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ALABAMA POWER
A SOUTHERN COMPANY

July 14, 2014

VIA ELECTRONIC FILING

Ms. Kimberly D. Bose
Secretary
Federal Energy Regulatory Commission
888 First Street NE
Washington, D.C. 20426

Re: Response to Schedule A of Additional Information Request for the Martin Dam Project (FERC No. 349-173)

Dear Secretary Bose:

On June 8, 2011, Alabama Power Company (Alabama Power) filed with the Federal Energy Regulatory Commission (FERC) a Final License Application for a new license for the Martin Dam Project (FERC No. 349-173). The application included a proposal to change project operations by increasing the winter pool elevation at Lake Martin from 481 ft msl to 484 ft msl and extending the summer pool lake elevation in years when certain water-availability criteria are met. This proposal was supported by property owners, business interests, community leaders, local officials, and others that use and enjoy Lake Martin. After FERC's June 2013 Draft Environmental Impact Statement (DEIS) recommended rejecting the proposed operational changes, FERC convened a public meeting in Alexander City, Alabama on July 17 to receive oral comments on the DEIS. Over 600 members of the public attended this meeting with the overwhelming majority strongly supporting Alabama Power's proposal regarding the operational changes and encouraging FERC to reconsider its preliminary recommendation.

On November 8, 2013, FERC staff issued an Additional Information Request (November 8 AIR). The purpose of the November 8 AIR was to obtain additional information from Alabama Power so that FERC staff could further assess the downstream effects of the proposed 3-foot increase in the Lake Martin reservoir winter pool elevation, and to confirm that the proposed changes will not affect dam safety up to the probable maximum flood (PMF). On November 13, 2013, Alabama Power and FERC staff participated in a conference call to clarify the technical details of the November 8 AIR. FERC staff then issued a Memo to Public Files on January 8, 2014 that documented the conference call and provided a number of clarifications to the AIR. As a result of these clarifications, on January 24, 2014, Alabama Power provided a proposed methodology to provide the requested information for Question 2, Parts (C) and (D) in Schedule A of the November 8 AIR. Subsequently, on February 14, 2014, FERC staff indicated the proposed methodology would not provide the level of information needed for their analysis and requested that Alabama Power develop a flood frequency analysis using U.S. Geological Survey stream gage data (February 14 AIR). The February 14 AIR included a revised Schedule A and directed Alabama Power to provide a response within 120 days. On March 5, 2014, Alabama Power and FERC staff met via video conference to clarify the technical points of the February 14 AIR, which is documented in an April 7, 2014 FERC Memo to Public Files. As a result of these technical clarifications, Alabama Power requested and received a 30 day extension of time to complete the modeling and response. Finally, on May 27, 2014, Alabama Power and FERC staff met via web conference to confirm the technical details of Alabama Power's methodology, which was documented in a June 19, 2014 FERC Memo to Public Files.

These technical conferences were essential to Alabama Power in developing the attached response to the additional information requested in Schedule A of the February 14, 2014 letter. We appreciate FERC staff's assistance in ensuring that we are providing the information it needs to fully evaluate our proposed operational changes, and we believe the information provided in this response will enable FERC staff to do so.

Based on the results of the modeling, which utilized the methods discussed in the technical conferences with FERC staff, there is no incremental increase in risk of annual flooding to downstream structures and roads resulting from our proposed operational changes at Martin Dam as compared to existing operations. As a result, the information we are submitting provides additional support for FERC's approval of our requested operational changes.

If you have any questions regarding this filing, please contact me at JFCREW@southernco.com or 205-257-4265.

Sincerely,

James F. Crew

James F. Crew
Manager, Hydro Services
Alabama Power Company

Attachment

cc(w/attachment): Martin Stakeholders
Stephen Bowler – FERC

Alabama Power Response to Schedule A of the February 14, 2014 Additional Information Request

Schedule A of the February 14, 2014 letter requests information to assess the downstream flooding effects of the proposed 3-foot increase in the Martin Dam reservoir winter pool elevation. The format of the response that follows is first the restating of the information requested in Schedule A (in bold text) followed by Alabama Power's specific response to that request.

Executive Summary

In the initial submittal of the Martin Project Final License Application, Alabama Power analyzed the potential downstream impacts of a proposed change in the flood control guideline by evaluating a flood event that equaled a 1% chance of exceedance (100 year return period) in annual inflow to the Martin Reservoir. FERC requested that Alabama Power conduct additional analysis of potential impacts of a change in the flood control guideline by evaluating the change in the frequency of the annual peak floods. The new approach was based on the annual peak records at the Montgomery Water Works (MWW) gage, which is the most downstream gage on the Tallapoosa River.

The methodology utilized for this analysis was a multi-step process that included assembly of data, simulation of the reservoir operation, then routing the releases downstream and applying the results to a frequency analysis. Structures and roads in the floodplain that would be potentially impacted were identified and the risk of inundation was determined based upon the frequency results. Reservoir operations were simulated using the existing Project Routing Model and simulation of the downstream flows was accomplished with the HEC-RAS model, both of which were described in Study Report 12a – Flood Control Guideline Change Modeling Analysis ("Study 12a") submitted with the Martin Project Final License Application.

The events that were analyzed were selected from the historical annual peak stages at the MWW gage. The USGS record at MWW is from water year 1973 to 2013 but also includes a 1961 event, since it was the historical maximum at that site. A total of ten (10) events occurred during the proposed winter pool period (mid-November through February). However, there was insufficient data to support an evaluation of the 1961 event, so 9 historical events were evaluated. Once the simulation process was complete, the results were input into HEC-SSP to generate stage frequency relationships at MWW by replacing the stages of the events that showed a change in the peak stages. To determine the risk of annual flooding at identified structures and roads, a correlation function was established for each area with the corresponding peak stages at MWW. The frequency of impact to the critical elevations of the structure or road was then projected from the MWW frequency relationship.

The analysis indicated that only 2 of the 9 events resulted in higher peak stages with the proposed winter pool at 484 feet mean sea level (ft msl). The peak elevations of these two events were increased in the record and a new frequency curve was generated with the HEC-SSP. The difference between the 481 ft msl and 484 ft msl frequency curves at MWW was no greater than

0.08 feet, which is negligible and well within the confidence limits of the stage-frequency analysis.

Based on the results of the analysis, the proposed change in the Martin winter pool from elevation 481 ft msl to 484 ft msl would not change the potential annual flooding frequency of the Tallapoosa River downstream of Martin Dam. Furthermore, there would be no additional impacts to the structures or roads downstream since the stage frequency relationship did not change due to raising the winter pool elevation by three feet. While the storm analyzed in Study 12a did result in some minor potential impacts to flooding, the chance of this storm occurring during the winter pool period is extremely unlikely and, as FERC staff is aware, all structures identified are well within the FEMA floodplain.

Additional concerns expressed by FERC staff in the AIR related to dam safety associated with the probable maximum flood (PMF), additional structures that could be affected by an increase of 50% in flood elevations, and additional spillage from Martin Dam are also addressed. Analyses indicated that raising the winter pool will have no effect on dam safety for floods up to the PMF. The requested 50% increase in flood elevations identified only 5 additional structures. Finally, there was only additional spillage from one event and it did not alter the downstream stage frequency relationship for the lower Tallapoosa River.

1. **Using existing hydraulic routing models, demonstrate that the effect of raising the winter pool to elevation 484 feet will have no effect on dam safety for floods up to the probable maximum flood (PMF). Alabama Power Company's (Alabama Power) response should address how the increased winter pool will affect Potential Failure Mode (PFM) 4, Flood Erosion of the Stilling Basin Walls, and PFM 16, Overtopping of the Left Embankment Due to Failure to Operate the Gates.**

Alabama Power is proposing to raise the winter pool level at Martin from elevation 481 ft msl to elevation 484 ft msl. A possible concern for dam safety would be the potential raising of the operating level of the reservoir above that used as the starting level in determination of extreme flood loading on the Martin structures.

For Martin, the inflow design flood (IDF) is the probable maximum flood (PMF). As a result, the flood loading condition assumed in the stability analyses of the Martin structures has been the loading resulting from the PMF, the theoretical worst case flood scenario.

As shown in the hydrographs from the PMF study for Martin, provided in Figure 1, the initial conditions assumed at Martin have the reservoir level at elevation 491 ft msl. Therefore, the change in the winter pool level to elevation 484 ft msl, still seven feet below the starting reservoir level assumed in the PMF study, will have no impact on the PMF determination or the resulting flood loading assumed in the Martin stability analyses.

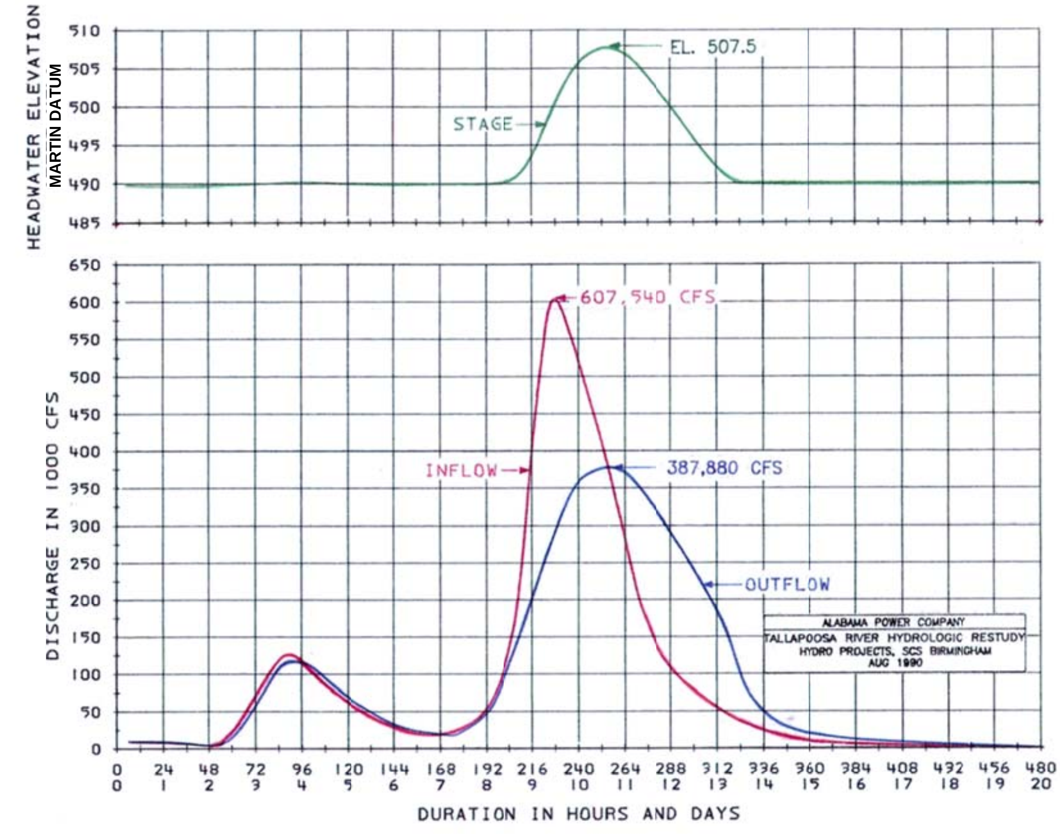
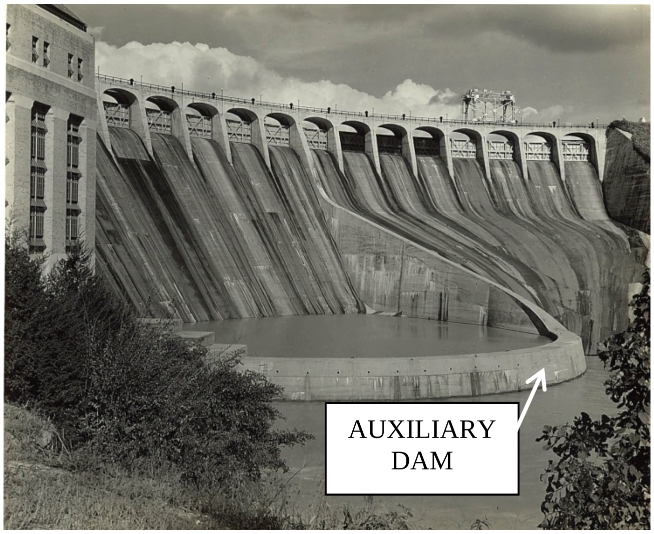


FIGURE 40 PMF Inflow, Outflow, and Stage Hydrographs at Martin Dam from 116% Elba Storm Critically Centered at EM_C Over Martin Basin

Figure 1: Martin Probable Maximum Flood Hydrographs

With Regard to PFM 4

Potential Failure Mode (PFM) 4 concerns failure of the stilling basin auxiliary dam due to flooding erosion, leading to a loss of support to the spillway and/or powerhouse. The full development of PFM 4 is shown in Figure 2. The auxiliary dam is a low structure downstream of gates 1-12 of the spillway (see photo below) intended to form a stilling basin for dissipation of energy from the discharge of these gates.



In the development of this PFM, the only unfavorable factor identified was that erosion had occurred (and been repaired) at times in the past. It was noted that the structure has performed satisfactorily since its construction, with no damage sustained that was critical with regard to the water-retaining integrity of the project. It was also noted that failure of the auxiliary dam would not result in incremental downstream impacts nor would it directly result in failure of the principal water retaining structures. Finally, it was noted that the repairs that have been done survived intact through significant flooding and spillway discharge in both June, 1989 and in May, 2003.

In the 2003 flood mentioned, the peak occurred on May 9, with 15 gates open. The reservoir was at about elevation 490.3 ft msl when the first spillway gate was raised in this event, which is within one foot of full pool. Many other spill events have occurred at Martin with the reservoir above both the current and proposed winter pool level.

Since the energy of the spillway discharge increases with the depth of flow, the critical conditions with regard to damage to the auxiliary dam are when the reservoir is at full pool (or higher in the case of a flood that exceeds the capacity of the spillway to hold the reservoir at full pool). While increasing the winter pool level to elevation 484 ft msl could result in spill events at a higher level than with the current winter pool level, these events do not represent those conditions with the greatest potential for damage to the auxiliary dam.

PFM 4 – Flood Erosion of Stilling Basin Wall/Auxiliary Dam

The PFM sequence involves a failure of the stilling basin auxiliary dam due to flooding erosion, leading to a loss of support to spillway and/or powerhouse.

Structure failure would have low immediate impact; however, its presence limits damage from erosion downstream and flood repair loading, and therefore is important to mitigate the flooding impacts to the project in the form of shoreline and channel erosion immediately downstream of the dam. The stilling basin/auxiliary dam has survived many flood events with little damage. Regular underwater inspection of the submerged portions of the stilling basin and auxiliary dam, particularly after major spill events will mitigate the potential for failure of the structures as a result of progressive erosion.

This PFM was classified as Category II to point out the importance of regular underwater inspection for erosion of submerged portions of the stilling basin and auxiliary dam. The following presents the tabular development of this failure mode:

PFM 4 – Flood Erosion of Stilling Basin Wall/Auxiliary Dam	
Conditions making PFM Likely Or Unfavorable Factors	***Conditions making PFM Unlikely Or Favorable Factors***
- The auxiliary dam and stilling basin have a history of erosion.	- 79 years of satisfactory operation. - Underwater inspections are regularly conducted. - Historical damage has not been critical with regard to the water-retaining integrity of the project. - Failure of the auxiliary dam would not result in incremental downstream impacts or failure of the principal water retaining structures of the project. - Repairs survived 1989 and 2003 floods
Category: II ***Reason:*** To point out the importance of regular underwater inspection for erosion of submerged portions of the stilling basin and auxiliary dam.	

Recommended Risk Reduction measures: The Core Team recommends that the owner continue regular underwater inspection of the submerged portions of the stilling basin and auxiliary dam particularly after major spill events.

Figure 2: Development of PFM 4

With Regard to PFM 16

PFM 16 concerns the inability or failure to operate spillway gates during a significant flood. The full development of PFM 16 is shown in Figure 3. Under certain flood conditions, this could result in the reservoir level rising to the point of overtopping the embankment flood wall and subsequent failure of the embankment.

In the development of this potential failure mode, it was noted that gate operation was necessary to prevent overtopping of the embankment section in the event of a significant flood up to the PMF. It was further noted that there is redundancy provided by both the number of gates (20) and the presence of two separate gantry hoists.

In the case of a rising reservoir and problems raising the needed gates, it is true that the higher the reservoir level at the start of the event, the less time there would be to solve whatever problem existed relative to operation of the spillway gates. However, the critical case for this scenario would be when the reservoir was at the top of the flood control guideline elevation of 491 ft msl, not when the reservoir was drawn down to the winter pool level. Therefore, the proposed raising of the winter pool level will have no significant impact on PFM 16.

PFM 16 – Failure of Left Embankment due to Failure to Operate the Gates during Significant Flood

This PFM scenario involves a failure to operate some or all of the lift gates during a significant flood, presumably due to an electrical failure. As a result, the reservoir rises sufficiently to overtop the PMF flood wall, washing out the downstream portions of the embankment, causing a breach and an uncontrolled release of reservoir. The consequences of this type of failure are possible downstream incremental impacts and loss of power generation (loss of reservoir).

This PFM was classified as Category II to point out the importance of continued testing, inspection and timely maintenance of the gates and hoists. The following presents the tabular development of this failure mode:

<table>
<tr><th colspan="2">PFM 16 – Failure of Left Embankment due to Failure to
Operate the Gates during Significant Flood</th></tr>
<tr><th>Conditions making PFM Likely
Or Unfavorable Factors</th><th>Conditions making PFM Unlikely
Or Favorable Factors</th></tr>
<tr><td>- Gate operation is necessary to prevent the embankment from overtopping during a significant flood up to the PMF.
- Gantry hoists are powered by line power or generator dependent on a fairly long circuit wire from the powerhouse to the gantry hoists; failure of the conduit would result in no power to the gantry hoists.</td><td>- There are two gantry hoists; one could be towed out of the way if a mechanical failure occurred.
- There is backup generation.
- There are numerous gates, which provides some redundancy in the case of a gate jam.
- It is likely they could be opened in an emergency with a portable crane.
- There is a program of testing, inspection and timely repair of the gates and hoist systems.</td></tr>
<tr><td colspan="2">Category: II
Reason: To point out the importance of continued testing, inspection and timely maintenance of the gates and hoists.</td></tr>
</table>

Possible Risk Reduction Measures, New Analyses or Other Actions:

- Continue the current program of testing, inspection and timely repair of the gates and hoist systems.
- Could provide a redundant circuit to the dam crest in case of loss of the primary circuit.

Figure 3: Development of PFM 16

2. Using a flood flow frequency analysis and maps of existing downstream developments, determine all significant floods (those that cannot be stored and safely released by Martin Dam without affecting downstream structures or infrastructure) where downstream development could be impacted by an increase in the winter pool elevation. Summarize your findings by providing the information in tables and figures as described below.

<u>Table 1:</u> This table should identify all structures downstream of Martin Dam to the Montgomery Water Works that would be subject to incremental flood increases resulting from raising the winter pool at Martin Dam. The table should include (A) the lower Tallapoosa River flow that first exceeds (B) the lowest adjacent grade or first floor elevation (Elevation of the lowest ground surface that touches any of the exterior walls of a building or structure) of each building or structure, and (C) the building use/building type (e.g., single family, warehouse, strip mall, stand-alone retail, vacant/occupied).

<u>Table 2:</u> This table should be completed for each affected building or structure, or group of affected structures at the same elevation, and include (D) the river flows, flooding depth at the identified structure(s) and annual exceedance probability (AEP) corresponding to the flows with a starting winter pool elevation of 481 feet, and (E) with a starting winter pool of 484 feet.

<u>Figure 1:</u> A version of this figure should be completed for each affected structure, or group of affected structures at the same elevation, and include a plot of flood depth at the affected structures versus the AEP for storms routed with the Martin Dam reservoir at the existing and proposed winter pool elevations. This plot would define the upper and lower limits of the storms for which flood depths at the affected structures are provided.

The AEP for the winter pool at 481 feet can be calculated by conducting flood frequency analyses using existing U.S. Geological Survey stream gauge data from a gage in the vicinity of the affected structures. To determine the AEP for the winter pool at 484 feet, the stream gauge records should be adjusted to reflect flows that would have occurred if the proposed rule curve was in effect at the time of the recorded storm event for each entry. A second flood frequency analysis can then be performed using the revised data.

<u>Introduction to Modeling</u>

As documented in the April 7, 2014 Memo to Public Files, Alabama Power developed a methodology to develop the information needed to respond to the February 14 AIR. This methodology is outlined below.

1. Identify flood events to model based on annual peak events during the winter pool period at the MWW gage over the period of record.
2. Assemble available data from MWW, Milstead, Uphapee, and Tallassee gages.

3. Assemble available data for Martin Reservoir.
4. Use the Project Routing Model to generate discharge hydrographs with the existing and proposed winter pool.
5. Derive the intervening flows from downstream data. Each event may require a different method, since different types of data are available.
    - Missing data will be estimated and adjusted during the HEC simulations to make the model reproduce the record peaks at MWW with Martin operating with the current winter pool.
6. Simulate the events with HEC-RAS and find the stages at the structures of interest (buildings, bridges, and roads).
7. Regenerate the MWW frequency curves.
    - Replace the peak elevations in the historical data for the period of record with the results of the simulations and regenerate the stage frequency relationship.
    - Replace the peaks flows and regenerate the relationship.
    - Compare results to original frequency relationship.
8. Documentation for final report.

The methodology was a multi-step process that included assembly of data and simulation of the reservoir operation, then routing the releases downstream through the reach of concern and finally applying the results to frequency analysis. Reservoir operations were simulated using the Project Routing Model and simulation of the downstream flows was accomplished with the HEC-RAS model, both of which were described in Study Report 12a submitted with the Martin Project Final License Application. The selection of events to analyze was based on the historical annual peak stages at the MWW gage, published by the USGS. Each step included a decision point for each event. First, only events that occurred during the proposed winter pool period (mid-November through February) were selected for modeling. Next, we determined if sufficient data were available at Martin in order to use the Project Routing Model. Once the model was completed for both winter pool alternatives, the hydrographs of the releases were compared. If the hydrographs were the same, then no further action was required for that event (i.e., that event was not modeled using HEC-RAS). If the release hydrographs were different, then the data for the downstream reach was assembled and the HEC-RAS model was completed. The HEC-RAS model was used to simulate three scenarios of each event: (1) historical releases, (2) releases based on a flat 481 ft msl pool and (3) releases based on a flat 484 ft msl pool. Once the simulation process was complete, the results were input into HEC-SSP to generate stage frequency relationships at MWW by replacing the stages of the events that showed change and the 481 ft msl pool results compared to the 484 ft msl results.

The following sections are organized to respond/report to each step in the methodology. Based on the results of the modeling exercise, the specific information requested by FERC follows the modeling results.

1. <u>Identify flood events to model based on annual peak events during the winter pool period at the MWW gage over the period of record.</u>

The USGS published 42 annual peak flow events but only 10 of the events occurred during the proposed winter pool period (mid-November through February). The USGS record is from water years 1973 to 2013 but also includes an event from 1961, which is the maximum of record. The identified events that occurred during the proposed winter pool period are listed in Table 1. The HEC-SSP software was used to generate the annual probability of occurrence (or risk) for the peak stages at the MWW gage. The risk is reflected in years in the column labeled "Return Period." It is interesting to note that all of these events, except the 1961 event, have return periods of 2 years or less. The high flow events in this basin generally occur during the spring period (March through May). In fact, 57% of the peak events at the MWW gage occurred during the spring period. When the annual peak event occurs during the winter pool period, it appears that the water year is a low flow year and, in some cases, a drought year.

Table 1: **Annual Peak Flow Events at the Montgomery Water Works Gage (USGS 02419890) That Occurred Between Mid-November through February**

Date	Peak Flow (cfs)	Peak Elevation (ft. NGVD)	Return Period (yrs)[1]
2/26/1961	170,347[2]	171.03	24.9
2/5/1982	45,839[2]	155.93	2.0
2/7/1985	26,858[2]	148.26	1.3
2/5/1988	24,768[2]	147.32	1.2
2/18/1992	27,380[2]	148.50	1.3
11/27/1992	45,181[2]	155.73	2.0
2/8/2002	17,100	143.84	1.0
1/8/2007	19,600	145.28	1.1
1/24/2012	19,500	145.21	1.0
2/14/2013	43,600	155.01	1.9

[1] Return period based on HEC-SSP analysis of the historical annual peaks at Montgomery Water Works gage.
[2] Peak flows for this event were estimated with the USGS current rating curve.

2. <u>Assemble available data from MWW, Milstead, Uphapee, and Tallassee gages.</u>

Four historical gages are located in the Tallapoosa River basin below Martin Dam (See Figure 4):

- **Montgomery Waters Works Gage** (USGS 02419890) is located on the intake for the pumping station at River Mile 12.9 and data are collected by the MWW. This is the most downstream gage on the Tallapoosa River and is at the downstream end of the reach of concern. Hourly flows and stages are available at the MWW gage from 2007 to present, daily records of flows from 1995 to present, daily stage records from 1989 to present, and annual peak stages from 1973 to present, plus the 1961 peak stage.
- The **Milstead Gage** (USGS 02419500) is located at River Mile 39.80 on the Alabama Highway 229 bridge and is maintained by Alabama Power. Only stage data is available for this station in the historical records. Hourly data is available from 2007 to present and daily data is available from 1994 to present. USGS also has a basic flow-stage rating, which was used to estimate the flow at the gage.
- The **Uphapee Gage** (USGS 02419000) is located on the Uphapee Creek approximately 10 miles upstream of its mouth; therefore, the gage only represents 79% of the Uphapee

runoff. Hourly flows and stage data are available from 2007 to present, daily average flows are available from 1939 to present, and daily stage values from 1974 to present.

- The **Tallassee Gage** (USGS 02418500) is located just downstream of Tallassee, AL at River Mile 47.93, which is at the upstream end of the reach of concern. Only daily flow data are available at this gage and are assumed to be defined by Thurlow Dam releases. These data are available from 1928 to 2013.

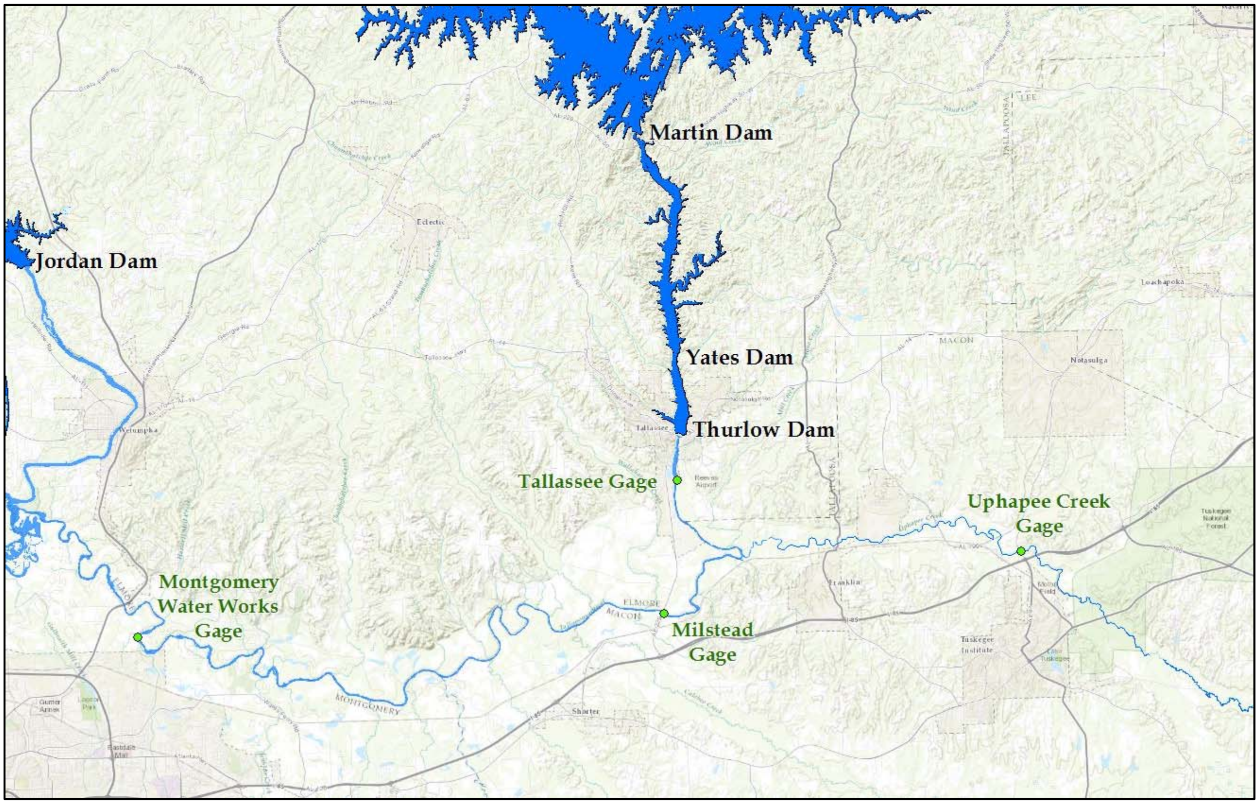


Figure 4: **Location of USGS Gages on the Tallapoosa River Below Martin Dam**

The stream flow and stage database is maintained by USGS but data are collected by others at some gages. Releases from the reservoirs were obtained from Alabama Power Company and the U. S. Army Corps of Engineers.

3. <u>Assemble available data for Martin Reservoir.</u>

Storage relationships and operational requirements along with data collected by Alabama Power, including pool elevation, discharges, and inflows are used to operate Martin Dam. These data are available hourly from 1990 to present and daily from 1963 to present. Prior to 1963, the operational data logged for Martin include turbine and gate operations only. Because the Project Routing Model requires reservoir inflows, the data available for the 2/26/1961 event were not sufficient to model this event. This event probably should not have been modeled since the upper basin of the Tallapoosa River has undergone significant change since 1961, including the construction of the Harris Reservoir by Alabama Power, improvement and operational changes at Martin, and other developments. The 1973 to present period best represents current conditions.

4. Use the Project Routing Model to generate discharge hydrographs with the existing and proposed winter pool.

The Alabama Power Project Routing Model is a spreadsheet model that combines the physical features of the dam with the flood control operations requirements to replicate the passage of a high inflow event through the Martin dam. Given an inflow hydrograph and initial conditions, the Project Routing Model produces the predicted stage of the Martin pool and the predicted outflow. Model inputs include the elevation/volume table, the spillway gate rating curves, turbine outflow ratings and the flood control guidelines. From a starting pool elevation, an equivalent volume is known. For the next hour of the simulated model there is a known inflow and elevation and based on the flood control rules there is a resulting outflow. The difference in the inflow and outflow added to the known previous hour volume produces the new current hour volume and thus a new current hour elevation. The calculation is performed on an hourly basis to best mimic actual operations during a flood event. A full description of the Project Routing Model can be found in Study Report 12a submitted with the Martin Project Final License Application. Applicable operational criteria used for this Project Routing Model were:

1. Current rule curve begins lowering summer pool in early September to winter pool by January 1 and begins the rise to summer pool about mid-February and reaches the summer pool in mid-April.
    a. With a winter pool at 484 ft msl, the winter pool would begin at the third week in November and end at the first of March.
2. Rules for Releases Used in the Project Routing Model
    a. If pool is below 486 ft msl, then release the Thurlow rate of 12,400 cfs through the Martin turbines to maintain rule curve.
    b. If the pool is between 486 ft msl and 489 ft msl, then release 13,200 cfs through the Martin turbines to maintain rule curve.
    c. For pool levels above 489 to 490.95 ft msl, go to full Martin turbines of 16,500 cfs and employ the spillway as needed to maintain the pool elevation below 491 ft msl. The number of gates to open is based on 2 gates per hour, changes in rate of rise of elevation and/or inflow as seen historically.
    d. Maximum pool is 491 ft msl, above which all outlets would be fully open and total release would be based on net head.

Martin Discharge Hydrographs of Identified Peak Flows

Using the Project Routing Model and available data as described in Step #2 and Step #3 above, discharge hydrographs (Martin releases) and pool levels were developed for each identified event. The following charts also include data on historical (actual) operations in order to compare model results.

The Project Routing Model was set up to best replicate the routing of a flood event through Martin Dam. Model logic was designed to match the rising limb and peak discharge of a known flood event. However, the model does not account for the variability in basin conditions for each storm event modeled. During real-time operations, Martin is operated as one component of a larger system, the Tallapoosa River and, ultimately, the Alabama-Coosa-Tallapoosa basin. Therefore, historical operations do not always match model results. Because of this, our comparison is based on results from the Project Routing Model at both starting elevations of 481 ft msl and 484 ft msl.

2/5/1982 Annual Peak Flow Event (February 1 to March 5)

This event, which modeled the greatest downstream elevation differences, has a 2 year return period. The Project Routing Model indicated that a peak release of 36,464 cfs would be required by the operating rules if the winter pool was a flat 484 ft msl (Figure 5) because the pool elevation reached the 491 ft msl elevation (Figure 6) and spillway gates were opened. A 481 ft msl flat pool would release a peak of 16,500 cfs. Since the peak flow associated with the discharge hydrographs (Martin releases) were different for this event, HEC-RAS modeling was required.

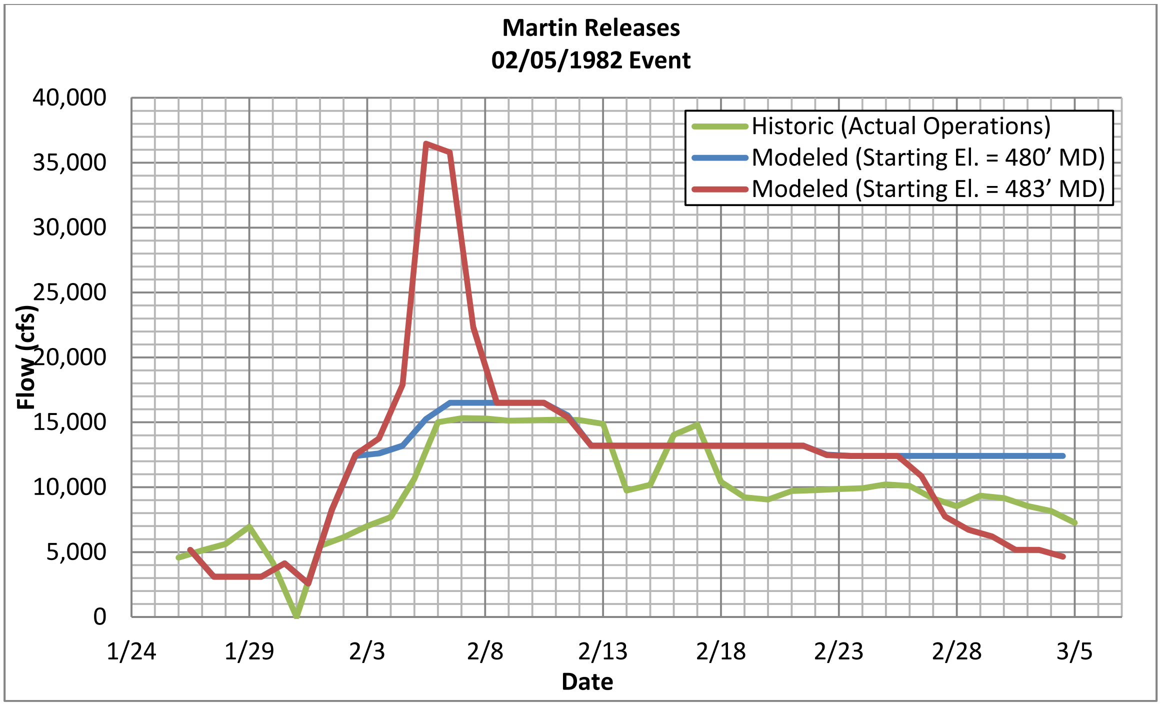


Figure 5: **Releases from Martin Dam During the February 5, 1982 Peak Flow Event at the Montgomery Water Works Gage (Note: Elevations are presented in Martin Datum, which is 1 foot lower than mean sea level.)**

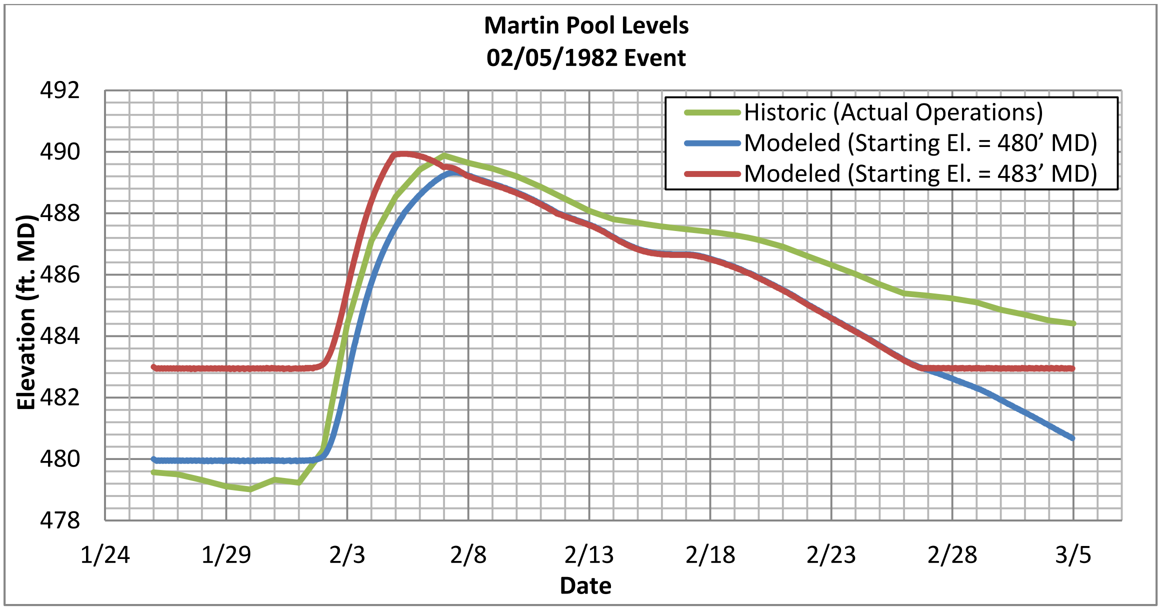


Figure 6: **Martin Reservoir Levels During the February 5, 1982 Peak Flow Event at the Montgomery Water Works Gage (Note: Elevations are presented in Martin Datum, which is 1 foot lower than mean sea level.)**

2/7/1985 Annual Peak Flow Event (February 1 to February 27)

The Project Routing Model indicated that the discharge hydrographs for Martin were not different for this event with a peak flow of 12,400 cfs (Figure 7) and the reservoir did not reach full pool under either scenario (Figure 8). Therefore, HEC-RAS modeling was not required.

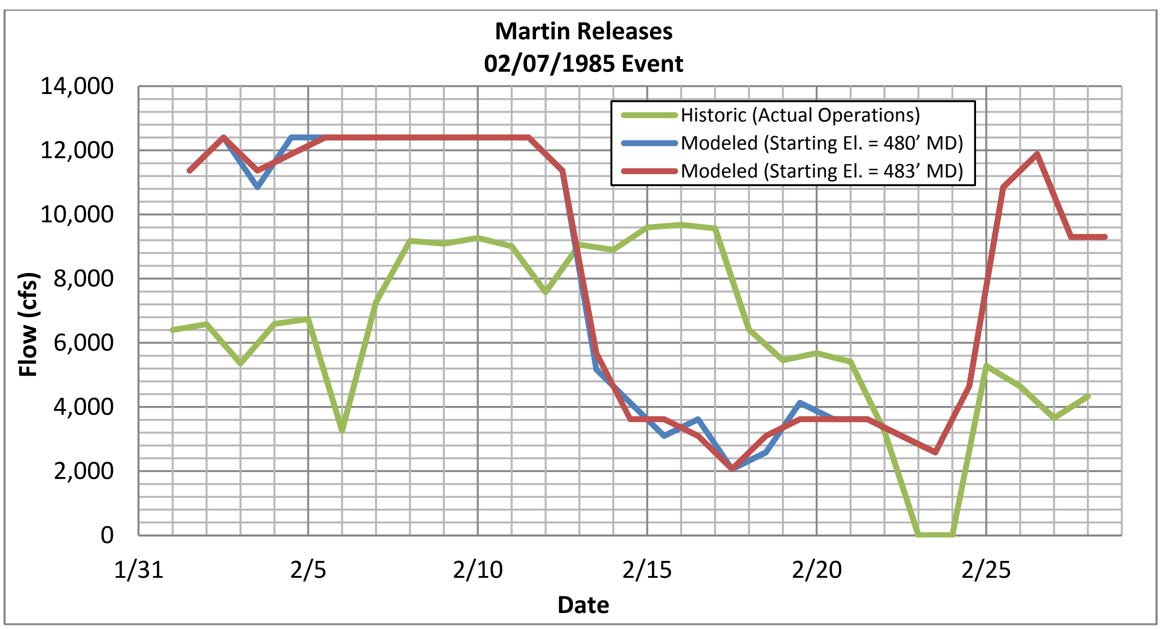


Figure 7: **Releases from Martin Dam During the February 7, 1985 Peak Flow Event at the Montgomery Water Works Gage (Note: Elevations are presented in Martin Datum, which is 1 foot lower than mean sea level.)**

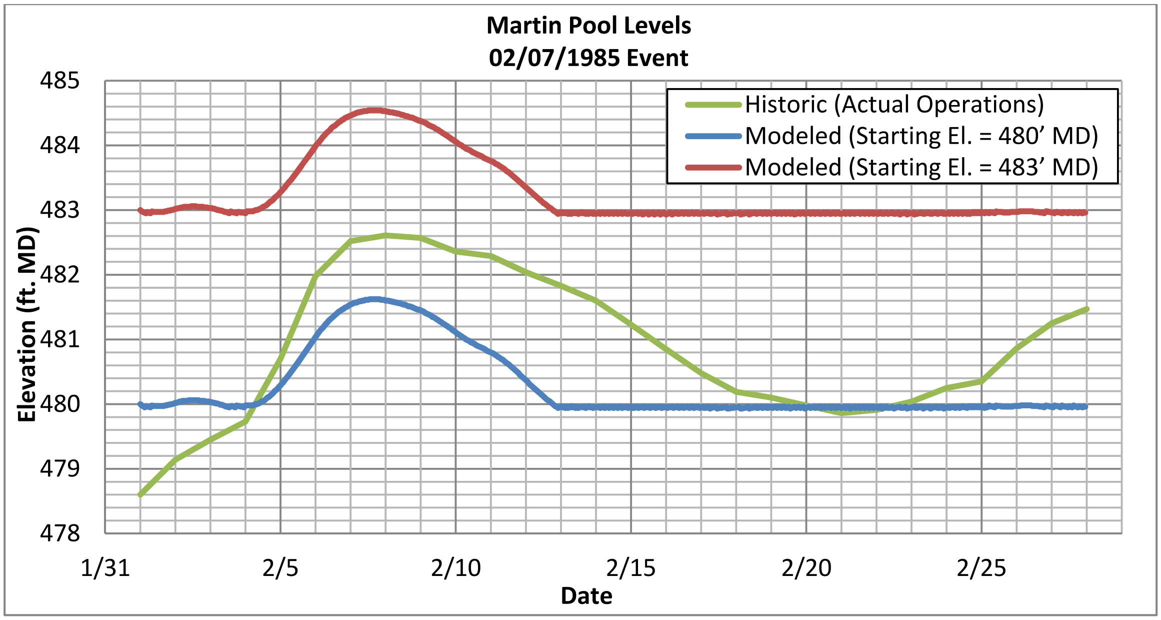


Figure 8: **Martin Reservoir Levels During the February 7, 1985 Peak Flow Event at the Montgomery Water Works Gage (Note: Elevations are presented in Martin Datum, which is 1 foot lower than mean sea level.)**

2/5/1988 Annual Peak Flow Event (January 11 to February 27)

The Project Routing Model indicated that the discharge hydrographs for Martin were not different for this event with a peak flow of 12,400 cfs (Figure 9) and the reservoir did not reach full pool under either scenario (Figure 10). Therefore, HEC-RAS modeling was not required.

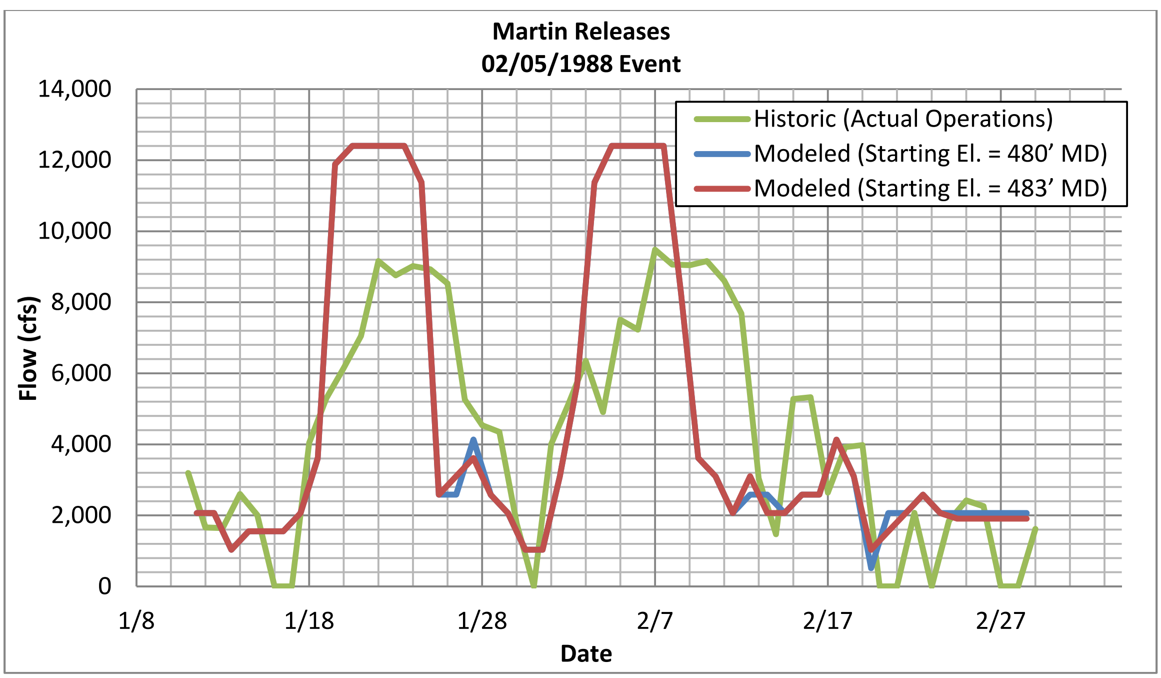


Figure 9: **Releases from Martin Dam During the February 5, 1988 Peak Flow Event at the Montgomery Water Works Gage (Note: Elevations are presented in Martin Datum, which is 1 foot lower than mean sea level.)**

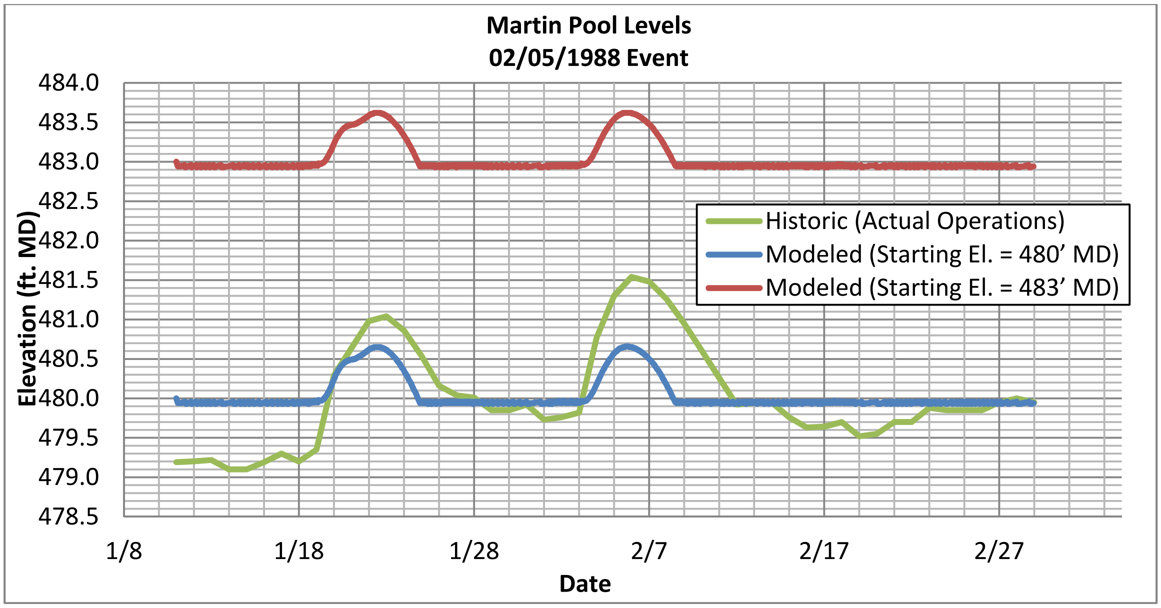


Figure 10: **Martin Reservoir Levels During the February 5, 1988 Peak Flow Event at the Montgomery Water Works Gage (Note: Elevations are presented in Martin Datum, which is 1 foot lower than mean sea level.)**

2/18/1992 Annual Peak Flow Event (February 1 to March 15)

This event is longer than the other events and has much more volume of inflow to the reservoir; however, because of the longer time and possibly lower downstream intervening flows, it only has a 1 year return period at MWW. The Project Routing Model indicated the pool maxed at 485.87 ft msl with the winter pool initially at 484 ft msl, but for an initial pool at 481 ft msl, it only reached a maximum of 482.97 ft msl (Figure 11). The daily average discharge hydrographs (releases) were the same (Figure 12); therefore, HEC-RAS modeling was not required. However, this event was modeled to verify that downstream sites experienced no change.

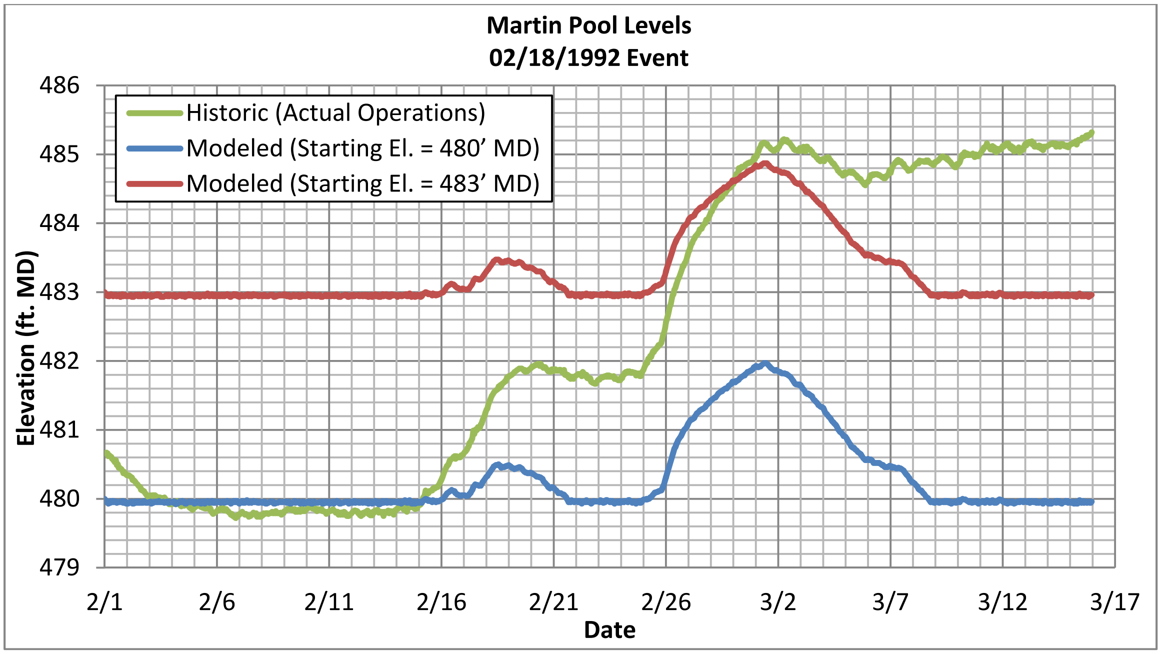


Figure 11: **Martin Reservoir Levels During the February 18, 1992 Peak Flow Event at the Montgomery Water Works Gage (Note: Elevations are presented in Martin Datum, which is 1 foot lower than mean sea level.)**

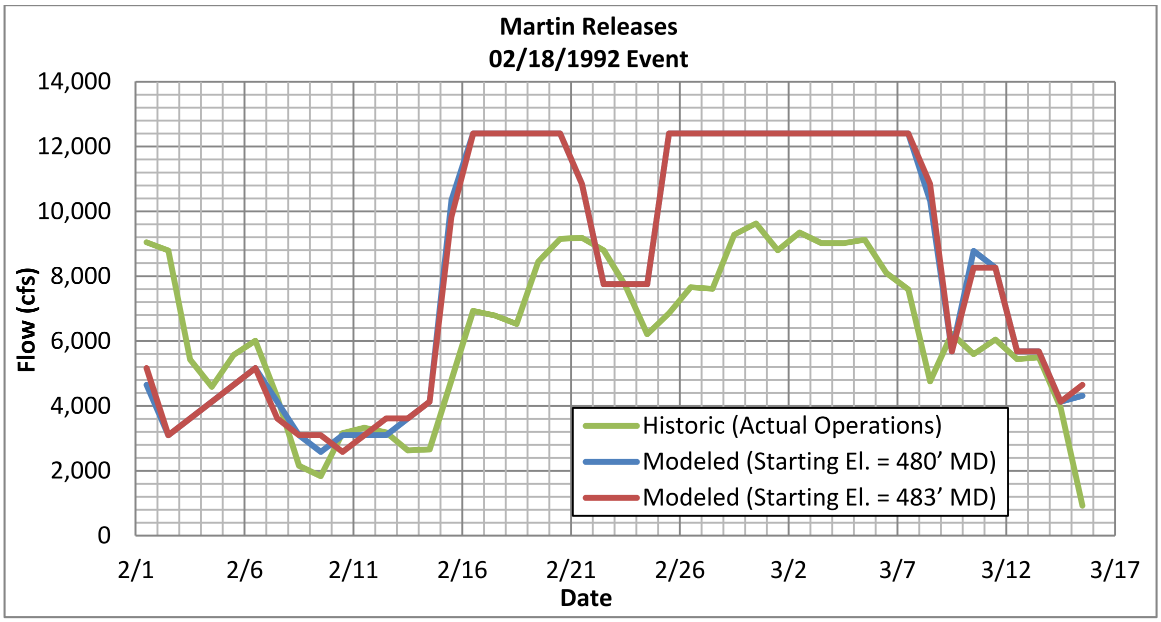


Figure 12: **Releases from Martin Dam During the February 18, 1992 Peak Flow Event at the Montgomery Water Works Gage (Note: Elevations are presented in Martin Datum, which is 1 foot lower than mean sea level.)**

11/27/1992 Annual Peak Flow Event (November 20 to January 5)

This event is a relatively long (46 days) event but it only rates as a 2.0 year return period at the MWW gage. The Project Routing Model indicated that a peak release of 13,200 cfs would be required by the operating rules if the winter pool was a flat 484 ft msl. A 481 ft msl flat pool would release a peak flow of 12,400 cfs (Figure 13). This is because under the 484 ft msl starting elevation, the pool maxed at 488.49 ft msl (Figure 14). Since the pool rose above 486 ft msl, different release rates in the Project Routing Model were required. Since the peak flow associated with the discharge hydrographs (Martin releases) were different for this event, HEC-RAS modeling was required.

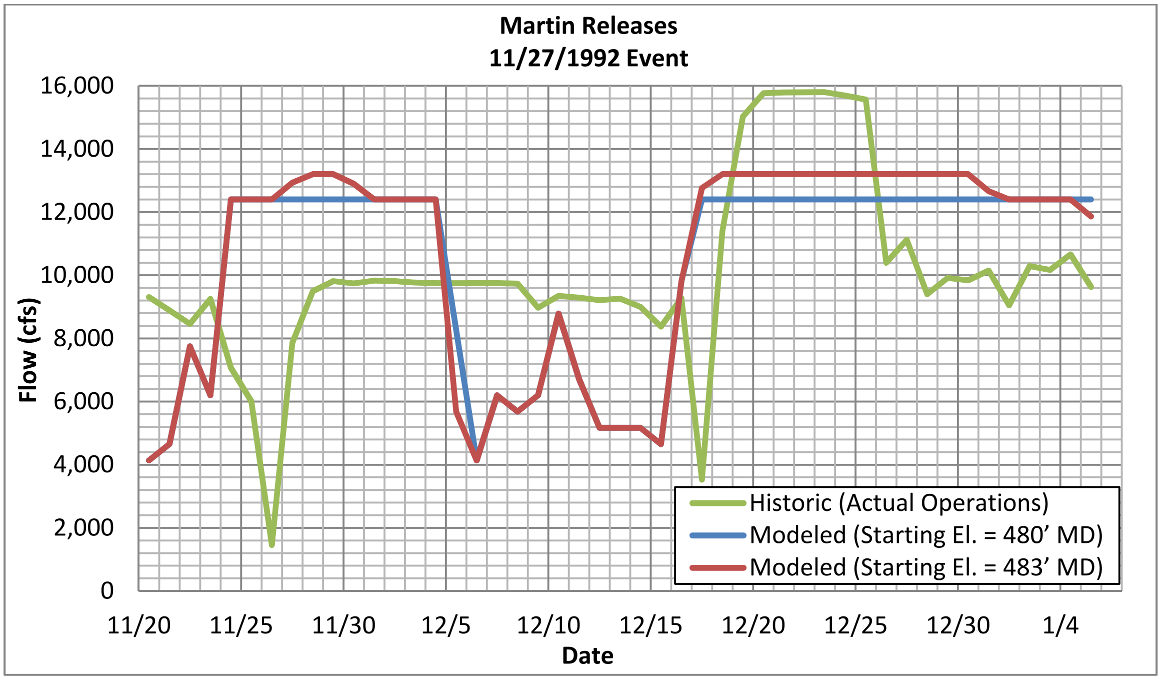


Figure 13: Releases from Martin Dam During the November 27, 1992 Peak Flow Event at the Montgomery Water Works Gage (Note: Elevations are presented in Martin Datum, which is 1 foot lower than mean sea level.)

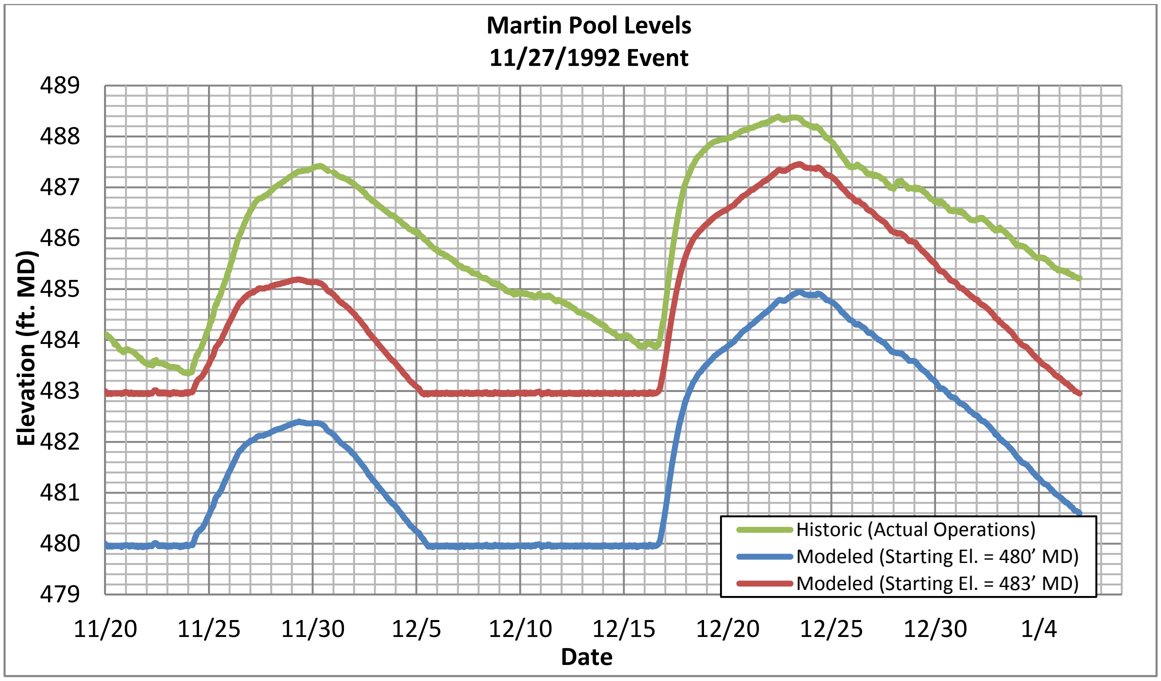


Figure 14: **Martin Reservoir Levels During the November 27, 1992 Peak Flow Event at the Montgomery Water Works Gage (Note: Elevations are presented in Martin Datum, which is 1 foot lower than mean sea level.)**

2/8/2002 Annual Peak Flow Event (February 1 to February 26)

This event has a 1 year return period at MWW and is a relatively small event, registering as an annual peak flow since 2002 was a drought year. The Project Routing Model indicated that the discharge hydrographs for Martin were not different for this event with a peak flow of 10,850 cfs (Figure 15) and the reservoir did not reach full pool under either scenario (Figure 16). Therefore, HEC-RAS modeling was not required.

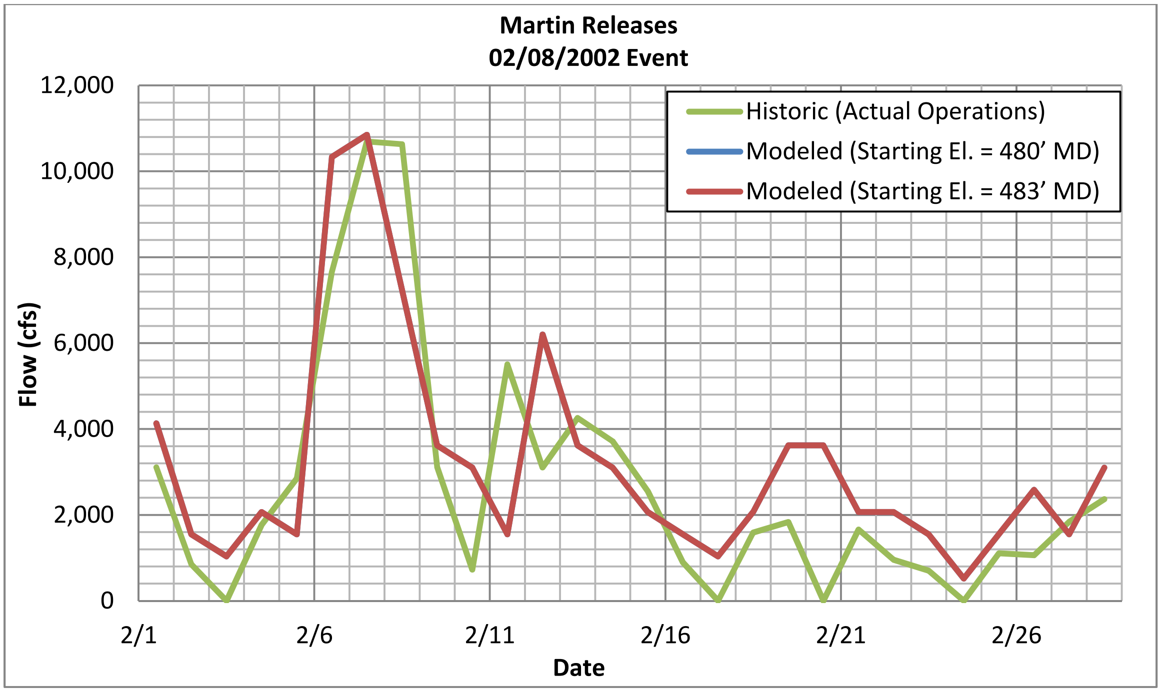


Figure 15: **Releases from Martin Dam During the February 8, 2002 Peak Flow Event at the Montgomery Water Works Gage (Note: Elevations are presented in Martin Datum, which is 1 foot lower than mean sea level.)**

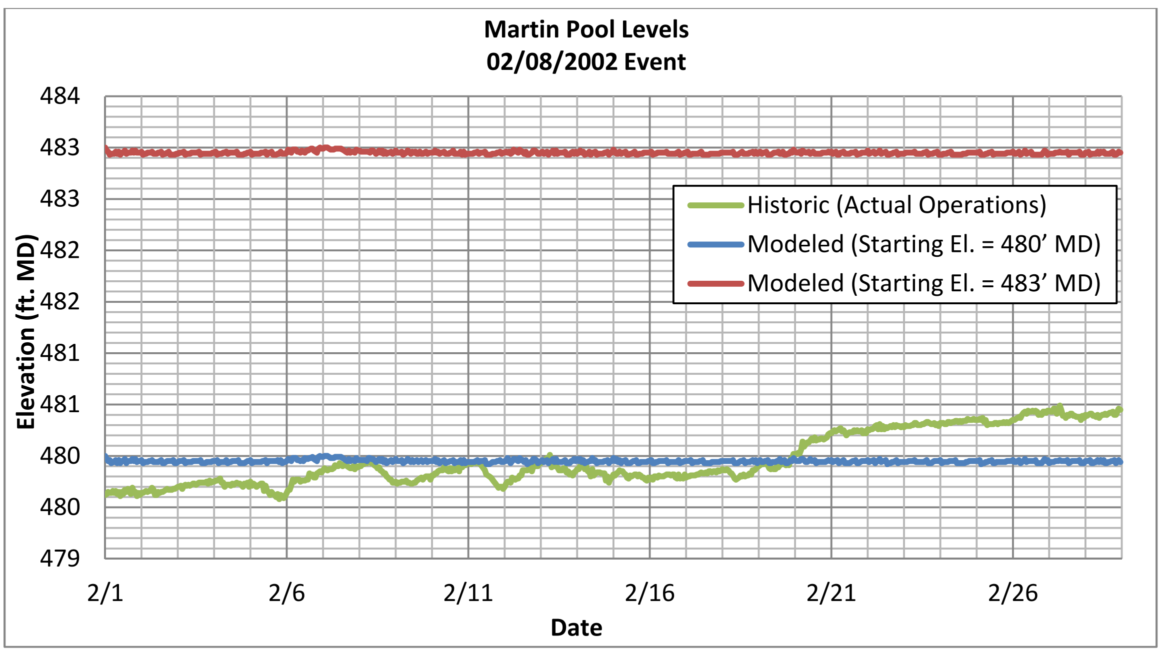


Figure 16: **Martin Reservoir Levels During the February 8, 2002 Peak Flow Event at the Montgomery Water Works Gage (Note: Elevations are presented in Martin Datum, which is 1 foot lower than mean sea level.)**

1/8/2007 Annual Peak Flow Event (January 3 through January 13)

This event has a return period of 1.1 years in the annual frequency relationship, registering as an annual peak flow since 2007 was a drought year. The Project Routing Model indicated that the discharge hydrographs for Martin were not different for this event with a peak flow of 12,400 cfs (Figure 17) and the reservoir did not reach full pool under either scenario (Figure 18). Therefore, HEC-RAS modeling was not required.

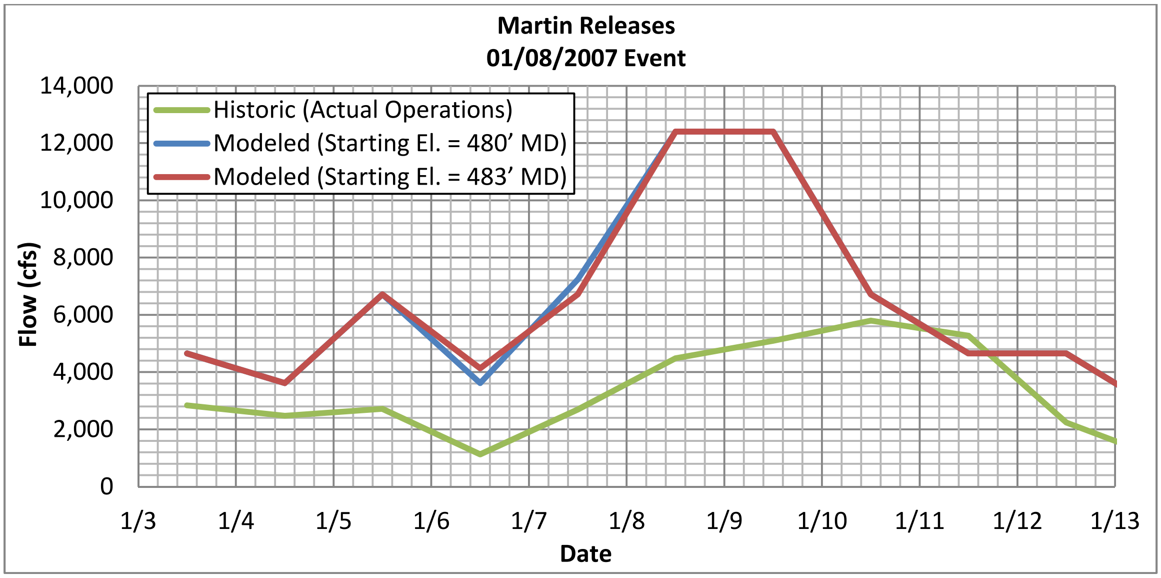


Figure 17: **Releases from Martin Dam During the January 8, 2007 Peak Flow Event at the Montgomery Water Works Gage (Note: Elevations are presented in Martin Datum, which is 1 foot lower than mean sea level.)**

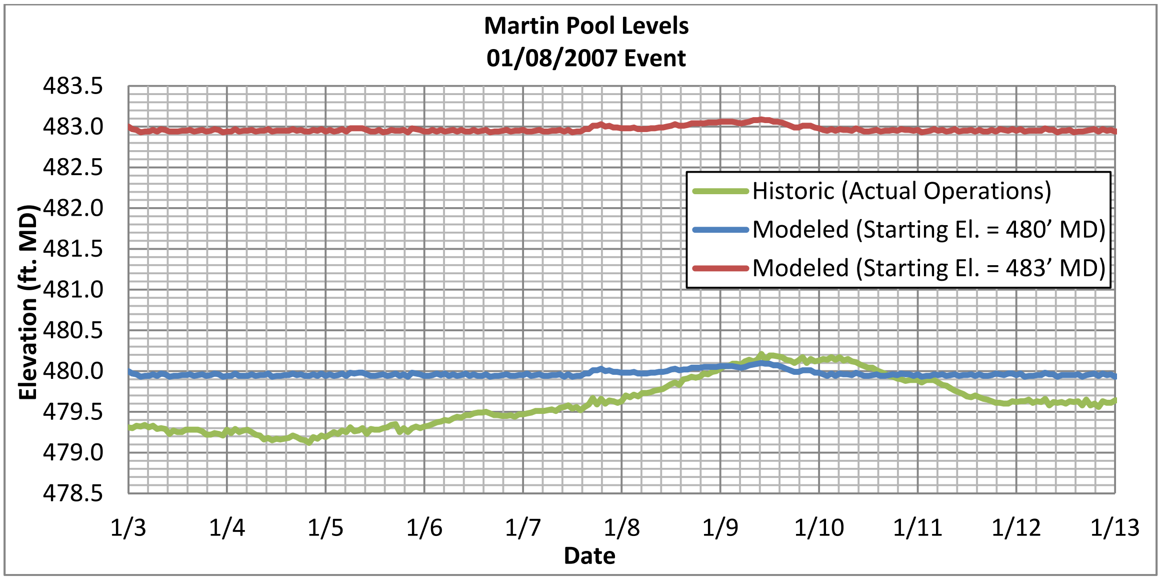


Figure 18: **Martin Reservoir Levels During the January 8, 2007 Peak Flow Event at the Montgomery Water Works Gage (Note: Elevations are presented in Martin Datum, which is 1 foot lower than mean sea level.)**

1/24/2012 Annual Peak Flow Event (January 16 through February 9)

This event has a return period of 1.0 year in the annual frequency relationship. The Project Routing Model indicated that the discharge hydrographs for Martin were not different for this event with a peak flow of 12,400 cfs (Figure 19) and the reservoir did not reach full pool under either scenario (Figure 20). Therefore, HEC-RAS modeling was not required. It should be noted that Martin was operating under a temporary variance at this time that allowed Alabama Power to hold the winter pool level at 484 ft msl.

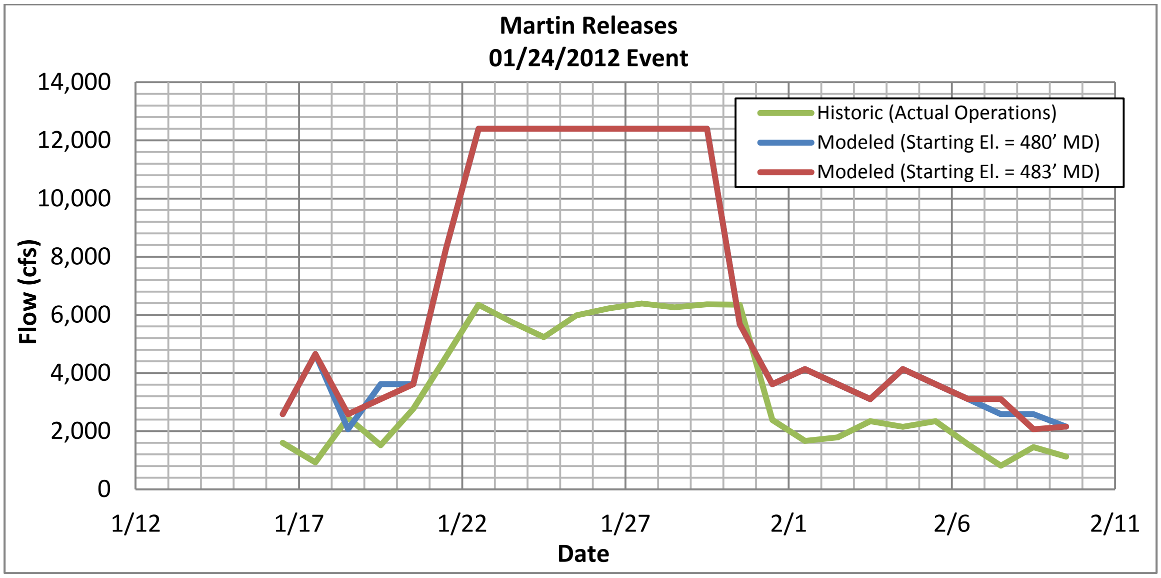


Figure 19: **Releases from Martin Dam During the January 24, 2012 Peak Flow Event at the Montgomery Water Works Gage (Note: Elevations are presented in Martin Datum, which is 1 foot lower than mean sea level.)**

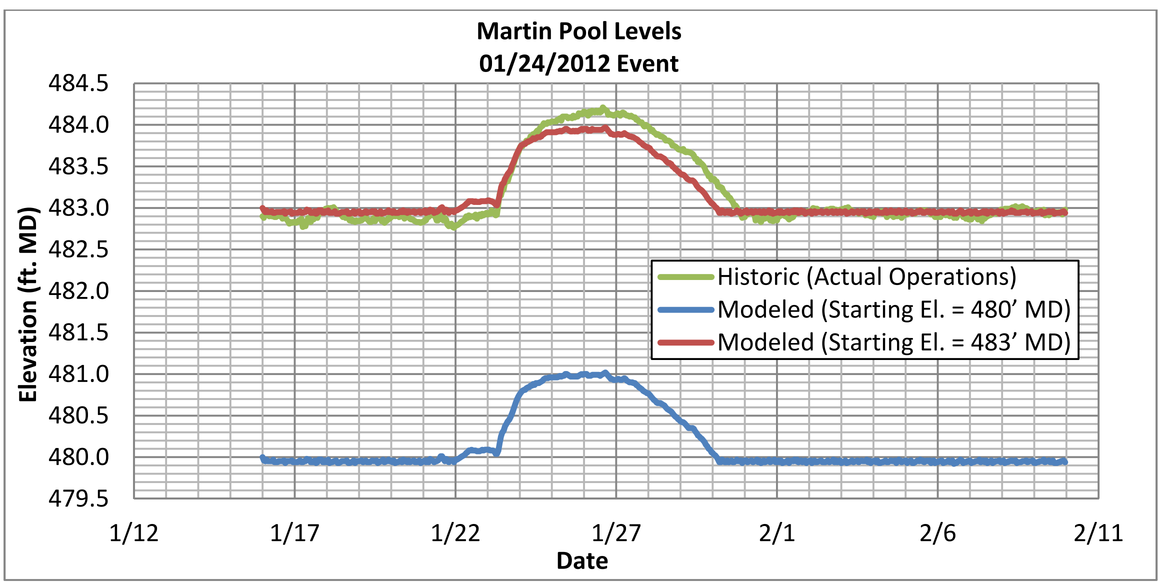


Figure 20: **Martin Reservoir Levels During the January 24, 2012 Peak Flow Event at the Montgomery Water Works Gage (Note: Elevations are presented in Martin Datum, which is 1 foot lower than mean sea level.)**

2/14/2013 Annual Peak Flow Event (February 1 through February 25)

This event has a return period of 1.9 years in the annual frequency relationship. The Project Routing Model indicated that the discharge hydrographs for Martin were not different for this event with a peak flow of 12,400 cfs (Figure 21) and the reservoir did not reach full pool under either scenario (Figure 22). Therefore, HEC-RAS modeling was not required.

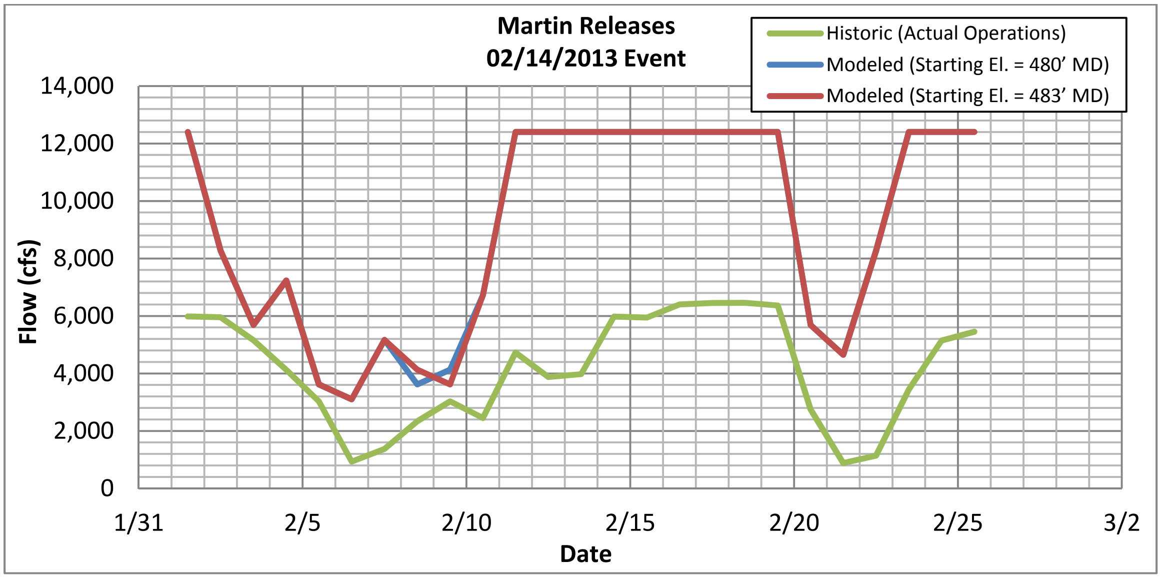


Figure 21: **Releases from Martin Dam During the February 14, 2013 Peak Flow Event at the Montgomery Water Works Gage (Note: Elevations are presented in Martin Datum, which is 1 foot lower than mean sea level.)**

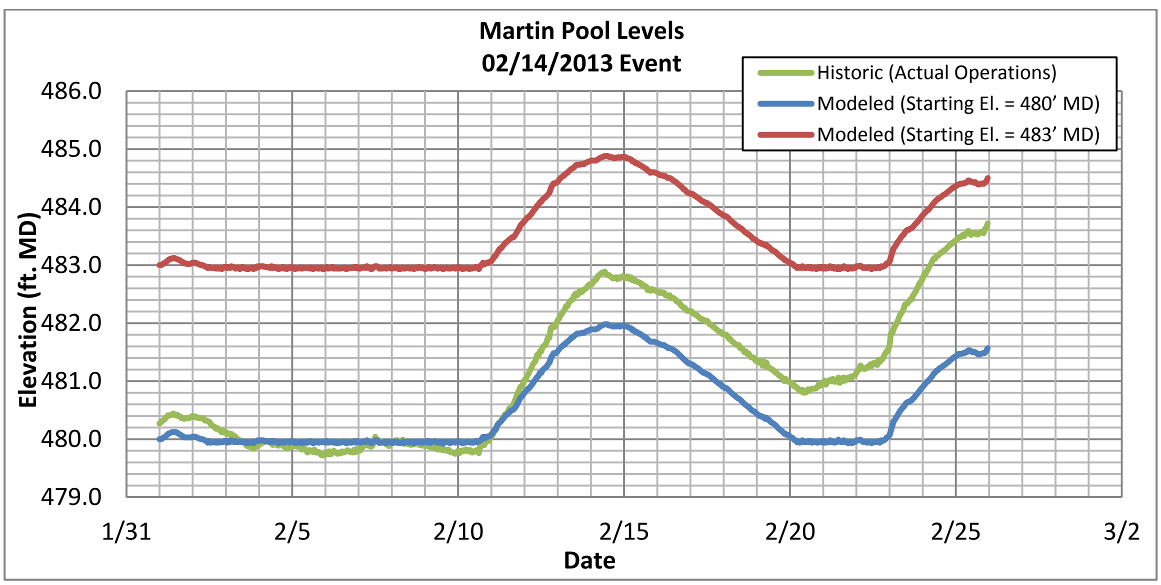


Figure 22: **Martin Reservoir Levels During the February 14, 2013 Peak Flow Event at the Montgomery Water Works Gage (Note: Elevations are presented in Martin Datum, which is 1 foot lower than mean sea level.)**

In conclusion, two events were selected for HEC-RAS modeling because the peak flows were different: the 2/5/1982 and 11/27/1992 peak flow events. One event was selected (2/18/1992) to test the impacts for an event that indicated the Martin Dam discharge hydrograph would be the same for both starting elevations.

5. Derive the intervening flows from downstream data.

The HEC-RAS model limits extend from the RM 48.12 on the Tallapoosa River to the Robert F. Henry Lock and Dam on the Alabama River and up the Coosa River to the tailwater of Jordan Dam. RM 48.12 is approximately 1.5 miles below Thurlow Dam on the Tallapoosa. The Tallassee gage is located at RM 47.98. The methodology to estimate or determine the intervening flows was dependent on the availability of data.

Only Uphapee and Tallassee flows were available for the 2/5/1982 event, so the flows at Montgomery Water Works were estimated using the USGS gage near Montgomery on the Alabama River, subtracting the Coosa River flows, and adjusting the results by drainage area ratios. The Milstead flows were estimated by adjusting the Tallassee flows by drainage area ratios and adding Uphapee flows, lagged by one day. Intervening flows were then determined. Negative flows were set to zero and all positive values adjusted to retain the total volumes.

For the 2/18/1992 and 11/27/1992 events, daily flows were available at Montgomery Water Works, Uphapee Creek, and Tallassee gages. Data were not available for Milstead. Local or intervening flows that entered the system between Martin Dam and Tallassee were determined by subtracting the historical releases from Martin from the Tallassee gage flows. Time steps with negative flows were set to zero but the positive hydrograph values were adjusted to retain the volume. Intervening flows between Tallassee and Milstead were determined by subtracting the Tallassee and the Uphapee values from the MWW flows. The results were then adjusted using drainage area ratios. The Milstead to Montgomery Water Works intervening flows were considered to be the residual of the Tallassee to Montgomery Water Works intervening flows after the Tallassee to Milstead and Uphapee values were removed.

It was found that daily flows produced results as good as hourly values and accommodated a more stable model. It was also realized that the hydropower releases from Martin were attenuated as they passed through Yates and Thurlow; therefore, daily average flows better represented the inflow to the upper boundary of the HEC-RAS model. Daily data were also used to develop the intervening flows for the events that were identified to be modeled.

6. Simulate the events with HEC-RAS and find the stages at the structures of interest (buildings, bridges, and roads).

Three scenarios were simulated for each event: (1) the historical event (for comparison to actual records at the MWW gage), (2) with Martin discharges from the 481 ft msl Project Routing Model results and (3) with Martin discharges from the 484 ft msl Project Routing Model results. The following schematic shows the basic layout of the HEC-RAS model and the inflow points (Figure 23).

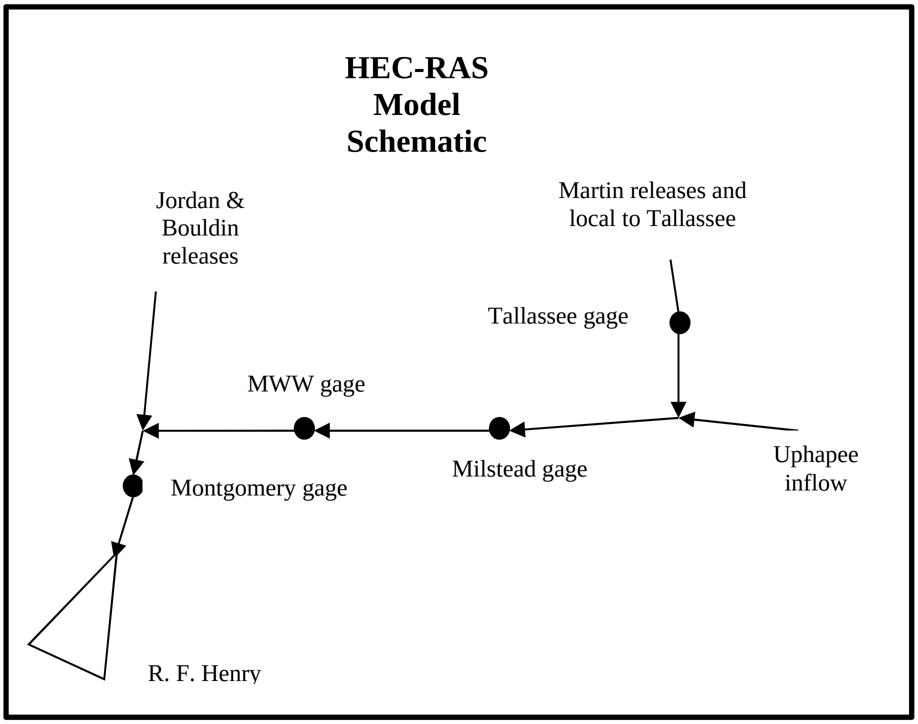


Figure 23: Basic Layout of the HEC-RAS Model and Inflow Points

2/5/1982 Annual Peak Flow Event (February 1 to March 5)

This simulation produced the greatest downstream elevation differences in the 481 ft msl and the 484 ft msl starting elevations at Martin. A peak increase of approximately 0.54 feet was produced at the MWW gage (Figures 24 and 25). Stage differences near Tallassee were up to 5.44 feet; however, the flows remained within the channel. The following are hydrographs for the simulations and the actual historical record at the MWW gage for those simulations.

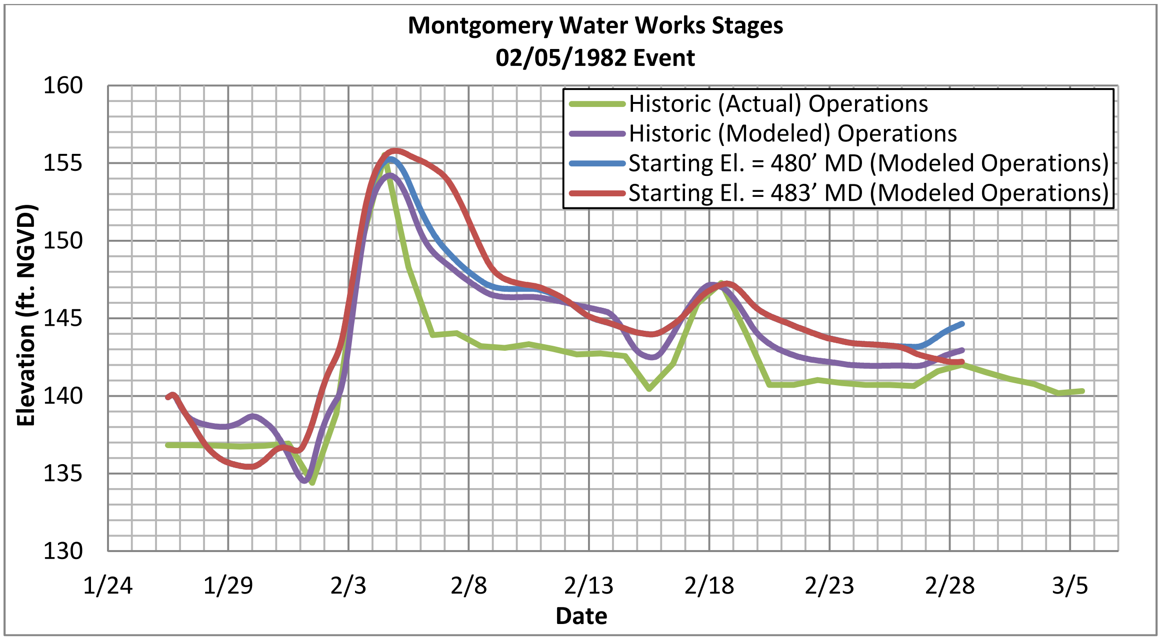


Figure 24: **Elevation at the Montgomery Water Works (Tallapoosa River Mile 12.9) During the February 5, 1982 Peak Flow Event at the Montgomery Water Works Gage**

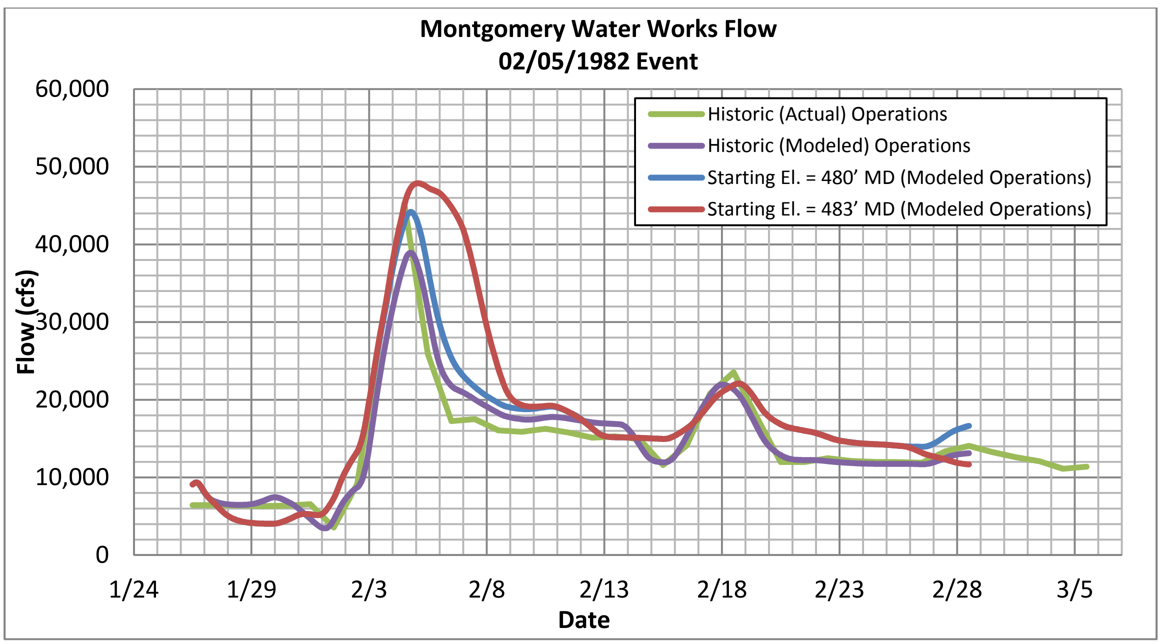


Figure 25: **Flow (in cfs) at the Montgomery Water Works (Tallapoosa River Mile 12.9) During the February 5, 1982 Peak Flow Event at the Montgomery Water Works Gage**

2/18/1992 Annual Peak Flow Event (February 1 to March 15)

This simulation was included to test the upstream impacts for an event that indicated the Martin Dam discharge hydrograph would be the same for both starting elevations. The results indicated that there were no differences in the peak stages over the entire downstream reach. The two starting elevations overlay each other in the following graphs (Figures 26 and 27) of the flows and stages at the MWW gage.

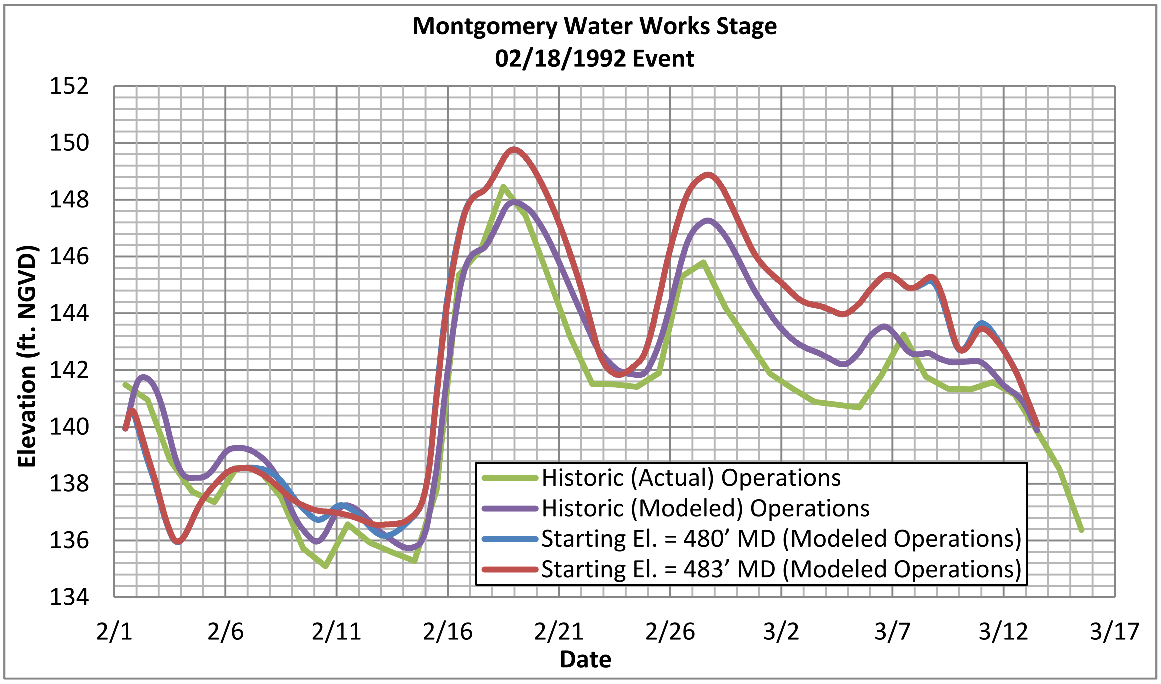


Figure 26: **Elevation at the Montgomery Water Works (Tallapoosa River Mile 12.9) During the February 18, 1992 Peak Flow Event at the Montgomery Water Works Gage**

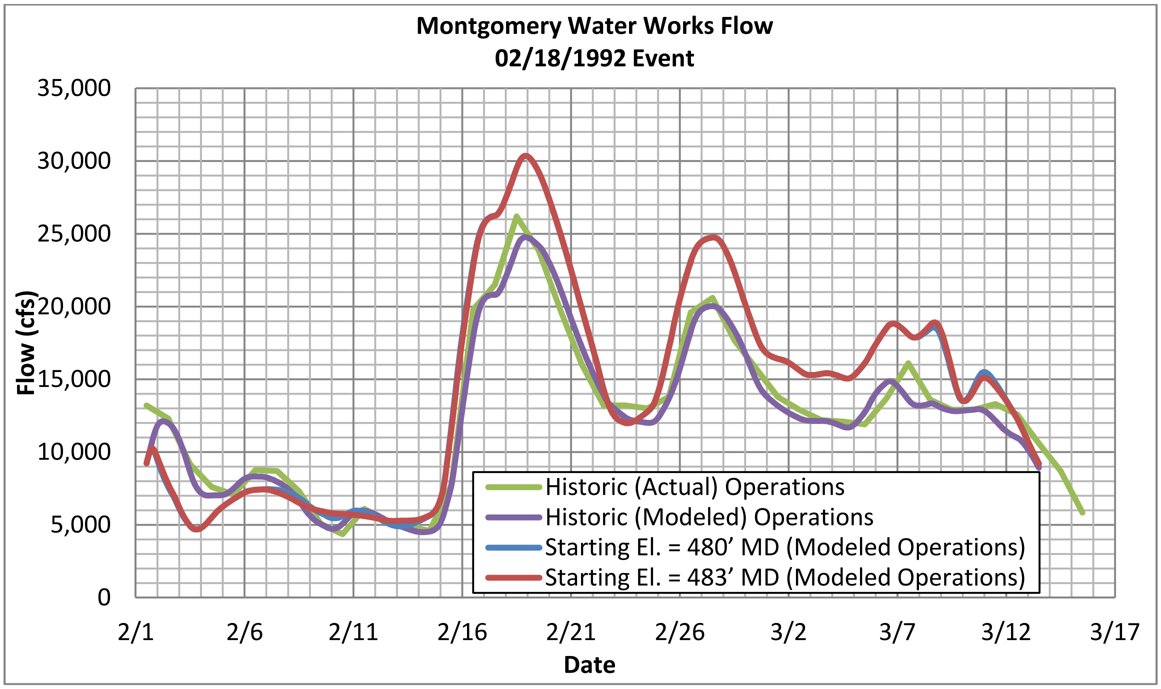


Figure 27: **Flow (in cfs) at the Montgomery Water Works (Tallapoosa River Mile 12.9) During the February 18, 1992 Peak Flow Event at the Montgomery Water Works Gage**

11/27/1992 Annual Peak Flow Event (November 20 to January 5)

The 11/27/1992 event was the only other event that resulted in different hydrographs for the Martin discharges. The 484 ft msl starting elevation resulted in a 0.06 foot higher peak stage at the MWW gage with only a maximum difference of 0.07 feet near River Mile 26 (Figures 28 and 29). Differences dropped to 0.01 feet near the Tallassee area.

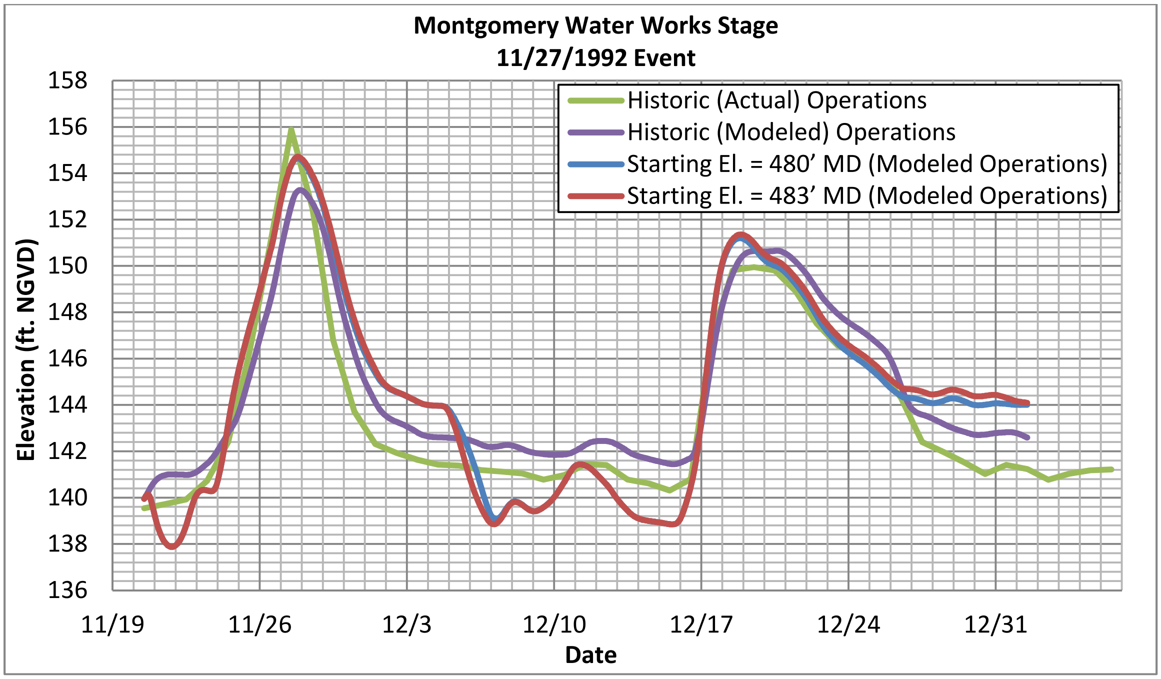


Figure 28: **Elevation at the Montgomery Water Works (Tallapoosa River Mile 12.9) During the November 27, 1992 Peak Flow Event at the Montgomery Water Works Gage**

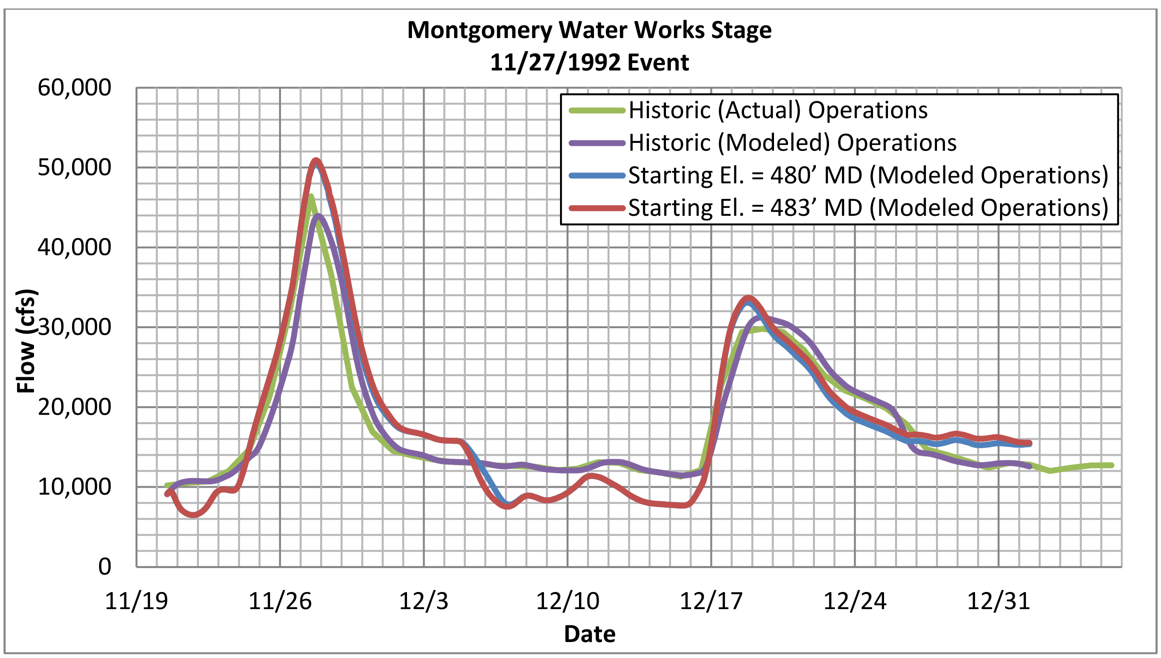


Figure 29: **Flow (in cfs) at the Montgomery Water Works (Tallapoosa River Mile 12.9) During the November 27, 1992 Peak Flow Event at the Montgomery Water Works Gage**

7. Regenerate the MWW frequency curves.

As discussed in Step #1, the published peak stages at the MWW gage were used to generate a stage-frequency relationship. This frequency curve was used as the basis for evaluating the net effect of the proposed operational changes for the Martin pool. The concept being that the events that resulted in different stages in the Tallapoosa River at the MWW gage would replace the observed peaks and a new stage-frequency relationship generated. The results would then be compared to project the potential flooding impacts with the new winter pool alternative. The evaluation process indicated that only 2 of the 9 modeled events would result in slight rises with the higher winter pool. Also, as noted above, the actual operations during the 9 events did not always strictly follow the operating criteria contained in the Project Routing Model; therefore, to make an appropriate comparison, a stage-frequency relationship with the events operated with a 481 ft msl winter pool was also generated. The two pool relationships were then compared. The following table compares the stage-frequency relationships at the MWW gage for the 481 ft msl elevation and 484 ft msl elevation (Table 2). As can be seen, the differences between these winter pool elevations are well within the 5% confidence limits of the stage-frequency analysis.

Table 2: Comparison of Stage-Frequencies at Montgomery Water Works Gage

Percent Chance	481 Pool Elevation (ft. NGVD)	484 Pool Elevation (ft. NGVD)	Difference (ft.)
0.2	186.53	186.45	-0.08
0.5	181.48	181.44	-0.04
1	177.72	177.7	-0.02
2	173.98	173.97	-0.01
5	168.97	168.98	0.01
10	165.04	165.06	0.02
20	160.83	160.86	0.03
50	154.11	154.13	0.02
80	148.72	148.73	0.01
90	146.31	146.32	0.01
95	144.5	144.5	0
99	141.47	141.46	-0.01

8. Documentation for Final Report

Qualification of Anticipated Effects of Larger Storms

In addition to answering AIR Question 2, in the Memo to Public Files dated June 19, 2014, FERC staff recommended, due to the absence of data to analyze larger storm events, that Alabama Power also include a qualitative discussion of the anticipated effects of larger storms based on (1) the results from the smaller storms; (2) hydrologic and hydraulic engineering principles; and (3) relevant, historic observations in the Martin and nearby systems.

Based on the historical operations and the Project Routing Model results of the smaller storms, Martin generally has sufficient storage to manage events with less than a 2 year return period using generation releases for the proposed winter pool elevation. As seen by the analysis in this study, these are the more common type events that would be seen in the winter pool time period. The 100 year event that was modeled in Study 12a was based on a March storm to give a worst case scenario evaluation to downstream flooding for the proposed winter pool. As shown in Study 12a, the greatest chance of the 100 year event during the winter pool period would be in December at 0.6%, which would be a return period of 167 years. Furthermore, the greatest risk of annual peak inflows is during the high flow season, March through May, which is not during the winter pool period.

The 42 historical annual peaks at the MWW gage range from a 1 year return period to a 60.6 year return period, with the March 1990 flood event as the event with the 60.6 year return period. The larger annual peak storms that occurred on the lower Tallapoosa River, during the period of record, are outside of the winter pool period (November through February) and were generally during the spring (March through May). The hydrologic climate of this region typically produces the greatest floods during the spring rain season with an occasional tropical event during the late summer and early fall period. The 9 annual peak events that occurred during the winter pool period only had return periods of 2 years or less but the frequency analysis included the other events also. While the annual peak events are used to define the annual risk of flooding, other significant events, that are smaller than the annual peak, could occur within the winter pool period during a given water year, but the annual peak is the primary indicator of flood damage during a water year. Inclusion of multiple events in a water year in the frequency analysis would not be consistent with standard hydrologic procedures and would produce biased results.

<u>Tables and Figure Requested by FERC in Schedule A</u>

FERC staff requested further information on structures downstream of Martin Dam to the MWW that would be subject to incremental flood increases resulting from increasing the winter pool at Martin Dam. In Study 12a, Alabama Power analyzed the potential downstream impacts of a proposed change in the flood control guideline by evaluating a flood event that equaled a 1% chance of exceedance (100 year return period) annual inflow to the Martin Reservoir. This event potentially affected 23 “baseline” and 18 “additional” structures downstream of Martin Dam. A “baseline” structure is one as being potentially affected under current conditions (winter pool elevation at Martin of 481 ft. msl). An “additional” structure is one identified as being potentially affected under proposed conditions (winter pool elevation at Martin of 484 ft. msl).

The request included two tables. The first table would list the identified structures, their first floor or relevant elevation, building type and the expected percent chance of exceeding the relevant elevation. Table 3 below provides this information. Determination of the percent chance of exceedance was accomplished by developing a correlation between the peak stages at each building with the stages at the MWW gage. Once the corresponding stage at the MWW gage was determined, the frequency was then determined by the historical frequency relationship at the MWW gage. It was found that linear correlations were sufficient with R^2 no less than 0.93. The 0.93 R^2 related to a single structure near Tallassee (Building ID 41) but that structure had a first floor elevation that was above the 100 year flood level (less than 1% chance of exceedance).

Other structures had R^2 values above 0.97. Since only three events were modeled during this analysis, other simulations with the 481 ft msl winter pool were used to establish the correlations. These included the 100 year inflow to Martin described in Study 12a, the 50 year inflow to Martin which was a scaled version of the 100 year inflow described in the Flood Frequency Analysis in Study 12a, the March 1990 flood, the March 2009 flood, and the 2/14/2013 peak flow event.

The second table requested by FERC staff would compare the stage frequencies for the 481 ft msl starting elevation to the 484 ft msl starting elevation. This would result in a table for each structure, or group of structures. However, as explained in Step #7 above, the comparison of the stage frequency curves at the MWW gage show that the difference in the stages for a given percent chance of occurrence is within the accuracy of the available data. This indicates that the proposed winter pool of 484 ft msl would not significantly alter the stage frequency relationship for the lower Tallapoosa River; therefore, there was no need to produce the second table.

Table 3: Buildings/Structures Potentially Affected by Current and Proposed Operations at Martin Dam

Building ID	Baseline or Additional[1]	Frequency of Event That First Impacts Structure (Percent Chance)	Building Adjacent Grade or First Floor Elevation[2]	Building Use/Type
1	Baseline	29.0	158.7	Water Works
2	Baseline	16.0	162.8	MISCIMP - Barn
3	Baseline	9.6	165.9	MISCIMP - Barn
4	Baseline	6.2	167.2	111- Single Family
5	Baseline	9.8	165.7	MANFHOME
6	Additional	<1	180.1	MANFHOME
7	Additional	7.0	167.8	111- Single Family
8	Additional	<1	180.3	MANFHOME
9	Additional	6.0	168.4	111- Single Family
10	Additional	5.0	169.3	111- Single Family
11	Additional	4.5	169.8	600- Service Shop (Low Partition)
12	Additional	5.0	169.5	600- Service Shop (Low Partition)
13	Additional	6.0	168.3	111- Single Family
14	Additional	<1	179.6	MANFHOME
15	Additional	6.5	167.9	111- Single Family
16	Additional	<1	180.2	MANFHOME
17	Baseline	5.2	170.9	610 - Office
18	Baseline	6.7	169.5	MISCIMP - Shed
19	Additional	9.1	169.4	637 - Warehouse, Storage
20	Additional	11.5	168.7	637 - Warehouse, Storage
21	Baseline	12.0	168.3	637 - Warehouse, Storage
22	Baseline	10.5	168.8	637 - Warehouse, Storage
23	Baseline	10.5	168.8	637 - Warehouse, Storage

Building ID	Baseline or Additional[1]	Frequency of Event That First Impacts Structure (Percent Chance)	Building Adjacent Grade or First Floor Elevation[2]	Building Use/Type
24	Baseline	12.0	168.3	637 - Warehouse, Storage
25	Baseline	11.7	168.5	637 - Warehouse, Storage
26	Baseline	10.5	168.8	637 - Warehouse, Storage
27	Baseline	9.8	169.3	637 - Warehouse, Storage
28	Additional	10.0	169.1	637 - Warehouse, Storage
29	Additional	10.5	168.8	637 - Warehouse, Storage
30	Additional	10.0	169.1	637 - Warehouse, Storage
31	Additional	7.8	170.3	637 - Warehouse, Storage
32	Additional	9.8	169.3	637 - Warehouse, Storage
33	Additional	7.8	170.3	637 - Warehouse, Storage
34	Additional	9.0	169.6	637 - Warehouse, Storage
35	Additional	8.9	169.7	637 - Warehouse, Storage
36	Baseline	9.0	169.6	610 - Office
37	Baseline	2.7	175.1	111- Single Family
38	Additional	2.8	170.7	610 - Office
39	Additional	8.3	169.9	600- Service Shop (Low Partition)
40	Baseline	1.7	177.1	637 - Warehouse, Storage
41	Baseline	<1	218.2	200 - Manufacturing, Light

[1] A “baseline” structure is one identified in Study 12a as being affected under current conditions (winter pool elevation at Martin of 481 ft. msl). An “additional” structure is one identified in Study 12a as being affected under proposed conditions (winter pool elevation at Martin of 484 ft. msl).

[2] For each identified structure, the lowest first floor elevation was derived from multiple sources. Thirty-eight of the 41 elevations were obtained from LiDAR data that had been acquired in 2006 during the initial relicensing efforts. Three structures were located just outside of the extent of the LiDAR data. Elevations for these three were obtained from the USGS’ National Elevation Dataset. Field surveys were conducted to factor any raised structures, and each first floor elevation was adjusted accordingly. Structures were further subdivided by type as defined by Montgomery and Elmore County tax records. Structures were also analyzed for any access limitations that may arise during the modeled flood event using a combination of aerial imagery, U.S. Census road data, and a generated flood model polygon.

FERC staff also requested a figure that includes a plot of flood depth at the affected structures versus the annual exceedance probability for the identified events at both starting elevations. This information is provided in Figure 30 for the MWW structure (Building ID 1). As discussed under Step #7, the difference between the stage frequencies was negligible; therefore, since there is no discernible shift in the curves it was not necessary to produce figures for each structure.

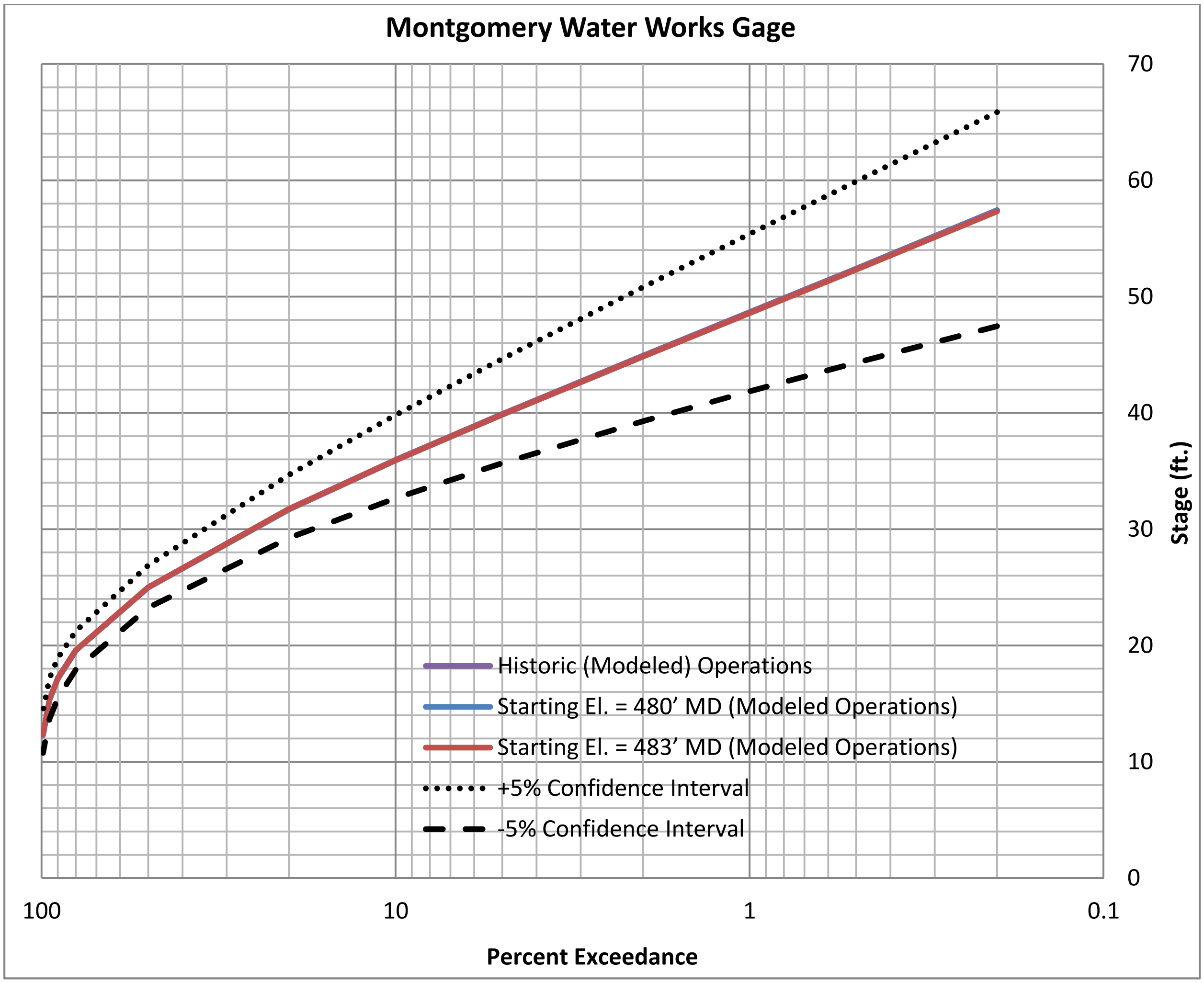


Figure 30: **Annual Exceedance Probability Curves Based on Stage Frequencies at the Montgomery Water Works Gage**

3. **Include tables and plots as described in item 2 above for any affected downstream roads and bridges. Also, describe any access limitations that would affect the structures in item 2 above and any roads or bridges which may be available to serve as alternatives to flooded roads or bridges.**

FERC staff requested further information on roads and bridges downstream of Martin Dam to the MWW that would be subject to incremental flood increases resulting from increasing the winter pool at Martin Dam. In Study 12a, Alabama Power analyzed the potential downstream impacts of a proposed change in the flood control guideline by evaluating a flood event that equaled a 1% chance of exceedance (100 year return period) annual inflow to the Martin Reservoir. Alabama Power used generated flood model polygons for this event to extract roads and bridges from U.S. Census road data. The request included the same tables and figures described in AIR #2. Table 4 provides this information.

Determination of the percent chance of exceedance was accomplished by developing a correlation between the peak stages at each low point along the road or bridge segment with the stages at the MWW gage. Once the corresponding stage at the MWW gage was determined, the frequency was then determined by the historical frequency relationship at the MWW gage. It was found that linear correlations were sufficient with minimum R^2 of 0.93. Since only three events were modeled during this analysis, other simulations with the 481 ft msl winter pool were used to establish the correlations. These included the 100 year inflow to Martin described in Study 12a, the 50 year inflow to Martin from the Flood Frequency Analysis described in Study 12a, the March 1990 flood, the March 2009 flood, and the 2/14/2013 peak flow event.

Based on the results in Step #7 under the response to AIR#2, there is no shift in the annual frequency curve from increasing the winter pool at Martin; therefore, there is no increased risk of annual inundation to the roads identified in Table 4. Because of this, there is no increase in risk to access any of the structures identified in Table 3 from the peak flow events that occurred during the study period.

Table 4: Roads/Bridges Identified to be Potentially Affected by Current and Proposed Operations at Martin Dam

Road ID	Road Description	Limits Access to Structures in Table 3?	Frequency of Event That First Impacts Structure (Percent Chance)	Elevation of Low Point Along Road Segment[1]	Road Type
1	Hunting Lodge Rd	Yes	18.69	162.81	Dirt
2	Rifle Range Rd	Yes	8.13	166.40	Paved
3	Rifle Range Rd	Yes	7.94	166.89	Paved
4	Rifle Range Rd	Yes	3.41	170.58	Paved
5	Unnamed Street	Yes	20.66	161.75	Dirt
6	Old Rifle Range Rd	Yes	35.84	159.94	Dirt
7	Davis Ln	Yes	13.64	165.51	Paved
8	Old Rifle Range Rd	Yes	20.77	163.78	Paved

Road ID	Road Description	Limits Access to Structures in Table 3?	Frequency of Event That First Impacts Structure (Percent Chance)	Elevation of Low Point Along Road Segment[1]	Road Type
9	Rifle Range Rd	Yes	5.09	173.23	Bridge
10	Peace Church Rd	Yes	5.02	172.56	Bridge
11	Brenson Branch Rd	Yes	7.94	170.15	Paved
12	Brenson Branch Rd	Yes	7.62	170.34	Paved
13	Rifle Range Rd	Yes	3.40	174.05	Bridge
14	Emerald Mountain Expy	Yes	1.16	179.41	Bridge
15	Dozier Rd	Yes	25.12	165.68	Paved
16	Dozier Rd	Yes	6.60	171.28	Paved
17	Wares Ferry Rd	Yes	8.89	169.63	Paved
18	Wares Ferry Rd	Yes	10.75	170.02	Paved
19	Unnamed Street	Yes	96.45	150.40	Dirt
20	Wares Ferry Rd	Yes	8.57	171.32	Paved
21	Unnamed Street	Yes	90.55	155.68	Dirt
22	Unnamed Street	Yes	83.11	157.41	Dirt
23	Unnamed Street	Yes	60.32	160.86	Dirt
24	Unnamed Street	Yes	52.25	161.99	Dirt
25	Jack Dr	No	8.29	166.30	Dirt
26	Unnamed Street	No	55.34	155.22	Dirt
27	Unnamed Street	No	34.22	157.62	Dirt
28	Lucys Trl	No	12.12	165.29	Paved
29	Unnamed Street	No	4.65	168.89	Dirt
30	Unnamed Street	No	3.77	169.86	Dirt
31	Unnamed Street	No	7.38	166.76	Dirt
32	Unnamed Street	No	91.42	148.74	Dirt
33	Unnamed Street	No	89.25	143.16	Dirt
34	Dozier Rd	No	9.85	168.32	Paved
35	Unnamed Street	No	13.29	168.87	Dirt
36	Eddie Tullis Dr	No	50.84	161.36	Dirt
37	Unnamed Street	No	34.37	164.40	Dirt
38	Unnamed Street	No	13.94	169.02	Dirt
39	Unnamed Street	No	9.51	170.66	Dirt
40	Unnamed Street	No	43.89	163.06	Dirt
41	Deer Range Rd	No	13.49	170.35	Dirt
42	Deer Range Rd	No	18.55	168.98	Dirt
43	Unnamed Street	No	66.72	161.06	Dirt
44	Unnamed Street	No	97.54	152.60	Dirt
45	Unnamed Street	No	27.49	164.06	Dirt
46	Unnamed Street	No	32.67	164.23	Dirt

Road ID	Road Description	Limits Access to Structures in Table 3?	Frequency of Event That First Impacts Structure (Percent Chance)	Elevation of Low Point Along Road Segment[1]	Road Type
47	Unnamed Street	No	98.86	153.00	Dirt
48	Unnamed Street	No	8.69	171.58	Dirt
49	Unnamed Street	No	54.32	162.00	Dirt
50	Unnamed Street	No	16.67	169.44	Dirt
51	Unnamed Street	No	75.20	160.63	Dirt
52	Unnamed Street	No	70.76	161.65	Dirt
53	Unnamed Street	No	49.35	165.65	Dirt
54	Unnamed Street	No	62.64	163.43	Dirt
55	Unnamed Street	No	6.94	176.32	Dirt
56	Unnamed Street	No	15.39	173.64	Dirt
57	Unnamed Street	No	33.58	169.76	Dirt
58	Unnamed Street	No	86.75	160.69	Dirt
59	Unnamed Street	No	13.84	175.49	Dirt
60	Unnamed Street	No	65.89	164.26	Dirt
61	Unnamed Street	No	6.62	178.71	Dirt
62	Unnamed Street	No	45.52	166.89	Dirt
63	Unnamed Street	No	90.61	161.40	Dirt
64	Unnamed Street	No	62.83	166.17	Dirt
65	Unnamed Street	No	32.69	171.61	Dirt
66	Unnamed Street	No	30.21	172.15	Dirt
67	Unnamed Street	No	29.68	172.27	Dirt
68	Unnamed Street	No	27.27	172.83	Dirt
69	Tysonville Loop	No	2.02	180.20	Paved
70	Alexander Rd	No	25.85	170.58	Bridge
71	Unnamed Street	No	8.33	195.03	Dirt
72	Unnamed Street	No	12.93	191.34	Dirt
73	County Road 40	No	3.81	200.44	Bridge
74	Unnamed Street	No	0.81	198.56	Dirt
75	State Hwy 229	No	16.54	193.42	Bridge
76	Taylor Rd	No	8.88	198.25	Paved
77	Taylor Rd	No	16.54	198.75	Bridge
78	State Hwy 229	No	12.82	195.55	Bridge
79	Unnamed Street	No	12.38	196.14	Dirt
80	Unnamed Street	No	11.00	200.62	Dirt
81	Unnamed Street	No	3.12	214.39	Dirt
82	Unnamed Street	No	2.18	217.35	Dirt
83	County Road 56	No	1.92	215.65	Bridge
84	State Hwy 49	No	1.70	216.60	Bridge
85	I 85N	No	0.98	191.51	Bridge
86	I 85S	No	1.18	190.48	Bridge
87	Unnamed Street	No	70.60	156.05	Dirt

Road ID	Road Description	Limits Access to Structures in Table 3?	Frequency of Event That First Impacts Structure (Percent Chance)	Elevation of Low Point Along Road Segment[1]	Road Type
88	Unnamed Street	No	6.61	171.27	Dirt
89	Unnamed Street	No	7.93	180.13	Dirt

[1] Affected downstream roads and bridges were extracted from a polyline file from the U.S. Census based on the intersection of a generated flood model polygon. Each individual segment was attributed elevation data at three points: beginning of the segment, end of the segment, and the lowest portion along the segment. Elevations were derived from multiple sources. Seventy-one of the 89 segment elevations were obtained from LiDAR data that had been acquired in 2006 during initial relicensing efforts. Eighteen segments were outside of the extent of the LiDAR data. Elevations for these segments were obtained from the USGS' National Elevation Dataset. Road segments were then analyzed for any access limitations that may arise during the modeled flood event, specifically in relation to the previously identified structures, using a combination of aerial imagery, U.S. Census road data, and a generated flood model polygon.

The second table requested by FERC staff would compare the stage frequencies for the 481 ft msl starting elevation to the 484 ft msl starting elevation. This would result in a table for each road or bridge, or group of roads and bridges. However, as explained in Step #7 in the response to AIR #2, the comparison of the stage frequency curves at the MWW gage show that the difference in the stages for a given percent chance of occurrence is within the accuracy of the available data. This indicates that the proposed winter pool of 484 ft msl would not significantly alter the stage frequency relationship for the lower Tallapoosa River; therefore, there was no need to produce the second table.

4. **Identify any additional buildings or structures that could be affected by a 50 percent increase in the flood elevations calculated in the "Flood Control Guideline Change, Modeling Analysis." Describe the building use/building type (e.g., single family, warehouse, strip mall, stand-alone retail, vacant/occupied) of these additional structures. The site of flood impact is the area of concern. The goal is to identify any significant structures in the vicinity of the area affected by a higher winter pool. The 50 percent increase is a guideline and Alabama Power can use some discretion in choosing the parameters for the evaluation if it provides its rationale.**

In Study 12a, profile elevations of the modeled flood for the Lower Tallapoosa River were given to AMEC (consulting firm) to be mapped using LIDAR and aerial photography. For the purposes of providing the information requested, Alabama Power calculated the difference between the profile elevation at the starting elevation of 481 ft msl and the profile elevation at the starting elevation of 484 ft msl and increased this difference by 50%. This new profile elevation was then provided to AMEC. AMEC then determined, using the same data used to identify the structures in Study 12a, the additional structures within this profile elevation (Table 5). The resulting report is also attached (AMEC AIR Report.pdf). Alabama Power then determined the building use/type (Table 6).

Table 5: Additional Structures Identified by Increasing Flood Elevations from Study 12a by 50%

Model Scenario	Additional Affected Structures	Additional Affected Structures by Land Use Category		
		Industrial	Commercial	Residential
483	41	3	21	17
483+50%	46	3	22	21
Additional Structures	5	0	1	4

Table 6: Building Use/Type of Additional Structures Identified by Increasing Flood Elevations from Study 12a by 50%

Building ID	Building Use/Type
1	637 – Warehouse, Storage
2	111 – Single Family
3	111 – Single Family
4	111 – Single Family
5	111 – Single Family

5. **In your August 13, 2012, filing, you stated that based on your modeled 100-year flood event, a 3-foot increase in the winter pool would result in additional spillage from the dam about 0.1 percent of the time. Please provide a similar analysis of spillage for the flow events identified in items 2 and 3 above.**

For the Study 12a, the HydroBudget was used to determine the increase to frequency of spill for the proposed alternative for the period of record. The baseline annual spill recurrence is approximately 0.85 %. In other words, for the time period of 1940 – 2007, spill occurred at Martin approximately 0.85% of the time. The frequency of spill for Martin with a 3 foot higher winter pool is approximately 0.10 % higher, meaning that spill occurred at Martin approximately 0.95% of the time. The spill analysis evaluation was performed for the period of record using the ACT unimpaired flow data set developed by the ACT/ACF Comprehensive Study technical team. The analysis was not performed on individual storms using historical data.

Spillage for the events evaluated in AIR# 2 is shown in Figures 5 through 22. The Project Routing Model shows spilling occurring for one of the 9 events identified, 2/5/1982. With the winter pool at 484 ft msl, the pool elevation reached 491 ft msl, resulting in three days of spillage. The additional spillage did not alter the downstream stage frequency relationship for the lower Tallapoosa River and would have been captured in the HydroBudget analysis of the period of record.

6. **Describe any operational measures Alabama Power is currently implementing to reduce the potential for downstream flooding. Also describe any additional operational measures that Alabama Power could implement to reduce potential impacts to downstream flooding associated with the proposed changes in the reservoir rule and flood discharge curves.**

Existing operations during high flow events are described in section 13.1.1, Exhibit H, of the application. When the reservoir is below full pool elevation (491 ft msl), Alabama Power has the ability to store floodwater to help control high flow events. As shown by Figure H-1, the winter flood control guideline elevation (December 31- February 17) is 481 ft msl. The summer flood control guideline elevation (April 28-August 30) is 491 ft msl. The reservoir is typically operated at an elevation between the flood control guideline and the operating guideline, provided there is sufficient inflow to the reservoir, and subject to use of the stored waters for Project purposes. Also, during the summer pool elevation period, the reservoir is typically operated at or near elevation 490.5 ft msl. Alabama Power routinely monitors weather conditions, and may alter its generation schedule in response to predicted precipitation. As explained in the application, during high flow events, turbine discharge is increased as the reservoir elevation rises, with spill gate operations above elevation 489 ft msl, provided however that the three-hour average rate of outflow does not exceed the concurrent three-hour rate of inflow (except to evacuate accumulated surcharge storage after peak inflow).

Also, during flood periods, Alabama Power can coordinate operations on the Tallapoosa River projects with the Corps of Engineers in connection with operations of Alabama Power's Coosa River Project and the Corps' Alabama River locks and dams for possible flood control benefits on the lower Tallapoosa River above the confluence with the Coosa River.

Proposed operations during high flow events are described in section 13.1.2, Exhibit H, of the application. Generally, operations are the same as described for current operations, except that the winter flood control guideline elevation is 484 ft msl, instead of 481 ft msl. As shown by Figure H-2, the operating guideline is one to four feet below the flood control guideline during the winter pool period, depending on the date. Accordingly, depending on actual reservoir elevations during the winter pool period (November 23rd through February 28th), there could be a decrease in flood storage during the winter pool elevation period, as compared to existing operations. Under the proposed guidelines, however, Alabama Power will have similar operational flexibility in the interest of flood control.

7. Provide the HEC-RAS files used to develop the information and tables above.

Attached to this filing are nine different versions of the Project Routing Models (one for each peak flow event) as well as the HEC-RAS project files for the three peak flow events that were necessary to model downstream. The Project Routing Models are provided as spreadsheets with the following names:

Peak Flow Event	Project Routing Model File
2/5/1982	Project Routing Model 19820205.xlxs
2/7/1985	Project Routing Model 19850207.xlxs
2/5/1988	Project Routing Model 19880205.xlxs
2/18/1992	Project Routing Model 19920218.xlxs
11/27/1992	Project Routing Model 19921127.xlxs
2/8/2002	Project Routing Model 20020208.xlxs
1/8/2007	Project Routing Model 20070108.xlxs
1/24/2012	Project Routing Model 20120124.xlxs
2/14/2013	Project Routing Model 20130214.xlxs

The HEC-RAS files are names "Lower_Tallaposa08.xxx" with the extension corresponding to the "Plan", "Geometry", and "Unsteady" columns below:

Plan	Geometry File	Unsteady Flow	Comment	Plan	Geometry	Unsteady
JAN1982(WP480)	08Lydar(Jan14)	JAN1982(WP480)	2/5/1982 event run with Martin outflow controlled to a flat 480 MD pool.	p27	g06	u22
JAN1982(WP483)	08Lydar(Jan14)	JAN1982(WP483)	2/5/1982 event run with Martin outflow controlled to a flat 483 MD pool.	p26	g06	u21
JAN1982(Hist)	08Lydar(Jan14)	JAN1982(Hist)	2/5/1982 event run with the Tallassee historical flow as the inflow.	p25	g06	u20
NOV1992(WP480)	08Lydar(Jan14)	NOV1992(WP480)	11/27/1992 event run with Martin outflow controlled to a flat 480 MD pool.	p23	g06	u18
NOV1992(WP483)	08Lydar(Jan14)	NOV1992(WP483)	11/27/1992 event run with Martin outflow controlled to a flat 483 MD pool.	p22	g06	u17
NOV1992(Hist)	08Lydar(Jan14)	NOV1992(Hist)	11/27/1992 event run with the Tallassee historical flow as the inflow.	p21	g06	u16
FEB1992(WP483)	08Lydar(Jan14)	FEB1992(WP483)	2/18/1992 event run with Martin outflow controlled to a flat 480 MD pool.	p20	g06	u15
FEB1992(WP480)	08Lydar(Jan14)	FEB1992(WP480)	2/18/1992 event run with Martin outflow controlled to a flat 483 MD pool.	p19	g06	u14
FEB1992(Hist)	08Lydar(Jan14)	FEB1992(Hist)	2/18/1992 event run with the Tallassee historical flow as the inflow.	p18	g06	u13