UPPER MISSISSIPPI RIVER RESTORATION FEASIBILITY REPORT WITH INTEGRATED ENVIRONMENTAL ASSESSMENT

GREEN ISLAND
HABITAT REHABILITATION AND ENHANCEMENT PROJECT

POOL 13, UPPER MISSISSIPPI RIVER
RIVER MILES 545.9 THROUGH 548.7
JACKSON COUNTY, IOWA

APPENDIX E
ENGINEERING

ATTACHMENT A
HYDROLOGY AND HYDRAULICS

APPENDIX E
ENGINEERING APPENDIX

ATTACHMENT A
HYDROLOGY AND HYDRAULICS

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1. INTRODUCTION

The following sections detail the hydrologic analysis completed for the Green Island Habitat Rehabilitation and Enhancement Project (HREP) Feasibility Study. Please refer to main report for the study description.

Initial hydraulic rise modeling has been completed to determine floodplain impacts of the proposed work. That analysis is not detailed within this appendix attachment but is available in a separate memo.

1.1 Vertical Datum

All elevations used in this appendix are expressed using the NAVD88 vertical datum unless otherwise stated. For hydraulic modeling, conversions between MSL12 and NAVD88 vertical datums is provided in Appendix E, *Engineering*, Attachment E, *Survey, Mapping, and Geospatial Data*

2. CLIMATE

Monthly climate data for the Bellevue Lock and Dam 12 U.S. Cooperative Network Station (gage #130608) from the Midwestern Regional Climate Center, is summarized in Tables A-1 and A-2 (Midwestern Regional Climate Center, 2022). The data for precipitation, snowfall, and temperature below is from the recent 30-year period, 1992-2021.

Table A-1. Average and Extremes of Monthly Precipitation and Snowfall (COOP gage #130608)

Precipitation							Snowfall		
Month	**Average (in)**	**Maximum**		**Minimum**			**Average (in)**	**Maximum**	
		(in)	**Year**	**(in)**	**Year**			**(in)**	**Year**
Jan	1.33	3.47	2005	0.17	2003		10.8	39	1996
Feb	1.56	3.61	2008	0.08	1995		8.5	30	2008
Mar	2.04	4.95	2004	0.5	1996		3.2	15	2002
Apr	3.76	7.91	2013	1.1	2018		0.4	3	1997/2003
May	4.26	7.97	1996	0.7	1992		0.1	4	1994
Jun	5.52	10.55	2015	1.21	1992		0.0	0.0	
Jul	4.59	9.66	2011	0.3	2012		0.0	0.0	
Aug	3.85	9.06	2018	0.84	2003		0.0	0.0	
Sep	3.44	9.63	2019	0.64	2021		0.0	0.0	
Oct	3.07	7.63	2009	0.39	2005		0.2	4.5	2019
Nov	2.20	5.42	2003	0.26	2007		1.2	9.5	2015
Dec	1.81	5.72	2015	0.1	1995		8.7	32.3	2010
Annual	**36.5**						**35.0**		

Fluctuation of temperatures in east-central Iowa can be extreme, evidenced by a minimum monthly temperature of -35° F in January and a maximum monthly temperature of 103° F in July. Precipitation is moderate, with an average annual value of 36.5 inches. The average annual snowfall is 35.0 in.

Table A-2. Average and Extremes of Monthly Temperature (COOP gage #131635)

Month	Average (°F)	Maximum (°F)	Minimum (°F)
Jan	20.27	60	-35
Feb	23.24	74	-34
Mar	35.49	83	-23
Apr	47.68	88	13
May	59.01	95	23
Jun	69.57	96	36
Jul	73.15	103	46
Aug	71.34	102	43
Sep	63.61	98	27
Oct	51.35	95	19
Nov	37.61	77	1
Dec	26.6	72	-23
Annual	**48.12**		

3. HYDROLOGY AND HYDRAULICS OVERVIEW

3.1 Historic and Current Mississippi River Hydrology

Table A-3 shows a summary of the nearby gages and their characteristics. The Green Island HREP is located between River Miles 545.9 and 548.7, approximately eight miles downstream of L&D 12. The District records continuous stages and computed flows at L&D 12 (Bellevue, IA) and L&D 13 (Fulton, IL). Historic and recent contemporary stage data is available at Sabula, IA. The L&D 12 gage (RM 556.7) with a drainage area of 82,400 square miles, is closest to the study area (USACE, 2022). Although located downstream in Pool 14, the USGS Mississippi River at Clinton, IA gage (05420500) provides the nearest long-term record of peak annual streamflow.

Table A-3. Summary of Available Stream Gages Near to Green Island

Gage Name	River Mile	Drainage Area (sq. miles)	Gage Zero Elevation (ft MSL12)	Gage Zero Elevation (ft NAVD88)	Period of Record	Flat pool stage (ft)	Flat pool elevation (ft MSL12)	Flat pool elevation (ft NAVD88)	Available data
Mississippi River at L&D 12, Bellevue, IA	556.7	82,400	580.2	579.5	1936-present	11.8	592.0	591.3	Stage and computed flow
Maquoketa River near Maquoketa, IA USGS 05418500	Confluence at 548.6	1,553	N/A	625.63	1989-present	N/A	N/A	N/A	Stage, discharge and water quality
Maquoketa River near Green Island, IA USGS 05418720	Confluence at 548.6	1,869	N/A	582.12	2014-2018	N/A	N/A	N/A	Stage, discharge and water quality
Mississippi River at Sabula, IA	535	N/A	572.3	571.6	1881-2010 2021-present	N/A	N/A	N/A	Stage
Mississippi River at L&D 13, Fulton, IL	522.4	85,500	568.7	568.1	1937-present	14.3	583.0	582.4	Stage and computed flow
Mississippi River at Clinton, IA USGS 05420500	511.8	85,600	N/A	562.54	1873-present	N/A	N/A	N/A	Stage, discharge, water quality and sediment

Figure A-1 shows average annual elevation hydrographs for Lock and Dam 13 (Fulton, IL) pool gage, the Green Island HREP, Sabula, IA gage and Lock and Dam 12 (Bellevue, IA) tail gage. Hydrographs were developed with data from 1993–2022. The LD 13 pool gage, located nearly 25 miles downstream of the study area, is strongly influenced by regulation of the dam. The L&D 12 tail gage, located 8 miles upstream of the study area, illustrates spring/summer flooding (typically peaking in late April), a slight pulse of higher flows in the fall, followed by more stable flows through the winter. The hydrograph for the Green Island study area (RM547.25) was generated through linear interpolation of the L&D 12 tail and L&D 13 pool daily stage records. Mississippi River conditions near the Green Island study area (RM 547.25) illustrate variability similar to the L&D 12 tail gage due to the location well upstream of the influence of L&D 13. The Maquoketa River enters the Mississippi on the upstream end of the study area.

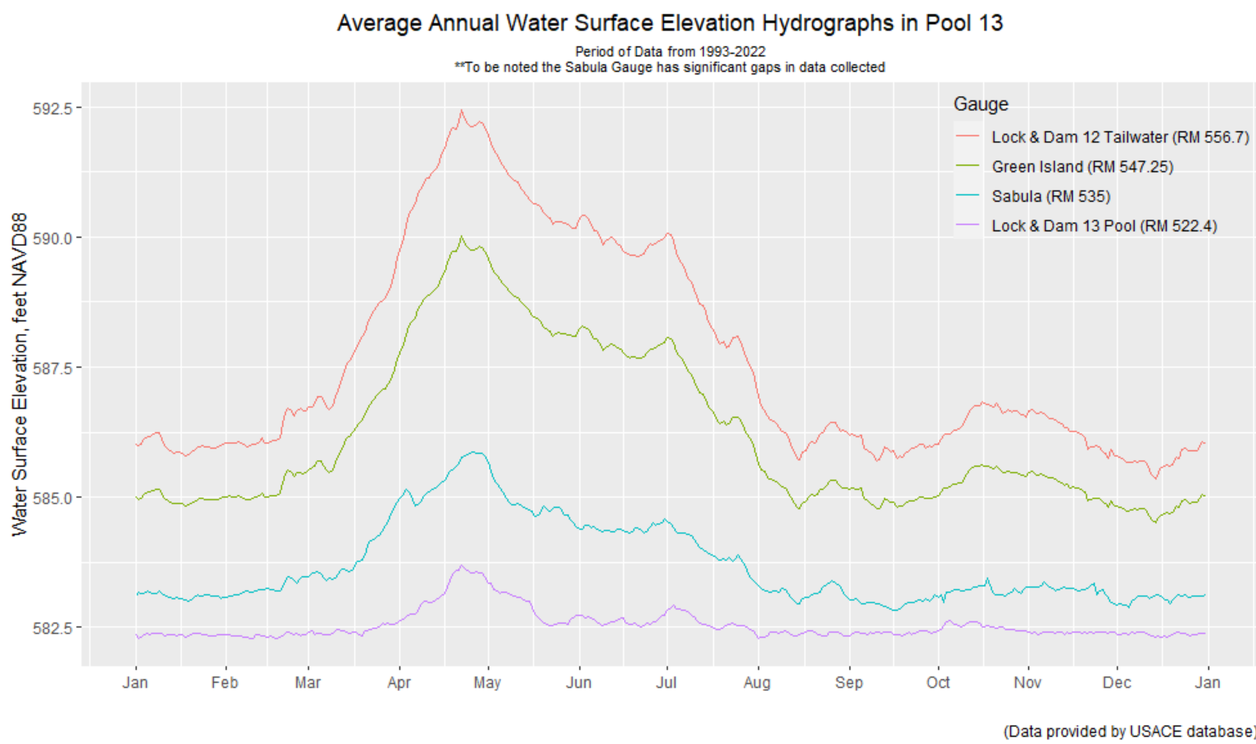


Figure A-1. Average Annual Elevation Hydrographs for Lock and Dam 12 (Bellevue, IA) Tail Gage, Green Island Study Area, Sabula, IA Gage and Lock and Dam 13 (Fulton, IL) Pool Gage (1993-2022).

3.2 Mississippi River Elevation Duration

Daily stage records at L&D 12 tail and L&D 13 pool were linearly interpolated to generate a stage hydrograph approximated for the mid-point of the study area (RM 547.25) to generate estimates of annual and seasonal stage duration. Seasonal duration curves were used to evaluate opportunities for gravity flow between the Mississippi River and the Green Island Management Area to support water level management goals for the Project. See Section 3.6 for a description of Green Island existing water level management and infrastructure, including an illustration of the Pool A and Pool B managed areas (Figure A-3). Section 3.6.2 provides a description of the desired water level management plans (WLMPs). Table A-4 and Figure A-2 show elevation duration curves for the most recent 30-year period (1993–2022) for (1) all data (2) only April 1–June 30 data, and (3) July 1- October 31 data based on the interpolated stage hydrograph at the mid-point of the study area (RM 547.25). The period from 1993–2022 was selected to characterize existing conditions. This 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 anticipated to change over time) and long enough to provide a representative sample size to adequately represent the population. Figure A-2 also shows the annual duration curve for the 1963-1992, 30-year period to illustrate the increase in stage that has occurred over the recent climatic period.

The curve developed with April 1–June 30 data characterizes the full extent of the growing season drawdown window throughout all of Green Island. The target with-Project minimum drawdown elevation (582.82’ NAVD88) for both Pools A & B plotted illustrates that the river exceeds the desired growing season elevation 100% of the time, thus providing extremely limited opportunity for gravity drainage to support this water level management goal. The curve developed for the September 1-October 31 data represents the fall raise period for waterfowl migration at Green Island. The target elevations for this raise in Pools A & B (587.72’ & 584.82’ NAVD88, respectively) illustrate that the river equals or exceeds the Pool A target elevation ~15% of the time and the Pool B target elevation ~43% of the time. Thus, gravity filling is shown to be a potential source to supplement pumping to meet the Pool B fall raise, however the higher target elevation necessary for the Pool A fall raise indicates that gravity filling only serves as a minimal supplement to pumping. These results demonstrate the sponsor’s regular need for pump operation to meet the annual fall raise in Pool A to elevation 587.72’ NAVD88. Detailed description of the existing Green Island infrastructure and water level management is presented in section 3.6 and 3.6.1, respectively. Details of with-Project WLMPs are found in section 3.6.3.

Table A-4. Elevation Duration Curves interpolated for the Green Island midpoint (RM 547.25) Developed from (1) All data 1993–2022, (2) April 1–June 30 1993–2022, and (3) September 1-Oct 31 1993-2022. Elevations in NAVD88 vertical datum.

% Days Exceeded	Elev, ft NAVD88			% Days Exceeded	Elev, ft NAVD88		
	All Months	April 1-June 30	Sept 1-Oct 31		All Months	April 1-June 30	Sept 1-Oct 31
99.90%	582.40	583.66	582.23	35%	586.59	589.26	585.4
99.50%	582.70	583.95	582.4	30%	587.03	589.6	585.71
99%	582.83	584.12	582.52	25%	587.65	589.98	586.14
98%	582.97	584.32	582.7	20%	588.27	590.48	586.74
97%	583.08	584.45	582.77	15%	588.86	591.24	587.79
96%	583.16	584.58	582.83	10%	589.72	592.21	588.7
95%	583.24	584.67	582.87	9%	589.92	592.43	588.91
94%	583.30	584.78	582.9	8%	590.16	592.74	589.12
93%	583.38	584.91	582.94	7%	590.46	593.01	589.43
92%	583.45	585.02	583.01	6%	590.78	593.32	589.72
91%	583.51	585.08	583.06	5%	591.24	593.78	590.1
90%	583.57	585.2	583.09	4.5%	591.51	593.99	590.39
85%	583.86	585.64	583.22	4%	591.80	594.46	590.75
80%	584.08	586.1	583.37	3.5%	592.07	594.93	591.19
75%	584.29	586.48	583.51	3%	592.30	595.29	591.44
70%	584.51	586.91	583.63	2.5%	592.73	595.59	591.67
65%	584.73	587.27	583.83	2%	593.19	596.18	591.86
60%	584.97	587.66	583.98	1.5%	593.92	596.74	592.07
55%	585.22	588.02	584.13	1%	595.03	597.23	592.38
50%	585.50	588.32	584.37	0.5%	596.68	597.82	592.71
45%	585.82	588.59	584.66	0.1%	597.97	599.11	594.29
40%	586.17	588.93	585				

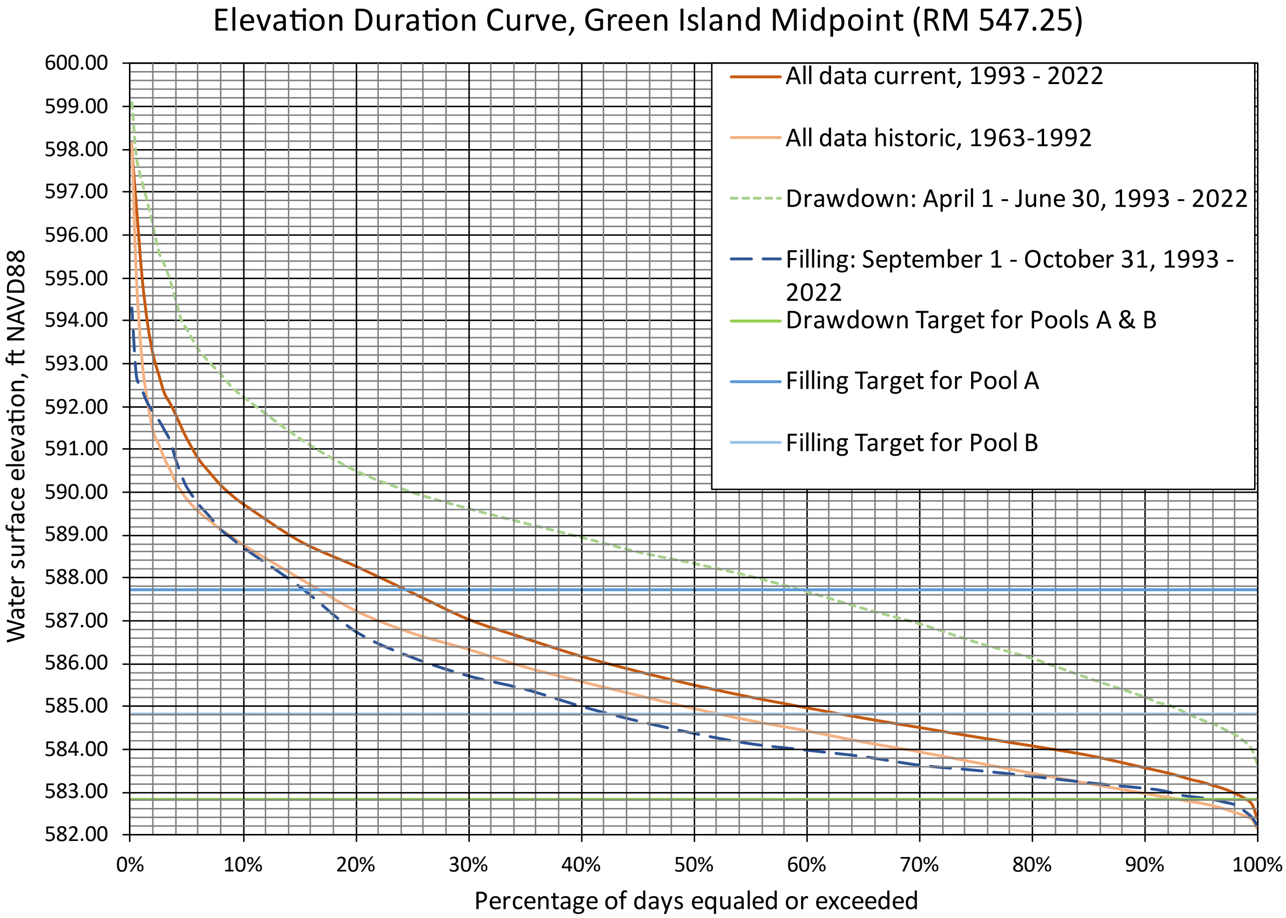


Figure A-2. Elevation Duration Curves at Green Island Midpoint (RM 547.25).

Duration curve developed from all data 1993–2022 is shown in rust. Duration curve developed from April 1–June 30 1993–2022 is shown in lime green (dashed). Duration curve developed from September 1-October 31 1993-2022 is shown in navy (dashed). Target drawdown elevation for Pools A & B is shown in light green. Target filling elevation for Pool A is shown in light blue. Target filling elevation for Pool B is shown in pale blue.

3.4 Mississippi River Flood Conditions

Table A-5 lists the 13 highest water events at the Mississippi River at L&D 12, Bellevue, IA gage; the highest flood on record occurred in late April 1965 and resulted in a water surface elevation of 603.03 feet NAVD88 (23.51 feet of stage).

The 2004 Upper Mississippi River System Flow Frequency Study (UMRSFFS) includes several cross-sections through the study area (USACE, 2004). Table A-6 shows results from the 2004 UMRSFFS that pertain to the study area and nearby gages. Cross sections that immediately bound and include the Green Island HREP are highlighted (RM 545.4 - RM 549.1)

Table A-5. Record High Stages at Mississippi River at L&D 12, Bellevue, IA Gage

Stage (ft)	Water Surface Elevation (ft NAVD88)	Date
23.51	603.03	04/26/1965
22.58	602.1	04/22/2001
21.78	601.3	04/29/2023
21.50	601.02	07/01/1993
20.87	600.39	04/29/2019
20.58	600.1	04/20/2011
20.16	599.68	04/17/1997
20.11	599.63	04/26/1952
20.06	599.58	04/24/1969
20.00	599.52	04/23/1951
19.95	599.47	05/06/1975
19.21	598.73	10/06/1986
19.13	598.65	04/24/1973
18.93	598.45	04/13/1967

Table A-6. 2004 Upper Mississippi River System Flow Frequency Study–Water Surface Elevations (WSE) and Discharges Corresponding to Each Annual Exceedance Probability (AEP) (USACE, 2004)

River Mile	0.5 AEP		0.2 AEP		0.1 AEP		0.04 AEP		0.02 AEP		0.01 AEP		0.005 AEP		0.002 AEP	
	WSE (ft NAVD88)	Flow (cfs)	WSE (ft NAVD88)	Flow (cfs)	WSE (ft NAVD88)	Flow (cfs)	WSE (ft NAVD88)	Flow (cfs)	WSE (ft NAVD88)	Flow (cfs)	WSE (ft NAVD88)	Flow (cfs)	WSE (ft NAVD88)	Flow (cfs)	WSE (ft NAVD88)	Flow (cfs)
522.6	582.4	131,000	585.1	174,000	587.3	202,000	589.8	235,000	591.5	259,000	593.1	283,000	594.3	306,000	595.6	337,000
522.7	582.4	131,000	585.2	174,000	587.4	201,000	589.8	235,000	591.5	259,000	593.1	283,000	594.3	306,000	595.6	337,000
522.8	582.5	131,000	585.3	174,000	587.4	201,000	589.9	235,000	591.6	259,000	593.2	283,000	594.4	306,000	595.6	337,000
523.1	582.6	131,000	585.4	174,000	587.5	201,000	590.0	235,000	591.7	259,000	593.3	283,000	594.4	306,000	595.7	337,000
523.6	582.7	131,000	585.4	174,000	587.6	201,000	590.0	235,000	591.7	259,000	593.3	283,000	594.5	306,000	595.7	337,000
524	582.8	131,000	585.5	174,000	587.6	201,000	590.1	235,000	591.8	259,000	593.3	283,000	594.5	306,000	595.7	336,000
524.5	583.0	131,000	585.6	174,000	587.7	201,000	590.1	235,000	591.8	259,000	593.4	283,000	594.5	306,000	595.8	336,000
525	583.2	131,000	585.7	174,000	587.7	201,000	590.2	235,000	591.8	259,000	593.4	283,000	594.5	306,000	595.8	336,000
526	583.4	131,000	585.8	174,000	587.8	201,000	590.3	235,000	591.9	259,000	593.4	283,000	594.6	306,000	595.8	336,000
526.6	583.6	131,000	586.0	174,000	587.9	201,000	590.3	235,000	591.9	259,000	593.5	283,000	594.6	306,000	595.9	336,000
527	583.8	131,000	586.1	174,000	588.0	201,000	590.4	235,000	592.0	259,000	593.5	283,000	594.6	306,000	595.9	336,000
528	584.0	131,000	586.2	174,000	588.1	201,000	590.4	235,000	592.0	259,000	593.5	283,000	594.7	306,000	595.9	336,000
528.5	584.3	131,000	586.4	174,000	588.2	201,000	590.5	235,000	592.1	259,000	593.6	282,000	594.7	306,000	596.0	336,000
529	584.4	131,000	586.6	174,000	588.3	201,000	590.6	235,000	592.2	259,000	593.7	282,000	594.8	306,000	596.0	336,000
529.7	584.6	131,000	586.7	174,000	588.4	201,000	590.7	235,000	592.2	259,000	593.7	282,000	594.8	306,000	596.1	336,000
530	584.8	130,000	586.9	174,000	588.6	201,000	590.8	235,000	592.3	259,000	593.8	282,000	594.9	306,000	596.1	336,000
530.9	585.1	130,000	587.1	174,000	588.8	201,000	591.0	235,000	592.5	259,000	593.9	282,000	595.0	306,000	596.3	336,000
531.7	585.4	130,000	587.4	174,000	589.0	201,000	591.1	235,000	592.6	259,000	594.0	282,000	595.1	306,000	596.4	336,000
532.3	585.6	130,000	587.6	174,000	589.2	201,000	591.3	235,000	592.7	259,000	594.1	282,000	595.3	306,000	596.5	336,000
532.55	585.7	130,000	587.7	174,000	589.3	201,000	591.4	235,000	592.9	259,000	594.2	282,000	595.3	306,000	596.6	336,000
532.8	585.8	130,000	587.8	174,000	589.4	201,000	591.5	235,000	593.0	259,000	594.3	282,000	595.4	306,000	596.6	336,000
533.5	586.0	130,000	588.1	174,000	589.7	201,000	591.8	235,000	593.2	259,000	594.5	282,000	595.6	306,000	596.8	336,000
534.1	586.2	130,000	588.4	174,000	590.0	201,000	592.0	234,000	593.4	259,000	594.7	282,000	595.7	305,000	596.9	336,000
535.1	586.4	130,000	588.6	174,000	590.3	201,000	592.3	234,000	593.7	259,000	594.9	282,000	595.9	305,000	597.1	336,000
535.5	586.6	130,000	588.8	174,000	590.4	201,000	592.5	234,000	593.8	259,000	595.0	282,000	596.1	305,000	597.3	336,000
535.7	586.6	130,000	588.9	174,000	590.6	201,000	592.6	234,000	594.0	259,000	595.1	282,000	596.2	305,000	597.4	336,000
535.9	586.7	130,000	589.0	173,000	590.7	200,000	592.7	234,000	594.1	258,000	595.2	281,000	596.3	305,000	597.5	335,000
536.4	586.9	130,000	589.3	173,000	590.9	200,000	593.0	234,000	594.4	258,000	595.5	281,000	596.6	305,000	597.7	335,000
537.1	587.2	130,000	589.6	173,000	591.3	200,000	593.4	234,000	594.7	258,000	595.8	281,000	596.8	305,000	598.0	335,000

River Mile	0.5 AEP		0.2 AEP		0.1 AEP		0.04 AEP		0.02 AEP		0.01 AEP		0.005 AEP		0.002 AEP	
	WSE (ft NAVD88)	Flow (cfs)	WSE (ft NAVD88)	Flow (cfs)	WSE (ft NAVD88)	Flow (cfs)	WSE (ft NAVD88)	Flow (cfs)	WSE (ft NAVD88)	Flow (cfs)	WSE (ft NAVD88)	Flow (cfs)	WSE (ft NAVD88)	Flow (cfs)	WSE (ft NAVD88)	Flow (cfs)
537.7	587.5	130,000	589.9	173,000	591.6	200,000	593.6	234,000	595.0	258,000	596.1	281,000	597.1	304,000	598.3	335,000
538.1	587.6	130,000	590.1	173,000	591.8	200,000	593.9	234,000	595.2	258,000	596.3	281,000	597.3	304,000	598.5	335,000
538.5	587.8	130,000	590.3	173,000	592.1	200,000	594.2	234,000	595.5	258,000	596.6	281,000	597.6	304,000	598.8	335,000
538.8	588.0	130,000	590.5	173,000	592.3	200,000	594.4	234,000	595.7	258,000	596.8	281,000	597.8	304,000	599.0	335,000
539.1	588.2	130,000	590.7	173,000	592.5	200,000	594.5	234,000	595.8	258,000	596.9	281,000	597.9	304,000	599.1	335,000
539.9	588.4	130,000	590.9	173,000	592.7	200,000	594.7	234,000	596.0	258,000	597.1	281,000	598.1	304,000	599.3	335,000
540.6	588.6	130,000	591.1	173,000	592.9	200,000	594.9	234,000	596.2	258,000	597.3	281,000	598.3	304,000	599.4	335,000
541.2	588.7	130,000	591.3	173,000	593.0	200,000	595.1	234,000	596.4	258,000	597.5	281,000	598.4	304,000	599.6	335,000
541.8	588.9	130,000	591.5	173,000	593.2	200,000	595.3	234,000	596.5	258,000	597.6	281,000	598.6	304,000	599.7	335,000
542.6	589.2	130,000	591.7	173,000	593.5	200,000	595.5	233,000	596.8	257,000	597.8	281,000	598.8	304,000	599.9	334,000
543.3	589.4	130,000	592.0	173,000	593.7	200,000	595.7	233,000	597.0	257,000	598.1	281,000	599.0	304,000	600.2	334,000
543.7	589.6	130,000	592.2	173,000	593.9	200,000	595.9	233,000	597.2	257,000	598.3	281,000	599.2	304,000	600.4	334,000
544.3	589.9	130,000	592.5	173,000	594.2	200,000	596.2	233,000	597.4	257,000	598.5	281,000	599.4	304,000	600.6	334,000
544.8	590.1	130,000	592.7	173,000	594.4	200,000	596.4	233,000	597.6	257,000	598.6	281,000	599.6	304,000	600.8	334,000
545.4	590.2	130,000	592.9	172,000	594.6	200,000	596.6	233,000	597.7	257,000	598.8	280,000	599.7	303,000	600.9	333,000
546	590.4	129,000	593.1	172,000	594.8	200,000	596.8	233,000	597.9	257,000	598.9	280,000	599.9	303,000	601.0	333,000
546.4	590.5	129,000	593.2	172,000	594.9	200,000	596.9	233,000	598.1	257,000	599.1	280,000	600.0	303,000	601.2	333,000
547	590.7	129,000	593.5	172,000	595.2	200,000	597.2	233,000	598.3	257,000	599.3	280,000	600.3	303,000	601.5	333,000
547.5	590.9	129,000	593.6	172,000	595.4	200,000	597.4	233,000	598.5	257,000	599.5	280,000	600.5	303,000	601.7	333,000
548.1	591.1	129,000	593.9	172,000	595.6	200,000	597.7	233,000	598.8	257,000	599.8	280,000	600.8	303,000	602.0	333,000
548.6	591.2	129,000	594.0	172,000	595.8	200,000	597.8	233,000	599.0	257,000	600.0	280,000	601.0	303,000	602.2	333,000
549.1	591.4	127,000	594.3	169,000	596.0	196,000	598.0	229,000	599.2	252,000	600.3	275,000	601.3	298,000	602.5	328,000
549.7	591.6	127,000	594.5	169,000	596.2	196,000	598.2	229,000	599.5	252,000	600.5	275,000	601.5	298,000	602.7	328,000
550.05	591.7	127,000	594.7	169,000	596.3	196,000	598.4	229,000	599.6	252,000	600.7	275,000	601.7	298,000	602.8	328,000
550.4	591.8	127,000	594.8	169,000	596.4	196,000	598.5	229,000	599.7	252,000	600.8	275,000	601.8	298,000	602.9	328,000
551	591.9	127,000	594.9	169,000	596.5	196,000	598.6	229,000	599.8	252,000	600.9	275,000	601.9	298,000	603.0	328,000
552	592.1	127,000	595.1	169,000	596.7	196,000	598.8	229,000	600.0	252,000	601.1	275,000	602.1	298,000	603.2	327,000
552.8	592.3	127,000	595.3	169,000	596.9	196,000	598.9	229,000	600.2	252,000	601.2	275,000	602.2	298,000	603.4	327,000
553.3	592.5	127,000	595.4	169,000	597.1	196,000	599.1	229,000	600.3	252,000	601.4	275,000	602.4	298,000	603.5	327,000
554	592.7	127,000	595.6	169,000	597.2	196,000	599.3	229,000	600.5	252,000	601.5	275,000	602.5	298,000	603.7	327,000
554.5	592.8	127,000	595.7	169,000	597.3	196,000	599.4	229,000	600.6	252,000	601.6	275,000	602.6	298,000	603.8	327,000

River Mile	0.5 AEP		0.2 AEP		0.1 AEP		0.04 AEP		0.02 AEP		0.01 AEP		0.005 AEP		0.002 AEP	
	WSE (ft NAVD88)	Flow (cfs)	WSE (ft NAVD88)	Flow (cfs)	WSE (ft NAVD88)	Flow (cfs)	WSE (ft NAVD88)	Flow (cfs)	WSE (ft NAVD88)	Flow (cfs)	WSE (ft NAVD88)	Flow (cfs)	WSE (ft NAVD88)	Flow (cfs)	WSE (ft NAVD88)	Flow (cfs)
555.2	592.9	127,000	595.8	169,000	597.4	196,000	599.5	229,000	600.7	252,000	601.7	275,000	602.7	298,000	603.9	327,000
555.7	593.0	127,000	595.9	169,000	597.5	196,000	599.6	228,000	600.8	252,000	601.8	275,000	602.8	298,000	604.0	327,000
556.2	593.1	127,000	596.0	169,000	597.7	196,000	599.7	228,000	600.9	252,000	601.9	275,000	602.9	298,000	604.1	327,000
556.6	593.2	127,000	596.1	169,000	597.7	196,000	599.8	228,000	601.0	252,000	602.0	275,000	603.0	298,000	604.2	327,000
556.65	593.2	127,000	596.1	169,000	597.8	196,000	599.8	228,000	601.0	252,000	602.1	275,000	603.0	298,000	604.2	327,000
556.7	593.2	127,000	596.1	169,000	597.8	196,000	599.8	228,000	601.0	252,000	602.1	275,000	603.0	298,000	604.2	327,000

3.5 Maquoketa River

Along the upstream boundary of the HREP, the Maquoketa River flows and enters into the Mississippi River near River Mile 548.5. The Maquoketa River watershed is 1,870 square miles in area with predominantly agricultural land cover. At the Mississippi River–Maquoketa confluence, the Maquoketa River drainage area makes up approximately 2% of the total drainage area. The Green Island Levee, described in greater detail below in section 3.6, manages flood risk for approximately 4 miles (between Sta. 202+00 and 0+00) along the Maquoketa River from the mouth, upstream to the tieback near 475th Avenue, approximately 2 miles upstream of the Highway 52 bridge (Figure A-11). The Green Island levee serves as the HREP boundary along the northwestern boundary between approximate Sta.202+00 and Sta.194+00. Flooding along the Maquoketa River reach of the Green Island Levee has been limited to the reach upstream of the Highway 52 and railroad bridge embankments (near Sta.107+00), primarily resulting from ice jams. According to the sponsor, the Highway 52 embankment acts as a hydraulic barrier that prevents upstream flooding from impacting downstream areas, north of Highway 52, including the Green Island HREP. In 2010, flooding resulted in breaching of the Green Island Levee in two locations along the Maquoketa River upstream of the Highway 52 bridge embankment but did not impact areas adjacent to the Green Island HREP, north of Highway 52. The flooding risk and history along other reaches of the Green Island Levee that border the HREP are discussed in more detail in sections 3.6.3 and 3.6.4, respectively.

As described below, a gate structure on the Maquoketa River allows flow to move between the river and Mooney Hollow Creek. The Maquoketa River contributes a substantial amount of suspended sediment to the Mississippi River sediment load at the confluence. However, this structure is primarily operated to allow flow from Mooney Hollow into the Maquoketa and thus is not considered a significant source for introducing sediment into the Project. Annual pumping and ingress gate flow from the Mississippi River into Green Island is just downstream of the Maquoketa River confluence. The influence of Maquoketa River water quality on water pumped annually into the Project from the Mississippi was accounted for in the sediment deposition estimates described in section 5.

3.6 Existing Infrastructure & Management

The Green Island HREP is located within the Green Island Wildlife Management Area (GIWMA) and is surrounded by berm features, limiting connectivity with the adjacent Mississippi and Maquoketa Rivers and allowing for moist soil management of the area (Figure A-3). The Green Island Levee serves as the outer berm along the Mississippi River (between approximate Sta.202+00 and Sta.361+00), the northwestern boundary with the Maquoketa River (between approximate Sta.194+00 and 202+00), and the southeastern boundary with Brown's Lake (between approximate Sta.361+00 to 421+00). The berm along the western and southern boundaries of the project area is not part of the Green Island Levee and thus maintains lower elevations. Flood risk to the Project from the Mississippi and Maquoketa Rivers is described in section 3.6.3. Figure A-3 illustrates how the Green Island HREP boundary does not exactly align with the perimeter berm. An interior berm that forms the GIWMA sub-impoundment 1 serves as the HREP boundary along the southwest corner. Sub-impoundment 1 is managed independently with portable pumps and a 30" in-line water control structure located along the 3rd ditch and was not evaluated as part of this feasibility study. According to the sponsor, elevations along the berm and levee that serve as the HREP boundary contain the maximum-managed water level within Pools A & B (587.72' NAVD88 and 584.82' NAVD88, respectively) with additional freeboard. Lidar-based survey available during feasibility shows a reach along the southern end of Pool A where the elevation is nearly 2 feet below the maximum managed water level, indicating that water would spill over the perimeter berm under the existing and proposed maximum-managed water surface elevation. Elevation data for the sub-impoundment 1 berm was not available during feasibility. Survey for the sub-impoundment 1 berm and the low reach along the southern boundary of Pool A will be obtained during design and is assumed to show elevations well above 587.72’ NAVD88. The lack of survey data will be documented in the risk register and can be managed through elevating the berm if deemed necessary during PED.

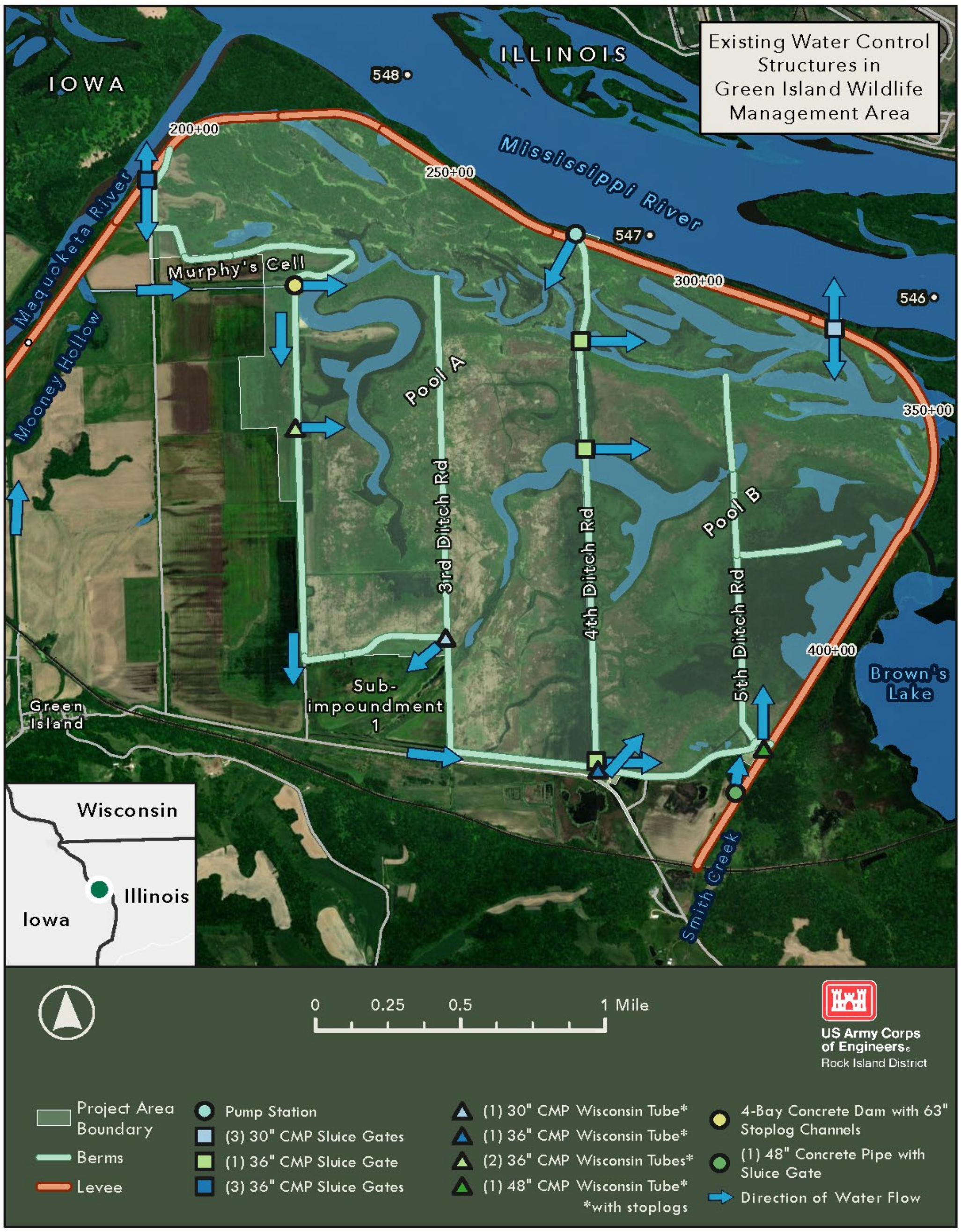


Figure A-3. Green Island HREP Water Management Flow Paths, Impounding Berms and Existing Water Control Structures.

The interior of the Green Island HREP is divided along the 4th Ditch Road into two subimpoundments: Pool A to the west and Pool B to the east (Figure A-3). Pool A and Pool B are managed independently, with three existing 36” in-line water control structures with sluice gates through the 4th Ditch Road providing connectivity between the two pools. Water levels within Murphy’s cell, located in the northwest corner of Pool A, and in sub-impoundment 1, referred to above, are managed independently from the rest of Pool A and were not evaluated as part the feasibility analysis described herein.

Water level management within the GIWMA is dictated by the agreement between Iowa DNR and the Green Island Levee and Drainage District (GILDD). The 2005 cooperative agreement allows for an annual three-foot rise in water level up to 587.72 ft NAVD88 (588.4 ft MSL1912) in Pool A from August 15 to December 15 (Figure A-3a). Outside of those dates, water is to be returned to 584.82 ft NAVD88 (585.5 ft MSL1912) (Figure A-3b). The maximum water level within Pool B, as specified by the agreement, is 584.82 ft NAVD88, which is generally held constant throughout the year (Figure A-3a and A-3b). Pool B is intended to serve as a reservoir to accommodate flow that is routed away from Pool A through the Mooney Hollow Creek.

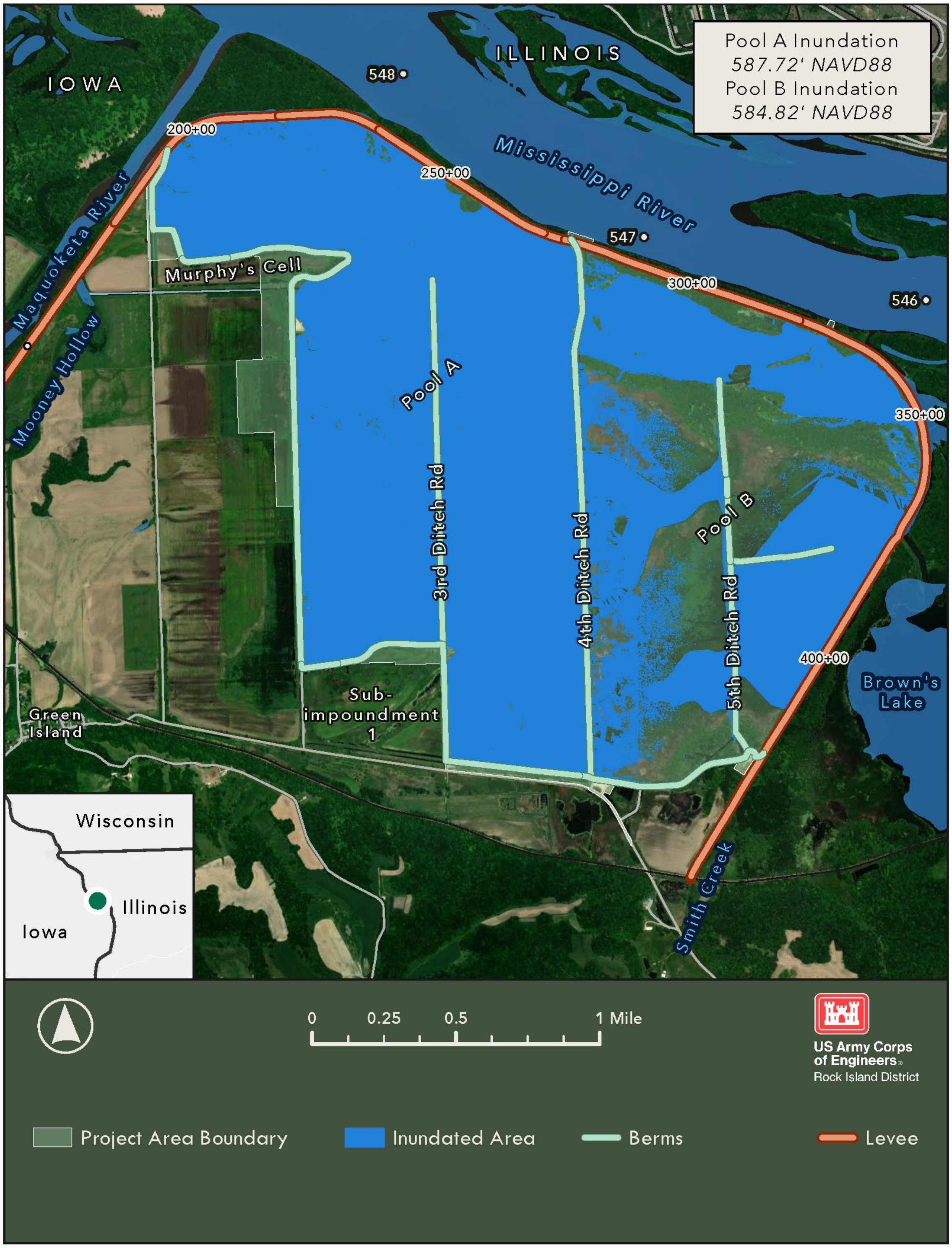


Figure A-3a. Fall maximum water surface elevation inundation in Pool A (587.72' NAVD88) & Pool B (584.82' NAVD88) under existing water level management, the Project's Typical WLMP and Drawdown WLMP. *Note the underlying terrain represents existing conditions and does not include Project features. Inundation mapping may include some unrealistic artifacts resulting from the LiDAR data. This mapping does not impact the Project's alternative evaluation.

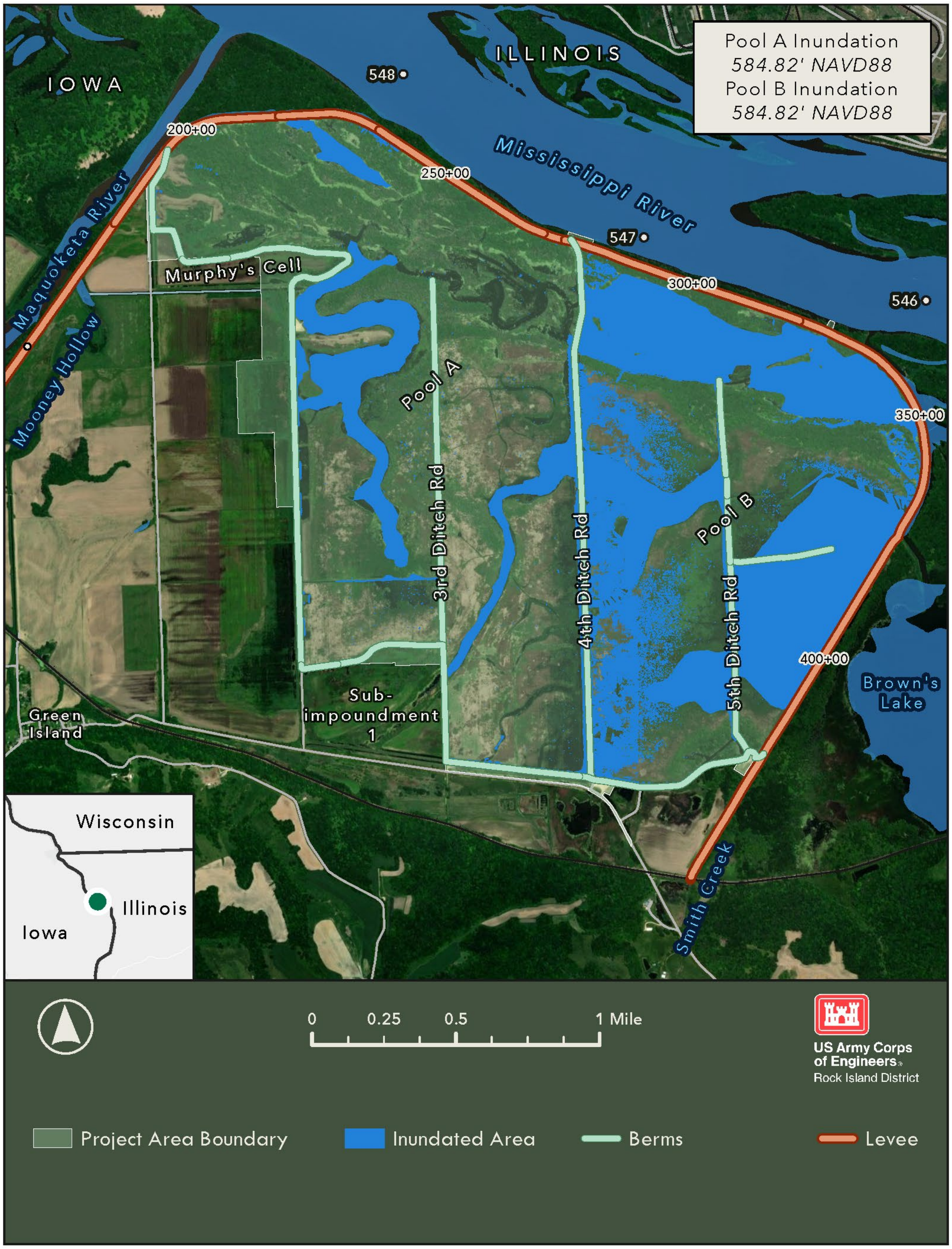


Figure A-3b. Growing season minimum water surface elevation inundation in Pool A (584.82' NAVD88) & Pool B (584.82' NAVD88) under existing water level management and the Project's Typical WLMP. *Note the underlying terrain represents existing conditions and does not include Project features. Inundation mapping may include some unrealistic artifacts resulting from the LiDAR data. This mapping does not impact the Project's alternative evaluation.

Surface water contributions to the Green Island Wildlife Area can include Mooney Hollow Creek, Smith Creek, the Mississippi River and the Maquoketa River (Figure A-3). Depending on Maquoketa River water levels, Mooney Hollow Creek can flow north into the Maquoketa River through a sluice gate structure, be routed east into Pool A through the Fish Lake Dam stoplog structure, or it can be routed south and east around the western and southern boundaries of the project area into Pool B near the 4th Ditch Road parking lot through an in-line structure with stop-logs (Figure A-3). Water from the Maquoketa River can also be routed through this Mooney Hollow drainage network and delivered to either Pool A or Pool B through these structures if water levels are conducive and there is a management need. Gravity flow into Pool A during the fall to meet migratory waterfowl needs is unusual as water levels in both Mooney Hollow and the Maquoketa River are typically low. An existing pump station with two-20,000 gallon per minute (GPM) pumps is used to pump water from the Mississippi River into Pool A to achieve the necessary fall rise. The existing pump station can only pump water into Pool A. As mentioned previously, connectivity between Pool A and Pool B is achieved by three in-line water control structures along the 4th Ditch Road. Pool B maintains an elevation of 584.82 ft and is managed as a source for storage when Mooney Hollow Creek cannot be discharged to either the Maquoketa River or into Pool A. Pool B has a sluice gate structure with three-30” culverts allowing gravity flow to and from the Mississippi River when water levels can accommodate. This Mississippi River sluice gate is the only means for draining Green Island.

3.6.1 Existing Water Level Management

Pumping operations under existing conditions typically begin in early-mid September, and water levels in Pool A are steadily increased to crest elevation (587.72 NAVD88) by approximately November 1st. Filling occurs in the fall to provide resting and feeding habitat for migratory waterfowl. Necessary depths for waterfowl cannot be met without the use of ingress pumps. Gravity filling to supplement pumping is rare as the Maquoketa River and Mooney Hollow Creek sources are typically low during the fall. Water levels in Pool A are drawn down slowly and steadily after the close of the duck season (typically mid-December) through gravity drainage into Pool B and through the Mississippi River gate. Existing water level management in Pool B maintains water levels around 584.82 NAVD88 (585.5 MSL1912) year-round. Gravity filling of Pool B through the Mississippi River gate for interior filling to meet the Pool A fall rise is inadequate due to low Mississippi River levels during the fall (Figure A-2, Table A-4). Furthermore, filling Pool B above the maintained water surface elevation (584.82 NAVD88) would conflict with the GILDD agreement. Each spring, the goal across the project area is to draw down water to the extent possible, ideally for a minimum of 30 days, to promote the growth of desirable aquatic vegetation and maintain diverse forest resources. However, in order to increase the abundance and diversity of aquatic and floodplain vegetation periodic, deeper drawdowns than what are currently managed for are necessary, giving rise to the primary objective of the projectP. Currently drawdowns can only be achieved through gravity drainage, and because Mississippi River levels are too high during the growing season drawdown period, there is little opportunity to achieve a significant drawdown (Figure A-2, Table A-4). Maximum and minimum inundation in Pools A & B under existing water level management are shown in Figures A-3a and A-3b. Existing water level management in Pools A & B is similar to the with-Project Typical WLMP as illustrated below in Figure A-4 and Figure A-5.

3.6.2 With-Project Water Level Management

As discussed previously, under existing conditions Pools A and B within the Green Island Wildlife Area are managed independently and the maximum managed water level in Pool A (587.72 ft NAVD88) is established by the 2005 agreement between Iowa DNR and the GILDD.

Minimum water surface elevations in both Pools A and B are constrained by high river levels which limit the extent of, or prevent gravity drainage, the only existing drainage infrastructure (Figure A-3). The proposed Project seeks to provide infrastructure to support a Typical WLMP, similar to the existing water level management, to be implemented approximately four out of every five years and a Drawdown WLMP to be opportunistically implemented approximately once every five years (Figures A-4 – A-7). The Tentatively Selected Plan (TSP) includes a 2-way pump station that allows flow both into and out of Green Island and a sluice gate structure for increased flexibility in gravity flow. The pump station would be located along the Green Island Levee near the 4th Ditch Road. This location will allow for separate channel construction into Pool A and Pool B, such that ingress and egress pumping into and out of Pool A and Pool B can be done independently or together. A 2-way pump station will allow the sponsor to manage for the deeper drawdowns illustrated in the Drawdown WLMP as described below (Figures A-5, A-7 & A-19). A sluice gate structure near Brown's Lake will provide additional Pool B filling capacity in the fall following the drawdown and modestly improve opportunities for gravity drainage through Pool B, based on the increased head differential in the backwater channel relative to the mainstem Mississippi. Additional details regarding the design of these two water level management features is provided in section 6. The Pool A Typical WLMP looks similar to the existing management plan described previously with a maximum elevation of 587.72 ft NAVD88 and inundation extents as shown in Figure A-3a (Figure A-4). The Drawdown WLMP in Pool A involves a 60-day drawdown beginning April 1st reaching a minimum elevation of 582.82 ft NAVD88 that is maintained over a 30-day period from June 1st-July 1st (Figure A-5). Pool A inundation under this minimum elevation under the Drawdown WLMP is shown in Figure A-5a. Water levels would begin increasing after July 1st and gradually rise to 584.82 ft NAVD88 on September 1st, continue filling to 586.72 on October 1st and reach a maximum level of 587.72 on November 1st that is maintained until December 15th. Maximum inundation in Pool A under the Drawdown WLMP (587.72 ft NAVD88) is the same maximum inundation that occurs under existing conditions as shown in Figure A-3a. After December 15th Pool A water levels are gradually drawn down through gravity, back to 584.82 by April 1st. The Pool B Typical WLMP looks similar to the existing management plan, whereby water levels are maintained at 584.82 (Figures A-6 & A-3b). The Pool B Drawdown WLMP includes a 60-day drawdown beginning May 1st until July 1st when a minimum elevation of 582.82 ft NAVD88 is reached and maintained for 45 days, until August 15th (Figure A-7). Minimum Pool B inundation under the Drawdown WLMP is illustrated in Figure A-5a. Water levels will then be gradually raised reaching 584.82 on October 1st. Drawdown WLMPs will utilize gravity drainage to the extent possible, however operation of the proposed (40,000 GPM) bi-directional pump station is anticipated to be necessary to successfully meet and maintain the drawdown as illustrated by the Mississippi River drawdown seasonal duration curve (Figure A-2). Supplemental filling of Pool B will be achieved through gravity structures such as the existing Mississippi River gate and the proposed Brown's Lake gate whenever possible, however it is anticipated that filling of both Pools A & B will continue to rely primarily on pumping.

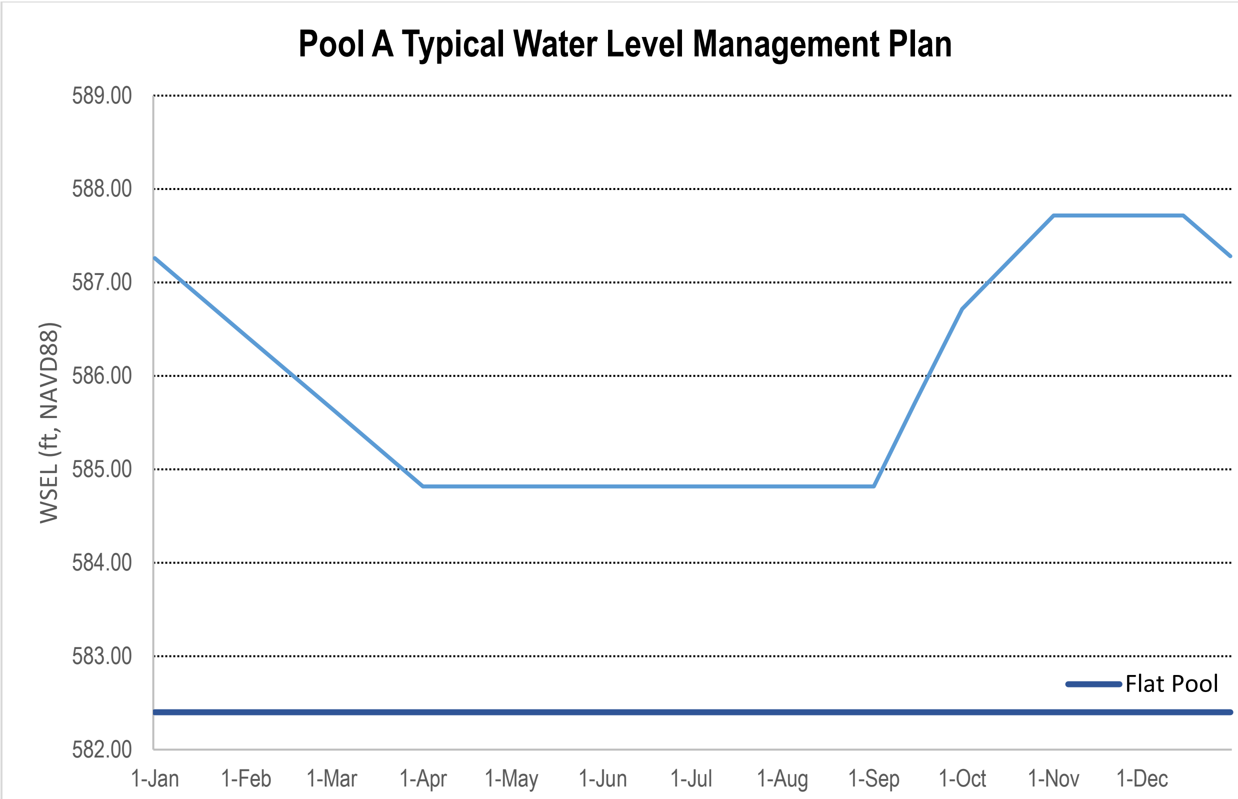


Figure A-4. Pool A Typical WLMP

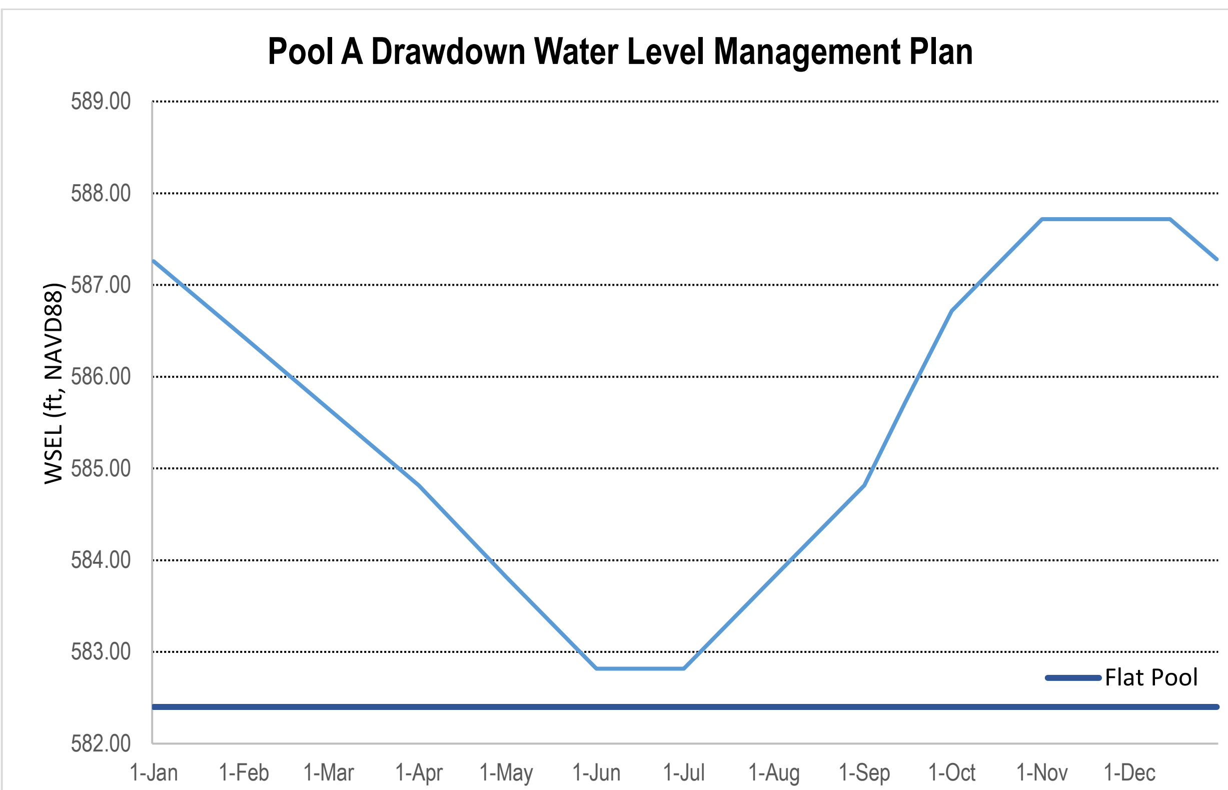


Figure A-5. Pool A Drawdown WLMP

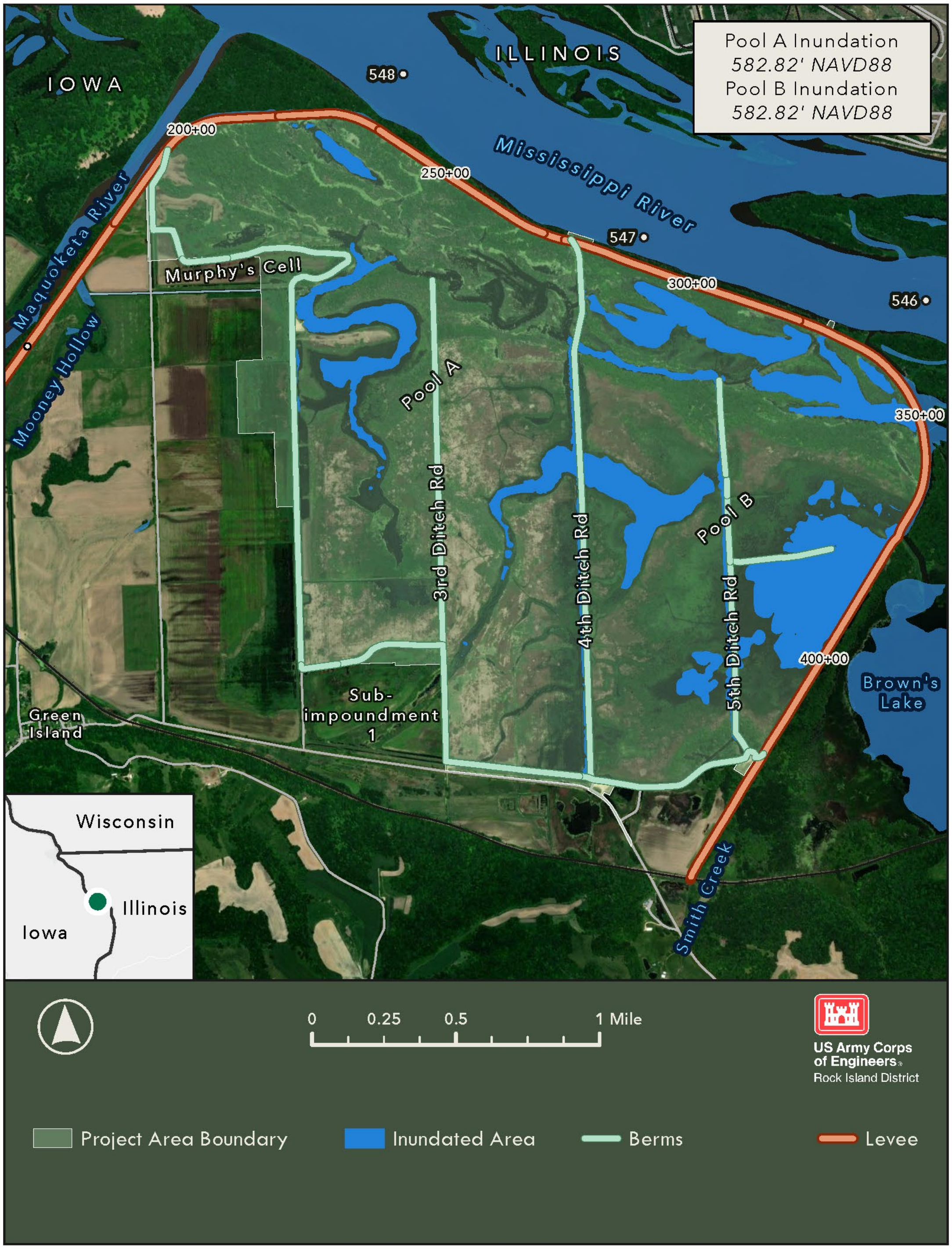


Figure A-5a. Growing season minimum water surface elevation in Pool A (582.82' NAVD88) & Pool B (582.82' NAVD88) under the Project's Drawdown WLMP.

*Note the underlying terrain represents existing conditions and does not include Project features. Inundation mapping may include some unrealistic artifacts resulting from the LiDAR data. This mapping does not impact the Project's alternative evaluation.

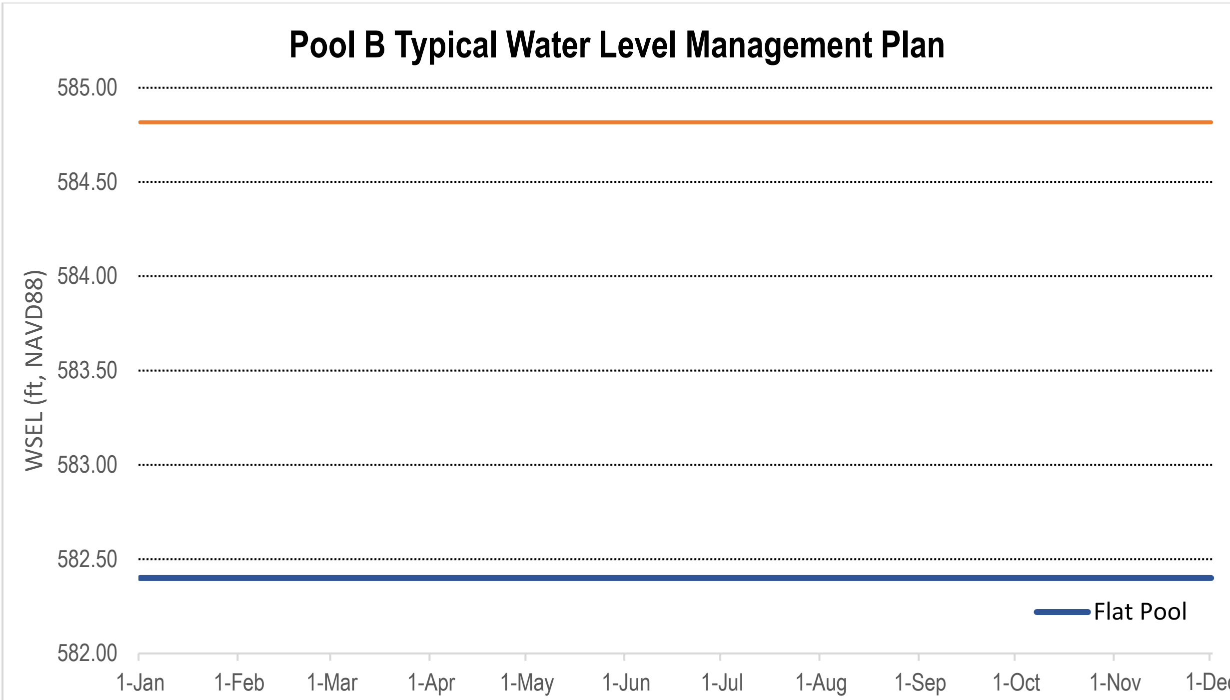


Figure A-6. Pool B Typical WLMP

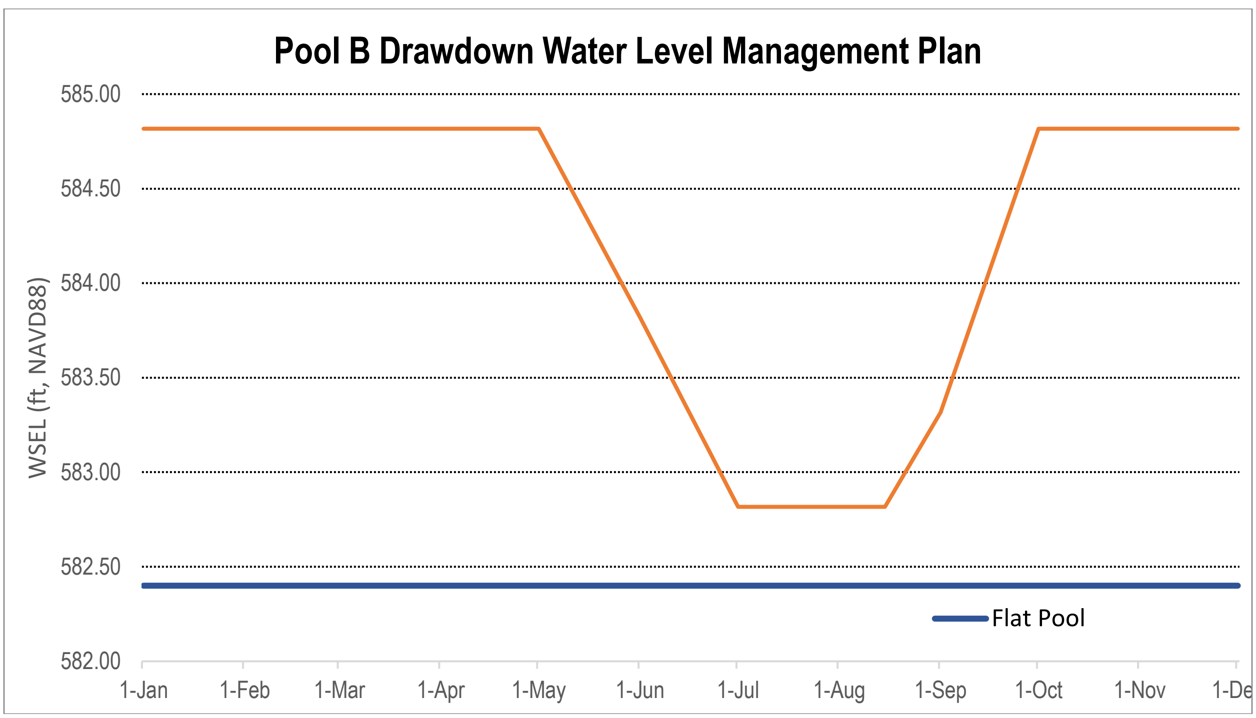
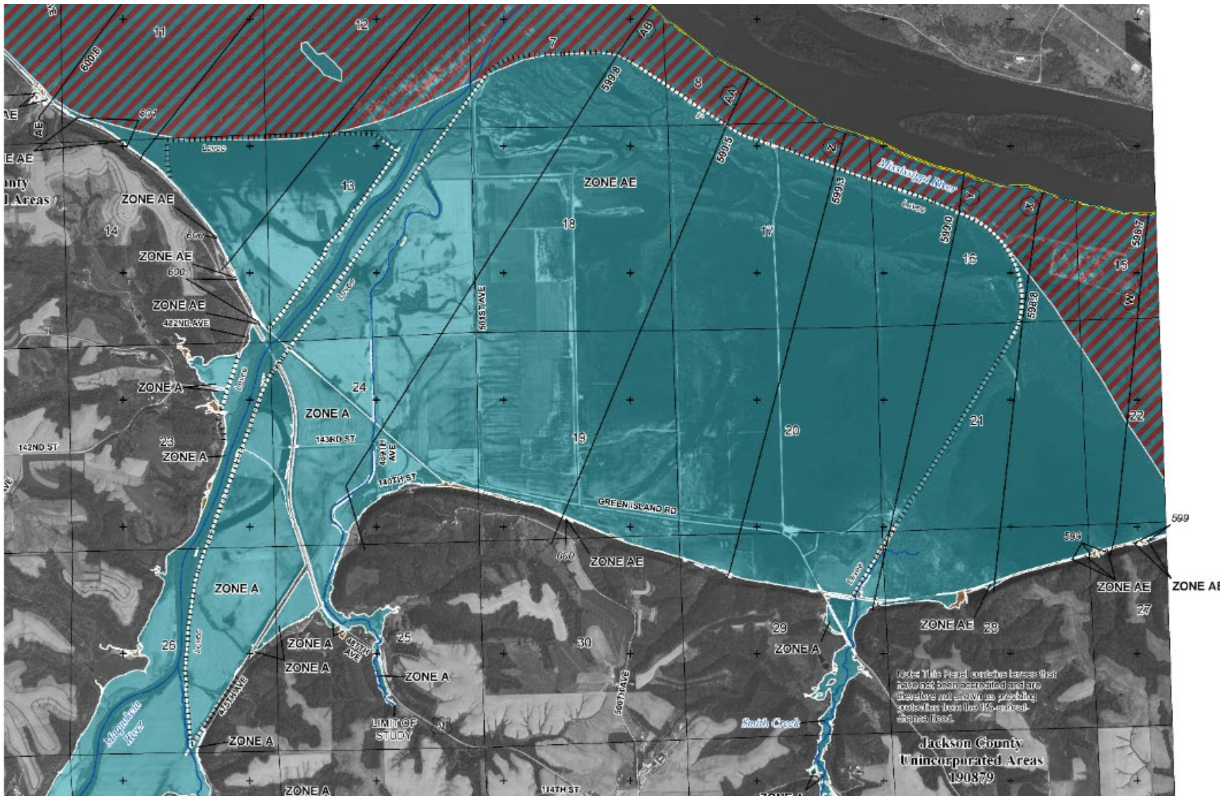


Figure A-7. Pool B Drawdown WLMP

3.6.3 Green Island Levee- Existing Hydraulic Superiority & Structural Superiority



Hydraulic superiority and structural superiority are important components of controlled overtopping and reducing the risk of damages resulting from levee overtopping. Early in the study the sponsor indicated the need to screen any levee degrading measures such as an armored spillway feature that would compromise their flood flighting and likely require decreasing the existing level of protection to provide controlled overtopping. Following the 2023 flood, the sponsor cited the 2005 GILDD agreement specifying that the levee maintain a minimum elevation of 596.5 feet MSL1912, a constraint to project planning. However, following the 2023 levee breach the PDT recognized the need to re-assess the risk of levee failure throughout the Project, therefore a controlled overtopping assessment will be evaluated during Planning Engineering and Design (PED). Existing hydraulic superiority and sequence of overtopping of the Green Island levee are described in the following Structural superiority surrounding the existing pump station and Mississippi gate as well as planned structural superiority around the 2-way pump station and Brown's Lake gate TSP features are also described herein.

Plots of perimeter berm elevations at 100' foot stationing increments with AEP water surface profiles, based on the 2004 UMRSFFS, were developed for the upstream tie-back, mainstem and downstream tie-back reaches of the perimeter levee that make up the Green Island HREP boundary (Figures A-3). The upstream tie-back reach is adjacent to the Maquoketa River that does not have computed flood profiles. To provide a reasonably conservative estimate of flood risk along this upstream tie-back reach, the flow frequency estimates for the Mississippi River cross-section upstream of the Maquoketa confluence were plotted on Figure A-10. Although the elevation variation is only captured every 100', the general sequencing of overtopping is apparent. The source of the elevation data along the mainstem and downstream tie-back reaches is the 2020 Project survey data collected by the Rock Island District survey branch. Plotted levee elevations along the downstream tie-back/Maquoketa River reach of the levee are based on the National Levee Database (NLD). Incipient overtopping along the Green Island Levee, based on 2020 survey data, is located along the downstream tie-back reach bordering Brown’s Lake near Sta. 376+40, RM 545.8, and has an elevation of 595.46 NAVD88 (596.14 MSL 1912). It is noteworthy that this incipient overtopping elevation is not reflected in Figure A-8, instead the approximate location of incipient overtopping is indicated by Sta. 376+00 with an elevation of 595.63 NAVD88, as identified by the gold circle. Overtopping initiates on the downstream end of the Project that borders Brown’s Lake, with approximately 3,000’ of the tie-back levee overtopping when river discharge is between the 10% and 4% AEP events (Figure A-8). The 2008 and 2023 breaches occurred along this reach. Overtopping remains constrained to this downstream tie-back reach until the river discharge reaches around the 2% AEP discharge, after which the mainstem and upstream tie-back levee reaches are overtopped (Figures A-9 & A-10). This general filling sequence from downstream to upstream illustrates that the existing levee profile provides some degree of hydraulic superiority. Early in the study, an armored spillway for controlled overtopping and perimeter berm grading for well-defined hydraulic superiority were discussed with the sponsor, however decreasing the levee elevation for a spillway would be considered a violation of the GILDD agreement and thus this feature was screened, as previously mentioned. The GILDD minimum level elevation is shown on Figure A-8.

The existing ingress pump station and gate structure are located on the mainstem levee and thus are not impacted during the initial perimeter berm overtopping. The proposed 2-way pump station will be located approximately 300’ downstream of the existing pump station along the mainstem levee with 2’ of structural superiority for 100’ on either side, per DIVR 1110-1-16 (Figure A-9). The proposed gate at Brown’s Lake, as part of the TSP, is located above the 2% AEP and will also be constructed with an additional 2’ of structural superiority for 100’ on either side. The location of the proposed gate with structural superiority (approx. Sta. 371+00 to 374+00) is near a previous levee breach and the incipient overtopping location. Levee materials and grading to optimize interior filling in this location should be evaluated during design. Structural superiority further delays overtopping of these structures until the downstream tie-back reach is fully overtopped and a greater length of the mainstem levee is overtopped.

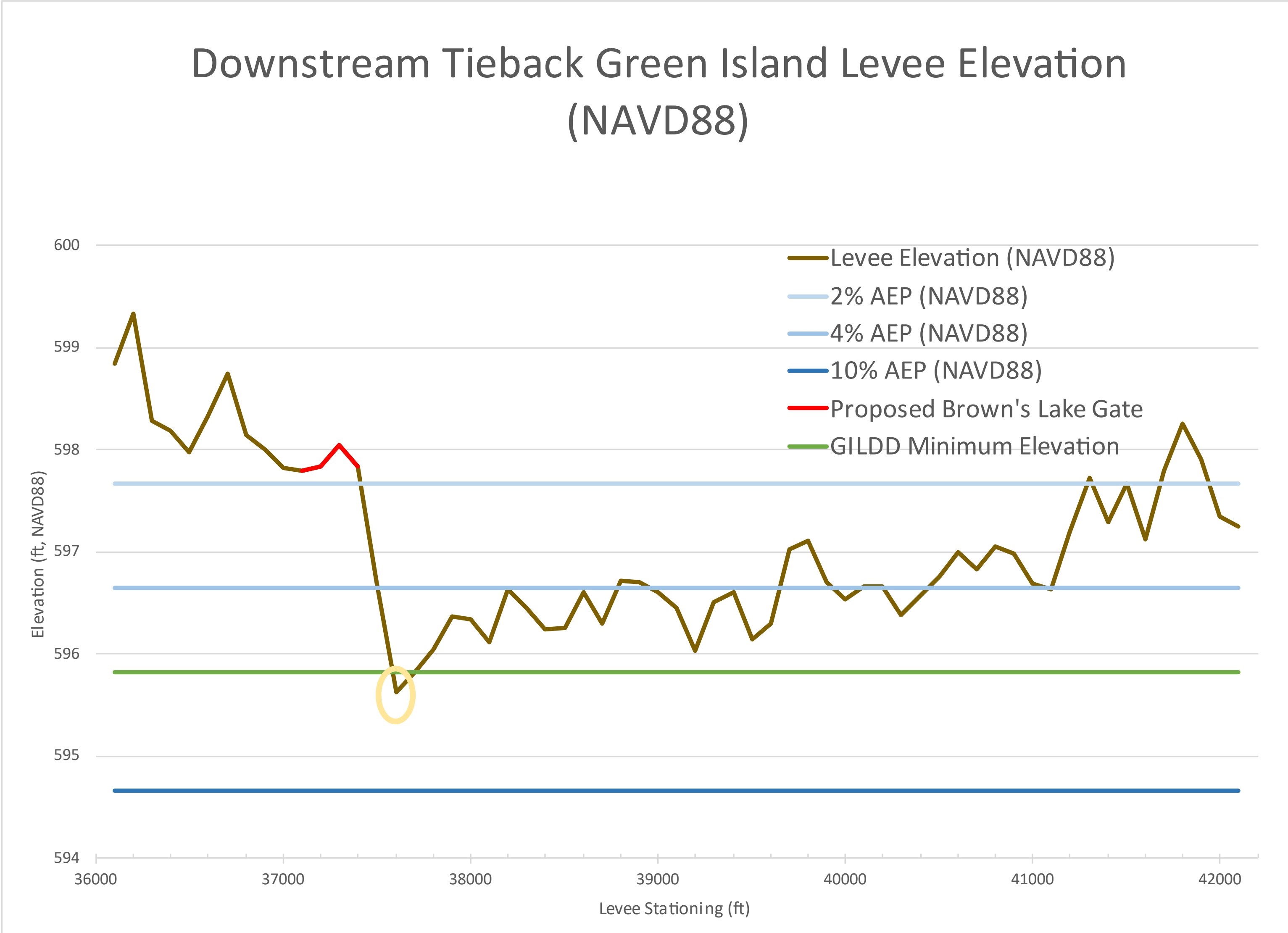


Figure A-8. Elevation and 2004 Flow Frequency Water Surface Profiles along the Downstream Reach of the Green Island Levee. The approximate location of incipient overtopping is identified by the gold circle and the GILDD minimum elevation identified in green.

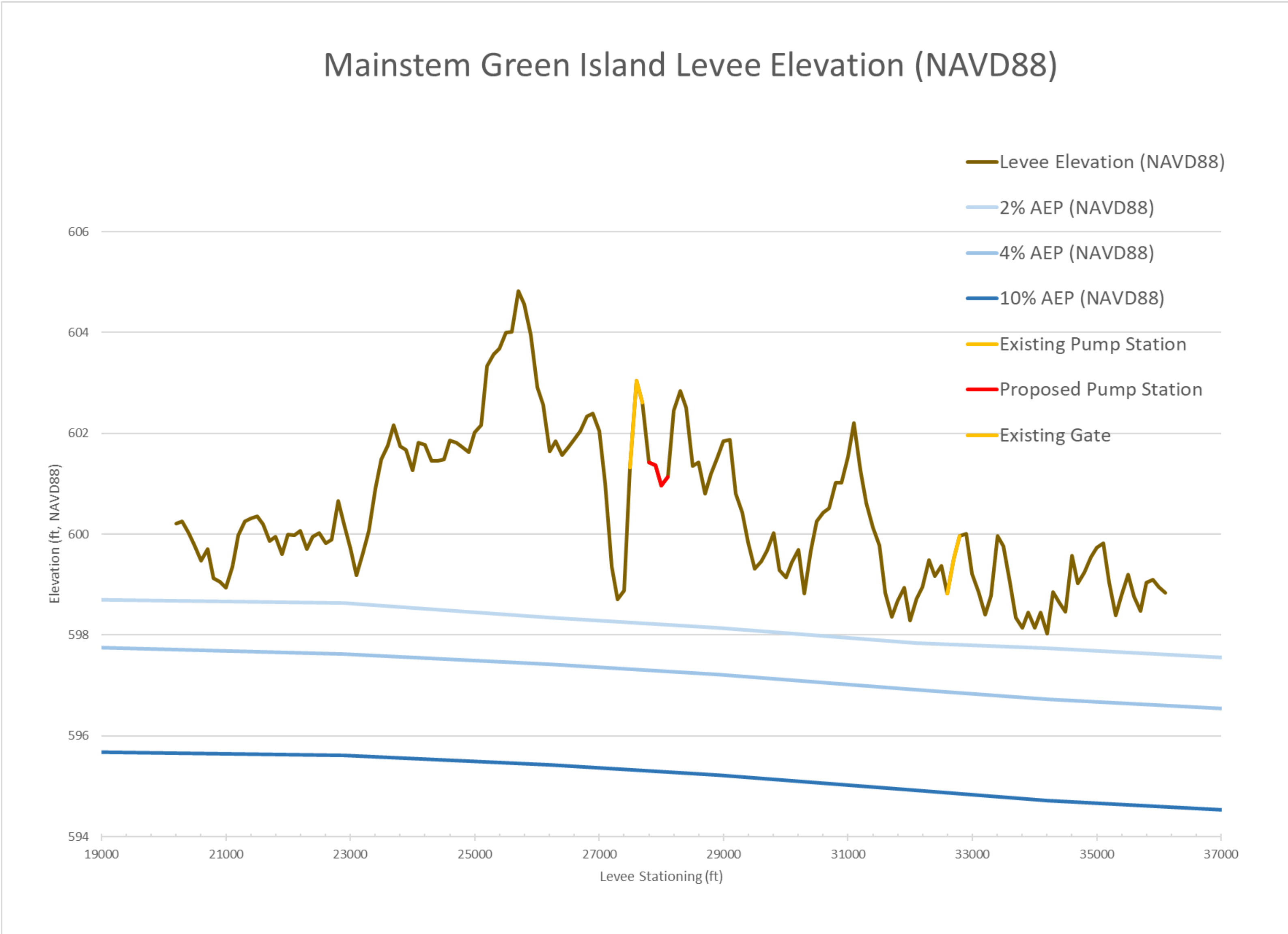


Figure A-9. Elevation and 2004 Flow Frequency Water Surface Profiles along the Mainstem Reach of the Green Island Levee

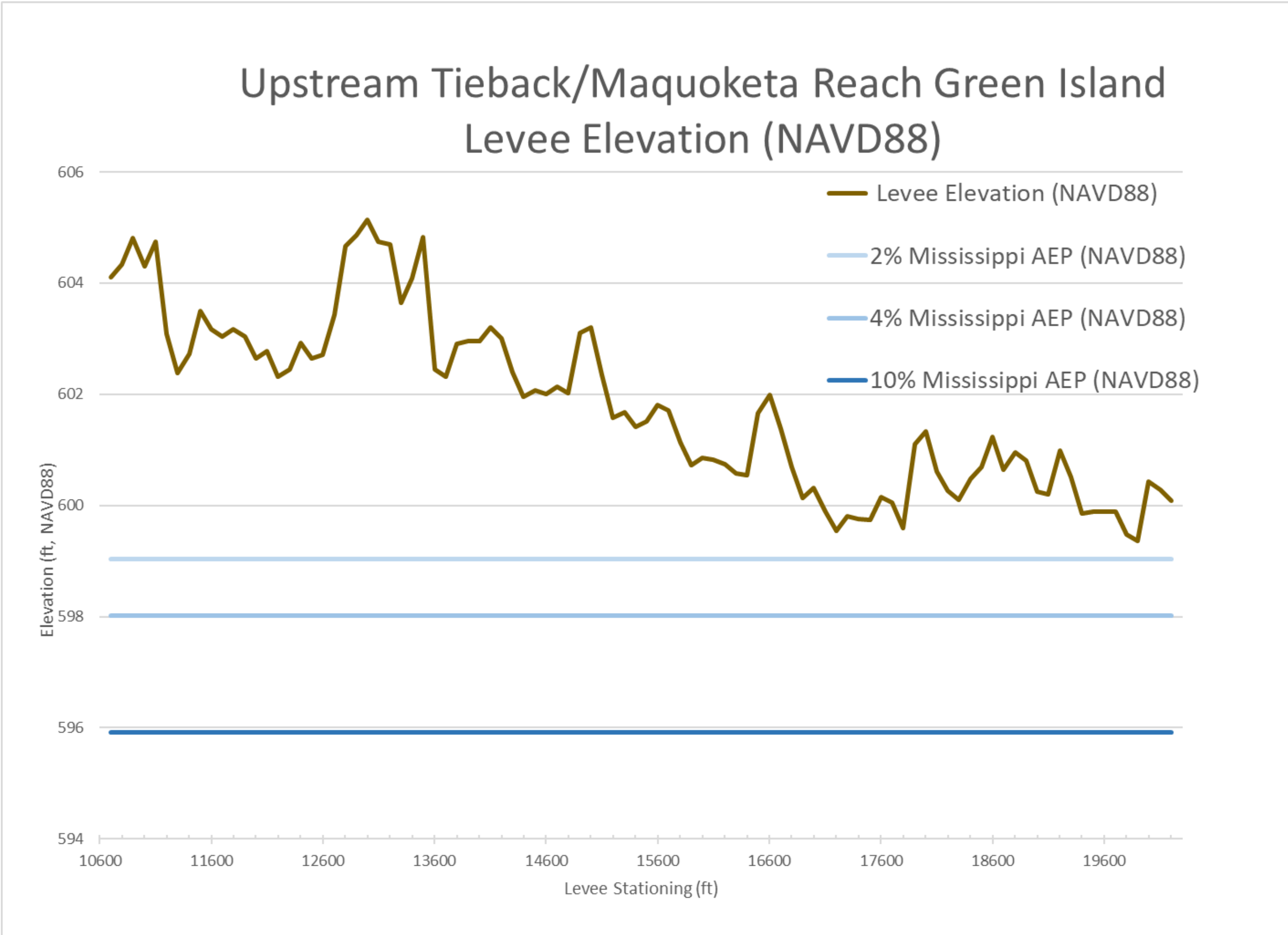


Figure A-10. Elevation and 2004 Flow Frequency Water Surface Profiles along the Upstream Tie-back/Maquoketa Reach of the Green Island Levee. The levee reach from approximately 194+00 to 202+00 serves as the northwest boundary of the Green Island HREP.

3.6.4 Green Island Levee Performance & Flood History

The Green Island Levee has a history of flood fighting, overtopping and breaches located both along the Mississippi and Maquoketa Rivers. As previously discussed, the incipient overtopping location along the Project boundary is at the downstream tie-back reach that borders Brown's Lake and is governed by Mississippi River flooding. Flooding history along the Green Island Levee reaches bordering the Green Island HREP is described herein, while flooding history along the Maquoketa River reach of the Green Island Levee is described above in section 3.5. The incipient overtopping elevation has changed over time due to overtopping and post-flooding levee repairs. A requirement specifying a minimum levee elevation of 596.5 feet MSL1912 be maintained, as detailed in the 2005 GILDD agreement, was provided to the PDT following the 2023 flood event. Thus, overtopping frequency information provided in the following paragraph is based off the levee survey data available at the time of this study. The incipient overtopping elevation (595.46 NAVD88 (596.14 MSL 1912)) at Sta. 376+40 and RM 545.8, based on 2020 survey data, corresponds to an approximate tailwater elevation of 599.34 MSL1912 (19.14 ft stage) at Lock and Dam 12 (RM 556.7), based on 2004 UMRSFFS profiles. Following the 2023 flood, the sponsor indicated that a LD12 tailwater stage of 21.0 feet is the benchmark they use for overtopping. This discrepancy in the corresponding LD12 tailwater elevation suggests that the incipient overtopping elevation may have increased as a result of sponsor levee improvement efforts, resulting from the 2019 flood and completed following the 2020 survey. Table A-7 provides a summary of documented overtopping and breach events at the Green Island Levee since 1993. Overtopping location and event details documented in the 2011 O&M Manual and provided by the sponsor are included in the table below. The 2011 O&M Manual documents additional flood repairs prior to 1993, indicative of past flood damages.

Table A-7. Green Island Levee Flood Events

Date	Water Surface Elevation at L&D12 TW, ft. NAVD88 (MSL1912) [stage]	Approximate Stationing	Flood Impact Description
7/1/1993	601.02 (601.7) [21.5]	Sta. 322+00 to 326+00	400’ breach
		Sta. 333+86 to 343+86	1000' overtopping and 7 scour holes
		Sta. 349+00 to 353+00	400' overtop
		Sta. 367+50 to 417+50	5000' riverside wave wash
		Sta. 345+00 to 420+00	7500' road rock erosion
4/22/2001	602.1 (602.78) [22.58]	Sta. 419+00 to 420+45	Breach
		Sta. 312+00 to 318+00	Breach
		Sta. 306+00 to 416+00	Landside benching
4/28/2008	597.86 (598.54) [18.34]	Sta. 374+80 to 375+20	Breach prior to overtopping resulted from piping due to animal burrow, significant head differential
		Various locations	Overtopping and wave wash, coincident Maquoketa flooding
4/29/2019	600.39 (601.07) [20.87]		Sandbagging prevented overtopping w/

Date	Water Surface Elevation at L&D12 TW, ft. NAVD88 (MSL1912) [stage]	Approximate Stationing	Flood Impact Description
			significant head differential
4/27/2023	600.71 (601.39) [21.19]	various locations between Sta. 375+00 to 400+00 where sandbagging occurred	Initial Overtopping Occurred at LD12 TW 21.0’
4/29/2023	601.3 (601.98) [21.78]	~Sta. 400+00	Overtopping Breach, >7’ differential *Pool A WSE ~588.7 NAVD88, Levee Elevation ~596.5 NAVD88*

*Denotes potential overtopping based on the current Green Island incipient overtopping elevation (595.46 ft NAVD88 (596.14 ft MSL 1912)) and corresponding L&D12 tailwater elevation at 598.66 ftNAVD88 [599.34 ft MSL1912, (19.14 ft stage)].

The current incipient overtopping elevation corresponds to a Mississippi River discharge of approximately 213,860 cubic feet per second (cfs) at RM 545.8, with a resulting annual exceedance probability (AEP) of 6.8% (6.8 percent flood) based on the 2004 UMRSFFS. A 6.8 percent flood has one chance in 14.7 of being exceeded in any given year. A Bulletin 17C analysis was completed using HEC-SSP v.2.3.00.25 to compute updated flow frequencies based on recent annual peak flows through 2021 for comparison to the 2004 UMRSFFS published values which only include an annual peak flow record through 1998. The Bulletin 17C analysis used the same weighted skew method and values as the 2004 flow frequency study (regional skew = -0.3, mean square error = 0.145), thus the resulting differences in frequency can be attributed to the extended flow record through 2021. No historic floods or perception thresholds were used in the updated Bulletin 17C analysis to remain consistent with the parameters used in the 2004 UMRSFFS. Annual peak flows for LD12 were computed based on peak flows from the Mississippi River at Clinton, IA gage (USGS 05420500, 85,500 sq. mile drainage area) in Pool 13 and scaling the values based on the published LD12 drainage area (82,400 sq. miles), located approximately eight miles upstream of Green Island. The results of this analysis suggests an increase in the Green Island levee AEP to 8.5% (an 8.5 percent flood has one chance in 11.8 of being exceeded in any given year) based on the extended record of analysis. Overtopping probabilities over the 50-year Project life based on the 2004 UMRSFFS and the updated Bulletin 17C analysis were computed and summarized in Tables A-8 & A-9 to illustrate the risk of overtopping to the Project.

Table A-8. Green Island Levee 50-yr exceedance probability based on the 2004 UMRSFFS.

Annual Exceedance Probability (AEP) = 6.8%					
Probability of Being Exceeded at Least:					
Once	**Twice**	**Three Times**	**Four Times**	**Five Times**	**Six Times**
97.54%	88.08%	70.26%	48.32%	28.49%	14.46%

Table A-9. Green Island Levee 50-yr exceedance probability reflective of updated Bulletin 17C frequency analysis results incorporating flows through 2021.

Annual Exceedance Probability (AEP) = 8.5%					
Probability of Being Exceeded at Least:					
Once	**Twice**	**Three Times**	**Four Times**	**Five Times**	**Six Times**
98.71%	92.85%	79.79%	60.79%	40.50%	23.53%

Risk of levee failure over the 50-year Project life is largely a function of levee materials and condition, rate of river rise and overtopping head differentials. As planned for evaluation during PED, an armored spillway would likely decrease the elevation along the downstream tie-back reach near the incipient overtopping location and potentially reduce the level of protection to achieve interior filling sufficient to meet a target overtopping head differential at the time of overtopping along the perimeter berm. This type of design would provide controlled overtopping to reduce the risk of overtopping damages and failures. Armored spillway design analysis requires assumptions regarding initial interior elevation, rate of river rise and interior filling computations to assess head differential at the time of perimeter levee overtopping. As discussed previously, the GILDD agreement excluded a spillway feature from the initial planning process. A filling analysis based on the existing perimeter levee was not completed as part of this study and the previous discussion illustrates the general hydraulic superiority that exists, however levee breaches that have occurred during historic and recent flood events demonstrate the need to acknowledge this risk (and associated consequences) in this study. Photographs of the 2023 overtopping and breach illustrate significant head differentials during this event, that according to the sponsor are typical during overtopping events (Figure A-11).

Figure A-11. 2023 Green Island Levee Breach near Brown's Lake around Sta. 400+00.

As-built drawings suggest, the mainstem Mississippi River reach of the Green Island Levee was constructed primarily of sand, and the downstream tie-back reach was constructed primarily of clay (Figure A-11). The Green Island Levee O&M Manual indicates the mainstem reach is a sand levee with a clay core, and the remaining levee reaches were constructed of clay (Figure A-12).

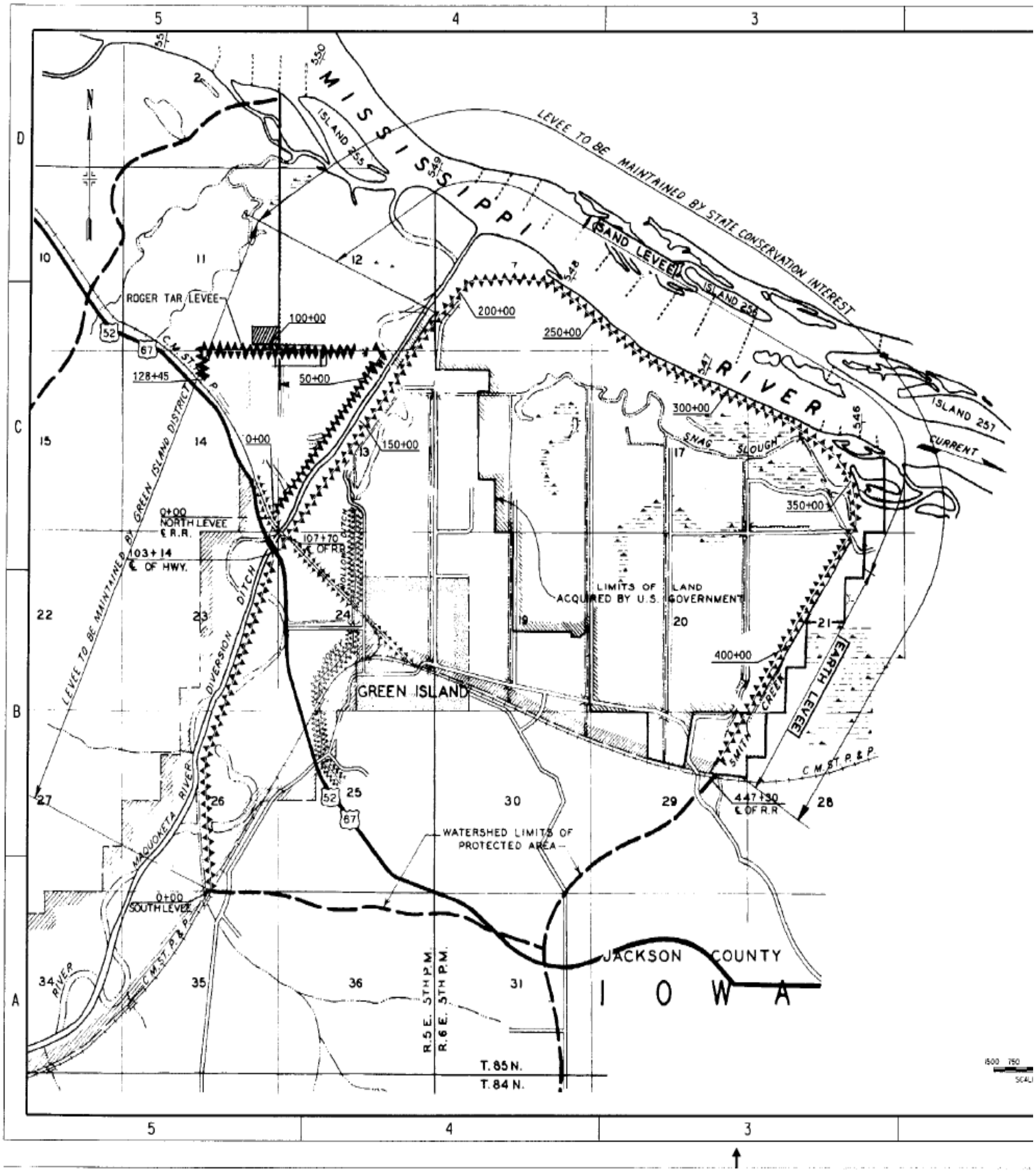


Figure A-11. Green Island As-Built Drawing Illustrating Construction Materials

(1). Levee

Reach	Type Material	River Slope H:V	Land Slope H:V	Height Feet	Top Width Feet	Berm Width Feet
Green Island						
Sta. 0+00 to 200+00	Clay	2:1	3:1	10	10	None
Sta. 200+00 to 350+00	Clay core and sand	4:1	4:1	15	10	None
Sta. 350+00 to 447+30	Clay	2:1	3:1	15	10	None

Figure A-12. Green Island Levee Construction Materials from O&M Manual

However, four borings taken in 2001 (periwinkle) along the tie-back levee reach bordering Brown's Lake boundary and one boring taken on the mainstem levee, upstream of the existing Mississippi River gate structure, show sandy lean clay, lean clay, fat clay, medium fine sand, and clayey medium fine sand (Figure A-13). Repairs from previous floods have also changed the geotechnical makeup of the levee.

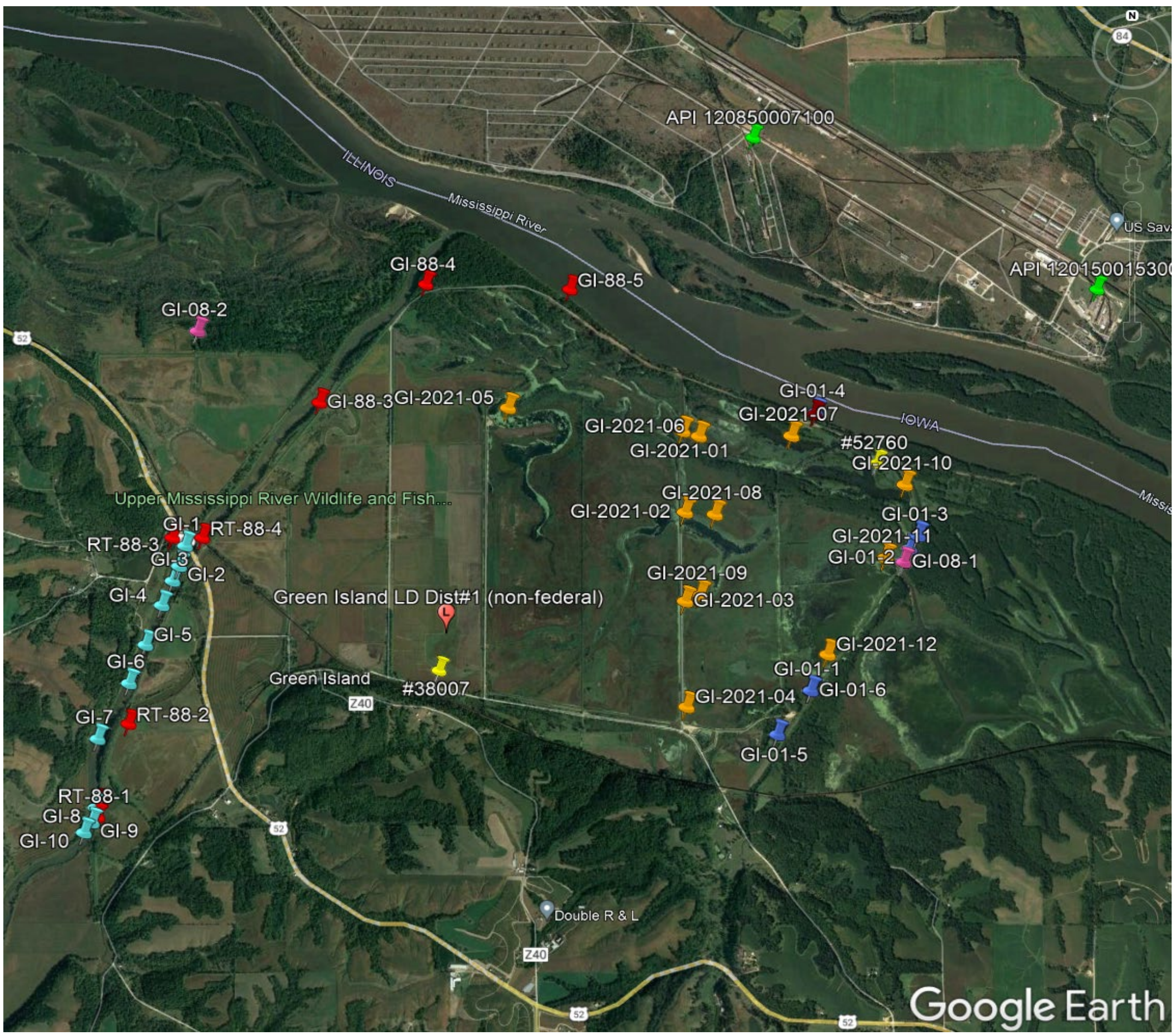


Figure A-13. Green Island Boring locations

Design considerations to reduce overtopping differentials and improve overtopping resiliency, such as those adopted for the Keithsburg Division HREP, will be evaluated during PED, including a potential spillway, levee regrading and a clay cap. Gate operation to provide interior filling prior to overtopping is not reliable or preferred and will not be a focus of the PED evaluation. Recent design for resilient levee overtopping at Keithsburg Division HREP included a 2' clay cap on a sand levee and interior filling to achieve overtopping differentials of 1' or less to ensure a minimum of 1' of clay below the interior water surface elevation at the time of initial overtopping.

4. WINDFETCH

An analysis of wind fetch was conducted to support placement of berms within the Green Island impounded areas to reduce wave-impacted erosion of the existing ditch berms and impounding water level management infrastructure as well as to reduce sediment-resuspension and deposition (Figure A-3). The proposed berm features reduce fetch length, thereby reducing potential for sediment resuspension and subsequent deposition which help to improve water clarity and maintain deep water habitat. Wind fetch reduction in Pool B is of primary concern, as there are greater depths and thus greater potential for wind-wave erosion during flooding conditions.

The wind gaging station at Lock & Dam 12 in Bellevue, IA (BLVI4) was selected for analysis due to its proximity to the Project (7.8 miles upstream), location in the floodplain and the relatively straight orientation of the river valley and direct fetch between the gage location and the study area (Figure A-14). For these reasons, wind conditions at the L&D 12 gage are assumed to be representative of those within the Project.

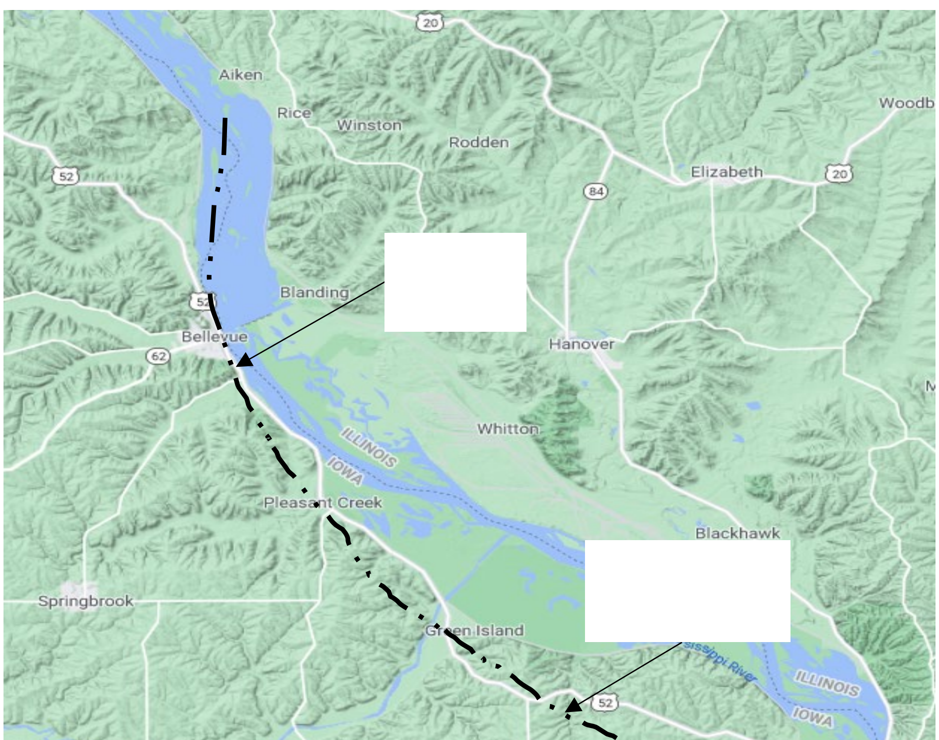


Figure A-14. Location of the BLVI4 wind gaging station relative to the Green Island HREP

Wind rose plots available from Iowa State University's Iowa Environmental Mesonet (IEM) for BLVI4, summarizing wind speed and direction using 10-minute samples of 2-minute averages of 5-second instantaneous recordings for the 2002-2020 period, served as the basis for this analysis.
https://mesonet.agron.iastate.edu/sites/windrose.phtml?station=BLVI4&network=IA_DCP

Wave energy increases with fetch length, wind speed and duration. Maximum managed water levels produce the greatest fetch lengths and occur within Pool A approximately between the months of October and January. Within Pool B the maximum managed water level is maintained year-round under the Typical WLMP. Flooding that often occurs in mid to late spring and mid summer also results in high water and potential overtopping. Therefore, summaries of annual, seasonal and monthly wind rose plots (Figures A-15-A-18) were evaluated to assess wind conditions coincident with maximum managed water levels and the greatest fetch length.

Figure A-15 below shows the annual trends in wind direction and speed for the 2002-2020 period, illustrating dominant wind directions from the southwest and south-southeast, as well as from the north-northwest. On an annual basis, the greatest proportion of higher wind speeds [> 7 miles per hour (mph)] are attributed to winds from the north-northwest and south-southeast directions.

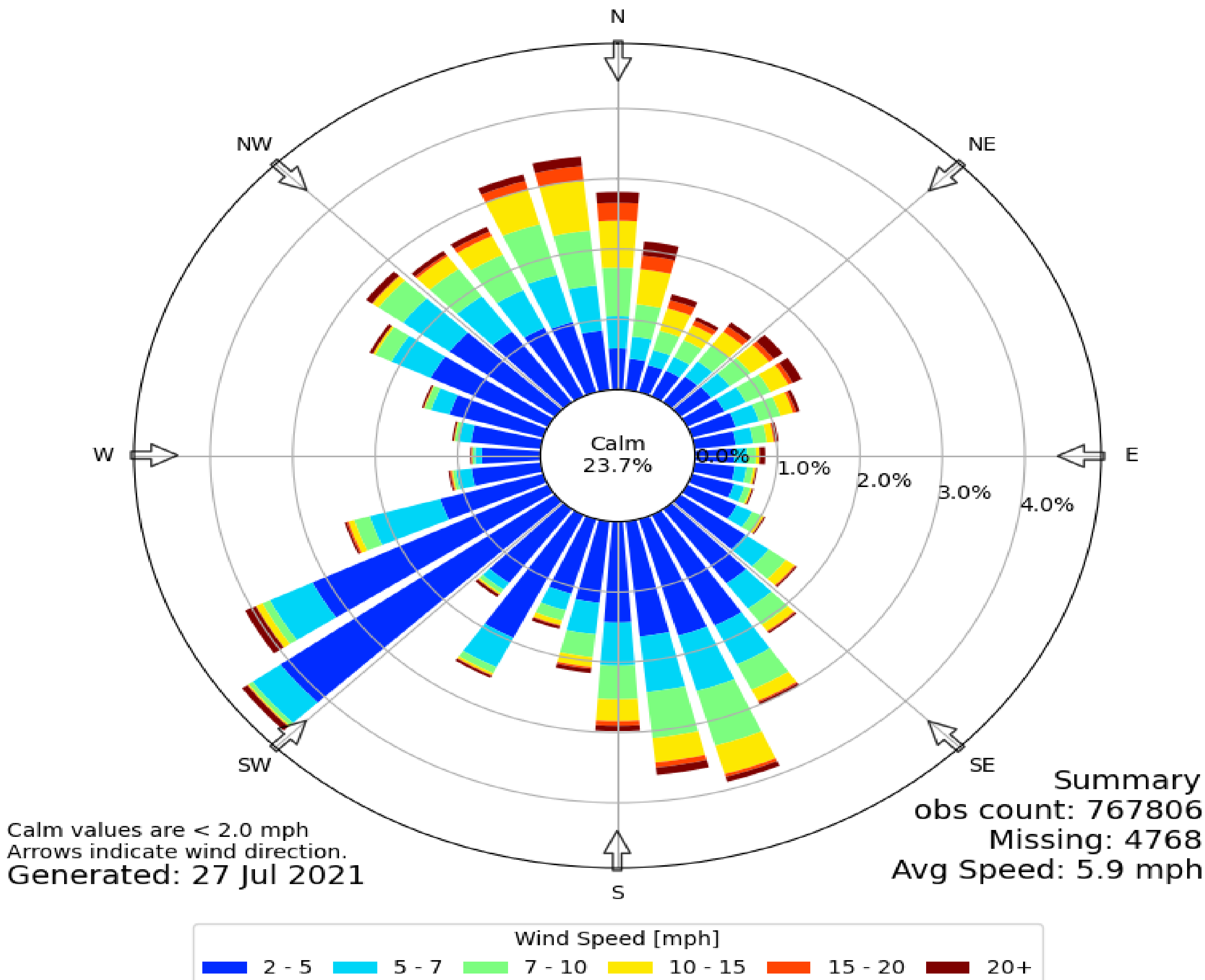


Figure A-15. Wind rose plot showing wind data summarized for all months of 2002-2020 from BLVI4 on Iowa State University's IEM.

Seasonal and monthly wind rose plots (Figures A-16 – A-18) show how the dominant wind direction shifts from predominantly the north-northwest/south-southeast in the late fall, winter, and early spring months (November – March) to having a somewhat more even distribution of wind direction from April - May, before shifting to a prevailing southwesterly wind direction through the summer and early fall (June – October).

The prevailing wind direction for wind speeds greater than 7 mph during the October through January period of maximum managed water levels in Pool A is south-southeasterly and north-northwesterly (Figures A-16 & A-18). The TSP includes strategic placement of dredge material (shown in gold) to create wind-wave mitigation structures in both Pools A & B (Figure A-19 and Sheet C-102). TSP features within Pool A include dredge material placement to elevate the existing 3rd Ditch Berm to an elevation of 589.72, two feet above the maximum-managed water level, reinforcing this north-south road embankment that effectively splits Pool A in half. Pool A also includes a dendritic pattern of ridge and swale features that will effectively reduce fetch length. The north-south orientation of the 3rd Ditch Berm placement is not the most effective orientation to reduce south-southeasterly and north-northwesterly prevailing winds, however the sponsor identified existing wind-wave erosion along this berm and thus it was prioritized. The Fish Lake Berm feature around Fish Lake is not likely to provide significant wind-wave mitigation.

The prevailing wind direction for wind speeds greater than 7 mph is again south-southeasterly and north-northwesterly, based on annual and spring (April and May) wind rose plots, when water levels are held high in Pool B and when flooding has historically occurred, respectively (Figure A-15). TSP features in Pool B include dredge material placement to widen the crown of the 4th Ditch Road Berm by an additional ten feet and maintain a crown elevation of approximately 593'. The 5th Ditch Berm will be elevated to 586.82, with a 50-foot top width. Additional placement features within Densmore Lake and Blake's Lake of Pool B have more of an east-west orientation and include Densmore Lake Upper Berm, Densmore Horseshoe Berm, Densmore Lake Lower Berm, Southeast Berm and Blake's Lake to Brown's Lake Berm. Each of these berm features include a crown elevation of 586.82 with varying crown widths no less than 50 feet. The Blake's Lake Lower Berm feature involves placement along the levee bordering Brown's Lake up to the levee crown to increase the levee template and protect against wind-wave erosion. The alignment of the TSP features described, effectively addresses the prevailing wind direction under the high water conditions of greatest concern in Pool B.

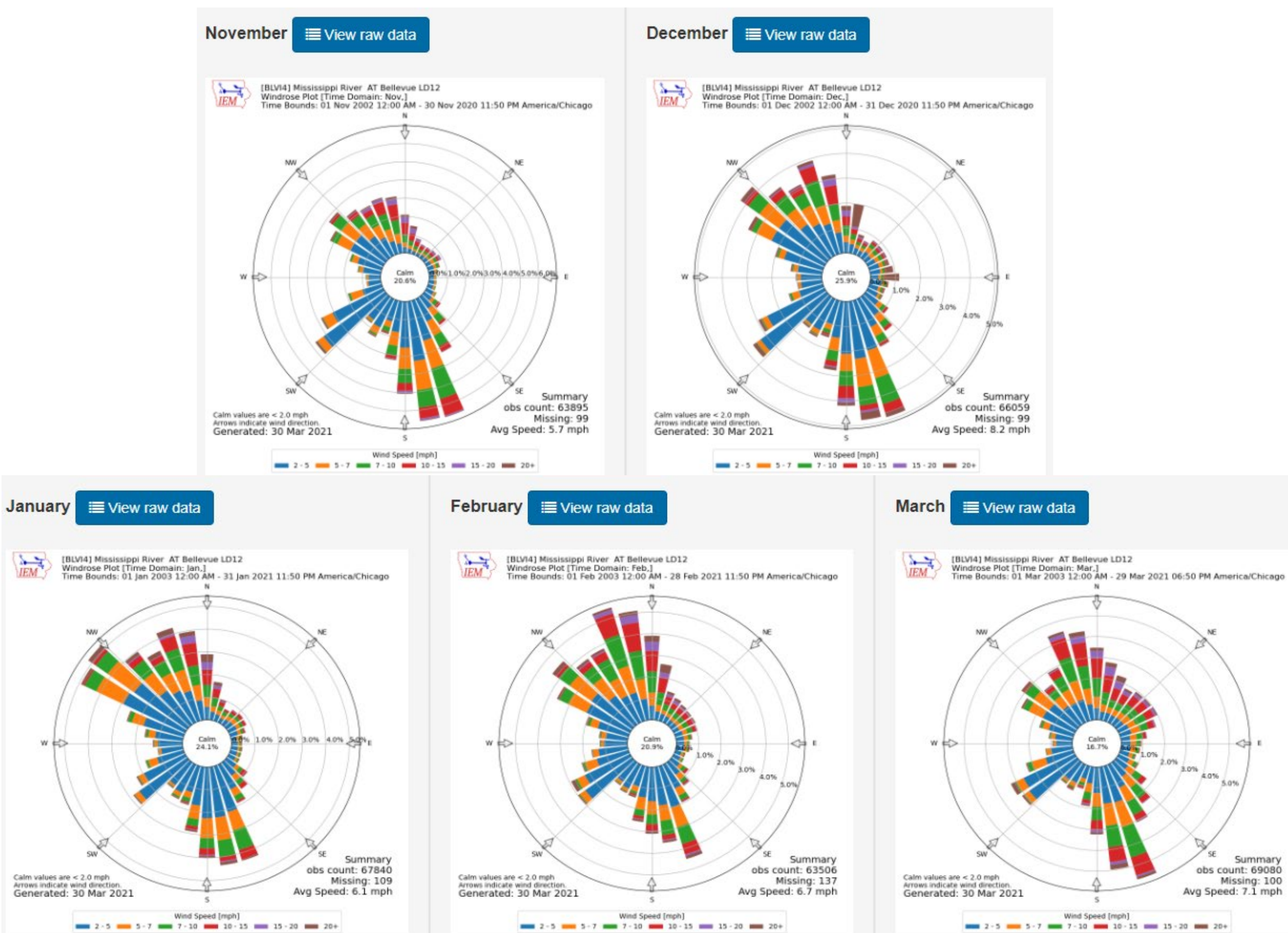


Figure A-16. Wind rose plots depicting wind speed and direction proportions from BLVI4 for late fall, winter, and early spring for years 2002 - 2020.

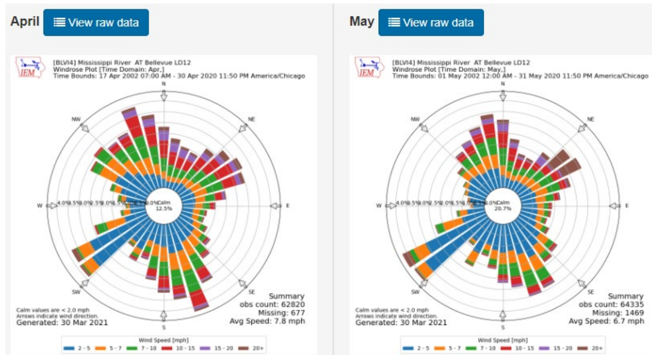


Figure A-17. Wind rose plots depicting wind speed and direction proportions from BLVI4 for the spring of years 2002 - 2020.

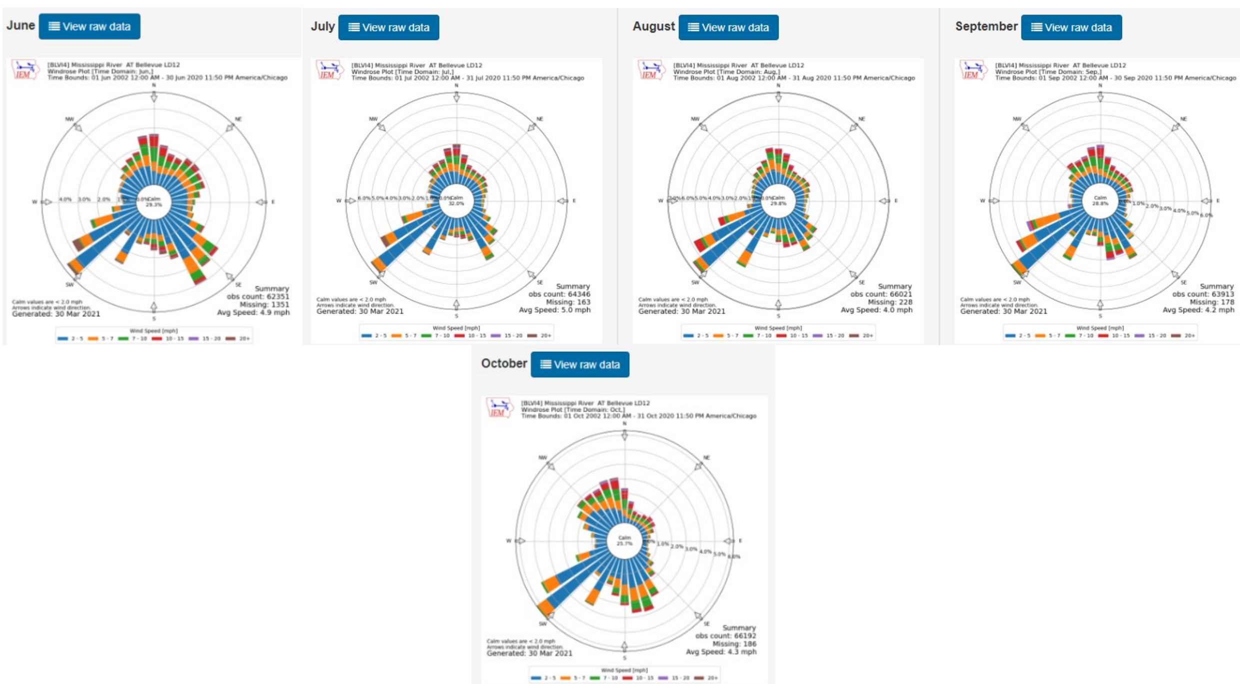


Figure A-18. Wind rose plots depicting wind speed and direction proportions from BLVI4 for the summer and early fall of years 2002 - 2020.

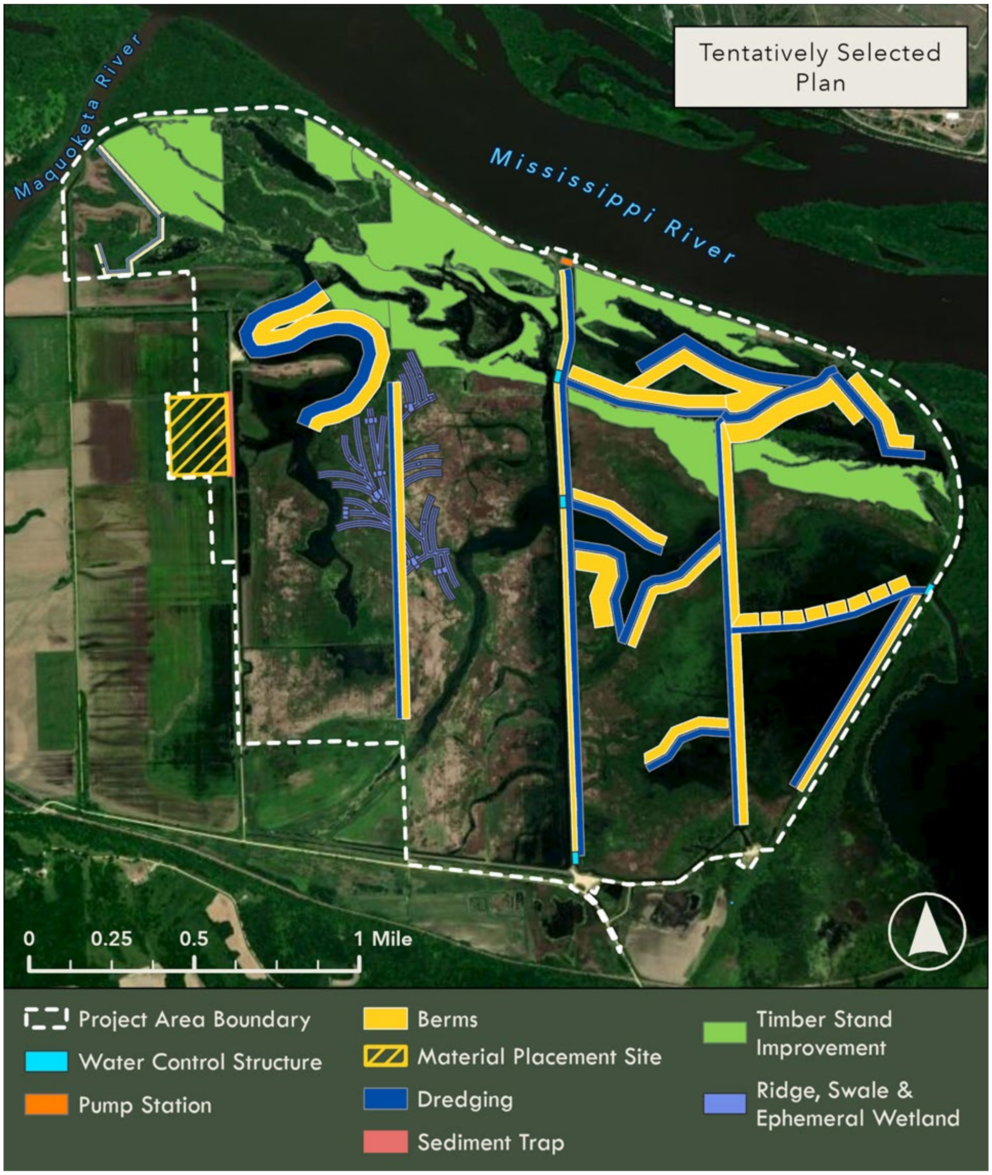


Figure A-19. TSP with dredge material placement berms shown in solid gold.

5. SEDIMENT DEPOSITION

Estimated sediment deposition rates within the Green Island HREP are used to support aquatic habitat benefits computed using the bluegill model under future without Project conditions, and to inform dredge cut design to support benefits over the 50-year Project life. The sediment assessment documented herein describes significant sources of sediment to the Project, provides estimates of contributions from these sources where possible and an estimated sediment deposition rate under both with- and without- Project conditions.

5.1 Primary Sources of Sediment Deposition

Hillslope soil loss from two contributing watersheds (Mooney Hollow and Smith Creek), suspended sediment from the Maquoketa River flowing into Mooney Hollow Ditch, suspended sediment from fall pumping of Mississippi River water into the Project, internal wind-wave erosion and perimeter levee overtopping were the primary sources of sediment deposition identified for Green Island.

5.1.1 Hillslope Soil Loss

The Water Erosion Prediction Project (WEPP) tool was used to estimate the annual average tons of soil erosion available for potential deposition within the Project (https://www.fs.usda.gov/ccrc/tool/watershed-erosion-prediction-project-wepp). The tool uses historical precipitation, watershed topography, soils, land use and management data, and runoff and soil detachment calculations, to estimate sediment delivered to the bottom of slopes, known as hillslope loss for a specified HUC 12 watershed. The WEPP tool was used to analyze hillslope loss for the Beaver Creek-Mississippi River HUC 12 (70600051201), within which the Green Island HREP is located. Table A-10 shows a summary of annual precipitation, rainfall runoff, soil detachment, and hillslope soil loss computed for the Beaver Creek-Mississippi River HUC 12 watershed for the 2007-2020.The resulting average annual hillslope loss rate of 8.74 tons/acre was multiplied by the acreages of the two contributing watersheds, Mooney Hollow and Smith Creek, to estimate potential contributions to an average annual sediment load to Green Island (Table A-11).

Table A-10. WEPP Analysis Summary for Beaver Creek-Mississippi River HUC 12

Year	Precipitation [inches]	Runoff [inches]	Soil Detachment [tons per acre]	Hillslope Soil Loss [tons per acre]
2007	37.86	3.71	1.84	1.71
2008	46.45	8.30	7.24	6.80
2009	48.40	7.82	9.65	9.48
2010	41.08	7.26	5.67	5.46
2011	38.91	6.26	3.99	3.87
2012	24.25	0.84	0.07	0.07
2013	34.41	7.61	0.99	0.96
2014	36.74	6.75	19.60	19.11
2015	51.43	11.32	25.16	24.50
2016	39.65	5.06	11.69	11.30
2017	36.71	3.89	10.72	10.39
2018	52.02	14.17	18.11	17.68
2019	48.19	11.29	8.58	8.26
2020	37.35	4.06	2.94	2.73
			Average (Tons/Acre):	8.74

Table A-11. Estimated Annual Hillslope Loss Potential Contributions to Green Island

Hillslope Soil Loss Source	Watershed Size (acres)	Potential Sediment Load (Tons/Year)	Potential Hillslope Sediment Depth in Pool B (inches/year) *(ft/50 years)*
Smith Creek	3,104	27,120	0.37 *(1.5)*
Mooney Hollow	1,472	12,861	0.17 *(0.7)*

Sediment loading from Smith Creek and Mooney Hollow hillslope loss enters Pool B and is assumed to deposit only within this pool. A depositional area within Pool B of 17,063,802 square feet was conservatively estimated based on the average managed water level (583.82 NAVD88). If 100% of available hillslope loss from both the Smith Creek and Mooney Hollow watersheds were deposited uniformly within this area of Pool B, this would result in 2.3 feet of deposition over the 50-year Project life. However, the Smith Creek gate is open only 15 days per year on average which reduces this load source significantly. Therefore, it was conservatively estimated that approximately 50% of the total hillslope loss from Smith Creek is deposited within Pool B (~0.8 feet over 50 years). After approaching the Maquoketa River, Mooney Hollow flows through a network of over 3 miles of drainage ditches as it is routed east and south to Pool B. Deposition along this lengthy flow path decreases the amount of hillslope loss sediment from Mooney Hollow available for deposition in Pool B. Hillslope loss estimates and sediment load reduction considerations, as discussed herein, were applied when estimating with- and without-Project sediment deposition rates as described later.

5.1.2 Internal Wind-Wave Erosion

Wind driven wave action during high water resulting in erosion of interior berm features has contributed to internal sediment loading. These materials are then available for resuspension and deposition. TSP berm features that effectively reduce fetch length and associated erosion were factored into the with-Project sediment deposition rate estimates.

5.1.3 Mississippi River Pumping

Annual fall pumping from the Mississippi River is assumed to have suspended sediment characteristics similar to the Maquoketa River, as the confluence is approximately one mile upstream of the pump station intake. Average annual suspended sediment concentration (SSC) data available for USGS gage 05418500 Maquoketa River near Maquoketa, IA, was used with annual pumping records provided by the sponsor to estimate an average annual sediment load contribution from fall pumping, based on years 1995-2004 (USGS 05418500 Maquoketa River near Maquoketa, IA, Table A-12).

The pump rate was calculated based on two pumps that operate at 20,000 gal/min. The pump rate in cubic feet per second is shown below.

$$Pump\ Rate\ (cfs) = \frac{2 * 20{,}000\left(\frac{gal}{min}\right)}{448.83\,\frac{(gal/min)}{(cfs)}} = 89.12\ cfs$$

The sediment load due to pumping was formulated based on annual total hours of pump operation and using the equation below. The average annual load was estimated as 2,594 tons/year. The pump station discharges into Pool A and thus it was assumed that this sediment source was deposited only within this pool. A depositional area within Pool A of 51,303,005 square feet was estimated based on the average managed water level during pumping (586.27 NAVD88). If sediment from pumping is deposited uniformly across this area of Pool A, this would result in 0.05 feet of deposition over the 50-year Project life.

$$Average\ load\ per\ year\ (Tons) = \frac{SSC\ \left(\frac{mg}{L}\right) * Pump\ Rate\ (cfs) * 28.32\left(\frac{L}{ft^3}\right) * Pumping\ Time\ (hrs) * 3600\ \left(\frac{seconds}{hour}\right)}{907{,}184{,}740\ \frac{mg}{Ton}}$$

Table A-12. Estimates of Average Annual Sediment Load to Pool A due to Fall Pumping

Water Year	80154, Suspended Sediment Concentration, (milligrams per liter)	Pumping Hours	Load (Tons)
1995	228.10	881.50	2013.84
1996	239.80	1519.70	3649.93
1997	238.90	1367.20	3271.34
2001	166.80	1147.80	1917.52
2002	235.90	813.40	1921.80
2003	151.60	2034.30	3088.81
2004	171.60	1334.30	2293.23
		Average Annual Load (Tons/Year)	2593.78

5.1.4 Minor Sources Not Included

Maquoketa River Gate

If Maquoketa River levels are high and the Maquoketa gate structure is opened, flow into the Mooney Hollow ditch can occur with the potential for flow into Pool A through the Fish Lake structure. The sponsor indicated that conditions and management goals rarely allow for this; therefore, this source of sediment was assumed insignificant and not included in the sediment load calculation.

Mississippi River Gate

As mentioned previously, Mississippi River levels are too low to provide reliable gravity filling necessary to meet the Pool A fall rise (Figure A2, Table A4). Thus, pumping is the primary filling source and Mississippi River gate flow was therefore excluded from the Green Island sediment load calculations.

5.1.5 Green Island Levee Overtopping

As previously described, there is a greater than 40% probability that the Green Island Levee will be overtopped five or more times during the 50-year Project life based on the annual peak flow record through 2021. Based on the TSP features, the risk of levee overtopping is the same for both with- and without- Project sedimentation. Sediment loading to Green Island due to levee overtopping was not explicitly estimated. Instead sediment deposition contributions from levee overtopping were qualitatively accounted for in the overall with and without-Project sediment deposition rate.

5.1.6 Green Island Levee Failure

Significant overtopping head differentials at Green Island pose a risk of levee failure that result in sediment deposition. The risk of levee failure under with- and without-Project conditions does not change given the TSP. Sediment contributions from levee failure were not explicitly estimated but were qualitatively accounted for in the overall with and without-Project sediment deposition estimates.

5.2 With- and Without-Project Sediment Deposition Rates

A sediment deposition rate for the Project was estimated to support dredge cut design and habitat benefit calculations under with and without-Project conditions for the 50-year life of the Project. Smith Creek and Mooney Hollow hillslope loss and fall pumping sediment sources were explicitly estimated, while contributions from levee overtopping and failure, as well as internal wind-waver erosion were qualitatively accounted for in the overall sediment deposition estimate. Consideration was given to the sediment deposition processes that result in increased deposition in deeper areas, Project features including Mooney Hollow sediment trap and berm features that provide reduced wind fetch, and published backwater sedimentation rates. The resulting without-Project sediment deposition rate for the Green Island Project Area was estimated as three feet over 50-years. The resulting with-Project sediment deposition rate for the Green Island Project Area was estimated as two feet over 50years.

6. WITH-PROJECT WATER LEVEL MANAGEMENT INFRASTRUCTURE AND INUNDATION ANALYSIS

The TSP includes a two-way pump station, a sluice gate structure at the downstream tie-back reach bordering Brown's Lake, conveyance, and aquatic habitat dredging and dredge material placement berms to facilitate implementation of the with-Project Drawdown WLMP described in Section 3.6.2 (Figure A-19). Tree, shrub, understory planting, various timber stand improvement measures, creation of ridge, swale features for ephemeral wetland habitat, and a sediment trap along Mooney Hollow are also features included in the TSP. Analysis to support the design of several of these TSP features is described earlier in the report: analysis of wind fetch in support of berm alignment for fetch reduction is described in Section 4; and analysis of sediment deposition in support of aquatic habitat benefit modeling and dredge cut design is described in Section 5. A discussion of the operational flexibility provided by the Brown's Lake gate; the associated structural superiority required as part of the gate design, and plans for design of the gate during PED are described in Section 6.1. An evaluation of pumping capacity for the proposed two-way pump station and inundation duration analysis conducted in support of floodplain forest benefit modeling are described in Sections 6.2 and 6.3, respectively.

6.1 Brown's Lake Gate

Inconducive river levels to facilitate gravity drainage, as discussed in Section 3.2 and illustrated in Figure A-19, demonstrate limited opportunity to meet the desired with-Project drawdown WLMP with the existing one-way ingress pump station and Mississippi River gate structure. To provide the water level management capability necessary to meet the Project's periodic increased growing season drawdowns and also meet the fall rise for migratory waterfowl, bi-directional pumping and an additional gate structure were identified as preferred water level management features considered during feasibility. Seasonal elevation duration analysis near the Green Island Project, shown in Figure A-19, demonstrated the limited opportunity for both gravity filling and draining. The sponsor pointed out the increased gravity drainage opportunities, due to the increased head differential between the mainstem Mississippi near the existing gate structure and the Brown's Lake backwater channel and considers the Brown's Lake gate structure a priority feature of the Project. Thus, the Brown's Lake gate is included in the TSP to provide increased capacity for gravity flow to be operated opportunistically in support of WLMPs, while the bi-directional pump station will serve as the primary means for reliably meeting the Project's drawdown WLMP.

The Brown's Lake gate structure is located near Sta. 372+50 to provide connection with Brown's Lake and increased gravity drainage and filling capacity. This structure can take advantage of the greater head drop into the backwater compared to the main channel and provide flexibility in meeting the with-Project drawdown WLMP. Flexibility in meeting water level management objectives is especially important under changing and uncertain future hydrology. Design of this structure will take place during PED and may include consideration of filling prior to levee overtopping, though operation of this structure for controlled overtopping purposes is not desirable.

The Brown's Lake gate structure will also include 2 ft of structural superiority for 100 ft either side of the structure to meet the requirements of DIVR 1110-1-16, Resiliency and Structural Superiority Requirements for Hydraulic Structures Within or Adjacent to Levees and Floodwalls. The gate location is represented as an ineffective flow area in the effective floodplain model;

thus, modeling of this feature for no rise was unwarranted.

6.2 Pumping Evaluation

A bi-directional pump station was identified as the primary means for reliably meeting the Project's drawdown WLMP and is the focus of the analysis described in the following narrative. An existing ingress diesel pump station with two- 20,000 gpm pumps is operated at full capacity during September and October to achieve the Pool A fall rise in support of waterfowl migration under the existing typical WLMP (Figures A-4). Pumping records from 1991-2021 provided by the sponsor indicate that on average the two pumps are operated concurrently for a duration of approximately 26 days each year to facilitate the fall rise. This existing pump station capacity has been adequate in meeting the filling needs under the current typical WLMP. Under the Project's drawdown WLMP, filling begins. The sponsor's preference is to replace the existing ingress diesel pump station with an electric bi-directional pump station with the same 40,000 gpm capacity. The assessment conducted to evaluate whether the existing pump capacity would adequately meet the desired drawdown WLMP is described herein.

Sources of water flux for the Green Island HREP include precipitation, evapotranspiration, shallow groundwater, levee seepage, surface water contributions from Mooney Hollow, Smith Creek, Maquoketa River and Mississippi River as well as pumping from the Mississippi River. Development of a water budget to better quantify these sources and evaluate pump sizing for a new bi-directional pump was initially considered. Given the limited geotechnical, geological and groundwater data available to support such a quantitative exercise, and the availability of empirical data from the existing ingress pump station that adequately meets the fall rise portion of the existing, typical WLMP, development of a water budget was not undertaken. Instead, a "bathtub" or elevation-storage rate analysis, neglecting all sources of inflow and outflow other than those from pumping, was conducted for the Green Island managed pools, to estimate the pump capacity necessary to achieve the volumetric rate of change specified by the Project's drawdown WLMP, assuming a closed system. Although this approach neglects important drivers in the Green Island water budget, there is value in terms of comparing the desired 40,000 gpm pumping capacity with the pumping capacity needs computed based on the Project's typical and drawdown WLMPs. This comparison provides a relative sense for the additional capacity available to account for these combined, unquantified sources of flux. Additionally, the 26-day average fall drawdown pumping duration provided by the sponsor, under the existing water level management, can be compared with the volumetric rate of change and duration specified by the existing or typical WLMP to assess the relative magnitude of cumulative losses that were not explicitly accounted for.

Storage elevation-volume curves were developed for both Pool A and Pool B, based on the Project terrain (Figures A-20 & A-21). The Project terrain was developed based on the following sources, listed in order of priority: 2007 State of Iowa LiDAR; topographic and bathymetric survey data collected in 2020 by the Rock Island District survey branch; and 2021 field verification measurements used to apply elevation adjustments to areas where the supporting LiDAR data had errors. Inundation plots based on the initial terrain revealed dry areas that should have been inundated, resulting from erroneously higher elevations in these areas indicating the need for this additional field verification effort.

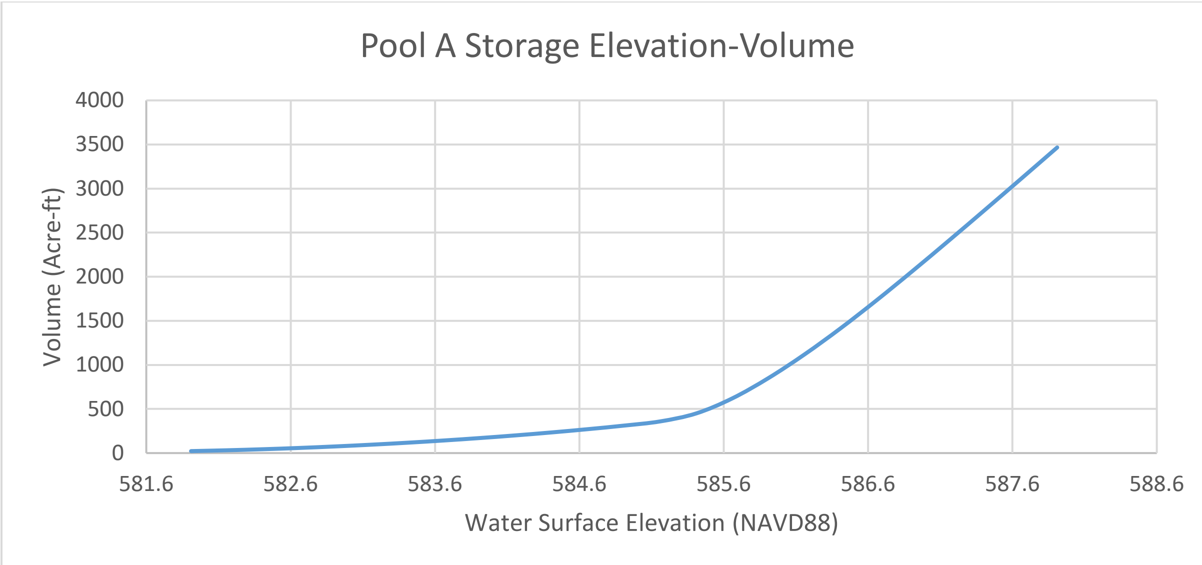


Figure A-20. Pool A Storage Elevation-Volume Curve

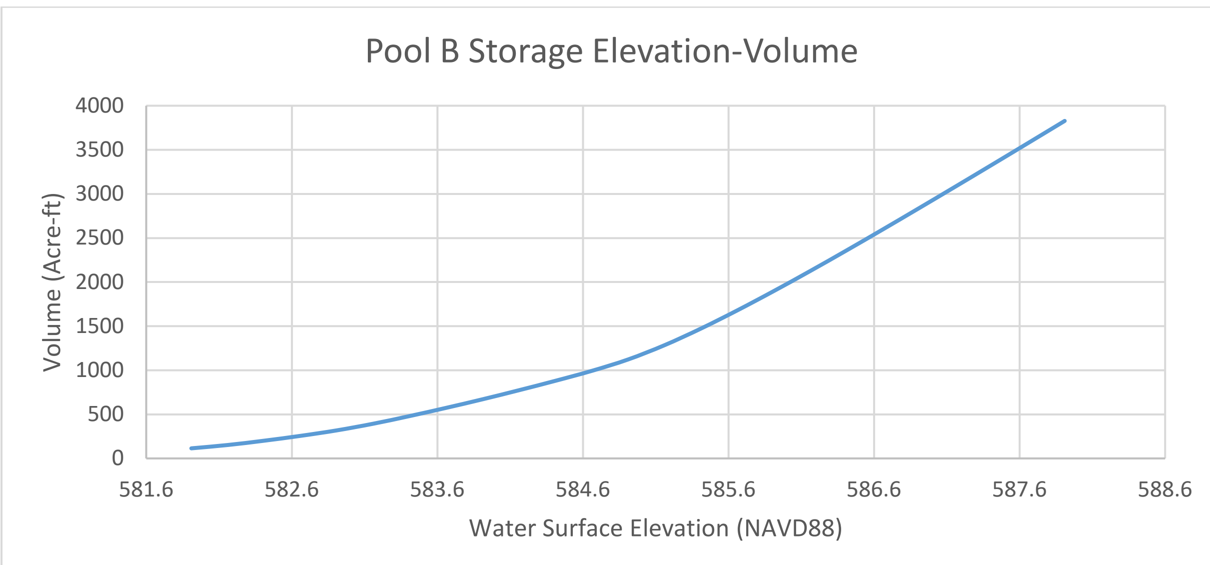


Figure A-21. Pool B Storage Elevation-Volume Curve

A goal of water level management is to make gradual and progressive changes to allow for the recruitment of an abundance and diversity of emergent vegetation species. Thus, pump capacity necessary to meet the desired drawdown WLMPs for Pools A & B was evaluated for 0.5'- to 1.0'- incremental changes in water surface elevation as specified by typical and drawdown WLMPs developed with the sponsor, assuming a linear rate of change in water level (Figures A-5 & A-7). Volumetric changes specified by discrete changes in water surface elevation were determined from the storage elevation-volume curve and divided by the duration determined from the WLMP to come up with a volumetric rate of change or pumping capacity. Water level management actions for the drawdown WLMP in both Pool A & Pool B are summarized by timing, volumetric and water surface elevation change, and pump rate in Table A-13. The resulting maximum pumping need (Pool A + Pool B) to meet the desired with-Project drawdown WLMP, based on this "bathtub" analysis, is less than 4,000 gpm. The "bathtub" analysis resulted in a maximum filling need (Pool A + Pool B) of approximately 21,000 gpm to meet the fall raise following the with-Project drawdown and less than 17,000 gpm to meet the existing or typical WLMP fall raise in Pool A. These results illustrate that the proposed 40,000 gpm capacity should adequately handle the drawdown and filling even with unaccounted for influxes and losses including precipitation, evapotranspiration, levee seepage and groundwater. These results also illustrate the variability in pump capacity necessary to achieve both the steady drawdown desired and the fall rise, highlighting the need for a pump configuration that provides flexibility in terms of variable pump rates. Additionally, the bathtub analysis does not account for rainfall events that could impact drawdown management actions and require temporary increases to pumping capacity to maintain emergent vegetation growth and avoid avian botulism. The appropriate pump configuration to meet these capacity and flexibility requirements will be explored during design.

Table A-13. Green Island Drawdown WLMP "Bathtub" Pumping Estimates with drawdown actions shown in green and filling actions shown in blue.

Pool Name	Change Period (Days)	Start WSEL (NAVD88)	End WSEL (NAVD88)	Volumetric Change (acre-ft)	Pool Pumping Needs (gpm)	Total Pumping Needs (gpm)	Notes
Pool A	15-Dec to 31-Dec (17)	587.72	587.26	647	8606	8606	Currently achieved with gravity
Pool A	1-Jan to 31-Jan (31)	587.26	586.43	1109	8094	8094	Currently achieved with gravity
Pool A	1-Feb to 28-Feb (28)	586.43	585.65	827	6686	6686	Currently achieved with gravity
Pool A	1-Mar to 31-Mar (31)	585.65	584.82	317	2312	2312	Currently achieved with gravity
Pool A	1-Apr to 30-Apr (30)	584.82	583.82	137	1032	1032	With-Project pumps*
Pool A	1-May to 31-May (31)	583.82	582.82	91	**666**	**3877**	With-Project pumps*
Pool B	1-May to 31-May (31)	584.82	583.82	440	**3212**		With-Project pumps*
Pool B	1-Jun to 30-Jun (30)	583.82	582.82	340	2561	2561	With-Project pumps*
Pool A	1-Jul to 31-Jul (31)	582.82	583.82	91	666	666	With-Project pumps*
Pool A	1-Aug to 31-Aug (31)	583.82	584.82	137	998	2931	With-Project pumps*
Pool B	15-Aug to 31-Aug (17)	582.82	583.32	145	1933		With-Project pumps*
Pool B	1-Sep to 30-Sep (30)	583.32	584.82	634	**4785**	**20882**	With-Project pumps*
Pool A	1-Sep to 14-Sep (14)	584.82	585.72	379	6122		Currently achieved with pumps
Pool A	15-Sep to 30-Sep (16)	585.72	586.72	1138	**16097**		Currently achieved with pumps
Pool A	1-Oct to 31-Oct (31)	586.72	587.72	1382	10091	10091	Currently achieved with pumps

*With-project pump operation is indicated only during Drawdown WLMP actions for the purposes of this feasibility-level pump capacity analysis.

The proposed pump station will include 2 ft of structural superiority for 100 ft on either side of the structure to meet the requirements of DIVR 1110-1-16, *Resiliency and Structural Superiority Requirements for Hydraulic Structures Within or Adjacent to Levees and Floodwalls*. This structural superiority levee raise was modeled to ensure no rise to the floodplain.

Preliminary pumping estimates for the five-year management cycle (four years under the typical plan and one year under the drawdown plan) were provided to the mechanical and electrical engineer to estimate preliminary operational costs based on the current utility rate structure. The sponsor saw no concerns regarding this preliminary cost estimate and their operating budget. However, there are risks that the current utility rate structure can change over the 50-year Project life even under the same utility company, which therefore poses a risk to the Project should pump operation become too costly for the sponsor. During design, these risks will be further explored and discussed as well as the option for portable diesel pumps, such as those used at the Keithsburg Division HREP. Additionally, pumping flexibility and pump configuration to provide the appropriate capacity for a slow and steady drawdown throughout the entire with-Project drawdown will be explored.

6.3 Floodplain Forest Inundation

Inundation duration, particularly during the growing season, is an important factor in restoring and maintaining a resilient floodplain forest. In support of floodplain forest habitat modeling for existing conditions, daily time series were developed based on available records of observed water surface elevation data collected by the sponsor from July 1991 to December 2021 and from September 1991 to December 2021 for Pools A and B, respectively. Water level observations generally included measurements recorded within 1-3 weeks of L&D 12 record flood events and within one month following the 2001 and 2008 Green Island breach and overtopping events. Frequency of data collection generally varied from weekly to monthly, thus linear interpolation was necessary to generate daily time series for both pools that were then used for development of growing season elevation duration curves for both Pools A & B, representing existing conditions. With-Project daily time-series were generated for Pools A and B based on 30 years of the idealized with-Project five-year water level management sequence (four years of the typical WLMP and one year of the drawdown WLMP) to compute with-Project growing season elevation duration curves for each pool. The with-Project growing season duration analysis did not account for interior water levels resulting from levee overtopping or failure, despite the likelihood of these events occurring over the 50-year Project life as described in section 3.6.4 (Tables A-8 & A-9). Growing season elevation duration curves for both existing and with-Project WLMPs for both Pools A & B were used to populate tables relating land elevation and number of growing season days inundated for both Pool A and Pool B (Table A-13a). These Hydraulics & Hydrology-generated outputs were provided to the PDT forester and Geographic Information System PDT member to compute acreages of suitable TSI habitat under both the existing and without-Project conditions. Under existing conditions there are 36 TSI-suitable acres in Pool A and 37 TSI-suitable acres in Pool B. Under with-Project conditions there are 145 TSI-suitable acres in Pool A and 87 TSI-suitable acres in Pool B. These TSI-suitable acres for the existing and with-Project conditions were input to the floodplain forest habitat model.

7. FLOOD IMPACT ASSESSMENT MODELING

1D hydraulic rise modeling has been completed for the TSP, demonstrating the no-rise criteria is met. The Floodplain Impact Analysis Memo will include documentation of this modeling effort and results.

8. ECB 2018-14 ANALYSIS OF POTENTIAL CLIMATE CHANGE VULNERABILITIES

This assessment is performed to highlight existing and future challenges facing the study area due to climate change and is conducted in accordance with United States Army Corps of Engineers' (USACE) Engineering Construction Bulletin (ECB) 2018-14, *Guidance For Incorporating Climate Change Impacts To Inland Hydrology In Civil Works Studies, Designs, and Projects*, revised 19 August 2022. In accordance with ECB 2018-14, this evaluation identifies potential climate change vulnerabilities for the Green Island Habitat Rehabilitation and Enhancement Project (HREP). The project area is located between Mississippi River miles 545.9 and 548.7 within Pool 13, approximately eight miles downstream of L&D 12 in Bellevue, IA. This assessment highlights existing and future climate change driven risks for the study area. Study background information can be found in Attachment A of Appendix E and the main report, and more general background information on climate change driven risk can be found in ECB 2018-14.

8.1 Study Background

The Green Island Habitat Rehabilitation and Enhancement Project (HREP) includes features to provide water level management capability to allow for management of habitat and associated plant and wildlife resources. Specific objectives include restoring bathymetric and topographic diversity, improving sediment management, restoring aquatic habitat for fish, and restoring emergent, submerged aquatic and floodplain forest vegetation.

Increased Mississippi River elevations have limited the ability of the sponsor to successfully conduct drawdowns during the growing season for vegetation recruitment with their existing infrastructure. Additionally, increasingly frequent, and longer duration floods present an increased risk of overtopping or breaching of the Green Island Levee which provides separation from the Mississippi River enabling independent water level management. Long duration flooding during the growing season prohibits water level management and adversely impacts the floodplain forest. Limited drawdown capability, sustained increased water levels and increased fetch lengths have resulted in sediment resuspension, decreased depth diversity and erosion to existing topographic features.

The proposed Project seeks to provide increased water level management capability to restore habitat and thus the ecosystem restoration business line is the focus of this analysis. Project features include dredging, berm construction, a bi-directional pump station, water control structures, and timber stand improvement measures. Future climate conditions may impact the establishment and design of Project features. As indicated by the U.S Geological Survey (USGS) in their 2022 report, *Ecological Status and Trends of the Upper Mississippi and Illinois Rivers*, hydrologic indicator variables most relevant to the ecological health of a watershed are defined as annual discharge (maximum, mean, and minimum), duration of high discharges (exceeding the 20% annual exceedance probability (AEP) discharge), and monthly mean discharge. Flood magnitude, frequency, duration and timing are streamflow characteristics relevant to the Project objectives and features, thus observed annual mean, annual peak streamflow and projections of annual maximum of mean-monthly annual mean streamflow, and annual streamflow volume were evaluated to analyze the effects of climate change on the Project objectives. Increased precipitation and rainfall intensity can result in increased flood volumes, flood frequency or flood duration. A lack of precipitation and increased temperatures can negatively impact water temperatures and dissolved oxygen. Projected precipitation variables including annual maximum 1-day, maximum 3-day, and accumulated precipitation, as well as a drought indicator were evaluated. Minimum, mean and maximum projected temperature were also evaluated to analyze the effects of climate change on the Project objectives.

8.2 Literature Review

The *Fourth National Climate Assessment* (NCA4) and the USACE *Civil Works Technical Report CWTS-2015-13,* as well as state and watershed specific resources published by the National Oceanic and Atmospheric Administration (NOAA) National Centers for Environmental Information (NCEI), and the U.S. Geological Survey (USGS) are the basis for this literature review. The focus of these references is on summarizing trends in historic, observed temperature, precipitation, and streamflow records, as well providing an indication of future, climate-changed hydrology based on the outputs from Global Climate Models (GCMs). For this assessment, background on observed and projected temperature and precipitation is provided primarily as context for the impact that they have on observed and projected streamflow and to qualitatively describe the potential impacts to water quality.

The NCA4 considers climate change research at both a national and regional scale (USGCRP, 2018). *Civil Works Technical Report CWTS-2015-13* was published as part of a series of regional summary reports covering peer-reviewed climate literature. The 2015 USACE Technical Reports cover 2-digit, United States Geological Survey (USGS), hydrologic unit code (HUC) watersheds in the United States (U.S). Green Island HREP is located in 2-digit HUC 07, the Upper Mississippi Region (USACE, 2015) and in the NCA4 Midwest climate region.

In many areas, temperature, precipitation, and streamflow have been measured since the late 1800s and provide insight into how the hydrology in the study area has changed over the past century. GCMs are used in combination with different representative concentration pathways (RCPs) reflecting projected radiative forcings up to year 2100 to model future climate. Radiative forcings encompass the change in net radiative flux due to external drivers of climate change, such as, for example changes in carbon dioxide or land use/land cover. Projected temperature and precipitation results can be transformed to regional and local scales (a process called downscaling) for use as inputs in precipitation-runoff models (Graham, Andreasson, and Carlsson, 2007). Uncertainty is inherent to projections of temperature and precipitation due to the GCMs, RCPs, downscaling methods, and many assumptions needed to create projections (USGCRP, 2017). When applied, precipitation-runoff models introduce an additional layer of uncertainty. However, these methods represent the best available science to predict future hydrologic variables (e.g. precipitation, temperature, streamflow). Many researchers use multiple GCMs and RCPs in their studies to understand how various model assumptions impact results (Gleckler et al., 2008).

Temperature. Based on observed temperature records, the annual, average air temperature between 1986 and 2016 for the Midwest has increased by 1.26°F from the 1901-1960 annual average temperature (USGCRP, 2017). Increasing temperatures can accelerate snowmelt and lengthen the frost-free season (Carelton and Hsiang, 2019; Liu, Goodrick, and Stanturf, 2013; Woodward, Perkins, and Brown, 2010). Many studies indicate a change in the seasonality in the region, marked by increasing winter temperatures and early spring melt (Schwartz, Ault, and Betancourt, 2013; Wang et al., 2009; Wolter et al., 2015; Westby, Lee, and Black, 2013). GCM based, projections of temperature for the Midwest show a statistically significant increase in both annual, average temperature and the number of extreme heat days over the next century (Vavrus and Behnke, 2014).

In Iowa, observed temperatures have risen more than 1°F since the beginning of the 20th

century. Warming is driven by increases in nighttime minimum temperatures, as daytime maximum temperatures show no trend. Like much of the Midwest, warming has been concentrated in winter and fall, with no substantial summer warming. This winter warming is reflected in a below average number of very cold nights (minimum temperature of 0°F or lower) since 1990, except for the 2010-2014 period. Under a lower emissions pathway, annual average temperatures are projected to exceed historical record levels by the middle of this century, while under a higher emissions pathway, historically unprecedented warming is projected during this century. Heat waves, accompanied by high humidity are projected to become more intense, while cold waves are projected to be less intense (Frankson et al., 2022).

Precipitation. Average, annual precipitation in the Midwest has increased by 5% to 15% from the first half of the last century (1901–1960) as compared to present day (1986–2015). The amount of rain falling in extreme rain events (1% AEP storm events), has increased by 42% from 1958 to 2016 (USGCRP, 2018). According to the NCA4, GCM based projections indicate that winter and spring precipitation in the Midwest could increase by up to 30% by the end of the century. Precipitation increases of 10-15% are projected in winter and spring for 2-digit HUC 07 from 2070–2099 relative to 1986–2015. However, in the summer and fall, projected precipitation amounts are not expected to change significantly. A northward shift in the rain–snow transition zone in the central and eastern United States is projected by end of the 21st century causing large areas that are currently snow dominated in the cold season to be rainfall dominated (USGCRP, 2017; Ning and Bradley, 2015).

In Iowa, spring precipitation has been above average since 1990, while summer and annual precipitation has been above average since 2005, resulting in increased flooding. The frequency of 2-inch extreme precipitation events has increased, with the greatest number occurring during the past 16 years. Increases in precipitation are projected for Iowa, most likely during the winter and spring. Increases in the frequency and intensity of extreme precipitation are also projected, and with that potentially the increased frequency and intensity of flooding. The intensity of future droughts is expected to increase as rising temperatures will increase evapotranspiration rates and soil moisture loss rates (Frankson et al., 2022).

Streamflow. Observed streamflow trends are strongly influenced by precipitation, temperature, and other factors such as land use and land cover in a region, groundwater dynamics, drainage patterns, channel geomorphology, and regulation. In the Upper Mississippi Region (2-digit HUC 07), multiple studies have identified increasing trends in the observed, annual, average streamflow (Novotny and Stefan, 2007; Mauget, 2004; Small, Islam, and Vogel, 2006) and in the observed, annual, mean/median baseflow (Juckem et al., 2008; Xu et al., 2013). Seasonally, studies have reported increasing annual, minimum, 7-day, low flows in the fall (Small, Islam, and Vogel, 2006) and annual, average, 7-day, low flows in the fall and winter (Novotny and Stefan, 2007). Some studies have found that annual peaks are increasing in the spring and summer (Novotny and Stefan, 2007).

The 2020, USACE *Mississippi River Geomorphology and Potamology* (MRG&P) *Study* also indicates that annual water yield, annual maximum daily water yield, and annual maximum 7-day water yield are increasing throughout the Upper Mississippi River Basin (USACE, 2020). Water yield represents discharge per unit of watershed area. For the 2020 USACE study, water yield was normalized by total annual precipitation to differentiate between the influence of altered precipitation versus other drivers of change in hydrologic response. Their evaluations of precipitation-normalized water yield indicate that changes to water management and land use/cover in the Upper Mississippi River Basin are exacerbating increases in water yield (Simon et al., 2020). There is little to no consensus in the literature regarding changes in projected streamflow in the Upper Mississippi Region.

Ecosystem Health. Based on a 2022 report generated by the USGS, the following variables are critical to ecosystem health and have changed overtime: annual discharge (maximum, mean, and minimum), duration of high discharges (exceeding the 20% AEP discharge), and monthly mean discharge. Results from the 2022 USGS report indicate that mean and minimum annual discharges are increasing at the USGS gages at Winona, Minnesota (05378500) and Keokuk, Iowa (05474500). The duration of high discharges has also increased from 1940 to 2019 for all gages analyzed. Significant increases in annual maximum discharges were detected for the Keokuk, Iowa (05474500) and Valley City, Illinois (05586100) USGS gages. Based on an analysis of monthly, mean discharges, large increases in May mean discharges were identified for all three Mississippi River gages analyzed. There is some evidence that the maximum in monthly, mean discharge for a given year has shifted from occurring in April to either May or June.

Water quality analysis presented in the 2022 USGS report indicates that total suspended sediment (TSS) concentrations associated with mean discharges have decreased long-term in many reaches and tributaries of the Upper Mississippi River. The most significant changes have been observed in L&D pools 4 and 8. Phosphorus loads in all the L&D pools analyzed (pools 4, 8, 13, and 26) on the Upper Mississippi River have also decreased long-term. Although there are no long-term, significant trends in dissolved oxygen (DO) for the portions of the Upper Mississippi River assessed, low DO in backwater areas has been observed more frequently in the summer than in winter.

The concentration of submersed aquatic vegetation (SAV) is considered the primary indicator of aquatic vegetative health in the Upper Mississippi River. High prevalence of SAV (generally >50-percent) indicates quality habitat for waterfowl. Aquatic vegetation analysis identified trends in SAV in L&D pools 4, 8, and 13. The prevalence of SAV in L&D pools 4 and 8 increased by 30% from 2002 to 2010. Since 2010, SAV concentrations at these two locations have plateaued. The prevalence of SAV in L&D 13 ‘s pool increased from 1998 to 2008. Since 2009, SAV concentrations have been decreasing in L&D 13’s pool. Additionally, since 2000, increases in aquatic plant species diversity have been observed in L&D pools 4 and 8. In the L&D 8 and 13’s pools, a positive trend in emergent vegetation has been recorded. Emergent vegetation provides habitat for aquatic species. No trends in aquatic vegetation were found within the lower portion of the Upper Mississippi River (L&D Pool 26).

Summary. Within the literature reviewed, there is evidence that temperature, precipitation, and streamflow have increased over the observed period of record within the Upper Mississippi Watershed. Trends in water quality within the Upper Mississippi Watershed indicate decreases in total phosphorus and total suspended solids. Aquatic vegetation analysis indicates increases in SAV in L&D pools 4, 8, and 13 in early 2000s through 2010. SAV concentrations have plateaued through 2019. Projections of future climate show strong consensus on increases in future temperature, and moderate consensus on increases in future precipitation. There is little to no consensus related to trends in future streamflow. Figure A-22 from the 2015 USACE *Civil Works Technical Report CWTS-2015-13* provides a visual summary of the trends in observed and projected hydrometeorological variables for 2-digit HUC 07, the Upper Mississippi Region.

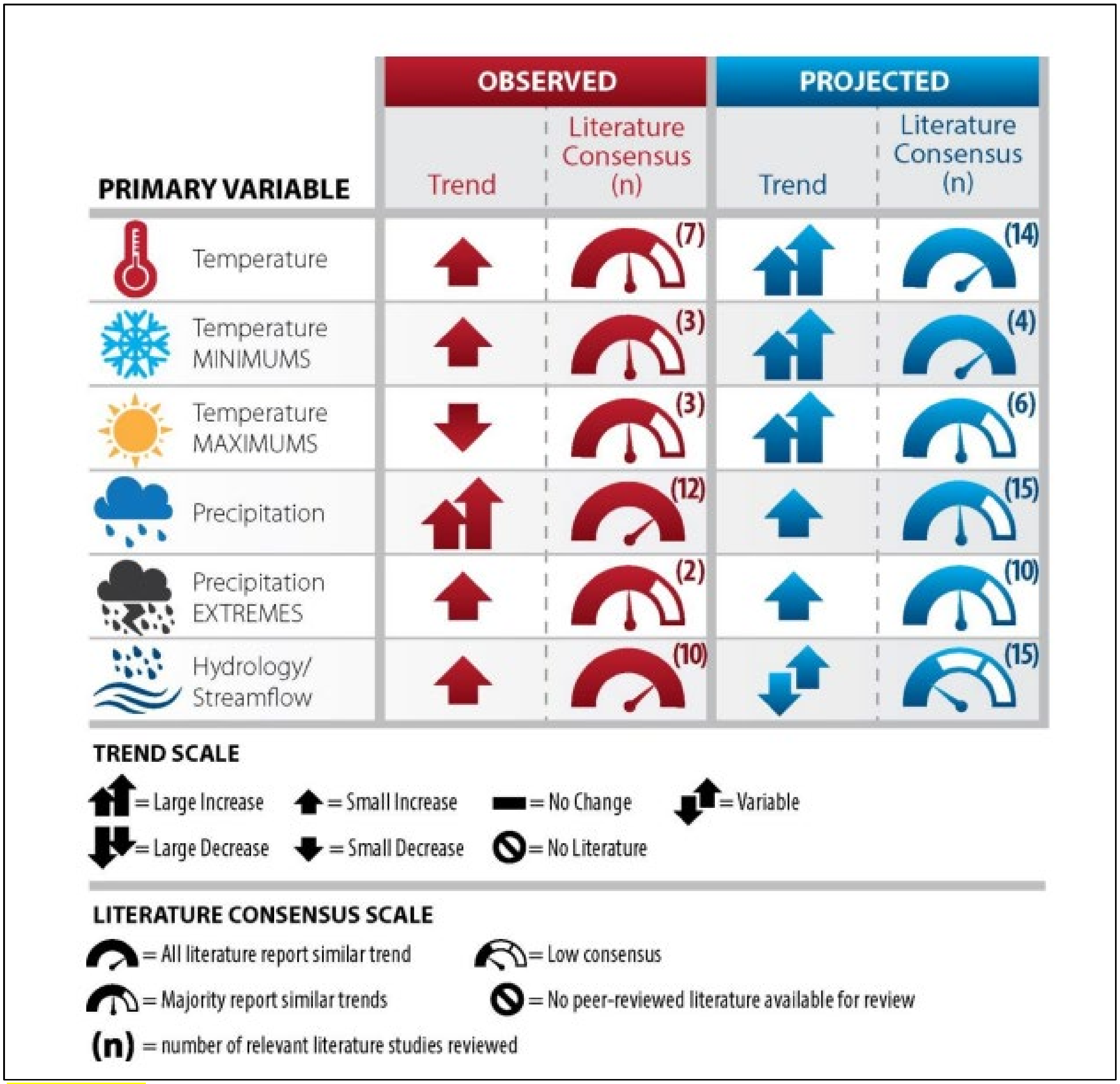


Figure A-22. Summary matrix of UMR (HUC 07) observed and projected climate trends (USACE, 2015)

8.3 Nonstationarity Detection and Trend Analysis

The assumption that hydrologic timeseries are stationary (their statistical characteristics are unchanging) in time underlies many traditional hydrologic analyses. Statistical tests can be used to test this assumption using the techniques outlined in USACE Engineering Technical Letter (ETL) 1100-2-3, *Guidance for Detection of Nonstationarities* (2017). The USACE Time Series Toolbox (TST) tool is a web-based tool that performs the statistical tests described in the guidance. Annual mean streamflow and annual peak streamflow are analyzed for the Green Island HREP because Project objectives related to water level management are vulnerable to flooding, particularly flooding resulting in levee overtopping and failures. Mean annual streamflow is most representative of flows features experience throughout the year (Van Appledorn, 2022). In the long-term, Project feature design needs to include resiliency so that features can perform under future conditions. More frequent overtopping or failure of the Green Island Levee and longer duration flooding will impact water level management capability and associated fisheries, aquatic vegetation, and floodplain forest.

Observed, annual mean daily discharge for L&D 12 was estimated in HEC-DSSVue v3.2.3 (HEC, 2021) from mean daily flow values for the Mississippi River at Clinton, IA gage (USGS 05420500, 85,500 sq. mile drainage area) in Pool 13 and scaling the values based on the L&D 12 drainage area (82,400 sq. miles). Observed, peak annual streamflow for L&D 12 was estimated using the same method based on peak streamflow for the Mississippi River at Clinton, IA gage.

The Mississippi River at USACE L&D 12 drains 82,400 square miles and at Clinton, IA drains 85,500 sq. miles. Both locations are influenced by regulation from the L&Ds on the Mississippi River. The L&Ds were constructed and placed into operation in the 1930's, with both L&D 12 and L&D 13 placed into operation in May 1939. The L&D 12 Water Control Manual states that the general objective of the L&Ds is to maintain the authorized, nine-foot, navigation channel upstream of L&D 12. The L&Ds maintain the minimum storage of water required for navigation at all times and any additional water volume is outflowed. Consequently, operation of the L&Ds does not have a significant impact on annual mean streamflow or peak annual streamflow. During flooding, the L&Ds go out of operation and regulation is no longer in place. The TST tool was applied to detect nonstationarities and trends in mean annual streamflow for the post L&D construction period of record from water years (WY) 1940 to 2022 and in peak annual streamflow for the WY1874 to WY2021 full observed period of record.

As shown in Figure A-23, the annual mean flow record for L&D 12 has strong evidence of a nonstationarity in water year 1990. A strong nonstationarity is one that demonstrates a degree of consensus, robustness and a significant increase or decrease in the sample mean and/or variance. The 1990 nonstationarity demonstrates consensus because it is identified by multiple tests targeted at identifying a change in the overall statistical distribution (light blue bars in Figure A-23). The 1990 nonstationarity can be considered robust because tests targeted at identifying nonstationarities in different statistical properties identify a change in mean (dark blue bar in Figure A-23) as well as distribution (light blue bars in Figure A-23). The magnitude of the mean, annual mean flow increases significantly, from 45,000 cfs between 1940-1979 to 61,000 cfs between 1991-2019. Linear and monotonic trends are evaluated using the t-test, Mann-Kendall and Spearman Rank Order tests. The significance of trends is evaluated using a 0.05 level of significance threshold (p-value<0.05 is considered statistically significant). Trend analysis indicates a statistically significant, positive trend for the 1940-2022 period of record by the t-Test (p-value$= 1.02 \times 10^{-5}$), Mann-Kendall test(p-value$=1.32 \times 10^{-4}$), and Spearman Rank-Order (p-value$=7.37 \times 10^{-5}$) test, see trendline in Figure A-24. Because there is strong evidence of a nonstationarity in water year 1990, the record was subset for analysis of monotonic trends. There is no statistically significant trend in the data recorded between 1940-1990, nor is there a statistically significant trend in the data recorded between 1990-2022.

As shown in Figure A-25, the annual peak streamflow record for L&D 12 has strong evidence of a nonstationarity in water year 1964. The 1964 nonstationarity demonstrates consensus because it is identified by multiple tests targeted at identifying a change in the mean (dark blue bars in Figure A-25). The 1964 nonstationarity can be considered robust because tests targeted at identifying nonstationarities in different statistical properties identify a change in the overall statistical distribution (light blue bars in Figure A-25) as well as the mean (dark blue bars in Figure A-25). The magnitude of the mean, annual peak streamflow increases significantly, from 130,000 cfs between 1874-1962 to 160,000 cfs between 1965-2021. Linear and monotonic trends are evaluated using the t-test, Mann-Kendall and Spearman Rank Order tests. The significance of trends is evaluated using a 0.05 level of significance threshold (p-value<0.05 is considered statistically significant). Linear and monotonic trend analysis does indicate a positive trend for the 1874-2021 period of record. However, each of the tests slightly exceed the 0.05 level of significance threshold: t-Test (p-value= 0.08); Mann-Kendall test (p-value=0.06); and Spearman Rank-Order (p-value=0.05) test; see trendline in Figure A-26. Because there is strong evidence of a nonstationarity in water year 1964, the record was subset for analysis of monotonic trends. Trend analysis indicates a statistically significant decreasing trend for the annual peak streamflow data recorded between 1874-1964 by the t-Test (p-value= 0.04) and Mann-Kendall test(p-value=0.04), however the Spearman Rank-Order (p-value=0.05) hypothesis test narrowly exceeded the significance threshold. No statistically significant trend in annual peak streamflow was identified for the for the data recorded between 1964-2021.

L&D 12 Mean Annual Streamflow

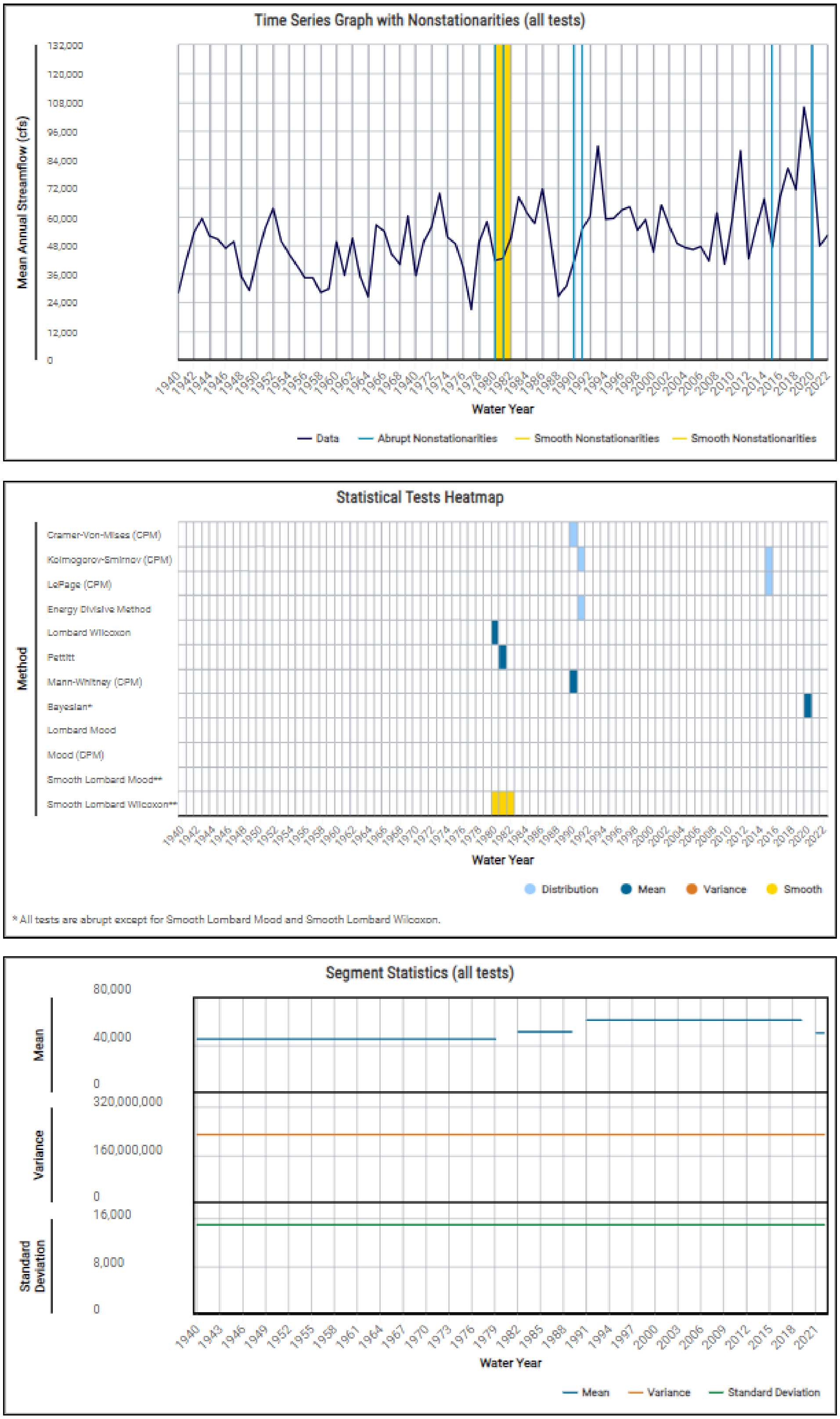


Figure A-23. Time Series Toolbox Output for Mean Annual Streamflow for L&D 12.

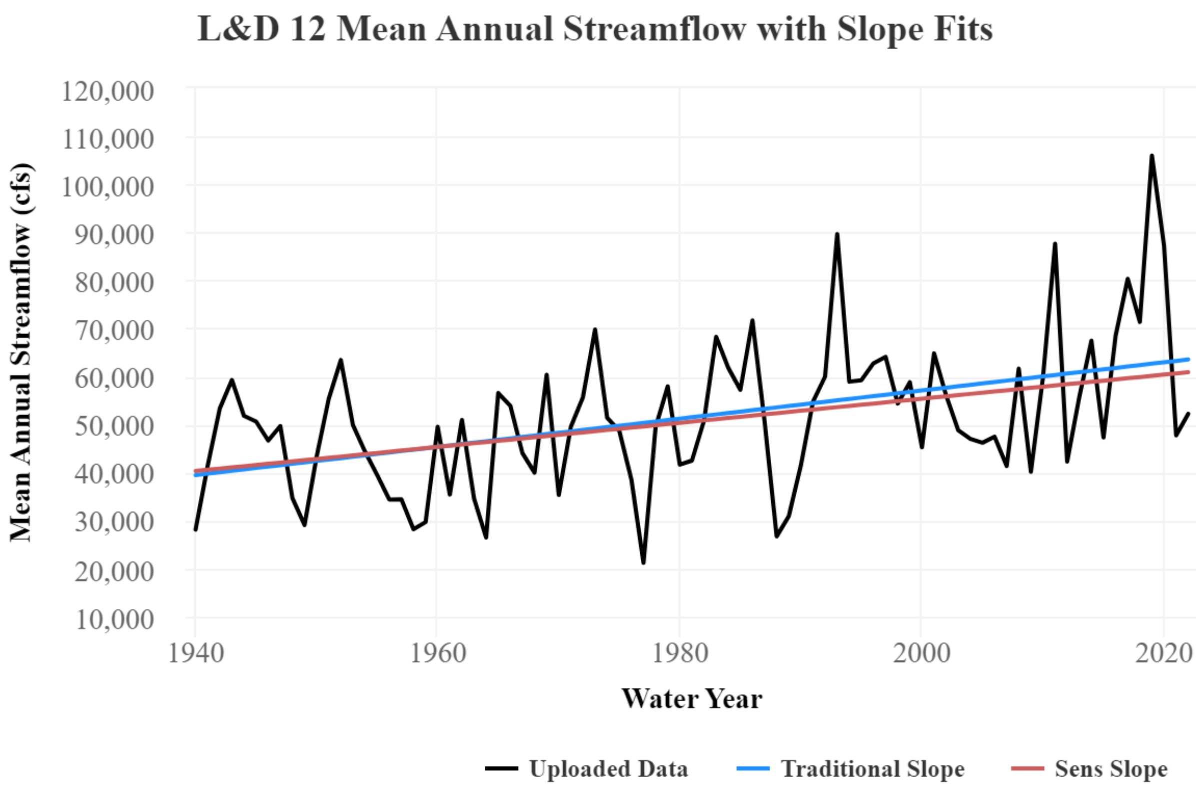


Figure A-24. Trend Analysis for Mean Annual Streamflow for L&D 12.

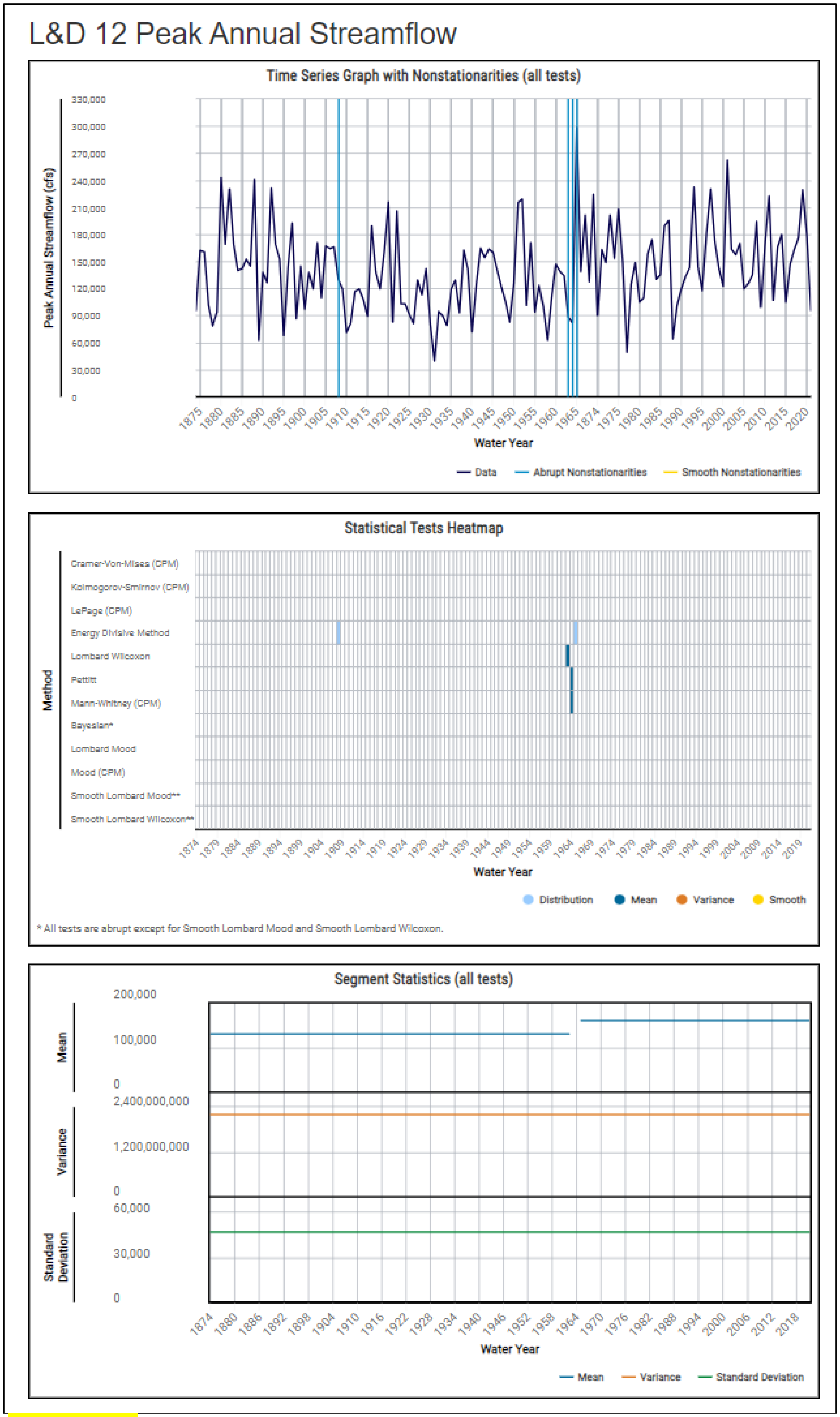


Figure A-25. Time Series Toolbox Output for Peak Annual Streamflow for L&D 12.

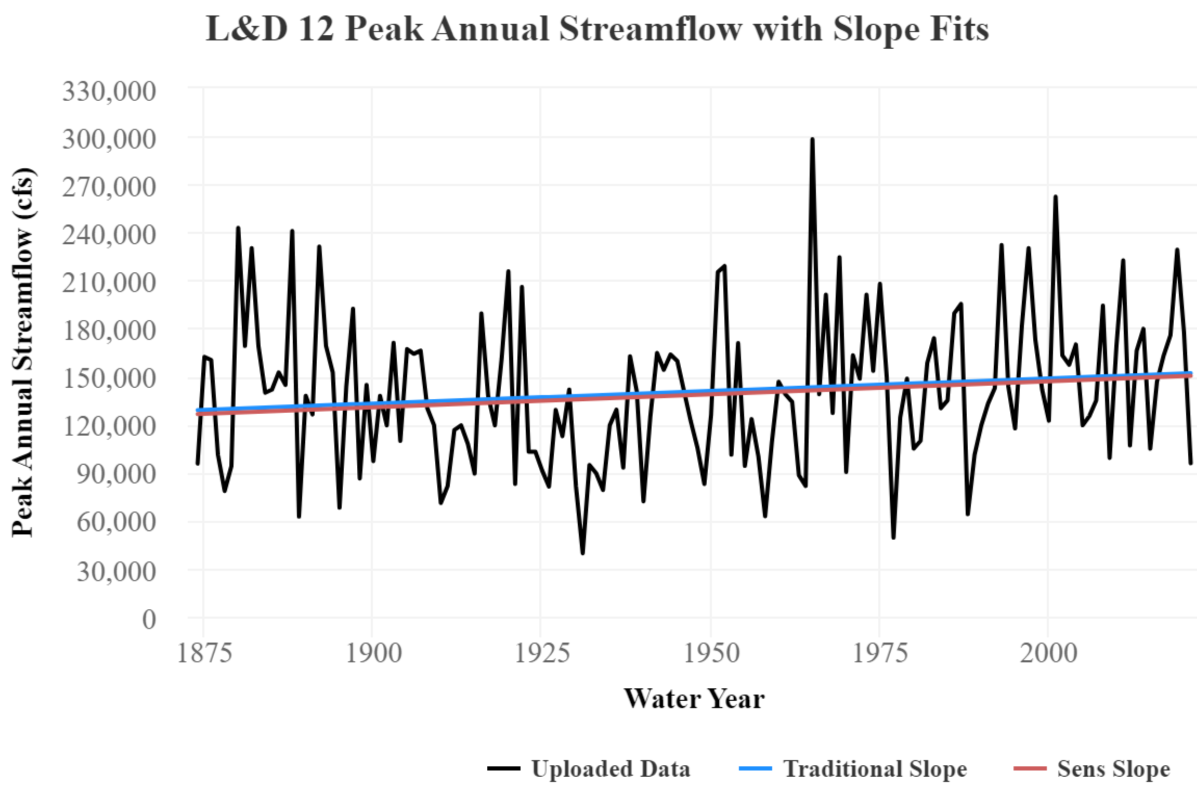


Figure A-26. Trend Analysis for Peak Annual Streamflow for L&D 12.

8.4 Climate Hydrology Assessment Tool (CHAT)

The USACE Climate Hydrology Assessment Tool (CHAT) displays various simulated, historic and future, climate-changed streamflow, temperature, and precipitation outputs derived from 32 GCMs. The CHAT uses Coupled Model Intercomparison Project Phase 5 (CMIP5) GCM meteorological data outputs that have been statistically downscaled using the Localized Constructed Analogs (LOCA) method. GCMs rely on scenarios representing different pathways to a given atmospheric concentration of greenhouse gas emissions (GHG) referred to as representative concentration pathways (RCPs). RCPs describe the change in radiative forcing at the end of this century, as compared with pre-industrial conditions. Projected hydroclimate data in the CHAT for 2006 to 2099 are produced using two future scenarios: RCP 4.5 (where greenhouse gas emissions stabilize by the end of the century) and RCP 8.5 (where greenhouse gas emissions continue to increase throughout the century). Simulated output representing the historic period of 1951 to 2005 is generated using a reconstitution of historic GHG emissions.

To analyze runoff, LOCA-downscaled GCM outputs are used to force an unregulated, Variable Infiltration Capacity (VIC) hydrologic model. Areal runoff from VIC is then routed through a stream network using MizuRoute. Outputs represent the daily in-channel, routed streamflow for each stream segment – valid at the stream segment endpoint. Since the runoff is routed, the streamflow value associated with each stream segment is a representation of the cumulative flow, including all upstream runoff, as well as the local runoff contributions to that specific segment. Within the CHAT, streamflow output can be selected by stream segment and precipitation/temperature output can be selected for a given 8-digit HUC watershed.

Green Island is in 4-digit HUC 0706 (Upper Mississippi Maquoketa-Plum). The 8-digit HUC of interest specific to the study area is the Apple-Plum watershed (HUC07060005). Mississippi River stream segment 07002612 encompasses the Project boundary. To provide insight into if and how potential, future flooding conditions may change, model results for the annual-maximum of mean monthly streamflow, annual-mean streamflow, annual streamflow volume, annual-maximum 3-day precipitation, and annual-accumulated precipitation were evaluated. These results are discussed below and presented in Figures A-27 through A-32. The range of data is indicative of the uncertainty associated with projected, climate-changed streamflow and precipitation. Figure A-27 shows the range of the modeled, annual maximum of mean monthly streamflow output presented for the historic period (1951-2005) and the future period (2006-2099). Figure A-28 illustrates robustness metrics for the RCP 8.5 model output, showing a positive signal in the data for the end-century epoch, however less than 66% of the models show a change greater than the variability threshold, as indicated by the weak robustness signal. Robustness metrics illustrate the level of agreement between the models in terms of the magnitude and direction of change between the historical and future periods. Figure A-29 and Figure A-30 show the range of the modeled, annual-mean streamflow output and modeled, annual streamflow volume output, respectively. No robustness signals were identified for either the future modeled, annual-mean streamflow or streamflow volume in either future period. A lack of robustness indicates less than 80% of the models were in agreement on the sign or direction of change over time and less than 66% of models show change greater than the variability threshold over time. Figure A-31 and Figure A-32 show the range of the modeled, annual-maximum 3-day precipitation output, and modeled, annual-accumulated precipitation output, respectively. Annual-maximum 3-day precipitation model output (under both RCPs) show weak positive robustness signals of change for both the mid-century and end-century epochs. Under RCP 4.5, annual-accumulated precipitation model output shows a weak positive robustness signal for change in both the mid-century and end-century epochs. Under RCP 8.5, model output for annual-accumulated precipitation shows a weak positive robustness signal for change in the mid-century epoch and a robust positive signal for the end-century epoch.

The drought indicator, defined as the annual-maximum of number of consecutive dry days, was analyzed for this assessment to identify how gravity drawdown opportunities may change in the future, and annual-mean temperature was analyzed as a proxy for water temperature. Warmer water holds less dissolved oxygen (DO) which affects the survival of aquatic life (USGS 2018). Figures A-33 and A-34 show the range of the modeled drought indicator output and modeled, annual-mean temperature output, respectively. Model output for the drought indicator shows a weak positive robustness signal during the end-century epoch under RCP 8.5 while annual-mean temperature output under both RCPs shows robust positive signals for both mid-century and late-century epochs, indicating strong model agreement in sign and magnitude of change.

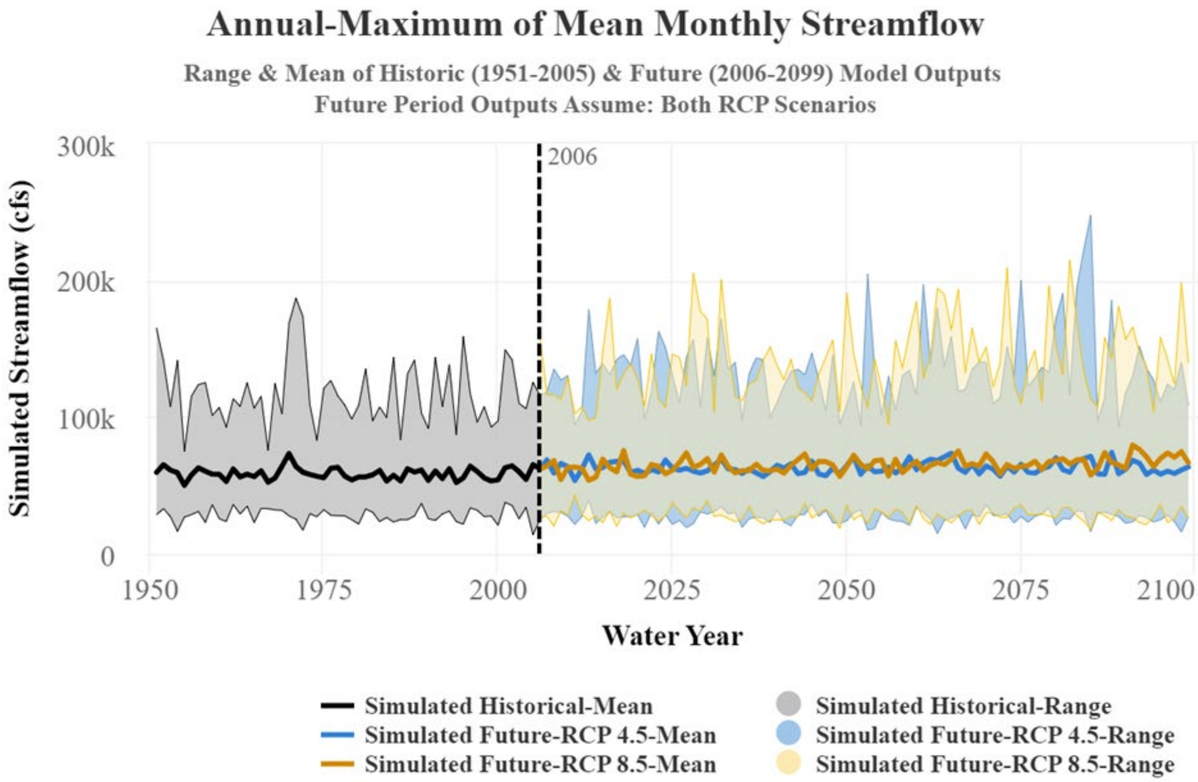
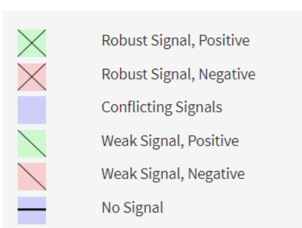


Figure A-27. Range of Annual-Maximum of Mean Monthly Streamflow Model Output for the Apple-Plum watershed (HUC07060005) Stream Segment: 07002612



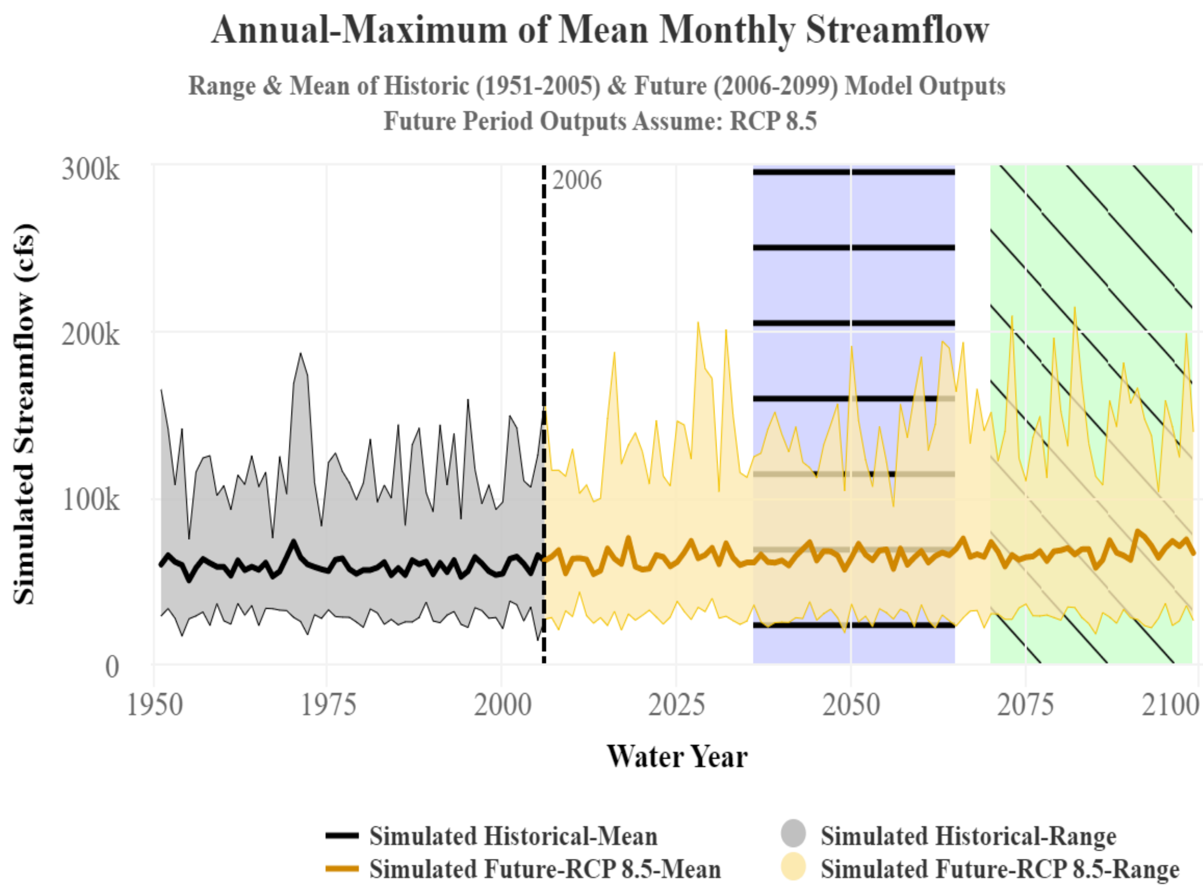


Figure A-28. Range and Robustness of Annual-Maximum of Mean Monthly Streamflow Model Output under RCP 8.5 for the Apple-Plum watershed (HUC07060005) Stream Segment: 07002612

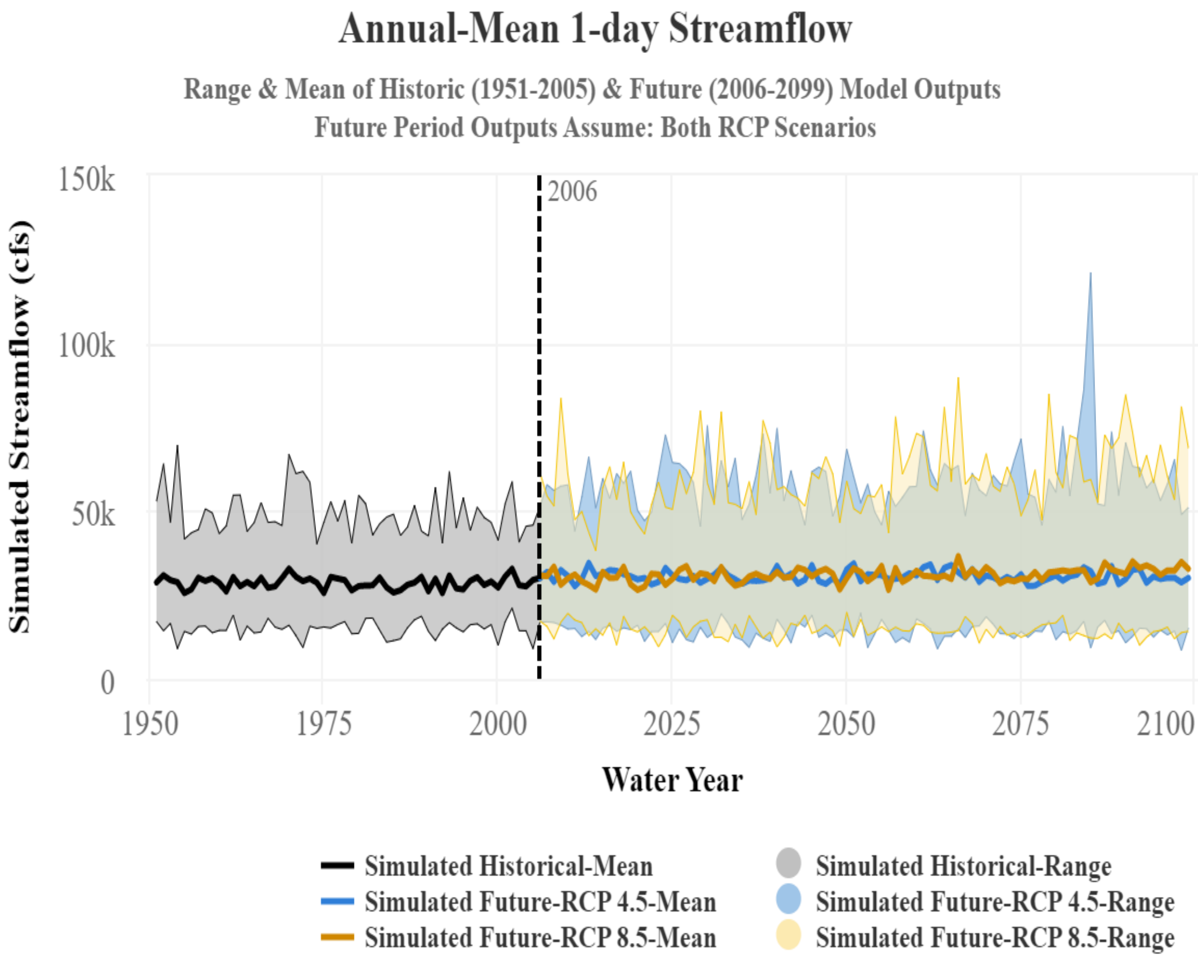


Figure A-29. Range of Annual-Mean Streamflow Model Output for the Apple-Plum watershed (HUC07060005) Stream Segment: 07002612

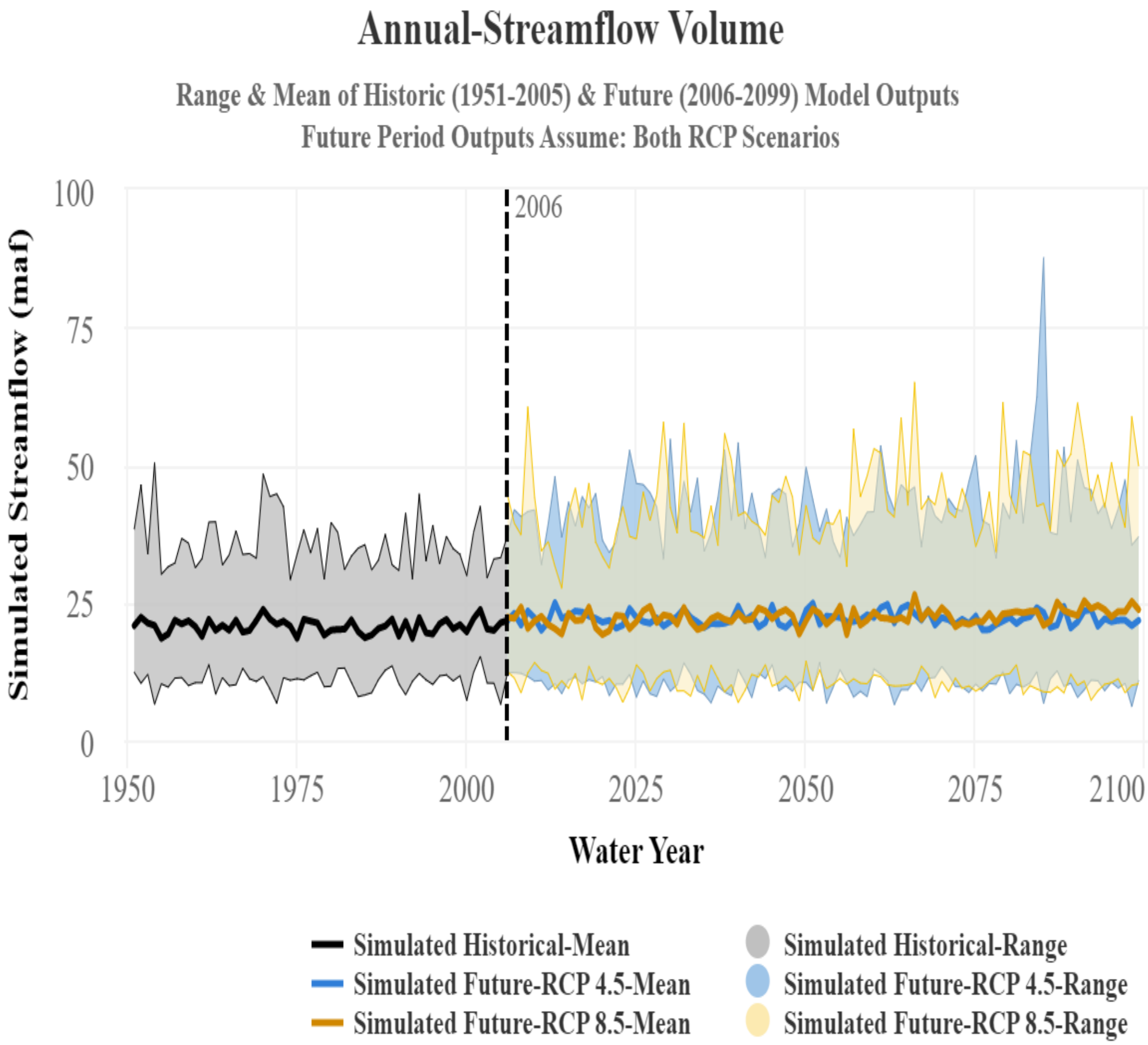


Figure A-30. Range of Annual Streamflow Volume Model Output for the Apple-Plum watershed (HUC07060005) Stream Segment: 07002612

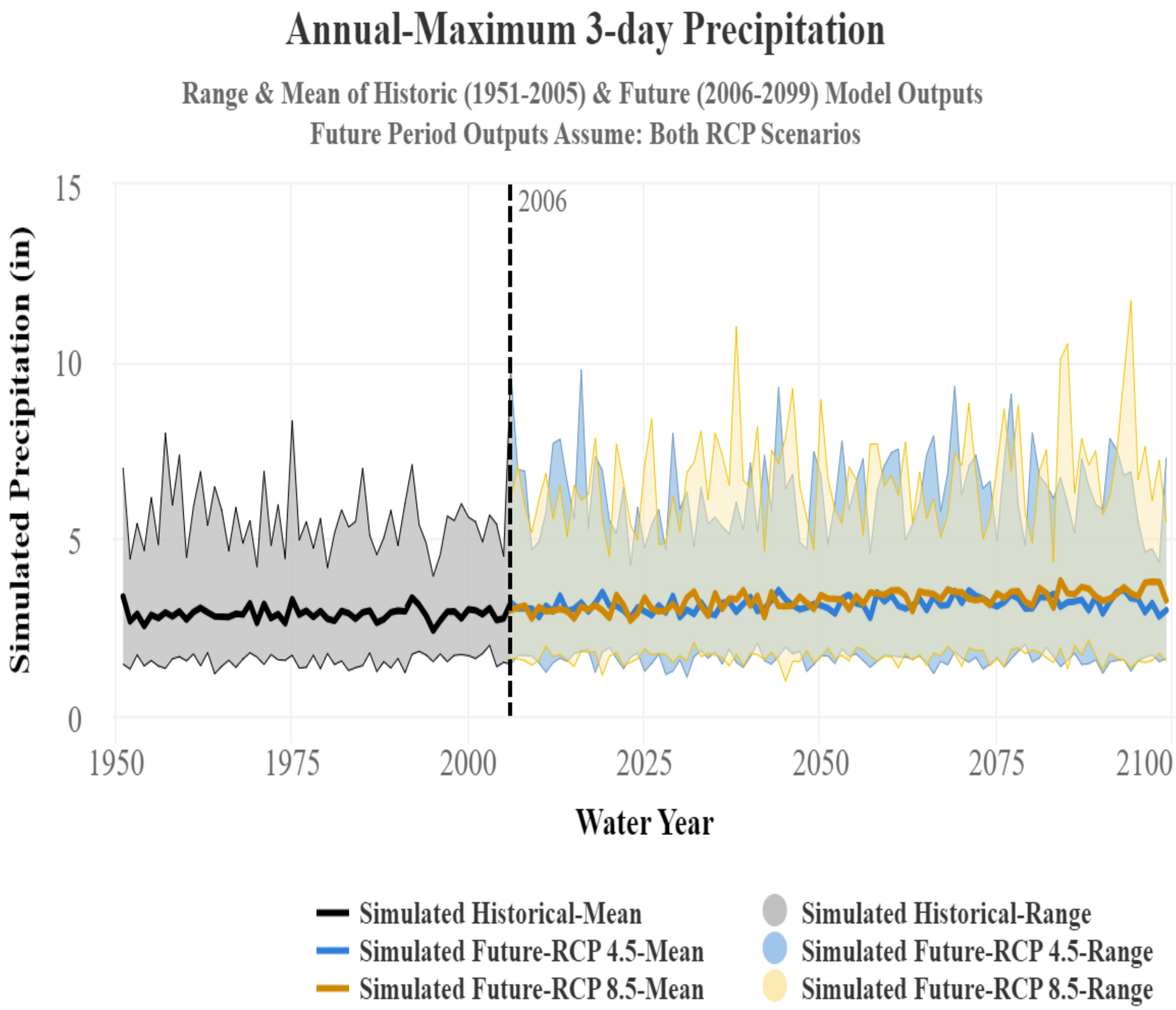


Figure A-31. Range of Annual Maximum 3-day Precipitation Model Output for the Apple-Plum watershed (HUC07060005)

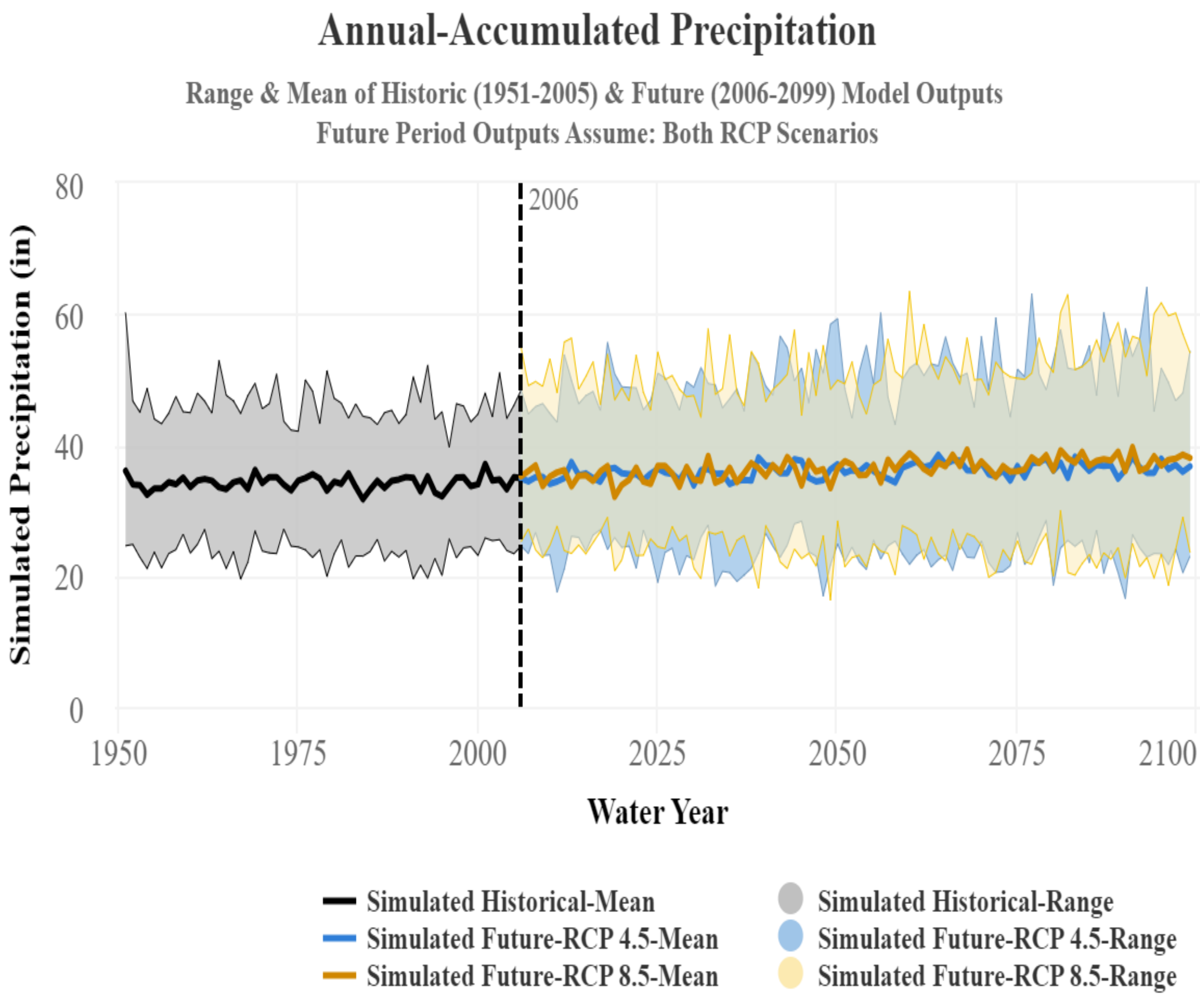


Figure A-32. Range of Annual Accumulated Precipitation Model Output for the Apple-Plum watershed (HUC07060005)

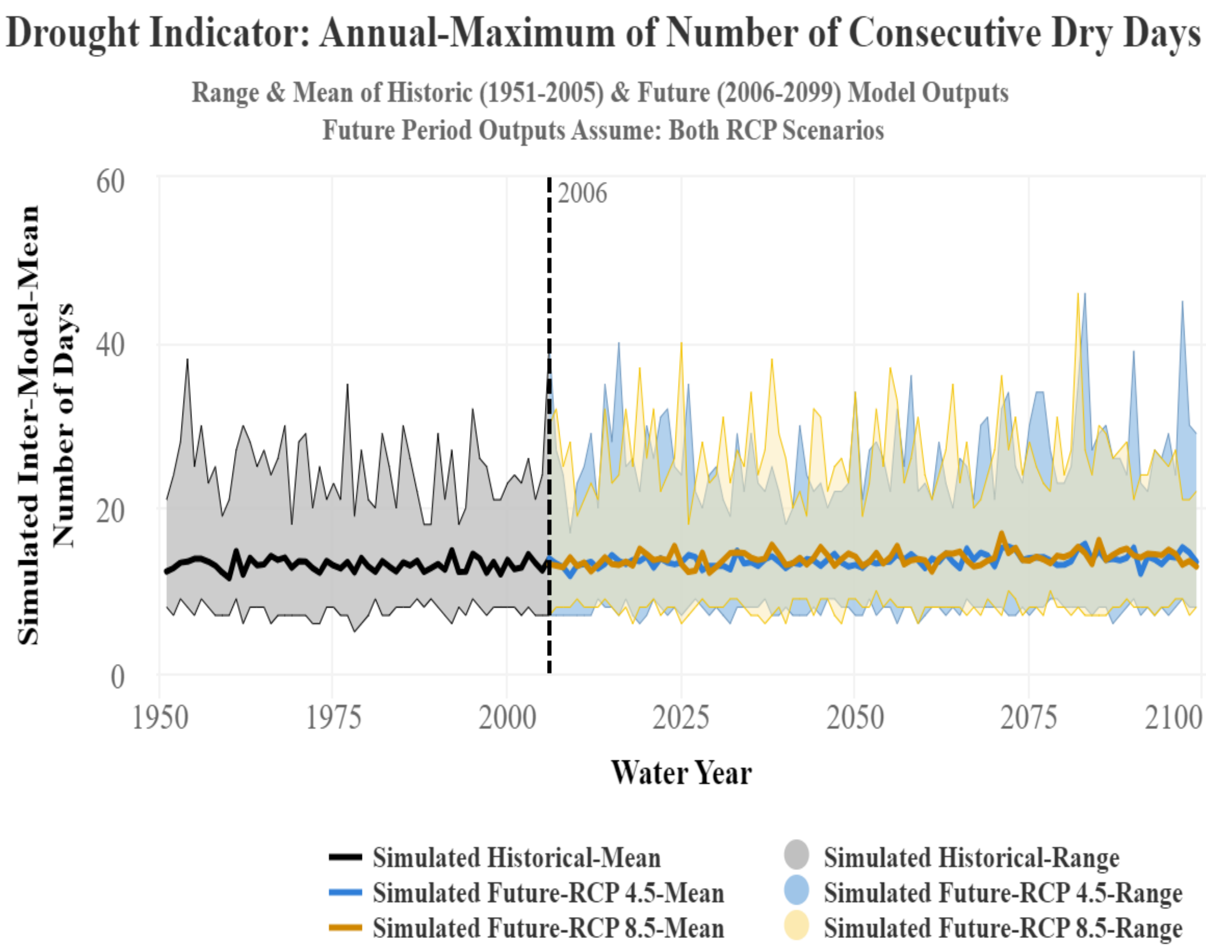


Figure A-33. Range of Drought Indicator Model Output for the Apple-Plum watershed (HUC07060005)

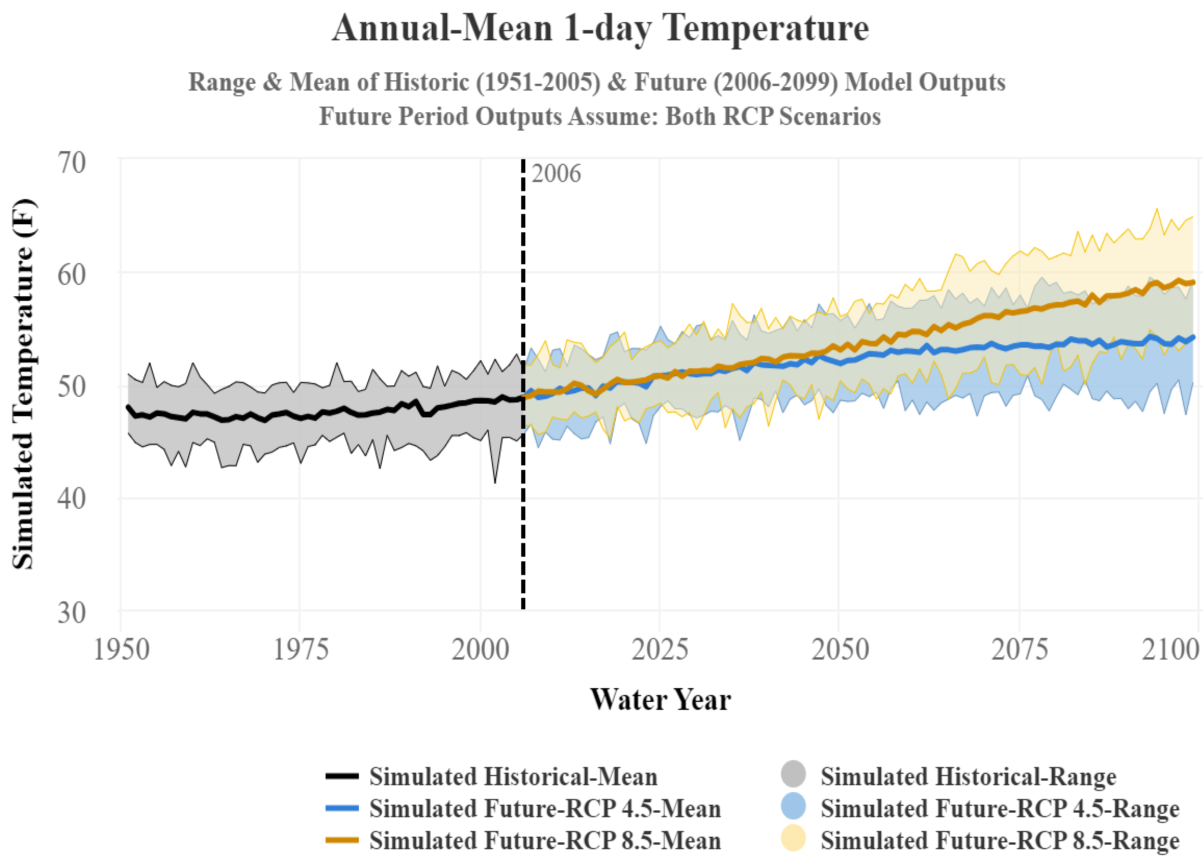


Figure A-34. Range of Annual Mean Temperature Model Output for the Apple-Plum watershed (HUC07060005)

For the Apple-Plum watershed (HUC07060005) trends in the mean model output are evaluated using the t-Test, Mann-Kendall and Spearman Rank-Order tests. All three statistical tests are applied using a 0.05 level of significance (p-values<0.05 are considered statistically significant). As displayed in Figure A-35, the direction and magnitude of change in statistically significant trends in mean annual-maximum of mean monthly streamflow are evaluated using the slope of the fitted linear regression relationship. The results of the three statistical tests and the slopes associated with identified, statistically significant trends are presented in Table A-14. The mean of the 32 projections of simulated, annual-maximum of mean monthly streamflow for the future period (2006-2099) shows a statistically significant, positive trend for the Apple-Plum watershed (HUC07060005) Stream Segment- 07002612 when RCP 8.5 is assumed. The trendline has a slope of 79 cfs a year, which equates to a 3,950 cfs change in the average of the 32 projections of annual-maximum of mean monthly streamflow over a 50-year period. When the CHAT is used to evaluate the change in Epoch-mean of simulated annual-maximum of mean monthly streamflow under RCP 8.5 it is found that the median change from the base Epoch (1976-2005) to the mid-century epoch (2035-2064) is 7.9%. By the end-century epoch (2070-2099) the change relative to the base period is 11.9%. There is no statistically significant trend in simulated, annual-maximum of mean monthly streamflow for the historic period (1951-2005) or for the future period (2006-2099) when RCP 4.5 is assumed.

Table A-14. Trend Analysis of Average Model Output: Annual – Maximum of Mean Monthly Streamflow Apple-Plum watershed (HUC07060005) Stream Segment: 07002612

Trend Analysis	Historic (1951-2005)	Future (2006-2099)		Historic (1951-2005)			Future (2006-2099)					
		RCP 4.5	RCP 8.5				RCP 4.5			RCP 8.5		
	p-values			Statistically Significant? (<0.05)	Slope (cfs/year)	Direction	Statistically Significant? (<0.05)	Slope (cfs/year)	Direction	Statistically Significant? (<0.05)	Slope (cfs/year)	Direction
t-Test	0.716	0.839	3.92e-5	No	Not applicable (no trend)		No	Not applicable (no trend)		Yes	79	↑
Mann-Kendall	0.632	0.695	3.86e-5	No			No			Yes		
Spearman Rank Order	0.725	0.699	4.8e-5	No			No			Yes		

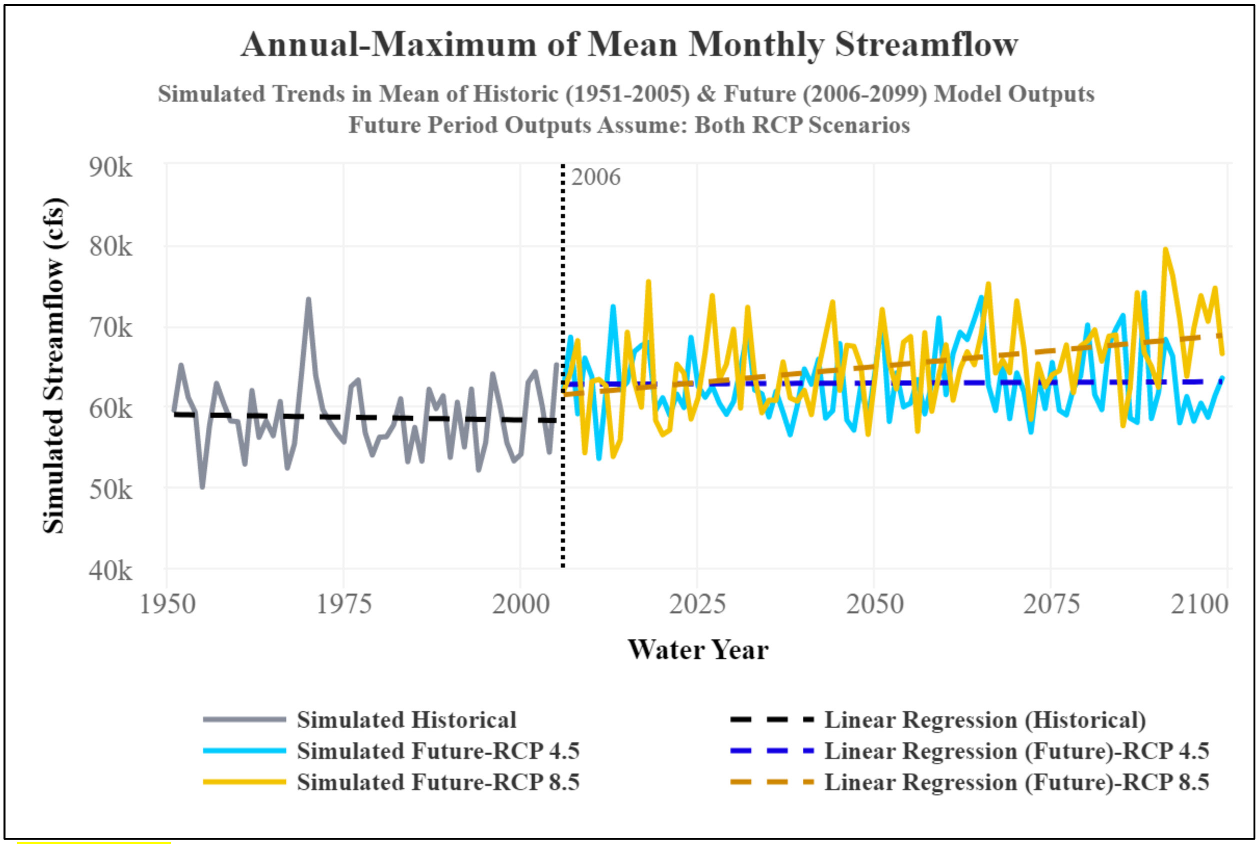


Figure A-35. Trend Analysis of Average Model Output: Annual-Maximum of Mean Monthly Streamflow Apple-Plum watershed (HUC07060005) Stream Segment: 07002612

For the mean of the 32 projections (per RCP) of annual-mean streamflow, the results of the three statistical tests and the slopes associated with statistically significant trends are presented in Table A-15 and Figure A-36. The mean of the simulated, annual-mean streamflow projections (future period: 2006-2099) show a statistically significant, positive trend for the Apple Plum watershed (HUC07060005) Stream Segment- 07002612 under the higher (RCP 8.5) emission scenarios. The CHAT computes a trendline slope of 27.6 cfs per year for the RCP 8.5 emission scenario, which would be a 1,380 cfs increase in annual-mean streamflow over a 50-year period. When the CHAT is used to evaluate the change in Epoch-Mean of simulated annual-mean streamflow it is found that the median change from the base Epoch (1976-2005) to the mid-century epoch (2035-2064) is 5.3% for RCP 8.5. By the end-century epoch (2070-2099) the change relative to the base period is 7.9% for RCP 8.5. There is no statistically significant trend in simulated, annual-mean streamflow for the historic period (1951-2005) or for the future period (2006-2099) when RCP 4.5 is assumed.

Table A-15. Trend Analysis of Average Model Output: Annual-Mean Streamflow for Apple-Plum watershed (HUC07060003) Stream Segment: 07002612

Trend Analysis	Historic (1951-2005)	Future (2006-2099)		Historic (1951-2005)			Future (2006-2099)					
		RCP 4.5	RCP 8.5				RCP 4.5			RCP 8.5		
	p-values			Statistically Significant? (<0.05)	Slope (cfs/year)	Direction	Statistically Significant? (<0.05)	Slope (cfs/year)	Direction	Statistically Significant? (<0.05)	Slope (cfs/year)	Direction
t-Test	0.832	0.648	0.000173	No	Not applicable (no trend)		No	Not applicable (no trend)		Yes	27.6	↑
Mann-Kendall	0.85	0.561	0.000409	No			No			Yes		
Spearman Rank Order	0.808	0.595	0.00046	No			No			Yes		

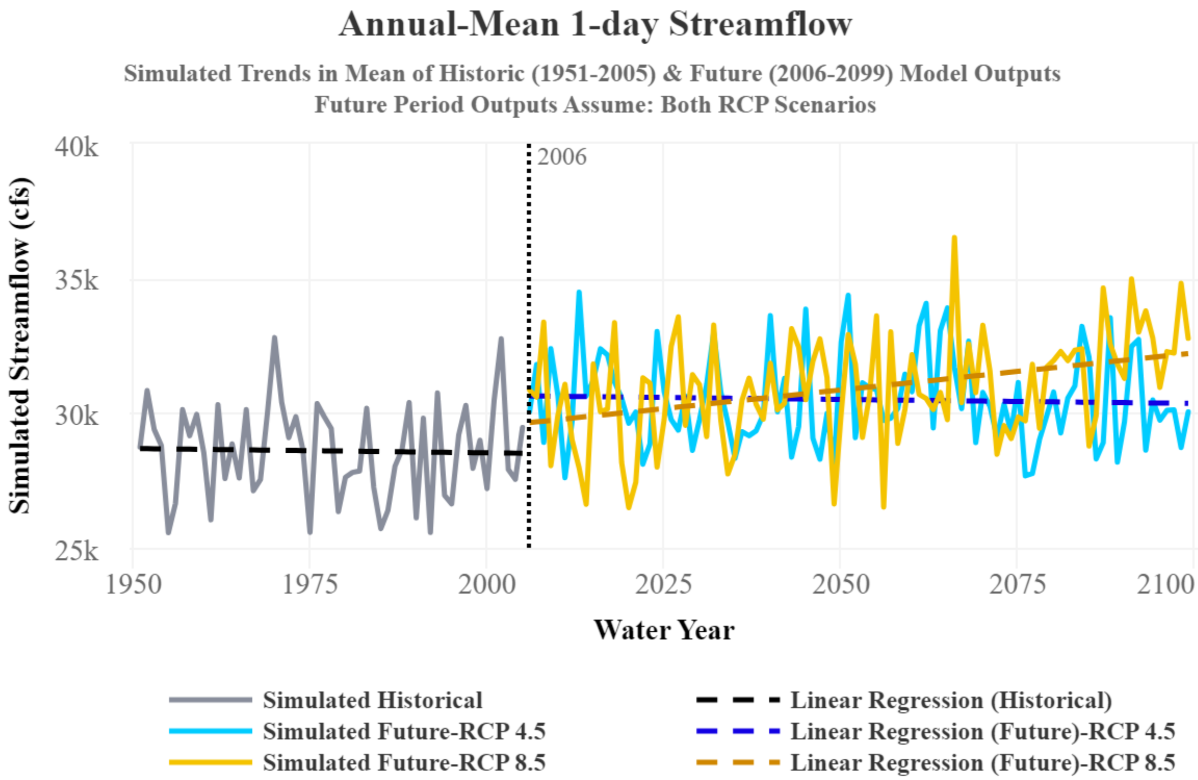


Figure A-36. Trend Analysis of Average Model Output: Annual-Mean Streamflow Apple-Plum watershed (HUC07060005) Stream Segment: 07002612

For the mean of the 32 projections (per RCP) of annual streamflow volume, the results of the three statistical tests and the slopes associated with statistically significant trends are presented in Table A-16 and Figure A-37. The mean of the simulated, annual streamflow volume projections (future period: 2006-2099) show a statistically significant, positive trend for the Apple Plum watershed (HUC07060005) Stream Segment- 07002612 under the higher (RCP 8.5) emission scenarios. The CHAT computes a trendline slope of 0.02 million acre-feet (maf) per year for the RCP 8.5 emission scenario, which would be a 1 maf increase in annual streamflow volume over a 50-year period. When the CHAT is used to evaluate the change in Epoch-Mean of simulated annual streamflow volume it is found that the median change from the base Epoch (1976-2005) to the mid-century epoch (2035-2064) is 5.3% for RCP 8.5. By the end-century epoch (2070-2099) the change relative to the base period is 7.9% for RCP 8.5. There is no statistically significant trend in simulated, annual streamflow volume for the historic period (1951-2005) or for the future period (2006-2099) when RCP 4.5 is assumed.

Table A-16. Trend Analysis of Average Model Output: Annual- Streamflow Volume for Apple-Plum watershed (HUC07060003) Stream Segment: 07002612

Trend Analysis	Historic (1951-2005)	Future (2006-2099)		Historic (1951-2005)			Future (2006-2099)					
		RCP 4.5	RCP 8.5				RCP 4.5			RCP 8.5		
	p-values			Statistically Significant? (<0.05)	Slope (maf/year)	Direction	Statistically Significant? (<0.05)	Slope (maf/year)	Direction	Statistically Significant? (<0.05)	Slope (maf/year)	Direction
t-Test	0.832	0.649	0.000174	No	Not applicable (no trend)		No	Not applicable (no trend)		Yes	0.02	↑
Mann-Kendall	0.85	0.554	0.000384	No			No			Yes		
Spearman Rank Order	0.804	0.589	0.000443	No			No			Yes		

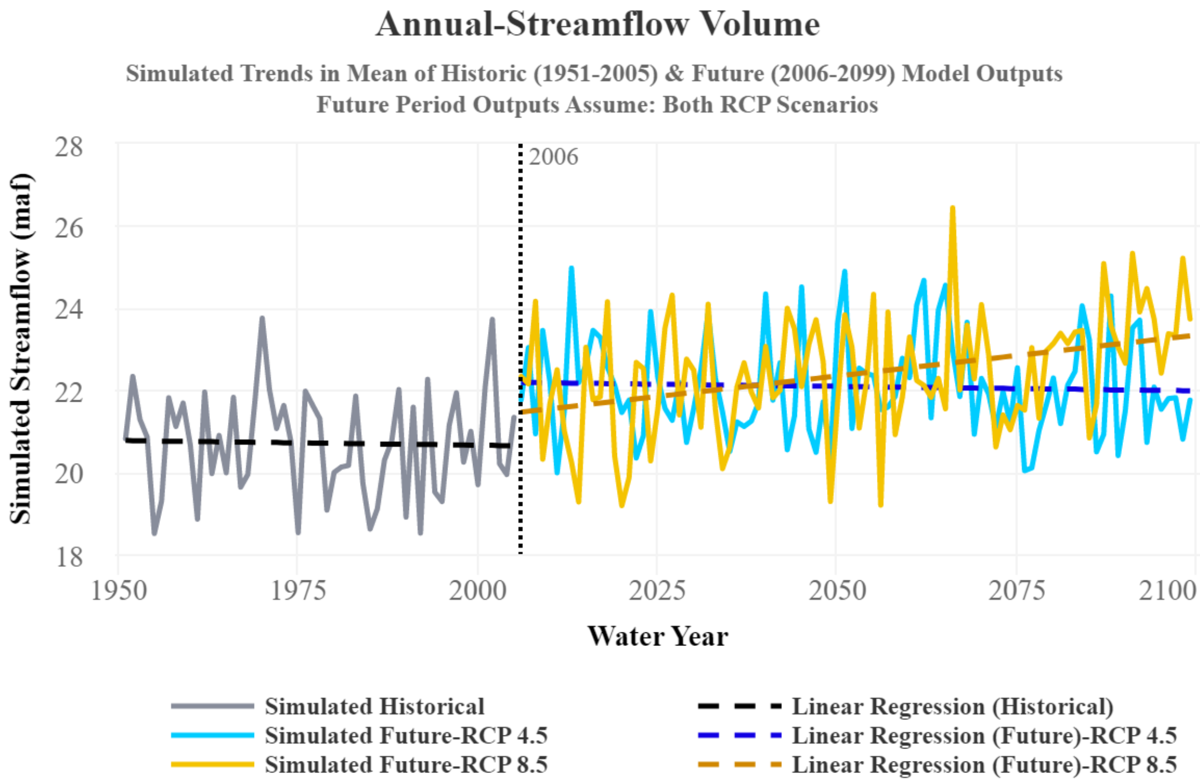


Figure A-37. Trend Analysis of Average Model Output: Annual- Streamflow Volume Apple-Plum watershed (HUC07060005) Stream Segment: 07002612

For the mean of the 32 projections (per RCP) of annual maximum 3-day precipitation, the results of the three statistical tests and the slopes associated with statistically significant trends are presented in Table A-17 and Figure A-38. The mean of the simulated, annual maximum 3-day precipitation projections (future period: 2006-2099) show a statistically significant, positive trend for the Apple Plum watershed (HUC07060005) under both emission scenarios. The CHAT computes a trendline slope of 0.002 inches per year for the RCP 4.5 emission scenario and 0.006 inches per year for the RCP 8.5 emission scenario. These projections indicate a 0.1 inch increase (RCP 4.5) and a 0.3 inch increase (RCP 8.5) in annual-maximum 3-day precipitation over a 50-year period. When the CHAT is used to evaluate the change in Epoch-Mean of simulated annual-maximum 3-day precipitation it is found that the median change from the base Epoch (1976-2005) to the mid-century epoch (2035-2064) is 0.26 inches for RCP 4.5 and 0.39 inches for RCP 8.5. By the end-century epoch (2070-2099) the change relative to the base period is 0.32 inches for RCP 4.5 and 0.53 inches for RCP 8.5.

Table A-17. Trend Analysis of Average Model Output: Annual- Maximum 3-day Precipitation for Apple-Plum watershed (HUC07060005)

Trend Analysis	Historic (1951-2005)	Future (2006-2099)		Historic (1951-2005)			Future (2006-2099)					
		RCP 4.5	RCP 8.5				RCP 4.5			RCP 8.5		
	p-values			Statistically Significant? (<0.05)	Slope (in/year)	Direction	Statistically Significant? (<0.05)	Slope (in/year)	Direction	Statistically Significant? (<0.05)	Slope (in/year)	Direction
t-Test	0.996	0.00194	5e-14	No	Not applicable (no trend)		Yes	0.002	↑	Yes	0.006	↑
Mann-Kendall	0.571	0.00185	<2.2e-16	No			Yes			Yes		
Spearman Rank Order	0.599	0.00159	5.8e-13	No			Yes			Yes		

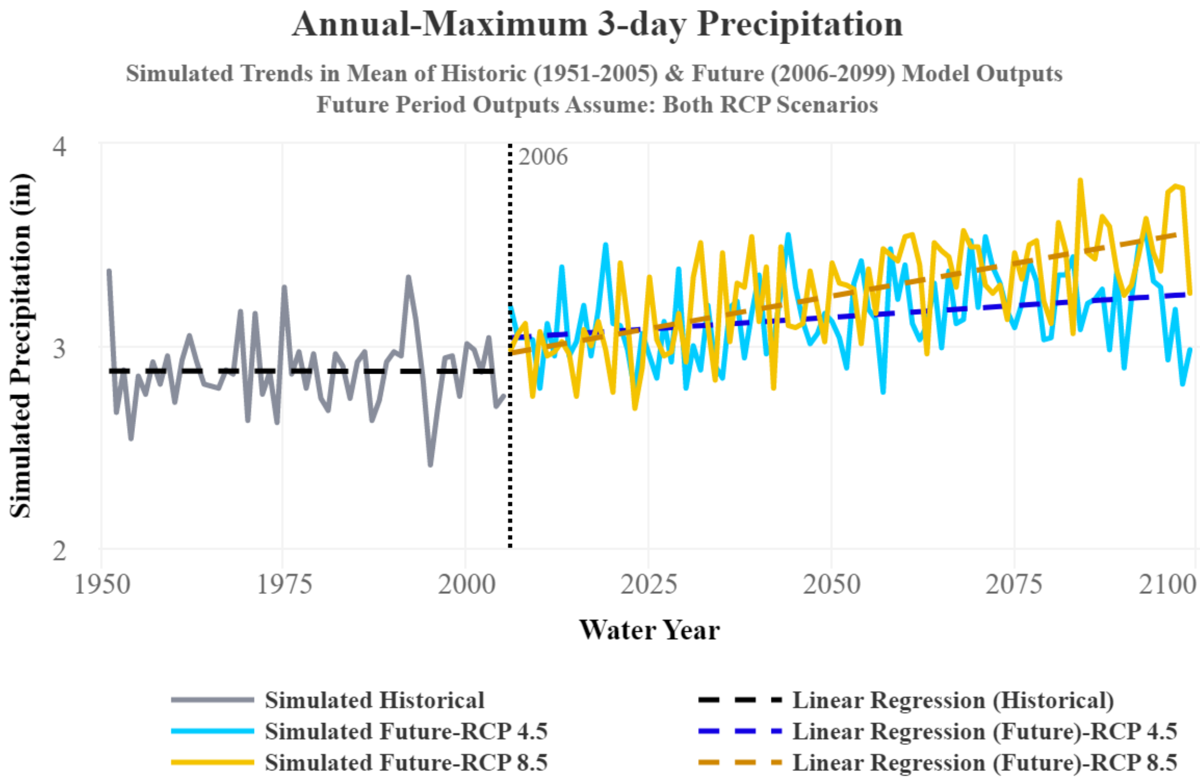


Figure A-38. Trend Analysis of Average Model Output: Annual- Maximum 3-day Precipitation for Apple-Plum watershed (HUC07060005)

For the mean of the 32 projections (per RCP) of annual-accumulated precipitation, the results of the three statistical tests and the slopes associated with statistically significant trends are presented in Table A-18 and Figure A-39. The mean of the simulated, annual-accumulated precipitation projections (future period: 2006-2099) show a statistically significant, positive trend for the Apple Plum watershed (HUC07060005) under both emissions scenarios. The CHAT computes a trendline slope of 0.02 inches per year for the RCP 4.5 emission scenario and 0.03 inches per year for the RCP 8.5 emission scenario. These trends indicate a 1 inch and 1.5 inch increase in annual-accumulated precipitation over a 50-year period under RCP 4.5 and RCP 8.5, respectively. When the CHAT is used to evaluate the change in Epoch-Mean of simulated annual-accumulated precipitation it is found that the median change from the base Epoch (1976-2005) to the mid-century epoch (2035-2064) is 2.6 inches for RCP 4.5 and 4.1 inches for RCP 8.5. By the end-century epoch (2070-2099) the change relative to the base period is 3.7 inches for RCP 4.5 and 6.1 inches for RCP 8.5.

Table A-18. Trend Analysis of Average Model Output: Annual- Accumulated Precipitation for Apple-Plum watershed (HUC07060005)

Trend Analysis	Historic (1951-2005)	Future (2006-2099)		Historic (1951-2005)			Future (2006-2099)					
		RCP 4.5	RCP 8.5				RCP 4.5			RCP 8.5		
	p-values			Statistically Significant? (<0.05)	Slope (in/year)	Direction	Statistically Significant? (<0.05)	Slope (in/year)	Direction	Statistically Significant? (<0.05)	Slope (in/year)	Direction
t-Test	0.638	4.46e-6	2.03e-10	No			Yes			Yes		
Mann-Kendall	0.477	9.18e-6	<2.2e-16	No	Not applicable (no trend)		Yes	0.02	↑	Yes	0.03	↑
Spearman Rank Order	0.527	2.16e-6	1.51e-10	No			Yes			Yes		

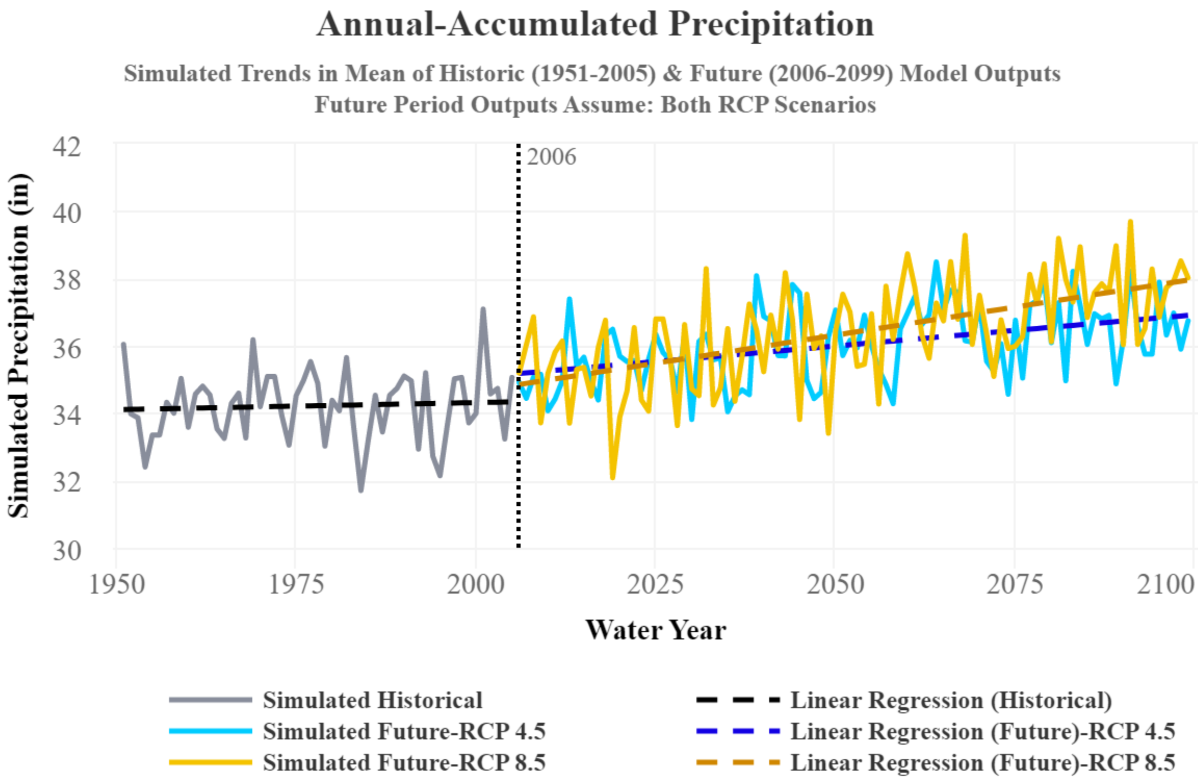


Figure A-39. Trend Analysis of Average Model Output: Annual- Accumulated Precipitation for Apple-Plum watershed (HUC07060005)

For the mean of the 32 projections (per RCP) of annual drought indicator, the results of the three statistical tests and the slopes associated with statistically significant trends are presented in Table A-19 and Figure A-40. The mean of the simulated, annual drought indicator projections (future period: 2006-2099) show a statistically significant, positive trend for the Apple Plum watershed (HUC07060005) under both emission scenarios. The CHAT computes a trendline slope of 0.01 days per year for both the RCP 4.5 and RCP 8.5 emissions scenarios. This results in a 0.5 day increase in annual drought indicator over a 50-year period under both emissions scenarios. When the CHAT is used to evaluate the change in Epoch-Mean of simulated annual drought indicator it is found that the median change from the base Epoch (1976-2005) to the mid-century epoch (2035-2064) is 0.5 days for RCP 4.5 and 0.8 days for RCP 8.5. By the end-century epoch (2070-2099) the change relative to the base period is 0.9 days for RCP 4.5 and RCP 8.5.

Table A-19. Trend Analysis of Average Model Output: Drought Indicator for Apple-Plum watershed (HUC07060005)

Trend Analysis	Historic (1951-2005)	Future (2006-2099)		Historic (1951-2005)			Future (2006-2099)					
		RCP 4.5	RCP 8.5				RCP 4.5			RCP 8.5		
	p-values			Statistically Significant? (<0.05)	Slope (days/year)	Direction	Statistically Significant? (<0.05)	Slope (days/year)	Direction	Statistically Significant? (<0.05)	Slope (days/year)	Direction
t-Test	0.736	7.14e-5	0.00171	No	Not applicable (no trend)		Yes	0.01	↑	Yes	0.01	↑
Mann-Kendall	0.481	7.81e-5	0.00195	No			Yes			Yes		
Spearman Rank Order	0.551	7.16e-5	0.00155	No			Yes			Yes		

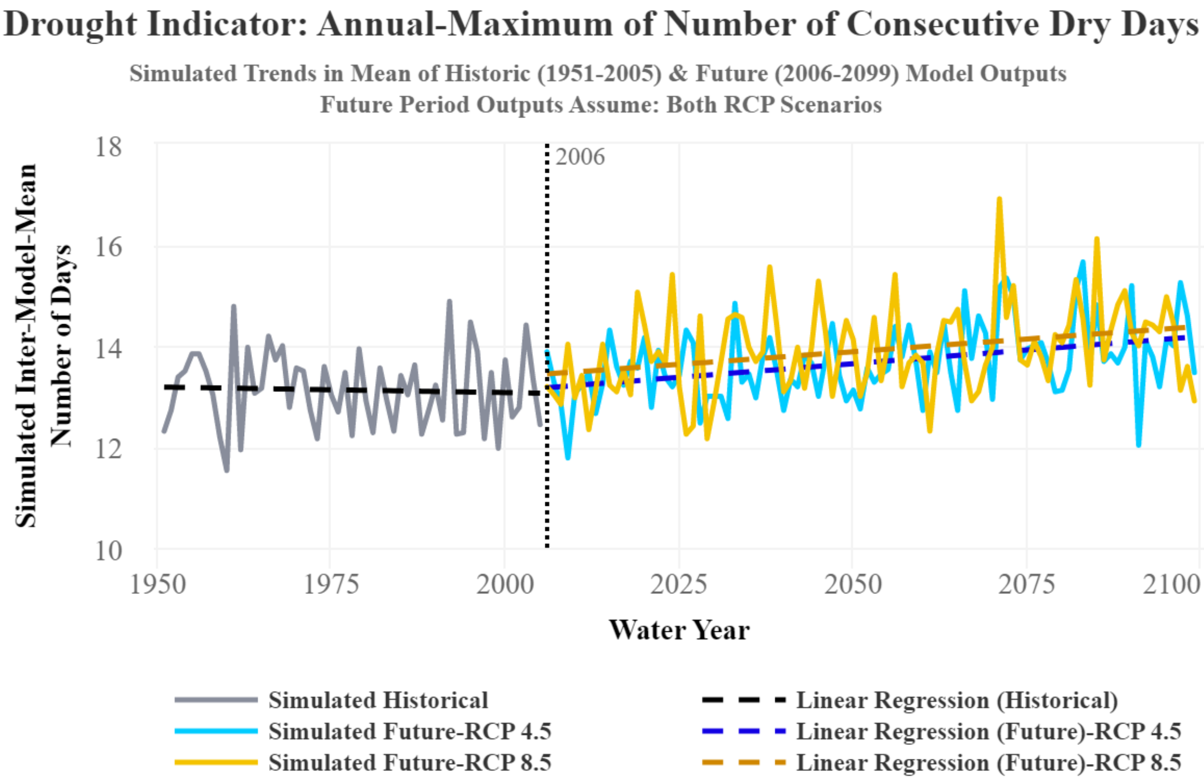


Figure A-40. Trend Analysis of Average Model Output: Drought Indicator for Apple-Plum watershed (HUC07060005)

For the mean of the 32 projections (per RCP) of annual-mean temperature, the results of the three statistical tests and the slopes associated with statistically significant trends are presented in Table A-20 and Figure A-41. The mean of the simulated historic (1951-2005), annual-mean temperature shows a statistically significant, positive trend, as does the simulated future (2006-2099), annual-mean temperature for the Apple Plum watershed (HUC07060005) under both emission scenarios. The CHAT computes a trendline slope of 0.03, 0.06, and 0.11° F per year for the historic period, future RCP 4.5 emission scenario, and future RCP 8.5 emission scenario, respectively. These trends indicate a 1.5, 3 and 5.5° F increase in annual-mean temperature over a 50-year period for the historic period, future RCP 4.5 emission scenario, and future RCP 8.5 emission scenario, respectively. When the CHAT is used to evaluate the change in Epoch-Mean of simulated annual-mean temperature it is found that the median change from the base Epoch (1976-2005) to the mid-century epoch (2035-2064) is 4.2° F for RCP 4.5 and 5.6° F for RCP 8.5. By the end-century epoch (2070-2099) the change relative to the base period is 5.8° F for RCP 4.5 and 10.0° F for RCP 8.5.

Table A-20. Trend Analysis of Average Model Output: Annual-Mean Temperature for Apple-Plum watershed (HUC07060005)

Trend Analysis	Historic (1951-2005)	Future (2006-2099)		Historic (1951-2005)			Future (2006-2099)					
		RCP 4.5	RCP 8.5				RCP 4.5			RCP 8.5		
	p-values			Statistically Significant? (<0.05)	Slope (°F/year)	Direction	Statistically Significant? (<0.05)	Slope (°F/year)	Direction	Statistically Significant? (<0.05)	Slope (°F/year)	Direction
t-Test	1.52e-11	<2.2e-16	<2.2e-16	Yes	0.03	↑	Yes	0.06	↑	Yes	0.11	↑
Mann-Kendall	<2.2e-16	<2.2e-16	<2.2e-16	Yes			Yes			Yes		
Spearman Rank Order	6.95e-10	<2.2e-16	<2.2e-16	Yes			Yes			Yes		

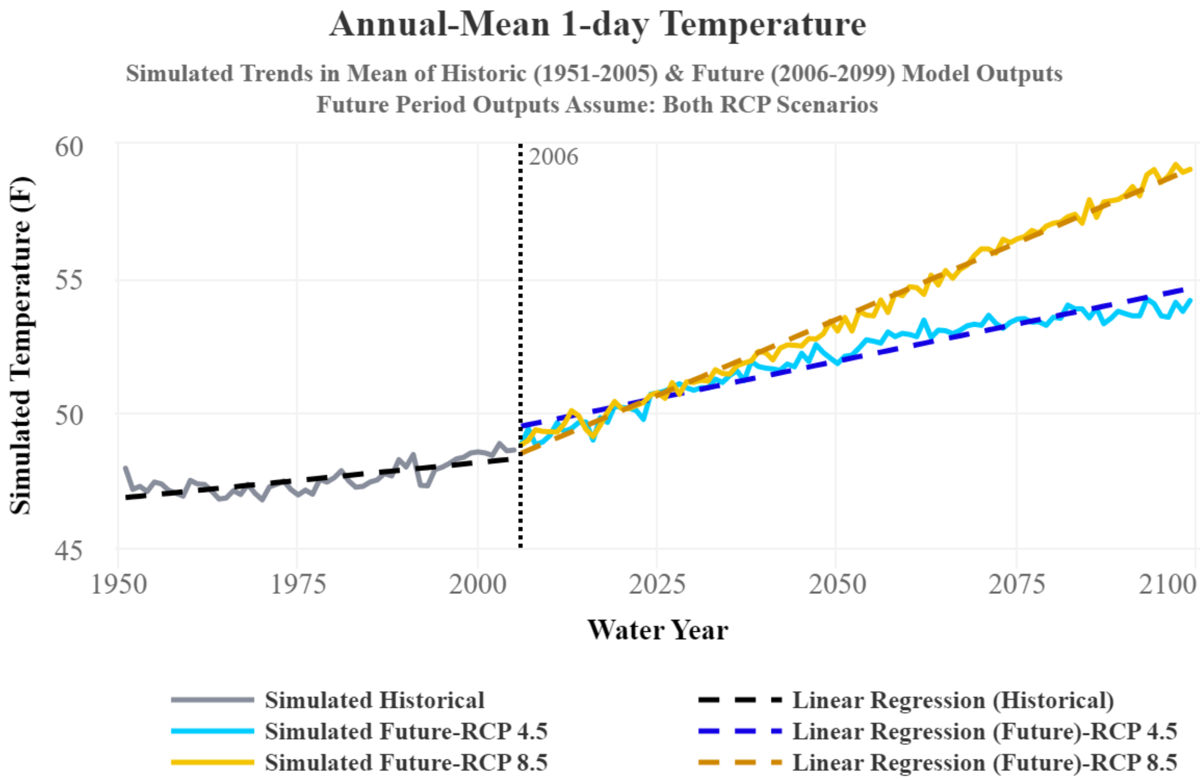


Figure A-41. Trend Analysis of Average Model Output: Annual-Mean Temperature for Apple-Plum watershed (HUC07060005)

Like the comparative analysis of simulated changes in annual output between different epochs (time periods), the CHAT provides streamflow, precipitation and temperature outputs analyzed comparatively by describing simulated changes in monthly streamflow, precipitation, and temperature output between different epochs. Monthly streamflow, precipitation, and temperature output is analyzed by determining the mean of the monthly value for the variable of interest for each GCM for three epochs: 1976-2005 (baseline), 2035-2064 (mid-century), and 2070-2099 (end of century). The difference between GCM/Month/Epoch means are determined for both the baseline vs. mid-century and baseline vs. end of century epochs and results are presented as boxplots. These boxplots provide insight into both the range of results and the seasonality of changes in streamflow, precipitation, and temperature over time.

For stream segment 07002612 in the Apple-Plum watershed (HUC07060005), changes in epoch-mean of simulated monthly-mean streamflow and simulated monthly streamflow volume are presented in Figures A-42 and A-43, respectively. The same seasonal trends observed for the changes in epoch-mean of simulated monthly-mean streamflow are present in the changes in epoch-mean of simulated monthly streamflow volume and are described in the following. Both the mid-century and end-century epochs show an increase (positive change) from November through June for the epoch-mean of monthly-mean streamflow and monthly streamflow volume output under both emission scenarios. December through April epoch-means show greater increase (positive change) under RCP 8.5 output than those simulated under RCP 4.5 for both the mid-century and end-century epochs. Simulated August monthly-mean streamflow and monthly streamflow volume output suggest a decreasing epoch-mean (negative change) under both emissions scenarios, for both epochs. Increased streamflow and streamflow volume during the spring growing season can result in flooding, impacting the sponsor's ability to drawdown water levels in support vegetation recruitment.

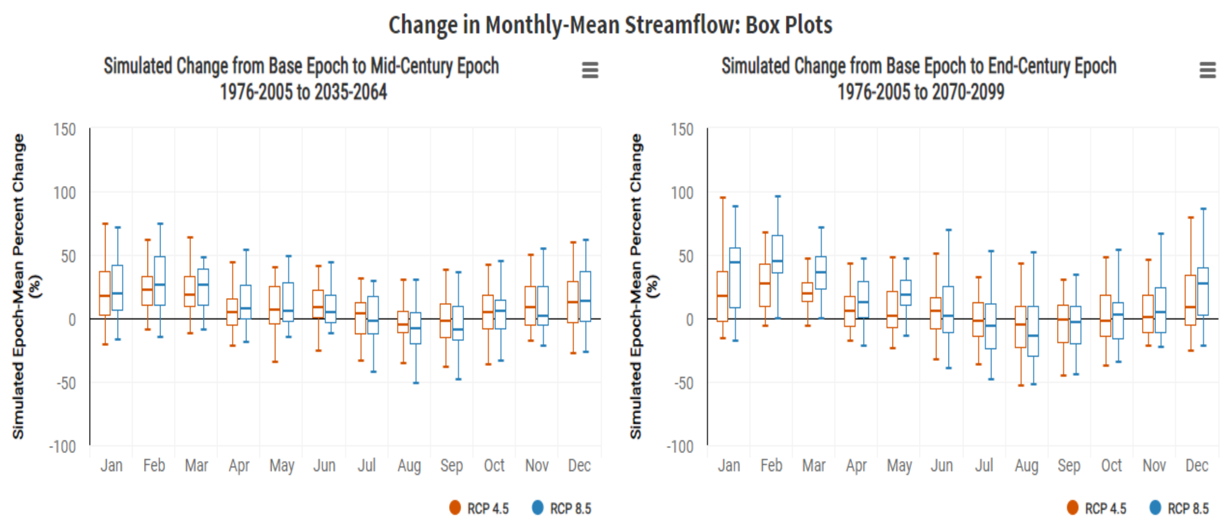


Figure A-42. Change in Epoch-Mean of Simulated Monthly-Mean Streamflow - HUC 07060005 – Apple-Plum- Stream segment ID: 07002612

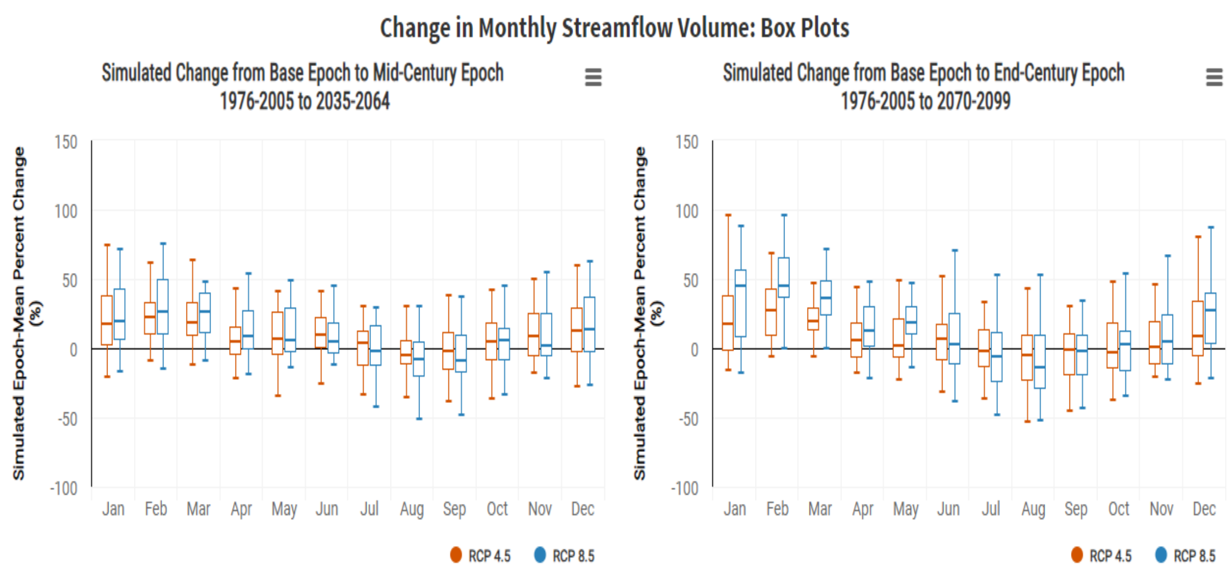


Figure A-43. Change in Epoch-Mean of Simulated Monthly Streamflow Volume - HUC 07060005 – Apple-Plum- Stream segment ID: 07002612

Changes in epoch-mean of simulated monthly accumulated precipitation for the Apple-Plum watershed (HUC07060005) are shown in Figure A-44. Both the mid-century and end-century epochs show an increase (positive change) from October through May for the epoch-mean of monthly accumulated precipitation under both emissions scenarios. Simulated August monthly accumulated precipitation output indicates a decreasing epoch-mean (negative change) under both emissions scenarios, for both epochs.

For the Apple-Plum watershed (HUC07060005), changes in epoch-mean of simulated monthly-maximum 3-day precipitation are shown in Figure A-45. Increases are shown in the epoch-mean (positive change) of simulated monthly-maximum 3-day precipitation from October through June for both the mid-century and end-century epochs, under both emissions scenarios. The greatest increase in simulated epoch-mean (0.44 inches) occurs in April during the end-century epoch, under RCP 8.5. Increased future precipitation can result in increased flood volumes, flood frequency and duration that could increase the frequency of levee overtopping.

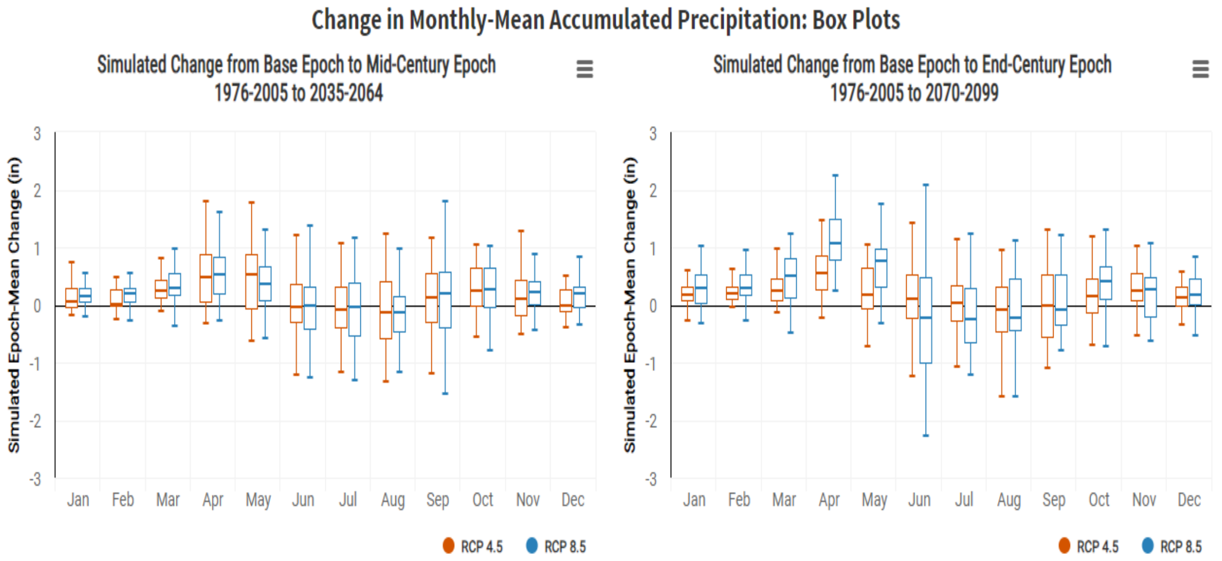


Figure A-44. Change in Epoch-Mean of Simulated Monthly Accumulated Precipitation - HUC 07060005 – Apple-Plum

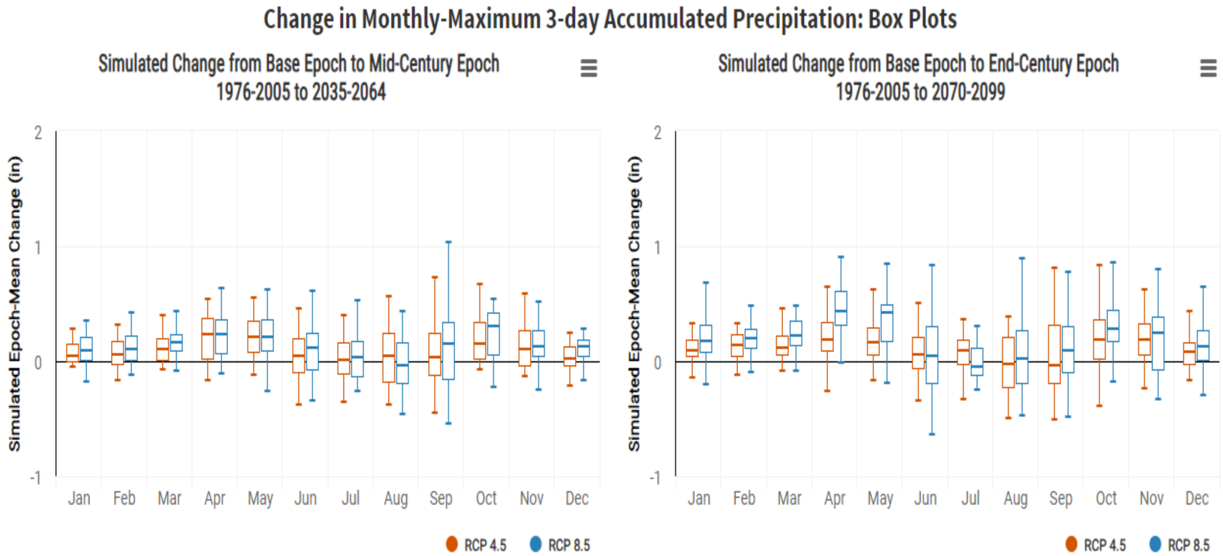


Figure A-45. Change in Epoch-Mean of Simulated Monthly-Maximum 3-day Accumulated Precipitation - HUC 07060005 – Apple-Plum

For the Apple-Plum watershed, simulated monthly-mean temperatures for both the mid-century epoch (2035-2064) and the end-century epoch (2070-2099) are increasing relative to historic temperature simulations (1950-2005) for all months and both RCPs (Figure A-46). For the mid-century comparisons, 4.7° F increases or greater in monthly-mean temperature are projected under RCP 8.5 for all months but April, May, and November. Larger changes in monthly-mean temperature are projected by the end-century. For the late-century epoch, there are larger discrepancies in epoch-mean change driven by emission scenario (RCP 8.5 vs. RCP 4.5), compared to those for the mid-century epoch. When RCP 8.5 is assumed, over 7.7° F of warming is projected for all months, with a maximum warming of 11.3° F projected for January. All RCP 8.5 comparisons show greater than 7.7° F of warming. When RCP 4.5 is assumed, between 4.4° F to 6.9° F of warming is projected for all months. Increases in mean-monthly air temperature during the summer months (June-August) are likely to increase surface water temperatures. This has the potential to adversely impact water quality by decreasing DO in backwater areas within the study area and impacting fisheries. Winter warming may result in accelerated and/or earlier spring snowmelt. These potential snowmelt characteristic changes and projected increases in spring precipitation could contribute to increased flooding and the risk of levee overtopping and failure.

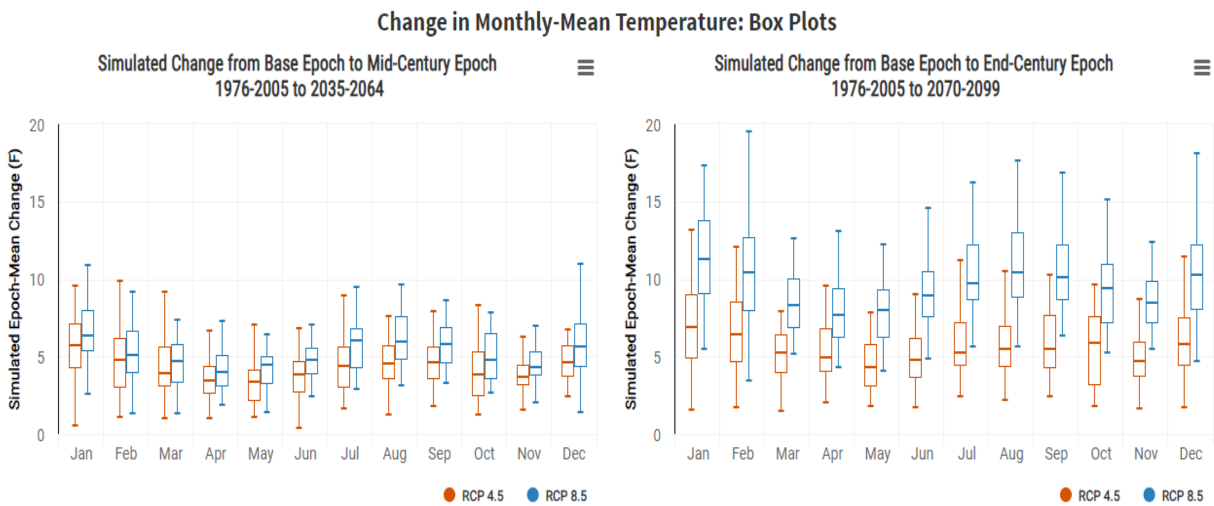


Figure A-46. Change in Epoch-Mean of Simulated Monthly-Mean Temperature- HUC 07060005 – Apple-Plum

8.5 Vulnerability Assessment

The USACE Climate Change Vulnerability Assessment (VA) Tool facilitates a screening level, comparative evaluation of climate change exposure to projects for a selected USACE business line in a given 4-digit HUC watershed relative to the other 4-digit HUC watersheds within the continental United States (CONUS). A series of indicator variables are computed and aggregated into a vulnerability score using the weighted-order, weighted-average (WOWA) approach. The tool uses the CMIP5 GCM-based Bias Corrected, Spatially Disaggregated (BCSD) VIC dataset (2014) to define projected, hydrologic, and meteorologic inputs to the tool's WOWA scores.

The WOWA scores and indicator variable values are available for two subsets of simulations (wet- top 50% by cumulative runoff projections and dry- bottom 50% by cumulative runoff projections). Data are available for three epochs. The epochs include a historic period (Base epoch) and two 30-year, future epochs (centered on 2050 and 2085). The Base epoch is not based on projections and it is not split into a wet and dry subset. Watersheds with WOWA scores specific to a given business line, that fall within the top 20% of WOWA scores for watersheds in the CONUS are identified as being vulnerable to climate change impacts. The projected datasets incorporated into VA scores contain considerable uncertainty. Some of this uncertainty is reflected by the differences in results for each of the subset-epoch combinations.

The tool is applied using the default, National Standards Settings and for the ecosystem restoration business line. Indicators used to compute the Ecosystem Restoration WOWA score include: change in sediment load due to change in future precipitation, cumulative monthly runoff variation relative to mean annual runoff, runoff elasticity (ratio of streamflow runoff change to precipitation change), macroinvertebrate index of biotic condition, local mean annual runoff, low flow reduction, percent of freshwater plant communities at risk, and two indicators of flood magnification (indicator of how much high flows are projected to change over time).

As shown in Figure A-47, compared to the other 4-digit HUC watersheds in the CONUS, the Upper Mississippi-Maquoketa-Plum (HUC 0706) watershed does not have a climate change vulnerability score in the top 20% for the ecosystem restoration business line. This is a comparative evaluation and thus does not imply that the watershed is not vulnerable to future, climate change impacts. Results indicate that for the select metrics incorporated into the tool, this watershed may be less exposed to potential climate change impacts relative to other watersheds in the CONUS. This is true for both the wet and dry subsets and both the 2050 and 2085 epochs.

As can be seen in Figure A-47 and Table A-21, the dominant indicator variable contributing to the Ecosystem Restoration business line VA score for the Upper Mississippi-Maquoketa-Plum (HUC 0706) watershed is (8) At Risk Freshwater Plants for all epoch and subset combinations. The WOWA score changes by less than 1% between the 2050 and 2085 epochs for both the wet and dry subsets. The percentage by which the indicator variable contributes to the VA score does not significantly change overtime. Because this indicator variable does not significantly vary dependent on computed, GCM-based changes in future hydrology (temperature, precipitation, streamflow) this indicator variable value is considered constant over time.

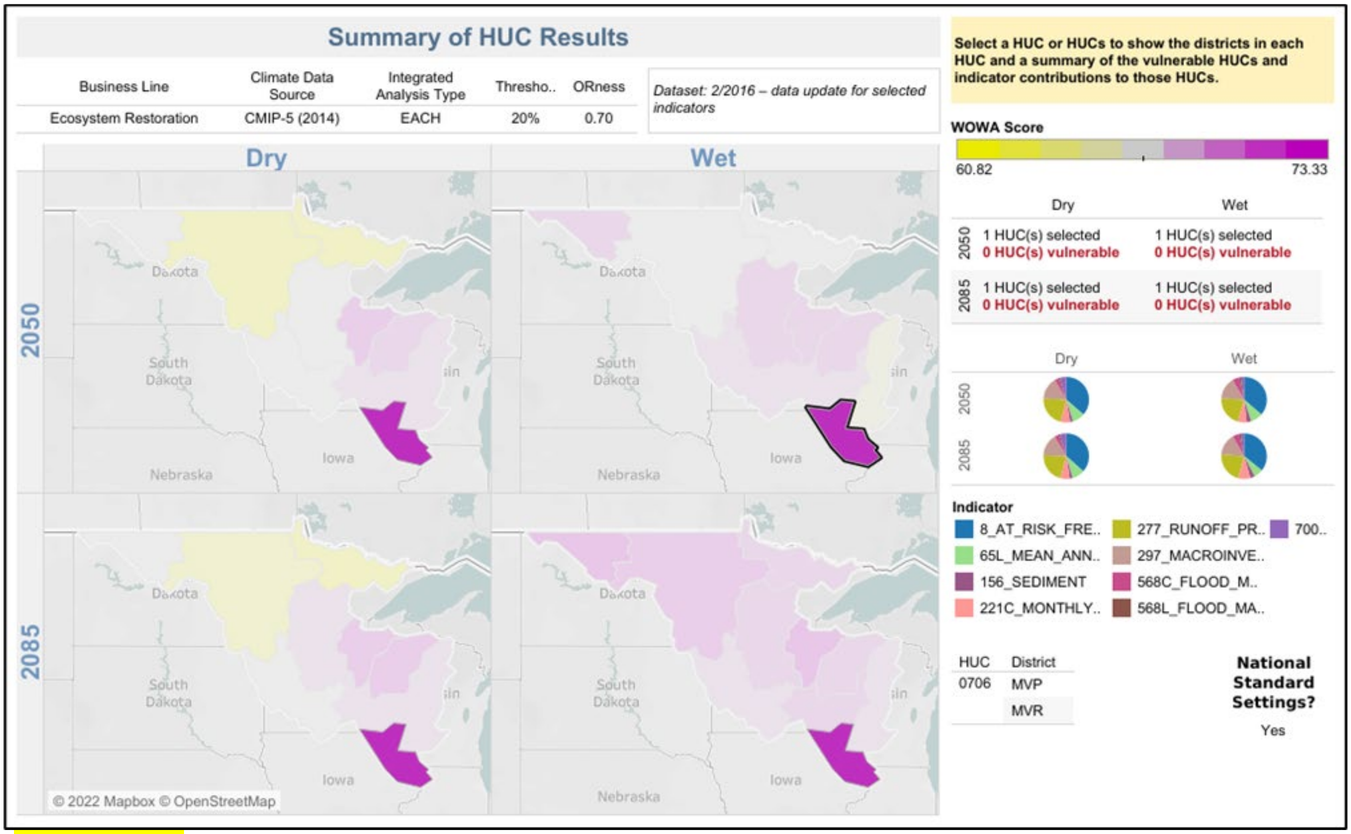


Figure A-47. Output of the Vulnerability Assessment tool - Upper Mississippi-Maquoketa-Plum watershed

Table A-21. VA Tool Output- HUC 0706 Upper Mississippi-Maquoketa-Plum Watershed- Ecosystem Restoration

Subset	Epoch	VA Score	% Change in VA Score (2050 to 2085)	Dominant Indicator (Value)	Dominant Indicator % Change (2050 to 2085)	
					Contribution to Overall WOWA Score	Indicator Value
WET	2050	71.28	+0.76%	8- At Risk Freshwater Plants (25.89)	-0.5%	Constant (-0.7%)
	2085	71.82		8- At Risk Freshwater Plants (25.71)		
DRY	2050	71.78	-0.56%	8- At Risk Freshwater Plants (26.11)	+0.2%	Constant (0%)
	2085	71.38		8- At Risk Freshwater Plants (26.11)		

8.6 Conclusion

The objective of the Green Island HREP is to restore the historic hydrologic cycle to improve the management and sustainability of existing habitat and associated plant and wildlife resources. The selected plan provides water level management capability to benefit wetland, floodplain forest, aquatic vegetation and fish habitat. The Project includes a bi-directional pump station, a sluice gate structure, island/berm creation and restoration, habitat and conveyance dredging, and timber stand improvement. Output based on both historic, observed hydrometeorological data and projected, climate-changed hydrometeorological data is reviewed to support qualitative statements that help identify ways resiliency to climate change impacts can be incorporated over the Green Island HREP's lifecycle.

Based on the weight of evidence presented in this assessment, climate change impacts are anticipated to affect the study area's hydrology over the Project's 50-year life cycle. Available climate change literature suggests a warmer and wetter climate in the future. There are statistically significant increasing trends or nonstationarities in observed annual-mean streamflow and all projected flow data analyzed specific to this study area. As streamflow increases, flood characteristics such as flood magnitude, flood frequency and flood duration can also increase resulting in a greater risk of levee overtopping or failure that would limit or inhibit interior water level management and thus impact the habitat of concern. There is also evidence that temperatures are increasing in the study area which may negatively affect water quality and aquatic habitat. Table A-22 indicates potential residual risks for this project due to climate change, along with a qualitative rating of how likely those residual risks are to materialize and undermine Project features resulting in harm to the study area.

Within the Upper Mississippi River Region climate change poses a potential risk to ecosystems due to the likelihood of the region experiencing shifts in the flow regime and increases in precipitation and temperature in the future. Projects, like the Green Island HREP will serve to offset some of this risk by providing water level management capability with operational flexibility to adapt to a greater variability of hydrologic conditions than is currently possible. The standard practices used to design and construct USACE, ecosystem restoration projects include a degree of resilience because features are typically designed to accommodate a wide range of flow conditions. The primary exception is a lack of levee overtopping resiliency, that was not evaluated during feasibility due to a minimum levee elevation constraint specified by the Green Island Levee and Drainage District Agreement. Thus, there is some likelihood that climate change induced increases in flow will undermine Project features, primarily due to the risk of increased levee failure. It is likely that increasing temperatures will place added stress on the ecosystem in the future. Ecosystem restoration standard design practices have been generated based on lessons learned from successful projects constructed between 1981 and 2015. The majority of these standards are listed in the 2012 *Upper Mississippi River Restoration (UMRR) Design Handbook* (USACE 2012).

Even though USACE ecosystem restoration projects provide some inherent resiliency due to the diversity of habitat objectives, opportunities to further incorporate innovative, resilient features into the final design without incurring a significant change in cost will be pursued during design. Added resilience will be targeted at ensuring Project features can withstand higher flows (and higher water surface elevations) and longer periods of inundation. Additionally, opportunities to reduce potential future, climate change driven water quality impacts from rising water temperatures should be considered. A potential mechanism by which to accomplish this is to explore the development of an adaptive management plan, whereby if conditions are observed to be changing in the future, certain Project features can be designed to accommodate flexibility for modification in response to changing future conditions. Adaptive management actions must take place within ten years following Project construction.

Table A-22. Residual Risk Due to Climate Change

Project Objective	Supporting Feature	Trigger	Hazard	Harm	Qualitative Likelihood	Justification of Likelihood Rating
Restore historic hydrologic cycle/Restore aquatic & floodplain forest vegetation.	Bi-directional Pump Station & Brown's Lake Gate	Increased discharge and water surface elevation (WSEL)	Future flood volume, frequency, and duration may increase	This will increase the risk of levee overtopping and failure, limiting the ability to meet the WLMP necessary for restoring vegetation	Likely	There is strong evidence in observed and projected data for the study area that annual maximum of mean monthly streamflow, flood volumes and precipitation will increase.
	Conveyance Dredging	Increased discharge, WSEL, fetch length, sedimentation and loss of depth	Future flood volume, frequency, and duration may increase along with sediment loading and deposition and wind-wave erosion	This will increase the risk of levee overtopping and failure, introducing additional sediment. Sustained flooding causing increased fetch lengths and sediment resuspension resulting in deposition in channels and reduced conveyance.	Unlikely	A loss of conveyance would take repeated long duration flood events and the ability to conduct periodic drawdowns in the future would help to consolidate sediments working to mitigate any reduced depth and conveyance
	Plantings & Timber Stand Improvement	Increased discharge and WSEL	Future flood volume, frequency, and duration may increase	This will increase the risk of levee overtopping and failure. Sustained flooding during the growing season can result in mortality in trees of all ages	Likely	There is strong evidence in observed and projected data for the study area that flood volumes and precipitation will increase. Levee failure is a significant risk associate with flooding and while pumping can help to increase rate of interior dewatering, levee failure prohibits this use of pumps
		Increased drought intensity/duration	Drought intensity/duration may increase	Sustained drought can impact new plantings and prevents forest regeneration	Unlikely	The likelihood of increased drought index may improve the success of periodic drawdowns without the use of pumps. Currently water levels are too high to achieve periodic drawdowns without the use of pumps.

Project Objective	Supporting Feature	Trigger	Hazard	Harm	Qualitative Likelihood	Justification of Likelihood Rating
Restore bathymetric diversity/Restore aquatic & fish habitat	Habitat Dredging	Increased discharge, WSEL, fetch length, sedimentation and loss of depth.	Future flood volume, frequency, and duration may increase	This will increase the risk of levee overtopping and failure, introducing additional sediment. Sustained flooding causing increased fetch length and wind-wave sediment resuspension can result in deposition within deep water reducing aquatic habitat.	Unlikely	A loss of deep water habitat would take repeated long duration flood events and the ability to conduct periodic drawdowns would help to consolidate sediments working to mitigate this loss of deep water.
		Increased air temperature	Increased surface water temperatures and decreased dissolved oxygen (DO)	Increased air temperatures present a risk for increased surface water temperatures that result in decreased DO	Unlikely	There is strong evidence in the literature, and observed and projected data that temperatures will increase. Habitat dredging in backwater areas will be designed to provide volumes that help to mitigate ambient temperature increases.
Restore topographic diversity/Improve sediment management	Berm Creation & Restoration	Increased discharge, WSEL, fetch length and wind-wave erosion.	Future flood volume, frequency, and duration may increase	This will extend the duration and extent of berm inundation, increasing fetch length and resulting in wind-wave erosion.	Unlikely	The berms are designed to perform under the typical WLMP and not under flood conditions. Submerged berms will provide reduced efficacy in limiting sediment resuspension, however sustained flooding is likely to result in erosion of the berms.
Improve sediment management	Sediment Trap	Increased extreme precipitation, erosion, and sediment delivery	Increased rainfall intensity can result in increased sediment deposition and sediment trap O&M		Unlikely	There is strong evidence in the literature, and observed and projected data that extreme precipitation events will increase. If the timing of these events occurs while agricultural fields are fallow, erosion is likely. Adapting O&M to accommodate increased sedimentation is an illustration of a resilient feature.

9. FURTHER ANALYSIS PLANNED

The following analyses will be completed or considered during post-TSP feasibility level of design (FLD) or during PED:

- Survey for the sub-impoundment 1 berm and the low reach along the southern boundary of Pool A will be obtained during design. It is assumed to show elevations well above the maximum managed water elevation (587.72' NAVD88) based on sponsor's management.
- Pump station evaluation
  - Ensure ability to directly pump to and from both Pool A & Pool B
  - Discussion with PDT regarding operational cost risk associated with electric pump station due to changing utility rate structure and strong consideration of portable pumps as designed for Keithsburg HREP
  - Pump configuration considerations to include (1) overall capacity needs; (2) flexibility in operation to provide resilience under uncertain future hydrology and (3) achieve desired slow drawdown and filling rates and the ability to adjust pumping rates as needed to adapt to changing conditions, i.e. rainfall.
- Siting the final location of the proposed Brown's Lake gate with structural superiority. The feasibility design is located at approx. Sta. 371+00 to 374+00, near previous levee breaches and the incipient overtopping location, thus levee materials and grading to optimize interior filling in this location should be evaluated during design to ensure resiliency of the structure.
- Interior filling and controlled overtopping evaluation.

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