Assembly of Design Flood Hydrographs for the Green River Basin
Summary Report for Flood Plain Management Services Program
Seattle District Army Corps of Engineers
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Project Overview

This report summarizes work performed under the Flood Plain Management Services Program (FPMS) to develop design flood hydrographs for the lower Green River in Washington State. The specific objective was to develop time-series of regulated flow and stage at the location of the Auburn USGS gage (Station 12113000) for seven design floods ranging from the 50 percent chance exceedance flood event (50 percent flood) to the 0.2 percent chance exceedance flood event (0.2 percent flood). In terms of probability, the "x" percent flood has an "x" percent chance of being exceeded in any given year. Accordingly, the 50 percent chance exceedance flood event (50 percent flood) has a 50 percent chance of being exceeded in any given year. As such, the 50 percent flood is a relatively high probability event yet is of relatively small magnitude (i.e., small associated peak and volume). Conversely, the 0.2 percent flood is a relatively low probability event but is of relatively large magnitude. In addition to the aforementioned flood events, the other events evaluated in this study were the 10 percent flood, 4 percent flood, 2 percent flood, 1 percent flood, and 0.5 percent flood.

Recent evaluations by the Corps, based on information from historical flood events, indicate that flood risk management operations using Howard Hanson Dam (HHD) can manage flows at the Auburn gage to a maximum desired flow of 12,000 cfs for events up to a 0.7 percent flood (this assumes the median discharge frequency function for that event). As discussed in the HHD Water Control Manual (September 2011 revision), the 12,000 cfs target represents the approximate channel capacity of the current levee system in the Auburn vicinity (observations from recent flood events suggest that 12,000 cfs is within the current channel capacity of the existing levee system). Levees and other flood control facilities along the lower Green River in King County have historically been designed and analyzed using a design flow of 12,000 cfs. These projects, cumulatively with HHD, protect billions of dollars of residential and commercial real estate and thousands of residents and workers in the Green River valley. Stakeholders have expressed an interest that a better and more complete understanding of the current hydrology for the lower Green River is developed to facilitate floodplain management, risk evaluations, and flood protection facility designs.

The need is made more urgent by several ongoing efforts to improve flood protection along the Green River. These include ongoing levee construction and reconstruction efforts by the newly formed King County Flood Control District. In addition, the City of Kent is pursuing extensive levee improvements to protect the flood-prone valley floor within their corporate boundaries. Further, the State of Washington has recently committed $10 million to Green River levee improvements in the Horseshoe Bend area, and is considering additional funding for similar projects elsewhere on the Green River. Each of these ongoing projects would benefit greatly from increased knowledge of flood event hydrology within the lower Green River valley. While it is acknowledged that the information provided in this study may be used in the design or evaluation of these projects, use of information from this study for these purposes in no way constitutes implicit approval of such design or construction nor does the use of this information imply a positive finding in regards to the National Flood Insurance Program levee system evaluation for levees constructed with this information.

A detailed description of the methodology used to develop the flow and stage hydrographs at Auburn is provided in subsequent sections of this report. A summary of the process is as follows. First, hypothetical hydrographs for HHD reservoir inflow and “local” inflow were developed based on statistical information from observed flood events. Local inflow is defined as the cumulative, natural tributary inflow to the Green River between HHD and the Auburn gage. To account for hydrologic uncertainty, three sets of hydrographs were created for each flood event. One set captured the lower confidence limit (95% exceedance), one set captured the median or expected hydrologic condition (50% exceedance), and one set captured the upper confidence limit (5% exceedance). Second, the District’s reservoir regulation software was used to simulate operation of HHD for flood risk management using the developed hydrographs. Reservoir operations followed the water control plan as outlined in the Project Water Control Manual. This step resulted in time-series of simulated outflow (regulated outflow) from HHD. Third, a calibrated, 1-dimensional HEC-RAS model of the Green River was used to route HHD outflow through the downstream channel and combine it with local tributary inflow to yield regulated flow in the river at Auburn. Additional modeling runs were used to evaluate the impacts of hydraulic uncertainty on simulated stage time-series at Auburn. The final result of all of these steps was the creation of flow and stage hydrographs at Auburn for the seven flood events, including hydrologic and hydraulic uncertainty.

It should be noted that the flow and stage hydrographs presented in this report are in no way intended to represent the full range of possibilities associated with the selected flood events. The presented range of uncertainty is only intended to capture a reasonable amount of uncertainty in the discharge-probability function and in the stage-discharge function consistent with the guidance in EM 1110-2-1619, Risk-Based Analysis for Flood Damage Reduction Studies. Uncertainty in other factors that could influence flow and stage hydrographs, including operational uncertainty, streamflow forecast uncertainty, and uncertainty related to infrastructure integrity (i.e., potential dam or levee safety issues) are not captured in the presented results.

Development of Unregulated Hydrographs

Purpose – The purpose of this task was to develop two sets of unregulated hydrographs. One set of hydrographs was developed to characterize the inflow to HHD. The second set of hydrographs was developed to characterize the unregulated tributary inflow (“local inflow”) to the Green River between HHD and the Auburn gage. Both sets of hydrographs were developed for the 50, 10, 4, 2, 1, 0.5, and 0.2 percent flood events.

Frequency Analyses – The first step in developing synthetic inflow hydrographs was to perform frequency analyses with observed streamflow data. Twelve separate frequency analyses were performed to determine the reservoir inflow and local inflow magnitudes for each of six durations during a hypothetical flood event: the instantaneous peak and the 1-, 3-, 5-, 7-, and 15-

day average flows. The results of these analyses of reservoir inflow and local inflow are included in Table 1 and Table 2, respectively. A further discussion of the data and methods used in the frequency analyses follows.

Average daily inflow to the HHD reservoir is available dating back to 1962. The dataset is very complete and none of the occasional missing days of daily inflow were during high flow events, which would have impacted a frequency analysis of peak flows. The period of record was extended back to 1932 by using streamflow data at the Green River near Palmer, Washington USGS gage (Station 12106500). The flow was adjusted to reflect estimated flow at the current site of HHD using a ratio of drainage areas. The drainage area above the HHD tailwater gage (the gage in the river just downstream of the dam) is 221 square miles, whereas the Palmer site has a drainage area of 230 square miles. Therefore, historic flow at the HHD tailwater site was assumed to be 96 percent of the flow at Palmer (221 mi^2 / 230 mi^2 = 96 percent). Combining the adjusted Palmer flow data with observed inflow data for HHD resulted in a 78-year period of record for the frequency curve analyses. Using the 1-day average flows, running averages for 3-, 5-, 7-, and 15-day periods were also calculated and the maximum annual flow averaged over each of these other five durations were determined for each of the 78 years.

Average hourly inflow to the HHD reservoir is available dating back to 1991. This hourly inflow data is considered to be the smallest reliable timestep for calculating reservoir inflow during a flood event. Maximum hourly inflows obtained from this database were assumed to be approximately equal to the peak instantaneous inflow. Natural peak flows at Palmer for water years 1932 through 1961, before the construction of HHD, were adjusted using the drainage area method described above. To estimate missing hourly reservoir inflow between water years 1962 and 1991, a regression analysis was performed on the overlapping hourly and daily reservoir inflow data collected since 1991. The annual peak (hourly) inflow to HHD was found to reliably be approximately 130 percent of the annual maximum daily inflow, and this ratio was used to estimate peak reservoir inflow for water years 1962 through 1991. The resulting dataset of annual maximum peak reservoir inflow at HHD encompassed a 78-year period of record.

Average daily local inflow to the Green River between HHD and the Auburn stream gage is available dating back to 1961. The local inflow data is based on observed discharge from HHD, observed flow at the Green River near Auburn, Washington USGS gage (Station 12113000), and an assumed diversion from the river for water supply by Tacoma Public Utilities. The data from water year 1971 was omitted from the analysis due to poor data quality during a period potentially spanning the highest flow period of that year. The maximum annual local inflow for water year 1971 was likely near average. As such, the omission of that year would not have a significant effect on the results of the frequency analysis. The resulting dataset of daily local inflow encompassed 50 years. Using the 1-day average flows, running averages for 3-, 5-, 7-, and 15-day periods were also calculated and the maximum annual flow averaged over each of these five durations were determined for each of the 50 years.

Average hourly local inflow to the Green River between HHD and the Auburn gage is available dating back to 1991. To estimate missing hourly local inflow between water years 1962 and 1991, a regression analysis was performed on the maximum hourly and daily local inflows during 19 high flow events recorded since 1991. The peak (hourly) local inflow between HHD and Auburn was found to reliably be approximately 125 percent of the daily inflow, and this ratio was used to estimate peak local inflow for water years 1961 through 1991. The resulting dataset of annual maximum peak local inflow between HHD and Auburn encompassed a 50-year period of record.

Flow frequency analyses were performed for each of the twelve datasets: peak (hourly), 1-, 3-, 5-, 7-, and 15-day duration average flows for both the reservoir inflow and local inflow datasets. The flow frequency analyses were performed according to the methods described in *Bulletin #17B of the Hydrology Subcommittee, Guidelines for Determining Flood Flow Frequency,* published revision by the USGS in 1981.

Balanced Hydrographs – For the purpose of this study, all hypothetical reservoir and local inflow hydrographs were designed to be balanced hydrographs. A balanced hydrograph is one that has an equal exceedance probability for all possible critical durations. The results presented in this report will likely be used for various studies and alternative analyses involving flood risk analysis and planning. For these types of follow-up studies, the critical flood durations will differ or will not necessarily be known. Balanced design flood hydrographs reflect degrees of protection and risk that are comparable regardless of critical duration. Therefore, balanced hypothetical flood hydrographs are useful for such planning and design purposes.

Reservoir and local inflow hydrographs were created for the 50, 10, 4, 2, 1, 0.5, and 0.2 percent flood events. To communicate the hydrologic uncertainty involved with these hypothetical floods, three hydrograph sets were created for each of the seven exceedance probabilities: the median discharge frequency, an upper confidence limit (5 percent), and a lower confidence limit (95 percent). The resulting products were 21 reservoir inflow hydrographs and 21 local inflow hydrographs, each identifiable by a percent chance exceedance and a confidence limit. A further discussion of the data and methods used in creating the balanced hydrographs follows.

For the purpose of this study, the balanced hydrographs were constructed using an hourly time-step and lasted 360 hours (15 days). For a given hydrograph, the peak flow was given by the selected percent chance exceedance flood and confidence limit. The frequency analyses defined the required maximum, 1-, 3-, 5-, 7- and 15-day averages of each hydrograph. The characteristics of balanced hydrographs that are subject to adjustments are the shape and timing of the hypothetical floods. To help determine the shape and timing, several hydrographs from observed flood events were analyzed. The flood hydrograph from January 2009 was deemed to be the most suitable flood event to guide development of the balanced hydrographs. The event in January 2009 was a large inflow event, approximately corresponding to a 2 percent flood, so the shape of the flood event best lends itself to the extremely large hypothetical floods developed for this study. The January 2009 event is the highest peak flood event that has occurred on the Green River since hourly flow data have been available. Additionally, the January 2009 event has a distinct rising limb, peak, and receding limb that are relatively free of perturbations from secondary rain events.

To create balanced hydrographs for hypothetical flood events based on the shape and timing of the January 2009 flood event, the observed hourly data was analyzed over the durations of interest (1-hour and 1-, 3-, 5-, 7-, and 15-day). The period of the maximum 15-day running average of the observed reservoir inflow was identified as hours 1 through 360. The timing of maximum flows for the other durations during the January 2009 event are as follows: the 7-day maximum inflow occurred from hours 14 to 181, the 5-day maximum inflow occurred from hours 16 to 135, the 3-day maximum inflow occurred from hours 17 to 88, the 1-day maximum inflow occurred from hours 26 to 49, and the peak inflow occurred at hour 38. This timing was adopted for all hypothetical hydrographs created for this study. Each of the 21 reservoir inflow hydrographs and the 21 local inflow hydrographs created for this study had peak (hourly) and 1-, 3-, 5-, 7-, and 15-day maximum flows set by the frequency analyses and timing based on the observed January 2009 flood event. To finalize each hypothetical hydrograph, the hourly inflows on the ascending and descending limbs of the hydrograph were smoothed while still preserving the correct average flow over a given time period. This was accomplished by taking the initially flat segments of the balanced hydrograph and rotating them about their center to change the slope while maintaining the same average flow for the period.

Figure 1 illustrates a graphical representation of the January 2009 flood event; an initial stair-stepped, balanced hydrograph; and a final smoothed, balanced hydrograph for the 1 percent flood (median discharge frequency function). Although the balanced hydrograph appears artificial with a very pronounced peak and somewhat angular transitions (i.e., angular changes in slope), this type of hydrograph is well-suited to studies such as the current one in which there is a desire for preservation of exceedance probability throughout the hydrograph. The final balanced hydrographs used for this study are included Figures 2 through 22, which also summarize reservoir simulations discussed in the subsequent section.

Operations Modeling to Produce Outflow Hydrographs from Hanson Dam

The purpose of this step was to develop outflow hydrographs from HHD using the unregulated, balanced hydrographs discussed in the previous section. Software used by the Corps for determining project operations during real-time operations (flood and non-flood conditions) was used to determine outflow hydrographs from the dam for the desired flood events. Operations modeling was performed using the procedures outlined in the Water Control Manual (WCM) for HHD. Per the guidance in the WCM, simulated HHD operations targeted 10,000 cfs at the Auburn gage on the rising limb of the hydrograph and a desired maximum of 12,000 cfs once the local inflow hydrograph peaked. Operations modeling also utilized guidance from the discharge regulation schedule per WCM procedures. As such, HHD releases in some scenarios, as required by the discharge regulation schedule, were sufficiently high to result in regulated discharges at Auburn in excess of 12,000 cfs.

Operations modeling was performed for coincident hydrologic data sets. For example, for the 50 percent flood, median discharge frequency scenario, operations modeling used the 50 percent, median discharge frequency inflow hydrograph and the 50 percent, median discharge frequency local inflow hydrograph. Simulation of each flood event therefore required three simulation runs, one each to cover the lower confidence (95%) hydrograph pairing, the median discharge frequency pairing, and the upper confidence (5%) pairing. Based on historical observations, local inflow hydrographs were assumed to peak 8 hours after the reservoir inflow hydrographs. A total of 21 operations simulations were therefore performed to simulate all 7 of the hypothetical flood events. The resultant work product generated by this task was 21 outflow hydrographs, one for each simulation. Each outflow hydrograph was developed at a 1-hour time step and had the same 15-day duration as the input hydrograph dataset.

Operations modeling was performed using a slightly modified version of the Excel-based spreadsheet that the Seattle District Corps uses for real-time regulation. It uses a mass balance approach and operates using an hourly timestep. Releases were determined per the Water Control Manual, Section 7.03a *Winter Flood Control* and Table 7-1, *Project Operating Limits*. In the most extreme events, higher required discharges were dictated by the Discharge Regulation Schedule (DRS), (Water Control Manual, Chart 7-3). The DRS is a family of curves that relate reservoir inflow on the rising limb of the hydrograph, pool elevation, and project discharge. Project discharge values obtained from the DRS are considered to be the minimum releases necessary to prevent the reservoir elevation from exceeding design conditions or to prevent premature filling of the reservoir that would result in higher subsequent releases that could exceed the peak that would have occurred under pre-project conditions. For the purposes of the operations modeling, a seven hour travel time was assumed between the dam and the Auburn gage (i.e., it takes seven hours for discharge changes from the dam to influence Auburn flow during a flood). Note that this assumption only applies to the operations modeling and does not impact hydraulic modeling discussed in a subsequent section of this report.

Further details regarding the operations modeling, including general assumptions are as follows:

- Operations modeling was based on the reservoir storage table from the 2001 Water Control Manual. This table was updated for the 2011 revision to the Water Control Manual after this project was underway. The new table differs from the previous one mostly only within the lower elevation range of the reservoir, where storage is limited. At normal full pool (elev. 1206 feet), the new table shows 1.1% less storage compared with the previous table. For flood regulation, including the current study, the results should not be significantly impacted by the selected storage table.
- Operations modeling was performed assuming high confidence in the availability and accuracy of observed real-time data.
- Operations decisions assumed perfect forecast of the local inflow hydrograph seven hours into the future, which allowed releases to be made to target both 10,000 and 12,000 cfs at the Auburn gage on the rising limb of the hydrograph. No other forecast knowledge was assumed.
- When following ramp rate criteria for increasing project discharge, criteria were applied to the downstream USGS gage closest to the project: Green River below Howard A. Hanson Dam, Washington (Station 12105900).

Details regarding operations during the rising limb of the inflow hydrograph are as follows:

- All scenarios start with the pool at elevation 1075 feet (NGVD 1929), the approximate level of the winter operating pool, and discharge approximately equal to inflow.

- In the initial phase of each simulation, the project passed inflow (discharge equal to outflow) up to the point where doing so was forecast to push Auburn above 10,000 cfs (+/-100 cfs).
- Discharge increases were limited by maximum downstream stage increases of 1 foot/hour at the downstream gage.
- Upon Auburn flow reaching 10,000 cfs, project discharge was adjusted as needed to maintain Auburn flows at roughly 10,000 cfs as local inflows continued to rise.
- In the 2nd hour after the local inflows peaked, project releases were increased to begin targeting 12,000 cfs flow (+/- 100 cfs) at the Auburn gage.
- The DRS was followed when necessary. Once the DRS was used, the maximum discharge it required was held until the pool had peaked.

Details regarding operations during the receding limb of the inflow hydrograph are as follows:

- When the total flow at Auburn was between 7,000 cfs and 12,000 cfs, to the extent possible, discharge reductions from the project were made to limit stage reductions at Auburn to a maximum 1 foot/day in order to protect levee stability.
- In situations where the flow at Auburn reached 12,000 cfs, flows at Auburn were maintained at this level for as long as possible to maximize discharge from the project. The duration that 12,000 cfs was maintained at Auburn was dictated by several factors including a desire to limit stage reductions at Auburn at flows above 7,000 cfs and a desire for a smooth transition to an empty reservoir. In many of the smaller flood events, the desire to limit stage reductions at Auburn prevented Auburn flow from ever reaching 12,000 cfs.
- In cases where the reservoir elevation exceeded 1206 feet (normal full pool using NGVD 1929 datum), project discharge was not reduced until the pool drafted back down to 1206 feet. Upon drafting the pool to elevation 1206 feet, the project then passed inflow (project discharge equal to inflow) until flows at Auburn receded to 12,000 cfs.
- When drafting the reservoir, an attempt was made to evacuate stored water in a timely fashion while having a relatively smooth transition back to an empty reservoir and passing inflows. When feasible, flood storage was fully evacuated within the 15 day window used for the operational simulations.

In addition to the aforementioned details regarding project flood operations, specific notes for each of the flood events are provided below. Graphical results of the operations modeling, including time-series of reservoir inflow, local inflow, reservoir elevation, and simulated reservoir outflow are shown on Figures 2 through 22 (one figure for each of the 21 reservoir operations scenarios).

50 percent flood

The event was sufficiently small such that Auburn flow peaked at 10,000 cfs or less. Project discharge was limited by a desire to avoid an excessive stage reduction in Auburn on the receding limb of the flood hydrograph.

10 percent flood
For all three confidence limit scenarios, project releases were made to target 12,000 cfs at Auburn. For all scenarios the pool was fully evacuated within the 15 day simulation window.

4 percent flood
For all three confidence limit scenarios, project releases were made to target 12,000 cfs at Auburn. For all scenarios the pool was fully evacuated within the 15 day simulation window.

2 percent flood
For all three confidence limit scenarios, project releases were made to target 12,000 cfs at Auburn. In the median and lower confidence limit scenarios the pool was fully evacuated within the 15 day simulation window. In the upper confidence limit scenario, the pool was not fully drafted within the simulation window but Auburn flows had been reduced to about 8,000 cfs.

1 percent flood
For the median and lower confidence limit scenarios project releases were made to target 12,000 cfs at Auburn. The pool was fully evacuated in the lower confidence limit scenario and partially evacuated at the end of the median confidence scenario. For the upper confidence limit scenario, releases were required per the DRS that resulted in a peak flow at Auburn of about 15,100 cfs. For this scenario, the pool peaks slightly above the normal full pool of elevation 1206 feet. Once the pool dropped to 1206 feet, it was possible to reduce project outflow at target 12,000 cfs at Auburn. The reservoir was not fully evacuated in the upper confidence scenario within the simulation window.

0.5 percent flood
For the lower confidence limit scenario project releases are made to target 12,000 cfs at Auburn. The reservoir elevation does not reach full pool and storage is fully evacuated within the 15 day simulation. For the median confidence scenario, releases are required per the DRS resulting in a peak at Auburn near 12,600 cfs. Subsequent releases are made to target 12,000 cfs at Auburn. The pool peaks less than a foot from normal full pool and is not fully evacuated within the simulation period. For the upper confidence limit scenario, releases required per the DRS result in a peak at Auburn of about 20,000 cfs. The pool peaks about one foot above normal full pool resulting in elevated project discharge for an extended period as the pool is evacuated. The pool is not fully evacuated at the end of the simulation and Auburn flows remain at 12,000 cfs.

0.2 percent flood
For the lower confidence limit scenario project releases are made to target 12,000 cfs at Auburn. The reservoir elevation does not reach full pool. Flows at Auburn remain at 12,000 cfs for an extended period but begin to decline near the end of the simulation window as pool evacuation progresses. For the median and upper confidence limit scenarios, releases are required per the DRS resulting in Auburn peaks well above 12,000 (Auburn peaks close to 27,000 cfs in the upper confidence limit scenario). In both scenarios, the pool peaks above normal full pool. Furthermore, in both scenarios pool evacuation is underway at the end of the simulation but Auburn flows remain at 12,000 cfs.

Hydraulic Modeling to Produce Design Hydrographs

Model Description and Purpose - Flood hydrograph routing was conducted using a 1-D unsteady HEC-RAS model which was developed for the King County River and Floodplain Management Section by Northwest Hydraulic Consultants Inc. in 2009. The purpose of the model is to route outflow flood hydrographs from HHD through the middle Green River and produce flow and stage hydrographs at the USGS Auburn gage with the assumption that levees in the lower Green River valley are sufficiently high to keep all water in the channel.

Existing Model Geometry - The existing model was developed as a high flow model which included all overbank areas up to the valley walls to better simulate emergency release water surfaces from HHD. The reach between river mile 3.8 and 44.4 was developed from a bathymetry survey for most cross sections surveyed by Minister-Glaeser Surveying in early 2006 and 2007, as well as topographic surveys and new aerial photogrammetric-based topography. The reach between river mile 44.4 and 64.3 was developed from several surveys including bathymetric and topographic point surveys performed by the City of Tacoma in 2009, cross section surveys by USACE in 2008, cross section surveys by NHC in 2009, and LIDAR data. 508 surveyed cross sections run from just below the outlet works at HHD (river mile 64.295) to just downstream of the 16th Ave. S. Bridge in Seattle, where the river stage is tidally dominated. The model includes one inline structure at river mile 60.983 representing the Tacoma Public Utilities diversion dam, as well as 42 bridges throughout the model. Ineffective flow areas and levees were also added to better represent actual flow conditions. The model utilizes the NAVD 1988 vertical datum, all output produced from the model is referenced to this datum.

Changes to Model - The existing model taken from NHC was originally a steady flow model and needed to be updated to allow for smooth and timely unsteady calculations. To accurately depict changes in energy gradient between cross sections and to increase model stability, cross sections were interpolated to a maximum of 500 feet apart, making a total of 931 cross sections. Tall levees were added to the model from RM 31.903 to the downstream boundary over the locations of existing levees to simulate the maximum water surface within the lower reach of the Green River if levees were sufficiently high to maintain all flow within the levee system (this same assumption was not made to any existing levees upstream of RM 31.903 [i.e., this assumption does not apply to levees in the middle Green River reach]). Note that depiction of the levees in this manner is not intended to show the existing levee configuration, but instead is a modeling technique to confine all river flow to within the channel, thereby resulting in conservatively high water surface profiles within the lower modeled reach (i.e., Auburn vicinity).

The bridge modeling approach also required modification due to the high flows modeled for this study. A large number of bridges overtop at this flow and required the modeling approach to be changed from the energy method to the weir/orifice method when the bridges just overtop, with weir flow going over the bridge deck and pressurized orifice flow going under the bridge deck. Other changes included adding ineffective flow areas where appropriate, and deleting some cross sections that were causing model instability issues because they were too close together.

Boundary Conditions - The downstream boundary was set at a constant 8 feet (NAVD 1988 datum), which is approximately mean high water for Puget Sound at Seattle, though the Auburn gage (point of interest) is far enough upstream that it is not affected by any tidal influence or by the downstream boundary condition. The upstream boundary is the appropriate outflow hydrograph applied just downstream of HHD (simulation of outflow hydrographs is discussed in the previous section) corresponding to a range of percent chance exceedance flood outflows, including the 95 percent and 5 percent confidence limits, for a total of 21 different hydrographs. There are two internal boundary conditions in the model representing local lateral inflows from Big Soos Creek at river mile 33.322, and Newaukum Creek at river mile 40.163. Calculated local inflows between the dam and the Auburn gage were split evenly between these two tributaries.

Model Calibration - Model calibration was checked for a range of flows between about 1,500 cfs to about 26,000 cfs based on a rating curve from the Green River near Auburn USGS gage (Station 12113000). Calibration was not expected to be exact because the final model differs (with assumed tall levees throughout the lower reach) from actual conditions. The maximum divergence of model results from observed stage (1.5 feet) occurred for only the most extreme events and is attributed to the assumed tall levees in the model.

Model Sensitivity - The model sensitivity to hydraulic roughness was tested by varying the Manning’s n value by +/- 10 percent from calibrated values. The resultant magnitude of change to this variation depended on the given flow rate. For a flow rate of 15,000 cfs, stages at the Auburn gage increase by about 0.9 ft when Manning’s n was increased 10 percent, and stages decreased by 0.8 ft when Manning’s n was decreased 10 percent. The high and low Manning’s n model run results encompass the rating curve at the Auburn gage showing that these runs probably capture the hydraulic uncertainty well.

Model Output - The final model output consists of flow and stage hydrographs at cross section 31.276 (the Auburn gage). For each hypothetical flood, results were collected for all associated scenarios pertaining to that event including the three hydrologic confidence limit scenarios and the hydraulic roughness (Manning’s n) sensitivity simulations. Results for all associated scenarios were grouped together and the maximum and minimum values were used to produce 95 and 5 percent confidence bounds for the final stage and flow hydrographs. Results of the modeling, including peak flow and stage at the Auburn gage and hydrograph duration above 10,000 and 12,000 cfs are summarized in Table 3. The flow and stage hydrographs are presented in Figures 23 through 36. All stages shown on these hydrographs and in Table 3 are referenced to the NAVD 1988 vertical datum.

Conclusions

The results suggest that for events up to the 2 percent flood, there is a very highly likelihood that flows at Auburn can be regulated to a maximum of about 12,000 cfs. For the 1 percent flood, the results suggest that the most probable outcome is an Auburn peak of about 12,000 cfs. However there is a chance that flows could peak significantly above this value (i.e., about 15,100 cfs for a confidence limit with a 5% chance of exceedance), which is reflective of the considerable uncertainty in the magnitude of the peak and volume of the inflow and local hydrographs for the 1 percent flood event.

Results for the 0.5 percent and 0.2 percent flood events also show considerable variability in the magnitude of the peak flow and stage. For the 0.5 percent flood, the range (90% confidence) in the likely peak flow at Auburn extends from 12,000 cfs up to about 20,000 cfs, with a median peak value of about 12,600 cfs. As such, there is a 50% probability that the peak at Auburn will be greater than 12,600 cfs. The probability of the peak reaching 20,000 cfs or greater is 5%. For the 0.2 percent flood, the range (90% confidence) in the likely peak flow at Auburn extends from 12,000 cfs up to about 26,800 cfs, with a median peak value of about 18,800 cfs. This suggests that it is very likely that the peak at Auburn will exceed 12,000 cfs, most likely by a considerable amount (50% probability of a peak of 18,800 cfs or greater).

Table 1 - Design flood flow magnitudes averaged over various durations, HHD reservoir inflow in cubic feet per second

Flood Event	Confidence Level	Instantaneous Peak	1-day	2-day	3-day	4-day	5-day	7-day	15-day
0.2 Percent Flood	Median Discharge	50,545	38,451	30,194	26,130	22,574	19,558	15,468	9,612
	Upper Confidence Limit (5%)	61,557	46,532	36,002	30,964	26,603	22,910	17,932	10,937
	Lower Confidence Limit (95%)	39,460	30,169	24,053	20,849	18,136	15,843	12,734	8,109
0.5 Percent Flood	Median Discharge	43,183	32,854	25,959	22,347	19,389	16,898	13,546	8,559
	Upper Confidence Limit (5%)	52,174	39,449	30,726	26,288	22,687	19,659	15,606	9,685
	Lower Confidence Limit (95%)	34,623	26,478	21,225	18,326	15,998	14,044	11,413	7,368
1 Percent Flood	Median Discharge	37,937	28,880	22,948	19,691	17,145	15,014	12,164	7,790
	Upper Confidence Limit (5%)	45,465	34,402	26,960	22,992	19,918	17,346	13,924	8,766
	Lower Confidence Limit (95%)	31,033	23,749	19,133	16,480	14,430	12,718	10,426	6,808
2 Percent Flood	Median Discharge	32,931	25,100	20,082	17,191	15,027	13,226	10,833	7,039
	Upper Confidence Limit (5%)	39,072	29,605	23,374	19,887	17,300	15,148	12,301	7,866
	Lower Confidence Limit (95%)	27,487	21,061	17,070	14,678	12,896	11,414	9,444	6,244
4 Percent Flood	Median Discharge	28,135	21,487	17,336	14,822	13,012	11,516	9,541	6,299
	Upper Confidence Limit (5%)	32,973	25,040	19,950	16,953	14,817	13,051	10,729	6,979
	Lower Confidence Limit (95%)	23,963	18,395	15,021	12,907	11,382	10,123	8,458	5,670
10 Percent Flood	Median Discharge	22,032	16,902	13,836	11,840	10,466	9,338	7,864	5,320
	Upper Confidence Limit (5%)	25,310	19,314	15,633	13,298	11,708	10,405	8,706	5,814
	Lower Confidence Limit (95%)	19,270	14,854	12,290	10,571	9,379	8,403	7,123	4,879
50 Percent Flood	Median Discharge	11,163	8,740	7,518	6,554	5,905	5,381	4,703	3,395
	Upper Confidence Limit (5%)	12,349	9,630	8,215	7,130	6,405	5,820	5,063	3,621
	Lower Confidence Limit (95%)	10,095	7,934	6,881	6,024	5,443	4,975	4,369	3,183

Table 2 - Design flood flow magnitudes averaged over various durations, local inflow to Green River in cubic feet per second

Flood Event	Confidence Level	Instantaneous Peak	1-day	2-day	3-day	4-day	5-day	7-day	15-day
0.2 Percent Flood	Median Discharge	10,196	8,171	7,361	6,393	6,038	5,632	5,155	3,895
	Upper Confidence Limit (5%)	12,372	9,919	8,957	7,737	7,324	6,807	6,229	4,646
	Lower Confidence Limit (95%)	7,836	6,274	5,680	5,000	4,703	4,427	4,064	3,113
0.5 Percent Flood	Median Discharge	8,883	7,113	6,458	5,671	5,349	5,017	4,608	3,497
	Upper Confidence Limit (5%)	10,719	8,588	7,813	6,826	6,453	6,031	5,538	4,150
	Lower Confidence Limit (95%)	7,044	5,636	5,131	4,554	4,280	4,044	3,722	2,861
1 Percent Flood	Median Discharge	7,941	6,356	5,801	5,137	4,840	4,560	4,198	3,199
	Upper Confidence Limit (5%)	9,518	7,622	6,972	6,144	5,801	5,446	5,014	3,773
	Lower Confidence Limit (95%)	6,445	5,155	4,711	4,208	3,952	3,746	3,454	2,664
2 Percent Flood	Median Discharge	7,035	5,628	5,162	4,609	4,338	4,104	3,788	2,900
	Upper Confidence Limit (5%)	8,357	6,688	6,149	5,466	5,156	4,862	4,487	3,394
	Lower Confidence Limit (95%)	5,842	4,671	4,284	3,851	3,615	3,437	3,175	2,458
4 Percent Flood	Median Discharge	6,158	4,924	4,535	4,083	3,839	3,647	3,373	2,597
	Upper Confidence Limit (5%)	7,230	5,783	5,340	4,789	4,512	4,274	3,954	3,010
	Lower Confidence Limit (95%)	5,230	4,179	3,846	3,480	3,265	3,113	2,881	2,242
10 Percent Flood	Median Discharge	5,022	4,012	3,711	3,378	3,172	3,029	2,809	2,185
	Upper Confidence Limit (5%)	5,780	4,618	4,285	3,889	3,658	3,485	3,233	2,489
	Lower Confidence Limit (95%)	4,389	3,505	3,236	2,955	2,771	2,652	2,460	1,931
50 Percent Flood	Median Discharge	2,886	2,302	2,126	1,973	1,849	1,780	1,654	1,332
	Upper Confidence Limit (5%)	3,194	2,548	2,359	2,184	2,050	1,970	1,832	1,464
	Lower Confidence Limit (95%)	2,609	2,081	1,918	1,783	1,669	1,609	1,496	1,213

Table 3 – Simulated regulated flow and stage[a] in the Green River at Auburn, WA (at USGS gage 12113000)

Flood Event	Confidence Level	Peak Flow (cfs)	Peak Stage (feet NAVD88)	Approximate Duration Above 12,000 cfs (days)	Approximate Duration Above 10,000 cfs (days)
0.2 Percent Flood	Median	18,800	70.5	3.8	> 13
	Upper Confidence Limit (5%)	26,800	76.0	4.3	>13
	Lower Confidence Limit (95%)	12,000	66.1	0.0	11.0
0.5 Percent Flood	Median	12,600	66.9	3.2	>13
	Upper Confidence Limit (5%)	20,000	71.7	4.3	>13
	Lower Confidence Limit (95%)	12,000	66.2	0.0	9.4
1 Percent Flood	Median	12,000	66.7	0.0	11.0
	Upper Confidence Limit (5%)	15,100	69.0	2.6	>13
	Lower Confidence Limit (95%)	12,000	66.0	0.0	7.5
2 Percent Flood	Median	12,000	66.8	0.0	9.0
	Upper Confidence Limit (5%)	12,000	67.4	0.0	11.7
	Lower Confidence Limit (95%)	12,000	66.1	0.0	6.3
4 Percent Flood	Median	12,000	66.7	0.0	5.7
	Upper Confidence Limit (5%)	12,000	67.4	0.0	8.9
	Lower Confidence Limit (95%)	12,000	66.1	0.0	4.5
10 Percent Flood	Median	12,000	66.7	0.0	3.5
	Upper Confidence Limit (5%)	12,000	67.3	0.0	5.7
	Lower Confidence Limit (95%)	11,900	65.9	0.0	2.8
50 Percent Flood	Median	9,200	65.2	0.0	0.0
	Upper Confidence Limit (5%)	9,900	66.1	0.0	0.0
	Lower Confidence Limit (95%)	9,200	64.7	0.0	0.0

a. Stage values assume hypothetical levees sufficiently tall to constrain all flow within the existing levee system

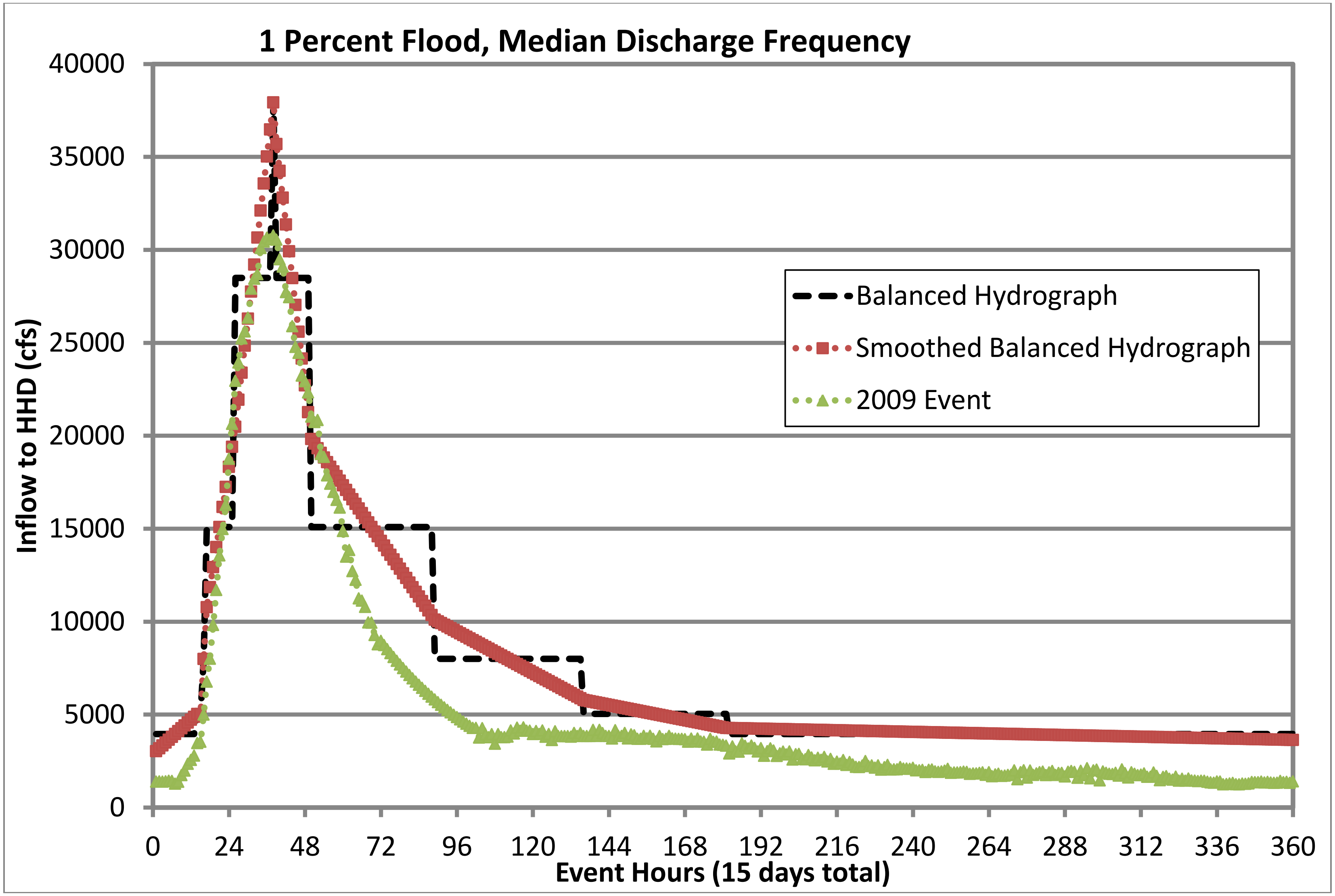

Figure 1 – Example balanced hydrograph, with shape and timing based on January 2009 flood event

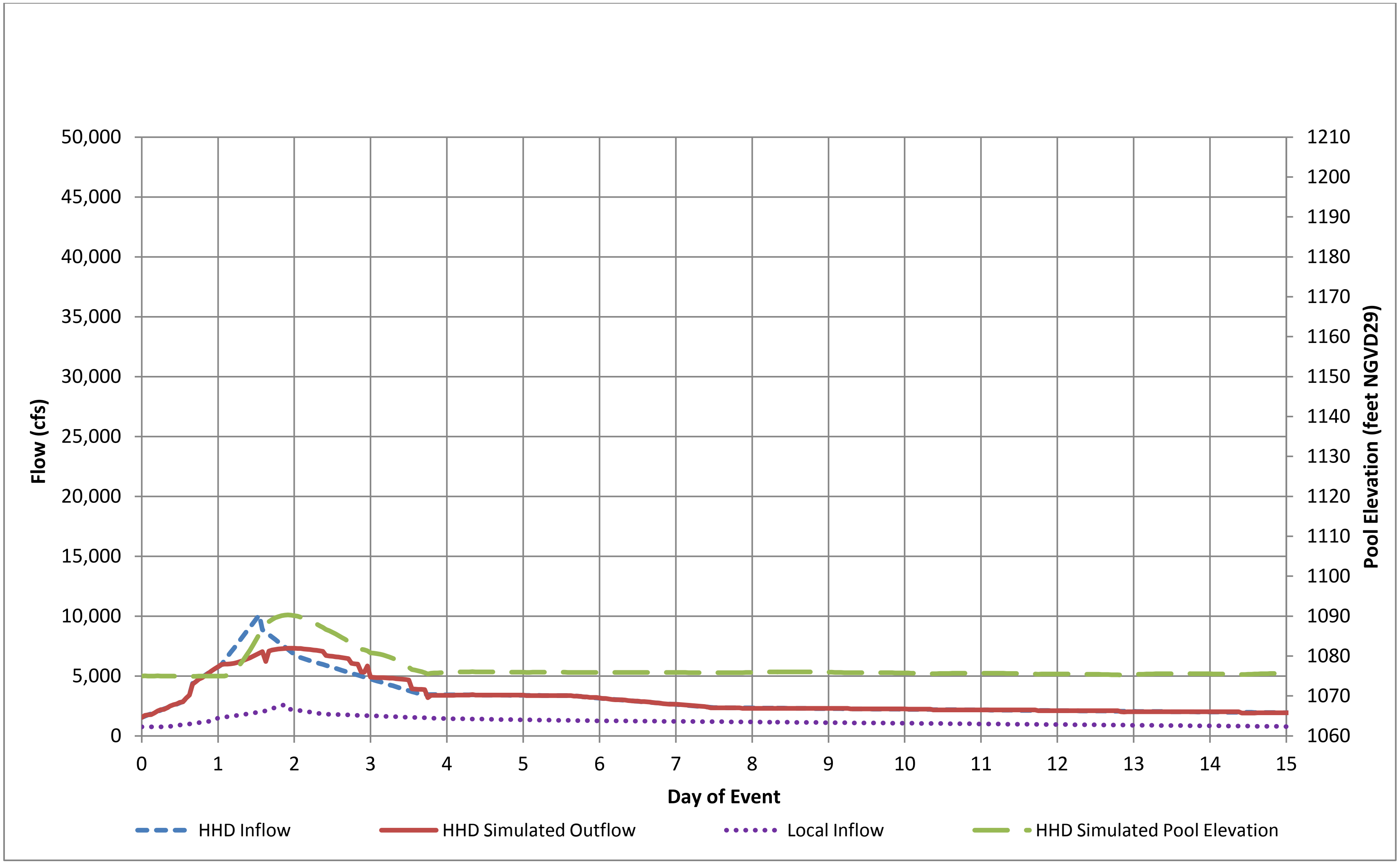

Figure 2 – Simulated Howard Hanson Dam Operations, 50 Percent Flood, Lower Confidence Limit (95%)

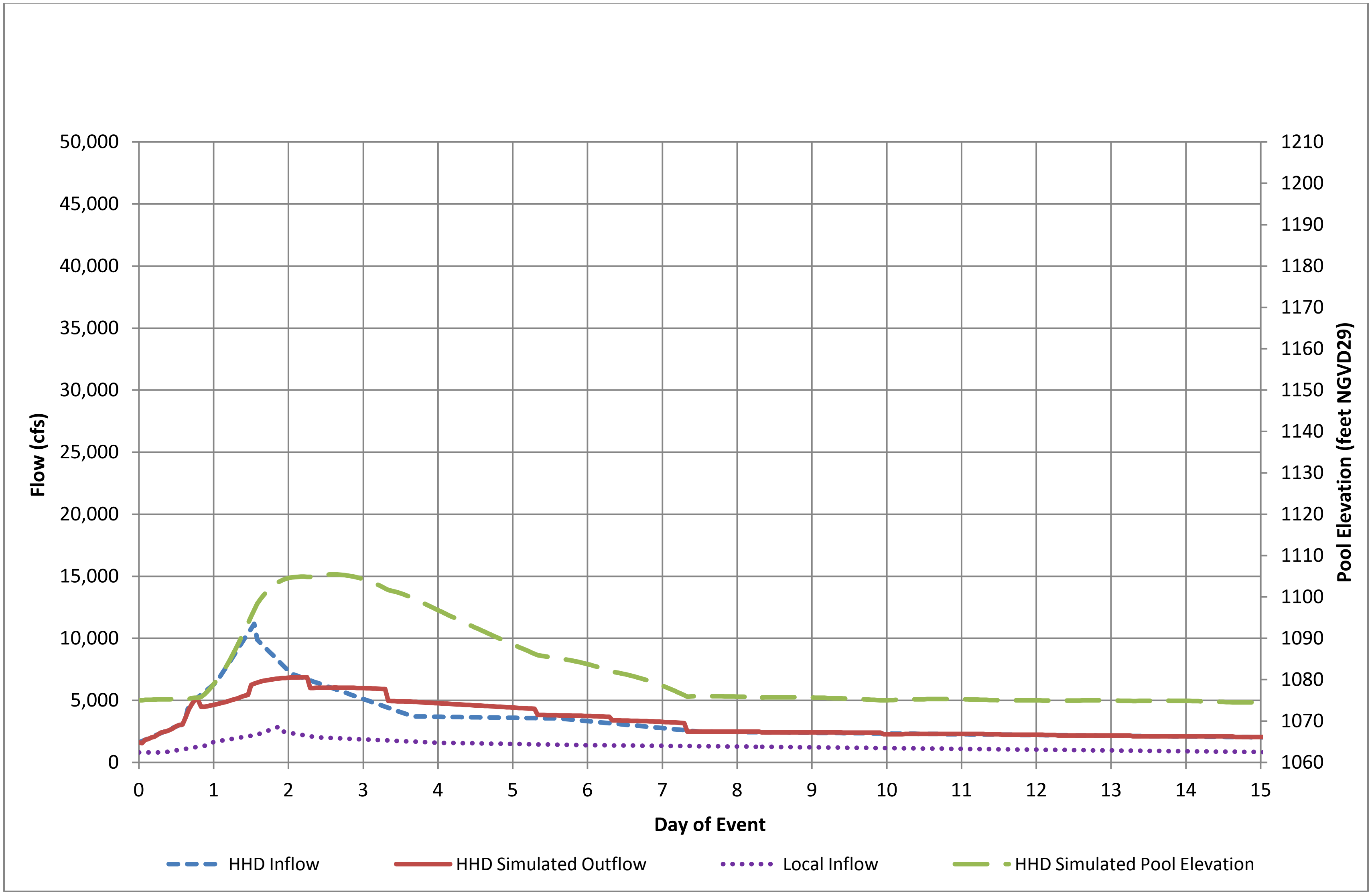

Figure 3 – Simulated Howard Hanson Dam Operations, 50 Percent Flood, Median Discharge Frequency Function

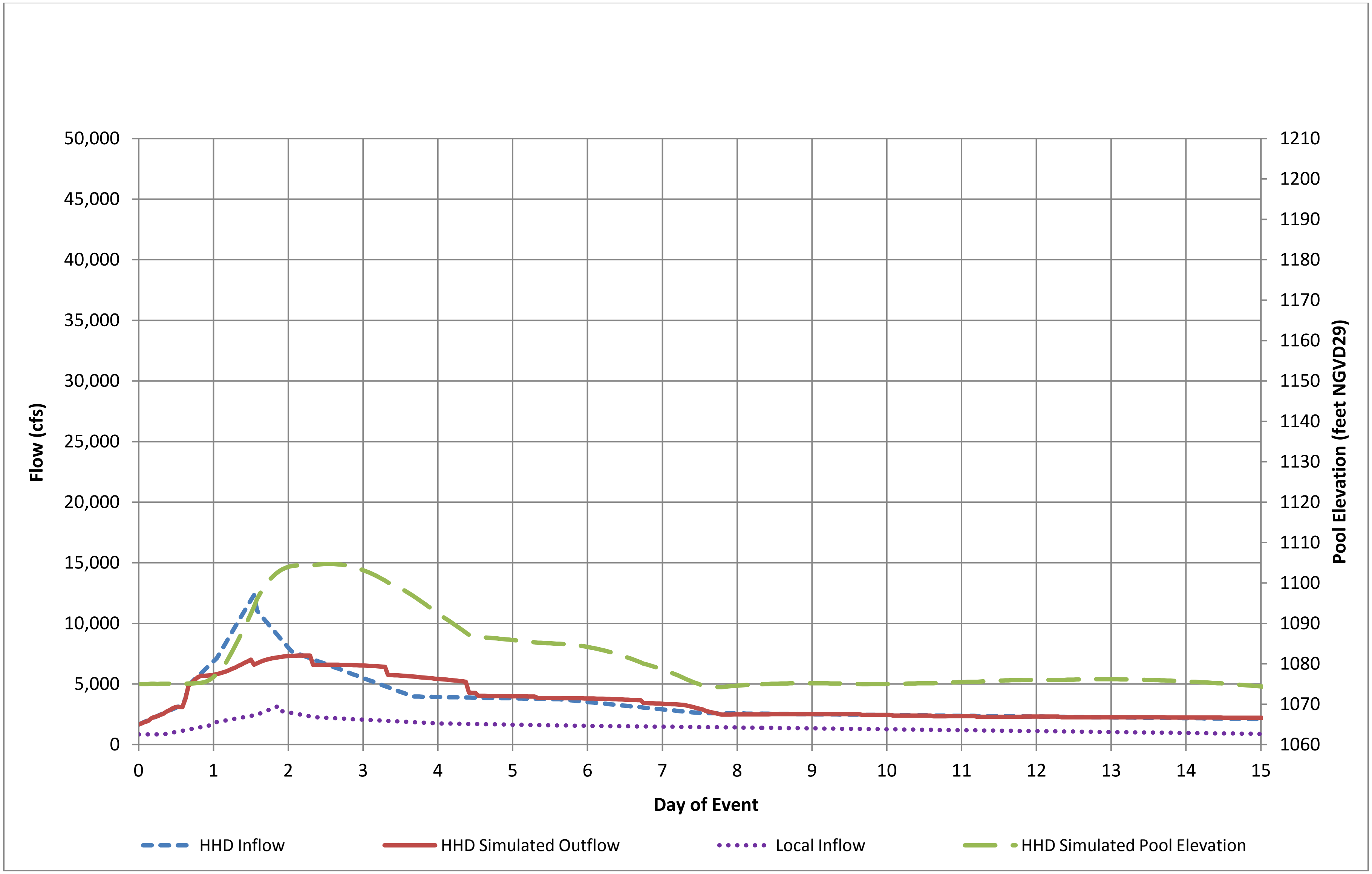

Figure 4 – Simulated Howard Hanson Dam Operations, 50 Percent Flood, Upper Confidence Limit (5%)

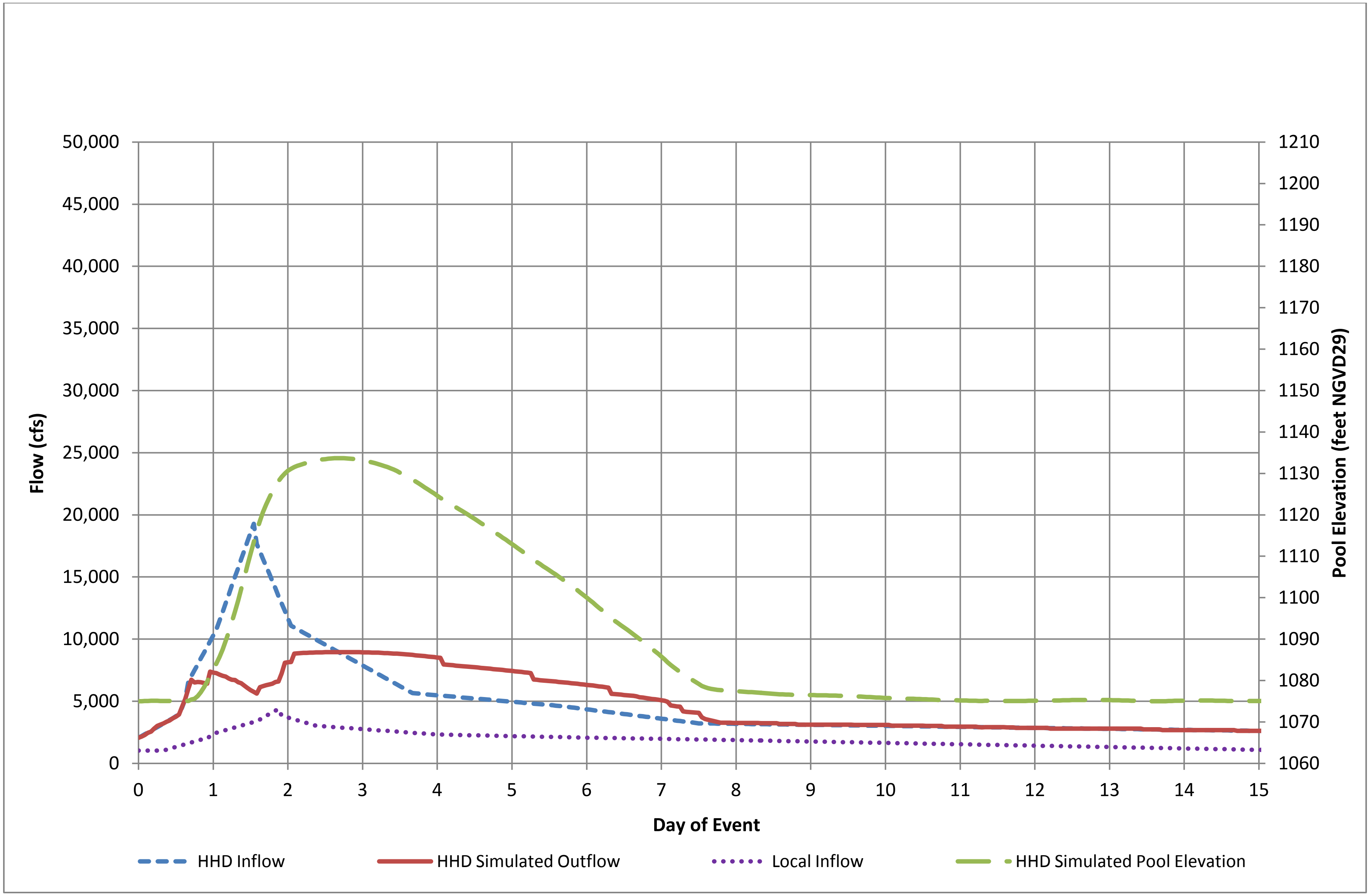

Figure 5 – Simulated Howard Hanson Dam Operations, 10 Percent Flood, Lower Confidence Limit (95%)

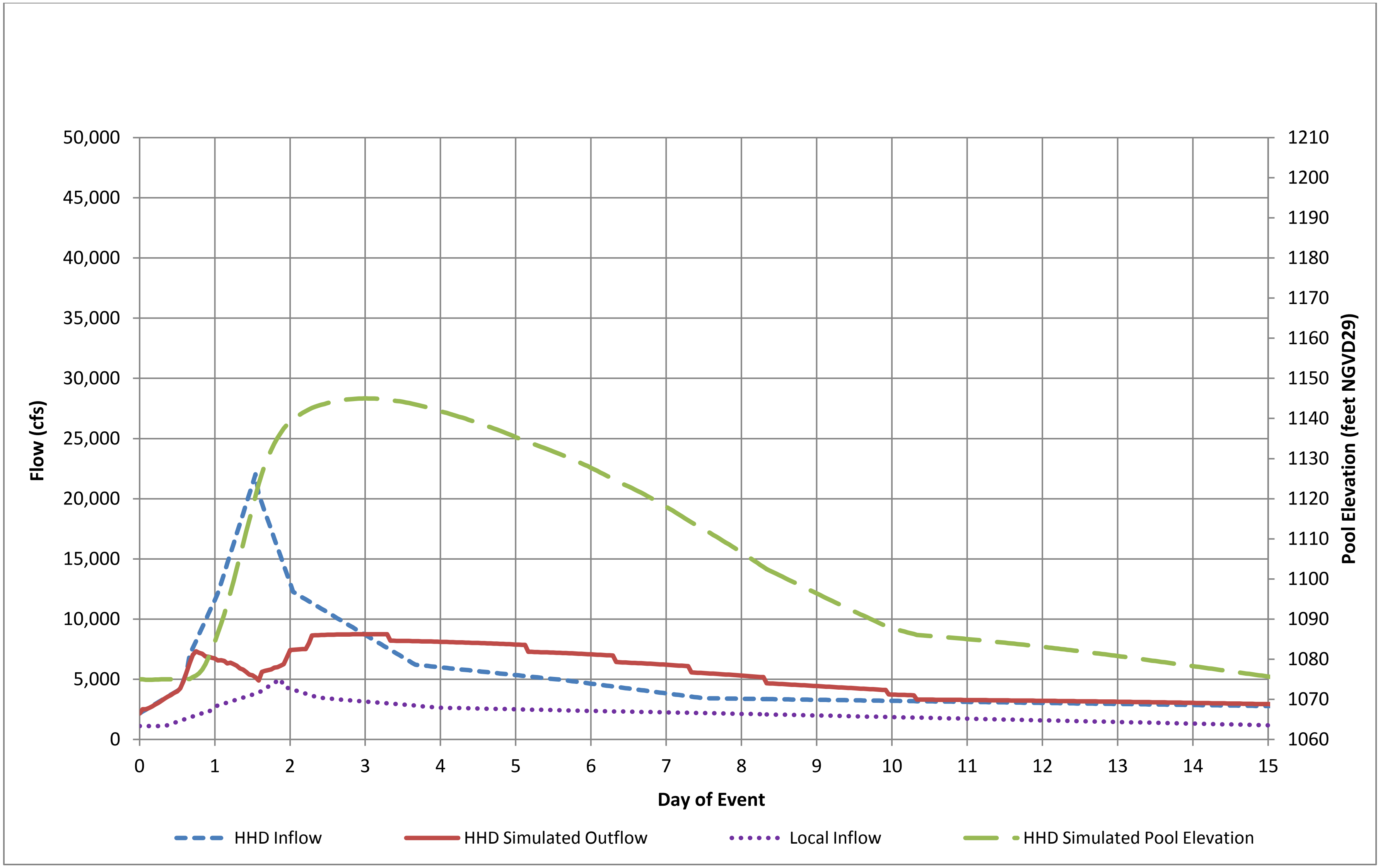

Figure 6 – Simulated Howard Hanson Dam Operations, 10 Percent Flood, Median Discharge Frequency Function

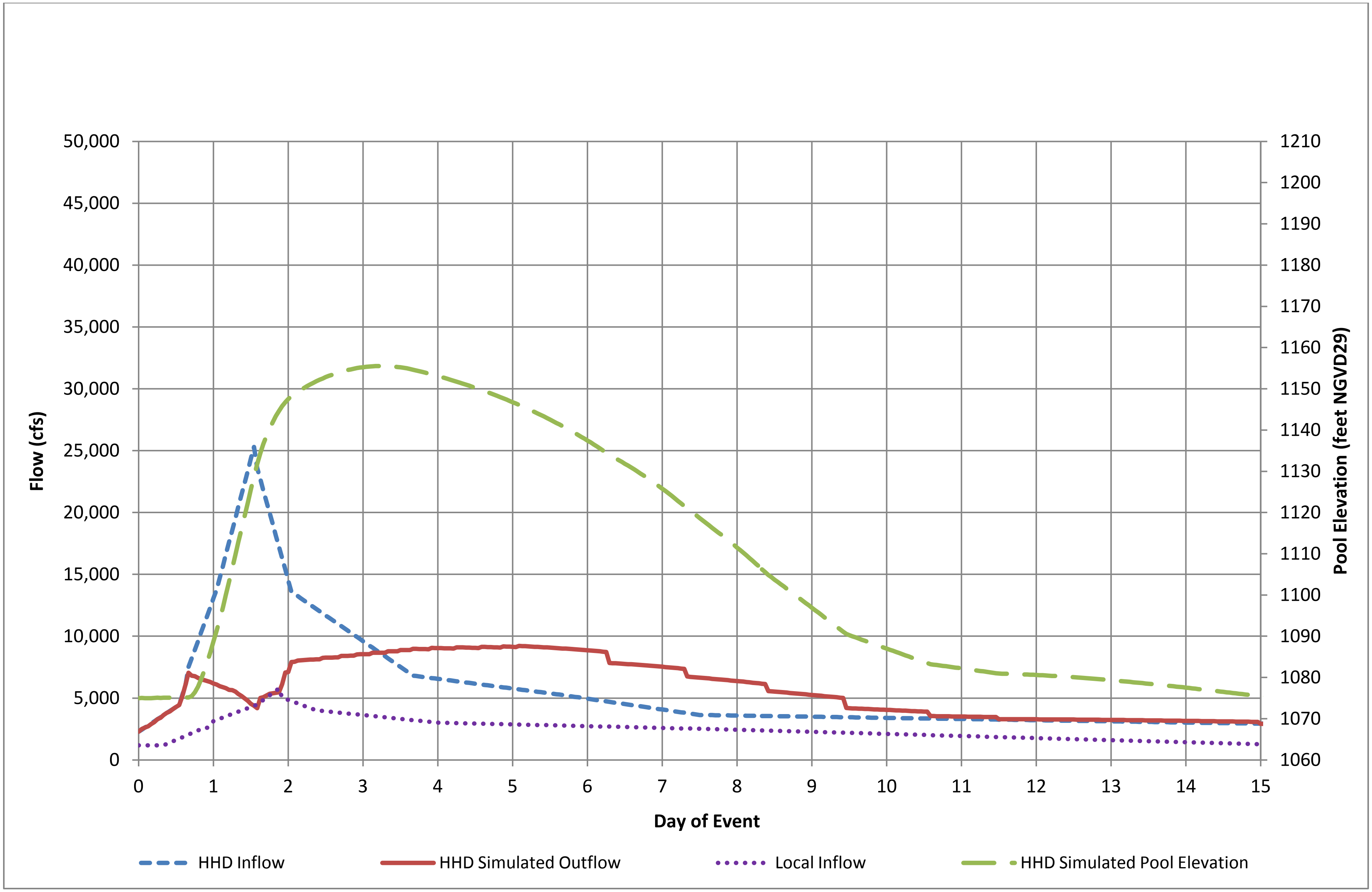

Figure 7 – Simulated Howard Hanson Dam Operations, 10 Percent Flood, Upper Confidence Limit (5%)

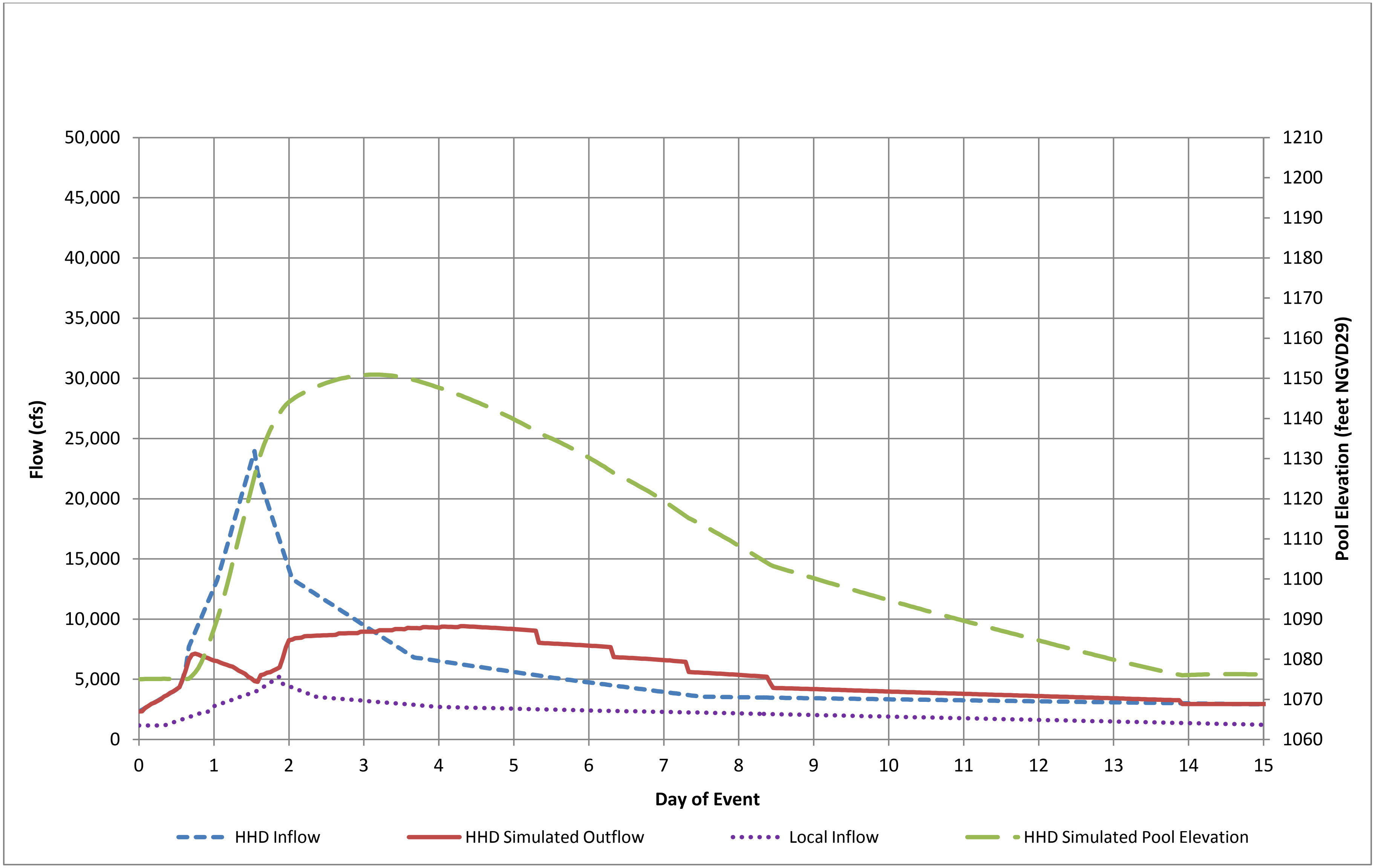

Figure 8 – Simulated Howard Hanson Dam Operations, 4 Percent Flood, Lower Confidence Limit (95%)

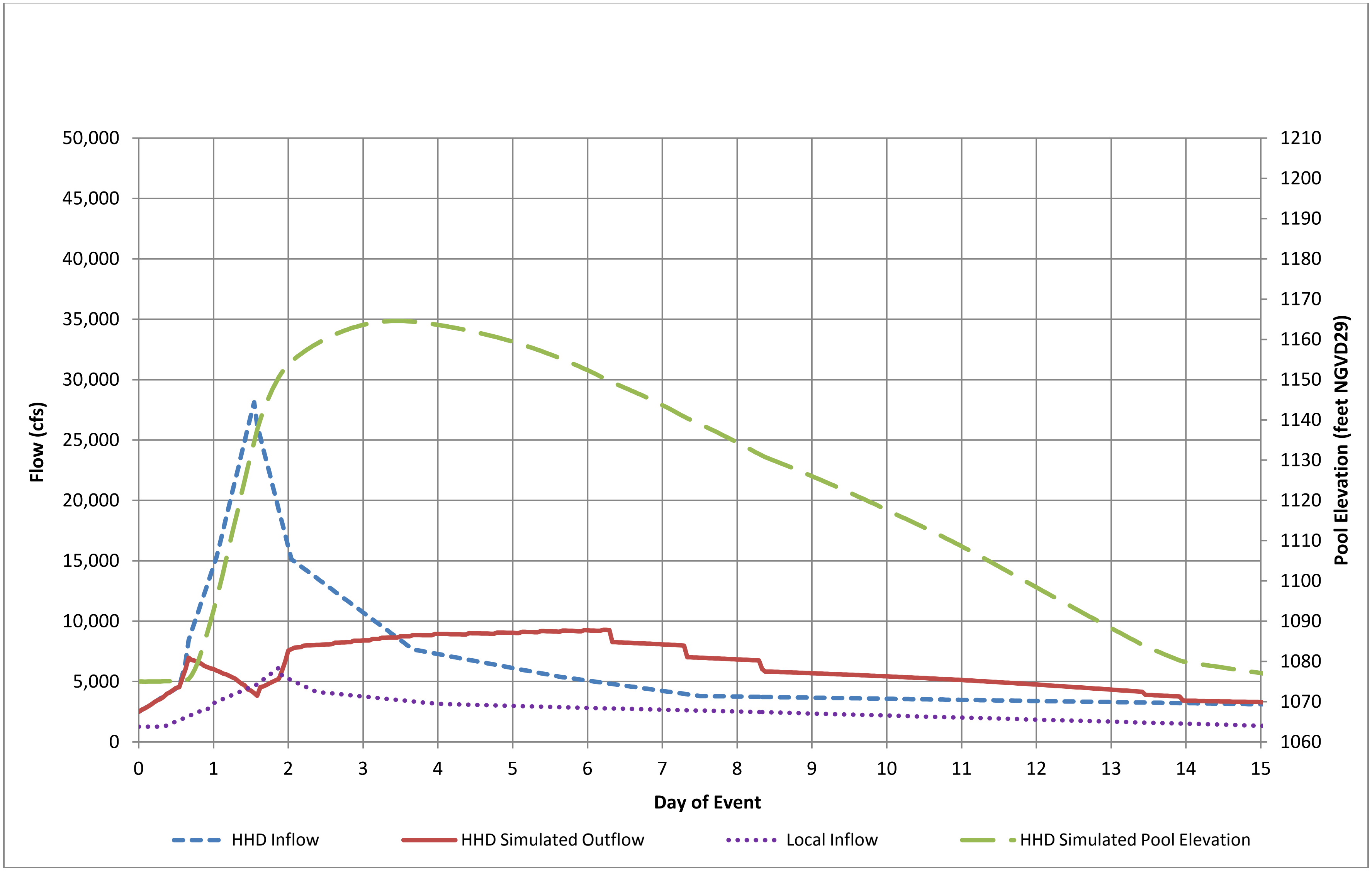

Figure 9 – Simulated Howard Hanson Dam Operations, 4 Percent Flood, Median Discharge Frequency Function

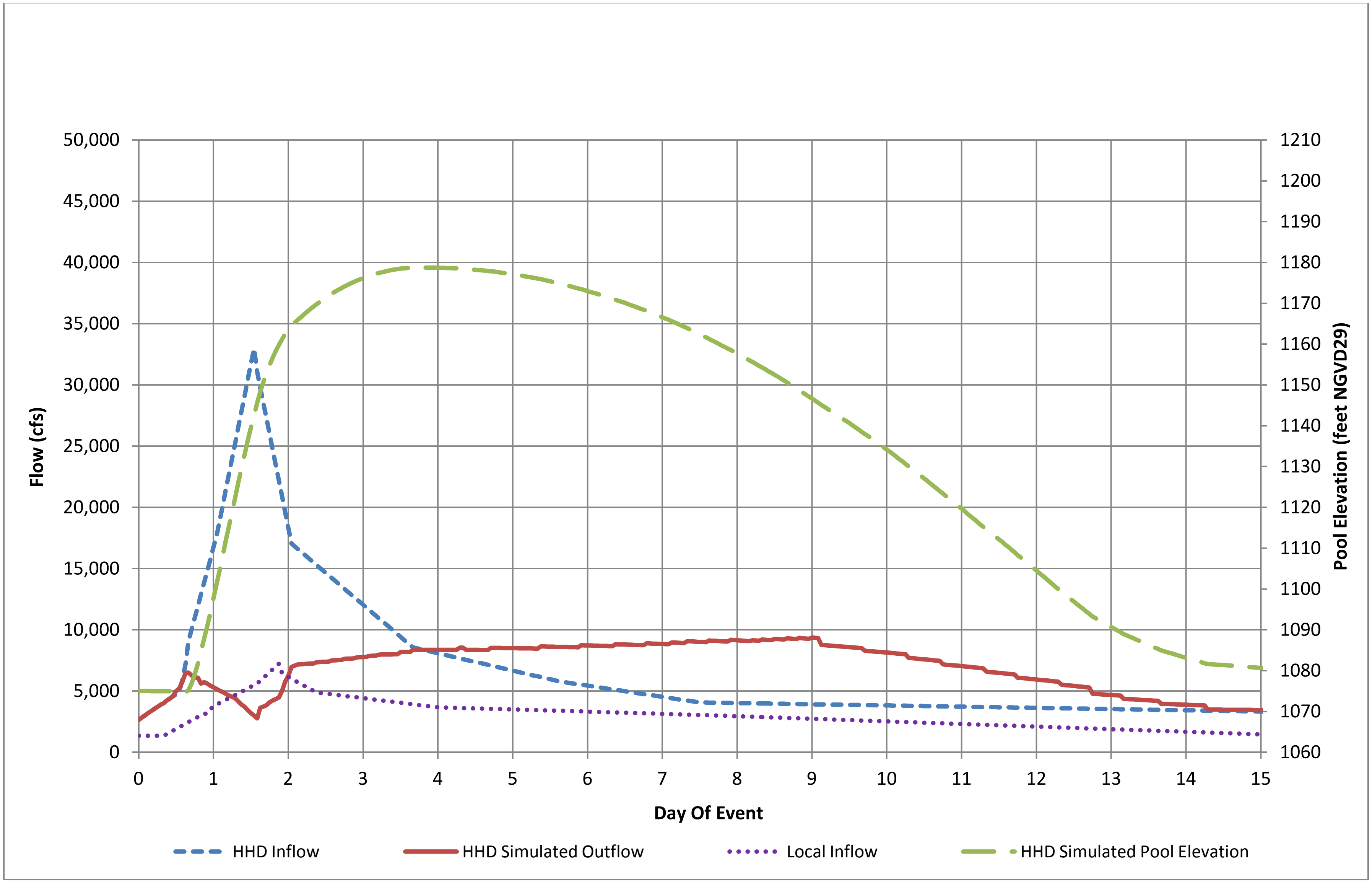

Figure 10 – Simulated Howard Hanson Dam Operations, 4 Percent Flood, Upper Confidence Limit (5%)

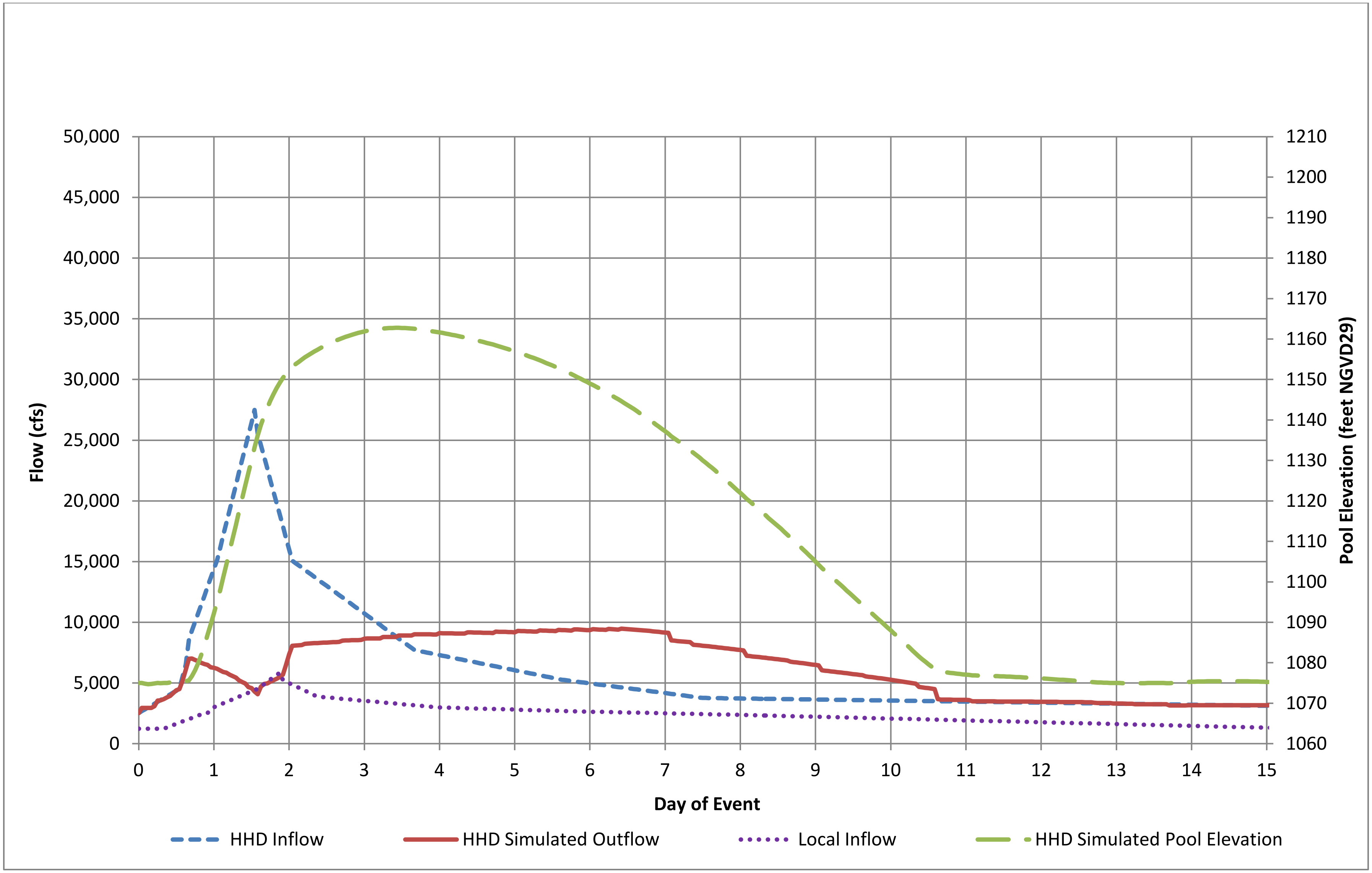

Figure 11 – Simulated Howard Hanson Dam Operations, 2 Percent Flood, Lower Confidence Limit (95%)

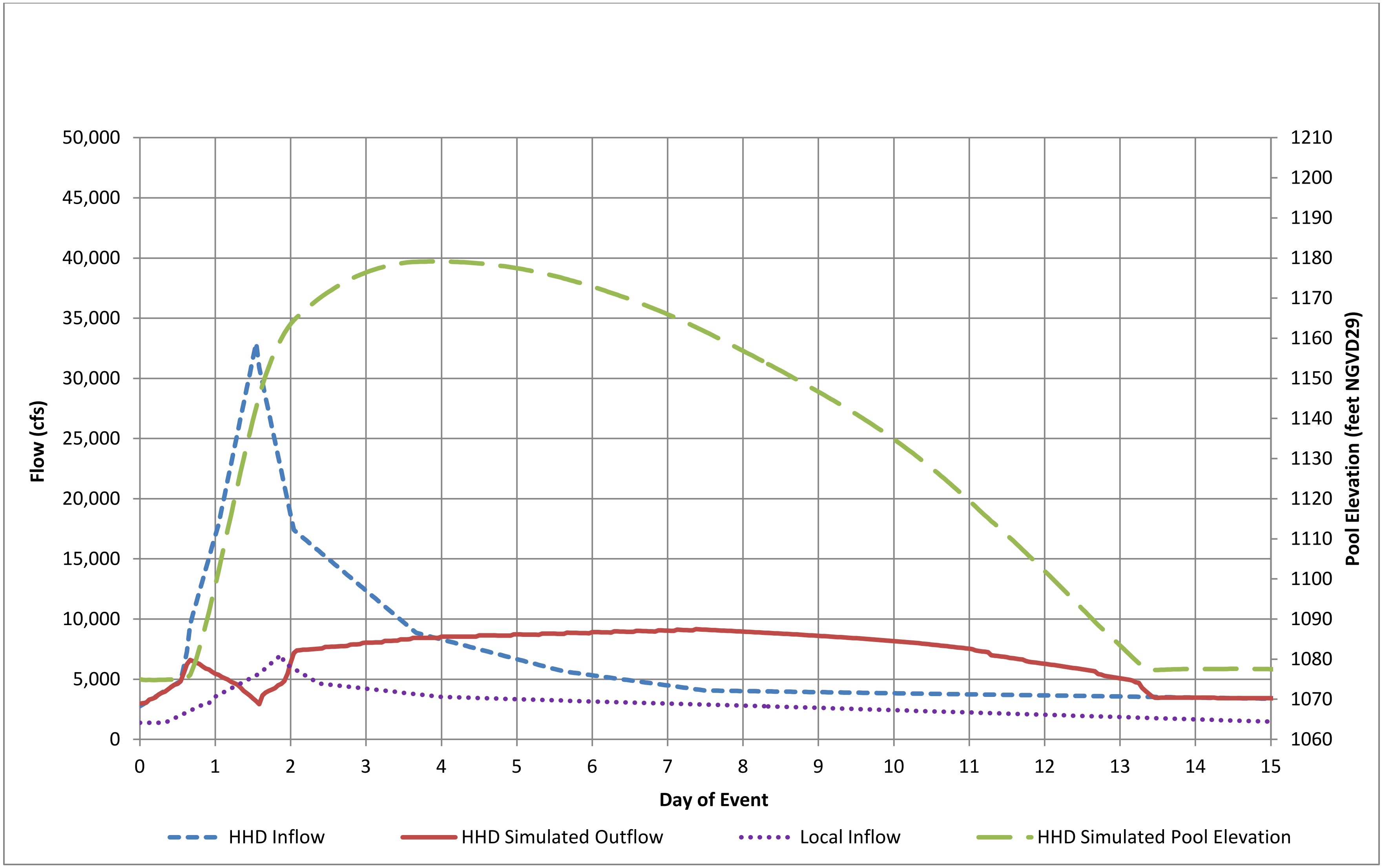

Figure 12 – Simulated Howard Hanson Dam Operations, 2 Percent Flood, Median Discharge Frequency Function

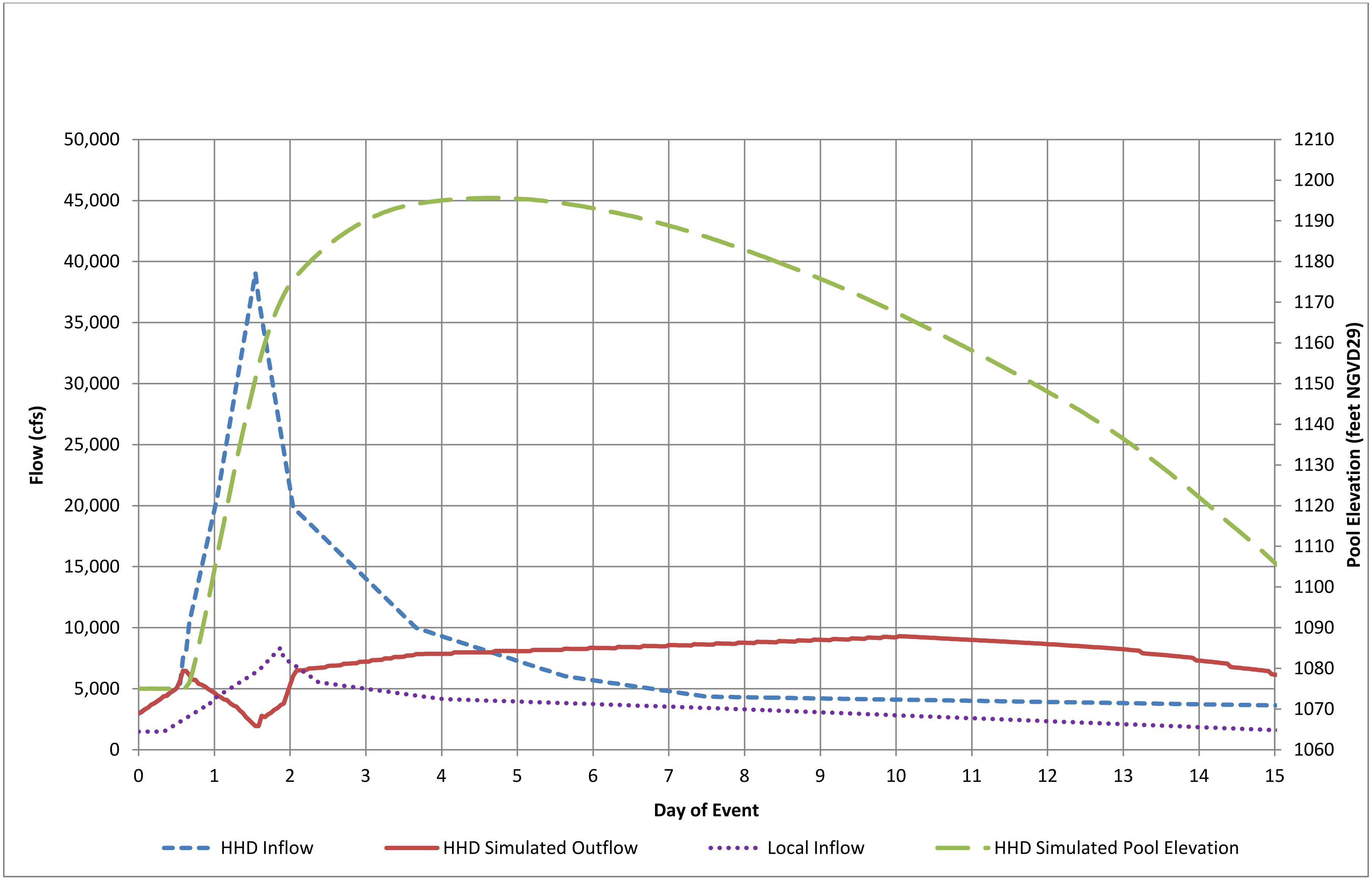

Figure 13 – Simulated Howard Hanson Dam Operations, 2 Percent Flood, Upper Confidence Limit (5%)

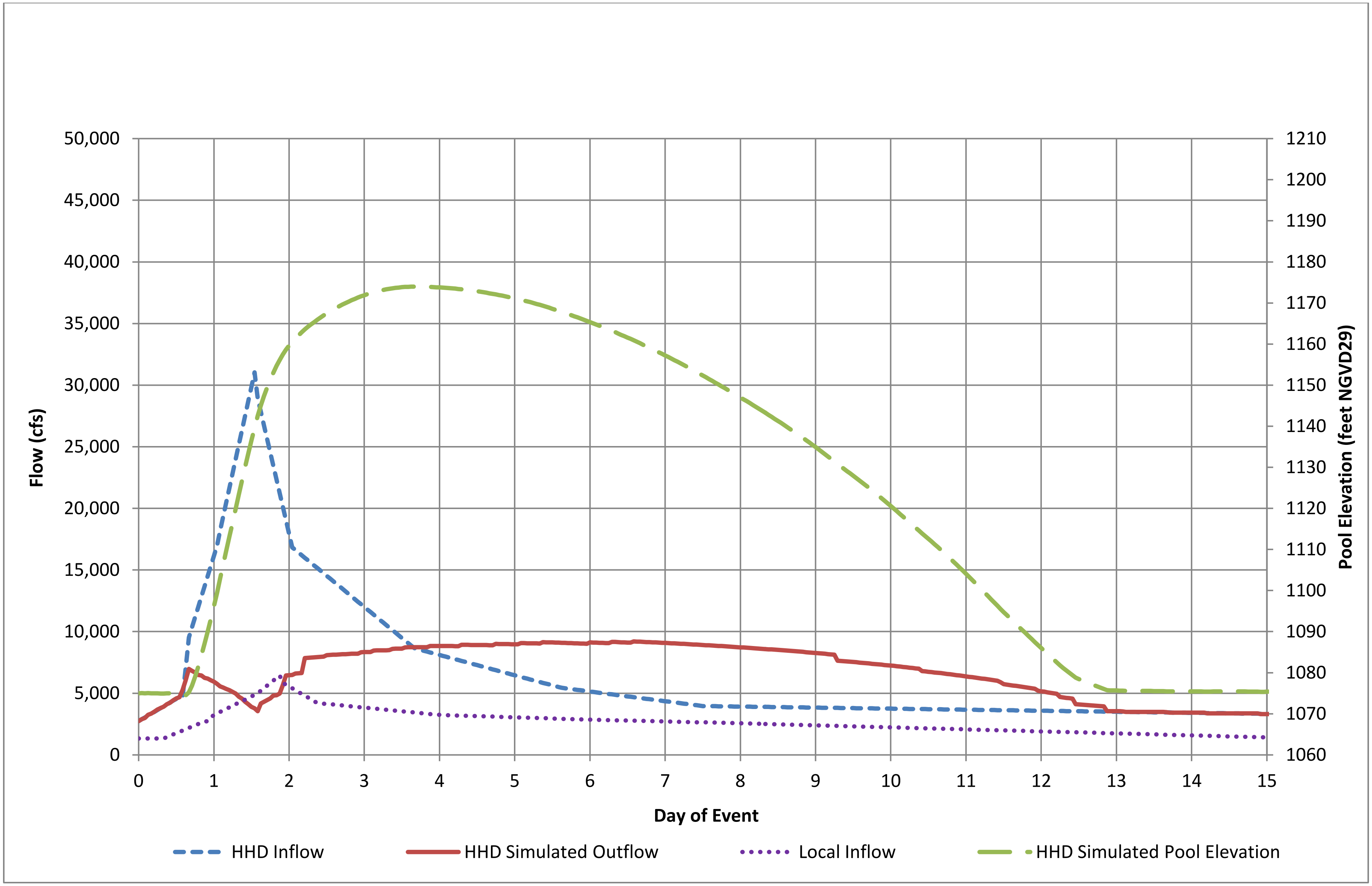

Figure 14 – Simulated Howard Hanson Dam Operations, 1 Percent Flood, Lower Confidence Limit (95%)

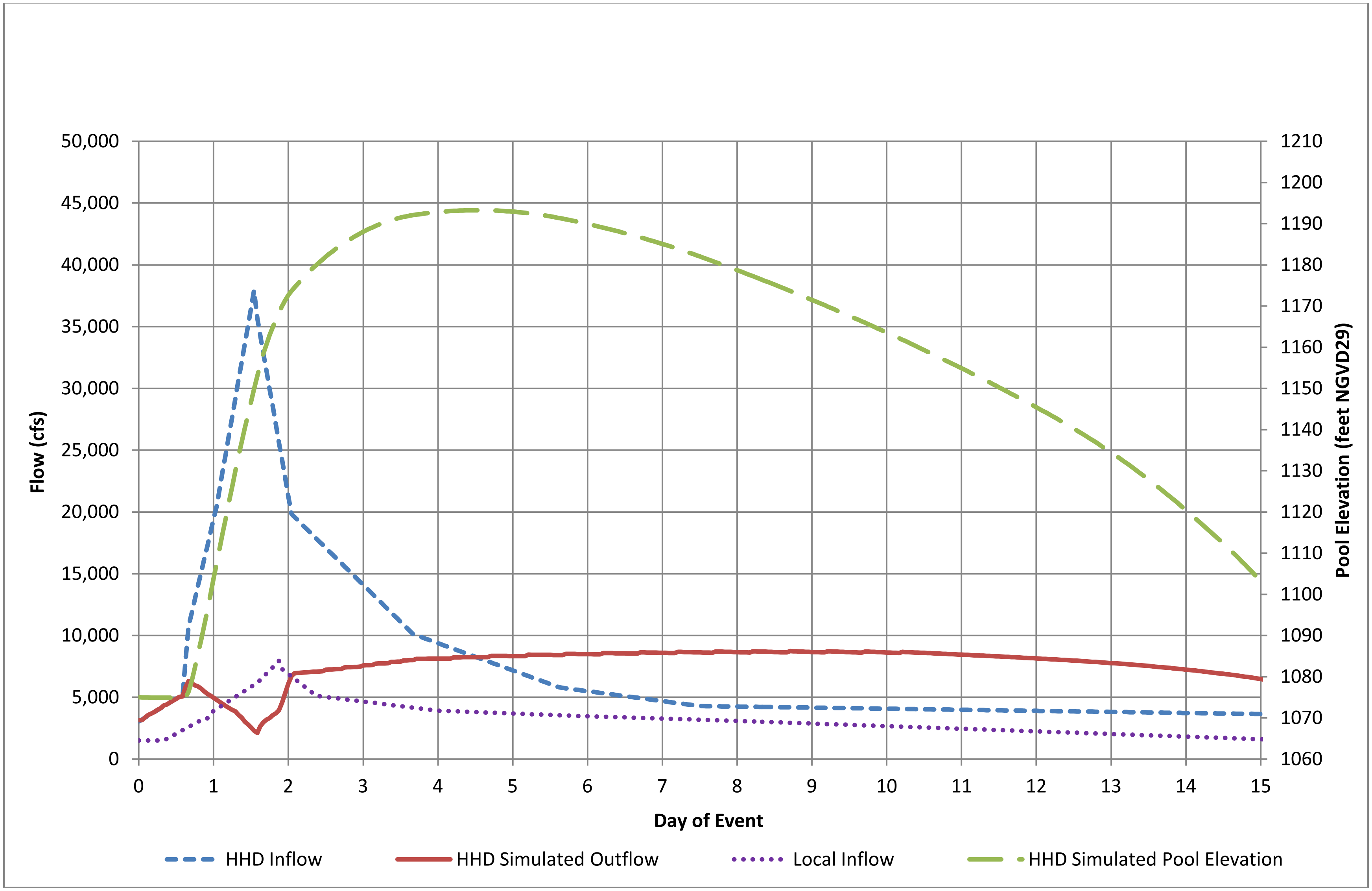

Figure 15 – Simulated Howard Hanson Dam Operations, 1 Percent Flood, Median Discharge Frequency Function

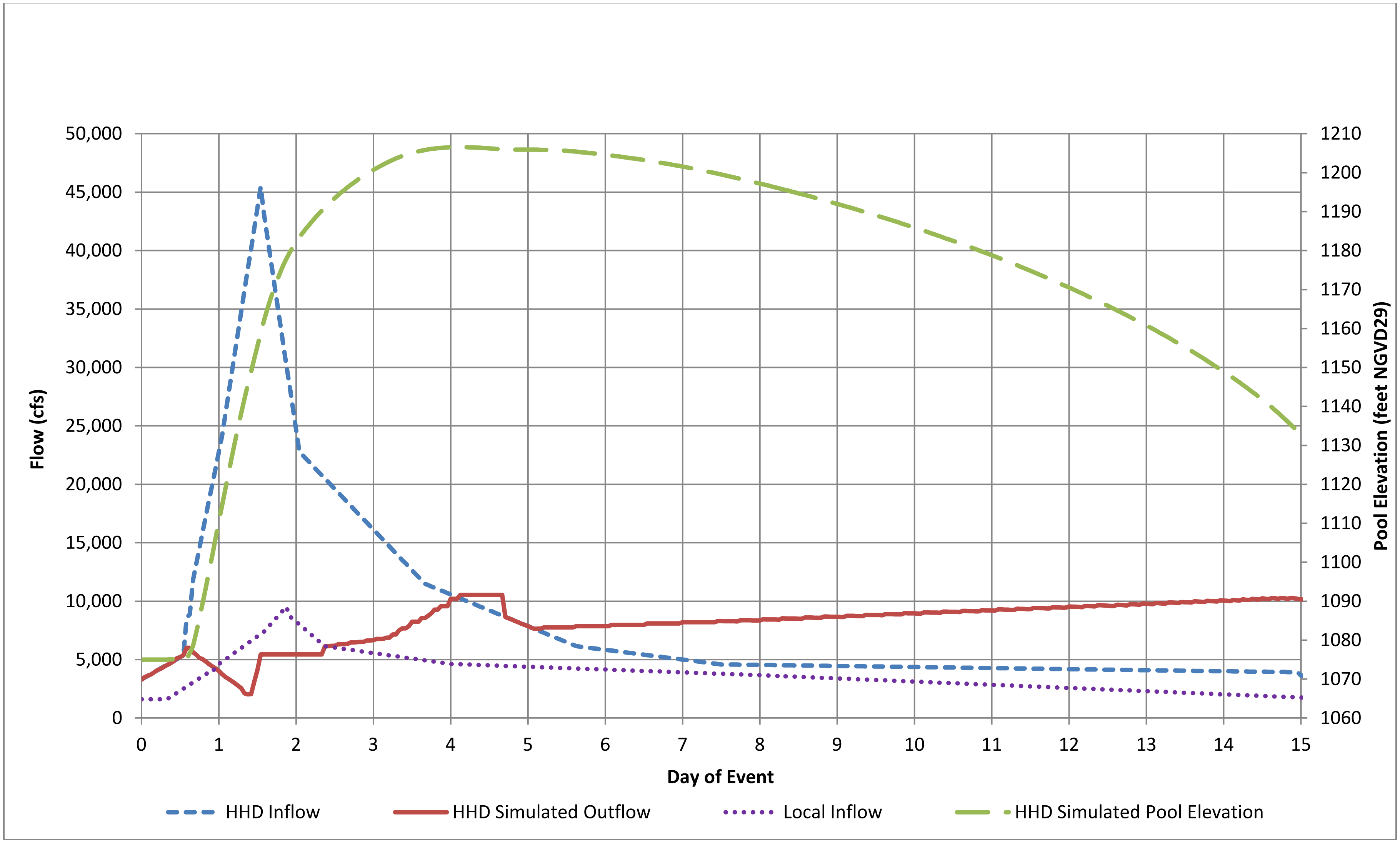

Figure 16 – Simulated Howard Hanson Dam Operations, 1 Percent Flood, Upper Confidence Limit (5%)

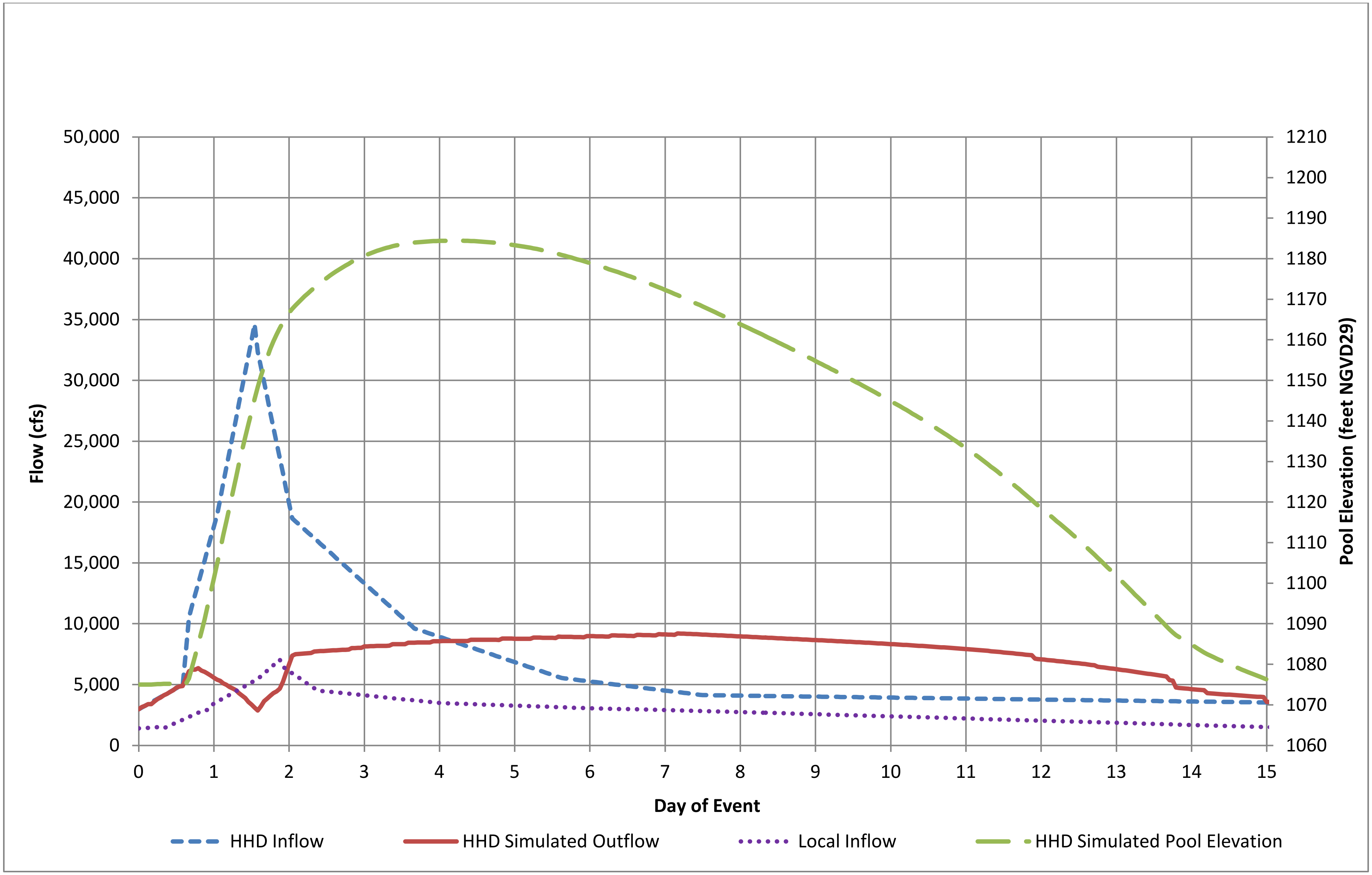

Figure 17 – Simulated Howard Hanson Dam Operations, 0.5 Percent Flood, Lower Confidence Limit (95%)

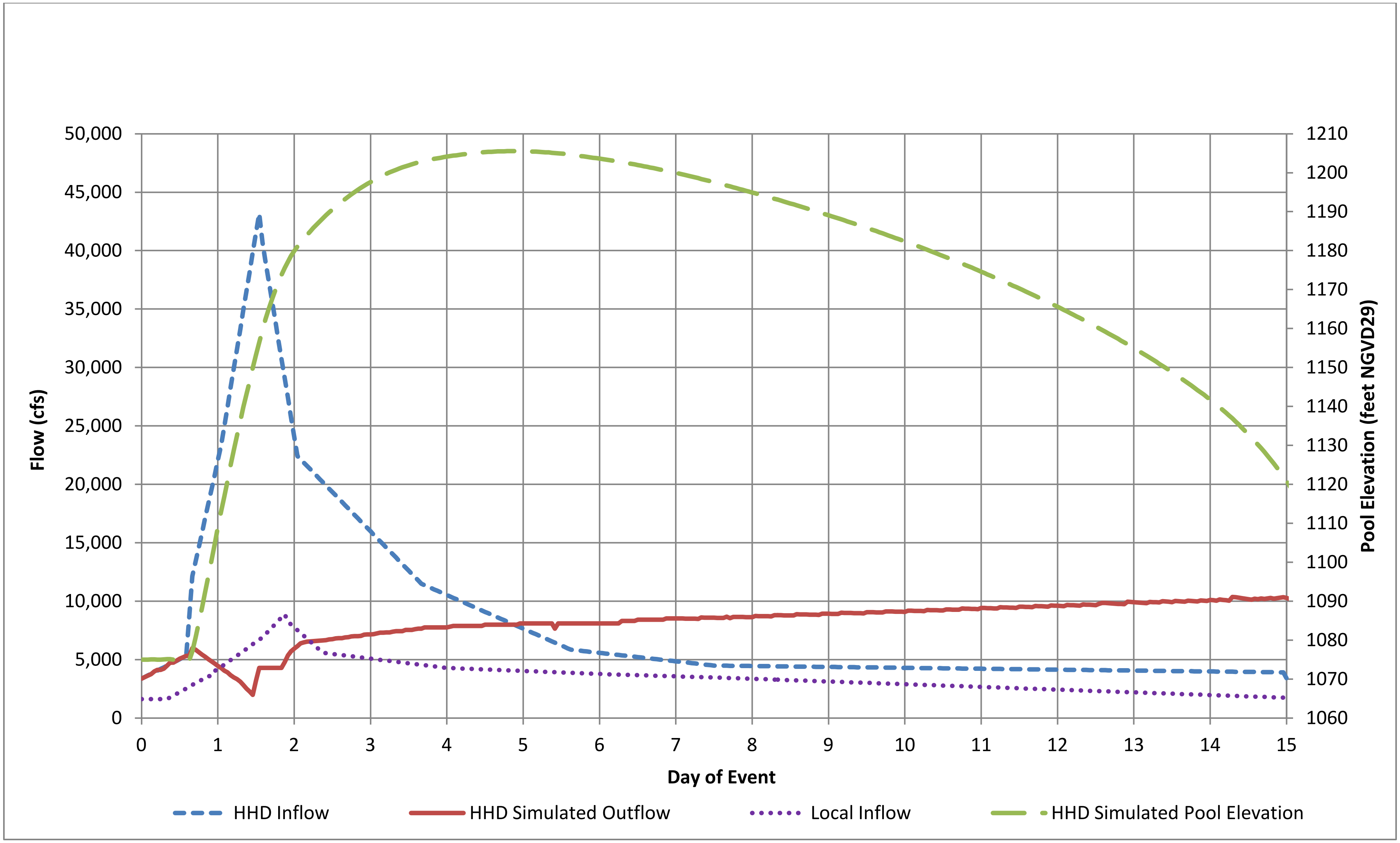

Figure 18 – Simulated Howard Hanson Dam Operations, 0.5 Percent Flood, Median Discharge Frequency Function

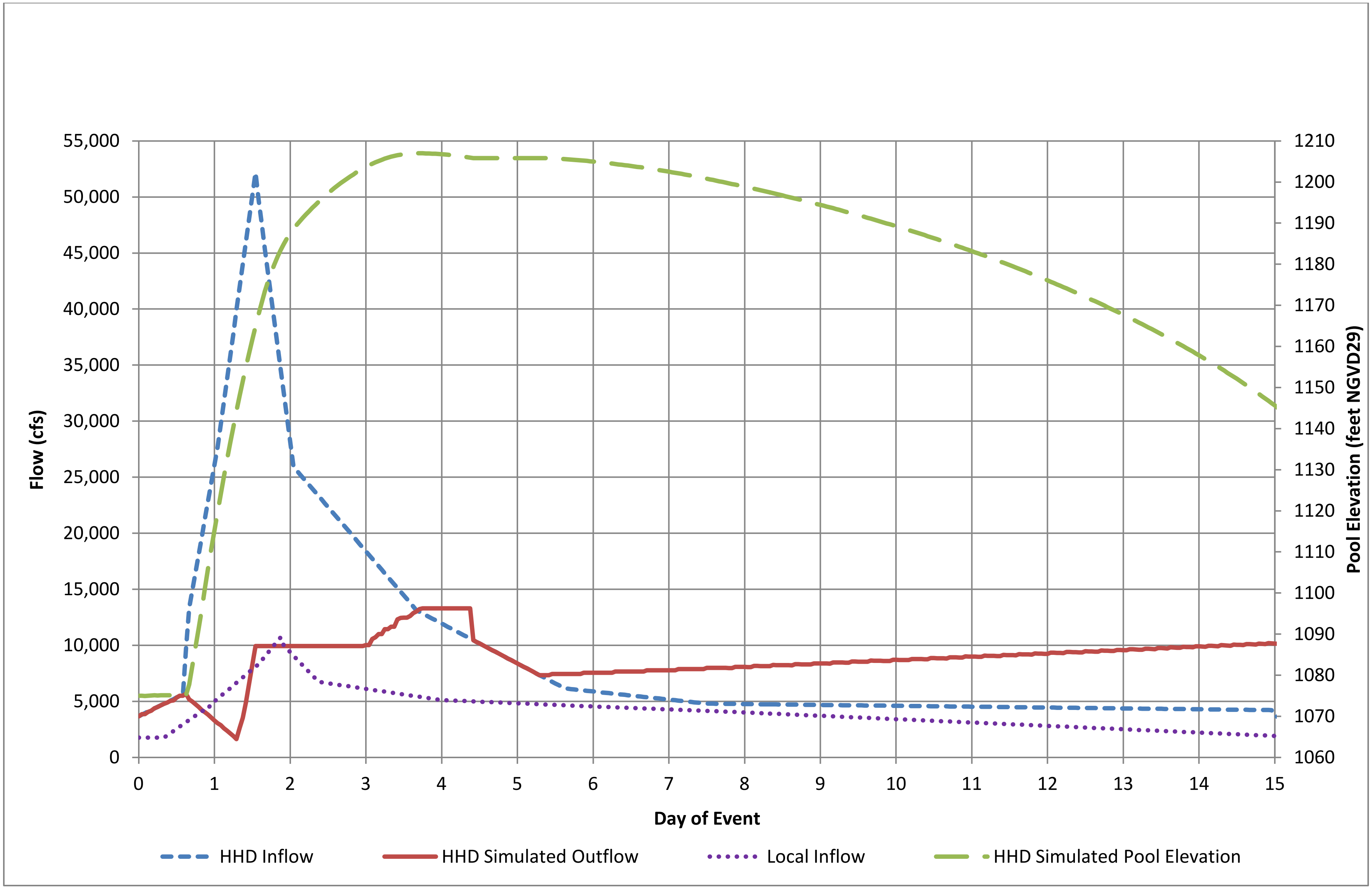

Figure 19 – Simulated Howard Hanson Dam Operations, 0.5 Percent Flood, Upper Confidence Limit (5%)

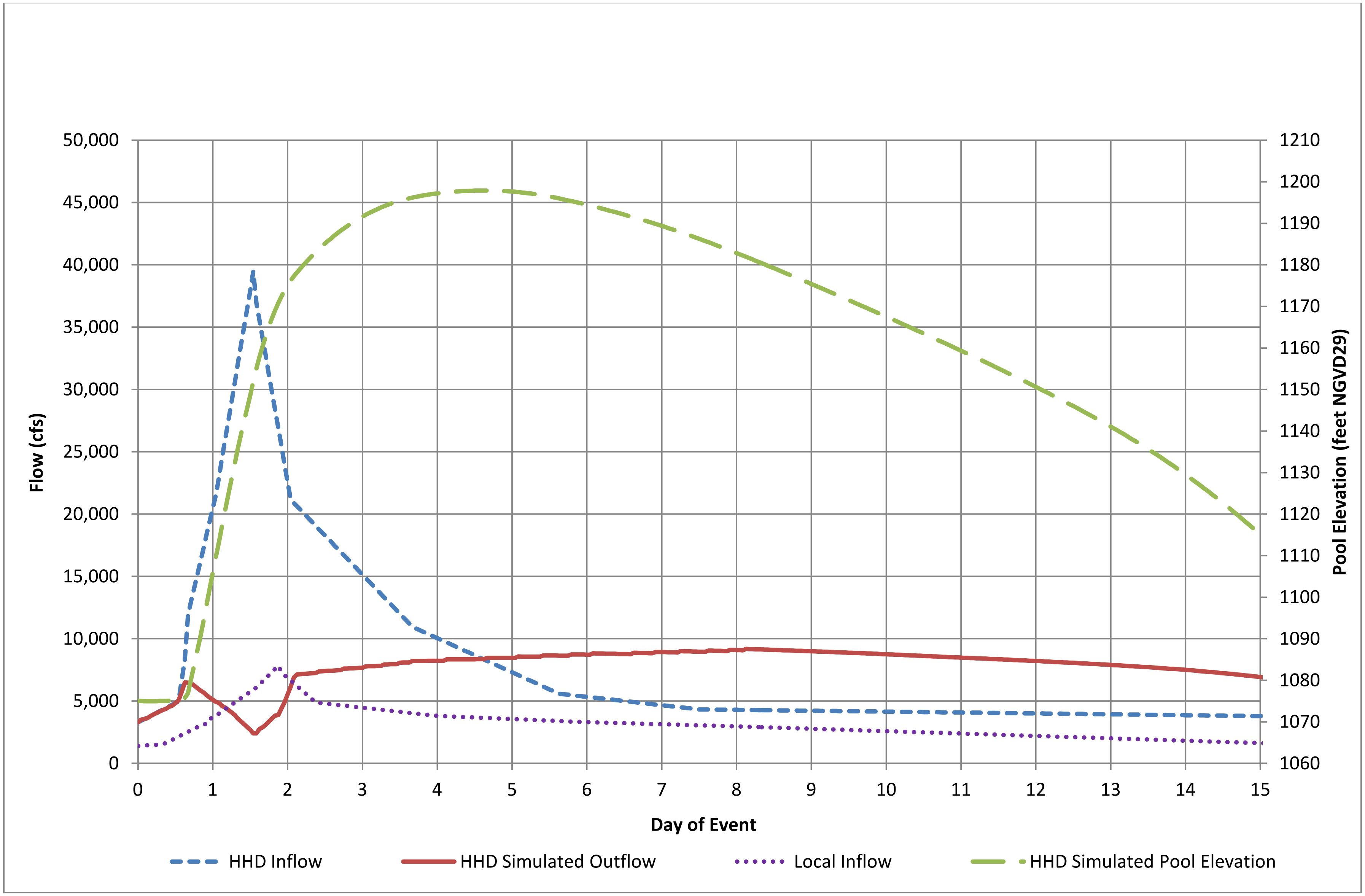

Figure 20 - Simulated Howard Hanson Dam Operations, 0.2 Percent Flood, Lower Confidence Limit (95%)

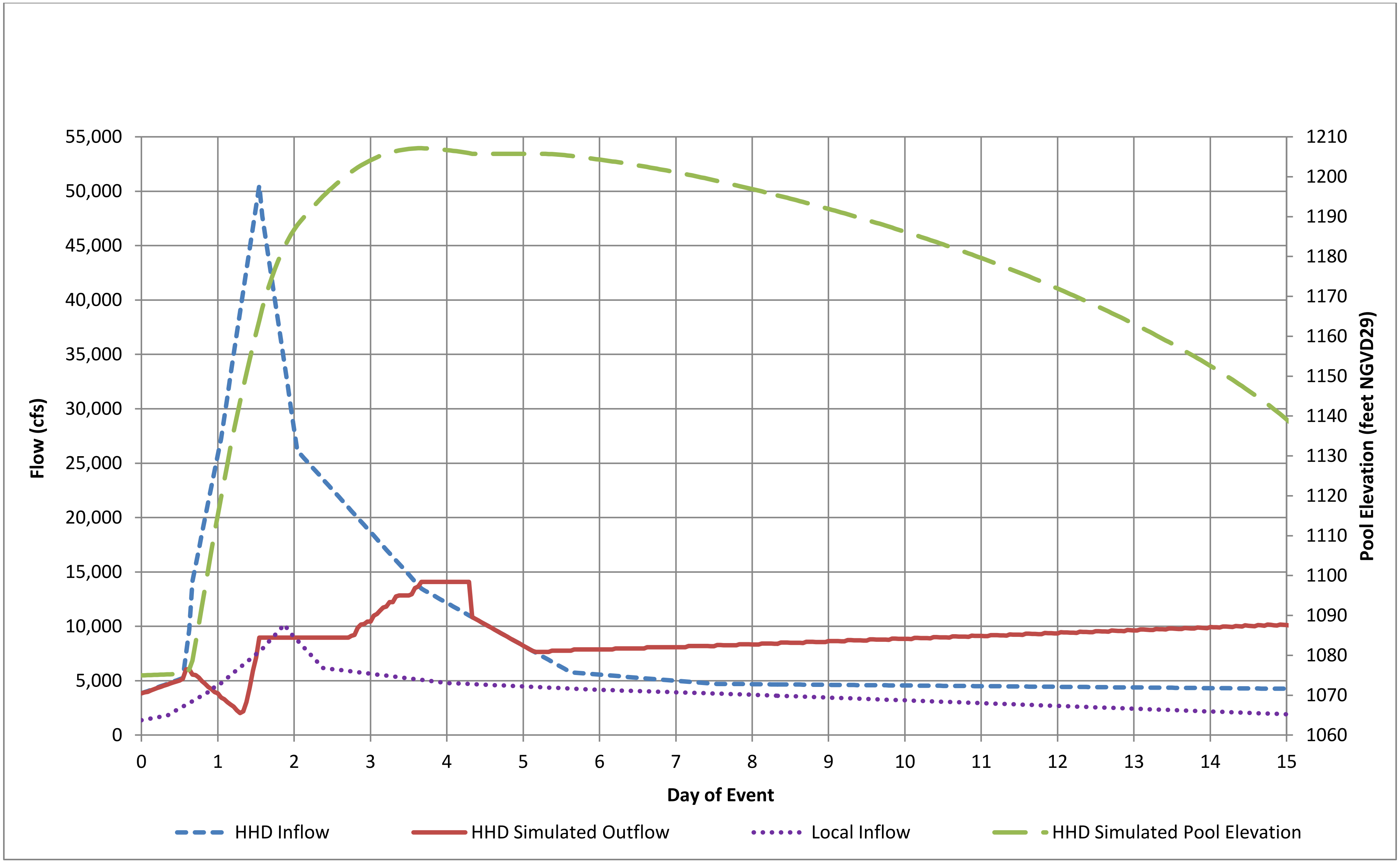

Figure 21 – Simulated Howard Hanson Dam Operations, 0.2 Percent Flood, Median Discharge Frequency Function

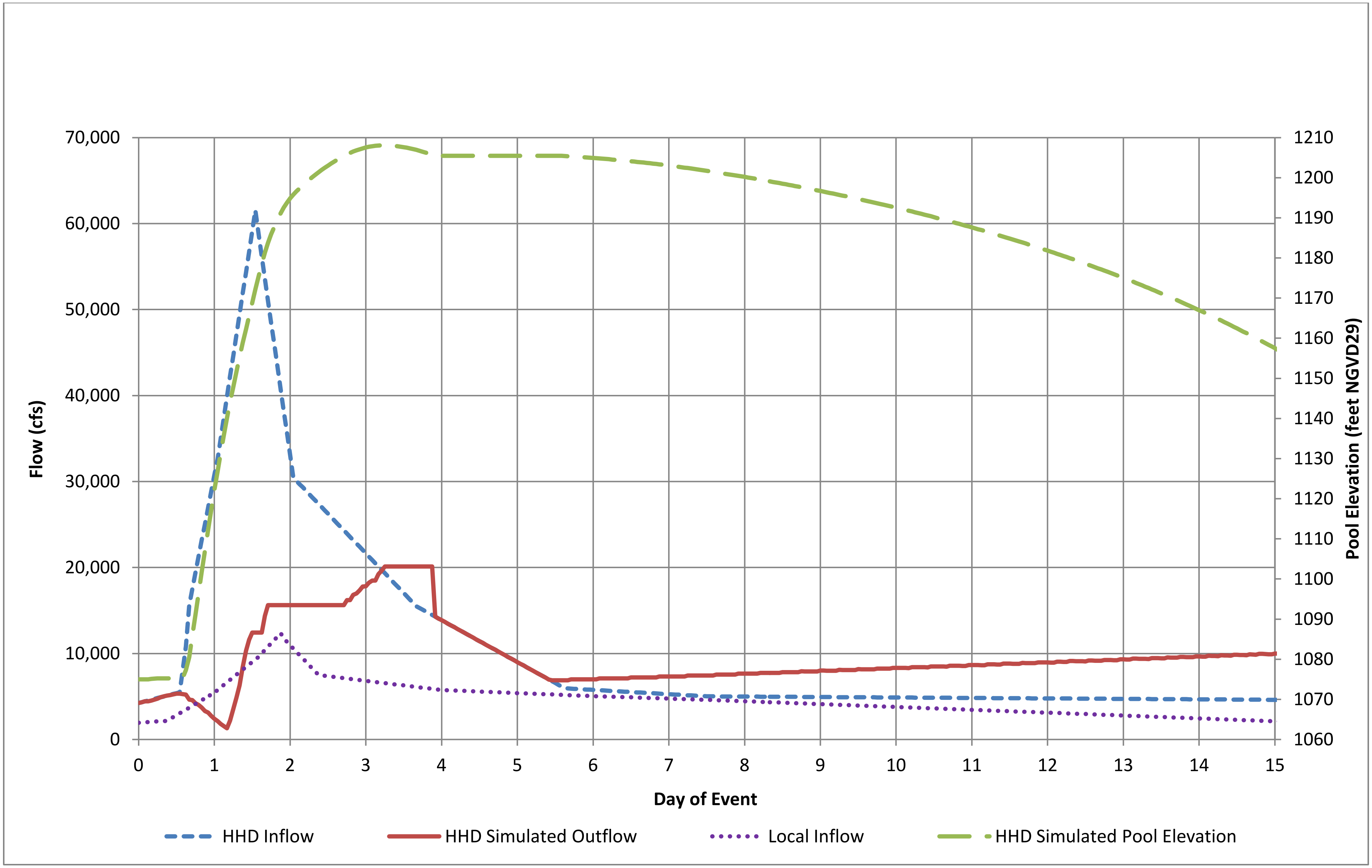

Figure 22 – Simulated Howard Hanson Dam Operations, 0.2 Percent Flood, Upper Confidence Limit (5%)

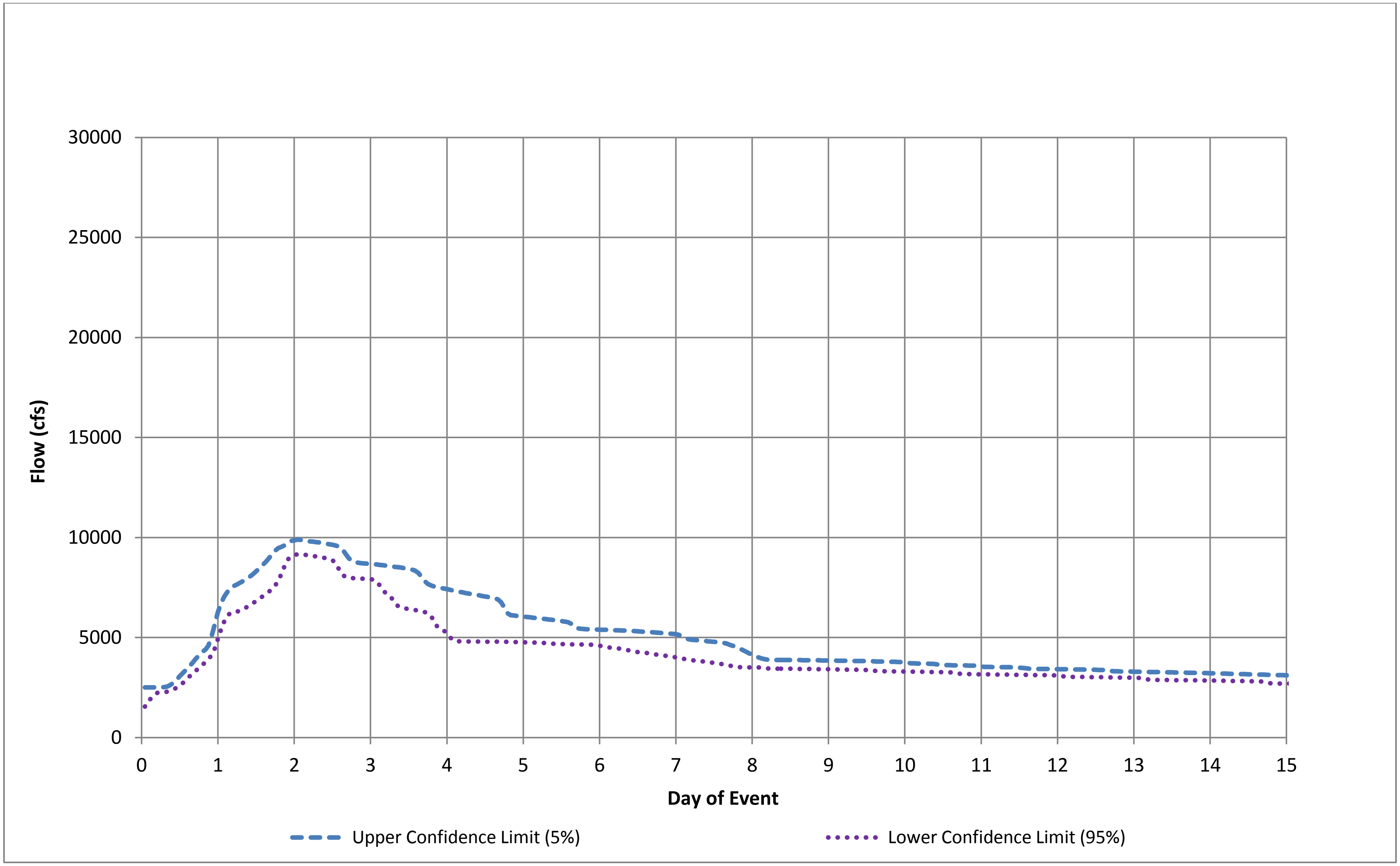

Figure 23 – Green River at Auburn Simulated Discharge, 50 Percent Flood

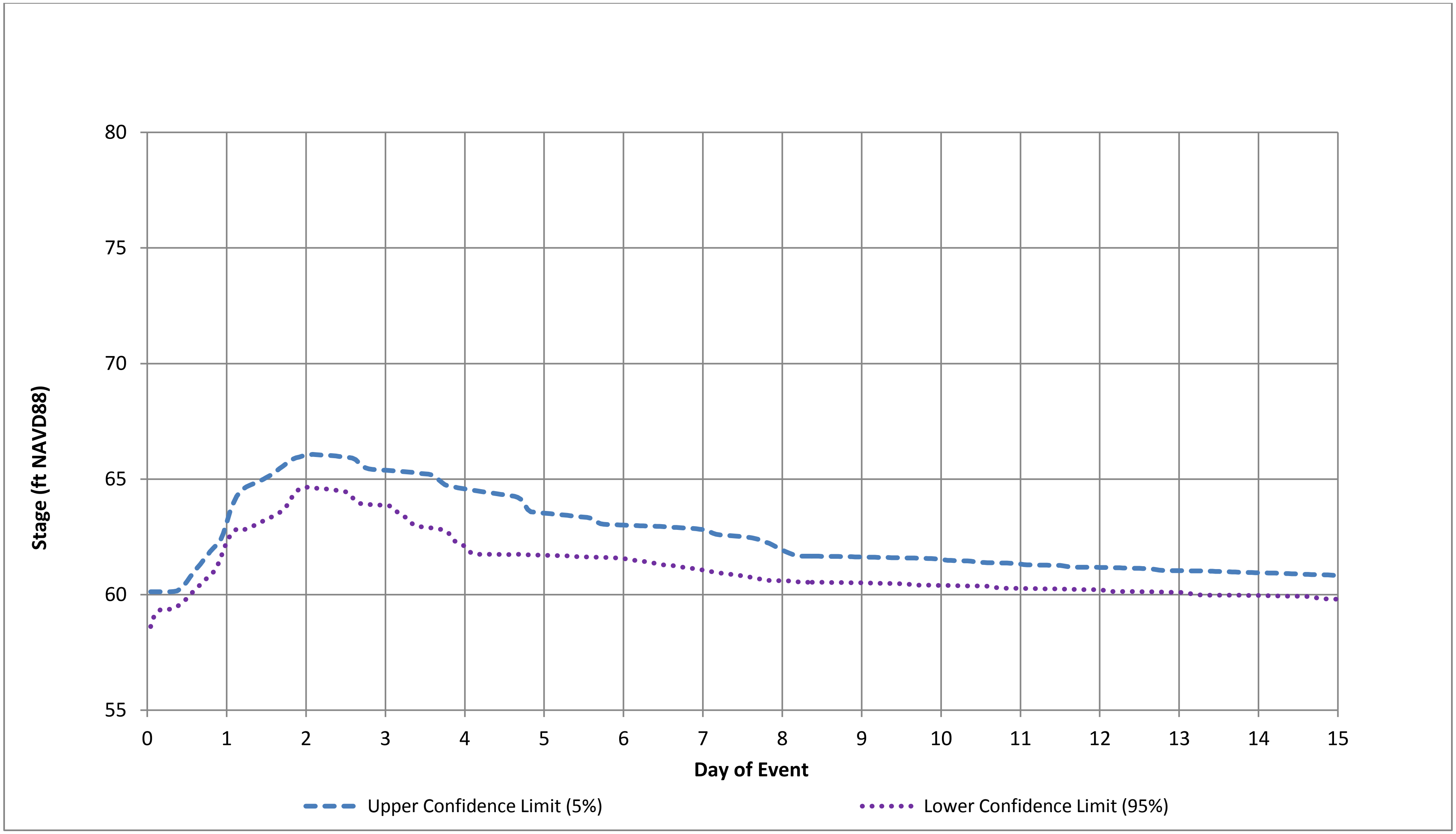

Figure 24 – Green River at Auburn Simulated Stage, 50 Percent Flood

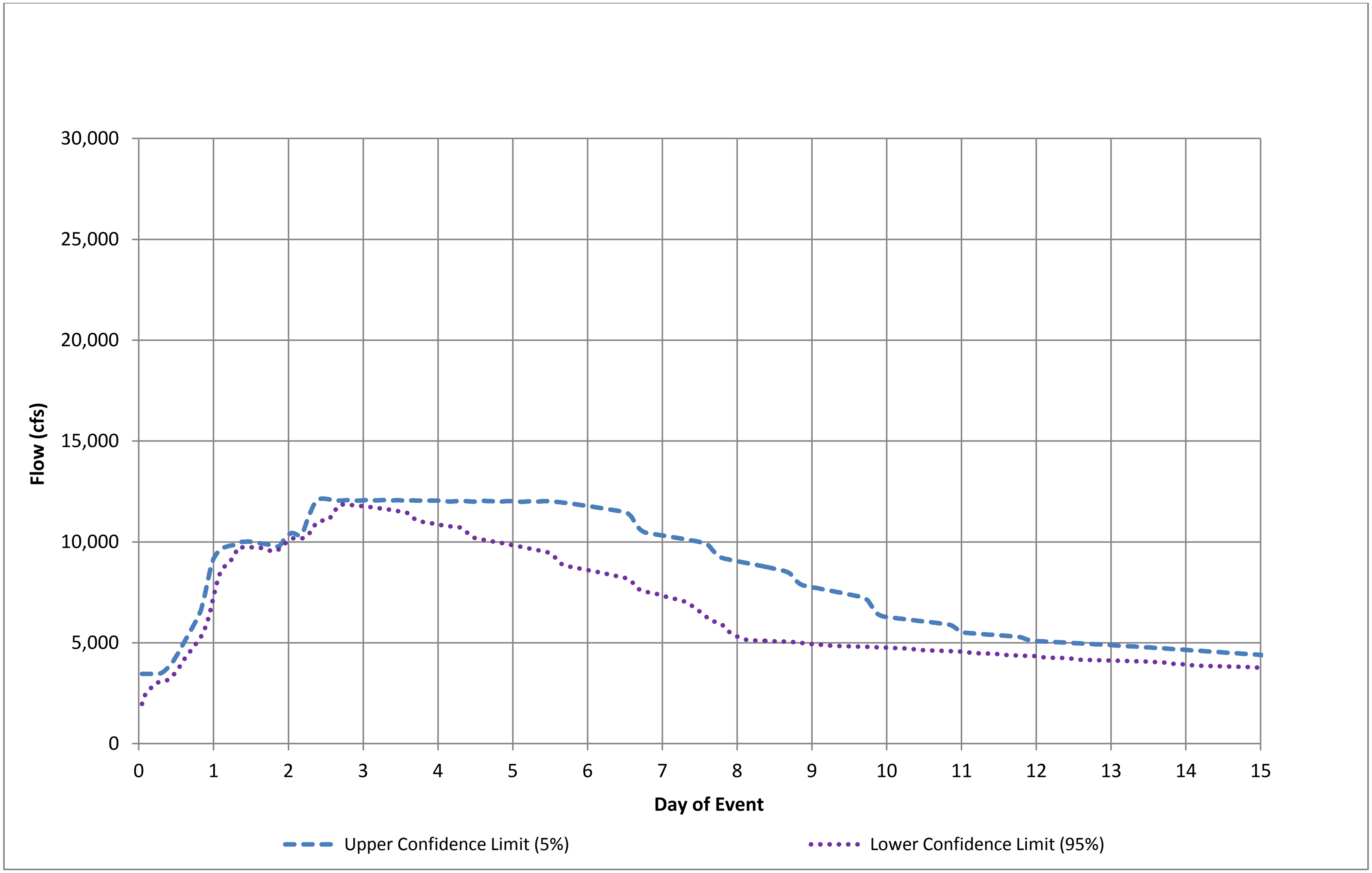

Figure 25 – Green River at Auburn Simulated Discharge, 10 Percent Flood

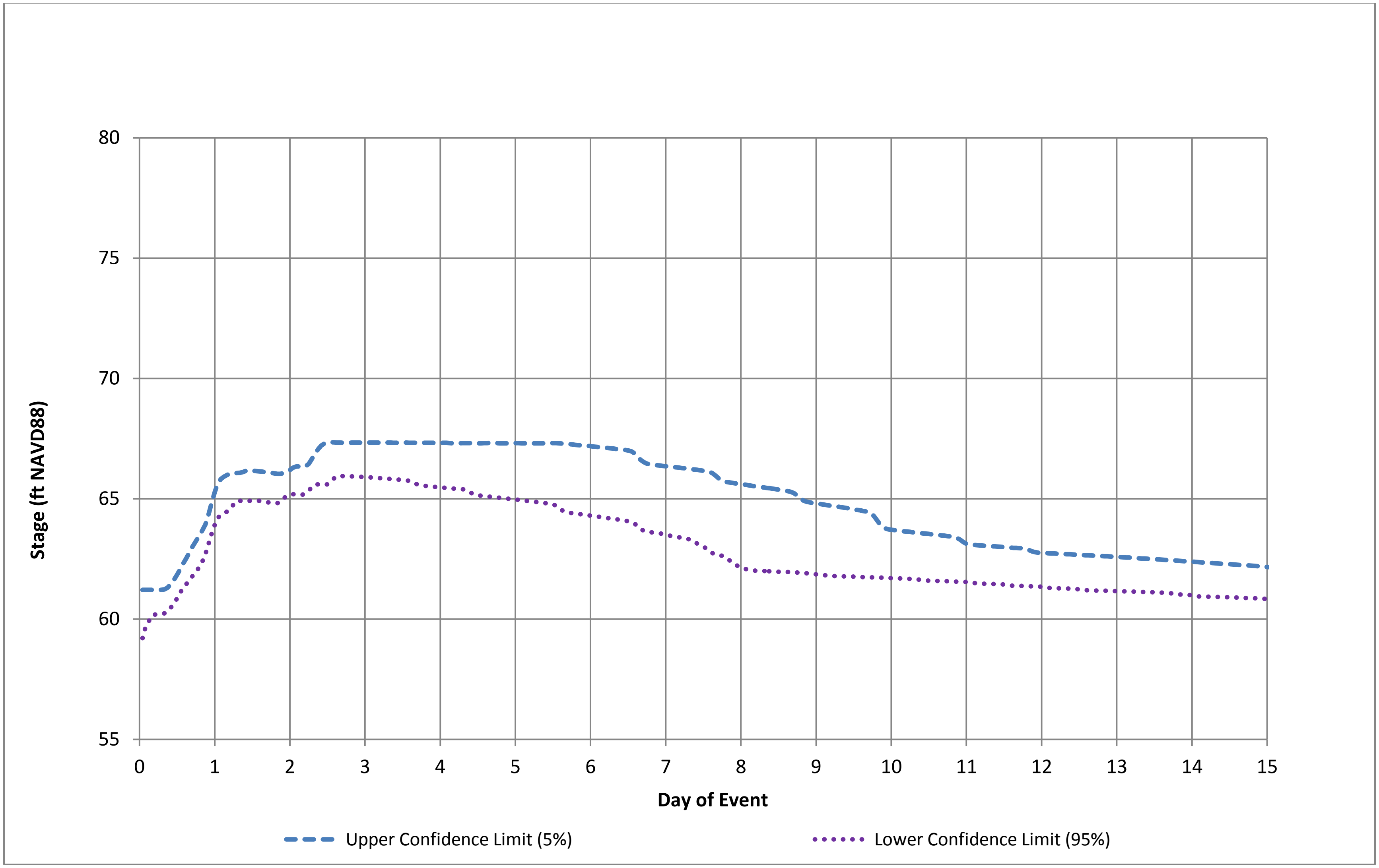

Figure 26 – Green River at Auburn Simulated Stage, 10 Percent Flood

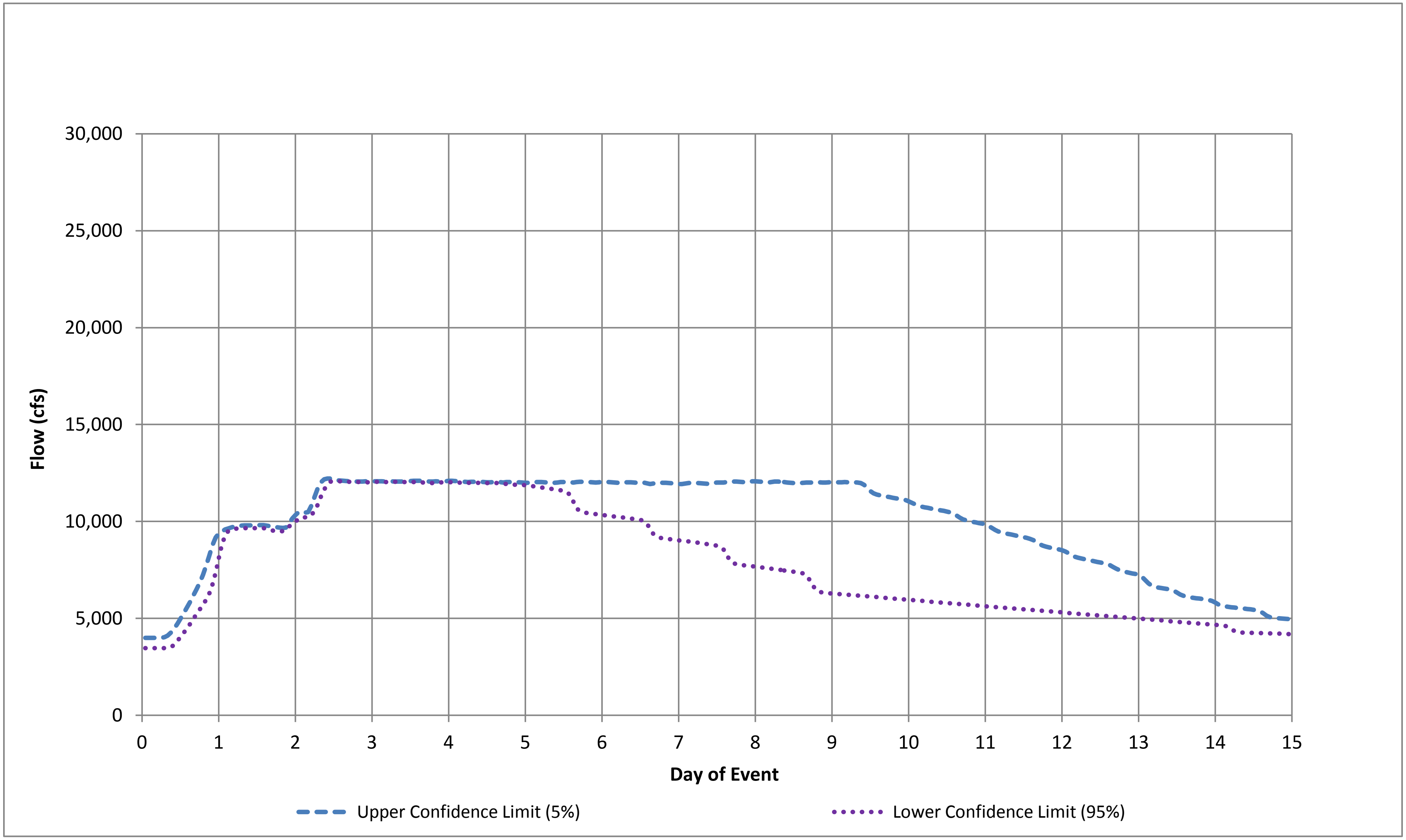

Figure 27 – Green River at Auburn Simulated Discharge, 4 Percent Flood

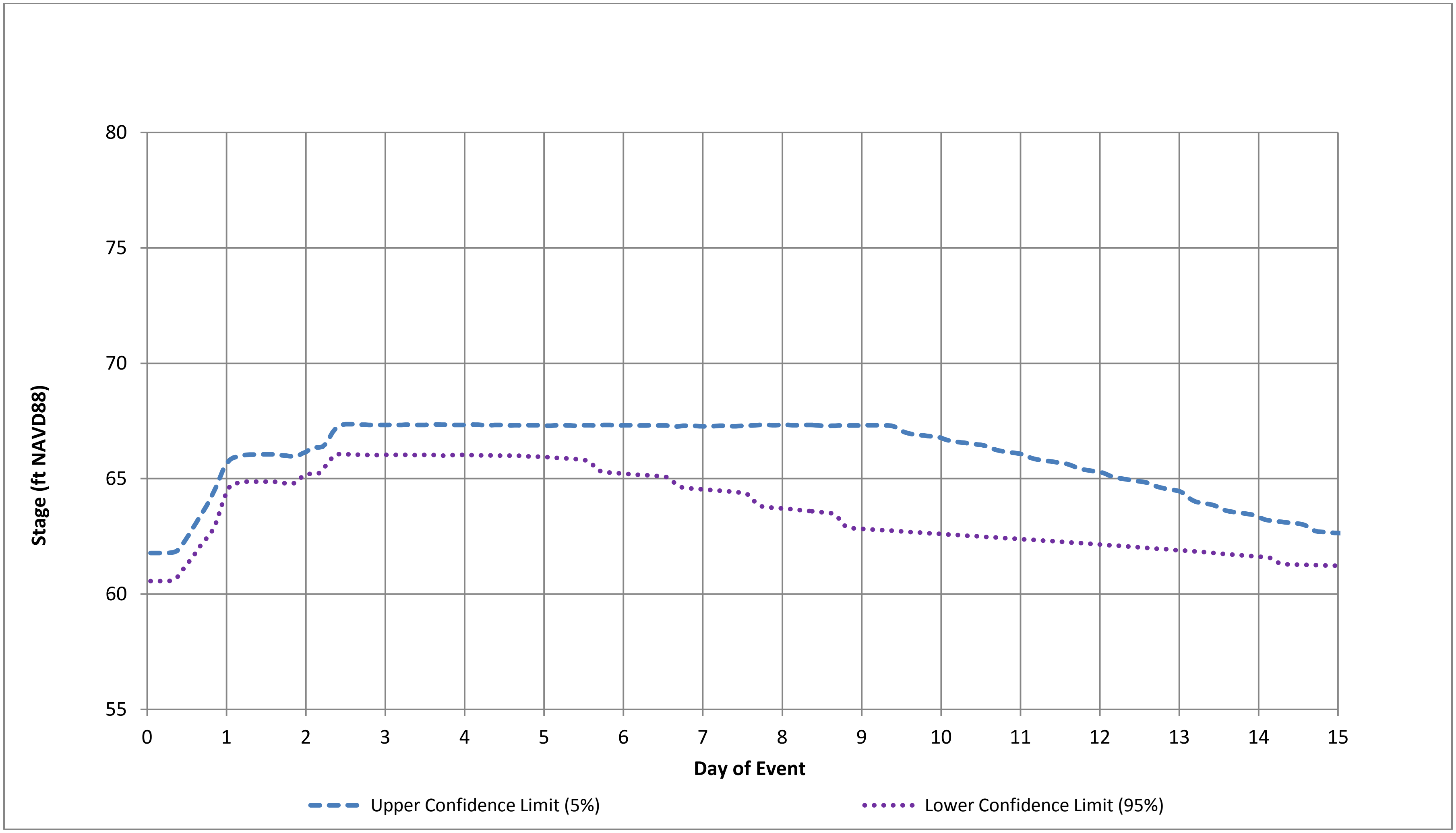

Figure 28 – Green River at Auburn Simulated Stage, 4 Percent Flood

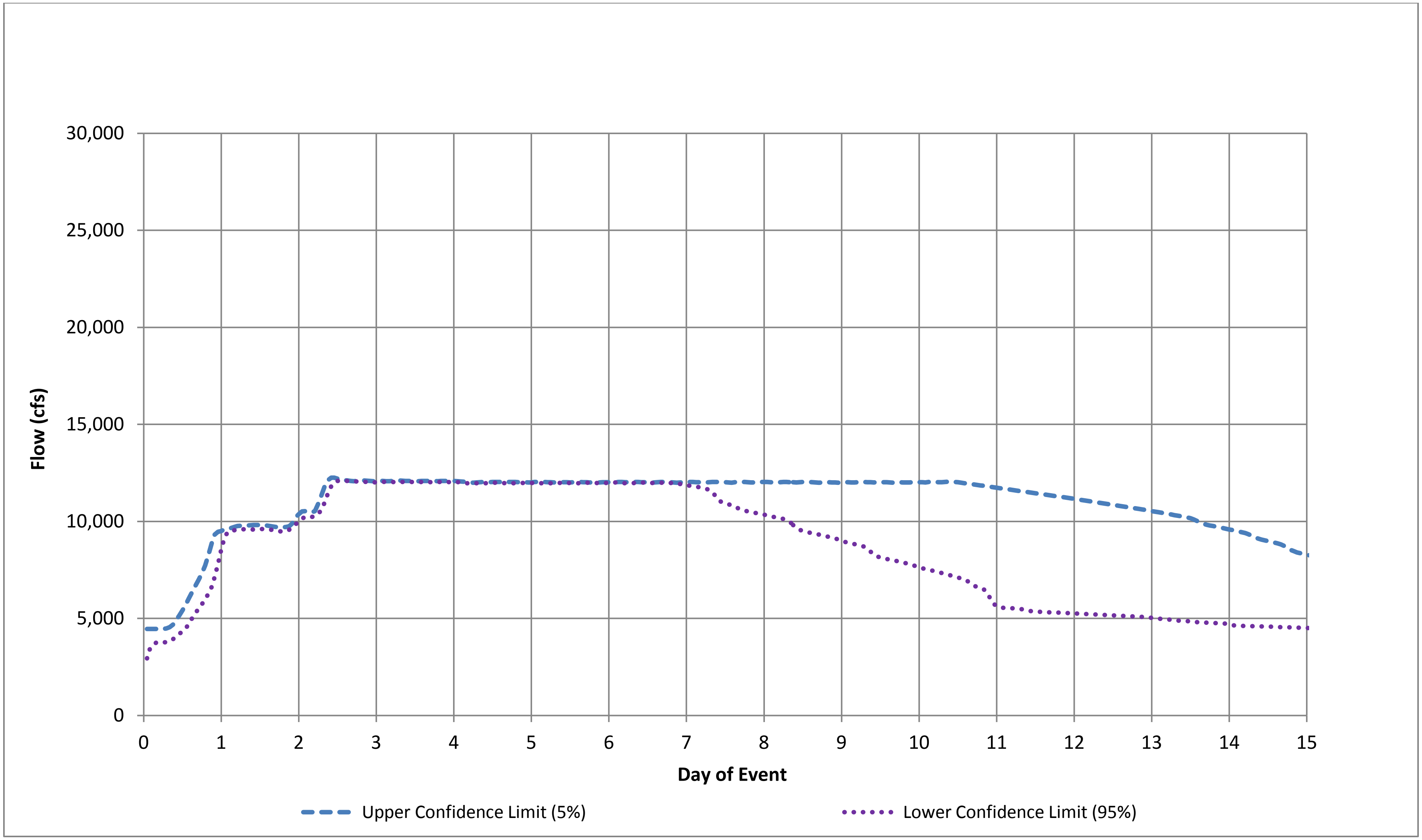

Figure 29 – Green River at Auburn Simulated Discharge, 2 Percent Flood

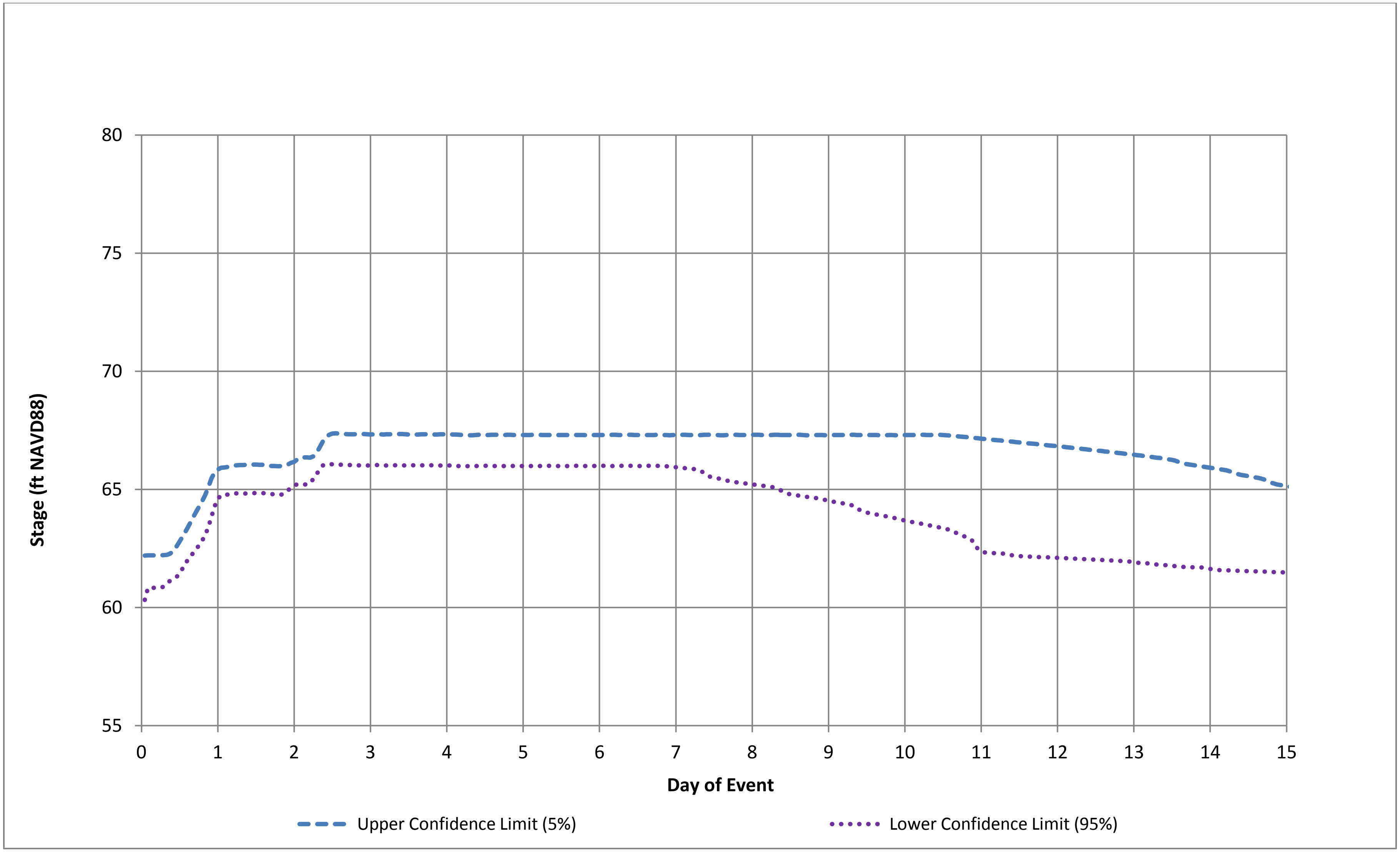

Figure 30 – Green River at Auburn Simulated Stage, 2 Percent Flood

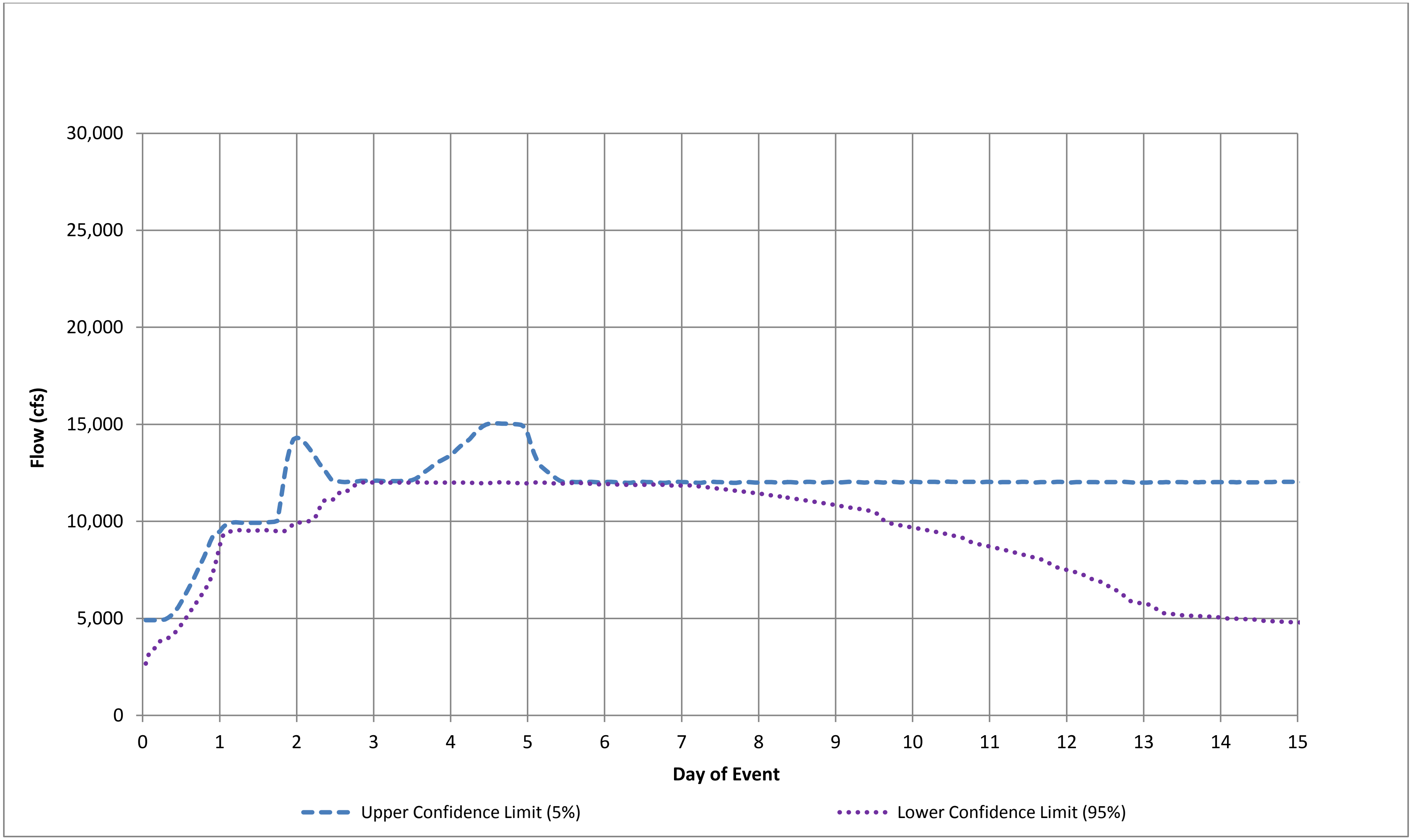

Figure 31 – Green River at Auburn Simulated Discharge, 1 Percent Flood

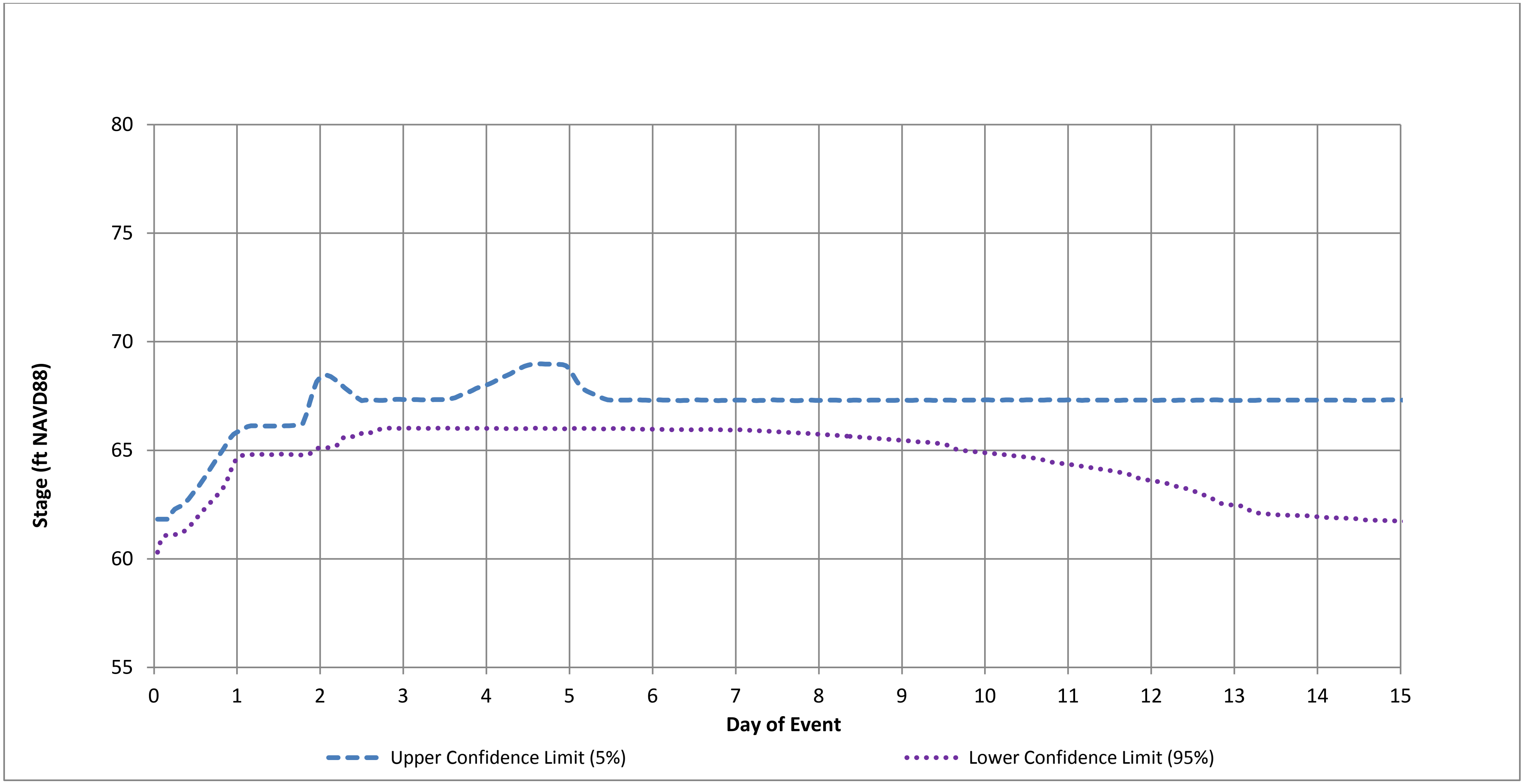

Figure 32 – Green River at Auburn Simulated Stage, 1 Percent Flood

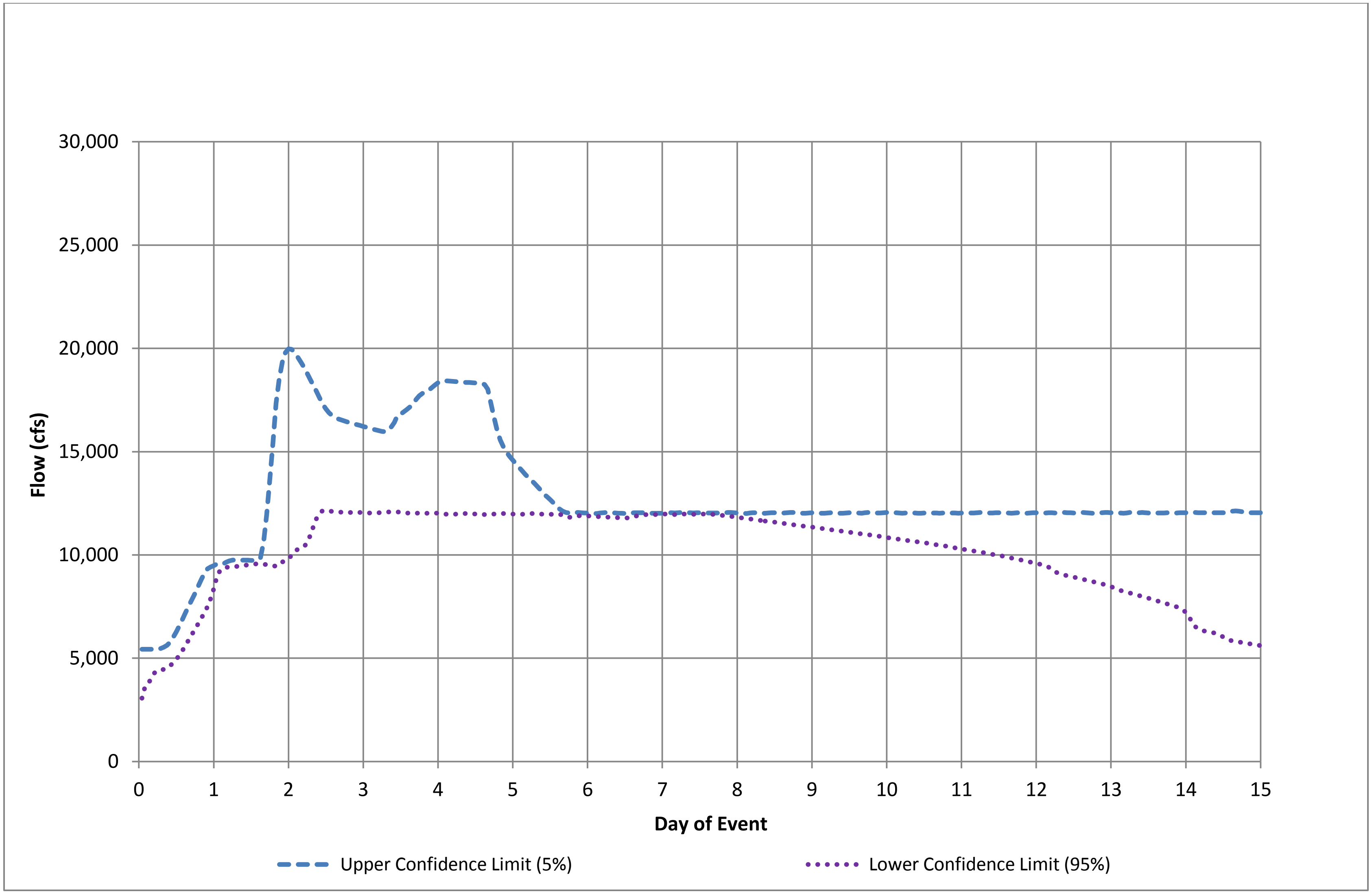

Figure 33 – Green River at Auburn Simulated Discharge, 0.5 Percent Flood

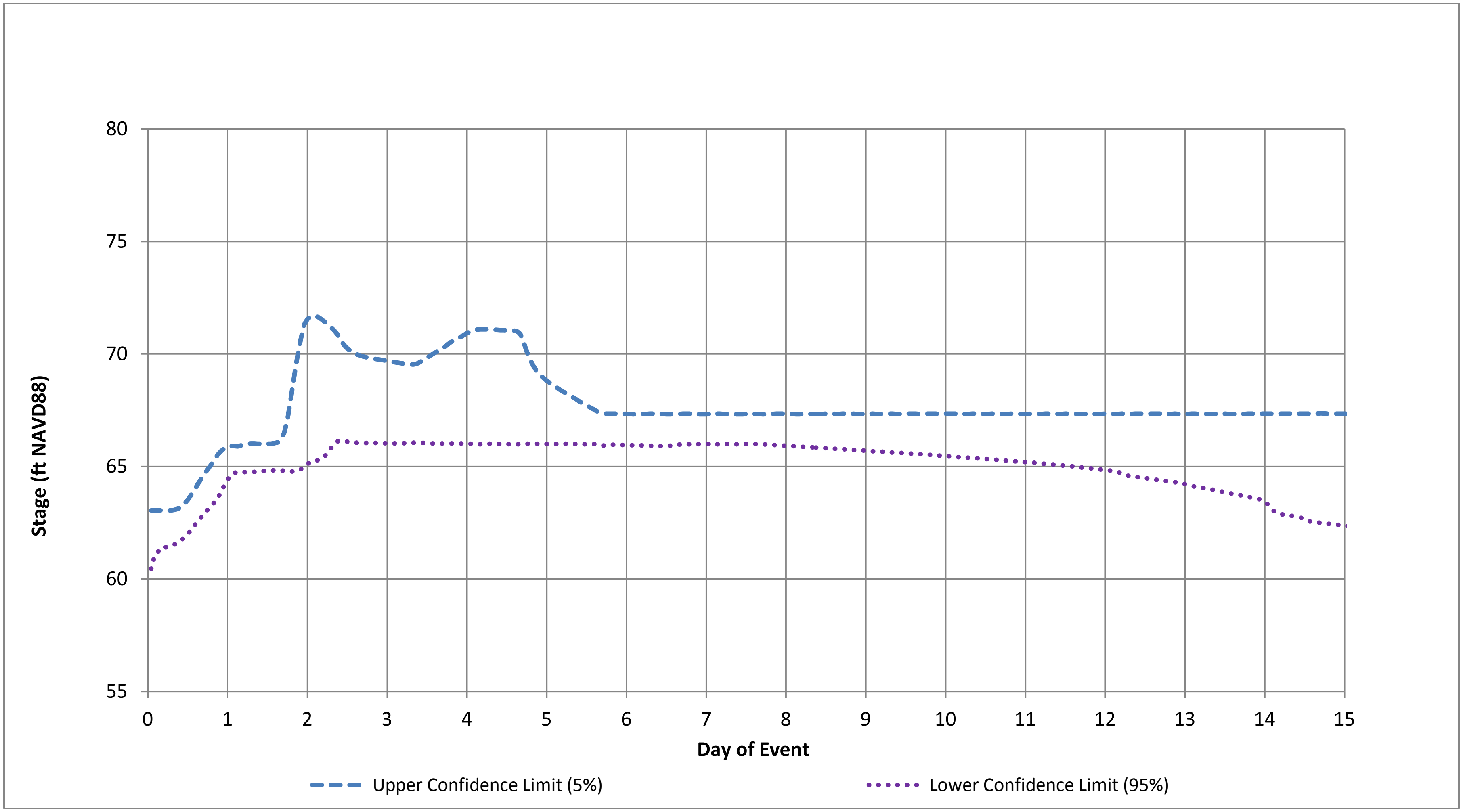

Figure 34 – Green River at Auburn Simulated Stage, 0.5 Percent Flood

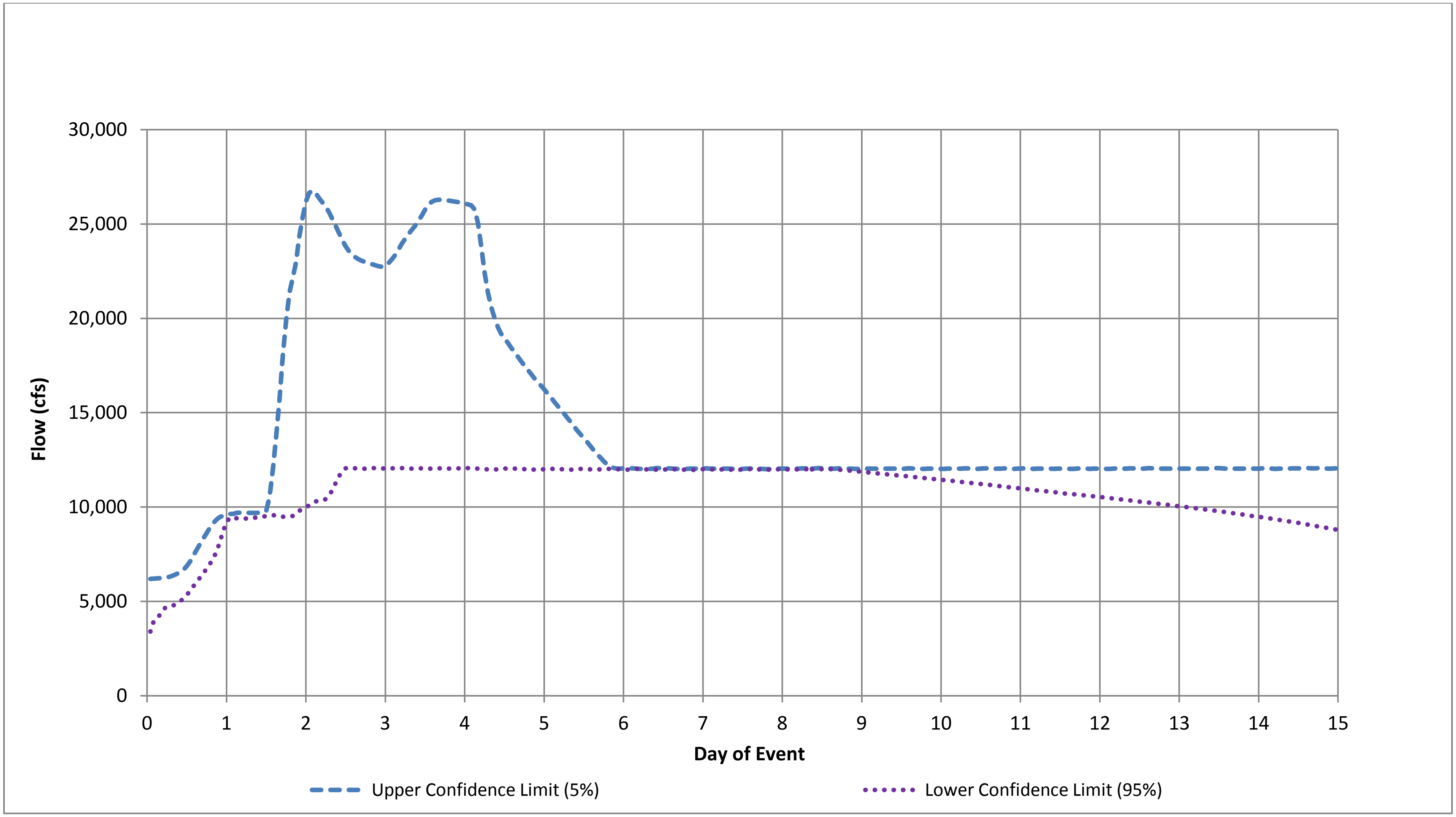

Figure 35 – Green River at Auburn Simulated Discharge, 0.2 Percent Flood

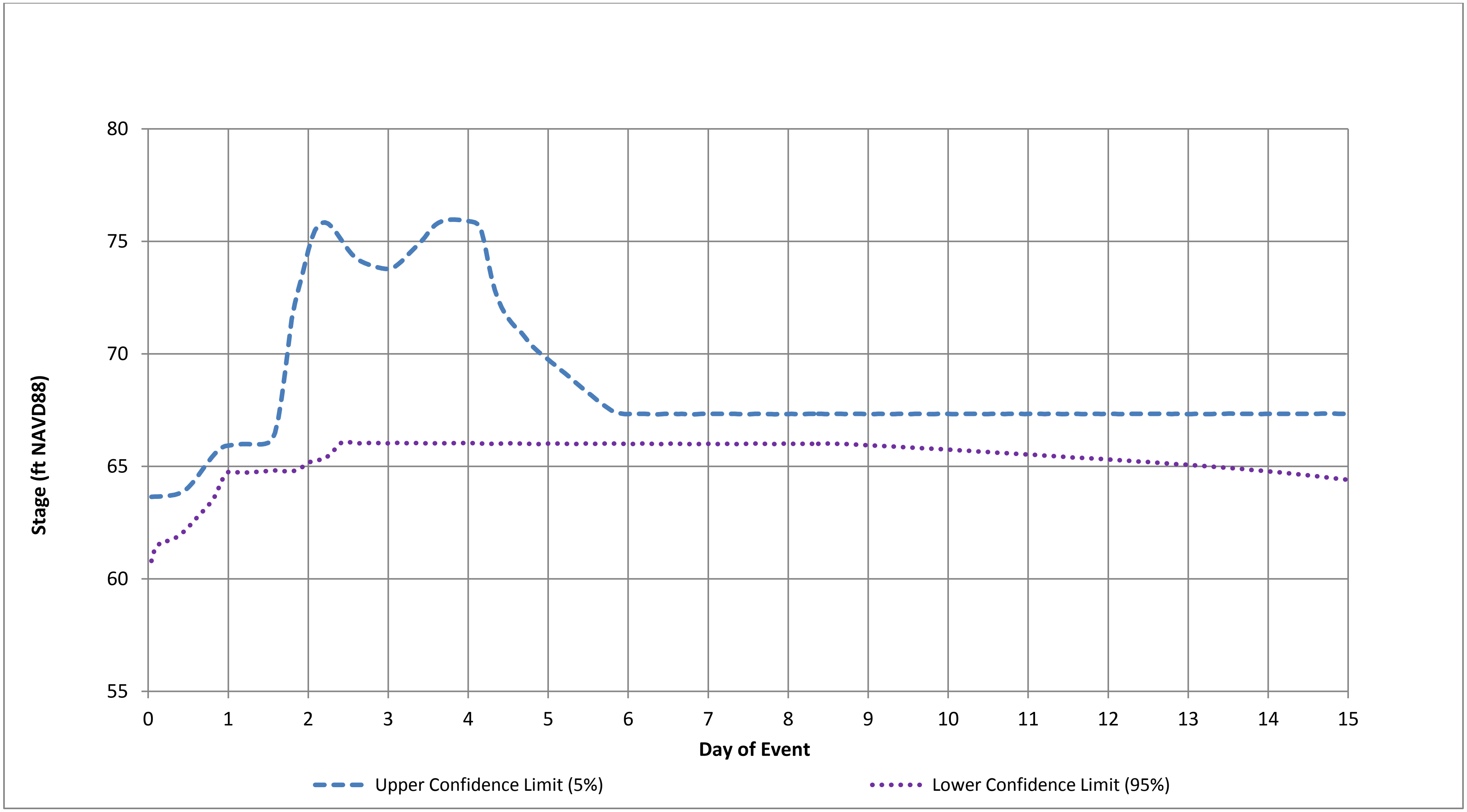

Figure 36 – Green River at Auburn Simulated Stage, 0.2 Percent Flood