ENGINEERING REPORT

CULLOWHEE DAM EVALUATION
WESTERN CAROLINA UNIVERSITY
TUCKASEIGEE WATER & SEWER AUTHORITY

JACKSON COUNTY, NORTH CAROLINA

Engineering • Planning • Finance

Post Office Box 2259

Asheville, North Carolina 28802

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EXECUTIVE SUMMARY

Cullowhee Dam is an approximately 7-foot tall, 160-foot long concrete dam on the Tuckasegee River approximately 600 feet upstream of the Old Cullowhee Road/SR 1002 bridge. It impounds a shallow, approximately 3.7-acre reservoir which serves as the main water source for two public water systems: Western Carolina University (WCU) and Tuckaseigee Water and Sewer Authority (TWSA).

Field inspections and structural modeling indicate that the dam has suffered deterioration over its service life, with an undercut area possibly permitting flow of water underneath the dam, a condition that could eventually lead to sliding or overturning failure of a portion of the dam. Localized erosion at the dam's right wall permits flow around the sidewall under heavy flow conditions, which could cause slope failure and the loss of a portion of Wayehutta Road/SR 1732 to the north. The overflow crest's timber flashboards also present a periodic maintenance need that could be reduced a more permanent solution.

The existing WCU raw water intake structure, located adjacent to the dam, consists of a screening chamber and wet well constructed in 1977 containing three pumps, two of which are older than the intake structure itself, and the third of which is relatively new, having been installed in 2010. This intake structure is in good condition with a pumping capacity of approximately one million gallons per day (1 MGD). The WCU water system's raw water demands are projected to more than double from 0.290 MGD on an average day in 2017 to 0.754 MGD on an average day in 2067, with a 2067 peak day demand of 1.462 MGD.

The existing TWSA raw water intake structure, located approximately 500 feet upstream of the WCU intake, consists of a wet well and screen constructed in 1994. Its two pumps are capable of pumping 1.5 MGD each. This intake structure is also in good condition. The TWSA water system, which serves off-campus areas of Cullowhee as well as the nearby towns of Sylva, Webster, and Dillsboro, is expected to see an increase in average day raw water demands from 1.111 MGD in 2017 to 2.116 MGD in 2067, with a 2067 peak day demand of 3.057 MGD.

The alternatives analysis for this project calls for three overall alternatives for responding to the dam's condition, one no-action and two intervention, with each of the intervention alternatives containing sub-alternatives as follows:

1. No Action	$0
2. Repair the Dam	
A. Address Safety Concerns Only	$500,000
B. Address Safety and Maintenance Concerns	$900,000
C. Incorporate Downstream Improvement Plans	$1,000,000
3. Remove the Dam	
A. Construct Two Separate Replacement Intake Structures	$7,000,000
B. Construct a Shared Replacement Intake Structure	$5,000,000

(The costs for Alternative 3, removal of the existing dam, do not include the loss of the dam itself, an asset with an estimated value of $1,500,000.)

Alternative 2.B, to repair the dam and address safety and maintenance concerns, is the recommended alternative, with a preliminary opinion of total project cost, including design, permitting, contract administration, and construction management, of approximately $900,000. This alternative increases dam stability without increasing the vulnerability of the public water supply systems. A preliminary opinion of total project cost for a variation of this alternative, Alternative 2.C, providing a 6-inch higher crest with a slot at the current normal pool elevation for potential kayak passage, is approximately $1,000,000. The No Action alternative is not recommended due to the continued risk of failure to the dam and adjacent road. The dam removal alternatives are not recommended due to the increased vulnerability of the public water supply systems, their high capital costs, and unknown but potentially costly regulatory consequences, including but not limited to the possible requirement of additional pre-treatment facilities at the WCU water treatment plant.

INTRODUCTION

The Tuckaseigee River extends from its headwaters approximately ten river miles upstream of the Town of Cullowhee in Jackson County, North Carolina to its mouth at Fontana Lake in Swain County, North Carolina. It serves as a primary water source for WCU and TWSA, which provides water to the Towns of Dillsboro, Sylva, and Webster as well as outlying Jackson County communities.

The Tuckasegee River has been the site of low head dams for centuries. The remains of early Native American stone dams can be seen throughout the river basin. As European settlers expanded through the region, dam construction continued, with low head dams providing head to drive mills and later provide hydroelectric power to local communities.

The dam located on the Tuckasegee River near Cullowhee approximately 600 feet upstream of Old Cullowhee Road was constructed in 1930 by Biltmore Concrete Company to replace a log dam that washed away in 1928. The remains of a grist mill that was the original site of WCU's first hydroelectric generator can be seen on the north bank of the stream immediately downstream of the dam. Adjacent to the south end of the dam, a powerhouse originally used for hydroelectric power and later as an intake structure for WCU's water system still stands. The dam was damaged in a 1940 flood, which washed away either the span or approaches of every bridge across the Tuckaseigee River in Jackson County. After its repair it has continued to function with minimal maintenance.

The dam is not currently used to provide head for power generation, but still serves two important purposes: it impounds water at a sufficient surface elevation to allow the function of the WCU and TWSA raw water intakes, and it lowers the flow velocity of the river, allowing some turbidity to settle out of the raw water prior to withdrawal.

The purpose of this study is to project future demands for the two raw water intakes; evaluate the existing dam structure, recreational facilities, and water systems; review regulations associated with dam improvements or removal; evaluate water quality and quantity issues and cultural and natural resources in the Tuckasegee River watershed; and analyze costs and other consequences associated with repair or removal of the existing dam.

CURRENT CONDITIONS

The project area's infrastructure components consist primarily of three elements: a low head dam, the WCU intake structure, and the TWSA intake structure. The fourth element of the project area to be discussed in this section is the Tuckasegee River itself.

I. Cullowhee Dam

A. General

The existing concrete dam was completed in 1930 on the Tuckasegee River, and has provided a dependable source of raw water for Western Carolina University since 1975, and for the Tuckaseigee Water and Sewer Authority since 1997 (Sylva Herald, September 21, 2016). The dam was originally constructed to provide hydroelectric power for WCU from a single turbine located within a brick powerhouse on the left abutment (looking downstream). With a structural height of less than 8 feet and a classification of Low Hazard, the dam is not currently regulated by the State of North Carolina. We estimate from available LiDAR (Light Detection and Ranging) data that the dam impounds a small reservoir along the river course with a surface area of less than 3.7 acres, and an estimated length of less than a quarter mile. The existing dam replaced an L-shaped (in plan view), timber crib dam constructed in the early 1920s that was later destroyed by a large flood in August 1928. Since construction, six hydropower developments have been built above Cullowhee Dam, including Cedar Cliff on the East Fork, and the Tuckasegee and Thorpe plants on the West Fork, each about six miles upstream and operated by Duke Energy (Sylva Herald). The structure is currently owned and operated by WCU as a run-of-river dam. A plan, including downstream elevation and cross-section of the dam, was prepared for this assessment based on available information and is provided as Figure 1 below. A recent photograph of the dam is provided as Figure 2.

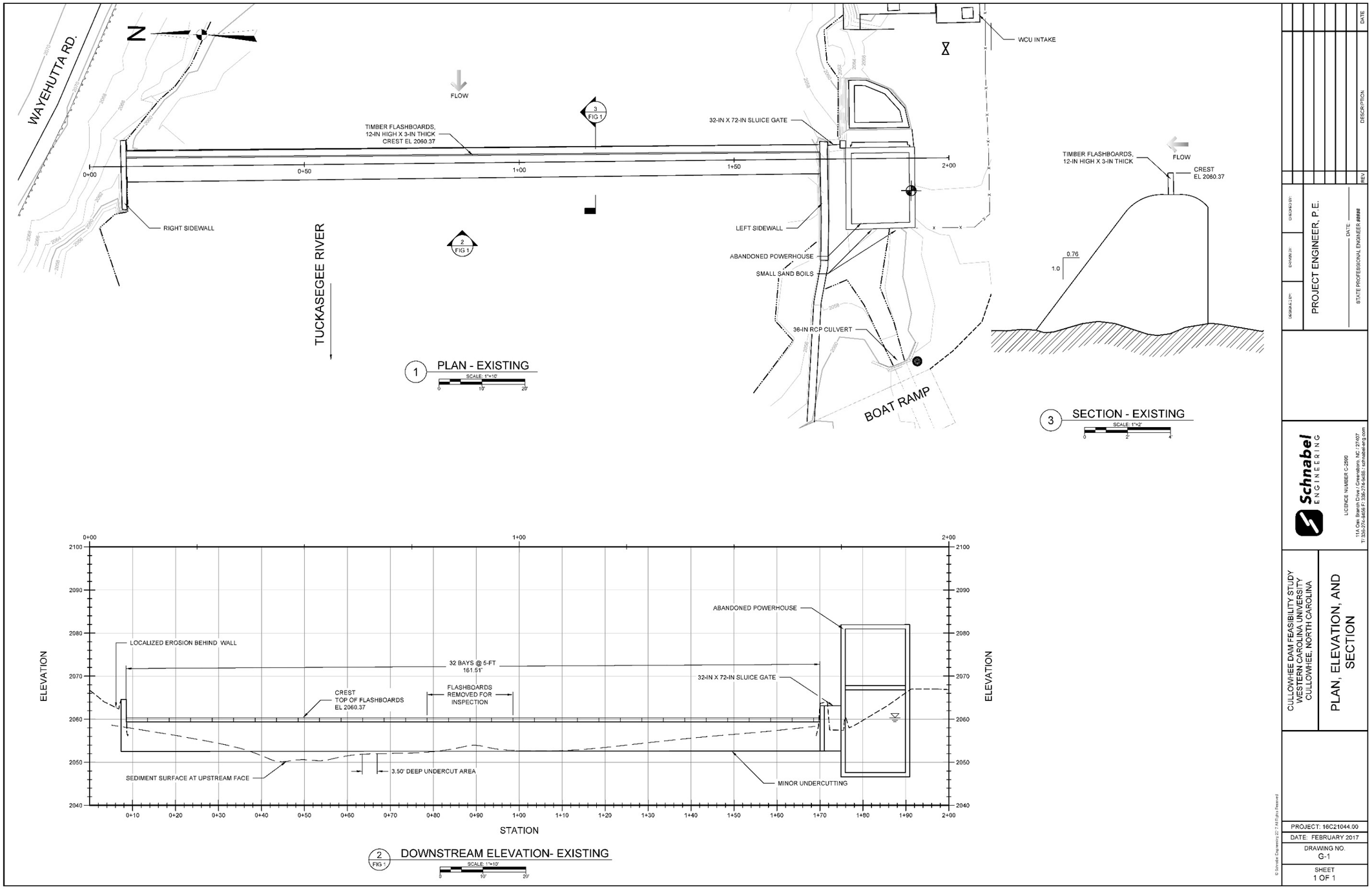

Figure 1. Cullowhee Dam Drawing

Figure 2. View of Cullowhee Dam

B. Overflow Section

The concrete overflow section (shown in Figure 3) is reported to have been founded on the "solid rock of the river bed" at depths of 7 to 8 feet below the dam crest, although the south (left) abutment beyond the powerhouse consisted of soil and was breached during flood flows on August 30, 1940 and subsequently restored. The 1940 flood was reported to peak at 18 feet above the dam crest, almost completely submerging the powerhouse on the left abutment and filling the tailrace with sand (Vanderhoof, 1941). The 1941 engineering report includes a scaled cross-section of the overflow section, indicating an approximate height of 7 feet, with a vertical upstream face, a curved crest, and an estimated 0.76H:1.0V downstream slope (based on scaled measurements). The report includes a reference to 2-foot high flashboards on the dam crest both in the text and on the drawing, and includes an allowance for "flashboards and holders" in a cost estimate to extend the dam through the breach to the south. The accompanying plan view of the dam in the report scales to a length of approximately 160 feet, which is consistent with our estimate of 32 flashboard bays with a length of 5 feet each. The estimate to extend the dam "of the same shape and elevation" appears to be based on a cross-sectional area of the overflow section of about 56 ft^2, which again is consistent with the cross-section shown on the accompanying drawing. Finally, the amount of cement in the concrete mix assumed by the 1941 report to extend the concrete dam to the south is about 590 lb/yd^3 (with correspondingly larger quantities of sand and crushed stone) which would have produced a relatively strong and durable concrete. While no information is available on the

actual concrete mix used in 1930, a design compressive strength of at least 3,000 psi is assumed based on the construction period and performance. Although the proposed concrete dam extension to the south was never constructed, the 1940 breach was repaired with earthen materials and operations were restored the following year.

Figure 3. General Alignment of Cullowhee Dam

Our field inspection on January 2, 2017, required the removal of flashboards from four of the 32 bays (shown on Figure 4), which was performed by In-Water Services of Greensboro under contract to WCU. These flashboards had been originally furnished and installed by In-Water Services in April 2016 near the middle of the overflow section (between stations 0+80 and 1+00, as shown on Figure 1), and consisted of three timber planks each 12 inches wide, 3 inches thick, and 10 feet, 4 inches long. The planks were bolted to existing pipe supports to span the four 5-foot openings on the dam crest, replacing older flashboards damaged or destroyed by previous floods. The total 20-foot wide opening was expected to pass between 70 and 80 cubic feet per second (cfs) below the top of the remaining flashboards; however, the river flow during the inspection exceeded this capacity, resulting in overtopping of the flashboards by about one inch across the remaining 140-foot length, for an estimated additional 15 to 20 cfs. Duke Energy had agreed in advance to suspend their normal daily upstream releases for peaking power, so that only base flow was passed through the dam-site during our inspection, which occurred between 10 am and 3 pm. The nearest (upstream) river gage (USGS 03508050) represented only 147 mi^2 of the estimated 207 mi^2 total drainage area at the dam site (see Figure 21), and did not include flows from Wayehutta and Caney Fork Creeks, which are ungaged.

Figure 4. Flashboards Removed from Mid-Section (4 Bays) of Cullowhee Dam

The field inspection of the concrete overflow section included visual examinations to the extent possible at selected areas of the downstream face, wading along the downstream toe to identify undercut areas, manual probing of undercut areas with a 4-foot long rod, and wading along the upstream face to determine sediment depths. A plywood sheet was used by In-Water Services at the upstream face to divert overtopping flow away from selected areas to permit observation of the concrete surface, which was found to be in generally satisfactory condition, but with isolated pockets of abrasion damage exposing larger cobbles used in the concrete (as shown on Figure 5). One or two irregular lift lines appeared to be present through the overtopping flow, but no evidence of disbonded or open joints were observed at the selected areas. Although a 160-foot-long concrete structure would be expected to require vertical joints or display vertical cracks (at least at the quarter-points), none were observed. Independent structural measurements and concrete strength testing could not be performed as originally planned due to the overtopping flow. The raw water intake and pumping station drawings indicate the top of weir (overflow section) to be at elevation 2060.75 (NAVD88), which is believed to represent the top of the 12-inch high flashboards. Survey data collected following our field inspection indicated the top of the flashboards to be at elevation 2060.37 (or about 0.4 feet lower), which has been adopted for our project drawing (see Figure 1) and structural analyses.

Figure 5. Close-up View of Unwatered Downstream Face of Dam

In-Water Services (Mike Brian) reported a large undercut area at the downstream toe of the overflow section approximately 60 feet from the right abutment (station 0+65, as shown on Figure 6), which extended at least 3.5 feet upstream beneath the downstream toe of the concrete structure. This area also generally coincided with an area of little or no sediment upstream, and may have originally represented the thalweg (line of lowest elevation within a watercourse) of the river. The undercut was estimated to be approximately 3-feet wide and 1.5-feet high at the toe, and appeared to be within the structure foundation. No other evidence of structure undercutting was observed, other than a reported 3 to 4 inches near the left abutment (station 1+50), which would be considered minor. Tailwater was generally waist-deep for the majority of the overflow section, with somewhat shallower depths closer to the abutments. In-Water Services also measured the depths below the reservoir water surface to the top of the impounded sediment at 5-foot intervals along the upstream face of the overflow section, which were recorded and are shown on Figure 1. Again, shallower water depths were reported closer to the abutments. Two samples collected of the reservoir sediment were visually classified as silty fine sand near the powerhouse intake and fine to medium sand closer to the overflow section.

Figure 6. In-Water Services Personnel at Location of 3.5-Foot Deep Undercut Area

Due to the overtopping flow, the current condition of the remaining flashboards and pipe supports could not be determined; however, significant leakage was observed below the flashboards at the concrete crest. WCU reported that selected flashboards have been replaced in the past, but that no records have been kept for the specific board locations. The flashboards replaced by In-Water Services in April 2016 had reportedly been missing for up to 8 months. Although the 1941 engineering report described 2-foot high flashboards, the existing flashboards appear to all be approximately 12 inches high (plus or minus one inch) and attached to steel pipes believed to be socketed into the concrete crest. The pipe supports are not much taller than the existing flashboards, but appeared to vary in diameter. The timber flashboards were typically attached to the supports using U-bolts. It is unknown why the height of the flashboards was reduced, or when it was done. Since the reservoir level, and available head, is directly dependent upon the height of the flashboards, it is likely the shorter flashboards were first installed sometime after power generation ceased, but before raw water diversions began.

C. Right Sidewall

An 18-inch thick concrete wall is located at the right end of the overflow section (as shown on Figure 7), extending approximately 2.5 feet upstream from the upstream face, and having a total length of 16 feet, 8 inches. The top of the wall is nearly 5 feet above the concrete crest, or nearly 4 feet above the top of the 12-inch high flashboards. Flows more than about one foot above the top of the flashboards, or greater than approximately 700 cfs, could produce flow behind the wall. Significant erosion has already occurred behind the wall and around a large boulder that may have slid downslope to rest against the outside face of the wall. It would appear that sustained erosion could undercut the adjoining hillslope and affect Wayehutta Road. Flow records suggest that Duke Energy could release flows exceeding 700 cfs on a daily basis at certain times of the year. The inside face and downstream end of the wall, downstream of the overflow section, was severely damaged due to abrasion and/or concrete deterioration. There was no evidence of reinforcing steel, and structural details of the wall (including the possible presence of a footing) are unknown. Thinner concrete walls were observed downstream of the right sidewall along the river bank, presumably from the foundations of structures originally located there and associated with the original timber crib dam.

Figure 7. Concrete Sidewall and Right Abutment of Cullowhee Dam

D. Left Sidewall and Sluice Gate

The left sidewall consists of a low concrete wall with a stone masonry cap, and is located between the left end of the concrete overflow section and a steel sluice gate. The concrete wall is about 12 inches thick and extends less than a foot above the top of the flashboards. The stone masonry cap is about 24 inches thick and extends about 3 feet above the top of the concrete wall, or to elevation 2064.53 (NAVD88), based on raw water intake and pumping station drawings, which appears to be the same as for the right sidewall. Much of the stone masonry on the outside of the concrete wall, within the sluice gate channel, is missing, leaving portions of the masonry cap unsupported (as shown on Figure 8). The sluice gate measures approximately 32 inches wide and 72 inches high, and consists of a thin steel sheet with a welded ring at the top (as shown on Figure 9). The sluice gate has apparently not been opened in many years, but appears to be intact and exhibits minor leakage at the gate seat and lower corners. The gate invert appears to be less than 3 feet below the reservoir water surface, and if removed would likely discharge less than 40 cfs at the normal pool level. The left sidewall currently terminates about 10 to 12 feet downstream of the powerhouse, but appears to have once extended much further downstream, with remnants

of a stone masonry wall visible downstream of the newly-constructed boat ramp for a distance of up to 150 feet.

Figure 8. Loss of Masonry Support Below Top of Wall on Left Abutment

Figure 9. Close-up View of Downstream Face of Sluice Gate on Left Abutment

E. Left Abutment Powerhouse and Intake

According to the 1941 engineering report (Vanderhoof, 1941), the hydroelectric plant was rebuilt in 1934 and contained a Leffel turbine, A-C generator of 125 kVA capacity, exciter, speed changer, speed and lubricant controls, and associated switch board controls. Available drawings and field measurements indicate the powerhouse structure to be 18 feet 9 inches long (upstream to downstream) and nearly 16 feet wide, with a structural height of nearly 36 feet above its foundation (approximately 6 feet below the dam foundation). The powerhouse superstructure consists of brick walls with a 4-foot wide entrance door on the south side, and two windows each on the north and west sides, with a concrete substructure. Flows would enter an upstream 12-foot wide trash racked intake structure parallel to the river (as shown on Figure 10), and pass through two 6-foot wide gated openings to the turbine. The intake and powerhouse substructure currently appear to be plugged with sediment, and there was no evidence of the draft tube outlet at the downstream face of the structure (likely due to its depth). This was confirmed by In-Water Services in April 2016 when they performed an underwater inspection of the site. The 2004 inspection report (Sutton-Kennerly & Associates, 2004) indicated that hydroelectric power generation continued until the late 1960s. Much of the original generating equipment appears to remain intact. Numerous sand

boils were observed at the downstream face of the powerhouse below about a foot of water depth (as shown on Figure 11), suggesting a hydraulic gradient between the inside and the outside of the structure with some interconnection. This is not of any concern, given the concrete structure is founded deep into bedrock and that some piping of sediment may be occurring from a buried structure opening under a few feet of head.

Figure 10. Powerhouse Intake and Left Abutment of Cullowhee Dam

Figure 11. Sand Boils at Downstream End of Powerhouse

II. Western Carolina University Raw Water Intake

Constructed in 1977, the WCU intake structure consists of three chambers: an 18.5’ x 14’ screening chamber containing both a bar screen and a traveling screen, a 10’ x 14” wet well, and a valve vault. (See Figure 12, valve vault not shown.) The wet well invert is 4’11” lower than the inlet, allowing the pumps to remain sufficiently submerged to operate at any water surface elevation that permits the screens to function.

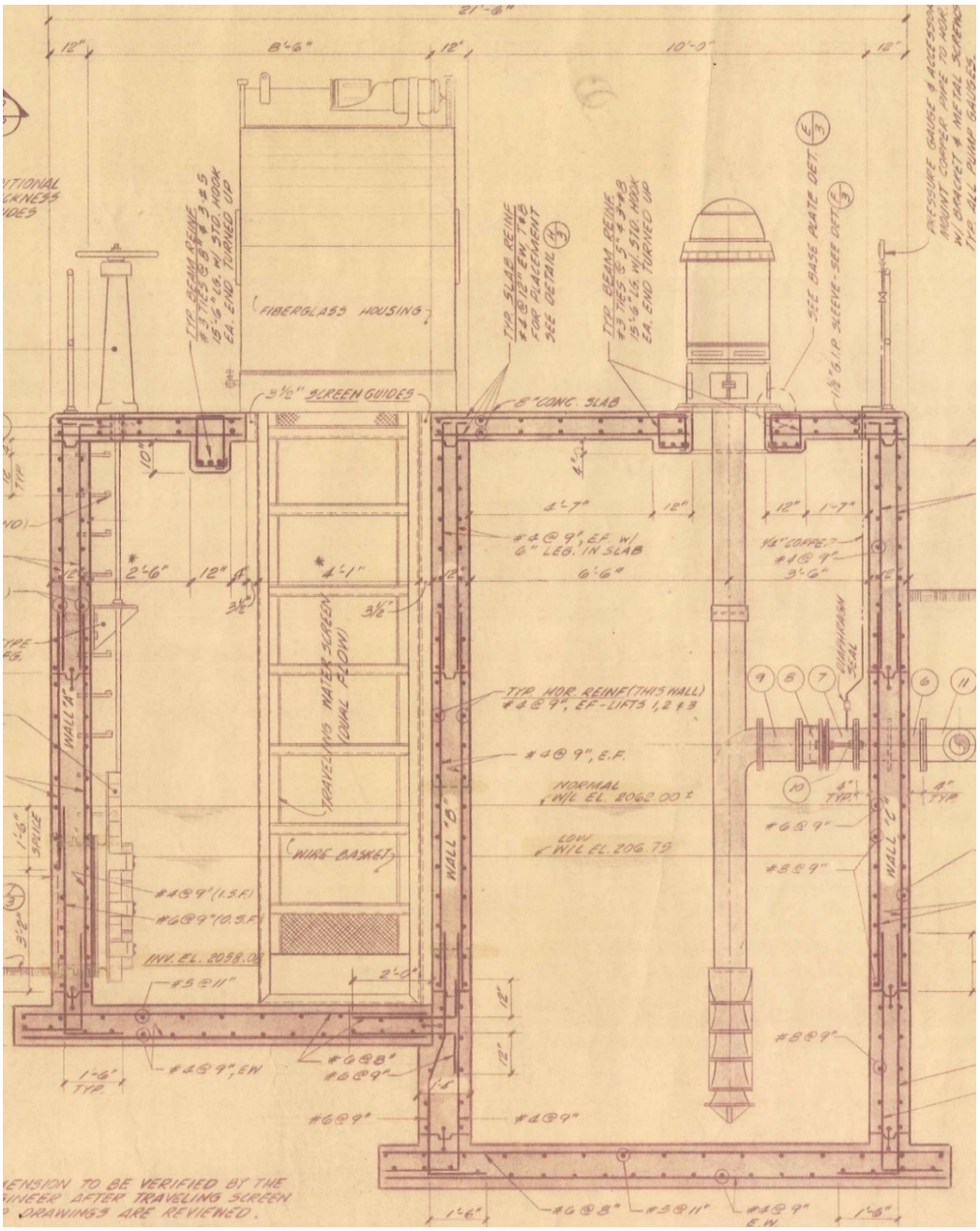

Figure 12. Section View of WCU Intake Structure

Raw water enters the screening chamber through a bar screen with an invert elevation of 2058.94' and an effective width and height of 3'1". The bar screen itself consists of eleven 2" x ½" bars. The bar screen is shielded from river flows by a screen gate that is intended to protect the bar screen from floating debris as shown in Figure 13.

Figure 13. WCU Intake Bar Screen

The screening chamber contains a manually operated traveling screen oriented perpendicular to the bar screen that catches debris and discharges it back to the river. The traveling screen was apparently intended to operate automatically and be triggered by head loss, but is currently operated locally approximately once per week by WCU staff.

Figure 14. WCU Intake Traveling Screen and Pumps

The WCU intake wet well has an invert elevation of 2054.06' and is drained by three pumps: two Worthington 30 hp 6-stage vertical turbine pumps that were relocated to the current WCU pump station in 1977 with a combined 1 MGD capacity (one pump is capable of pumping 550 gallons per minute (gpm) alone and the other is capable of pumping 500 gpm), and a newer and larger Ruhrpumpen 50 hp 6-stage vertical turbine pump installed in 2010 capable of pumping 690 gpm alone. The pumps, control panel, and traveling screen, (shown in Figure 14), are located outside but are protected from tampering by a chain link fence around the intake building. There is no backup generator for this pump station, however WCU does have the ability to connect a portable generator available on campus.

The intake is functioning adequately at present despite the age of two of its pumps. If conditions do not change it is likely to continue functioning until an expansion is needed. At that time, the wet well and screens are likely to continue to be adequate to permit inflow of raw water so that only pump replacement and piping modifications would be necessary. A more detailed in-stream study should be performed before any capacity increase is designed.

III. Tuckaseigee Water and Sewer Authority Raw Water Intake

The TWSA intake structure was constructed in 1994 with a single chamber design consisting of a 12’8” x 16’ concrete wet well whose screen takes in flow directly from the Tuckasegee River. Its controls and mechanical equipment are housed in an enclosed room directly above the wet well.

Figure 15. TWSA Intake Structure and Generator

Raw water flows into the wet well through a 4’ wide, 5’ tall aluminum mesh screen with an invert elevation of 2057.76’, as shown in Figure 16. An aluminum plate can be raised or lowered behind the mesh screen to block influent flow to permit dewatering of the wet well for maintenance. The wet well invert is 3’ below the screen invert at 2054.76’.

Figure 16. TWSA Intake Screen and Wet Well

The wet well is drained by two 100 hp vertical turbine pumps, each capable of pumping 1,050 gallons per minute. The pumps and controls can be powered by a backup generator located on an elevated platform behind the intake building (visible in Figure 15).

This intake structure is also functioning adequately. With proper maintenance it is likely to continue to perform acceptably until increased demand necessitates expansion. At that time, the existing screen should be capable of permitting sufficient raw water inflow so that pump replacement will be the only necessary modification. As with the WCU intake, an in-stream flow study should be performed prior to any capacity increase.

IV. Tuckasegee River

A. Classification

According to the 2014 NCDEQ Division of Water Resources Integrated Report, from the West Fork Tuckasegee River to a point 0.6 mile upstream of Cullowhee Dam, the Tuckasegee River, Stream Index 2-79-(29.5), Little Tennessee River basin, is currently classified as WS-III,B;Tr (water supply, primary and secondary recreational use, trout waters). WS-III indicates these waters are used as sources of water supply for drinking, culinary, or food processing purposes where a more protective WS-I or II classification is not feasible. WS-III waters are generally in low to moderately developed watersheds. Class B indicates that these waters are protected for uses such as secondary recreation, fishing, wildlife, fish consumption, aquatic life including propagation, survival and maintenance of biological integrity, and agriculture. Secondary recreation includes wading, boating, and other uses involving human body contact with water where such activities take place in an infrequent, organized, or incidental manner. Additionally, these waters are protected for primary recreation use. Primary recreational activities include swimming, skin diving, water skiing, and similar uses involving human body contact with water where such activities take place in an organized manner or on a frequent basis. Trout waters is a supplemental classification intended to protect freshwaters which have conditions that shall sustain and allow for trout propagation and survival of stocked trout on a year-round basis. From a point 0.6 mile upstream of Cullowhee Dam to Cullowhee Dam, the Tuckasegee has an additional classification of Critical Area. Critical Area is the land adjacent to a water supply intake where risk associated with pollution is greater than from remaining portions of the watershed.

Figure 17. Water Supply Critical Area

B. Withdrawals and Releases

Although Duke Energy withdraws several million gallons per day (mgd) from the Tuckasegee River through hydroelectric power plants, the discharges from these facilities result in no net withdrawal from the basin. Old Edwards Golf Club reported an average daily withdrawal of 0.141 mgd from a pond on that property. Trillium Links and Lake Club reported an average daily withdrawal of 0.250 mgd from Norton Creek (a tributary to the Tuckasegee River) and 0.090 mgd from an irrigation pond. The sum of these withdrawals is minimal in comparison to the flow in the Tuckasegee River.

The Tuckasegee River's tributaries upstream of the Cullowhee dam are controlled by a total of six dams. Duke Energy administers those six dams according to the terms of two 30-year licenses granted by the Federal Energy Regulatory Commission (FERC). Releases from the dams upstream of Cullowhee are prescribed by the terms of the East Fork and West Fork Hydro Project licenses and the Tuckasegee Cooperative Stakeholder Team Settlement Agreement signed by Duke Energy and 16 other stakeholders in October 2003. The licenses and settlement agreement

state the minimum flows Duke Energy is expected to release from each dam in the forks of the Tuckasegee River in order to meet environmental, recreational, power generation, and water supply concerns, as well as a Low Inflow Protocol that permits staged reductions in lake levels, power generation, and river flows in the event of a drought.

A letter sent from Duke Energy on March 13, 2017 commenting on the potential removal of the dam outlined these target flows and stated the following warning regarding Duke Energy's future target flows and contractual obligations to stakeholders:

"*WCU can certainly make its own decisions regarding the future of Cullowhee Dam. While Duke Energy has no direct interest in any choice WCU makes concerning Cullowhee Dam, we believe the Tuckasegee River's relatively small size and the inherent flow limitations within the FERC Project Licenses should be strong considerations in the decision-making process. Three years of negotiations with stakeholders were required to develop the Settlement Agreement which established the current balance of lake levels, recreation flow releases, continuous minimum flow releases and generation for electric customers. If Cullowhee Dam is removed only to find out later the Tuckasegee River does not have enough water flow during certain periods of the year to meet the growing water supply needs of WCU, TWSA and other entities, Duke Energy would not be able to support releasing higher prescribed flows to meet those needs. In fact, Duke Energy and other signatory parties to the Settlement Agreement are contractually prevented from supporting flow prescriptions which are inconsistent with the Settlement Agreement (see Paragraph 12.2 in the Settlement Agreement). Rebalancing the water uses in the Tuckasegee River would require reopening and successfully renegotiating the Settlement Agreement's water balance, followed by revising the Water Quality Certifications issued by the North Carolina Department of Environmental Quality and finally reopening and amending the FERC Project Licenses. Based on the expected degree of difficulty, existing knowledge and operating experience, Duke Energy could not support pursuit of a different water balance than the one currently provided by the Settlement Agreement and FERC Project Licenses.*"

Effectively, Duke Energy has stated that they cannot be expected to exceed the target flow releases given below, and that in the event of drought the Low Inflow Protocol does not ensure that Duke Energy must even meet those targets. An examination of United States Geological Survey (USGS) data later in this study demonstrates that the potential for releases of less than these target flows has been realized on multiple occasions.

Table 1. Duke Energy Target Flows

Project	Dam	Target Flow Releases	Purpose	Target Location	Timing
East Fork	Cedar Cliff	about 500 cfs	Recreation	Dillsboro	10:30 AM to 4:30 PM, ~40 days/year
East Fork	Cedar Cliff	35 cfs	Aquatic Habitat	Downstream of Cedar Cliff Dam	Continuous July 1 - November 30
East Fork	Cedar Cliff	10 cfs	Aquatic Habitat	Downstream of Cedar Cliff Dam	Continuous December 1 - June 30
West Fork	Tuckasegee	about 200 cfs	Recreation	Dillsboro	10:30 AM to 4:30 PM at Dillsboro ~40 days/year
West Fork	Tuckasegee	20 (or inflow, depending on conditions)	Aquatic Habitat	Downstream of Tuckasegee Dam	Continuous

At least eight named and several unnamed tributaries flow into the Tuckasegee River downstream of the Cedar Cliff and Tuckasegee dams, which should increase the available flows downstream of those dams as a result of the expansion of the drainage area as the river continues downstream.

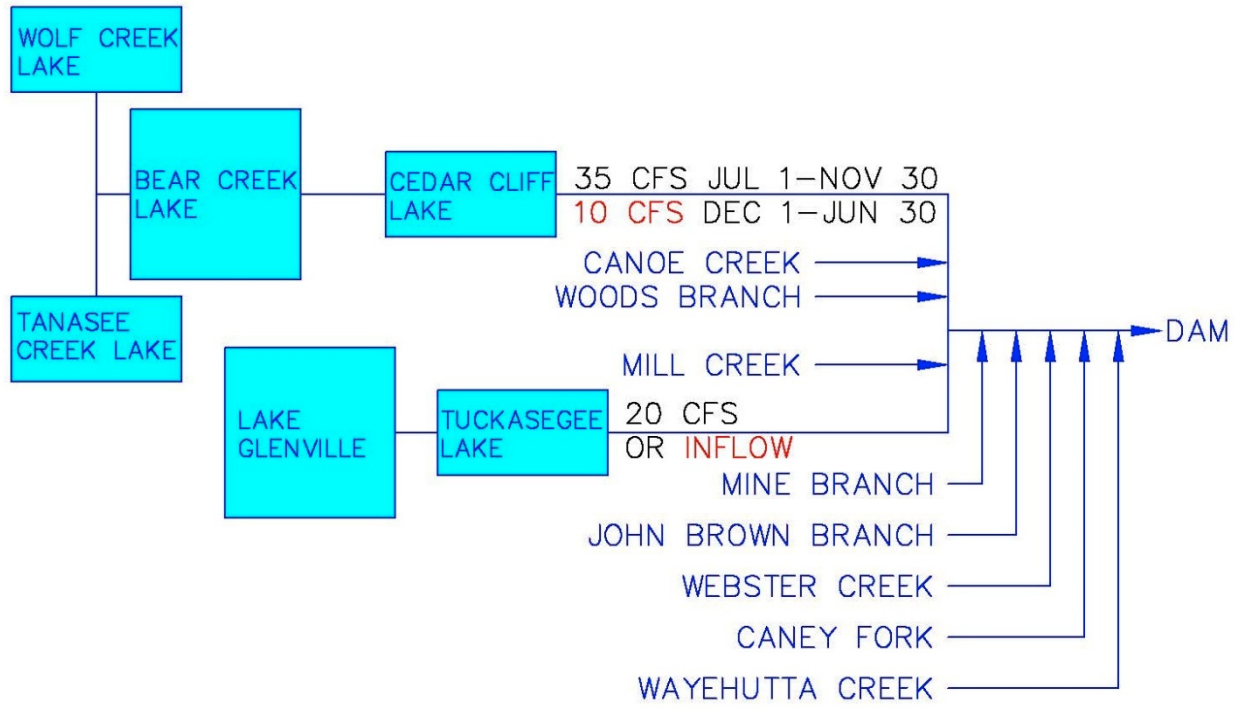

Figure 18. Tuckasegee River Flow Schematic

USGS stream gage 03508050, located approximately 6.25 river miles upstream of the dam at SR 1172, has complete daily flow data available online from September 1, 2004, to March 16, 2017, a period which includes significant floods and droughts. While mean daily flows at the gaging station are approximately 396 cfs, which would imply that even higher flows should be observed at the Cullowhee dam, flows vary significantly from day to day. See Figure 18 and Figure 19 for illustrations of variations in flow over the entire reporting period and most recent full calendar year, which includes a period of extreme drought.

See Future Conditions, Section II-C: Raw Water Availability below for a discussion of potential low flows at the project site, which will affect intake function. See Structural Analysis and Need for Project, Section I. A: Hydrologic Loading Assumptions below for a discussion of normal and high flows, which will affect dam stability.

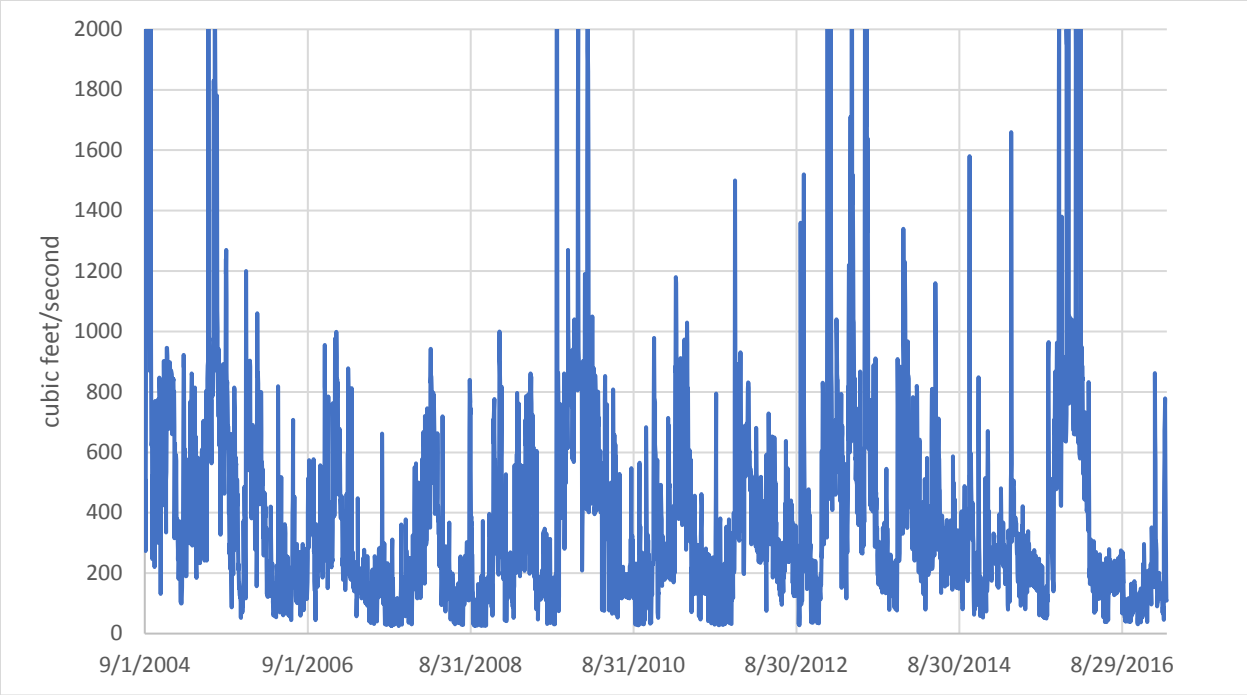

Figure 19. Daily Mean Flows at Nearest Stream Gage, 9/1/2004 - 3/16/2017

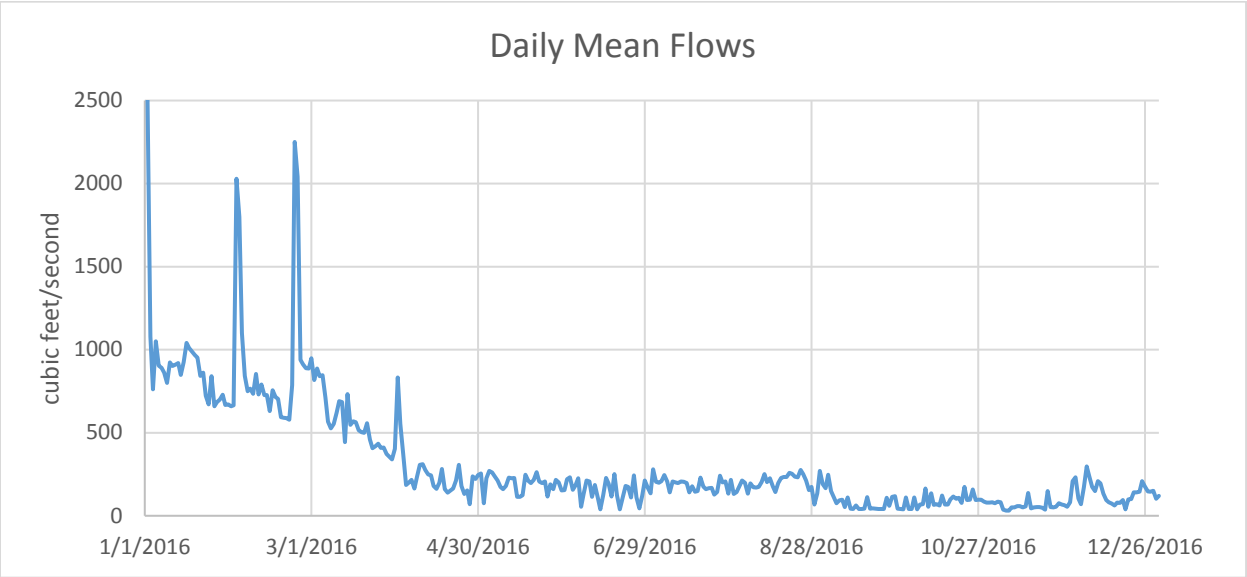

Figure 20. Daily Mean Flows at Nearest Stream Gage, 2016 Calendar Year

C. Recreational Use

The Lena Davis Landing Site is a popular access point to the Tuckasegee River at Cullowhee Dam and serves as a kayak and canoe portage around the dam. For the past seven years, the pool above the dam has been used by the Western Carolina University Parks and Recreation Management Program and Base Camp Cullowhee for an annual Canoe Slalom event. The flat, but moving, water created by the dam creates a course which is suitable for use by children and families.

Downstream recreational facilities include the Cullowhee Community Garden and adjacent South Painter Park, Jackson County Greenway, and Locust Creek river access.

D. Natural and Cultural Resources

Consultation with the North Carolina Department of Cultural Resources, North Carolina Office of State Archaeology, and Eastern Band of the Cherokee Indians Tribal Historic Preservation Office revealed that no architectural, archaeological, or other culture resources are present in the project area or are likely to be affected by any modification of the Cullowhee Dam or adjacent water intake structures. However, any proposed modifications of those structures may require formal cultural resource assessments as required by law during the permitting process. See Appendix for details of this consultation.

Consultation with the US Fish and Wildlife Service (USFWS) and North Carolina Wildlife Resources Commission (NCWRC) indicated that two species of likely concern are present in or are likely to be affected by any proposed removal or modification of the dam: the Appalachian Elktoe Mussel, a current federally listed endangered species, and the Sicklefin Redhorse, a candidate for the endangered species list not yet formally listed. If either of the dam repair options or the dam removal option is chosen, consultation with USFWS and NCWRC would be appropriate. The dam removal option may possibly extend the critical habitat of the Appalachian Elktoe Mussel as well as restore habitat and improve passage for the Sicklefin Redhorse. See Appendix for details of this consultation.

FUTURE CONDITIONS

In order to evaluate the effect of dam removal, modification, or repair might have on the performance of the intakes going forward, an evaluation of the expected future demands for both utilities must be made. These projections depend heavily on the information reported by both WCU and TWSA in their Local Water Supply Plans (LWSPs). These reports were filed in 1997, 2002, 2007, and yearly thereafter with NCDEQ and contain estimates of distribution system sizes, raw water withdrawals and finished water use, and planning projections, as well as similar information for wastewater collection and treatment systems.

I. Raw Water Demand Projections

A. Western Carolina University

According to information presented in its 2015 LWSP, WCU's existing maximum daily water withdrawals were approximately 0.522 MGD, which represents a considerable decrease in per capita water demand since 2007. Despite this trend, WCU anticipates 2% growth per year in enrollment, which should eventually lead to an increase in raw water demands over time.

According to WCU's 2014 Master Plan, the university's 2013 enrollment was 8,148 students. While staff and faculty can also be expected to contribute to water use, and off-campus students (including both those who reside off-campus but are physically present for classes and those enrolled in distance learning programs) can be expected to contribute less to demand, the assumption made for this demand projection was that increases in student enrollment would correlate directly with raw water demands, i.e., that raw water demands would increase 2% per year as student enrollment increased.

Table 2. WCU Raw Water Demand Projection

Year	Enrollment	Demand (MGD)	
		Avg. Day	Max. Day
2017	9,107	0.280	0.543
2022	10,055	0.309	0.600
2027	11,101	0.342	0.662
2032	12,256	0.377	0.731
2037	13,531	0.416	0.807
2042	14,940	0.460	0.891
2047	16,495	0.508	0.984
2052	18,212	0.560	1.086
2057	20,108	0.619	1.199
2062	22,200	0.683	1.324
2067	24,511	0.754	1.462

According to the current projection, WCU's 50-year average day raw water demand is expected to be approximately 0.754 MGD while its maximum day raw water demand is expected to approach 1.462 MGD. Since this average daily demand does not exceed 80% of existing capacity, the water supply is considered to be adequate to meet demands within the 50-year planning period, and it is likely that on peak days the current intake structure and storage are also adequate. It is likely that the age of the existing WCU treatment plant and intake equipment will necessitate renovations and possibly accompanying expansions over the course of the next fifty years before capacity concerns drive such decisions.

B. Tuckaseigee Water and Sewer Authority

TWSA's raw water demands have fluctuated since 2007, but maximum daily water withdrawals for 2015 were the second highest seen in the study period after 2010. Three factors contributed to TWSA's projected service population increase: the anticipated population growth of Jackson County, the likely expansion of TWSA's service area over time, and the likelihood that WCU's off-campus student body, which is anticipated to increase steadily over the planning period, is likely to contribute additional customers to TWSA. In the course of developing its LWSP, TWSA has made year-round service population projections at 10-year intervals through 2060. These population projections have been interpolated linearly through 2060 and extended along a best-fit line for the remaining seven years of the planning period.

TWSA's 2015 per capita water use is much higher than that of WCU, largely owing to the fact that TWSA's customers make greater and more varied use of finished water for both domestic and commercial purposes. While the 2015 TWSA LWSP assumes a reduction in future per capita water demand, no such assumption has been made in this projection, which maintains the 2015 per capita water demand and applies it to all future service populations.

Table 3. TWSA Raw Water Demand Projection

Year	Service Population	Demand (MGD)	
		Avg. Day	Max. Day
2017	7,730	1.111	1.605
2022	9,309	1.338	1.933
2027	9,765	1.404	2.028
2032	10,240	1.472	2.126
2037	10,742	1.544	2.231
2042	11,264	1.619	2.339
2047	11,816	1.699	2.454
2052	12,390	1.781	2.573
2057	12,998	1.869	2.699
2062	14,025	2.016	2.912
2067	14,721	2.116	3.057

According to these projections, TWSA's permitted capacity of 1.5 MGD is expected to become inadequate by 2034 on average days and may already be exceeded on peak days. As early as 2019 TWSA can be expected to exceed 80% of capacity on average days, at which time NC General Statute 143-355(i) prescribes that a revised plan be submitted that specifies how the water system intends to address foreseeable future water needs. TWSA has already stated in its LWSP that it plans to expand its existing water treatment plant. Its existing intake structure would continue to be adequate for some time longer, although the current pumps would need to be upgraded as part of the treatment expansion project.

C. Raw Water Availability

In order to determine how much water is available for domestic use, it is first necessary to determine how much water is present in the stream. The best flow data available are from USGS gaging station 03508050, which offers complete data online for a 12.5-year period from which several potentially useful measures of low flow can be taken.

Table 4. Tuckasegee River Low Flows

Measure at USGS Gage 03508050	CFS	MGD
Minimum Instantaneous	16.0	10.3
Minimum Daily Average	25.0	16.2
Minimum 7-day Average	33.6	21.7
7Q10 at gage for reporting period	125.1	80.9

From the above table it can be seen that the projected 2067 maximum day withdrawal for the two water systems of 4.52 MGD (6.99 cfs) constitutes a significant fraction of the minimum instantaneous flow of 10.3 MGD (16 cfs) that was observed twice in October of 2007, and the minimum daily average flow of 16.2 MGD (25 cfs) that was observed twice in September of 2007 and once in October of the same year. September of 2007 also had the lowest seven-day average flow, 21.7 MGD (33.6 cfs). NCDEQ permits withdrawal of 20% of the 7Q10, or minimum 7-day average flow with a 10% chance of recurrence, to be used in determining safe yield of a surface water source without additional studies.

Farther upstream of USGS Stream Gage 03508050, Gage 03508000 (Tuckasegee River at Tuckasegee, NC) reported from 1934-1976 for a slightly smaller drainage area of 143 mi^2. The published 7Q10 for that period was 57 cfs, or 0.488 cfs/mi^2. USGS Stream Gage 03508240 (Caney Fork at East Laport, NC), located on a tributary of the Tuckasegee River between the current Tuckasegee gage and the project area, reported a 7Q10 of 25 cfs for a drainage area of 51.2 mi^2, or 0.399 cfs cfs/mi^2. USGS Water Supply Paper 2403, "Low-Flow Characteristics of Streams in North Carolina," notes that the median 7Q10 value for the Western Piedmont and Mountains region of North Carolina is approximately 0.317 cfs cfs/mi^2, with 7Q10 values in the region ranging from over 1 cfs/mi^2 to some streams going completely dry in a similar drought period. The extreme variation in flow rates is due to the high variability of topographic and climatic factors in

Western North Carolina, including orographic precipitation and rain shadow effects commonly observed in the region due to the mountainous terrain. These figures imply that a range of possible low flow measurements may be calculated from the intake site's 207 square mile drainage area.

Table 5. Estimates of 7Q10 from Drainage Area

Low Flow Measure	CFS/Mi^2	CFS	MGD	20% of 7Q10 (MGD)
7Q10 at USGS Gage 03508000	0.399	82.5	53.3	10.7
7Q10 at USGS Caney Fork Gage	0.488	101.1	65.3	13.1
7Q10 at USGS Gage 03508050	0.850	176.0	113.8	22.8
median 7Q10 for region	0.317	65.6	42.4	8.5

Consequently, even the lowest estimate of 7Q10 at the dam site, 42.4 MGD, is still more than sufficient to permit the combined maximum-day withdrawal of 4.519 MGD within the 50-year planning period from the perspective of NCDEQ regulations for raw water withdrawal.

Figure 21 below shows the locations of the upstream gages in relation to the project area. The intakes and dam are located approximately 6.5 river miles downstream of the Cullowhee stream gage and nearly 11 river miles downstream of the Tuckasegee, NC stream gage.

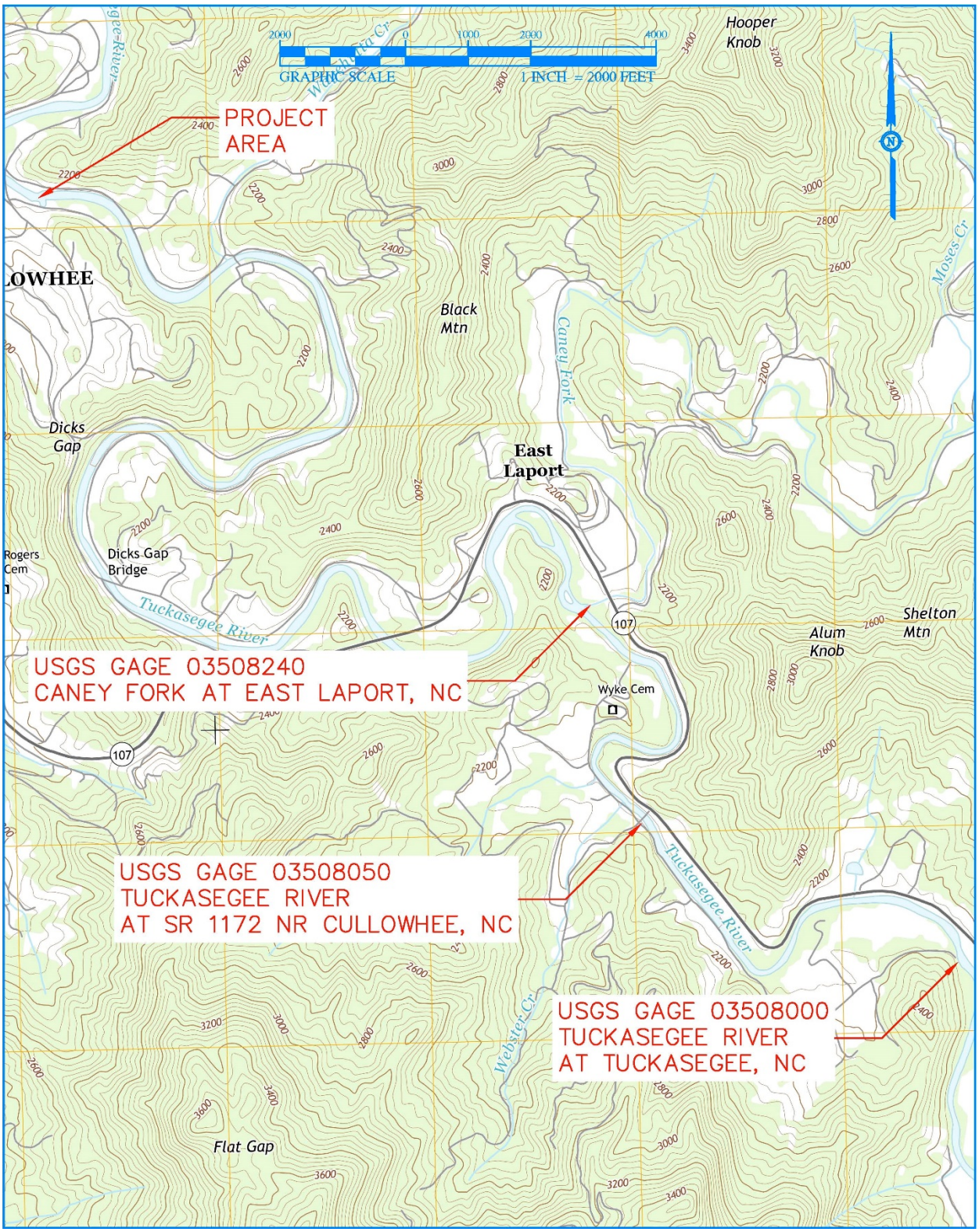

Figure 21. Tuckasegee River Stream Gage Locations

II. Recreational Use

The Cullowhee Revitalization Endeavor, Inc. (CuRvE) is a community organization whose mission is to facilitate the beautification and revitalization of downtown Cullowhee from State Highway 107 to the former main entrance of the Western Carolina University campus. CuRvE considers the Tuckasegee River to be Cullowhee's core natural asset and views this corridor as central to the revitalization of a multi-community area in the heart of Jackson County. As part of the Cullowhee River Corridor project, CuRvE conducted an economic impact study in 2014 of the 3 ½ mile length of the Tuckasegee River as it flows through old Cullowhee. The target area begins above the Cullowhee Dam at the Lena Davis Landing and, below the dam, to the proposed Cullowhee River Park, downstream to the Cullowhee Community Garden and adjacent South Painter Park, continuing along the Jackson County Greenway to the Locust Creek river access.

From a survey of Western Carolina University students, faculty and staff, it is estimated that more than $4.1 million was spent outside the county on recreational activities that the Cullowhee River Corridor development will offer. Local spending would increase by $800,000 simply by capturing about 2 recreational trips for each participant that would have traveled outside the county. The Cullowhee Dam in its current form is considered by CuRvE to be an impediment to river recreation due to the necessity for portage of kayaks and canoes around the existing obstacle.

Equinox contacted Jackson County Recreation & Parks Director Rusty Ellis to discuss potential recreational uses along the river both upstream and downstream of the dam. No future park sites are proposed along the Tuckasegee River. While a greenway is proposed further downstream, current plans do not include a greenway extension past the dam. If a greenway were proposed for this location in the future the removal of the dam facilities along the bank would provide more space to accommodate a greenway as well as a small pocket park in the future. If the dam is modified there appears to be sufficient space for a greenway pathway. A review of the Jackson County Recreation Master Plan was also conducted. In addition to the Lena Davis Landing and Access Area, one other recreational facility, the South Painter Community Park, is proposed to be located downstream of the Cullowhee Dam. This park is located far enough away from the dam that it would not be impacted by any potential modification of the dam.

The Cullowhee Revitalization Endeavor also conducted a Conceptual Design Study in 2015 prepared by Scott Shipley of S20 Design and Engineering. The proposed whitewater park with in-river wave structures is proposed at the existing Cullowhee Dam. According to Shipley's study, the dam "features adequate drop and flow to host a river park". Scott Shipley stated the following in his report:

"*The current diversion is eroded and in need of repair with, at a minimum, a scoured and failing right abutment. The proposed project would include refurbishment of the dam. The design solutions suggested allow the operator to focus the power of the drop into a central channel using*

Obermeyer-type head gates that allow for constriction during low flows and that can be lowered to minimize impacts at higher flows. The proposed solutions include three wave/drop features".

This plan is naturally predicated on the continued presence of the dam as well as availability of sufficient excess water in the reservoir to permit the discharge of recreational flows while still retaining a surface water elevation that will permit the existing intakes to meet demand. When contacted for comments on this study, Scott Shipley of S2O Design, the writer of the initial CuRvE report, stated that the purpose of the Obermeyer gate was to ensure continued recreational flows during periods of low inflow and that its final design could be as short as 18 inches. It was his opinion that the gate's design and operation would take all stakeholders' concerns into account.

The Conceptual Design Study notes that flows in the Tuckasegee River average 550 cfs in a month or less and suggests that 1200 cfs would be preferable for competition-level use. An examination of available flow data discussed in the following section shows that these competition-level flows were met or exceeded around 2.5% of days in the last 12.5 years, or approximately nine days a year, while the lower "average" flows were met or exceeded 82 days a year.

River sports are some of the fastest growing sports and recreation activities in the US and there is increasing interest from many communities to activate their river corridors by creating "whitewater wave parks". This includes two other communities in western North Carolina that are looking to develop "whitewater wave parks" in the region for economic and ecotourism potential. This includes a site at the former Dillsboro Dam in Jackson County and another in Woodfin, NC. If such a park was developed at the Cullowhee Dam and downstream the county would want to extend the current greenway system to connect to this park.

In the absence of a whitewater park, CuRvE is still committed to the creation of a "river park" near the dam's current location. The river park option would not have in-river structures to generate whitewater waves, but would include recreational facilities and improve access to the river for recreational opportunities, including potentially becoming a part of a proposed American Rivers Blue Trail. A Blue Trail or Blueway is "a river adopted by communities that are dedicated to improving family friendly recreation such as fishing, boating, hiking, and wildlife watching, and conserving rivers and lands".

STRUCTURAL ANALYSIS AND NEED FOR PROJECT

I. Hydrologic and Seismic Loading Conditions

A. Hydrologic Loading Assumptions

The existing dam has survived two major flood events, the August 30, 1940 flood, and the floods associated with the remnants of Hurricanes Frances and Ivan in September 2004. Structure inspections and engineering reports were prepared following each event, by Vanderhoof in 1941 and by Sutton-Kennerly & Associates in 2004.

Available streamflow gage data include USGS 03508000 at Tuckasegee, NC from 1844 to 1976, with a drainage area of 143 mi^2; and USGS 03508050 at SR 1172 near Cullowhee, NC from 2004 to date, with a drainage area of 147 mi^2. Corresponding streamflow at Cullowhee Dam, with a drainage area of 207 mi^2, can be approximated using a factor determined by the square root of the ratio of the drainage areas, or an increase of around 20 percent. Applying this factor to the historical streamflow gage data suggests a peak flow at Cullowhee Dam of approximately 49,000 cfs in 1940, and 14,000 cfs in 2004. An analysis of the stream gage data for mean daily base flow produced an estimate of approximately 500 cfs for the site, which includes daily fluctuations resulting from peaking power releases by Duke Energy at their upstream hydroelectric facilities. A FEMA river routing model was first developed for the Tuckasegee River by the Tennessee Valley Authority (TVA) in 1983 to produce flood profiles for various return periods. The data provided (TVA, 1983) are summarized below for comparison with the historical flood events, and suggest a return period slightly greater than 500 years for the 1940 flood, and greater than 10 years for the 2004 flood, based on the estimated peak discharges. Using the FEMA model to determine headwater and tailwater conditions for various floods (with the exception of the normal operating conditions which were based on survey data), the following hydrologic loading conditions were prepared:

Table 6. Hydrologic Loading Flows

Frequency Flood	Peak Discharge (CFS)	Headwater (feet)*	Tailwater (feet)	Net Head (feet)
Daily base flow	500	2061.0	2054.0	7.0
10-year	12,185	2067.3	2064.6	2.7
2004 Flood	14,000	N/A	N/A	N/A
50-year	22,680	2071.7	2070.4	1.3
100-year	29,070	2073.5	2073.5	0
500-year	47,950	2080.4	2080.4	0
1940 Flood	49,000	N/A	N/A	N/A

* based on 1-foot tall flashboards; 2-foot tall flashboards assumed for 1940 flood

Note that the net head on the overflow section decreases with increasing discharge until the structure is fully submerged and no longer serves as a hydraulic control in the river. For purposes of the stability analyses, the overflow crest was increased by 1 foot for passage of the 1940 flood only to replicate the conditions assumed with 2-foot tall flashboards installed.

B. Seismic Loading Assumptions

The dam is located in Jackson County, North Carolina, with Latitude 35.31523° N and Longitude 83.17593° W. The USGS provides seismic hazard curves based on site location at https://geohazards.usgs.gov/hazardtool/application.php. Based on this information, peak ground accelerations for Cullowhee Dam are estimated to be approximately 0.05g for an earthquake having a return period of 500 years, 0.1 g for a return period of 1,000 years, and 0.45g for a return period of 10,000 years.

The USGS also provides a GIS database of geologic units and structural features in North Carolina. The bedrock at the dam site is biotite gneiss of the Coweeta Group (Middle/Late Proterozoic), a foliated metamorphic rock. This rock type is generally very strong (requiring many hammer blows to fracture), with a compressive strength greater than 15,000 psi and with a friction angle of at least 45 degrees (from Evert Hoek's "Practical Rock Engineering"). A regional geologic map of the dam site is shown on Figure 21.

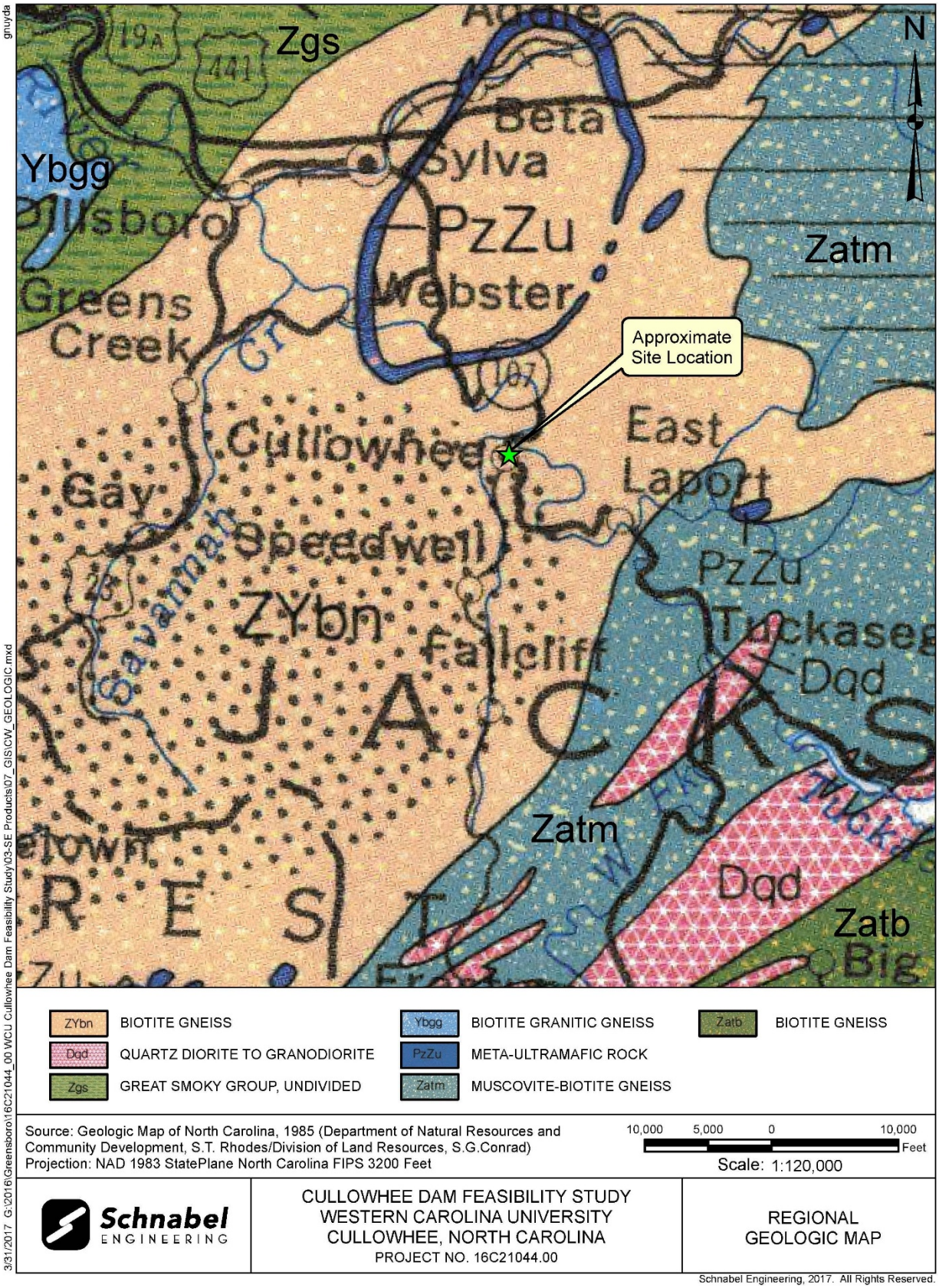

Figure 22. Regional Geology

II. Structural Stability Analyses of Overflow Section

A. General

For the structural stability analysis of the concrete gravity overflow section, an in-house spreadsheet (GRAVDAM) was used that was developed specifically for two-dimensional analysis of gravity dam stability. The spreadsheet includes an evaluation of sliding stability on the dam foundation, and requires that the resultant of all forces acting on the dam lie within the middle ⅓ of the base to avoid tensile stresses in the heel of the dam for a rigid body analysis. When the resultant lies outside of the middle ⅓ of the base, calculated tensile stresses are assumed to result in the formation of a crack propagating from the heel of the dam. Full uplift is then assumed to act on the cracked section of the base, and the analysis is revised to reflect this modified uplift distribution, with any shear strength acting only along the uncracked portion of the base. The evaluation was performed for the overflow section shown on Figure 1, with assumptions of both 1-foot and 2-foot tall flashboards, for various load combinations as described below. The two-dimensional analyses neglect three-dimensional effects and are therefore conservative, but reasonable for a long, straight concrete gravity structure. The GRAVDAM analysis sheets are included in the Appendix.

B. Normal Operating Conditions

The estimated mean daily base flow of 500 cfs with 1-foot tall flashboards was used for analysis of normal operating conditions. The new survey data were used to establish a headwater at elevation 2061 and a tailwater at elevation 2054, for a net head on the overflow section of 7 feet. Assuming no cohesion on the structure foundation, but with a surface friction angle of 45 degrees, a nominal unit weight of 145 pcf for unreinforced concrete, and an upstream sediment depth of 2 feet, the structure is stable, with a sliding factor of safety of 1.39 and a ratio of stabilizing moment to overturning moment of 1.52. For normal operating conditions, a higher sliding factor of safety is typically desired (generally 2.0, but varies with regulatory agency and other factors), but from a risk standpoint, the structure has a long history of satisfactory performance under these conditions. If the upstream sediment depth is increased to the top of the concrete, the structure remains stable but with a reduced sliding factor of safety of 1.12.

For the undercut section, the tailwater would extend an additional 3.5 feet upstream beneath the structure, and the foundation contact area for sliding would be reduced by about 43 percent (from 8.2 feet to 4.7 feet). To represent the reduced sliding resistance, a surface friction angle (phi) of 30 degrees was used for a reduced tangent phi factor of 0.577, instead of a tangent phi factor of 1.0 for a surface friction angle of 45 degrees. Assuming no upstream sediment at this location (based on field observations), the undercut section is unstable for a net head of 7 feet (with a sliding factor of safety of 0.91) and would slide downstream (or overturn) if not for the three-dimensional effects of the adjoining sections. With a slight increase in net head, this section could lose contact with the reduced foundation surface and develop full uplift beneath the structure. This could

produce a large hydraulic gradient (or large head over a short distance) and a potential to move sediment downstream at the foundation surface, as has apparently been observed at this location.

For the previous operating conditions with 2-foot tall flashboards (reported in 1941), the sliding factor of safety remains above 1.0 for a net head of 7 feet (with a computed value of 1.09), but a partial crack could occur at the foundation contact and produce increased uplift and a possible sliding failure under the assumed conditions. Since the structure did not fail under these conditions during the 1940 flood, one or more of the assumptions may be incorrect. For example, a slight increase in the unit weight of the concrete from 145 to 147 pcf maintains the sliding factor of safety above 1.0 for the cracked section and prevents sliding. The actual unit weight of the concrete is unknown. Regardless, this suggests that 2-foot tall flashboards could provide unfavorable conditions for sliding and should never again be used on the overflow section.

C. Flood Operating Conditions

Due to the development of high tailwater downstream of this small structure during the passage of floods, the net head on the structure actually reduces with larger flows. A 10-year flood routed through the FEMA river model produces a net head of less than 3 feet, compared to a maximum of 7 feet under normal operating conditions. This produces a sliding factor of safety of 1.89, but a reduced ratio of stabilizing moment to overturning moment of 1.17, primarily due to the increased uplift beneath the structure. For comparison purposes, a 500-year flood produces a sliding factor of safety of over 50, but a ratio of stabilizing moment to overturning moment of 1.14, for no net head. The existing structure withstood complete submergence during the 1940 flood with an overtopping depth of 18 feet, and with higher flashboards, for a peak discharge comparable to the 500-year flood.

D. Earthquake Conditions

Performance of the overflow section during an earthquake was also evaluated using GRAVDAM, with assumed average base flow of 500 cfs. For an assumed horizontal acceleration of 0.05g, associated with an estimated 500-year return period, the structure remains stable, with a sliding factor of safety of 1.18 and ratio of stabilizing moment to overturning moment of 1.43. For a larger acceleration of 0.1g, associated with a 1000-year return period, the sliding factor of safety remains above 1.0 for the assumed conditions but cracking may initiate at the upstream heel, increasing uplift at the foundation. If cracking occurs at the heel due to a 1000-year earthquake, as predicted, the sliding factor of safety under subsequent normal operating conditions is 0.97 and sliding would occur, indicating post-earthquake instability.

III. Engineering Opinion of Structural Stability and Inherent Weaknesses

A. Opinion Based on Available Information

The overflow section constructed in 1930, having a vertical upstream face, curved crest, and estimated 0.76:1 sloping downstream face, is a reasonable design for a low concrete gravity structure founded on competent bedrock. Although the addition of flashboards reduces structural stability by increasing hydrostatic loads without a corresponding increase in dead weight, the existing structure has performed satisfactorily for many years with 1-foot-high flashboards in place. The structure is exhibiting evidence of wear, however, with abrasion damage on the downstream face gradually reducing the cross-section, and with potentially serious foundation scour and headcutting erosion occurring at one location. The existence of a 3.5-foot deep undercut approximately 60 feet from the right abutment, identified during our field investigation, represents an unstable condition for the overflow section without help from the adjoining sections. If there are formed vertical joints, or cracks, within the 160-foot-long structure, as would be expected, the global stability of that concrete block would be seriously compromised. Without further exploration, it is difficult to determine the exact cause and full extent of the undercutting, and whether or not the foundation will continue to headcut toward the reservoir. It is possible the bedrock foundation had a discontinuity at that location, perhaps associated with a presumed location along the thalweg of the river, which contained weaker material subject to erosion. If a flow path exists beneath the overflow structure between the tailwater and reservoir, a higher gradient and flow velocity would be present that could continue to erode the foundation and enlarge the cavity sufficient to produce a localized failure. The apparent lack of impounded reservoir sediment at this location may be evidence of this condition. The existing structure may also be subject to cracking and instability following an earthquake producing peak ground accelerations of 0.1g or more. Although the potential breach outflow for this Low Hazard structure should be relatively small, depending upon the size of the breach, the reservoir could be drained sufficiently to eliminate the head on the WCU and TWSA raw water intakes and prevent normal water withdrawals. A sudden breach of the undercut section under normal reservoir conditions would probably be less than 20 feet wide (depending on vertical joint and crack locations) and 8 feet deep, with a peak outflow of less than 1,000 cfs (or twice assumed average base flow). However, a larger breach could be possible. Any failure of Cullowhee Dam would garner media attention and could reflect poorly on those responsible for its operation and maintenance. It may also be possible that the breach outflow could endanger fishermen or recreationalists immediately below the dam at the time of failure.

Another potential area of concern is the right sidewall. The existing concrete sidewall displays obvious deterioration downstream of the overflow crest, with some evidence of bank erosion, and could eventually collapse. Furthermore, adequate closure is not provided at the upstream end and flows can easily pass behind the wall at higher flood levels that can occur rather frequently. This can produce erosion at the base of the adjacent hillslope and cause a sudden slope

failure below Wayehutta Road, taking out at least a portion of the roadway with it. This condition may have existed for a long time, but will only get worse if not addressed. The sudden loss of a portion of Wayehutta Road would also garner media attention, and could pose a serious risk to anyone driving along the road at that time, as well as limit access for emergency responders.

Other features do not appear to represent any serious concerns at this time. Individual flashboards have failed in the past, only to be replaced (although with some difficulty, and by contractors). The existing sluice gate, powerhouse, and intake have been essentially abandoned in place and appear to remain in stable condition. A chain link fence is provided for security of the powerhouse and intake site, and signs are posted both upstream and downstream as a warning of potential hazards to fishermen and recreationalists. The existing left sidewall appears to consist of a low concrete wall with a masonry cap. Lower portions of the masonry have disappeared over time, leaving a portion of the cap unsupported. If this cap were to collapse, it could fill the sluice gate channel with some debris, but without significant damage or impact to reservoir operations. The wall would probably have to be repaired to contain larger flood flows through the overflow section.

B. Potential Additional Field Investigations

The structural assessment performed for this report is based on available information, field observations, and engineering judgment. Numerous assumptions were made for the structural stability analyses performed, including the unit weight of the concrete in the overflow section, the friction angle for the structure foundation contact, and the absence of foundation cohesion. While these assumptions are believed to be reasonable, a couple of concrete corings into bedrock at the concrete overflow crest would provide samples of the concrete for density and strength testing, and could indicate the conditions at the foundation contact. The location of a core hole at the overflow crest in the undercut area could help determine the extent and possible cause of the conditions observed in the field. A new survey was completed of the area following our field visit to help establish structure locations and elevations, and has been incorporated in our drawings and final results.

C. Potential Future Regulatory Requirements

It may be possible for the classification of Cullowhee Dam to be changed from Low Hazard to Intermediate Hazard in the future (based on the economic value of potential damage), if the planned downstream developments for a river walk or water park are completed. Low Hazard suggests economic damage in the event of dam failure of less than $30,000, whereas Intermediate Hazard suggests economic damage of up to $200,000. This could result in closer regulation by the State and the need for periodic dam safety inspections and maintenance. A High Hazard classification would require probable loss of life, or significant damage to infrastructure which appears unlikely for this dam. NCDEQ states that "Hazard classification refers to damage potential downstream and does not relate to the condition of a dam."

IV. Repair and Stabilization Needs for Existing Dam

A. Concrete Overflow Section

As described above, the existence of a 3.5-foot deep undercut section beneath a portion of the overflow structure represents a potentially serious situation that should be addressed, either by localized repairs or as part of a larger modification to the entire structure. A high priority should be placed to correct this structural deficiency and safety concern.

B. Flashboards

The existing flashboards are routinely replaced when damaged or destroyed, but with some difficulty and only through the efforts of a specialty contractor. It is understood that the flashboards installed in April 2016 by In-Water Services replaced older flashboards that had been missing for several months. Although the periodic loss of a flashboard has negligible downstream impact and apparently a limited effect on reservoir operations for the diversion of water, it does represent a maintenance cost that could be averted through a modification to permanently raise the concrete overflow crest. Alternatively, all flashboards and pipe supports could be removed and replaced with new flashboards and slotted supports (e.g. short H-piles) for ease of future replacement. At a minimum, the existing flashboards and pipe supports should be closely inspected and replaced as needed. Note that the use of flashboards greater than 12 inches tall reduces the stability of the overflow structure and must be avoided.

C. Right Sidewall

The right sidewall currently allows flood flows to pass behind the wall and erode the adjoining earthen slope, creating a potential slope failure and loss of a portion of Wayehutta Road. The wall is also severely deteriorated and could eventually collapse. This is a safety concern and should be given a high priority.

D. Left Sidewall

Some repairs to the existing masonry of the left sidewall may be required in the future to prevent a collapse of the masonry cap. Although not considered critical, this maintenance work could be done from the dry side of the wall to shore up the existing cap to prevent future collapse.

E. Sluice Gate

The existing steel sluice gate has been welded closed and abandoned in place and appears to remain intact with minor leakage. However, if this gate were to fail or be removed for any reason, the reservoir level could drop due to the flow through this feature, but likely not to the extent that water deliveries would be impacted. At full pool, this feature would have a maximum discharge capacity of approximately 40 cfs, which is well below average base flow and would have the effect under normal conditions of only reducing the depth of overflow across the flashboards.

F. Powerhouse and Intake

The existing powerhouse and intake have essentially been abandoned in place. The four windows are boarded up, the door is locked, and the site is surrounded by chain link fence. Warning signs are posted to prohibit entry. The intake and draft tube areas are plugged full of sediment. These structures do not represent any issues of particular concern relative to the dam.

ALTERNATIVES ANALYSIS

Broadly speaking there are three alternatives for responding to the current condition of the dam. The first alternative is to take no action. The second alternative is to repair the dam. The third alternative is to remove the dam from the stream. Within the second alternative, Schnabel Engineering has identified three approaches to repair: repair of only the at-risk portions of the dam, complete repair of the dam, and integration into either of those two sub-alternatives of modifications to the dam intended to support recreational activities. The third general alternative, removing the dam, is technically feasible, but the consequences of dam removal on water supply must be mitigated by the construction of new intake structures. McGill Associates has identified two alternatives for replacement of the existing intake structures: construction of two separate replacement intake structures at a new location, and construction of a single combined replacement intake structure that would supply both public water systems.

I. Alternative 1: No Action

The No Action alternative would retain the structure as it currently exists, with a serious condition of foundation undercutting at Station 0+65 that could eventually result in the sliding or overturning failure of a portion of the concrete overflow crest, and with a deteriorating right sidewall and eroding backfill that is subject to slope failure and loss of a portion of Wayehutta Road. These two conditions alone make this alternative unfavorable. The existing timber flashboards would also remain under this alternative, requiring periodic replacement as portions become damaged due to large floating debris or succumb to gradual deterioration. There is no construction cost for this alternative, but substantial risk of failure and associated consequences. Should the overflow crest's failure result in a significant drop in water surface elevation, both the WCU and TWSA raw water intakes could immediately be left without access to the river until emergency repairs or intake modifications could be performed.

Under the no action alternative, no significant improvements to aquatic habitat or increases in recreational opportunities are likely to occur.

II. Alternative 2: Structural Repairs and Improvements to the Existing Dam

Potential repairs to the existing structure would involve in-stream work and restoration or improvement of the existing dam's stability. Under repair alternatives 2.A and 2.B, no significant improvements to aquatic habitat are likely to occur. Proper permits would have to be obtained and further environmental and biological assessment would be required. A consultation with USFWS and NCWRC would be necessary given that critical Appalachian Elktoe Mussel habitat exists immediately downstream of the dam.

Any dam modification or repair alternative would require, at minimum, the following permits from the United States Army Corps of Engineers (USACE) and NCDEQ Division of Water Resources.

1. USACE 404 Nationwide Permit 3a – Maintenance
   - Does not authorize maintenance dredging for navigation
   - Limits stream channel modification to the minimum necessary for the maintenance activity.
   - Stream channel activities beyond those immediately adjacent to the dam may require other permits
2. NCDEQ Division of Water Resources 401 General Certification 4085
   - May require written approval depending on proposed activities.
   - Mitigation required if impacts exceed 150 linear feet of stream or 1 acre of wetland.

A. Alternative 2.A: Address Safety Concerns Only

This sub-alternative would consist only of targeted repairs to backfill the undercut structure foundation, replace the damaged right sidewall, and replace any damaged or deteriorated flashboards. The right half of the concrete overflow structure would be isolated by the installation of cofferdams consisting of stacked super-sacks (large polypropylene fabric bags full of gravel or cobbles) upstream and downstream to a maximum height of 10 feet and for an estimated total length of about 180 feet. The existing flashboards outside this area would first be removed to produce a lower reservoir water surface, to facilitate installation of the super-sacks using a mobile crane from a constructed gravel bench downstream near the right bank of the river. Removal of the flashboards would also replace the discharge capacity lost by isolation of nearly one-half of the overflow crest. The undercut foundation would be inspected, cleaned to competent bedrock, and backfilled with concrete and/or cement grout (probably requiring less than one cubic yard). The exposed portion of the concrete overflow section would be inspected and any damage repaired as deemed prudent. The right sidewall (estimated volume of 12 cubic yards) would be demolished and a new reinforced concrete wall of the same height (about 12 feet), but extending further upstream and with a concrete footing, would be constructed to provide increased flood protection for the adjoining backfill and hillslope (for an estimated 18 cubic yards of concrete). The flashboards and pipe supports on the isolated portion of the overflow crest could be removed and replaced in the dry. Following completion of the work, the super-sacks would be removed to restore normal operations. Rather than reinstall the existing flashboards, new timber planks would be purchased and installed for greater longevity (assumed to require twenty four 3- by 12-inch planks with lengths of 10 feet each). Estimated total project cost = $500,000.

B. Alternative 2.B: Address Safety and Maintenance Concerns

This sub-alternative (2.B) would include the safety repair work described in sub-alternative 2.A. above, but would replace the function of the existing timber flashboards and pipe supports

with the addition of a reinforced concrete overlay on the overflow crest and downstream face of the entire structure to match the existing weir crest elevation. Following isolation of the right half of the concrete overflow structure using super-sack cofferdams, treatment of the undercut foundation, and replacement of the right sidewall, the surface of the overflow structure would be cleaned and anchors would be installed for the placement of the one-foot-thick reinforced concrete overlay with joints. The concrete mix and reinforcing steel would be designed to improve durability and reduce shrinkage cracking of the overlay. Fiber reinforcement could also be considered for additional abrasion resistance. Any seepage at existing cracks, joints, or lift lines (identified by controlled filling of the upstream area between dam and cofferdam) would be treated by pressure grouting, sealing, or internal drainage. After work within the isolated portion of the structure is complete, the super-sack cofferdams would be removed and replaced to isolate the left half of the concrete overflow structure, to a maximum height of 10 feet and for an estimated total length of 240 feet, using a mobile crane on the upstream and/or downstream boat ramps. This would allow for a close inspection of the remainder of the existing structure, foundation treatment as required, and completion of the concrete overlay (probably requiring a total of less than 70 cubic yards of concrete and less than 200 anchors for the entire structure). Concrete repairs to support the masonry cap for the left sidewall would also be performed. Long-term maintenance requirements would be reduced by permanently removing the timber flashboards and pipe supports, and by providing a more abrasion-resistant concrete surface. Stability of the overflow structure would also be improved by increasing the dead weight. The sliding factor of safety under mean daily base flow conditions would improve to 1.78, and the ratio of stabilizing moment to overturning moment would improve to 1.76. Seismic stability would be improved sufficiently to safely withstand the 1,000-year earthquake with a peak horizontal acceleration of 0.1g, rather than the post-earthquake instability expected for the existing structure (see GRAVDAM results in Appendix). Estimated total project cost = $900,000. A cross-section of the modified overflow crest showing details of the proposed concrete overlay and foundation repair is shown on Figure 22.

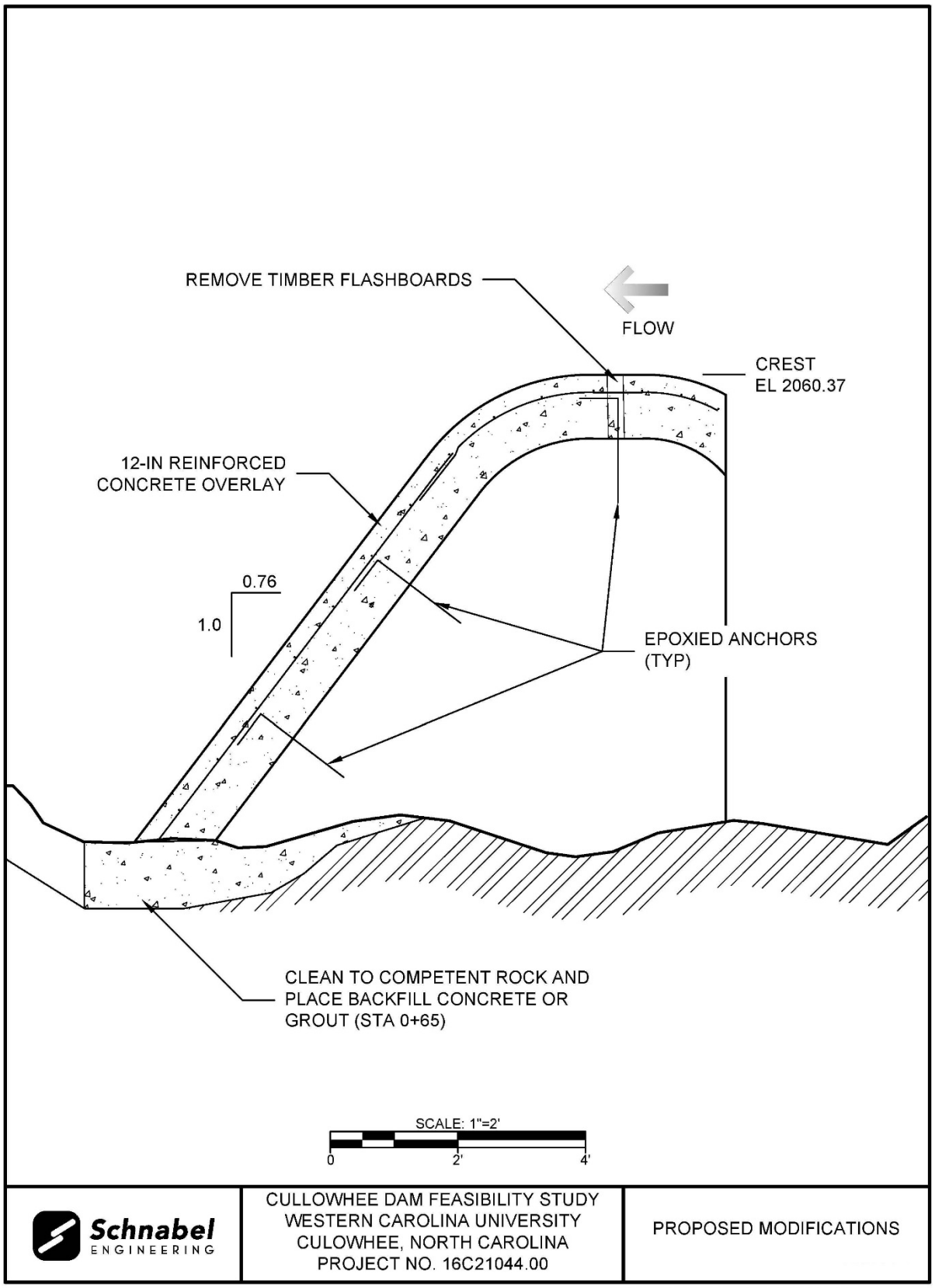

Figure 23. Cross Section of Alternative 2.B Crest Overlay

C. Alternative 2.C: Incorporate Downstream Improvement Plans

A third sub-alternative could incorporate potential plans for recreational use of the downstream river channel. This would likely require the comprehensive repairs identified in sub-alternative 2.B above to support the construction of a proposed family river park possibly consisting of a downstream rock ramp or series of low drop structures, or the installation of an Obermeyer gate within the existing overflow crest. It is assumed that WCU would still be responsible for the costs of the repairs to their dam to address safety concerns, but not for other downstream improvements, which would be the responsibility of others and could impact the schedule and scope of the overall work. WCU's total potential investment for this sub-alternative could be similar to that of 2.B above, with a third party covering the remainder of the project costs.

Another alternative which WCU staff have stated may be preferable would be a modification of alternative 2.B to include a concrete crest overlay of 18" rather than 12" thickness, with a reduction of 6" between approximately Sta. 0+40 and 0+70. This passive improvement would permit kayaks to pass through the dam without the operational complexity of the Obermeyer gate described in the CuRvE report. Because an increase in overlay thickness will add weight in addition to head, this alternative is not expected to increase the probability of overturning or sliding failure. This potential variation of alternative 2.B would, however, raise the effective height of the dam, necessitating floodplain and floodway mapping revisions. This could add an additional 30 cubic yards of concrete to the concrete crest overlay, plus possible safety improvements for kayak passage, for an estimated additional cost of less than $100,000 (if performed as part of the crest overlay work).

It is possible, depending on the dimensions and function of the gate or chute installed, and on the potential addition of a rock ramp, that Alternative 2.C could result in the restoration of fish passage. Sicklefin Redhorse is a focal species that could benefit from access to upstream habitat. Appalachian Elktoe Mussel habitat would however not be restored. This alternative could generate significant additional recreational opportunities and economic uplift to the area. However, the higher expected cost of this option and the lack of apparent funding mechanisms must be taken into consideration.

An additional regulatory concern is that this alternative would promote recreational activities in close proximity to the intake in contravention of 15A NCAC 18C .1201 (c):

> *(c) Only those recreational activities specifically authorized in the [previously referenced] resolution will be allowed. No recreational activities shall be permitted within 50 yards of any public water system intake.*

It is likely that this concern can be disregarded since the Tuckasegee River has well established existing historical recreational uses, and the Lena Davis Landing boat ramp is itself within 50 yards of the WCU intake structure.

III. Alternative 3: Dam Removal

This alternative would remove the existing dam in its entirety. Modifications would be required for continued operation of both raw water intakes for continued water withdrawals (discussed below). Sediment management requirements would have to be determined, ranging from mechanical removal for off-site disposal, to natural erosion and flushing downstream. Based on the relatively small volume of impounded sediment (estimated to be approximately 10,000 cubic yards) consisting mostly of granular material (ranging from sand near the dam to gravel and cobbles at the upper end of the small reservoir) with no evidence of contaminants, natural erosion is currently believed to be permissible. For estimating purposes, super-sacks would first be installed upstream and downstream of the right half of the overflow structure as described for Alternative 2.A above, and the timber flashboards would be removed from the entire structure. The existing right sidewall (with an estimated concrete volume of 12 cubic yards) and exposed portion of the concrete overflow section would then be demolished by mechanical methods (e.g. hydraulic hoe-ram), and the right bank below Wayehutta Road would be stabilized and armored using concrete rubble. Excess concrete rubble would be removed from the site. The super-sacks would then be removed and replaced to isolate the remaining portion of the overflow section, as described for Alternative 2.B above, to complete the demolition of the structure (for a total concrete volume of less than 340 cubic yards). The intake structure and powerhouse would also be demolished to the extent required (depending upon final removal limits), and the site restored as desired. The structure volumes of the existing intake and powerhouse are estimated to be less than 30 cubic yards and 170 cubic yards, respectively; however, the lower portions of the powerhouse substructure could be buried in place to reduce demolition costs. All concrete debris, bricks, reinforcing steel, timber, miscellaneous metal, and mechanical items would be removed for disposal, or salvaged for possible reuse or display (e.g. original generating equipment). Concrete debris may be buried onsite or used for bank protection. The super-sacks would be removed and the site restored to complete the dam removal work. The approximate project cost of only the dam removal portion of this alternative is $700,000. Additional costs would be incurred if sediment dredging or mechanical excavation and disposal is required in connection with any dam removal work. These estimated costs do not include costs to modify the existing raw water intakes for operation at a lower head. This cost estimate does not include the loss of the dam itself, which is a tangible asset belonging to Western Carolina University, and could cost as much as $1.5 million if constructed today.

A. Fluvial Geomorphology

A fluvial geomorphic assessment of the potential project area was conducted to provide an assessment of the effects of potential alternatives on the fluvial processes, channel morphology, and habitat characteristics of the Tuckasegee River.

Bed material in the channel is generally coarse, consisting of cobble, gravel, and sand with little silt or clay. Coming out of a deep pool in the outside bend of the river, the channel thalweg is located closer to the north bank and appears to align with the section of the dam which has been identified as being undercut. Scouring has occurred in the plunge pool created by the dam. Much of the larger material removed during scouring appears to have been deposited about 100 to 150 feet downstream of the dam.

FEMA hydraulic models indicate the length of the river upstream of the dam which is influenced by the impoundment is slightly more than 0.4 mile. For the scenario of the dam being completely removed, a drop in water surface elevation of 0.5 foot was calculated at a distance of 0.4 mile upstream of the current dam location.

Sediment impounded by a dam is an issue to be considered if the dam is removed. The amount of sediment, potential for mobilization, and potential for contamination are of particular concern. Sediment management is not always a major issue if there is not a large quantity of sediment trapped by the dam or if the sediment release is of a similar magnitude to natural loads.

The volume of sediment which may need to be removed was estimated to be in the range of 5,000 to 15,000 cubic yards. Recent cross section information from just upstream of the dam was used to calculate an average depth of sediment. It was assumed the impounded sediment has a wedge shape terminating about 400 feet upstream of the dam. Sediment thickness appears to diminish significantly farther upstream of that point.

Due to the drop in water surface elevation if the removal option is chosen, a variety of bank stabilization techniques could be put in place to prevent erosion of the river banks. Sections of the north bank of the Tuckasegee would need to be monitored closely for failure due to their close proximity to Wayehutta Road. Boulder toe or rock toe treatment may be required in these sections. It is worth noting that boulders (possibly put in place during construction of Wayehutta Road) and bedrock were observed along this section at the base of the bank below the water surface. The presence of boulder and bedrock could help significantly reduce the need for bank stabilization. It is estimated that an additional $200,000 for stream bank stabilization would be necessary for the dam removal alternative. Along the south bank of the Tuckasegee River upstream of the dam, bank heights are generally much lower and the banks are less steep. Most of the south side banks would need only revegetation or light grading and revegetation to be stabilized.

The volume and potential for contamination of the impounded sediment is low (see Appendix 3 and Figure 23 below), therefore if the dam is removed, flushing of the sediment rather than dredging is likely a viable strategy for management. Resource agency consultation would be needed to confirm this.

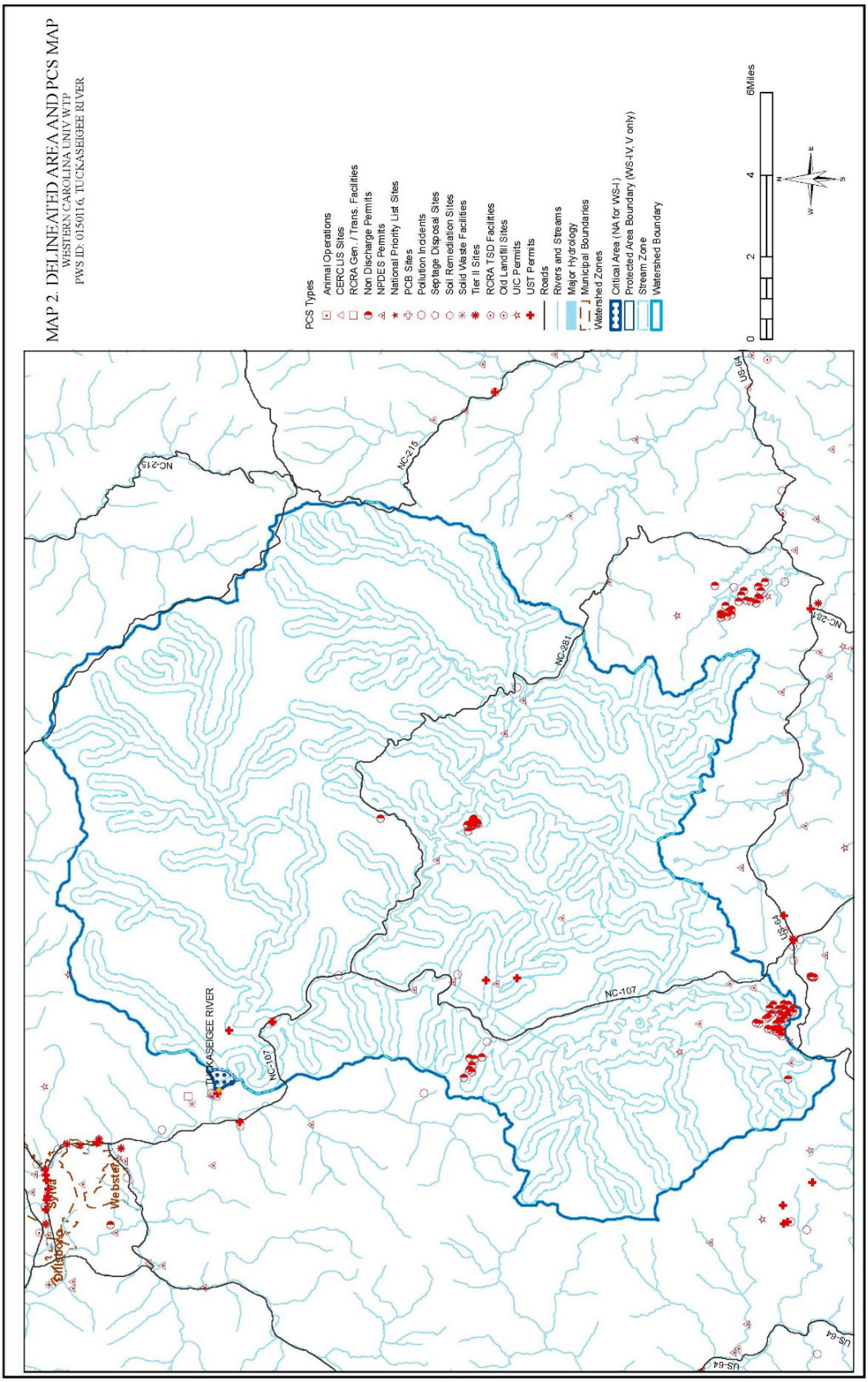

Figure 24. Potential Contaminant Sources Map

B. Environmental and Recreational Effects

Under the dam removal option fish passage and access to upstream habitat for focal species would be restored. Habitat which has been inaccessible for Appalachian elktoe since the dam's construction could also possibly be restored. Overall, this option provides the most potential for ecological uplift.

Recreational opportunities could be improved in the form of greenway access, a Blue Trails paddle trail, and a river park near the dam's current location. Although no economic impact study was conducted for this scenario, it is likely there would be significant economic uplift for Cullowhee. Additionally, funding mechanisms are in place for dam removal and the creation of the Blue Trails paddle trail that could help offset the costs for dam removal and provide added economic and recreational opportunities.

Whitewater parks are a function of drop, flow, and shape. If the dam is removed, some drop in the river will necessarily be removed. This means the whitewater park design depicted in the CuRvE Conceptual Design Study would not be feasible. It is possible that the dam was created at a drop in the river, so there may still be whitewater downstream, or there may be a dam removal scenario in which enough drop is left to accommodate one or more of the proposed drop structures.

The Lena Davis Landing would be the most likely candidate to be affected by dam removal. The concrete boat ramp extends far enough into the river that it could still be used with a drawdown of approximately five feet in stage at base flow. The steps extending into the river upstream of the dam would likely be unusable without modification. However, without the need to portage around the dam, it is likely that these steps would be used much less often. While CuRvE considers the dam an impediment to river recreation, it is unclear at this time whether the removal of the dam and possible loss of the Lena Davis Landing's put in and take out location would result in a net increase or decrease in recreational use of the Tuckasegee River at this location.

The Base Camp Cullowhee Canoe Slalom is likely to be affected as well by the removal of the dam. While this alternative would not preclude the use of this reach of the Tuckasegee River for the event, removal of the dam may create a swifter, whitewater type course less suitable for casual canoe, kayak, and paddle board users.

C. Raw Water Intake Function

The removal of the dam is projected to affect water levels at the intake sites. With the crest elevation of the existing dam set at 2060.37', assuming a base flow of approximately 500 cfs, the water surface elevation at the WCU intake drops from 2061.6' to 2055.8', which is over 3 feet lower than the bottom of the intake screen. The water surface elevation at the TWSA intake drops from 2061.6' to 2056.6', around 14 inches below the bottom of the intake screen.

Modeling the intake screens as rectangular contracted weirs and assuming minimal head loss through the intake screens due to the low flow velocity, minimum necessary water levels can be calculated for a given inflow. The required minimum water levels for the continued function of the existing screens as well as projected water levels with and without the current dam at 500 cfs in-stream flow are shown in the table below.

Table 7. Projected Water Surface Elevations at 500 CFS In-Stream Flow

	2067 Max Day Demand	Minimum Water Level	with Dam	without Dam	Wet Well Invert	Screen Invert
WCU Intake	1.462	2060.5	2061.6	2055.8	2054.06	2058.98
TWSA Intake	3.057	2058.6	2061.6	2056.6	2054.76	2057.76

As shown in the table, the screens are expected to be dry with the dam removed. It is also unlikely that the wet wells can be reused if new screens are constructed due to their invert elevations. This necessitates the development of two sub-alternatives within the dam removal alternative, in which a new intake structure or structures must be constructed to ensure continued water service to the two utilities. Note that this projection has not been repeated at any of the low flows discussed above to determine likely water surface elevation during drought conditions. While the water surface elevation can be expected to be lower during drought conditions, the model used here is best suited for higher flows and floods. Consequently, any dam removal project should only be considered in conjunction with a project intended to replace the raw water intakes of both WCU and TWSA with an alternative water source.

Discussions with NCDEQ indicate that replacement of the existing intakes near their current locations would have minor effects on the classification of the Tuckasegee River, whose current WS-III classification is stated in terms of distance from the WCU intake. Any relocation of the WCU intake upstream of its current location could cause the section of the Tuckasegee River between the new and old intake locations to be reclassified from WS-III to WS-V, a less restrictive classification reserved for industrial water supplies or waters formerly used as water supply. Jackson County would have the opportunity to petition NCDEQ to refrain from reclassifying this section of the river if the more restrictive WS-III classification were still preferable. Some additional reach of the Tuckasegee River is likely to become classified as Critical Area if one of the intakes is moved upstream. Moving either intake upstream of its current location cannot, however, cause additional areas to be reclassified as WS-III, since the WS-III classification of the Tuckasegee River and its East and West Forks already continues to the top of the watershed.

D. Permitting

Either dam removal alternative would require, at minimum, the following permits from the United States Army Corps of Engineers (USACE) and NCDEQ Division of Water Resources.

1. USACE 404 Nationwide Permit (NWP) 53 – Low Head Dam Removal
   - New nationwide permit; requires a PCN (Pre-Construction Notice) and stream/wetland delineation.
   - Generally will not require mitigation.
   - Does not authorize stream restoration (NWP 27) or bank stabilization (NWP 13).
2. NCDEQ Division of Water Resources 404 General Certification 4091
   - Requires written approval from DWR.
   - Mitigation required if impacts exceed 150 linear feet of stream or 1 acre of wetland.
3. Dam Safety Program: A consultation with the Division of Energy, Minerals, and Land Resources Dam Safety Program will need to occur to determine any required approvals. Structural changes to the dam will likely require their approval.
4. Removal constraints due to fish spawning requirements (NCWRC)
   - January through end of April only
5. NCDEQ Public Water Supply Section (PWSS) Plan Review
   - PWSS approval is required for the construction of any new or expanded water intake structure.

E. Alternative 3.A: Separate Replacement Intake Structures

This alternative would call for the construction of two independent intake structures for TWSA and WCU. Each intake structure would have its own mid-stream intake screen with an airburst system to clear obstructions located at a low point in the stream with an intake pipe leading to a wet well located on the stream bank.

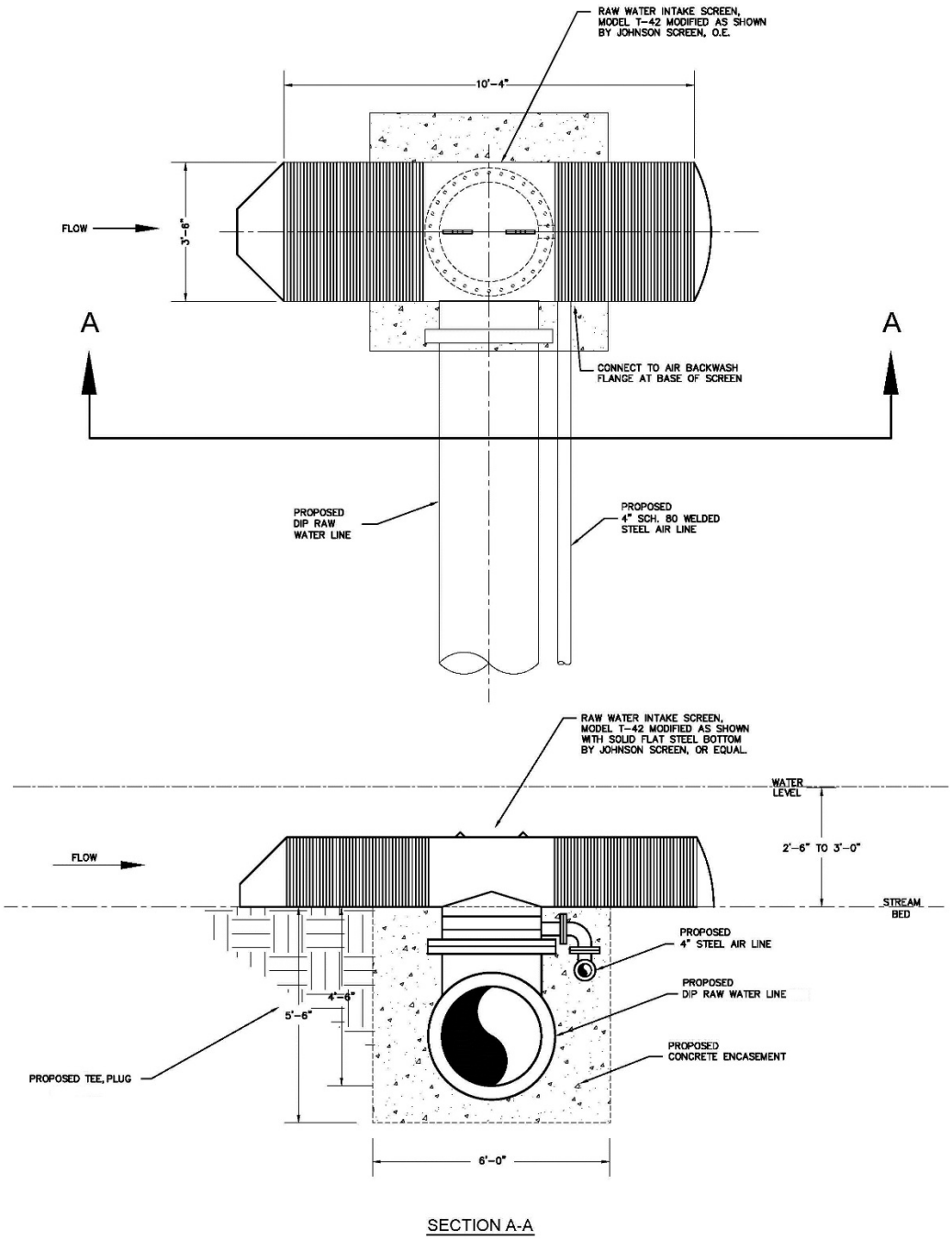

Figure 25. Typical In-Stream Intake Screen Detail

Each intake building would be constructed similarly to the existing TWSA intake building, with a rectangular concrete wet well located below ground level and two vertical turbine pumps withdrawing raw water from the wet well. In order to protect the pump motors and control panels from weather and tampering it is recommended that they be housed above the wet well in an enclosed room as in the existing TWSA intake building.

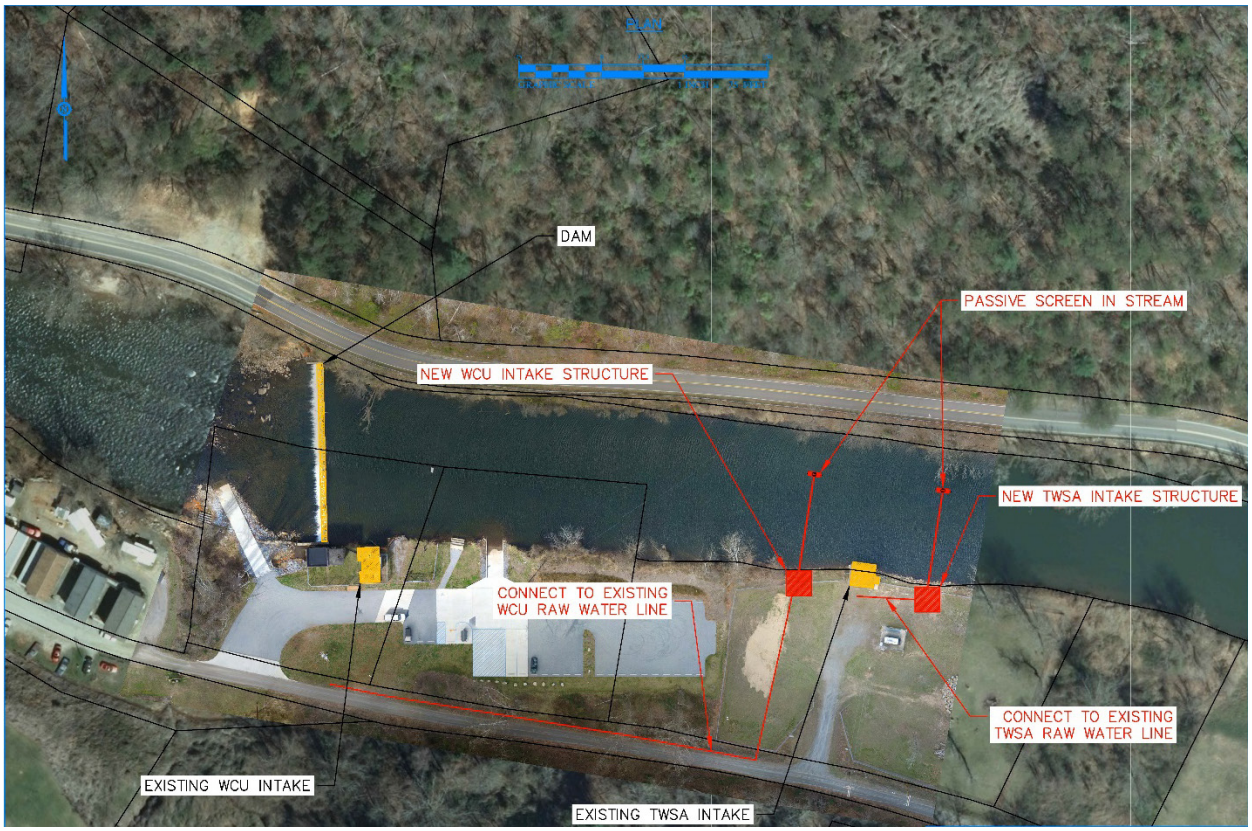

Figure 26. Dam Removal Alternative - Separate Replacement Intake Structures Layout

The pump buildings would be constructed to similar dimensions with pumps and controls sized for each utility. The screens and wet wells would be sized for the eventual 50-year demand. Raw water transmission lines from the new intake would connect to the existing raw water lines to each treatment plant.

The TWSA generator and its platform could be reused for the new intake facility, but it is recommended that WCU install its own generator to be similarly elevated above the 100-year flood elevation. The eventual 50-year max day demand would require that TWSA upgrade its two existing 100 hp raw water pumps to two 200 hp raw water pumps. WCU could replace its existing 50 hp and two 30 hp pumps with two 75 hp pumps. This pump upgrade may be better performed in a future capacity expansion project rather than at initial intake construction.

This alternative has an estimated project cost of $6,100,000, not including the dam removal itself and stream bank stabilization. Total estimated project cost for this alternative is $7,000,000,

which includes $700,000 for dam removal and $200,000 for stream bank stabilization described above.

F. Alternative 3.B: Combined Replacement Intake Structure

This alternative would call for the construction of a single replacement intake structure to be operated by TWSA and WCU, with separate raw water supply lines to the two existing water treatment plants. A single in-stream intake screen with airburst system and raw water intake pipe sized to accommodate the combined flows of the TWSA and WCU water systems would enter a wet well containing four pumps: two 100 hp pumps for TWSA and two 50 hp pumps for WCU. Space for a third pump for each system would be provided in the wet well for future expansion. The pumps would connect to existing raw water lines for each water system.

Maintenance and operations costs would be shared between the two water systems for the intake screen and building itself while separate power supplies and meters would be provided for each control panel and set of pumps. A single backup generator capable of supplying emergency power to both intakes would be constructed on the existing TWSA backup generator platform.

The lowest elevations noted in recent bathymetric surveys of the project area are approximately 85 feet offshore of the TWSA intake structure, so the best location for an in-stream intake and new pump building would be the same as the location for the replacement WCU intake equipment shown in Figure 25 above.

Figure 27. Dam Removal Alternative - Combined Replacement Intake Structure Layout

This alternative has an estimated project cost of $4,100,000, not including the dam removal itself and stream bank stabilization. Total estimated project cost for this alternative is $5,000,000, which includes $700,000 for dam removal and $200,000 for stream bank stabilization described above.

IV. Discussion of Alternatives

A. Need for Dam Removal

Whether Cullowhee Dam's removal is warranted from an economic or political standpoint is beyond the scope of this report. However, while the removal itself has a cost between those of the minor and major repair alternatives, the environmental mitigation and raw water intake replacements that dam removal would necessitate constitute a significant expenditure.

B. Comparison of Alternatives

Table 8. Alternatives Comparison

Alternative 1 - No Action	
Project Cost	$0
Operations & Maintenance	continued replacement of flashboards as needed
Risk	potential for partial dam failure due to existing and/or undiscovered issues
Regulatory Concerns	none
Alternative 2 - Structural Repairs and Improvements to the Existing Dam	
2.A - Address Safety Concerns Only	
Project Cost	$500,000
Operations & Maintenance	continued replacement of flashboards as needed
Risk	reduced
Regulatory Concerns	USACE and NCDEQ permits needed to perform work
2.B - Address Safety and Maintenance Concerns	
Project Cost	$900,000
Operations & Maintenance	minimal
Risk	minimal
Regulatory Concerns	USACE and NCDEQ permits needed to perform work
2.C - Incorporate Downstream Improvement Plans	
Project Cost	$1,000,000
Operations & Maintenance	minimal
Risk	minimal
Regulatory Concerns	USACE and NCDEQ permits needed to perform work; floodplain and floodway map revision if dam height is increased; potential violation of prohibition against recreational activities within 50 yards of public water supply

Alternative 3 - Dam Removal	
3.A. Separate Intake Structures	
Project Cost	$7,000,000; loss of $1,500,000 value of existing dam
Operations & Maintenance	none for dam; potential reduction in O&M costs for raw water intakes (similar to 3.B)
Risk	potential increase in vulnerability to drought for both water systems
Regulatory Concerns	USACE and NCDEQ permits needed to perform work in stream; environmental mitigation necessary for sediment control and stream bank stabilization; potential reclassification of river as water supply; potential pre-settling and/or offstream raw water storage requirements for WTPs
3.B. Combined Intake Structure	
Project Cost	$5,000,000; loss of $1,500,000 value of existing dam
Operations & Maintenance	none for dam; potential reduction in O&M costs for raw water intakes (similar to 3.A)
Risk	potential increase in vulnerability to drought for both water systems
Regulatory Concerns	USACE and NCDEQ permits needed to perform work in stream; environmental mitigation necessary for sediment control and stream bank stabilization; potential reclassification of river as water supply; potential pre-settling and/or offstream raw water storage requirements for WTPs

C. Water System Vulnerability

As with any attempt to extrapolate future conditions from historical trends, an underlying assumption is that those trends will continue in a predictable manner. There is no officially published "drought of record" for Jackson County, North Carolina, but the 2016 drought shown in Figure 19 has been noted as the lowest rainfall for nearby Buncombe County since the late 1800s. If additional "record" droughts continue to occur in the future, low flows not reflected in this report could be observed in the Tuckasegee River.

Assuming a surface elevation for the reservoir of 3.7 acres and a water surface at the same elevation as the dam crest, only around 1.7 million gallons of water is available above the screen invert of the WCU intake, and only 3.1 million gallons is available to the TWSA intake. These amounts are not cumulative. In effect, if inflow into the reservoir ceased entirely, WCU and TWSA would compete for the first 1.7 million gallons of water (less than their projected combined 2067 average day demand), after which the remaining 1.5 million gallons would be available only to

TWSA, and would be depleted early in the second day. Therefore, the dam does provide a small buffer of storage in case of periods of extreme drought.

If the No Action alternative is taken, any risk of dam failure should also be interpreted as an equal risk of immediate and total failure of both raw water intakes. If a section of the dam were to collapse and cause the normal pool surface elevation to drop below the intake screen inverts, the intakes would be unable to function normally until dam repairs could be completed. It is possible that emergency pumps and intakes could be employed in the interim, however due to the lack of a pre-settling reservoir (see below), WCU's water treatment plant may be unequal to the task of treating any water withdrawn in such a manner.

D. Water Treatment

The primary function of the current dam is not to provide a reservoir of on-stream raw water storage for the water systems, but as an aid to the process of raw water collection and treatment. It supplies sufficient head to allow the intakes to function at their current locations. It also reduces the velocity of the stream and reduces turbidity, serving as an on-stream pre-settling reservoir for both water treatment plants.

Two North Carolina laws concerning off-stream storage of raw water constitute potential treatment process concerns as dam removal is considered. The first, 15A NCAC 18C .0403, paragraph (b), concerns pre-settling reservoirs and states the following:

> *(b) Pre-settling Reservoirs. Construction of a pre-settling or pre-treatment reservoir shall be required where wide and rapid variations in turbidity, bacterial concentrations or chemical qualities occur or where the following raw water quality standards are not met: turbidity – 150 NTU, coliform bacteria – 3000/100 ml, fecal coliform bacteria – 300/100 ml, color – 75 CU.*

15A NCAC 18C .0601 adds the following language:

> *Where impoundment of the water supply stream does not or will not provide a raw water of acceptable quality, a pre-settling or pre-treatment reservoir located outside the watershed or catchment area may be required.*

NCDEQ engineer Randy Hintz clarified that the pre-settling requirements are triggered by specific conditions in the raw water supply that he does not anticipate occurring if the existing dam is removed, and that he would not recommend that this planning study attempt to take into consideration any changes in treatment processes for this project.

Paragraph (c) of the same North Carolina law states the following regarding raw water storage:

(c) Impoundments. Raw water storage capacity shall be sufficient to reasonably satisfy the designed water supply demand during periods of drought.

North Carolina law does not appear to further clarify this requirement or give specific design criteria for the duration of drought or amount of raw water storage capacity relative to demand or capacity required. The current land use and topography of the project area does not suggest any practicable locations for the construction of raw water storage ponds. NCDEQ engineers have expressed the opinion that the department would scrutinize any plans to remove the dam and construct new intakes carefully in terms of potential increased vulnerability of the water systems.

It is possible that the relocation and replacement of the existing raw water intakes for WCU and TWSA will necessitate review of both systems' treatment processes in light of current North Carolina regulations and design standards. During the initial permitting process for TWSA's existing water treatment plant, McGill Associates design engineers and NCDEQ (then NCDEHNR) regulators debated various sizes of pre-settling reservoirs before agreeing on the flocculating clarifier currently in service at the TWSA plant. Should its intake be relocated, the WCU water treatment plant is likely to also be scrutinized by current standards. Extensive modifications necessary to bring the WCU water treatment process into compliance with standards that were not in effect at the time of the plant's original construction may be required. These modifications would involve significant capital as well as operations and maintenance costs that are beyond the scope of this report and have not been included in the above discussion of intake replacement alternatives.

CONCLUSION

Alternative 2.B, the dam repair alternative that includes repair of the undercut and right sidewall as well as the installation of a 12" concrete overlay on the crest and downstream face of the dam in order to improve stability and eliminate the timber flashboards, is the recommended alternative. This alternative adds weight to the dam, increasing its factor of safety for both sliding and overturning for the potential loading conditions considered, while also addressing its structural deficiencies and lowering maintenance costs. As a partial encapsulation of the entire dam, this alternative is more comprehensive than Alternative 2.A and reduces the possibility of additional deficiencies that may require further repairs arising at a later date. Completion of these modifications should increase the expected service life of Cullowhee Dam by at least 50 years.

Alternative 2.B also preserves the current overflow elevation of the dam and eliminates the risk of flashboard failure. Any alternative that either intentionally reduces the pool surface elevation or fails to decrease the risk of that outcome should be avoided due to the probability of adverse consequences for the continued operation of the raw water intakes, which may necessitate their replacement.

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NO.	DATE	BY	REVISION DESCRIPTION

McGill ASSOCIATES
ENGINEERING·PLANNING·FINANCE
55 BROAD STREET ASHEVILLE, NC 28801 PH. (828) 252-0575 FIRM LICENSE # C-0459

CULLOWHEE DAM EVALUATION
WESTERN CAROLINA UNIVERSITY
TUCKASEIGEE WATER & SEWER AUTHORITY
JACKSON COUNTY, NORTH CAROLINA

JOB NO.: 16.02000
DATE: JANUARY, 2017
DESIGNED BY: MJW2
CADD BY: MJW2
DESIGN REVIEW: ___
CONST. REVIEW: ___
FILE NAME:
16.02000 WCU TWSA Cullowhee Dam.dwg

ALTERNATIVE 3.A
SEPARATE REPLACEMENT
INTAKE STRUCTURES

FIGURE
A - 30

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ALTERNATIVE 3.B
COMBINED REPLACEMENT
INTAKE STRUCTURE

FIGURE
A - 31

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Appendix 2

Cultural and Environmental Resources Assessment

CULTURAL AND ENVIRONMENTAL RESOURCES ASSESSMENT

Cultural Resources Assessment

For purposes of determining the impact of modifications to the intake structure, the project boundary was defined as follows:

- Downstream Tuckasegee River Boundary - Cullowhee Road (SR 1002) Bridge
- Northern Boundary - Wayehutta Road (SR 1732)
- Upstream Tuckasegee River Boundary – a point perpendicular to the river located near the end of Casey Road
- Southern Boundary - Casey Road (Private)

The North Carolina Department of Cultural Resources (NCDCR) HPOWEB online interactive mapping tool was consulted for important architectural elements that may be within the project boundary. Additional information was obtained from the Western Regional NCDCR office Preservation Specialist Annie McDonald.

The North Carolina Office of State Archaeology Western Office was consulted for the presence of known archaeological resources within the project boundary. Archaeological records are not publicly available.

The Eastern Band of the Cherokee Indians Tribal Historic Preservation Office was consulted to obtain information on the presence of known Indian cultural resources within the project boundary.

Findings

Architectural – No architectural elements are known to exist within the project boundary; however, the existing Cullowhee Road (SR 1002) bridge has been identified by the NCSHPO as a potential candidate for the National Register of Historic Places. To date, it has not been evaluated to determine its eligibility.

Review of the NCSHPO web map also revealed that the entire project falls within the Western Carolina University Historic District. Closer examination of the boundary indicates that the boundary may have been arbitrarily defined and not based on the presence or absence of historic structures. The Western SHPO representative reviewed the files associated with the establishment of the historic district and concluded "that the Study Listed WCU Historic District may extend only to the historic buildings documented in 1992 and included in the survey file for [site identifier] JK0585 (and thus limited to the area outlined in red on the accompanying map). Were we to reevaluate the campus for National Register eligibility today, I expect that the boundaries may extend farther out. At the greatest possible extent, the Study List boundaries may be limited to only those buildings constructed through the mid-1960s".

Based on the information provided in the NCSHPO letter it appears that the Cullowhee Dam project would not fall within a revised historic district boundary. As such, there does not appear to be any

architectural resources that would be impacted by any proposed modification to the Cullowhee Dam. Should any modifications be proposed, a formal environmental review of cultural resources would be required to confirm these preliminary findings.

Archaeological – Based on information provided by Linda Hall of the Office of State Archaeology, there are "no previously recorded [archaeological] sites within or adjacent to the project boundary". Furthermore, she stated "that as long as ground disturbing activities remain within the project boundary, it is unlikely that significant resources would be affected". As such, there does not appear to be any archaeological resources that would be impacted by any proposed modification to the Cullowhee Dam. Should any modifications be proposed, a formal environmental review of all cultural resources would be required to confirm these preliminary findings.

A response from the EBCI-THPO indicates that they believe there would be no negative impacts to cultural resources or historic properties within or adjacent to the project boundary. They stated that there are no signs of historic EBCI features within the Tuckasegee River and that the nearby Cullowhee Mound and village has been destroyed and lacks integrity.

Conclusions

Based on the preliminary research no architectural, archaeological, or other cultural resources are likely to be affected by any modification of the Cullowhee Dam and adjacent water intake structures. However, any proposed modifications of those structures may require formal cultural resource assessments as require by law during the permitting process.

Natural Resources Assessment

The U.S. Fish and Wildlife Service (USFWS) and North Carolina Wildlife Resources Commission (NCWRC) were consulted to identify any plant or animal species of concern that may be affected by activities associated with modification or removal of the existing dam and appurtenant structures.

Findings

A table of the USFWS threatened and endangered species list within the project area is presented below. Critical habitat for the Appalachian elktoe mussel is mapped as beginning immediately downstream of the dam. However, a Natural Heritage Element Occurrence search does not include any documented occurrences of Appalachian elktoe within one mile of the project area.

Table 1. Threatened and Endangered Species List

Common Name	Scientific Name	Federal Status	Record Status
Arachnids			
Spruce-Fir Moss Spider	*Microhexura montivaga*	Endangered	Current
Clams			
Appalachian elktoe	*Alasmidonta raveneliana*	Endangered	Current
Flowering Plants			
Small Whorled pogonia	*Isotria medeoloides*	Threatened	Current
Swamp pink	*Helonias bullata*	Threatened	Current
Lichens			
Rock Gnome lichen	*Gymnoderma lineare*	Endangered	Current
Mammals			
Carolina Northern Flying squirrel	*Glaucomys sabrinus coloratus*	Endangered	Current
Indiana bat	*Myotis sodalis*	Endangered	Current
Northern long-eared Bat	*Myotis septentrionalis*	Threatened	Current

Species Descriptions

Swamp Pink (*Helonias bullata*)

A perennial member of the lily family, swamp pink has smooth, oblong, dark green leaves that form an evergreen rosette. In spring, some rosettes produce a flowering stalk that can grow over 3 feet tall. The stalk is topped by a 1 to 3-inch-long cluster of 30 to 50 small, fragrant, pink flowers dotted with pale blue anthers. The evergreen leaves of swamp pink can be seen year round, and flowering occurs between March and May.

An obligate wetland species, swamp pink occurs in a variety of palustrine forested wetlands including swampy forested wetlands bordering meandering streamlets, headwater wetlands, sphagnous Atlantic white-cedar swamps, and spring seepage areas. Specific hydrologic requirements of swamp pink limit its occurrence within these wetlands to areas that are

perennially saturated, but not inundated, by floodwater. The water table must be at or near the surface, fluctuating only slightly during spring and summer months. Groundwater seepage with lateral groundwater movement are common hydrologic characteristics of swamp pink habitat.

Swamp pink is a shade-tolerant plant and has been found in wetlands with canopy closure varying between 20-100%. Sites with minimal canopy closure are less vigorous due in part to competition from other species. Common vegetative associates of swamp pink include Atlantic white-cedar (*Chamaecyparis thyoides*), red maple (*Acer rubrum*), pitch pine (*Pinus rigida*), American larch (*Larix laricina*), black spruce *(Picea mariana*), red spruce (*P. rubens*), sweet pepperbush (*Clethra alnifolia*), sweetbay magnolia (*Magnolia virginiana*), sphagnum mosses (*Sphagnum spp*.), cinnamon fern (*Osmunda cinnamomea*), skunk cabbage (*Symplocarpus foetidus*), and laurels (*Kalmia spp*.). Swamp pink is often found growing on the hummocks formed by trees, shrubs, and sphagnum mosses, and these micro-topographic conditions may be an important component of swamp pink habitat.

Small Whorled Pogonia (*Isotria medeoloides)*

The small whorled pogonia is a perennial orchid with long, pubescent roots and a smooth, hollow stem 9.5 to 25 centimeters tall terminating in a whorl of 5 or 6 light green, elliptical leaves that are somewhat pointed and measure up to 8 by 4 cm. A flower, or occasionally two flowers, is produced at the top of the stem. Small whorled pogonia's nearest relative is *I. verticillata*, which is similar looking but can be distinguished by its purplish stem and by differences in the flower structure. *I. verticillata* is much more common and widespread than the small-whorled pogonia. When not in flower, young plants of Indian cucumber-root (*Medeola virginiana*) also resemble small whorled pogonia, however, the hollow stout stem of *Isotria* will separate it from the genus *Medeola*, which has a solid, more slender stem. Flowering occurs from about mid-May to mid-June.

In North Carolina, this species is typically found in montane oak-hickory or acidic cove forests. The understory structure and composition of occupied sites can be quite variable, ranging from dense rhododendron thickets to open/sparse shrub and sub-shrub strata. Herbaceous cover tends to be sparse, however at least two sites are characterized by fairly dense stands of New York fern (*Thelypteris noveboracensis*). Sites currently or historically known to support this species range from 2,000 to 4,000 feet in elevation. The species does not appear to exhibit strong affinities for a particular aspect, soil type, or underlying geologic substrate. Habitat manipulation experiments in New England indicate that the species responds favorably to canopy openings, and may therefore be light-limited, however this remains to be observed in the southern Blue Ridge portion of the species' range.

Northern Long-Eared Bat (*Myotis septentrionalis*)

The northern long-eared bat is a medium-sized bat about 3 to 3.7 inches in length but with a wingspan of 9 to 10 inches. As its name suggests, this bat is distinguished by its long ears, particularly as compared to other bats in its genus *Myotis*, which are actually bats noted for their small ears (*Myotis* means “mouse-eared”). The northern long-eared bat is found across much of

the eastern and north central United States and all Canadian provinces from the Atlantic coast west to the southern Northwest Territories and eastern British Columbia.

Northern long-eared bats spend winter hibernating in caves and mines, called hibernacula. They use areas in various sized caves or mines with constant temperatures, high humidity, and no air currents. Within hibernacula, surveyors find them hibernating most often in small crevices or cracks, often with only the nose and ears visible. During the summer, northern long-eared bats roost singly or in colonies underneath bark, in cavities or in crevices of both live trees and snags (dead trees). Males and non-reproductive females may also roost in cooler places, like caves and mines. Northern long-eared bats seem to be flexible in selecting roosts, choosing roost trees based on suitability to retain bark or provide cavities or crevices. This bat has also been found rarely roosting in structures, like barns and sheds.

The northern long-eared bat's range includes much of the eastern and north central United States, and all Canadian provinces from the Atlantic Ocean west to the southern Yukon Territory and eastern British Columbia. The species' range includes the following 37 States and the District of Columbia: Alabama, Arkansas, Connecticut, Delaware, Georgia, Illinois, Indiana, Iowa, Kansas, Kentucky, Louisiana, Maine, Maryland, Massachusetts, Michigan, Minnesota, Mississippi, Missouri, Montana, Nebraska, New Hampshire, New Jersey, New York, North Carolina, North Dakota, Ohio, Oklahoma, Pennsylvania, Rhode Island, South Carolina, South Dakota, Tennessee, Vermont, Virginia, West Virginia, Wisconsin, and Wyoming.

White-nose syndrome, a fungal disease (*Pseudogymnoascus destructans*) known to affect bats, is currently the predominant threat to this bat, especially throughout the Northeast where the species has declined by up to 99 percent from pre-white-nose syndrome levels at many hibernation sites. Although the disease has not yet spread throughout the northern long-eared bat's entire range (white-nose syndrome is currently found in at least 25 of 37 states where the northern long-eared bat occurs), it continues to spread. Experts expect that where it spreads, it will have the same impact as seen in the Northeast.

Indiana Bat (*Myotis sodalis)*

Indiana bats are quite small, weighing only one-quarter of an ounce although in flight they have a wingspan of 9 to 11 inches. Their fur is dark-brown to black. *Myotis* means "mouse eared" and refers to the relatively small, mouse-like ears of the bats in this group. *Sodalis* is the Latin word for "companion." The Indiana bat is a very social species; large numbers cluster together during hibernation.

Indiana bats hibernate during winter in caves or, occasionally, in abandoned mines. For hibernation, they require cool, humid caves with stable temperatures, under 50° F but above freezing. After hibernation, Indiana bats migrate to their summer habitat in wooded areas where they usually roost under loose tree bark on dead or dying trees. During summer, males roost alone or in small groups, while females roost in larger groups of up to 100 bats or more. Indiana bats eat a variety of flying insects found along rivers or lakes and in uplands. Indiana bats also forage in or along the edges of forested areas.

Indiana bats are found over most of the eastern half of the United States. Almost half of all Indiana bats hibernate in caves in southern Indiana. In 2005, other states known to support populations of over 40,000 individuals included Missouri, Kentucky, Illinois, and New York. Other states within the current range of the Indiana bat include Alabama, Arkansas, Connecticut, Iowa, Maryland, Michigan, New Jersey, North Carolina, Ohio, Oklahoma, Pennsylvania, Tennessee, Vermont, Virginia, and West Virginia.

Carolina Northern Flying Squirrel (*Glaucomys sabrinus)*

Northern flying squirrels are about one-third larger than the very common southern species. Northern flying squirrels are brown on their backs and their fur fades to a buff white on their belly. The endangered Carolina northern flying squirrel is a subspecies of the northern flying squirrel.

Flying squirrels are nocturnal and have large eyes to help them see at night. They cannot actually fly, but glide by extending a fold of skin that stretches from their wrists to their ankles. The flattened tail acts as a rudder. Carolina northern flying squirrels are relicts of the last ice age. As the glaciers retreated northward and temperatures rose, remnant populations remained in the suitable habitat left behind on the high mountain tops along the ridges of the Southern Appalachian Mountains. Northern flying squirrels principally feed on certain fungi and lichens, though they do occasionally eat some fruits and nuts. They are active year-round, but more so in the warmer summer months. They nest in tree cavities in nests made almost exclusively of yellow birch (*Betula alleghaniensis*) bark, where two to six young are born in early spring. Groups of squirrels often occupy the same tree cavity, particularly in the colder winter months.

Northern flying squirrels are typically found in areas where northern hardwoods, such as yellow birch (*Betula alleghaniensis*), are adjacent to the higher-elevation red spruce-Fraser fir forest. These habitats are often moist and cool. Carolina northern flying squirrels are found on high mountain peaks in southwest Virginia, western North Carolina, and eastern Tennessee.

Appalachian Elktoe

The Appalachian elktoe has a thin, kidney-shaped shell, extending to about 4 inches. Juveniles generally have a yellowish-brown outer shell surface, while the outer shell surface of the adults is usually dark brown to greenish-black in color. Although rays are prominent on some shells, particularly in the posterior portion of the shell, many individuals have only obscure greenish rays. The shell inside shell surface is shiny, often white to bluish-white, changing to a salmon, pinkish, or brownish color in the central and beak cavity portions of the shell; some specimens may be marked with irregular brownish blotches. The species has been reported from relatively shallow, medium-sized creeks and rivers with cool, clean, well-oxygenated, and moderate to fast-flowing water. The species is most often found in riffles, runs, and shallow flowing pools with stable, relatively silt-free, coarse sand and gravel substrate associated with cobble, boulders, and/or bedrock. Stability of the substrate appears to be critical to the Appalachian elktoe, and the species is seldom found in stream reaches with accumulations of silt or shifting sand, gravel, or cobble. Individuals that have been encountered in these areas are believed to have been

scoured out of upstream areas during periods of heavy rain, and have not been found on subsequent surveys.

The Appalachian elktoe is known only from the mountain streams of western North Carolina and eastern Tennessee. Currently, the Appalachian elktoe has a very fragmented, relict distribution. The species still survives in scattered pockets of suitable habitat in portions of the Little Tennessee River system, scattered reaches of the main stem of the Tuckasegee River, Cheoah River, Toe River, Cane River, North Toe River, Pigeon River system, Mills River, and Little River in North Carolina, and the Nolichucky River system in North Carolina and Tennessee. The majority of the surviving occurrences of the Appalachian elktoe appear to be small to extremely small and restricted to scattered pockets of suitable habitat. Critical habitat for the Appalachian elktoe has been federally designated from Lake Emory downstream to Fontana Lake.

Rock Gnome Lichen (*Gymnoderma lineare*)

The rock gnome lichen of the reindeer moss family grows in dense colonies of narrow, strap-like lobes, called squamules. The squamules are blue-gray on the upper surface and usually shiny white on the lower surface. Near the base of the lobe, the color darkens to black. The slightly branched squamules are less than 0.04 inch (1 mm) across near the tip, and are usually 0.4-0.8 inch (1-2 cm) long. The squamules grow parallel to the substrate, but the tips curl up almost perpendicularly. The small fruiting bodies (apothecia) occur at the tips of the squamules from July-September. They are colored black or brown, and are no larger than 1 mm across. The fruiting bodies may be sessile, or they may be carried on short stalks (podetia) less than 0.1 in. height. The fruiting bodies are shaped like cylinders. Similar-looking lichens in the genus *Cladonia* do not blacken near the base and have brown or red fruiting bodies.

The rock gnome lichen only grows in areas with a great deal of humidity, such as high elevations above 5,000 feet where there is often fog, or in deep river gorges at lower elevations. Habitat is restricted to vertical rock faces occasionally exposed to seepage water. It does well on moist, generally open sites with northern exposures but needs partial canopy coverage on southern or western aspect because it is intolerant of high-intensity solar radiation. High-elevation coniferous forests, red spruce (*Picea ruben*) and Fraser fir (*Abies fraseri*), usually on rocky outcrop or cliff habitat.

Rock gnome lichen is endemic to the southern mountains of Tennessee, North Carolina, South Carolina, and Georgia. Only 35 populations are known to exist and most are 1 square meter or less in size. It is the only member of its genus in North America. Populations have been reported in Ashe, Avery, Buncombe, Graham, Haywood, Jackson, Macon, Mitchell, Rutherford, Swain, Transylvania, and Yancey counties.

Spruce-fir Moss Spider (*Mirohexura montivaga*)

The spruce-fir moss spider coloration ranges from light brown to darker reddish brown with no markings on the abdomen. The carapace is generally yellowish brown with chelicerae that project forward well beyond the anterior edge of the carapace. It has a pair of very long posterior

spinnerets, and a second pair of book lungs that appear as light patches posterior to the genital furrow. Adult spiders measure only 0.10 - 0.15 inches (2.5 – 3.8 mm) in length.

The spider constructs its tube-shaped webs in the interface between the moss mat and rock surface. Although there are no records of prey being found in the webs, the species has been observed taking prey in the wild. The abundant springtales in the moss mats provide the most likely food source for the spider. Males of the species mature during September and October, and females lay eggs in June. The thin-walled, transparent egg sac may contain seven to nine eggs. The female remains with the egg sac and will carry it with her fangs if disturbed. Spiderlings emerge in September. It is estimated that it may take three years for the species to reach maturity.

The spruce-fir moss spider is found in damp but well-drained moss and liverwort mats growing on rocks or boulders and in well-shaded areas of mature, high-elevation Fraser fir and fir dominated spruce-fir forests. The moss mats cannot be to dry, as the species is very sensitive to desiccation, or too wet because large drops of water pose a threat to the spider. The spider is known to only exist is six locations: Mount Collins and Clingmans Dome in Swain County, NC; Grandfather Mountain in Avery, Watauga, and Caldwell Counties; Roan Mountain in Avery and Mitchell counties, NC and Carter County, TN; and, Mount LeConte and Mount Buckley in Sevier County, TN. It is believed to be extirpated from Mount Mitchell in Yancey County, NC.

Conclusions

If either of the dam repair options or the dam removal option is chosen, a consultation with USFWS and the NWRC would be appropriate given that critical Appalachian elktoe habitat exists immediately downstream of the dam. The dam removal option is the only option that possibly extends the critical habitat of the Appalachian elktoe mussell. The removal option would also likely restore habitat and improve passage for the Sicklefin redhorse. Removal of the dam would provide other habitat improvements and restore the free flowing nature of the river. Ecological uplift would be enhanced by removal of the dam.

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Appendix 3

Land Use and Potential Contaminant Sources

LAND USE AND POTENTIAL CONTAMINANT SOURCES

In terms of physical characteristics, the drainage area for the watershed of the Tuckasegee River above the Cullowhee dam is 207 square miles and the perimeter of the basin is 101 miles. The mean annual precipitation for the watershed is 68.7 inches, and the maximum 24-hour precipitation that occurs on average once in 50 years is 10 inches.

Land Cover and Land Use

Land cover for the water supply basin is taken from the National Land Cover Database 2011 (NLCD 2011), the most recent national land cover product created by the Multi-Resolution Land Characteristics (MRLC) Consortium. The percentage of forest for the water supply basin is 87 percent. Approximately 5 percent of the basin is developed (urban) land. The average percentage of impervious area determined from NCLD 2011 data is 0.3 percent. A map of land cover and land use characteristics is provided (Figure 1).

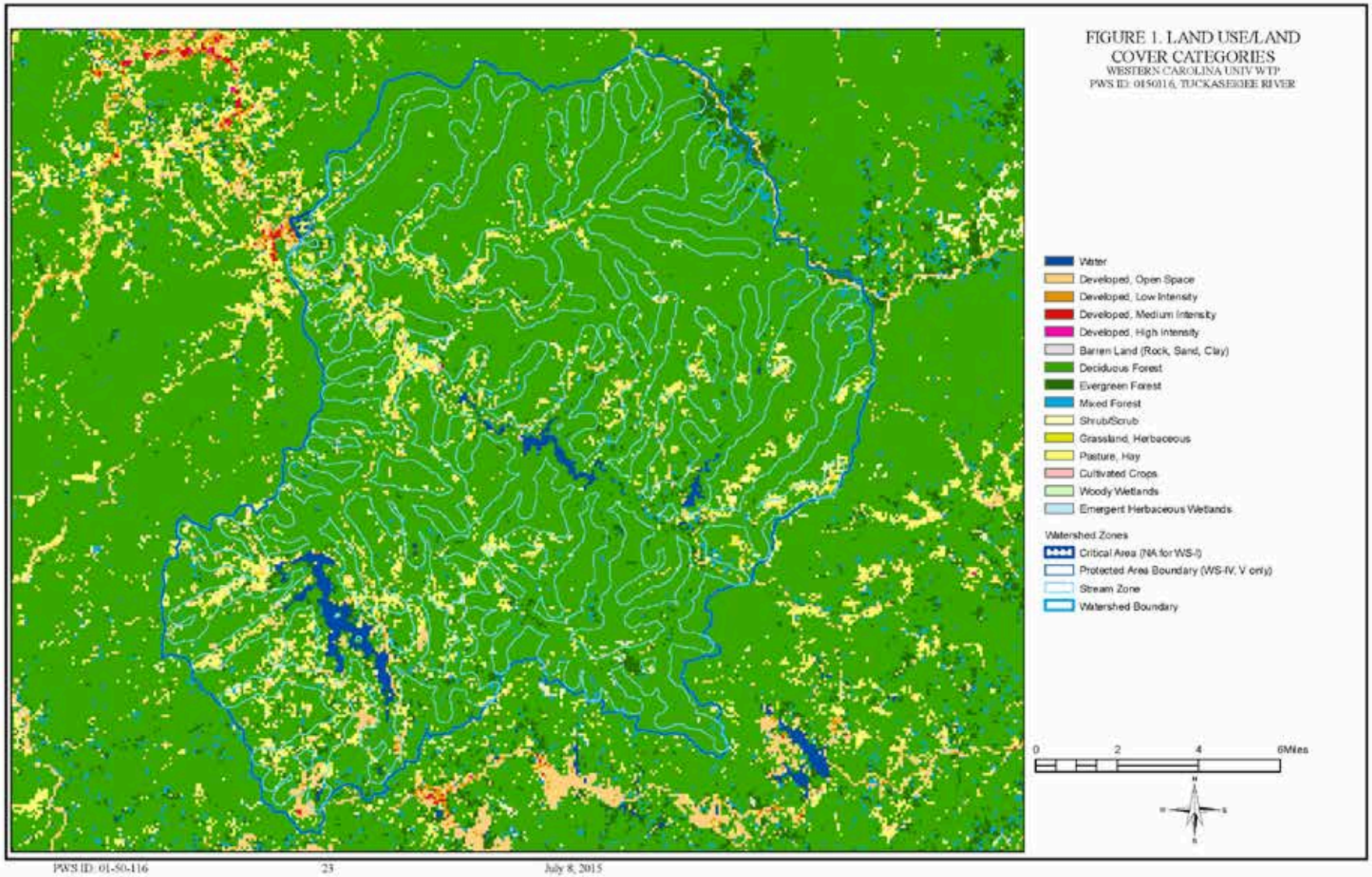
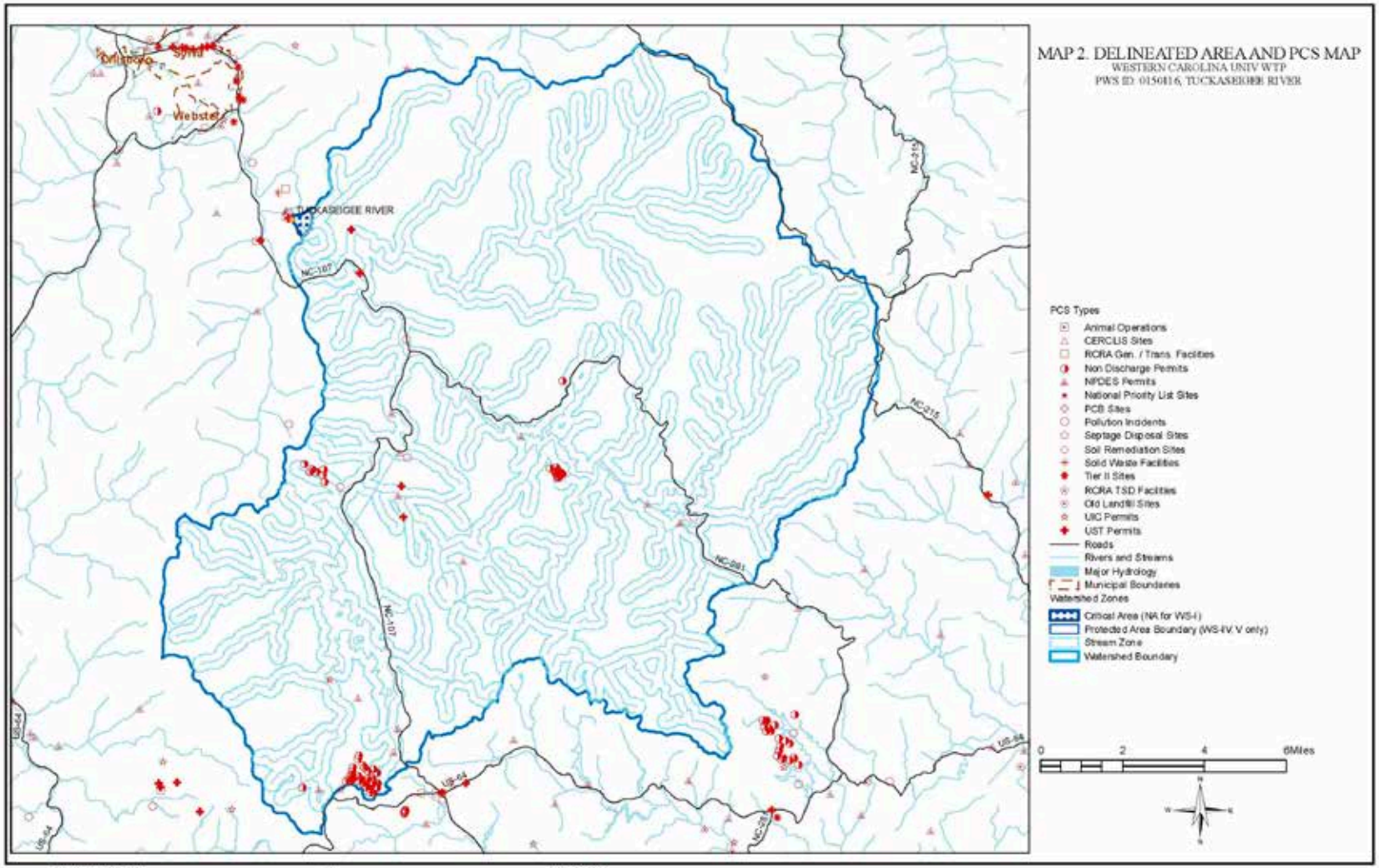


FIGURE 1. LAND USE/LAND COVER CATEGORIES
WESTERN CAROLINA UNIV WTP
PWS ID: 0150116, TUCKASEGEE RIVER
Water
Developed, Open Space
Developed, Low Intensity
Developed, Medium Intensity
Developed, High Intensity
Barren Land (Rock, Sand, Clay)
Deciduous Forest
Evergreen Forest
Mixed Forest
Shrub/Scrub
Grassland, Herbaceous
Pasture, Hay
Cultivated Crops
Woody Wetlands
Emergent Herbaceous Wetlands
Watershed Zones
Critical Area (NA for WS-I)
Protected Area Boundary (WS-IV, V only)
Stream Zone
Watershed Boundary
0
2
4
6Miles
N
W
E
S


MAP 2. DELINEATED AREA AND PCS MAP
WESTERN CAROLINA UNIV WTP
PWS ID: 0150116, TUCKASEIGEE RIVER
Dillsboro
Sylva
Webster
TUCKASEIGEE RIVER
NC-107
NC-215
NC-281
US-64
PCS Types
Animal Operations
CERCLIS Sites
RCRA Gen. / Trans. Facilities
Non Discharge Permits
NPDES Permits
National Priority List Sites
PCB Sites
Pollution Incidents
Septage Disposal Sites
Soil Remediation Sites
Solid Waste Facilities
Tier II Sites
RCRA TSD Facilities
Old Landfill Sites
UIC Permits
UST Permits
Roads
Rivers and Streams
Major Hydrology
Municipal Boundaries
Watershed Zones
Critical Area (NA for WS-I)
Protected Area Boundary (WS-IV, V only)
Stream Zone
Watershed Boundary
0
2
4
6Miles
PWS ID: 01-50-116
7
July 8, 2015

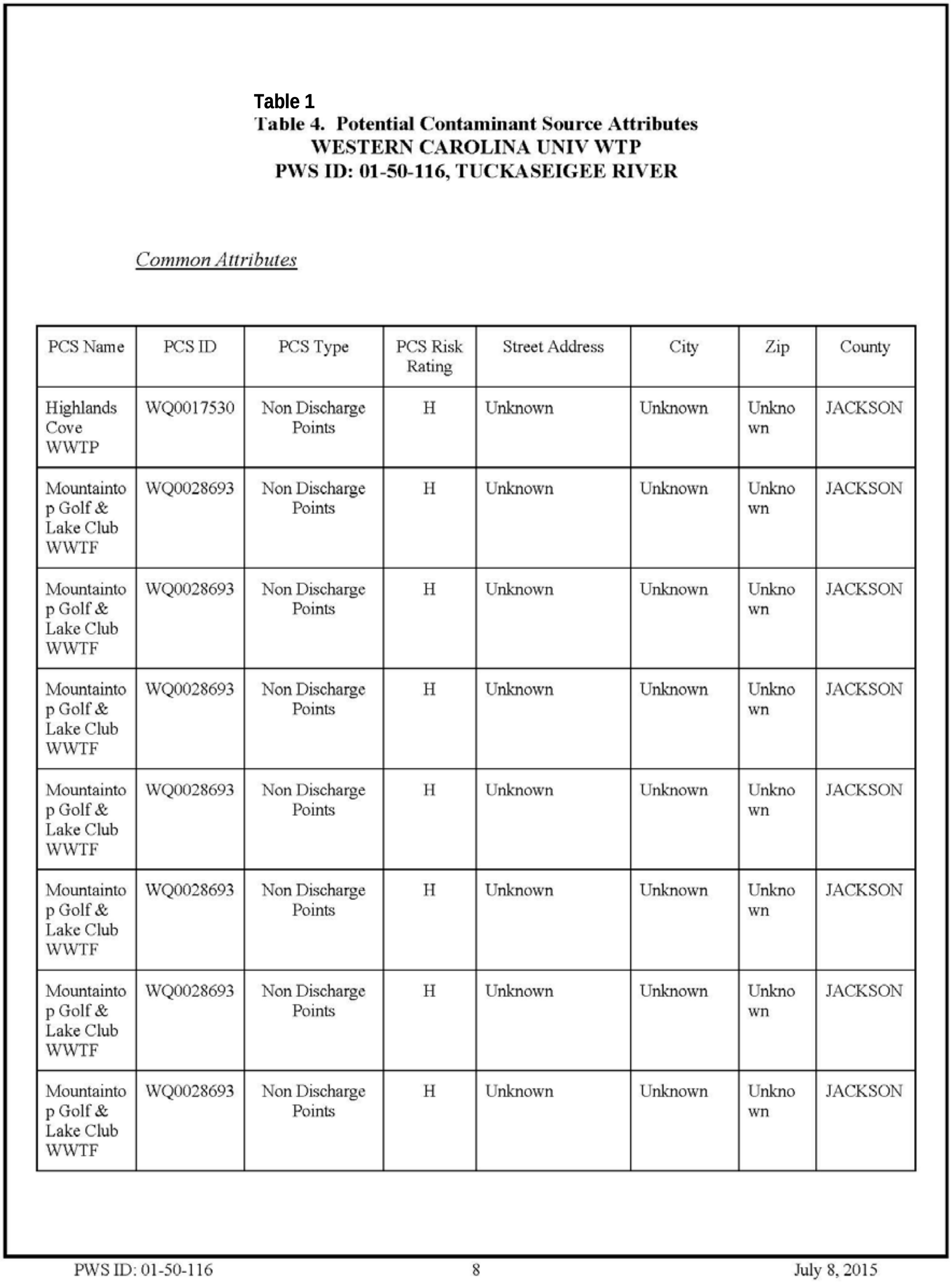


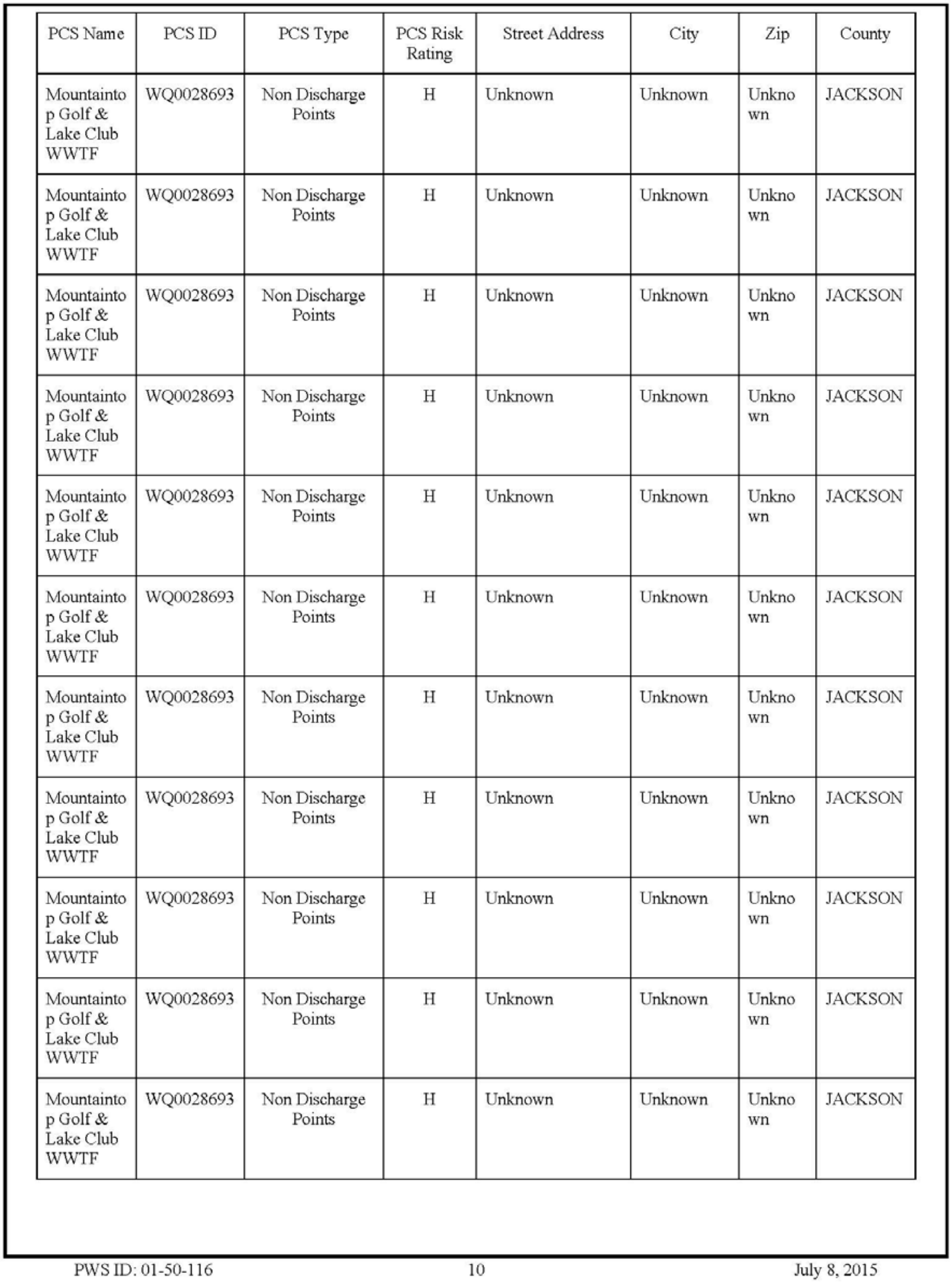
Table 1

Table 4. Potential Contaminant Source Attributes
WESTERN CAROLINA UNIV WTP
PWS ID: 01-50-116, TUCKASEIGEE RIVER

Common Attributes

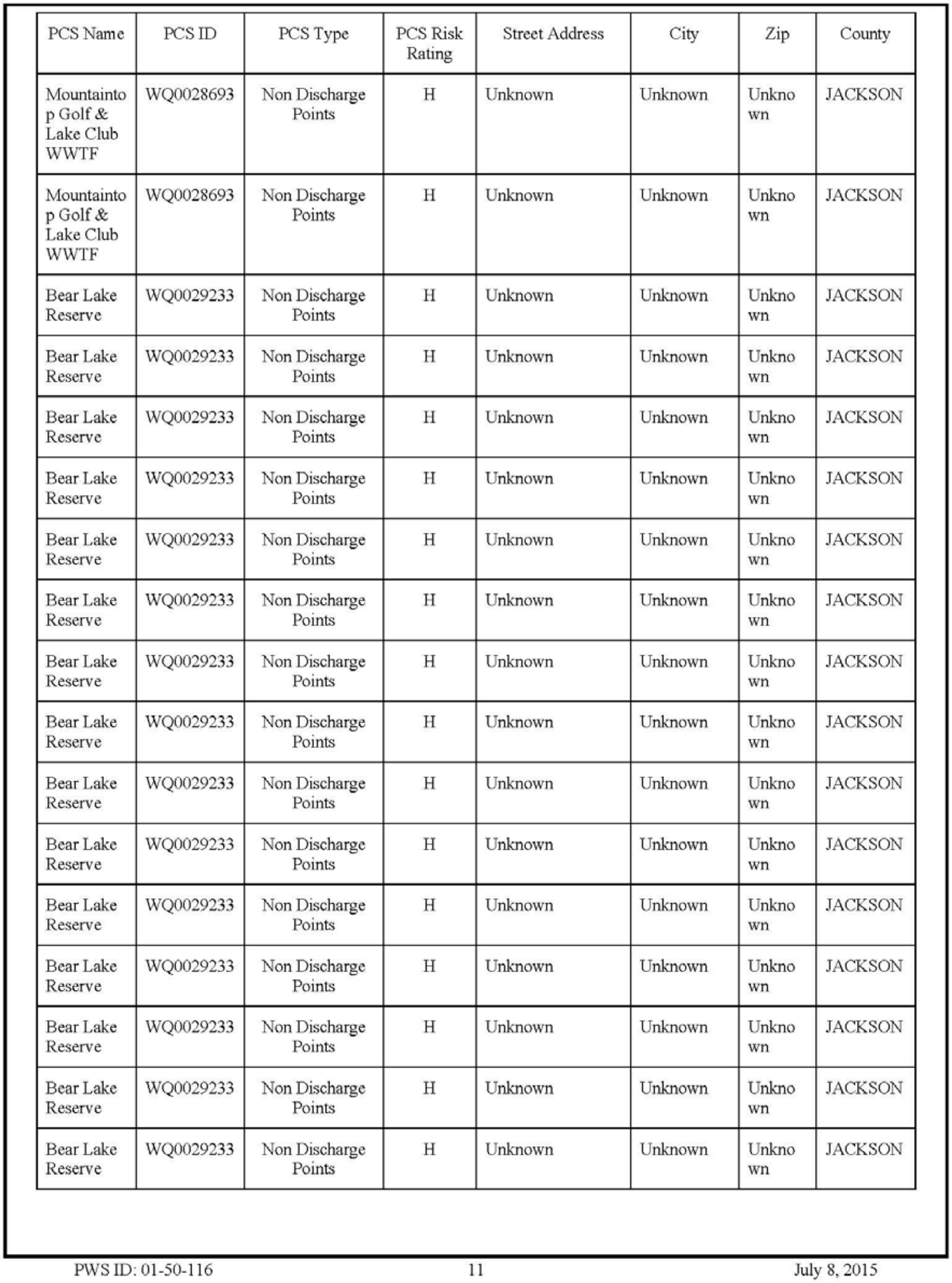
PCS Name	PCS ID	PCS Type	PCS Risk Rating	Street Address	City	Zip	County
Highlands Cove WWTP	WQ0017530	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON

PCS Name	PCS ID	PCS Type	PCS Risk Rating	Street Address	City	Zip	County
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON



PCS Name	PCS ID	PCS Type	PCS Risk Rating	Street Address	City	Zip	County
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON

PWS ID: 01-50-116 10 July 8, 2015



PCS Name	PCS ID	PCS Type	PCS Risk Rating	Street Address	City	Zip	County
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Mountaintop Golf & Lake Club WWTF	WQ0028693	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Bear Lake Reserve	WQ0029233	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Bear Lake Reserve	WQ0029233	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Bear Lake Reserve	WQ0029233	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Bear Lake Reserve	WQ0029233	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Bear Lake Reserve	WQ0029233	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Bear Lake Reserve	WQ0029233	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Bear Lake Reserve	WQ0029233	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Bear Lake Reserve	WQ0029233	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Bear Lake Reserve	WQ0029233	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Bear Lake Reserve	WQ0029233	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Bear Lake Reserve	WQ0029233	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Bear Lake Reserve	WQ0029233	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Bear Lake Reserve	WQ0029233	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Bear Lake Reserve	WQ0029233	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Bear Lake Reserve	WQ0029233	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON

PCS Name	PCS ID	PCS Type	PCS Risk Rating	Street Address	City	Zip	County
Bear Lake Reserve	WQ0029233	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Bear Lake Reserve	WQ0029233	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Bear Lake Reserve	WQ0029233	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Bear Lake Reserve	WQ0029233	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Tuckasegee WWTF	WQ0031427	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Tuckasegee WWTF	WQ0031427	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Tuckasegee WWTF	WQ0031427	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Tuckasegee WWTF	WQ0031427	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Tuckasegee WWTF	WQ0031427	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Tuckasegee WWTF	WQ0031427	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
Tuckasegee WWTF	WQ0031427	Non Discharge Points	H	Unknown	Unknown	Unknown	JACKSON
James and Hazel Boyd 5QM SFR	WI0100074	UIC Permits	H	3471 N Norton Rd	Cashiers	28717	JACKSON
GREAT SMOKY MOUNTAINS RAILROAD	89100	Pollution Incidents	H	119 FRONT STREET	DILLSBORO		JACKS
FORMER KERMIT PHILLIPS PROPERTY	21821	Pollution Incidents	H	13737 NC HIGHWAY 281	TUCKASEGEE		JACKS
MIDDLETON'S BP	28218	Pollution Incidents	H	7032 HIGHWAY 107 SOUTH	CULLOWHEE		JACKS
former tuckasegee mill	28862	Pollution Incidents	H	657 scotts creek road	sylva		JACKS

PWS ID: 01-50-116 12 July 8, 2015

PCS Name	PCS ID	PCS Type	PCS Risk Rating	Street Address	City	Zip	County
Chastain property	41258	Pollution Incidents	H	9824 NC Hwy 107	Tuckasegee		JACKS
TEAGUES SUPERETTE	00-0-0000010241	UST Sites	H	130 SOCO ROAD	MAGGIE VALLEY	Unknown	JACKSON
MOUNTAIN ENERGY 2	00-0-0000031632	UST Sites	H	200 E MAIN STREET	SYLVA	Unknown	JACKSON
SCHOOL BUS GARAGE	00-0-0000009871	UST Sites	H	439 WEBSTER RD HWY 116	SYLVA	Unknown	JACKSON
CANEY FORK GENERAL STORE	00-2-0000001750	UST Sites	H	7032 HWY 107 SOUTH	CULLOWHEE	Unknown	JACKSON
Whiteside Estates WWTP	NC0075736	NPDES Permits	H	NCSR 1143 Norton Rd	Highlands	28741	JACKSON
Blue Ridge School	NC0066958	NPDES Permits	H	NC Hwy 107	Glenville	28736	JACKSON
DD of Cashiers LLC	NCG020799	NPDES Permits	H	5121 Cashiers Rd	Highlands	28741	JACKSON
Trillium Links & Village WWTP	NC0059200	NPDES Permits	H	NCSR 1145	Cashiers	28717	JACKSON
Westview Estates WWTP	NC0087700	NPDES Permits	H	4317 Big Rdg Rd	Cashiers	28717	JACKSON
Cedar Cliff Hydroelectric Station	NCG500125	NPDES Permits	H	1478 Canada Rd	Tuckasegee	28783	JACKSON
Singing Waters Camping Resort	NC0038687	NPDES Permits	H	1006 Trout Creek Rd	Tuckasegee	28783	JACKSON
Tennessee Creek Hydroelectric Station	NCG500123	NPDES Permits	H	NC Hwy 281	Tuckasegee	28783	JACKSON

PCS Name	PCS ID	PCS Type	PCS Risk Rating	Street Address	City	Zip	County
Thorpe Creek Hydroelectric Station	NCG500127	NPDES Permits	H	13201 NC Hwy 107	Tuckasegee	28783	JACKSON
Bear Creek Hydroelectric Plant	NCG500124	NPDES Permits	H	NC Hwy 281	Tuckasegee	28783	JACKSON
Tuskasegee Hydroelectric Station	NCG500126	NPDES Permits	H	NC Hwy 107	Tuckasegee	28783	JACKSON

Appendix 4

Gravity Dam Stability Analysis

Project	Cullowhee Dam	**GRAVITY DAM STABILITY ANALYSIS**	
Client	WCU	By: TEH	Date: 02/28/17
Location	Jackson County, NC	Program Ver: SE 1a	Sheet 1
Structure:	Overflow Section with 1-ft Flashboards	Checked	Chk Date:
Load Condition:	Mean Daily Baseflow = 500 cfs, Normal Section	Job No.	

Schnabel ENGINEERING

Input Parameters

Base Elevation	2052.37	ft
Top Elevation	2060.37	ft
Base Width	8.21	ft
Headwater Elevation	2061	ft
Tailwater Elevation	2054	ft
% Tailwater for Retrogression	100	%
Tailwater for Analysis Purp.	2054.0	ft (calculated)
Sta.of D/S Edge of Crest	6.8	for TW Wt
Station of U/S Edge of Crest	7.0	for HW Wt
Uplift Linear from HW to TW?	Y	(Y/N)
Perform Cracked Sect. Analysis?	Y	(Y/N)
Horz. EQ Accel.		g
Westergaard's Ce (Usually=0.051	0.051	Kip-sec-ft
Ice Load		kips/LF
Depth below HW where Ice acts	0.5	feet
Silt Elevation	2054.4	ft
Silt Density (Submerged)	50	pcf
Silt Pressure Coef. k	0.5	
D/S Backfill Elevation	2052.4	ft
D/S Backfill Density (Submerged)	60	pcf
D/S Backfill Pressure Coef. k	0.5	
Concrete Unit Weight	145	pcf
Key Shear Strength	0	psi
Key Shear Width	0	ft
Base Shear Strength	0	psi
Downstream Sta. of Key	8.21	ft
Passive Force (Rock Wedge, etc	0	Kips/LF
Friction Angle	45	degrees

Stationing of Dam

Station	Elevation
0.0	2052.4
0.8	2053.4
1.5	2054.4
2.3	2055.4
3.0	2056.4
3.8	2057.4
4.6	2058.4
5.2	2059.0
6.8	2059.4
6.8	2060.4
7.0	2060.4
7.0	2059.4
8.2	2059.0
8.2	2052.4

Manually Input Uplift

Station	Elev. Head

Misc Load No. =>	1	2	3	4
Name of Load				
Station				
Elevation				
CCW Angle from Vert				
Magnitude (kips)				
Horiz Force	0.00	0.00	0.00	0.00
Vert Force	0.00	0.00	0.00	0.00

Summation of Forces

Loading	Horizontal Forces Force (Kips)	Horizontal Forces Arm (ft)	Vertical Forces Force (Kips)	Vertical Forces Arm (ft)	Stabilizing Moment (K-ft)	Overturning Moment (K-ft)
Weight of Dam			5.6	5.2	29.1	
Tailwater Weight			0.1	0.4	0.0	
Silt Weight			0.0	8.2	0.0	
Headwater Weight			0.1	7.6	1.0	
Downstream Backfill Weight			0.0	0.0	0.0	
Uplift			-2.6	5.0		13.2
Headwater	2.3	2.8				6.6
Silt	0.1	0.7				0.0
Tailwater	-0.1	0.5			0.0	
Downstream Backfill	0.0	0.0			0.0	
Ice Load	0.0	0.0				0.0
Earthquake Inertia Force	0.0	0.0				0.0
Earthquake Hydrodynamic Force	0.0	0.0				0.0
Sum	**2.3**		**3.2**		**30.2**	**19.9**

Net Moment 10.4

Safety Factor Evaluation

Water Pressure at Toe	0.10	KSF
Water Pressure at Heel	0.54	KSF
Eccentricity	0.83	FT
Pressure at Toe (Incl. Uplift)	0.62	KSF
	4.30	PSI
Pressure at Heel (Incl. Uplift)	0.15	KSF
	1.06	PSI
%Base in Compression 100%	8.21	FT
Sliding Factor of Safety	1.39	Friction Only
Sliding Factor of Safety	1.39	Incl. Shear & Passive Force

Cracked Section Analysis if Tension at Heel (i.e., neg. pres.)

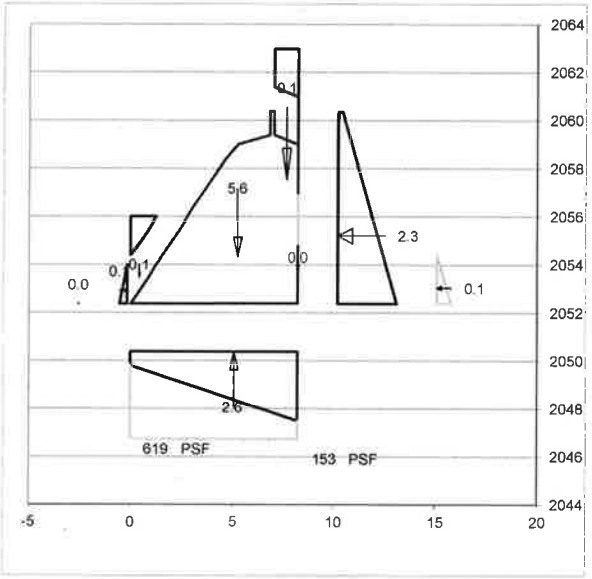
Crack Length		N/A	FT
%Base in Compression	N/A	N/A	FT
Reaction at the Toe		N/A	KSF
		N/A	PSI
Modified Uplift Force		N/A	KIPS
Modified Uplift Arm		N/A	FT
Modified Total Vertical Force		3.17	KIPS
Modified Sliding F.S.		N/A	Friction Only
Modified Sliding F.S.		N/A	Incl. Shear & Passive Force

Compute Shear Strength Needed for Required FS

Req. F.S.:	2	
Revised Shear Width	8.21	ft*

Φ	Req. Residual Shear Strength (psi)	
30	2.3	full base
35	2.0	full base
40	1.6	full base
45	1.2	full base

*Revised Shear Width = (Input Shear Width - Crack Length)



The analysis at left is a sensitivity analysis that may be called for in certain circumstances.

Project	Cullowhee Dam	**GRAVITY DAM STABILITY ANALYSIS**	
Client	WCU	By: TEH	Date: 02/28/17
Location	Jackson County, NC	Program Ver: SE 1a	Sheet 1
Structure:	Overflow Section with 1-ft Flashboards	Checked	Chk Date:
Load Condition:	Average Baseflow = 500 cfs, Undercut Section, No Sediment Load	Job No.	

Schnabel ENGINEERING

Input Parameters

Base Elevation	2052.37	ft
Top Elevation	2060.37	ft
Base Width	8.21	ft
Headwater Elevation	2061	ft
Tailwater Elevation	2054	ft
% Tailwater for Retrogression	100	%
Tailwater for Analysis Purp.	2054.0	ft (calculated)
Sta.of D/S Edge of Crest	6.8	for TW Wt
Station of U/S Edge of Crest	7.0	for HW Wt
Uplift Linear from HW to TW?	N	(Y/N)
Perform Cracked Secl. Analysis?	Y	(Y/N)
Horz. EQ Accel.		g
Westergaard's Ce (Usually=0.051	0.051	Kip-sec-ft
Ice Load		kips/LF
Depth below HW where Ice acts	0.5	feet
Silt Elevation	2052.4	ft
Silt Density (Submerged)	50	pcf
Silt Pressure Coef. k	0.5	
D/S Backfill Elevation	2052.4	ft
D/S Backfill Density (Submerged)	60	pcf
D/S Backfill Pressure Coef. k	0.5	
Concrete Unit Weight	145	pcf
Key Shear Strength	0	psi
Key Shear Width	4.71	ft
Base Shear Strength	0	psi
Downstream Sta. of Key	3.50	ft
Passive Force (Rock Wedge, etc	0	Kips/LF
Friction Angle	30	degrees

Stationing of Dam

Station	Elevation
0.0	2052.4
0.8	2053.4
1.5	2054.4
2.3	2055.4
3.0	2056.4
3.8	2057.4
4.6	2058.4
5.2	2059.0
6.8	2059.4
6.8	2060.4
7.0	2060.4
7.0	2059.4
8.2	2059.0
8.2	2052.4

Manually Input Uplift

Station	Elev. Head
0	2054.0
3.5	2054
8.2	2061

Misc Load No. =>	1	2	3	4
Name of Load				
Station				
Elevation				
CCW Angle from Vert				
Magnitude (kips)				
Horiz Force	0.00	0.00	0.00	0.00
Vert Force	0.00	0.00	0.00	0.00

Summation of Forces

Loading	Horizontal Forces Force (Kips)	Horizontal Forces Arm (ft)	Vertical Forces Force (Kips)	Vertical Forces Arm (ft)	Stabilizing Moment (K-ft)	Overturning Moment (K-ft)
Weight of Dam			5.6	5.2	29.1	
Tailwater Weight			0.1	0.4	0.0	
Silt Weight			-0.4	7.6	-3.1	
Headwater Weight			0.1	7.6	1.0	
Downstream Backfill Weight			0.0	0.0	0.0	
Uplift			-1.9	5.5		10.3
Headwater	2.3	2.8				6.6
Silt	0.0	0.0				0.0
Tailwater	-0.1	0.5			0.0	
Downstream Backfill	0.0	0.0			0.0	
Ice Load	0.0	0.0				0.0
Earthquake Inertia Force	0.0	0.0				0.0
Earthquake Hydrodynamic Force	0.0	0.0				0.0
Sum	**2.2**		**3.5**		**27.2**	**16.8**

Net Moment 10.3

Safety Factor Evaluation

Water Pressure at Toe	0.10	KSF
Water Pressure at Heel	0.54	KSF
Eccentricity	1.18	FT
Pressure at Toe (Incl. Uplift)	0.80	KSF
	5.56	PSI
Pressure at Heel (Incl. Uplift)	0.06	KSF
	0.41	PSI
%Base in Compression 100%	8.21	FT
Sliding Factor of Safety	**0.91**	Friction Only
Sliding Factor of Safety	**0.91**	Incl. Shear & Passive Force

Cracked Section Analysis if Tension at Heel (i.e., neg. pres.)

Crack Length	N/A	FT
%Base in Compression N/A	N/A	FT
Reaction at the Toe	N/A	KSF
	N/A	PSI
Modified Uplift Force	N/A	KIPS
Modified Uplift Arm	N/A	FT
Modified Total Vertical Force	3.53	KIPS
Modified Sliding F.S.	**N/A**	Friction Only
Modified Sliding F.S.	**N/A**	Incl. Shear & Passive Force

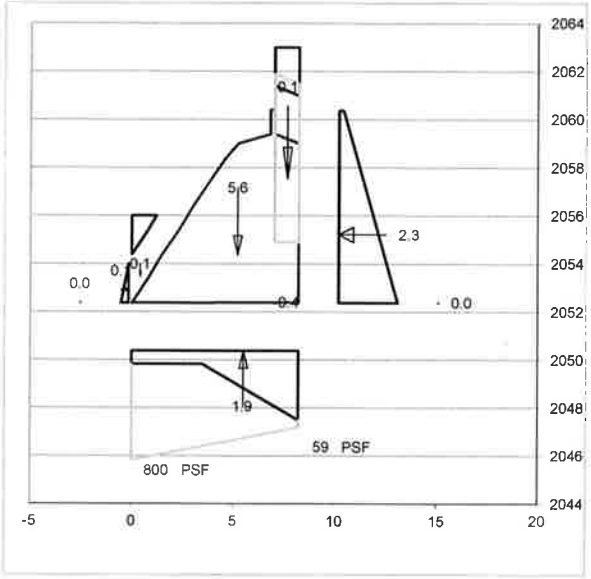
Compute Shear Strength Needed for Required FS

Req. F.S.: 2

Revised Shear Width 8.21 ft*

Φ	Req. Residual Shear Strength (psi)	
30	4.8	full base
35	3.9	full base
40	3.0	full base
45	1.8	full base

*Revised Shear Width = (Input Shear Width - Crack Length)



The analysis at left is a sensitivity analysis that may be called for in certain circumstances.

Project	Cullowhee Dam	**GRAVITY DAM STABILITY ANALYSIS**	
Client	WCU	By: TEH	Date: 02/28/17
Location	Jackson County, NC	Program Ver: SE 1a	Sheet 1
Structure:	Overflow Section with 2-ft Flashboards	Checked	Chk Date:
Load Condition:	Mean Daily Baseflow = 500 cfs, 1940 Section	Job No.	

Schnabel ENGINEERING

Input Parameters

Base Elevation	2052.37	ft
Top Elevation	2060.37	ft
Base Width	8.21	ft
Headwater Elevation	2062	ft
Tailwater Elevation	2054	ft
% Tailwater for Retrogression	100	%
Tailwater for Analysis Purp.	2054.0	ft (calculated)
Sta.of D/S Edge of Crest	6.8	for TW Wt
Station of U/S Edge of Crest	7.0	for HW Wt
Uplift Linear from HW to TW?	Y	(Y/N)
Perform Cracked Sect. Analysis?	Y	(Y/N)
Horz. EQ Accel.		g
Westergaard's Ce (Usually=0.051	0.051	Kip-sec-ft
Ice Load		kips/LF
Depth below HW where Ice acts	0.5	feet
Silt Elevation	2054.4	ft
Silt Density (Submerged)	50	pcf
Silt Pressure Coef. k	0.5	
D/S Backfill Elevation	2052.4	ft
D/S Backfill Density (Submerged)	60	pcf
D/S Backfill Pressure Coef. k	0.5	
Concrete Unit Weight	145	pcf
Key Shear Strength	0	psi
Key Shear Width	0	ft
Base Shear Strength	0	psi
Downstream Sta. of Key	8.21	ft
Passive Force (Rock Wedge, etc	0	Kips/LF
Friction Angle	45	degrees

Stationing of Dam

Station	Elevation
0.0	2052.4
0.8	2053.4
1.5	2054.4
2.3	2055.4
3.0	2056.4
3.8	2057.4
4.6	2058.4
5.2	2059.0
6.8	2059.4
6.8	2061.4
7.0	2061.4
7.0	2059.4
8.2	2059.0
8.2	2052.4

Manually Input Uplift

Station	Elev. Head

Misc Load No. =>	1	2	3	4
Name of Load				
Station				
Elevation				
CCW Angle from Vert				
Magnitude (kips)				
Horiz Force	0.00	0.00	0.00	0.00
Vert Force	0.00	0.00	0.00	0.00

Summation of Forces

Loading	Horizontal Forces Force (Kips)	Horizontal Forces Arm (ft)	Vertical Forces Force (Kips)	Vertical Forces Arm (ft)	Stabilizing Moment (K-ft)	Overturning Moment (K-ft)
Weight of Dam			5.6	5.2	29.3	
Tailwater Weight			0.1	0.4	0.0	
Silt Weight			0.0	8.2	0.0	
Headwater Weight			0.2	7.6	1.6	
Downstream Backfill Weight			0.0	0.0	0.0	
Uplift			-2.9	5.1		14.6
Headwater	2.8	3.1				8.6
Silt	0.1	0.7				0.0
Tailwater	-0.1	0.5			0.0	
Downstream Backfill	0.0	0.0			0.0	
Ice Load	0.0	0.0				0.0
Earthquake Inertia Force	0.0	0.0				0.0
Earthquake Hydrodynamic Force	0.0	0.0				0.0
Sum	**2.8**		**3.0**		**31.0**	**23.3**
					Net Moment	
					7.8	

Safety Factor Evaluation

Water Pressure at Toe	0.10	KSF
Water Pressure at Heel	0.60	KSF
Eccentricity	1.53	FT
Pressure at Toe (Incl. Uplift)	0.78	KSF
	5.41	PSI
Pressure at Heel (Incl. Uplift)	-0.04	KSF
	-0.31	PSI
%Base in Compression	See Below	FT
Sliding Factor of Safety	1.09	Friction Only
Sliding Factor of Safety	1.09	Incl. Shear & Passive Force

Cracked Section Analysis if Tension at Heel (i.e., neg. pres.)

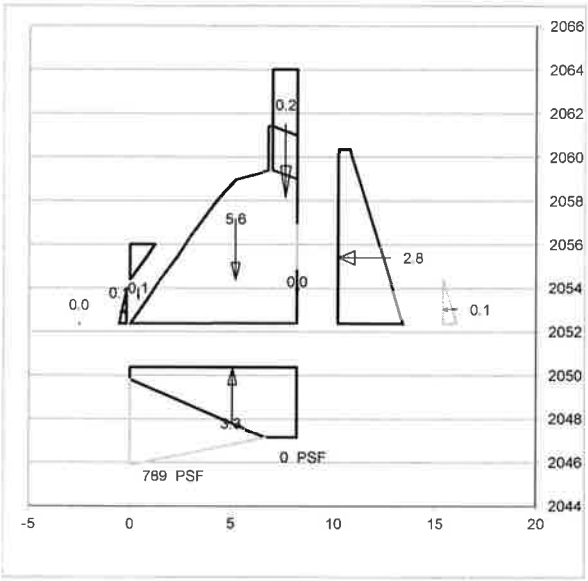
Crack Length		1.54	FT
%Base in Compression	81%	6.67	FT
Reaction at the Toe		0.79	KSF
		5.48	PSI
Modified Uplift Force		3.27	KIPS
Modified Uplift Arm		5.06	FT
Modified Total Vertical Force		2.63	KIPS
Modified Sliding F.S.		0.95	Friction Only
Modified Sliding F.S.		0.95	Incl. Shear & Passive Force

Compute Shear Strength Needed for Required FS

Req. F.S.:	2	
Revised Shear Width	6.67	ft*

Φ	Req. Residual Shear Strength (psi)	
30	4.2	uncracked width
35	3.9	uncracked width
40	3.5	uncracked width
45	3.0	uncracked width

*Revised Shear Width = (Input Shear Width - Crack Length)



The analysis at left is a sensitivity analysis that may be called for in certain circumstances.

Project	Cullowhee Dam	**GRAVITY DAM STABILITY ANALYSIS**	
Client	WCU	By: TEH	Date: 02/28/17
Location	Jackson County, NC	Program Ver: SE 1a	Sheet 1
Structure:	Overflow Section with 1-ft Flashboards	Checked	Chk Date:
Load Condition:	10-Year Flood, Normal Section	Job No.	

Schnabel ENGINEERING

Input Parameters

Base Elevation	2052.37	ft
Top Elevation	2060.37	ft
Base Width	8.21	ft
Headwater Elevation	2067.3	ft
Tailwater Elevation	2064.6	ft
% Tailwater for Retrogression	100	%
Tailwater for Analysis Purp.	2064.6	ft (calculated)
Sta.of D/S Edge of Crest	6.8	for TW Wt
Station of U/S Edge of Crest	7.0	for HW Wt
Uplift Linear from HW to TW?	Y	(Y/N)
Perform Cracked Sect. Analysis?	Y	(Y/N)
Horz. EQ Accel.		g
Westergaard's Ce (Usually=0.051	0.051	Kip-sec-ft
Ice Load		kips/LF
Depth below HW where Ice acts	0.5	feet
Silt Elevation	2054.4	ft
Silt Density (Submerged)	50	pcf
Silt Pressure Coef. k	0.5	
D/S Backfill Elevation	2052.4	ft
D/S Backfill Density (Submerged)	60	pcf
D/S Backfill Pressure Coef. k	0.5	
Concrete Unit Weight	145	pcf
Key Shear Strength	0	psi
Key Shear Width	0	ft
Base Shear Strength	0	psi
Downstream Sta. of Key	8.21	ft
Passive Force (Rock Wedge, etc	0	Kips/LF
Friction Angle	45	degrees

Stationing of Dam

Station	Elevation
0.0	2052.4
0.8	2053.4
1.5	2054.4
2.3	2055.4
3.0	2056.4
3.8	2057.4
4.6	2058.4
5.2	2059.0
6.8	2059.4
6.8	2060.4
7.0	2060.4
7.0	2059.4
8.2	2059.0
8.2	2052.4

Manually Input Uplift

Station	Elev. Head

Misc Load No. =>	1	2	3	4
Name of Load				
Station				
Elevation				
CCW Angle from Vert				
Magnitude (kips)				
Horiz Force	0.00	0.00	0.00	0.00
Vert Force	0.00	0.00	0.00	0.00

Summation of Forces

Loading	Horizontal Forces Force (Kips)	Horizontal Forces Arm (ft)	Vertical Forces Force (Kips)	Vertical Forces Arm (ft)	Stabilizing Moment (K-ft)	Overturning Moment (K-ft)
Weight of Dam			5.6	5.2	29.1	
Tailwater Weight			3.4	2.9	9.7	
Silt Weight			0.0	8.2	0.0	
Headwater Weight			0.6	7.6	4.6	
Downstream Backfill Weight			0.0	0.0	0.0	
Uplift			-7.0	4.2		29.5
Headwater	5.5	3.5				19.2
Silt	0.1	0.7				0.0
Tailwater	-4.1	3.4			13.8	
Downstream Backfill	0.0	0.0			0.0	
Ice Load	0.0	0.0				0.0
Earthquake Inertia Force	0.0	0.0				0.0
Earthquake Hydrodynamic Force	0.0	0.0				0.0
Sum	**1.4**		**2.6**		**57.2**	**48.7**

Net Moment: 8.5

Safety Factor Evaluation

Water Pressure at Toe		0.76	KSF
Water Pressure at Heel		0.93	KSF
Eccentricity		0.88	FT
Pressure at Toe (Incl. Uplift)		0.53	KSF
		3.67	PSI
Pressure at Heel (Incl. Uplift)		0.12	KSF
		0.81	PSI
%Base in Compression	100%	8.21	FT
Sliding Factor of Safety		1.89	Friction Only
Sliding Factor of Safety		1.89	Incl. Shear & Passive Force

Cracked Section Analysis if Tension at Heel (i.e., neg. pres.)

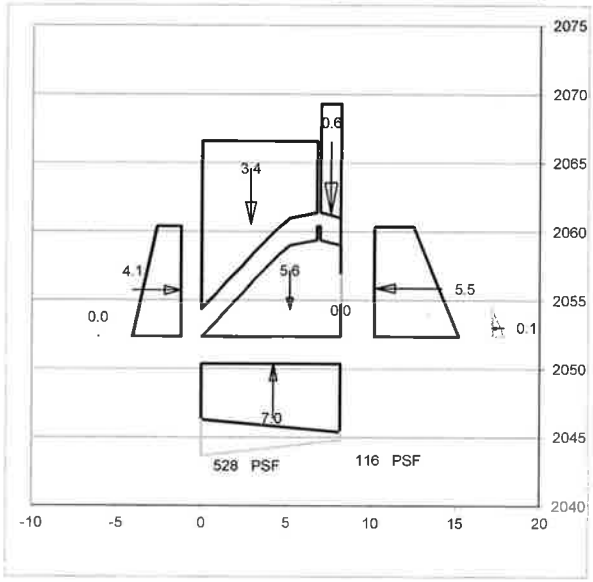
Crack Length		N/A	FT
%Base in Compression	N/A	N/A	FT
Reaction at the Toe		N/A	KSF
		N/A	PSI
Modified Uplift Force		N/A	KIPS
Modified Uplift Arm		N/A	FT
Modified Total Vertical Force		2.64	KIPS
Modified Sliding F.S.		N/A	Friction Only
Modified Sliding F.S.		N/A	Incl. Shear & Passive Force

Compute Shear Strength Needed for Required FS

Req. F.S.:	1.5	
Revised Shear Width	8.21	ft*

Φ	Req. Residual Shear Strength (psi)	
30	0.5	full base
35	0.2	full base
40	0.0	full base
45	0.0	full base

*Revised Shear Width = (Input Shear Width - Crack Length)



The analysis at left is a sensitivity analysis that may be called for in certain circumstances.

Project	Cullowhee Dam	GRAVITY DAM STABILITY ANALYSIS			
Client	WCU	By	Date:	Program Ver:	Sheet
Location	Jackson County, NC	TEH	02/28/17	SE 1a	1
Structure:	Overflow Section with 1-ft Flashboards	Checked	Chk Date:	Job No.	
Load Condition:	500-Year Flood, Normal Section				

Schnabel ENGINEERING

Input Parameters

Base Elevation	2052.37	ft
Top Elevation	2060.37	ft
Base Width	8.21	ft
Headwater Elevation	2080.4	ft
Tailwater Elevation	2080.4	ft
% Tailwater for Retrogression	100	%
Tailwater for Analysis Purp.	2080.4	ft (calculated)
Sta.of D/S Edge of Crest	6.8	for TW Wt
Station of U/S Edge of Crest	7.0	for HW Wt
Uplift Linear from HW to TW?	Y	(Y/N)
Perform Cracked Sect. Analysis?	Y	(Y/N)
Horz. EQ Accel.		g
Westergaard's Ce (Usually=0.051	0.051	Kip-sec-ft
Ice Load		kips/LF
Depth below HW where Ice acts	0.5	feet
Silt Elevation	2054.4	ft
Silt Density (Submerged)	50	pcf
Silt Pressure Coef. k	0.5	
D/S Backfill Elevation	2052.4	ft
D/S Backfill Density (Submerged)	60	pcf
D/S Backfill Pressure Coef. k	0.5	
Concrete Unit Weight	145	pcf
Key Shear Strength	0	psi
Key Shear Width	0	ft
Base Shear Strength	0	psi
Downstream Sta. of Key	8.21	ft
Passive Force (Rock Wedge, etc	0	Kips/LF
Friction Angle	45	degrees

Stationing of Dam

Station	Elevation
0.0	2052.4
0.8	2053.4
1.5	2054.4
2.3	2055.4
3.0	2056.4
3.8	2057.4
4.6	2058.4
5.2	2059.0
6.8	2059.4
6.8	2060.4
7.0	2060.4
7.0	2059.4
8.2	2059.0
8.2	2052.4

Manually Input Uplift

Station	Elev. Head

Misc Load No. =>	1	2	3	4
Name of Load				
Station				
Elevation				
CCW Angle from Vert				
Magnitude (kips)				
Horiz Force	0.00	0.00	0.00	0.00
Vert Force	0.00	0.00	0.00	0.00

Summation of Forces

Loading	Horizontal Forces Force (Kips)	Horizontal Forces Arm (ft)	Vertical Forces Force (Kips)	Vertical Forces Arm (ft)	Stabilizing Moment (K-ft)	Overturning Moment (K-ft)
Weight of Dam			5.6	5.2	29.1	
Tailwater Weight			10.1	3.2	32.5	
Silt Weight			0.0	8.2	0.0	
Headwater Weight			1.6	7.6	12.1	
Downstream Backfill Weight			0.0	0.0	0.0	
Uplift			-14.4	4.1		58.9
Headwater	12.0	3.8				45.3
Silt	0.1	0.7				0.0
Tailwater	-12.0	3.8			45.3	
Downstream Backfill	0.0	0.0			0.0	
Ice Load	0.0	0.0				0.0
Earthquake Inertia Force	0.0	0.0				0.0
Earthquake Hydrodynamic Force	0.0	0.0				0.0
Sum	**0.1**		**2.9**		**119.0**	**104.3**
					Net Moment	
					14.7	

Safety Factor Evaluation

Water Pressure at Toe		1.75	KSF
Water Pressure at Heel		1.75	KSF
Eccentricity		-0.93	FT
Pressure at Toe (Incl. Uplift)		0.11	KSF
		0.78	PSI
Pressure at Heel (Incl. Uplift)		0.60	KSF
		4.17	PSI
%Base in Compression	100%	8.21	FT
Sliding Factor of Safety		**56.82**	Friction Only
Sliding Factor of Safety		**56.82**	Incl. Shear & Passive Force

Cracked Section Analysis if Tension at Heel (i.e., neg. pres.)

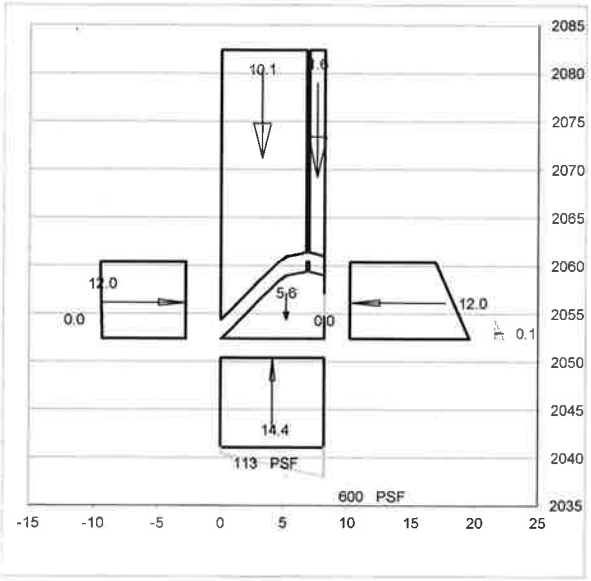
Crack Length		N/A	FT
%Base in Compression	N/A	N/A	FT
Reaction at the Toe		N/A	KSF
		N/A	PSI
Modified Uplift Force		N/A	KIPS
Modified Uplift Arm		N/A	FT
Modified Total Vertical Force		2.93	KIPS
Modified Sliding F.S.		**N/A**	Friction Only
Modified Sliding F.S.		**N/A**	Incl. Shear & Passive Force

Compute Shear Strength Needed for Required FS

Req. F.S.:	1.5	
Revised Shear Width	8.21	ft*

Φ	Req. Residual Shear Strength (psi)	
30	0.0	full base
35	0.0	full base
40	0.0	full base
45	0.0	full base

*Revised Shear Width = (Input Shear Width - Crack Length)



The analysis at left is a sensitivity analysis that may be called for in certain circumstances.

Project	Cullowhee Dam	**GRAVITY DAM STABILITY ANALYSIS**	
Client	WCU	By: TEH	Date: 02/28/17
Location	Jackson County, NC	Program Ver: SE 1a	Sheet 1
Structure:	Overflow Section with 1-ft Flashboards	Checked	Chk Date:
Load Condition:	Mean Daily Baseflow = 500 cfs, Plus 500-Year Earthquake, Normal Section	Job No.	

Schnabel ENGINEERING

Input Parameters

Base Elevation	2052.37	ft
Top Elevation	2060.37	ft
Base Width	8.21	ft
Headwater Elevation	2061	ft
Tailwater Elevation	2054	ft
% Tailwater for Retrogression	100	%
Tailwater for Analysis Purp.	2054.0	ft (calculated)
Sta.of D/S Edge of Crest	6.8	for TW Wt
Station of U/S Edge of Crest	7.0	for HW Wt
Uplift Linear from HW to TW?	Y	(Y/N)
Perform Cracked Sect. Analysis?	Y	(Y/N)
Horz. EQ Accel.	0.05	g
Westergaard's Ce (Usually=0.051	0.051	Kip-sec-ft
Ice Load		kips/LF
Depth below HW where ice acts	0.5	feet
Silt Elevation	2054.4	ft
Silt Density (Submerged)	50	pcf
Silt Pressure Coef. k	0.5	
D/S Backfill Elevation	2052.4	ft
D/S Backfill Density (Submerged)	60	pcf
D/S Backfill Pressure Coef. k	0.5	
Concrete Unit Weight	145	pcf
Key Shear Strength	0	psi
Key Shear Width	0	ft
Base Shear Strength	0	psi
Downstream Sta. of Key	8.21	ft
Passive Force (Rock Wedge, etc	0	Kips/LF
Friction Angle	45	degrees

Stationing of Dam

Station	Elevation
0.0	2052.4
0.8	2053.4
1.5	2054.4
2.3	2055.4
3.0	2056.4
3.8	2057.4
4.6	2058.4
5.2	2059.0
6.8	2059.4
6.8	2060.4
7.0	2060.4
7.0	2059.4
8.2	2059.0
8.2	2052.4

Manually Input Uplift

Station	Elev. Head

Misc Load No. =>	1	2	3	4
Name of Load				
Station				
Elevation				
CCW Angle from Vert				
Magnitude (kips)				
Horiz Force	0.00	0.00	0.00	0.00
Vert Force	0.00	0.00	0.00	0.00

Summation of Forces

Loading	Horizontal Forces Force (Kips)	Horizontal Forces Arm (ft)	Vertical Forces Force (Kips)	Vertical Forces Arm (ft)	Stabilizing Moment (K-ft)	Overturning Moment (K-ft)
Weight of Dam			5.6	5.2	29.1	
Tailwater Weight			0.1	0.4	0.0	
Silt Weight			0.0	8.2	0.0	
Headwater Weight			0.1	7.6	1.0	
Downstream Backfill Weight			0.0	0.0	0.0	
Uplift			-2.6	5.0		13.2
Headwater	2.3	2.8				6.6
Silt	0.1	0.7				0.0
Tailwater	-0.1	0.5			0.0	
Downstream Backfill	0.0	0.0			0.0	
Ice Load	0.0	0.0				0.0
Earthquake Inertia Force	0.3	2.9				0.8
Earthquake Hydrodynamic Force	0.1	3.5				0.4
Sum	**2.7**		**3.2**		**30.2**	**21.1**
					Net Moment	
					9.1	

Safety Factor Evaluation

Water Pressure at Toe		0.10	KSF
Water Pressure at Heel		0.54	KSF
Eccentricity		1.22	FT
Pressure at Toe (Incl. Uplift)		0.73	KSF
		5.07	PSI
Pressure at Heel (Incl. Uplift)		0.04	KSF
		0.29	PSI
%Base in Compression	100%	8.21	FT
Sliding Factor of Safety		1.18	Friction Only
Sliding Factor of Safety		1.18	Incl. Shear & Passive Force

Cracked Section Analysis if Tension at Heel (i.e., neg. pres.)

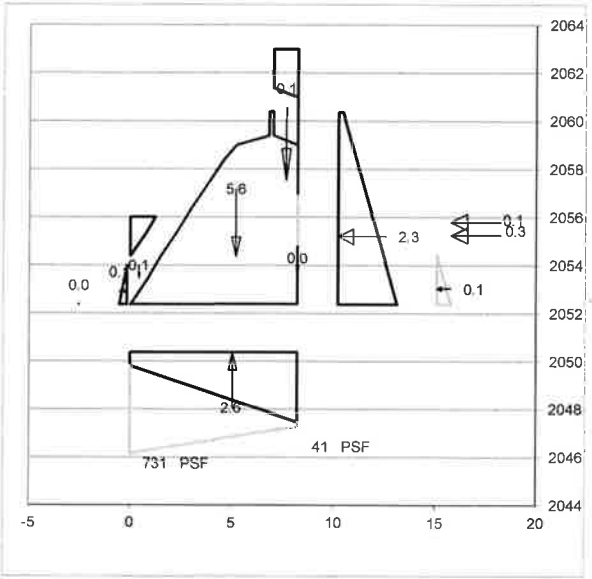
Crack Length		N/A	FT
%Base in Compression	N/A	N/A	FT
Reaction at the Toe		N/A	KSF
		N/A	PSI
Modified Uplift Force		N/A	KIPS
Modified Uplift Arm		N/A	FT
Modified Total Vertical Force		3.17	KIPS
Modified Sliding F.S.		N/A	Friction Only
Modified Sliding F.S.		N/A	Incl. Shear & Passive Force

Compute Shear Strength Needed for Required FS

Req. F.S.:	1.1	
Revised Shear Width	8.21	ft*

Φ	Req. Residual Shear Strength (psi)	
30	1.0	full base
35	0.6	full base
40	0.3	full base
45	0.0	full base

*Revised Shear Width = (Input Shear Width - Crack Length)



The analysis at left is a sensitivity analysis that may be called for in certain circumstances.

Project	Cullowhee Dam	**GRAVITY DAM STABILITY ANALYSIS**	
Client	WCU	By: TEH	Date: 02/28/17
Location	Jackson County, NC	Program Ver: SE 1a	Sheet 1
Structure:	Overflow Section with 1-ft Flashboards	Checked	Chk Date:
Load Condition:	Mean Daily Baseflow = 500 cfs, Plus 1000-Year Earthquake, Normal Section	Job No.	

Schnabel ENGINEERING

Input Parameters

Base Elevation	2052.37	ft
Top Elevation	2060.37	ft
Base Width	8.21	ft
Headwater Elevation	2061	ft
Tailwater Elevation	2054	ft
% Tailwater for Retrogression	100	%
Tailwater for Analysis Purp.	2054.0	ft (calculated)
Sta. of D/S Edge of Crest	6.8	for TW Wt
Station of U/S Edge of Crest	7.0	for HW Wt
Uplift Linear from HW to TW?	Y	(Y/N)
Perform Cracked Sect. Analysis?	Y	(Y/N)
Horz. EQ Accel.	0.1	g
Westergaard's Ce (Usually=0.051	0.051	Kip-sec-ft
Ice Load		kips/LF
Depth below HW where Ice acts	0.5	feet
Silt Elevation	2054.4	ft
Silt Density (Submerged)	50	pcf
Silt Pressure Coef. k	0.5	
D/S Backfill Elevation	2052.4	ft
D/S Backfill Density (Submerged)	60	pcf
D/S Backfill Pressure Coef. k	0.5	
Concrete Unit Weight	145	pcf
Key Shear Strength	0	psi
Key Shear Width	0	ft
Base Shear Strength	0	psi
Downstream Sta. of Key	8.21	ft
Passive Force (Rock Wedge, etc	0	Kips/LF
Friction Angle	45	degrees

Stationing of Dam

Station	Elevation
0.0	2052.4
0.8	2053.4
1.5	2054.4
2.3	2055.4
3.0	2056.4
3.8	2057.4
4.6	2058.4
5.2	2059.0
6.8	2059.4
6.8	2060.4
7.0	2060.4
7.0	2059.4
8.2	2059.0
8.2	2052.4

Manually Input Uplift

Station	Elev. Head

Misc Load No. =>	1	2	3	4
Name of Load				
Station				
Elevation				
CCW Angle from Vert				
Magnitude (kips)				
Horiz Force	0.00	0.00	0.00	0.00
Vert Force	0.00	0.00	0.00	0.00

Summation of Forces

Loading	Horizontal Forces Force (Kips)	Horizontal Forces Arm (ft)	Vertical Forces Force (Kips)	Vertical Forces Arm (ft)	Stabilizing Moment (K-ft)	Overturning Moment (K-ft)
Weight of Dam			5.6	5.2	29.1	
Tailwater Weight			0.1	0.4	0.0	
Silt Weight			0.0	8.2	0.0	
Headwater Weight			0.1	7.6	1.0	
Downstream Backfill Weight			0.0	0.0	0.0	
Uplift			-2.6	5.0		13.2
Headwater	2.3	2.8				6.6
Silt	0.1	0.7				0.0
Tailwater	-0.1	0.5			0.0	
Downstream Backfill	0.0	0.0			0.0	
Ice Load	0.0	0.0				0.0
Earthquake Inertia Force	0.6	2.9				1.6
Earthquake Hydrodynamic Force	0.3	3.5				0.9
Sum	**3.1**		**3.2**		**30.2**	**22.4**
					Net Moment	
					7.9	

Safety Factor Evaluation

Water Pressure at Toe	0.10	KSF
Water Pressure at Heel	0.54	KSF
Eccentricity	1.62	FT
Pressure at Toe (Incl. Uplift)	0.84	KSF
	5.85	PSI
Pressure at Heel (Incl. Uplift)	-0.07	KSF
	-0.49	PSI
%Base in Compression	See Below	FT
Sliding Factor of Safety	1.02	Friction Only
Sliding Factor of Safety	1.02	Incl. Shear & Passive Force

Cracked Section Analysis If Tension at Heel (i.e., neg. pres.)

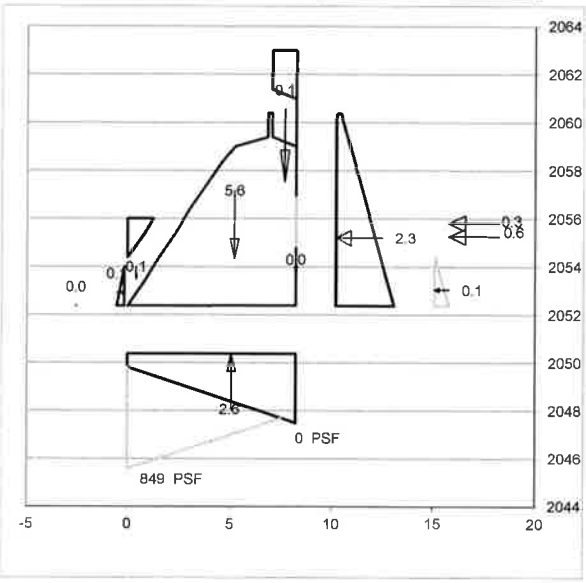
Crack Length		0.75	FT
%Base in Compression	91%	7.46	FT
Reaction at the Toe		0.85	KSF
		5.90	PSI
Modified Uplift Force		N/A	KIPS
Modified Uplift Arm		N/A	FT
Modified Total Vertical Force		3.17	KIPS
Modified Sliding F.S.		1.02	Friction Only
Modified Sliding F.S.		1.02	Incl. Shear & Passive Force

Compute Shear Strength Needed for Required FS

Req. F.S.:	1.1	
Revised Shear Width	7.46	ft*

Φ	Req. Residual Shear Strength (psi)	
30	1.5	uncracked width
35	1.1	uncracked width
40	0.7	uncracked width
45	0.2	uncracked width

*Revised Shear Width = (Input Shear Width - Crack Length)



The analysis at left is a sensitivity analysis that may be called for in certain circumstances.

Project	Cullowhee Dam	**GRAVITY DAM STABILITY ANALYSIS**	
Client	WCU	By: TEH	Date: 02/28/17
Location	Jackson County, NC	Program Ver: SE 1a	Sheet 1
Structure:	Overflow Section with 1-ft Flashboards	Checked	Chk Date:
Load Condition:	Mean Daily Baseflow = 500 cfs, After 1000-Year Earthquake, Normal Section	Job No.	

Schnabel ENGINEERING

Input Parameters

Base Elevation	2052.37	ft
Top Elevation	2060.37	ft
Base Width	8.21	ft
Headwater Elevation	2061	ft
Tailwater Elevation	2054	ft
% Tailwater for Retrogression	100	%
Tailwater for Analysis Purp.	2054.0	ft (calculated)
Sta. of D/S Edge of Crest	6.8	for TW Wt
Station of U/S Edge of Crest	7.0	for HW Wt
Uplift Linear from HW to TW?	N	(Y/N)
Perform Cracked Sect. Analysis?	Y	(Y/N)
Horz. EQ Accel.	0.1	g
Westergaard's Ce (Usually=0.051	0.051	Kip-sec-ft
Ice Load		kips/LF
Depth below HW where ice acts	0.5	feet
Silt Elevation	2054.4	ft
Silt Density (Submerged)	50	pcf
Silt Pressure Coef. k	0.5	
D/S Backfill Elevation	2052.4	ft
D/S Backfill Density (Submerged)	60	pcf
D/S Backfill Pressure Coef. k	0.5	
Concrete Unit Weight	145	pcf
Key Shear Strength	0	psi
Key Shear Width	0	ft
Base Shear Strength	0	psi
Downstream Sta. of Key	8.21	ft
Passive Force (Rock Wedge, etc)	0	Kips/LF
Friction Angle	45	degrees

Stationing of Dam

Station	Elevation
0.0	2052.4
0.8	2053.4
1.5	2054.4
2.3	2055.4
3.0	2056.4
3.8	2057.4
4.6	2058.4
5.2	2059.0
6.8	2059.4
6.8	2060.4
7.0	2060.4
7.0	2059.4
8.2	2059.0
8.2	2052.4

Manually Input Uplift

Station	Elev. Head
0	2054.0
7.46	2061
8.2	2061

Misc Load No. =>	1	2	3	4
Name of Load				
Station				
Elevation				
CCW Angle from Vert				
Magnitude (kips)				
Horiz Force	0.00	0.00	0.00	0.00
Vert Force	0.00	0.00	0.00	0.00

Summation of Forces

Loading	Horizontal Forces Force (Kips)	Horizontal Forces Arm (ft)	Vertical Forces Force (Kips)	Vertical Forces Arm (ft)	Stabilizing Moment (K-ft)	Overturning Moment (K-ft)
Weight of Dam			5.6	5.2	29.1	
Tailwater Weight			0.1	0.4	0.0	
Silt Weight			0.0	8.2	0.0	
Headwater Weight			0.1	7.6	1.0	
Downstream Backfill Weight			0.0	0.0	0.0	
Uplift			-2.8	5.0		14.1
Headwater	2.3	2.8				6.6
Silt	0.1	0.7				0.0
Tailwater	-0.1	0.5			0.0	
Downstream Backfill	0.0	0.0			0.0	
Ice Load	0.0	0.0				0.0
Earthquake Inertia Force	0.6	2.9				1.6
Earthquake Hydrodynamic Force	0.3	3.5				0.9
Sum	**3.1**		**3.0**		**30.2**	**23.2**
					Net Moment	
					7.0	

Safety Factor Evaluation

Water Pressure at Toe	0.10	KSF
Water Pressure at Heel	0.54	KSF
Eccentricity	1.77	FT
Pressure at Toe (Incl. Uplift)	0.84	KSF
	5.82	PSI
Pressure at Heel (Incl. Uplift)	-0.11	KSF
	-0.74	PSI
%Base in Compression	See Below	FT
Sliding Factor of Safety	0.97	Friction Only
Sliding Factor of Safety	0.97	Incl. Shear & Passive Force

Cracked Section Analysis if Tension at Heel (i.e., neg. pres.)

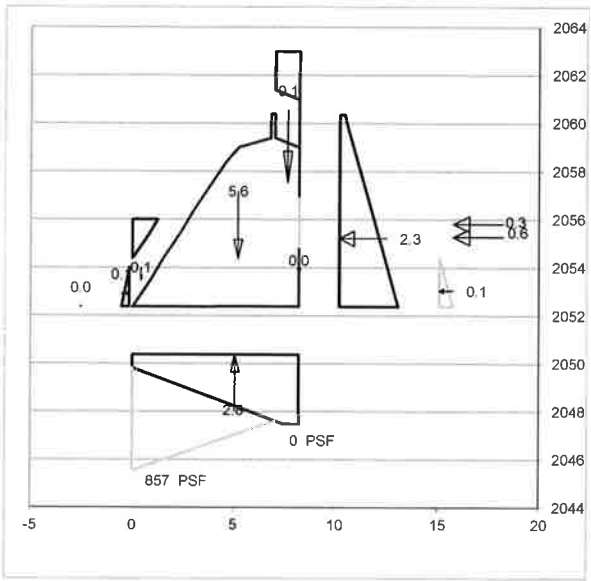
Crack Length		1.19	FT
%Base in Compression	85%	7.02	FT
Reaction at the Toe		0.86	KSF
		5.95	PSI
Modified Uplift Force		N/A	KIPS
Modified Uplift Arm		N/A	FT
Modified Total Vertical Force		3.01	KIPS
Modified Sliding F.S.		0.97	Friction Only
Modified Sliding F.S.		0.97	Incl. Shear & Passive Force

Compute Shear Strength Needed for Required FS

Req. F.S.:	1.1	
Revised Shear Width	7.02	ft*

Φ	Req. Residual Shear Strength (psi)	
30	1.7	uncracked width
35	1.3	uncracked width
40	0.9	uncracked width
45	0.4	uncracked width

*Revised Shear Width = (Input Shear Width - Crack Length)



The analysis at left is a sensitivity analysis that may be called for in certain circumstances.

Project	Cullowhee Dam	**GRAVITY DAM STABILITY ANALYSIS**	
Client	WCU	By: TEH	Date: 02/28/17
Location	Jackson County, NC	Program Ver: SE 1a	Sheet 1
Structure:	New 1-Foot Crest Overlay	Checked	Chk Date:
Load Condition:	Mean Daily Baseflow = 500 cfs, Modified Section	Job No.	

Schnabel ENGINEERING

Input Parameters

Base Elevation	2052.37	ft
Top Elevation	2060.37	ft
Base Width	9.01	ft
Headwater Elevation	2061	ft
Tailwater Elevation	2054	ft
% Tailwater for Retrogression	100	%
Tailwater for Analysis Purp.	2054.0	ft (calculated)
Sta of D/S Edge of Crest	6.8	for TW Wt
Station of U/S Edge of Crest	7.6	for HW Wt
Uplift Linear from HW to TW?	Y	(Y/N)
Perform Cracked Sect. Analysis?	Y	(Y/N)
Horz. EQ Accel.		g
Westergaard's Ce (Usually=0.051	0.051	Kip-sec-ft
Ice Load		kips/LF
Depth below HW where Ice acts	0.5	feet
Silt Elevation	2054.4	ft
Silt Density (Submerged)	50	pcf
Silt Pressure Coef. k	0.5	
D/S Backfill Elevation	2052.4	ft
D/S Backfill Density (Submerged)	60	pcf
D/S Backfill Pressure Coef. k	0.5	
Concrete Unit Weight	145	pcf
Key Shear Strength	0	psi
Key Shear Width	0	ft
Base Shear Strength	0	psi
Downstream Sta. of Key	9.01	ft
Passive Force (Rock Wedge, etc	0	Kips/LF
Friction Angle	45	degrees

Stationing of Dam

Station	Elevation
0.0	2052.4
0.8	2053.4
1.5	2054.4
2.3	2055.4
3.0	2056.4
3.8	2057.4
4.6	2058.4
5.4	2059.4
6.0	2060.0
7.6	2060.4
9.0	2060.0
9.0	2052.4

Manually Input Uplift

Station	Elev. Head

Misc Load No. =>	1	2	3	4
Name of Load				
Station				
Elevation				
CCW Angle from Vert				
Magnitude (kips)				
Horiz Force	0.00	0.00	0.00	0.00
Vert Force	0.00	0.00	0.00	0.00

Summation of Forces

Loading	Horizontal Forces Force (Kips)	Horizontal Forces Arm (ft)	Vertical Forces Force (Kips)	Vertical Forces Arm (ft)	Stabilizing Moment (K-ft)	Overturning Moment (K-ft)
Weight of Dam			6.8	5.7	39.3	
Tailwater Weight			0.1	0.4	0.0	
Silt Weight			0.0	9.0	0.0	
Headwater Weight			0.0	8.6	0.4	
Downstream Backfill Weight			0.0	0.0	0.0	
Uplift			-2.9	5.5		15.9
Headwater	2.3	2.8				6.6
Silt	0.1	0.7				0.0
Tailwater	-0.1	0.5			0.0	
Downstream Backfill	0.0	0.0			0.0	
Ice Load	0.0	0.0				0.0
Earthquake Inertia Force	0.0	0.0				0.0
Earthquake Hydrodynamic Force	0.0	0.0				0.0
Sum	**2.3**		**4.1**		**39.7**	**22.6**

Net Moment: 17.2

Safety Factor Evaluation

Water Pressure at Toe		0.10	KSF
Water Pressure at Heel		0.54	KSF
Eccentricity		0.28	FT
Pressure at Toe (Incl. Uplift)		0.54	KSF
		3.73	PSI
Pressure at Heel (Incl. Uplift)		0.37	KSF
		2.54	PSI
%Base in Compression	100%	9.01	FT
Sliding Factor of Safety		1.78	Friction Only
Sliding Factor of Safety		1.78	Incl. Shear & Passive Force

Cracked Section Analysis if Tension at Heel (i.e., neg. pres.)

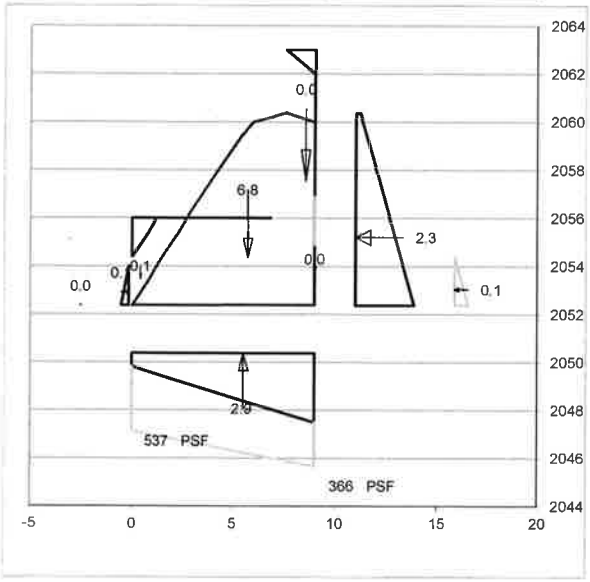
Crack Length		N/A	FT
%Base in Compression	N/A	N/A	FT
Reaction at the Toe		N/A	KSF
		N/A	PSI
Modified Uplift Force		N/A	KIPS
Modified Uplift Arm		N/A	FT
Modified Total Vertical Force		4.07	KIPS
Modified Sliding F.S.		N/A	Friction Only
Modified Sliding F.S.		N/A	Incl. Shear & Passive Force

Compute Shear Strength Needed for Required FS

Req. F.S.:	1.1	
Revised Shear Width	9.01	ft*

Φ	Req. Residual Shear Strength (psi)	
30	0.1	full base
35	0.0	full base
40	0.0	full base
45	0.0	full base

*Revised Shear Width = (Input Shear Width - Crack Length)



The analysis at left is a sensitivity analysis that may be called for in certain circumstances.

Project	Cullowhee Dam	**GRAVITY DAM STABILITY ANALYSIS**	
Client	WCU	By: TEH	Date: 02/28/17
Location	Jackson County, NC	Program Ver: SE 1a	Sheet 1
Structure:	New 1-Foot Crest Overlay	Checked	Chk Date:
Load Condition:	Mean Daily Baseflow = 500 cfs, Plus 1000-Year Earthquake, Modified Section	Job No.	

Schnabel ENGINEERING

Input Parameters

Base Elevation	2052.37	ft
Top Elevation	2060.37	ft
Base Width	9.01	ft
Headwater Elevation	2061	ft
Tailwater Elevation	2054	ft
% Tailwater for Retrogression	100	%
Tailwater for Analysis Purp.	2054.0	ft (calculated)
Sta. of D/S Edge of Crest	6.8	for TW Wt
Station of U/S Edge of Crest	7.6	for HW Wt
Uplift Linear from HW to TW?	Y	(Y/N)
Perform Cracked Sect. Analysis?	Y	(Y/N)
Horz. EQ Accel.	0.1	g
Westergaard's Ce (Usually=0.051	0.051	Kip-sec-ft
Ice Load		kips/LF
Depth below HW where Ice acts	0.5	feet
Silt Elevation	2054.4	ft
Silt Density (Submerged)	50	pcf
Silt Pressure Coef. k	0.5	
D/S Backfill Elevation	2052.4	ft
D/S Backfill Density (Submerged)	60	pcf
D/S Backfill Pressure Coef. k	0.5	
Concrete Unit Weight	145	pcf
Key Shear Strength	0	psi
Key Shear Width	0	ft
Base Shear Strength	0	psi
Downstream Sta. of Key	9.01	ft
Passive Force (Rock Wedge, etc.	0	Kips/LF
Friction Angle	45	degrees

Stationing of Dam

Station	Elevation
0.0	2052.4
0.8	2053.4
1.5	2054.4
2.3	2055.4
3.0	2056.4
3.8	2057.4
4.6	2058.4
5.4	2059.4
6.0	2060.0
7.6	2060.4
9.0	2060.0
9.0	2052.4

Manually Input Uplift

Station	Elev. Head

Misc Load No. =>	1	2	3	4
Name of Load				
Station				
Elevation				
CCW Angle from Vert				
Magnitude (kips)				
Horiz Force	0.00	0.00	0.00	0.00
Vert Force	0.00	0.00	0.00	0.00

Summation of Forces

Loading	Horizontal Forces Force (Kips)	Horizontal Forces Arm (ft)	Vertical Forces Force (Kips)	Vertical Forces Arm (ft)	Stabilizing Moment (K-ft)	Overturning Moment (K-ft)
Weight of Dam			6.8	5.7	39.3	
Tailwater Weight			0.1	0.4	0.0	
Silt Weight			0.0	9.0	0.0	
Headwater Weight			0.0	8.6	0.4	
Downstream Backfill Weight			0.0	0.0	0.0	
Uplift			-2.9	5.5		15.9
Headwater	2.3	2.8				6.6
Silt	0.1	0.7				0.0
Tailwater	-0.1	0.5			0.0	
Downstream Backfill	0.0	0.0			0.0	
Ice Load	0.0	0.0				0.0
Earthquake Inertia Force	0.7	3.3				2.2
Earthquake Hydrodynamic Force	0.3	3.5				0.9
Sum	**3.2**		**4.1**		**39.7**	**25.7**

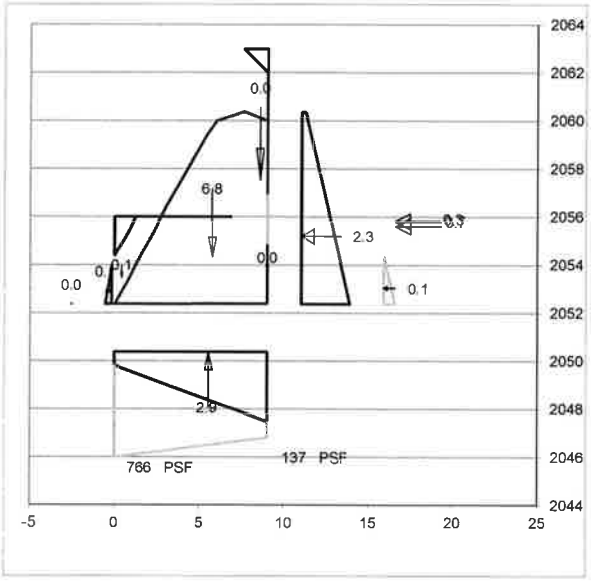
Net Moment: 14.1

Safety Factor Evaluation

Water Pressure at Toe	0.10	KSF
Water Pressure at Heel	0.54	KSF
Eccentricity	1.05	FT
Pressure at Toe (Incl. Uplift)	0.77	KSF
	5.32	PSI
Pressure at Heel (Incl. Uplift)	0.14	KSF
	0.95	PSI
%Base in Compression 100%	9.01	FT
Sliding Factor of Safety	1.26	Friction Only
Sliding Factor of Safety	1.26	Incl. Shear & Passive Force

Cracked Section Analysis if Tension at Heel (i.e., neg. pres.)

Crack Length	N/A	FT
%Base in Compression N/A	N/A	FT
Reaction at the Toe	N/A	KSF
	N/A	PSI
Modified Uplift Force	N/A	KIPS
Modified Uplift Arm	N/A	FT
Modified Total Vertical Force	4.07	KIPS
Modified Sliding F.S.	N/A	Friction Only
Modified Sliding F.S.	N/A	Incl. Shear & Passive Force



Compute Shear Strength Needed for Required FS

Req. F.S.:	1.1	
Revised Shear Width	9.01	ft*

Φ	Req. Residual Shear Strength (psi)	
30	0.9	full base
35	0.5	full base
40	0.1	full base
45	0.0	full base

*Revised Shear Width = (Input Shear Width - Crack Length)

The analysis at left is a sensitivity analysis that may be called for in certain circumstances.

Appendix 5

Raw Water Demand Projections

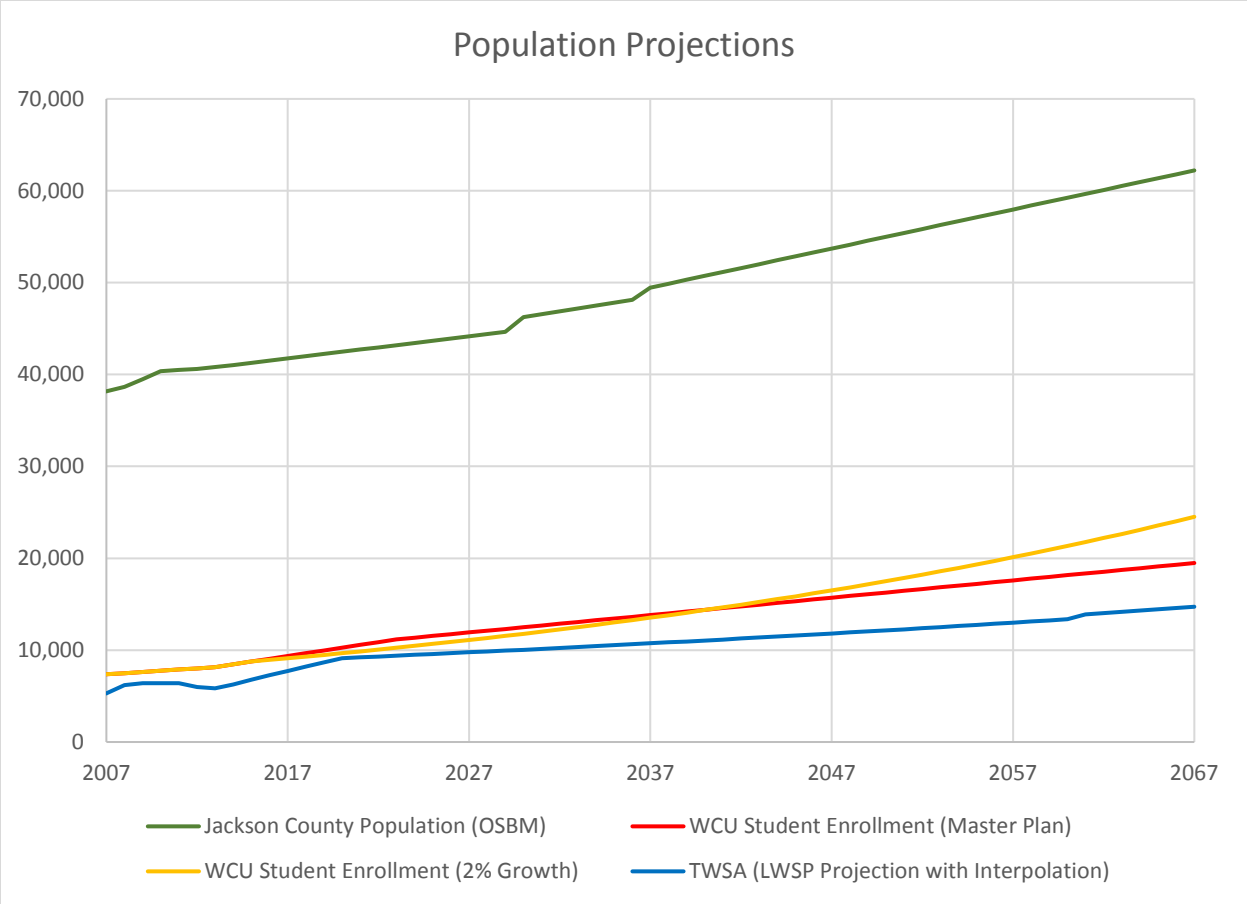
List of Tables

List of Figures

Table A - 1 - Population Projections

Year	Jackson County Population (OSBM Estimate)	WCU Enrollment Projections (2014 Master Plan)	WCU 2% Growth Projection	WCU LWSP Service Population	TWSA LWSP Service Population
1991	**27,622**				
1997	**31,221**				**4,950**
1998	**31,848**				*5,160*
1999	**32,492**				*5,370*
2000	**33,275**	**6,444**	**6,444**		*5,580*
2001	**33,892**	*6,575*	**6,575**		*5,790*
2002	**34,570**	*6,706*	**6,706**	**8,065**	**6,000**
2003	**35,594**	*6,837*	**6,837**		*5,856*
2004	**36,381**	*6,968*	**6,968**		*5,713*
2005	**36,751**	*7,099*	**7,099**		*5,569*
2006	**37,849**	*7,230*	**7,230**		*5,426*
2007	**38,181**	*7,362*	**7,362**	**8,500**	**5,282**
2008	**38,659**	*7,493*	**7,493**	**8,500**	**6,200**
2009	**39,483**	*7,624*	**7,624**	**8,500**	**6,400**
2010	**40,351**	*7,755*	**7,755**	**8,500**	**6,400**
2011	**40,504**	*7,886*	**7,886**	**8,500**	**6,400**
2012	**40,622**	*8,017*	**8,017**	**8,400**	**6,000**
2013	**40,809**	**8,148**	**8,148**	**8,500**	**5,850**
2014	**41,039**	*8,450*	**8,450**	**8,800**	**6,262**
2015	**41,279**	*8,753*	**8,753**	**8,900**	**6,800**
2016	**41,516**	*9,055*	*8,928*		*7,265*
2017	**41,756**	*9,357*	*9,107*		*7,730*
2018	**41,998**	*9,660*	*9,289*		*8,196*
2019	**42,237**	*9,962*	*9,475*		*8,661*
2020	**42,477**	*10,264*	*9,665*		*9,126*
2021	**42,716**	*10,566*	*9,858*		*9,217*
2022	**42,956**	*10,869*	*10,055*		*9,309*
2023	**43,196**	**11,171**	*10,256*		*9,400*
2024	**43,436**	*11,360*	*10,461*		*9,491*
2025	**43,674**	*11,549*	*10,670*		*9,583*
2026	**43,917**	*11,738*	*10,883*		*9,674*
2027	**44,156**	*11,927*	*11,101*		*9,765*
2028	**44,395**	*12,115*	*11,323*		*9,856*
2029	**44,635**	*12,304*	*11,549*		*9,948*
2030	**46,259**	*12,493*	*11,780*		*10,039*
2031	**46,569**	*12,682*	*12,016*		*10,139*
2032	**46,879**	*12,871*	*12,256*		*10,240*

Year	Jackson County Population (OSBM Estimate)	WCU Enrollment Projections (2014 Master Plan)	WCU 2% Growth Projection	WCU LWSP Service Population	TWSA LWSP Service Population
2033	**47,187**	*13,060*	*12,501*		*10,340*
2034	**47,499**	*13,249*	*12,751*		*10,441*
2035	**47,810**	*13,438*	*13,006*		*10,541*
2036	**48,119**	*13,627*	*13,266*		*10,641*
2037	*49,458*	*13,815*	*13,531*		*10,742*
2038	*49,884*	*14,004*	*13,802*		*10,842*
2039	*50,309*	*14,193*	*14,078*		*10,943*
2040	*50,734*	*14,382*	*14,360*		*11,043*
2041	*51,159*	*14,571*	*14,647*		*11,153*
2042	*51,585*	*14,760*	*14,940*		*11,264*
2043	*52,010*	*14,949*	*15,239*		*11,374*
2044	*52,435*	*15,138*	*15,544*		*11,485*
2045	*52,860*	*15,327*	*15,855*		*11,595*
2046	*53,285*	*15,515*	*16,172*		*11,705*
2047	*53,711*	*15,704*	*16,495*		*11,816*
2048	*54,136*	*15,893*	*16,825*		*11,926*
2049	*54,561*	*16,082*	*17,162*		*12,037*
2050	*54,986*	*16,271*	*17,505*		*12,147*
2051	*55,412*	*16,460*	*17,855*		*12,269*
2052	*55,837*	*16,649*	*18,212*		*12,390*
2053	*56,262*	*16,838*	*18,576*		*12,512*
2054	*56,687*	*17,026*	*18,948*		*12,633*
2055	*57,112*	*17,215*	*19,327*		*12,755*
2056	*57,538*	*17,404*	*19,714*		*12,876*
2057	*57,963*	*17,593*	*20,108*		*12,998*
2058	*58,388*	*17,782*	*20,510*		*13,119*
2059	*58,813*	*17,971*	*20,920*		*13,241*
2060	*59,239*	*18,160*	*21,338*		*13,362*
2061	*59,664*	*18,349*	*21,765*		*13,885*
2062	*60,089*	*18,538*	*22,200*		*14,025*
2063	*60,514*	*18,726*	*22,644*		*14,164*
2064	*60,939*	*18,915*	*23,097*		*14,303*
2065	*61,365*	*19,104*	*23,559*		*14,443*
2066	*61,790*	*19,293*	*24,030*		*14,582*
2067	*62,215*	*19,482*	*24,511*		*14,721*



Graph A - 1 - Population Projections

Table A - 2 - WCU Existing Water Demands

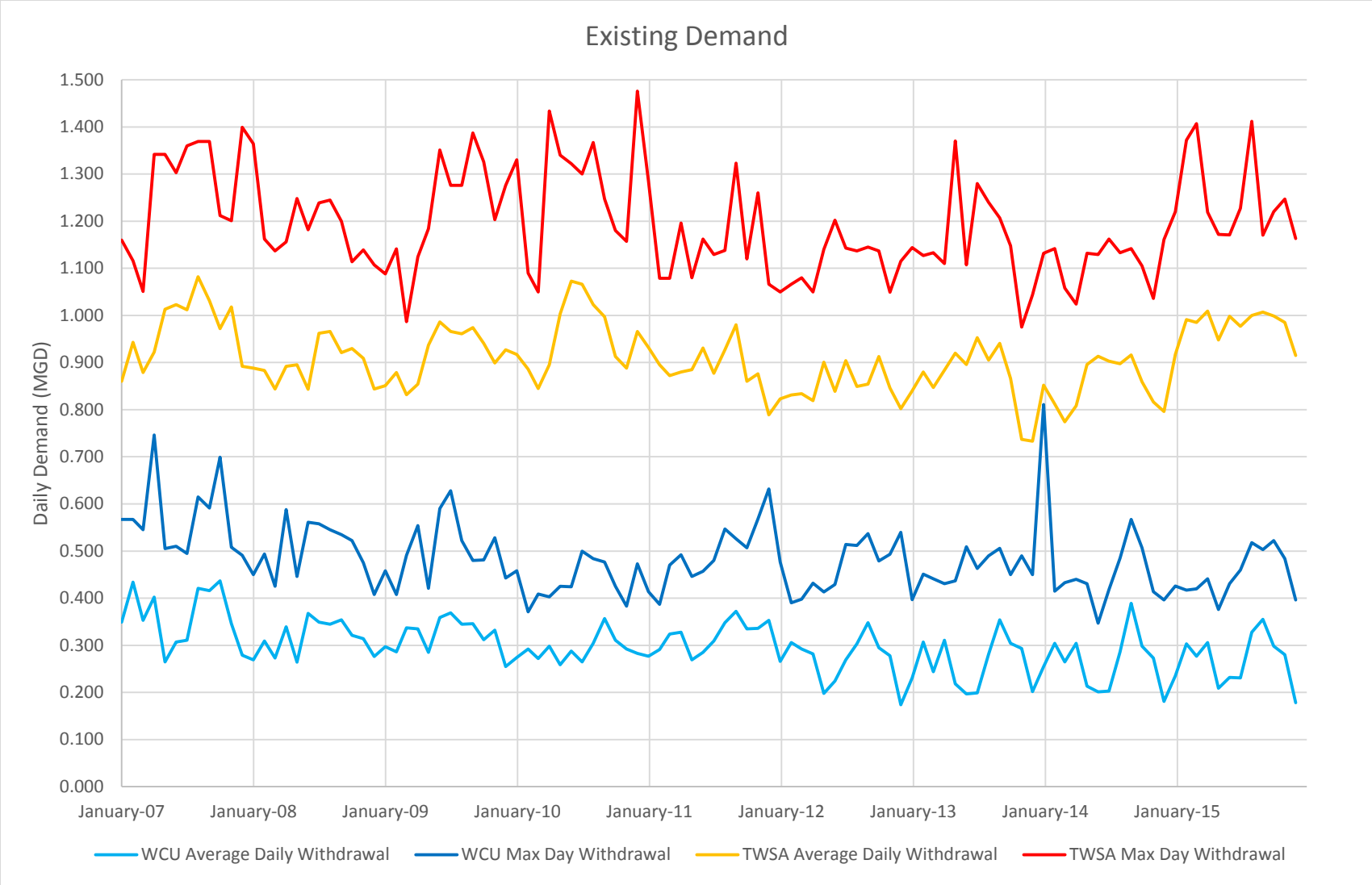
Year	WCU Withdrawals (MGD)		WCU Student Enrollment	avg. per capita (gpcd)	max. per capita (gpcd)	WCU Service Population	avg. per capita (gpcd)	max. per capita (gpcd)
	Average Daily	Max Day						
1997								
2002	0.376	0.726	6,706	56.0	108.3	8,065	46.6	90.0
2007	0.360	0.746	7,362	48.9	101.3	8,500	42.4	87.8
2008	0.315	0.588	7,493	42.1	78.5	8,500	37.1	69.2
2009	0.322	0.628	7,624	42.2	82.4	8,500	37.8	73.9
2010	0.291	0.500	7,755	37.6	64.5	8,500	34.3	58.8
2011	0.319	0.632	7,886	40.4	80.1	8,500	37.5	74.4
2012	0.270	0.540	8,017	33.6	67.4	8,400	32.1	64.3
2013	0.262	0.509	8,148	32.1	62.5	8,500	30.8	59.9
2014	0.264	0.811	8,450	31.3	96.0	8,800	30.0	92.2
2015	0.269	0.522	8,753	30.8	59.6	8,900	30.3	58.7
Average				39.5	80.1		35.9	72.9
2007-2015 Increase	-14.5%	-11.2%	16.8%	-26.8%	-24.0%	4.7%	-18.4%	-15.2%
Increase per Year	-1.8%	-1.4%	2.1%	-3.4%	-3.0%	0.6%	-2.3%	-1.9%

Table A - 3 - TWSA Existing Water Demands

Year	TWSA Withdrawals (MGD) Average Daily	TWSA Withdrawals (MGD) Max Day	TWSA Service Population	avg. per capita (gpcd)	max. per capita (gpcd)
1997	0.810	1.089	4,950	163.7	220.0
2002	0.914	1.350	6,000	152.3	225.0
2007	0.971	1.399	5,282	183.8	264.9
2008	0.898	1.364	6,200	144.9	220.0
2009	0.917	1.387	6,400	143.3	216.7
2010	0.956	1.476	6,400	149.4	230.6
2011	0.892	1.323	6,400	139.4	206.7
2012	0.851	1.202	6,000	141.9	200.3
2013	0.867	1.370	5,850	148.2	234.2
2014	0.854	1.162	6,262	136.3	185.6
2015	0.978	1.412	6,800	143.8	207.6
Average				148.3	219.2
2007-2015 Increase	8.9%	3.5%	9.7%	-0.8%	-5.6%
Increase per Year	1.1%	0.4%	1.2%	-0.1%	-0.7%

Table A - 4 - Combined Existing Water Demands

Year	Total Withdrawals (MGD) Average Daily	Total Withdrawals (MGD) Max Day	Total Service Population	avg. per capita (gpcd)	max. per capita (gpcd)
1997	0.810	1.089	4,950	163.7	220.0
2002	1.289	2.076	14,065	91.7	147.6
2007	1.331	2.145	13,782	96.5	155.6
2008	1.213	1.952	14,700	82.5	132.8
2009	1.239	2.015	14,900	83.1	135.2
2010	1.247	1.976	14,900	83.7	132.6
2011	1.211	1.955	14,900	81.3	131.2
2012	1.121	1.742	14,400	77.8	121.0
2013	1.129	1.879	14,350	78.6	130.9
2014	1.118	1.973	15,062	74.2	131.0
2015	1.247	1.934	15,700	79.4	123.2
Average				82.9	134.1
2007-2015 Increase	2.8%	-0.9%	6.8%	-3.8%	-7.2%
Increase per Year	0.3%	-0.1%	0.9%	-0.5%	-0.9%

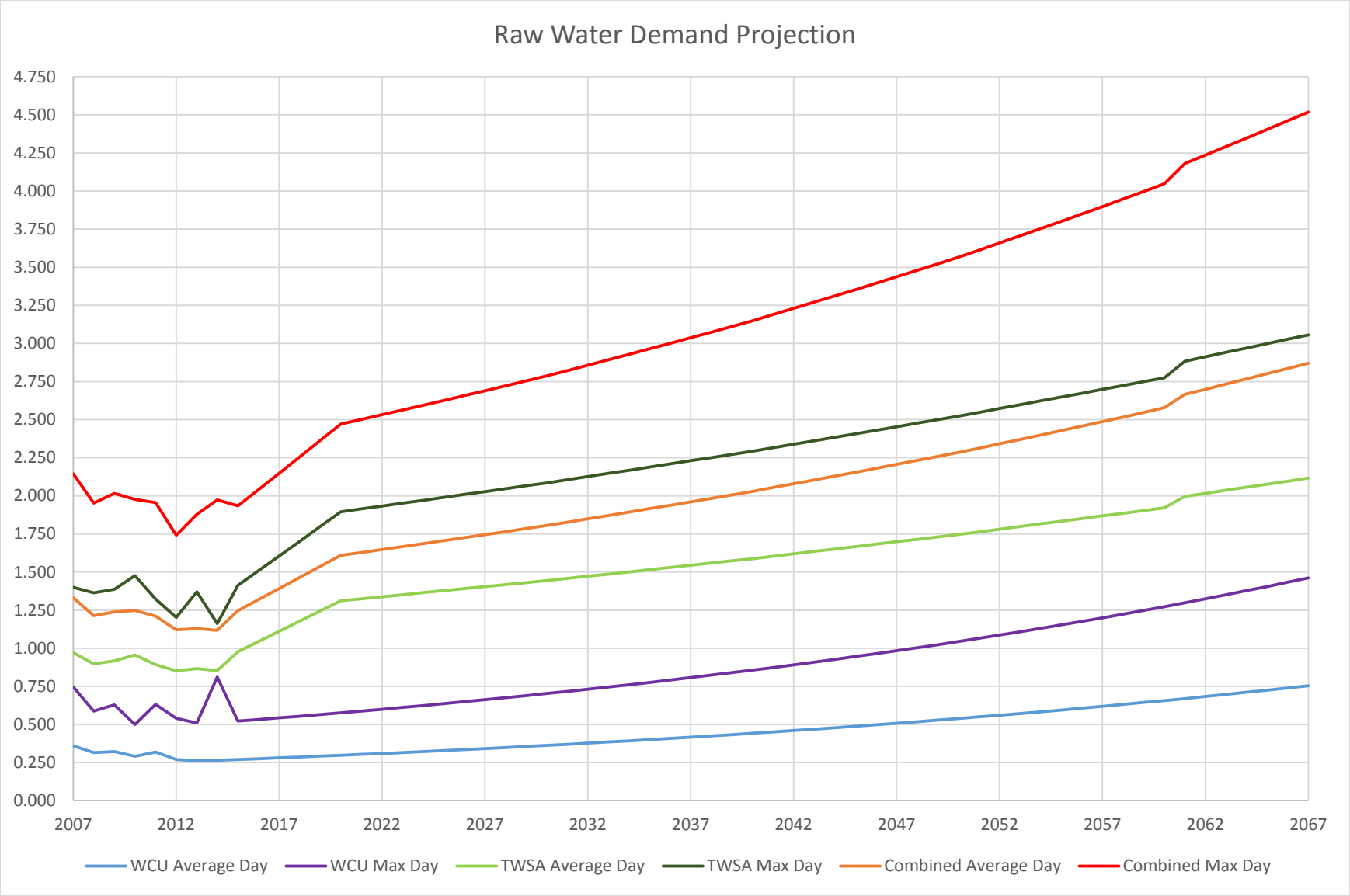


Graph A - 2 - Existing Demand 2007-2015

Table A - 5 - Demand Projection

Western Carolina University				Tuckaseigee Water & Sewer Authority			Combined Withdrawals	
Per Capita Water Use (gpcd)		30.8	59.6		143.8	207.6		
Year	Enrollment	Demand (MGD)		Service Population	Demand (MGD)		Demand (MGD)	
		Avg. Day	Max. Day		Avg. Day	Max. Day	Avg. Day	Max. Day
2007	7,362	**0.360**	**0.746**	5,282	**0.971**	**1.399**	**1.331**	**2.145**
2008	7,493	**0.315**	**0.588**	6,200	**0.898**	**1.364**	**1.213**	**1.952**
2009	7,624	**0.322**	**0.628**	6,400	**0.917**	**1.387**	**1.239**	**2.015**
2010	7,755	**0.291**	**0.500**	6,400	**0.956**	**1.476**	**1.247**	**1.976**
2011	7,886	**0.319**	**0.632**	6,400	**0.892**	**1.323**	**1.211**	**1.955**
2012	8,017	**0.270**	**0.540**	6,000	**0.851**	**1.202**	**1.121**	**1.742**
2013	8,148	**0.262**	**0.509**	5,850	**0.867**	**1.370**	**1.129**	**1.879**
2014	8,450	**0.264**	**0.811**	6,262	**0.854**	**1.162**	**1.118**	**1.973**
2015	8,753	**0.269**	**0.522**	6,800	**0.978**	**1.412**	**1.247**	**1.934**
2016	8,928	0.275	0.532	7,265	1.044	1.509	1.319	2.041
2017	9,107	0.280	0.543	7,730	1.111	1.605	1.392	2.148
2018	9,289	0.286	0.554	8,196	1.178	1.702	1.464	2.256
2019	9,475	0.292	0.565	8,661	1.245	1.798	1.537	2.363
2020	9,665	0.297	0.576	9,126	1.312	1.895	1.609	2.471
2021	9,858	0.303	0.588	9,217	1.325	1.914	1.628	2.502
2022	10,055	0.309	0.600	9,309	1.338	1.933	1.648	2.533
2023	10,256	0.316	0.612	9,400	1.351	1.952	1.667	2.564
2024	10,461	0.322	0.624	9,491	1.364	1.971	1.686	2.595
2025	10,670	0.328	0.636	9,583	1.378	1.990	1.706	2.626
2026	10,883	0.335	0.649	9,674	1.391	2.009	1.726	2.658
2027	11,101	0.342	0.662	9,765	1.404	2.028	1.745	2.690
2028	11,323	0.348	0.675	9,856	1.417	2.047	1.765	2.722
2029	11,549	0.355	0.689	9,948	1.430	2.066	1.785	2.754
2030	11,780	0.362	0.703	10,039	1.443	2.085	1.806	2.787
2031	12,016	0.370	0.717	10,139	1.458	2.105	1.827	2.822
2032	12,256	0.377	0.731	10,240	1.472	2.126	1.849	2.857
2033	12,501	0.385	0.746	10,340	1.487	2.147	1.871	2.893
2034	12,751	0.392	0.760	10,441	1.501	2.168	1.893	2.928
2035	13,006	0.400	0.776	10,541	1.515	2.189	1.916	2.964
2036	13,266	0.408	0.791	10,641	1.530	2.210	1.938	3.001
2037	13,531	0.416	0.807	10,742	1.544	2.231	1.961	3.037
2038	13,802	0.425	0.823	10,842	1.559	2.251	1.983	3.074

Western Carolina University				Tuckaseigee Water & Sewer Authority			Combined Withdrawals	
Per Capita Water Use (gpcd)		30.8	59.6		143.8	207.6		
Year	Enrollment	Demand (MGD)		Service Population	Demand (MGD)		Demand (MGD)	
		Avg. Day	Max. Day		Avg. Day	Max. Day	Avg. Day	Max. Day
2039	14,078	0.433	0.840	10,943	1.573	2.272	2.006	3.112
2040	14,360	0.442	0.856	11,043	1.588	2.293	2.029	3.149
2041	14,647	0.451	0.874	11,153	1.603	2.316	2.054	3.190
2042	14,940	0.460	0.891	11,264	1.619	2.339	2.079	3.230
2043	15,239	0.469	0.909	11,374	1.635	2.362	2.104	3.271
2044	15,544	0.478	0.927	11,485	1.651	2.385	2.129	3.312
2045	15,855	0.488	0.946	11,595	1.667	2.408	2.155	3.353
2046	16,172	0.498	0.964	11,705	1.683	2.431	2.180	3.395
2047	16,495	0.508	0.984	11,816	1.699	2.454	2.206	3.437
2048	16,825	0.518	1.003	11,926	1.715	2.476	2.232	3.480
2049	17,162	0.528	1.024	12,037	1.730	2.499	2.259	3.523
2050	17,505	0.539	1.044	12,147	1.746	2.522	2.285	3.566
2051	17,855	0.549	1.065	12,269	1.764	2.548	2.313	3.612
2052	18,212	0.560	1.086	12,390	1.781	2.573	2.342	3.659
2053	18,576	0.572	1.108	12,512	1.799	2.598	2.370	3.706
2054	18,948	0.583	1.130	12,633	1.816	2.623	2.399	3.753
2055	19,327	0.595	1.153	12,755	1.834	2.648	2.428	3.801
2056	19,714	0.607	1.176	12,876	1.851	2.674	2.458	3.849
2057	20,108	0.619	1.199	12,998	1.869	2.699	2.487	3.898
2058	20,510	0.631	1.223	13,119	1.886	2.724	2.517	3.947
2059	20,920	0.644	1.248	13,241	1.903	2.749	2.547	3.997
2060	21,338	0.657	1.273	13,362	1.921	2.775	2.578	4.047
2061	21,765	0.670	1.298	13,885	1.996	2.883	2.666	4.181
2062	22,200	0.683	1.324	14,025	2.016	2.912	2.699	4.236
2063	22,644	0.697	1.350	14,164	2.036	2.941	2.733	4.292
2064	23,097	0.711	1.377	14,303	2.056	2.970	2.767	4.348
2065	23,559	0.725	1.405	14,443	2.076	2.999	2.801	4.404
2066	24,030	0.739	1.433	14,582	2.096	3.028	2.836	4.461
2067	24,511	0.754	1.462	14,721	2.116	3.057	2.871	4.519



Graph A - 3 - Raw Water Demand Projection

Appendix 6

Letter from Duke Energy re: Cullowhee Dam

DUKE ENERGY.

WATER STRATEGY, HYDRO LICENSING AND LAKE SERVICES

Duke Energy Corporation
526 South Church Street
Charlotte, NC 28202

March 13, 2017

Mr. Joe Walker
Associate Vice Chancellor for Facilities Management
Western Carolina University
3476 Old Cullowhee Road
Cullowhee, NC 28723

RE: Cullowhee Dam

Dear Mr. Walker:

Thank you for our recent discussion concerning Western Carolina University's (WCU) engineering evaluation of options for Cullowhee Dam including the possible removal of the structure. The conference call on March 10 with you and Mr. Dan Harbaugh, Executive Director of the Tuckaseigee Water and Sewer Authority (TWASA) was very informative and appreciated. The Tuckasegee River is a key component of the quality of life in Jackson County and Duke Energy shares common interests in the Tuckasegee River with WCU, TWASA and other local entities.

As you know Duke Energy has two hydropower projects on the Tuckasegee River upstream of Cullowhee Dam. The East Fork Hydro Project includes the Tennessee Creek Development (Tanasee Creek and Wolf Creek Lakes), the Bear Creek Development (Bear Creek Lake) and the Cedar Cliff Development (Cedar Cliff Lake). The West Fork Project includes the Glenville / Thorpe Development (Lake Glenville) and the Tuckasegee Development (Tuckasegee Lake). The East Fork and West Fork Hydro Projects received 30-year licenses from the Federal Energy Regulatory Commission (FERC) in May, 2011. These FERC Project Licenses were based largely on the Tuckasegee Cooperative Stakeholder Team Settlement Agreement (Settlement Agreement) signed by Duke Energy and 16 other stakeholders in October 2003. The Tuckasegee Cooperative Stakeholder Team worked diligently for three years to develop a balanced approach to utilize the water resources in the Tuckasegee River for multiple purposes and across multiple jurisdictions.

The Settlement Agreement and FERC Project Licenses achieve a robust balance for the water uses in the Tuckasegee River. Duke Energy generates electricity and releases water from the East Fork and West Fork Projects in a coordinated manner to achieve this water balance. Flow releases required by the FERC Project Licenses and the Settlement Agreement from the Projects' most downstream dams are as follows:

Project	Dam	Target Flow Releases from the Dam in cubic feet per second (cfs)	Purpose	Target Location	Timing
East Fork	Cedar Cliff	about 500	Recreation	Dillsboro	10:30 AM to 4:30 PM at Dillsboro (required approximately 40 days per year)
East Fork	Cedar Cliff	35	Aquatic Habitat	Downstream of Cedar Cliff Dam	Continuous July 1 through November 30
East Fork	Cedar Cliff	10	Aquatic Habitat	Downstream of Cedar Cliff Dam	Continuous December 1 through June 30
West Fork	Tuckasegee	about 200	Recreation	Dillsboro	10:30 AM to 4:30 PM at Dillsboro (required approximately 40 days per year)
West Fork	Tuckasegee	20 (or inflow, depending on conditions)	Aquatic Habitat	Downstream of Tuckasegee Dam	Continuous

Also incorporated into the Settlement Agreement and FERC Project Licenses is the Low Inflow Protocol (LIP) for the East Fork and West Fork Projects. During drought situations when Duke Energy is unable to maintain the Projects' reservoirs at water levels required by the FERC Project Licenses, Duke Energy must invoke the LIP in order to equitably share the reduced water supply available in the Tuckasegee River. The LIP involves staged reductions in required lake levels, generation for electric customers, recreation flows and continuous minimum flows. For complete information, the FERC Project Licenses for the East Fork and West Fork Hydro Projects and the Settlement Agreement can be viewed at https://www.duke-energy.com/community/lakes/hydroelectric-relicensing/nantahala.

WCU can certainly make its own decisions regarding the future of Cullowhee Dam. While Duke Energy has no direct interest in any choice WCU makes concerning Cullowhee Dam, we believe the Tuckasegee River's relatively small size and the inherent flow limitations within the FERC Project Licenses should be strong considerations in the decision-making process. Three years of negotiations with stakeholders were required to develop the Settlement Agreement which established the current balance of lake levels, recreation flow releases, continuous minimum flow releases and generation for electric customers. If Cullowhee Dam is removed only to find out later the Tuckasegee River does not have enough water flow during certain periods of the year to meet the growing water supply needs of WCU, TWASA and other entities, Duke Energy would not be able to support releasing higher prescribed flows to meet those needs. In fact, Duke Energy and other signatory parties to the Settlement Agreement are contractually prevented from supporting flow prescriptions which are inconsistent with the Settlement Agreement (see Paragraph 12.2 in the Settlement Agreement). Rebalancing the water uses in the Tuckasegee River would require reopening and successfully renegotiating the Settlement Agreement's water balance, followed by revising the Water Quality Certifications issued by the North Carolina Department of Environmental Quality and finally reopening and amending the FERC Project Licenses. Based on the expected degree of difficulty, existing knowledge and operating experience, Duke Energy could not support pursuit of a different water balance than the one currently provided by the Settlement Agreement and FERC Project Licenses.

In closing, Duke Energy respectfully requests WCU consider the above information when making decisions concerning the future of Cullowhee Dam. If you have any further questions about this matter please contact Steve Johnson at (704) 382-4240.

Sincerely,

Jeff Lineberger

Jeffrey G. Lineberger, PE
Director, Water Strategy and Hydro Licensing
Duke Energy Carolinas, LLC

cc: Dan Harbaugh, TWASA
Lisa Leatherman, Duke Energy
Steve Johnson, Duke Energy
Phil Fragapane, Duke Energy
Lynne Dunn, Duke Energy