APPENDIX B

ENGINEERING APPENDIX

Augusta Rocky Creek, Georgia
Flood Risk Management
Section 205 Feasibility Study
Augusta-Richmond County, Georgia

June 2017

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C-1. GENERAL SUMMARY

In 2004, USACE and selected contractors underwent a detailed feasibility study to evaluate many different alternatives to reduce flooding impacts in Augusta, Georgia. USACE studied Rocky Creek, Rae's Creek, Augusta Canal and Phinizy Swamp. Upon the completion of this study, USACE and the City of Augusta discussed which of these alternatives would be feasible for construction. The majority of the recommended solutions were decided against, for reasons such as low BCR ratio, HTRW issues, and others. The purpose of this Engineering Appendix is to re-evaluate and expand upon specific selected alternatives from the 2004 feasibility study for Rocky Creek, and to provide concept designs and ROM (Rough Order of Magnitude) cost estimates for each of the project features that are considered to be feasible potential solutions. The flood improvement features for Rocky Creek have been carried over from the previous feasibility study. Rae's Creek, Augusta Canal and Phinizy Swamp will not be evaluated in this appendix. Engineering recommendations are based on the analysis of data acquired through field investigation and from existing data provided by Augusta – Richmond County and from Corps of Engineers archive files. The engineering investigations and evaluations meet the requirements for a section 205 Feasibility Study. All elevations within this report are stated in NAVD88.

C-2. HYDROLOGY AND HYDRAULICS

C-2.1 INTRODUCTION

In 2004, HEC-RAS and HEC-HMS modeling were performed by Engineering Methods & Applications, Inc./Watershed Concepts, (WSC), as part of the Corps of Engineers (Savannah District) Augusta-Richmond County Flood Control Project. The purpose of this portion of the overall study was to develop hydrologic models of both existing and future conditions for Rocky Creek and to evaluate improvement alternatives. The results of all the models are tied to economic models in order to quantify the existing and future impacts of flood events, and then to select which alternatives would be most beneficial to the Community.

This CAP study has used the previous modeling as a baseline to update and validate specific selected design alternatives with new data and information. The alternatives that were selected for construction are the Rosedale Detention area, and Kissingbower home property buyouts. The Rosedale Detention Area project will consist of installing a new weir/box culvert structure in-line with the existing creek and partially re-constructing an existing earth embankment which is approximately 900 feet in length and about 20 feet in height.

Technical details of the model development conducted in 2004 have been condensed in this report, but can be found in full in the 2004 reports.

C-2.2 PHYSICAL DESCRIPTION

Rocky Creek lies in the central portion of the City of Augusta (
Figure 1). The project area is in the headwaters of Rocky Creek, as shown in **Figure 2** The majority of the stream is south of U. S. Route 78 (Gordon Highway) and north of Interstate 520 (Bobby Jones Expressway). Rocky Creek has numerous small tributaries flowing into it, eventually emptying into Phinizy Swamp approximately 1.2 miles downstream of Georgia Highway 56 Spur (Doug Barnard Parkway). Rocky Creek drains approximately 11,024 acres (17.23 square miles) of Augusta. The Creek is 47,030 feet (8.91miles) in length from its headwaters north of Gordon Highway to its mouth at Phinizy Swamp. Elevations within the Rocky Creek basin range from a high of about 490' to as low as 115' at Phinizy Swamp. The channel has a slope of 0.0021 ft/ft downstream of Milledgeville Road; upstream of Milledgeville Road the channel quickly rises to a slope of 0.012 ft/ft.

Figure 1 : Rocky Creek Vicinity Map

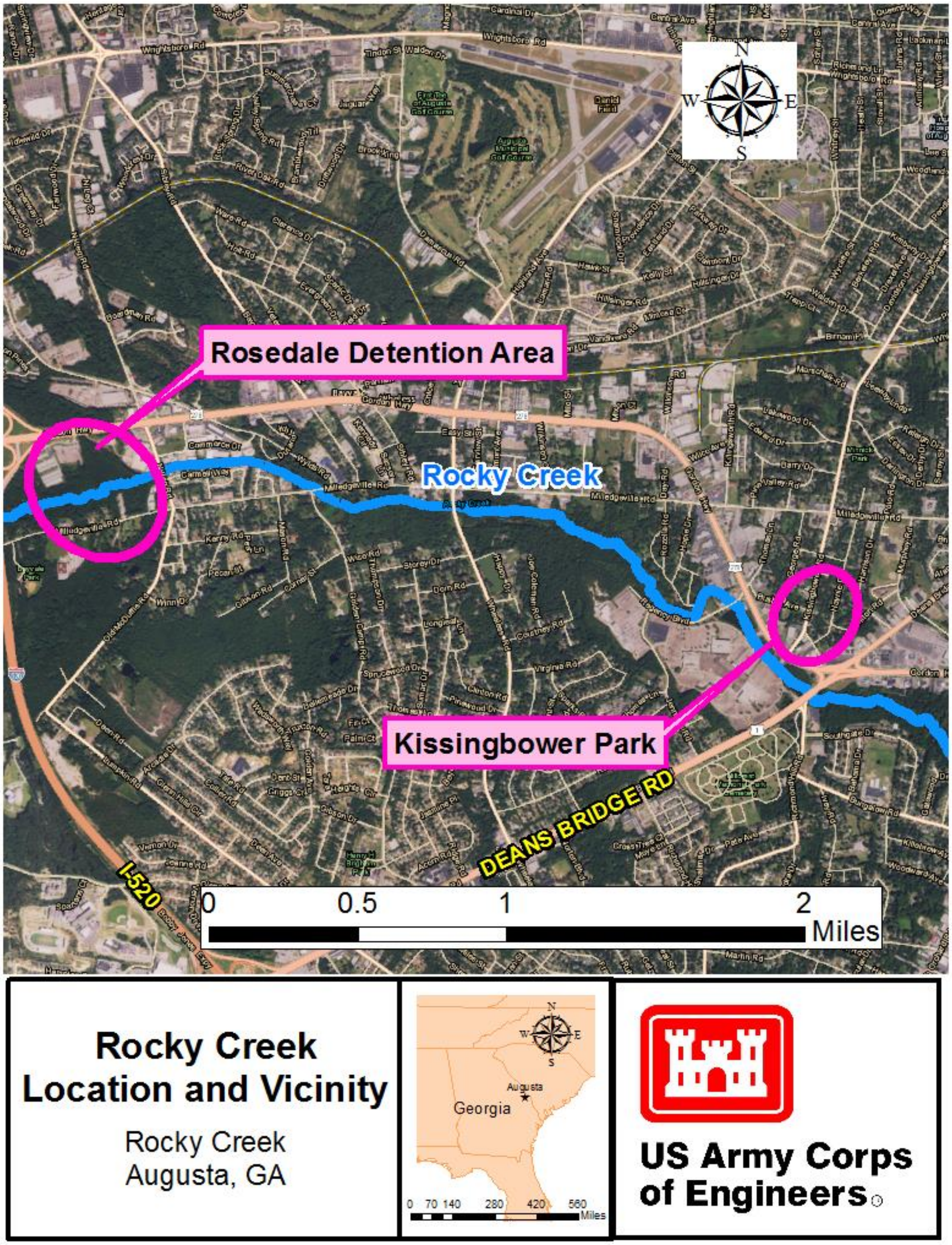

Figure 2 : Project Location Map

C-2.3 PLAN ACCOMPLISHMENTS

The Rosedale Detention area will primarily provide temporary storage for small to medium size flood events. Once constructed, the area will provide additional attenuation time for rainfall runoff, and the peak downstream flow will be reduced by 200-250 CFS, with the most reduction observed at the 25-year event. Exact peak reductions are shown in **Table 11.** Flood elevations will be reduced downstream, particularly immediately downstream. The Kissingbower property buyouts will remove five residential structures from the floodplain. The Kissingbower properties sustain water damage on a fairly frequent interval due to the proximity to Rocky Creek. Residents will be relocated to more suitable location(s), and the area will be converted to recreation such that flooding will not cause further damages to property.

The selected Rosedale Detention area will reduce the peak flow downstream for all rain events. The structure design is targeted to have the largest flood reduction impact up to the 10-year and 25-year flood event. At flows of the 10 year flood event and greater, the overflow weir will be engaged and pass water in addition to culvert flow. The detention structure will still provide a reduction in peak flows and water surface elevations downstream at flows greater than the 10 year event, however the incremental water surface elevation reduction will decrease as flow increases.

C-2.4 HYDROLOGY

Topographic data consisted of digital files with 1-foot interval contours in some areas and 5-foot interval contours elsewhere. WSC was also provided a 30-meter Digital Elevation Model, GIS soils coverage, land use coverage, transportation coverage, and digital aerial images. The Savannah District Corps of Engineers provided the existing conditions hydrology for the Rocky Creek basin for the 2-, 5-, 10-, 25-, 50-, 100-, 250-, and 500-year 24-hour storm events. Calculations of future conditions are based on these models; existing conditions are assumed to reflect land uses in the year 2005, and future conditions are based on estimated land uses in the year 2030. Rainfall totals were obtained from TP-40; a summary is shown in **Table 1.**

Table 1 : TP-40 Rainfall

Duration	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	250-yr	500-yr
5 min	0.50	0.56	0.61	0.69	0.75	0.81	0.89	0.95
15 min	1.05	1.21	1.33	1.51	1.66	1.80	2.00	2.20
1 hr	1.90	2.36	2.69	3.17	3.53	3.90	4.35	4.70
2 hr	2.20	2.75	3.20	3.60	4.00	4.40	4.90	5.30
3 hr	2.45	3.00	3.50	3.90	4.35	4.80	5.20	5.65
6 hr	2.70	3.60	4.25	4.90	5.50	5.90	6.70	7.25
12 hr	3.30	4.25	4.80	5.75	6.15	6.95	7.70	8.40
24 hr	3.75	4.75	5.80	6.60	7.40	8.00	8.90	9.70

For this analysis, the Rocky Creek basin was divided into 33 subbasins - 24 subbasins by the Savannah District (SBx) and 9 subbasins by WSC (ROCKYx) - using the Corps' HEC Geo-HMS GIS extension. For each subbasin, SCS Curve Numbers (CN) were calculated based on land use and soil types assuming Type II antecedent moisture conditions (average conditions). The Rocky Creek basin is composed primarily of Type C soils (98%) and only 2% of Type B soils. Type C soils are characterized by clay loams, shallow sandy loam, soils low in organic content, and soils usually high in clay, while Type B soils are characterized by shallow loess and sandy loam.

Table 2 shows a hydrologic parameter summary for current and future basin conditions, generated with the GIS datasets described above.

Table 2 : Hydrologic Parameter Summary

Basin	Area (sq. mi.)	Tc (hrs)	CN (existing)	CN (future)
SB1a	0.14	1.5	67	70
SB1b	0.14	1.3	67	70
SB1c	0.10	1.2	67	70
SB2	0.62	1.9	72	76
SB3	0.43	1.9	75	79
SB4	0.16	1.4	70	73
SB5	1.05	3.0	66	69
SB6	0.63	2.0	67	70
SB7	1.65	3.0	77	81
SB8	0.11	1.7	80	84
SB9	0.54	1.7	69	72
SB10	0.18	1.3	72	76
SB11	0.23	1.5	67	70
SB12	1.39	2.8	73	77
SB13	0.08	1.0	74	78
SB14	1.51	2.7	77	81
SB15	0.28	1.4	74	78
SB16	0.57	1.6	82	86
SB17	0.58	2.1	73	77
SB18	0.81	3.0	74	78
SB19	4.06	4.0	76	80
SB20	0.83	1.9	73	77
SB21	0.47	2.7	78	82
SB22	0.67	4.8	74	78
ROCKY1	0.10	0.32	90	95
ROCKY2	0.33	0.97	84	88
ROCKY3	0.87	0.76	84	88
ROCKY4	0.13	0.97	76	80
ROCKY5	0.14	0.73	76	80
ROCKY6	0.43	0.71	76	80
ROCKY7	1.22	1.90	81	85
ROCKY8	0.16	0.15	71	75
ROCKY9	0.11	0.15	76	80

The Rocky Creek basin is well developed. Approximately 58 percent of the basin is either residential or commercial development.

The Curve Numbers were obtained by combining the soils and land use datasets, and then calculating a Curve Number for each combination. As expected from the degree of development, the average existing conditions Curve Number for Rocky Creek is 75.

The future conditions Curve Numbers were calculated by WSC from the same soils coverage, but the land use coverage was adjusted to reflect future Augusta-Richmond County planning and zoning maps (year 2030). The average future conditions Curve Number is 79.

The Hydrologic Engineering Center's *Hydrologic Modeling System* (HEC-HMS) version 2.2.2 was used to calculate runoff and to generate hydrographs. Within HEC-HMS, hydrograph generation was based on the NRCS (SCS) Lag Method.

The Lag Time parameters, T_L , for the "SBx" subbasins were calculated based on the relationship given in the USGS publication, *Lagtime Relations for Urban Streams in Georgia:*

$$T_L = 7.86 * DA^{0.35} * TIA^{-0.22} * S^{-0.31} * QV$$

T_L = lagtime (hrs)
DA = drainage area (sq mi)
TIA = measured total impervious area (%)
S = channel slope (ft/mi)
QV = qualitative variable (set to 1)

The Lag Time parameters for the "ROCKYx" subbasins were calculated from the empirical formula,

$$T_L = 3/5 * T_c$$

where T_L = Lag Time and T_c = Time of Concentration.

The times of concentration were calculated using the NRCS (SCS) velocity method. The different flow regimes in the velocity method include:

sheet flow:

$$T_c \text{ (hours)} = 0.007(nL)^{0.8} / (P_2)^{0.5}S^{0.4}$$

T_c = time of concentration
n = manning's n for sheet flow
L = length of sheet flow path (ft); note: this is typically less than 200 feet
P_2 = 2-year 24-hour rainfall (in)
S = slope of path (ft/ft)

shallow concentrated flow (unpaved):

$$T_c \text{ (hours)} = L / 16.1345(S)^{0.5}$$

T_c = time of concentration
L = length of shallow concentrated flow path (ft)
S = slope of path (ft/ft)

channel flow:

$$T_c \text{ (hours)} = L / ((1.49R^{0.67}S^{0.5})/n)$$

T_c = time of concentration
L = length of channel flow path (ft)
R = hydraulic radius (ft)
S = slope of path (ft/ft)
n = manning's n for channel flow

The total time of concentration for a subbasin is the sum of the individual times.

The Rocky Creek existing conditions model was compared to regional regression equations adjusted for urbanization by the Savannah District. The comparison location was selected so the effects of backwater from Phinizy Swamp would not influence the results. The location of comparison is just downstream of Wheeless Road, just upstream of the abandoned Regency Mall, and at the headwater of SB18. The sum of these subbasins is approximately 9.8 mi^2. A comparison of flows calculated by both HMS and regression equations from 2002 are shown in **Table 3.** Discharges at various locations are given in **Table 4.** Detailed HMS output is available in USACE archives.

Table 3 : Discharge Calibration Comparison (Existing Conditions)

	10-yr	50-yr	100-yr	500-yr
Regression (2002) (cfs)	3121	4435	5028	6612
HEC-HMS (cfs)	3017	4441	5023	6576
% Error	3.3	-0.1	0.1	0.5

Table 4 : Rocky Creek Base Condition Discharges (CFS)

Location	*2-yr ex*	*2-yr fu*	*10-yr ex*	*10-yr fu*	*100-yr ex*	*100-yr fu*	*500-yr ex*	*500-yr fu*
Mike Padgett Hwy	1187	1677	3452	4814	5766	7002	7532	8363
Dean's Bridge Rd	1102	1373	3034	3410	5051	5482	6616	7121
Wheeless Rd	786	985	2247	2526	3799	4123	4998	5334
North Leg Rd	221	283	603	680	1008	1086	1301	1377
Barton Chapel Rd	33	51	110	133	187	195	226	238

During the 2014-2015 CAP study, the HEC-HMS basin model was not adjusted, enhanced, or recalibrated in any way. Previous modeled flows were assumed to be adequate and accurate for this design. Various different versions of the HMS model were available for analysis. Each of these models produced similar output to the table above, however none of the model configurations were able to exactly reproduce the outputs. However, since no modifications were made to the HMS model, the outputs don't change, and therefore the inputs into HEC-RAS do not change either.

Regression equations have been updated using data through 2006. New regression equations output could be useful if the basin model were to be recalibrated. Impervious area data would have to be obtained and calculated for each sub basin, and input into the following equations for region 3. Regression values will be re-computed as part of updating the hydrology during additional studies.

Recalculation of regression flows was not done as part of this effort due to 1) limited availability of basin delineations used in previous studies 2) high average standard error (54% to 74.5%) associated with output. 3) Augusta located right on the border of Region 3 and Region 1 (to the north) and Region 4 (to the south).

Percent annual exceedance probability	3 $0.20\ mi^2 < DRNAREA < 5.5\ mi^2$
50	$35.2(DRNAREA)^{0.632}10^{(0.0297IMPNLCD01)}$
20	$56.1(DRNAREA)^{0.634}10^{(0.0270IMPNLCD01)}$
10	$72.1(DRNAREA)^{0.636}10^{(0.0257IMPNLCD01)}$
4	$94.6(DRNAREA)^{0.637}10^{(0.0243IMPNLCD01)}$
2	$113(DRNAREA)^{0.639}10^{(0.0234IMPNLCD01)}$
1	$132(DRNAREA)^{0.639}10^{(0.0227IMPNLCD01)}$
0.5	$153(DRNAREA)^{0.641}10^{(0.0220IMPNLCD01)}$
0.2	$184(DRNAREA)^{0.642}10^{(0.0212IMPNLCD01)}$

Figure 3 : Current Regression Equations

According to the 2002 WCS report, the year 2030 land use data was developed by WSC using the Augusta-Richmond County planning and zoning maps. There is no Future land use dataset available for download on the MRLC (Multi-Resolution Land Characteristics Consortium.) New existing Land Use datasets for 2006 & 2011 are available. An analysis could be done on those differences and further projected than year 2030, and to see if the difference between year 2001 and year 2011 was more or less than previously projected. Additional studies and completely updated hydrology could be done considering new land use data in the future.

C-2.5 CLIMATE CHANGE

USACE screening level climate change vulnerability assessment (VA) tool was utilized to assess the potential impacts and likelihood of climate change impacts to this region. The tool operates on a HUC-4 level spatial scale, and it used to quickly assess climate change vulnerably. The tool can be found on https://maps.crrel.usace.army.mil/apex/f?p=170:2:963367691217::NO:::

The parameters that were used are as follows:
Division: South Atlantic
District: Savannah
Business line: Flood Risk Reduction
Indicators under selected business line: Annual Cov, Runoff Precipitation, Flood Magnification C & L, Urban 500 Yr Floodplain area.
Climactic Data Source: CMIP-5 (2014)
Threshold: 20%
ORness: .7[1]

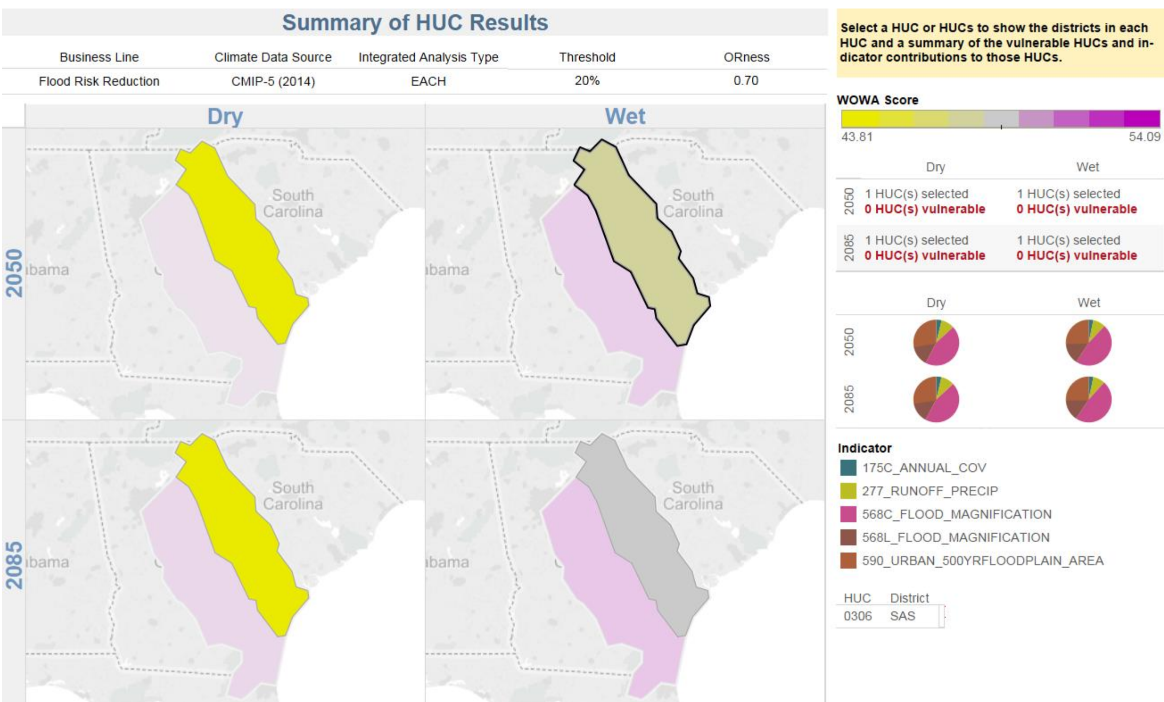

Figure 4 : HUC0306 Summary Results

WOWA Score[2]: 46.17

[1] Specifies how risk-averse the analysis should be. Value should be between 0.5 and 1.0. Higher ORness values weigh the more vulnerable indicators more heavily, resulting in greater perceived vulnerability overall (more risk-averse). Lower ORness values weigh all indicators in a business line more equally, resulting in lower perceived vulnerability overall because less vulnerable indicators average out more vulnerable indicators (less risk-averse). Typical value is 0.7

[2] WOWA stands for “Weighted Ordered Weighted Average,” which reflects the aggregation approach used to get the final score for each HUC. After normalization and standardization of indicator data, the data are weighted with “importance weights” determined by the Corps (the first “W”). Then, for each HUC-epoch-scenario, all indicators in a business line are ranked according to their weighted score, and a second set of weights (which are the OWA weights,” are applied, based on the specified ORness level. This yields a single aggregate score for each HUC-epoch-scenario called the WOWA score. WOWA contributions/indicator contributions are calculated after the aggregation to give a sense of which indicators dominate the WOWA score at each HUC.

The WOWA Score of the Savannah-Ogeechee watershed is a standardized way to compare climate change vulnerability to other basins throughout the United States. The WOWA score for the basins throughout the country under the Flood Risk Reduction Business line ranges from 35.15 to 92.85. **Figure 5** shows how the project basin is related to the rest of the country.

The Savannah-Ogeechee watershed is at a relatively low risk for impacts to climate change within Flood Risk Reduction projects, compared to the rest of the continental United States.

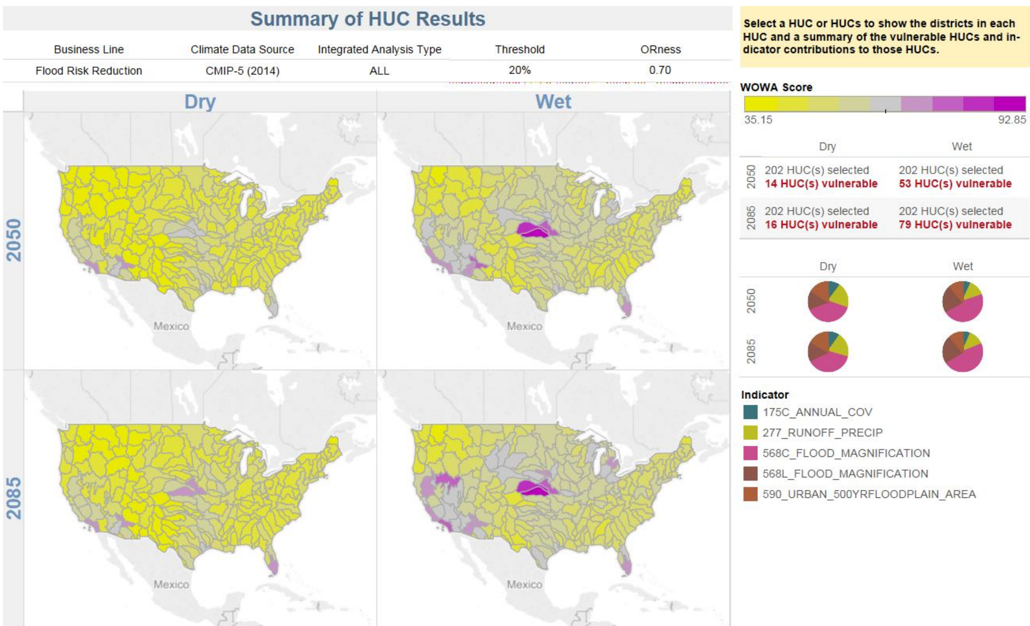

Figure 5 : Nationwide HUC Comparison

The vulnerability WOWA score was also evaluated over time, from the period 2050 to 2085. During a wet hypothetical future scenario, the WOWA score can be expected to increase approximately 1.93%. Suring a dry hypothetical future scenario, the WOWA score can be expected to increase by 0.91%.

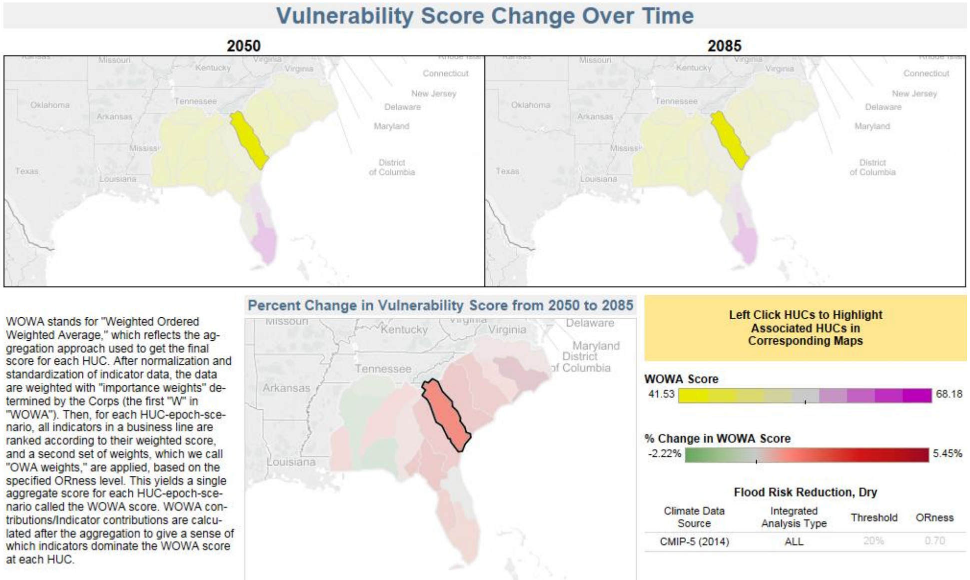

Figure 6 : HUC Vulnerability over time

C-2.6 HYDRAULICS

The Hydrologic Engineering Center River Analysis System (HEC-RAS) version 3.0.1 and 3.1.1, in conjunction with the Corps' HEC Geo-RAS GIS extension were used to calculate the water surface elevations for each storm event. WSC was provided the steady-state existing conditions RAS model for Rocky Creek along with survey data for natural cross-sections and structures at stream crossings from the Savannah District, and additional cross-sections were interpolated based on these surveyed cross-sections and digital topographic data.

The HEC-RAS model was used with the discharges from the HEC-HMS model taking in to account existing and future land use. The HEC-RAS model extended from the outfall at Phinizy Swamp to a point approximately 1,500 feet upstream of Barton Chapel Road. Due to the dramatic difference in channel slope downstream of Doug Bernard Parkway, where the slope is 0.0021 ft/ft compared to 0.012 ft/ft further upstream, both the steady and unsteady options were utilized within HEC-RAS.

The entire stream was initially modeled in steady-state assuming initial conditions at Phinizy Swamp which were based on the flood levels published in the effective Flood Insurance Study for the City of Augusta. The downstream portion of Rocky Creek, from approximately 3,150 feet upstream of Doug Bernard Parkway to the confluence with Phinizy Swamp, was included in an unsteady HEC-RAS model of Phinizy Swamp (Note that details of the Phinizy Swamp modeling can be found in the 2004 Engineering Appendix).

A steady-state methodology assumes that peak flood levels are coincident with peak runoff discharges. This is applicable for most of Rocky Creek, except the lower section. Lower Rocky Creek is flat enough where the backwater effects of Phinizy Swamp will dictate the flooding characteristics. In this lower section, an unsteady HEC-RAS model allows for peak flood stages to occur independent of the time of peak discharge. With flood stage results of the entire stream from the steady-state model, and flood stages for the lower section from the unsteady model, the total picture of the Rocky Creek flooding is a combination of the two methods.

The flood profiles and inundation mapping for the existing and future conditions for Rocky Creek are given in the 2004 Feasibility Study Engineering Appendix. All elevations are referenced to the North American Vertical Datum of 1988, and in units of US Survey Feet. Flood elevations for the base condition at specific points along the stream are shown in **Table 5.** This data is directly from the 2004 Feasibility Study. Detailed digital HEC-RAS outputs are available in USACE archives.

Table 5 : Rocky Creek Base Condition Flood Elevations (feet NAVD)

Location	*2-yr ex*	*2-yr fu*	*10-yr ex*	*10-yr fu*	*100-yr ex*	*100-yr fu*	*500-yr ex*	*500-yr fu*
Mike Padgett Hwy U/S	130.8	132.1	134.5	135.7	136.3	136.5	136.6	136.8
Dean's Bridge Rd U/S	151.9	152.6	155.9	156.4	158.6	160.3	162.6	162.6
Wheeless Rd U/S	172.8	173.6	176.9	177.4	180.7	180.9	181.4	181.5
North Leg Rd U/S	204.8	205.7	209.7	210.6	213.8	214.6	216.0	216.2
I-520 (Bobby Jones Expwy) U/S	241.9	242.2	243.9	244.3	245.5	245.8	246.4	246.7
Nolan Connector U/S	243.2	243.5	244.5	244.8	246.0	246.2	246.9	247.1
Gordon Highway U/S	285.6	286.2	288.0	288.6	289.9	290.1	290.8	291.0
Barton Chapel Rd U/S	301.2	301.9	306.4	307.1	307.3	307.4	307.5	307.5

C-2.6.1 2015 UPDATED HYDRAULIC MODEL

The HEC-RAS modeling done in 2004 was obtained from archives and deciphered. There were hundreds of different combinations of geometric data and flow data to represent all of the previously analyzed alternatives. However, since all of the structural alternatives except for the Rosedale detention area have been eliminated, those plans are not relevant for this study. The relevant geometry and flow files below were copied over into a new project, as the base conditions to begin model updates.

- Plan: Existing conditions 2004 w/o project, 2004 geometry with 2004 flow.
- Plan: Future conditions 2030 w/o project, 2004 geometry with 2030 flow
- Plan: Future Rosedale CEO 2004, 2004 geometry with 2004 dam design recommendation, with modified 2030 flow to simulate routing.

Since the HEC-HMS computed flows did not change, the primary element of the model that was revised was the geometry. It was necessary to go revisit all of the structure crossings on Rocky Creek to validate that they did still in fact exist. Additionally, aerial imagery suggested that there had been some additional crossings constructed since 2004. All modeled crossings were photographed and measured; new data was incorporated into the 2014 geometric conditions. See **Figure 7 and Table 6**, in order beginning in Phinizy swamp and progressing upstream.

Additional cross sections were extracted from new LiDAR data in the following locations:

- Behind Rosedale detention area to define ponded area as accurately as possible
- Downstream of model Station 15000 (or ½ mile downstream of Peach Orchard Road) for more accurate mapping
- Various locations on the reach when prior sections were spaced >1000 ft apart.

See section **C-3.** For additional details regarding LiDAR Data.

Table 6 : Rocky Creek Structure Crossings

Structure Name	2004 Model	2014 Model	Update Notes
Gravel Pit Road	Yes	Yes	No change
Doug Barnard Road	Yes	Yes	Added Pier caps, channel realignment, and smaller abutments
N & S RR Bridge #1	Yes	Yes	Added Pier Caps
N&S Bridge #2	Yes	Yes	Added culvert obstruction due to siltation, and additional culverts off main channel
Mike Padgett Hwy	Yes	Yes	Added Pier Caps, abutments, and additional culverts off main channel
Peach Orchard Rd	Yes	Yes	Added pier caps and guardrail
Deans Bridge Road	Yes	Yes	Added 1.5’ to guardrail
Regency Mall East Entrance	Yes	Yes	Added abutments and 1’ to guardrail
Regency Mall Middle Entrance	Yes	Yes	Added 1’ to guardrail
Regency Mall West Entrance	Yes	Yes	No change
Wheeless Rd	Yes	Yes	Added 1’ to guardrail
Milledgeville Rd	Yes	Yes	No change
North Leg Road	Yes	Yes	No change
I-520	Yes	Yes	No change
Nolan Connector	Yes	Yes	Siltation blockage removed
American Tire Distribution Driveway	No	Yes	New construction. Added 3 RCP, wing walls, sedimentation blockage and road deck.
Gordon Hwy	Yes	Yes	Box culvert dimension change
Barton Chapel Road	Yes	Yes	Roadway width updated
Mobile Home Park	Yes	No	Mobile home park no longer exists
SBD RR	Yes	Yes	Blocked conveyance updated

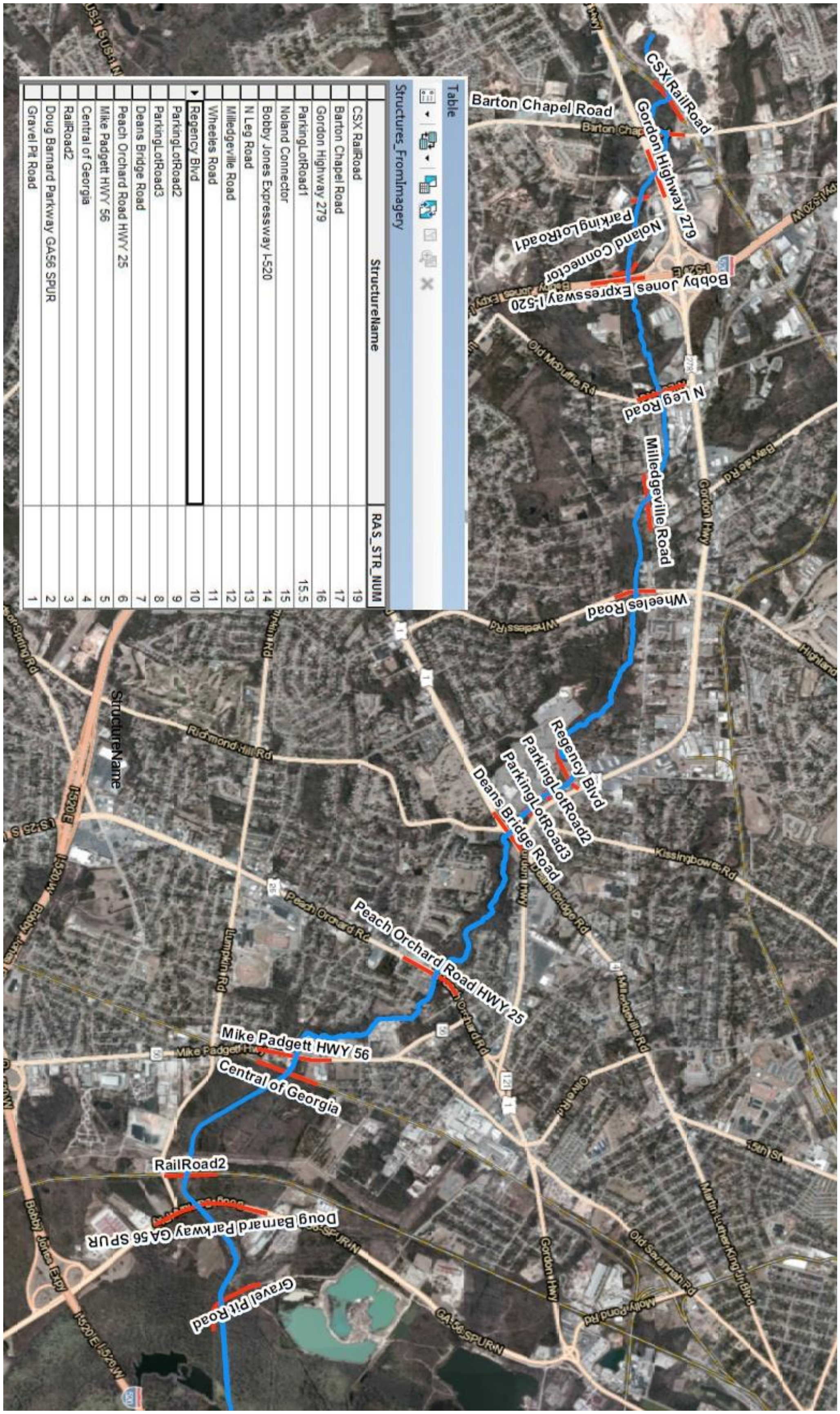

Figure 7 : Stream Crossing Structures

C-2.6.2 DETENTION AREA SIZE AND HAZARD CLASSIFICATION

According to USACE publication ER_1110-2-1156: Engineering and Design, Safety of Dams Policy and Procedures, any artificial barrier constructed for the control of water which is either 1) 25' in height from natural stream bed or 2) has an impounding capacity of 50 ac-ft or greater is considered to be a dam. This definition applies whether the dam is a permanent reservoir or a detention dam for temporary storage of floodwaters. The Rosedale detention area will not be a permanent impoundment of water, but rather a dry storage area to temporarily impound storm water and reduce the peak flow loading downstream.

Table 7 : USACE Hazard Potential Classification

Category	Rating	Description
Direct Loss of Life	Low	No direct loss of life is expected
Lifeline Losses	Low	No disruption of services can be expected. Repairs would be cosmetic and rapidly repairable
Property Losses	Low	Isolated buildings and equipment.
Environmental Losses	Low	Minimal incremental damage

In order to confidently assign a DSAC rating to the completed dam, additional modeling, mapping and investigations must be done, including a dam breach analysis. However, due to the small size, no permanent impounding, and new construction with suitable soils and riprap, a DSAC 5 rating would be the likely recommendation.

The Rosedale detention area would have approximately 161 ac-ft of storage and 23.3' foot head height under full pool conditions. According to Georgia Safe Dams criteria, a "small dam" will have between 100 ac-ft and 500 ac-ft of storage capacity, and not more than 25' of head differential. The Rosedale detention area would fall into this category. The required design storm for a "small dam" is 25% of the PMP.

HMR-51 records for the PMP rainfall for 10 square miles at Augusta Georgia are shown below in **Table 8.** The 6-hr rainfall depths are very similar, within ½" of rainfall. The HEC-RAS model was run with flows generated for the 500-yr and 24-hr duration, or 9.7" rainfall. The water surface elevation within the detention area during this case was 235.44, still over 4.5' of freeboard to the top of dam elevation of 240'. During final design, the 25% PMP can be modeled to ensure that the dam will not be overtopped, and even that 3' of freeboard will remain.

Table 8 : HMR51 Rainfall

	HMR51 PMP	25%PMP	500-yr
6-hour	31"	7.75"	7.25"
12-hour	37"	9.25"	8.4"
24-hour	44"	11"	9.7"

C-2.7 SELECTED AND ELIMINATED ALTERNATIVES

The 2004 Feasibility Study utilized the base condition HEC-RAS model to identify areas of high flooding potential. A total of seventeen potential actions for improvement were identified. To quantify the effectiveness of the alternatives, Rocky Creek was divided into seven distinct sub-reaches. These sub-reaches and the possible improvement alternatives in each are described in **Table 9.**

Initial models were constructed for each alternative to gage its effectiveness. Channel improvements were initially modeled for all reaches, except for R7, as stand-alone models. Structural (culvert, bridge, and levee) improvements were then modeled for each relevant reach. Finally, possible detention ponds were modeled to determine their effectiveness in attenuating the floods downstream. If the initial modeling produced favorable results, three other alternative design plans were modeled. Based on the most promising plans, combinations of channel improvements, structural improvements, and detention ponds were considered. This produced combination improvement models with the contribution of each component to be evaluated once again.

These modeling efforts produced eleven combinations of detention ponds, structural improvements, and channel improvements based on flood prevention and cost. The combinations are shown in **Table 10.** Details regarding each potential improvement can be found in the 2004 Feasibility Report.

During the course of the last ten years, virtually all of these alternatives were eliminated for a variety of reasons; most commonly the flood reduction benefits were nominal or negligible. A brief summary of reasons for elimination of alternatives are shown below.

The modeling analysis for each of these improvements was performed in the previous 2004 Feasibility study, and was not part of this section 205 effort. This section 205 study includes the evaluation of measures that were included in the project management plan (PMP).

Eliminated Due to nominal or negligible flood benefits

- Gravel Pit Road Culvert/Bridge
- Norfolk & Southern Railroad #2
- Chester St Levee
- Dean's Bridge Improvements
- Rozella Berm
- Wheeless Rd Culverts
- Milledgeville Rd Culvert/Bridge Replacement
- North Leg Rd Culvert Replacement
- Barton Chapel Rd Culvert Replacement
- Channel improvement along Rocky Cr.
- Barton Chapel Rd Trailer Park
- Noland Detention Basin

Eliminated Due to Cost

- North Leg Rd Detention Basin

Eliminated due to Sponsor request

- Wheeless Rd Detention Basin

Eliminated due to HTRW issues

- Nixon Street Levee

The remaining alternatives that are being reevaluated for construction include:

- Rosedale Detention Area
- Kissingbower home property buyouts.

Table 9 : Sub-Reaches

Stream	Sub reach	Improvement
Rocky	R1 – mouth to Mike Padgett Hwy	- Channel improvements - Improve culvert at Gravel Pit Rd - Levee along Suffolk Rd and Nixon Rd - Bridge improvements at Mike Padgett Hwy and D/S RR - Non-structural buyout along Dan Bowles Rd area
Rocky	R2 – Mike Padgett Hwy to Regency Mall	- Repair Old Mill Dam - Channel improvements - Improve/Remove three bridges at Regency Mall - Berm along Chester Avenue - Berm along Gordon Highway opposite mall
Rocky	R3 – Regency Mall to Wheeless Rd	- Detention pond U/S of mall (at Rozella Road) - Channel improvements - Buy out homes
Rocky	R4 – Wheeless Rd to Rosedale Dam	- Bridge improvements at Wheeless Rd - Detention pond and buyout of residential and commercial structures U/S of Wheeless Rd - Channel improvements - Culvert improvements at Milledgeville Rd - Culvert improvements at North Leg Rd - Detention pond U/S of North Leg Rd
Rocky	R5 – Rosedale Dam to Bobby Jones Expwy	- Rosedale Dam repair or rebuild - Channel improvements
Rocky	R6 – Bobby Jones Expwy to Barton Chapel Rd	- Detention pond U/S of Nolan Connector - Culvert improvements at Nolan Connector - Channel improvements
Rocky	R7 – Barton Chapel Rd to U/S limit	- Culvert improvements at Barton Chapel Rd - Develop relocation or buyout plan for trailer park at Barton Chapel Rd - Establish maintenance program

Table 10 : Improvement Alternative Combinations

Rocky Creek Features	RY1	RY2	RY3	RY4	RY5	RY6	RY7	RY8	RY9	RY10	RY11
Lombard Detention Pond		X									
Rozella Detention Pond	X	X	X	X	X	X	X	X			
Wheeless Detention Pond	X	X	X	X	X	X	X	X	X	X	X
Rosedale Dam Detention Area	X	X		X	X	X	X	X			X
Nolan Connector Detention Basin		X									
Excavation & Berm at Regency Mall							X	X			
Bridge/Culvert Improvement at Milledgeville	X	X									
Culvert Improvements at North Leg	X	X				X	X	X			
Bridge Improvements at Wheeless	X	X									
Culvert Improvements at ‘s’s Chapel	X	X	X	X	X	X	X	X			
Nixon Street Levee	X	X	X	X	X	X	X	X	X	X	X
Chester Avenue Berm					X	X	X	X			
Remove 3 Mall Crossings								X			
Channel Improvements	X	X									
Clear & Even Channel Inverts at Dean’s Bridge and Peach Orchard	X	X	X	X	X	X	X	X			X
Priority III Channel Improvements U/S and D/S of Peach Orchard			X	X	X	X	X	X			
Priority III Channel Improvements with Meandering between Wheeless and Milledgeville			X	X	X	X	X			X	

C-2.7.1 SUMMARY OF RESULTS

The resulting flood discharge reductions at several locations along Rocky Creek are shown in **Table 11** for the with-project improvement condition.

The Rosedale-only detention area produced positive, yet somewhat limited flood reduction benefits. There are critical levels that the flood elevations would have to be below to capture visible improvements in areas not targeted by the Nixon Levee. One critical elevation is based on the overflow level between Deans Bridge Road and Peach Orchard Road. Both the stand-alone detention pond options still result in overflow across Bungalow Road and continued residential flooding, as compared to the RY11 results, which prevent overflow. The economic calculations should support these conclusions.

Table 11 : Rosedale Detention Improvement Discharges (CFS)

	Future without-project	Future with-project	Delta
2-year	282.6	257	25.6
10-year	680.1	445	235.1
25-year	825	580	245
100-year	1086.1	913	173.1

With the decrease in flood discharges from the proposed Rosedale Detention area, flood elevations at critical locations were reduced as shown in **Table 12.**

Table 12 : Rosedale Improvement Flood Elevations (feet NAVD)

		2-year future		10-year future		25-year future		100-year future	
	Location	w/o Project	w/ project	w/o Project	w/ project	w/o Project	w/ project	w/o Project	w/ project
Downstream → Upstream	Barton Chapel Rd U/S	301.43	301.43	304.15	304.15	305.82	305.82	308.13	308.13
	Gordon Highway U/S	286.19	286.19	288.58	288.58	289.25	289.25	289.89	289.89
	Nolan Connector U/S	243.33	243.33	244.72	244.72	245.30	245.30	246.14	246.14
	I-520 (Bobby Jones Expwy) U/S	242.21	242.21	244.25	244.25	244.87	244.87	245.78	245.77
	Rosedale	Dam is approx 1/3 miles down from I-520 and ¼ mile up from North Leg Road							
	North Leg Rd U/S	205.74	205.28	210.56	207.78	212.06	209.36	214.55	212.79
	Wheeless Rd U/S	174.07	174.07	178.08	177.63	178.99	178.38	180.99	180.71
	Dean's Bridge Rd U/S	152.62	152.50	156.39	156.09	157.47	157.10	160.28	159.57
	Mike Padgett Hwy U/S	135.61	135.54	136.13	135.93	136.25	136.08	137.02	136.29

Although the flood reduction improvements for the various combination scenarios are evident based on direct comparisons of water surface profiles, the true evaluation of the resulting benefits can only be seen in the analysis of its economic impact, which is discussed in a separate section.

C-2.7.2 ROSEDALE WETTING DURATION

Although there were not any parameter updates done to the HMS basin model for future (2030) and existing (2004) flow calculations, additional model runs were developed to determine the duration of wetting that could be expected during various hypothetical events. The following hydrographs were calculated utilizing future conditions curve numbers, a 5x5 effective flow box culvert and a 50' overflow weir at an elevation of 232', and a top of dam crest at elevation 240'. A sketch of the proposed structure is shown in **Figure 8**. The impoundment duration summary is shown below in **Table 13,** and the hydrographs are shown in **Figure 9 - Figure 14.** Impoundment durations were calculated using synthetic 24-hour storms, over a 48-hour simulation window to capture the whole hydrograph.

Table 13 : Rosedale Impoundment Duration Summary

Frequency	Hypothetical Event Duration	Peak Inflow (CFS)	Peak Outflow (CFS)	Peak detention elevation (NAVD88-ft)*	Total impoundment duration (hours)
2-Yr	24-hour	286	256	222.5	~18
5-Yr	24-hour	504	371	231.24	~18.5
10-Yr	24-hour	687	442	233.12	~21
25-Yr	24-hour	835	487	233.68	~21
50-Yr	24-hour	976	591	234.15	~21.5
100-Yr	24-hour	1098	666	234.52	~22

* HEC-RAS Elevations

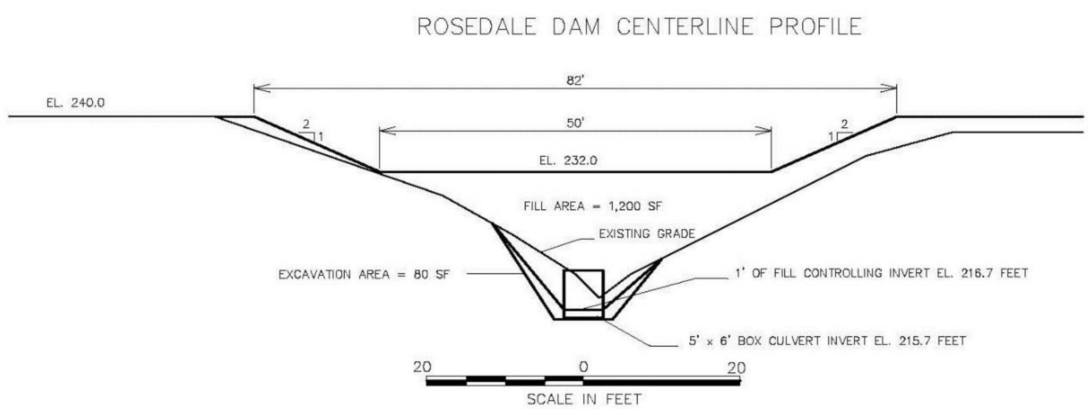

Figure 8 : Dam Profile Sketch

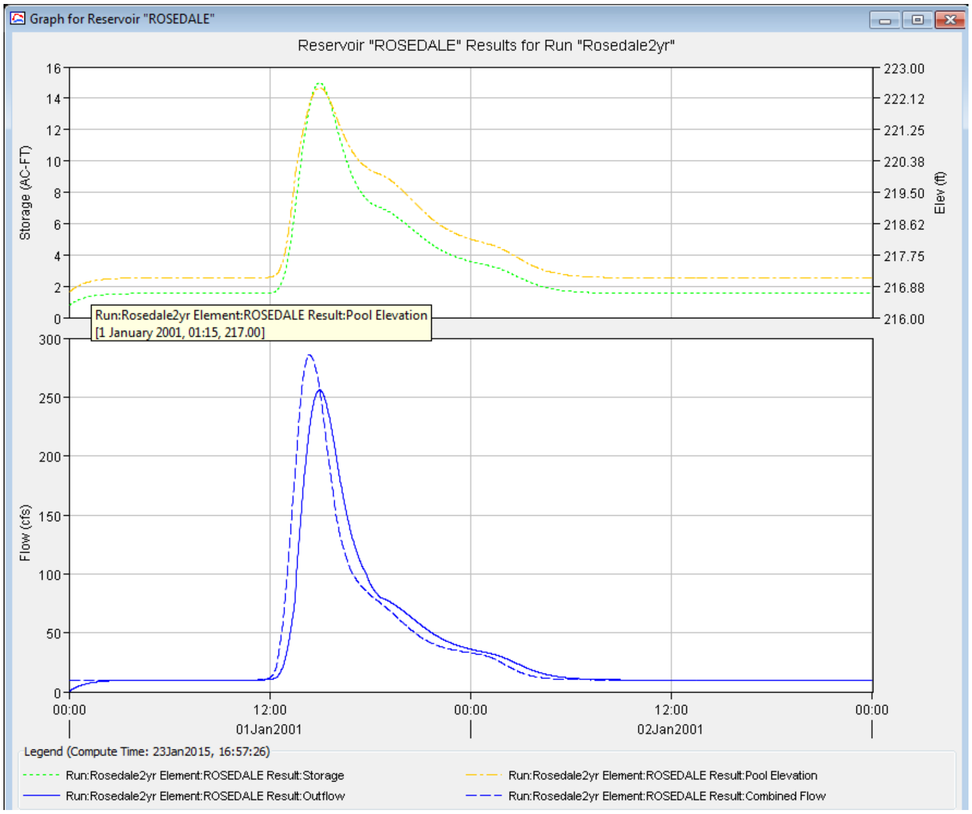

Figure 9 : 2-year Impoundment Hydrograph

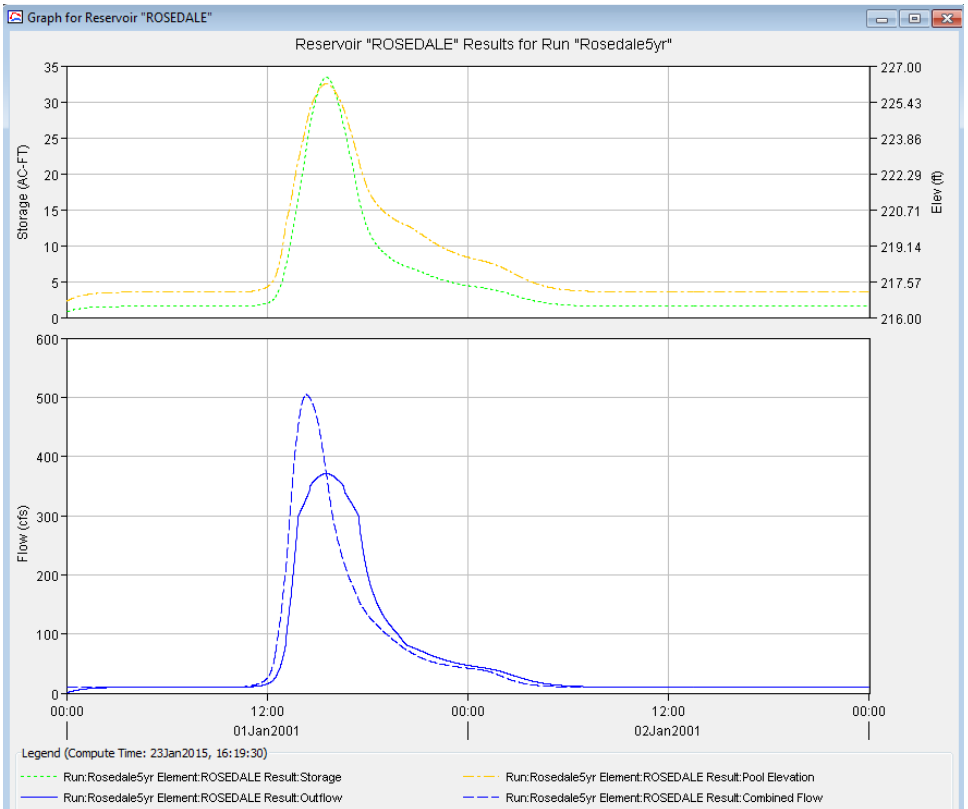

Figure 10 : 5-year Impoundment Hydrograph

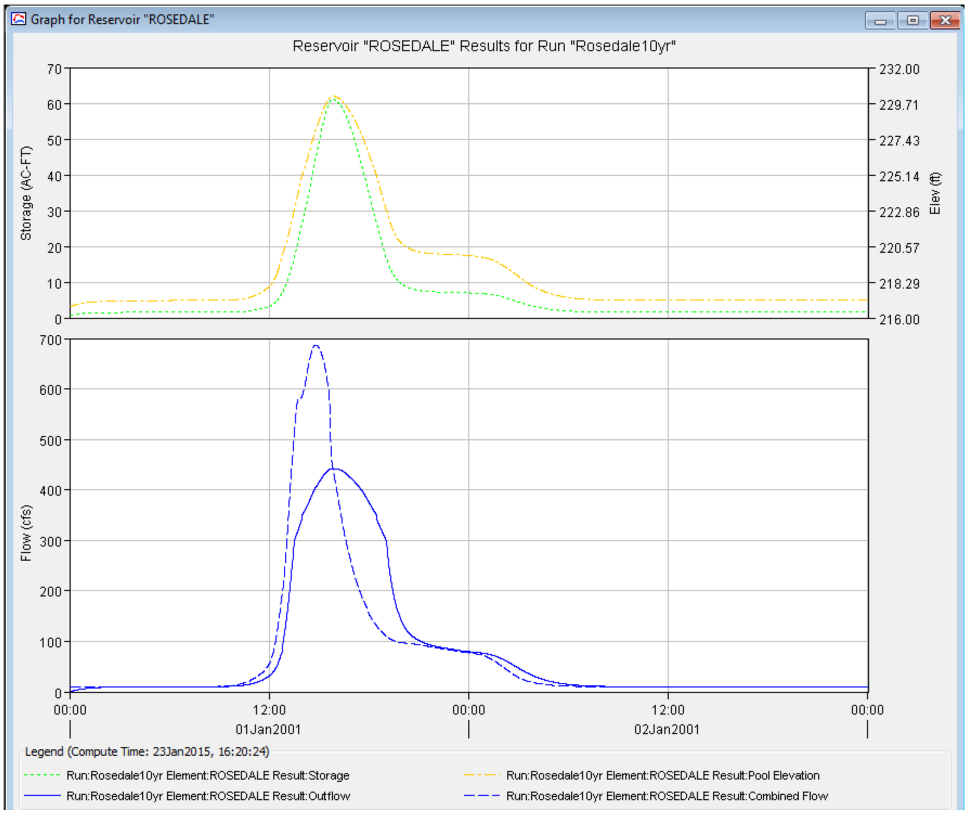

Figure 11 : 10-year Impoundment Hydrograph

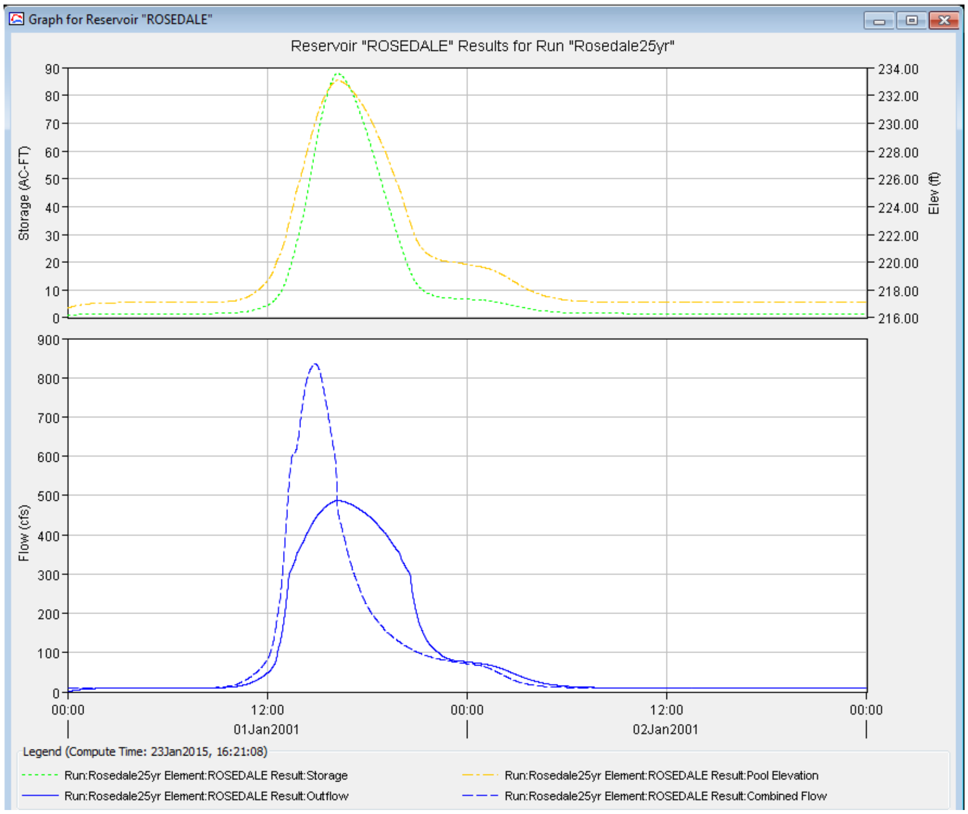

Figure 12 : 25-year Impoundment Hydrograph

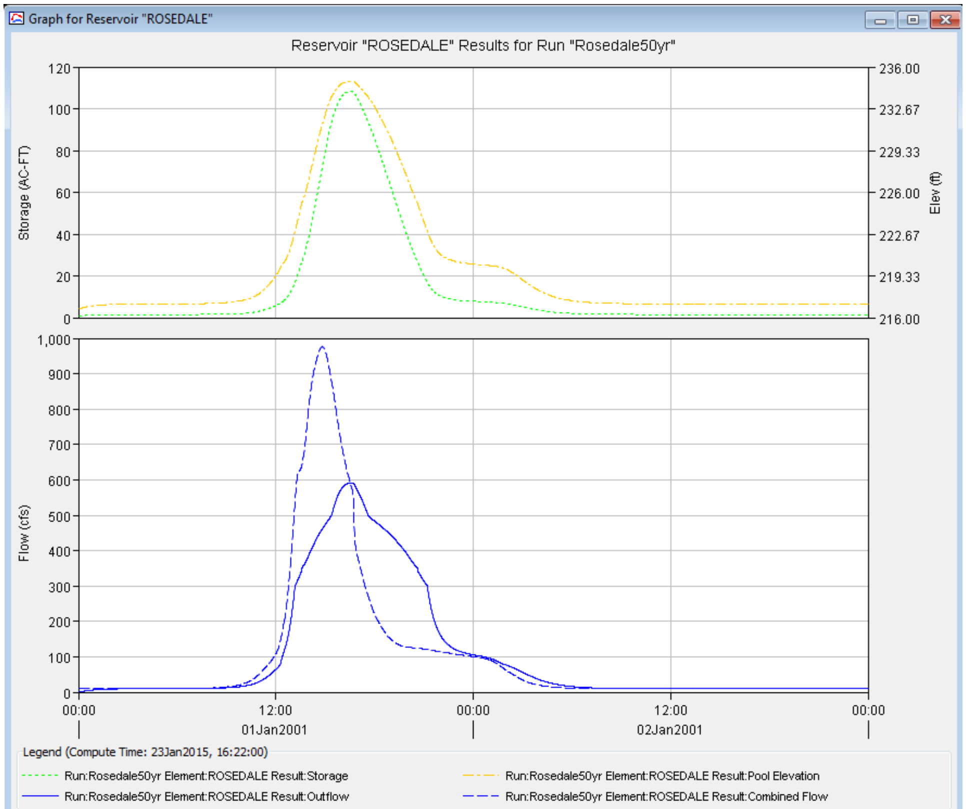

Figure 13 : 50-year Impoundment Hydrograph

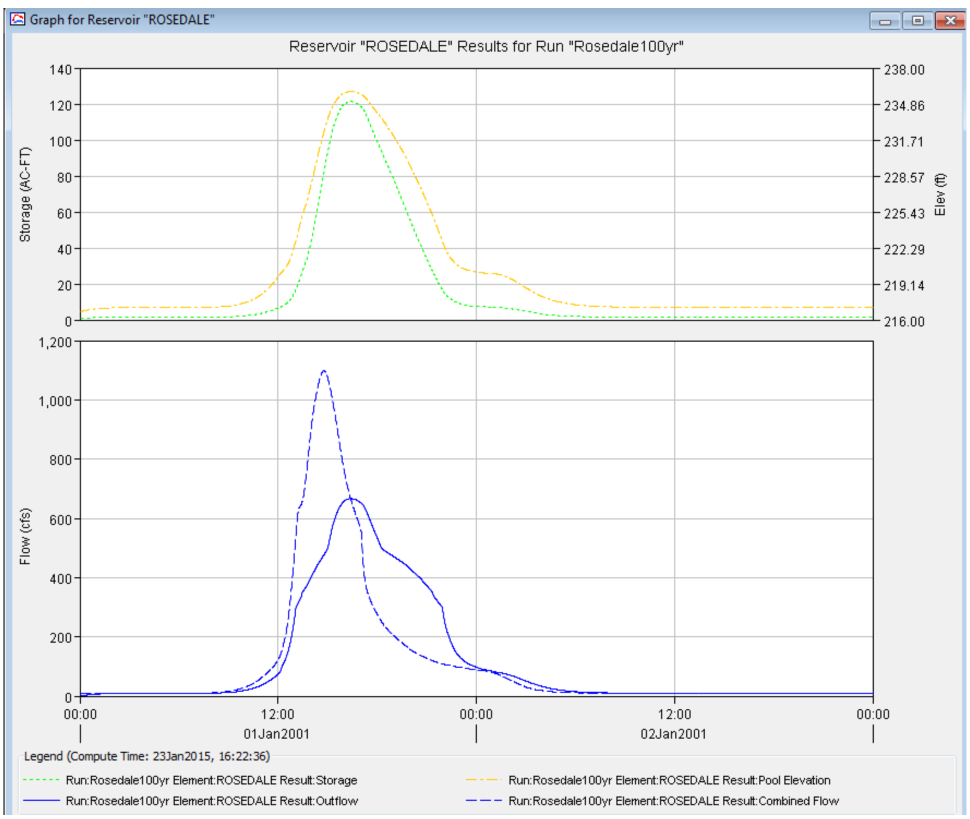

Figure 14 : 100-year Impoundment Hydrograph

C-2.7.3 PROFILE PLOTS

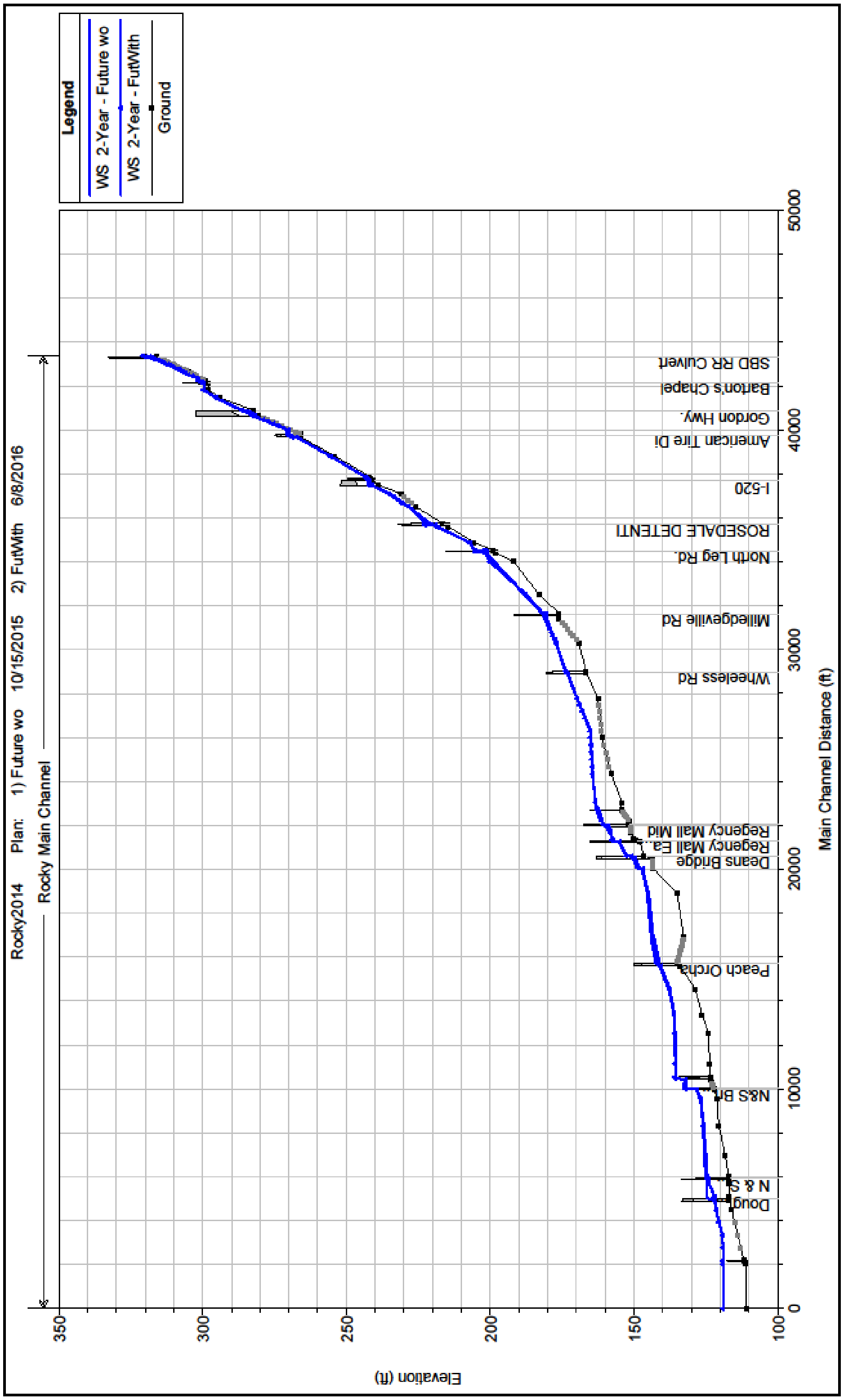

Figure 15 : 2-Year with and without Profiles

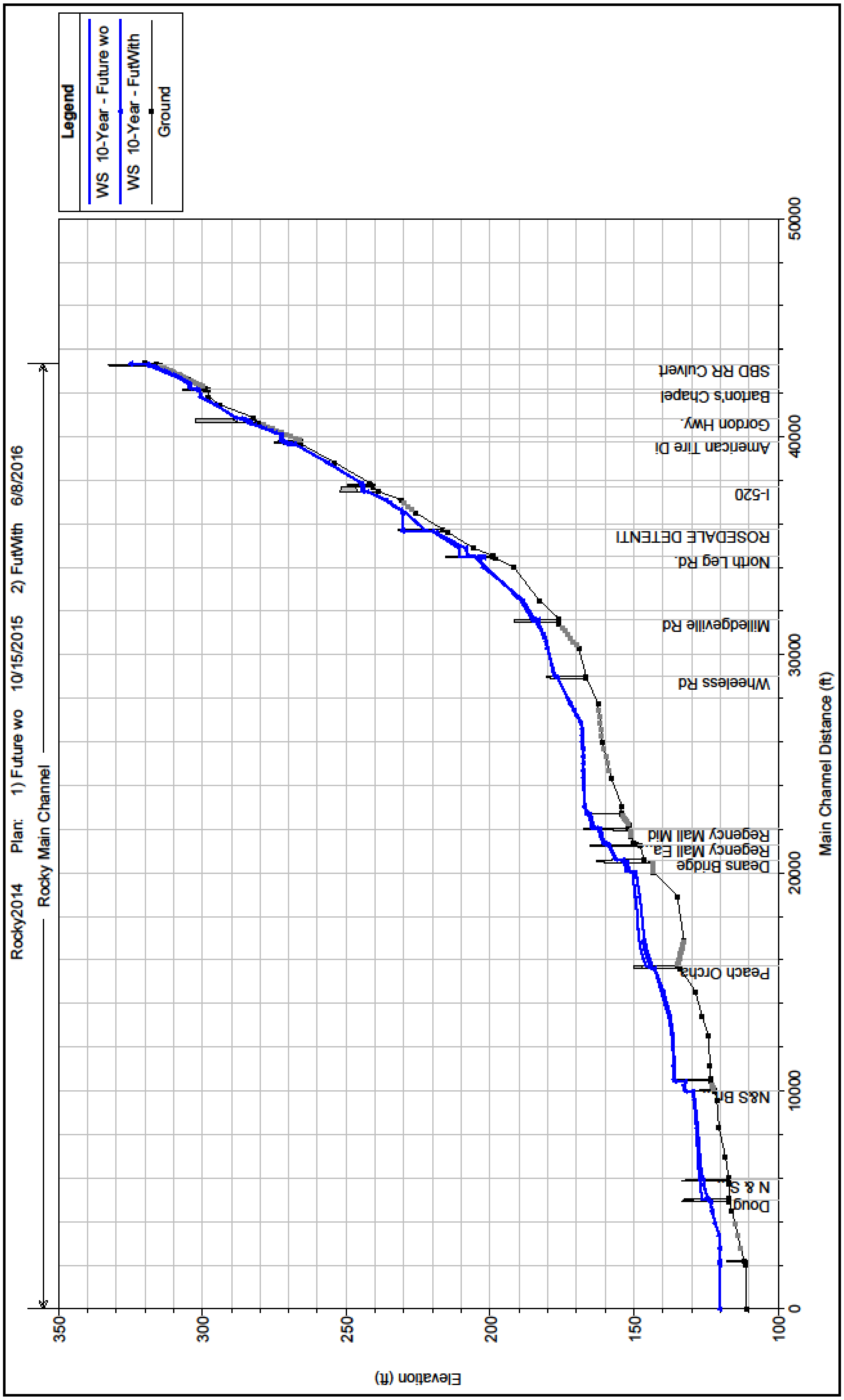

Figure 16 : 10-Year with and without Profiles

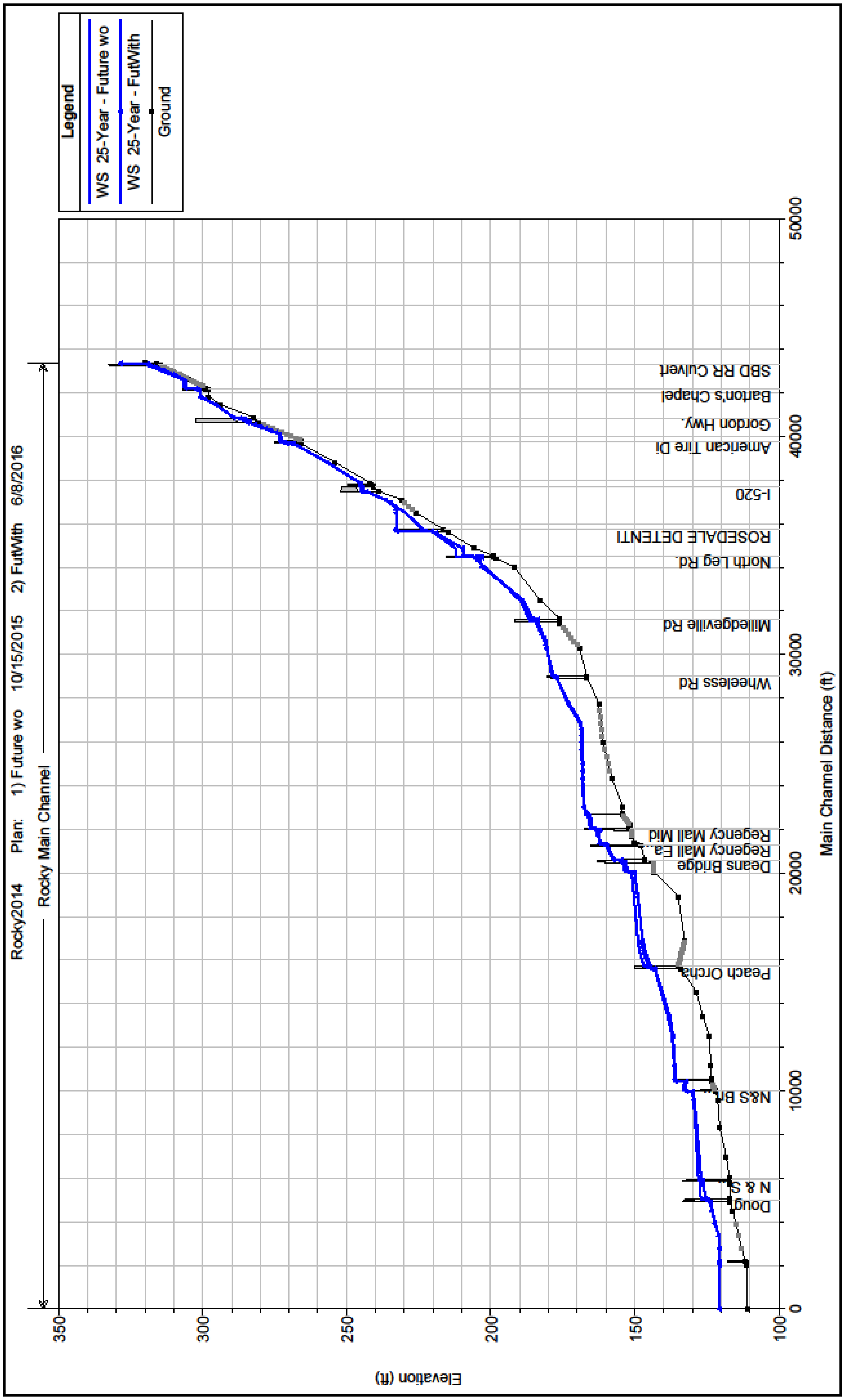

Figure 17 :25-Year with and without Profiles

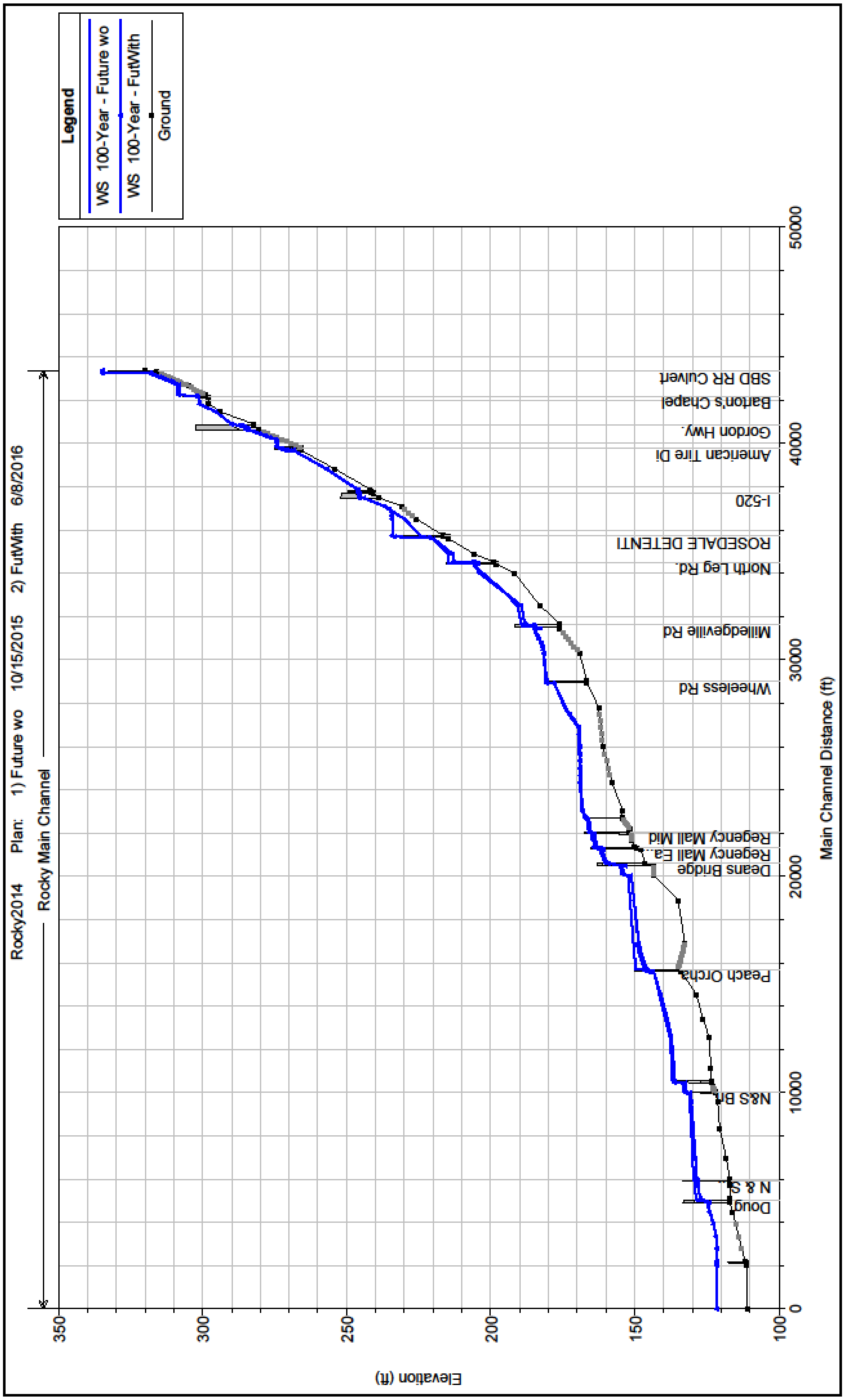

Figure 18 : 100-Year with and without Profiles

C-2.7.4 IMPOUNDMENT MAPPING PLOTS

Shown below in **Figure 19** is the amount of ponding that can be expected behind the structure during a 100-year flood event. Inundation limits below the dam were not mapped. At the deepest portion of the pond, the upstream toe of the structure, the water surface elevation will increase from approximately 224.5'. to 234.5'. Shown in **Figure 20** is the 500-year mapping and 100-year mapping, illustrating the minimal difference between the two.

Figure 19 : 100-YR Impoundment Extents

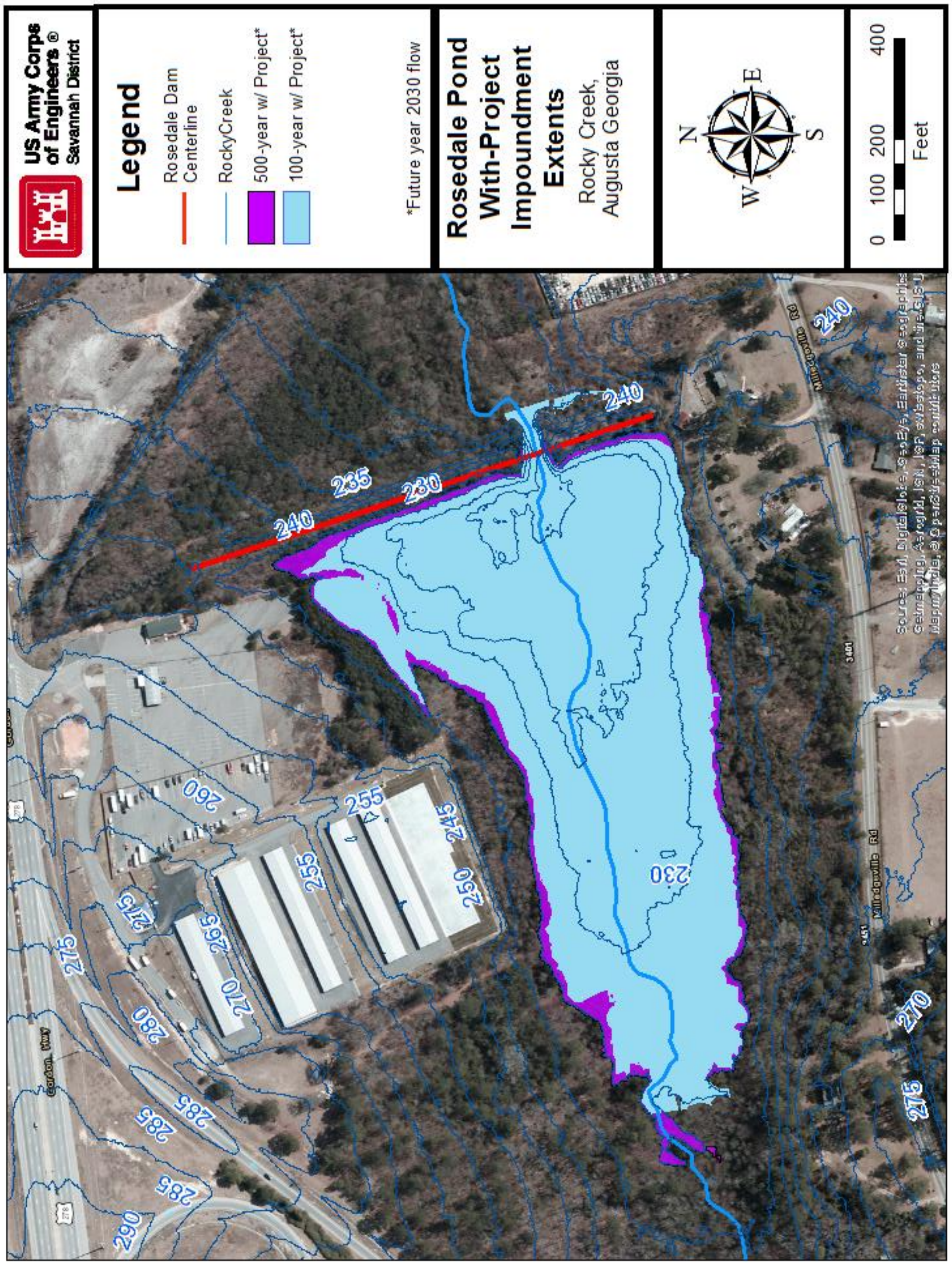

Figure 20 : 500-YR Impoundment Extents

C-2.7.5 REAL ESTATE SUMMARY

Areas behind the dam that are going to be inundated at various event levels must have real estate easements purchased from the parcel owners. A detailed evaluation of these takings can be found in the Real Estate Appendix. A brief summary of impacted parcels can be seen below in **Table 14** and **Figure 21.**

Table 14 : Parcel Easement Areas

Impacted Parcel	Total Area (AC)	100-yr w/o Area (AC)	100-yr w Area (AC)	Increase (AC)
680029000	6.34	0	0.61	0.61
680030000	10.18	4.48	7.73	3.25
694001000	4.12	1.29	1.96	0.67
691012000	10.58	.08	0.41	0.33
690015000	6.5	1.45	2.45	1

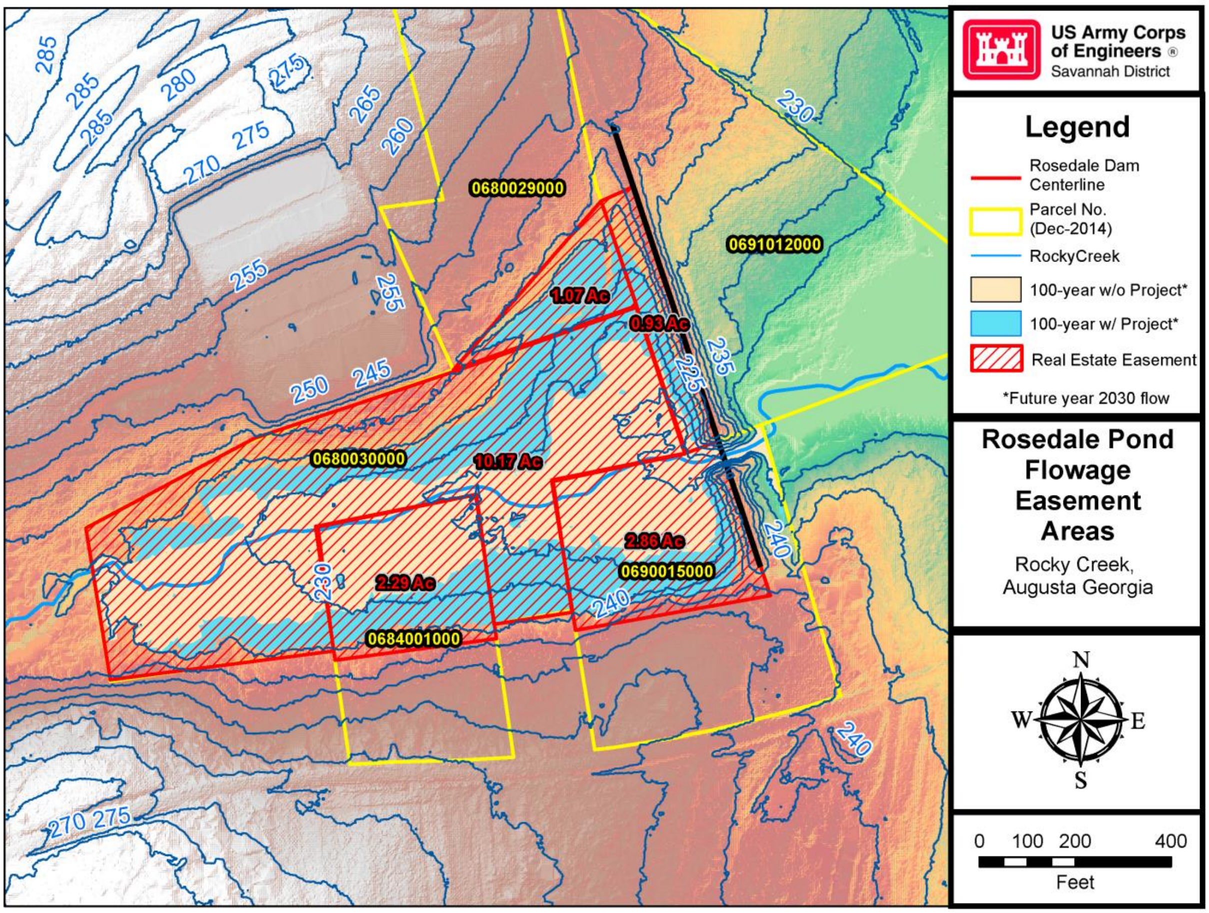

Figure 21 : Flowage Easements

C-3. SURVEYING, MAPPING, AND OTHER GEOSPATIAL DATA

Aerial photography was flown in 2000, digital orthophotos were produced, and 1-foot interval topographic contours developed for the lowland portion of Rocky Creek and Phinizy Swamp. Cross-sections and stream structures were surveyed in January 2001 by Continental Aerial Surveys, Inc., all under contract DACW21-98-D-0017. The Savannah District in-house survey crew surveyed first floor elevations of all structures in the Rocky Creek 500 year flood plain. The GIS database developed and maintained by the City of Augusta, which contains data on topography, structure location, vegetation, roads, etc., was used as the base information for the flood maps and concept design layouts. The additional topographic and structure elevation data collected as part of this study will be added to the GIS database and provided back to Augusta – Richmond County for their future use.

Terrain data was also updated and validated using new LiDAR data. Initially, Army Geospatial Center data (AGC) was used. New cross sections were cut and compared to the current model. Some of the sections were very similar in shape, but not in absolute magnitude. Some sections and top-of-road data was close, but some were off by 5ft +/-. High accuracy overbank data is important for mapping and accurate water surface profile computations. Published benchmarks in the domain of the dataset were analyzed and compared to the data. It became clear that the errors were not systematic errors, such as a datum conversion, but simply low quality data collection techniques, resulting in random error within every data point. The AGC was contacted, and noted that errors of +/- 3 feet were not unheard of, and that the terrain was better used for other purposes where this level of error was not as critical as H&H applications.

An alternate source of LiDAR terrain data was located in the USACE-SAS database. The data was collected for a GADNR project in 2012. The point cloud was processed for the Rocky Creek project area, and a DEM was created at a resolution of 3.28ft grid cell resolution, in NAD_1983_StatePlane_Georgia_East_FIPS_1001_Feet datum. The data had previously undergone rigorous QA/QC from the data collection contractor. However, given the problems with the AGC dataset, it was also compared to benchmarks and to existing model cross sections. The standard error was within ~.1 ft +/-, increasing confidence in this dataset for use. An overview of the terrain data used is shown in **Figure 22.**

In accordance with SMART planning guidance, the data that was used consisted predominately of readily available data. No additional survey grade data was collected as part of this Section 205 study. USACE conducted field reconnaissance to assure that all of the structures in the old model were ground truthed to make sure they were still in place and there were no large obvious discrepancies. Tape down measurements were also taken at any new structures that have been constructed since the 2004 model.

Given the availability of two separate LiDAR datasets to choose from, the ability to compare the LiDAR to a maintained benchmark database published by the National Geodetic Survey, and the ability to compare cross sections to the old model (which contained some surveyed sections), there is a high level of confidence in the terrain data used. This data is considered fit for this level of hydraulic analysis, and is not a significant source of uncertainty in the hydraulic analysis.

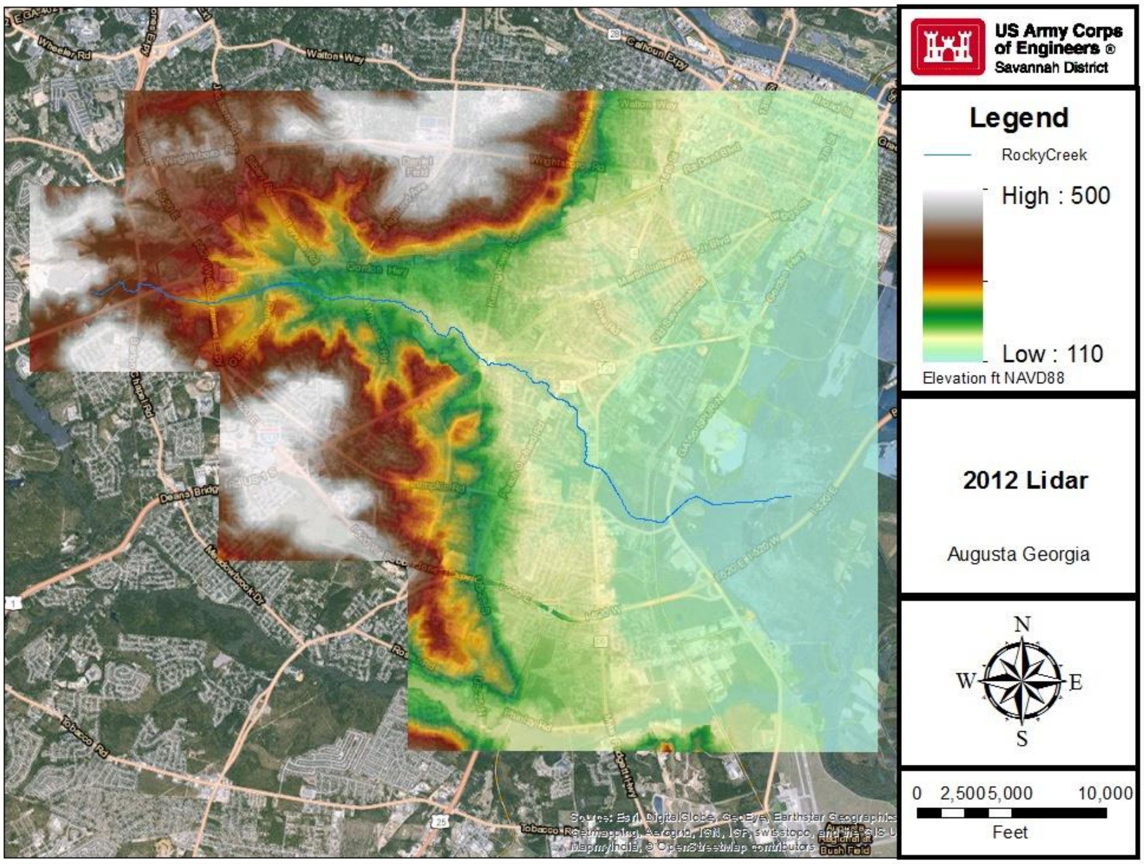

Figure 22 : LiDAR

C-4. GEOTECHNICAL

C-4.1 DESIGN REQUIREMENTS:

In 2002, Savannah District Geotechnical and HTRW Branch performed subsurface exploration and prepared a geotechnical assessment of soil conditions for a number of the alternative project sites identified at that time. Standard Penetration Test borings were drilled at the proposed locations of Lombard, Rozella, Wheeless, and Noland Connector detention basin. However, none of these sites were selected for construction. Those boring locations, drilling log sheets and approximate soil profiles can be found in the 2004 Engineering Appendix. In 2009, Savannah District Geotechnical and HTRW Branch mobilized to the proposed Rosedale Detention Structure location to perform subsurface exploration for geotechnical assessment. Presented in this report are the results of the field and laboratory investigation. The geotechnical information obtained regarding site and soil conditions were used to determine the retaining structure type and size and estimate material quantities for a rough order magnitude cost estimate.

C-4.2 SITE GEOLOGY:

The headwaters of the Rocky Creek basin start in the southeast edge Piedmont area of Georgia. The basin ends in Phinizy Swamp which is in the northwestern edge of the upper Coastal Plain area of Georgia.

The Fall Line is the boundary between the Piedmont and the Coastal Plain. Its name arises from the occurrence of waterfall and rapids that are the inland barriers to navigation on Georgia's major rivers. The Fall Line is a boundary of bedrock geology, but it can also be recognized from stream geomorphology. Upstream from the Fall Line, rivers and streams typically have very small floodplains, if they have any at all, and they do not have well-developed meanders. Within approximately a mile downstream from the Fall Line, rivers and streams typically have floodplains or marshes across which they flow, and within three or four miles they meander. The most pronounced example of this is in the Savannah River's course at Augusta.

The Coastal Plain is a region of Cretaceous and Cenozoic sedimentary rocks and sediments. These strata dip toward the southeast, and so they are younger nearer the coast. At least near the Fall Line, they are ultimately underlain by igneous and metamorphic rocks like those of the Piedmont. The sedimentary rocks of the Coastal Plain partly consist of sediment eroded from the Piedmont over the last 100 million years or so, and partly of limestone generated by marine organisms and processes at sea. One could generalize that buried Triassic rocks in the subsurface are various rift-basin siliciclastics, the Cretaceous strata are sandstones and shales, the Tertiary strata are limestones and shales, and that the Quaternary strata are sands and muds.

The outcrops near the Phinizy Swamp area are mostly Quaternary alluvium composed of unconsolidated sand and gravel located primarily on the river's flood plain. Underlying the alluvium are sediments of Cretaceous to Eocene in age. They are dominantly terrestrial to shallow marine in origin and consist of sand, kaolinitic sand, kaolin, and pebbly sand. The sediments are underlain by metamorphic and igneous rocks including granite, biotite gneiss, granite gneiss, and amphibolite.

C-4.3 ROSEDALE SUBSURFACE EXPLORATION

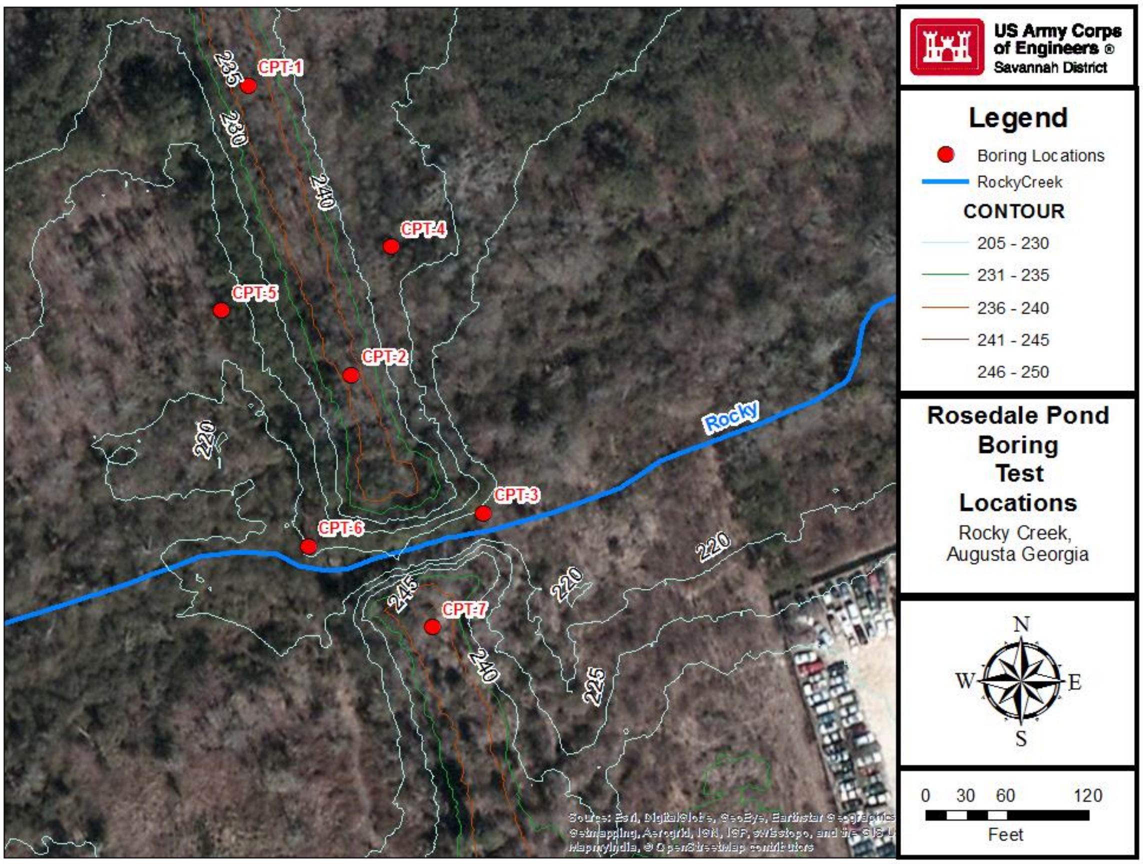

Figure 23 : Boring Locations (2009)

In 2002, the Rosedale detention area was unable to be tested for soils due to right of entry obstacles. In 2009, Savannah District Geotechnical and HTRW Branch completed seven Cone Penetrometer Tests (CPT) and obtained soil samples for lab analysis. The locations for the CPT tests are shown in **Figure 23**. Grain Size Distribution analysis, gradation curves, and liquid limit/plastic limit tests were performed at the Environmental Testing Unit lab in Marietta, Georgia. The results from these tests are attached in section **C-12. GEOTECH EXPLORATION RESULTS .**

C-4.4 BORROW/DISPOSAL SITES

Based on the geotechnical assessment it is anticipated that borrow materials will be required for construction of the new Rosedale storm water detention structure. These materials will come from required excavations on-site and also from City/County owned borrow sources. Haul distances for borrow and disposal are assumed to be between five and ten miles.

C-4.5 SLOPE STABILITY AND SEEPAGE

The Rosedale detention basin embankment was analyzed and designed for slope stability in accordance with EM 1110-2-1902. The factor of safety for slope stability was higher than the minimum requirements identified.

The detention area does not hold a permanent pool, as such a transient analysis was utilized for seepage and embankment design in accordance with EM 1110-2-1901.

EM 1110-2-1902
31 Oct 03

Table 3-1
Minimum Required Factors of Safety: New Earth and Rock-Fill Dams

Analysis Condition[1]	Required Minimum Factor of Safety	Slope
End-of-Construction (including staged construction)[2]	1.3	Upstream and Downstream
Long-term (Steady seepage, maximum storage pool, spillway crest or top of gates)	1.5	Downstream
Maximum surcharge pool[3]	1.4	Downstream
Rapid drawdown	1.1-1.3[4,5]	Upstream

[1] For earthquake loading, see ER 1110-2-1806 for guidance. An Engineer Circular, "Dynamic Analysis of Embankment Dams," is still in preparation.
[2] For embankments over 50 feet high on soft foundations and for embankments that will be subjected to pool loading during construction, a higher minimum end-of-construction factor of safety may be appropriate.
[3] Pool thrust from maximum surcharge level. Pore pressures are usually taken as those developed under steady-state seepage at maximum storage pool. However, for pervious foundations with no positive cutoff steady-state seepage may develop under maximum surcharge pool.
[4] Factor of safety (FS) to be used with improved method of analysis described in Appendix G.
[5] FS = 1.1 applies to drawdown from maximum surcharge pool; FS = 1.3 applies to drawdown from maximum storage pool.
For dams used in pump storage schemes or similar applications where rapid drawdown is a routine operating condition, higher factors of safety, e.g., 1.4-1.5, are appropriate. If consequences of an upstream failure are great, such as blockage of the outlet works resulting in a potential catastrophic failure, higher factors of safety should be considered.

Table 15 : Minimum Factor of Safety

C-5. ENVIRONMENTAL ENGINEERING

The majority of the environmental enhancement features were in the form of channel improvements and restoration measures that have been eliminated as alternatives.

The box culvert that would be installed as part of the Rosedale detention structure would be buried 1 foot below grade to avoid the potential for scouring of the channel bottom along the edge of the culvert that would create a barrier to wildlife passage through the culvert. The required conveyance area is 25 square feet, which is accomplished with a 5x5 culvert. However, with the invert being buried, the culvert will need to be 5'x6' to achieve the required flow.

Rock cross vanes were part of the channel improvement alternatives that were previously evaluated and eliminated. The proposed detention structure is not intended to change (increase or decrease) typical daily stream flows. A stone/rip rap apron will be included at the discharge point of the culvert to reduce scour potential and protect the structure from undermining.

C-6. CIVIL DESIGN ROCKY CREEK PROJECT FEATURES

C-6.1 DESIGN REQUIREMENTS

The proposed project features presented in this section are limited to the concept level of the Rosedale Detention area. Prior alternatives in the 2002 Feasibility Report have been eliminated from consideration, such as retaining structures, culverts, and channel improvements. The designs were developed to a sufficient level that cost could be reasonably estimated. This section discusses all structural features considered for Rocky Creek. Non-structural features are discussed in section C-7.

C-6.2 OUTLET DISCHARGE VELOCITIES

HEC-RAS model output data from the future conditions with-project plan were used to determine a range of expected discharge velocities from the box culvert at Rosedale. Culvert discharge flows for each simulated event were taken from the model, and the Hazen-Williams friction loss method was used to predict velocities. The Hazen-Williams method is valid for water flowing at ordinary temperatures of 40 to 75 ^{o}F through pressurized pipes. Therefore, this approximation is valid when the culvert is submerged on the upstream side and acting as orifice flow, at the 10-year event and higher. The Hazen Williams equation is shown below.

$V = k*C*(D/4)^{0.63}S^{0.54}$

Where
k = conversion factor for English units = 1.318
C = Hazen Williams roughness coefficient for concrete pipes = 130
D= equivalent circular diameter = 5.64’
S = energy slope = h_f/L
L = pipe length = 150’

Table 16 : Outlet Velocity

Frequency	HEC-RAS flow through culvert (CFS)	Hazen Williams velocity (ft/s)	Hazen Williams head loss (ft)
25-Yr	483	19.3	1.77
50-Yr	495	19.81	1.85
100-Yr	502	20.1	1.9
250-Yr	510	20.41	1.95
500-Yr	517	20.7	2.00

C-6.3 ROSEDALE DETENTION STRUCTURE

The Rosedale Dam embankment is located along Rocky Creek between Milledgeville Road and Gordon Highway upstream of North Leg Road. Many years ago the owners deliberately breached the dam in a controlled manner at the approximate location of the creek channel. It is understood that after the owners were made aware of deficiencies regarding insufficient/undersized outlet works a decision was made to breach the dam instead of making repairs to bring the in-place outlet works into compliance with the current dam safety regulations at the time. The remaining embankment is approximately 800 linear feet in length. The existing crest width is approximately 15-20 feet and the height of the embankment is approximately 20-25 feet. Results of the CPT soundings and laboratory tests indicate that the top 5-10 feet of the existing embankment is constructed of predominantly sand with the rest of the embankment consisting of clays and clayey silts. The CPT results also indicate that the embankment is founded on medium to dense silty sands and very stiff fine grained soils. The renovations proposed include placing a reinforced concrete pipe or box culvert through the breach in the embankment, at the location of the creek bed for normal flow. The culvert invert will be at an elevation of 215.7’, with 1’ of backfill to minimize biological impacts. The controlling hydraulic elevation will be 216.7’. The breach will then be filled to elevation 232.0 to form a notch for all flows between the 10- and 500-year flood events. The bottom width of the overflow notch will be 50’, and the tope width will be 82’. The side slopes will be at 2:1. At no time should the entire structure be overtopped. The entire structure will require clearing and grubbing and establishment of grass cover. A plan view of the existing dam and proposed modifications and a profile of the dam are shown on **Figure 24** and **Figure 25** respectively.

The majority of the existing embankment will be deconstructed and reconstructed according to USACE publication ER_1110-2-1156: Engineering and Design, Safety of Dams Policy and Procedures. Unsuitable material will be disposed of, and suitable material will be reused. Additional fill will be brought in to replace unsuitable material. These quantities estimates are reflected in C-6.4 Quantity Estimate Summary.

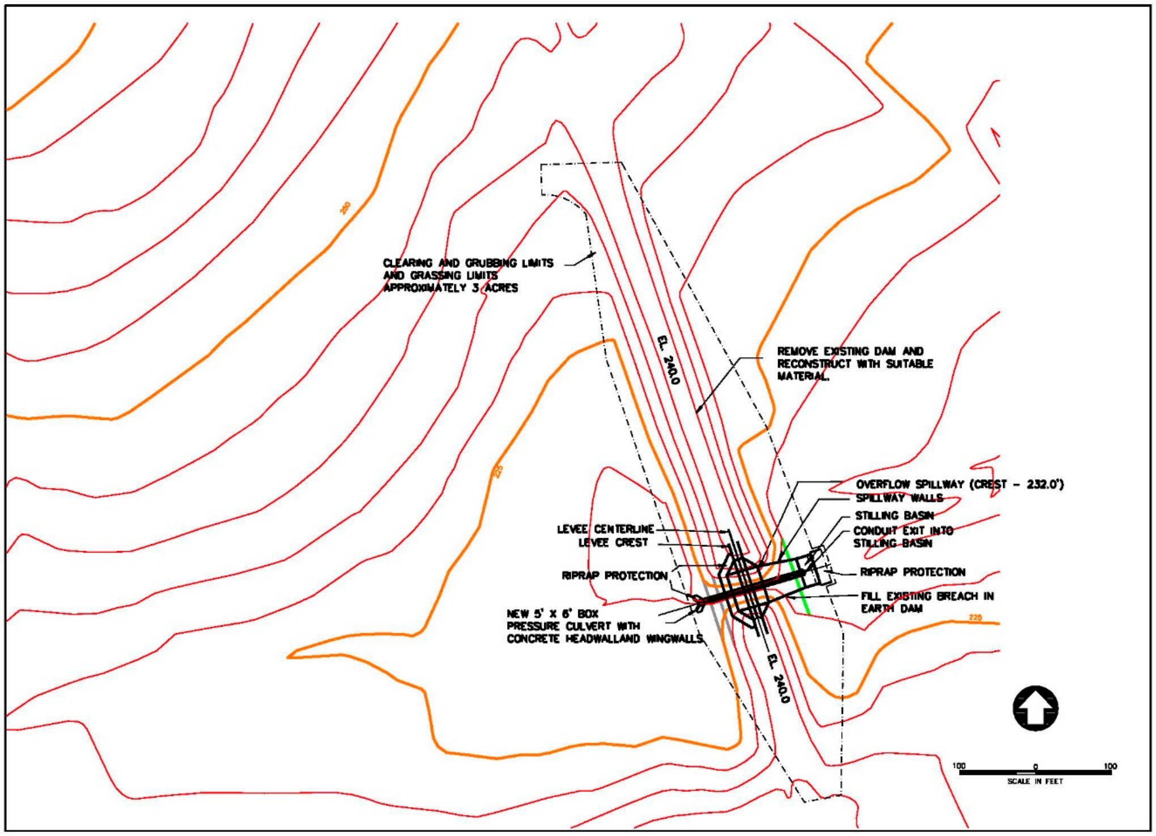

Figure 24 : Rosedale Detention Structure Renovations

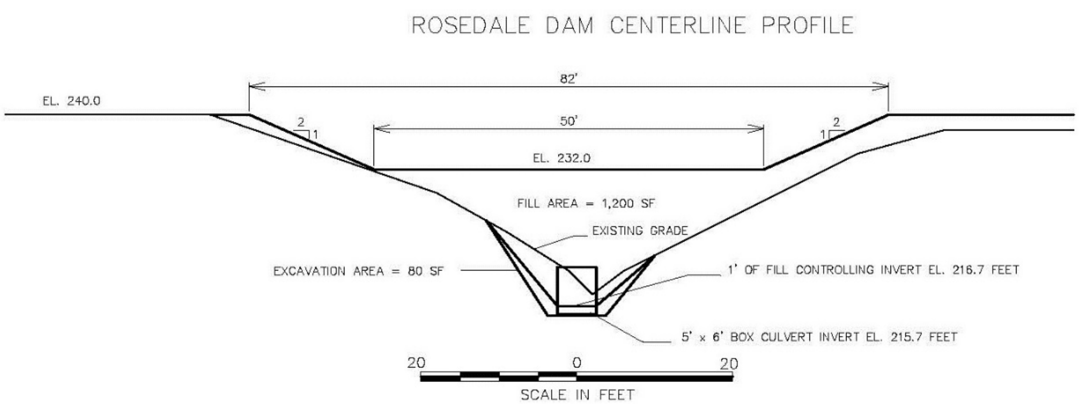

Figure 25 : Rosedale Center Line Dam Profile

C-6.4 QUANTITY ESTIMATE SUMMARY

Rosedale Dam is an existing earth dam that was breached at the creek channel many years ago. The renovations proposed include placing a reinforced concrete pipe or box culvert through the breach in the embankment, at the location of the creek bed for normal flow. The breach will then be filled to elevation 232.0 to form an overflow weir for all flows from the 10 to the 500-year flood event. The crest and downstream slope at the notch will be protected from erosion with articulated concrete blocks (ACB) slope protection or cast in place concrete. The entire structure will require clearing and grubbing and establishment of grass cover.

Clearing and Grubbing:

Clearing and grubbing will include trees of all sizes (up to 40-inch diameter) and woody vegetation. Clearing and grubbing will occur within the footprint of the new embankment, as well as area as required for ingress and egress.

Stripping & Hauling:

The area is heavily wooded and vegetated. Stripping and hauling quantity of material estimates are assumed to fairly high due to dense vegetation.

Excavation:

Common excavation quantities were estimated using readily available topographical data and concept design parameters discussed within this document. The entire existing embankment will be excavated and rebuilt, to assure structure stability. Cone Penetrometer Test results indicate that approximately 20% of excavated material will be suitable for reuse.

Dewatering/Diversion of Water:

During construction, assume temporary coffer dikes will be built both upstream and downstream of the existing breach and tied to the embankment at both ends. The common existing low flow rate is approximately 25-40 CFS. The existing creek flow can be pumped around the dam during construction. Within the fill placement area, water can be controlled by temporary ditches and sumps. Water from sumps will be pumped downstream of construction area. The volume of material used to construct coffer can later be used as fill in the permanent construction once the fill is several feet above the new RCP.

Reinforced Concrete Pipe, Wing Walls, and Slabs:

The design incorporates a reinforced 5' X 6' box culvert. New concrete wing walls will be required on both ends of the culvert. Wing walls can be precast or cast-in-place. A concrete apron/slab will also be required between the wing walls.

Earthwork:

Backfill will be placed and compacted in layers to 95% standard proctor density. Spreading and compaction will require both conventional earthwork equipment and hand placement and compaction around the RCP. Moisture control will be required. Compaction of the surface of the entire dam will be required after clearing and grubbing is complete and prior to seeding. Suitable material from the construction of the coffer dike can be included in the quantity.

Outfall Protection:

A stilling basin with riprap protection will be placed at the downstream toe of the emergency spillway and at the outfall of the concrete box culvert to prevent scour and undercutting.

Geotextile:

Geotextile will be required beneath the concrete spillway and between the riprap and existing ground.

Reinforced Concrete Spillway

The reinforced concrete spillway area as described in the concept design is assumed to be 12” thick. The concrete spillway will be cast in place concrete.

Topsoil, Grassing, Mulching, Fertilizing:

Topsoil will be stockpiled separately from other excavation (but is included in excavation volume). Topsoil will be considered the top four inches of existing grade. Topsoil placement will only be required in areas of fill placement. All disturbed areas will be grass seeded, fertilized, and mulched. There will be no topsoil or grassing required inside the dry impoundment area.

Maintenance:

Regular maintenance will include items such as mowing, reseeding, and minor earthwork to repair rutting and erosion as needed. Vegetation removal and herbicide application within the riprap outfall protection will be required. Inspections of the embankment should be scheduled periodically and should also occur after large rain events.

C-7. NON-STRUCTURAL FEATURES

The only nonstructural feature proposed for Rocky Creek are home buyouts at Kissingbower, near the Regency Mall. This feature provides for the removal of five residential buildings within the existing floodplain. See the Economics Appendix for the full description of this feature, with a full analysis of benefits, costs and B/C ratio. A general vicinity location map of the parcels, with the existing FEMA 100-year floodplain can be seen in **Figure 26.**

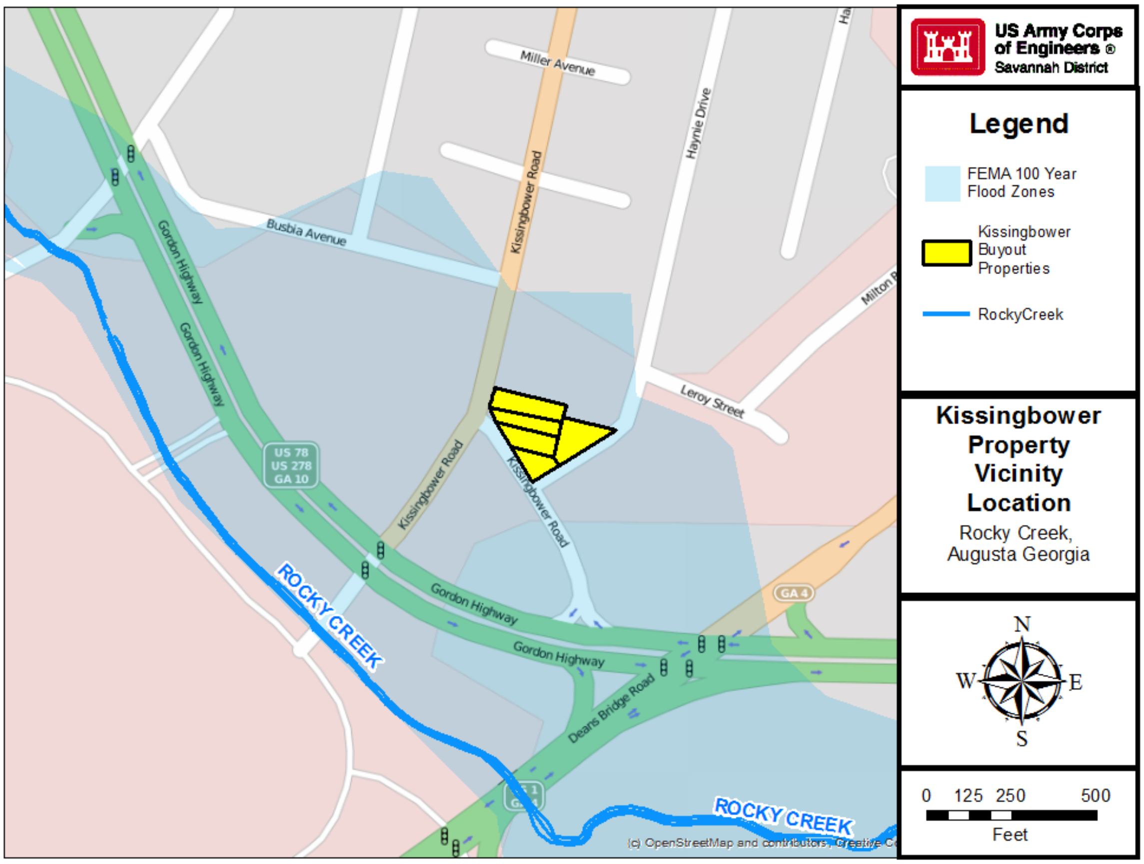

Figure 26 : Kissingbower Vicinity

	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	250-yr	500-yr
Future with Project	155.1	157.12	158.46	159.37	160.78	161.49	163.82	164.19
Future Without Project	155.29	157.45	158.87	160.22	161.06	162.33	164.09	164.45
Existing	154.51	156.72	158.23	159.25	160.42	161.16	163.31	163.56

Table 17: Water Surface Elevations at Kissingbower[3]

[3] HEC-RAS Station 23210

A zoomed in view of the five parcels, with parcel numbers can be seen in **Figure 27**. HEC-RAS river stationing is also shown for reference. Kissingbower is approximately 700 feet south of the Regency Mall Middle Entrance, and 500 feet north of the Dean's Bridge Road crossing. Rocky Creek is about 300 feet to the west, other the other side of Gordon Highway. See **Table 18** for a list of parcels and addresses. Full appraisals can be found in the Real Estate Appendix.

Table 18 : Kissingbower addresses and parcel names

Parcel Number	Address	Parcel Acreage
086-1-023-00-0	1956 ½ Kissingbower Rd	.2
086-1-022-00-0	1958 Kissingbower Rd	.22
086-1-024-00-0	1956 Kissingbower Rd	.27
086-1-020-00-0	1957 Haynie Dr	.28
086-1-021-00-0	1960 Kissingbower Rd	.16

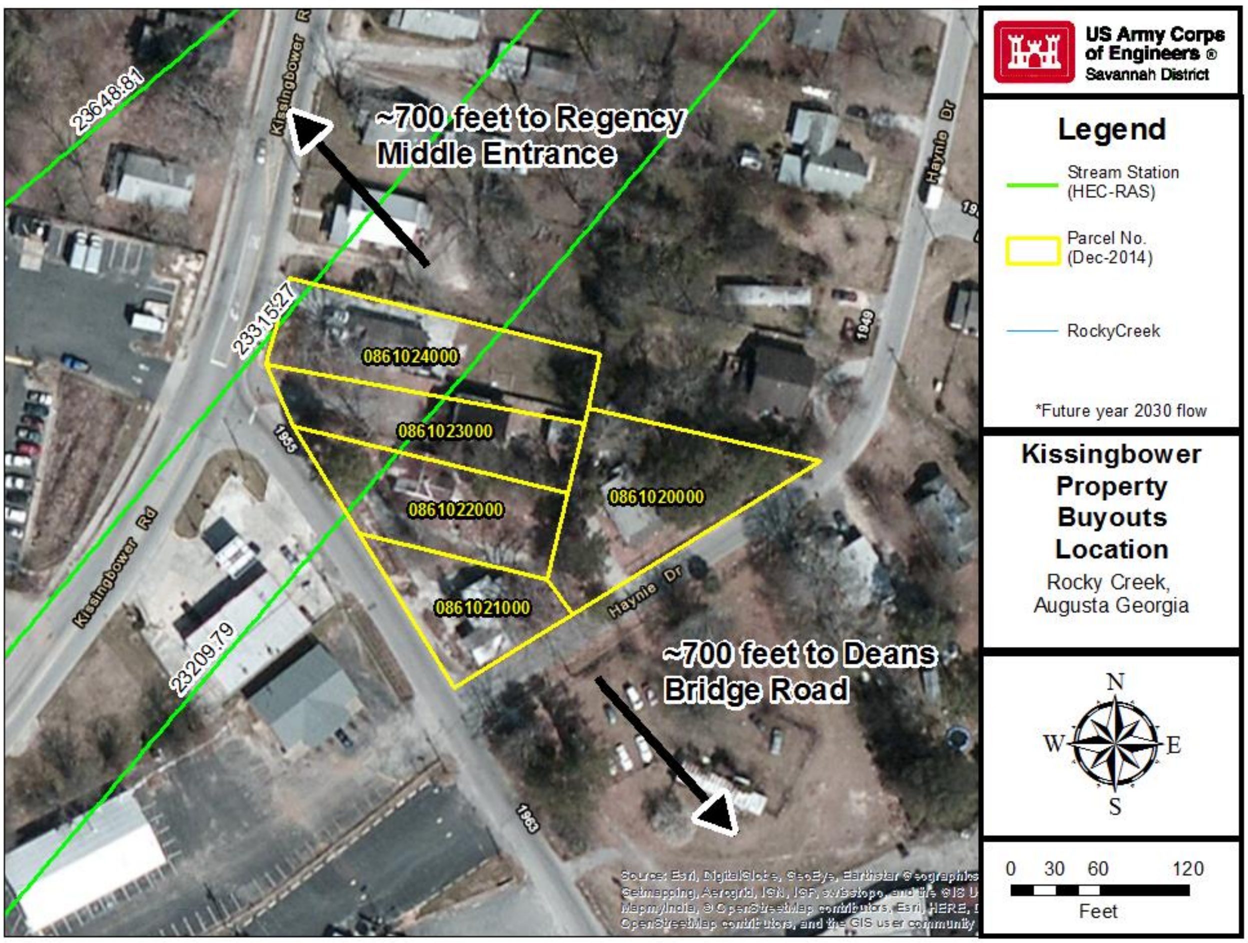

Figure 27 : Kissingbower Park Parcels

C-8. HAZARDOUS AND TOXIC MATERIALS

A historical database search was done to determine whether an expectation of contamination existed for the planned construction areas of the Augusta Flood Control Project. The database search showed no major historical factors, but several possible minor contamination issues. Based on these issues, as well as, an inclusive site visit, it was determined that extensive sampling along the planned Rocky creek detention pond area should be conducted. Analytical results indicated that no contamination exists that should interfere with planned construction activities. Therefore, it was recommended that flood control activities should continue as planned. Please refer to the "Environmental Assessment Augusta Canals" report sections 3 & 4 for summaries of hazardous waste issues. The HTRW report will be made available upon request.

C-9. OPERATION AND MAINTENANCE

In 2002, the design team, with input from the local sponsor on some issues, analyzed each project feature and determined what would be their individual operation and maintenance requirements as well as what would be the frequency of maintenance. A full matrix of O&M requirements can be found in the 2004 Engineering Report. In the case of the Rosedale detention structure, it was estimated that the following maintenance was required

- Mowing of 6.5 acres x 7 times per year
- Debris removal of 10 cy per year
- Erosion repair @ 50 sq yard seeding and 15 cy soil per 5 years

The cost engineer estimated the annual costs of these requirements, as well as contingency and construction management, the estimated cost of O&M was approximately 10,000 $ /year in 2002 These costs were not escalated to 2015 dollars.

C-10. HISTORICAL PHOTOGRAPHS

The following photographs are scanned images from USACE archives. They are associated with a Phase I Inspection Report as part of the national dam safety efforts conducted in the late 1970's and early 1980's. These photographs are from prior to the designed breach, and still show the spillway and low level control structures.

Photo 1: Overview from right side of Reservoir

Photo 2 : Dam Crest view from left end of dam

Photo 3 : Dam Crest view from right end of dam

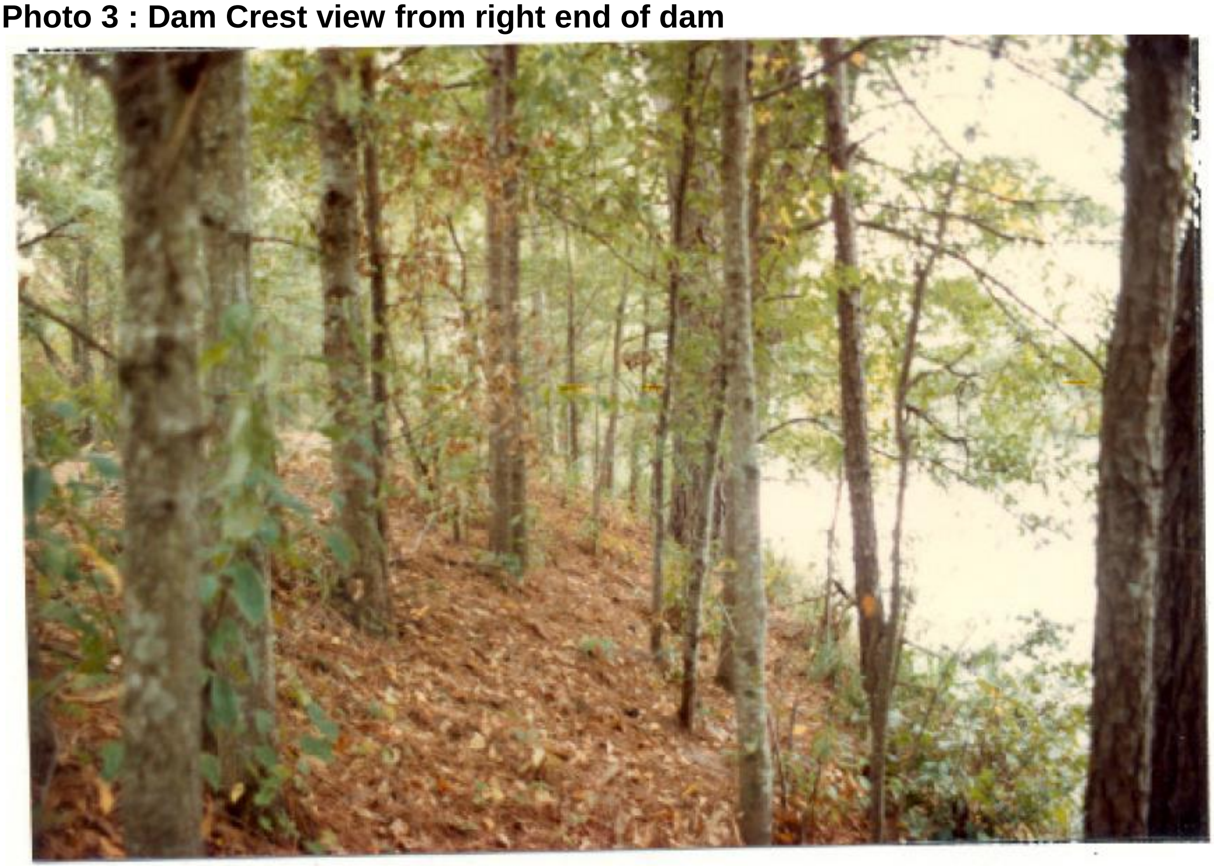

Photo 4 : Upstream slope

Photo 5 : Downstream Slope

Photo 6 : Downstream Slope

Photo 7 : Spillway entrance viewed from spillway channel

Photo 8 : Low flow outlet

C-11. RECENT PHOTOGRAPHS

These photographs were taken by EN-GS on 20 March 2015. Additional photos from the trip can be accessed in the Savannah District archives.

Photo 9 : Old Outlet Structure

Photo 10

Photo 11

C-12. GEOTECH EXPLORATION RESULTS

C-12.1 CONE PENETRATION TEST LOGS

C-12.2 GRADATION CURVES

C-12.3 PARTICLE SIZE DISTRIBUTION REPORT

C-12.4 LIQUID AND PLASTIC LIMIT

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