FINAL STUDY REPORT
EFFECT OF PROJECT OPERATIONS ON DOWNSTREAM FLOODING
RSP 3.29

CONOWINGO HYDROELECTRIC PROJECT
FERC PROJECT NUMBER 405

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EXECUTIVE SUMMARY

Exelon Generation Company, LLC has initiated with the Federal Energy Regulatory Commission (FERC) the process of relicensing the 573-megawatt Conowingo Hydroelectric Project (Conowingo Project). The current license for the Conowingo Project was issued on August 14, 1980 and expires on September 1, 2014. FERC issued the final study plan determination for the Project on February 4, 2010, approving the revised study plan with certain modifications. The final study plan determination required Exelon to determine how project operations affect flooding levels and durations downstream of the Conowingo Project. Although the current license permits water levels in Conowingo Pond to range from 101.2 to 110.2 ft NGVD 1929, the water surface elevation (WSE) in Conowingo Pond is normally maintained above 105.2 ft to allow the Muddy Run Project pumps to operate. The WSE in Conowingo Pond is generally maintained at 109.2 ft, and contains a maximum effective storage capacity of 42,300 acre-ft.

The final study plan determination required Exelon to determine how project operations may impact downstream flood elevations. In addition, Exelon was required to identify opportunities and costs of measures that may reduce downstream flooding. The study objectives are to:

1) Use a hydraulic model to estimate WSEs for a full range of flood events at Port Deposit;

2) Document the areas of inundation and flooding depths during these events;

3) Document the flow conditions during which flooding of the Port Deposit area has occurred;

4) Identify the impact of the project on downstream WSEs; and

5) Determine the operational feasibility, generation effects, and implementation costs of any procedures that might attenuate flooding conditions.

An initial study report (ISR) was filed on February 22, 2011, containing Exelon's 2010 study findings. An ISR meeting was held on March 9, 10 and 11, 2011 with resource agencies and interested members of the public. Formal comments on the ISR including requested study plan modifications were filed with FERC on April 27, 2011 by Commission Staff, several resource agencies and interested members of the public. Exelon filed responses to the ISR comments with FERC on May 27, 2011. On June 24, 2011, FERC issued a study plan modification determination order. The order specified what, if any, modifications to the ISRs should be made. For this study, FERC's June 24, 2011 order required no modifications to the original study plan. An updated study report (USR) was filed on January 23, 2012 addressing comments from stakeholders received at the March ISR meeting, those comments addressed by Exelon in the May 27, 2011 responses to ISR comments, as well as editorial and minor text changes. This final study report is being filed with the Final License Application for the Project.

This study used a hydraulic model to examine existing operating conditions during several flood events (i.e., 10, 50, 100 and 500-year floods) and under a no-dam scenario. Three alternative operating scenarios were investigated for their potential to reduce downstream flooding. The first alternative simulated drawing down Conowingo Pond prior to high-flow events arriving. The second alternative simulated the impact of targeting lower pond levels during the storm. The third alternative analyzed using the reservoir storage during the storm peak to reduce downstream flows. The no-dam scenario simulated Port Deposit stage time series to estimate what conditions would be like if Conowingo Dam did not exist.

The HEC-RAS model results indicated that none of the considered alternative operating scenarios substantially reduced downstream flooding. The first alternative was found to have no effect on downstream flooding magnitude, and only a slight reduction in flooding duration. The second alternative had no considerable impact on flooding magnitude or duration (< 15 min). The third alternative only slightly reduced flooding magnitudes (< 0.02 ft) and had very little impact on flooding duration (< 15 min). The no-dam scenario had slightly increased (0.00 to 0.08 ft) flooding magnitudes, and slightly decreased flooding durations relative to existing conditions.

It appears that Conowingo Dam operations have little impact on downstream flooding. The three alternatives investigated represent the best possible mitigation alternatives, yet they showed negligible or no improvements over the current conditions. The storage available in Conowingo Pond is not enough to mitigate even relatively small events such as the 10-yr flood. The no-dam scenario showed current flooding durations and magnitudes are only slightly different than if the dam did not exist. Significantly higher storage capacity would be required in Conowingo Pond in order for Conowingo Dam to be a viable flood control mechanism. There do not appear to be any operational changes that could be made that would reduce Port Deposit flooding for the 10, 50, 100 or 500-yr storm events.

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DEM	Digital Elevation Model
FEMA	Federal Emergency Management Agency
FERC	Federal Energy Regulatory Commission
ILP	Integrated Licensing Process
ISR	Initial Study Report
MW	Megawatt
NGO	Non-Government Organization
PAD	Pre-Application Document
PMF	Probable Maximum Flood
PSP	Proposed Study Plan
RSP	Revised Study Plan
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey
USR	Updated Study Report
WSE	Water Surface Elevation

1. INTRODUCTION

1.1 Background

Exelon has initiated with the Federal Energy Regulatory Commission (FERC) the process of relicensing the 573-megawatt (MW) Conowingo Hydroelectric Project (Project). Exelon is applying for a new project license using the FERC's Integrated Licensing Process (ILP). The current license for the Conowingo Project was issued on August 14, 1980 and expires on September 1, 2014.

Exelon filed its Pre-Application Document (PAD) and Notice of Intent (NOI) with FERC on March 12, 2009. On June 11 and 12, 2009, a site visit and two scoping meetings were held at the Project for resource agencies, local communities and interested members of the public. Following these meetings, formal study requests were filed with FERC by several resource agencies, local communities and non-government organizations (NGOs). Many of these study requests were included in Exelon's Proposed Study Plan (PSP), which was filed on August 24, 2009. On September 22 and 23, 2009, Exelon held a meeting with resource agencies, local communities, NGOs and interested members of the public to discuss the PSP.

Formal comments on the PSP were filed with FERC on November 22, 2009 by Commission staff and several resource agencies. Exelon filed a Revised Study Plan (RSP) for the Project on December 22, 2009. FERC issued the final study plan determination for the Project on February 4, 2010, approving the RSP with certain modifications.

The final study plan determination required Exelon to determine how project operations may impact downstream flood elevations. In addition, Exelon was required to identify opportunities and costs of measures that may reduce downstream flooding. The study objectives are to:

1) Use a hydraulic model to estimate WSEs for a full-range of flood events at Port Deposit;
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An initial study report (ISR) was filed on February 22, 2011, containing Exelon's 2010 study findings. An ISR meeting was held on March 9, 10 and 11, 2011 with resource agencies and interested members of the public. Formal comments on the ISR including requested study plan modifications were filed with FERC on April 27, 2011 by Commission Staff, several resource agencies and interested members of the public. Exelon filed responses to the ISR comments with FERC on May 27, 2011. On June 24, 2011, FERC issued a study plan modification determination order. The order specified what, if any, modifications to the ISRs should be made. For this study, FERC's June 24, 2011 order required no modifications to the original study plan. However, FERC's order did request that the USR include:

1) Figures for each of the model's cross-sections;

2) A stage-storage curve defining the relationship between the depth of water and storage volume for Conowingo dam to calculate the volume of Conowingo pond;

3) Figures and/or tables relating to the model calibration;

4) A table of the HEC-RAS model parameters;

5) Comparisons between modeled stages and the Conowingo USGS gage rating curve;

6) Inflow-Outflow hydrographs for alternatives 1, 2 and 3 and the no-dam scenario for the 10, 50, 100 and 500-year storm events or actual storm events to compare attenuation of flood peaks. Exelon will provide hydrographs for the 10, 50, 100 and 500-year storm events (rather than actual storm events) to be consistent with previously reported results.

An updated study report (USR) was filed on January 23, 2012. This final study report is being filed with the Final License Application for the Project.

1.2 Project Description

The Conowingo Project is the most downstream of the four hydroelectric projects located on the lower Susquehanna River. The upstream projects are Safe Harbor, Holtwood and Muddy Run, which are located at river miles 56, 32, 24 and 22, respectively. The Conowingo Project is located at river mile 10.

The Conowingo Project is a 573 MW hydroelectric facility that uses limited active storage within the 9,000 acre Conowingo Pond for generation purposes. The current license has no flood control requirements. The current license permits water levels from 101.2 to 110.2 ft (NGVD 1929), but the pond rarely fluctuates over the full range. The WSE in Conowingo Pond must be maintained above 105.2 ft for normal operation of the Muddy Run Project pumps, and above 104.2 ft for the operation of Peach Bottom Atomic Power Station. The effective storage based on the Muddy Run minimum required elevation and the maximum allowable elevation (full pond) is 42,300 acre-ft. This storage amounts to slightly less than 6 hours of generation at full capacity (86,000 cfs), assuming no inflow.

The water surface in Conowingo Pond is typically maintained at an elevation of 109.2 ft, which is considered normal pond. When inflow is above the project's generation capacity (86,000 cfs), crest gates are opened to maintain the water at normal pond. The dam spillway has 50 crest gates which are opened and closed individually by three gantry cranes located permanently on the dam. Normally two gantry cranes are active, and each takes 7 minutes to open a gate, which averages to 3.5 minutes per gate. Each gate is 38 ft wide and 22.5 ft tall, with gate crests at 86.7 ft. The latching system that holds the gates open only allows the gates to be either fully opened or fully closed. In addition to the crest gates, there are two 38 ft wide 10 ft tall regulation gates, with gate crests at 98.5 ft. Figure 1.2-1 contains a plan of the Project and sections through the dam.

A HEC-RAS hydraulic model was developed that extends from approximately 1,350 feet downstream of Holtwood Dam (the upstream limit of the Conowingo Pond) to the mouth of the Susquehanna River at the Chesapeake Bay. Currently, the model contains 28 cross sections from Conowingo Dam to the Susquehanna River mouth, and is capable of estimating WSEs for the expected flow range at these locations. The HEC-RAS hydraulic model was developed using two previous FEMA Flood Insurance Study models, digital elevation models (DEMs) from the USGS, historic bathymetric mapping, and project drawings.

Susquehanna River hydrology has been studied in previous work. In 1971, the Baltimore District of the United States Army Corps of Engineers (USACE) developed a probable maximum flood (PMF) hydrograph for the Susquehanna River at Harrisburg, PA (USACE, 1971). This study produced 2-hour hydrographs for conditions current at the time and future conditions including upstream flood storage reservoirs, which have since been fully constructed[1]. The PMF methodology was recently reviewed by Gomez and Sullivan Engineers, as part of Exelon's on-going FERC Part 12 dam safety requirements. Part of the review involved flow comparisons between the Harrisburg and Marietta USGS gages. Based on a flow and drainage area comparison between the Harrisburg and Marietta USGS gages, the review found that a factor of 1.078 could be used to prorate flows from the Harrisburg USGS gage location to Conowingo Dam.

[1] This is referring to flood control dams in the Susquehanna basin, not the hydroelectric dams on the Lower Susquehanna, which are assumed to have no flood flow influence.

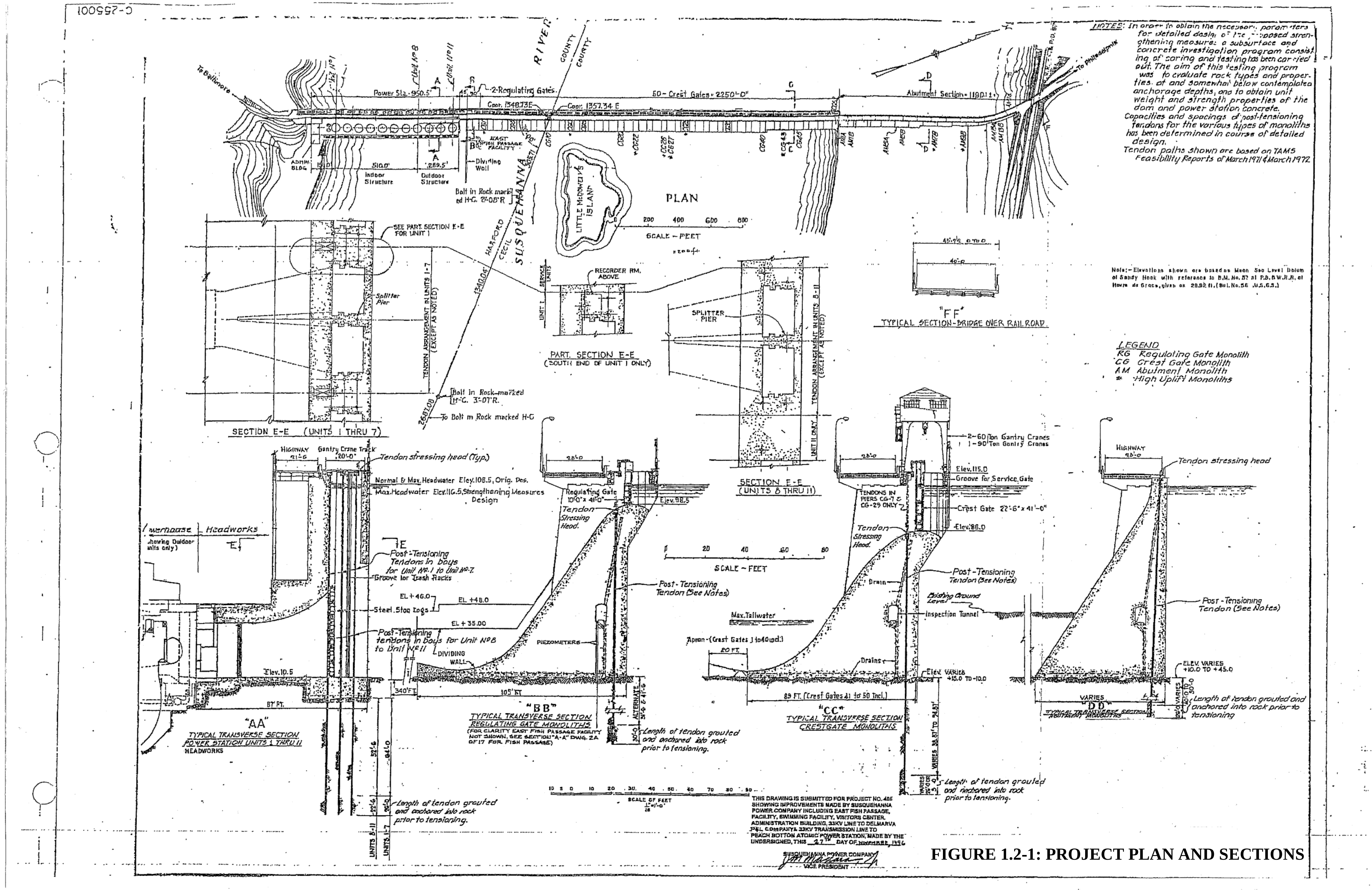

FIGURE 1.2-1: PROJECT PLAN AND SECTIONS

2. METHODOLOGY

2.1 Inflow Hydrology

The study methodology specified that Exelon would use its OASIS operations model to generate hydrographs for the 10, 50, 100 and 500-yr flood events[2], which correspond to an occurrence probability of 10, 2, 1 and 0.2 % in any given year. The RSP also specified that OASIS would be used to examine alternative pond management schemes at the four hydropower projects located on the Susquehanna River. The USACE's Hydrologic Engineering Centers River Analysis System (HEC-RAS) hydraulic model would then use the generated Conowingo Dam outflow hydrographs to determine WSEs at Port Deposit.

While the objectives and main tasks have not changed, some of the methodology specified in the RSP was modified. The OASIS model was chosen because it outputs revenue and generation estimates with generated flow hydrographs, and also because it can estimate upstream project influences (i.e. Holtwood, Safe Harbor). Conowingo Project operation guidelines state that the turbines should not be operating when river flows exceed 400,000 cfs. This means that using the OASIS model to estimate generation impacts from pond operation changes during high flow events is not necessary, since there are none. Additionally, Exelon believes that evaluating the Holtwood Project's impacts is not useful, given the relatively small storage upstream of Holtwood Dam (15,000 acre-ft), and the current expiration date of the Safe Harbor FERC license (2030) makes it unlikely that operational changes could be required at this time. Thus, the primary reasons for using OASIS for this study are irrelevant. Therefore, it was concluded that a HEC-RAS model could be used to meet study objectives by simulating Conowingo Pond management alternatives and gate operations. Since the OASIS model was not used, alternative pond management schemes upstream of Conowingo Pond were not simulated. Focus was thus placed solely on management alternatives within Conowingo Pond.

There is no USGS streamflow gage at the Conowingo Pond inlet. Therefore, flow data from other USGS gages along the Lower Susquehanna were used to estimate the hydrograph magnitude and shape for each storm event.

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[2] Events stated as a specific return interval in years are calculated from the probability of this event to occur in any given year. This is calculated given as (Return Interval in Years) = 1/(P), where P is the probability of the event occurring in a given year, based on historic flow records.

2.1.1 Flood Frequency Analysis

Storm magnitudes were generated for the 10, 50, 100 and 500-year flood events at Conowingo based on a flow proration from the Harrisburg gage frequency analysis. This study used prorated flows from the Harrisburg, PA gage (USGS No. 01570500) rather than the Marietta, PA gage (USGS No. 01576000) or Conowingo, MD gage (USGS No. 01578310) because the period of record for Harrisburg (111 years) was longer than Marietta (78 years) or Conowingo (42 years). The proration factor was estimated by comparing Harrisburg flows to Marietta flows, and then scaling the relative flow increase to the basin area increase between Harrisburg (drainage area = 24,100 mi^2) and the Conowingo Dam outlet (drainage area = 27,100 mi^2). Based on the flood frequency analysis, it was estimated that Conowingo peak flows were 1.078 times the Harrisburg peak flows. Though there are several dams between the Harrisburg gage and Conowingo Dam, none are operated as flood control dams, and they all have relatively small storage capacities in terms of flood control. Therefore it is unlikely that they would substantially impact the hydrograph of any large storm event.

Peak flows for this study's four design storms were calculated using the USGS program PKFQWIN, which follows the methodology listed in Bulletin 17B (U.S. Interagency Advisory Committee on Water Data, 1982). Streamflow data from the Harrisburg gage was used for the peak flow computations. The peak flows calculated from the Harrisburg gage were scaled using the 1.078 ratio from above to determine the peak flow at the Conowingo location. The Harrisburg and prorated Conowingo peak flow results are summarized in Table 2.1.1-1.

2.1.2 Inflow Hydrograph

The lack of a USGS streamflow gage at the Conowingo Pond inlet meant there was no individual storm inflow data. Hydrographs from upstream (Harrisburg and Marietta USGS gages) and downstream (Conowingo USGS gage) showed that each storm had a uniquely-shaped hydrograph. Rather than use a single storm's hydrograph, this study utilized the PMF hydrograph developed by the USACE to define the hydrograph shape. The hydrograph represented the general shape that most high-flow events would have in the Lower Susquehanna, and was easily scalable for different peak flows. The USACE tabulated the 144 hour hydrograph on a 2 hour increment.

The 10, 50, 100 and 500-yr storm inflow hydrographs at Conowingo were calculated by using the peak flow as calculated by the flood frequency analysis and fitting the PMF hydrograph shape to match each event's peak flow. A minimum baseflow of 40,000 cfs was used to replace any lower values that were produced by the proration. Each inflow hydrograph is shown in Figure 2.1.2-1.

2.2 Hydraulic Model

2.2.1 Model Development

WSEs were modeled using the USACE Hydrologic Engineering Centers River Analysis System (HEC-RAS) hydraulic model. The model was based on two previous FEMA Flood Insurance Studies, and used data from USGS digital elevation models (DEM), previously surveyed reservoir depths, and project drawings. The HEC-RAS hydraulic model is based on the model used in Gomez and Sullivan's IDF study model (Gomez and Sullivan 2008) The model contained 28 cross-sections downstream of Conowingo Dam, excluding two "false" boundary cross-sections. The model's cross-sections downstream of Conowingo Dam (excluding the false sections) are included in Appendix A. The model was created using NGVD 1929 elevations, which is the datum this study's results are reported in[3]. A 15 second computation time step and 10 minute output time step was used.

Since cross-sections were available for Conowingo Pond, the model dynamically routed water through Conowingo pond, rather than as a storage reservoir. This allowed a more accurate representation of water movement through the pond. As a result of this, there is no set stage-storage curve used in the model. To address FERC's request to provide Conowingo Pond's stage-storage curve, Figure 2.2.1-1 shows the stage-storage curve that was collected as part of Conowingo RSP 3.12: Water Level Management Study. The curve provides Conowingo Pond's stage-storage curve and stage-surface area curve for the FERC-permitted reservoir elevation range, which is from 101.2 ft to 110.2 ft[4].

The HEC-RAS model grouped the 50 crest gates into ten groups. One gate group is for the two regulating gates, while nine gate groups are for the 50 crest gates. The crest gate groups consist of one group of one, two, three and four gates, two groups of five gates and three groups of ten gates. This configuration allows any number (0-50) of crest gates to be open at one time. An eleventh gate group, with one gate, was used to model the railroad opening in the left abutment, but it did not pass flow in any of these simulations.

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[3] This is consistent with other Conowingo relicensing study reports, but contrasts past flooding studies conducted on the dam, which have been reported in Conowingo datum. Conowingo datum is 0.702 ft lower than the NVGD 1929 datum (i.e. NGVD 1929 datum = Conowingo Datum elevation + 0.702 ft).

[4] Muddy Run cannot fully operate in pumping mode below elevation 105.2 ft, so normal operations maintain the pond above this level. Additionally, Peach Bottom's critical pool level is defined as elevation 104.2 ft.

The gates were set up to maintain the pond at a constant elevation by changing the number of open crest gates. The number of crest gates open at any time is set by the pond inflow, which is shown in Table 2.2.1-1.

It is conservatively assumed that Conowingo ceases turbine operation (86,000 cfs maximum capacity) during the modeled high-flow events due to debris and clogging risks that could damage the turbines. Though the turbines may improve the dam's ability to pass smaller flow events, they are not explicitly modeled in this analysis. Rather, the two regulating gates were assumed to be open for the entire model run. This was done in order to pass a portion of the flow that may normally pass through the turbines.

The downstream boundary of the hydraulic model is tidally influenced. The tidal range is typically in the range of -0.5 ft to 2.5 ft. To account for this influence, the model conservatively assumed a constant downstream boundary of 2.5 ft (i.e., high tide), which slightly increased Port Deposit flood levels.

2.2.2 Model Calibration

The hydraulic model was calibrated to three storms of record as well as an average daily flow. The three storms used in the calibration were Hurricane Agnes (June 1972), Hurricane Eloise (Sept. 1975) and Hurricane Ivan (Sept. 2004). The high flow event from January 1996 was not utilized in the calibration of the hydraulic model since the event involved river ice, which is not accounted for in the model. For each of the high flow events, gate opening, reservoir water surface records and spillway rating data were used to determine the flow over the spillway, rather than the USGS gage recorded flows. For Hurricane Ivan, turbine flows were available from station operation records, which were used to determine total flow at the dam for the event. For Hurricane Eloise, an estimated constant turbine flow was added to the calculated spillway flow in order to estimate the total station flow. Turbine flow during Hurricane Agnes is assumed to be negligible due to plugging and damage to the intakes sustained during the event.

For each event, there are peak stages recorded at the Conowingo stream gage station operated by the USGS. Flows are also reported at this gaging station; however these flows differ significantly from the flows computed at Conowingo Dam under high flow conditions. It is assumed that the depths and flows for the high flow events are well outside the calibrated range of the gaging station, therefore flows computed at Conowingo Dam were used in the calibration. The calibration results are shown in Table 2.2.2-1.

Due to the fact that the only gaging station on the river, within the modeled reach, is at the downstream side of the dam, the flows for Hurricanes Agnes and Eloise and the average daily flow could only be used to calibrate the roughness coefficients in the downstream reach. For Hurricane Ivan, elevation data was collected at the tailrace of the Muddy Run project, which is at the upper end of the Conowingo Pond. The elevation data for Hurricane Ivan, in addition to an estimated travel time from Muddy Run to Conowingo Dam, based on historic project data, were used to calibrate the roughness coefficients within the Pond. The four flow events gave a wide flow range by which to calibrate the vertically varying roughness coefficients in the downstream reach. However, water surface elevations are only specifically known at the gaging station.

The model was also run using an approximate flow for the 1996 Ice Jam flood. The results of that run were compared to an approximate high water level during that storm, in the Town of Port Deposit, in order to check the downstream reach for reasonableness. The 1996 ice jam storm was not specifically used to calibrate the hydraulic model due to the fact that the model can not reproduce ice laden flows. Also, the fact that the flows during this event were rapidly changing makes it difficult to replicate the storm. The final calibrated model parameters for each cross-section downstream of Conowingo Dam are shown with the cross-section plots in Appendix A.

2.3 Inundation Mapping

Inundation maps were created in GIS using a 30-m DEM upstream of Conowingo Dam and a 1-m grid downstream of Conowingo Dam. The 30-m DEM came from the USGS seamless website (www.seamless.usgs.gov). LIDAR data was provided in the form of 2-ft contours by Harford County on the Western side of the Susquehanna. Multipoint-form LIDAR data on the Cecil County (Eastern) side of the Susquehanna was available through NOAA’s Digital Coast website. The two forms of LIDAR data were input into an ESRI terrain geodatabase to build the downstream terrain model.

TABLE 2.1.1-1: RETURN INTERVAL PEAK FLOWS

Return Period (years)	Annual occurrence probability (%)	Harrisburg Peak Flow (cfs)	Conowingo Peak Flow (cfs)
10	10	446,600	481,000
50	2	653,400	704,000
100	1	755,700	815,000
500	0.2	1,033,000	1,114,000

TABLE 2.2.1-1 : EXISTING CONDITIONS GATE OPENING SCHEME

Open Gates	Flow (cfs)	Open Gates	Flow (cfs)
0	0	26	404,550
1	17,050	27	420,050
2	32,550	28	435,550
3	48,050	29	451,050
4	63,550	30	466,550
5	79,050	31	482,050
6	94,550	32	497,550
7	110,050	33	513,050
8	125,550	34	528,550
9	141,050	35	544,050
10	156,550	36	559,550
11	172,050	37	575,050
12	187,550	38	590,550
13	203,050	39	606,050
14	218,550	40	621,550
15	234,050	41	637,050
16	249,550	42	652,550
17	265,050	43	668,050
18	280,550	44	683,550
19	296,050	45	699,050
20	311,550	46	714,550
21	327,050	47	730,050
22	342,550	48	745,550
23	358,050	49	761,050
24	373,550	50	776,550
25	389,050		

Table 2.2.2-1: Comparison of Modeled and Observed Peak WSE for HEC-RAS Model Calibration Events

		Measured Peak Stage (ft NGVD 1929)		Modeled Peak Stage (ft NGVD 1929)	
Calibation Event	Peak Flow (cfs)	At Muddy Run	Below Conowingo Dam	At Muddy Run	Below Conowingo Dam
Hurricane Agnes (1972)	987,200	N/A	41.88	128.07	42.24
Hurricane Eloise (1975)	609,500	N/A	35.91	123.12	35.96
Hurricane Ivan (2004)	565,300	123.50	35.19	122.50	35.11
Average Daily Flow	34,000	N/A	17.52	110.22	18.19

FIGURE 2.1.2-1: RETURN INTERVAL EVENT HYDROGRAPHS

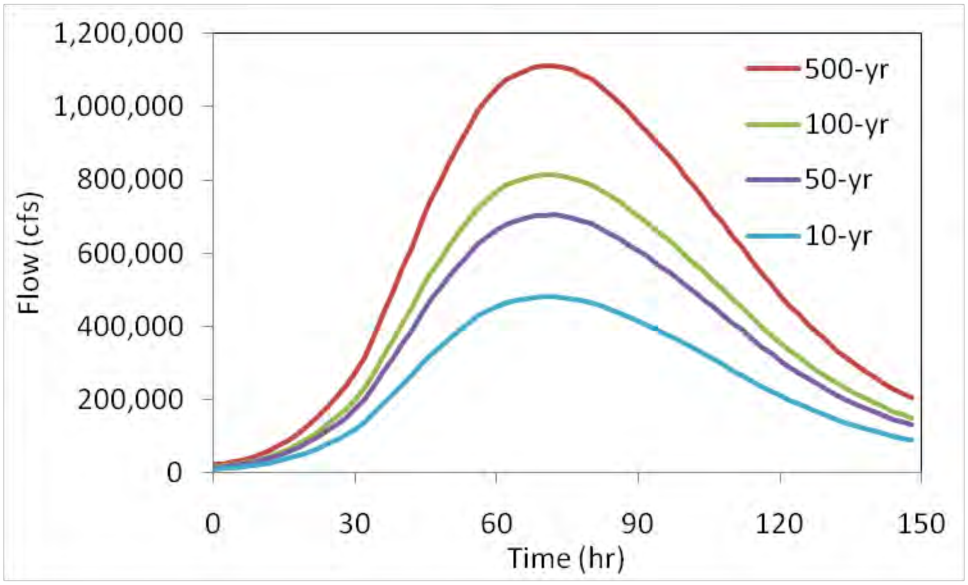


FIGURE 2.2.1-1: CONOWINGO POND STAGE-STORAGE AND STAGE-SURFACE AREA CURVE

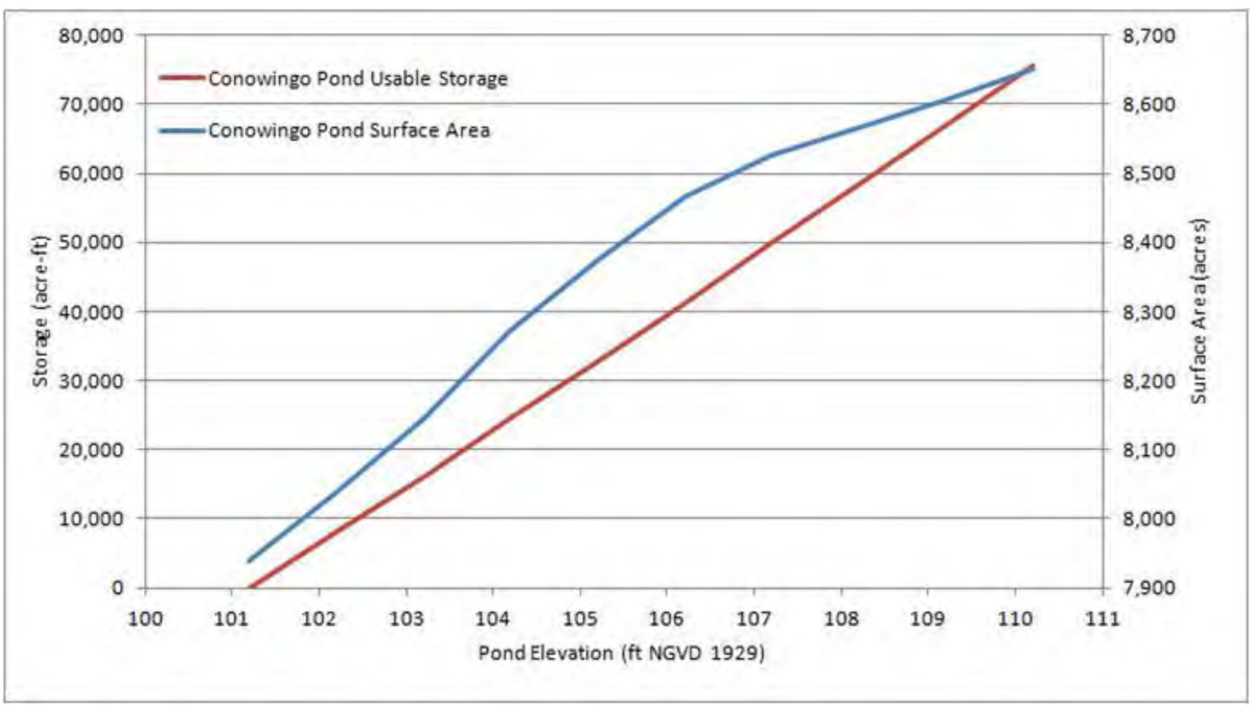
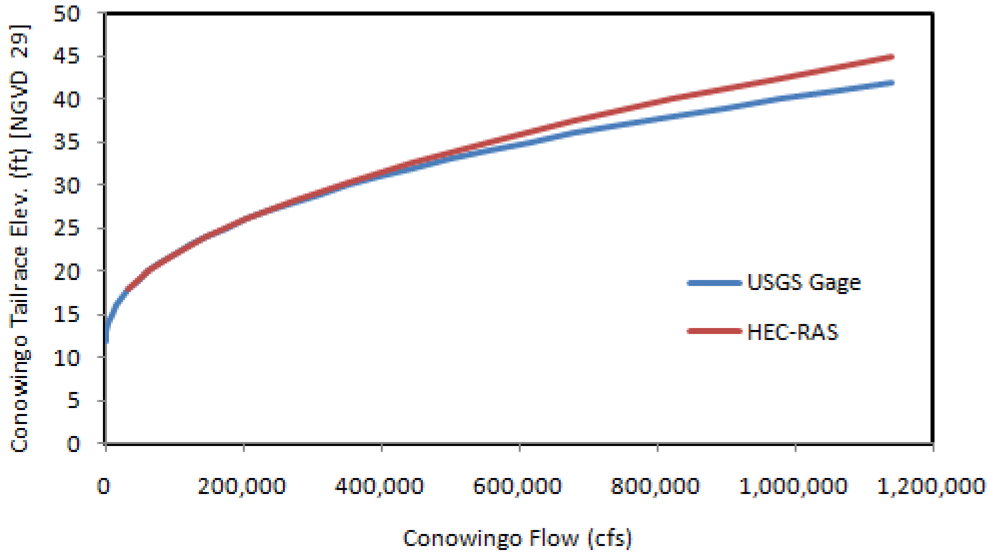


FIGURE 2.2.2-1: HEC-RAS AND USGS GAGE STAGE-DISCHARGE RELATIONSHIPS ON THE DOWNSTREAM FACE OF CONOWINGO DAM



3. RESULTS AND DISCUSSION

3.1 Existing Operations

The existing conditions hydraulic model was run assuming Conowingo Pond was at normal pond (109.2 ft) prior to the event. In addition to the design flow events, three additional flows (125,000 cfs, 250,000 cfs and 350,000 cfs) were modeled to find Port Deposit's flooding threshold. The model results are summarized for three model cross sections in Table 3.1-1. Results showed that WSEs near Port Deposit are moderately sensitive to flow changes. At Port Deposit, the WSE was nearly 10 ft higher for the 500-yr flood than it was for the 10-yr flood. This is equivalent to an additional foot of water for every 70,000 cfs on average. This appears to result from the steep bank and overbank areas.

Stage time series were created for each event (Figure 3.1-1). Each stage hydrograph shows that the WSEs increased in a stepwise manner, coinciding with crest gates opening and closing. In addition, inflow-outflow hydrographs for all modeled scenarios (existing operations, no-dam, and alternatives 1, 2 and 3) are shown in Appendix B.

Inundation maps were created to show flooding extents and depths in Port Deposit. Figure 3.1-2 shows that slight flooding began during the 250,000 cfs event, which corresponded to a WSE of 5.00 ft at Port Deposit. Port Deposit is expected to experience flooding during all four target events (Figure 3.1-3, Figure 3.1-4, Figure 3.1-5, and Figure 3.1-6).

Flooding depths at select locations throughout Port Deposit (Figure 3.1-7) are shown in Table 3.1-2. While only minor flooding and water depths are expected for the 10-yr event, the 500-yr event is expected to cause a large portion of Port Deposit to be inundated with 10 ft of water.

Using the identified 5.00 ft flooding threshold, flooding durations at Port Deposit were calculated for the target storms (Table 3.1-2). All of the events were over the flooding threshold for 72 hours (3 days) or more.

3.2 No-Dam Scenario

Hydraulic simulations were run at the four target flows to estimate the Port Deposit flooding levels as if Conowingo Dam did not exist. The model assumed channel geometry was not changed by removing the dam. Upstream and downstream boundary conditions were not changed (inflows, rating curves, etc). This was far below the flooding threshold (250,000 cfs), and it did not influence flooding durations or magnitudes.

Stage hydrographs showed that the dam slightly altered the flood timing, but did not substantially change the flood magnitude, as exemplified by the 10-yr event (Figure 3.2-1). The existing conditions tended to have an earlier rising and falling limb, but the peaks' magnitudes were nearly identical.

Table 3.2-1 compares no-dam flood magnitudes and durations to existing conditions. The results show that the no-dam scenario showed slightly higher peak stages and slightly lower flooding durations. Yet, the relative differences between the two scenarios are rather small, as neither metric changed more than 0.5% for any flow event.

3.3 Pond Management Alternatives

Several Conowingo Pond management alternatives were investigated to assess peak flood level reductions at Port Deposit. While flooding durations were compared, the limited storage available (2.0 hr at 250,000 cfs) shows that the dam cannot substantially change flooding durations that are days long, and that managing the pond to do so would be ineffective. The first alternative investigated lowering pre-storm pond elevations. The second alternative investigated maintaining the pond at lower levels throughout the storm to retain additional storage. The third alternative investigated maintaining the pond at a lower elevation for most of the storm, and then utilizing the pond storage to reduce downstream flow when inflow peaked.

The first pond management alternative investigated lowering the pre-storm pond elevation to the lowest practical level of 105.2 ft. The rationale behind this was that the pond would have more total storage capacity to buffer the peak flow. The model results showed that this had no upstream or downstream peak stage impacts, as the storage was used up in the beginning of the storm. This alternative did slightly reduce flooding durations by delaying initial flooding. This is shown in the downstream hydrograph (Figure 3.3-1). The peak WSEs did not change for the 10-yr, 50-yr, 100-yr and 500-yr event, though flooding durations were slightly reduced (Table 3.3-1). Results showed that lowering the pond prior to a high flow event provides little stand alone flood mitigation benefit.

The second pond management alternative involved changing the elevation that the pond was attempted to be maintained at throughout the storm. Instead of maintaining the pond at its normal elevation (109.2 ft), the gate opening criteria were modified so that the pond would be maintained at lower pond elevations, which would increase the available storage throughout the storm. Pond levels of 108.2, 107.2, 106.2 and 105.2 ft were targeted. Results showed that this did not substantially impact Port Deposit flooding levels (Table 3.3-2 and Figure 3.3-2), as all trials were within 0.04 ft of the existing conditions.

The third alternative investigated maintaining the reservoir at minimum pond (105.2 ft) for most of the storm, and then using the excess storage up to normal pond (109.2 ft) during the storm's peak to reduce the downstream flooding magnitude. This was accomplished by restricting the number of gates open during the storm peak. Results showed that the effect on peak flood levels was minimal (Table 3.3-3). The 10-yr event hydrograph is shown in Figure 3.3-3, which shows how the pond elevation rises during the storm peak, utilizing all available storage.

Table 3.3-4 summarizes each scenarios' estimated flooding impacts. None of the alternatives investigated had a substantial impact on flooding magnitude or duration. The best peak WSE reduction was almost negligible (0.02 ft), and the greatest flooding duration reduction was 0.83 hr (50 min) over the course of a nearly 3-day long storm.

TABLE 3.1-1: ESTIMATED EXISTING CONDITIONS PEAK WSES (FT) AT THREE CROSS SECTIONS

Event Return Interval (years)	Event Probability (%)	Peak Flow (cfs)	Upstream of Dam	Downstream of Dam	Port Deposit
1	100	125,000	109.41	23.07	2.67
1.6	64.0	250,000	109.41	27.82	5.00
3.3	30.0	350,000	109.41	30.31	6.59
10	10.0	481,400	109.42	33.40	8.87
50	2.0	704,400	109.44	37.79	12.72
100	1.0	814,600	109.61	39.64	14.45
500	0.2	1,113,600	115.82	43.99	18.71

TABLE 3.1-2: FLOODING DEPTHS AT SELECT LOCATIONS IN PORT DEPOSIT

Location	250,000 cfs Event	10-yr Event	50-yr Event	100-yr Event	500-yr Event
1	0.00	1.92	5.79	7.51	11.72
2	0.00	0.00	0.00	0.00	3.99
3	0.00	0.54	4.40	6.14	10.36
4	0.00	2.05	5.91	7.64	11.89
5	1.36	5.23	9.07	10.80	15.07

TABLE 3.1-3: EXISTING OPERATIONS FLOODING DURATIONS

Event Return Interval (years)	Event Probability (%)	Peak Flow (cfs)	Flooding Duration (hr)[5]
10	10.0	481,400	72.00
50	2.0	704,400	90.83
100	1.0	814,600	97.33
500	0.2	1,113,600	111.33

[5] The flooding durations are calculated as the time that Port Deposit WSEs are above 5.00 ft.

TABLE 3.2-1: EXISTING CONDITIONS FLOODING DURATION AND MAGNITUDE VERSUS NO DAM

Event Return Interval (years)	Event Probability (%)	Peak Flow (cfs)	Existing Conditions		No Dam		Reduction	
			Peak WSE (ft)	Duration (hr)	Peak WSE (ft)	Duration (hr)	Peak WSE (ft)	Duration (hr)
10	10.0	481,400	8.87	72.00	8.90	71.67	-0.03	0.33
50	2.0	704,400	12.72	90.83	12.72	90.67	0.00	0.16
100	1.0	814,600	14.45	97.33	14.47	97.17	-0.02	0.16
500	0.2	1,113,600	18.71	111.33	18.79	111.17	-0.08	0.16

TABLE 3.3-1: ALTERNATIVE 1 FLOOD MAGNITUDE AND DURATION COMPARISON

Event Return Interval (years)	Event Probability (%)	Peak Flow (cfs)	Existing Conditions		Alternative 1		Reduction	
			Peak WSE (ft)	Duration (hr)	Peak WSE (ft)	Duration (hr)	Peak WSE (ft)	Duration (hr)
10	10.0	481,400	8.87	72.00	8.87	71.17	0.00	0.83
50	2.0	704,400	12.72	90.83	12.72	90.33	0.00	0.50
100	1.0	814,600	14.45	97.33	14.45	96.83	0.00	0.50
500	0.2	1,113,600	18.71	111.33	18.71	110.66	0.00	0.66

TABLE 3.3-2: ALTERNATIVE 2 FLOOD MAGNITUDE AND DURATION COMPARISON

Event Return Interval (years)	Event Probability (%)	Peak Flow (cfs)	Existing Conditions		Alternative 2		Reduction	
			Peak WSE (ft)	Duration (hr)	Peak WSE (ft)	Duration (hr)	Peak WSE (ft)	Duration (hr)
10	10.0	481,400	8.87	72.00	8.91	71.66	-0.04	0.16
50	2.0	704,400	12.72	90.83	12.70	90.67	0.02	0.16
100	1.0	814,600	14.45	97.33	14.45	97.16	0.00	0.16
500	0.2	1,113,600	18.71	111.33	18.71	111.17	0.00	0.16

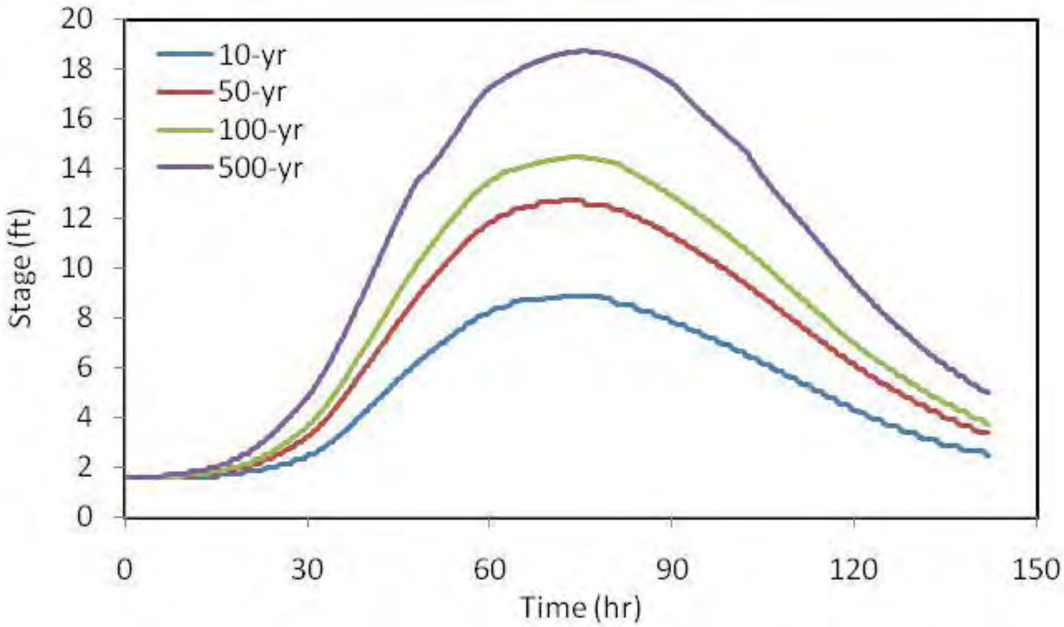
TABLE 3.3-3: ALTERNATIVE 3 FLOOD MAGNITUDE AND DURATION COMPARISON

Event Return Interval (years)	Event Probability (%)	Peak Flow (cfs)	Existing Conditions		Alternative 3		Reduction	
			Peak WSE (ft)	Duration (hr)	Peak WSE (ft)	Duration (hr)	Peak WSE (ft)	Duration (hr)
10	10.0	481,400	8.87	72.00	8.86	71.83	0.01	0.17
50	2.0	704,400	12.72	90.83	12.70	90.67	0.02	0.16
100	1.0	814,600	14.45	97.33	14.45	97.16	0.00	0.16
500	0.2	1,113,600	18.71	111.33	18.71	111.17	0.00	0.16

TABLE 3.3-4: 10-YR EVENT FLOODING MAGNITUDE AND DURATIONS - POND MANAGEMENT ALTERNATIVES COMPARED TO EXISTING CONDITIONS

Scenario	Peak WSE (ft)	Flooding Duration (hr)	Peak WSE Reduction (%)	Duration Reduction (%)
Existing Conditions	8.87	72.00	N/A	N/A
No Dam	8.90	71.67	-0.34	0.46
Alternative 1	8.87	71.17	0.00	1.15
Alternative 2	8.91	71.66	-0.45	0.47
Alternative 3	8.86	71.83	0.11	0.24

FIGURE 3.1-1: EXISTING CONDITIONS PORT DEPOSIT STAGE TIME SERIES



Exelon.

N

EXELON GENERATION COMPANY, LLC
CONOWINGO HYDROELECTRIC PROJECT
PROJECT NO. 405

1 inch = 250 feet

0 125 250 500 Feet

Figure 3.1-4
50 Yr Event Inundation Area

Path: X:\GISMaps\project_maps\study_plan\conowingo\Study_3.29\C.29 Figure 3.1-4.mxd

Exelon.

N

EXELON GENERATION COMPANY, LLC
CONOWINGO HYDROELECTRIC PROJECT
PROJECT NO. 405

1 inch = 250 feet

0 125 250 500 Feet

Figure 3.1-3
10 Yr Event Inundation Area

Path: X:\GISMaps\project_maps\study_plan\conowingo\Study_3.29\C.29 Figure 3.1-3.mxd

EXELON GENERATION COMPANY, LLC
CONOWINGO HYDROELECTRIC PROJECT
PROJECT NO. 405

1 inch = 250 feet

0 125 250 500 Feet

N

Exelon.

Figure 3.1-4
50 Yr Event Inundation Area

Path: X:\GISMaps\project_maps\study_plan\conowingo\Study_3.29\C.29 Figure 3.1-4.mxd

Exelon.

N

EXELON GENERATION COMPANY, LLC
CONOWINGO HYDROELECTRIC PROJECT
PROJECT NO. 405

1 inch = 250 feet

0 125 250 500 Feet

Figure 3.1-5
100 Yr Event Inundation Area

Path: X:\GISMaps\project_maps\study_plan\conowingo\Study_3.29\C.29 Figure 3.1-5.mxd

Exelon.

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EXELON GENERATION COMPANY, LLC
CONOWINGO HYDROELECTRIC PROJECT
PROJECT NO. 405

1 inch = 250 feet

0 125 250 500 Feet

Figure 3.1-6
500 Yr Event Inundation Area

Path: X:\GISMaps\project_maps\study_plan\conowingo\Study_3.29\C.29 Figure 3.1-6.mxd

N

1 inch = 250 feet

0 125 250 500 Feet

EXELON GENERATION COMPANY, LLC
CONOWINGO HYDROELECTRIC PROJECT
PROJECT NO. 405

Figure 3.1-7
Flooding Depth Locations

Path: X:\GISMaps\project_maps\study_plan\conowingo\Study_3.29\C.29 Figure 3.1-7.mxd

FIGURE 3.2-1: STAGE TIME SERIES COMPARING EXISTING CONDITIONS TO A NO DAM SCENARIO

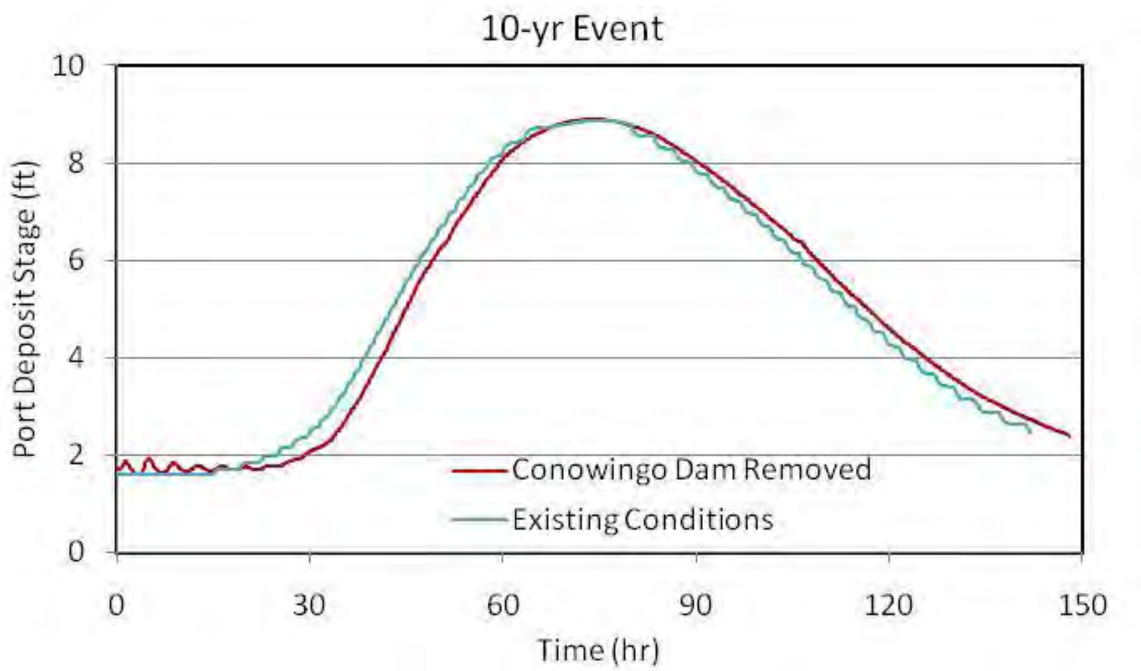


FIGURE 3.3-1: ALTERNATIVE ONE-PORT DEPOSIT STAGE TIME SERIES COMPARISON

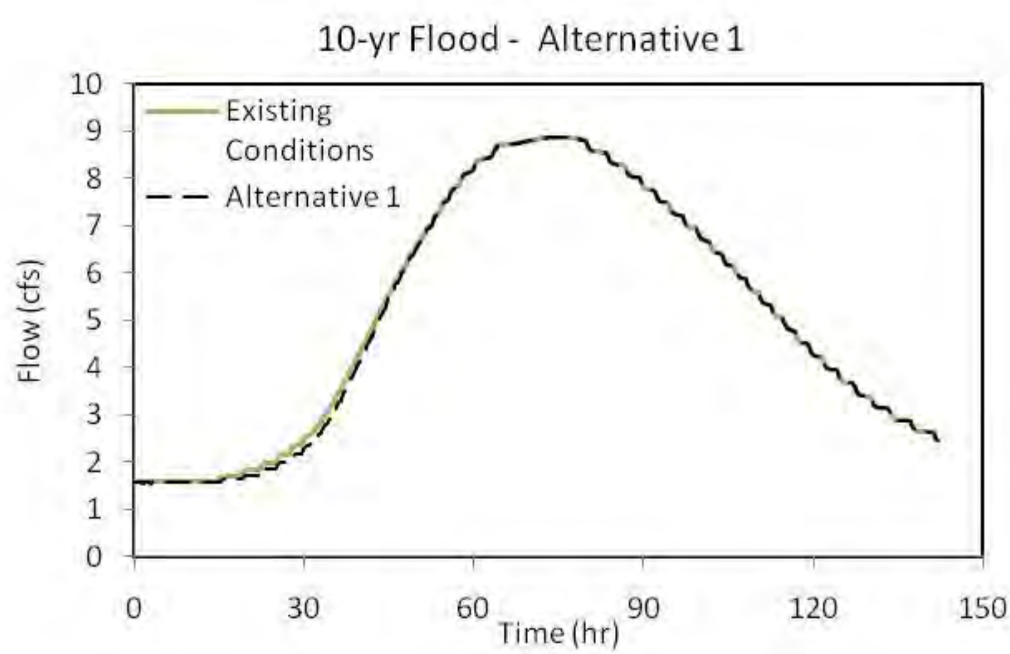


FIGURE 3.3-2: ALTERNATIVE TWO RESULTS SHOWING CONOWINGO DOWNSTREAM PEAK WSES FOR DIFFERENT INITIAL CREST GATE OPENING ELEVATIONS. ELEVATION 109.2 (FAR RIGHT) REPRESENTS EXISTING CONDITIONS

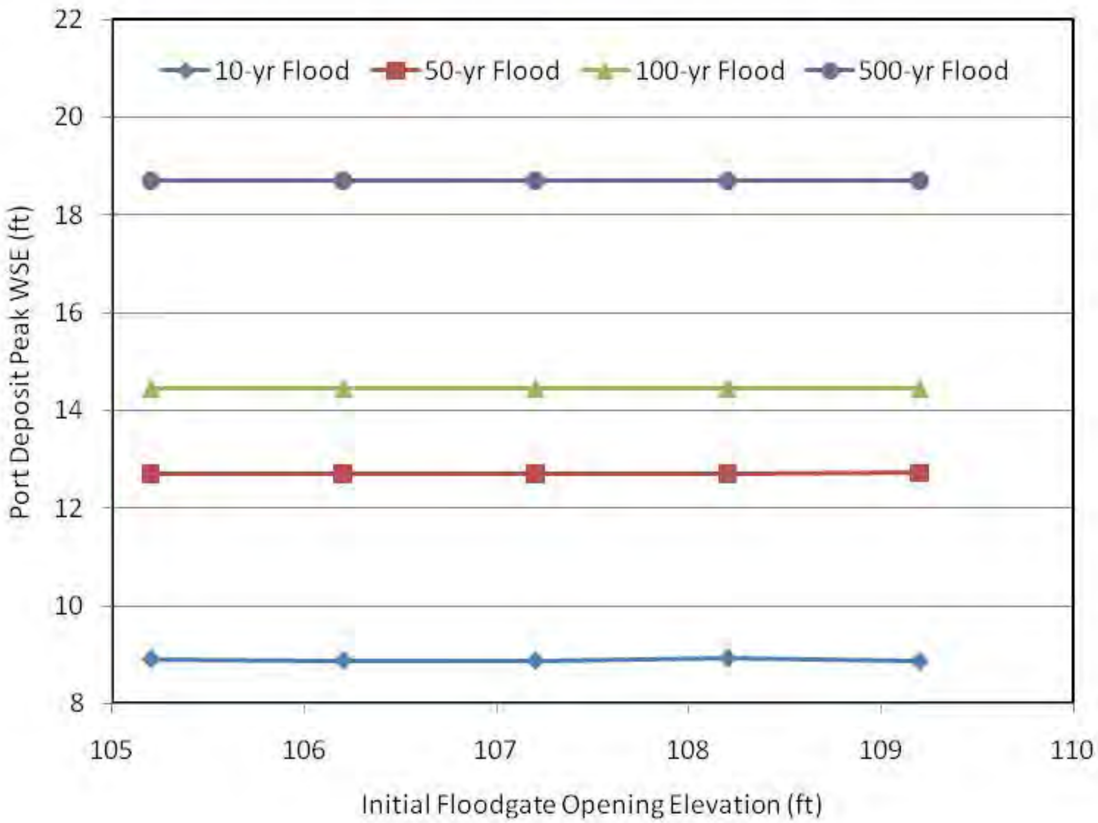
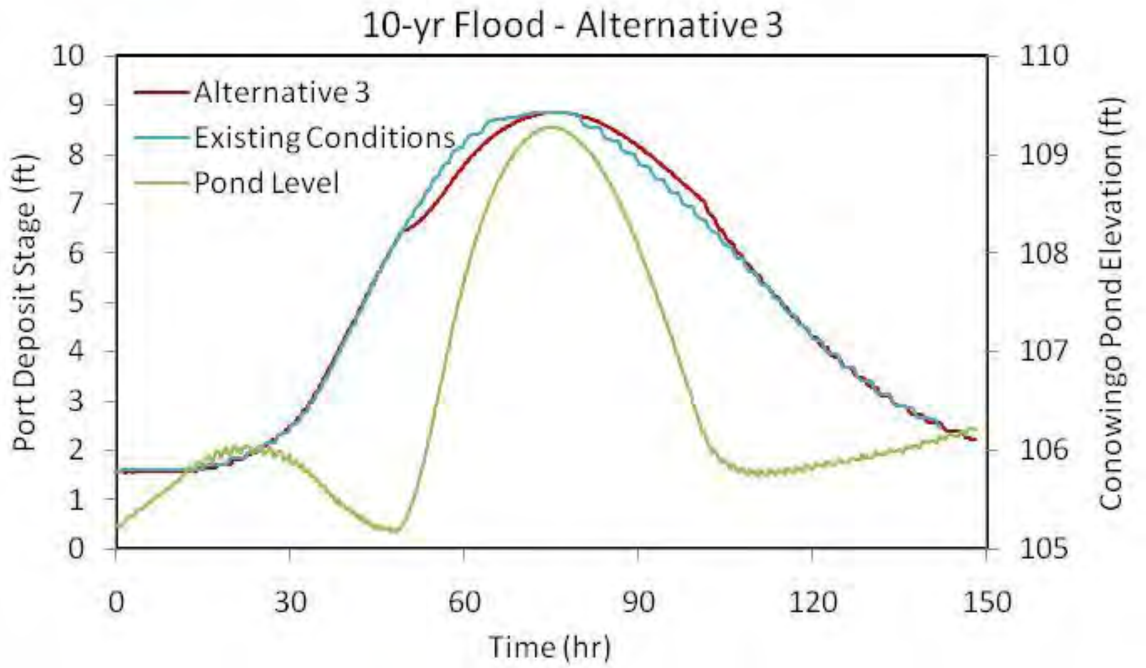


FIGURE 3.3-3: ALTERNATIVE THREE RESULTS COMPARED TO EXISTING CONDITIONS. POND LEVEL SHOWN IS FOR ALTERNATIVE 3.



4. CONCLUSIONS

The existing condition results showed that Port Deposit is susceptible to flooding over a wide range of flows. Overall the lower flow modeling predicts that some minor flooding occurs at 250,000 cfs, and more major inundation begins to occur between 350,000 cfs and 481,000 cfs (10-yr event). While less frequent flood events (100-yr, 500-yr) would cause widespread flooding, relatively frequent flows such as the 10-yr event would also cause moderate flooding in downtown Port Deposit. Flooding durations were on the scale of days for the target events, with longer floods associated with larger storms.

The first alternative pond management scheme of lowering the reservoir to 105.2 ft before high flows arrive was ineffective at reducing downstream flooding. Even smaller events such as the 10-yr flow were not influenced. This is logical, as Figure 3.3-1 shows that the hydrograph is over 400,000 cfs for several hours and was at 481,000 cfs for 2 hours. Even if the pond was at its lowest possible level (105.2 ft) just prior to the peak flow, 481,400 cfs would fill the reservoir to full pond in slightly less than 65 minutes. For the 50-yr, 100-yr and 500-yr events, the reservoir would fill in 44, 38 and 28 minutes at peak flow, respectively. This alternative does slightly reduce the total flooding duration by approximately 50 minutes, which the hydrograph shows comes from delaying flood conditions at the storm beginning.

The second alternative investigated simulating the effects of maintaining Conowingo Pond at levels lower than normal pond throughout the storm to increase pond storage. The results showed nearly negligible (0.01 to -0.04 ft) peak flood elevation changes for the 10-yr and 50-yr events. For the 100-yr and 500-yr events, the water volume flowing through the river was too much for altered crest gate operations to have any impact. This alternative's overall impact is rather small and does not represent a viable flood reduction method.

The third alternative investigated using the pond storage during peak flows by restricting the number of open crest gates during the storm peak. This filled the pond and used available storage during the peak flow, but only resulted in slightly decreased downstream peak elevations. This is because though no more crest gates were opened, the open crest gates passed more flow as the pond level rose. In order to hold reservoir output to a constant flow, crest gates would have to be closed while pond inflow was still rising. This is not a practical or safe way to operate the dam during flood conditions, as it is possible that too many gates could be closed, or inflows to Conowingo Pond could be underestimated in real-time. If pond levels rise above the top of the gates they can no longer be opened. This could potentially lead to overtopping the dam, which is a major safety hazard. The third alternative shows that the nearly negligible peak stage decrease is likely the most the dam could reduce downstream flooding levels, given the Project's equipment and operation methods.

Part of the final study objective was to determine the generation effects and implementation costs of any procedures that might attenuate flooding conditions. Since the station does not operate the turbines when the flow is above 400,000 cfs, and the four target storms all had peaks above 400,000 cfs, there are no generation effects. Since there would be no generation loss, there would likewise be no implementation costs beyond opening and closing the crest gates, which is normally done throughout a storm already.

Overall, the study results show that the main reason why Conowingo Dam is not an effective flood mitigation tool is the limited amount of storage in Conowingo Pond. The pond's actively used storage is small relative to the flows experienced in the river. The three alternatives investigated represented a wide range of operational changes that could be made to Conowingo Dam, but none of these changes would substantially reduce flooding in Port Deposit.

5. REFERENCES

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APPENDIX A: HEC-RAS MODEL CROSS-SECTIONS DOWNSTREAM OF CONOWINGO DAM

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 2410 ft	Station 5085 ft
15	0.068	0.035	0.068
24	0.065	0.034	0.065
40	0.065	0.033	0.065

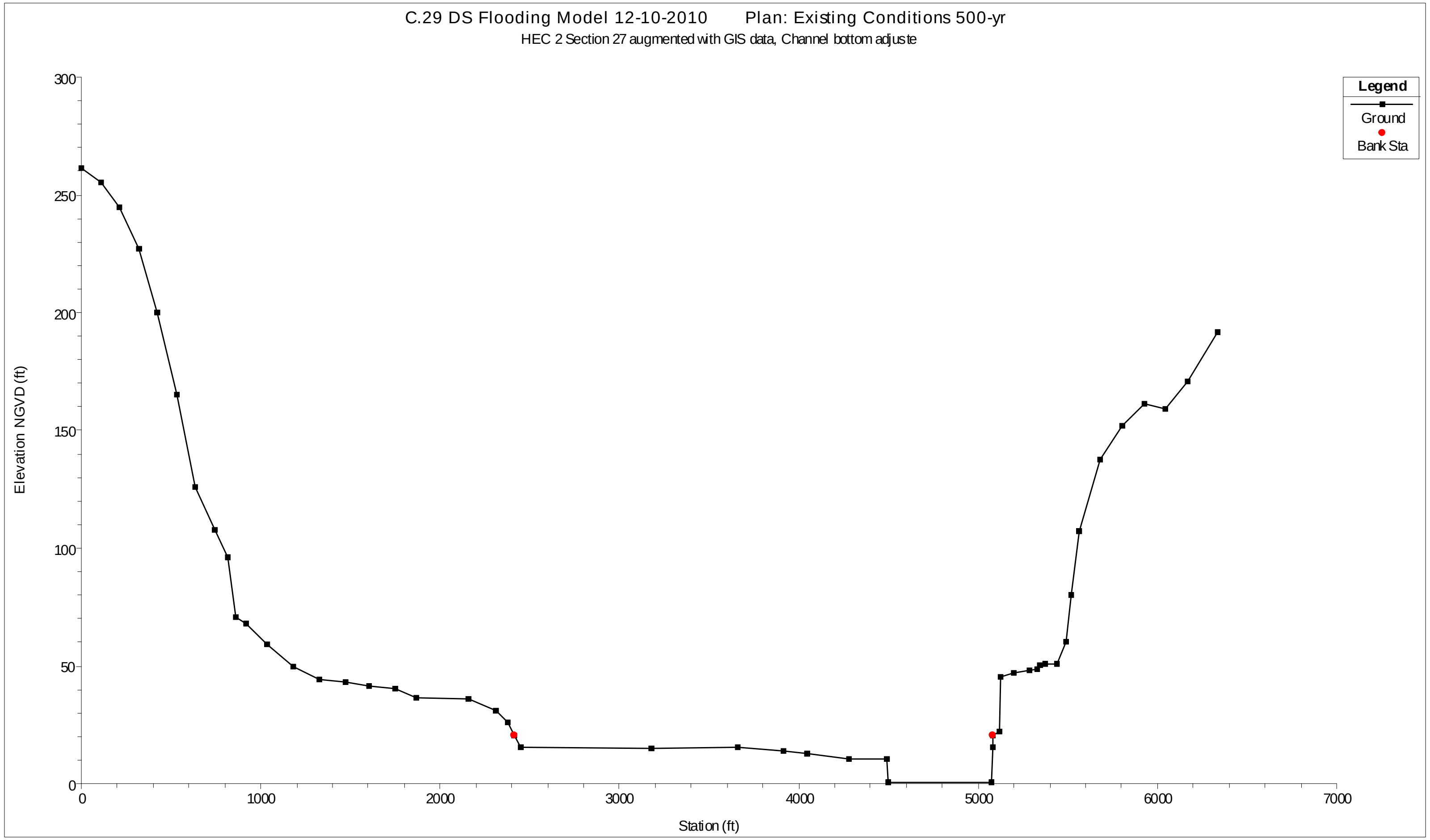


Figure 1: River Station 49609

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 2760 ft	Station 5320 ft
15	0.068	0.035	0.068
23	0.065	0.034	0.065
40	0.065	0.033	0.065

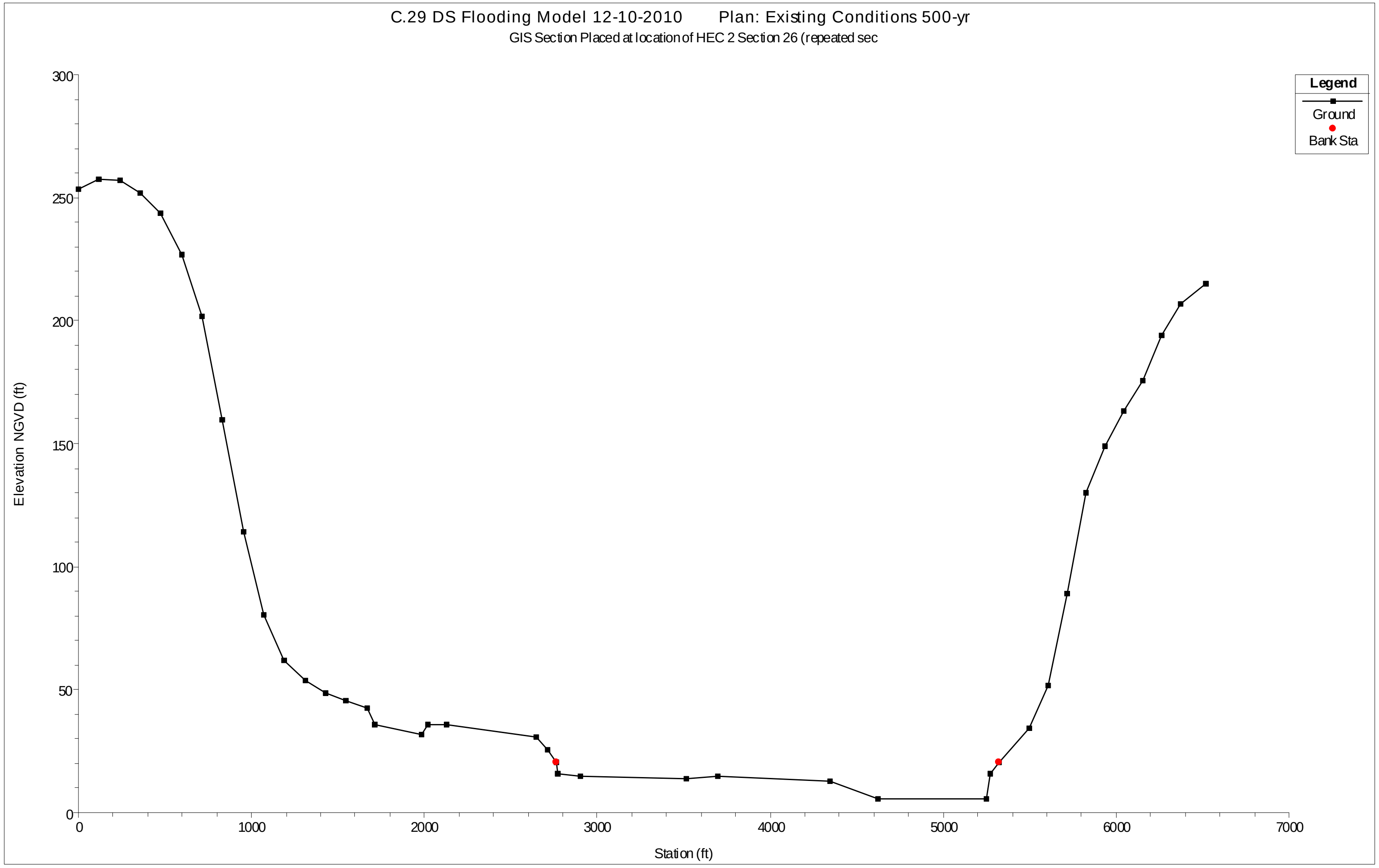


Figure 2: River Station 48888

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 2785 ft	Station 5391 ft
14	0.068	0.035	0.068
23	0.065	0.034	0.065
39	0.065	0.033	0.065

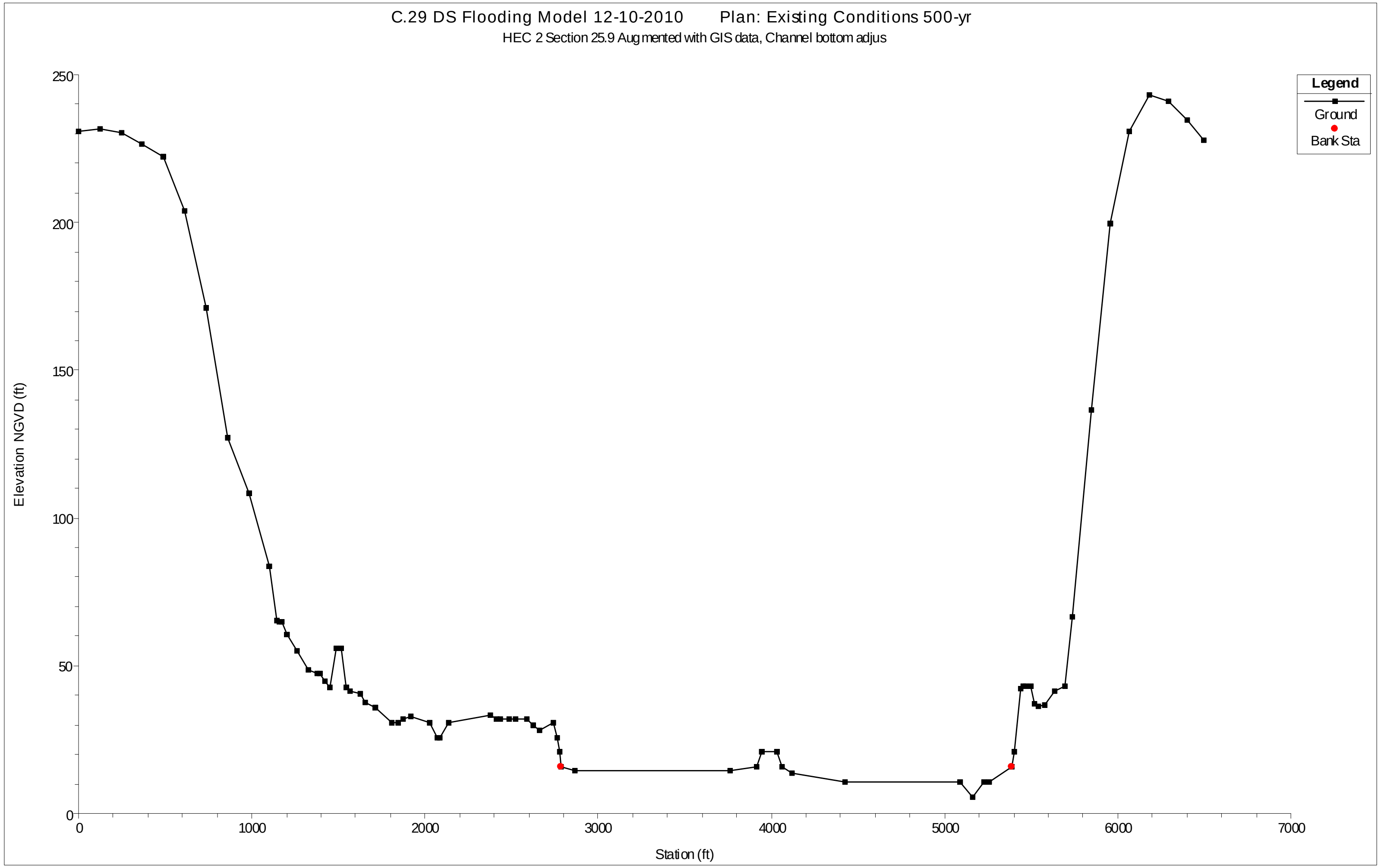


Figure 3: River Station 48135

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 2770 ft	Station 4600 ft	Station 4830 ft	Station 5521 ft
13	0.068	0.035	0.050	0.035	0.068
23	0.065	0.034	0.048	0.034	0.065
39	0.065	0.033	0.043	0.033	0.065

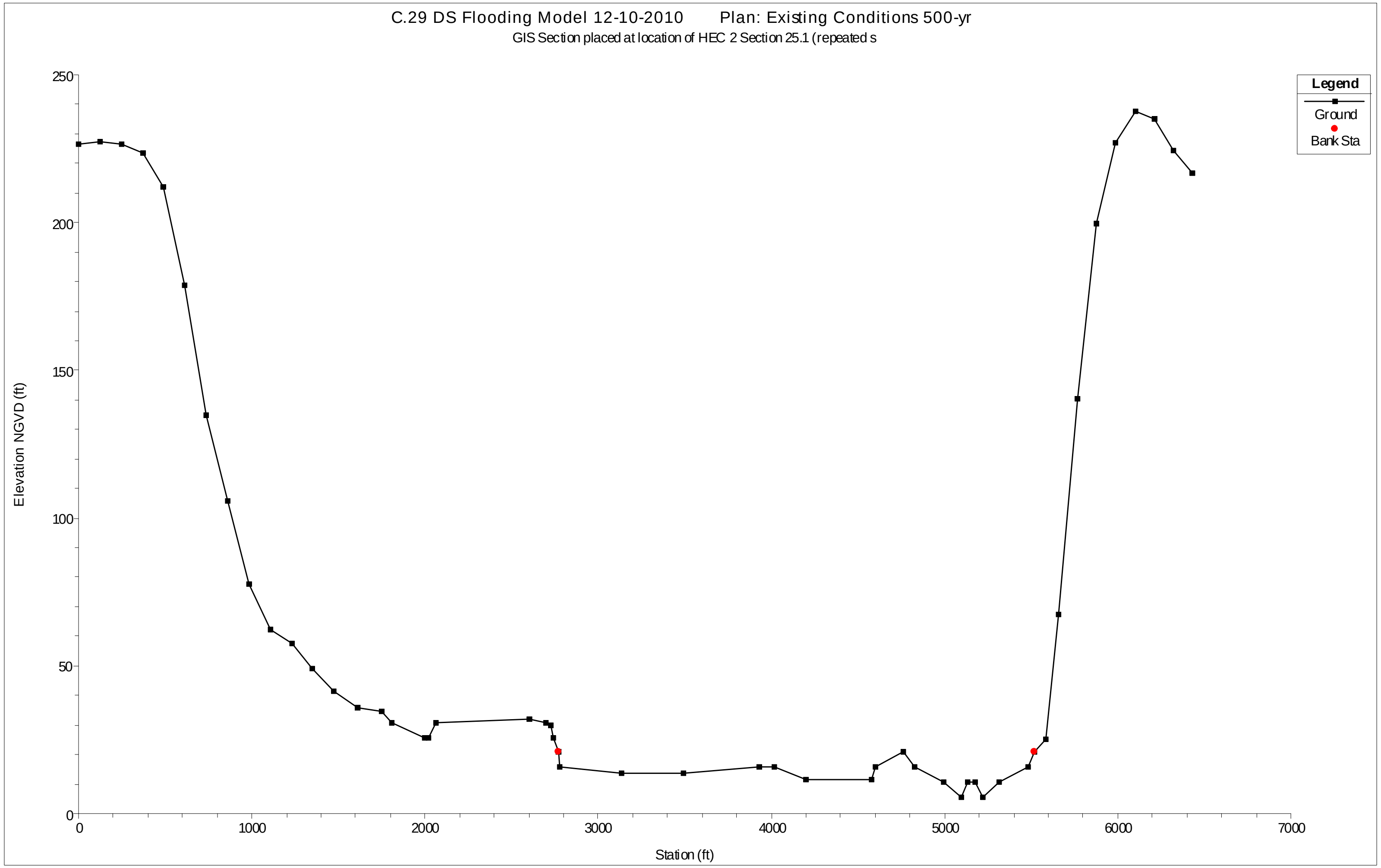


Figure 4: River Station 47990

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 2965 ft	Station 4720 ft	Station 5035 ft	Station 5786 ft
12	0.068	0.036	0.050	0.036	0.068
23	0.065	0.034	0.048	0.034	0.065
38	0.065	0.034	0.043	0.034	0.065

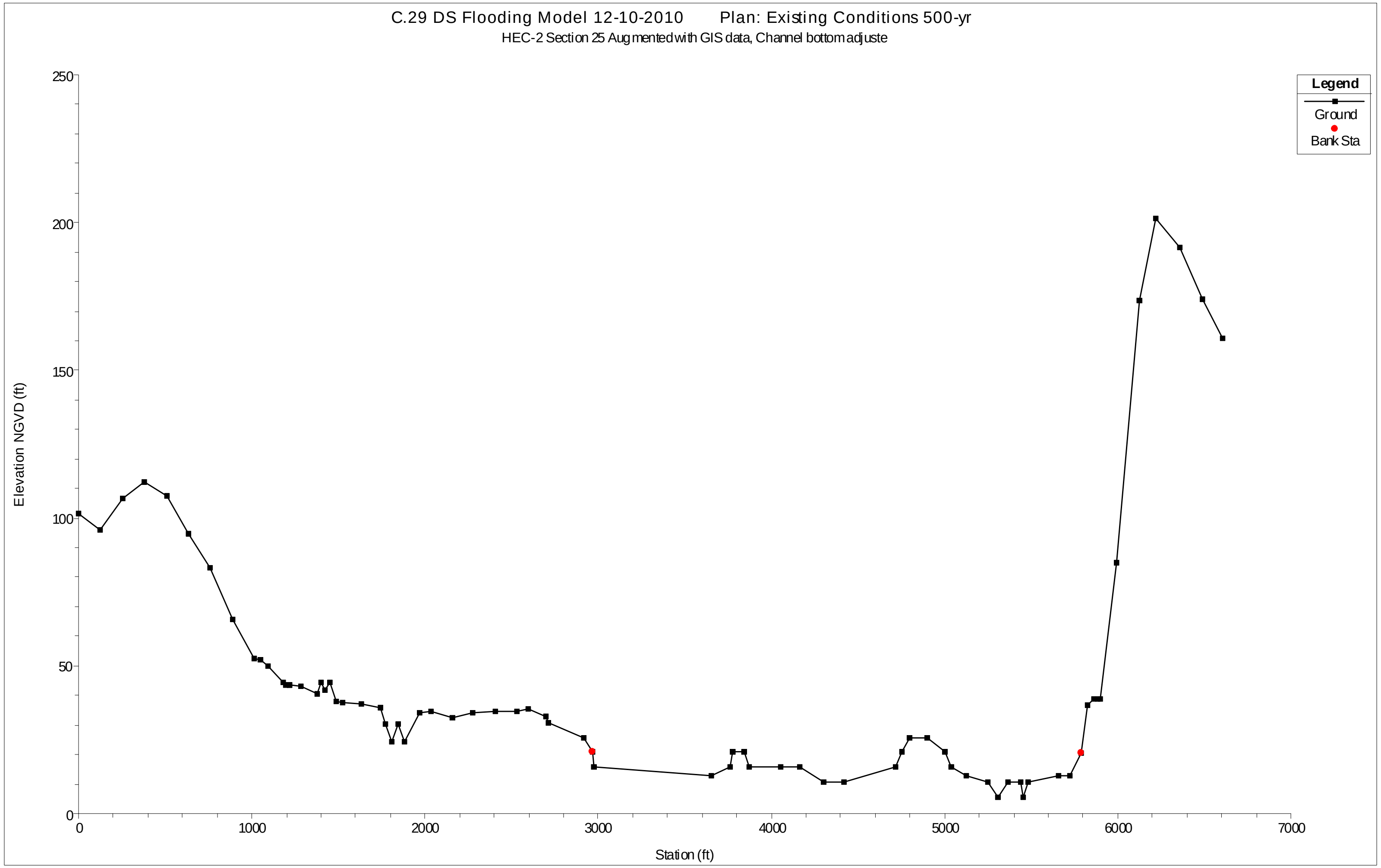


Figure 5: River Station 47243

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 1694 ft	Station 4951 ft
11	0.068	0.036	0.068
22	0.065	0.034	0.065
38	0.065	0.034	0.065

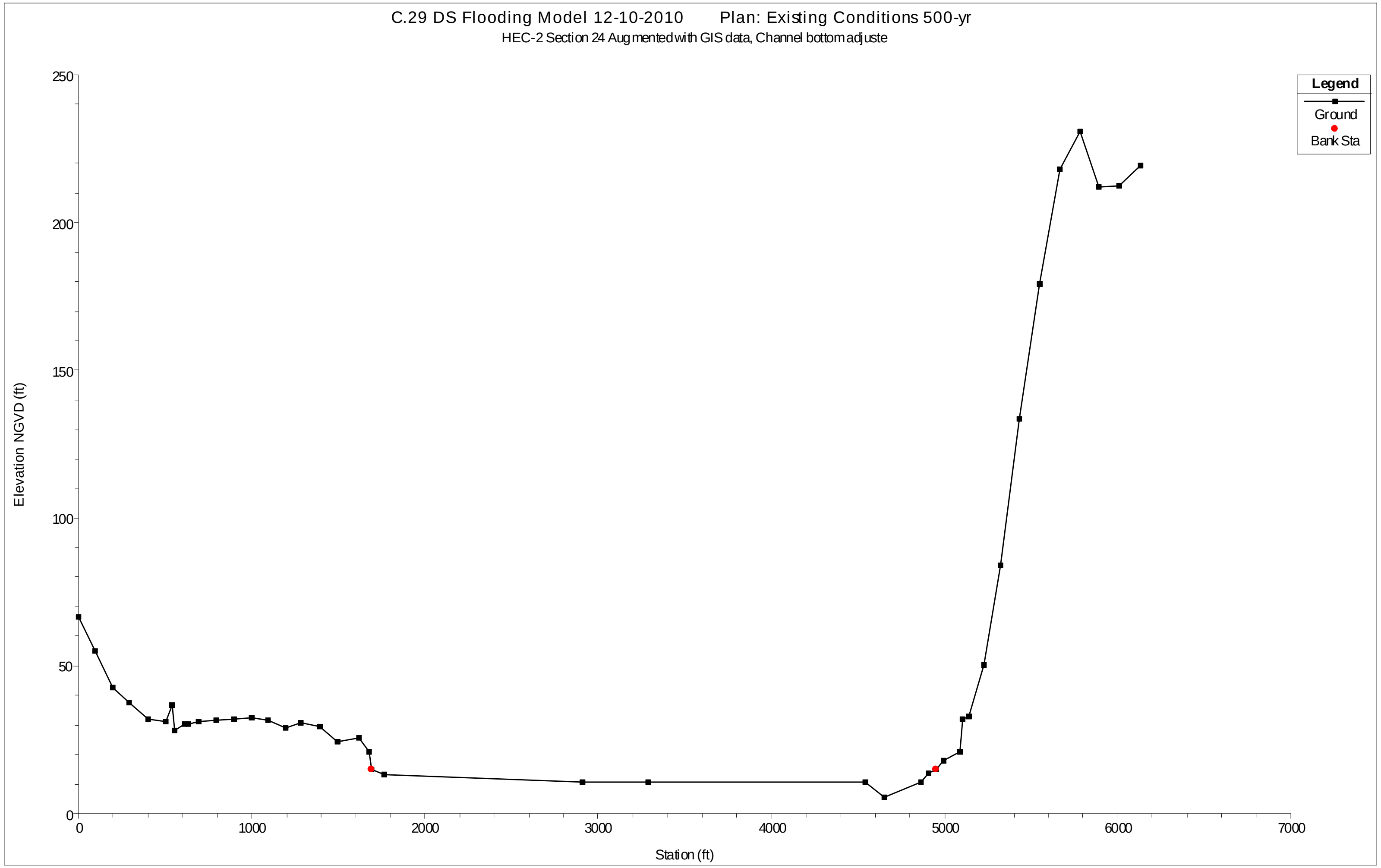


Figure 6: River Station 45442

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 2920 ft	Station 5590 ft
8.5	0.068	0.036	0.075
22	0.065	0.035	0.070
37	0.065	0.034	0.070

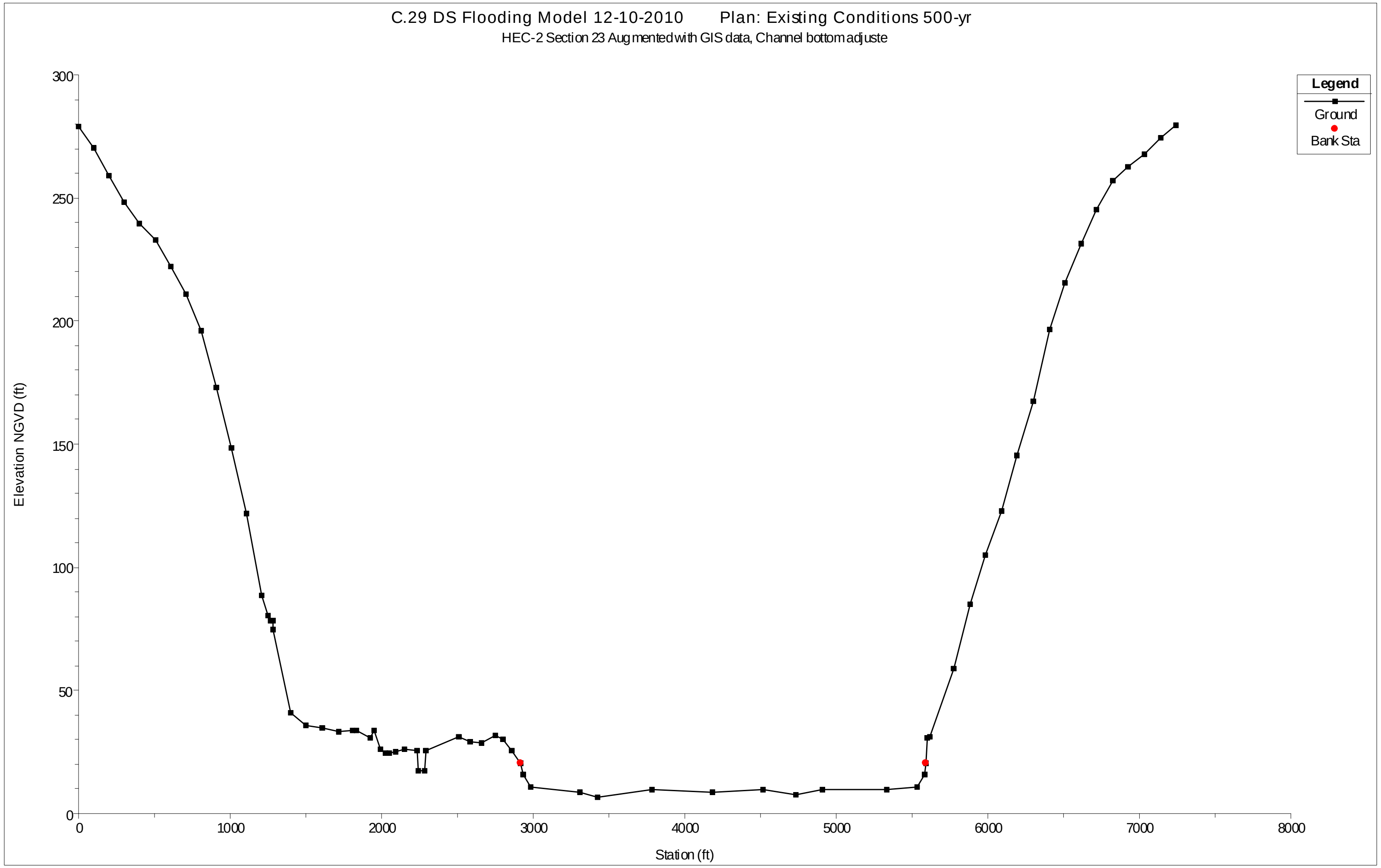


Figure 7: River Station 42929

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 2210 ft	Station 5285 ft
8	0.070	0.042	0.075
18	0.065	0.036	0.070
36	0.065	0.034	0.070

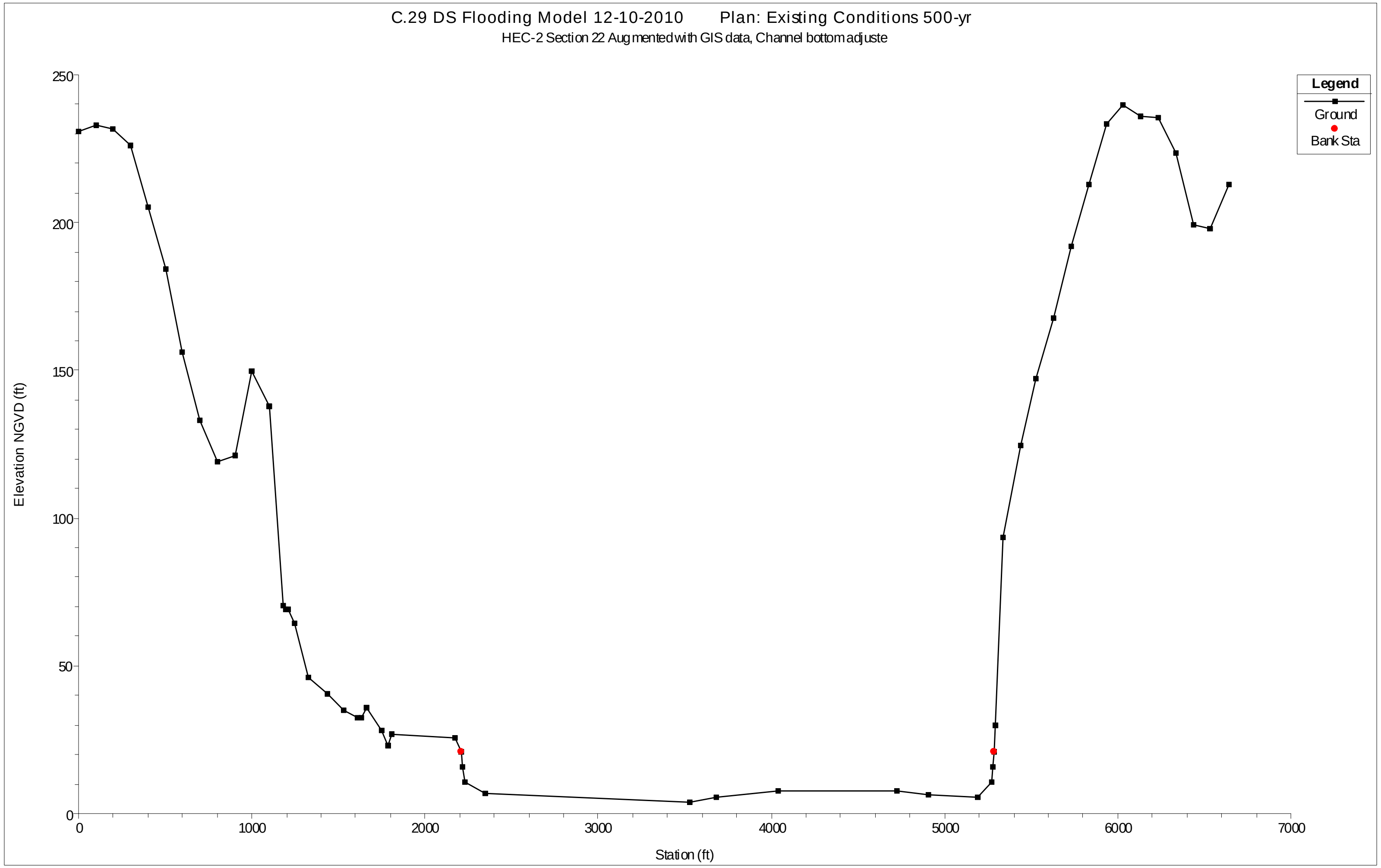


Figure 8: River Station 40670

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 2041 ft	Station 4977 ft
7	0.070	0.043	0.075
18	0.065	0.037	0.070
35	0.065	0.035	0.070

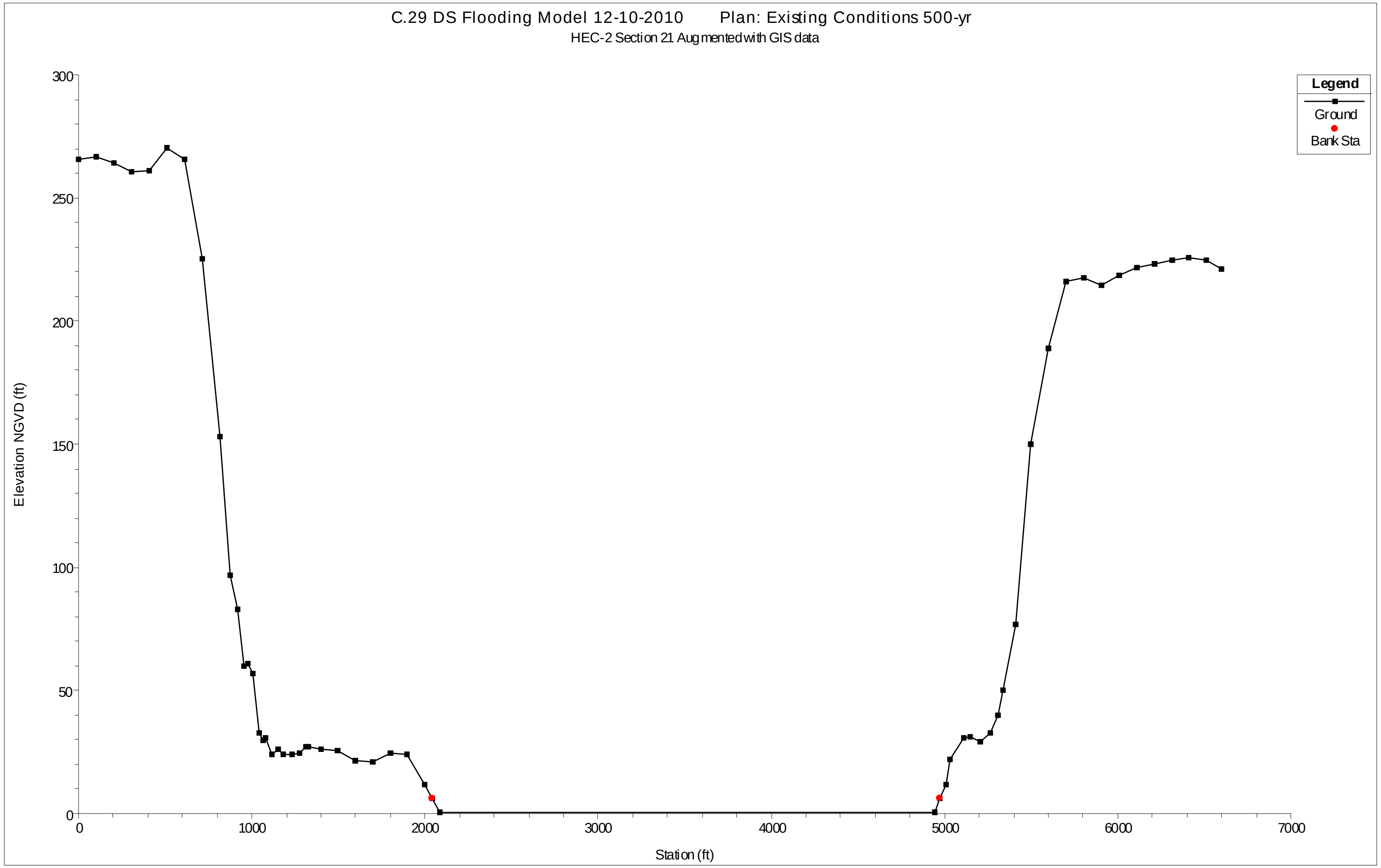


Figure 9: River Station 37659

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 2079 ft	Station 5197 ft
6	0.065	0.043	0.075
17	0.065	0.039	0.070
34	0.065	0.036	0.070

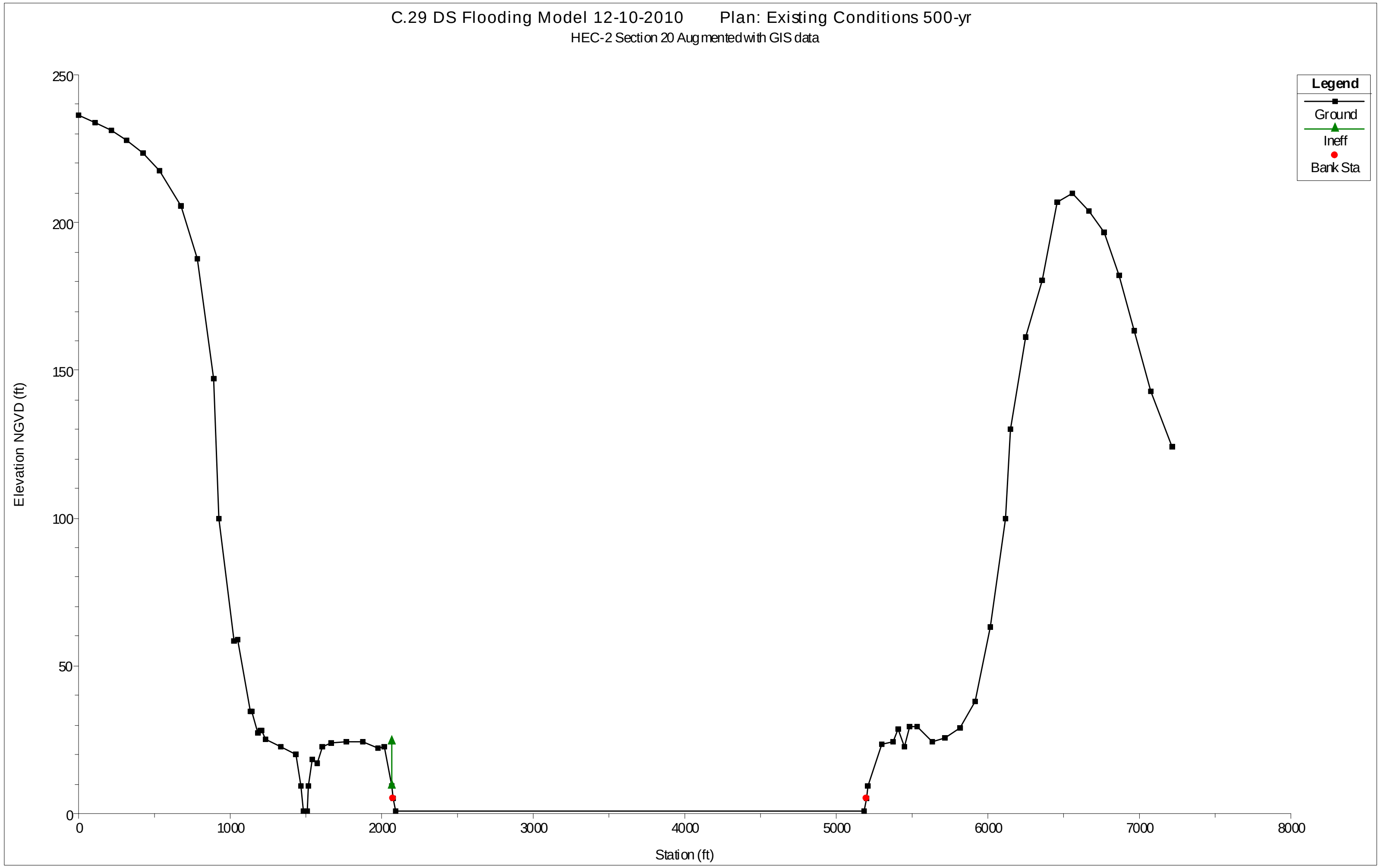


Figure 10: River Station 35938

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 1682 ft	Station 1950 ft	Station 2355	Station 4911 ft
4	0.070	0.043	0.070	0.043	0.075
17	0.065	0.039	0.065	0.039	0.070
33	0.065	0.036	0.065	0.036	0.070

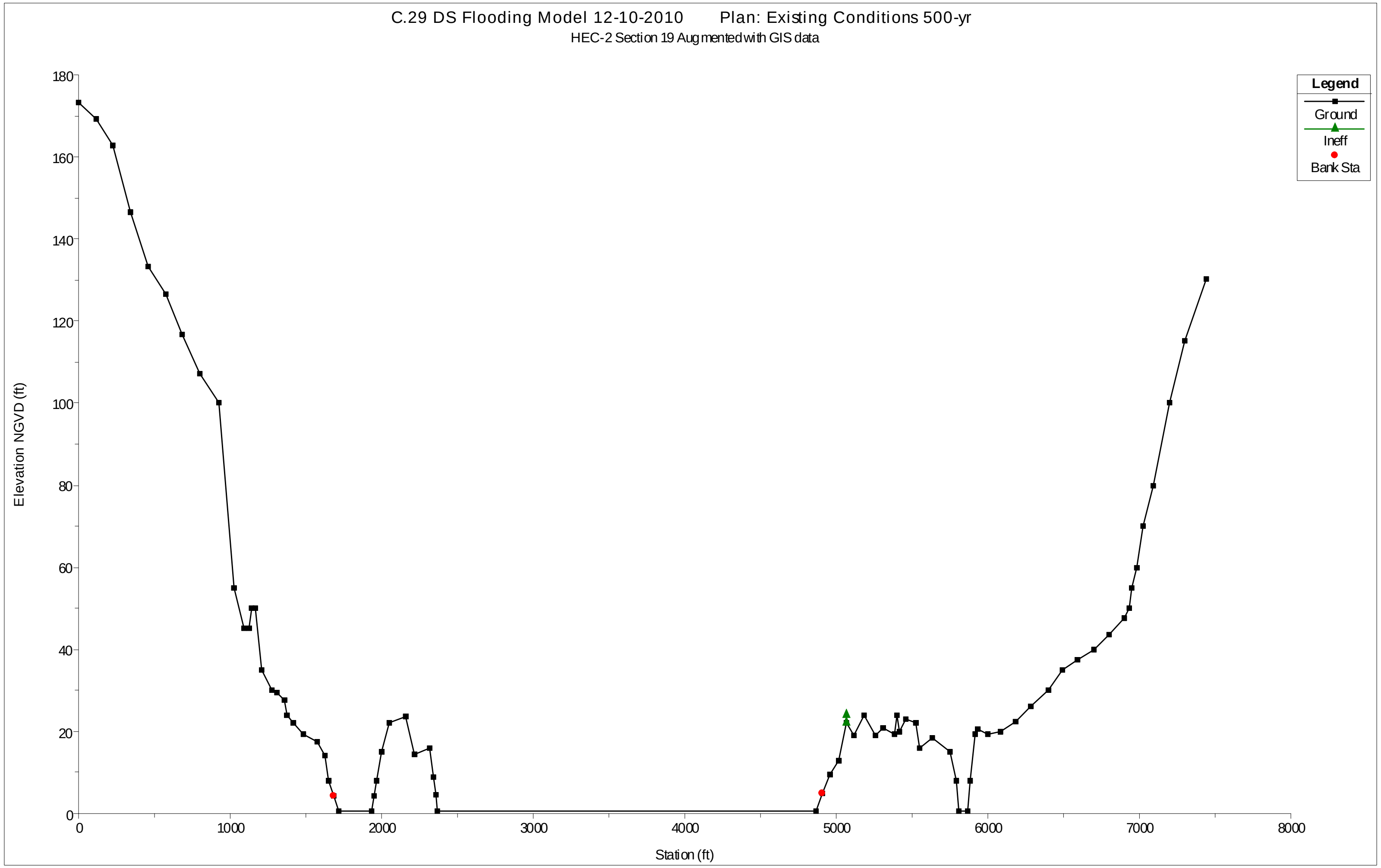


Figure 11: River Station 33967

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 2410 ft	Station 5085 ft
3	0.070	0.043	0.075
16	0.065	0.039	0.070
32	0.065	0.036	0.070

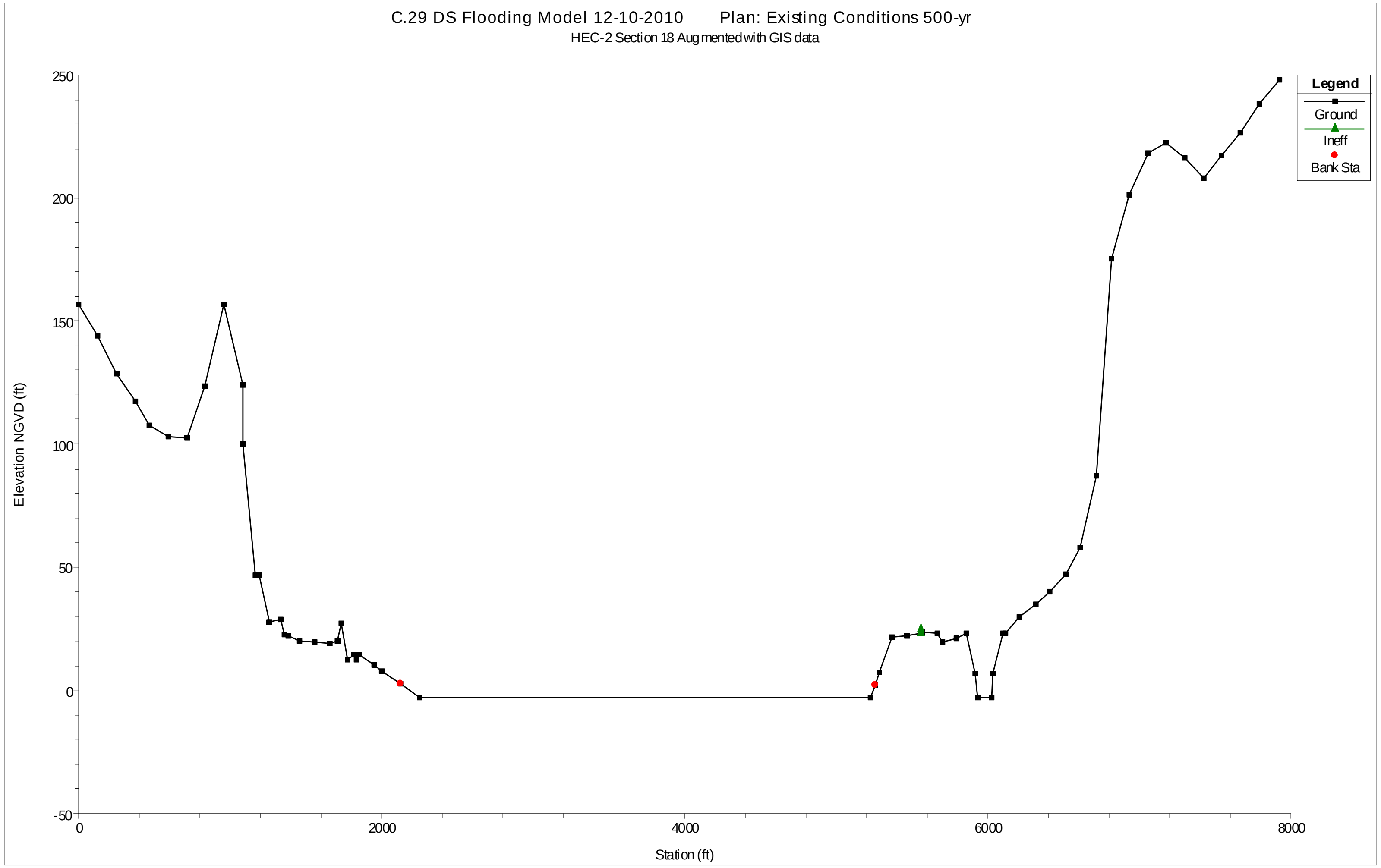


Figure 12: River Station 32707

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 1942 ft	Station 4069 ft	Station 4806 ft	Station 5128 ft	Station 5153 ft	Station 6317 ft
2	0.070	0.043	0.070	0.043	0.070	0.043	0.075
15	0.065	0.039	0.065	0.039	0.065	0.039	0.070
32	0.065	0.036	0.065	0.036	0.065	0.036	0.070

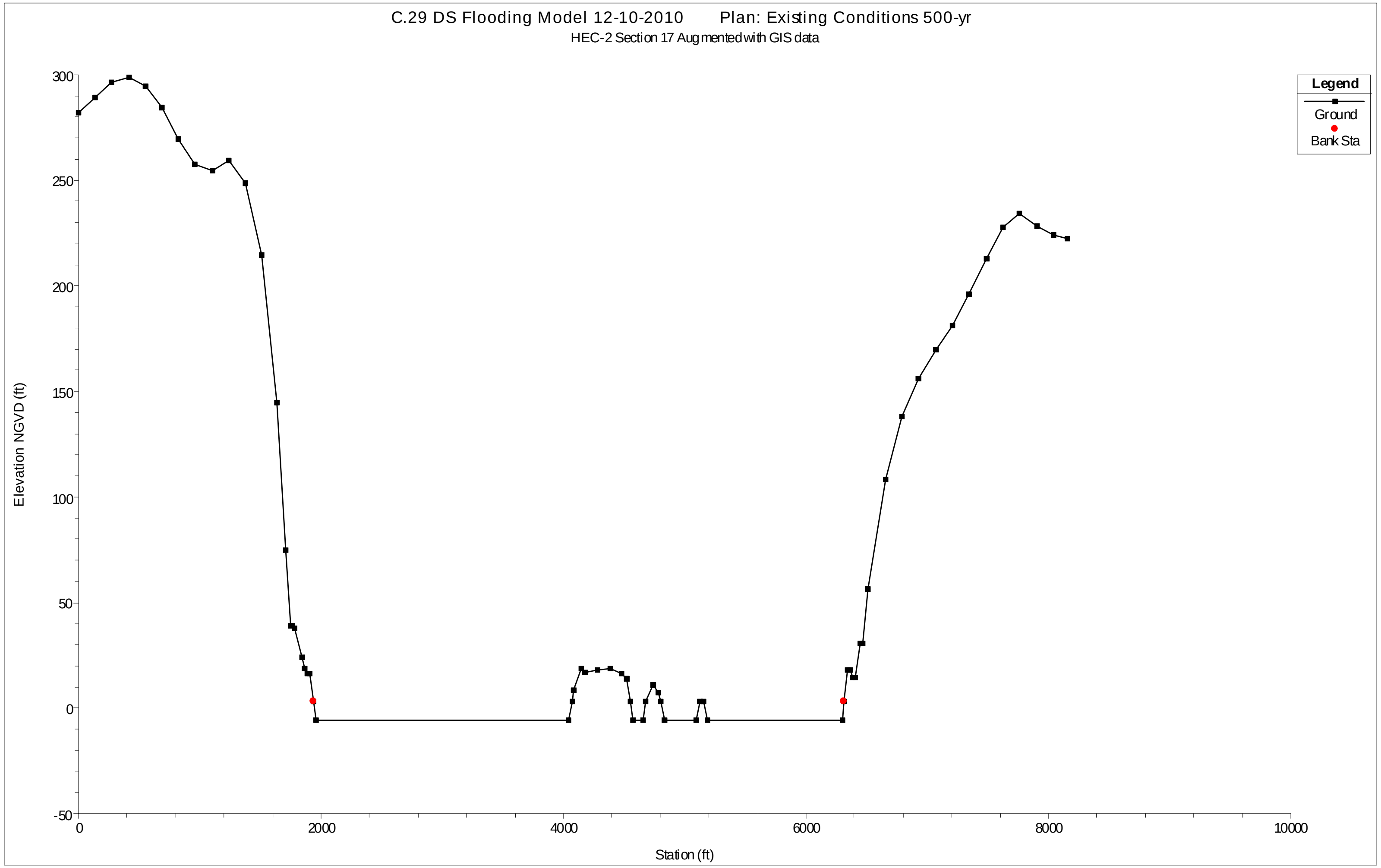


Figure 13: River Station 30307

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 1438 ft	Station 1886 ft	Station 1975 ft	Station 5771 ft
2	0.068	0.043	0.070	0.043	0.075
14	0.065	0.039	0.065	0.039	0.070
31	0.065	0.036	0.065	0.036	0.070

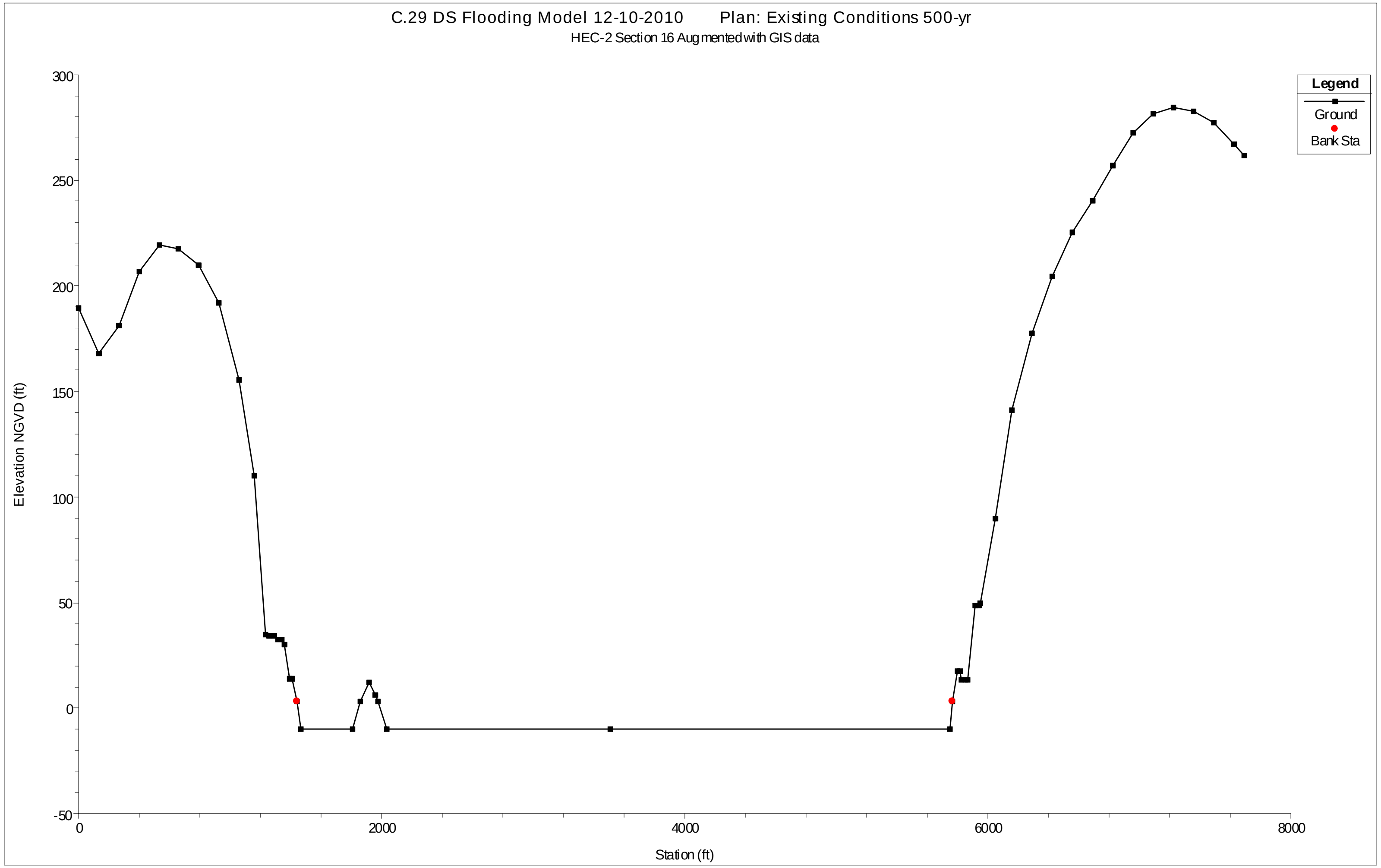


Figure 14: River Station 28071

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 1737 ft	Station 3962 ft	Station 4524 ft	Station 5075 ft	Station 5136 ft	Station 5793 ft
2	0.070	0.043	0.070	0.043	0.070	0.043	0.075
12	0.065	0.039	0.065	0.039	0.065	0.039	0.070
30	0.065	0.036	0.065	0.036	0.065	0.036	0.070

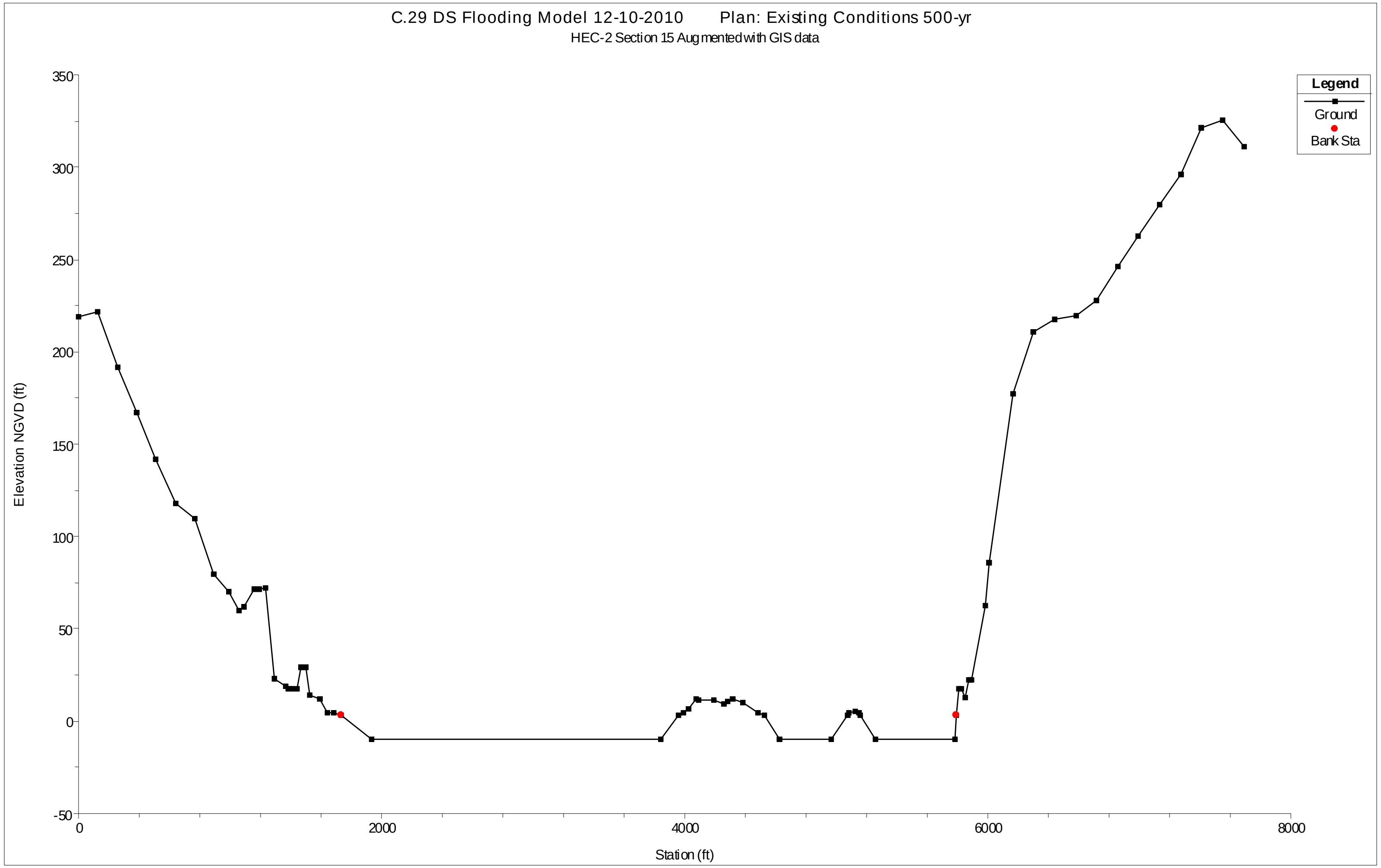


Figure 15: River Station 27239

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 2810 ft	Station 6801 ft
2	0.075	0.043	0.075
10	0.070	0.039	0.070
29	0.070	0.036	0.070

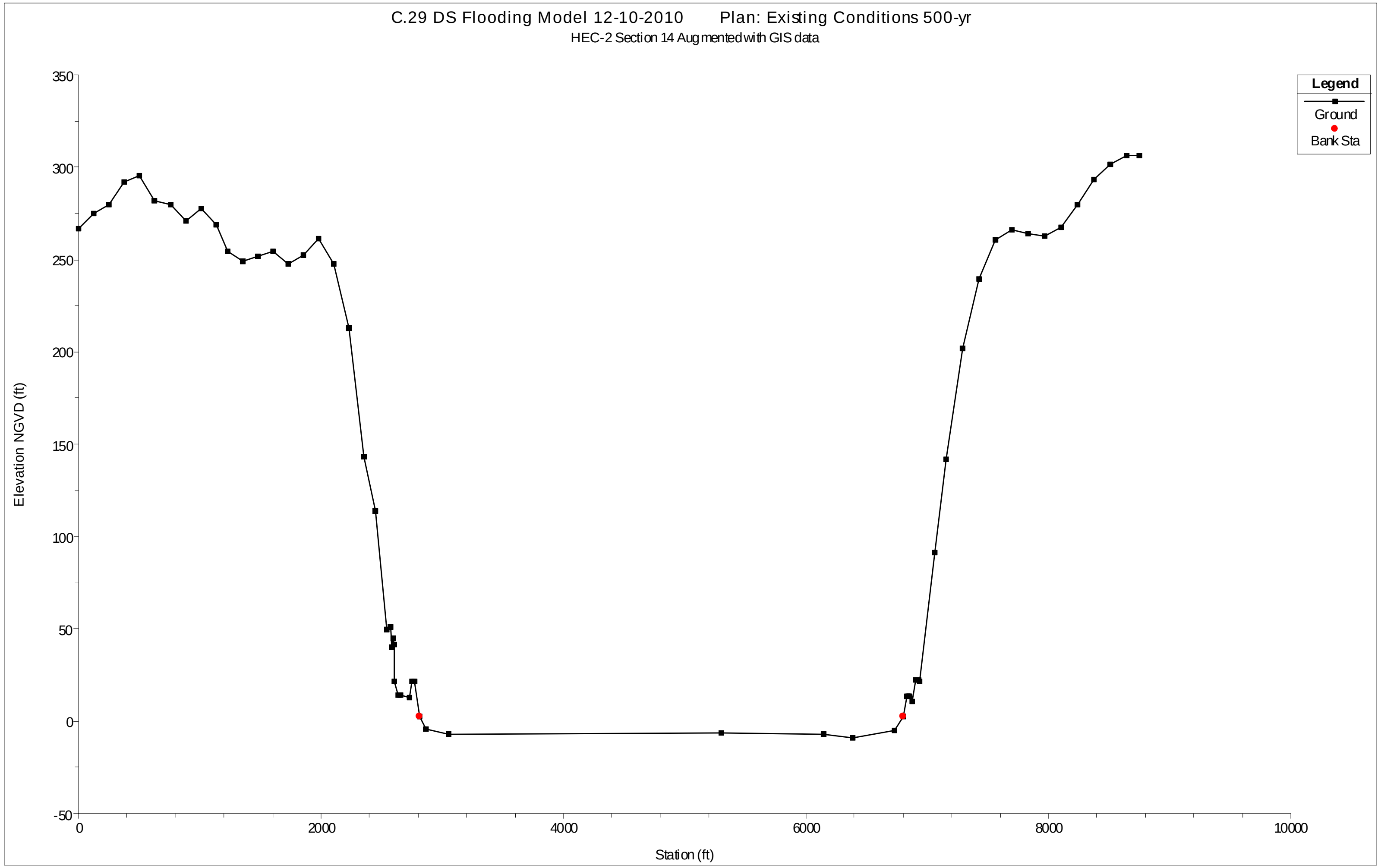


Figure 16: River Station 25219

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 2522 ft	Station 7032 ft
4	0.075	0.042	0.075
9	0.070	0.038	0.070
28	0.070	0.035	0.070

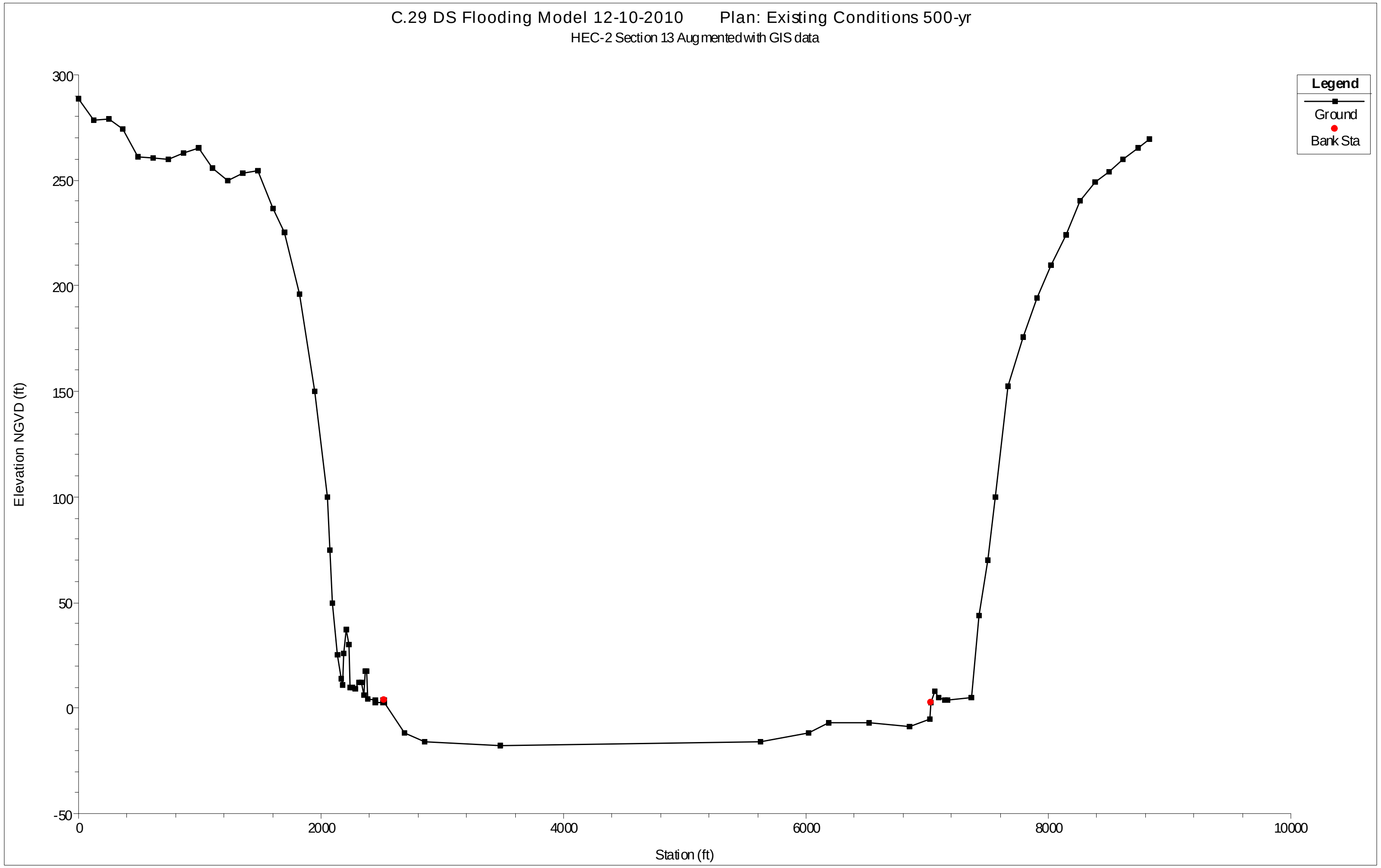


Figure 17: River Station 21166

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 3073 ft	Station 7574 ft
4	0.075	0.042	0.075
8	0.070	0.038	0.070
27	0.070	0.035	0.070

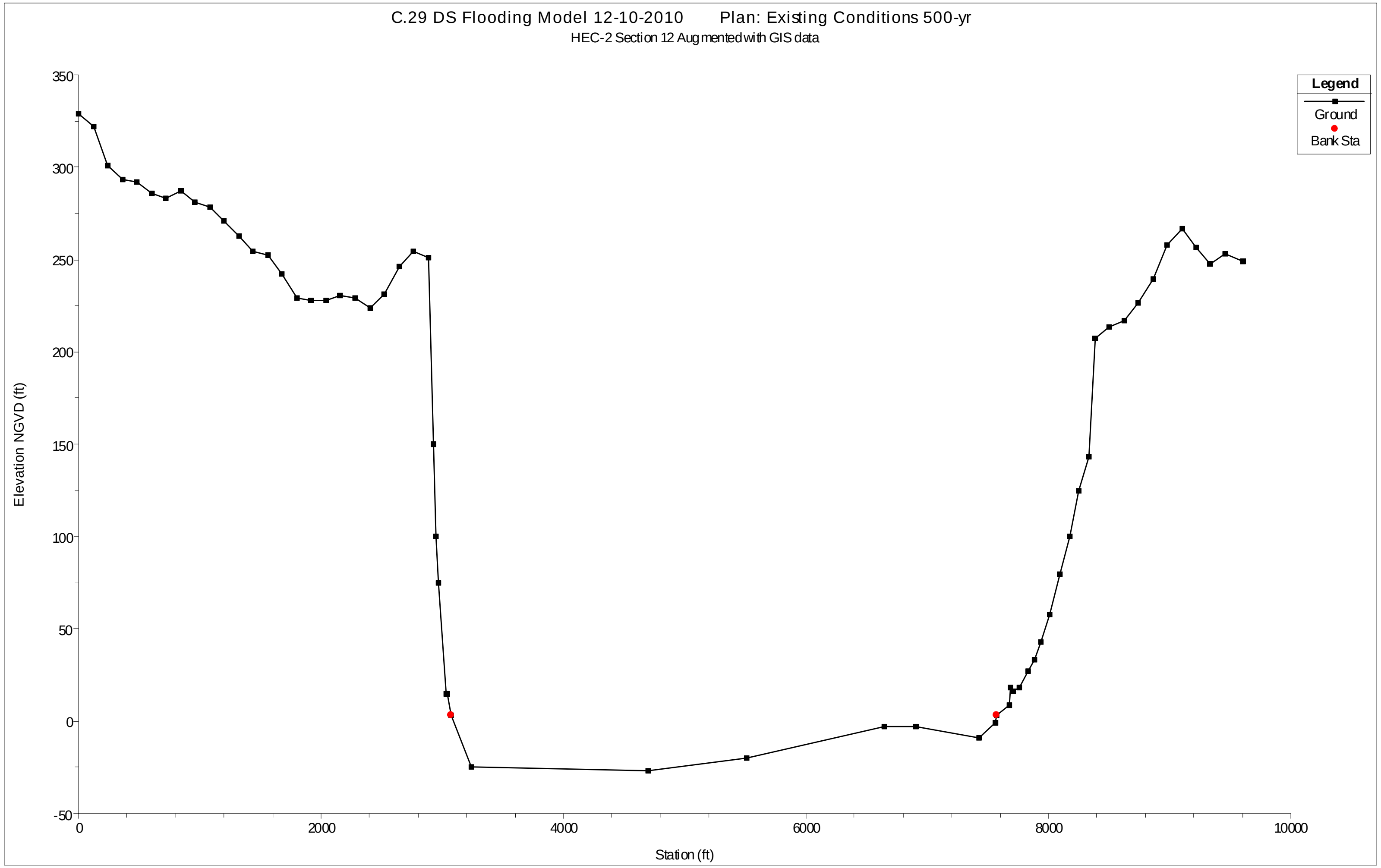


Figure 18: River Station 17251

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 2713 ft	Station 6995 ft
3	0.075	0.042	0.075
7	0.070	0.038	0.070
26	0.070	0.035	0.070

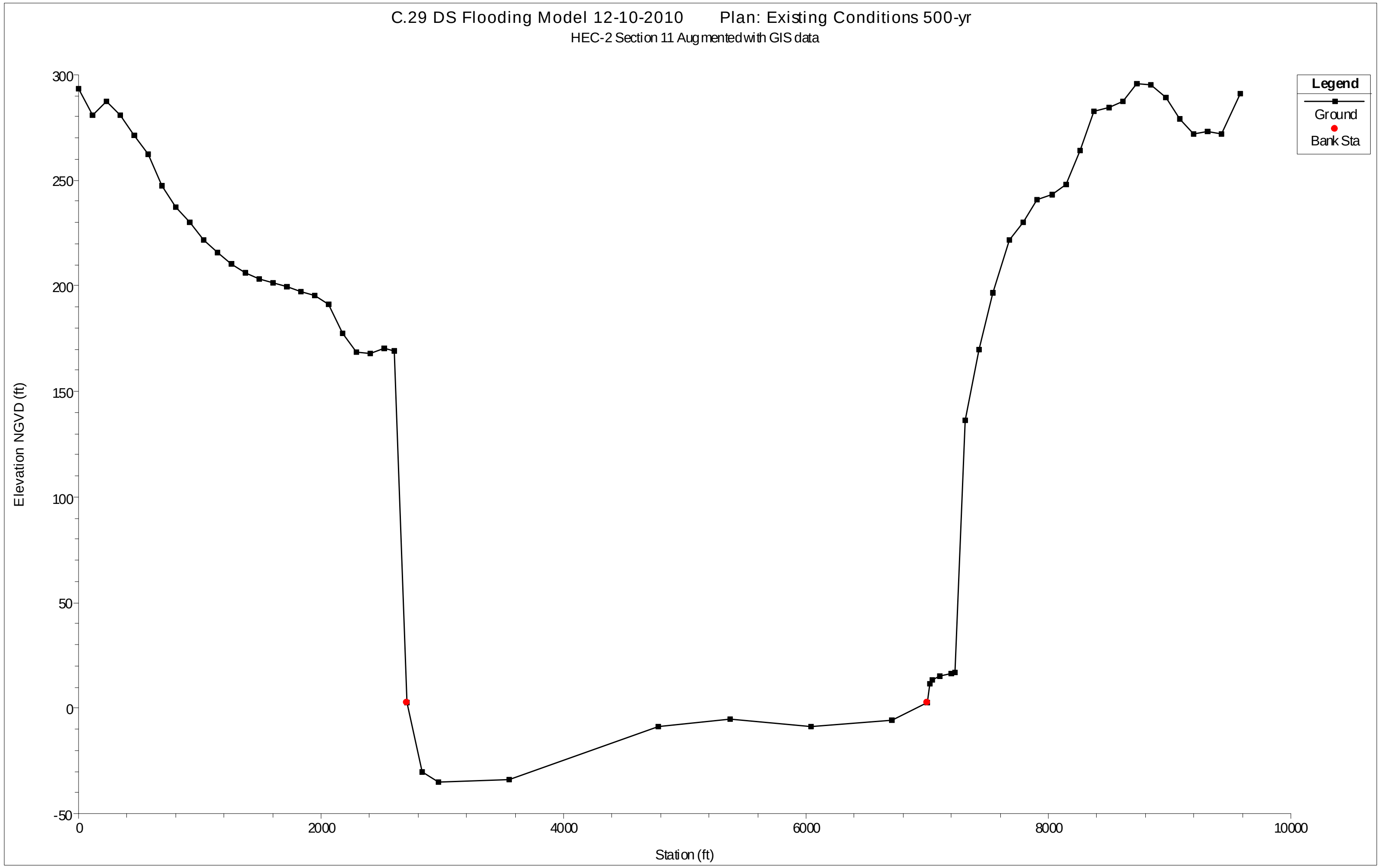


Figure 19: River Station 13989

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 2816 ft	Station 7642 ft
3	0.075	0.042	0.075
5	0.070	0.038	0.070
25	0.070	0.035	0.070

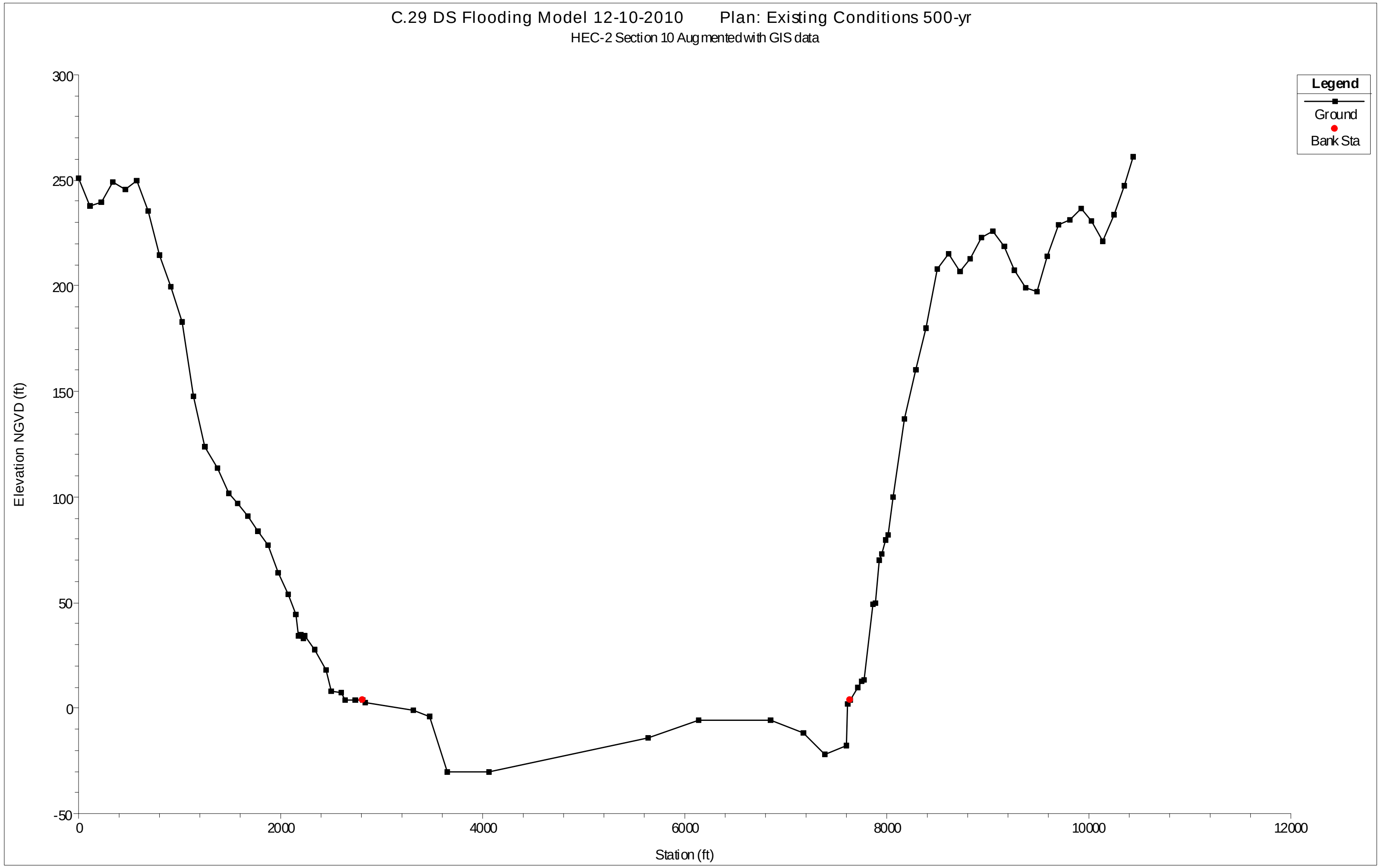


Figure 20: River Station 10038

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 2233 ft	Station 3508 ft	Station 5455 ft	Station 7810 ft
2	0.070	0.042	0.070	0.042	0.075
5	0.065	0.038	0.065	0.038	0.070
24	0.065	0.036	0.065	0.036	0.070

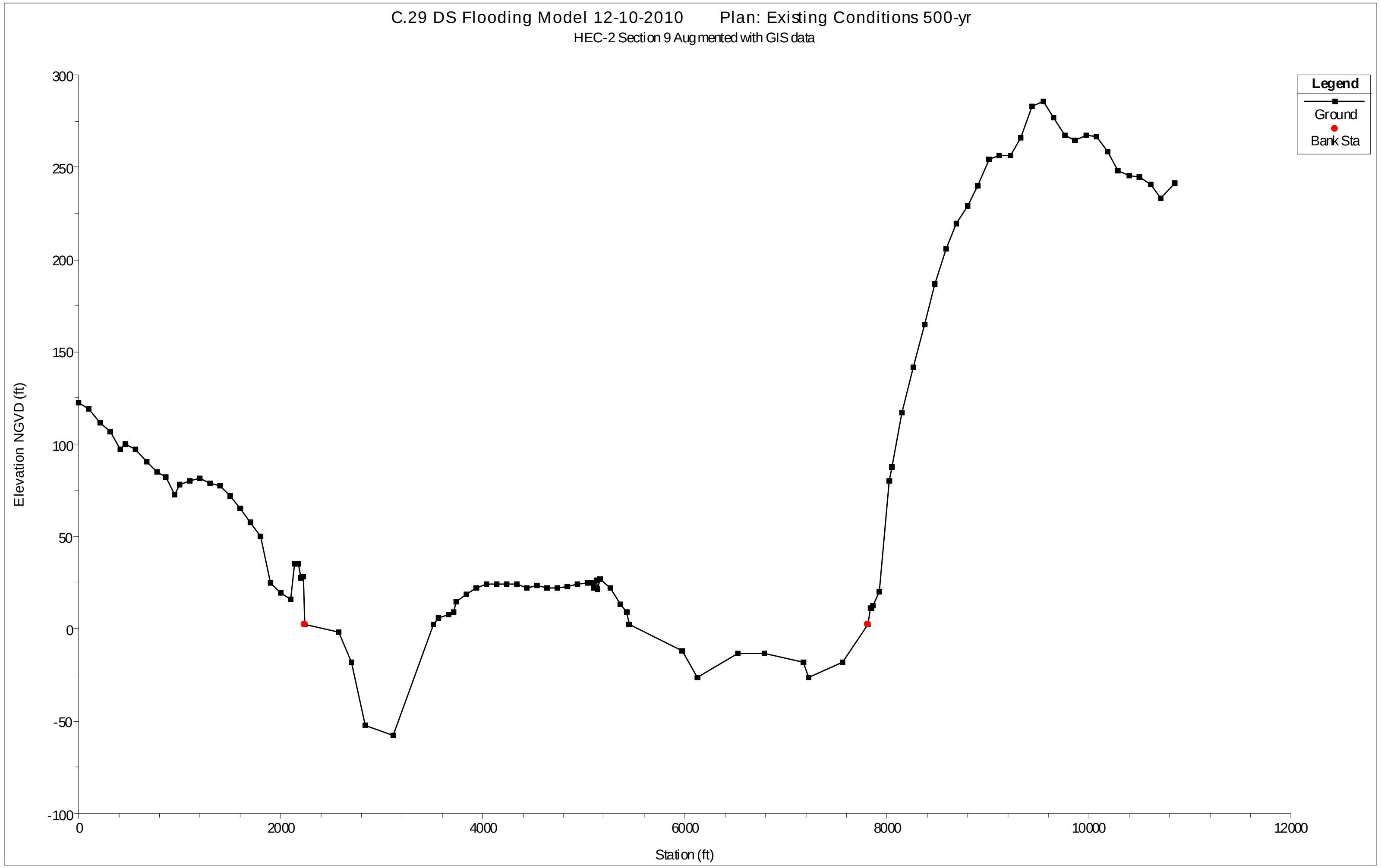


Figure 21: River Station 8017

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 2374 ft	Station 3398 ft	Station 5505 ft	Station 7750 ft
2	0.075	0.042	0.070	0.042	0.075
5	0.070	0.038	0.065	0.038	0.070
22	0.070	0.036	0.065	0.036	0.070

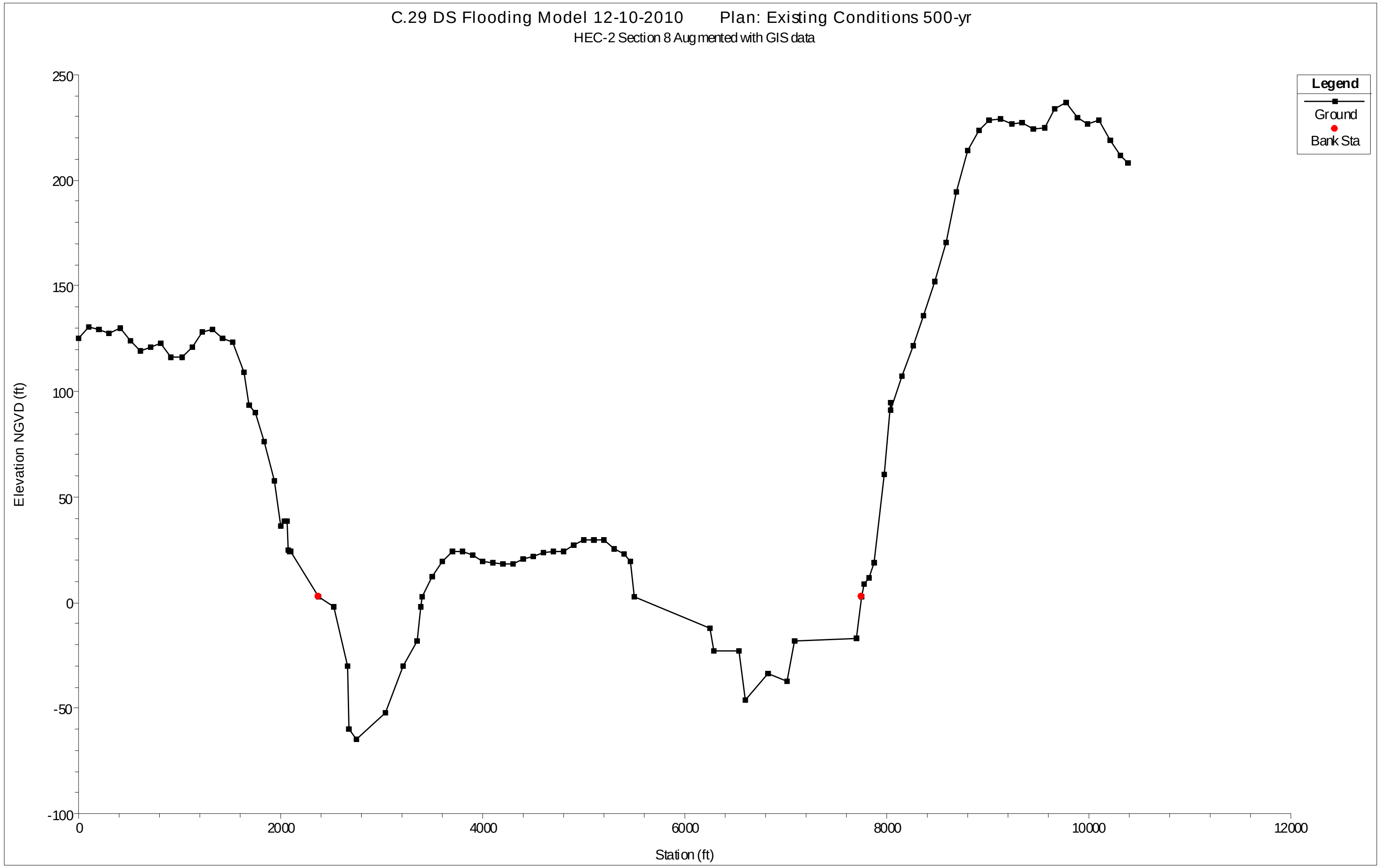


Figure 22: River Station 7457

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 2251 ft	Station 3677 ft	Station 5809 ft	Station 7733 ft
2	0.070	0.042	0.070	0.042	0.075
4	0.065	0.038	0.065	0.038	0.070
40	0.065	0.036	0.062	0.036	0.070

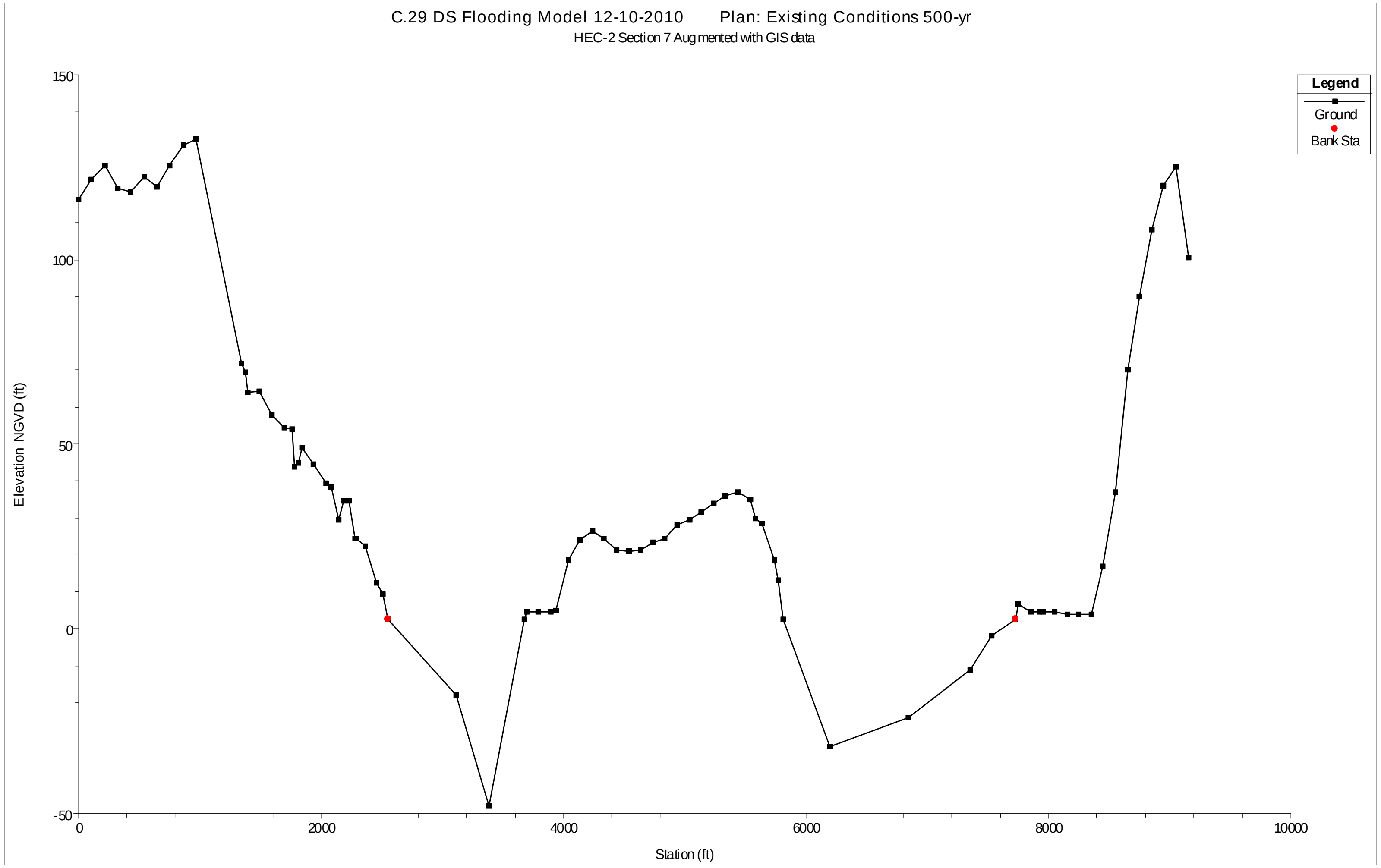


Figure 23: River Station 5919

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 3194 ft	Station 4156 ft	Station 5760 ft	Station 7696 ft
2	0.070	0.042	0.070	0.042	0.075
4	0.065	0.038	0.065	0.038	0.070
18	0.065	0.036	0.065	0.036	0.070

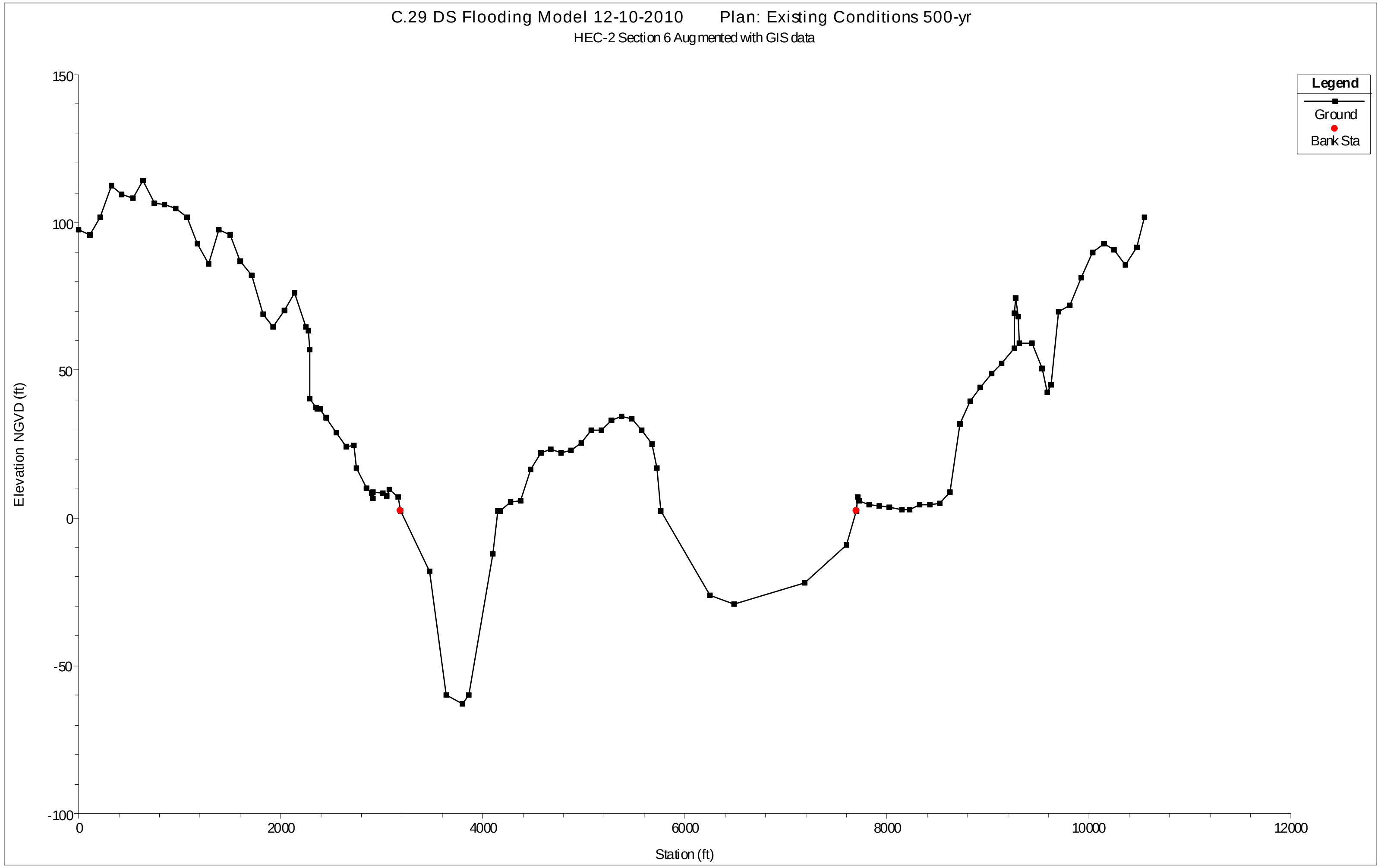


Figure 24: River Station 5353

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 5717 ft	Station 6686 ft	Station 8058 ft	Station 10084 ft
1	0.070	0.042	0.070	0.042	0.075
4	0.065	0.038	0.065	0.038	0.070
16	0.065	0.035	0.065	0.036	0.070

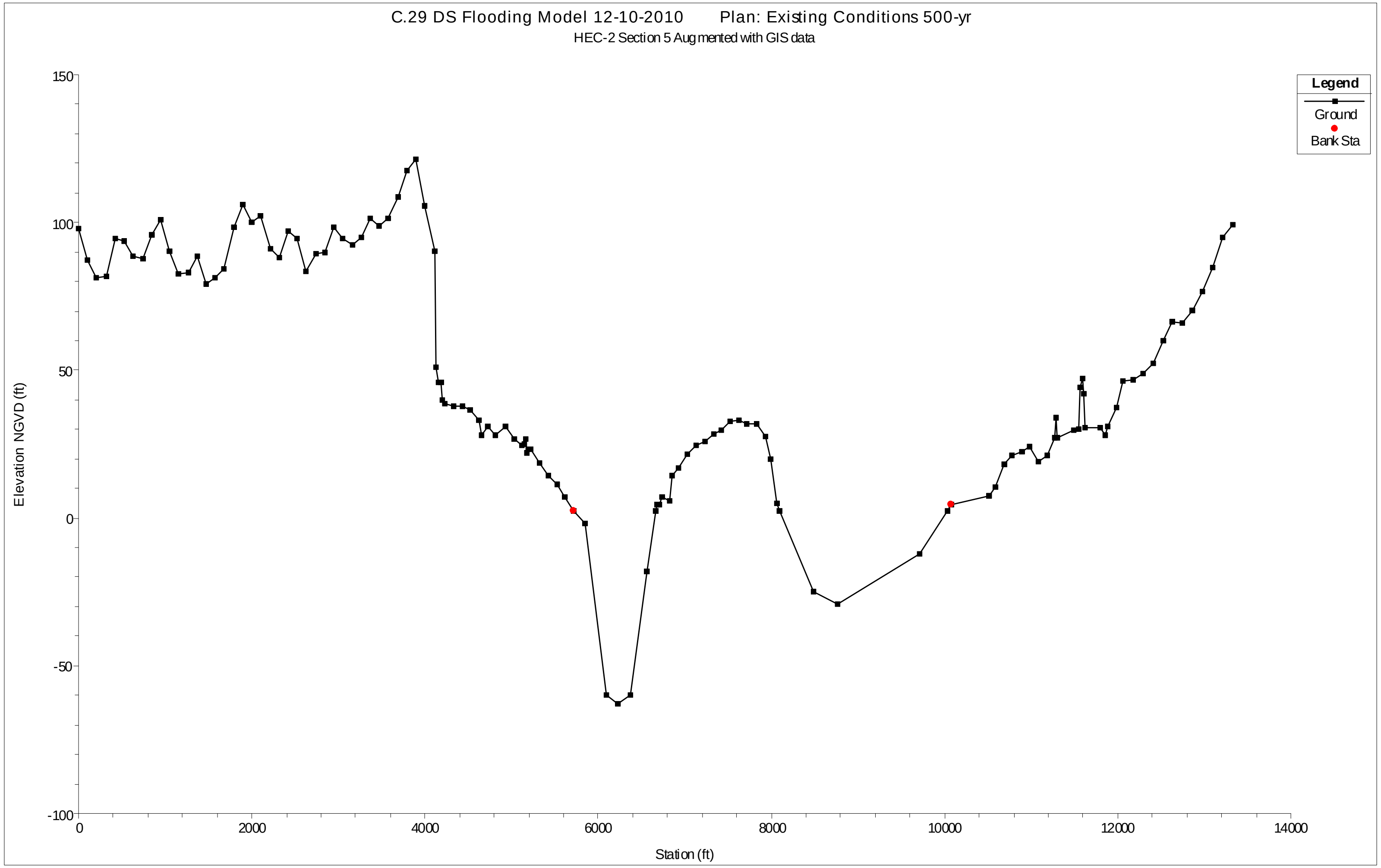


Figure 25: River Station 5032

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 7880 ft	Station 11200 ft
1	0.070	0.042	0.075
3	0.065	0.038	0.070
14	0.065	0.034	0.070

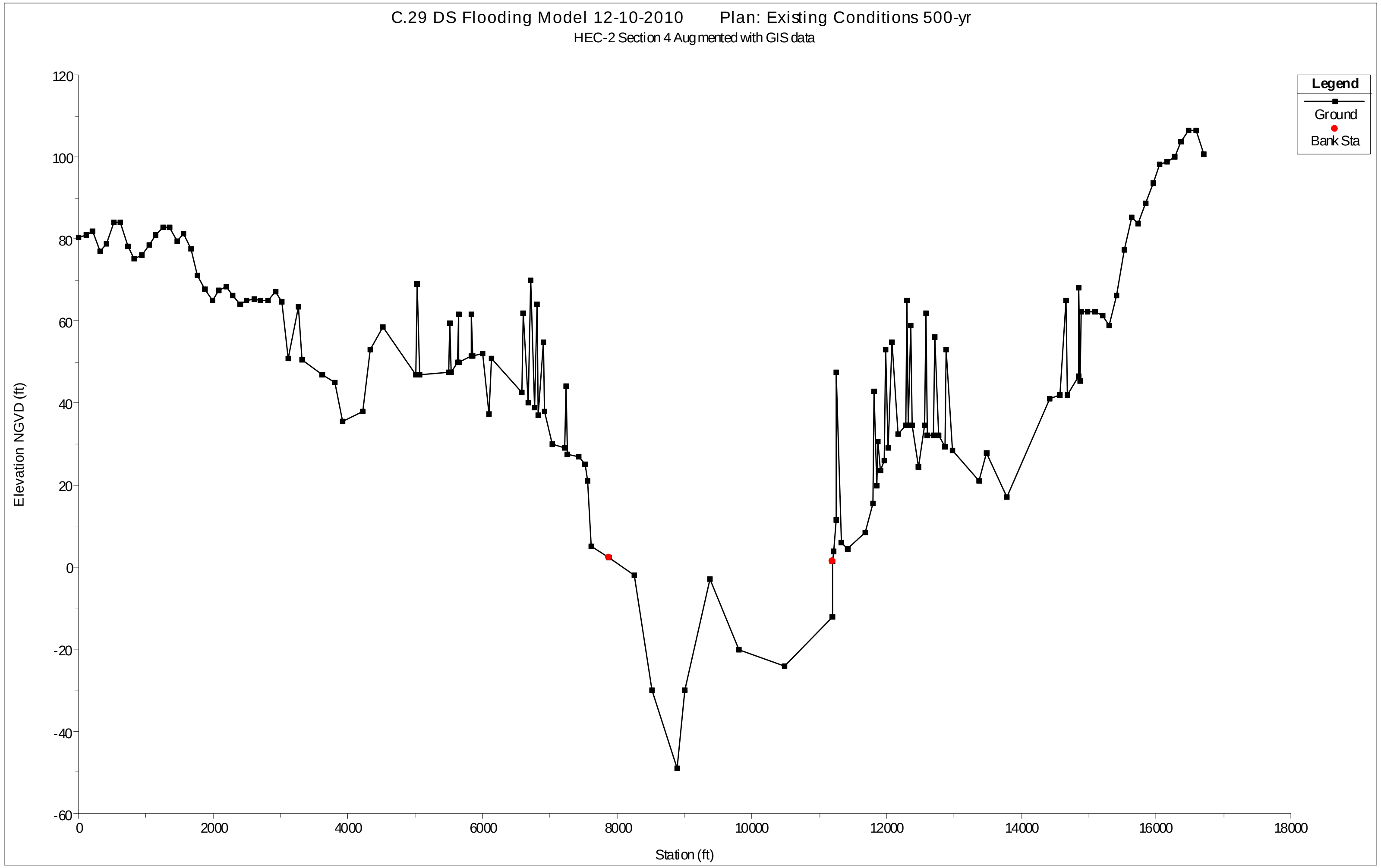


Figure 26: River Station 2907

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 8610 ft	Station 11778 ft
0	0.070	0.042	0.075
3	0.065	0.038	0.070
12	0.065	0.033	0.070

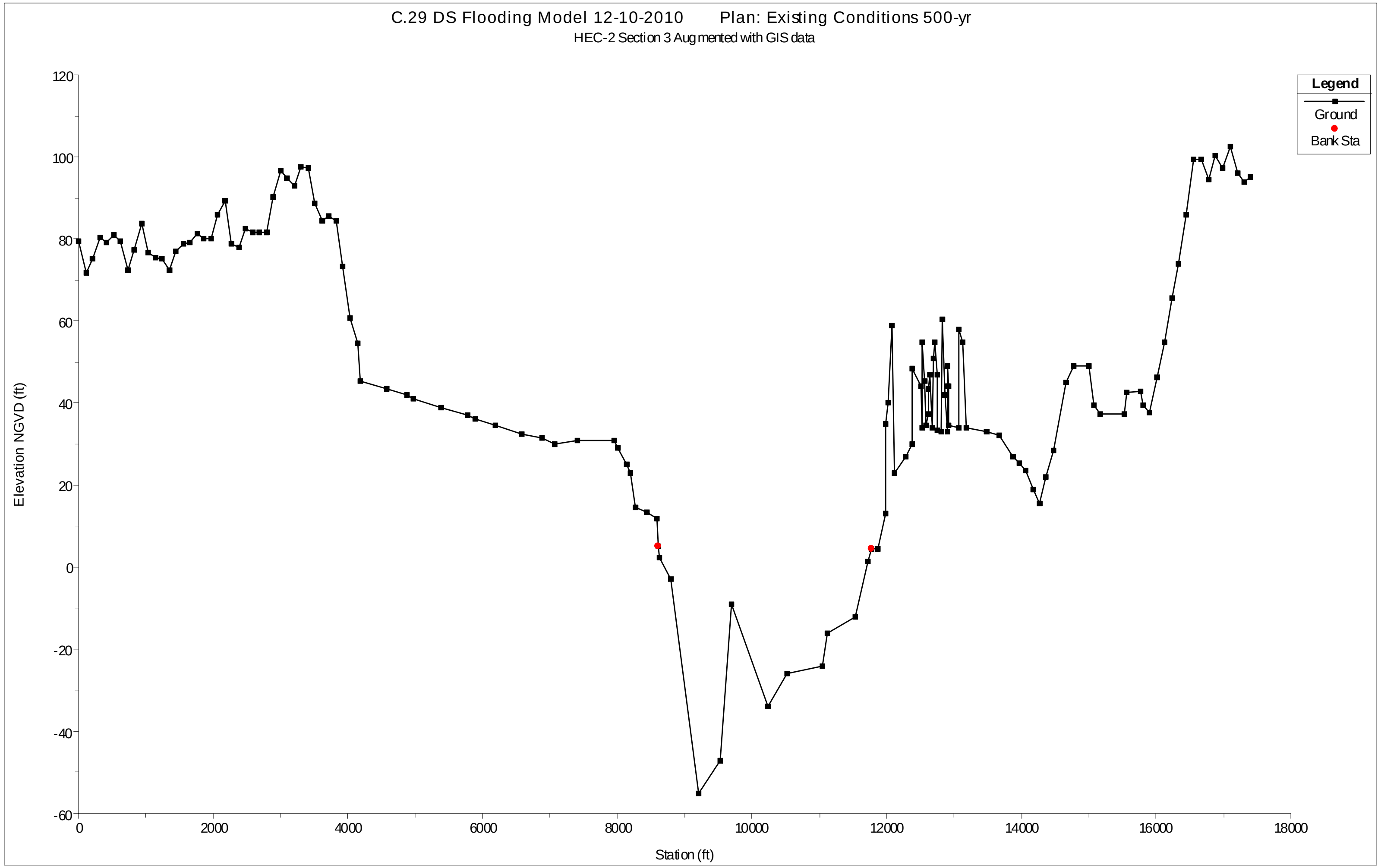


Figure 27: River Station 2435

Manning's N Values – Vertically Varying

Water Surface Elevation (ft NGVD 1929)	Station 0 ft	Station 8265 ft	Station 11817 ft
0	0.070	0.042	0.075
2	0.065	0.038	0.070
10	0.065	0.033	0.070

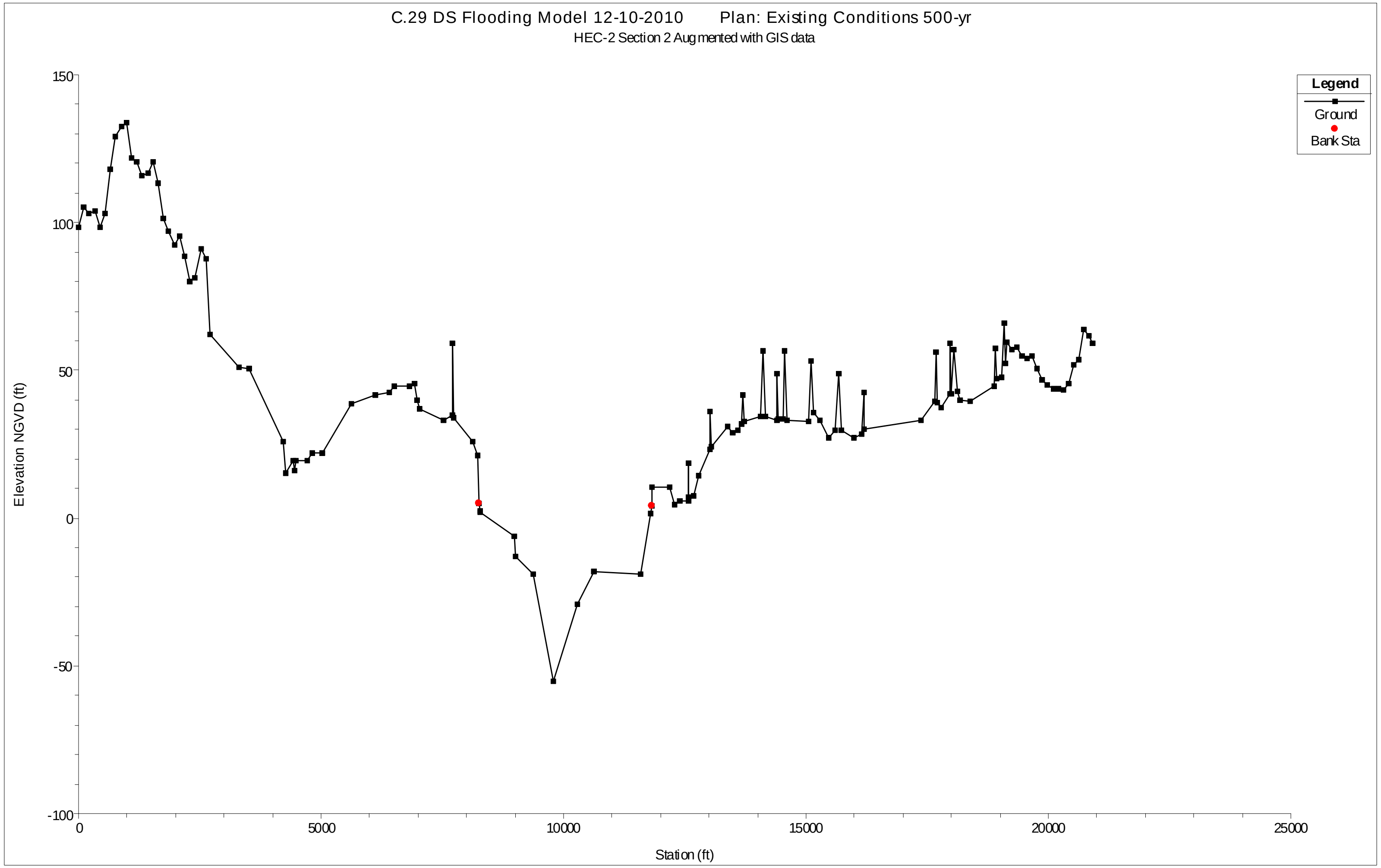
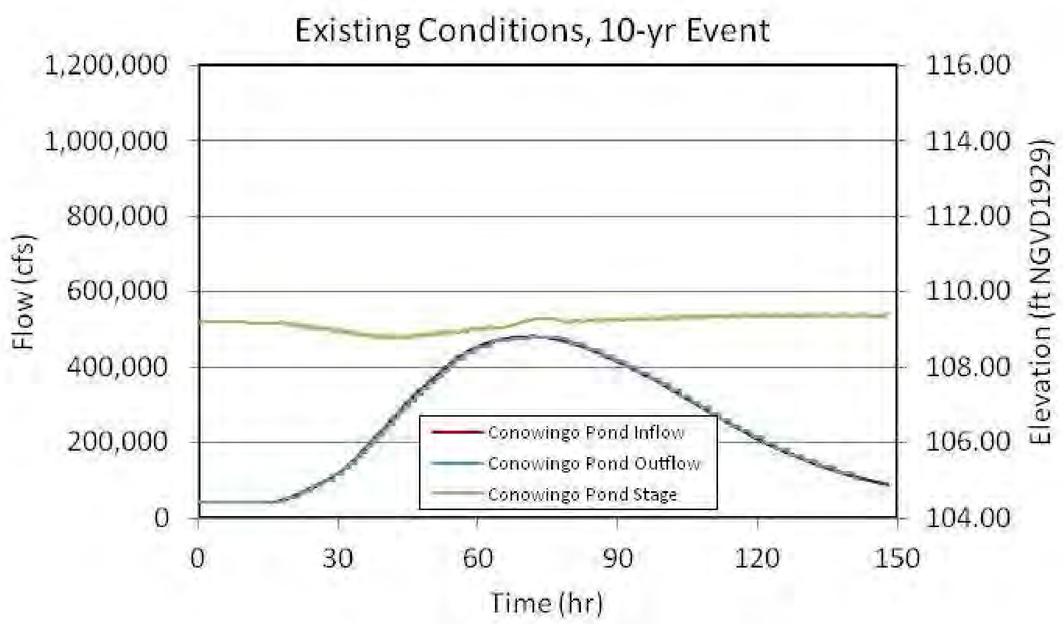
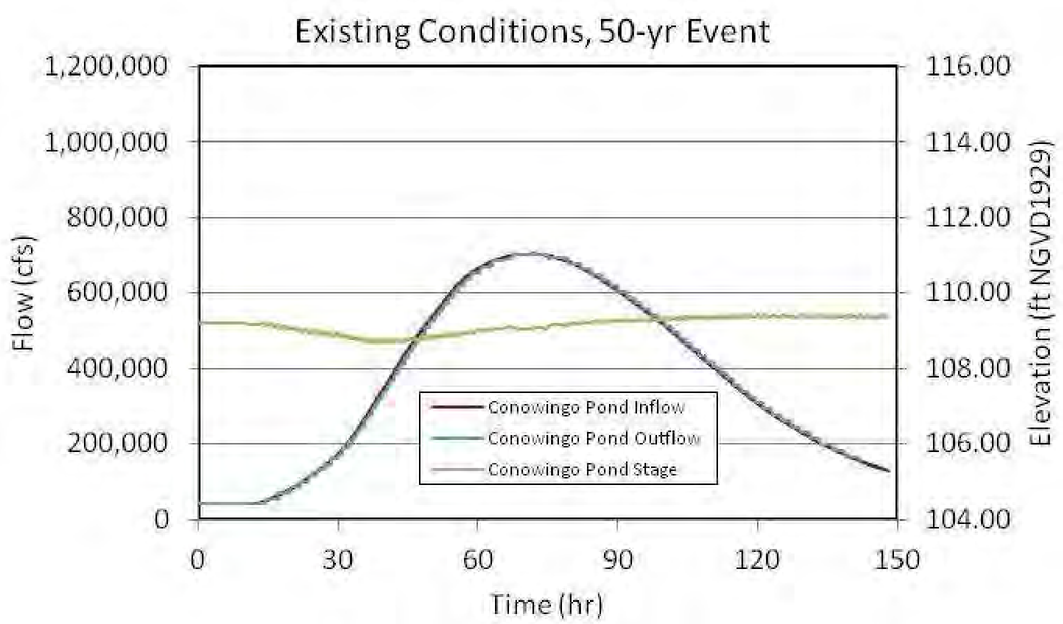
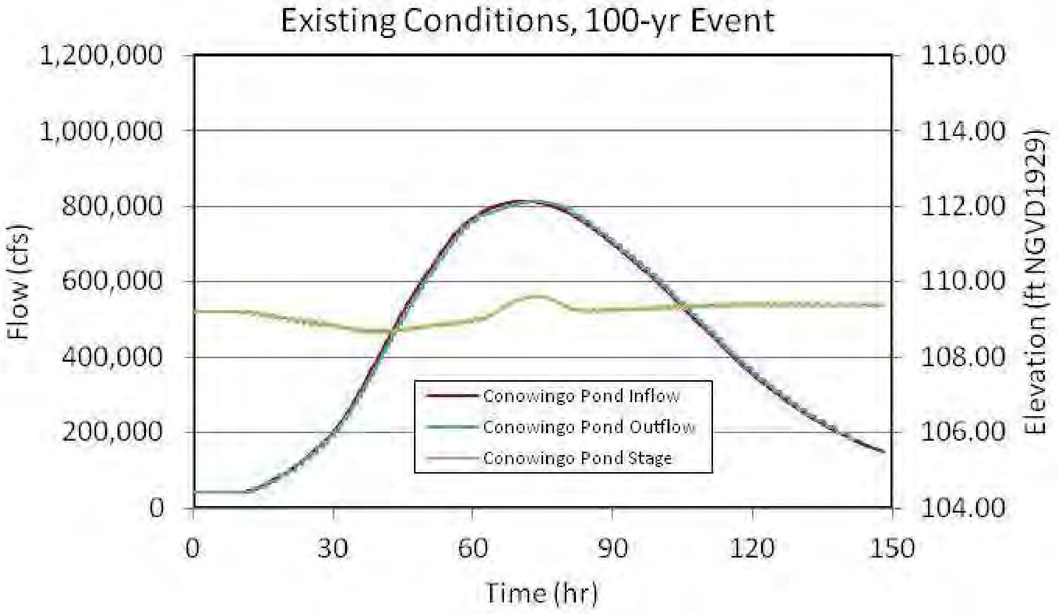


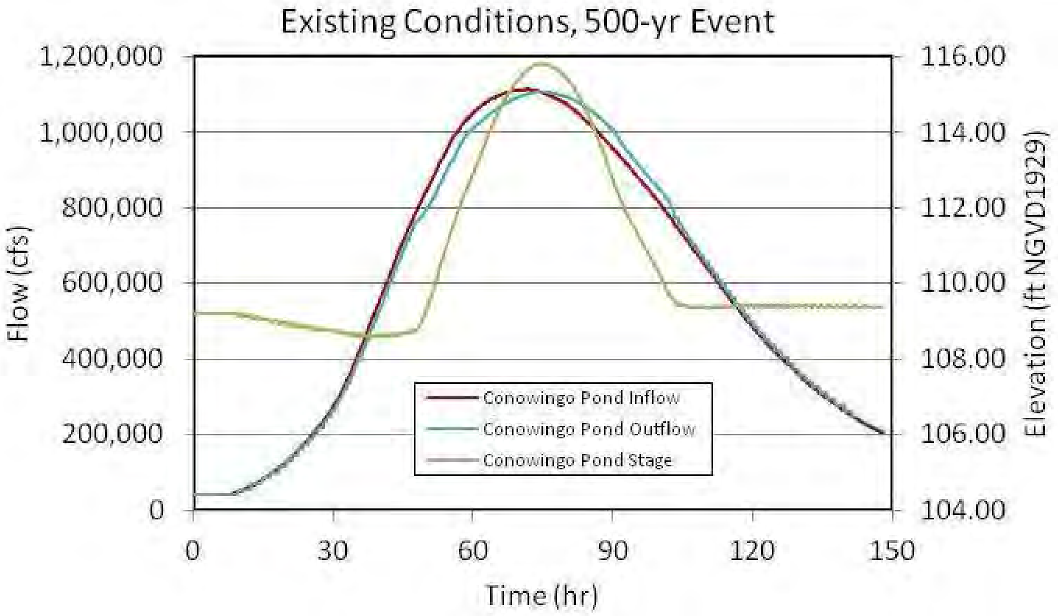
Figure 28: River Station 500

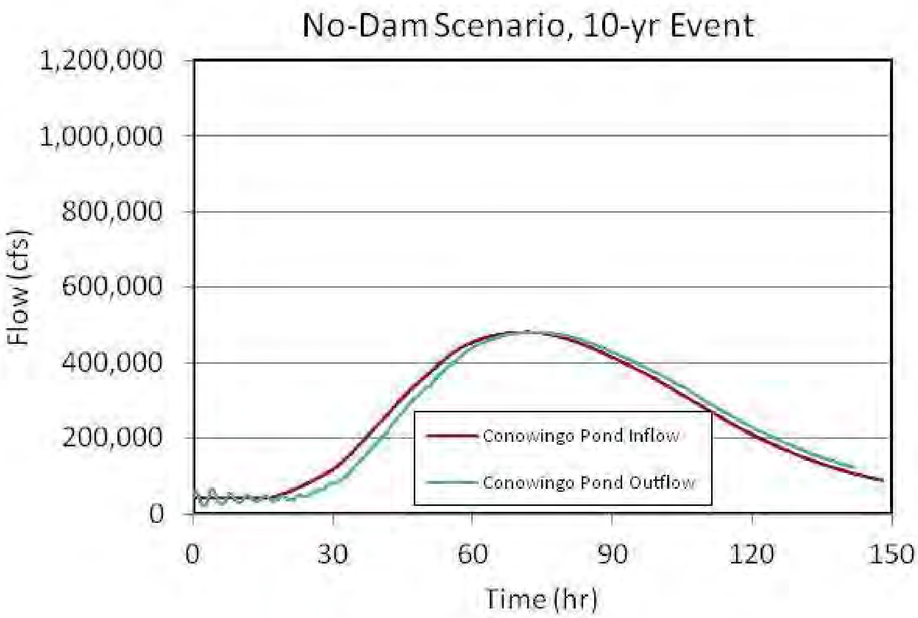
APPENDIX B: CONOWINGO POND INFLOW-OUTFLOW HYDROGRAPHS FOR ALL MODELED SCENARIOS

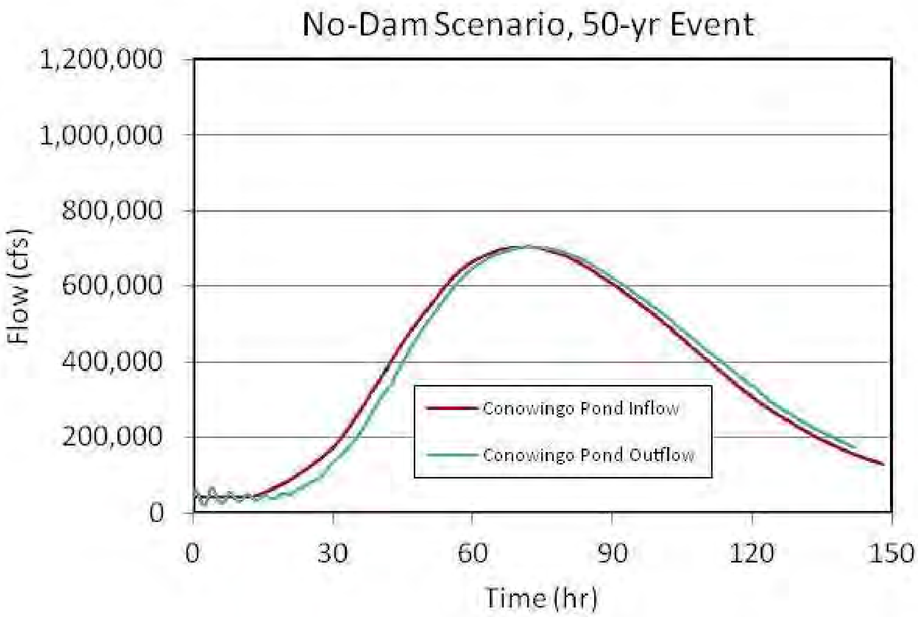

Existing Conditions, 10-yr Event
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
Flow (cfs)
116.00
114.00
112.00
110.00
108.00
106.00
104.00
Elevation (ft NGVD1929)
0
30
60
90
120
150
Time (hr)
Conowingo Pond Inflow
Conowingo Pond Outflow
Conowingo Pond Stage

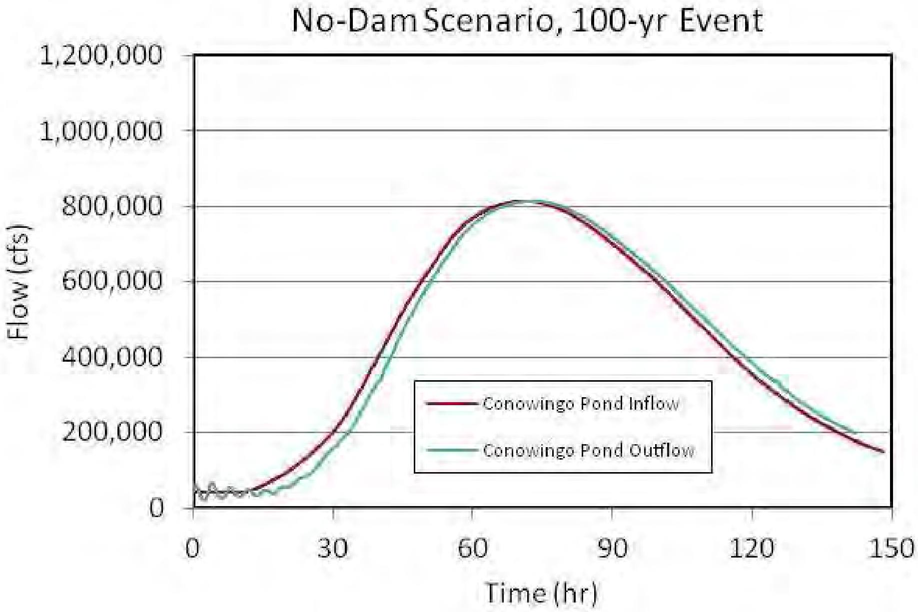

Existing Conditions, 50-yr Event
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
Flow (cfs)
116.00
114.00
112.00
110.00
108.00
106.00
104.00
Elevation (ft NGVD1929)
0
30
60
90
120
150
Time (hr)
Conowingo Pond Inflow
Conowingo Pond Outflow
Conowingo Pond Stage

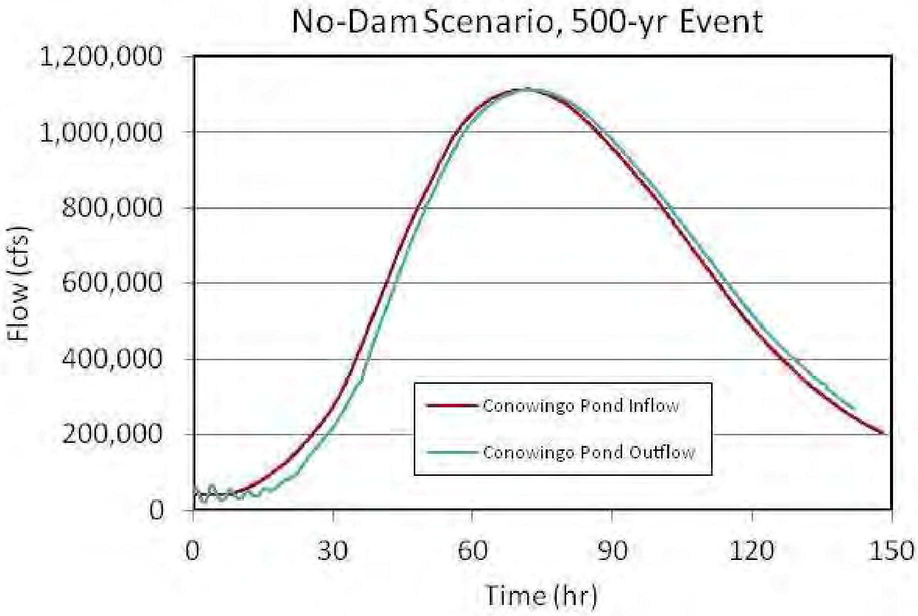

Existing Conditions, 100-yr Event
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
Flow (cfs)
116.00
114.00
112.00
110.00
108.00
106.00
104.00
Elevation (ft NGVD1929)
0
30
60
90
120
150
Time (hr)
Conowingo Pond Inflow
Conowingo Pond Outflow
Conowingo Pond Stage

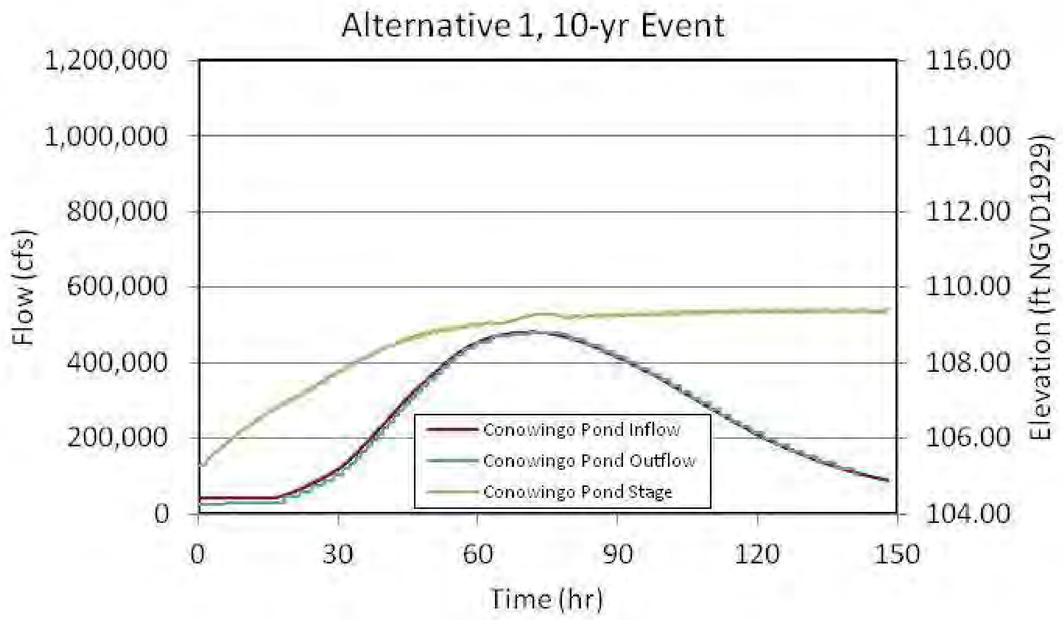

Existing Conditions, 500-yr Event
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
Flow (cfs)
116.00
114.00
112.00
110.00
108.00
106.00
104.00
Elevation (ft NGVD1929)
0
30
60
90
120
150
Time (hr)
Conowingo Pond Inflow
Conowingo Pond Outflow
Conowingo Pond Stage


No-Dam Scenario, 10-yr Event
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
Flow (cfs)
0
30
60
90
120
150
Time (hr)
Conowingo Pond Inflow
Conowingo Pond Outflow

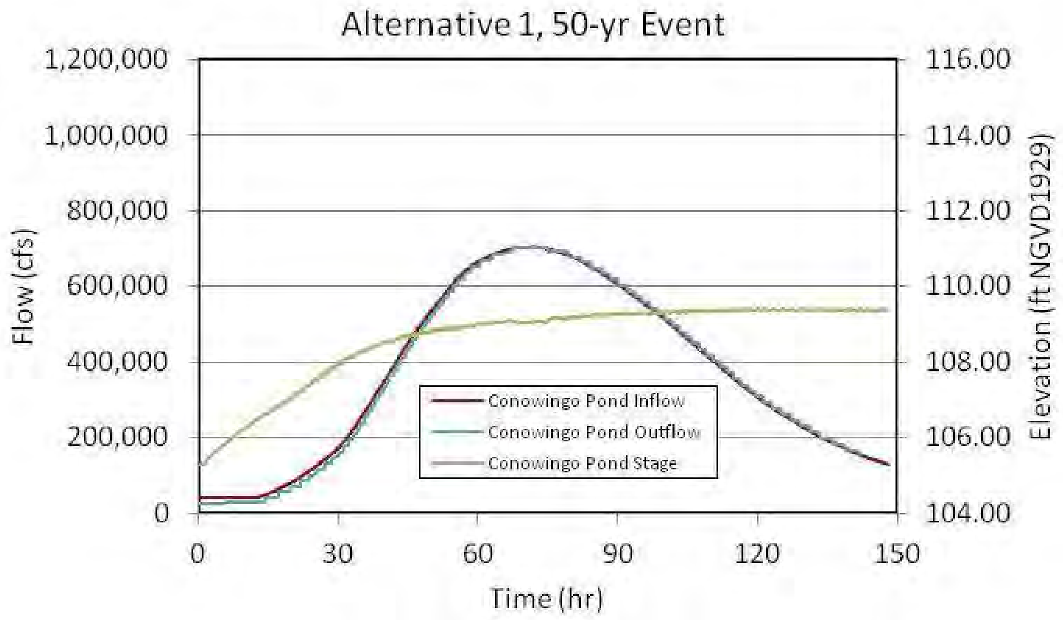

No-Dam Scenario, 50-yr Event
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
Flow (cfs)
0
30
60
90
120
150
Time (hr)
Conowingo Pond Inflow
Conowingo Pond Outflow


No-Dam Scenario, 100-yr Event
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
Flow (cfs)
0
30
60
90
120
150
Time (hr)
Conowingo Pond Inflow
Conowingo Pond Outflow

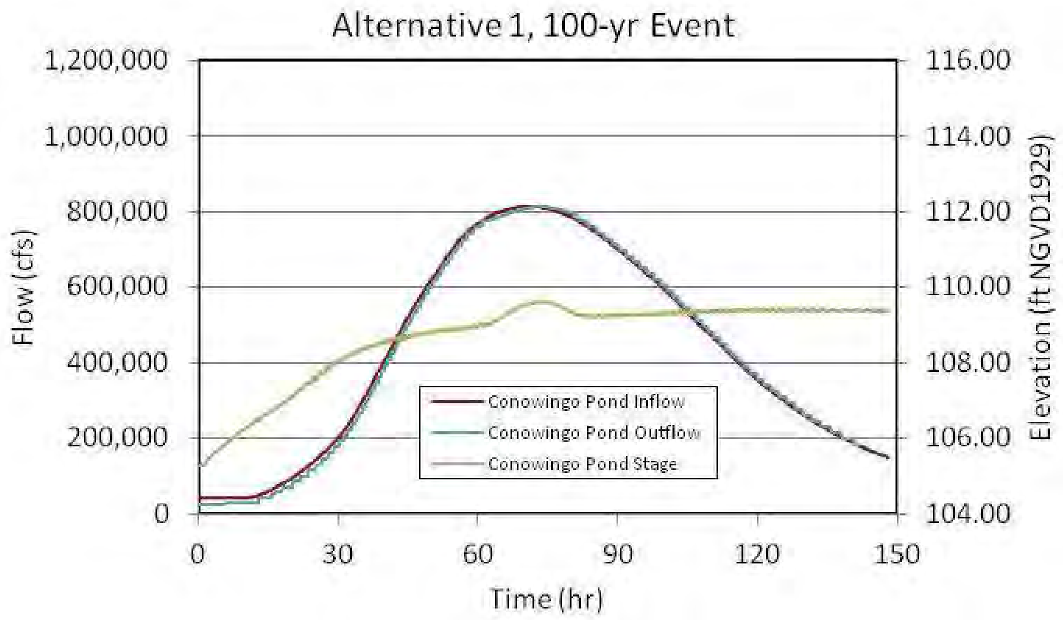

No-Dam Scenario, 500-yr Event
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
Flow (cfs)
0
30
60
90
120
150
Time (hr)
Conowingo Pond Inflow
Conowingo Pond Outflow

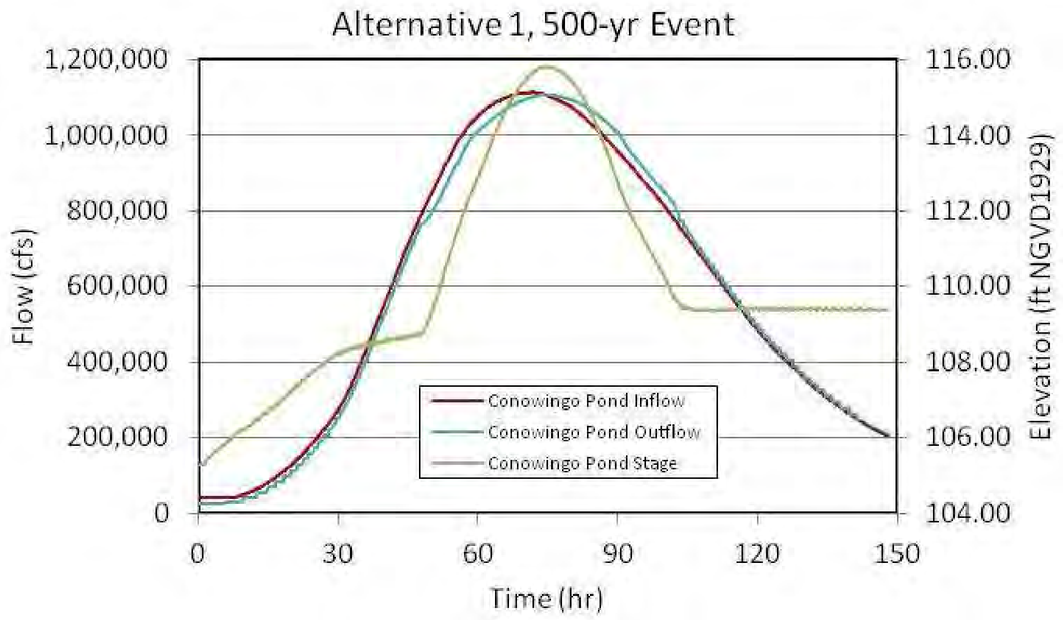


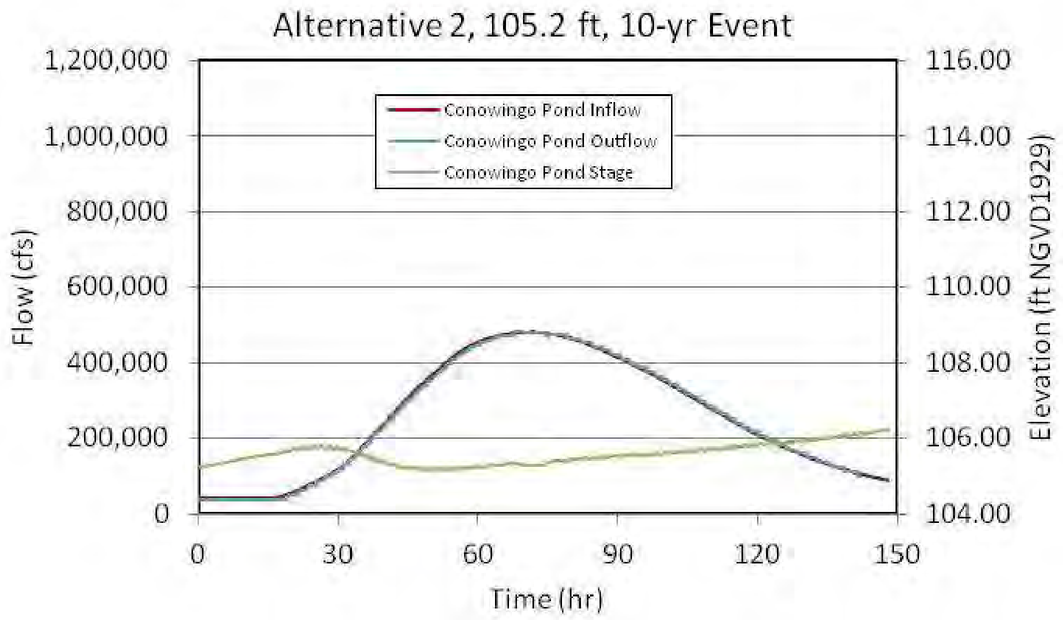
Alternative 1, 10-yr Event
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
116.00
114.00
112.00
110.00
108.00
106.00
104.00
0
30
60
90
120
150
Time (hr)
Flow (cfs)
Elevation (ft NGVD1929)
Conowingo Pond Inflow
Conowingo Pond Outflow
Conowingo Pond Stage



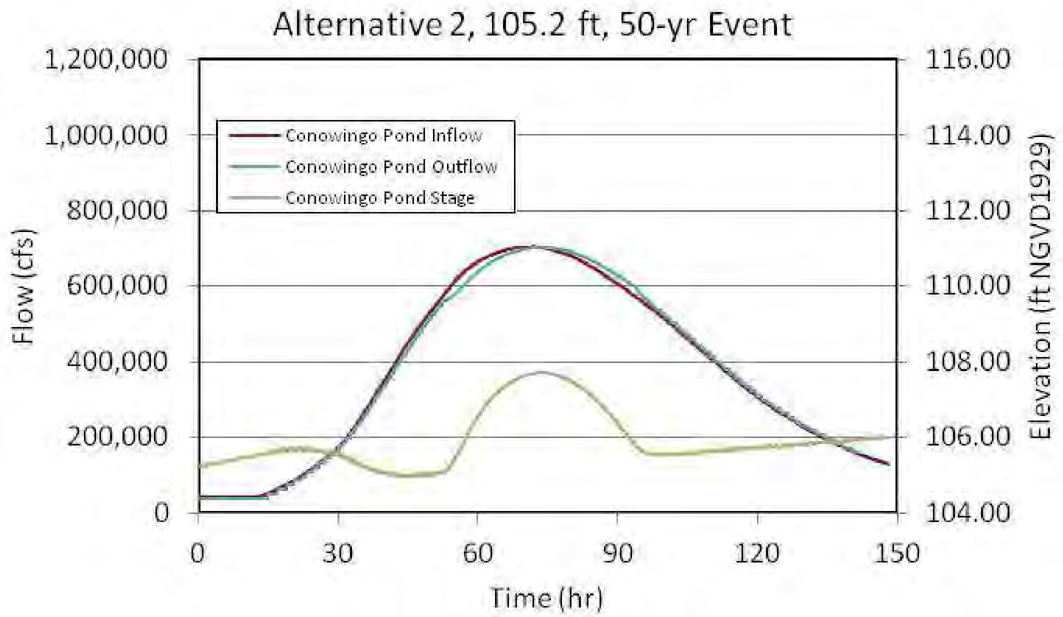
Alternative 1, 50-yr Event
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
116.00
114.00
112.00
110.00
108.00
106.00
104.00
0
30
60
90
120
150
Time (hr)
Flow (cfs)
Elevation (ft NGVD1929)
Conowingo Pond Inflow
Conowingo Pond Outflow
Conowingo Pond Stage


Alternative 1, 100-yr Event
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
Flow (cfs)
116.00
114.00
112.00
110.00
108.00
106.00
104.00
Elevation (ft NGVD1929)
0
30
60
90
120
150
Time (hr)
Conowingo Pond Inflow
Conowingo Pond Outflow
Conowingo Pond Stage

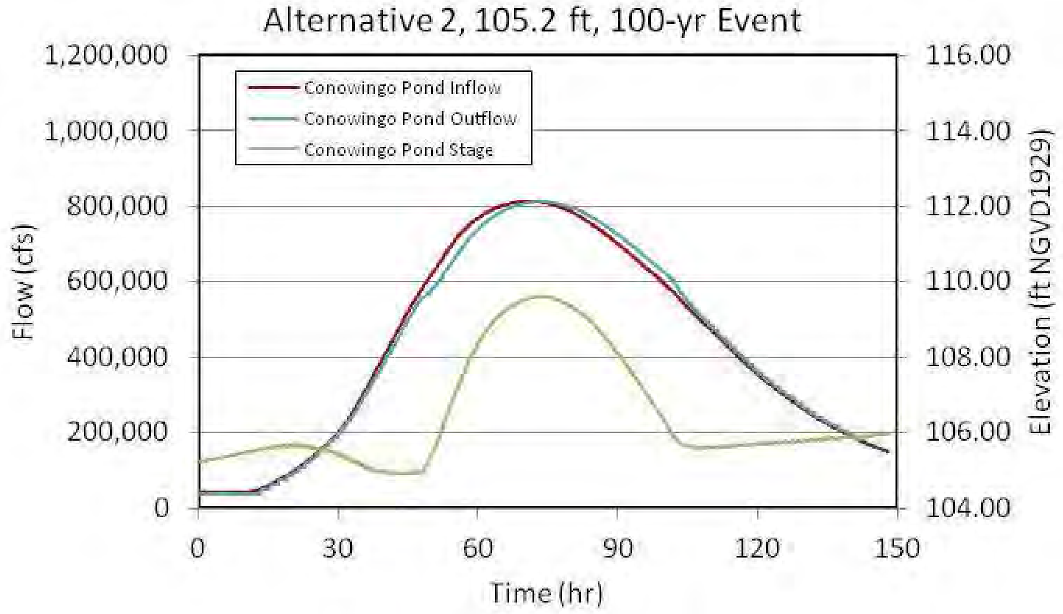

Alternative 1, 500-yr Event
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
Flow (cfs)
116.00
114.00
112.00
110.00
108.00
106.00
104.00
Elevation (ft NGVD1929)
0
30
60
90
120
150
Time (hr)
Conowingo Pond Inflow
Conowingo Pond Outflow
Conowingo Pond Stage

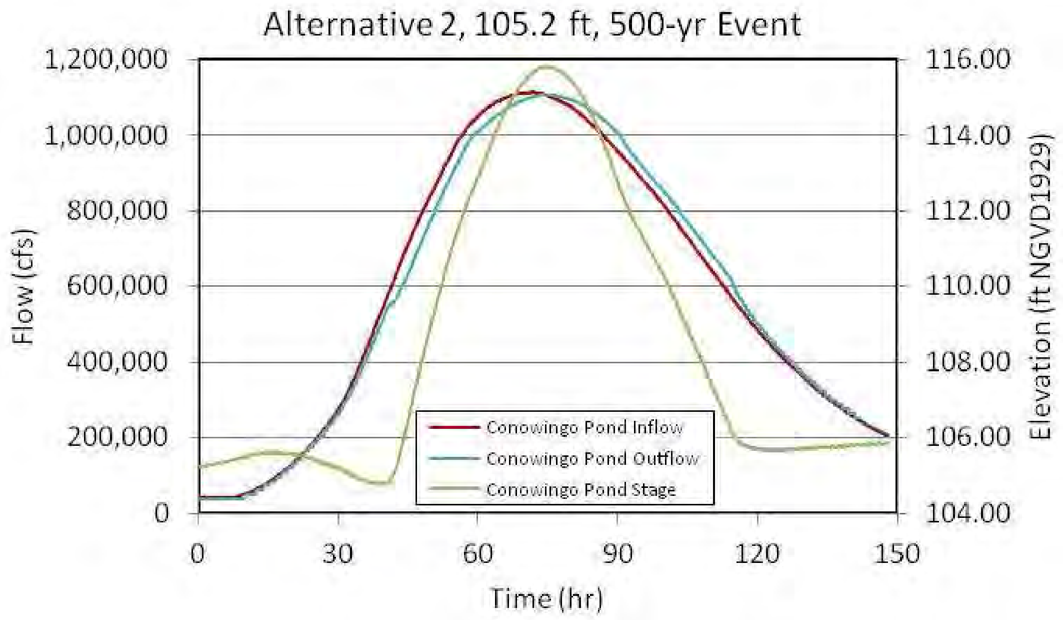


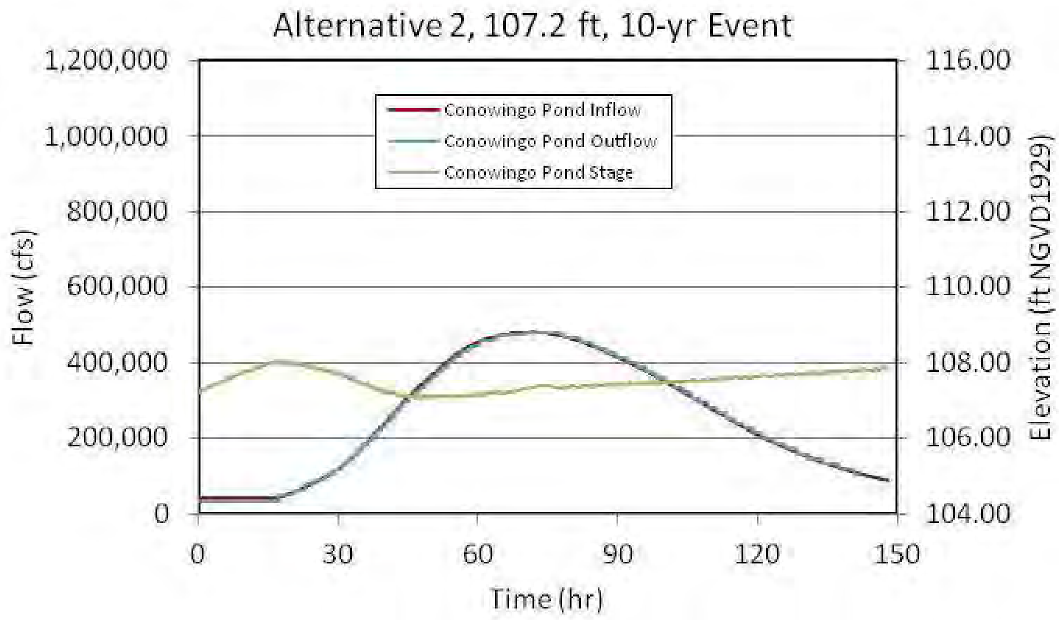
Alternative 2, 105.2 ft, 10-yr Event
Conowingo Pond Inflow
Conowingo Pond Outflow
Conowingo Pond Stage
Flow (cfs)
Elevation (ft NGVD1929)
Time (hr)
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
116.00
114.00
112.00
110.00
108.00
106.00
104.00
0
30
60
90
120
150

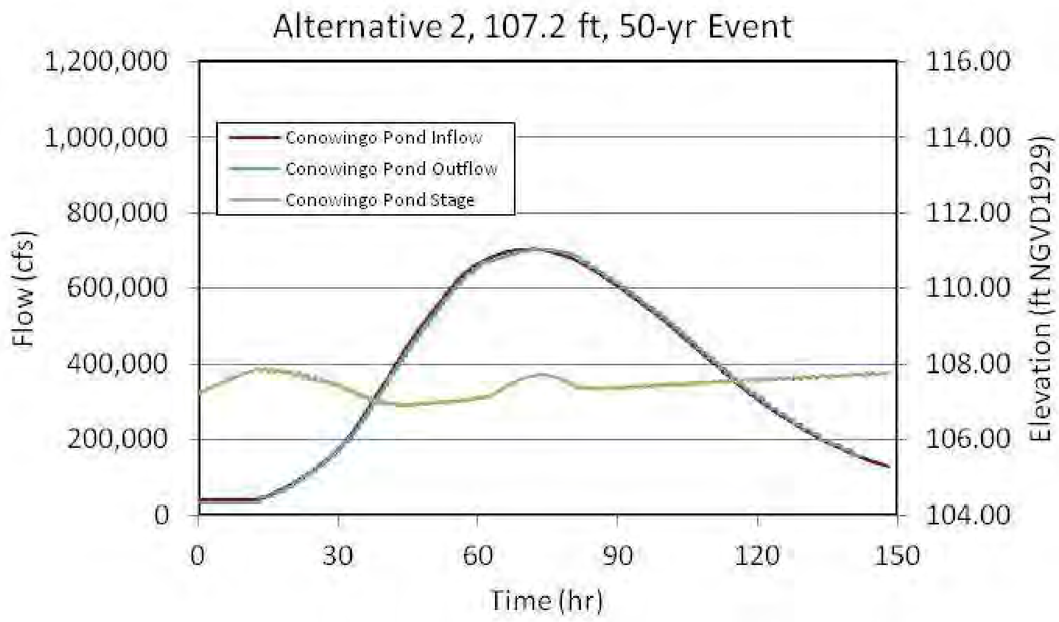


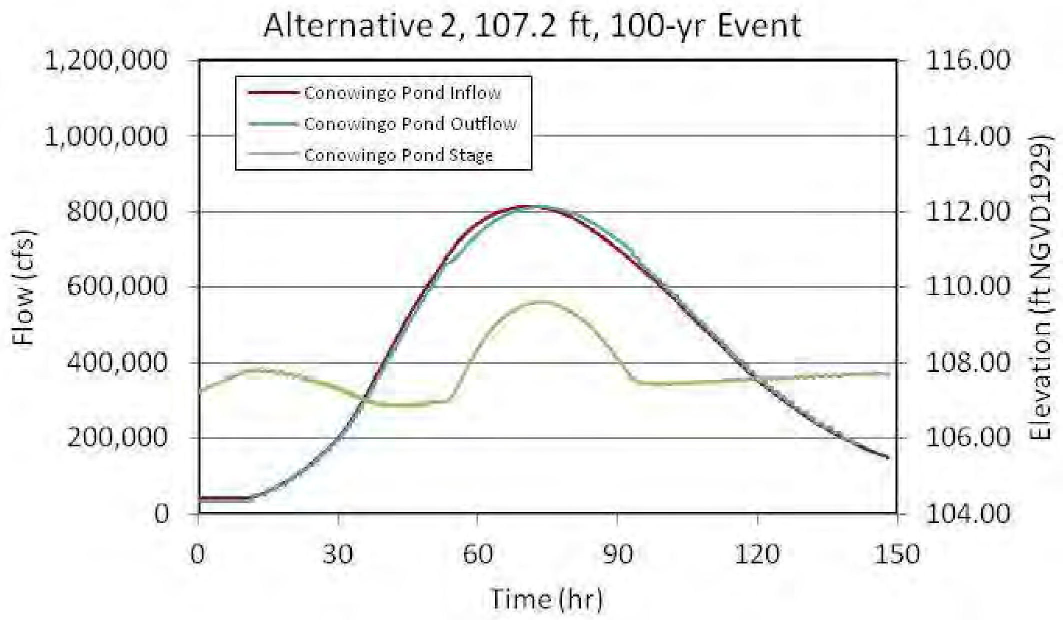
Alternative 2, 105.2 ft, 50-yr Event
Conowingo Pond Inflow
Conowingo Pond Outflow
Conowingo Pond Stage
Flow (cfs)
Elevation (ft NGVD1929)
Time (hr)
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
116.00
114.00
112.00
110.00
108.00
106.00
104.00
0
30
60
90
120
150

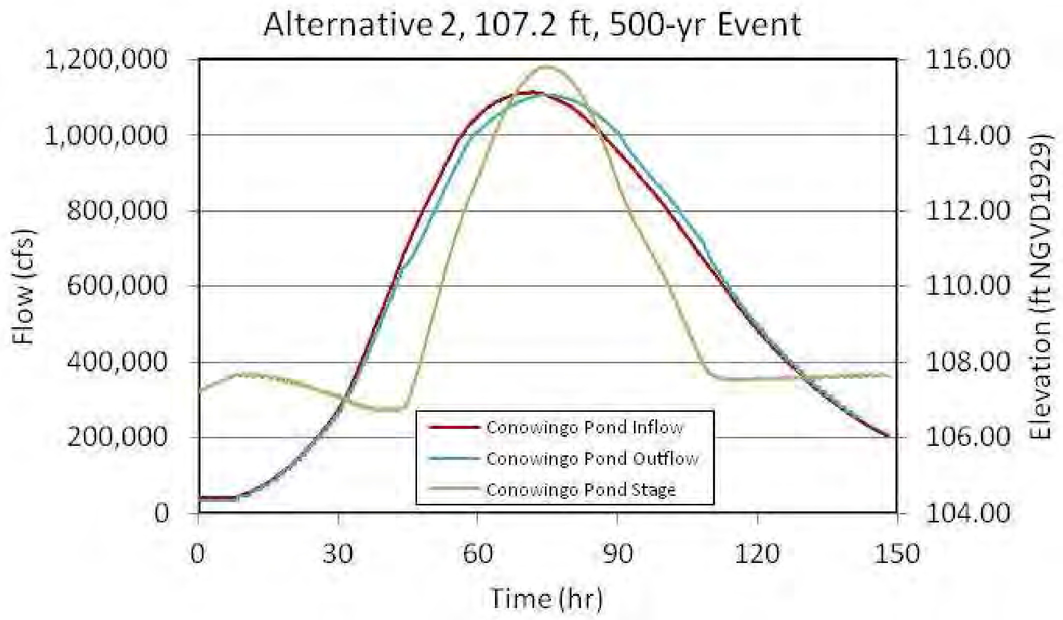

Alternative 2, 105.2 ft, 100-yr Event
Conowingo Pond Inflow
Conowingo Pond Outflow
Conowingo Pond Stage
Flow (cfs)
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
Elevation (ft NGVD1929)
116.00
114.00
112.00
110.00
108.00
106.00
104.00
0
30
60
90
120
150
Time (hr)

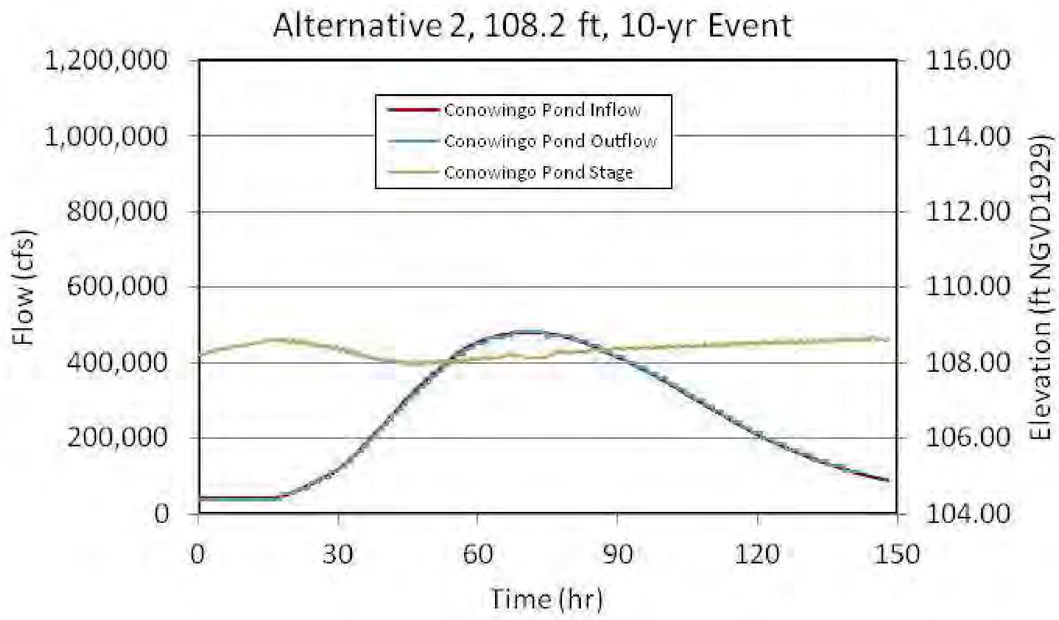

Alternative 2, 105.2 ft, 500-yr Event
Conowingo Pond Inflow
Conowingo Pond Outflow
Conowingo Pond Stage
Flow (cfs)
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
Elevation (ft NGVD1929)
116.00
114.00
112.00
110.00
108.00
106.00
104.00
0
30
60
90
120
150
Time (hr)

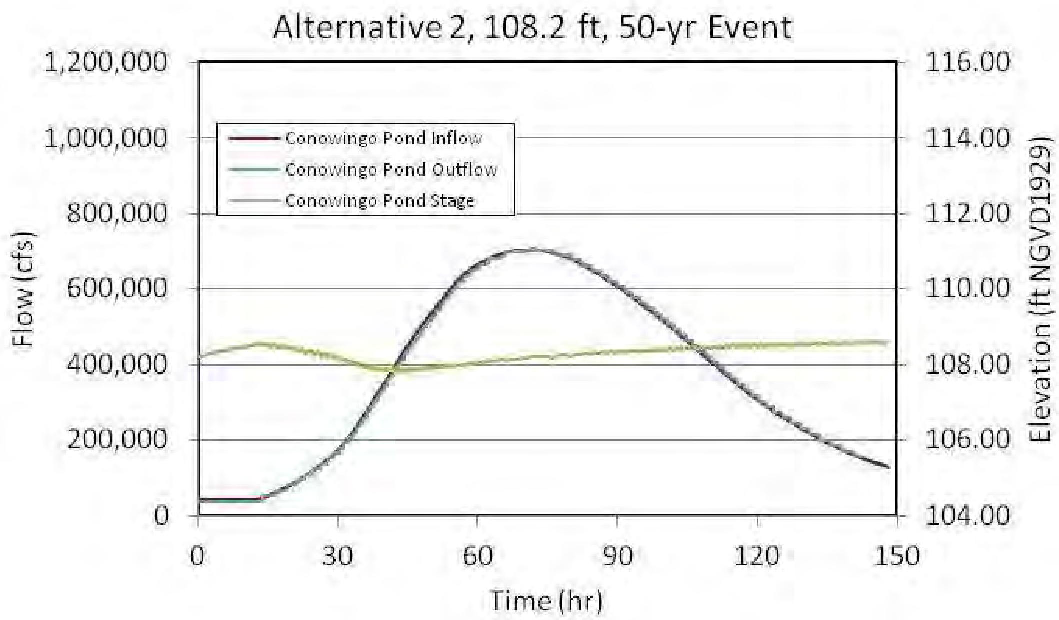

Alternative 2, 107.2 ft, 10-yr Event
Conowingo Pond Inflow
Conowingo Pond Outflow
Conowingo Pond Stage
Flow (cfs)
Elevation (ft NGVD1929)
Time (hr)
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
116.00
114.00
112.00
110.00
108.00
106.00
104.00
0
30
60
90
120
150

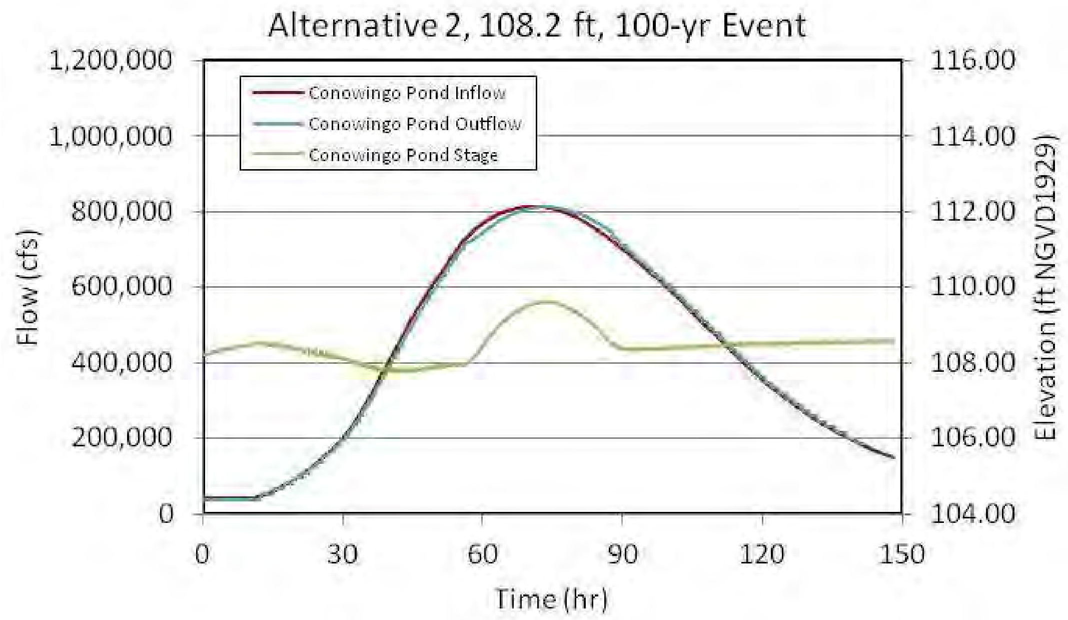

Alternative 2, 107.2 ft, 50-yr Event
Conowingo Pond Inflow
Conowingo Pond Outflow
Conowingo Pond Stage
Flow (cfs)
Elevation (ft NGVD1929)
Time (hr)
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
116.00
114.00
112.00
110.00
108.00
106.00
104.00
0
30
60
90
120
150

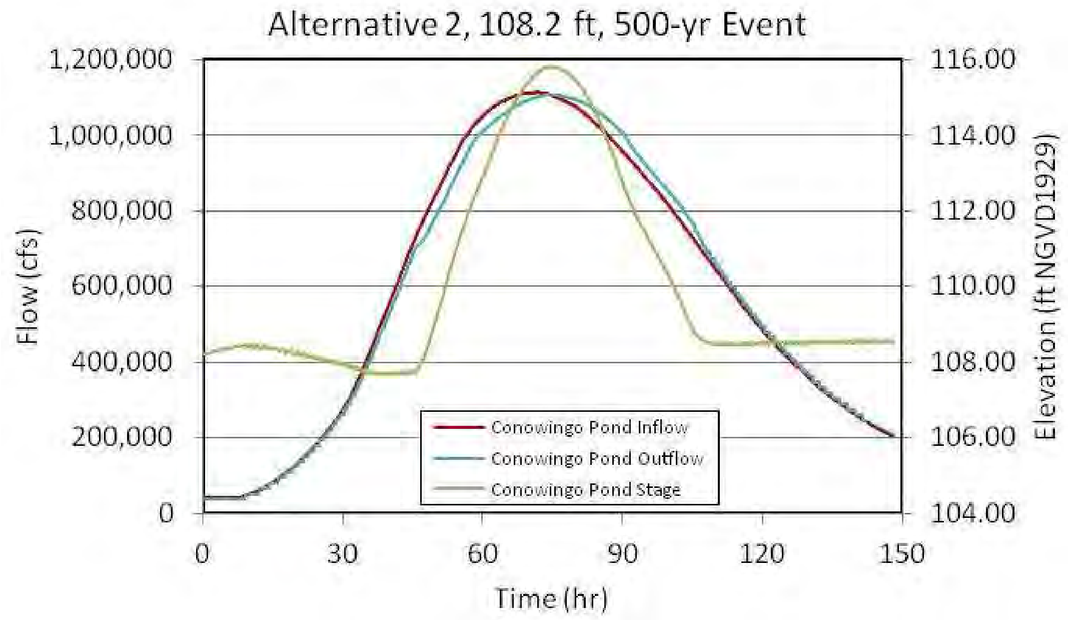

Alternative 2, 107.2 ft, 100-yr Event
Conowingo Pond Inflow
Conowingo Pond Outflow
Conowingo Pond Stage
Flow (cfs)
Elevation (ft NGVD1929)
Time (hr)
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
116.00
114.00
112.00
110.00
108.00
106.00
104.00
0
30
60
90
120
150

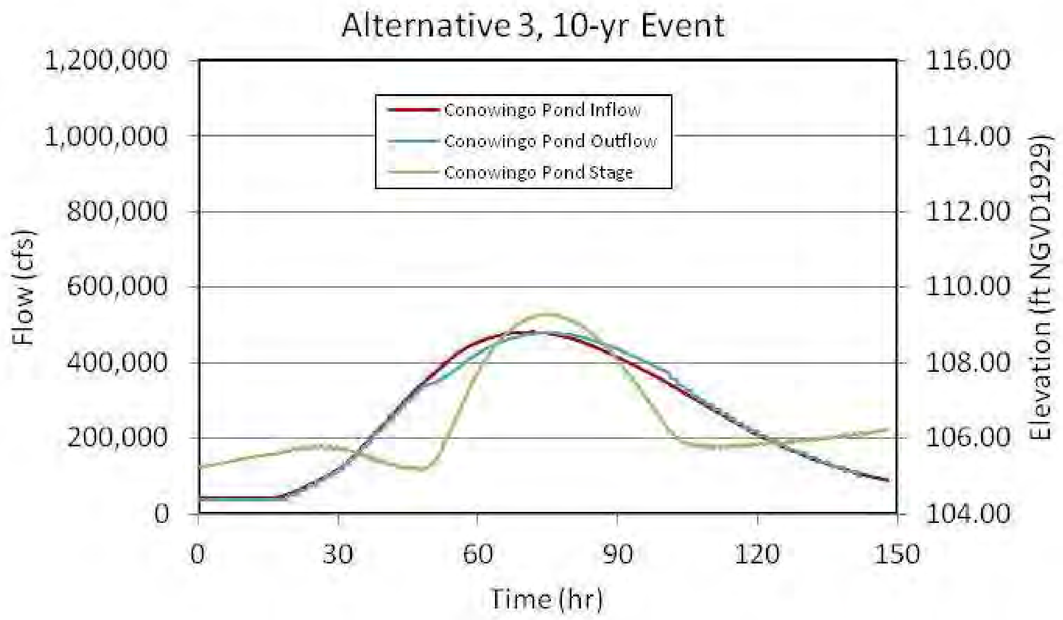

Alternative 2, 107.2 ft, 500-yr Event
Conowingo Pond Inflow
Conowingo Pond Outflow
Conowingo Pond Stage
Flow (cfs)
Elevation (ft NGVD1929)
Time (hr)
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
116.00
114.00
112.00
110.00
108.00
106.00
104.00
0
30
60
90
120
150

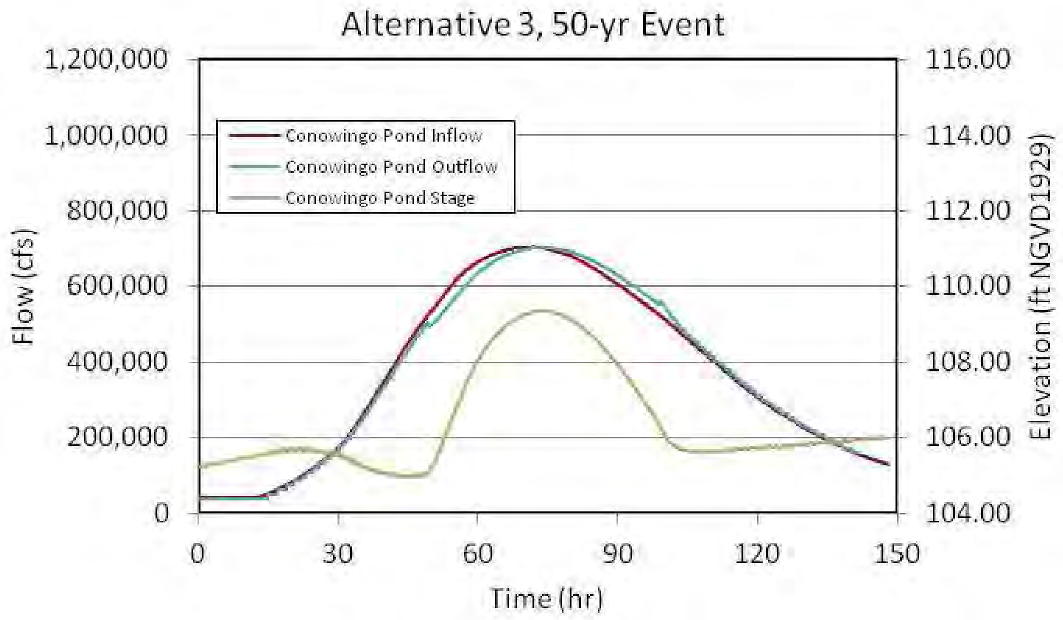

Alternative 2, 108.2 ft, 10-yr Event
Conowingo Pond Inflow
Conowingo Pond Outflow
Conowingo Pond Stage
Flow (cfs)
Elevation (ft NGVD1929)
Time (hr)
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
116.00
114.00
112.00
110.00
108.00
106.00
104.00
0
30
60
90
120
150

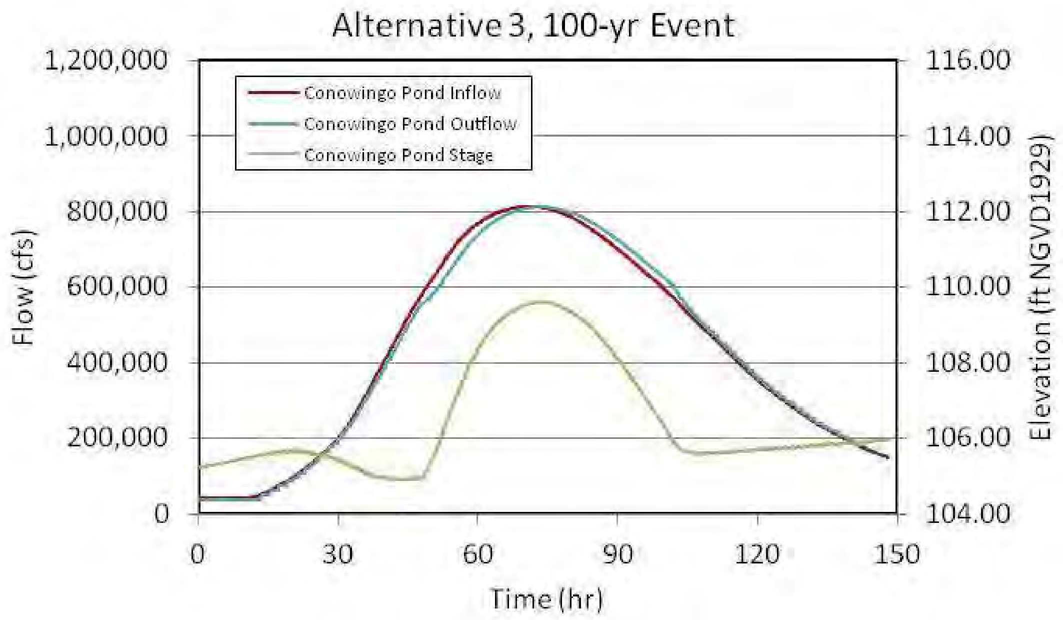

Alternative 2, 108.2 ft, 50-yr Event
Conowingo Pond Inflow
Conowingo Pond Outflow
Conowingo Pond Stage
Flow (cfs)
Elevation (ft NGVD1929)
Time (hr)
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
116.00
114.00
112.00
110.00
108.00
106.00
104.00
0
30
60
90
120
150

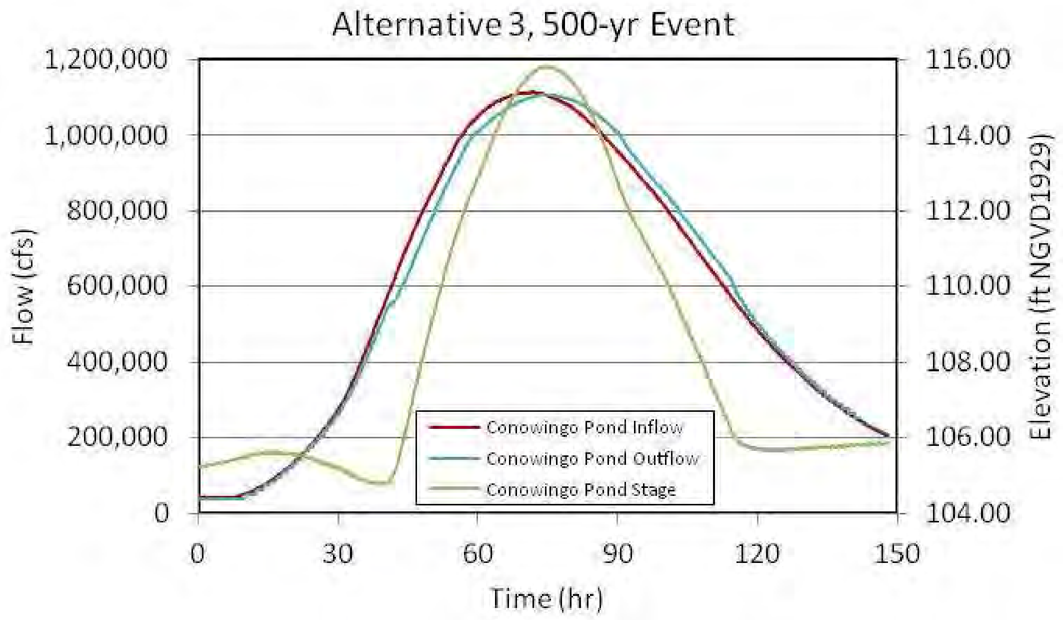

Alternative 2, 108.2 ft, 100-yr Event
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
Flow (cfs)
116.00
114.00
112.00
110.00
108.00
106.00
104.00
Elevation (ft NGVD1929)
0
30
60
90
120
150
Time (hr)
Conowingo Pond Inflow
Conowingo Pond Outflow
Conowingo Pond Stage


Alternative 2, 108.2 ft, 500-yr Event
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
Flow (cfs)
116.00
114.00
112.00
110.00
108.00
106.00
104.00
Elevation (ft NGVD1929)
0
30
60
90
120
150
Time (hr)
Conowingo Pond Inflow
Conowingo Pond Outflow
Conowingo Pond Stage


Alternative 3, 10-yr Event
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
Flow (cfs)
116.00
114.00
112.00
110.00
108.00
106.00
104.00
Elevation (ft NGVD1929)
0
30
60
90
120
150
Time (hr)
Conowingo Pond Inflow
Conowingo Pond Outflow
Conowingo Pond Stage


Alternative 3, 50-yr Event
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
Flow (cfs)
116.00
114.00
112.00
110.00
108.00
106.00
104.00
Elevation (ft NGVD1929)
0
30
60
90
120
150
Time (hr)
Conowingo Pond Inflow
Conowingo Pond Outflow
Conowingo Pond Stage


Alternative 3, 100-yr Event
Conowingo Pond Inflow
Conowingo Pond Outflow
Conowingo Pond Stage
Flow (cfs)
Elevation (ft NGVD1929)
Time (hr)
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
116.00
114.00
112.00
110.00
108.00
106.00
104.00
0
30
60
90
120
150


Alternative 3, 500-yr Event
Conowingo Pond Inflow
Conowingo Pond Outflow
Conowingo Pond Stage
Flow (cfs)
Elevation (ft NGVD1929)
Time (hr)
1,200,000
1,000,000
800,000
600,000
400,000
200,000
0
116.00
114.00
112.00
110.00
108.00
106.00
104.00
0
30
60
90
120
150