# UPPER MISSISSIPPI RIVER RESTORATION
# FEASIBILITY REPORT
# WITH INTEGRATED ENVIRONMENTAL ASSESSMENT

## KEITHSBURG DIVISION
## HABITAT REHABILITATION AND ENHANCEMENT PROJECT

## POOL 18, UPPER MISSISSIPPI RIVER
## RIVER MILES 428.0 THROUGH 431.0
## MERCER COUNTY, ILLINOIS

## APPENDIX H

## HYDROLOGY AND HYDRAULICS

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# APPENDIX H
# HYDROLOGY AND HYDRAULICS

This appendix presents the hydrologic and hydraulic assessment of the *Keithsburg Division Habitat Rehabilitation and Enhancement Project* (Project) and summarizes the hydrologic and hydraulic evaluations of various Project features conducted by the U.S. Army Corps of Engineers (Corps), Rock Island District (District). The Project is managed by the U.S. Fish and Wildlife Service (USFWS) as part of the Port Louisa National Wildlife Refuge (NWR).

## I. INTRODUCTION AND LOCATION

The Project is a former levee and drainage district located on the left-descending bank of the Mississippi River in the upper portion of Pool 18, between River Miles (RM) 428.0 and 431.0 (Figure H-1). The City of Keithsburg, Illinois is located just south of the Project. It is bordered on the northern end by the Edwards River, on the southern end by Pope Creek, and on the west side by the Mississippi River. A 3-mile long berm separates the NWR from the Mississippi River which ties into two short berm sections on the north and south ends. On the northern berm section along the Edwards River there is a hardened spillway in addition to an earthen spillway along the southern end of the Mississippi River main berm reach. A water control structure with two 36-inch screw gates is located on the south end of the main berm downstream of the main berm spillway. The southern berm reach along Pope Creek is a former railroad embankment and was breached during the flood of 1993. The Iowa River enters the Mississippi River approximately 3 miles upstream of the Project area (~RM 434.0).

The upper fifth of the interior of the Project is separated by a road that crosses from the bluff to the berm. A culvert located near the eastern end of the road allows water to pass from the northern portion, (herein referred to as Spring Slough), to the southern portion of the Project area (Keithsburg South). The interior is made up of a mosaic of wetlands and bottomland forest. A hard mast tree component still survives at the northern end of the Project.

All elevations used in this appendix are expressed using the North American Vertical Datum of 1988 (NAVD88) unless otherwise stated. The conversion from MSL1912 to NAVD88 throughout the project river reach is NAVD88 +0.85 foot = MSL1912.

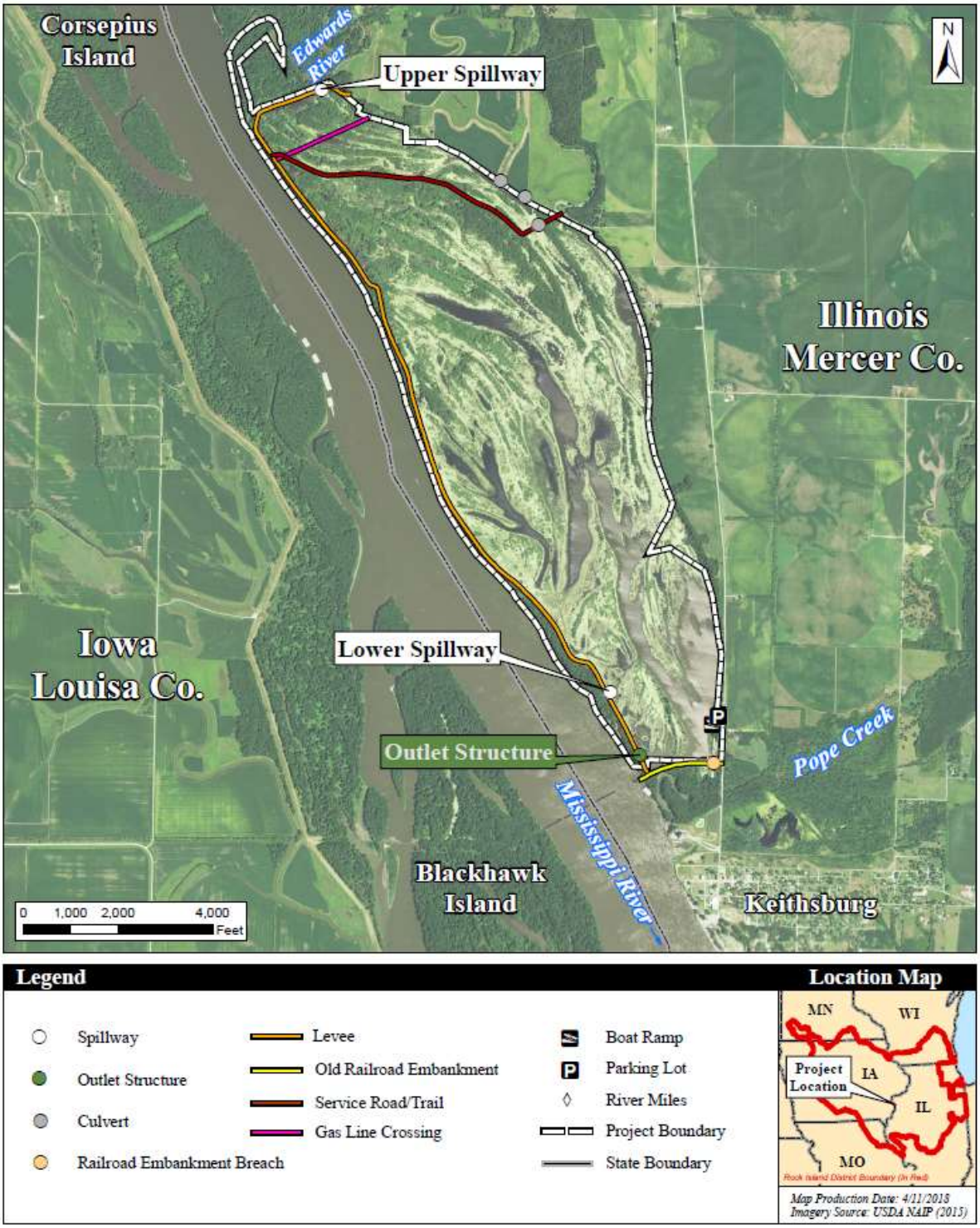


**Figure H-1:** Existing Features

## II. CLIMATE

Annual climate data for the Aledo, Illinois, U.S. Cooperative Network Station (gage #110072) provides statistics for precipitation, snowfall, and temperature data used for the Project site (Tables H-1 and H-2). The period of record for the reported values is 1985 through 2014. Minimum and maximum precipitation, snowfall, and temperature values are included in this period of record.

**Table H-1:** Average Mean and Extremes of Monthly Precipitation (COOP Gage #110072)

| | Precipitation | | | | | Snow | | |
|---|---|---|---|---|---|---|---|---|
| | Average | Maximum | | Minimum | | Average | Maximum | |
| **Month** | **(in)** | **(in)** | **Year** | **(in)** | **Year** | **(in)** | **(in)** | **Year** |
| Jan | 1.47 | 3.6 | 1996 | 0.16 | 1986 | 6.6 | 18.0 | 1999 |
| Feb | 1.74 | 4.00 | 2001 | 0.44 | 1987 | 4.7 | 20.4 | 2014 |
| Mar | 2.56 | 6.64 | 1991 | 0.98 | 1992 | 2.1 | 8.0 | 2013 |
| Apr | 3.84 | 6.84 | 1999 | 1.05 | 1985 | 0.2 | 6.0 | 1997 |
| May | 4.69 | 11.6 | 2001 | 0.45 | 1992 | 0.0 | 0.0 | |
| Jun | 4.67 | 11.97 | 1993 | 0.97 | 1988 | 0.0 | 0.0 | |
| Jul | 3.42 | 11.04 | 1992 | 0.44 | 1991 | 0.0 | 0.0 | |
| Aug | 4.13 | 10.35 | 1993 | 0.04 | 2013 | 0.0 | 0.0 | |
| Sep | 3.29 | 10.6 | 2008 | 0.75 | 2002 | 0.0 | 0.0 | |
| Oct | 2.79 | 8.10 | 1998 | 0.50 | 1998 | 0.1 | 3.0 | 1997 |
| Nov | 2.25 | 5.72 | 1992 | 0.10 | 2002 | 0.7 | 4.3 | 2014 |
| Dec | 2.00 | 5.75 | 1987 | 0.42 | 1995 | 5.2 | 31.0 | 2000 |
| **Annual** | **36.97** | | | | | **19.9** | | |

**Table H-2:** Average Mean and Extremes of Monthly Temperature (COOP Gage #110072)

| | Temperature | | |
|---|---|---|---|
| | Average | Maximum | Minimum |
| **Month** | **(°F)** | **(°F)** | **(°F)** |
| Jan | 22.41 | 69 | -25 |
| Feb | 25.90 | 68 | -28 |
| Mar | 38.49 | 87 | -10 |
| Apr | 50.44 | 91 | 17 |
| May | 61.20 | 93 | 26 |
| Jun | 70.32 | 103 | 42 |
| Jul | 73.76 | 102 | 49 |
| Aug | 72.16 | 102 | 45 |
| Sep | 64.22 | 99 | 31 |
| Oct | 52.18 | 91 | 18 |
| Nov | 39.07 | 78 | 0 |
| Dec | 26.40 | 70 | -20 |
| **Annual** | **49.65** | | |

## III. TOPOGRAPHY

The Project is a backwater complex located within the Mississippi River floodway, but separated from the River by a berm. Topographic diversity is significant enough to provide for several different habitat types, including scrub-shrub, submersed and emergent aquatic vegetation, wet meadow, wet floodplain forest and open water. It is bordered to the east by a bluff; however, the northeastern-most boundary is separated from the bluff by low-lying agricultural fields that drain into the Project's Spring Slough area (Figure H-2).

The District Survey Branch conducted extensive surveys that included survey of the interior that was used to develop storage volume-elevation relationships for both Spring Slough and Keithsburg South. Ground survey data that was collected as part of the feasibility study also included the berm and spillway structures, as well as the Pope Creek breach (old railroad embankment).

LiDAR data for Mercer County was available at the time of the study and had improved vertical accuracy over the Long-Term Resource Monitoring Program (LTRMP) Bluff to Bluff LiDAR dataset. The supplemental vertical accuracy at the 95th percentile was 0.966 foot for the Brush landcover category. Ground-truthing of the Mercer County LiDAR was completed after an extensive land-survey collection effort was completed. The site-specific ground-truthing results indicated an additional dataset correction was warranted. The LiDAR points were adjusted down by 0.42 feet.

## IV. WATERSHED CHARACTERISTICS

The watershed that contributes to the Project is approximately 800 acres (1.25 square miles) in size. The watershed is located on an upland stream terrace landform. The dominant soil types within the watershed include Sparta loamy sand (88B) (35%), Littleton silt loam (81A) (14%), Coloma Sand (689B) (12%) and Dickinson sandy loam (87B) (10%). The drainage class of these soils is excessively drained, somewhat poorly drained, excessively drained and well drained, respectively. The Sparta loamy sands and Coloma sands have a high and very high susceptibility to wind erosion, respectively. The Littleton silt loam has a slight susceptibility to wind erosion and the Dickinson sandy loam has a moderately high susceptibility to wind erosion.

Land use throughout the watershed is dominantly cultivated crops (74%) and deciduous forest (17%). Center-pivot irrigation is a common practice within the watershed to sustain row crop production within the generally sandy, well-drained soils.

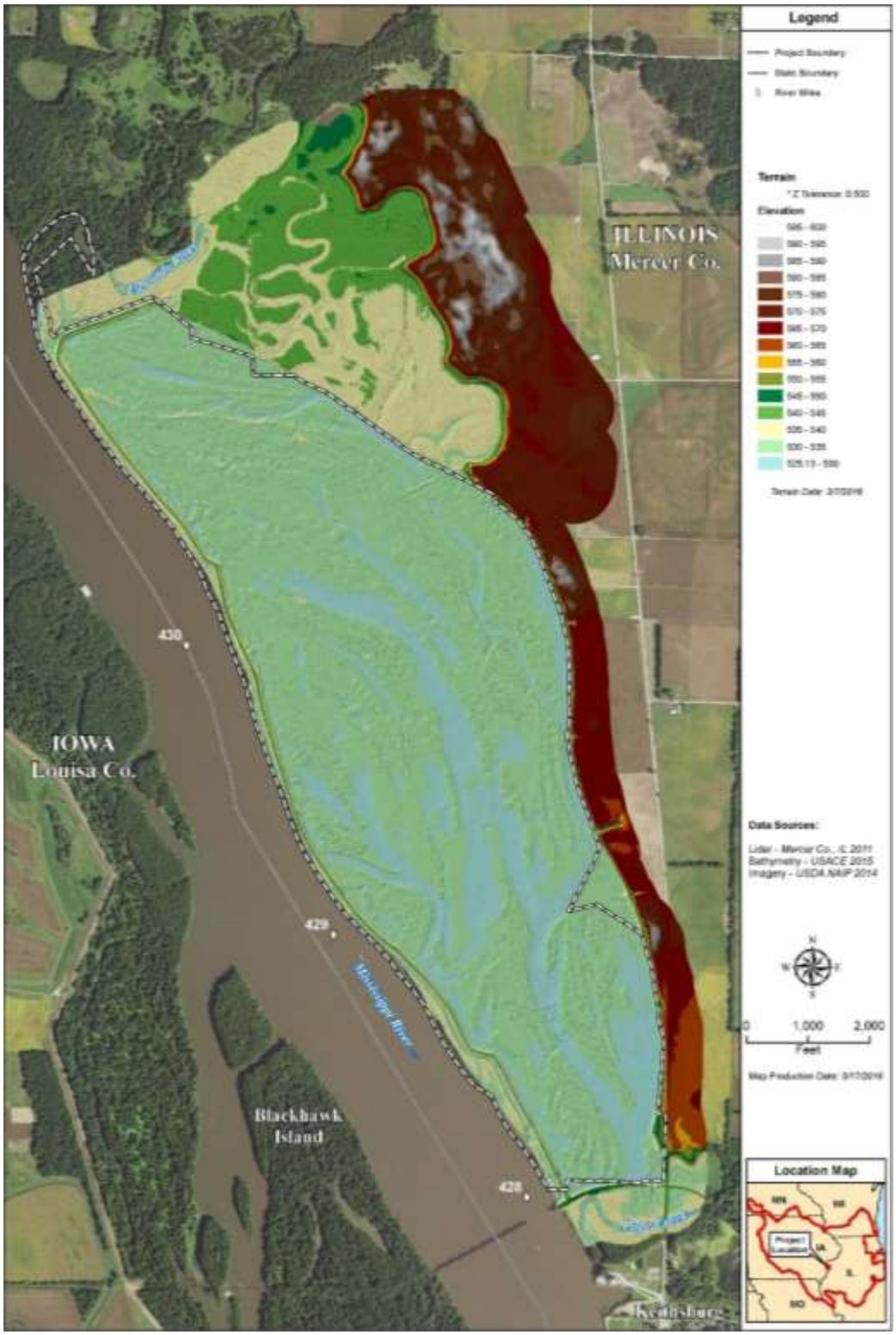


**Figure H-2:** Elevation Model

## V. GROUNDWATER

The surficial geology of the watershed is dominated by alluvial sediments from the Hudson Glacial Episode, specifically the Cahokia Fm. which is primarily river sand, gravel and silt. Thickness of the Quaternary deposits throughout the watershed is thought to be 50-100 feet.

Based on well logs from nearby irrigation wells, the sands and gravels of the Cahokia Fm. serve as the source aquifer for irrigation of adjacent agricultural fields. These well logs indicate the first ~60 feet from surface (~570 feet) is sand, but in varying grain sizes. The sand and gravel aquifer is located below a medium grain sand layer, however these units are likely hydraulically connected. The logs show medium grain sand present at roughly the same elevation as the Project and the streams near Spring Slough (~ 530 feet), suggesting this is the same aquifer that is discharging to the Project. This supports the occurrence of groundwater seeps along the toe of the bluff. The relatively high hydraulic conductivity of the surficial aquifer materials provide seeps when the surface water elevation in the Project area decreases. Additionally, the regional groundwater flow pattern is toward the Mississippi River and the Project.

Additional groundwater discussion is included in Section IX, *Interior Water Level Modeling*.

## VI. MISSISSIPPI RIVER-POOL 18

**A. Historic and Current Mississippi River Hydrology.** The District maintains continuous stage records at Lock and Dam 17 and Lock and Dam 18. The USGS manages stream gages on the Edwards River near New Boston, Illinois (USGS 05466500), the Iowa River near Wapello, Iowa (USGS 05465500), and Pope Creek near Keithsburg, Illinois (Gage 05467000). A summary of the nearby gages and their properties is provided in Table H-3.

**Table H-3:** Stream Gages adjacent to the Project Area

| Gage Name | River Mile | Drainage Area (sq. miles) | Period of Record |
|---|---|---|---|
| Mississippi River @ L&D17 Tail | 437.1 | 99,600 | 1932-present |
| Iowa River at Wapello, Iowa (#5465500) | Enters Mississippi at RM 434 | 12,499 | 1915-present |
| Edwards River near New Boston, IL (#5466500) | Enters Mississippi at RM 431.3 | 445 | 1934-present |
| Pope Creek near Keithsburg, IL (#5467000) | Enters Mississippi at RM 427.7 | 174 | 1934-present |
| Mississippi River @ Keithsburg | 427.4 | 112,870 | 1900-present |
| Mississippi River @ L&D18 Pool | 410.5 | 113,600 | 1936- present |

Lock and Dam 18 provides navigable channel depths by maintaining a water surface elevation of 528 feet MSL 1912 (flat pool) or higher. Pool 18 is regulated using a dam control point. The annual river stage hydrograph is affected by river regulation such that low river stages are maintained higher by the dam during low discharge periods, thereby limiting overall fluctuations in river stage. However, the degree of influence of the impounding dam diminishes as you move upstream of the dam, where

greater variation in river stage occurs (Figure H-3). The Project is located less than one mile upstream of the Keithsburg gage where the river does experience some annual fluctuation in river stage. Additionally, the Iowa River, which has a drainage area of nearly 12,500 square miles, enters the Mississippi River three miles upstream of the Project. The Keithsburg gage drainage area is 112,870 square miles.

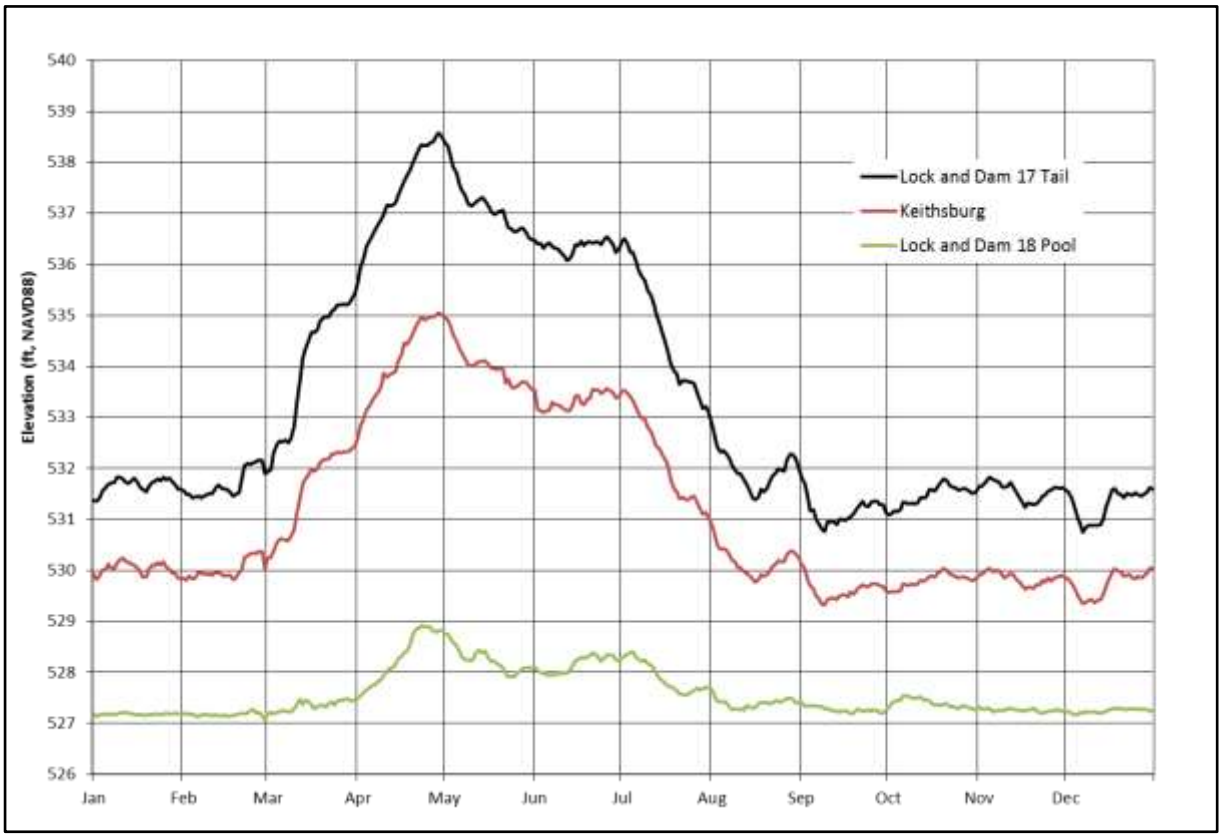


**Figure H-3:** Average Annual Elevation Hydrographs for the Upper, Middle, and Lower Portions of Pool 18 (1986-2015)

**B. Flood Conditions.** The 2004 Upper Mississippi River System Flow Frequency Study (2004 UMRS Flow Frequency Study) includes several cross sections through the three-mile Project reach (see references in Section XIX). Results from the 2004 UMRS Flow Frequency Study that pertain to the Project are shown in Table H-4. Based on historic records from past flood events and accounts from the USFWS of more recent spillway overtopping events, flooding of the interior through the Edwards River spillway has always occurred due to high Mississippi River stages. In other words, the Mississippi River stages typically drive overtopping on the Edwards River Spillway rather than high stages on the Edwards River that occur during low stages on the Mississippi River. Table H-5 provides a summary of the record high stages on the Mississippi River as observed at the Keithsburg gage. Tables H-6 and H-7 summarize record high stages and corresponding discharge at the Pope Creek and Edwards gages, respectively.

**Table H-4:** Upper Mississippi River Flow Frequency Study (Elevations in MSL 1912)

| | Exceedance Probability | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| River | 0.5 | | 0.2 | | 0.1 | | 0.04 | | 0.02 | | 0.01 | | 0.005 | | 0.002 | |
| Mile | feet | cfs | feet | cfs | feet | cfs | feet | cfs | feet | cfs | feet | cfs | feet | cfs | feet | cfs |
| 422.4 | 534.7 | 168,000 | 537.3 | 219,000 | 538.9 | 251,000 | 540.6 | 290,000 | 541.8 | 319,000 | 542.8 | 347,000 | 543.7 | 373,000 | 544.6 | 405,000 |
| 423 | 535.0 | 168,000 | 537.6 | 219,000 | 539.2 | 251,000 | 540.9 | 290,000 | 542.1 | 319,000 | 543.1 | 347,000 | 543.9 | 373,000 | 544.9 | 405,000 |
| 423.5 | 535.2 | 168,000 | 537.8 | 219,000 | 539.4 | 251,000 | 541.1 | 290,000 | 542.2 | 319,000 | 543.2 | 347,000 | 544.1 | 373,000 | 545.0 | 406,000 |
| 424 | 535.5 | 168,000 | 538.1 | 219,000 | 539.6 | 251,000 | 541.3 | 290,000 | 542.5 | 319,000 | 543.5 | 347,000 | 544.3 | 373,000 | 545.3 | 406,000 |
| 424.4 | 535.6 | 168,000 | 538.2 | 219,000 | 539.8 | 251,000 | 541.5 | 290,000 | 542.6 | 319,000 | 543.6 | 347,000 | 544.5 | 373,000 | 545.4 | 406,000 |
| 425 | 535.9 | 168,000 | 538.5 | 219,000 | 540.1 | 251,000 | 541.8 | 290,000 | 542.9 | 319,000 | 543.9 | 347,000 | 544.7 | 373,000 | 545.7 | 406,000 |
| 425.5 | 536.1 | 168,000 | 538.7 | 219,000 | 540.3 | 251,000 | 542.0 | 290,000 | 543.1 | 319,000 | 544.1 | 347,000 | 544.9 | 373,000 | 545.9 | 406,000 |
| 426 | 536.4 | 168,000 | 539.0 | 218,000 | 540.6 | 251,000 | 542.3 | 290,000 | 543.4 | 319,000 | 544.4 | 347,000 | 545.2 | 373,000 | 546.1 | 406,000 |
| 427 | 536.7 | 168,000 | 539.3 | 218,000 | 540.9 | 251,000 | 542.6 | 290,000 | 543.7 | 319,000 | 544.7 | 347,000 | 545.5 | 373,000 | 546.5 | 406,000 |
| 427.6 | 536.9 | 168,000 | 539.6 | 218,000 | 541.1 | 251,000 | 542.8 | 289,000 | 543.9 | 319,000 | 544.9 | 346,000 | 545.8 | 373,000 | 546.7 | 406,000 |
| 427.95 | 537.0 | 168,000 | 539.7 | 218,000 | 541.3 | 251,000 | 542.9 | 289,000 | 544.1 | 319,000 | 545.1 | 347,000 | 545.9 | 373,000 | 546.8 | 406,000 |
| 428 | 537.0 | 167,000 | 539.7 | 218,000 | 541.3 | 250,000 | 543.0 | 289,000 | 544.1 | 318,000 | 545.1 | 346,000 | 545.9 | 372,000 | 546.9 | 406,000 |
| 428.05 | 537.1 | 167,000 | 539.8 | 218,000 | 541.4 | 250,000 | 543.1 | 289,000 | 544.2 | 318,000 | 545.2 | 346,000 | 546.0 | 372,000 | 547.0 | 406,000 |
| 428.7 | 537.4 | 167,000 | 540.0 | 218,000 | 541.6 | 250,000 | 543.3 | 289,000 | 544.5 | 318,000 | 545.5 | 346,000 | 546.3 | 372,000 | 547.3 | 406,000 |
| 429 | 537.5 | 167,000 | 540.2 | 218,000 | 541.8 | 250,000 | 543.5 | 289,000 | 544.7 | 318,000 | 545.7 | 346,000 | 546.6 | 372,000 | 547.5 | 406,000 |
| 429.5 | 537.7 | 167,000 | 540.3 | 218,000 | 542.0 | 250,000 | 543.7 | 289,000 | 544.9 | 318,000 | 545.9 | 346,000 | 546.8 | 372,000 | 547.8 | 406,000 |
| 430 | 537.9 | 167,000 | 540.6 | 218,000 | 542.2 | 250,000 | 543.9 | 289,000 | 545.2 | 318,000 | 546.2 | 346,000 | 547.1 | 372,000 | 548.1 | 406,000 |
| 430.5 | 538.1 | 167,000 | 540.8 | 218,000 | 542.5 | 250,000 | 544.2 | 289,000 | 545.5 | 318,000 | 546.6 | 346,000 | 547.5 | 372,000 | 548.5 | 406,000 |
| 431 | 538.3 | 167,000 | 541.1 | 218,000 | 542.8 | 250,000 | 544.6 | 289,000 | 545.8 | 318,000 | 546.9 | 346,000 | 547.8 | 372,000 | 548.8 | 406,000 |
| 431.5 | 538.7 | 166,000 | 541.4 | 217,000 | 543.2 | 248,000 | 545.1 | 287,000 | 546.4 | 315,000 | 547.5 | 342,000 | 548.5 | 370,000 | 549.4 | 406,000 |
| 432 | 539.0 | 166,000 | 541.8 | 217,000 | 543.5 | 248,000 | 545.5 | 287,000 | 546.8 | 315,000 | 547.9 | 342,000 | 548.9 | 370,000 | 549.9 | 406,000 |
| 432.5 | 539.2 | 166,000 | 542.1 | 217,000 | 543.8 | 248,000 | 545.8 | 287,000 | 547.1 | 315,000 | 548.2 | 342,000 | 549.2 | 370,000 | 550.2 | 406,000 |
| 433 | 539.6 | 166,000 | 542.4 | 216,000 | 544.2 | 248,000 | 546.2 | 287,000 | 547.5 | 315,000 | 548.6 | 342,000 | 549.7 | 370,000 | 550.7 | 406,000 |
| 433.4 | 539.8 | 166,000 | 542.6 | 216,000 | 544.4 | 248,000 | 546.5 | 287,000 | 547.7 | 315,000 | 548.9 | 342,000 | 550.0 | 370,000 | 551.0 | 406,000 |
| 434 | 540.0 | 166,000 | 543.0 | 216,000 | 544.7 | 248,000 | 546.8 | 287,000 | 548.1 | 315,000 | 549.4 | 342,000 | 550.5 | 370,000 | 551.5 | 406,000 |
| 434.4 | 540.2 | 166,000 | 543.3 | 216,000 | 544.9 | 248,000 | 547.1 | 287,000 | 548.5 | 315,000 | 549.7 | 342,000 | 550.8 | 370,000 | 551.9 | 406,000 |
| 435 | 540.4 | 149,000 | 544.1 | 197,000 | 545.2 | 227,000 | 547.6 | 264,000 | 549.2 | 291,000 | 550.4 | 315,000 | 551.5 | 338,000 | 552.5 | 370,000 |
| 435.6 | 540.5 | 149,000 | 544.4 | 197,000 | 545.3 | 227,000 | 547.8 | 264,000 | 549.5 | 291,000 | 550.7 | 315,000 | 551.7 | 338,000 | 552.7 | 370,000 |
| 436 | 540.6 | 149,000 | 544.4 | 197,000 | 545.4 | 227,000 | 547.8 | 264,000 | 549.5 | 291,000 | 550.7 | 315,000 | 551.8 | 338,000 | 552.8 | 370,000 |
| 436.5 | 540.7 | 149,000 | 544.5 | 197,000 | 545.5 | 227,000 | 548.0 | 264,000 | 549.6 | 291,000 | 550.9 | 315,000 | 551.9 | 338,000 | 553.0 | 370,000 |
| 437 | 540.8 | 149,000 | 544.6 | 197,000 | 545.7 | 227,000 | 548.1 | 264,000 | 549.8 | 291,000 | 551.1 | 315,000 | 552.1 | 338,000 | 553.2 | 370,000 |
| 437.1 | 541.0 | 149,000 | 544.8 | 197,000 | 545.9 | 227,000 | 548.4 | 264,000 | 550.1 | 291,000 | 551.3 | 315,000 | 552.4 | 338,000 | 553.5 | 370,000 |
| 437.5 | 541.2 | 149,000 | 545.0 | 197,000 | 546.1 | 227,000 | 548.6 | 264,000 | 550.3 | 291,000 | 551.6 | 315,000 | 552.6 | 338,000 | 553.7 | 370,000 |
| 438 | 541.4 | 149,000 | 545.2 | 197,000 | 546.3 | 227,000 | 548.8 | 264,000 | 550.5 | 291,000 | 551.8 | 315,000 | 552.8 | 338,000 | 553.9 | 371,000 |
| 438.4 | 541.5 | 149,000 | 545.3 | 197,000 | 546.4 | 227,000 | 548.9 | 264,000 | 550.6 | 291,000 | 551.9 | 315,000 | 552.9 | 338,000 | 554.0 | 371,000 |
| 439 | 541.6 | 149,000 | 545.4 | 197,000 | 546.6 | 227,000 | 549.1 | 264,000 | 550.7 | 291,000 | 552.0 | 315,000 | 553.1 | 338,000 | 554.2 | 371,000 |
| 439.5 | 541.7 | 149,000 | 545.5 | 197,000 | 546.7 | 227,000 | 549.2 | 264,000 | 550.8 | 291,000 | 552.1 | 315,000 | 553.2 | 338,000 | 554.3 | 371,000 |
| 440 | 541.8 | 149,000 | 545.6 | 197,000 | 546.8 | 227,000 | 549.3 | 264,000 | 550.9 | 291,000 | 552.3 | 315,000 | 553.3 | 338,000 | 554.4 | 371,000 |

**Table H-5:** Record High Stages at Keithsburg Gage for the 1900-2014 Period of Record

| Stage | Elevation | Date |
|---|---|---|
| 24.49 | 547.68 | 06/17/2008 |
| 24.15 | 547.34 | 07/09/1993 |
| 20.72 | 543.91 | 05/12/2001 |
| 20.56 | 543.75 | 04/22/2013 |
| 20.46 | 543.65 | 04/28/1965 |
| 19.35 | 542.54 | 04/25/1973 |
| 19.19 | 542.38 | 04/24/2011 |
| 19.10 | 542.29 | 04/25/1993 |
| 18.71 | 541.90 | 04/14/1998 |
| 17.99 | 541.18 | 04/20/1997 |
| 17.46 | 540.65 | 10/07/1986 |
| 17.35 | 540.54 | 05/08/1975 |
| 17.24 | 540.43 | 04/26/1969 |
| 17.10 | 540.29 | 04/29/1951 |
| 16.90 | 540.09 | 06/26/1974 |
| 16.90 | 540.09 | 04/06/1979 |

**Table H-6:** Record High Stages at Pope Creek (NGVD29)

| Discharge | Stage | Elevation | Date |
|---|---|---|---|
| 9640 | 29.60 | 553.67 | 04/19/2013 |
| 8900 | 27.88 | 551.95 | 04/22/1973 |
| 8290 | 28.68 | 552.75 | 10/18/1998 |
| 7270 | 29.08 | 553.15 | 07/24/1993 |
| 7200 | 28.36 | 552.43 | 07/07/1982 |
| 6360 | 28.48 | 552.55 | 02/24/2001 |
| 6320 | 29.54 | 553.61 | 05/14/2010 |
| 5900 | 27.92 | 551.99 | 02/19/1986 |
| 5480 | 27.76 | 551.83 | 02/23/1985 |
| 5380 | 29.11 | 553.18 | 05/01/2003 |

**Table H-7:** Record High Stages at Edwards River (NAVD88)

| Discharge | Stage | Elevation | Date |
|---|---|---|---|
| 18000 | 23.33 | 552.45 | 04/22/1973 |
| 10300 | 22.49 | 551.61 | 06/20/1990 |
| 10300 | 24.13 | 553.25 | 09/14/2008 |
| 8630 | 23.82 | 552.94 | 04/20/2013 |
| 8590 | 22.37 | 551.49 | 08/16/1993 |
| 8090 | 23.70 | 552.82 | 12/30/2008 |
| 7860 | 22.22 | 551.34 | 08/07/1982 |
| 7510 | 23.57 | 552.69 | 05/15/2010 |
| 7280 | 20.56 | 549.68 | 04/26/1950 |
| 7280 | 20.63 | 549.75 | 05/12/1951 |

The most recent Flood Insurance Study (FIS) completed for Mercer County is dated April 19, 2010, and includes the Project area. The flow frequency values referenced by the 2010 FIS are from the 2004 UMRS Flow Frequency Study.

**C. Stage Hydrographs and Elevation Duration.** Historical water levels have not been regularly documented. A staff gage located near the existing water control structure served as the water level gaging device, but no datum information was available. In June and September of 2015 the USFWS installed three pressure transducers with data loggers to record lake stages every fifteen minutes at three locations: Spring Slough; just downstream of the culvert connecting Spring Slough and Keithsburg South (north transducer); and in the southern portion of Keithsburg South (south transducer). The location of these gages is shown in Figure H-1 and the interior water level data recorded is shown in Figure H-4.

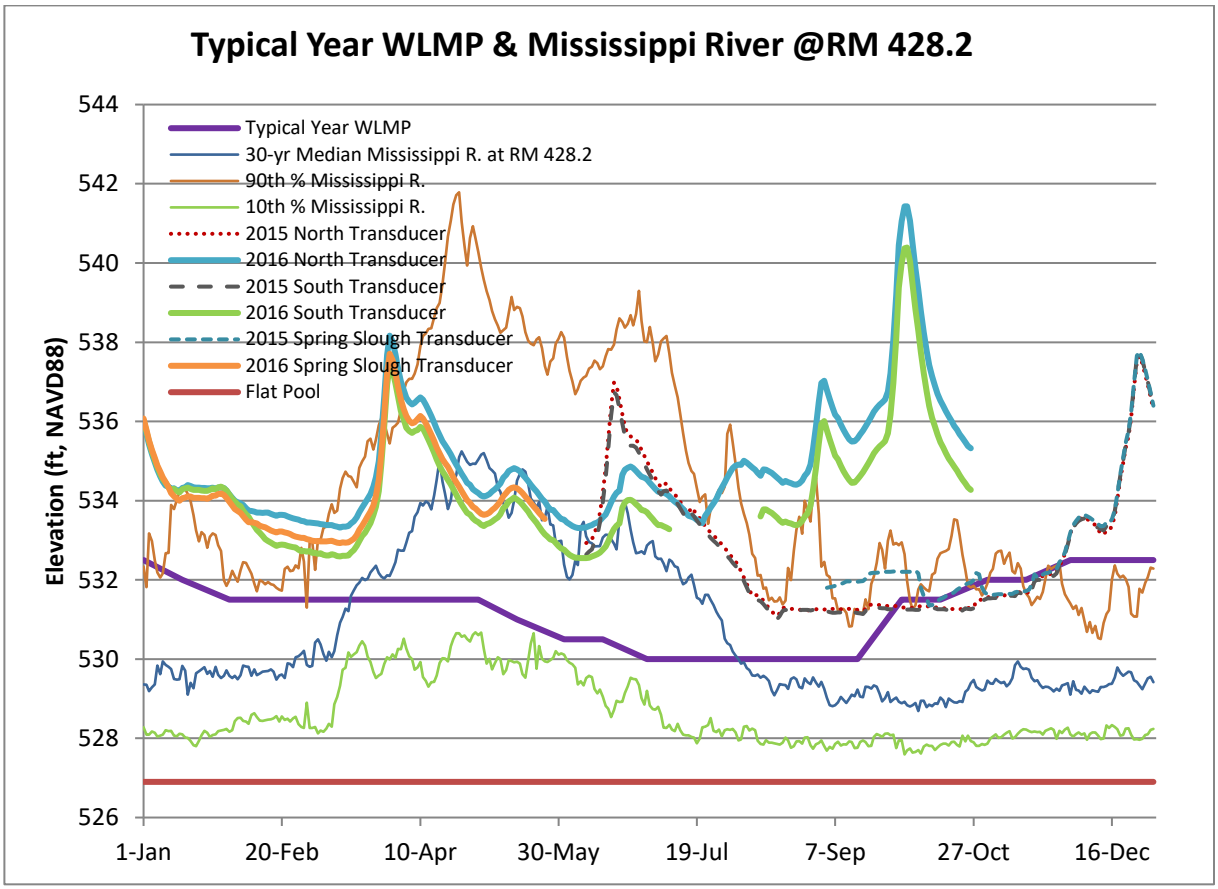


**Figure H-4:** Interior Water Level Data

Since the Pope Creek breach occurred in 1993, water level fluctuations are largely influenced by the Mississippi River, seen in Figure H-4 above, with very limited ability to manage water levels on the interior. A goal of the Project is to restore the berm so that water levels can be managed independently from the river.

As previously stated, the Keithsburg gage on the Mississippi River is located at RM 427.4, less than one mile downstream of the Project on the left-descending bank. The average annual elevation hydrograph is shown in Figure H-5. The hydrograph illustrates a spring rise due to early rains and snowmelt. The stage remains high through late spring to early summer as rain on snow or rain only events can produce floods. Stages typically decline sharply in July marking the beginning of a low-flow period that extends into mid-September before fall rains result in a modest rise that extends into the early winter months when flows drop off as tributary flows begin to decline due to freezing conditions. Figure H-6 illustrates the difference in the pre- and post-impoundment average annual hydrographs at the Keithsburg gage.

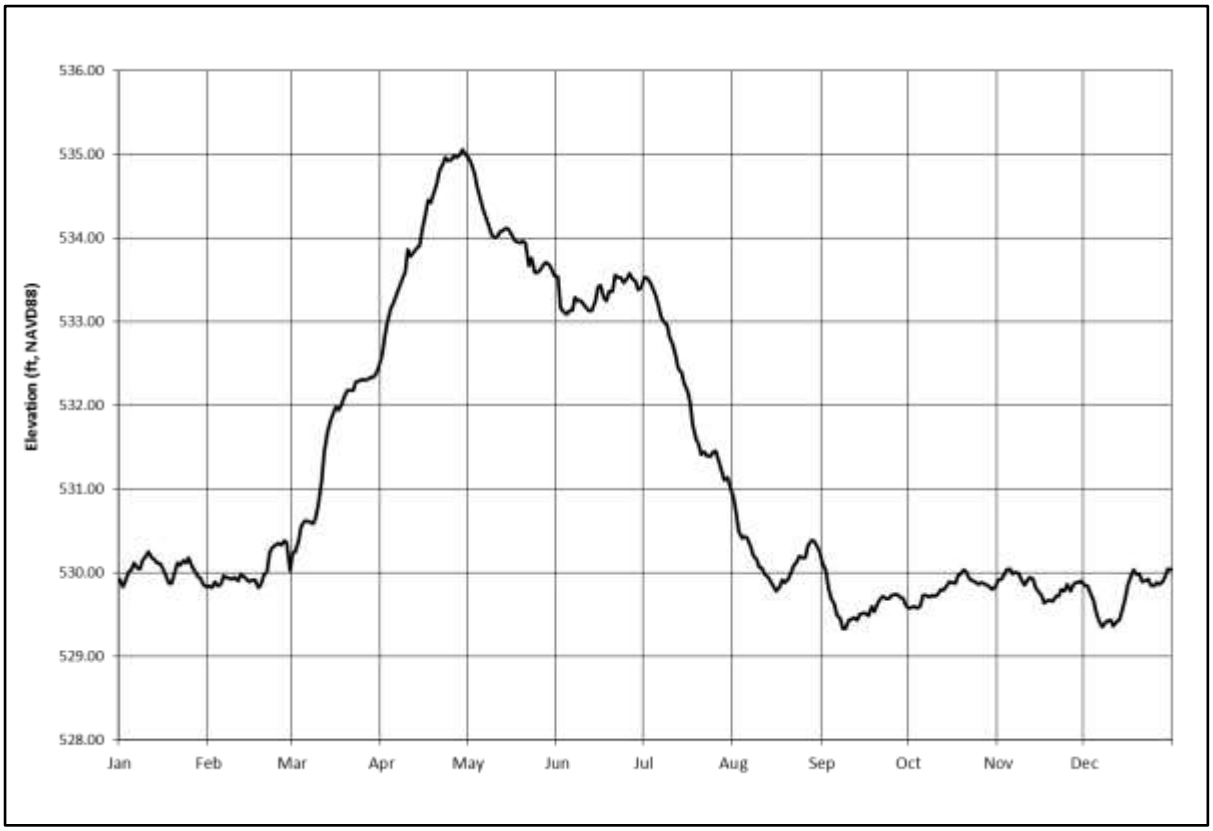


**Figure H-5:** Average Annual Hydrograph at the Keithsburg Gage for the 1986-2015 Period

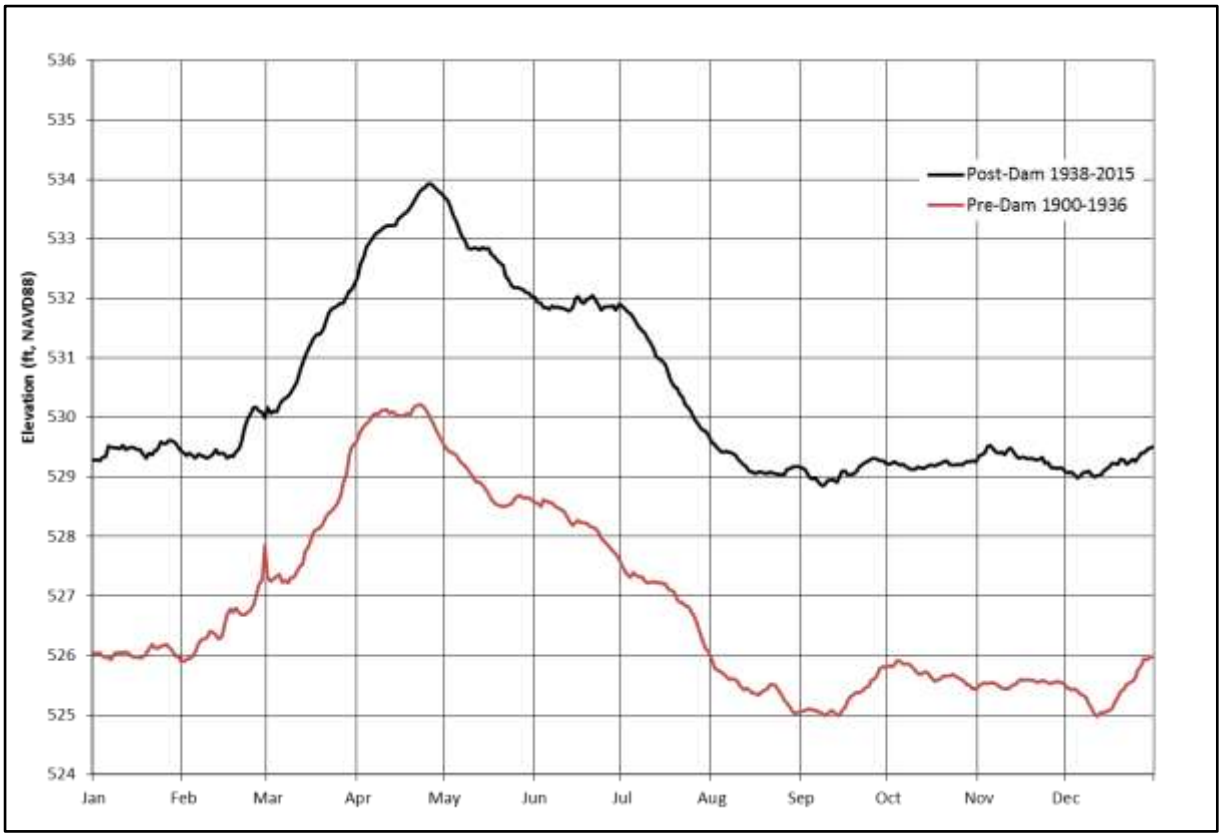


**Figure H-6:** Comparison of the Average Annual Hydrographs at the Keithsburg Gage for Pre- and Post-Impoundment

The hydrologic period from 1986-2015 was selected to characterize existing conditions and as the basis for design. The most recent 30-year period was selected because it is considered short enough to represent a stationary dataset (i.e. statistical properties of the data are not changing over time) and long enough to provide a large enough sample size to adequately represent the population. A comparison of annual elevation-duration curves for the current and previous 30-year periods (Figure H-7) indicates a consistent increase in stage across all durations and greater than a one foot increase in stage for the less frequent elevations.

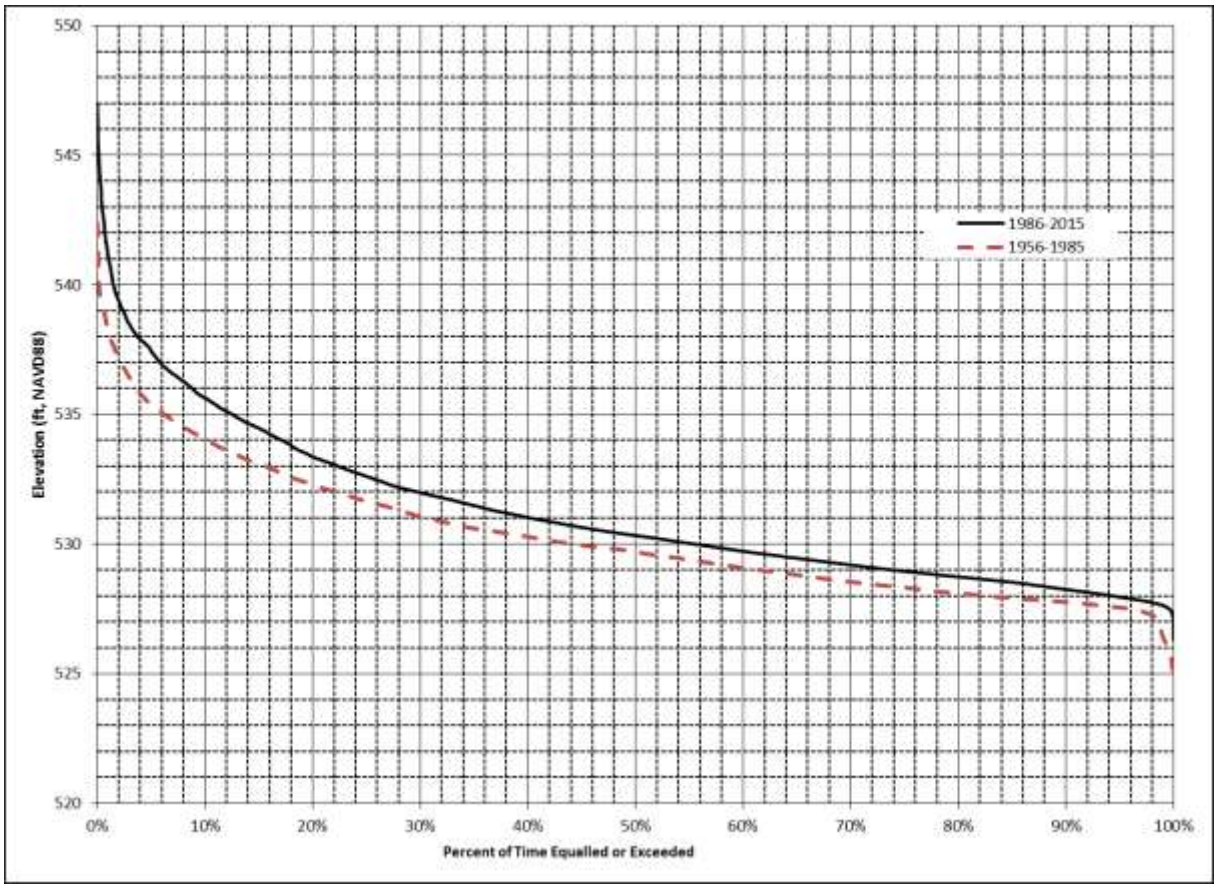


**Figure H-7:** Comparison of Annual Elevation Duration Curves at the Keithsburg Gage for the Current and Previous 30-year Periods

## VII. SEDIMENT DEPOSITION

The Iowa River confluence is located approximately 3 miles upstream of the Project. Pool 18 is one of the District's most heavily dredged pools. Historic Pool 18 dredge cuts in the Project vicinity are shown in Figure H-8.

To date, studies of backwater sedimentation rates within the Upper Mississippi River (UMR) have focused within Navigation Pools 4-10, and 13 (see references in Section XIX). Sedimentation rates from these studies range from as little as 0.2 cm/yr (Navigation Pool 7) to as high as 4 cm/yr (Navigation Pools 4-10). A sedimentation rate of 0.8 cm/yr for Navigation Pool 13 was reported by Rogala & Boma (see references in Section XIX). The Cumulative Effects Study indicates a backwater sediment deposition rate for Pools 12-19 of 0.05 cm/yr, as derived from the sediment budget (see references in Section XIX).

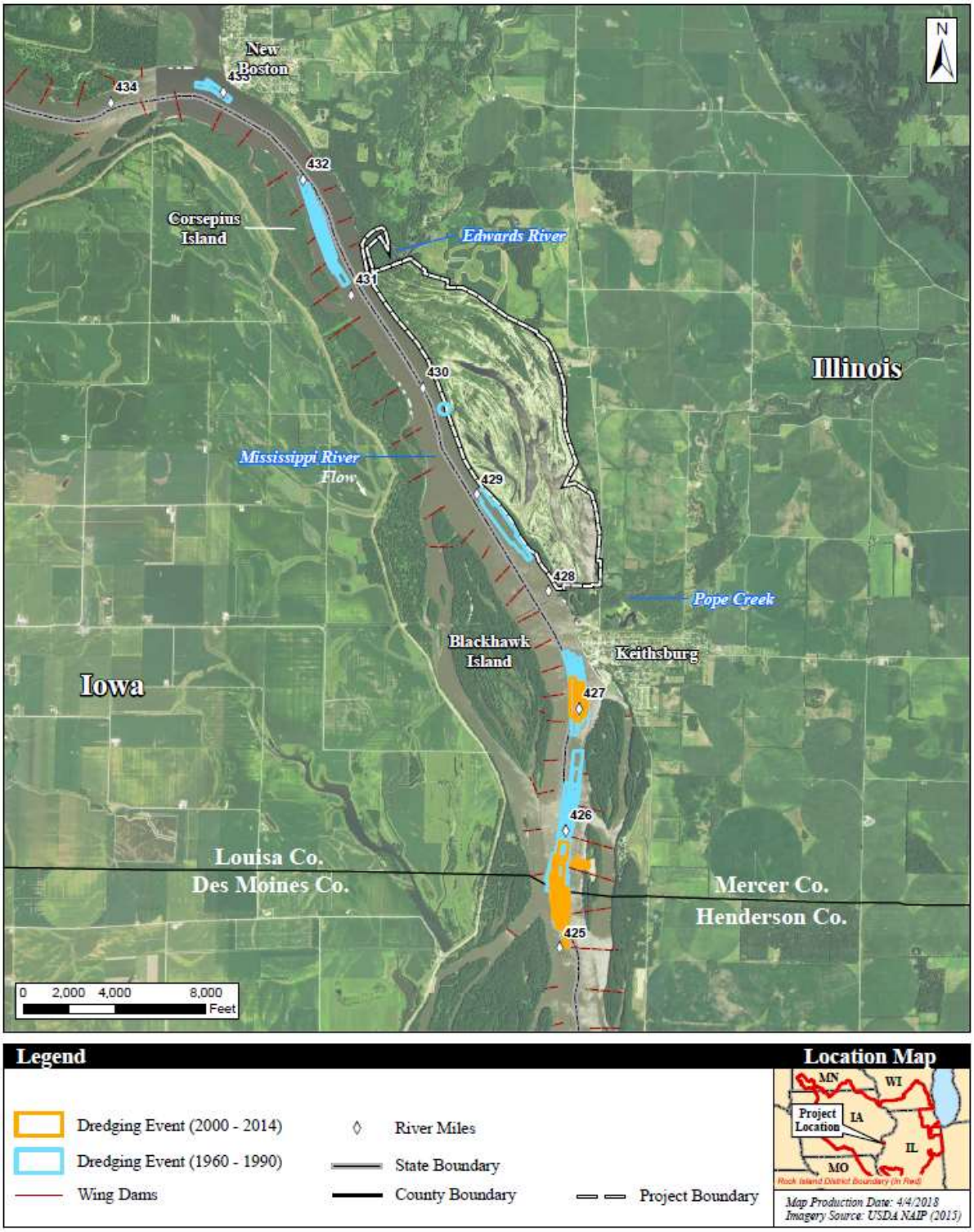


**Figure H-8:** Historic Dredge Cuts near the Project

Sources of sediment deposition within the bermed Project area include suspended sediments from the Mississippi and Edwards Rivers during spillway and/or berm overtopping as well as sediment originating in the watershed that may be delivered through surface runoff or wind erosion. Sediment is also introduced through the Pope Creek breach. The dominant soils within the watershed include the Sparta loamy sand, Littleton silt loam, Coloma Sand and Dickinson sandy loam. Each of these soils have a slight susceptibility to water erosion. The Sparta loamy sands and Coloma sands have a high and very high susceptibility to wind erosion, respectively. The Littleton silt loam has a slight susceptibility to wind erosion and the Dickinson sandy loam has a moderately high susceptibility to wind erosion.

## VIII. EXISTING PERIMETER BERM AND SPILLWAY FEATURES

The existing Edwards River spillway along the northern berm reach is overtopped at an elevation of 539.36' (540.21 feet MSL1912), characterized as an incipient overtopping frequency of 27%. The Mississippi River reach spillway is overtopped at an elevation of 540.18' (541.03 feet MSL 1912 at RM 428.2), characterized as an incipient overtopping frequency of 12%. The Pope Creek breach which occurred in 1993, has a bottom elevation of 529.65 feet. Based on the annual elevation duration plots from the Keithsburg Gage, the Mississippi River exceeds the breach elevation approximately 62% of the time. Assuming repair of the Pope Creek breach were completed, the overall level of protection currently afforded to the system would be a 27% exceedance probability elevation, as defined by the Edwards River spillway elevation. Although there is not an official or complete record of overtopping events, limited records provided by the sponsor indicate that the Edwards River spillway was overtopped in 1969 and 1993 and anecdotal information suggests overtopping also occurred in 2001, 2008, 2013 and 2014. Figure H-1 provides an illustration of the existing berm features and Table H-8 summarizes the location and elevation of key protection features along the existing berm. The existing berm profile is shown in the drawings, Appendix Q.

**Table H-8:** Summary of Existing Berm Features

| Existing Berm Feature | RM | Elevation NAVD88 |
|---|---|---|
| US Perimeter Berm | 431.07 | 543.5 |
| Edwards Spillway | 431.07 | 539.36 |
| Mississippi Spillway | 428.2 | 540.18 |
| DS Perimeter Berm | 427.95 | 542 |
| Pope Creek Breach | 427.95 | 529.65 |

## IX. INTERIOR WATER LEVEL MODELING

Current water level management infrastructure is very limited and does not reliably allow the sponsor to meet habitat goals and objectives as defined by the Water Level Management Plans (WLMPs) discussed below. Existing infrastructure relies exclusively on gravity drainage through the stoplog gates, and river stages significantly limit opportunities for gravity flow required to meet the desired water level management. During the spring and summer growing-season drawdown period, river stages are often too high to achieve the full drawdown and during the fall migration period river stages

are frequently too low to meet the desired filling schedule. Proposed water control structures include two sluice gate structures and a pump station to provide reliable water level management, however the operating cost of these capabilities as well as the incremental habitat benefit achieved by these features was a significant concern voiced by the sponsor. A more complete description of the proposed alternatives is included in Section XI, *Project Alternatives Evaluated.*

Interior water level modeling was identified by the PDT as the most appropriate tool to address the following two critical questions: 1) what level of success in meeting the Project goals (desired WLMP) can be expected under existing conditions and the proposed Project alternatives; 2) what is the expected (pumping) cost associated with the proposed alternatives.

The following discusses a 30-year simulation of interior water levels that was done using Hydrologic Engineering Center-River Analysis System Model v 5.0.2 (HEC-RAS). Watershed inputs were estimated using the Hydrologic Engineering Center-Hydrologic Modeling System (HEC-HMS) and groundwater contributions were estimated based on available well data. Operation of water level management features associated with the different Project alternatives was coded into HEC-RAS using the Rules Editor. The results from the 30-year simulation were used to assess the amount of pumping required to meet the desired water level management plan over a 50-year project life and to assess the level of success the different alternatives had in achieving the desired WLMP.

**A. Water Level Management Plans.** The PDT, with significant input provided by the USFWS, developed three annual WLMPs: Typical WLMP; Hardwood/Buttonbush1 WLMP; and Hardwood/Buttonbush2 WLMP, each designed to meet different habitat objectives (Figures H-9 through H-11). During the 50-year project life the site will most often be managed based on the Typical WLMP. However, when habitat conditions require or when water levels are conducive, the site will be drawn down for a longer period and managed according to the Hardwood and Buttonbush WLMPs. For the purposes of the habitat calculations, it is estimated that the Typical (1-year) WLMP will be implemented 30 times, amounting to a total of 30 years, and the 2-year Hardwood and Buttonbush WLMP will be implemented 10 times, amounting to a total of 20 years, during the 50-year project life. A more in-depth discussion of the specific habitat benefits attributed to the different segments of the WLMP can be found in Appendix D.

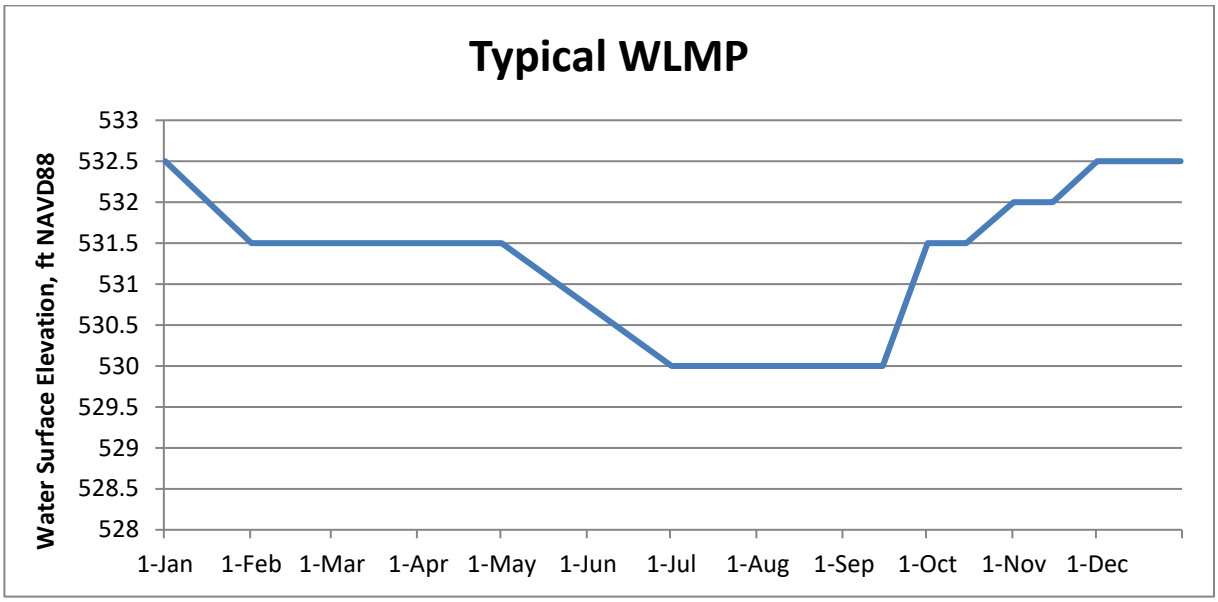


**Figure H-9:** Typical Water Level Management Plan

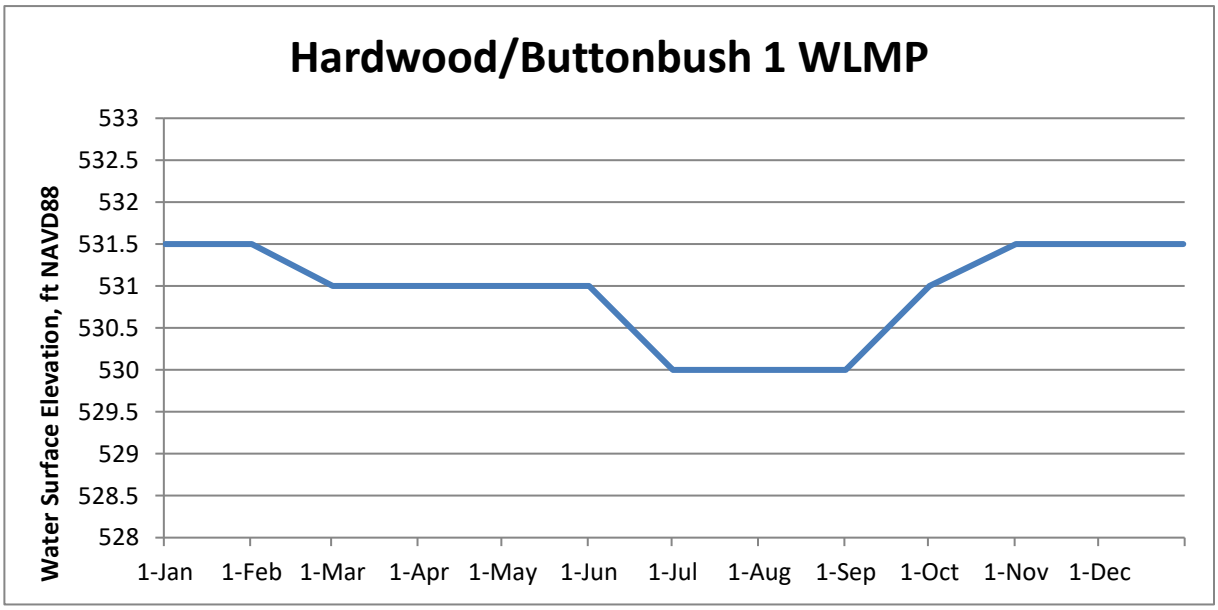


**Figure H-10:** Hardwood/Buttonbush 1 WLMP

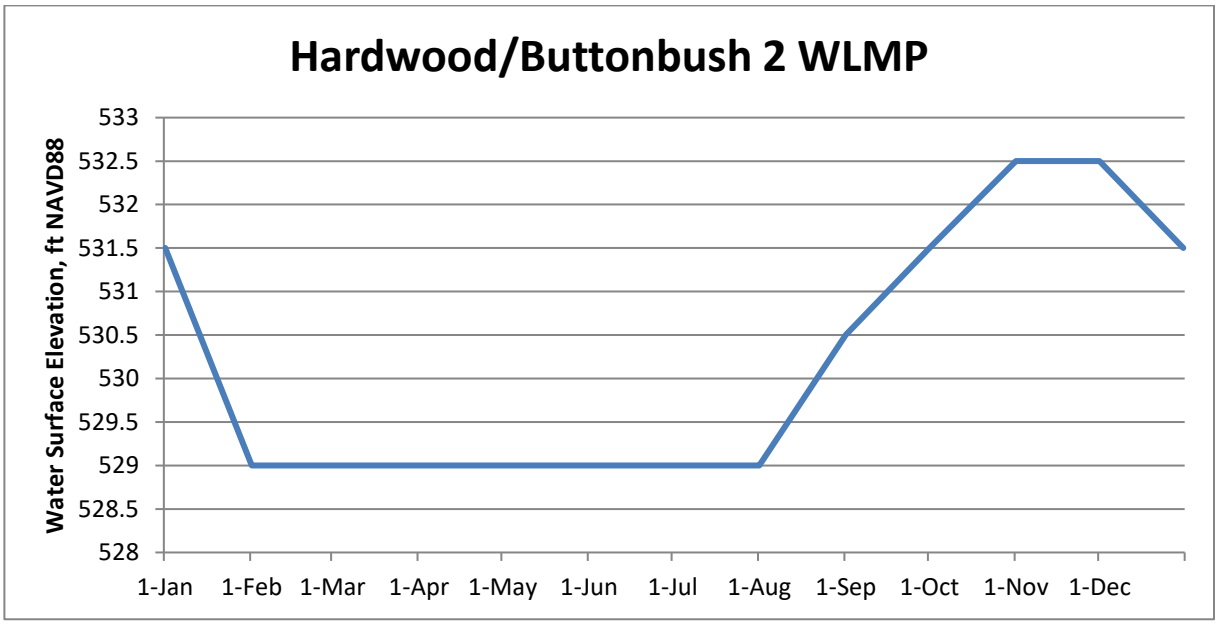


**Figure H-11:** Hardwood/Buttonbush 2 WLMP

**B. Rainfall-Runoff Estimation Using HEC-HMS.** A simplistic hydrologic model was developed using HEC-HMS v. 4.1 to develop a daily rainfall runoff time series to determine runoff entering the Project. The model includes four sub-basins: Spring Slough Watershed; Spring Slough; Keithsburg South Watershed and Keithsburg South; as well as two reservoirs: Spring Slough; and Keithsburg South (Figure H-12). The model was simulated with an hourly timestep from January 1986 through December 2015.

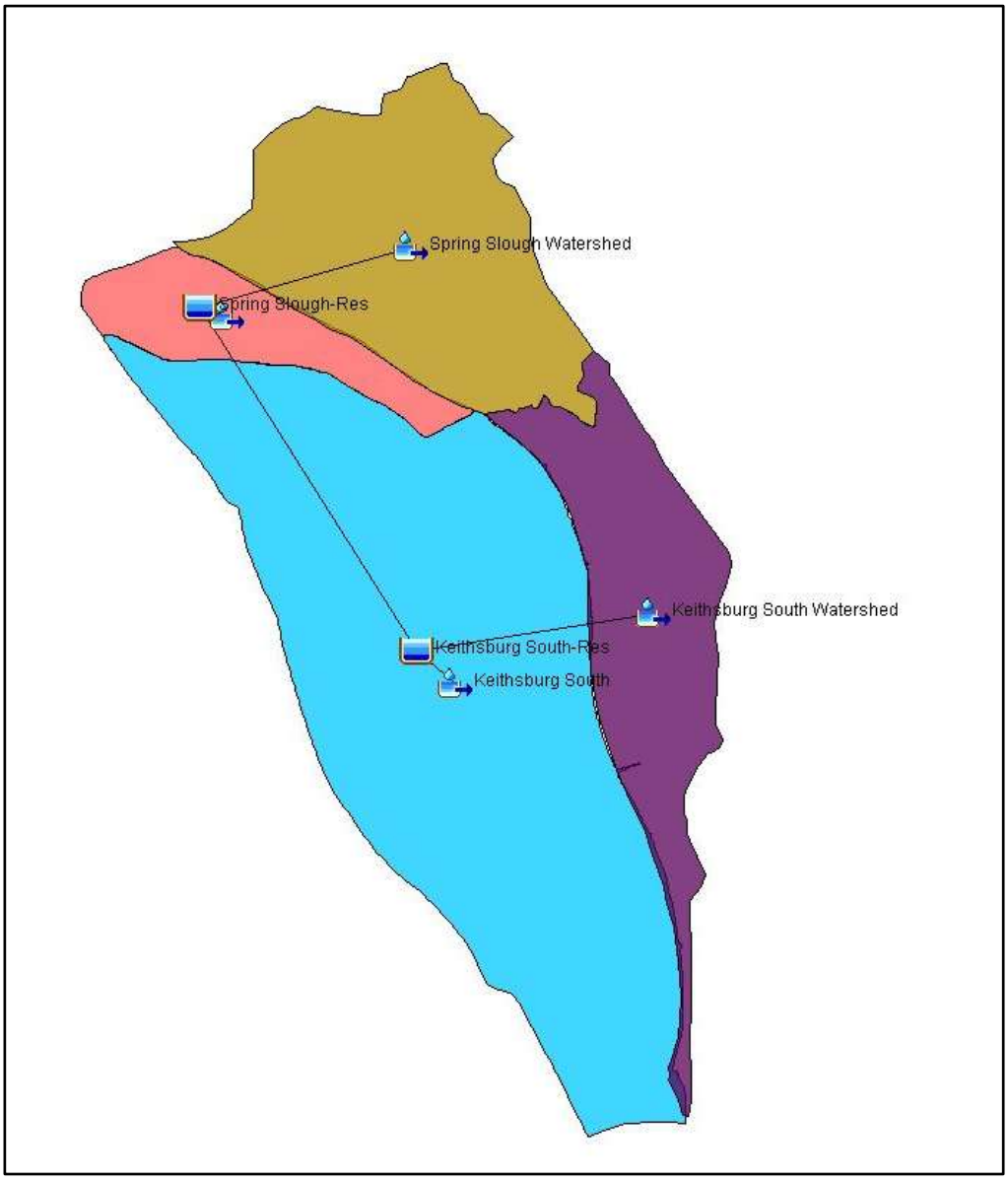


**Figure H-12:** HEC-HMS Sub-basins

**1. Precipitation.** Hourly precipitation data was obtained at the NWS Coop gage 131060 (Burlington 2S, Iowa) for the period of record beginning 1-1-1986 through 12-22-2013. Missing and incomplete monthly data throughout the period were completed using data from the same gage record, based on identifying the year with the most similar total monthly precipitation. The NWS Coop gage 114655 (Keithsburg) was referenced to compare monthly precipitation totals to determine the most representative year for replacing missing values. Precipitation in the HMS meteorologic model was defined based on gage weight, with full gage weight assigned to each of the four subbasins.

**2. Evapotranspiration.** Daily potential evapotranspiration (PET) data for the period from 1-1-2000 to 4-7-2014 was obtained through the Iowa Environmental Mesonet ISU AgClimate Legacy Network for the Muscatine gage. Data for the period from 4-2-14 to 12-31-15 was available through the Iowa Environmental Mesonet ISU Soil Moisture Muscatine gage. These two datasets were used to determine an average daily PET value for each month. These monthly averages were used to populate a full 30-year daily PET dataset, specifically filling in the missing period from January 1986 through December 1999. Evapotranspiration in the HMS meteorologic model was specified based on this daily evapotranspiration time series.

**3. Loss Method.** The deficit and constant loss method was selected.

**4. Canopy Method.** The simple canopy method was applied.

'Flow-Direct' output from the Spring Slough watershed and the Keithsburg South watershed from HEC-HMS were used as lateral inflow boundary conditions to the Spring Slough and Keithsburg South storage areas, respectively, to represent runoff into the Project area within the HEC-RAS model. 'Precip-Excess' output from the Spring Slough subbasin and the Keithsburg South subbasin were used as precipitation boundary conditions for their respective subbasins in HEC-RAS.

**C. Groundwater Inputs.** Drillers' well logs, available through the Illinois State Geological Survey, were used to estimate groundwater flow to the Project. Static and pumping water levels were applied to the Dupuit Equation for unconfined flow. The Dupuit Equation assumes 1) the hydraulic gradient is equal to the slope of the water table and 2) for small water-table gradients, the streamlines are horizontal and the equipotential lines are vertical.

$$q' = \frac{1}{2}K\left(\frac{h_1^2 - h_2^2}{L}\right)$$
$$q' = flow\ per\ unit\ width\ (ft^2/d)$$
$$K = hydraulic\ conductivity\ (ft/d)$$

The location of the two wells (ISGS 72-API 121312157200 and ISGS 58-API 121312181100) which were used for the analysis described below are shown in Figure H-13 (highlighted in yellow). (ISGS 72-API 121312157200 and ISGS 58 Northern-API 121312181100) The westernmost well is located within the NRCS-defined watershed, however the easternmost well is located over 1,500 feet outside of the watershed boundary. For unconfined aquifer flow, the aquifer extent/boundaries are generally approximated by the watershed boundaries. The use of the easternmost well to approximate the groundwater divide may be an underestimate of the hydraulic gradient and therefore could result in an underestimate of groundwater flow to the site.

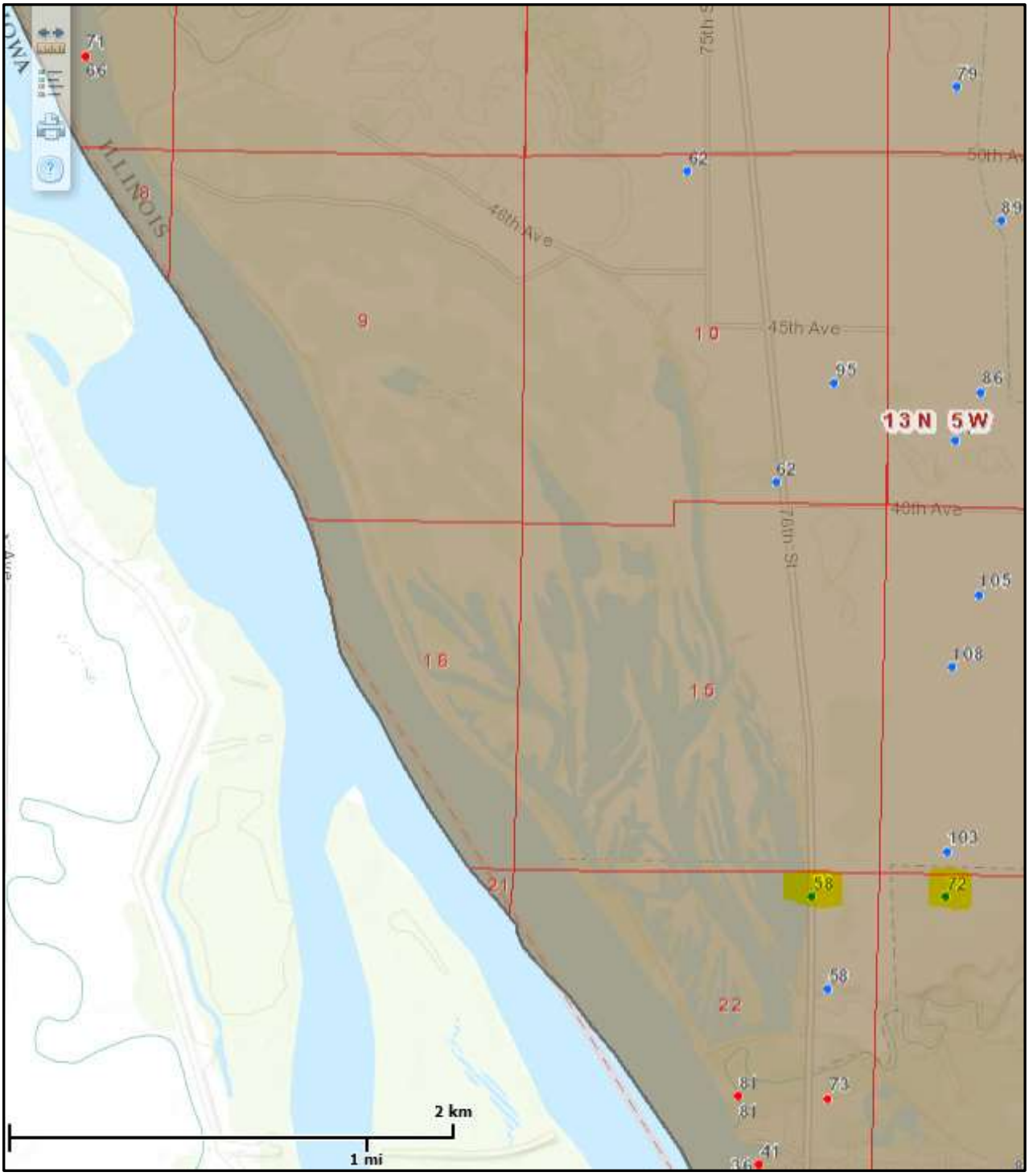


**Figure H-13:** Location of Well Logs Used for Groundwater Flow Estimate

## D. Interior Filling and Draining Simulation Using HEC-RAS

**1. HEC-RAS Model Geometry-Existing Conditions.** An existing unsteady Pool 18 Flow Frequency model was modified for application to the Project analysis. The model was re-projected into State Plane, Illinois West, U.S. Feet and the vertical datum was converted from MSL 1912 to NAVD88. Storage areas were added where the original model cross-sections had been inappropriately extended beyond existing berms to high ground. Three storage areas were created to represent the Project: Northeast Ag Land; Spring Slough; and Keithsburg South (Figure H-14). The storage areas were defined at the 548 feet elevation contour which is well above the maximum managed water surface elevation.

Lateral structures and storage area connections were added to represent segments of the berm, spillways, the Pope Creek breach, roadways and high ground separating the storage areas from the river reach and the storage areas from each other. The perimeter berm profile plates are included in Appendix Q. Figure H-15 illustrates the elevation profile of the interior road which serves as the connection between the Spring Slough and Keithsburg South storage areas.

The existing condition stoplog gate structure is the only control structure for the existing condition model. This gate is a combination of two 4-foot wide stoplog gates, an open top entrance box structure which supports the stoplogs, two 3-foot diameter concrete culverts connecting the entrance box to the river and two fully open and inoperable slide gates on the riverside end of the two culverts. The dimensions of the stoplog structure were extracted from the 1959 design drawings. The elevations on that drawing were assumed to reference NGVD29, based on some recent surveys collected on the structure. The stoplog gate structure was modeled as a small storage area (Stoplog Entrance), a storage area connection, a lateral structure and two culverts. The Pope Creek breach, however, significantly limits the ability of the stoplog gate to provide reliable water level management.

HEC-RAS geometries were created for the two Project Alternatives (Berm and Gates, and Berm, Gates and Pumps) as well. The specific geometry features included for each of these alternatives is discussed in Section XI, *Project Alternatives Evaluated.*

**Figure H-14:** HEC-RAS Model Storage Areas

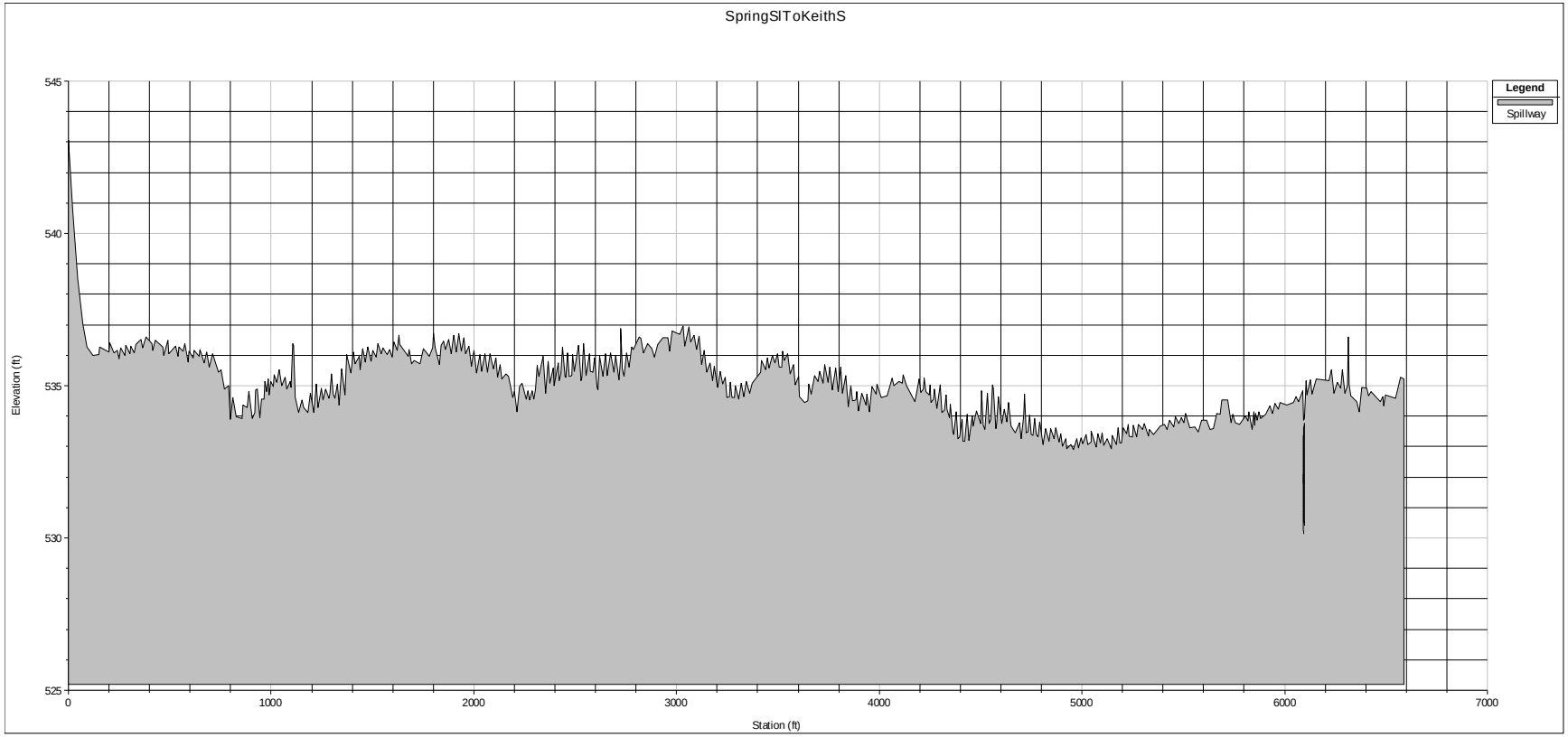


**Figure H-15:** Interior Road/Spring Slough-Keithsburg South Storage Area Connection Elevation Profile

## 2. HEC-RAS Model Boundary Conditions

**a. Discharge and Stage.** A downstream water surface elevation based on the observed stage record at L&D 18 was specified as the downstream boundary condition. The upstream boundary condition is the computed discharge through Lock and Dam 17 as computed by District Water Management personnel. USGS flow records supply lateral inflows to the Mississippi River reach for three tributaries. These inflows represent the Iowa River at Wapello, Iowa entering the Mississippi River at RM 434.4, Edwards River at New Boston, Illinois entering the Mississippi River at RM 431.0 and Pope Creek at Keithsburg, Illinois entering the Mississippi River at RM 427.95. The observed stage record at the Lock and Dam 18 Pool gage is used as the downstream boundary for the model simulations.

**b. Precipitation.** Direct precipitation onto Spring Slough and Keithsburg South storage areas was simulated using a precipitation boundary condition. The precipitation boundary condition daily time series dataset was based on the output from the HEC-HMS model whereby precipitation less evapotranspiration is computed and output in the 'Precip-Excess' file. The 'Precip-Excess' dataset for the Spring Slough HEC-HMS subbasin was applied to the Spring Slough HEC-RAS storage area and the 'Precip-Excess' dataset for the Keithsburg South subbasin was applied to the Keithsburg South HEC-RAS storage area as precipitation boundary condition files.

**c. Watershed Runoff.** Watershed runoff into Spring Slough and Keithsburg South storage areas was simulated using lateral inflow boundary conditions. Lateral inflow boundary condition daily time series datasets were based on the output from the HEC-HMS model whereby runoff computed and output in the 'Flow-Direct' file. The 'Flow-Direct' datasets for the Spring Slough and Keithsburg South HEC-HMS subbasins were applied to their respective HEC-RAS storage areas as lateral inflow boundaries.

**d. Simulation Period and Model Calibration.** A 30-year simulation period (01Jan1986 to 31Dec2015) was chosen in order to provide a long enough record to capture the variability that can reasonably be expected during a 50-year project life. Capturing the variability in river stage and watershed contributions provides a broad range of conditions to evaluate the performance and operational cost of different alternatives in terms of meeting the desired WLMP. The 1986-2015 period also represents the most recent climatic period, whereby an assumption of stationarity is reasonable and there are enough data points to produce a statistically significant sample size. Calibration and verification of the model focused primarily on reproducing observed stages at the Keithsburg Gage, with a secondary focus on reproducing observed stages at the Oquawka, Illinois and Lock and Dam 17 Tailwater Gages. Stage at Lock and Dam 18 was used as the downstream model boundary condition and thus could not provide an independent calibration source. Note that the gage location for Keithsburg changed in 1997 from the Railroad Bridge at RM 428 to a site near the City of Keithsburg, Illinois at RM 427.4. Calibration to interior water levels within the Project was not completed due to limited data and resource constraints. Although, variability of stage reproduction is expected to be low over the 30 years of simulated inflow, the model calibration was focused primarily on reproducing the most recent observed stages. Calibration was accomplished in 5 year periods, with the most weight given to the period between 01Jan2011 to 31Dec2016. Lesser weight was given to each of the preceding 5 year periods. Model parameter adjustments to improve stage reproduction started with the Manning's N-value roughness factors. Additional adjustments were made using Flow-

roughness factors with two application reaches, one being from L&D 17 to the Keithsburg gage and the second from the Keithsburg gage to L&D 18.

**e. Operational Rules and Assumptions.** In order to accurately model the current operation of the existing stoplog structure and intended future operation of proposed sluice gates and pumps, extensive coordination with the sponsor took place. These operational constraints provided by the sponsor were used when developing the operational rules for the existing stoplog structure, and proposed sluice gates and pumps within the model.

Currently the USFWS has no real time interior stage gage. Additionally, the commute from the NWR Office to the Project area requires a bridge crossing at either Muscatine, Iowa or Burlington, Iowa and takes over one hour's time. Considering commute time and additional resource demands, the staff visits the site no more frequently than once every seven days. Therefore, stoplog setting changes simulated by the model can only be made every 7-9 days, defined as an 'actionable day'. A table of actionable days for the 30-year simulation period was defined as part of the rules. A target interior stage table was created to define the target interior stage for each day as defined by the different WLMPs described previously. A highest forecast river stage table and a lowest forecast river stage table were also defined, based on the observed stage record, so that the model could evaluate the river stage forecast prior to making a stoplog or gate setting change. Stoplog gate openings, which are from the top down in 6 inch increments are typically set to the next target interior elevation of the WLMP unless the gates are closed in an attempt to limit backflow into the interior from the Mississippi River. However, based on the construction and current condition of the stoplog structure, water surfaces above elevation 534 feet on either the interior or river sides cannot be controlled by the stoplog gates. A flow chart showing the logic for the stoplog operation is shown in Figure H-16.

The modeled gate configuration consists of two separately located 5 feet wide by 5 feet tall sluice gates, one upstream (RM 430.17) and one downstream (RM 428.12) that can control all gravity flow up to the elevation of the spillway crest. Sluice gates are operated manually on Actionable Days. Both gates are operated equally. As part of the rules set, the gate sluice gate setting is determined as a function of average discharge between actionable days, the average forecasted river stage between actionable days and the average interior stage between actionable days. A more detailed description of the gate design and analysis results are included in Section XI, *Project Alternatives Evaluated*.

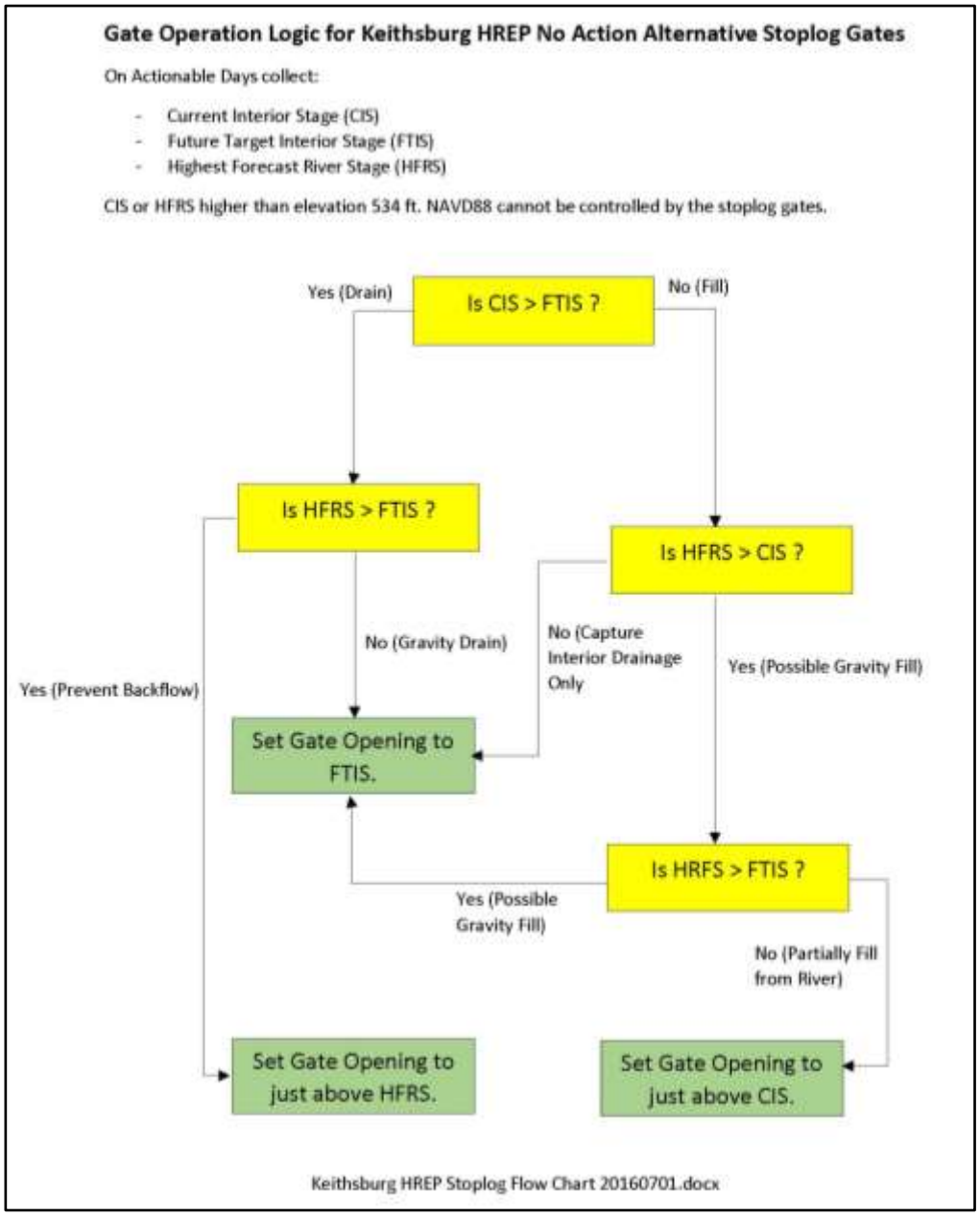


**Figure H-16:** HEC-RAS Logic for Stoplog Operation

The proposed pump configuration consists of two collocated 6,000 GPM pumps, operated independently and capable of pumping water either direction. Pump design information is included in Section XI, *Project Alternatives Evaluated.* Preference is given to gravity drainage whenever possible, therefore both pumps are turned off if the sluice gates are opened. A determination on pumping direction and on whether two pumps, one pump or no pumps are running is made only on actionable days, which occur every 7-9 days throughout the year. Pumps started on actionable days monitor both the interior and river stage and continue pumping between

actionable days to follow the WLMP's interior elevations, similar to what a float system would allow. When pumps are operating they are allowed to turn on and off automatically to maintain the interior stage within +/- 0.2 foot of the WLMP target stage for the current day at every computational time step (10 minutes). This threshold of +/- 0.2 foot prevents the pumps from cycling on for short durations of time, which was observed sporadically during model sensitivity tests. A flow chart illustrating the logic of the proposed sluice gates and pumps is shown in Figure H-17.

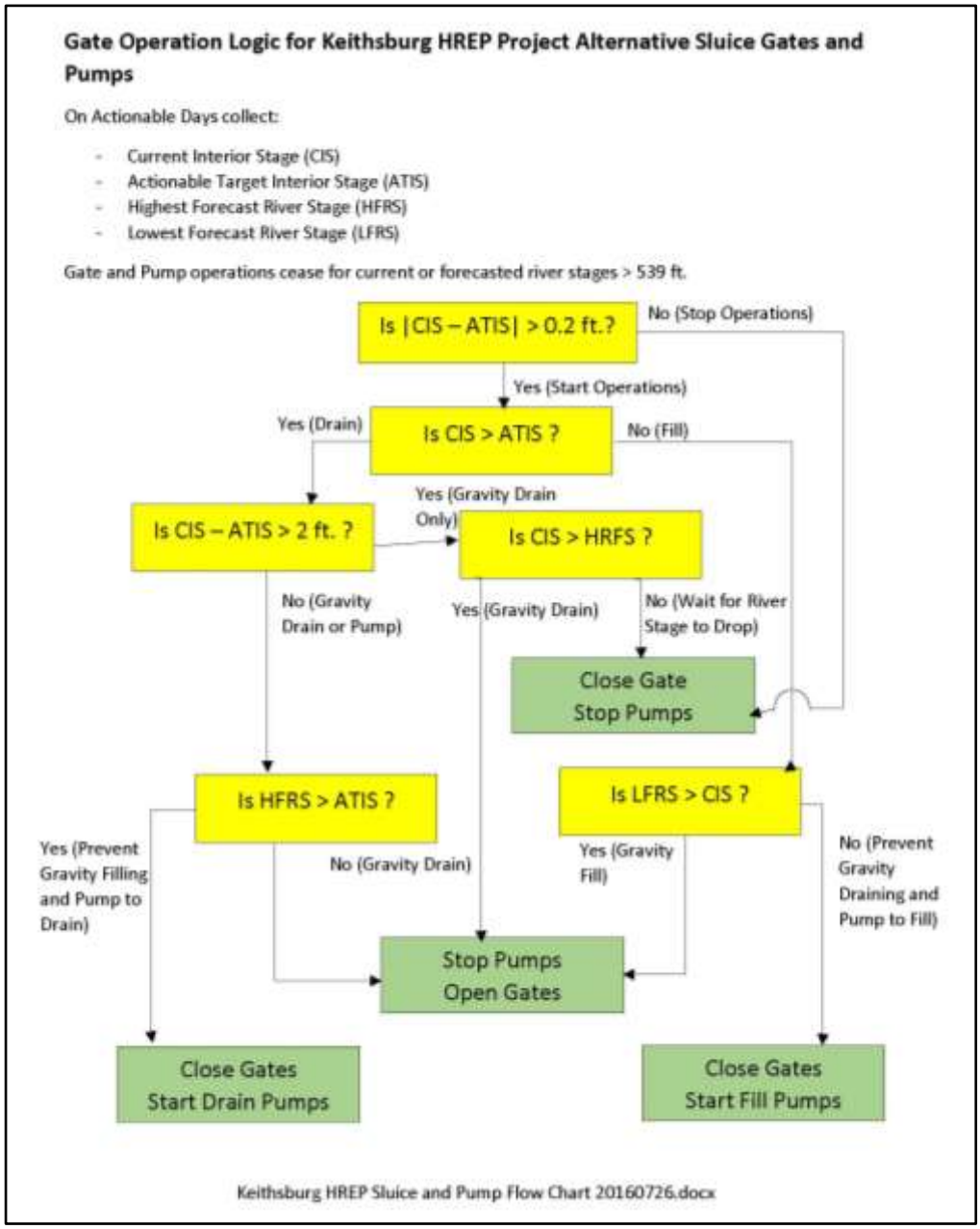


**Figure H-17:** HEC-RAS Logic for Sluice Gates and Pump Operation

## X. PERIMETER BERM AND CONTROLLED OVERTOPPING DESIGN

Hydraulic superiority ensures berm overtopping from the downstream end up in order to minimize damage to the interior during an overtopping event. The existing berm does not provide hydraulic superiority as the initial filling location is the upstream Edwards Spillway. In order to provide hydraulic superiority for controlled overtopping the proposed Project alternatives include the following features/modifications: 1) a set-back berm in lieu of repairing the Pope Creek breach due to real estate constraints; 2) eliminate the Edwards River spillway (currently initial interior filling location); 3) regrade the berm to ensure initial uncontrolled overtopping toward the downstream end of system; and 4) replace existing Mississippi spillway with new reinforced spillway sized to provide interior filling to within one foot of the initial overtopping elevation (Figure H-18).

The initially proposed perimeter berm design grade sloped from a 2% exceedance probability (50-year) elevation at the upstream end down to a 4% exceedance probability (25-year) at the downstream end. The tie-back reaches of the berm along Pope Creek and the Edwards River were sloped at 0.0005 foot/foot based upon an assumed upper limit to water surface profile slope. Proposed water control structures including a pump station and two sluice gate structures necessary to provide interior water level management capability were evaluated and are described in Section XI, *Project Alternatives Evaluated* (Figure H-18). The downstream end of the proposed spillway was to be located 500 feet upstream of the proposed pump station and adjacent downstream sluice gate structure. The pump station and adjacent downstream sluice gate structure were located 500 feet upstream of the southwest corner of the berm.

Downstream of the lower spillway, the initial berm design elevation followed the 4% exceedance probability (25-year) water surface profile to the southwest corner. Beginning at the upstream end of the lower spillway the berm sloped from a 4% exceedance probability (25-year) water surface elevation design grade up to a 2% exceedance probability (50-year) water surface elevation design grade at the northwest corner of the berm. The upstream sluice gate structure was located nearly 800 feet upstream of the upstream spillway. In accordance with DIVR 1110-1-15, a 2-foot berm superiority surrounding the spillway structures and gate and pump station structures extended for 100 feet upstream and downstream. The minimum elevation along the perimeter berm (542.23) was located at the southwest corner. This elevation is 0.02’ above the 25-year elevation, but was fixed based on the interior filling/spillway design analysis. Under this preliminary berm design profile, initial berm overtopping was assumed to occur at some location either between the downstream spillway and gate/pump structure or between the gate/pump structure and the southwest corner of the berm. Table H-9 provides a summary of these initial design berm feature locations and elevations.

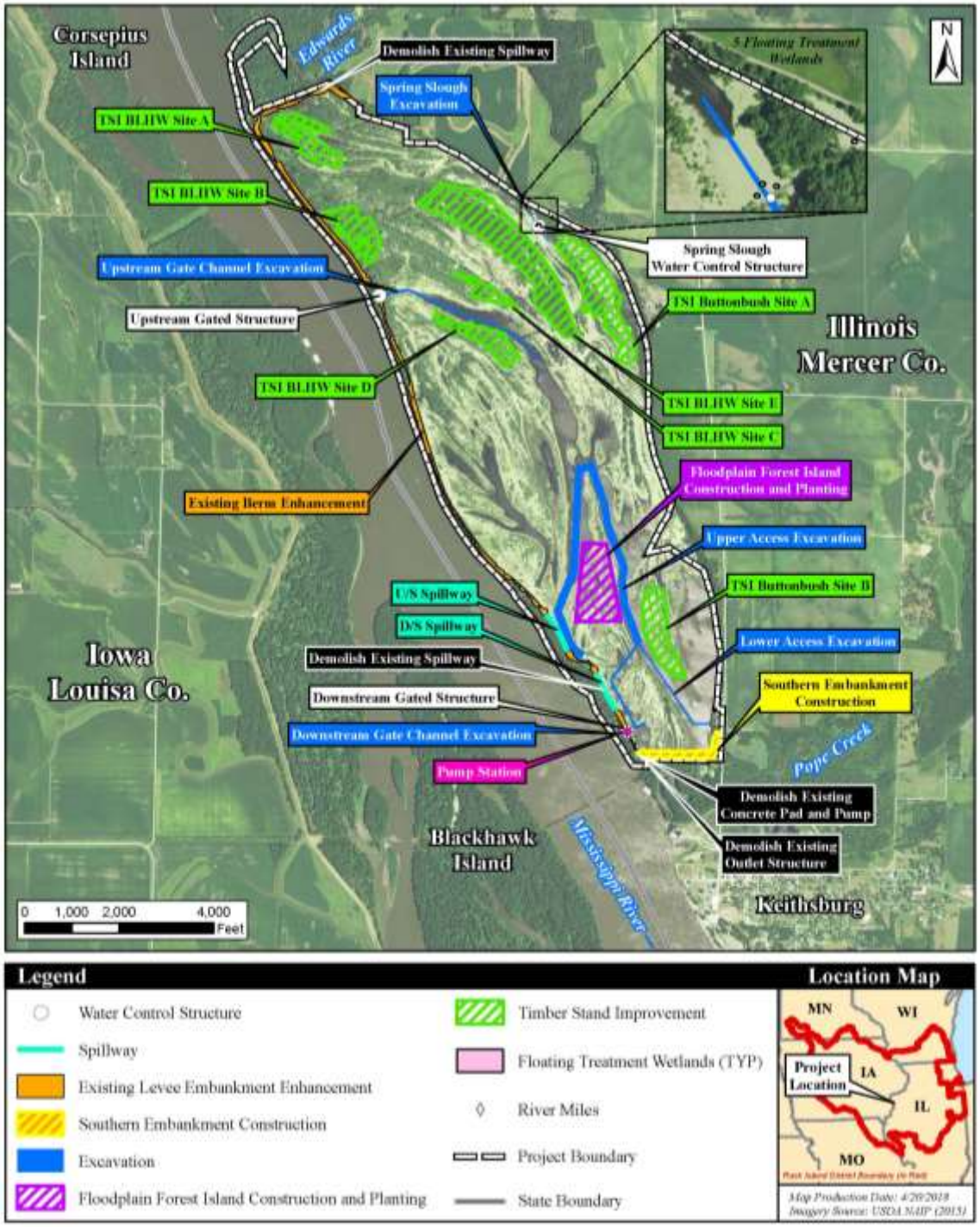


**Figure H-18:** Evaluated Project Features

**Table H-9:** Summary of Initially Proposed Design Berm Features

| Design Berm Feature | RM | Elevation NAVD88 | Design Criteria |
|---|---|---|---|
| Tie-back Berm (NE corner tie-in) | 431.07 | 546.03 | 0.0005 ft/ft slope along Edwards River |
| US Perimeter Berm (NW corner) | 431.07 | 545.03 | 2% EP (50-yr) elevation |
| US Gates | 430.17 | 546.18 | 2%/4% EP (50/25-yr) interpolated elevation +2' Superiority |
| US SpillwayTie-in | 428.67 | 544.74 | 2%/4% EP (50/25-yr) interpolated elevation +2' Superiority |
| US 900' Spillway | 428.67 | 540.00 | 1' overtopping differential w/ 2.42 ft/day rate of rise, initial interior at 529' and overtopping elev. of 542.23 |
| US 900' Spillway | 428.51 | 540.00 | 1' overtopping differential w/ 2.42 ft/day rate of rise, initial interior at 529' and overtopping elev. of 542.23 |
| US SpillwayTie-in | 428.5 | 544.58 | 2%/4% EP (50/25-yr) interpolated elevation +2' Superiority |
| DS Spillway Tie-in | 428.38 | 544.47 | 2%/4% EP (50/25-yr) interpolated elevation +2' Superiority |
| DS 900' Spillway | 428.38 | 540.00 | 1' overtopping differential w/ 2.42 ft/day rate of rise, initial interior at 529' and overtopping elev. of 542.23 |
| DS 900' Spillway | 428.21 | 540.00 | 1' overtopping differential w/ 2.42 ft/day rate of rise, initial interior at 529' and overtopping elev. of 542.23 |
| DS Spillway Tie-in | 428.2 | 544.30 | 4% EP (25-yr) + 2' Superiority |
| DS Gates | 428.12 | 544.27 | 4% EP (25-yr) + 2' Superiority |
| DS Perimeter Berm (SW corner) | 428.03 | 542.23 | ~4% EP (25-yr) (min. elev. used in spillway design) |
| Tie-back Berm (SE tie-in) | 427.97 | 543.10 | 0.0005 ft/ft slope along Pope Creek |

The design elevation of the proposed Mississippi River spillways were based on input from the sponsor indicating their preference not to decrease the system's existing level of protection. In order to determine the spillway size/capacity necessary to fill the interior to within the required overtopping differential, prior to the perimeter berm overtopping, a filling model was developed using excel. Assumptions of the filling model included: (1) a spillway crest elevation; (2) a rate of rise of the Mississippi River; (3) an initial interior water surface elevation; and (4) a 2-foot clay cap along the berm crest. The spillway crest was initially set at 540 feet NAVD88 (nearly a 14% exceedance probability or a 7-year return interval at RM 428.67). (This is an improvement over the existing Edwards' Spillway which has a 27% annual exceedance probability.) Conservative values were chosen for rate of rise and the initial interior water surface elevation in order to allow for a conservative analysis. The rate of rise for the Mississippi River of 2.42 feet/day was assumed based on analysis of historic rates of rise for events between the 20% and 10% exceedance probability elevations. The 20% to 10% exceedance probability corresponds to the frequency of an event that would potentially result in spillway flow. The maximum rate of rise observed during the historical gage record is 2.42 feet/day which occurred during the June 2008 flood event. The 2008 flooding impacted much of eastern Iowa and other areas throughout the Midwest and set many records. An initial interior elevation of 529 feet was assumed, based upon the minimum managed interior water level associated with the Hardwood/Buttonbush WLMP. These constraints resulted in two-900-foot long spillways, required to fill the interior to within 1 foot of the initial overtopping elevation prior to overtopping. A similar berm design with a sand core, vegetated 2-foot clay cap and controlled overtopping with a 1-foot head differential is located 4 miles upstream and has performed without damage during overtopping events. The conservative assumptions used for this design limit the risk of overtopping differentials greater than 1-foot and the demonstrated success of a similar berm illustrates the limited risk for damages.

The initially proposed design, as described above, was evaluated for floodplain impacts and the results indicated the berm produced an increase to the 1% water surface profile. Revisions to the berm design grade were made to ensure the Project would not result in floodplain impacts (Table H-10). Design changes that were incorporated into the final feasibility design in order to reduce the floodplain impacts include the following:

1. the berm design elevation slopes from a 4% exceedance probability design elevation at the northwest corner to a 4% exceedance probability minus 1-foot design elevation upstream of the downstream spillway;
2. downstream of the downstream spillway the berm design elevation will follow the 4% exceedance probability elevation as in the original design.

Additional revisions made to the initial feasibility design in order to ensure hydraulic superiority that must be re-evaluated for floodplain impacts during plans and specifications include:

3. the entire berm reach between the upstream and downstream spillways will maintain the 2-foot berm superiority;
4. the location of initial perimeter berm overtopping is now upstream of the upstream spillway (Sta.144+73);
5. the spillway crest elevation has been lowered to 539.5 feet (approximately a 17% exceedance probability event or 6 year return interval) in order to meet the uncontrolled overtopping criteria as described in the previous paragraph.

**Table H-10:** Summary of Final Feasibility Design Berm Features

| Design Berm Feature | RM | Elevation NAVD88 | Design Criteria |
|---|---|---|---|
| Tie-back Berm (NE corner tie-in) | 431.07 | 544.82 | 0.0005 ft/ft slope along Edwards River |
| US Perimeter Berm (NW corner) | 431.07 | 543.82 | 4% EP (25-yr) elevation |
| US Gates | 430.17 | 545.03 | 4% EP/4% EP-1' (25/25-yr-1') interpolated elevation +2' Superiority |
| US Spillway Tie-in | 428.67 | 543.71 | 4% EP/4% EP-1' (25/25-yr-1') interpolated elevation +2' Superiority |
| US 900' Spillway | 428.67 | 539.5 | 1' overtopping differential w/ 2.42 ft/day rate of rise, initial interior at 529' and overtopping elev. of 541.73 |
| US 900' Spillway | 428.51 | 539.5 | 1' overtopping differential w/ 2.42 ft/day rate of rise, initial interior at 529' and overtopping elev. of 541.73 |
| US Spillway Tie-in | 428.5 | 543.56 | 4% EP/4% EP-1' (25/25-yr-1') interpolated elevation +2' Superiority |
| DS Spillway Tie-in | 428.38 | 543.45 | 4% EP/4% EP-1' (25/25-yr-1') interpolated elevation +2' Superiority |
| DS 900' Spillway | 428.38 | 539.5 | 1' overtopping differential w/ 2.42 ft/day rate of rise, initial interior at 529' and overtopping elev. of 541.73 |
| DS 900' Spillway | 428.21 | 539.5 | 1' overtopping differential w/ 2.42 ft/day rate of rise, initial interior at 529' and overtopping elev. of 541.73 |
| DS Spillway Tie-in | 428.2 | 544.30 | 4% EP (25-yr) + 2' Superiority |
| DS Gates | 428.12 | 544.27 | 4% EP (25-yr) + 2' Superiority |
| DS Perimeter Berm (SW corner) | 428.03 | 542.23 | ~4% EP (25-yr) (min. elev. used in spillway design) |
| Tie-back Berm (SE tie-in) | 427.97 | 543.10 | 0.0005 ft/ft slope along Pope Creek |

This revised design analysis assumes that when the river reaches a stage of 539.5' initiating spillway flow, while the interior is held at its minimum managed elevation of 529'. The Mississippi River stage is assumed to continue rising at a rate of 2.42 ft/day. In order to fill the interior to within 1' of the initial overtopping elevation (541.73') at Sta. 144+73, the spillways were sized to fill ~558 million ft^3 (529' to 540.73') within a 32 hour period. Results of the filling model indicate that when the Mississippi River reaches an elevation of 541.73, the interior water surface elevation will be 541' (0.73' overtopping differential). The maximum spillway capacity (~18,000 cfs) occurs just prior to uncontrolled overtopping. Once berm overtopping occurs, the overtopping differential is eliminated in less than one hour. Initial overtopping begins at Sta. 144+73 (upstream of the upstream spillway) and progresses upstream toward the northwest corner of the berm. The storage volume-elevation curve for the Project interior was determined using a GIS terrain developed based on conventional survey and Mercer County LiDAR. The storage volume-elevation curve was defined using 0.25' resolution (Figure H-19). As previously mentioned, these conservative assumptions used for design analysis support a limited risk for exceeding the 1' overtopping differential.

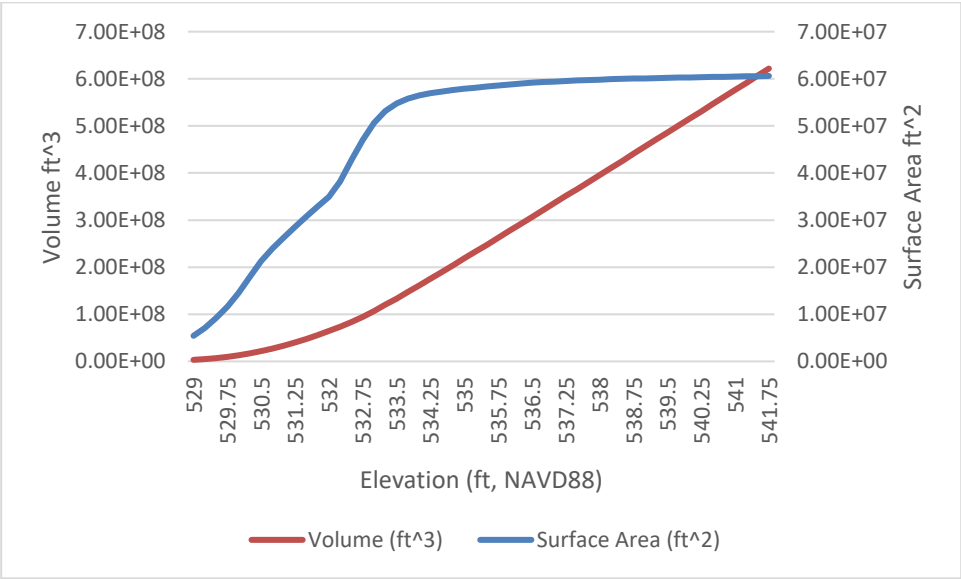


**Figure H-19:** Interior Storage-Volume Elevation Curve

The increased frequency of overtopping (from 14% to 17% annual exceedance probability) due to lowering the spillway elevation by 0.5 foot was not considered by the PDT to be a detriment to project success. Periodic overtopping simulates the natural flooding process that is essential to the habitat and therefore desirable. The PDT adopted this revised design for the feasibility design and cost estimate with the exception of the 2-foot berm superiority that occurs between the upstream and downstream spillways. During plans and specifications, the design incorporating all revisions as described above will be evaluated for floodplain impacts and the controlled overtopping analysis will be verified with the filling spreadsheet model or using an HEC-RAS model to ensure the final design results in no impacts to the 1% exceedance probability flood profile and that controlled overtopping with a maximum 1' overtopping differential is met. Appendix Q includes drawings of the existing conditions berm profile and the feasibility design profile.

## XI. PROJECT ALTERNATIVES EVALUATED

**A. Existing Conditions Alternative.** In the main report this alternative corresponds to A30. Interior water levels within the Project were simulated over the 30-year period (1986-2015) in HEC-RAS to determine how successful the existing infrastructure would be in achieving the desired water level management plans (Plates 1-13). The existing conditions HEC-RAS model contains the perimeter berm features including the Pope Creek breach, Edwards River Spillway, Mississippi Spillway and the stoplog structure. As would be expected, the Pope Creek Breach significantly limits the ability to control interior water levels. Additionally river stages above 534 feet cannot be controlled by the stoplog gates as explained previously. These physical limitations prevent the existing project from meeting the desired water level management plan. Figure H-20 illustrates the simulated interior water level during 1998 relative to the desired typical WLMP. The stoplog elevation is shown by the black line. As demonstrated, without pumping it is difficult to maintain interior water levels during the winter and the berm breach further inhibits the ability to prevent inflow from rising river stages.

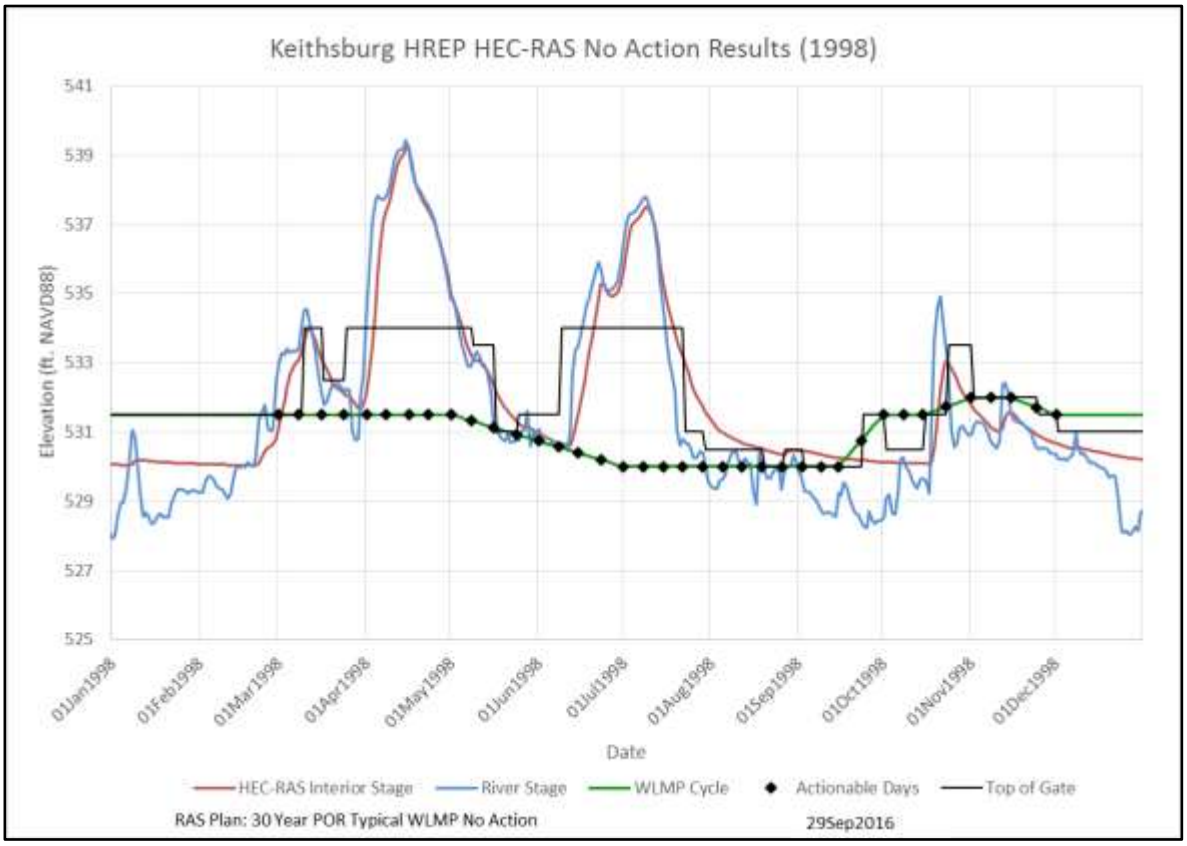


**Figure H-20:** HEC-RAS Results for 1998 Under Existing Conditions Alternative and the Typical WLMP

Figure H-21 illustrates the simulated interior water level during the 1998-1999 period relative to the desired hardwood WLMP. HEC-RAS results for the full 30-year period under both the typical and hardwood WLMPs are shown in Plates 1-12.

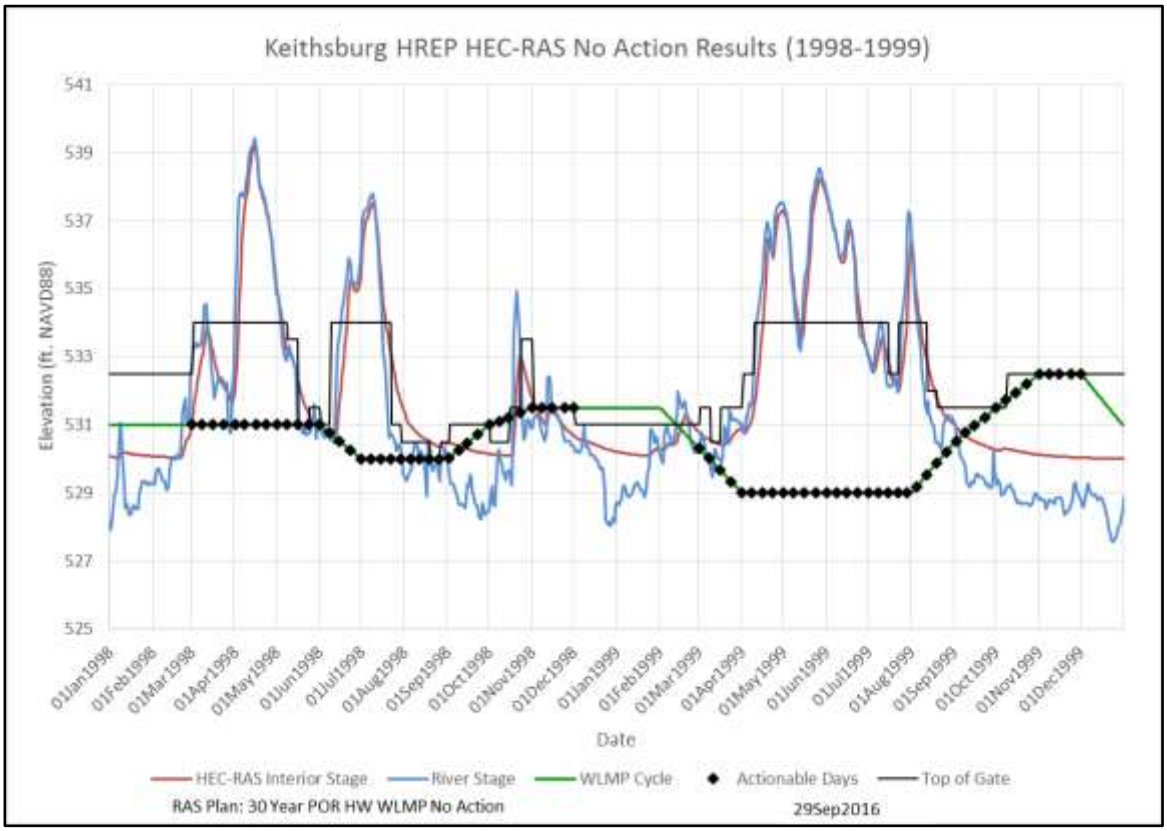


**Figure H-21:** HEC-RAS Results for 1998-1999 Under Existing Conditions Alternative and the Hardwood WLMP

**B. Berm Improvements with Gates Alternative.** In the main report this alternative corresponds to alternatives A2, A18, A20, A21, A22, A36, and A37. The geometry for this alternative was added to the HEC-RAS model and similarly, interior water levels were simulated for the 30-year period (1986-2015). The Berm Improvements with Gates Alternative includes many of the existing conditions HEC-RAS geometry features in addition to the berm improvements as described in Section X, *Perimeter Berm and Controlled Overtopping Design*, elimination of the stoplog structure and replacement with two sluice gate structures; one at an upstream location (RM 430.17) and one at a downstream location (RM 428.12). Figure H-18 illustrates all of the evaluated Project features.

The HEC-RAS model includes one 5 foot x 5 foot sluice gate at each location. The downstream sluice gate was located near the existing stoplog gate to utilize existing natural drainage (from up-river to down-river within the complex) while the upstream sluice gate was located near an existing channel feature, where it can take advantage of higher Mississippi water surfaces during filling periods. Gate sizing for this initial design was determined based on the capacity necessary to meet the idealized WLMP requirements (Figures H-9, H-10, and H-11). A combined storage volume-elevation relationship for Keithsburg South and Spring Slough was used to determine volumetric change requirements necessary to achieve the WLMPs-specified water surface elevation on a bi-monthly basis. The volumetric calculations were determined without consideration of surface water gains or losses. The fully submerged Orifice equation was used to compute the number of days necessary to

pass the required volume of water and then evaluated against the duration specified by the WLMPs. Gate width (5') and height (5') were the design variables in this analysis and initial design assumptions regarding sill elevation, gate opening, tailwater and headwater were based on PDT input and engineering judgment. The sluice gate design analysis assumptions and rationale used are summarized in Table H-11. Results from this conservative gate sizing analysis indicated that a single 5 foot x 5 foot sluice gate has the capacity to gravity fill and drain the interior as specified by the WLMPs, assuming river levels are conducive.

**Table H-11:** Gate Sizing Analysis Assumptions

| Assumption | Value | Rationale/Notes |
|---|---|---|
| ***Gravity Analysis for WLMP Operations*** | | |
| Gate Sill Elevation | 528 ft/ 523.5 ft | The initial sill elevation of 528' was included in the HEC-RAS model, however the final feasibility design was revised down to 523.5 ft. |
| Gate Width | 5 ft | The initial design included 1-5'x 5' gate at each location, as was modeled in HEC-RAS. However the final feasibility design includes 2-5'x 5' gates at the upstream location and additional use of the pump station 5' x 5' intake channel at the downstream location. |
| Gate Height | 5 ft | The initial design included 1-5'x 5' gate at each location, as was modeled in HEC-RAS. However the final feasibility design includes 2-5'x 5' gates at the upstream location and additional use of the pump station 5' x 5' intake channel at the downstream location. |
| WLMP Draining Tailwater Elevation | varies | Conservative assumptions were made for each WLMP draining action, whereby the assumed tailwater elevation was 0.1' below the target interior (the maximum that would still allow for meeting the target exclusively through gravity drainage). |
| WLMP Filling Headwater Elevation | varies | Conservative assumptions were made for each WLMP filling action, whereby the assumed headwater elevation was 0.1' above the target interior (the minimum that would still allow for meeting the target exclusively through gravity drainage). |
| Gate Opening | 1 ft | The sluice gate was set 1' above the gate sill. |
| WLMP Discharge Coefficient | (0.75-0.8) | Chatterton's formula for discharge coefficient. |
| ***Gravity Analysis for Interior Surge Drawdown*** | | |
| Gate Sill Elevation | 528 ft/ 523.5 ft | |
| Gate Width | 5 ft | The final feasibility design includes 2-5'x 5' gates at the upstream location and additional use of the pump station 5' x 5' intake channel at the downstream location. |
| Gate Height | 5 ft | The final feasibility design includes 2-5'x 5' gates at the upstream location and additional use of the pump station 5' x 5' intake channel at the downstream location. |
| Surge Drawdown Interior (Headwater) Elevation | 531 ft | Average interior water surface elevation during drawdown action. 532' to 530' drawdown assumed to represent a late growing season runoff event. |
| Surge Drawdown River (Tailwater) Elevation | 529.5 ft | Tailwater river levels were assumed to be 0.5' below the interior drawdown target (530'), which also approximates the median late growing season stage. |
| Gate Opening | 3-4 ft | Gate opening was varied as a function of number of gates at each location. |
| Surge Drawdown Discharge Coefficient | 0.69 | Chatterton's formula for discharge coefficient. |

In addition to simply meeting the needs of the WLMPs, sluice gate capacity was also evaluated for its ability to rapidly drawdown the interior following a significant runoff event during the growing season (2-foot surge in interior water surface elevation) so to avoid impacts to vegetation (Table H-11). These results indicated that in order to drain an interior 2-foot surge within a 2-day period during the growing season (as specified by the sponsor) 2-5 foot by 5 foot sluice gates are required at each the upstream and downstream locations. The 2-foot surge was assumed to occur late during the growing season when the interior would need to be drained from 532 feet to 530 feet. A river elevation of 529.5 ft was assumed, which approximates the median late growing season stage.

Note that the sluice gate sizing analyses described above assumed a preliminary sill elevation design of 528 ft, however the final design (Recommended Plan) includes a gate sill elevation of 523.5 feet necessary to maintain adequate submergence for pump operation. The gate sizing calculations were revised with the (523.5 feet) sill elevation change to verify that the gate dimensions (5 feet x 5 feet) and total number of gates (4) were still able to provide adequate drainage under routine and storm runoff event management conditions. Gate openings were varied to ensure that drainage goals, as specified by the sponsor, were met. The results verified that the final feasibility design for gravity drainage capacity at both sites was adequate. Knowing that the gate design changes are not expected to significantly impact routine WLMP operation, HEC-RAS modeling which was completed using the preliminary design (sill elevation of 528 feet and only one 5 foot x 5 foot sluice gate at each location), was not updated with the final feasibility design changes.

The sluice gates can control all gravity flow in and out of the Project up to the spillway crest elevation. The HEC-RAS geometry used a spillway crest elevation at 540 feet. Late in the feasibility design, the spillway elevation was changed to 539.5 feet. Figures H-22 and H-23 illustrate the simulated interior water level during 1998 under the Berm Improvements with Gates Alternative relative to the desired typical WLMP and hardwood WLMP, respectively. The sluice gate elevation setting is shown by the black line. Elimination of the Pope Creek Breach provides significantly more control over interior water levels; however, even with the new sluice gates, exclusive gravity draining and filling capability relies on river stages, which often are too high to drain or too low to fill. The plotted river stage data is from the upstream gate location which shows a peak stage of 540.72 feet (just above the spillway elevation) during the April flooding event. However the river stage near the spillway, downstream of the plotted stage, is below the 540 feet spillway crest location and therefore the spillway is not overtopped.

The results of the 30-year HEC-RAS simulations under the Berm Improvements with Gates Alternative are shown in Plates 13 through 24.

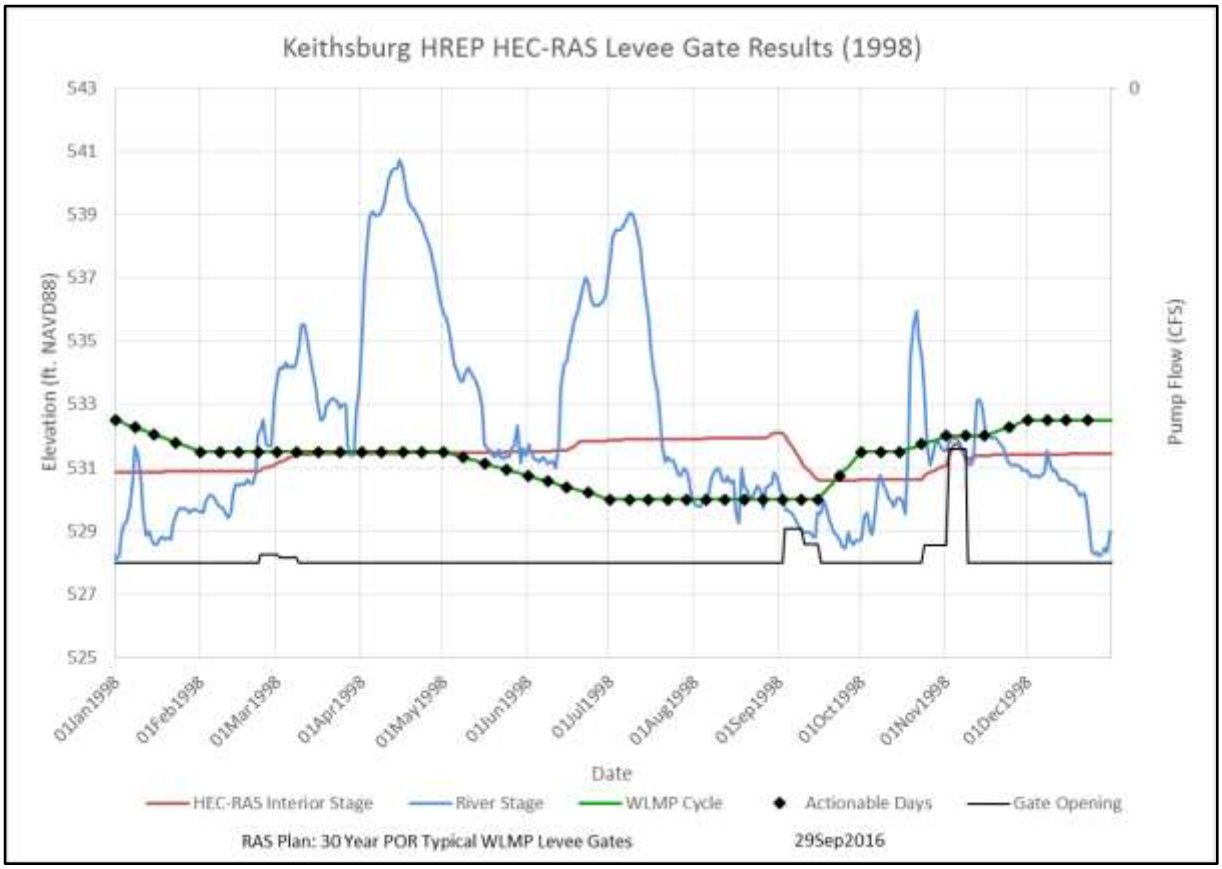


**Figure H-22:** HEC-RAS Results for 1998 Under the Berm Improvements with Gates Alternative and the Typical WLMP

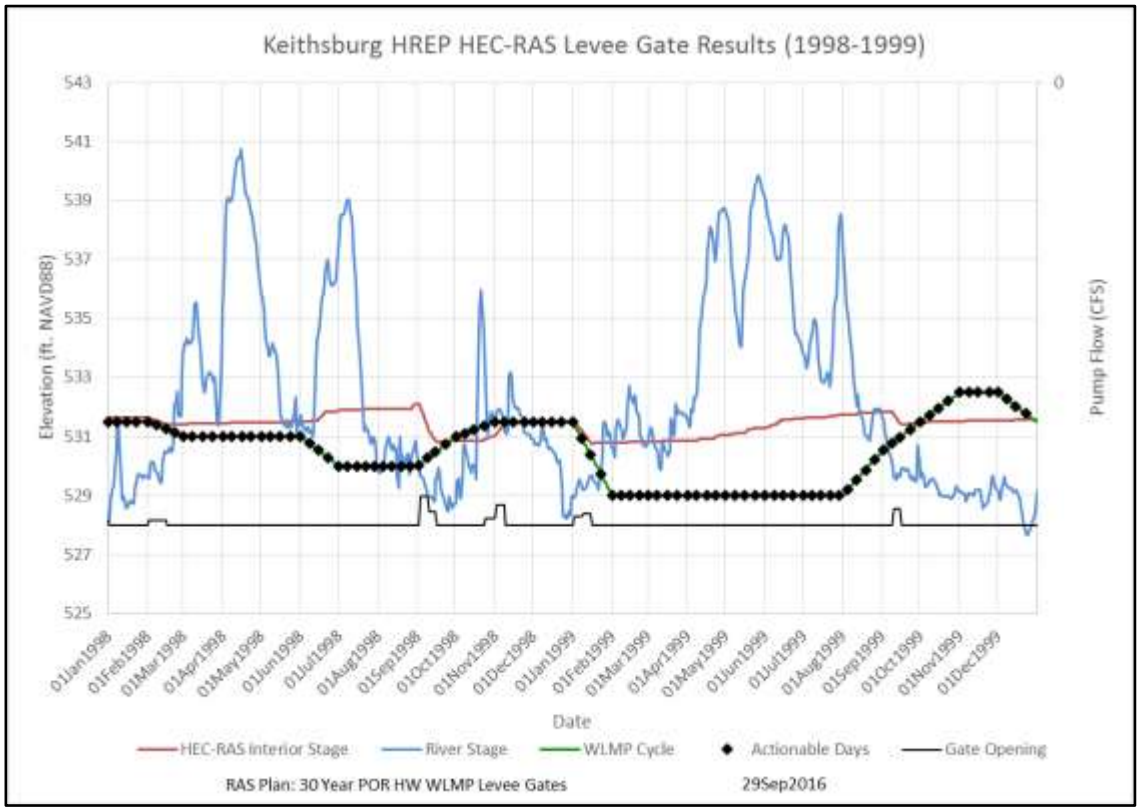


**Figure H-23:** HEC-RAS Results for 1998-1999 Under the Berm Improvements with Gates Alternative and the Hardwood WLMP

**C. Berm Improvements with Gates and Pumps Alternative.** This alternative corresponds to alternatives A3, A6, A7, A10, A11, A31, and A32. The Berm Improvements with Gates and Pumps Alternative geometry was added to the HEC-RAS model and similarly, interior water levels were simulated for the 30-year period (1986-2015). This alternative includes all of the HEC-RAS geometry features in the Berm Improvements with Gates Alternative plus a 12,000 GPM capacity pump station. The pump station includes 2-6K GPM pumps that are operated independently and capable of pumping in both directions.

Similar to the gate sizing analysis, pump capacity was determined based upon the capacity required to meet the idealized WLMP (Figures H-9, H-10, and H-11). The storage volume-elevation relationship was used to compute volumetric changes between each inflection point on the WLMPs. Volumetric calculations were determined without consideration of surface water gains or losses. Pump capacity necessary to follow the WLMP was computed based on the incremental volume changes and associated durations, as specified by the WLMP, assuming exclusive pumping (no gate augmentation). In addition to evaluating the WLMPs, the pump capacity analysis also considered the extremes in the operational flexibility range, as provided by the sponsor, which account for the greatest pumping demands. The resulting pump capacity required by each incremental water level management change ranges from ~2.5K GPM to ~12.4K GPM. Two-6K GPM fixed rate pumps were specified in order to provide the necessary operational flexibility within the WLMP the sponsor indicated would be implemented based on river levels and weather (Figures H-9, H-10, and H-11). Additionally, smaller

pump sizes facilitate more gradual water level changes as specified in the WLMP. As stated in the Mechanical Engineering Appendix, the pumping units will be operated using a Programmable Logic Controller (PLC)/Human Machine Interface control utilizing a pressure transducer to determine water level. This will allow the pump station to operate automatically. Additionally, the sponsor will have remote monitoring capabilities.

As previously described in the *Operational Rules and Assumptions* section, preference is always given to gravity drainage when river stages allow and the pumps are turned off whenever the gates are open. Details of the pump operation are also provided in the *Operational Rules and Assumptions* section. Figures H-24 and H-25 illustrate the simulated interior water level and required gate and pump operation during 1998, relative to the desired typical WLMP and hardwood WLMP. As shown, sluice gates with the addition of pumps offer greater control of the interior stages than both the existing stoplog gates and the sluice gates alone. As discussed above, the sluice gate elevation setting is shown by the black line. The vertical axis on the right side of the graph shows the pump flow computed by the model and the orange line indicates pumping out of (draining) the interior and a dark blue line indicates pumping into (filling) the interior. The results of the 30-year HEC-RAS simulations under Berm Improvements with Gates and Pumps Alternative are shown in Plates 25-36.

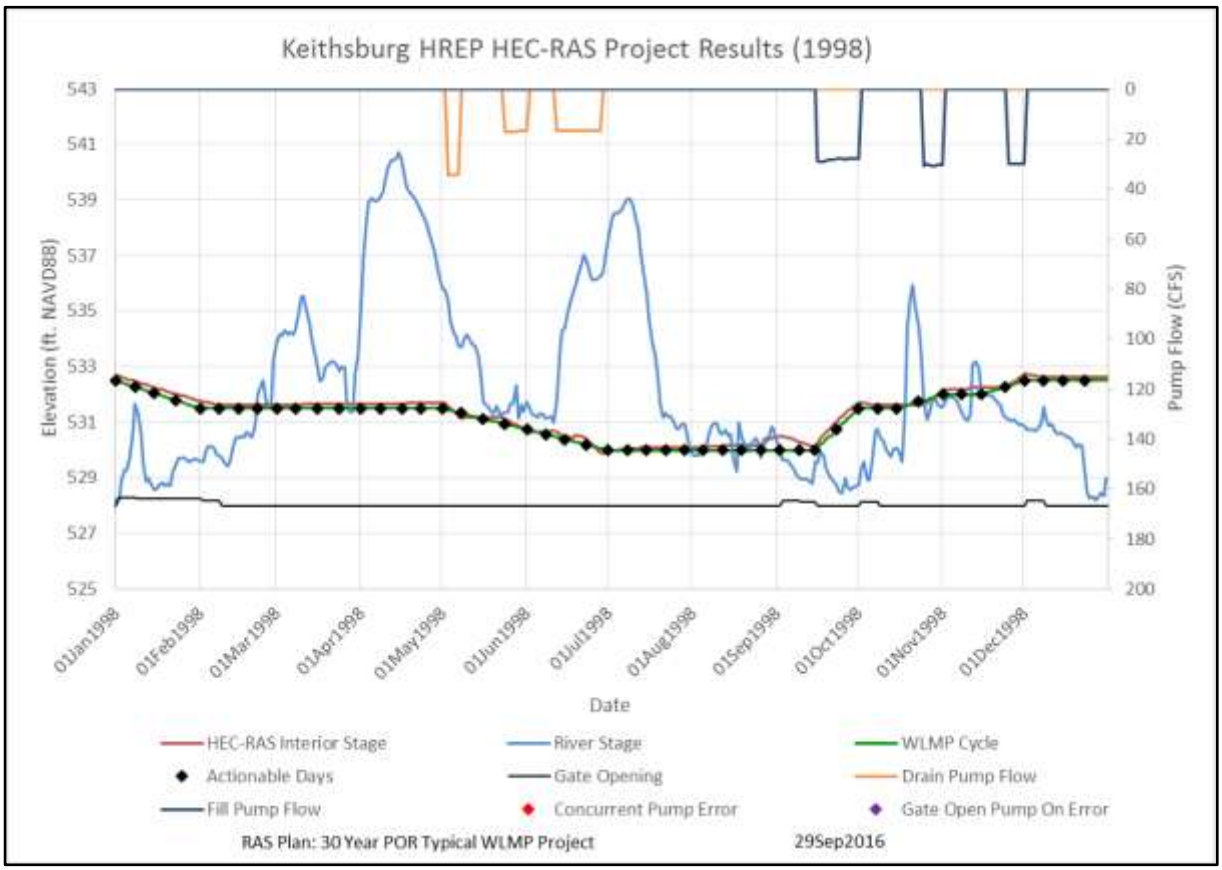


**Figure H-24:** HEC-RAS Results for 1998 Under the Berm Improvements with Gates and Pumps Alternative and the Typical WLMP

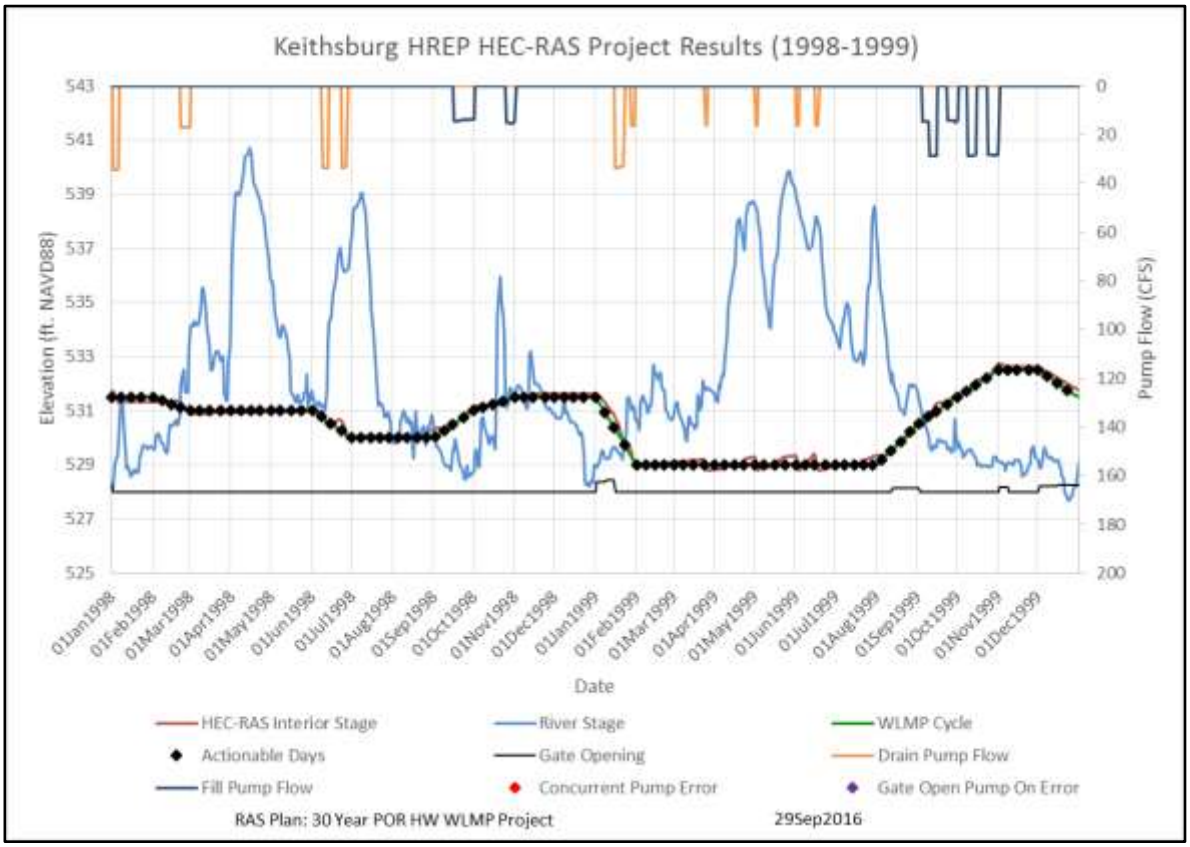


**Figure H-25:** HEC-RAS Results for 1998-1999 Under the Berm Improvements with Gates and Pumps Alternative and the Hardwood WLMP

## XII. SUCCESS CRITERIA FOR HABITAT BENEFIT CALCULATION

The desired WLMPs illustrate the idealized water depths and timing needed to maximize the habitat benefits for the various target species. However, during the feasibility study the sponsor indicated that even with moderate deviations from the desired WLMPs there can still be habitat benefits. In order to identify whether or not habitat goals were met during each of the 30 years, the biologists identified the critical WLMP attributes that define habitat success. This quantifiable success criteria was used to evaluate each of the thirty years of simulated interior water levels under the two different WLMPs and the three different alternatives (Plates 1-36).

For the Typical WLMP, seven different attributes of the annual hydrograph were identified for benefits to diving duck, dabbling duck and submerged and emergent aquatic vegetation habitat types. For the Hardwood/Buttonbush WLMP, four different attributes of the annual hydrograph were identified for benefits to hard mass tree, buttonbush and diving duck habitat types. Table H-12 identifies the WLMP of interest, the specific habitat criteria, and how success is quantified.

Typical and Hardwood/Buttonbush WLMP success criteria, as summarized in Table H-12, are shown in Figures H-26 and H-27, respectively.

**Table H-12:** WLMP Habitat Criteria and Success

| WLMP | Habitat Quantification Criteria | Success Criteria |
|---|---|---|
| Typical | **Aquatic Veg:** b/w 1-May and 15-September, find the WSEL that represents the minimum of the 90-day maximums. This elevation will be referenced to the wood duck acres table that relates water level to acres of vegetation. Main goal: 90 day growing season for aquatic vegetation. | +/- 0.5' of idealized WSEL =530' |
| Typical | **Dabbler1:** compute the increase in WSEL from Shorebird WSEL (above solution) and interior WSEL on 1 October to better evaluate quality of duck habitat. Main goal: 1.5' increase in WSEL. | 0'< WSEL increase <1.75' |
| Typical | **Dabbler2:** b/w 1 October and 15-October, find the 15-day average. The objective for these dates is to keep water levels stable for dabblers. | +/- 0.5' of idealized WSEL =531.5' |
| Typical | **Dabbler/Diver1:** compute the increase in WSEL from Dabbler2 WSEL (above solution) and interior WSEL on 1 November to better evaluate quality of dabbler/diver habitat. | 0.45'< WSEL increase <1' |
| Typical | **Dabbler/Diver2:** b/w 1 November and 15 November, find the 15-day average. The objective for these dates is to keep water levels stable for dabblers/divers. | +/- 0.5' of idealized WSEL =532' |
| Typical | **Dabbler/Diver3:** compute the increase in WSEL from Dabbler/Diver2 WSEL (above solution) and interior WSEL on 1 December to better evaluate quality of dabbler/diver habitat. | 0.25'< WSEL increase <1' |
| Typical | **Diver:** b/w 1 December and 15 December, find the 15-day average (and standard deviation). The objective for these dates is to keep water levels stable for divers. | +/- 0.5' of idealized WSEL =532.5' |
| HW/BB1 | **Buttonbush1:** b/w 1-June and 1-October during Hardwood WLMP1, find the WSEL that represents the maximum during the entire 120-day period. | +/- 0.5' of idealized WSEL =531' |
| HW/BB1 | **Hardwood1:** b/w 25-August and 1-November during Hardwood WLMP1, find the maximum WSEL that is not exceeded during the entire period. | +/- 0.5' of idealized WSEL =531.5' |
| HW/BB2 | **Buttonbush/Hardwood2:** b/w 15-April and 15-October during Hardwood WLMP2, find the 120-day maximum and return the minimum of the maximums. | +/- 0.5' of idealized WSEL =529.75' |
| HW/BB2 | **Diver2:** b/w 1-November and 1-December during Hardwood WLMP2, find the minimum elevation during the entire period. | +/- 0.5' of idealized WSEL =532.5' |

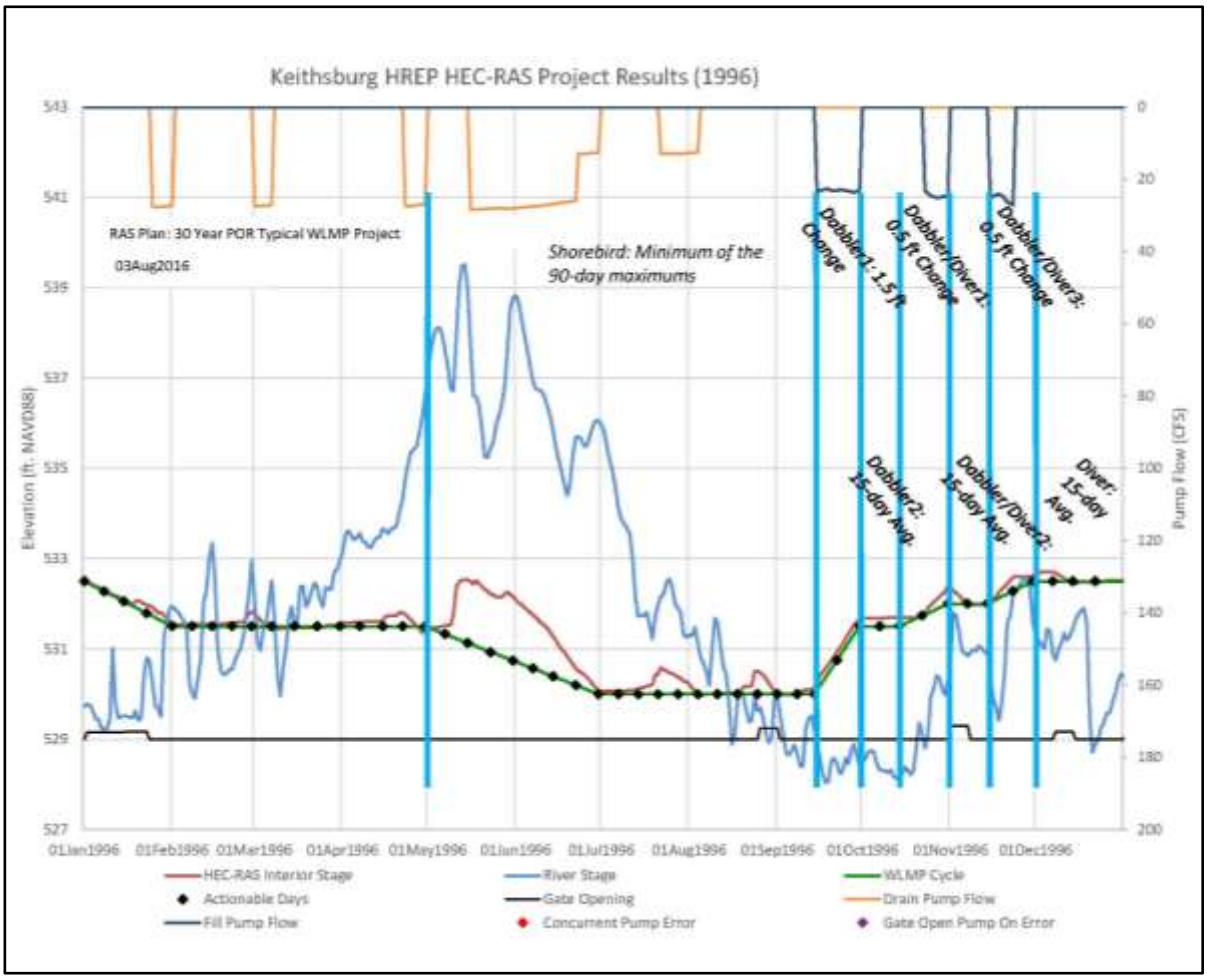


**Figure H-26:** Typical WLMP Success Criteria for Example Year 1996

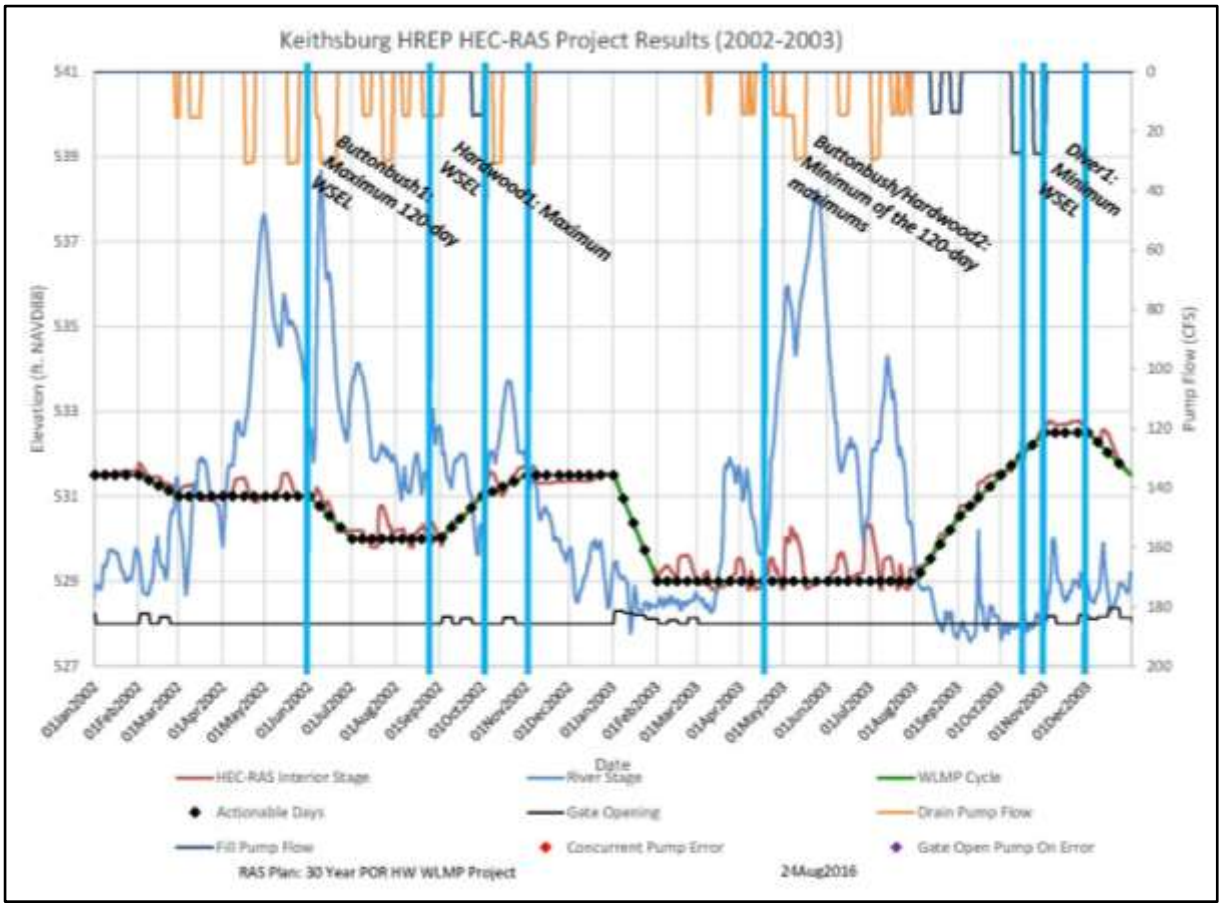


**Figure H-27:** Hardwood/Buttonbush WLMP Success Criteria for Example Years 2002-2003

The success evaluation process looks at each of the eleven different criteria from Table H-12 and computes the number of successful years (out of the 30-year simulation) and repeats this for each of the three Project alternatives. Success for each individual criteria is expressed in terms of percentage or success rate. Success rates are broken down for each WLMP by taking an average of the success rates for all the criteria specific to that WLMP. A summary of the success rates based on the thirty year simulation period (1986-2015) is shown in Table H-13. These success rates were factored into the calculation of habitat benefits for each of the different Project alternatives.

**Table H-13:** Summary of Habitat Success Rates for 30 Year Simulation

| | **No Action Alternative Success** | | **Berm & Gates Alternative Success** | | **Gates & Pumps Alternative Success** | |
|---|---|---|---|---|---|---|
| | **%** | **# years during 1986-2015 period where success would have been met** | **%** | **# years during 1986-2015 period where success would have been met** | **%** | **# years during 1986-2015 period where success would have been met** |
| **Typical WLMP** | | | | | | |
| *Aquatic Veg* | 7% | 2 | 3% | 1 | 50% | 15 |
| *Dabbler1* | 0% | 0 | 10% | 3 | 73% | 22 |
| *Dabbler2* | 10% | 3 | 37% | 11 | 97% | 29 |
| *Dabbler/Diver1* | 10% | 3 | 17% | 5 | 80% | 24 |
| *Dabbler/Diver2* | 7% | 2 | 33% | 10 | 100% | 30 |
| *Dabbler/Diver3* | 7% | 2 | 13% | 4 | 93% | 28 |
| *Diver* | 7% | 2 | 23% | 7 | 100% | 30 |
| ***OVERALL*** | **7%** | 2 | **20%** | 6 | **85%** | 26 |
| **HW/BB1** | | 0 | | 0 | | 0 |
| *Buttonbush1* | 0% | 0 | 27% | 8 | 87% | 26 |
| *Hardwood1* | 7% | 2 | 47% | 14 | 100% | 30 |
| ***OVERALL*** | **3%** | 1 | **37%** | 11 | **93%** | 28 |
| **HW/BB2** | | 0 | | 0 | | 0 |
| *Buttonbush/Hardwood2* | 0% | 0 | 27% | 8 | 73% | 22 |
| *Diver1* | 0% | 0 | 27% | 8 | 100% | 30 |
| ***OVERALL*** | **0%** | 0 | **27%** | 8 | **87%** | 26 |

## XIII. PUMPING COST ANALYSIS

As part of the incremental costs analysis, an assumption regarding operational costs for the pump station for a 50-year project life was needed. The pumping required during the 30 year HEC-RAS simulation is assumed to be representative of the variability in hydrologic conditions that the Project will see during a 50-year project life. Hourly pumping output from the HEC-RAS model was used to compute pumping costs during the 30 year simulation.

Total pumping discharge in cubic feet per second (cfs) is output from RAS for each hour and is specified as 'Draining' or 'Filling'. RAS output specifies whether 1 or 2 pumps are operating during each hour. Total pumping discharge (cfs) is divided by the number of pumps operating (1 or 2) to come up with discharge per pump (cfs). A lookup function (provided by the pump manufacturer) was used to determine horsepower (hp) for each pump based on the discharge in cfs. Total hp is then summed for both pumps. Total hp is then converted to total kW using the formula [hp*0.7457/0.902*1.1=kW]. Total kWh is then computed by multiplying kW by the number of hours (1). The utility rate structure was obtained by the electrical engineer from a typical bill provided by the utility company. Peak demand occurs from 10 am through 10 pm.

Variable costs under the typical WLMP were initially computed by assuming 50% of the total pumping occurred during on-peak hours. The total variable cost for the typical WLMP for ~30 yrs of operation (Jan 2, 1986 to Dec 21, 2015) is ~$8,900 for draining and ~$12,600 for filling ($0.007674/hr). This variable rate is comprised of a distribution delivery charge, a peak charge and an EDT cost recovery charge. Additional modeling could be done to evaluate potential cost savings by reducing on-peak pump operation and the associated changes in the ability to meeting the desired WLMPs. Clarification regarding the distribution delivery charge was provided late during the incremental cost analysis, so it was used to verify the assumption that 50% of pumping occurred during peak hours and 50% of pumping occurred during off-peak hours. The results indicate that slightly more than 50% of pumping occurred during peak hours for both filling and draining, therefore the total variable cost for the Typical WLMP increased to ~$9,400 for draining and ~$13,400 for filling. Total variable costs for the Hardwood/Buttonbush WLMP for ~30 yrs of operation (Jan 2, 1986 to Dec 21, 2015) is $6,600 for draining and $8,400 for filling.

Fixed costs are assessed for every hour the pumps are online (whether or not they are pumping). Fixed costs were calculated based upon information provided by the utility including ($0.51388889/hour customer charge, $0.01604167/hour meter charge, and $0.06818056/hour transformation charge). The total fixed cost for ~30 years of operation (1-2-1986 to 12-21-2015), under either WLMP, is ~$157,100 ($0.59811111/hour). These costs reported above do not include taxes.

## XIV. ALTERNATIVE ANALYSIS RESULTS AND RECOMMENDED PLAN

The PDT identified the Berm, Gates and Pumps Alternative as part of the Recommended Plan. This includes the upstream and downstream sluice gate structures and pump station. As discussed in Section XI, *Success Criteria for Habitat Benefit Calculation*, the addition of a pump station, albeit costly to operate, makes a dramatic impact in the ability to meet the desired WLMPs necessary for habitat benefits. The sponsor spent considerable time weighing the pump station operational costs, for

which they are responsible, and the additional habitat benefits that come with that cost before they provided concurrence on the Recommended Plan.

## XV. FLOWAGE EASEMENT ANALYSIS

Along the northeast corner, within Spring Slough, there are several culverts that pass water from the Northeast Ag Land HEC-RAS Storage Area into Spring Slough. Two of these culverts (culvert #2 and culvert #4) have an invert elevation below the maximum proposed managed water surface elevation of 532.5 feet (Figures H-28 and H-29). Under existing conditions, the site sees greater water level fluctuations than what is proposed by the Project due to the Pope Creek Breach. The Mississippi River stage equals or exceeds the breach elevation 60% of the year and the Mississippi River stage equals or exceeds 532.5 feet ~25% of the year. Nevertheless an evaluation to assess flowage easement extents was determined by the PDT to be necessary. The focus of this assessment was to ensure that the Project's maximum managed water level would not limit the capacity of the culverts.

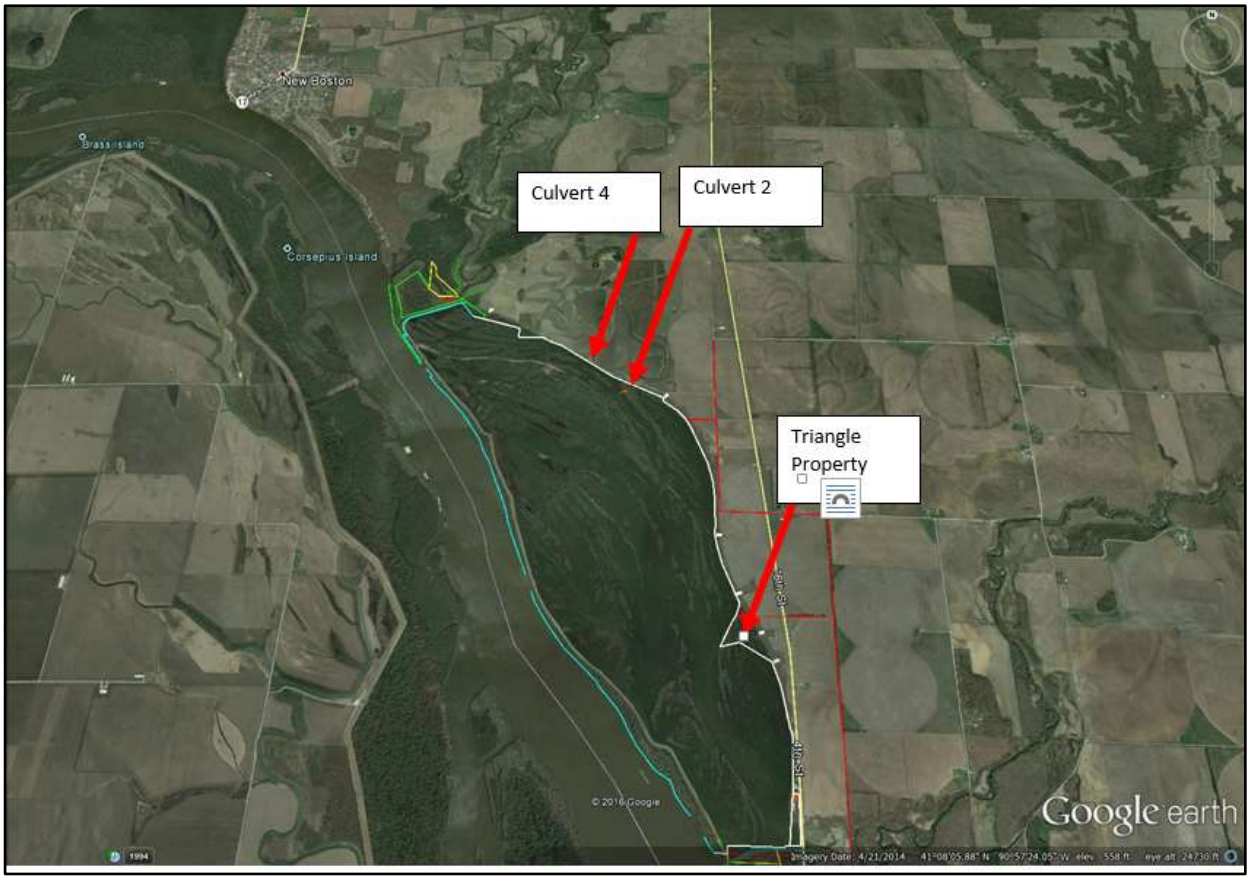


**Figure H-28:** Location of Culverts Examined for Flowage Easement Assessment

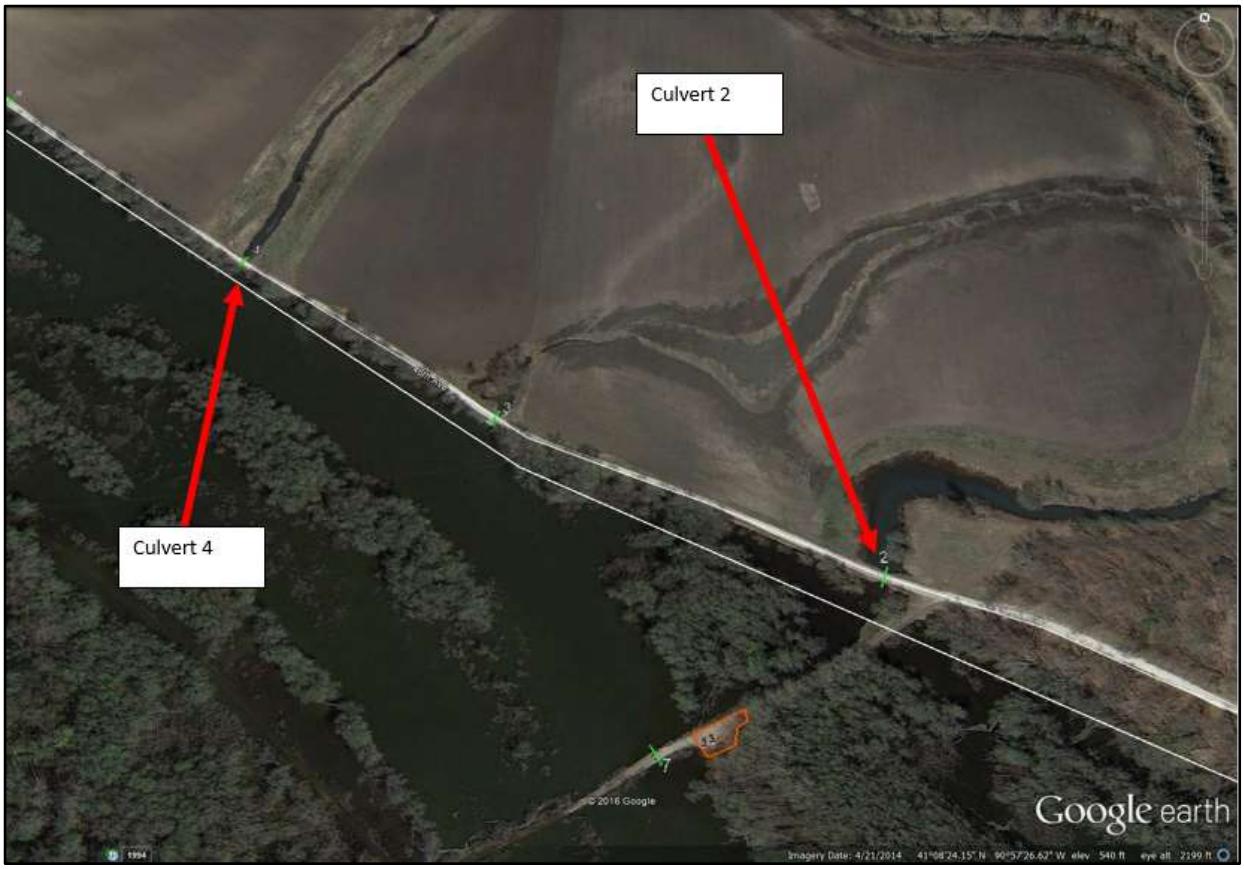


**Figure H-29:** Culverts 2 and 4 Examined for Flowage Easement Assessment

The two culverts were evaluated to determine whether maximum discharge through the culverts is limited by a tailwater condition at 532.5 feet (i.e. outlet control). The fixed-headwater method was used to determine inlet versus outlet control for the two culverts under design head conditions. The design head at each culvert was assumed to be a maximum headwater condition just below the road crown elevation and the tailwater condition was fixed at 532.5 feet for the analysis, the maximum managed interior water surface elevation with-Project.

Results of the fixed-headwater analysis for culvert #4 indicated that discharge under maximum headwater and maximum tailwater (as managed with-Project) conditions is inlet-controlled. Therefore the proposed Project will not limit discharge capacity through culvert #4.

2016 survey indicated that culvert #2 was replaced and no longer has a positive, downstream slope. As a result, the fixed-headwater analysis results for culvert #2 were inconclusive. The recommendation of the PDT is to design a replacement culvert at this location during plans and specifications that will ensure inlet control.

## XVI. CLIMATE CHANGE ASSESSMENT

Engineering Construction Bulletin (ECB) No. 2016-25 (USACE 2016) provides guidance on conducting a qualitative assessment of climate change impacts related to inland hydrologic analyses for civil works studies, designs and projects. The intent of the qualitative analysis is to describe the current and potential future climate threats, vulnerabilities and impacts of climate change specific to the study goals and engineering designs. Consideration of both past (observed) changes as well as potential future (projected) changes to relevant climatic and hydrologic variables is required as part of this qualitative assessment.

### A. Literature Review: Regional Climate Change Trends

Observed and projected hydrometeorologic data were evaluated to assess potential changes to climate over the 50 year project life. Hydrometeorologic variables with the potential to impact the performance of the Project were examined; including temperature, precipitation, and streamflow.

Regional summary reports prepared by the Corps in 2015 summarize observed and projected trends reported in the literature. Findings for Water Resources Region 07, the Upper Mississippi Region (UMR), which includes the Project area, include statistically significant increases in mean air temperature during winter, spring, and summer; however, a slight decreasing trend was observed for fall mean air temperatures (Wang et al., 2009). Westby et al. (2013) found during the period of 1949 to 2011, statistically significant warming occurred in the northern UMR.

In the UMR, spring onset is occurring at least a few days earlier for the current period (2001 to 2010), as compared to an earlier baseline reference decade (1951 to 1960). This denotes an apparent small shift in seasons, with spring warming occurring earlier than in the past (Schwartz et al., 2013). Increases in air and water temperature across the UMR over the past few decades have led to earlier ice-out dates and later ice-in dates for lakes and earlier spring runoff (Johnson and Stefan, 2006).

Regional results from the U.S. Global Research Program's *Third National Climate Assessment for the Upper Midwest* (NCA3) suggest the rate of warming in the Midwest has markedly accelerated over the past few decades. Between 1900 and 2010, the average Midwest air temperature increased by more than 1.5°F (Figure H-30). However, between 1950 and 2010, the average temperature increased twice as quickly, and between 1980 and 2010, it increased three times as quickly as it did from 1900 to 2010. Warming has been more rapid at night and during winter. The literature shows consensus among authors indicating increasing trends in observed air temperatures for the Midwest and UMR.

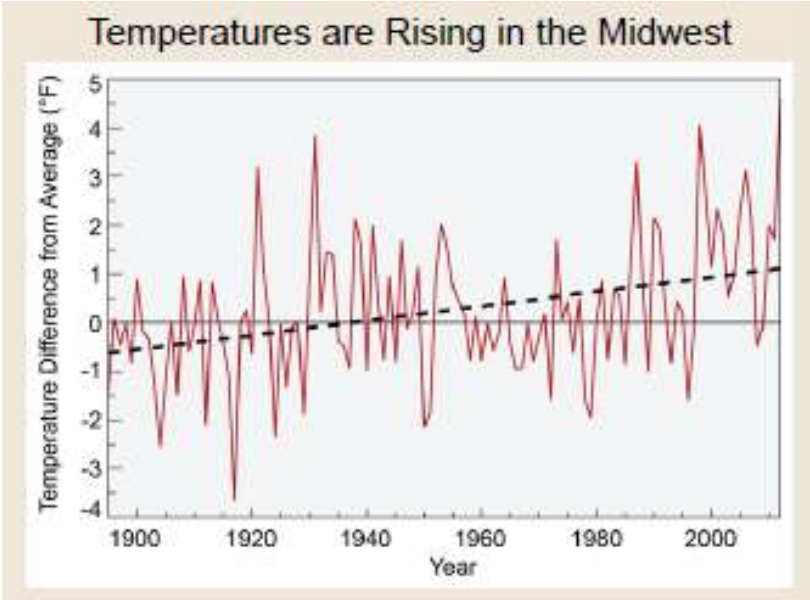


**Figure H-30:** Increasing Annual Average Temperatures in the Midwest (NCA3)

There is a high level of consensus across multiple Global Circulation Models (GCMs) and emissions scenarios projecting a significant warming trend across the entire UMR. Compared to the baseline time frame of 1971 to 2000, a study by Liu et al. (2013) using a single GCM and assuming an A2 greenhouse gas emissions scenario (worst case) projected an increase in maximum air temperature of 1.5 to 4.5 ºC (2.7 to 8.1 ºF) for a 2055 planning horizon in the UMR. Additionally, the study predicted an increase in the Keetch Byrum Drought Index, which is a measure of soil moisture index.

In 2014, Scherer and Diffenbaugh published a study in which they projected a steady increase in both summer and winter air temperatures throughout the whole 21st century for the UMR, using a multi-member GCM and assuming an A1B emissions scenario (middle of the road). Compared to a baseline timeframe of 1980 to 2009, the study projected that by the year 2090, the air temperature would increase by 5.7°C (10.3°F) in the summer and 3.6°C (6.5°F) during the winter. Cai et al. (2009) and Wilson and Weng (2011) both produced similar results when evaluating central Illinois using GCMs and shorter planning horizons.

Using two different GCMs and assuming high greenhouse gas emissions (A2 and A1f), Kunkel et al. (2010) projected a 4.0 to 6.5 ºC (7.2 to 11.7 ºF) increase in three-day heat wave temperatures and a 15 to 50 day increase in the annual number of heat wave days for a 2090 planning horizon compared to a recent historical baseline for the UMR.

Projections of future extreme climate events were summarized in the report. Gao et al. (2012) used a planning horizon of 2058 and a single GCM with a high greenhouse gas emission assumption to project increases in heat wave intensity, duration, and frequency. Results show an increase of up to 4.0 ºC (7.2 ºF) in extreme heat wave temperatures in the UMR and the duration of heat waves is projected to increase by 2 to 4 days per event, compared to the baseline period of 2001 to 2004. The overall frequency of heat waves are projected to increase by 1 to 4 events per year.

Pryor et al. (2014) used GCMs to project statistically significant increases in both annual average temperature and the number of extreme heat days over the next century for the Midwest (Figure H-31). Additionally, projections are presented showing an increase in the frost free season and an increase in the number of "cooling degree days," defined as the number of degrees that a day's average temperature is above 65 °F (18.3 °C).

According to the NCA3, the amount of future warming will depend on changes in the atmospheric concentration of heat-trapping gases. Projections for regionally-averaged temperature increases by the middle of the century (2046-2065) relative to 1979-2000 are approximately 3.8°F for a scenario with substantial emissions reductions (B1) and 4.9°F with continued growth in global emissions (A2). See Figure H-31. The projections for the end of the century (2081-2100) are approximately 5.6°F for the lower emissions scenario and 8.5°F for the higher emissions scenario.

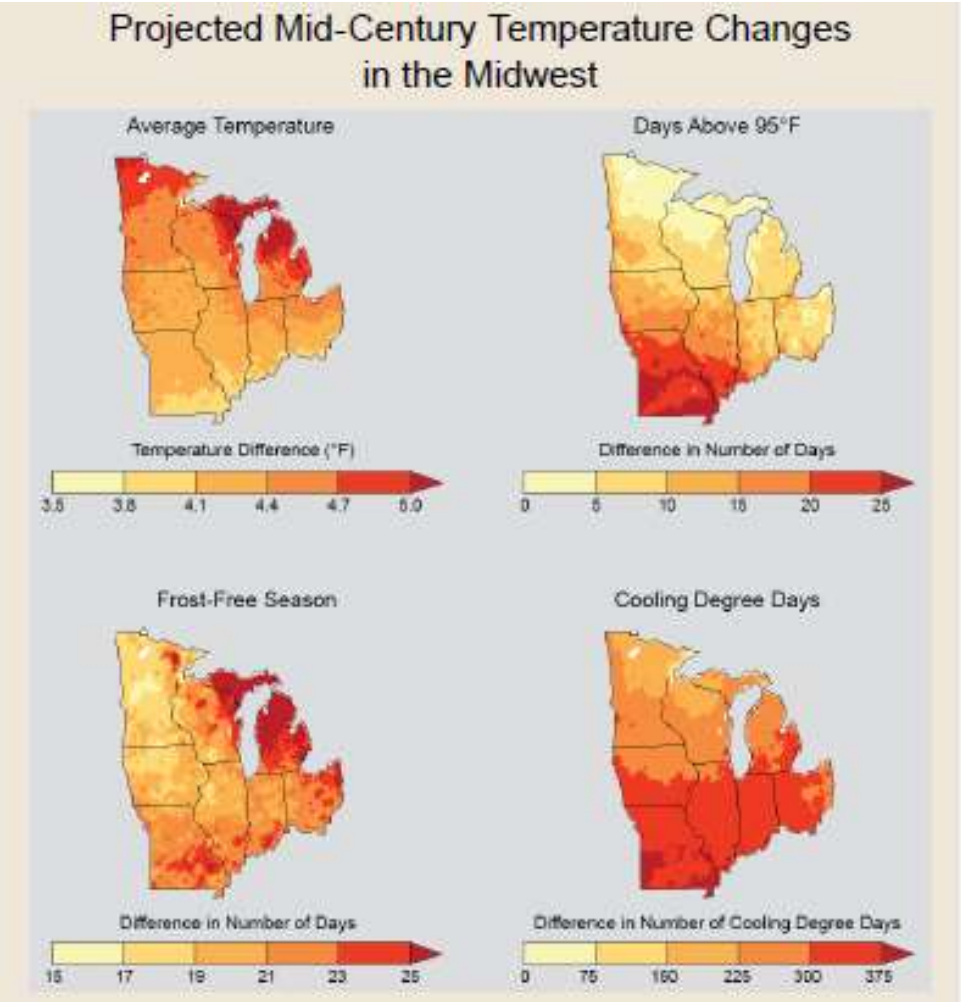


**Figure H-31:** GCM Projections Showing Increasing Annual Average Temperatures, Number of Hottest Days, Length of Frost-Free Season, and Cooling Degree Days (Pryor et al., 2014)

Statistically significant increasing trends in total annual precipitation have been reported in several studies. An increasing trend in winter storm precipitation total (1972-2002) was observed by Palecki et al. (2005), while Grundstein (2009) identified positive linear trends in both total annual precipitation and soil moisture index during the 1895-2006 period. During the 1950 to 2000 period, a significant increasing trend in precipitation, particularly in the summer and fall, was observed for the UMR (Wang et al., 2009). During the winter and spring, a mild decreasing trend was identified for the northern half of the UMR. McRoberts and Nielsen-Gammon (2011) quantified an increasing trend in annual precipitation (1895-2009) for the UMR as between 5% to 10% per century.

Moderate increases (33%) in the frequency of 20-year storm events in the UMR where observed by Wang and Zhang (2008) when comparing the period 1949 to 1976, to the period 1977 to 1999. Within the UMR, statistical analysis of 20$^{th}$ century rainfall data showed generally increasing and statistically significant trends in total annual precipitation and the number of precipitation days per year (Pryor et al., 2009). For multiple climate stations in the UMR with at least 50 years of historical record, statistically significant increasing trends in the frequency of occurrence of heavy rainfall were identified by Villarini et al. (2013).

The NCA3 report concluded that annual precipitation increased during the past century (by up to 20% in some locations), with much of the increase driven by intensification of the heaviest rainfalls. Multiple authors have identified a mild upward trend in observed precipitation for the Midwest and UMR.

Projections of future changes in precipitation in the UMR generally concur that both annual precipitation and extreme precipitation totals are going to increase. Using a planning horizon of 2055, Liu et al. (2013) projects an increase in spring, summer, and winter precipitation. Despite the projected increase in precipitation, the study also projects an increase in the severity of future droughts.

Applying a planning horizon of 2058 to a GCM, Gao et al. (2012) generally projects increases in the magnitude of annual and daily extreme (defined as 95$^{th}$ percentile) storm events, and in the frequency of precipitation events.

A study of the Illinois watershed by Wilson and Weng (2011) used A1B and B1 emission scenarios and a 2020 planning horizon to project changes in monthly precipitation. Results indicate a general agreement of drier summer months and wetter winter months.

Multiple studies present future projections of extreme events, which include storm events and droughts. Tebaldi (2006), Wang and Zhang (2008), Wang et al. (2011), Schuster et al. (2012), and Joetzjer et al. (2013) all generally predict increases in the number of high (>10 mm) precipitation days for the region, the number of storm events greater than the 95th percentile of the historical record, and the daily precipitation intensity index (annual total precipitation divided by number of wet days). In other words, the projections forecast small increases in the occurrence and intensity of storm events by the end of the 21st century for the general study region. Wang and Zhang (2008) used downscaled GCMs and a high emissions scenario (A2) to quantify a significant increase (50 to 100%) in the recurrence of the current 20 year 24-hour storm event for the planning horizon of 2075. In 2011, Wang et al. used multiple Regional Climate Models (RCMs) and two emissions scenarios to characterize the intensity and frequency of projected droughts in Illinois for the end of the 21st century using the

Standardized Precipitation Index (SPI). Results show significant increases in the frequency and intensity of short duration (1 to 4 weeks) droughts due primarily to increased air temperatures. Joetzjer et al. (2013) did a similar study, but focused on the whole UMR, with results concurring with Wang et al.'s earlier study. These results reflect the impacts of projected temperature and evapotranspiration increases in the basin, which appear to exceed the projected impacts of increased precipitation.

Pryor et al. (2014) studied the Midwest. Results generally support all of the other studies' findings. Climate model projections presented in this study indicate a statistically significant increase in annual average precipitation (2.4 to 4.0 inches), wettest 5-day total (0.4 to 1.0 inches), and the number of heavy precipitation days by the middle of this century. Additionally, the duration of consecutive dry days is expected to increase by up to 3 days.

According to the NCA3 assessment, projections indicate increased spring precipitation (9% in 2041-2062 relative to 1979-2000) and decreased summer precipitation (by an average of about 8% in 2041-2062 relative to 1979-2000), particularly in the southern portions of the Midwest. Increases in the frequency and intensity of extreme precipitation are projected across the entire region, and these increases are generally larger than the projected changes in average precipitation. Although the total amount of water from rainfall and snowfall is projected to increase across the entire Midwest, models also indicate an increase in consecutive dry days and chances of drought.

From 1939 to 1998, the Mississippi River watershed saw an increase in river flow (Mauget, 2004). An increase in surplus flow days increased and the number of drought incidence decreased for the UMR during the same study period, with the greatest change occurring during the latter part of the century. A majority of the 42 gage stations in the UMR showed statistically positive trends in both annual streamflow and baseflow (Duan et al., 2006). Mean flow and peak flows showed the same positive trend during the period 1913 to 2002.

Statistically significant increasing trends in both annual 7-day low flow and annual mean flow were detected for multiple sites. These were based on analysis of USGS stream gage data, part of the Hydroclimatologic Data Network. Studies of surface water trends, including runoff, for the Mississippi River Region, which includes the UMR, quantified statistically increasing trends in runoff in the region for the period 1948 to 2004 (Qian et al., 2007).

The NCA3 assessment looked at climate change impacts on the water cycle. It found river flows have increased across the Midwest, however, the length of dry spells and the number of short term droughts have also increased. The timing of peak river levels has changed in response to warming trends. Snowpack and snowmelt-fed rivers in much of the western U.S. have earlier peak flow trends since the middle of the last century, including the past decade. The change in total annual precipitation and heavy precipitation is projected to lead to an increase in the magnitude and frequency of flooding, especially flash floods.

In order to project future climate trends in hydrology, many studies of the UMR have relied upon the use of GCMs and macro-scale hydrologic models. There is no clear consensus in the literature, with some studies projecting an increase in future streamflow as a result of increased precipitation in the UMR, while others project a decrease in flows as a result of increased evapotranspiration. Seasonally, multiple studies suggest increased flows in the winter and spring and decreased flows in the summer. An example of contradictory projections can be found in the study performed by Thomson in 2005,

where two GCMs with one set of input assumptions, yielded different results. One model predicts significant decreases in water yield, the other projects significant increases in water yield. Studies by Hagemann et al. (2013) and Döll and Zhang (2010) illustrate how climate change is expected to have as much, or more, of an impact to ecologically relevant flow characteristics as dams and withdrawals over the next century. For the UMR, projections show mild (relative to global results) impacts to both low and average annual flows.

Results of other studies such as Jha (2006) and Wu et al. (2012) highlight the significant uncertainty associated with future hydrologic projections. However, these studies can be used to show the potential for large-scale changes in either direction.

Projected changes to streamflow are not covered in great detail for the Project area within the NCA3 report; however, the report does highlight projected changes in spring peak river flows as a result of shifts of the amount and timing of snow pack and snow melt in much of the U.S. This shift is in response to warming trends. The NCA3 notes that projecting future flooding is difficult due to variables such as river level and soil moisture prior to a rain event, yet data suggests an increase in flooding frequency.

There is strong consensus that air temperatures will increase in the UMR, with studies generally agreeing on an increase in mean annual air temperature of approximately 2 to 6 ºC (3.6 to 10.8 ºF) by the latter half of the 21st century. A reasonable consensus is also seen on projected increases in extreme temperature events. This includes more frequent, longer, and more intense summer heat waves in the long term future compared to the recent past.

A majority of the precipitation projections in the studies forecast an increase in both annual precipitation and in the frequency of large storm events. Seasonally, though, some studies indicate a potential for drier summers despite the overall increase in annual precipitation totals. As a result of increased air temperature and evapotranspiration rates, droughts are also projected to increase in the UMR.

In regards to streamflow and hydrology projections, there is no clear consensus in the literature. Projections generated by coupling GCMs with macro scale hydrologic models in some cases indicate a reduction in future streamflow, but in other cases indicate a potential increase in streamflow. Figure H-32 summarizes the trends and literary consensus of observed and projected primary variables of temperature, temperature extremes, precipitation, precipitation extremes, and streamflow (hydrology).

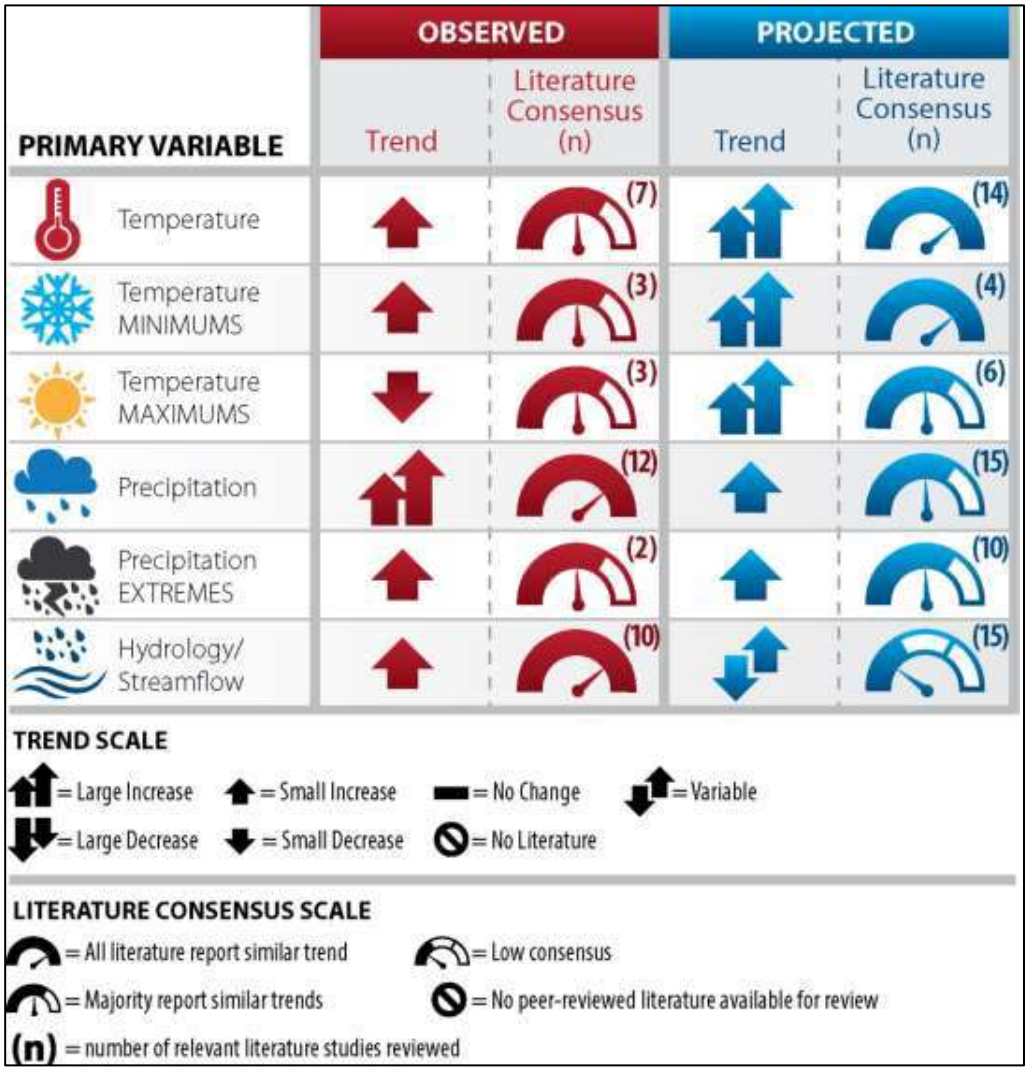


**Figure H-32:** Summary and Literature Consensus of Observed and Projected Trends in Important Meteorologic Variables Potentially Impacted by Climate Change

Observed and projected trends in meteorologic parameters, as summarized above, suggest that climate change poses potential threats and impacts to ecosystem restoration efforts in the region, such as those proposed as part of this Project. Increased air temperatures and increased frequency of drought during the summer months will result in increased water temperatures. These conditions may impact water quality, including reduced dissolved oxygen concentrations which are a necessary component for aquatic ecosystem habitat. Occurrence of nuisance algal blooms associated with increased water temperatures also results in decreased light penetration and decreased nutrient availability. Light and nutrient availability within the water column are two key factors influencing aquatic vegetation. Restoring migratory waterfowl habitat, including their aquatic vegetation food source, is a common ecosystem restoration objective. Therefore negative impacts to water quality due to climate change can affect the ability to meet such restoration objectives.

Increasing mean annual precipitation may pose potential impacts to assumed pump operation and estimated O&M costs. Pump capacity flexibility, provided by two smaller pumps, in the proposed Project offers resilience in the face of increased variability in precipitation (interior rainfall and runoff) and streamflow (Mississippi River stage) variability.

**B. First Order Statistical Analysis: Regional Scale Trends in Streamflow and Climate Change**

In order to evaluate projected trends in hydrology for the Project area, the USACE Climate Hydrology Assessment Tool was used to analyze streamflow for the Upper Mississippi-Iowa-Skunk-Wapsipinicon watershed (HUC 0708) (Figure H-33). The Climate Hydrology Assessment Tool provides qualitative information at the HUC 4 watershed level about future climate conditions and allows the Corps to produce repeatable analytical results using consistent information.

**Figure H-33:** Location of Project Area within HUC2 and HUC4

Observed trends in annual peak instantaneous streamflow for the Mississippi River at Keokuk, Iowa (USGS gage 05474500) evaluated using the USACE Climate Hydrology Assessment Tool indicate a statistically significant increasing trend for the period 1878-2016 (p-value =0.0009347) (Figure H-34).

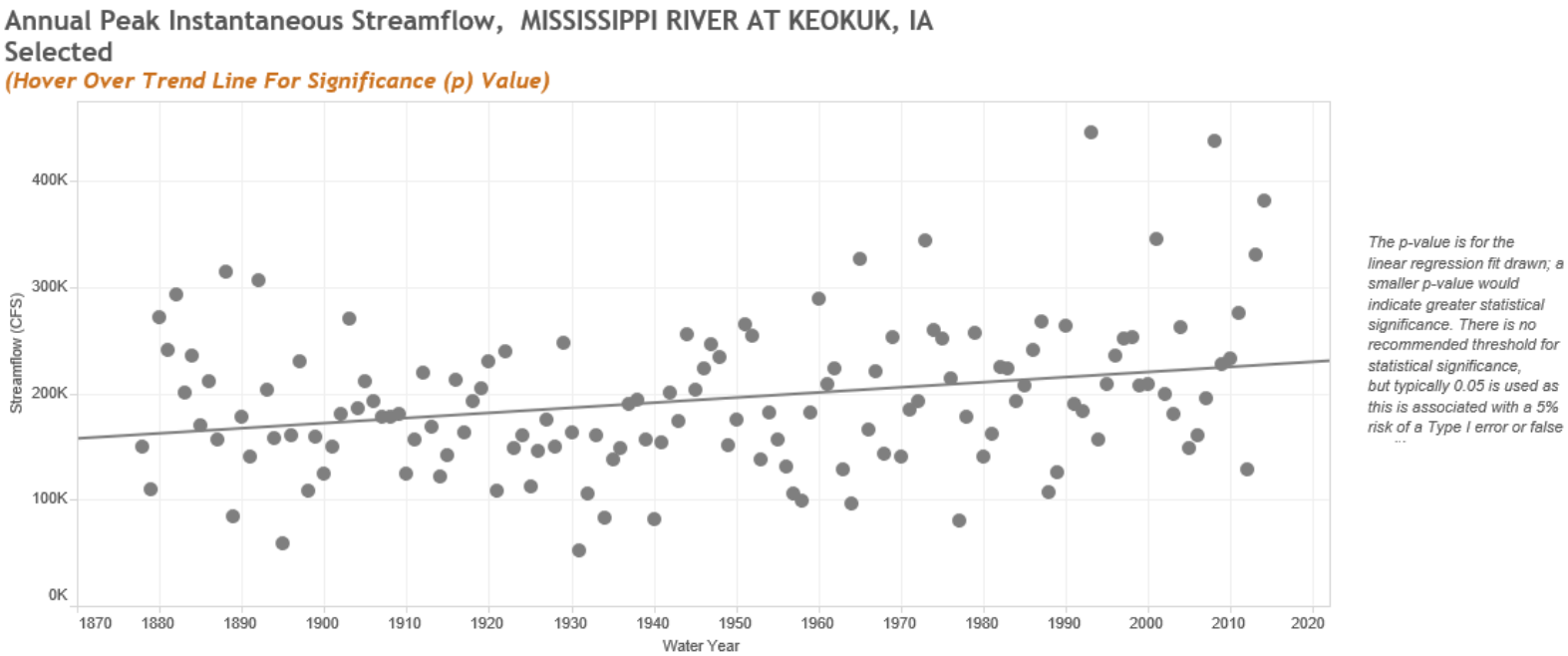


**Figure H-34:** Increasing Trend in Observed Annual Peak Instantaneous Streamflow for Mississippi River at Keokuk, IA (p-value=0.0009347)

Projected hydrology under future climate conditions is generated using a hydrologic model with precipitation and temperature input parameters derived from GCM output. The range in projected annual maximum monthly streamflow is computed based on 93 different climate changed hydrologic model simulations for the 1981-2099 period. Results for the Upper Mississippi-Iowa-Skunk-Wapsipinicon watershed shown in Figure H-35 indicate there is a lot of uncertainty in projected climate changed hydrology. Nevertheless, there is a statistically significant increasing trend in the mean projected annual maximum monthly streamflow (Figure H-36). Based on the trendline, over the 50 year project life annual maximum monthly flows could increase by nearly 2,500 cfs (Figure H-36). Average annual mean discharge over the most recent 30-year period (1987-2016) for USGS gage 05474500 (Mississippi River at Keokuk, Iowa) is 82,500 cfs and the minimum annual mean discharge is 40,000 cfs. Therefore, the projected increase in mean annual maximum monthly streamflow is not considered to have any operationally significant impacts. These results are qualitative only.

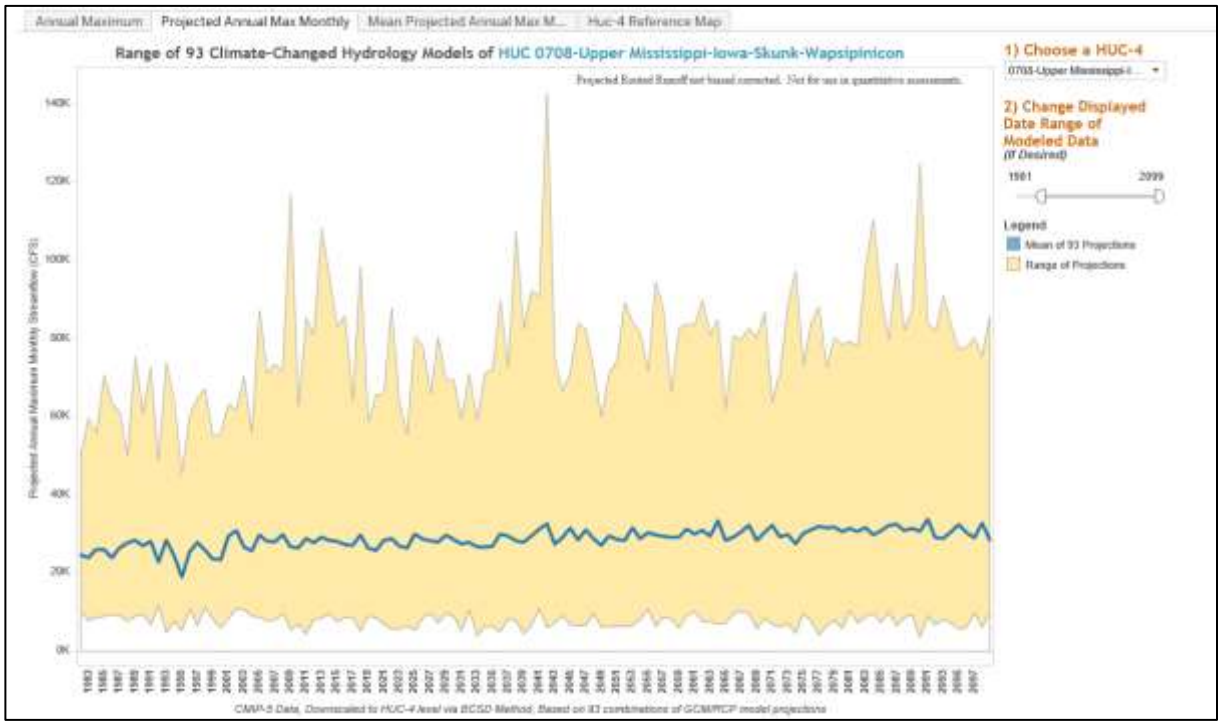


**Figure H-35:** Range in Projected Annual Maximum Monthly Streamflow for the Upper Mississippi-Iowa-Skunk-Wapsipinicon Watershed (HUC 0708)

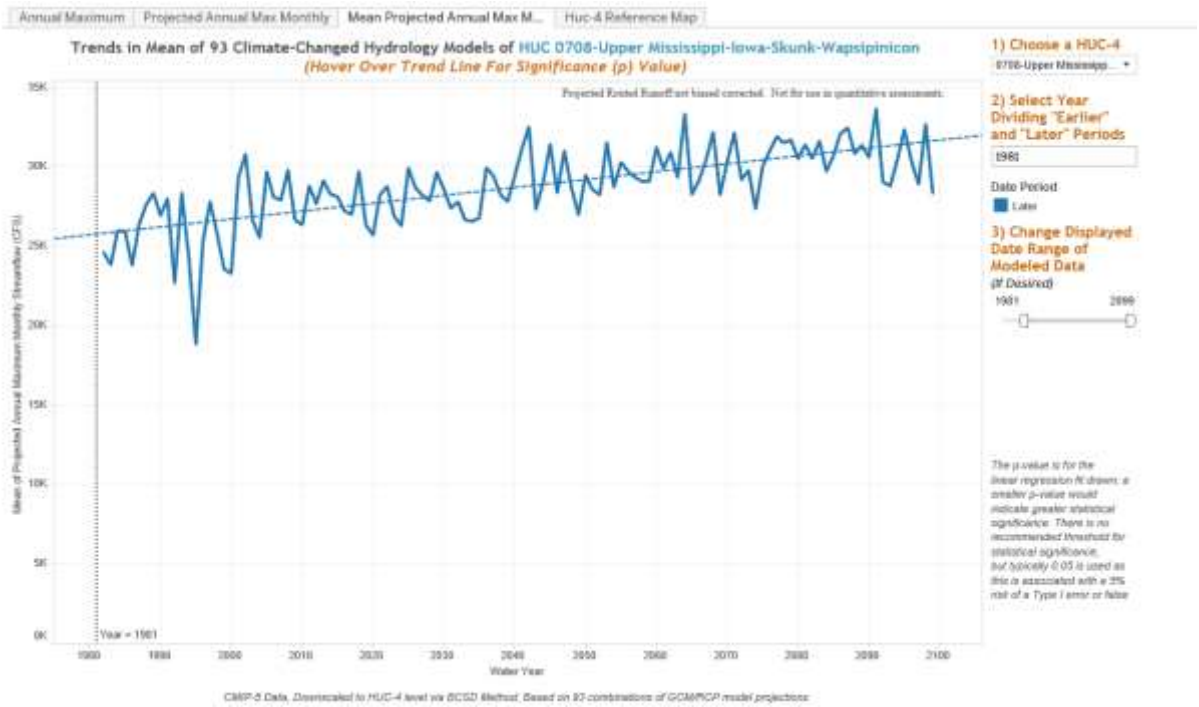


**Figure H-36:** Mean Projected Annual Maximum Monthly Streamflow for the Upper Mississippi-Iowa-Skunk-Wapsipinicon Watershed (HUC 0708). Trendline Equation: Q = 48.9904*[Water Year] – 71232.5, $p < 0.0001$

## C. Screening Level Vulnerability Assessment to Climate Change Impacts

The purpose of the USACE Watershed Climate Vulnerability Assessment Tool is to compare the relative vulnerability of the Project's HUC 04 watershed to climate change with that of the other 202 HUC 04 watersheds throughout the continental United States (CONUS). The tool provides a screening level comparative assessment of vulnerability to climate change according to business line. The tool is intended to identify climate threats and vulnerabilities for a specified region and business line using the Weighted Order Weighted Average (WOWA) method to calculate a composite index (Vulnerability Score).

The Project is located within HUC 0708, the Upper Mississippi-Iowa-Skunk-Wapsipinicon watershed and the UMRR Program is part of the "Ecosystem Restoration" business line (Figure H-33). Indicators including change in sediment load, short-term variability in hydrology, runoff elasticity (ratio of streamflow runoff to precipitation), macroinvertebrate index (sum score of six metrics indicating biotic condition), two indicators of flood magnification (indicator of how much high flows are projected to change overtime), mean annual runoff, change in low runoff, and percent of at risk freshwater plant communities are used to calculate WOWA scores under Ecosystem Restoration. HUC-4 watersheds with the top 20% of WOWA scores are flagged as being vulnerable. The default National Standards setting was used when conducting this vulnerability assessment.

Results of the Vulnerability Assessment Tool suggest that ecosystem restoration efforts in HUC0708, the Upper Mississippi-Iowa-Skunk-Wapsipinicon watershed, are relatively less vulnerable to climate change compared to the other 202 HUC 04 watersheds in the CONUS (Figure H-37). Vulnerability

scores for the two 30 year epochs indicate little change between epochs and little change between the wet and dry traces (Table H-14). Contributions to the overall vulnerability score from each of the different indicators for the ecosystem restoration business line and the 2050 epoch are shown in Table H-15. The dominant indicators contributing to the HUC0708 vulnerability scores are At-Risk Freshwater Plants (38%), Runoff Elasticity (24%), and Macroinvertebrate Index (12%).

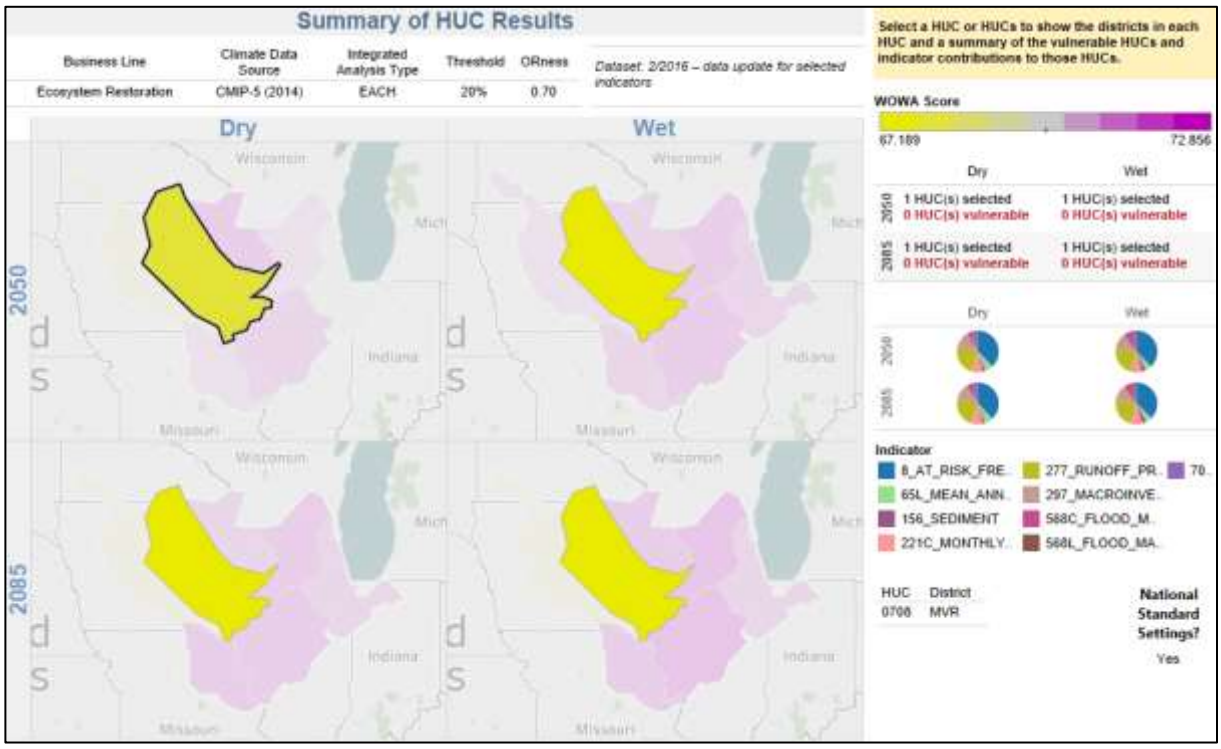


**Figure H-37:** Projected Vulnerability for Ecosystem Restoration within the Upper Mississippi-Iowa-Skunk-Wapsipinicon Watershed (0708)

**Table H-14:** Projected Vulnerability for Ecosystem Restoration Business Line

| | Ecosystem Restoration Vulnerability Score | | | |
|---|---|---|---|---|
| **HUC 04 Watershed** | **2050 Dry** | **2050 Wet** | **2085 Dry** | **2085 Wet** |
| Mississippi River (0708) | 67.965 | 67.189 | 67.540 | 67.779 |

**Table H-15:** Individual Indicator Contributions Related to Ecosystem Restoration

| **2050 Epoch** | **Mississippi River 0708** | |
|---|---|---|
| | Dry | Wet |
| **Indicator** | Contribution to WOWA Ecosystem Restoration Vulnerability Score | |
| Sediment | 1.45 | 2.21 |
| Monthly Covariance | 6.17 | 5.97 |
| Runoff Elasticity | 16.02 | 15.28 |
| Macroinvertebrate | 8.36 | 8.27 |
| Flood Magnification - Cumulative | 1.99 | 4.32 |
| Flood Magnification - Local | 0.82 | 1.02 |
| Mean Annual Runoff | 4.36 | 3.23 |
| Change in Low Runoff | 3.12 | 1.48 |
| At Risk Freshwater Plants | 25.68 | 25.41 |

## D. First Order Statistical Analysis: Site Specific Trends in Streamflow

Water surface elevation (stage) and discharge are the most important hydrologic variables in terms of the performance of the Project. Factors other than climate change can impact discharge and stage values over time including changes in land use, changes to Lock and Dam operation and long term geomorphic change. Consequently, identifying impacts to local hydrology attributable exclusively to climate change presents a challenge. The intent of the analysis described herein is to determine whether the hydrologic variables most relevant to Project performance have changed throughout the observed hydrologic record or are projected to change in the future. Frequency of spillway overtopping during the growing season was considered to be the most important hydrologic factor influencing the performance of the Project, because interior flooding during the growing season results in a loss of vegetation to support the fall migration.

The Non-Stationarity Detection Tool was used to analyze the 1939-2014 period for discharge at the USGS Gage 05474500, Mississippi River at Keokuk, Iowa using twelve different statistical tests to detect nonstationarity in the peak annual streamflow record. The gage is located downstream, in the tailwater of Lock and Dam 19. The first dam was constructed in 1913, resulting in the initial impoundment of Pool 19 and minor regulation of flows. Adjustments have been made to the flow record through September 1937. For this reason, the analysis period for the Non-Stationarity Detection Tool was restricted to 1938-2014. The dam at Lock and Dam 19 is operated for hydroelectric power production as well as to maintain a nine foot depth for navigation. At flows above 360,000-380,000 cfs the hydroelectric power company is unable to control the pool elevation and the lock and dam operates as a run-of-the-river structure.

The Energy Divisive Method detected a statistically significant nonstationarity in 1982 and the Lombard Wilcoxon Test detected a statistically significant nonstationarity in 1963 (Figure H-38). However, a lack of consensus between the statistical tests indicate no significant operational nonstationarities or change points in the flow record can be detected.

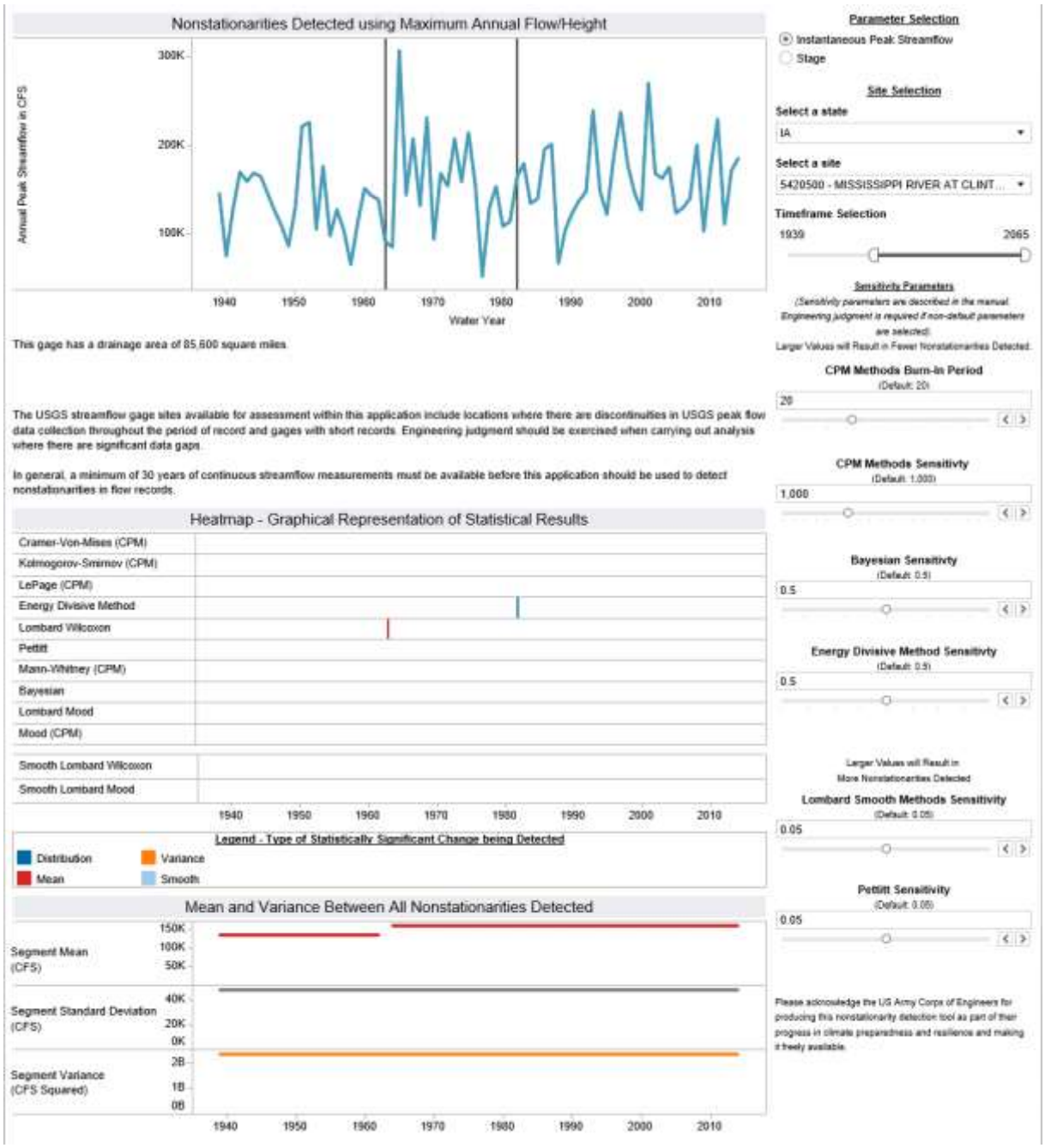


**Figure H-38:** Nonstationary Analysis of Peak Annual Discharge for the Mississippi River at Keokuk, Iowa (USGS Gage 05474500) for the 1939 to 2014 Period

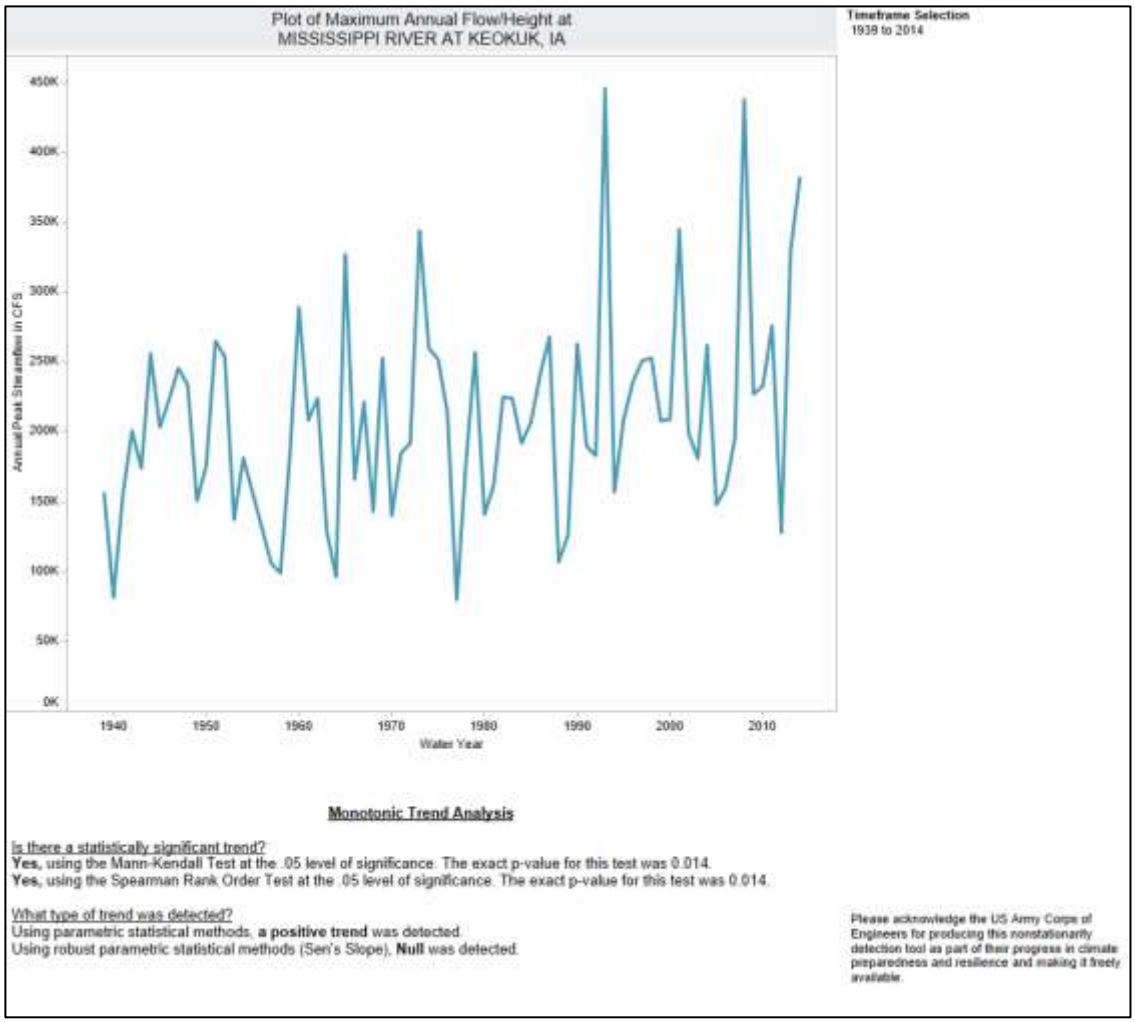


**Figure H-39:** Monotonic Trend Analysis of Peak Annual Discharge for the Mississippi River at Keokuk, Iowa (USGS Gage 05474500) for the 1939 to 2014 Period.

Figure H-39 shows the Monotonic Trend Analysis results which illustrate a statistically significant upward trend in peak annual discharge at the Keokuk gage. The Mann-Kendall and Spearman Rank Order Tests both resulted in p-values of 0.014. Increasing trends in peak annual streamflow suggest that an upward shift in the flow frequency curve has been occurring since impoundment (1939). If this upward trend continues then the assumed frequency of spillway overtopping could be increasing during the life of the project. This suggests that there is a risk that the spillways could be overtopped more than the assumed 8.5 times (17% annual chance exceedance) during the 50-year project life as a result of climate change. A risk of increased overtopping frequency during the growing season increases the risk of more frequent loss in vegetation and associated migratory waterfowl habitat. Floodplain impact and controlled overtopping design constraints prevent opportunities for increasing the spillway elevation to mitigate for this potential risk to increased overtopping frequency. The results from this analysis are qualitative.

### E. Climate Change Assessment Conclusions

Available literature suggests a wetter and warmer climate in the future for the Project area. Changes in discharge frequency results in potential changes to the frequency of spillway overtopping at the Project. Current USACE tools for the qualitative assessment of climate change indicate there have been increases in peak streamflow and that increases to peak streamflow are expected to continue. The Non-stationarity Detection Tool identified a statistically significant increasing trend in observed annual peak streamflow (p-value=0.014) and the Climate Hydrology Assessment Tool identified a statistically significant upward trend in both observed annual peak instantaneous streamflow (p-value=0.0009347) and mean projected annual maximum monthly streamflow (p-value<0.0001).

As previously discussed, opportunities to improve the Project's resilience by decreasing the risk of spillway overtopping are limited due to floodplain and controlled overtopping design constraints. Resilience to the potential of increased overtopping frequency is provided by the controlled overtopping design that reduces the risk of berm and interior damages associated with overtopping events.

## XVII. FLOODPLAIN IMPACT ANALYSIS

The Project is located within the floodway within which the Federal Emergency Management Agency requires "no rise" to the 1 percent exceedance probability water surface profile. The State of Illinois requires an evaluation of the maximum impact, rather than impacts at a specified frequency event (i.e. 1 percent exceedance probability event). The maximum impact cannot exceed 0.04 foot. Five different historic events that produce water surface profiles ranging in elevation from below the proposed spillway elevation to well above the perimeter berm elevation were selected to evaluate the maximum water surface profile impact (Figure H-40).

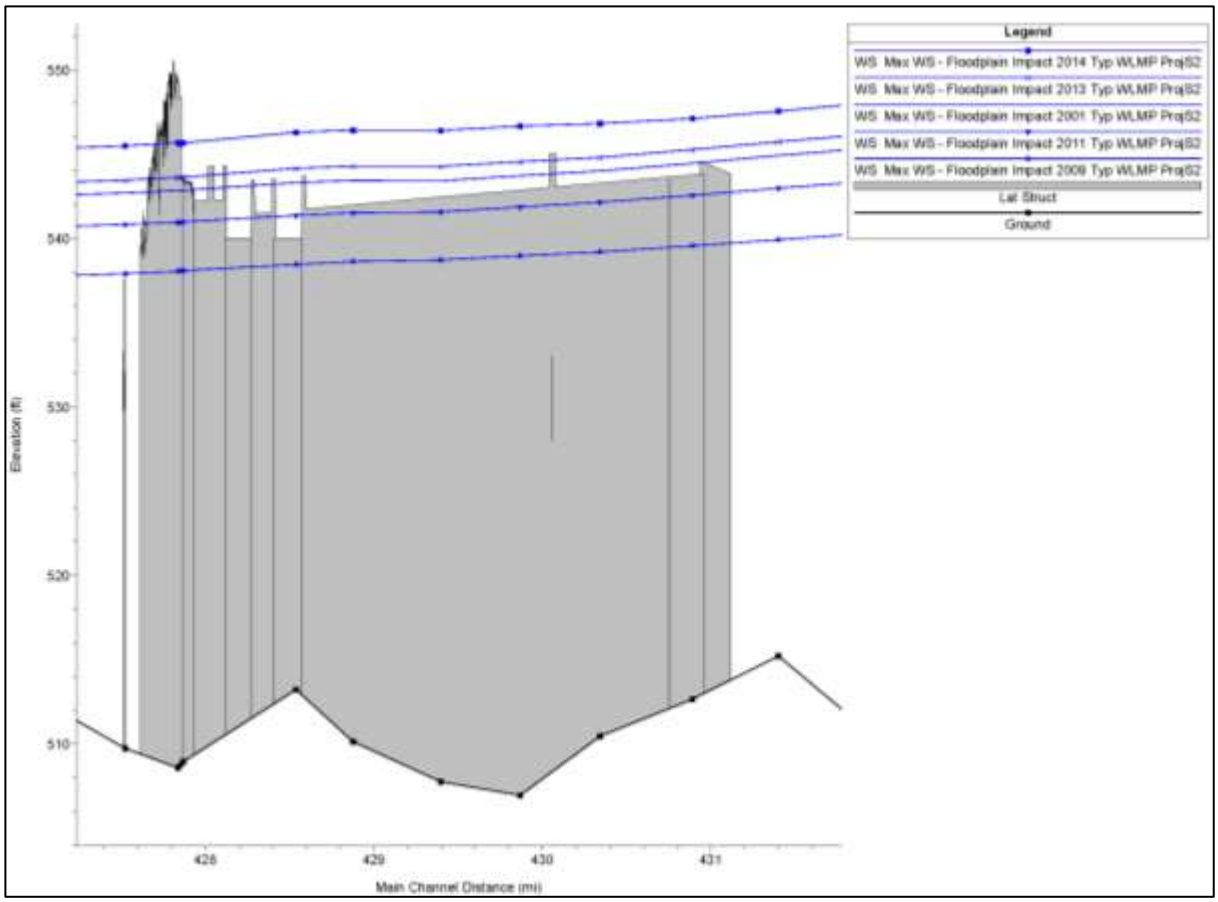


**Figure H-40.** Computed Water Surface Profiles Evaluated for Floodplain Impact Analysis

The initial perimeter berm design was evaluated for floodplain impacts and resulted in a maximum water surface profile impact greater than 0.04 foot. Design revisions were made to the perimeter berm design as described in *X. Perimeter Berm and Controlled Overtopping Design* and re-evaluated for floodplain impacts to show "no-rise" to the percent exceedance probability water surface elevation. This revised design includes a 4% exceedance probability design elevation at the northwest corner of the berm sloping down to a 4% exceedance probability minus 1-foot design elevation at the upstream end of the downstream spillway. Downstream of the downstream spillway the berm design elevation will follow the 4% exceedance probability design elevation as in the original design. This is the perimeter berm design that was adopted for the feasibility design and cost estimate. The floodplain analysis of this feasibility design resulted in a maximum impact of 0.04 foot from the 2014 event at river stations 432 and 433.4, located upstream of the Project (Figures H-41 and H-42).

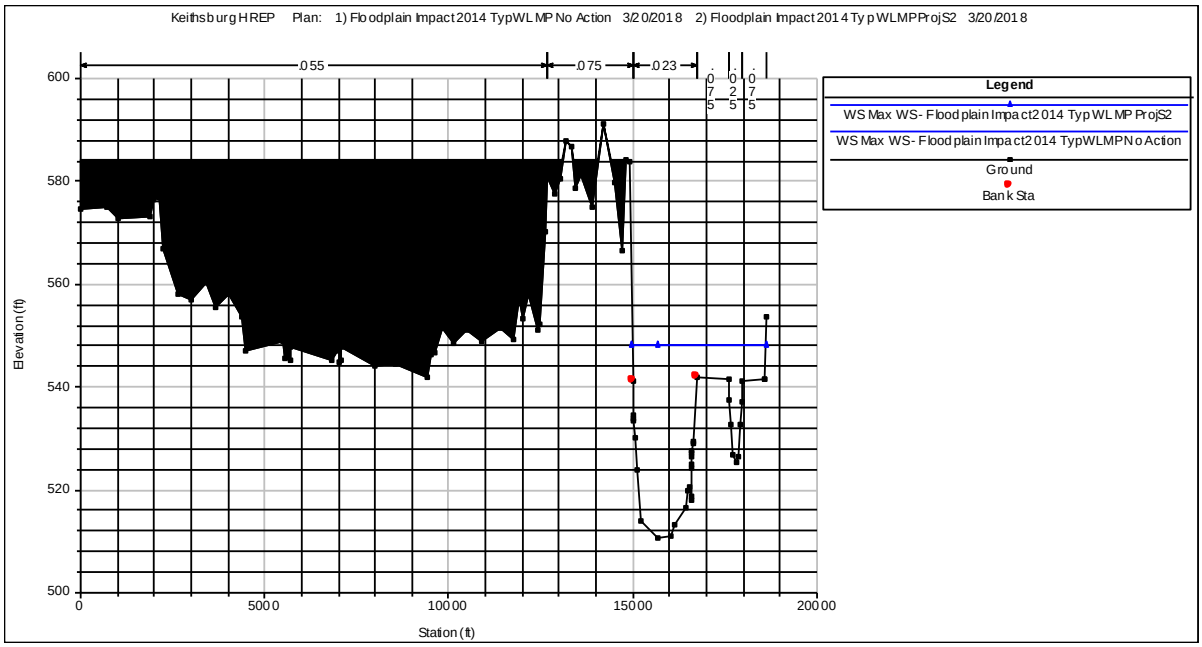


**Figure H-41:** 2014 Water Surface Profile Comparison of Existing vs. With-Project for River Station 432 Showing Maximum Impact

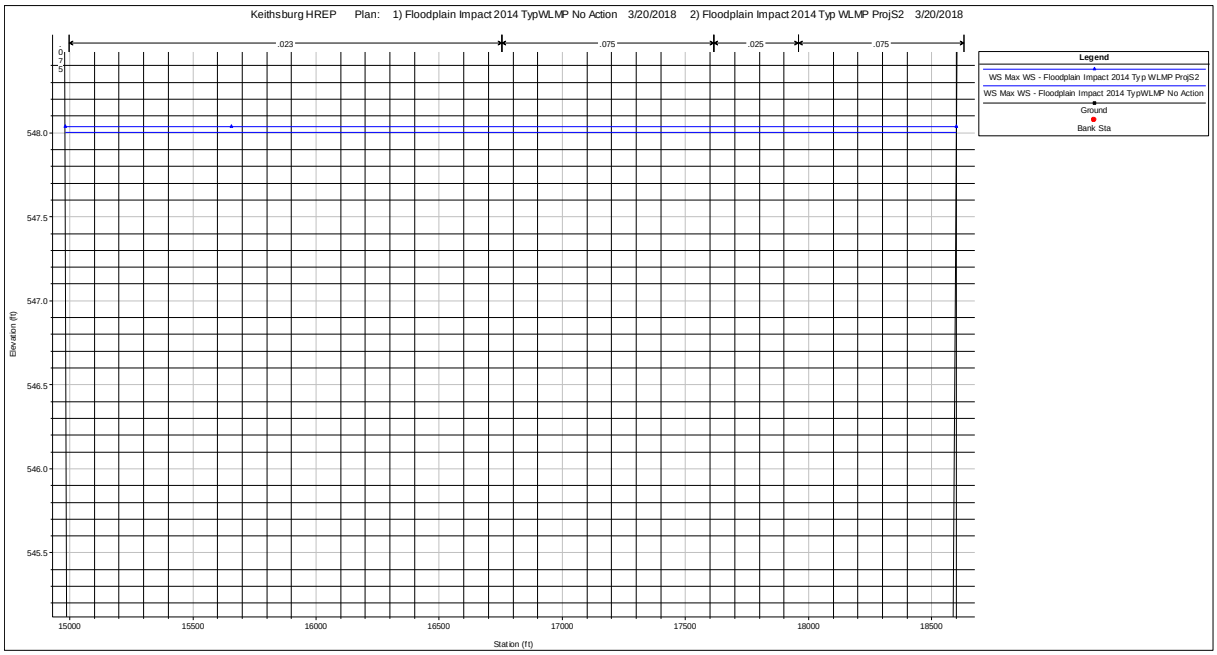


**Figure H-42:** Zoomed-In Comparison of 2014 Water Surface Profiles Showing 0.04' Maximum Impact at River Station 432

The floodplain analysis results above are for the feasibility design. However, additional revisions to be incorporated during plans and specifications, (as described in Section X, *Perimeter Berm and Controlled Overtopping Design)*, necessary to ensure hydraulic superiority were not re-evaluated for floodplain impacts. These revisions include maintaining 2' of berm superiority between the upstream and downstream spillways in order to force initial overtopping to occur upstream of the upstream spillway. Additionally, lowering the spillway elevation to 539.5' was not included in the floodplain evaluation. This design incorporating all revisions as described above will be evaluated for floodplain impacts during plans and specifications.

## XVIII. OUTSTANDING DESIGN FOR PLANS AND SPECIFICATIONS

Additional analysis of the following project components will be required to support the design phase of the project:

- Culvert design analysis is needed for the replacement of Culvert #2 to ensure a positive sloping culvert that provides inlet control. Recent 2016 survey indicated this culvert was replaced and does not have a positive slope in the downstream direction.
- Final sluice gate design requires a sedimentation evaluation.
- Final spillway design should be verified using the filling spreadsheet model or HEC-RAS to simulate spillway overtopping and interior filling to ensure the uncontrolled overtopping head differential (1') design constraint is met.
- Final floodplain impact analysis of the feasibility design that incorporates all revisions as described in *X. Perimeter Berm and Controlled Overtopping Design* must be completed.

## XIX. REFERENCES

Cai X, Wang D, Laurent R. (2009). *Impact of climate change on crop yield: A case study of rainfed corn in central Illinois.* Journal of Applied Meteorology and Climatology 48:1868-1881.

Chin, D.A. (2013). Water Resources Engineering. Third Edition

Döll P, Zhang J. (2010). *Impact of climate change on freshwater ecosystems: a global-scale analysis of ecologically relevant river flow alterations.* Hydrol. Earth Syst. Sci. Discuss. 7:1305-1342.

Duan Q, Schaake J, Andreassian V, Franks S, Goteti G, Gupta HV, Gusev YM, Habets F, Hall A, Hay L, Hogue T, Huang M, Leavesley G, Liang X, Nasonova ON, Noilhan J, Oudin L, Sorooshian S, Wagener T, Wood EF. (2006). *Model Parameter Estimation Experiment. (MOPEX): An overview of science strategy and major results from the second and third workshops.* Journal of Hydrology 320: 3-17.

Eckblad, J. W., N. L. Petersen, K. Ostlic, and A. Tempte. (1977). The morphometry, benthos, and sedimentation rates of a floodplain lake in Pool 9 of the Upper Mississippi River. American Midlands Naturalist 97:433-443.

Gao Y, Fu JS, Drake JB, Liu Y, Lamarque JF. (2012). *Projected changes of extreme weather events in the eastern United States based on a high resolution climate modeling system.* Environmental Research Letters 7.

Grundstein A. (2009). *Evaluation of climate change over the continental United States using a moisture index.* Climatic Change 93:103-115.

Hagemann S, Chen C, Clark DB, Folwell S, Gosling SN, Haddeland I, Hanasaki N, Heinke J, Ludwig F, Voss F, Wiltshire AJ. (2013). *Climate change impact on available water resources obtained using multiple global climate and hydrology models.* Earth System Dynamics 4:129-144.

http://corpsmapu.usace.army.mil/cm_apex/f?p=257:2:0::NO:

Illinois Administrative Code Title 17, Chapter I h, Part 3700, *Construction in floodways of rivers, lakes and streams.*

Jha M, Arnold JG, Gassman PW, Giorgi F, Gu RR. (2006). *Climate change sensitivity assessment on Upper Mississippi River Basin streamflows using SWAT.* Journal of the American Water Resources Association 42:997-1016.

Joetzjer E, Douville H, Delire C, Ciais P, Decharme B, Tyteca S. (2013). *Hydrologic benchmarking of meteorological drought indices at interannual to climate change timescales: A case study over the Amazon and Mississippi river basins.* Hydrology and Earth System Sciences 17:4885-4895.

Johnson SL, Stefan HG. (2006). *Indicators of climate warming in Minnesota: Lake ice covers and snowmelt runoff.* Climatic Change 75:421-453.

Korschgen, C. E., G. E. Jackson, L. F. Muessig, & D. C. Southworth. (1987). Sedimentation in Lake Onalaska, Navigation Pool 7, Upper Mississippi River, since impoundment. Water Resources Bulletin 23:221-226.

Kunkel KE, Liang X-Z, Zhu J. (2010). *Regional climate model projections and uncertainties of U.S. summer heat waves.* Journal of Climate 23:4447-4458.

Liu Y, Goodrick SL, Stanturf JA. (2013). *Future U.S. wildfire potential trends projected using a dynamically downscaled climate change scenario.* Forest Ecology and Management 294:120-135.

Mauget SA. (2004). *Low frequency streamflow regimes over the central United States: 1939-1998.* Climatic Change 63:121-144.

McHenry J. R., J. C. Ritchie, C. M. Cooper, & J. Verdon. (1984). Recent rates of sedimentation in the Mississippi River. Pages 99-117 in J. G. Wiener, R. V. Anderson, and D. R. McConville, editors. Contaminants in the Upper Mississippi River. Butterworth Publishers, Stoneham, Massachusetts.

McRoberts DB, Nielsen-Gammon JW. (2011). *A new homogenized climate division precipitation dataset for analysis of climate variability and climate change.* Journal of Applied Meteorology and Climatology 50:1187-1199.

Palecki MA, Angel JR, Hollinger SE. (2005). *Storm precipitation in the United States. Part I: Meteorological characteristics.* Journal of Applied Meteorology 44:933-946.

Pryor SC, Howe JA, Kunkel KE. (2009). *How spatially coherent and statistically robust are temporal changes in extreme precipitation in the contiguous USA?* International Journal of Climatology 29:31-45.

Pryor, S. C., D. Scavia, C. Downer, M. Gaden, L. Iverson, R. Nordstrom, J. Patz, and G. P. Robertson. (2014). Ch. 18: Midwest. *Climate Change Impacts in the United States: The Third National Climate Assessment*, J. M. Melillo, Terese (T.C.). Richmond, and G. W. Yohe, Eds., U.S. Global Change Research Program, 418-440. doi:10.7930/J0J1012N.

Qian T, Dai A, Trenberth KE. (2007). *Hydroclimatic trends in the Mississippi River basin from 1948 to 2004.* Journal of Climate 20:4599-4614.

Rogala, J. R. & P. J. Boma. (1996). Rates of sedimentation along selected backwater transects in Pools 4, 8, and 13 of the Upper Mississippi River. U.S. Geological Survey, Environmental Management Technical Center, Onalaska, Wisconsin, October 1996. LTRMP 96-T005. 24 pp. (NTIS-#PB97-122105).

Rogala, J. R., W. F. James, & H. L. Eakin. (1997). Rates of net fine sediment accumulation in selected backwaters of Pool 8, Upper Mississippi River. Interim report for the Upper Mississippi River–Illinois Waterway System Navigation Study prepared for the U.S. Army Corps of Engineers, October 1997. 28 pp.

Scherer M, Diffenbaugh N. (2014). *Transient twenty-first century changes in daily-scale temperature extremes in the United States.* Climate Dynamics 42:1383-1404.

Schuster ZT, Potter KW, Liebl DS. (2012). *Assessing the effects of climate change on precipitation and flood damage in Wisconsin.* Journal of Hydrologic Engineering 17:888-894.

Schwartz MD, Ault TR, Betancourt JL. (2013). *Spring onset variations and trends in the continental United States: Past and regional assessment using temperature-based indices.* International Journal of Climatology 33:2917-2922.

Tebaldi C. (2006). *Going to the Extremes: An Intercomparison of Model-Simulated Historical and Future Changes in Extreme Events.* Climate Change 79:185-211. 2006

U.S. Army Corps of Engineers. (June 2000). Upper Mississippi River and Illinois Waterway Cumulative Effects Study, Volume I: Geomorphic Assessment.

(January 2004). *Upper Mississippi River System Flow Frequency Study, Final Report*, Rock Island District, Rock Island, Illinois.

http://www.mvr.usace.army.mil/Portals/48/docs/FRM/UpperMissFlowFreq/Upper%20Mississippi%20River%20System%20Flow%20Frequency%20Study%20Main%20Report.pdf

(2015). *Recent US Climate Change and Hydrology Literature Applicable to US Army Corps of Engineers Missions – Water Resources Region 07, Upper Mississippi*. Civil Works Technical Report, CWTS-2015-13, Washington, DC.

(July 2015). Hydrologic Engineering Center's Hydrologic Modeling System. Version 4.1.

(February 2016). Hydrologic Engineering Center's River Analysis System. Version 5.0.

(September 2016). *ECB 2016-25: Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs and Projects*.

USACE Climate Hydrology Assessment Tool
http://corpsmapu.usace.army.mil/cm_apex/f?p=313:2:0::NO

USACE Non-Stationarity Detection Tool

USACE Vulnerability Assessment Tool https://maps.crrel.usace.army.mil/apex/f?p=201

Villarini G, Smith JA, Vecchi GA. (2013). *Changing Frequency of Heavy Rainfall over the Central United States.* Journal of Climate 26:351-357.

Wang D, Hejazi M, Cai X, Valocchi AJ. (2011). *Climate change impact on meteorological, agricultural, and hydrological drought in central Illinois.* Water Resources Research 47.

Wang H, Schubert S, Suarez M, Chen J, Hoerling M, Kumar A, Pegion P. (2009). *Attribution of the seasonality and regionality in climate trends over the United States during 1950-2000.* Journal of Climate 22:2571-2590.

Wang J, Zhang X. (2008). *Downscaling and projection of winter extreme daily precipitation over North America.* Journal of Climate 21:923-937.

Westby RM, Lee Y-Y, Black RX. (2013). *Anomalous temperature regimes during the cool season: Long-term trends, low-frequency mode modulation, and representation in CMIP5 simulations.* Journal of Climate 26:9061-9076.

Wilson CO, Weng Q. (2011). *Simulating the impacts of future land use and climate changes on surface water quality in the Des Plaines River watershed, Chicago Metropolitan Statistical Area, Illinois.* Science of the Total Environment 409:4387-4405.

Wu Y, Liu S, Abdul-Aziz OI. (2012). *Hydrological effects of the increased CO2 and climate change in the Upper Mississippi River Basin using a modified SWAT.* Climatic Change 110:977-1003.

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# UPPER MISSISSIPPI RIVER RESTORATION
# FEASIBILITY REPORT
# WITH INTEGRATED ENVIRONMENTAL ASSESSMENT

## KEITHSBURG DIVISION
## HABITAT REHABILITATION AND ENHANCEMENT PROJECT

**POOL 18, UPPER MISSISSIPPI RIVER**
**RIVER MILES 428.0 THROUGH 431.0**
**MERCER COUNTY, ILLINOIS**

## APPENDIX H
## HYDROLOGY AND HYDRAULICS

## PLATES
## HEC-RAS RESULTS

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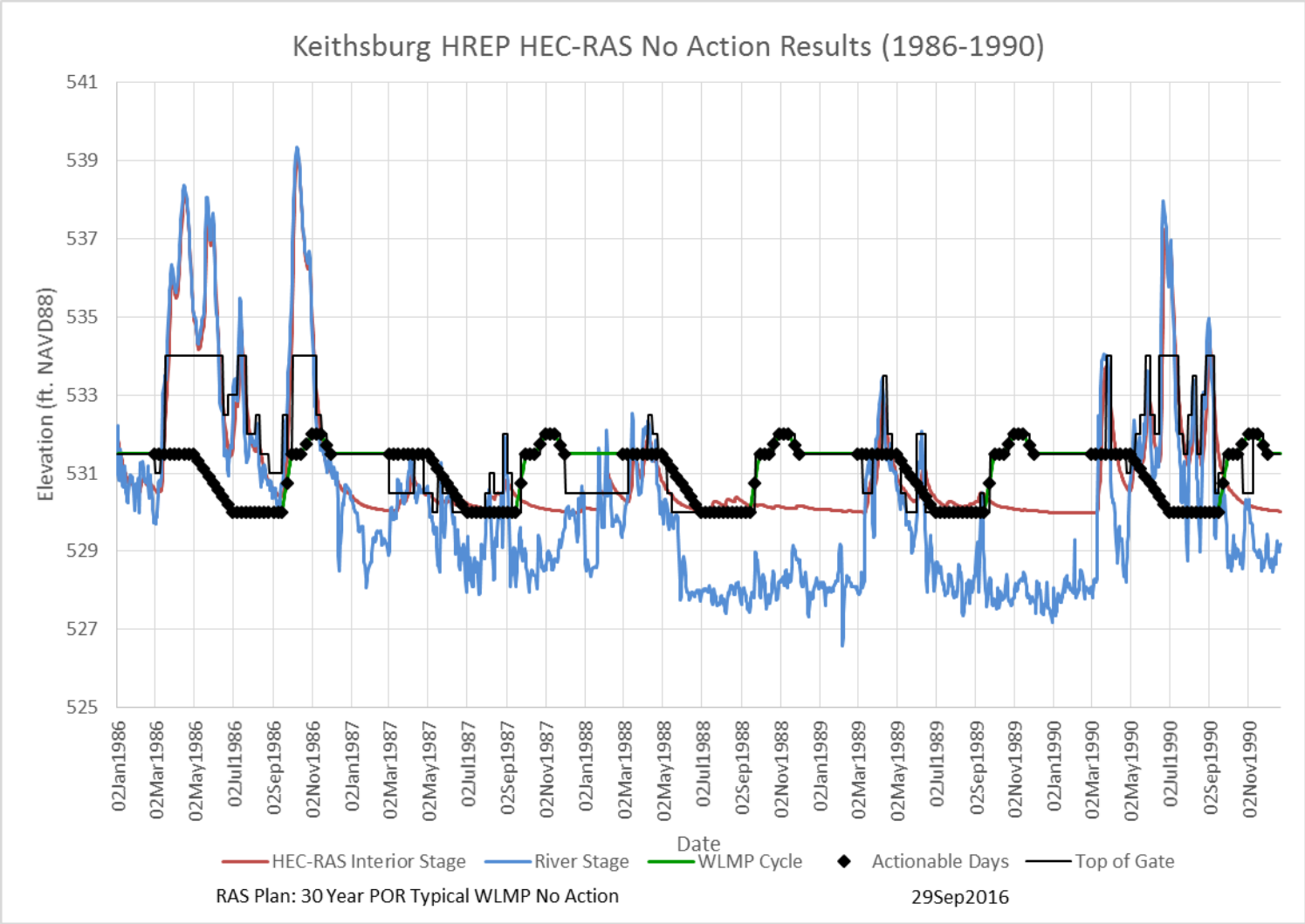


**Plate 1: HEC-RAS Results for 1986-1990 Under the Existing Conditions Alternative and the Typical WLMP**

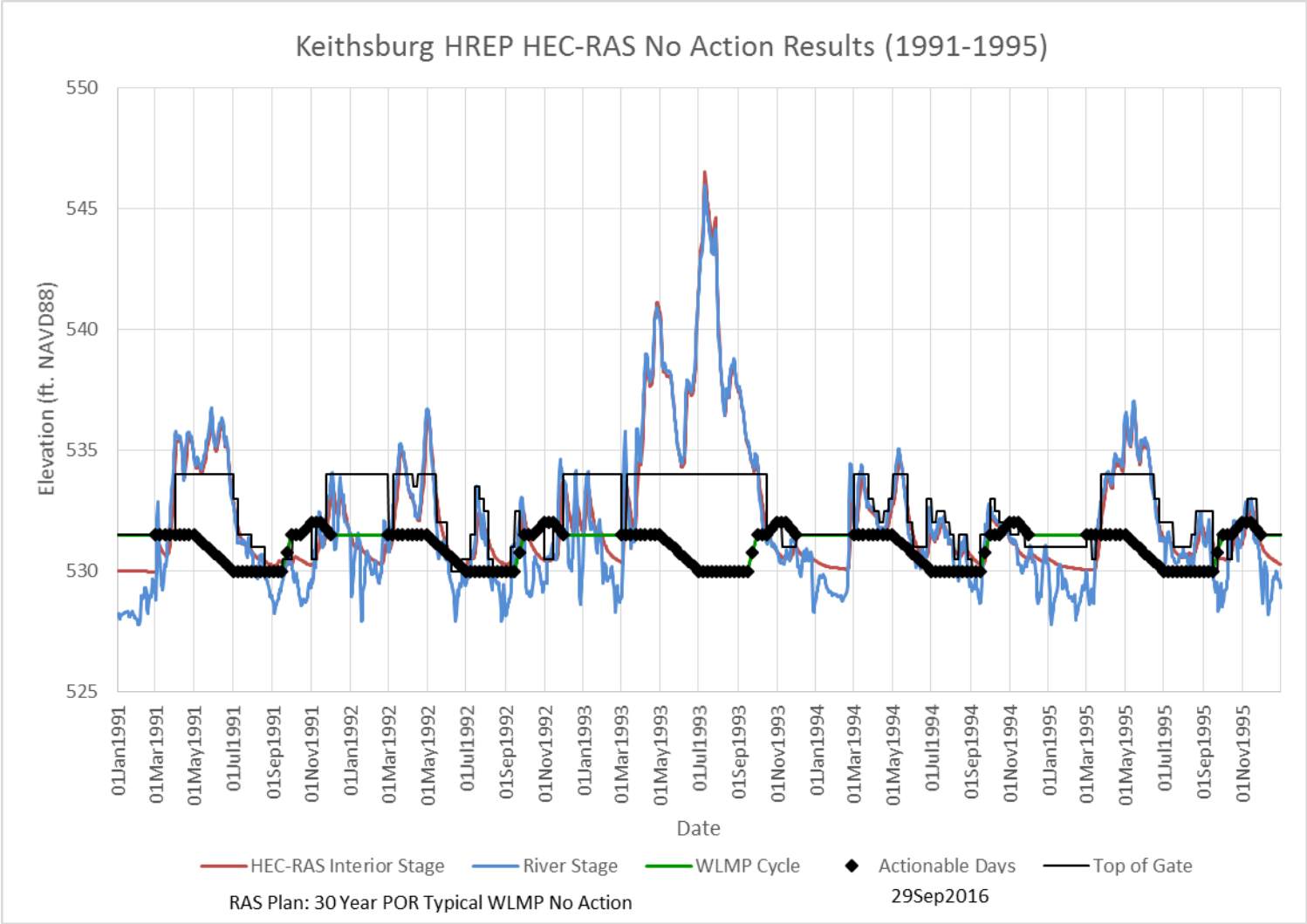


**Plate 2: HEC-RAS Results for 1991-1995 Under the Existing Conditions Alternative and the Typical WLMP**

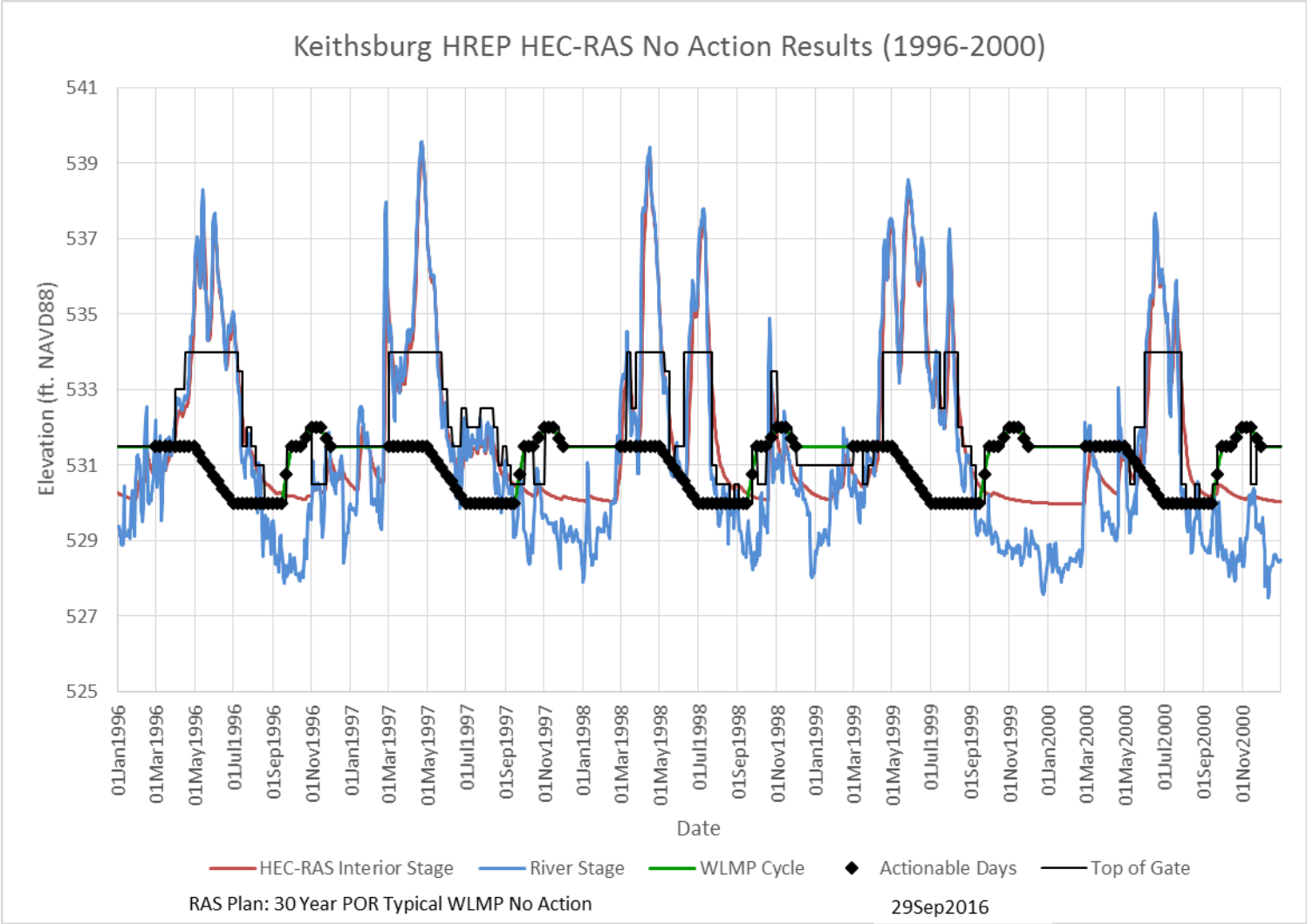


**Plate 3: HEC-RAS Results for 1996-2000 Under the Existing Conditions Alternative and the Typical WLMP**

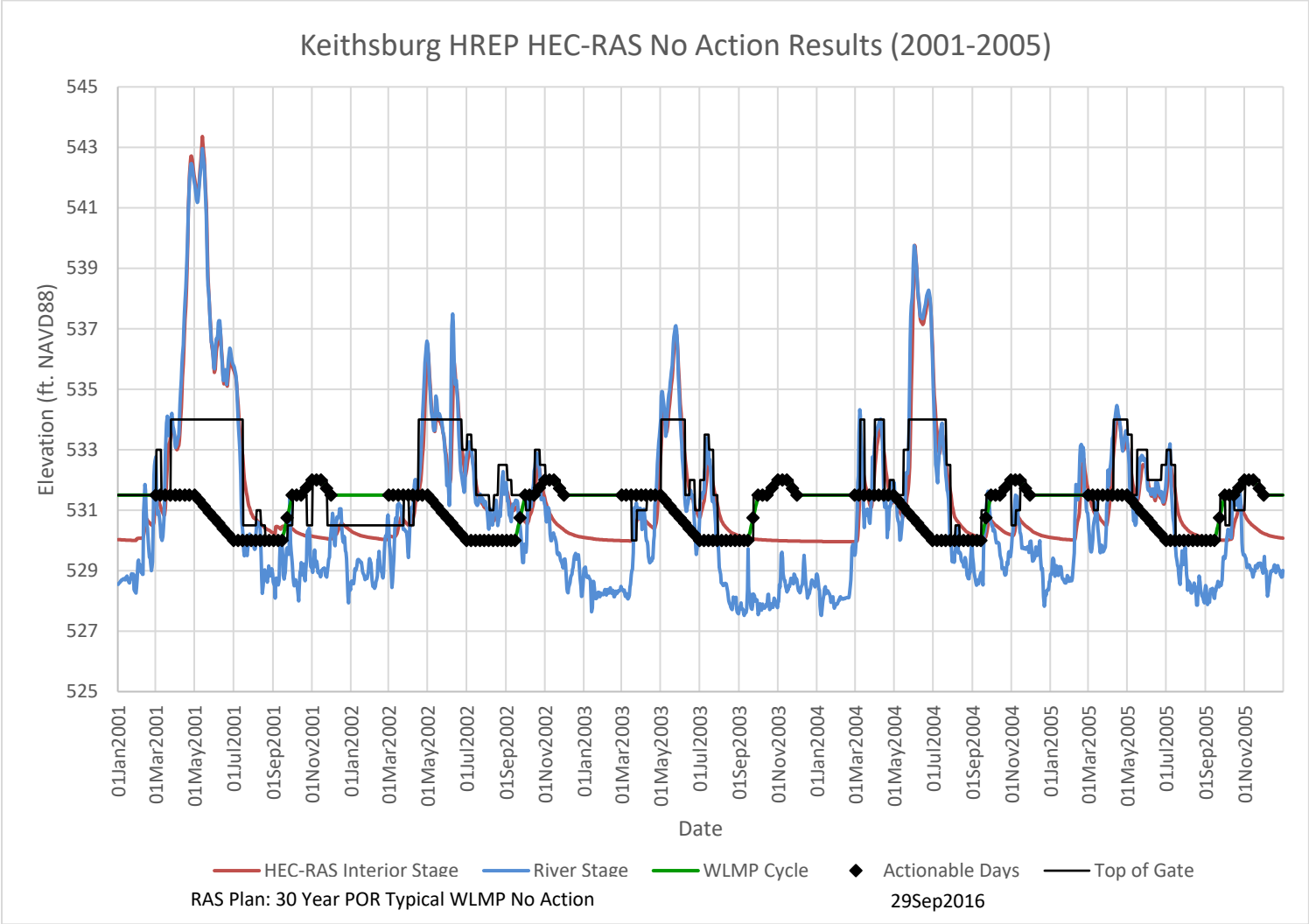


**Plate 4: HEC-RAS Results for 2001-2005 Under the Existing Conditions Alternative and the Typical WLMP**

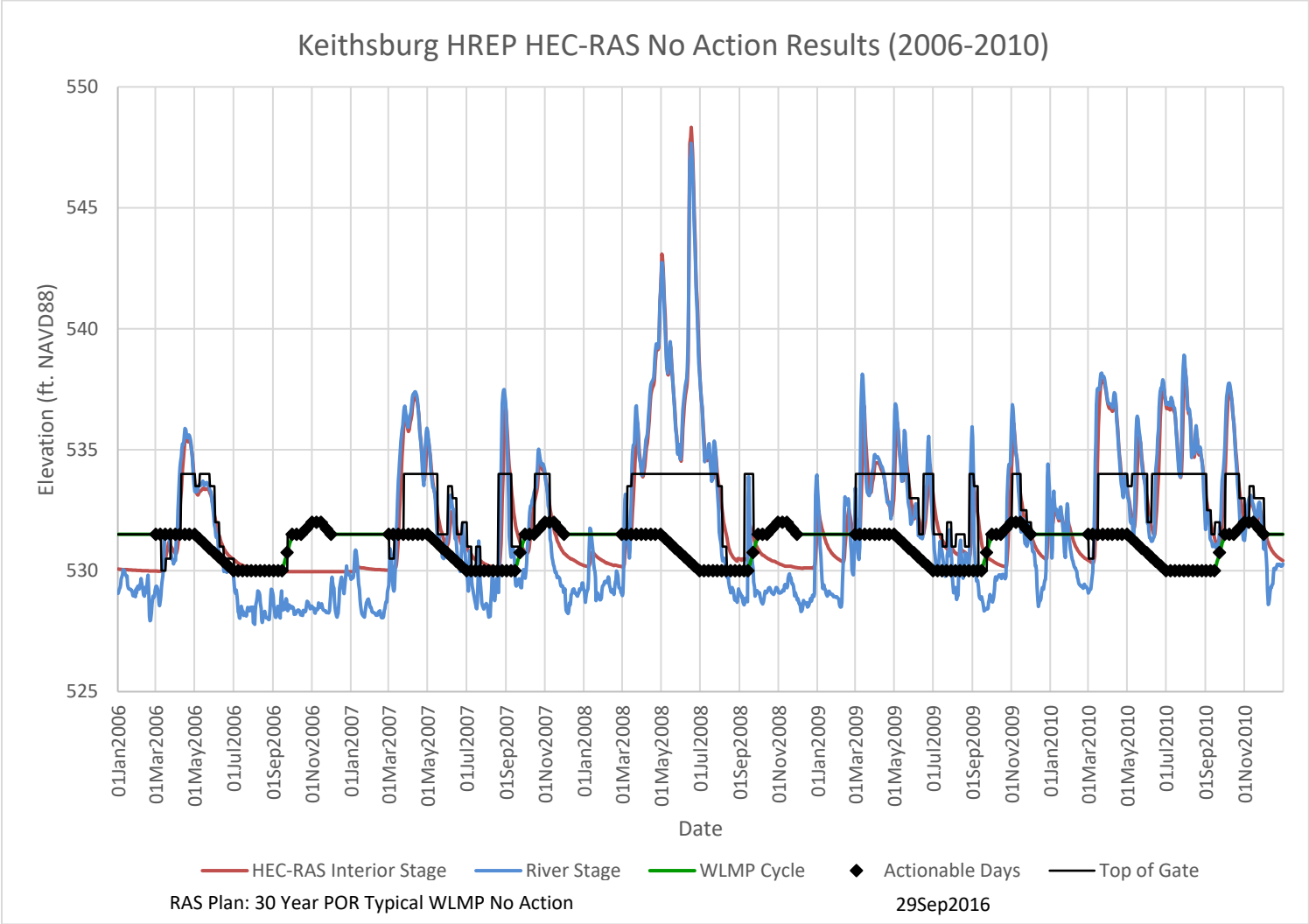


**Plate 5: HEC-RAS Results for 2006-2010 Under the Existing Conditions Alternative and the Typical WLMP**

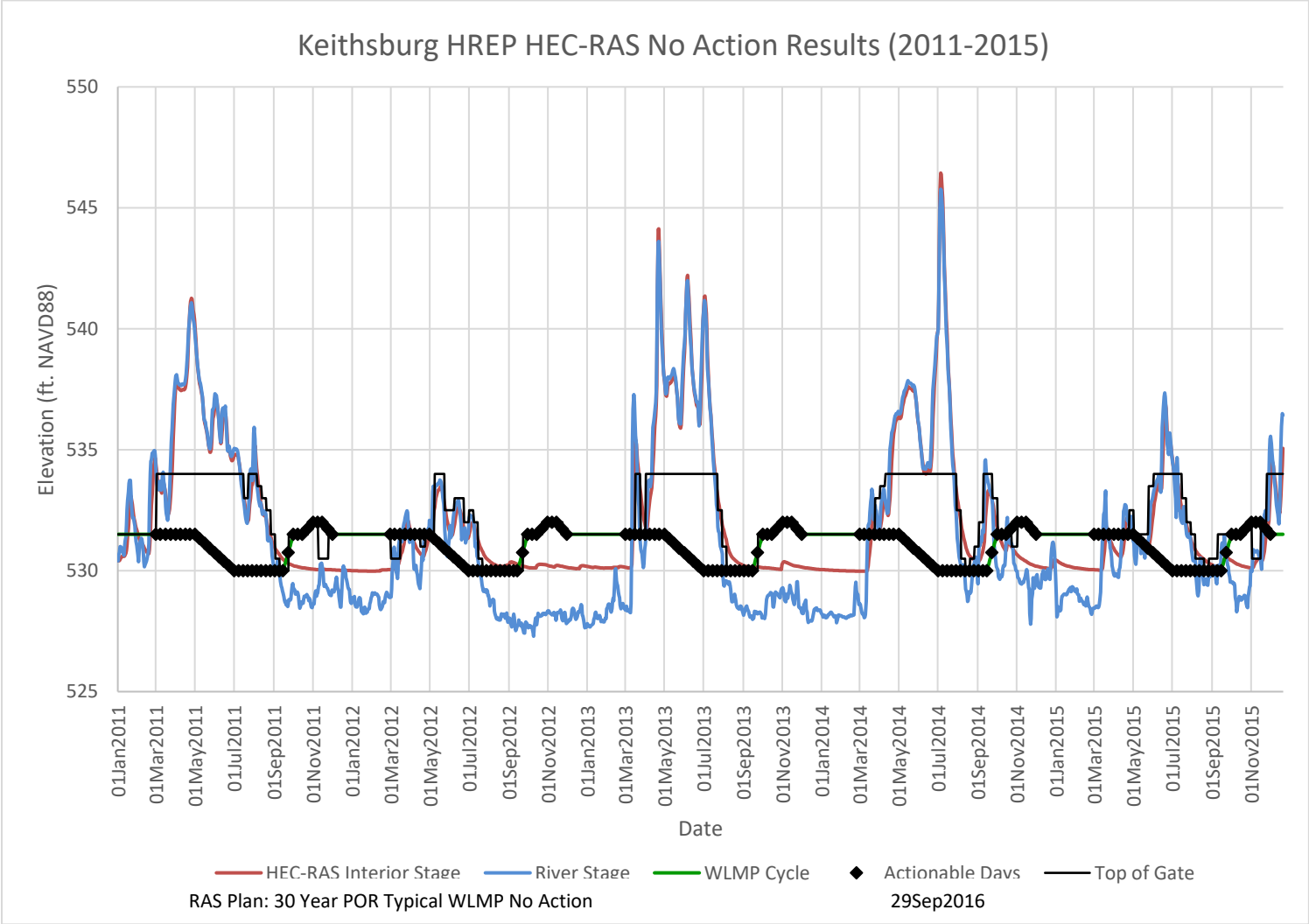


**Plate 6: HEC-RAS Results for 2011-2015 Under the Existing Conditions Alternative and the Typical WLMP**

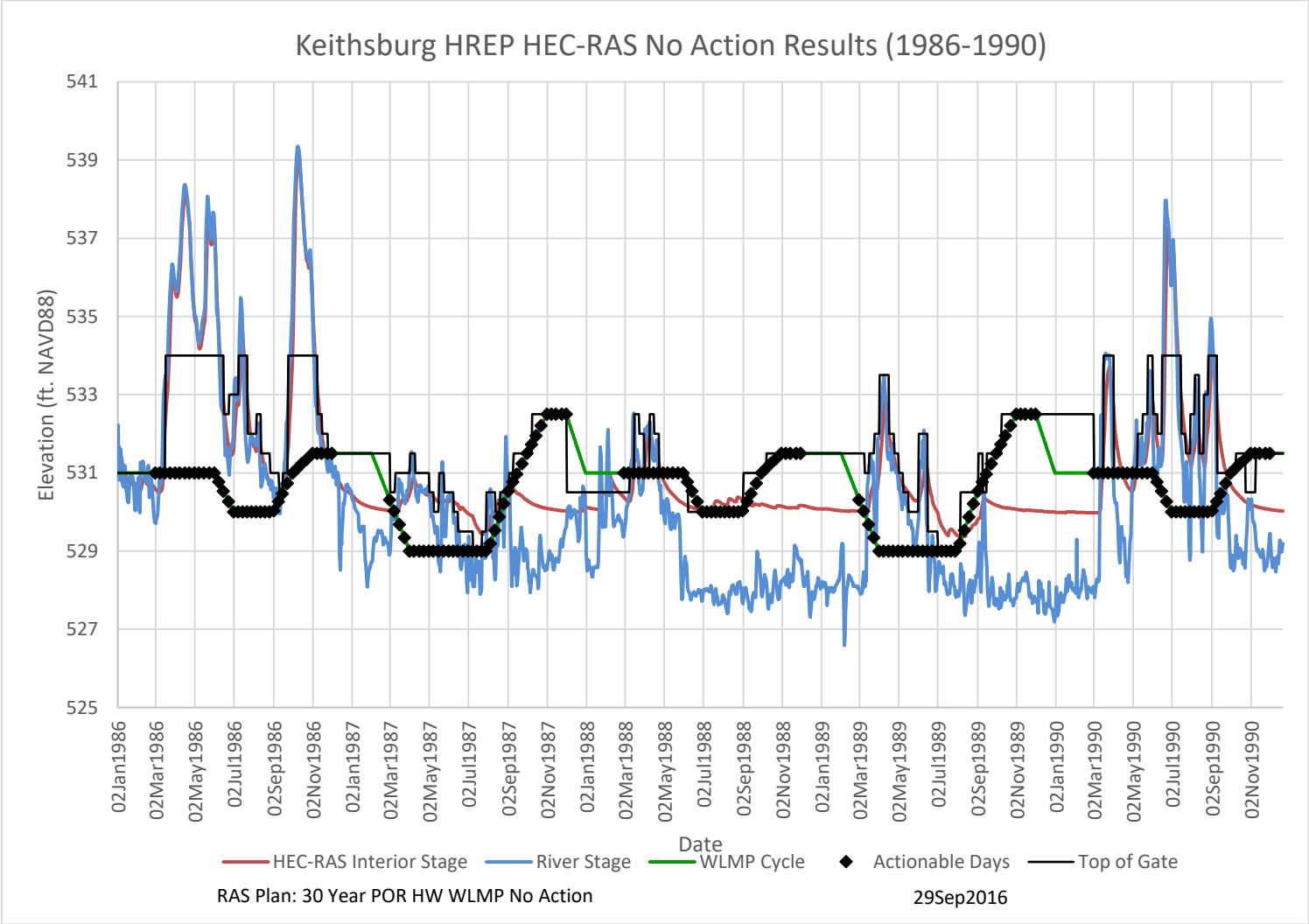


**Plate 7: HEC-RAS Results for 1986-1990 Under the Existing Conditions Alternative and the Hardwood WLMP**

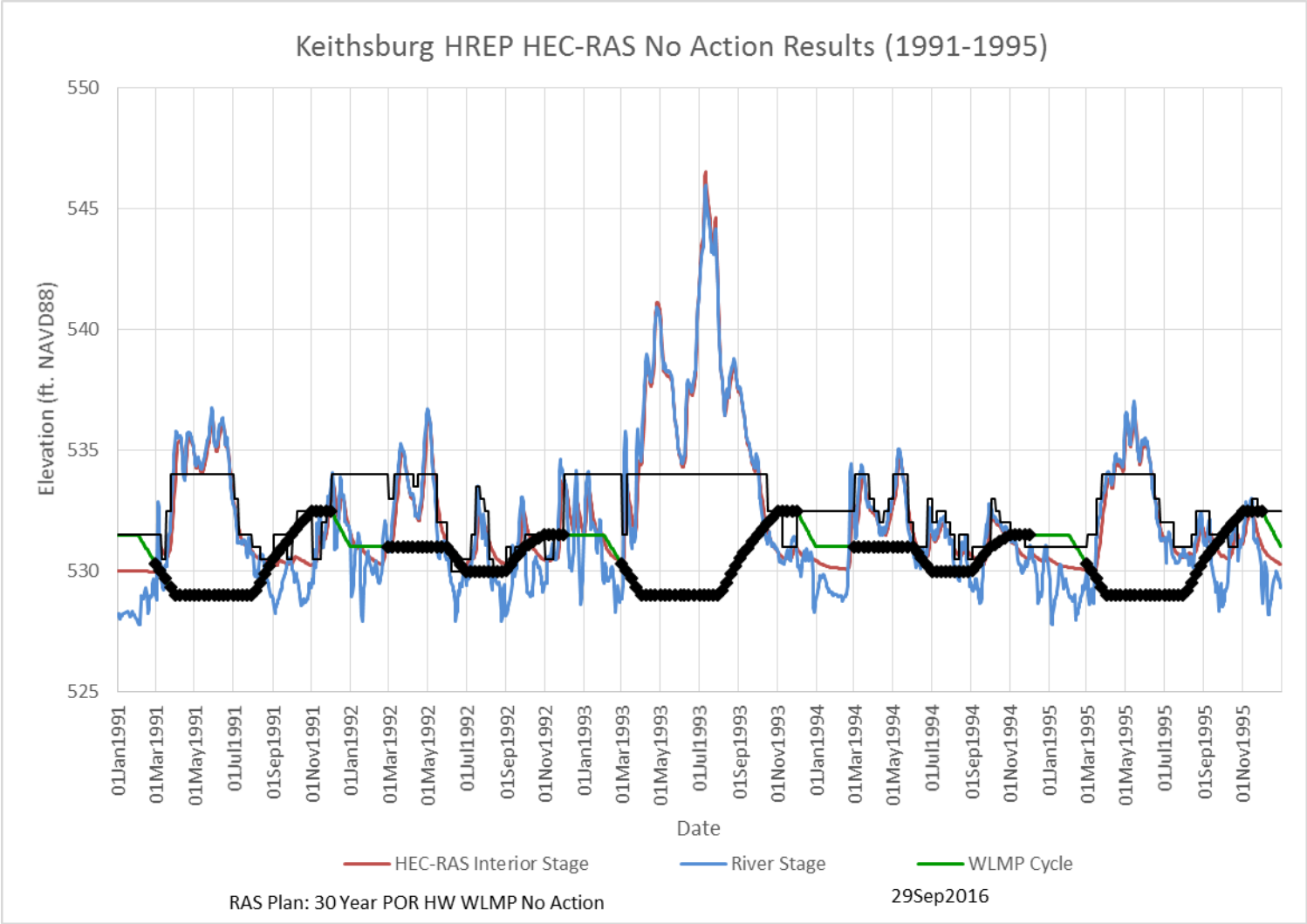


**Plate 8: HEC-RAS Results for 1991-1995 Under the Existing Conditions Alternative and the Hardwood WLMP**

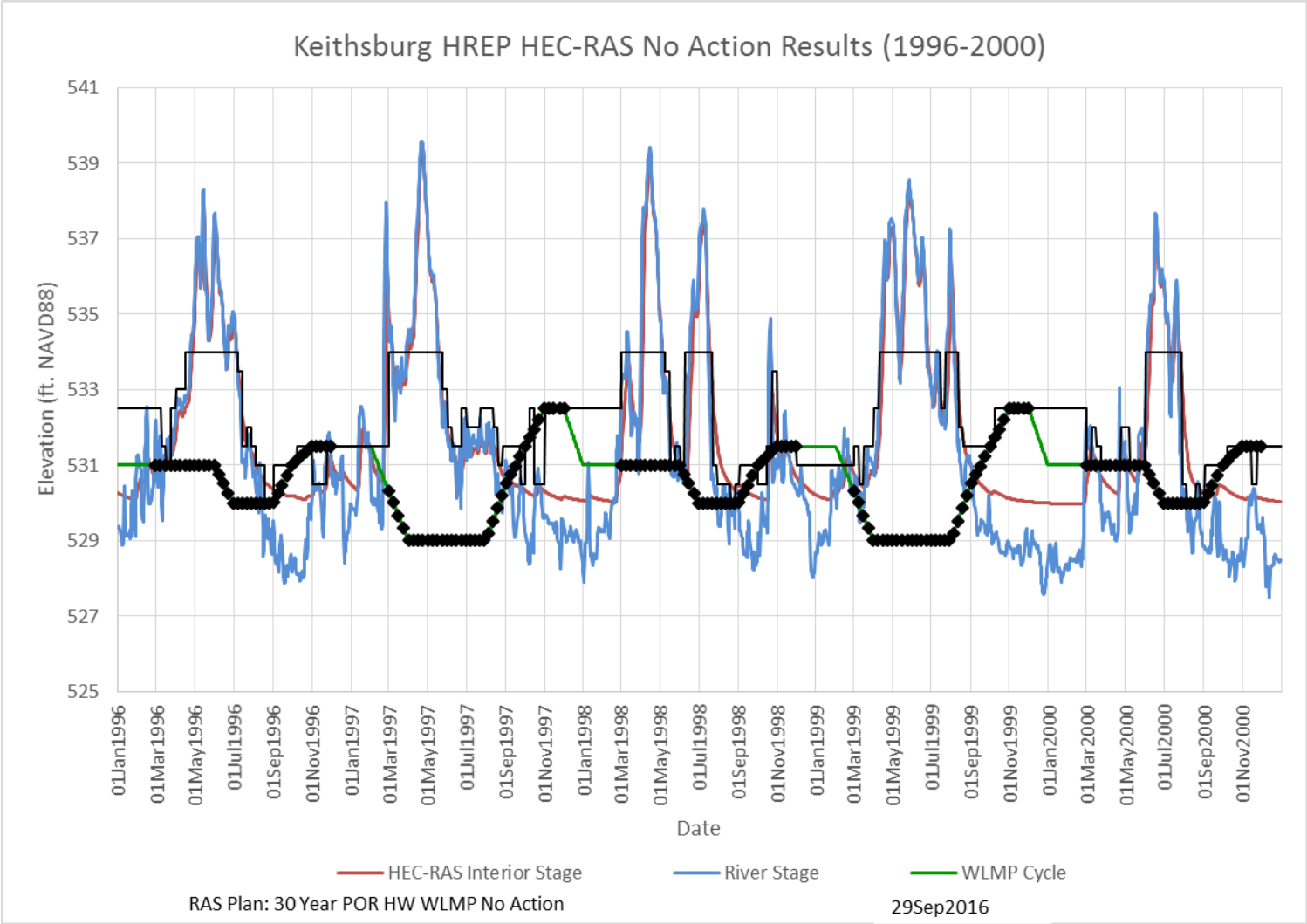


**Plate 9: HEC-RAS Results for 1996-2000 Under the Existing Conditions Alternative and the Hardwood WLMP**

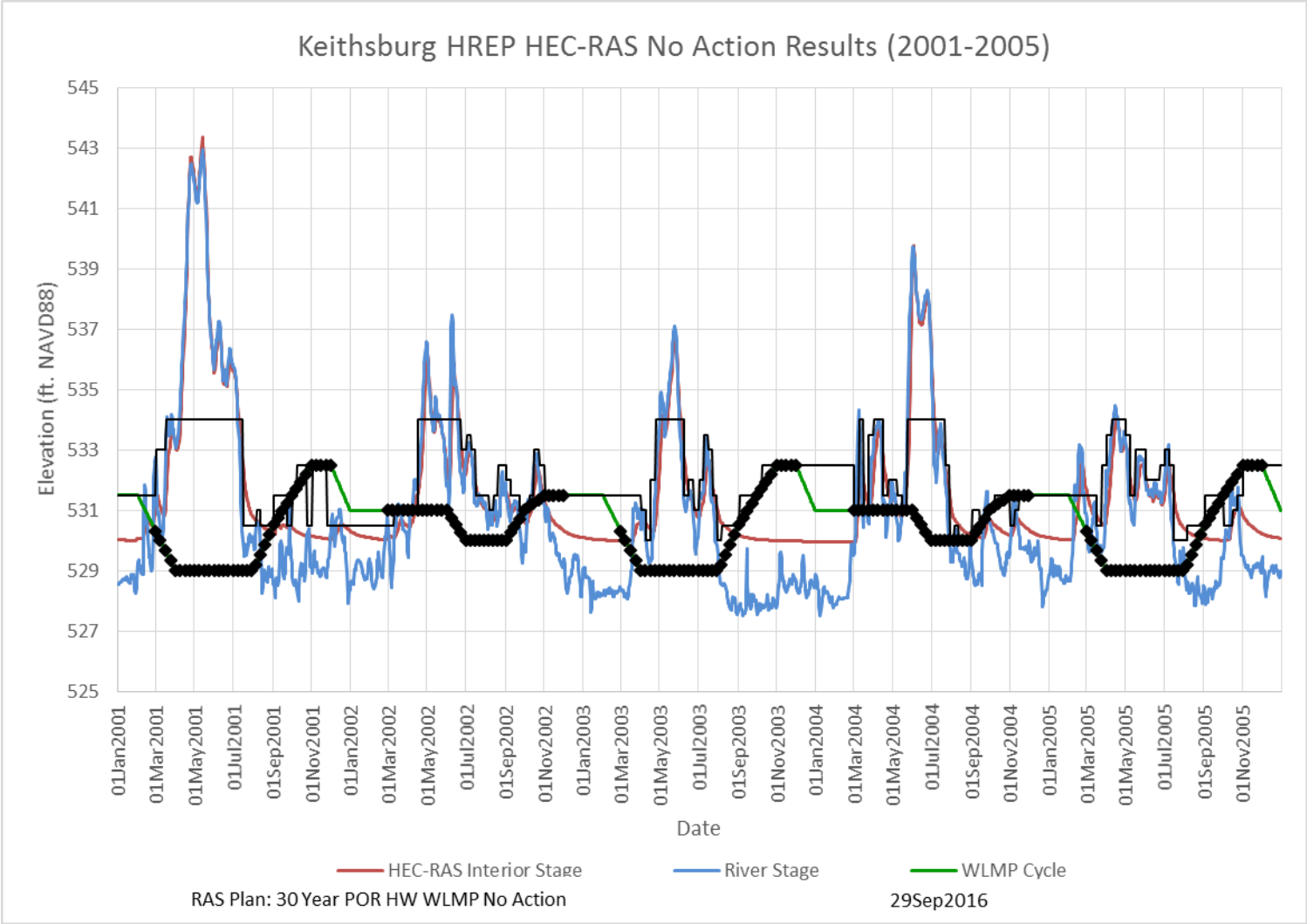


**Plate 10: HEC-RAS Results for 2001-2005 Under the Existing Conditions Alternative and the Hardwood WLMP**

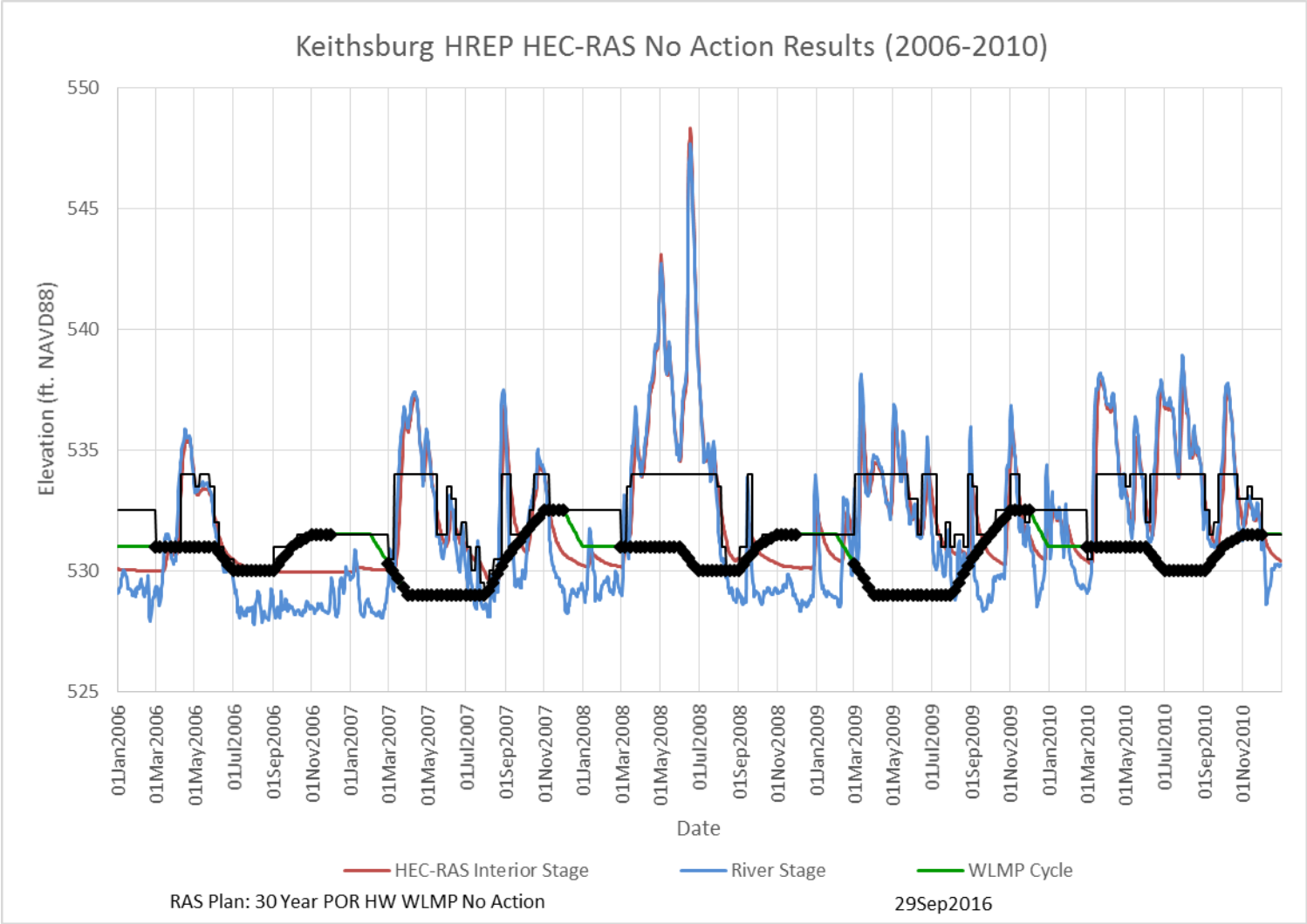


**Plate 11: HEC-RAS Results for 2006-2010 Under the Existing Conditions Alternative and the Hardwood WLMP**

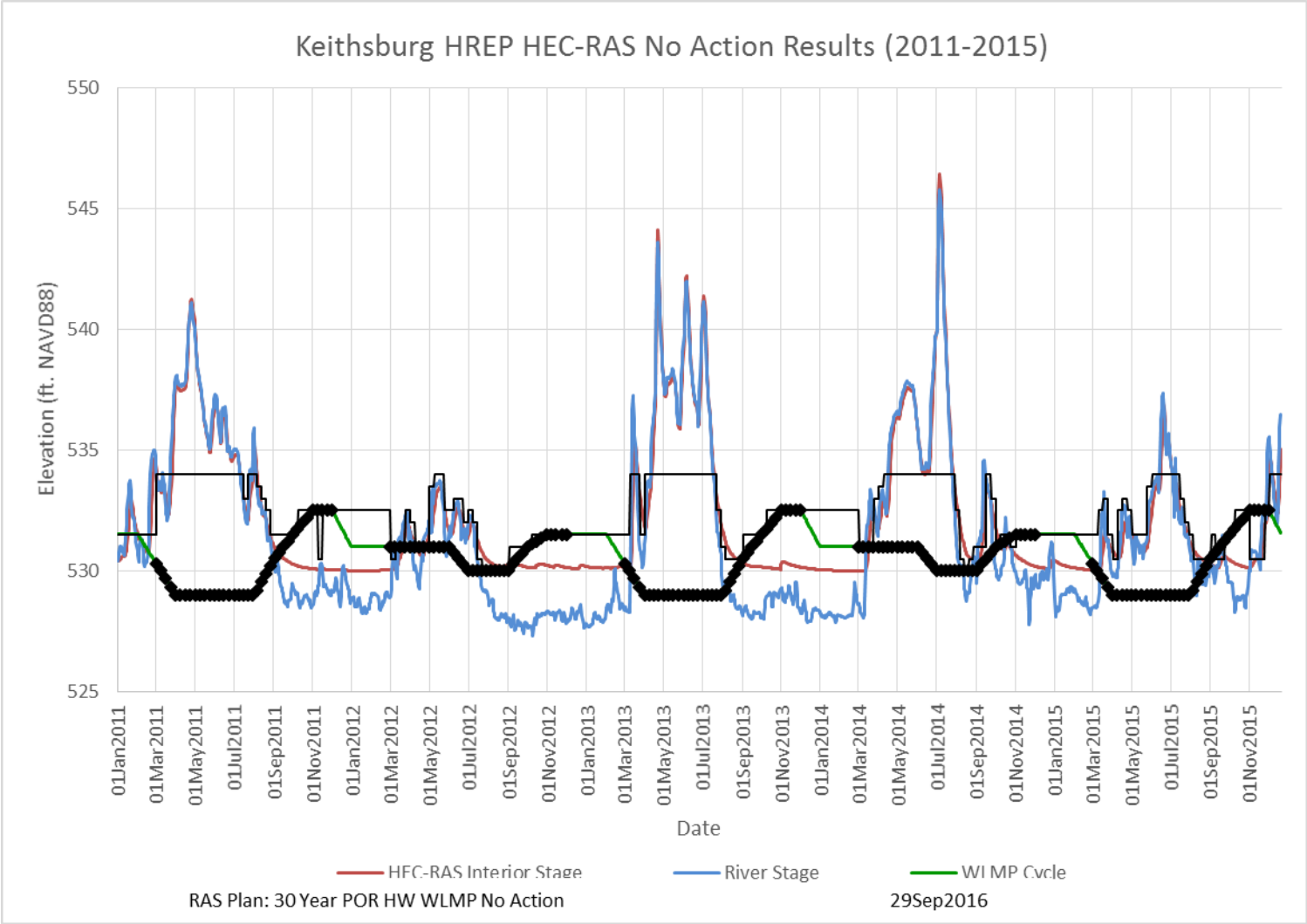


**Plate 12: HEC-RAS Results for 2011-2015 Under the Existing Conditions Alternative and the Hardwood WLMP**

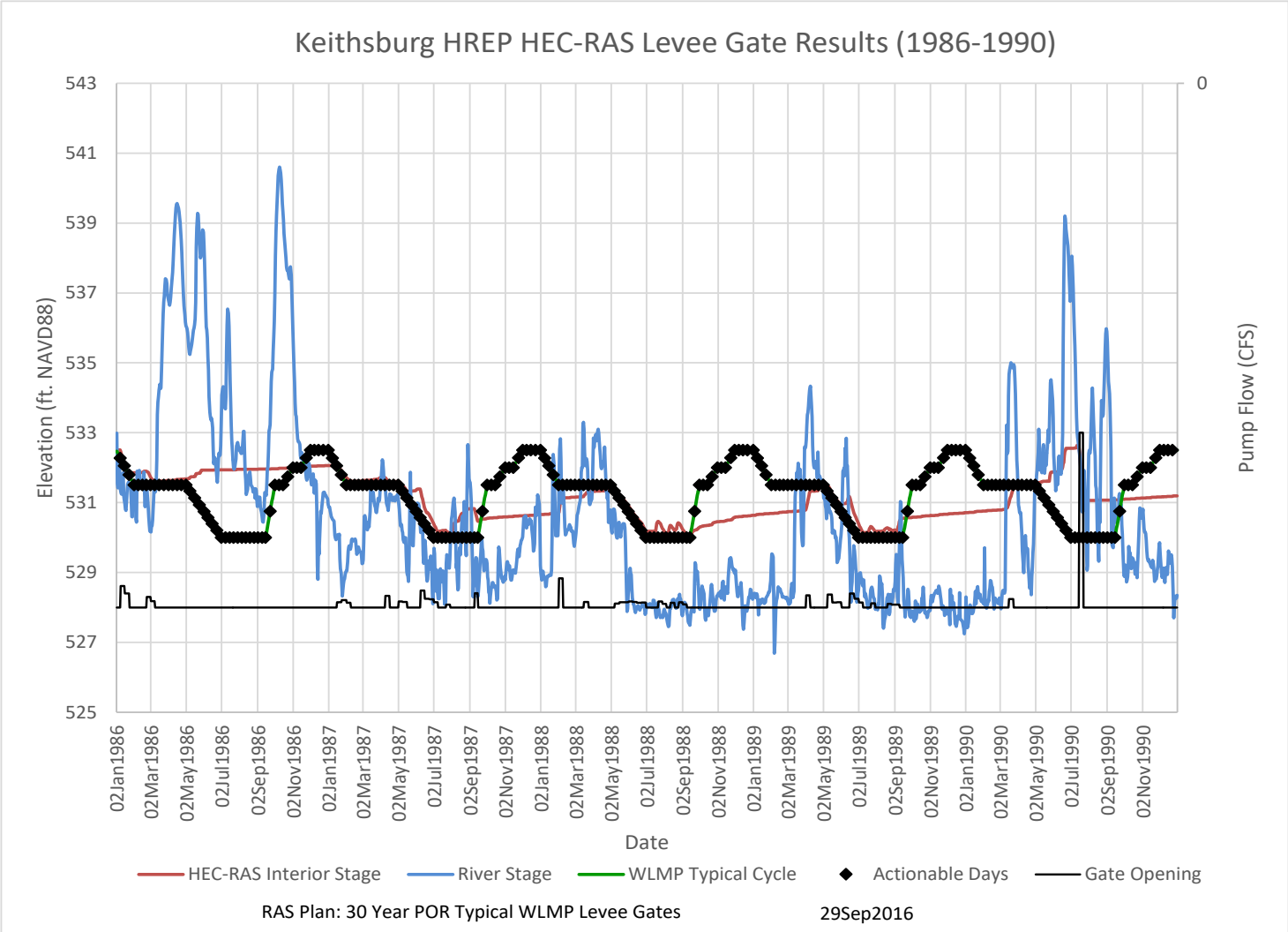


**Plate 13: HEC-RAS Results for 1986-1990 Under the Berm Improvements with Gates Alternative and the Typical WLMP**

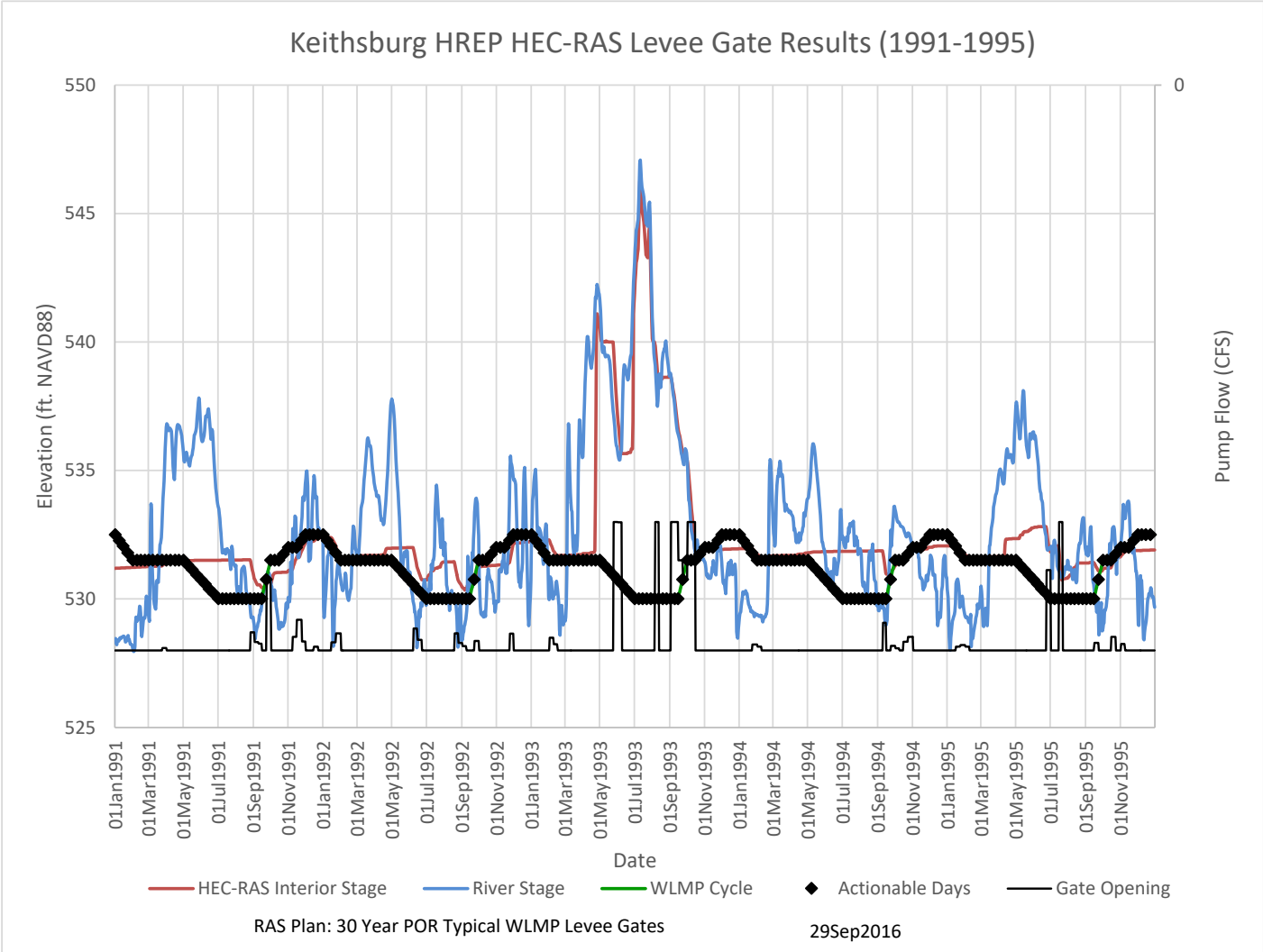


**Plate 14: HEC-RAS Results for 1991-1995 Under the Berm Improvements with Gates Alternative and the Typical WLMP**

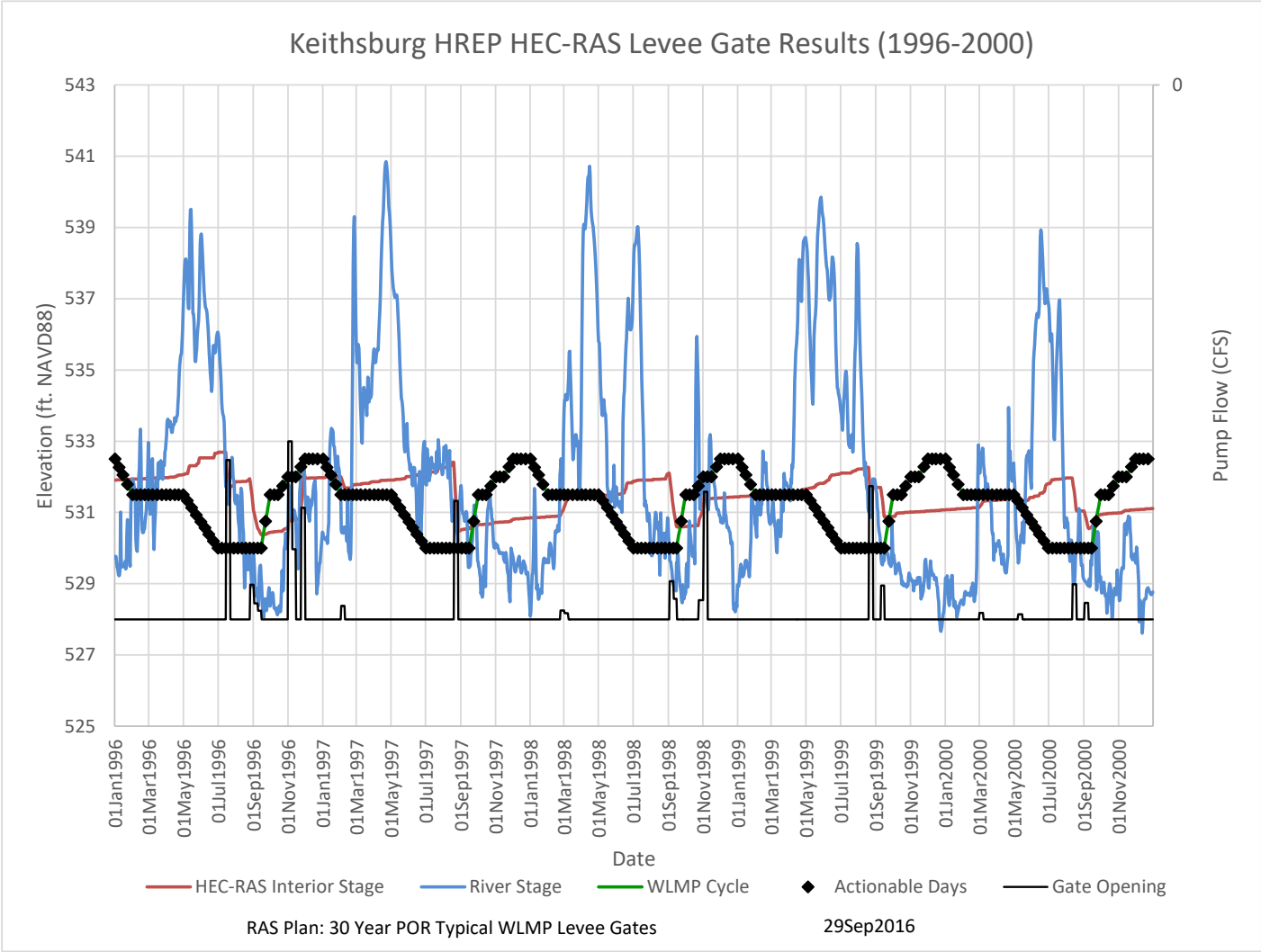


**Plate 15: HEC-RAS Results for 1996-2000 Under the Berm Improvements with Gates Alternative and the Typical WLMP**

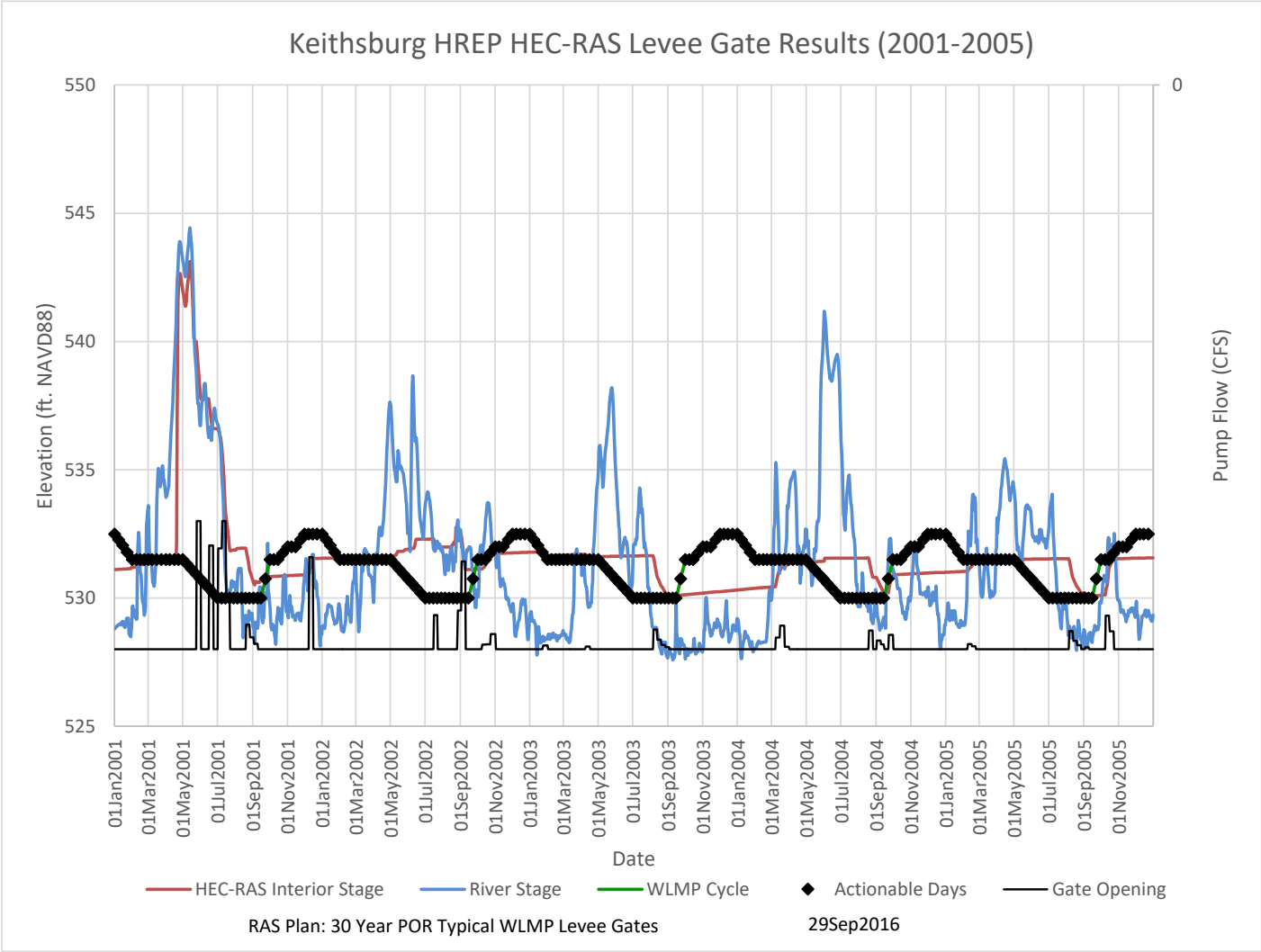


**Plate 16: HEC-RAS Results for 2001-2005 Under the Berm Improvements with Gates Alternative and the Typical WLMP**

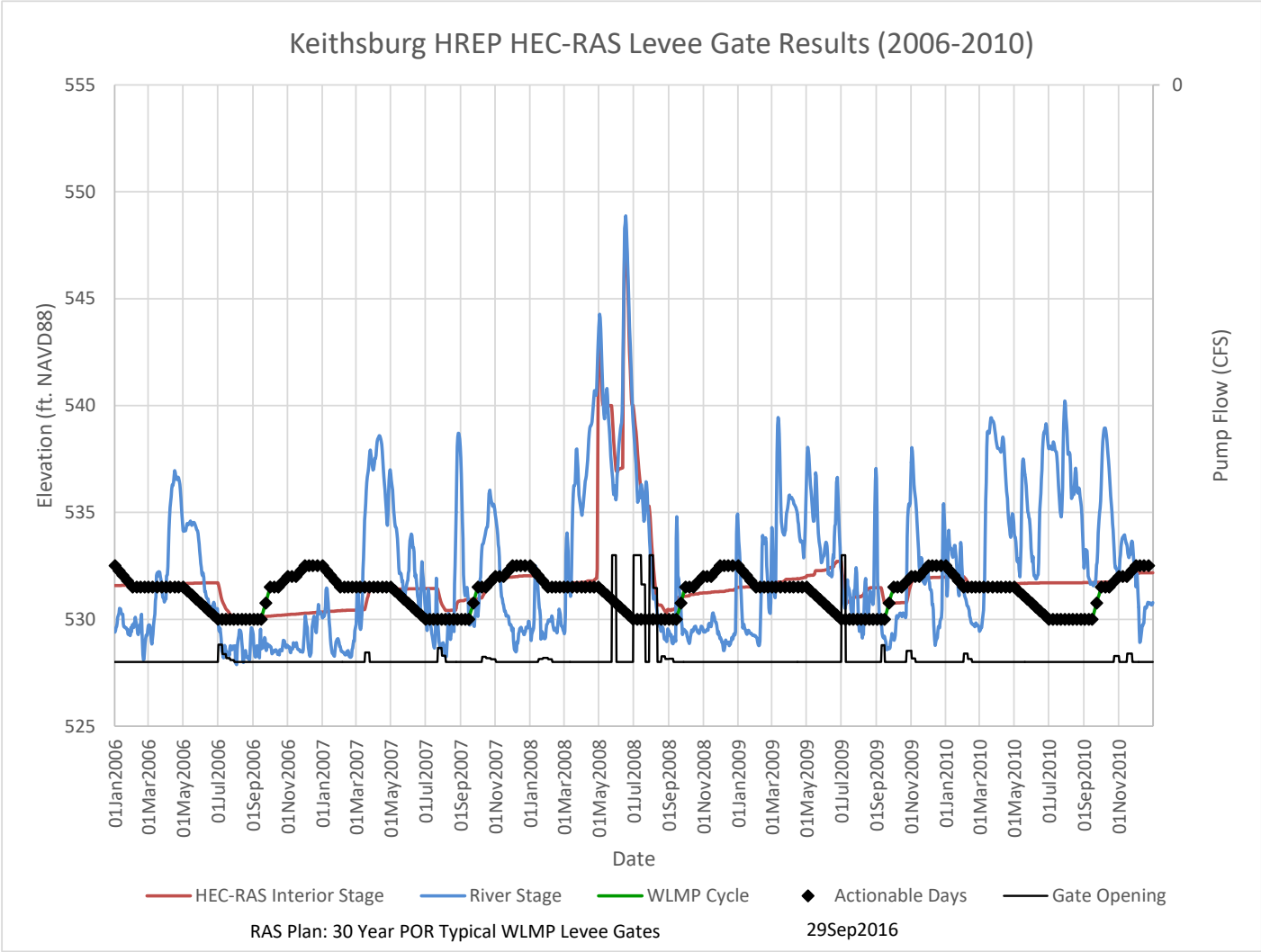


**Plate 17: HEC-RAS Results for 2006-2010 Under the Berm Improvements with Gates Alternative and the Typical WLMP**

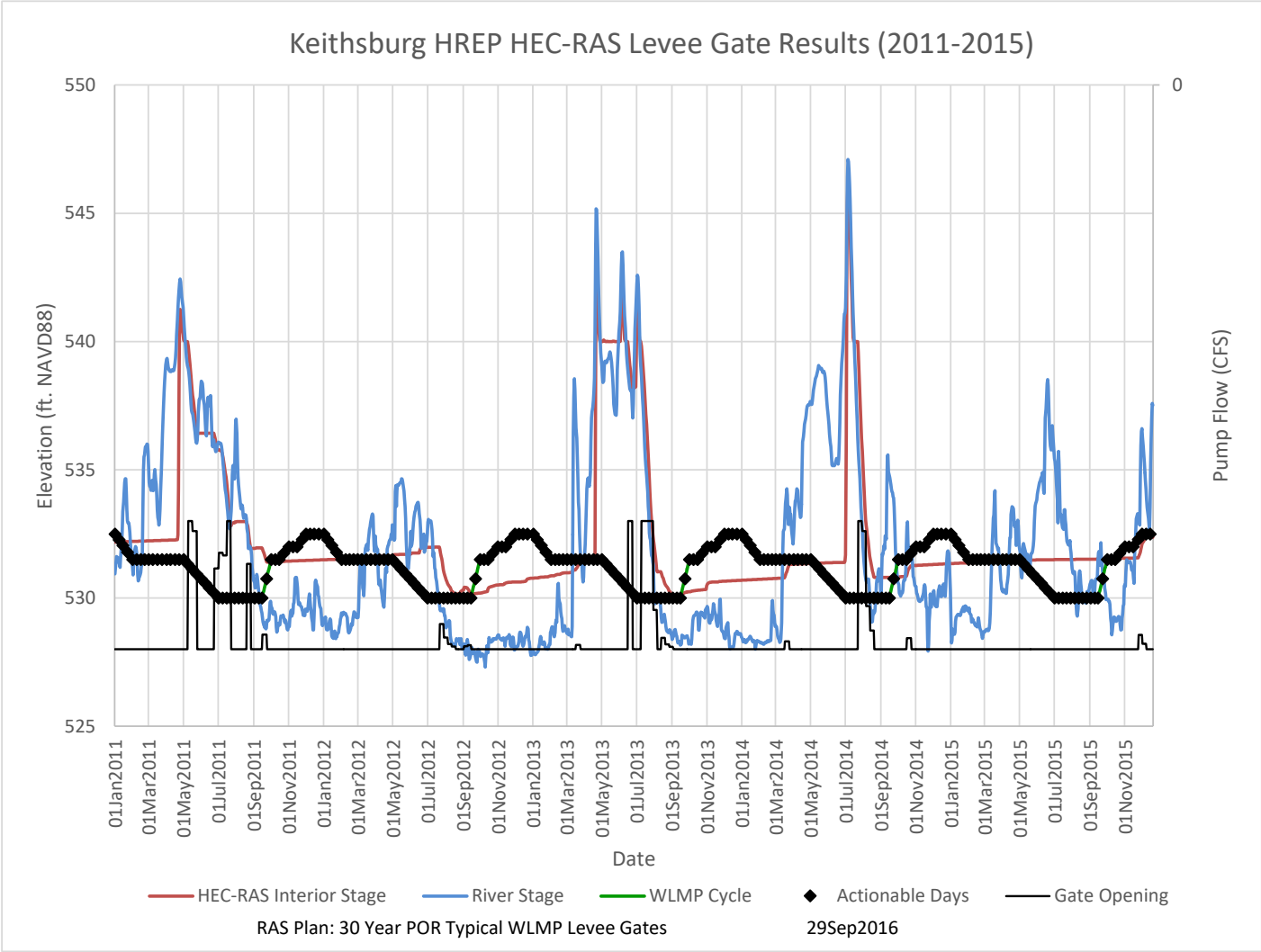


**Plate 18: HEC-RAS Results for 2011-2015 Under the Berm Improvements with Gates Alternative and the Typical WLMP**

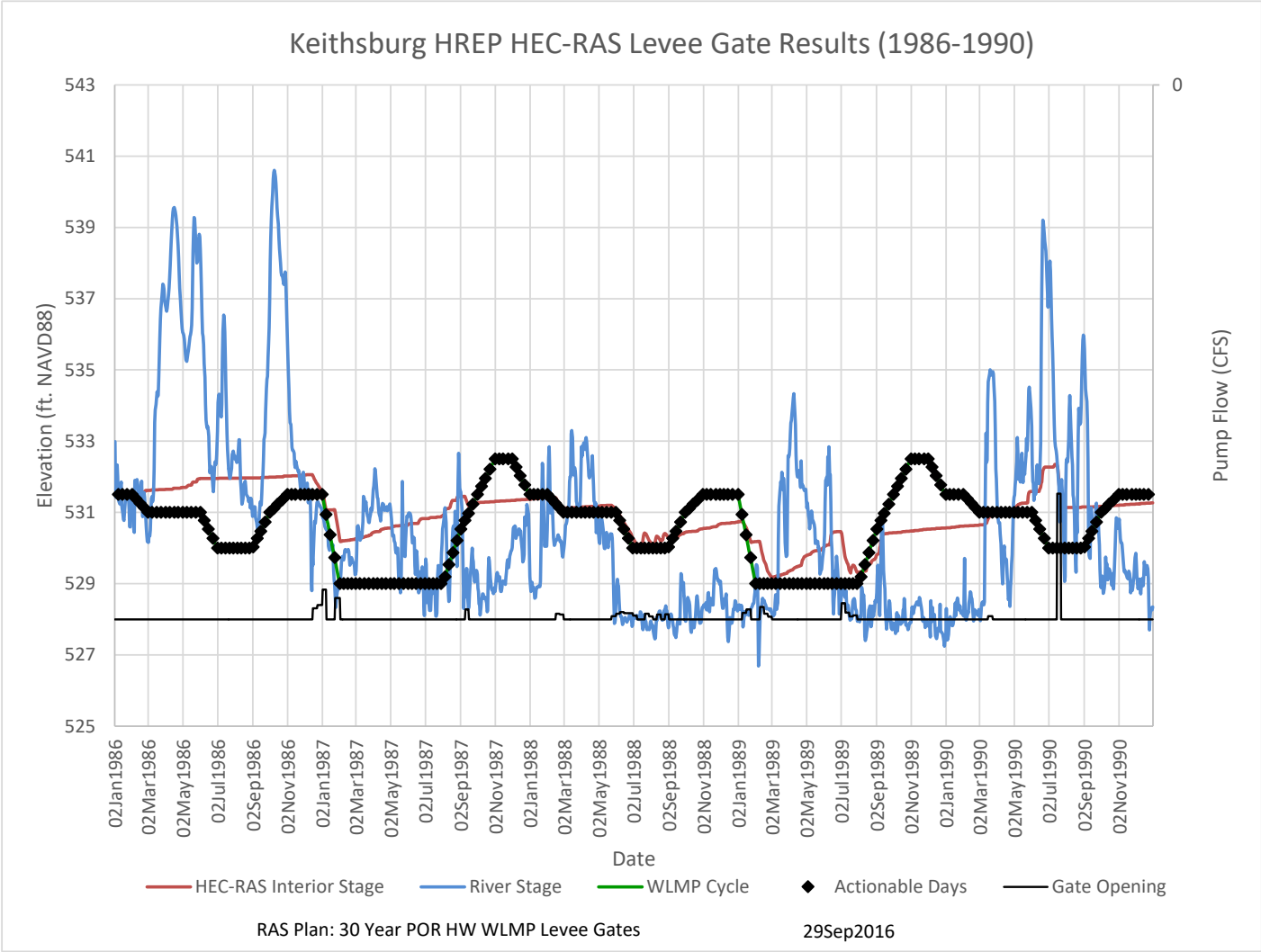


**Plate 19: HEC-RAS Results for 1986-1990 Under the Berm Improvements with Gates Alternative and the Hardwood WLMP**

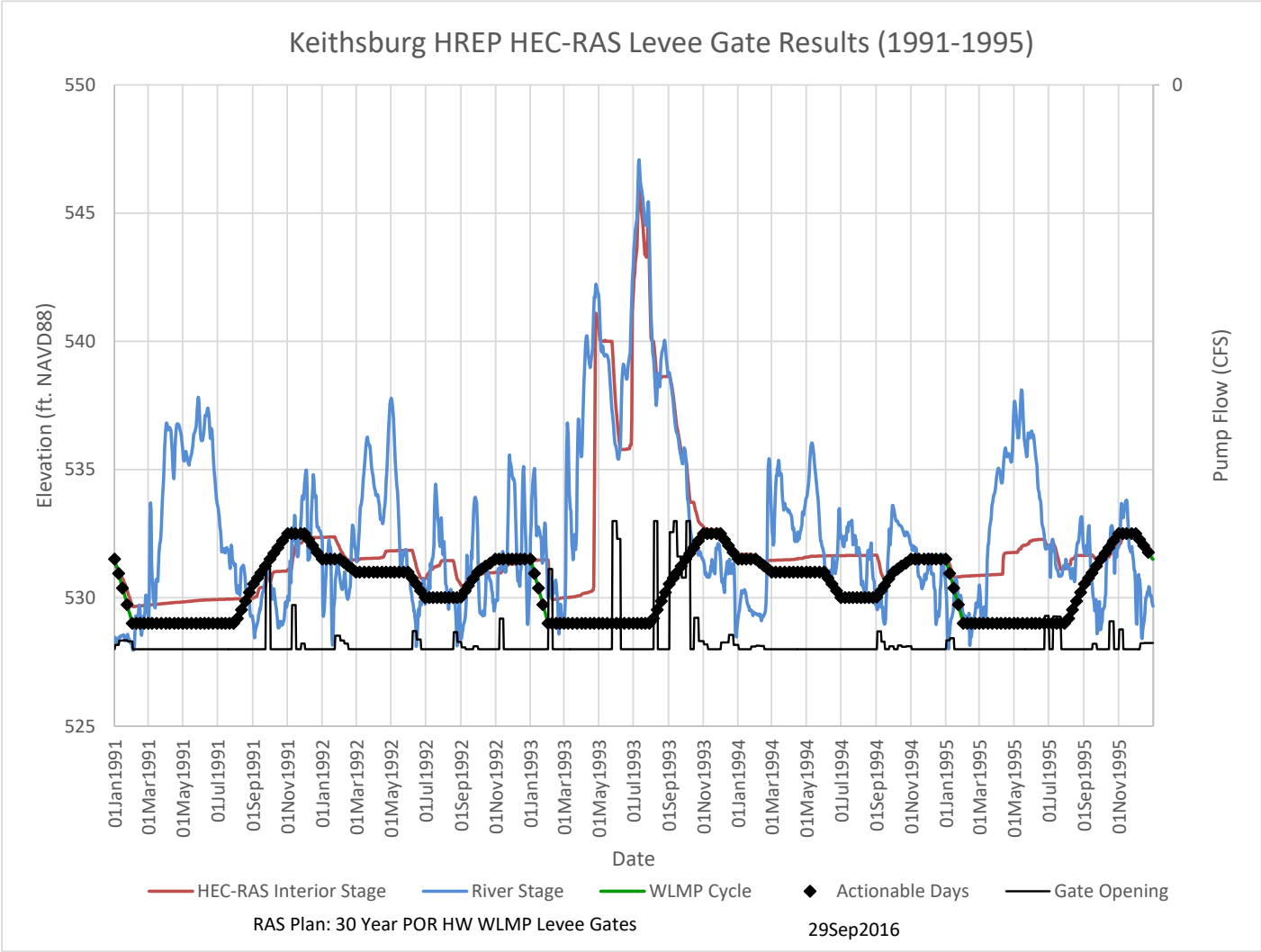


**Plate 20: HEC-RAS Results for 1991-1995 Under the Berm Improvements with Gates Alternative and the Hardwood WLMP**

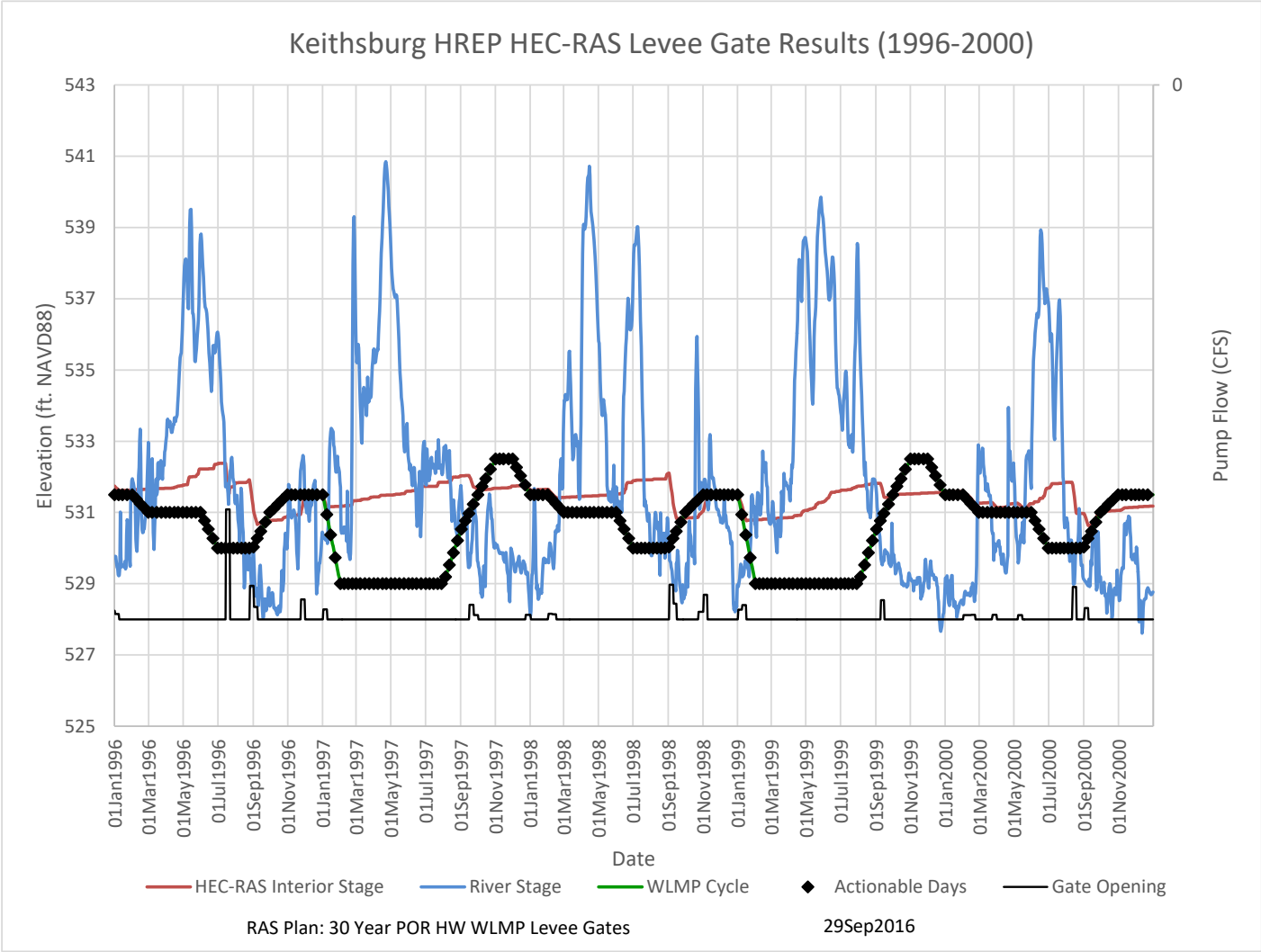


**Plate 21: HEC-RAS Results for 1996-2000 Under the Berm Improvements with Gates Alternative and the Hardwood WLMP**

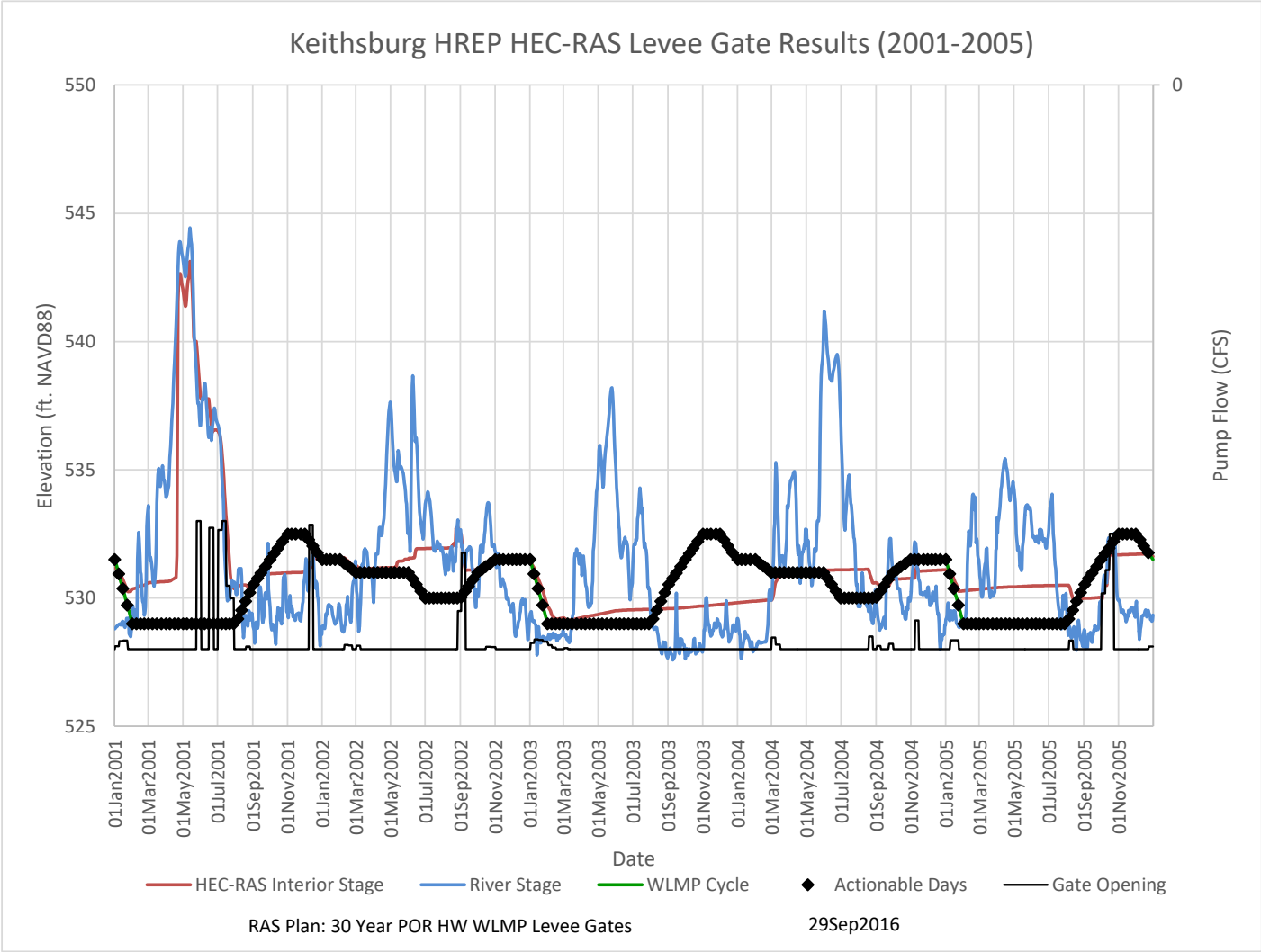


**Plate 22: HEC-RAS Results for 2001-2005 Under the Berm Improvements with Gates Alternative and the Hardwood WLMP**

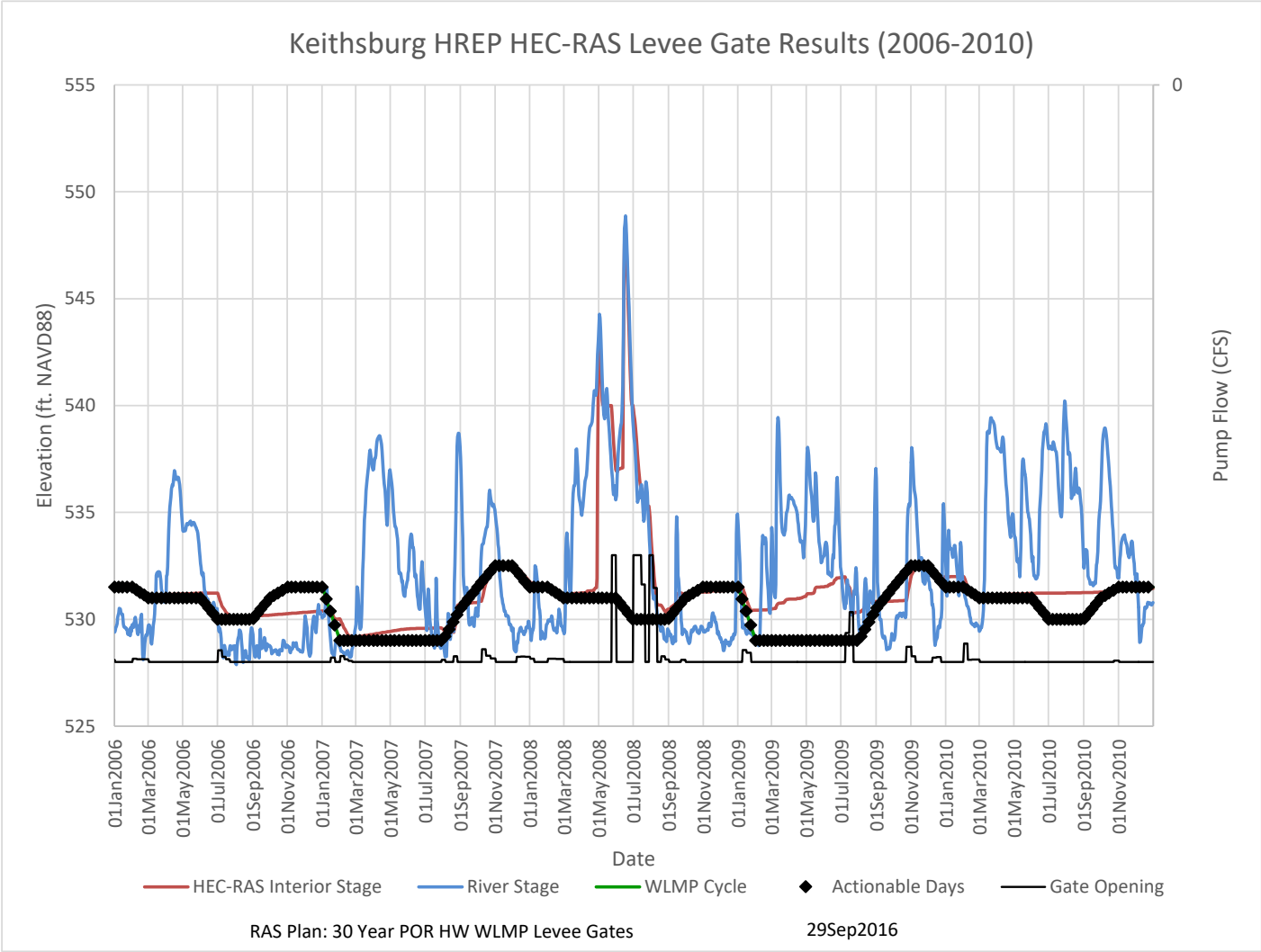


**Plate 23: HEC-RAS Results for 2006-2010 Under the Berm Improvements with Gates Alternative and the Hardwood WLMP**

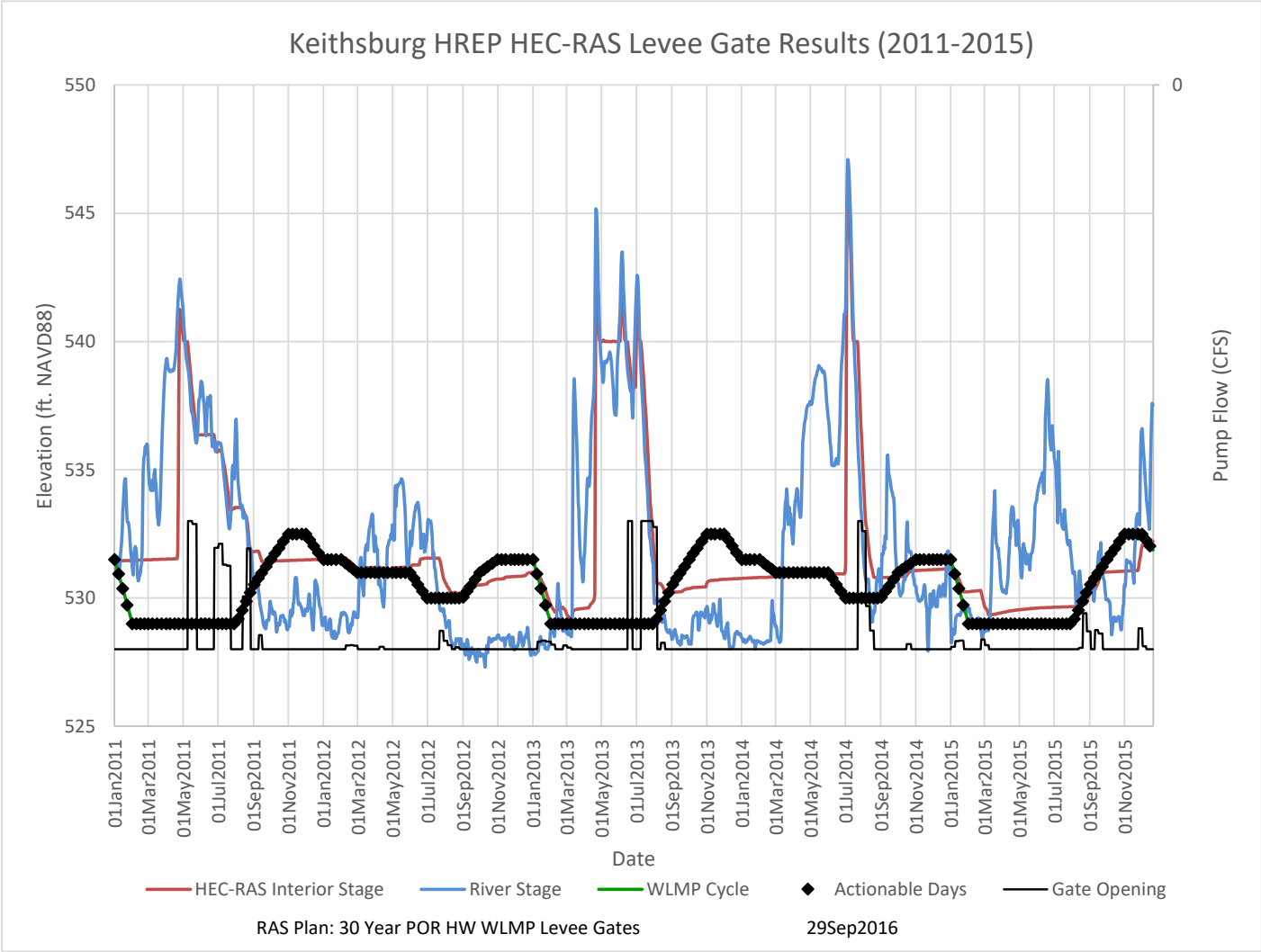


**Plate 24: HEC-RAS Results for 2011-2015 Under the Berm Improvements with Gates Alternative and the Hardwood WLMP**

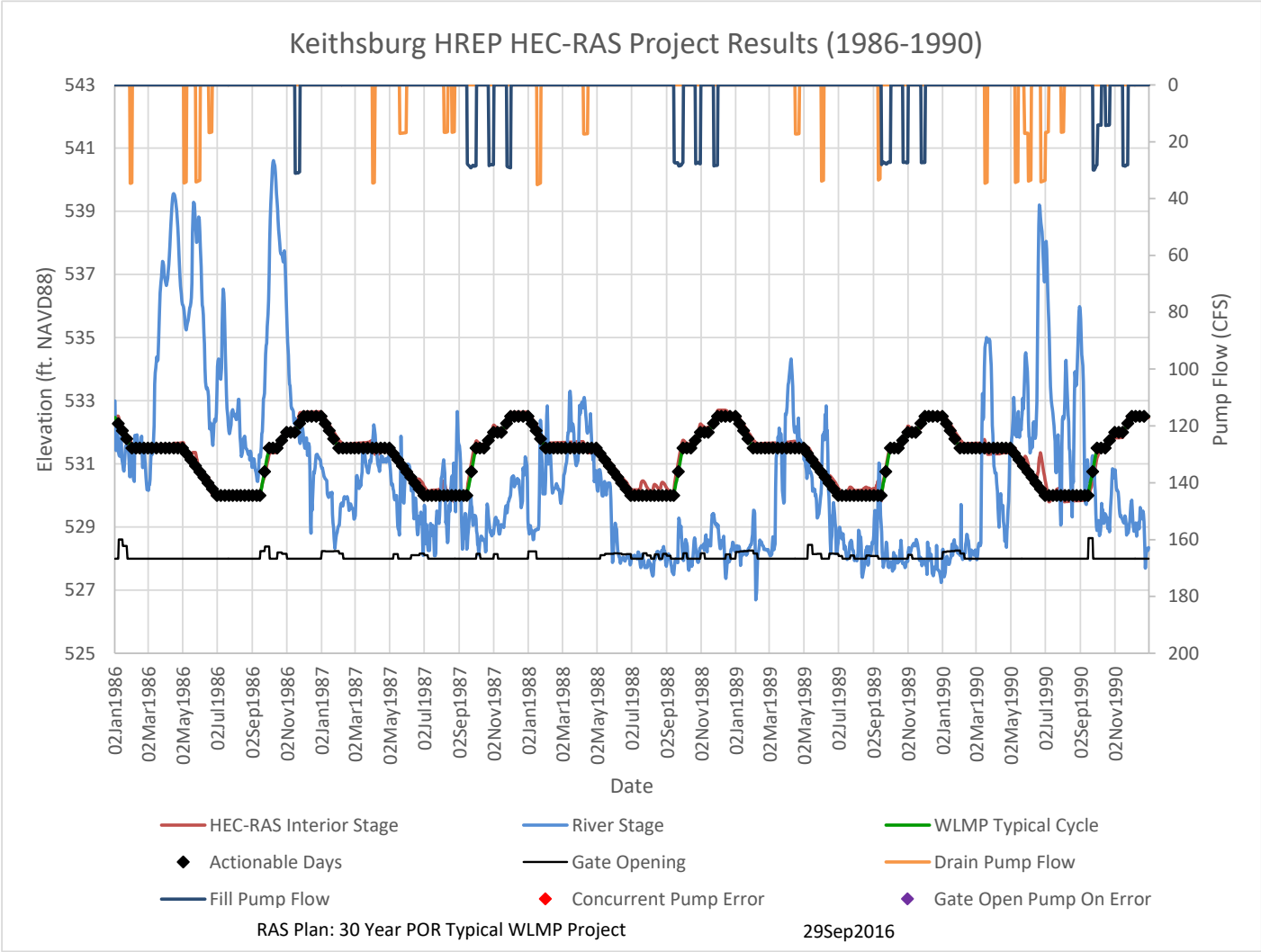


**Plate 25: HEC-RAS Results for 1986-1990 Under the Berm Improvements with Gates and Pumps Alternative and the Typical WLMP**

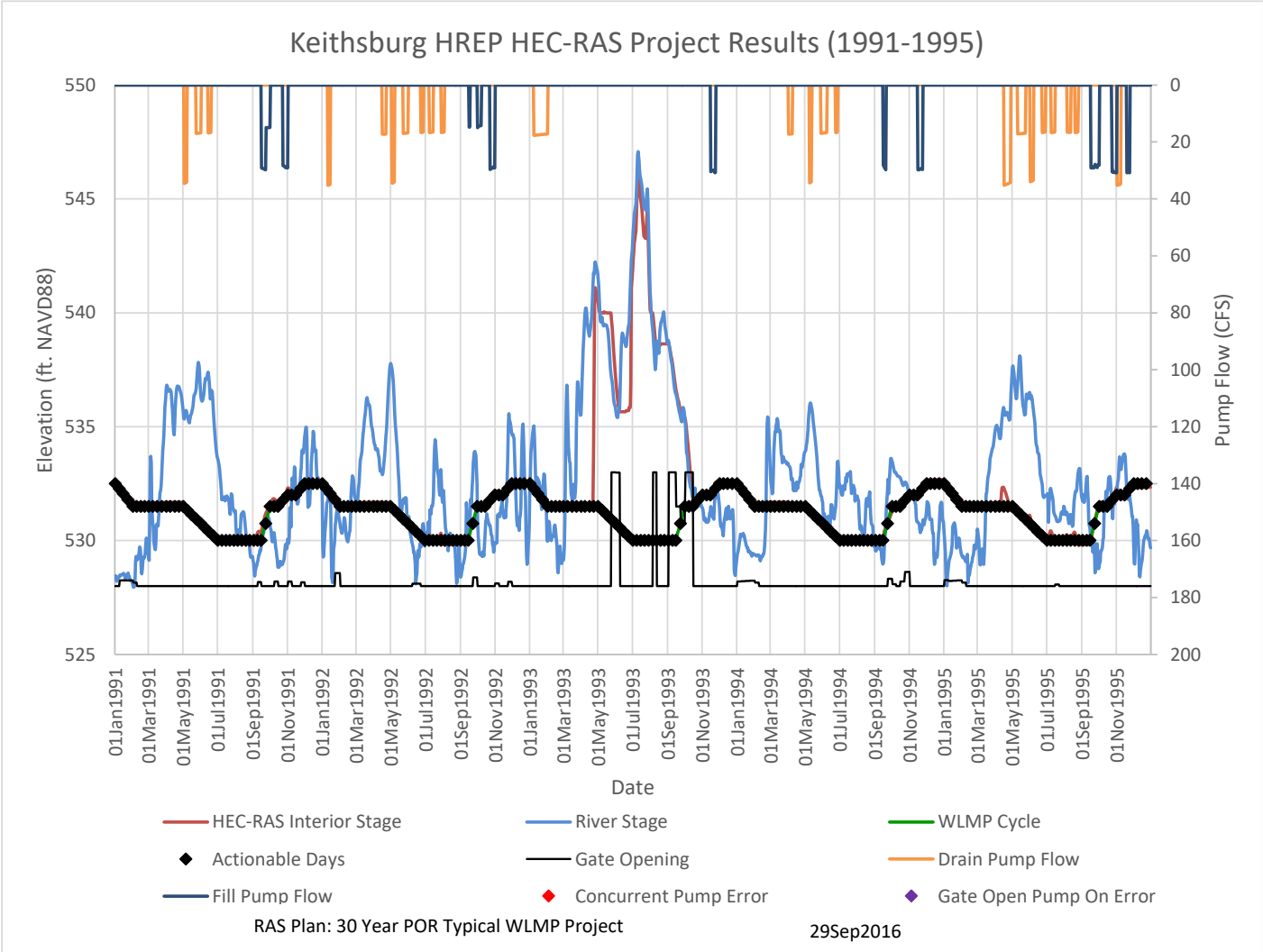


**Plate 26: HEC-RAS Results for 1991-1995 Under the Berm Improvements with Gates and Pumps Alternative and the Typical WLMP**

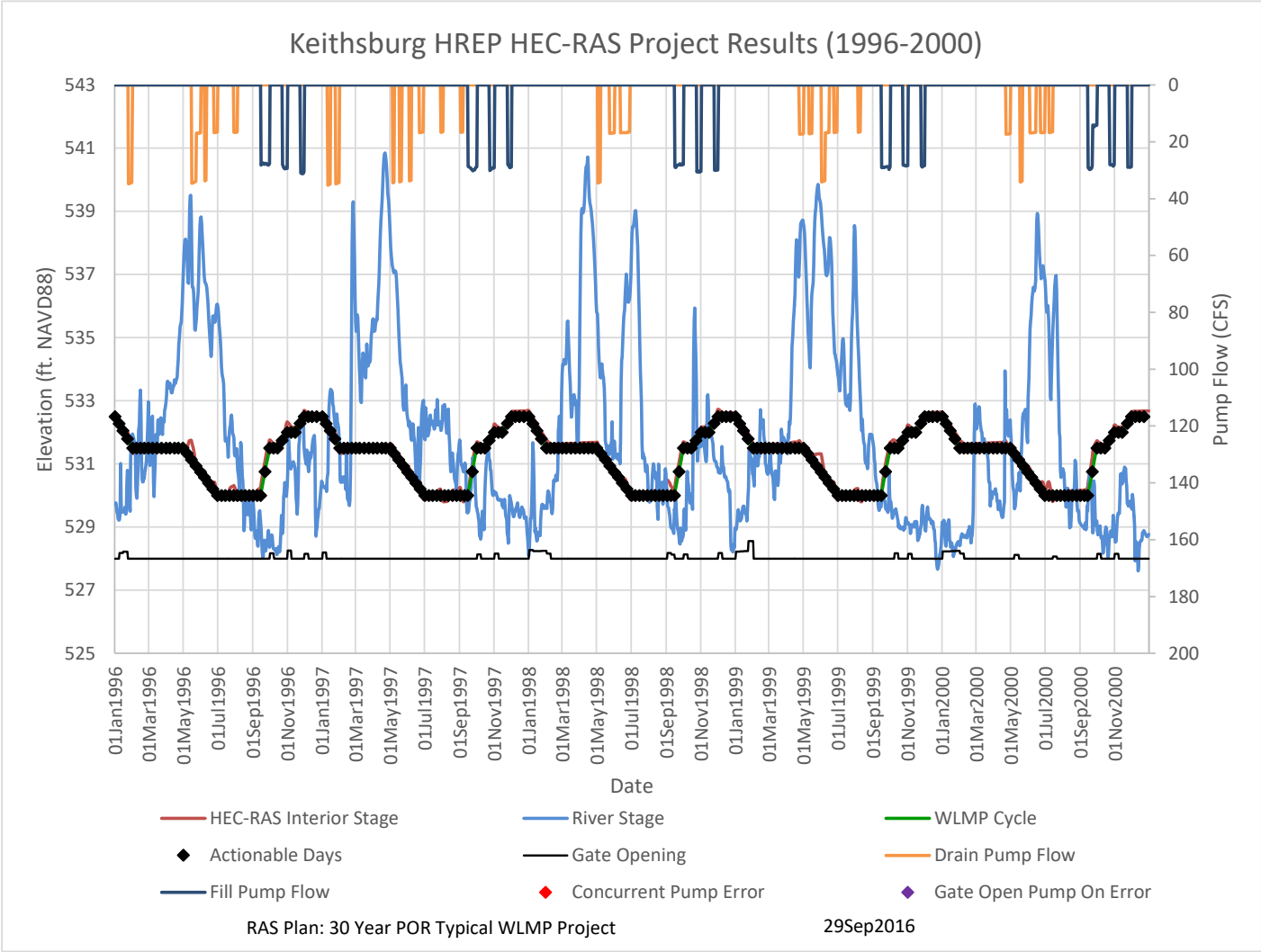


**Plate 27: HEC-RAS Results for 1996-2000 Under the Berm Improvements with Gates and Pumps Alternative and the Typical WLMP**

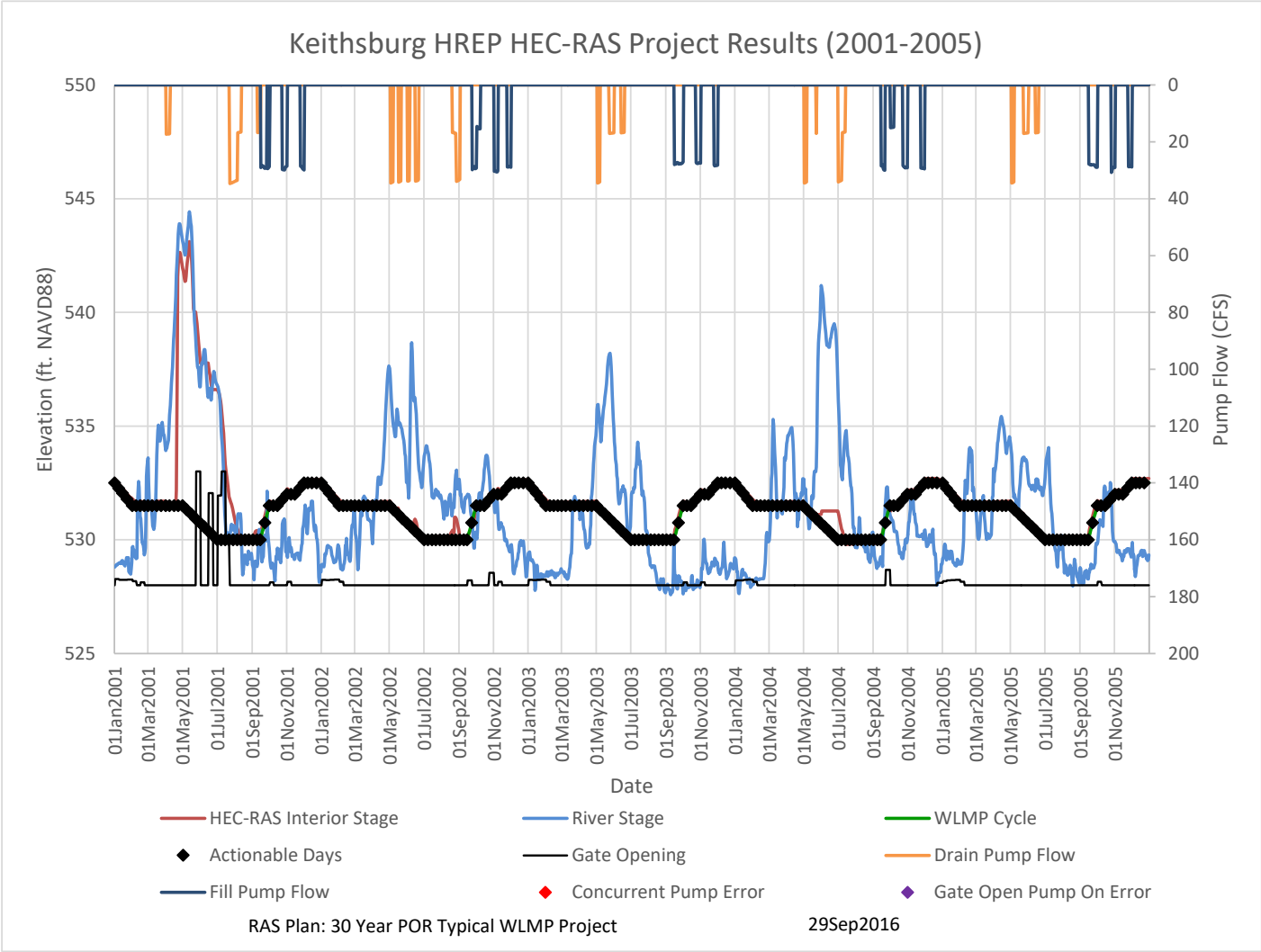


**Plate 28: HEC-RAS Results for 2001-2005 Under the Berm Improvements with Gates and Pumps Alternative and the Typical WLMP**

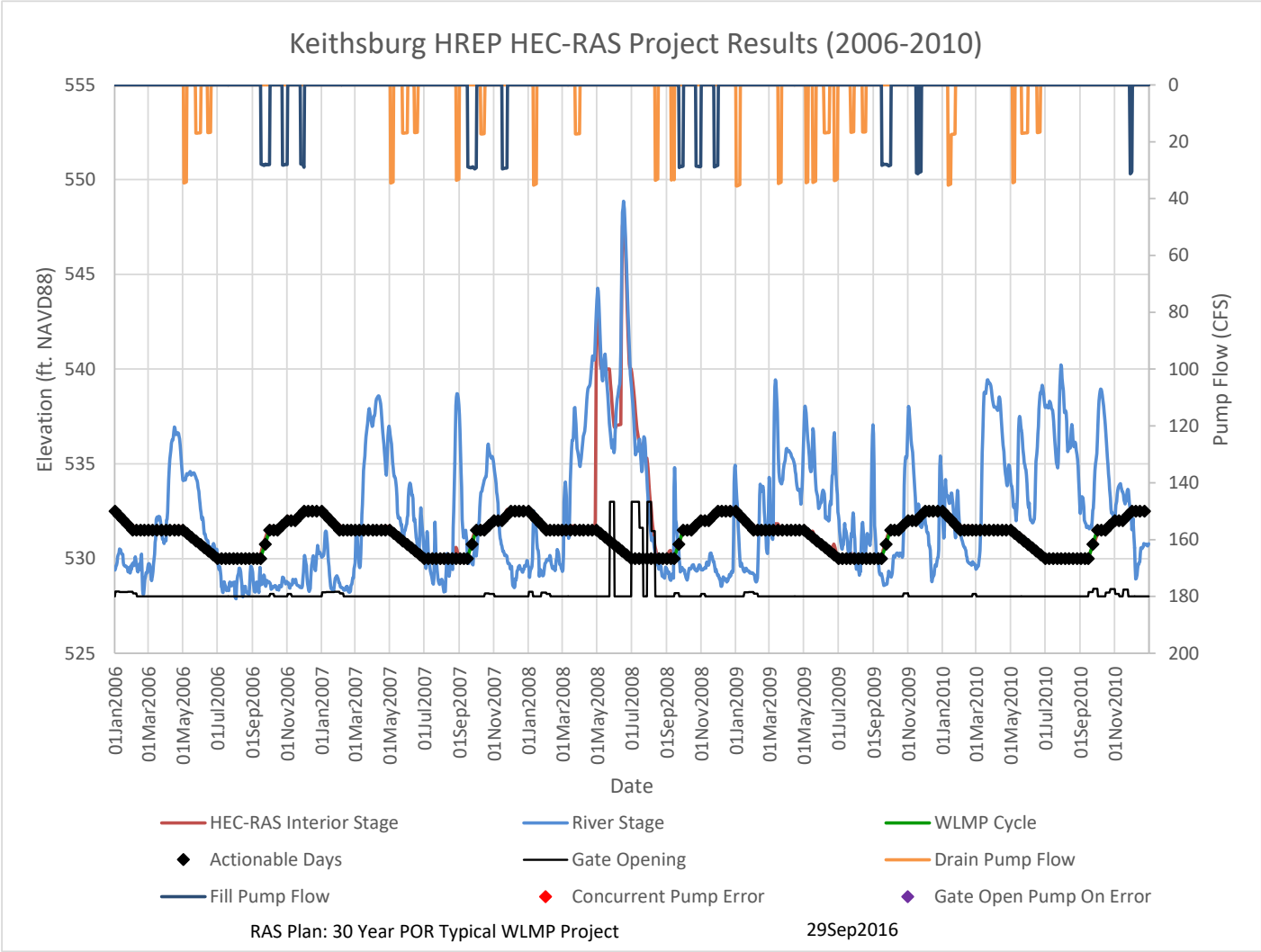


**Plate 29: HEC-RAS Results for 2006-2010 Under the Berm Improvements with Gates and Pumps Alternative and the Typical WLMP**

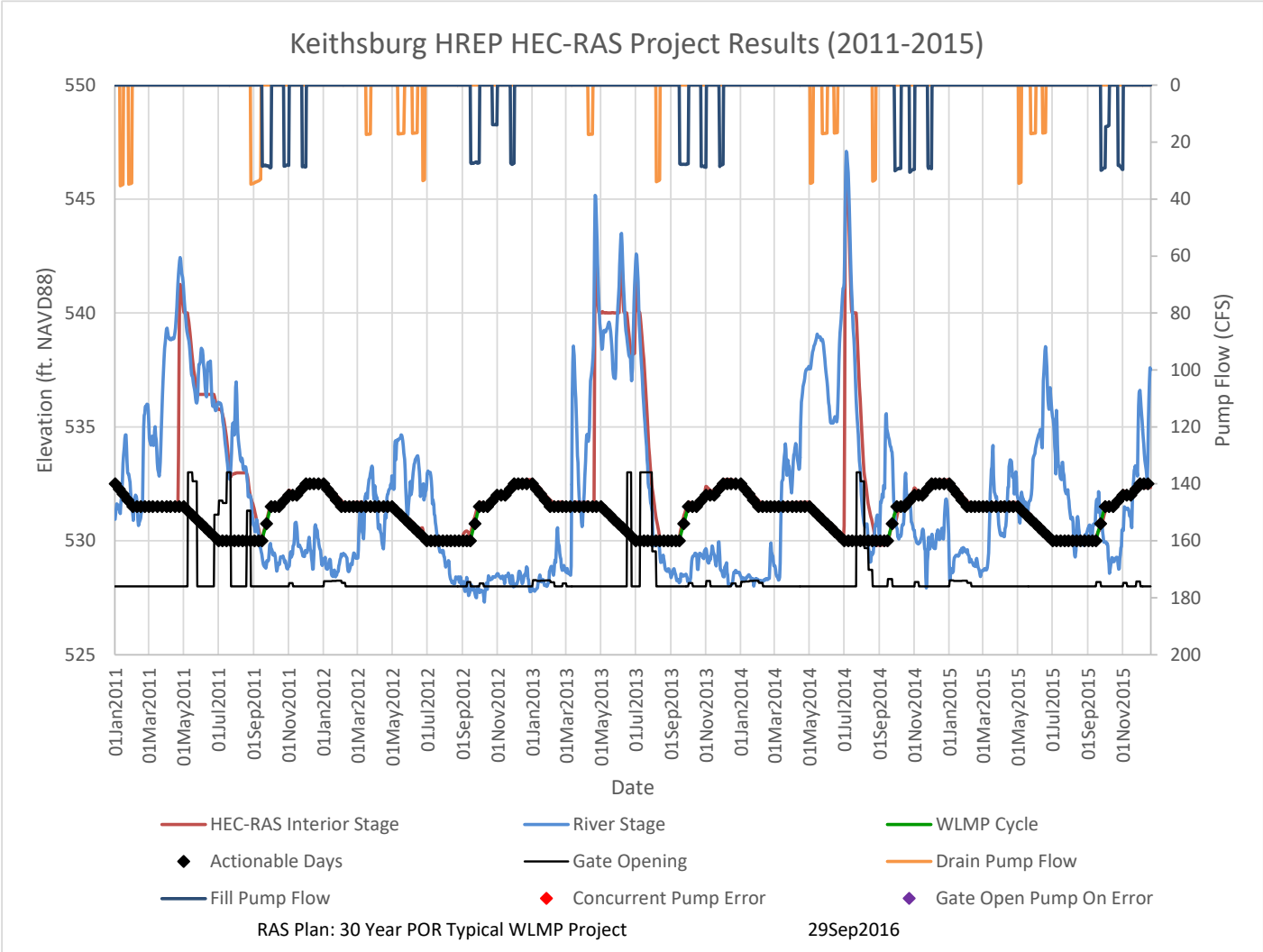


**Plate 30: HEC-RAS Results for 2011-2015 Under the Berm Improvements with Gates and Pumps Alternative and the Typical WLMP**

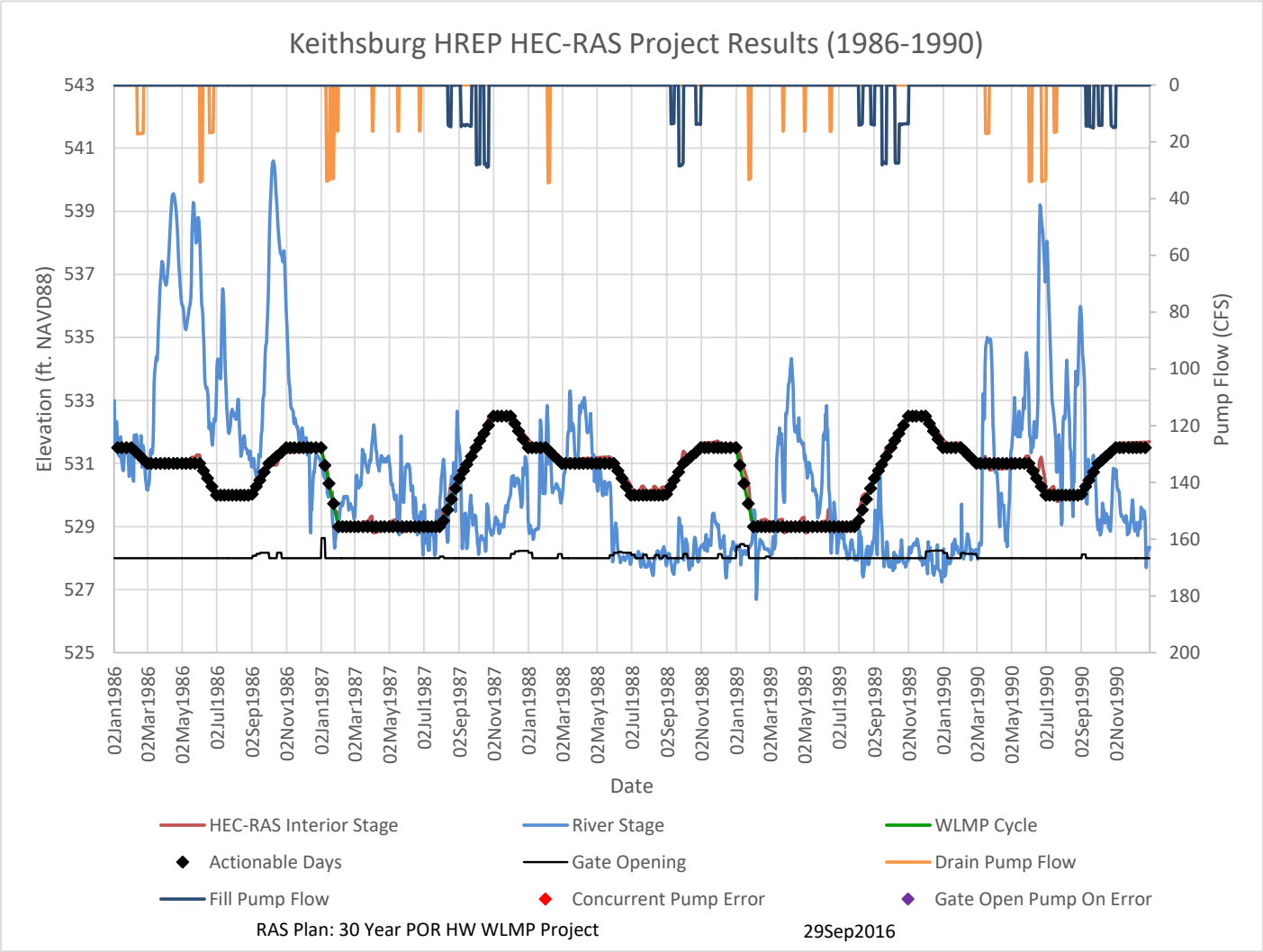


**Plate 31: HEC-RAS Results for 1986-1990 Under the Berm Improvements with Gates and Pumps Alternative and the Hardwood WLMP**

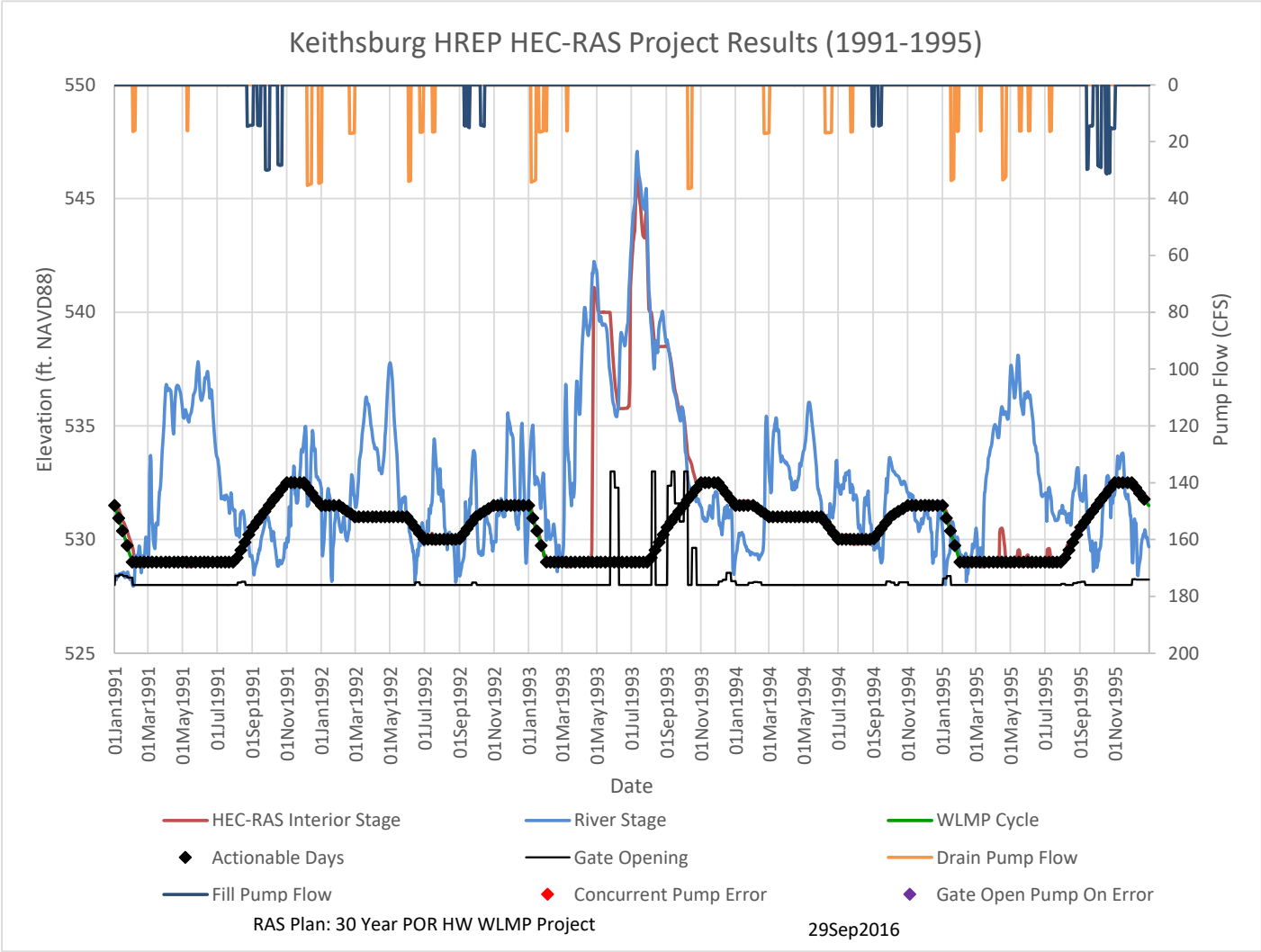


**Plate 32: HEC-RAS Results for 1991-1995 Under the Berm Improvements with Gates and Pumps Alternative and the Hardwood WLMP**

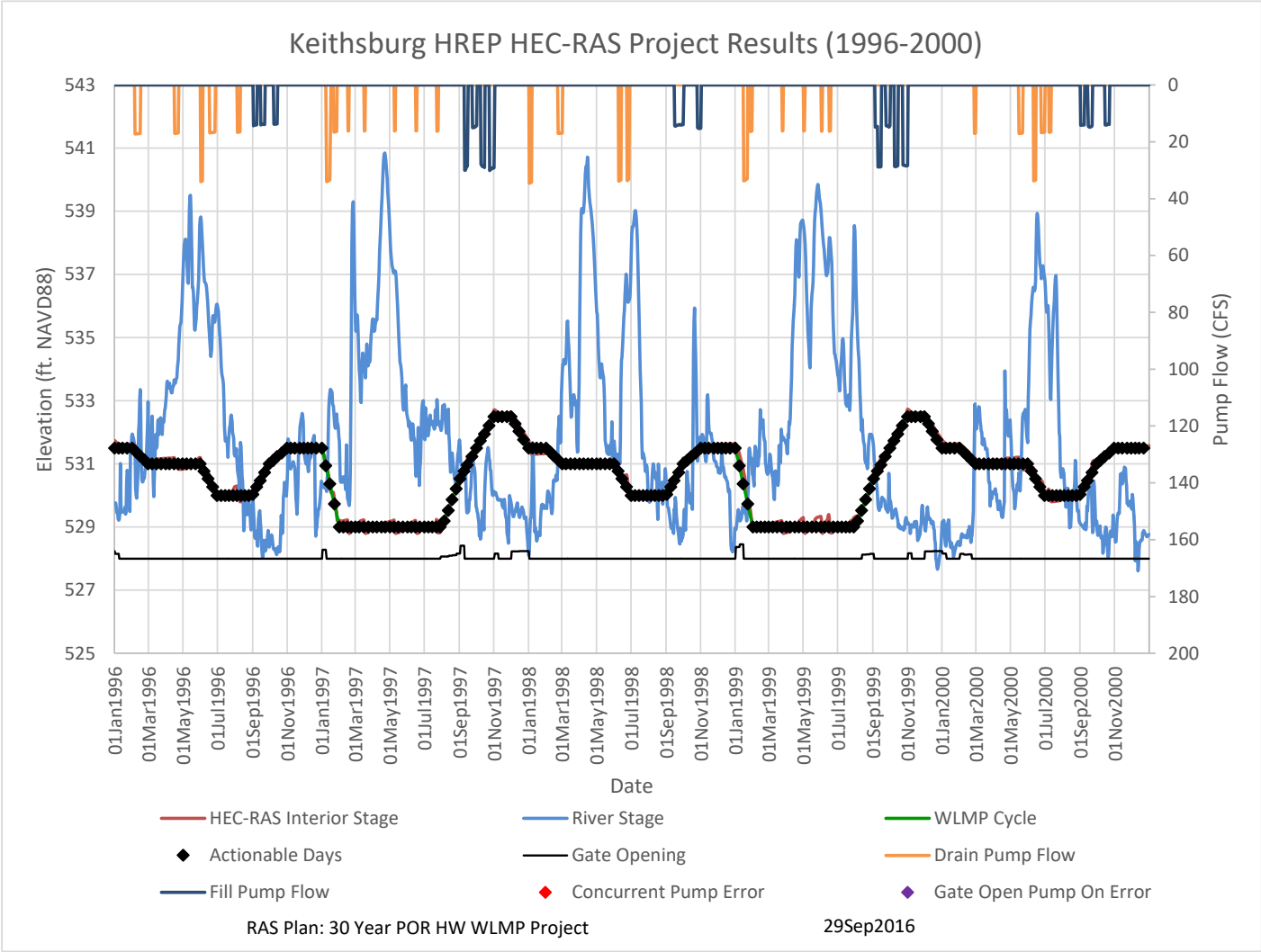


**Plate 33: HEC-RAS Results for 1996-2000 Under the Berm Improvements with Gates and Pumps Alternative and the Hardwood WLMP**

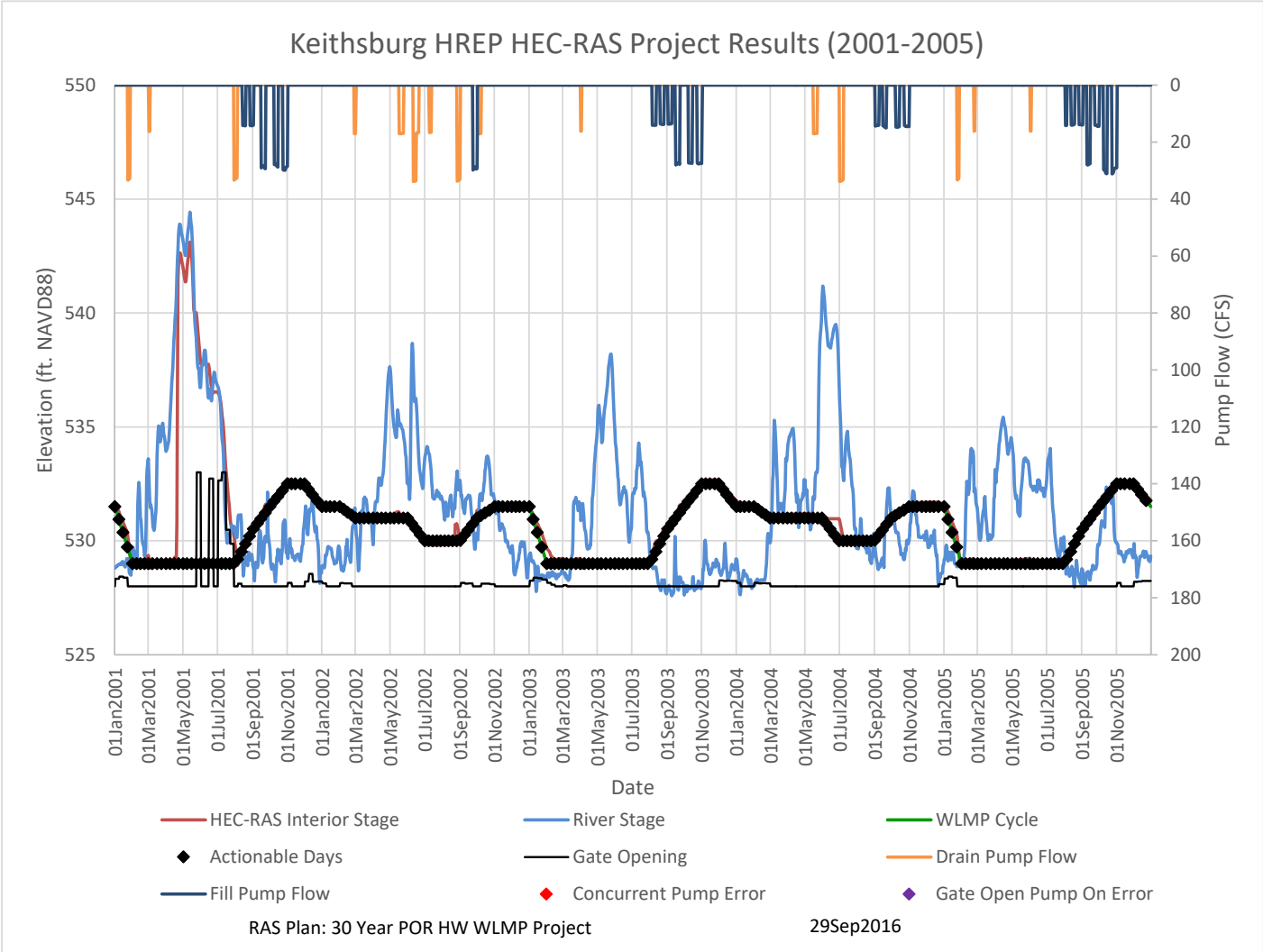


**Plate 34: HEC-RAS Results for 2001-2005 Under the Berm Improvements with Gates and Pumps Alternative and the Hardwood WLMP**

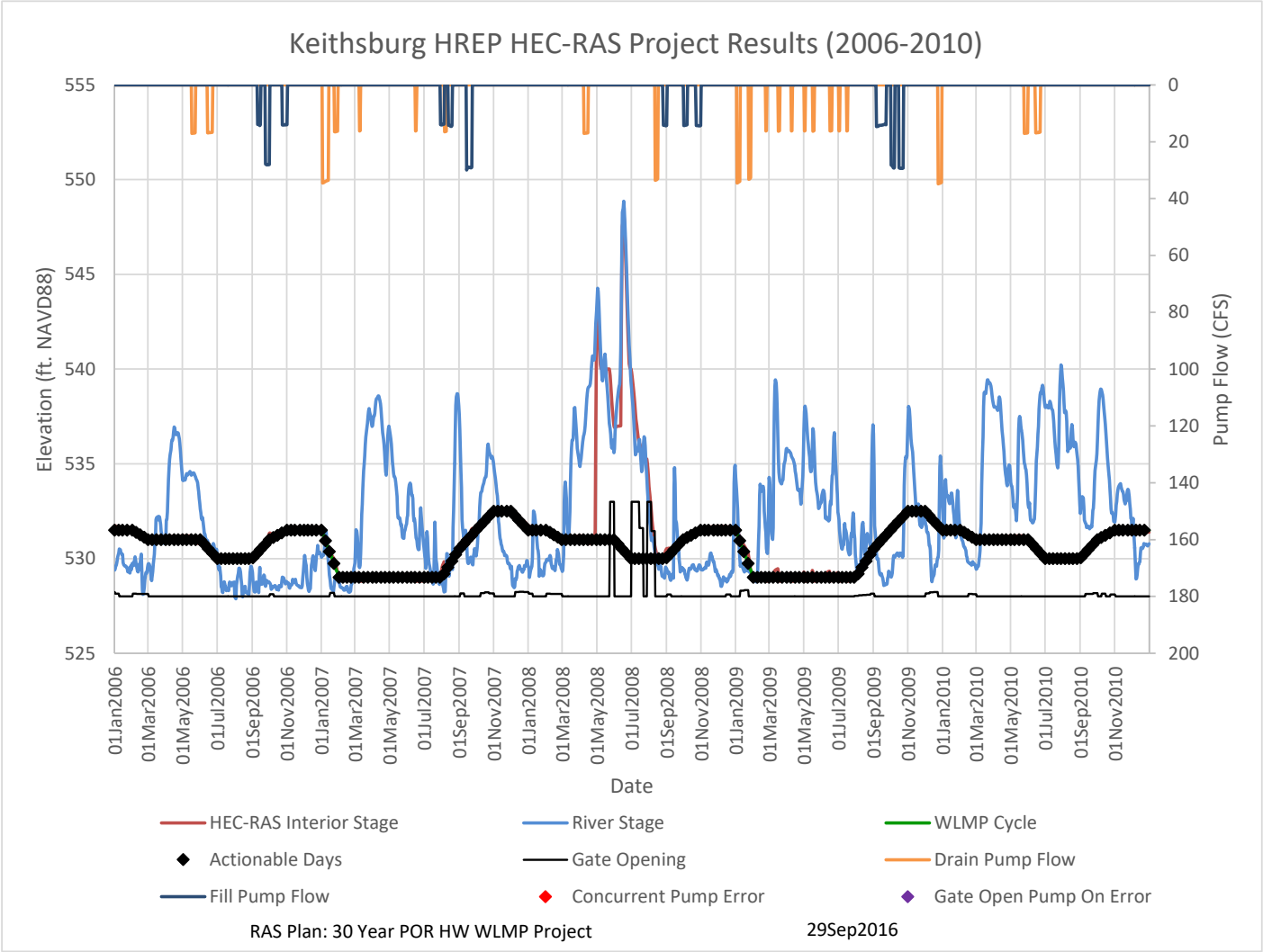


**Plate 35: HEC-RAS Results for 2006-2010 Under the Berm Improvements with Gates and Pumps Alternative and the Hardwood WLMP**

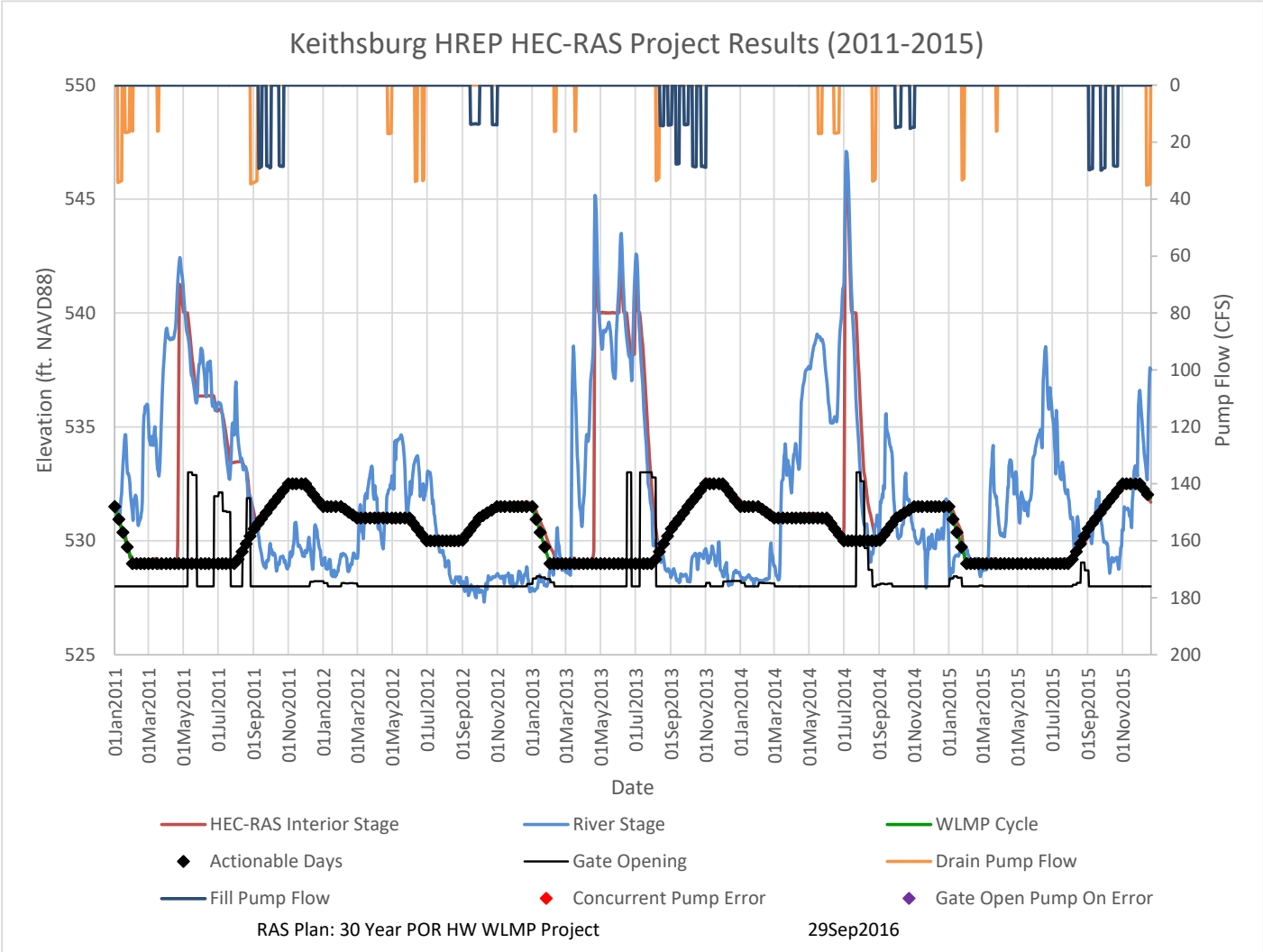


**Plate 36: HEC-RAS Results for 2011-2015 Under the Berm Improvements with Gates and Pumps Alternative and the Hardwood WLMP**