EXHIBIT E

APPENDIX K

HISTORIC STORM ANALYSIS

Scott’s Mill Hydropower Project

FERC Project No. 14867

APPENDIX K

SCOTT'S MILL HITORIC STORM ANALYSIS

APPROACH

Storms Analyzed

Applicant analyzed three historic storms using USGS published Holcomb Rock data: (1) November 1985, (2) January 1996, and (3) September 2004. FERC staff had requested that 10 storms be analyzed. However, because of the steepness of the headpond area, Applicant determined that flows reach equilibrium quickly and three storms were sufficient to understand inflow/outflow and headpond relationships.

The November 1985 storm is the storm of record. Maximum average daily flow was 180,000 cfs with an estimated peak flow of 207,000 cfs. This equates to a flood level between 100 and 500 years, likely a recurrence interval of about 300 years. The January 1996 storm had a maximum average daily flow of 93,590 cfs,and a peak of 116,000 cfs. It was the third largest flood between 1927 and 2020. (Second largest daily flood had a maximum daily average flow of 96,700 cfs.) The flood equates to a flood with recurrence interval between 10 and 50 years (i.e., recurrence interval of about 40 years). The September 2004 storm had a maximum daily flow of 62,200 cfs with a peak of 71,900 cfs. It was the 10th largest flood from 1927 to 2020. (Floods ranked six through nine were not much higher, ranging from 68,700 to 63,200 cfs.) This is equivalent to about a 7-year recurrence interval based on FEMA's analysis and the Holcomb Rock gauge.

Inflows for each storm were divided into 15-minute intervals. For the 1985 storm, 15 minute interval data was unavailable. The daily flows were plotted on a graph. The average daily flow was assumed to occur at noon. A smooth line was drawn between each daily flow to provide a reasonable approximation of a rising and a falling hydrograph. Flow estimates at 6-hour increments were then made. Flows were then linearly interpolated between these intervals on a 15-minute basis. Average daily flows were then calculated and compared to USGS daily averages. As necessary, minor adjustments were made in the inflow data to obtain the average daily inflow rates.

For the 1996 and 2004 storms, USGS publishes flow data on a 15-minute basis. These data were used to develop the inflow profile. Average daily flows were then calculated and compared to the published daily flow data. The 15-minute and average flow data compared almost exactly for both storms.

The Holcomb Rock drainage area is 3256 square miles. Scott's Mill drainage area is about 3300 square miles. (Downstream at the junction of Blackwater Creek and the James River, the drainage area is 3370 square miles and includes Blackwater Creek.) Since there is only about a 1.5 percent difference in drainage area, the Holcomb Rock flows were not adjusted for inflow to Scott's Mill. (See also discussion on overland flow, below.)

Reservoir-Area Relationships

Reservoir area - elevations were determined for three reservoir elevations (516, 518, and 525 feet NAVD88) using GIS tools. These headpond areas were measured to the upstream project boundary with Reusens Dam. The area from the upstream project boundary to the Reusens Dam was estimated from large scale contour maps as 27 acres, 30 acres and 33 acres for the 516, 518, and 525 contours, respectively. Resulting areas were 305, 318, and 414 acres or 6.5 acres per foot between 516 and 518, and 13.7 acres per foot between elevations 518 and 525. Area - elevation data was interpolated between 516 and 525 feet. (Previous estimates made in 1981 indicated a reservoir surface area of 316 acres at normal pool level of about 516 feet. The primary difference between the areas between 518 and 525 feet is the submergence of portions of the islands in the James River.)

Above 525 feet, cross sections along the headpond axis were measured to the 540-foot contour and compared to the 525-foot contour cross section widths. Based on the increased widths, it was estimated that there are an additional 101 acres between these contours, for a total of 515 acres, an average increase of 7 acres per foot. (The James River islands are fully submerged between 525 and 540 feet. The small increase per foot is indicative of the shoreline steepness.)

Overland Flow Component

Since the shorelines are steep, the river is confined and very high flows do not have much of an overland component. Therefore, the overland flow component was ignored, and all flow was assumed to pass over the 735- foot-long main spillway and the 140-foot-long horseshoe spillway under existing conditions. (Discounting the overland flow component essentially counters the slight increase in flow from the additional drainage area between Holcomb Rock and Scott's Mill Dam.)

Flow Pathways

Under proposed project conditions, the first 4,500 cfs is assumed to go through the powerhouse up to a river flow of 25,000 cfs at which point the powerhouse is shut down due to debris load. Flow over the main spillway is minor and is ignored below the capacity of the powerhouse. Above 4,500 cfs, all flow above the powerhouse discharge goes over the 735 foot-long main spillway only. (This ignores any flow through the fish bypass in the intake area.) As water elevations increase above 521.5 feet flow begins to pass over the powerhouse, which becomes submerged. The powerhouse plus fish bypass/debris stack is 168 feet-wide. Thus, above elevation 521.5 there are two pathways for flow to pass over the dam.

Tailwater Effects

Tailwater levels are assumed to develop instantaneously based on total outflow from Scott's Mill. In a rising flood, the tailwater would lag the steady state condition and during a falling hydrograph, the tailwater would fall slower.

Tailwater levels below flows of 25,000 cfs are based on field measurements. At higher flows. tailwater levels are based on the FEMA Flood Insurance Study for the City of Lynchburg, dated

June 3, 2008. Elevations for the 10, 50, 100 and 500-year floods are based on FEMA's analysis. Tailwater elevations for the high flows were interpolated. Tailwater/flow relationships were assumed to be the same for existing and project conditions since tailwater is controlled downstream. (To develop tailwater relationships through a modeling analysis would have required extensive data from downstream locations. Since existing conditions are being compared to project conditions, the tailwater level/flow relationships are considered appropriate for the analysis.)

Spillway Flow Relationships

Spillway head-flow relationships for flows below 25,000 cfs are based on actual measurements. The weir equation (i.e., $Q=CLH^{1.5}$) was used for all spillway flows. However, the coefficient was developed based on actual head-flow measurements. In general, the coefficient varies from 1.55 for flows of 1,000 cfs to 3.5 for higher flows. For all high flows, a coefficient of 3.5 provided excellent results and was used to calculate upstream water levels for the storm events. The powerhouse acts like a broad crested weir when upstream water levels exceed 521.5 feet. Because the over-powerhouse flow component is much less than flow over the main spillway, the same weir equation and coefficients are used.

At tailwater levels equal to and greater than the spillway crest, submergence affects the spillway discharge. The effect of submergence is complicated but can be approximated based on the weir equation multiplied by a coefficient which in turn is based upon the depth of submergence (i.e., tailwater water level above the dam crest) divided by the water level over the spillway. Figure 219 from the 4th edition of Elementary Fluid Mechanics by John Vennard (copy right 1961) illustrates the effects of submergence. This figure provides the ratio of the actual measured discharge to the weir equation discharge for given headwater (H1) and tailwater (H2) elevations above the dam crest (i.e., for given H2/H1 ratios). The coefficient was crosschecked with the FEMA study results for each of the 4 flood events presented in FEMA's analysis. The coefficient extracted from Vennard showed excellent agreement with the coefficient calculated based on the FEMA study. The coefficients from the FEMA study were therefore used and interpolated for other flows.

For project conditions, the spillway crest is raised by 2 feet. This decreases both H1 and H2. For a given tailwater elevation (and downstream flow), H2 decreases by 2 feet. For the same discharge per foot along the spillway, the upstream water level correspondingly increases but that increase must be less than 2 feet, because if upstream levels were to increase by 2 feet the ratio H1/H2 would be less and the discharge coefficient would increase. Hence the H2/H1 coefficient was increased for the project condition based upon the existing condition coefficient and then cross checked once the upstream head was determined. As appropriate the coefficient was adjusted iteratively and the calculation repeated to get the best fit.

Effect of River Slope and Delay in Inflow Wave Reaching

In the approximate 3.6-mile length of the headpond, there is some delay in flows from Reusens to Scott's Mill. Wave celerity is determined as $V=(2g \times h)^{1/2}$ where V is the velocity in feet per second, g is the acceleration of gravity and h is the average depth. Based on an average depth of

10 feet, the wave celerity would be about 25 feet per second. Therefore in 15 minutes the wave would travel about 4.3 miles. Thus, the flow wave would reach Scott's Mill within 15 minutes.

Secondly there is also some elevation difference (i.e., river slope) from the upstream end to Scott's Mill Dam. Based on FEMA profiles for the 10, 50, 100 and 500 year floods the elevation change between Scott's Mill is about 9 feet, or an average slope of 0.00047 feet per feet. Manning's equation can be used to determine how slope would change in the proposed project condition. Manning's equation is $Q=1.49*A*R^{2/3}*S^{½}/n$, where Q is the flow in cfs, A is the cross sectional area in square feet, R is the hydraulic radius, and S is the slope or energy gradient. The hydraulic radius is the area divided by the wetted perimeter. In a wide river like the James, the wetted perimeter can be represented by the river width, so the hydraulic radius becomes the average depth. For a flow of 159,000 cfs (i.e., 100-year flood) and using the FEMA elevations (532.6 feet and bottom elevation of about 507 feet for an average depth of 25 feet and average cross sectional width of 800 feet, and a Manning's n of 0.035, yields a slope of 0.000477 which is essentially the slope found in FEMA's studies. Since area is D*W. where D is the average depth and W is the river width which changes insignificantly between the existing conditions and proposed conditions and R is the average depth D, the formula can be rewritten as $Q=1.49*W*D*D^{2/3}*S^{½}/n$. Since Q is the same for both existing and proposed conditions and W is essentially the same, the formula can be rewritten as $D_1^{(5/3)}S_1^{(1/2)}= D_2^{(5/3)}S_2^{(1/2)}$, where D_2 is about 2 feet higher than D_1. Substituting $D_2=(D_1+2)$ yields the following: $S_2=(D_1/(D_1+2))^{10/3}*S_{1.}$ By inspection it can be seen that slope (S_2) is less than S_1. Therefore the water level differential will decrease with distance upstream. However, for each increment upstream, say 1,000 feet, the project conditions slope will need to be recalculated since the increment to D_1 will be less than 2 (i.e., gradually varied flow). At some point upstream the differential will cease to exist. However, for this study that was ignored and the water level differential at Scott's Mill was assumed to propagate upstream.

The approximate 9 foot water level differential over the length of the headpond (i.e., slope effects) were ignored in the model since they would be similar for both existing and project conditions and since the incremental area increases with elevation are relatively linear. However, water level increases would attenuate with distance upstream.

When inflows exceed outflow, the excess flow is stored in the headpond and any reservoir changes are accounted for in the next 15 minute interval (i.e., Muskingum method).

<u>Spreadsheet Calculations</u>

For the existing conditions, the initial spillway head was used to calculate the main spillway and horseshoe spillway discharge using the weir equation. The weir coefficient was determined for each spillway head based on the spillway-head relationship developed from field measurements. The coefficient varies from 1.55 for heads less than 1 foot to 3.5 for heads of 4 feet or greater. The outflow was modified by a correction factor based on the level of submergence. When water levels were below the dam crest the factor was one. When water levels were above the crest, the factor was decreased to a fraction less than one based on FEMA's analysis and cross checking with Vennard.

The outflow of each spillway (horseshoe and main spillway) was then summed to yield total outflow. Outflow was then subtracted from the inflow to obtain the change in storage in acre-feet for the 15-minute interval. Based on the initial water level, the reservoir area was determined from the reservoir area-elevation relationship. The net change in storage was divided by the reservoir area to yield the change in water level. The change in water level was added to the existing water level and that water level was used as the starting point for the next 15-minute interval.

For the project conditions, the horseshoe reservoir was no longer available to convey flow downstream. The powerhouse was assumed to be the only means of discharge for flows up to the hydraulic capacity of the plant (4,500 cfs). The minimal flow over the dam of about 25 to 30 cfs was ignored. Above 4,500 cfs, excess flow was passed over the main spillway. At a flow of 25,000 cfs, the powerhouse was shut down and all flow was assumed to pass over the spillway. However, above a headpond elevation of 521.5 feet the powerhouse becomes submerged and flow can pass over it. At that point, the powerhouse acts like a broad crested weir.

Similar to the existing conditions, when tailwater levels where higher than the spillway crest elevation (which was 2 feet higher than under existing conditions), a submergence correction factor was incorporated into the calculation to obtain the discharge. The same correction factor was used for the flow over the powerhouse because its discharge coefficient is likely lower than the main spillway coefficient. This results in a more conservative analysis. Similar to the calculation for existing conditions, the discharges were added to yield total outflow. Outflow was subtracted from inflow to yield the net storage volume, which was converted to acre-feet. Using the area-elevation table, the headpond area was extracted from the table. The storage change in acre-feet was divided by the area to obtain the change in water level. This was added to the previous water level to obtain the water level for the next calculation interval.

The existing condition water levels were subtracted from project operation water levels to obtain the change in elevation between existing and project conditions. In comparing the water levels for existing conditions with the FEMA levels, there were minor differences, but generally very good agreement. However, the more important result was the difference in water levels between the existing conditions and project operations.

RESULTS

The modeling analysis demonstrates that because of the limited storage in the Scott's Mill headpond, inflow and outflow reach equilibrium conditions rapidly. Assuming a linear increase in reservoir area from elevation 516 feet to 540 feet, the total volume stored amounts to about 9,840 acre-feet or 4,944 cfs days. If just the three highest days of flow in the 1985 flood are considered, the flow volume is 317,600 cfs days. That means that the storage volume is only about 1.5 percent of the inflow. This suggests that routing of the flow in the headpond is unnecessary to determine upstream water levels. The weir equations as affected by submergence are sufficient to determine heapond elevation flow relationships at Scott's Mill Dam.

Commission staff requested an analysis of 10 storms. Applicant analyzed only three storms. The water levels at a given flow are similar for each storm. For example, at a flow of 70,000 cfs in the rising limb of the hydrograph, the model predicted the following results:

Storm	Existing Conditions Elev. (ft)	Proposed Conditions Elev. (ft)	Difference (ft)
1985	522.6	525.1	2.5
1996	522.4	525.0	2.6
2004	522.6	525.1	2.5

For the 1985 and 1996 floods, at a flow of 50,000 cfs the "existing conditions" headpond elevation is 520.9 feet for both storms. In the "with project condition", the headpond levels are also the same at 523.5 feet. At a flow of 100,000 cfs, the "existing conditions" headpond levels were calculated to be 525.3 and 525.0 feet and for "with project conditions" the headpond levels were 527.5 and 527.4 feet for the 1996 and 1985 storms respectively. On the falling hydrograph, at 100,000 cfs, "existing condition" and "with project" water levels were the same for both storms at 525.4 and 527.6 feet, respectively.

The primary purpose of the storm analyses was to determine water level changes between existing conditions and project conditions. Water level differences were determined for each 15 minute interval and are presented in the model spreadsheet. A summary of water levels for each storm showing the highest water level for the peak storm flow and the water levels at the flows which show the greatest water level difference are illustrated below.

Storm	Flow (cfs)	Existing Conditions	Proposed Conditions	Difference
1985	207,000	536.2	537.6	1.4
	61,000	521.9	524.5	2.6
1996	116,000	526.6	528.9	2.3
	58,000	521.5	524.1	2.6
2004	71,900	522.7	526.3	2.6

At the highest flow analyzed (i.e., 207,000 cfs, the water level difference at Scott's Mill was 1.4 feet. This was the approximate expected result. As tailwater levels increase above the dam crest elevation, the submergence of the water flowing over the crest of the dam has a greater effect. Since the crest elevation is 2 feet higher during project conditions, the submergence effect is less per foot length of spillway. Thus, the increase in headpond elevation between existing and project conditions must be less than 2 feet. The maximum delta between existing and project conditions and was 2.6 feet at flows of about 50,000 to 70,000 cfs.

The presence of the powerhouse and spillway cap does not affect the duration of the storms. Floods on the James River in the vicinity of Scott's Mill last only from 1 to 3 days.

Commission staff requested inundation maps for the storms analyzed. Because of the steepness of the shoreline, the lines illustrating the different inundation levels between proposed project and existing conditions were indistinguishable on the contour maps. For the 1985 flood, under existing conditions the headpond elevation varies from 536.2 feet at Scott's Mill to about 545 feet at Reusens. Applicant selected the 540 foot elevation contour to illustrate the maximum extent of flooding during the 1985 storm. At Scott's Mill Dam, the 540 contour overestimates flooding, while slightly underestimating flooding upstream at Reusens Dam. (The 540-foot contour also corresponds with the predicted 500-year flood at Scott's Mill.) Even with an approximate 24-foot elevation change in water level from the normal headpond level, there are many areas where the inundation level does not exhibit large changes from the normal operating level (see Figure 1 of Appendix K).

The Exhibit G map does not show existing structures. Applicant therefore examined the Google Earth map of the area to locate the existing structures and mark them on a large scale map of the area. (Appendix K, Figure 1 scale was too small to provide meaningful information.) The effect of potential flooding of these structures is described below.

On the west side of the James River, there are no structures except for a number of structures owned by CSX railroad located about one half mile downstream from Reusens Dam. These structures are situated between elevation 540 and 560 feet and could be affected by storms greater than the 100-year flood. A storm like the one which occurred in November 1985 flood could affect some of these CSX structures, but storms less than the 100-year flood, would not be impacted by either the existing or proposed project conditions. Since these structures are close to the upstream end of the headpond, as described in the Approach section under river slope, the 1.4-foot difference in elevation would be attenuated because the river energy gradient would be less under the proposed conditions.

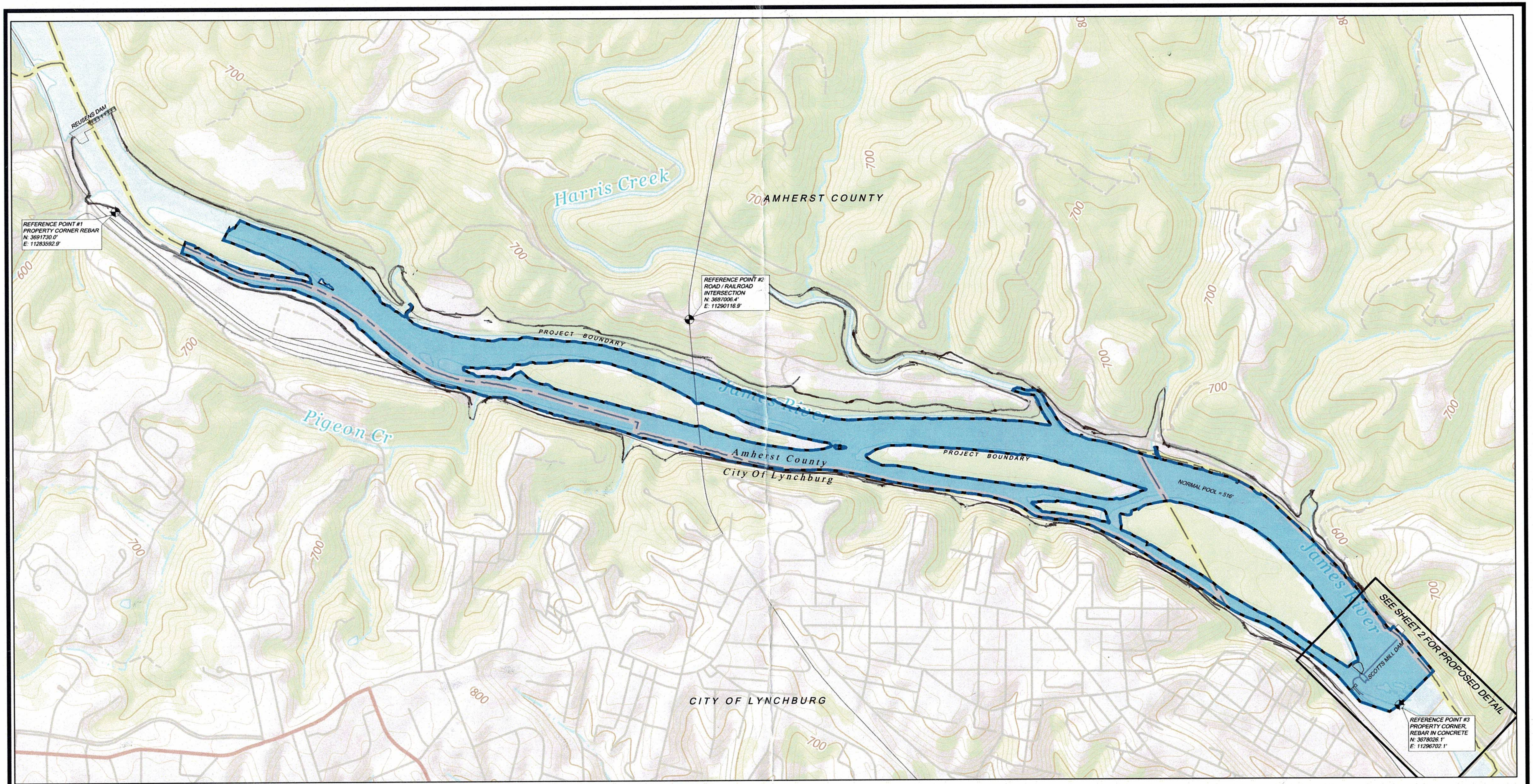
On the east side of the James River, there are about 4 structures about a mile downstream of Reusens Dam at the upstream end of Woodruff Island as observed from Google Earth. They are situated at about elevation 540 feet or greater. The horizontal distance between the 520 and 540-foot contours is about 150 feet. This is the least sloped area between these contour intervals along the entire headpond. For each foot increase, there is about 15 additional feet of ground that would be inundated. Thus, these structures could be minimally affected during a flood such as occurred in 1985. However, at smaller floods (e.g., the 1996 flood which has a recurrence interval of about 40 years), the structures appear to be at a sufficiently high elevation to not be affected. Due to the accuracy of the analysis, a detailed survey of elevations would be needed to determine if there were any effects to these structures.

Downstream adjacent to Woodruff Island, the ground slope between the 520 to 540-foot contours is on the order of 1:10. For each foot rise there are 10 horizontal feet. Structures are set back about 100 to 600 feet from the river at elevation 540 feet or greater. Only at floods greater than 100-year floods, would these structures see effects of the proposed project conditions. Again, detailed surveys would be needed to determine the additional extent of flooding, but it would likely be on the order of 10 to 20 feet horizontally and only for floods greater than a 100-year recurrence interval.

There are additional structures to the east of Treasure Island. They are set back about 150 feet or more from the James River and also appear to be at an elevation higher than 540 feet. Similar 1:10 slopes can be found here. Again, only at the larger floods greater than 100 years would the structures potentially be affected. This area is about 2 ½ miles downstream from Reusens Dam. Per FEMA, the existing conditions flood level for the 100-year flood is 532.5 feet in this area. The proposed project conditions could increase the water level about 2 feet at a flow of 159,000 cfs (i.e., 100-year flood) to about 534.5. For the 1985 storm, modeled elevations at 159,000 cfs indicate headpond levels at Scott's Mill of 531 and 533 feet for existing and proposed conditions, respectively.

There are up to a dozen structures adjacent to Daniel Island between River Road and the James River. The structures on the east side of River Road away from the James River are at an elevation of about 540 feet and again only would be affected by the floods greater than 100 years. The structures between the James River and River Road appear to be at about elevation 530 feet. It appears these structures could be affected by the 100-year flood under existing conditions. There would be an additional increase of 2 feet in water levels under the proposed project conditions.

The incremental increase of 1.4 feet for floods approaching the 500-year recurrence interval and 2 feet for floods approaching the 100-year recurrence interval (i.e., 159,000 cfs) could exacerbate flooding. During the detailed design process, the elevation of nearby structures west of River Road will be surveyed to determine the potential for effects. At the lower frequency floods (i.e., 2 to 7 year recurrence intervals) there could be up to a 2.6 foot increase in water levels. However, the elevation change will raise water levels from about 522.7 to 525.3. Based on the elevations of the existing structures, they should be above any of the flood levels and therefore not impacted.



NOTES:

THIS DRAWING HAS BEEN PREPARED FOR THE PURPOSE OF AIDING IN THE LICENSING APPLICATION FOR SCOTT'S MILL DAMN IN LYNCHBURG, VIRGINIA. THE PROPOSED CHANGES SHOWN HEREON HAVE BEEN DIGITIZED FROM PROVIDED DRAWINGS AND ARE INTENDED TO BE USED FOR ILLUSTRATIVE PURPOSES ONLY. NO FIELD SURVEY HAS BEEN PERFORMED AT THIS TIME.

OVERLAY: USGS QUADRANGLE MAP, LYNCHBURG, DATED 2019.

HORIZONTAL COORDINATES SHOWN ON THIS DRAWING ARE BASED ON VIRGINIA STATE PLANE GRID, SOUTH ZONE, NAD83, US SURVEY FEET.

SURVEYOR'S STATEMENT:

I HEREBY STATE THAT THE FERC P-14867 PROJECT BOUNDARY DELINEATION FOR THE LICENSING PROJECT AS SHOWN ON THIS EXHIBIT "G" IS DEVELOPED WITHIN REASONABLE ACCURACIES TO ITS GEOGRAPHIC LOCATION, AS REQUIRED IN 18 CFR 4.41, BASED ON A GRAPHICAL OVERLAY IN REFERENCE TO USGS QUADRANGLE MAPPING, HURT & PROFFITT RECORD DRAWINGS, PUBLIC GEOGRAPHIC INFORMATION SYSTEMS, AND PREVIOUS FIELD SURVEYS FOR THIS PROJECT, WITHIN ±40 FEET. THE FERC P-14867 PROJECT BOUNDARY LINE IS BASED ON EXISTING CONTOURS FROM THE CITY OF LYNCHBURG PUBLIC GEOGRAPHIC INFORMATION SYSTEM, FOLLOWING THE NORMAL MAXIMUM WATER SURFACE ELEVATION OF APPROXIMATELY 516 FEET ABOVE SCOTT'S MILL DAM, WITH THE PROPOSED PROJECT BOUNDARY BEING UTILIZED BELOW THE DAM. NO FIELD SURVEY HAS BEEN PERFORMED AT THIS TIME.

J. F. SCHUPPE, LS
REGISTERED LAND SURVEYOR NO. 3340
IN THE STATE OF VIRGINIA

DATE OF DRAWING

Apr 27, 2021 - 4:53pm V:\LandProjects\20150824\dwg\FERC_r2.dwg

LEGEND:

PROJECT BOUNDARY

REFERENCE POINT (COORDINATES SHOWN ARE VIRGINIA STATE PLANE GRID, SOUTH ZONE, NAD83)

VA STATE PLANE GRID, SOUTH ZONE, NAD83
NORTH

PROJECT NO.	20150824
FILE NO.:	LS-15304
DATE:	4/20/21
DRAWN BY:	JFS

SHEET 1 OF 2

COMMONWEALTH OF VIRGINIA
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EXHIBIT G - PROJECT BOUNDARY
SCOTT'S MILL PROJECT
FERC NO. P-14867

600 300 0 600 1200
GRAPHIC SCALE IN FEET

Appendix K - Figure 1
Inundation at 540 foot
Contour Elevation.