 2,280 cfs. The water surface elevation (WSE) at the bridge for a 1,900 cfs streamflow was 1,140.60 feet. For the climate change peak flow, the WSE was found to be at 1,141.43 feet. This is an additional 0.83 feet higher for the climate change flow compared to our design flow.

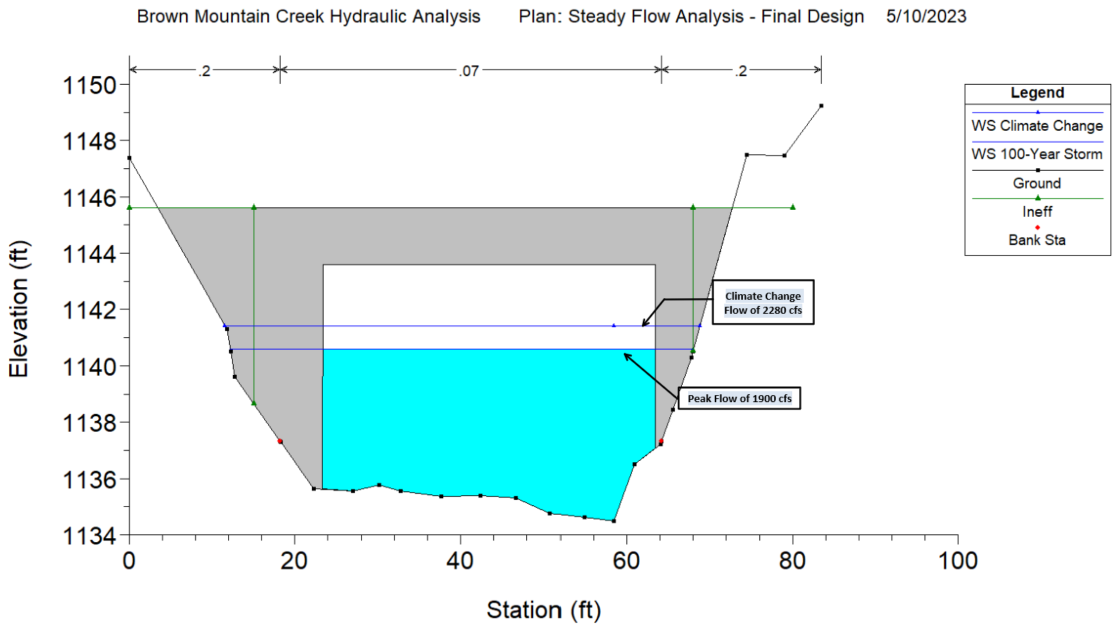

Figure 6. Cross-sectional profile including bridge design and steady flow results.

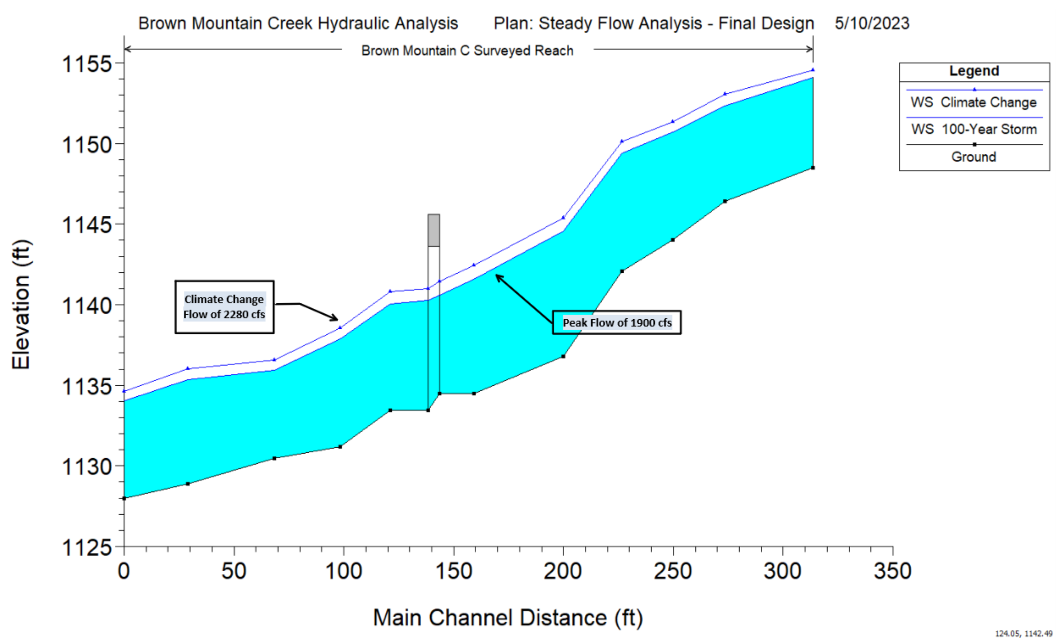

Figure 7. Longitudinal profile including bridge design and steady flow results. Flow is from right to left in the figure.

From the results of this hydraulic analysis, it was concluded that the dimensions of the pedestrian bridge design are appropriate for this site. Placing the bridge on the original crib wall on the left side of the bank as well as the naturally existing bedrock on the right side of bank would yield an acceptable elevation (having a low chord elevation at 1,144 feet) for the pedestrian bridge that allows for both the 100-year design flow and the climate change streamflow to pass under the superstructure. However, it will be necessary to increase the height of the existing crib wall by about 3 feet in order to meet these requirements. In the cross-sectional profile above (Figure 6), the constraint of having at least three feet of freeboard was met. The design flow with a WSE of 1,140.6 feet provides 3.4 feet of freeboard, exceeding the minimum 3 feet of freeboard requirement. The climate change peak flow of 1,141.43 feet allows for 2.57 feet of freeboard. An example calculation for determining freeboard is provided in Equation B-3 located in Appendix B. Furthermore, this analysis verifies that the bridge can remain in the location it was when it was destroyed in 2019 due to the placement of the original crib wall abutment and existing bedrock on the sides of the bank.

Final Bridge Design and Plan Set

The final plan set was based on the standards from the "USFS Standard Trail Plans & Specifications," and the standards in the plans are as follows:

- STD_961-20-01 (Structure Table);
- STD_961-20-02b (General Notes; Southern Pine);
- STD_961-20-03b (Typical Superstructure; Southern Pine);
- STD_961-20-04 (Cross Sections);
- STD_961-20-05 (Cross Sections/Abutments);
- STD_961-20-06 (Abutments).

The plan set begins with a cover page that includes a project location and vicinity map. The following figures are pages X-02 (Figure 8) and X-03 (Figure 9) from the full design plan set of eight pages (see Appendix C) that was delivered to our stakeholders. Following these pages the plan set includes the bridge design standards listed above.

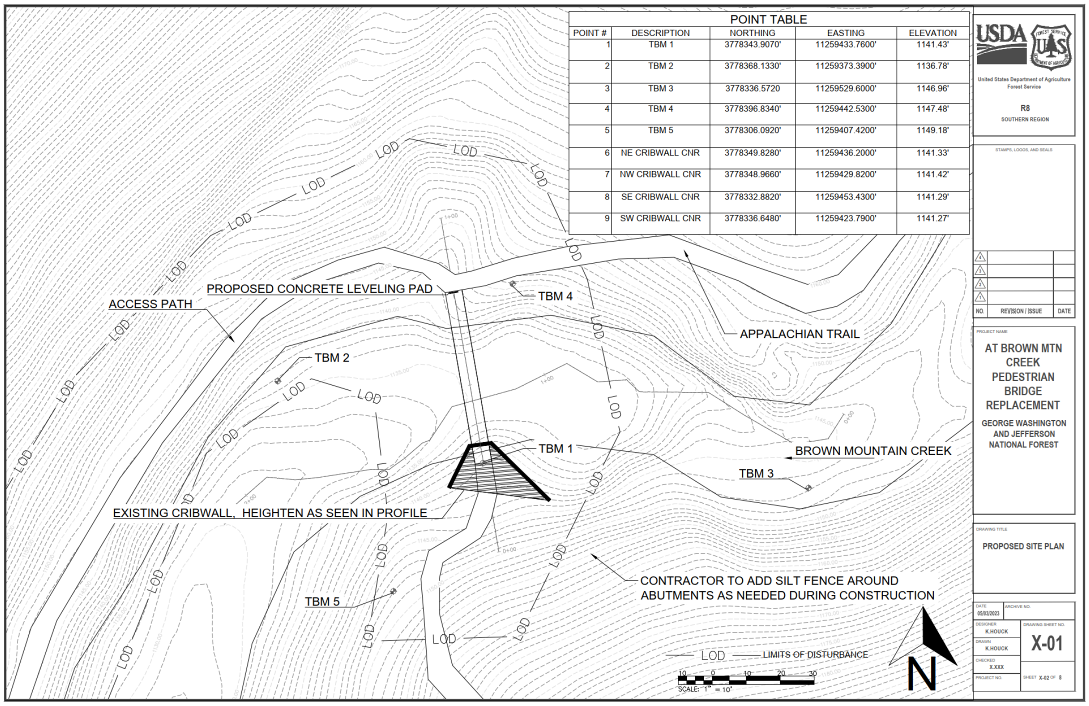

Figure 8. Page X-02 from the design plan set of the bridge location and overall site plan.

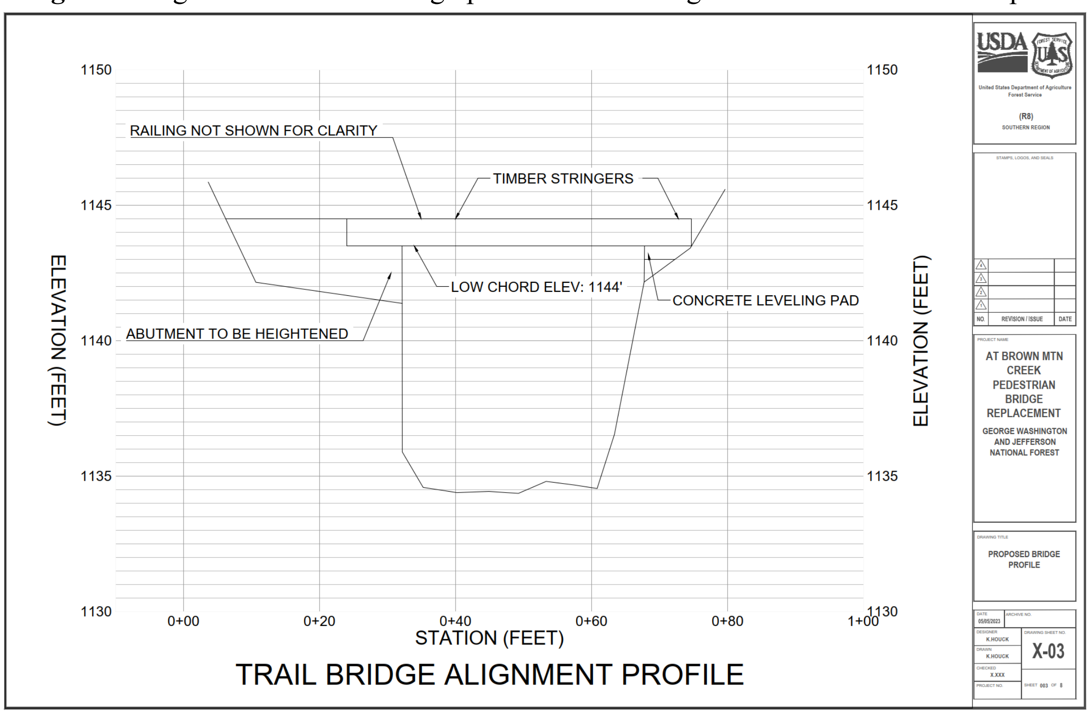

Figure 9. Page X-03 from the plan set of the alignment profile showing the elevation of the bridge.

The final bridge design dimensions are 5 ft x 50 ft, with 3.40 feet of freeboard between the design storm flow and the bottom of the bridge; the climate change simulation allowed for 2,57 feet of freeboard. The bridge material was chosen within the standard as southern pine, in order to use local wood materials.

Cost Estimate

The cost estimate for this project was provided to us, based on our design, by a local contractor with prior experience building pedestrian bridges for the ATC, Josh Thomas from High Country, LLC (2023). The cost breakdown provided to us included an estimate for clearing the trail and the old bridge and moving equipment and materials. A total lump sum for the materials was also given; the contractor suggests a 30% minimum larger sum set aside for unanticipated changes. The total cost amounted to $113,200, and the breakdown is provided in Table 3.

Table 3. The cost breakdown in categories with a description of their function and the subtotals for each.

Item	Quantity	Function	Amount	Cost Source
Mobilization, logistics	1	~11 days: clearing trail access, moving materials	$18,000	HCC, April 2023
Materials	1	Logs, boards, fasteners, and crib materials	$29,600*	HCC, April 2023
Abutment repair and estimated changes, bridge construction	1	~21 days: abutment, deck, and railing construction work	$41,000	HCC, April 2023
Contingency	1	Unexpected costs	$24,000	HCC, April 2023
Total Cost			$113,200	

*Log stringer price may vary based on project timeline due to availability and market variability.

Conclusion

We have developed a new design plan for a multiple log stringer trail bridge on the AT over Brown Mountain Creek in Amherst County, Virginia for the ATC and USFS that is approximately 5-ft wide by about 50-ft long. The bridge meets all criteria (must include 3 ft of freeboard during the peak flow of a 100-year flood event and have a design life of 30 years) and constraints (must have a rustic appearance, be accessible to transport materials and equipment, be resilient to environmental hazards, and tie into the existing abutments). While the hydraulic analysis confirmed the bridge design will meet the freeboard criterion for the 100-year design storm, it will not support the climate change value of 2,280 cfs. A geotechnical investigation is also recommended to verify the existing abutments will suffice. Per our analysis, the bridge will remain in its original location and will be constructed on the existing abutments that will be refurbished to extend the overall design life. The final cost of the project was estimated at around $113,000, and a further cost breakdown can be provided once a contractor is selected for the project.

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dredwards@vt.edu

April 28th, 2023

Gunlogson Environmental Design Student Competition
American Society of Agricultural and Biological Engineers
2950 Niles Road
St. Joseph, Michigan 49085

Dear Competition Judges,

I affirm that this undertaking is an original project conceived and completed as part of our capstone Comprehensive Design course. Neither the project nor its report is a part of any other ASABE student design competition.

Thank you for your service to ASABE and to our students.

With best regards,

Dwayne R. Edwards

Dwayne R. Edwards, Ph.D., P.E.
Professor and Department Head

Appendices

Appendix A. Decision Matrices

Table A-1. Decision matrix for pedestrian bridge construction materials.

Criteria	Weight	Steel Bridge	Concrete Bridge	Timber Bridge
Aesthetic	0.2	3	3	5
Design life	0.3	5	4	4
Cost	0.3	3	3	4
Constructability	0.1	1	1	5
Resilience to fallen trees	0.1	5	4	3
Total	1	3.6	3.2	4.2

Table A-2. Decision matrix for the comparison of different timber bridge designs.

Criteria	Weight	Nail-laminated Trail Bridge	Glulam Trail Bridge	Log Stringer Trail Bridge	Sawn Timber Trail Bridge
Aesthetic	0.2	4	4	5	4
Design life	0.3	2	3	3	3
Cost	0.3	3	3	5	4
Constructability	0.1	3	4	5	4
Resilience to fallen trees	0.1	3	3	3	3
Total	1	2.9	3.3	4.2	3.6

Appendix B. Hydrologic and Hydraulic Analyses

Table B-1. List of stream gages that were used in the 100-year peak flow analysis, and the stream flow values obtained for each of them. The stream flow values were either from HEC-SSP or from the USGS. The start and end dates mark when the flow measurements were conducted, and the number of counts identifies the number of peak flow events measured during that time range.

Stream Gage	Gage ID	Drainage Area (mi2)	Start Date	End Date	Number of Counts	SSP Peak Flow? Y/N	Value (cfs)
South River Near Riverside, VA	2023500	112.00	1936-03	8/20/1969	14	N	35,000
Pedlar River Near Pedlar Mills, VA	2025000	94.60	8/8/1942	8/20/1969	16	N	32,000
Piney River at Piney River, VA	2027500	47.70	6/18/1949	5/27/2022	74	Y	962
Rockfish River Near Greenfield, VA	2028500	94.80	7/22/1998	2/21/2023	80	Y	30,600

Stockton Creek Near Afton, VA	2030800	2.70	3/7/1967	11/1/2019	52	Y	858
White Oak Run Near Grottoes, VA	1628060	1.95	10/1/1990	9/6/1996	17	Y	853
Holiday Creek Near Toga, VA	2038840	1.67	6/21/1972	1/14/2005	33	Y	1,850
Buffalo Creek Near Hampden Sydney, VA	2039000	69.60	10/1/1990	Current	77	N	810
Hardware River BL Briery Run NR Scottville , VA	2030000	116.00	2/11/1939	3/24/2022	84	Y	26,900
Christians Creek Near Fishersville, VA	1624800	73.00	3/12/1968	3/3/1997	30	Y	13,100
Bunch Creek Near Boswells, VA	1671500	4.34	12/4/1948	9/5/1979	31	Y	3,670
Holiday Creek Near Andersonville, VA	2080207	8.54	6/23/1967	11/11/2020	55	N	2,930
Falls Creek Trib Near Victoria, VA	3010201	0.37	1/14/1968	2/16/2021	54	N	532
Pamunkey Creek at Lahore, VA	2080106	41.90	11/10/1990	12/8/2011	21	N	6,900

Equation B-1. Estimation Method to Determine Manning’s n.
The first Estimation Method used to calculate Manning’s n with, for the hydraulic analysis in HEC-RAS is as follows:

$$n = (n_b + n_1 + n_2 + n_3 + n_4) * m \tag{1}$$

where,
n_b = base n, base material,
n_1 = degree of irregularity,
n_2 = variation in channel cross section,
n_3 = effect of obstruction,,
n_4 = amount of vegetation
m = degree of meandering, $m = \frac{channel\ length}{valley\ length}$.

Equation B-2. Back-Calculation Method to Determine Manning’s n.
The equations used to back-calculate Manning’s n were as follows (Arcement & Schneider, 1989):

$$n = \frac{K*A*R_h^{2/3}*S^{1/2}}{Q} = \frac{K*R_h^{2/3}*S^{1/2}}{v} \tag{2}$$

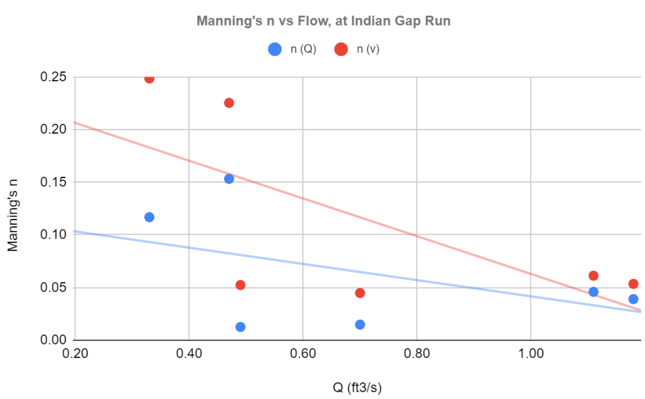
where,
n = Manning’s roughness coefficient,
K = conveyance of the channel = 1.486 for USCS,
A = cross-sectional area (ft^2),

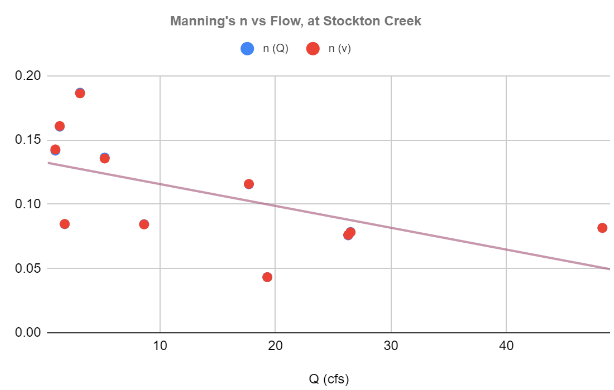
R_h = hydraulic radius, $R_h = \frac{Area}{Top\ Width}$ (ft),
S = slope, $S = \frac{horizontal\ run\ (m)}{vertical\ rise\ (m)}$,
Q = discharge (ft^3/s),
v = velocity (ft/s).

The three chosen sites at Indian Gap Run, Stockton Creek, and Holiday Creek for backtracking Manning's n were gathered like the example Table B-2 below.

Table B-2. Stream gage: Indian Gap Run at Modine MFG at Buena Vista, VA, with the Gage ID: 2024208, and drainage area of 4.10 sq mi. This gage was chosen because it is slightly larger than our drainage area of 3.15 sq mi. There was no peak flow value available from the USGS or StreamStats.

ID	Q (ft3/s)	v (ft/s)	Ax (ft2)	Top Width (ft)	Rh (ft)	Horizontal run across site (m)	Manning's n (Q)	Manning's n (v)
1	0.33	0.33	0.47	0.7	0.674	277.26	0.111	0.248
2	1.18	1.18	0.73	1.61	0.453	Vertical rise across site (m)	0.039	0.053
3	1.11	1.11	0.75	1.48	0.506	1.43646	0.045	0.061
4	0.21	NA	NA	NA	NA	Slope (m/m)	NA	NA
5	0.49	0.49	0.24	2.04	0.117	0.00518	0.012	0.052
6	0.27	NA	NA	NA	NA		NA	NA
7	0.47	0.47	0.68	0.69	0.985		0.153	0.225
8	0.21	NA	NA	NA	NA		NA	NA
9	0.7	0.7	0.33	2.08	0.158		0.014	0.044
10	0.22	NA	NA	NA	NA		NA	NA





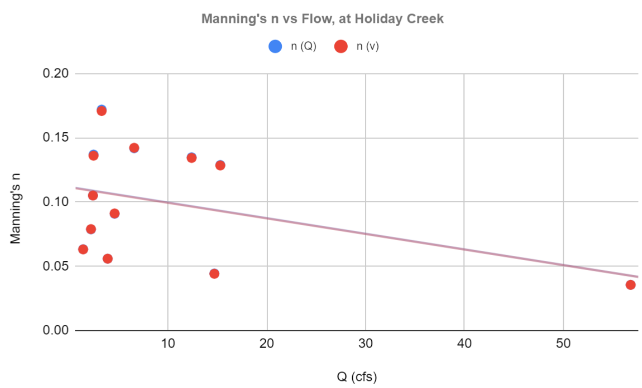


Figure B-1. Selection of Manning's n by plotting river flow versus Manning's n values, at three sites: Indian Gap Run, Stockton Creek, and Holiday Creek.

Equation B-3. Example calculation of the freeboard between the 100-year peak flow water surface elevation (WSE) and lower chord of Brown Mountain Creek pedestrian bridge.

$$Freeboard \ = Low\ Chord\ Elevation\ (ft)\ -\ WSE_{100\ Year\ Peak\ Flow}\ (ft) \tag{3}$$

$$Low\ Chord\ Elevation\ =\ 1,148.00\ ft;\ WSE_{100\ Year\ Peak\ Flow}\ =\ 1,140.65\ ft$$

$$Freeboard\ =\ 1,1480.00\ -\ 1,140.65\ =\ 7.35\ feet$$

Appendix C. AutoCAD Plan Set

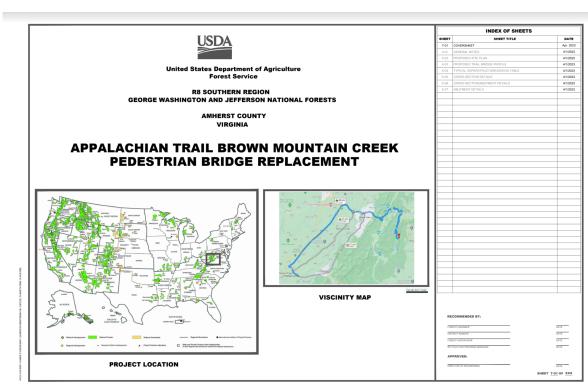


Figure C-1. Cover page for pedestrian bridge replacement AutoCAD plan set, page T-01.

Figure C-2. General notes for pedestrian bridge replacement AutoCAD plan set, page X-01.

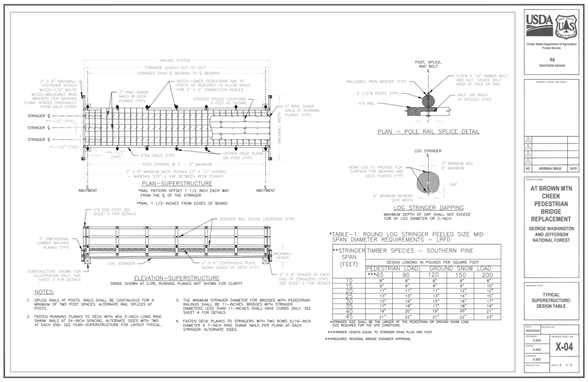


Figure C-3. USFS standard for typical superstructure of a multiple log stringer trail bridge, page X-04.

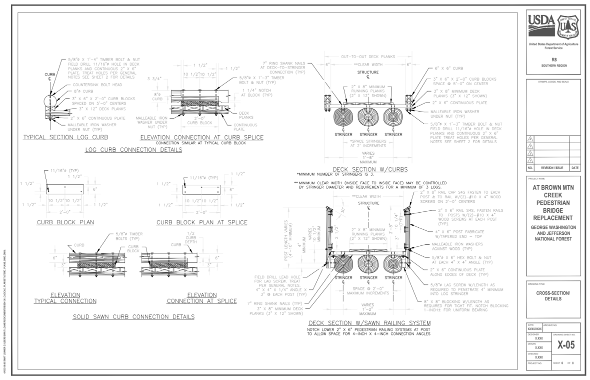


Figure C-4. USFS standard for cross section details of a multiple log stringer trail bridge, page X-05.

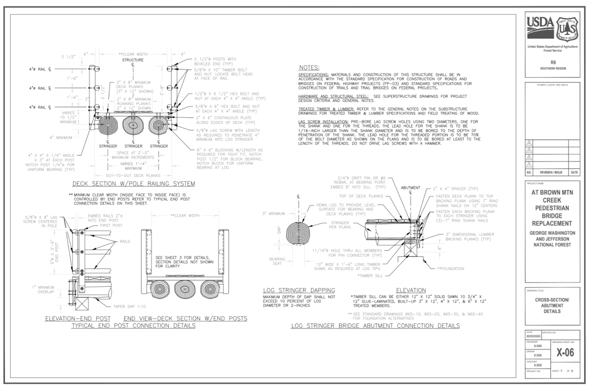


Figure C-5. USFS standard for cross section/abutment details of a multiple log stringer trail bridge, page X-06.

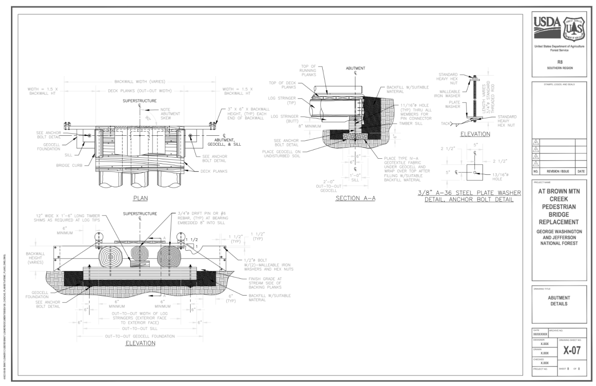


Figure C-6. USFS standard for abutment details of a multiple log stringer trail bridge, page X-07.