Appendix D.2 – Hydrology & Hydraulics Investigations & Analyses
Jewell Watershed Dam Sites #1, #2, #3 and #5.
Supplemental Watershed plan and Environmental Assessment

January 10, 2023

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LIST OF ATTACHED PLATES

LIST OF ACRONYMS

1-D	One-Dimensional
2-D	Two-Dimensional
ARC	Antecedent Runoff Condition
cfs	Cubic Feet per Second
CL	Clay Low-Plasticity
CN	Curve Number
DM	Departmental Management
DEM	Digital Elevation Model
EM	Engineering Manual
ESRI	Environmental Systems Research Institute
fps	Feet per Second
FBH	Freeboard Hydrograph
FWOFI	Future Without Federal Investment
GIS	Geographic Information System
HEC-RAS	Hydrologic Engineering Center – River Analysis System
HSG	Hydrologic Soil Group
LiDAR	Light Detection and Ranging
ML	Silt Low-Plasticity
Na	Not Applicable
NAVD88	North American Vertical Datum 1988
NED	National Elevation Dataset
NHD	National Hydrography Dataset
NEH	National Engineering Handbook
NRCS	National Resources Conservation Service
NOAA	National Oceanic and Atmospheric Administration
NWIS	National Water Information System
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
PSH	Principal Spillway Hydrographs
RCC	Roller Compacted Concrete
Q	Runoff or Discharge
QRF	Quick Return Flow
S	Storage
SCS	Soil Conservation Service
ASH	Auxiliary Spillway Hydrograph
SM	Silty Sand
SP	Sand Poorly Graded
SW	Sand Well-Graded
Tc	Time of Concentration
TR	Technical Release
USDA	U.S. Department of Agriculture
USGS	U.S. Geologic Survey
WSEL	Water Surface Elevation
V	Velocity

1.0 INTRODUCTION

1.1 Project Background

The Jewell Brook Watershed Project is comprised of four flood control structures located in Ludlow, Vermont. The project was developed as identified in the Watershed Work Plan for Watershed Protection, Flood Prevention, and Recreation, Jewell Brook Watershed, dated April 1964. The four flood control structures were constructed between 1968 and 1972. with flood control as the primary purpose. The 1964 plan also identifies Dam Site #3 as a multi-purpose structure and includes Basic Recreational Facilities. The four flood control structures which comprise the Jewell Brook Watershed Project are the following:

- Jewel Brook Site No. 1 (High Hazard);
- Jewell Brook Site No. 2 (High Hazard);
- Jewell Brook Site No. 3 (High Hazard);
- Jewell Brook Site No. 5 (High Hazard).

Each of the four dams are located on their own respective tributaries and therefore function independently of each other. Each of the four dam tributaries discharges into Jewell Brook which flows northeast alongside Andover Street (VT RT. 100) before discharging into the Black River near the center of the Village of Ludlow. The dams are owned and operated by the Town of Ludlow who is the Project Sponsor.

Both the NRCS and State of Vermont classify all four dams as High Hazard (Class I). None of the dams currently meet NRCS dam safety performance standards which have been updated since their original design and construction. The State of Vermont dam safety regulations are currently under development (anticipated completion 2022) and for the time being the State of Vermont is defaulting to federal guidance from agencies such as the Federal Emergency Management Agency (FEMA), United States Army Corps of Engineers (USACE), & National Resources Conservation Service (NRCS) to establish dam safety criteria.

The Sponsor's objective is to continue to provide flood protection in an environmentally responsible and cost-effective manner.

1.2 Purpose

The purpose of this hydrologic and hydraulic analysis is to provide an assessment of the Jewell Brook flood control dams compared to the hydraulic criteria from NRCS and the State of Vermont Dam Safety Program. The analysis will gather information on current and future watershed characteristics such as precipitation and land cover. The dams will be evaluated to determine estimated sediment storage, and hydrologic capacity. Outflow from the dams and breach analysis will be routed through the downstream floodplain to generate possible impacts. The primary analyses are summarized below.

- Freeboard Hydrograph (FBH) for 6-hr and 24-hr precipitation durations,
- Auxiliary Spillway Hydrograph (ASH) for 6-hr and 24-hr precipitation durations,
- Rainfall and Runoff Principal Spillway Hydrographs (PSH),
- 10-, 25-, 50-, 100-, 200-, and 500-year storm event hydrographs for economic analysis,
- Hydrologic, Static, and Seismic Breaches,
- HEC-RAS 2-D Modeling of the downstream floodplain.

The hydrologic and hydraulic analyses were performed in accordance with NRCS requirements outlined in Earth Dams and Retarding pools - TR 210-60 (NRCS 2019), applicable sections of the NRCS National Engineering Handbook Part 630 – Hydrology, applicable sections of the NRCS National Engineering Handbook Part 628 – Dams. Hydrologic data, methods, assumptions and results are documented herein.

2.0 BASIC DATA

2.1 Mapping and Survey

Mapping, aerial photography, previous construction drawings, and ground-based data used in the hydrologic and hydraulic analyses are listed below:

- Jewell Brook Watershed Project –Dam Site No. 1 - As Built Drawings (SCS 1966),
- Jewell Brook Watershed Project –Dam Site o. 2 - As Built Drawings (SCS 1967),
- Jewell Brook Watershed Project –Dam Site No. 3 - As Built Drawings (SCS 1967),
- Jewell Brook Watershed Project – Dam Site No. 5 - As Built Drawings (SCS 1970),
- Bathymetry Survey (Dams 1, 2, 3, & 5) (DDK Survey 2020),
- Ground Survey (Dams 1, 2, 3, & 5) (DDK Survey 2019),
- Hydro-flattened Digital Elevation Model (0.7m) (VCGI 2013, 2016),
- Google Imagery (Google),
- NAIP Imagery (USDA, 2016),
- Southern Windsor County Regional Planning Commission Future Land Use Planning Areas (SWCRPC, Updated Dec. 2018, GIS data obtained from VCGI,
- Rutland Regional Planning Commission Future Land Use Planning Areas (RRPC, Updated June 2018, GIS data obtained from VCGI,
- Town & Village of Ludlow Municipal Plan (Ludlow, Oct. 2019),
- Andover Town Plan, VT Future Land Use Map (Andover, Sept. 2018),
- Mount Holly Town Plan (Mt. Holly, 2018).

2.2 Previous Watershed Studies

Previous watershed studies provided by NRCS include:

- Watershed Work Plan for Watershed Protection and Flood Prevention, Jewell Brook Watershed, Ludlow, Windsor County Vermont. (SCS 1979),
- Dam Assessment Report - Jewell Brook Dam #1 (McMillen Jacobs Associates, Oct 2015),
- Dam Assessment Report - Jewell Brook Dam #2 (McMillen Jacobs Associates, Oct 2015),
- Dam Assessment Report - Jewell Brook Dam #3 (McMillen Jacobs Associates, Oct 2015),
- Dam Assessment Report - Jewell Brook Dam #5 (McMillen Jacobs Associates, Oct 2015),

2.3 Stream Gaging Station Data

Jewell Brook flows into the Black River which is a tributary to the Connecticut River. The nearest downstream gauge is located on the Black River in North Springfield, VT (USGS 01153000 Black River at North Springfield, VT). Flood frequency analysis of the stream gauge data was not performed to compute peak discharges in the Back River for this study due to the gauge being located downstream of the North Springfield Dam. The North Springfield Dam is a flood control dam owned and operated by the US Army Corps of Engineers.

2.4 Existing Dams

Key information about each dam is outlined in Table 1 through Table 4. All elevations listed in the tables correspond to the NAVD88 ft vertical datum unless specified otherwise.

Table 1: Jewell Brook Site No. 1 Existing Geometry

Principal Spillway (Two Stage Riser, Circular Conduit)		
	Elevation	**Description**
Low Level Orifice (Ungated)	1584.77	1.5 ft horizontal x 1 ft vertical rectangle
High Stage Weir (Ungated)	1605.17	15 ft sharp crested
Outlet Conduit	1568.7 (inlet) *	30 Dia. RCP, approx. 254 ft long
Auxiliary Spillway (Vegetated Channel)		
Control Section Elevation	1612.93 (surveyed average)	
Control Section Breadth	30 ft	
Bottom Width	250 ft	
Side Slopes	3 H: 1 V	
Valley Floor Elevation	1557.10	
Dam (Earthen Embankment)		
Structural Height	approx. 57.7 ft	
Dam Crest Elevation	1620.00 (Surveyed Average), 1618.25 (Surveyed Low Point)	
Crest Width	28 ft (varies)	
Crest Length	410 ft	
Upstream Slope	2.7 H: 1 V	
Downstream Slope	2.6 H: 1 V	
Saddle Dike Elevation	1618.75 (Surveyed Low Point)	

* Elevation not surveyed. Referenced from as-built drawings with approximate datum conversion from NGVD29 to NAVD88.

Table 2: Jewell Brook Site No. 2 Existing Geometry

Principal Spillway (Two Stage Riser, Circular Conduit)		
	Elevation	**Description**
Low Level Orifice (Ungated)	1531.51	1 ft. 1 in horizontal x 1 ft vertical rectangle
High Stage Weir (Ungated)	1558.73	15 ft sharp crested
Outlet Conduit	1522.2 (inlet) *	30” Dia. RCP
Auxiliary Spillway (Vegetated Channel)		
Control Section Elevation	1565.67 (surveyed average)	
Control Section Breadth	30 ft	
Bottom Width	300 ft	
Side Slopes	2.9 H: 1 V	
Valley Floor Elevation	1537.70	
Dam (Earthen Embankment)		
Structural Height	70.4 ft	
Dam Crest Elevation	1573.50 (surveyed average), 1572.58 (surveyed low point)	
Crest Width	22 ft	
Crest Length	1000 ft	
Upstream Slope	2.9 H: 1 V	
Downstream Slope	2.6 H: 1 V	

* Elevation not surveyed. Referenced from as-built drawings with approximate datum conversion from NGVD29 to NAVD88.
* Site No. 2 includes a gated 4” C.I. water supply pipe operated by a valve box in front of the principal spillway riser.

Table 3: Jewell Brook Site No. 3 Existing Geometry

Principal Spillway (Two Stage Riser, Circular Conduit)		
	Elevation	**Description**
Low Level Orifice (Ungated)	1229.47	1.5 ft horizontal x 1 ft vertical rectangle
High Stage Weir (Ungated)	1239.16	15 ft sharp crested
Outlet Conduit	1220.17 (inlet) *	30” Dia. RCP
Auxiliary Spillway (Vegetated Channel)		
Control Section Elevation	1243.03	
Control Section Breadth	30	
Bottom Width	200	
Side Slopes	3 H: 1 V	

Valley Floor Elevation	1214.2
Dam (Earthen Embankment)	
Structural Height	64 ft
Dam Crest Elevation	1251.80 (surveyed average), 1250.37 (surveyed low point)
Crest Width	23 ft
Crest Length	650 ft
Upstream Slope	3 H: 1 V
Downstream Slope	2.5 H: 1 V
Saddle Dike Crest Elevation	1250.37 (surveyed low point)

* Elevation not surveyed. Referenced from as-built drawings with approximate datum conversion from NGVD29 to NAVD88.

Table 4: Jewell Brook Site No. 5 Existing Geometry

Principal Spillway (Two Stage Riser, Circular Conduit)		
	Elevation	**Description**
Low Level Orifice (Ungated)	N/A	N/A
High Stage Weir (Ungated)	1445.90	15 ft sharp crested
Outlet Conduit	1412.90 (inlet) *	30" Dia. RCP
Auxiliary Spillway (Vegetated Channel)		
	Right	**Left**
Control Section Elevation	1489.68	1489.58
Control Section Breadth	30	30
Bottom Width	150	150
Side Slopes	2.5 H: 1 V	2.5 H: 1 V
Valley Floor Elevation	1467.00	1474.20
Dam (Earthen Embankment)		
Structural Height	112 ft	
Dam Crest Elevation	1496.80 (surveyed average), 1495.62 (surveyed low point)	
Crest Width	15 ft	
Crest Length	680 ft	
Upstream Slope	3 H: 1 V	
Downstream Slope	2.5 H: 1 V	

* Elevation not surveyed. Referenced from as-built drawings with approximate datum conversion from NGVD29 to NAVD88.

3.0 WATERSHED HYDROLOGIC ANALYSES

Hydrologic analysis of the Jewell Brook watershed was completed to collect data for hydraulic modeling of the flood control dams and downstream floodplain. The analysis included precipitation data, composite runoff curve number, time of concentration, base flow & quick return flow. In addition, information about the individual dams was collected such as estimated sedimentation, elevation-stage rating curve for the retarding pool.

3.1 Watershed Delineation

The watershed that flows to the Jewell Brook flood control dams is approximately 7.1 square miles. The overall watershed area of Jewell Brook at its confluence with the Black River is 9.4 square miles. Table 5 below depicts the watershed area captured by each individual dam.

Table 5: Jewell Brook Watershed Project Drainage Areas

Location	Drainage Area (sq. mi.)
Site No. 1	1.92
Site No. 2	1.94
Site No. 3	1.40
Site No. 4	1.83

Drainage areas were delineated using a digital elevation model (DEM) which created from a combination of 2013 & 2016 hydro-flattened DEMS obtained from the Vermont Center of Geographic Information (VCGI) and 2019/2020 DDK field survey. ESRI ArcGIS spatial analyst tools, ArcHydro tool, and HEC-GeoHMS tools were used to delineate the watershed within GIS software (ESRI ArcMap version 10.6.0). The final drainage area delineations were selected following review of the GIS software delineations with recent aerial imagery and topographic contours and applying engineering judgement. Plate H-1 depicts the watershed delineations for each dam. In general, the watersheds are steep and are primarily covered by forest. In the Site 3 watershed, there is some residential development and a portion of a commercial ski area.

3.2 Rainfall Distribution Inputs

Rainfall inputs for the FBH, ASH, PSH, and recurrence interval storm events were developed using the following guidance documents.

- NOAA Hydrometeorological Report No. 51 Probable Maximum Precipitation Estimates – United States East of the 105th Meridian (NOAA 1978).
- NOAA Hydrometeorological Report No. 52 Application of Probable Maximum Precipitation Estimates – United States East of the 105th Meridian (NOAA 1982).
- NOAA Atlas 14 Volume 10 (NOAA 2015, Revised 2019).
- Earth Dams and Reservoirs, TR 210-60 (NRCS 2019).
- National Engineering Handbook Part 630 Chapter 21 Design Hydrographs (NEH-630.21) (NRCS 2019).
- National Engineering Handbook Part 630 Chapter 4 Storm Rainfall Depth and Distribution (NEH-630.04) (NRCS 2019).

- Regional Estimation of Baseflow for the Conterminous United States by Hydrologic Landscape Regions (Journal of Hydrology, 2008).

3.2.1 Freeboard Hydrograph

Figure 2-2 of TR 210-60 defines the precipitation data for the freeboard hydrograph at a High Hazard classified dam to be the probable maximum precipitation (PMP). The PMP depths for the 6-and 24-hour duration events were obtained from HMR-51 and were used to create the 5-point rainfall distribution provided in NEH-630.2103.

The 5-point method is an NRCS approved 24-hour distribution in instances where local PMP studies are not available. Table 6 shows the PMP depths from HMR-51 and the development of the 5-point rainfall distribution for the 24-hour PMP storm event. The NRCS dimensionless design storm distribution for FBH design storms from NEH-630.2103 Figure 21-9 was applied to the 6-hour PMP event. All four dam share the same PMP rainfall depths and distributions. In accordance with NEH-630.2103 Figure 21-8, the areal reduction factor for drainages areas less than 10 square miles is 1.0 (no areal reduction).

Table 6: HMR-51 Rainfall Values

Storm Duration	HMR-51 PMP Rainfall Depth (in)
6-hr	23.70
12-hr	26.90
24-hr	29.10

Table 7: 24-hr NRCS-5pt Distribution of PMP Rainfall

Rainfall Duration (hours)	6-hour block	Incremental Depth (Inches)	Cumulative Depth (Inches)
0	---	---	---
6	a	1.10	1.10
12	b	23.70	24.80
18	c	3.20	28.00
24	d	1.10	29.10

3.2.2 Auxiliary Spillway Hydrograph

The ASH inputs were developed for the 6- and 24-hour events. The ASH rainfall input for a high hazard dam was computed using the following formula from TR 210-60:

$$P_{ASH} = P_{100} + 0.26(PMP - P_{100})$$

Where P_{100} is the X-hour, 100-year rainfall depth from NOAA Atlas 14 and PMP corresponds to the 24-hour general storm event from the HMR-51. The calculated rainfall inputs for the ASH are summarized in Table 8.

Table 8: ASH Rainfall Values

Storm Duration	Site 1 ASH Rainfall Depth (in)	Site 2 ASH Rainfall Depth (in)	Site 3 ASH Rainfall Depth (in)	Site 5 ASH Rainfall Depth (in)
6-hr	9.11	9.11	9.11	9.12
12-hr	10.87	10.89	10.91	10.92
24-hr	12.44	12.47	12.49	12.52

In accordance with NEH-630.2103 Figure 21-8, the areal reduction factor for drainages areas less than 10 square miles is 1.0 (no areal reduction). The dimensionless 6-hr storm distribution from NEH-630.21 Figure 21-9 was applied to the 6-hour ASH event. The 5-point distribution was applied to the 24-hour ASH event. The NOAA Atlas 14 rainfall values differed slightly between each site, which causes the ASH values from each drainage area also slightly differ.

3.2.3 Principal Spillway Hydrograph

The PSH was developed following NRCS procedures in NEH-630.21 as incorporated in the SITES computer program. Two scenarios are considered for the principal spillway hydrograph (rainfall & runoff). Rainfall inputs include the 100-year, 10-day event and the 100-year, 1-day (24-hour) event as referenced from NOAA Atlas 14. Runoff inputs include the 100-yr, 10-day event and the 100-yr, 1-day (24-hour) event as referenced from NEH 630.21 Figure 21-2 and Figure 21-3. Table 9 below summarizes the PSH rainfall and runoff inputs.

Table 9: PSH Distribution Inputs

	Site No. 1	Site No. 2	Site No. 3	Site No. 5
100-yr 10-day Rainfall Depth (in)	11.1	11.1	11.0	11.1
100-yr 1 day Rainfall Depth (in)	6.58	6.63	6.66	6.69
100-yr 10-day Runoff Depth (in)	8.70	8.70	8.70	8.70
Q1/Q10 Runoff Ratio	0.3	0.3	0.3	0.3
100-yr 1 day Runoff Depth (in)	2.61	2.61	2.61	2.61

In accordance with NEH-630.2102 there is no aerial reduction for PSH volume drainage areas less than 10 square miles. In addition to rainfall inputs the PSH requires baseflow and quick return flow. A general baseflow value based on a regional regression method outlined in *Regional Estimation of Base Flow for the Conterminous United States by Hydrologic Landscape Regions (Journal of Hydrology, 2008)* was calculated for each drainage area (**Table 22**).

3.2.4 Frequency Storm Events

The 2-, 5-, 10-, 25-, 50-, 100-, 200-, and 500-year recurrence interval average depths for a 24-hour duration were obtained from NOAA Atlas 14 for each dam (Table 10). Point precipitation values were taken at the centroid coordinates of each drainage area. The depth-duration values for each recurrence interval were smoothed in accordance with procedures described in Appendix 4B in NEH-630.04. The smoothed precipitation depths were then sorted to create a balanced cumulative rainfall distribution in accordance with procedures described in Appendix 4C of NEH-630.04. The calculations were performed using NRCS WinTR-20 v3.20 hydrologic & hydraulic modeling software, which incorporates both methods. WinTR-20 outputs a rainfall distribution that can be directly copied and pasted into NRCS SITES.

Table 10: 24-hr Frequency Storm Event Rainfall Depths

24-hr rainfall depth	Site No. 1	Site No. 2	Site No. 3	Site No. 5
2-yr	3.06	3.08	3.10	3.11
5-yr	3.81	3.83	3.85	3.87
10-yr	4.42	4.45	4.48	4.49
25-yr	5.27	5.31	5.34	5.36
50-yr	5.91	5.95	5.98	6.00
100-yr	6.58	6.63	6.66	6.69
200-yr	7.34	7.39	7.43	7.46
500-yr	8.44	8.49	8.54	8.57
1000-yr	9.34	9.40	9.46	9.49

3.3 Hydrologic Soil Groups

Hydrologic soils data was obtained using NRCS Web Soil Survey contains the most recent soil survey data for Windsor County Vermont (SSURGO Sept. 2019). A map of the soil units within the drainage areas of each of the four dams is shown in **Plate H-2**. The hydrologic soil groups (HSG) in the watershed are shown on **Plate H-3**. A breakdown of HSGs by percentage of watershed area is provided in Table 11 below.

Table 11: Jewell Brook Watershed Hydrologic Soil Group Classifications

HSG Classification	Site No. 1 Drainage Area	Site No. 2 Drainage Area	Site No. 3 Drainage Area	Site No. 5 Drainage Area
A	1.0%	2.8%	3.2%	1.5%
B	27.4%	17.6%	30.6%	17.0%
C	38.9%	66.4%	41.4%	68.0%
D	32.7%	13.2%	24.9%	13.6%

Soil surveys in this area contain dual HSGs (i.e. A/D, B/D, and C/D) as well as areas without HSG rating. Dual HSGs occur in areas where the water table is estimated to be within 24 inches of the surface; the first letter applies to the drained condition and the second to the undrained condition as noted in *NEH Part 630-Hydrology, Chapter 7, Hydrologic Soil Groups* (NEH 630.07) (NRCS 2009). During large rain events, the water table is likely to rise and with it the potential for runoff generation. In dual HSG units, the HSG that corresponds to a higher potential for runoff was chosen for computation of composite CNs. Areas without a HSG classification were conservatively assigned a HSG of D.

3.4 Land Cover

Land cover for the existing land use condition was based on reviewing recent aerial imagery in accordance with NEH Part 630 Chapters 8 & 9. Land cover for the future land use condition was created by modifying areas for future development based on the following three sources of information.

- Mt. Holly Future Land Use (Rutland Regional Planning Commission),
- Town/Village of Ludlow Future Land Use (South Windsor County Regional Planning Commission),
- Town of Andover Future Land Use (South Windsor County Regional Planning Commission),

3.5 Rainfall Losses

Rainfall losses were based on the NRCS runoff curve number (CN) method from *NEH Part 630 Chapter 10 – Estimation of Direct Runoff from Storm Rainfall* (NEH-630.10) (NRCS 2004b):

$$Q = (P-0.2S)^2/(P+0.8S) \quad \text{for } P > 0.2S \qquad \text{(Eq. 10-1)}$$
$$Q = 0 \quad \text{for } P \leq 0.2S$$

Where Q is the runoff (in), P is the total storm rainfall (in), S is the potential maximum watershed retention (in) after runoff begins, and 0.2S is an initial abstraction that includes interception losses, depression storage, and infiltration before runoff begins. The CN and S are related by:

$$CN = 1000/(10+S) \qquad \text{(Eq. 10-12)}$$

Watershed CNs were calculated as the area-weighted average of the individual CNs based on soil type, cover type, and cover condition. Base CNs for each soil/cover complex were determined using Tables 9-1 and 9-5 in *NEH Part 630 Chapter 9 – Hydrologic Soil Cover Complexes* (NEH-630.09) (NRCS 2004). Aerial photographs depict a well-vegetated watershed and the cover condition is considered to be "good" for most applicable cover types. The antecedent runoff condition (ARC) class II CNs were used for modeling all storm events.

3.5.1 Composite Curve Number – Existing Watershed Condition

The following steps were taken (for each of the four drainage areas part of the Jewell Brook Watershed Project) within ArcGIS,

- Existing land use categories were delineated based on visual inspection of recent aerial imagery and observations made during a site visit. Land use categories were converted to NEH-630.09 land cover types.

- NRCS SSURGO HSG values were prepared as described in Section 3.4 Land Cover.

- The SSURGO HSG shapefile (**Plate H-3**) and the existing land use delineation (**Plate H-4**) shapefiles were merged to create a new shapefile which contained all of the sub areas made up of each possible combination of HSG value and land use type with each dams' drainage area.

- HSG and land use complex combinations were used to determine curve numbers based on NEH- 630.09 Tables 9-1 & 9-5.

- Each dam's HSG and land cover combination shapefile was given a curve number attribute field which was populated with curve numbers for each sub area combination of HSG value and land use type using the ArcMap field calculator.

- The curve numbers for each sub area combination of HSG and land use type were than multiplied by the respective area in acres using the ArcMap field calculator.

- The sum of the products of "curve number" * "Area" was then divided by the total drainage area, resulting in a composite number for existing watershed condition at each dam.

Table 12: Jewell Brook Site No. 1 Existing Watershed Curve Number Determination

DDK Land Use Determination	NEH-630.09 Land Use Pairing	Hydrologic Condition of Land Use	Hydrologic Soil Group	Curve Number	Area (Acres)	Percent of Total Area
Forest (Good)	Woods	Good	A	30	2.33	0.2%
			B	55	289.60	23.5%
			C	70	441.74	35.9%
			D	77	329.84	26.8%
			None	100	0.00	0.0%
Forest (Poor)	Woods	Poor	A	45	0.00	0.0%
			B	66	0.00	0.0%
			C	77	1.33	0.1%
			D	83	6.57	0.5%
			None	100	0.00	0.0%
Field (Mowed)	Meadow-continuous grass, protected from grazing an generally mowed for hay	Good	A	30	6.42	0.5%
			B	58	4.73	0.4%
			C	71	0.00	0.0%
			D	78	20.02	1.6%
			None	100	0.21	0.0%
Residential	Residential	1 Acre Lot / 20% Impervious	A	51	2.49	0.2%
			B	68	37.78	3.1%
			C	79	32.87	2.7%
			D	84	35.66	2.9%
			None	100	0.00	0.0%
Road	Streets and Roads (paved with curbs, excluding right of way)	Good	A	98	0.58	0.0%
			B	98	4.65	0.4%
			C	98	3.31	0.3%
			D	98	5.40	0.4%
			None	100	0.00	0.0%
Open Water	-	-	A	100	0.64	0.1%
			B	100	0.41	0.0%
			C	100	0.00	0.0%
			D	100	0.02	0.0%
			None	100	4.41	0.4%
				Total Area (Acres)	**1231**	
				Weighed CN	**69.2**	

Table 13: Jewell Brook Site No. 2 Existing Watershed Curve Number Determination

DDK Land Use Determination	NEH-630.09 Land Use Pairing	Hydrologic Condition of Land Use	Hydrologic Soil Group	Curve Number	Area (Acres)	Percent of Total Area
Forest (Good)	Woods	Good	A	30	16.48	1.3%
			B	55	201.07	16.2%
			C	70	803.34	64.6%
			D	77	145.28	11.7%
			None	100	0.00	0.0%
Field (Mowed)	Meadow-continuous grass, protected from grazing an generally mowed for hay	Good	A	30	13.48	1.1%
			B	58	6.22	0.5%
			C	71	6.66	0.5%
			D	78	1.24	0.1%
			None	100	0.00	0.0%
Residential	Residential	1 Acre Lot / 20% Impervious	A	51	1.60	0.1%
			B	68	9.27	0.7%
			C	79	11.47	0.9%
			D	84	13.67	1.1%
			None	100	0.00	0.0%
Road	Streets and Roads (paved with curbs, excluding right of way)	Good	A	98	0.94	0.1%
			B	98	2.11	0.2%
			C	98	3.14	0.3%
			D	98	4.00	0.3%

			None	100	0.00	0.0%
Open Water	-	-	A	100	2.77	0.2%
			B	100	0.00	0.0%
			C	100	0.00	0.0%
			D	100	0.00	0.0%
			None	100	0.00	0.0%
				Total Area (Acres)	1243	
				Weighted CN	67.9	

Table 14: Jewell Brook Site No. 3 Existing Watershed Curve Number Determination

DDK Land Use Determination	NEH-630.09 Land Use Pairing	Hydrologic Condition of Land Use	Hydrologic Soil Group	Curve Number	Area (Acres)	Percent of Total Area
Forest (Good)	Woods	Good	A	30	11.05	1.2%
			B	55	214.40	23.9%
			C	70	221.29	24.7%
			D	77	133.08	14.8%
			None	100	0.44	0.0%
Forest (Poor)	Woods	Poor	A	45	0.00	0.0%
			B	66	5.37	0.6%
			C	77	13.93	1.6%
			D	83	0.35	0.0%
			None	100	0.00	0.0%
Field (Mowed)	Meadow-continuous grass, protected from grazing an generally mowed for hay	Good	A	30	7.24	0.8%
			B	58	5.10	0.6%
			C	71	10.76	1.2%
			D	78	18.04	2.0%
			None	100	0.72	0.1%
Ski Slope	Meadow-continuous grass, protected from grazing an generally mowed for hay	Good	A	30	0.00	0.0%
			B	58	31.08	3.5%
			C	71	3.99	0.4%
			D	78	35.24	3.9%
			None	100	0.00	0.0%
Brush	Brush-brush-forbs-grass mixture with brush the major element	Poor	A	48	0.36	0.0%
			B	67	0.00	0.0%
			C	77	0.00	0.0%
			D	83	0.00	0.0%
			None	100	0.32	0.0%
Residential	Residential	1 Acre Lot / 20% Impervious	A	51	9.94	1.1%
			B	68	18.26	2.0%
			C	79	120.90	13.5%
			D	84	22.65	2.5%
			None	100	2.40	0.3%
Open Water	-	-	A	100	0.00	0.0%
			B	100	0.00	0.0%
			C	100	0.00	0.0%
			D	100	0.00	0.0%
			None	100	9.58	1.1%
				Total Area (Acres)	**896**	
				Weighed CN	**68.5**	

Table 15: Jewell Brook Site No. 5 Existing Watershed Curve Number Determination

DDK Land Use Determination	NEH-630.09 Land Use Pairing	Hydrologic Condition of Land Use	Hydrologic Soil Group	Curve Number	Area (Acres)	Percent of Total Area
Forest (Good)	Woods	Good	A	30	8.63	0.7%
			B	55	180.44	15.3%
			C	70	740.35	63.0%
			D	77	147.22	12.5%
			None	100	0.00	0.0%
Forest (Poor)	Woods	Poor	A	45	0.00	0.0%
			B	66	0.00	0.0%
			C	77	58.39	5.0%
			D	83	0.00	0.0%
			None	100	0.00	0.0%
Ski Slope / Forest	Woods	Fair	A	36	0.00	0.0%
			B	60	19.21	1.6%
			C	73	0.00	0.0%
			D	79	10.66	0.9%
			None	100	0.00	0.0%
Field (Mowed)	Meadow-continuous grass, protected from grazing an generally mowed for hay	Good	A	30	6.97	0.6%
			B	58	0.00	0.0%
			C	71	0.00	0.0%
			D	78	1.93	0.2%
			None	100	0.00	0.0%
Brush	Brush-brush-forbs-grass mixture with brush the major element	Poor	A	48	0.58	0.0%
			B	67	0.00	0.0%
			C	77	0.23	0.0%
			D	83	0.00	0.0%
			None	100	0.00	0.0%
Open Water	-	-	A	100	0.93	0.1%
			B	100	0.00	0.0%
			C	100	0.00	0.0%
			D	100	0.00	0.0%
			None	100	0.00	0.0%
				Total Area (Acres)	**1176**	
				Weighed CN	**68.3**	

3.5.2 Composite Curve Number – Future Watershed Conditions

The future conditions watershed curve numbers for the Jewell Brook Watershed Project were determined using the same process used to determine the existing conditions curve number described above except in areas where to local governing bodies had identified anticipated future land use changes; the future land use overwrote the previously determined existing conditions land use.

- The future land use types were converted to NEH-630.09 land use types.
- The future land use shapefile is depicted on **Plate H-5**.
- The future conditions curve numbers based on future land use and SSURGO HSG combination.
- The future conditions land use delineations indicated a minor to moderate amount of planned development primarily weighted towards an anticipated increase in residential development along the base / lower elevations of the mountain. The majority of the drainage areas remain as resource / conserved forest.

Table 16: Jewell Brook Site No. 1 Future Watershed Curve Number Determination

Future Land Use	NEH-630.09 Land Use Pairing	Hydrologic Condition of Land Use	Hydrologic Soil Group	Curve Number	Area (Acres)	Percent of Total Area
Resource	Woods	Good	A	30	0.34	0.0%
			B	55	136.30	11.1%
			C	70	361.71	29.4%
			D	77	275.31	22.4%
			None	100	0.41	0.0%
Political Boundary Data Gap	Woods	Good	A	30	0.00	0.0%
			B	55	1.44	0.1%
			C	70	3.75	0.3%
			D	77	0.00	0.0%
			None	100	0.00	0.0%
Rural	Residential	1 Acre Lot / 20% Impervious	A	51	12.10	1.0%
			B	68	199.66	16.2%
			C	79	114.21	9.3%
			D	84	121.56	9.9%
			None	100	4.21	0.3%
				Total Area (Acres)	**1231**	
				Weighted CN	**71.7**	

Table 17: Jewell Brook Site No. 2 Future Watershed Curve Number Determination

Future Land Use	NEH-630.09 Land Use Pairing	Hydrologic Condition of Land Use	Hydrologic Soil Group	Curve Number	Area (Acres)	Percent of Total Area
Resource	Woods	Good	A	30	0.01	0.0%
			B	55	46.17	3.7%
			C	70	699.74	56.3%
			D	77	97.87	7.9%
			None	100	0.00	0.0%
Conserved & Open	Woods	Good	A	30	0.00	0.0%
			B	55	28.62	2.3%
			C	70	7.07	0.6%
			D	77	0.00	0.0%
			None	100	0.00	0.0%
Political Boundary Data Gap	Woods	Good	A	30	0.00	0.0%
			B	55	10.55	0.8%
			C	70	19.13	1.5%
			D	77	0.00	0.0%
			None	100	0.00	0.0%
Rural	Residential	1 Acre Lot / 20% Impervious	A	51	35.26	2.8%
			B	68	133.54	10.7%
			C	79	98.57	7.9%
			D	84	66.32	5.3%
			None	100	0.00	0.0%
				Total Area (Acres)	**1243**	
				Weighted CN	**70.2**	

Table 18: Jewell Brook Site No. 3 Future Watershed Curve Number Determination

Future Land Use	NEH-630.09 Land Use Pairing	Hydrologic Condition of Land Use	Hydrologic Soil Group	Curve Number	Area (Acres)	Percent of Total Area
Resource	Woods	Good	A	30	1.57	0.2%
			B	55	137.32	15.3%
			C	70	121.05	13.5%
			D	77	59.96	6.7%
			None	100	8.83	1.0%
Conserved & Open	Woods	Good	A	30	0.00	0.0%
			B	55	2.05	0.2%
			C	70	0.00	0.0%
			D	77	44.07	4.9%
			None	100	0.00	0.0%
Political Boundary Data Gap	Woods	Good	A	30	0.00	0.0%
			B	55	5.24	0.6%
			C	70	0.00	0.0%
			D	77	23.13	2.6%
			None	100	0.00	0.0%
Rural	Residential	1 Acre Lot / 20% Impervious	A	51	13.99	1.6%
			B	68	129.08	14.4%
			C	79	249.37	27.8%
			D	84	81.79	9.1%
			None	100	1.82	0.2%
Medium Density Neighborhood	Residential	1/2 Acre Lot / 25% Impervious	A	54	12.67	1.4%
			B	70	0.49	0.1%
			C	80	0.44	0.0%
			D	85	2.68	0.3%
			None	100	0.09	0.0%
				Total Area (Acres)	**896**	
				Weighted CN	**71.9**	

Table 19: Jewell Brook Site No. 5 Future Watershed Curve Number Determination

Future Land Use	NEH-630.09 Land Use Pairing	Hydrologic Condition of Land Use	Hydrologic Soil Group	Curve Number	Area (Acres)	Percent of Total Area
Resource	Woods	Good	A	30	2.53	0.2%
			B	55	102.92	8.8%
			C	70	463.22	39.4%
			D	77	32.60	2.8%
			None	100	0.00	0.0%
Political Boundary Data Gap	Woods	Good	A	30	0.00	0.0%
			B	55	24.75	2.1%
			C	70	55.84	4.8%
			D	77	3.87	0.3%
			None	100	0.00	0.0%
Conserved & Open	Woods	Good	A	30	0.00	0.0%
			B	55	59.83	5.1%
			C	70	175.22	14.9%
			D	77	43.64	3.7%
			None	100	0.00	0.0%
Rural	Residential	1 Acre Lot / 20% Impervious	A	51	14.58	1.2%
			B	68	11.43	1.0%
			C	79	112.20	9.5%
			D	84	72.94	6.2%
			None	100	0.00	0.0%
				Total Area (Acres)	**1176**	
				Weighted CN	**69.5**	

3.6 Time of Concentration

The time of concentration for the watershed was calculated using the velocity method procedure in the *NEH Part 630, Chapter 15 – Time of Concentration* (NEH-630.15) (NRCS 2010). The velocity method assumes that time of concentration is the sum of travel times for segments along the hydraulically most distant flow path also referred to as the longest watercourse. The segment types used in the velocity method are sheet flow, shallow concentrated flow, and open channel flow computed using the following are the equations.

Velocity Method:

$$T_c = Tt_1 + Tt_2 + Tt_3 + \ldots Tt_n \quad \text{(Eq. 15–7)}$$

Where:

T_c = time of concentration, h

Tt_n = travel time of a segment n, h

n = number of segments comprising the total hydraulic length

Sheet flow:

$$Tt = \frac{0.007(nl)^{0.8}}{(P_2)^{0.5} s^{0.4}} \quad \text{(Eq. 15–8)}$$

Where:

T_t = travel time, h

n = Manning's roughness coefficient (table 15–1)

l = sheet flow length, ft

P_2 = 2-year, 24-hour rainfall, in

Shallow Concentrated Flow:

$$V=20.328(s)^{0.5}$$

Where:

V = average velocity, ft/s

S = slope of the hydraulic grade line (channel slope), ft./ft.
(Flow type: Upland Gully Table 15-3)

Open Channel Flow:

$$V = \frac{1.49(r)^{2/3}s^{1/2}}{n} \quad \text{(Eq. 15-10)}$$

Where:

V = average velocity, ft./s
r = hydraulic radius, ft
s = slope of the hydraulic grade line (channel slope), ft/ft
n = Manning's *n* value for open channel flow

Lag Time:

$$T_l = 0.6 * T_C \quad \text{(Eq. 15-3)}$$

Where:

T_l = Lag Time
T_c = Time of Concentration

The longest watercourse (within the delineated drainage areas) for each of the four Jewell Brook Dams was defined using the LIDAR DEM and ArcMap Spatial Analyst tools (ArcHydro toolbar version 10.6.0 & HEC-GeoHMS toolbar version 10.6.0) with refinement for false flow paths caused by culverts and diversions. The longest watercourse was divided into three main segments. Guidance in NEH-630.15 recommends assuming sheet flow occurs for a maximum distance of 100 ft starting from the highest point in the watershed before transitioning to shallow concentrated flow. NEH-630.16 recommends after the depth of flow reaches 0.5 ft that it be represented as open channel flow. The determination for the transition from shallow concentrated flow to open channel flow was determined via inspection of the LIDAR DEM to gauge channel width and depth, and then creating a 2D HEC-RAS model of the remaining flow path to gauge flow characteristics and geometry.

Flow characteristics and geometry from the 2D HEC-RAS model were used in the open channel flow calculations. Manning's n value for the time of concentration flow path calculations were based on site observations made from nearby streams in the Jewell Brook watershed (typ. 0.04 - 0.05). **Plate H-6** shows the longest watercourse and the three different flow segment types used for each dam. All slopes used in the time of concentration calculations were obtained by stationing the longest watercourse shapefile and extracting elevation points from the LIDAR DEM. The time of concentration and travel times for each segment are listed in

Table 20 below.

Table 20: Jewell Brook Watershed Project Time of Concentration (ToC)

	Site No. 1	Site No. 2	Site No.3	Site No. 5
Sheet Flow				
Length (ft)	104	99.9	100	120.3
ToC (hr.)	0.04	0.05	0.14	0.06
Shallow Concentrated Flow				
Length (ft)	1,099	3,551	2,670	2,607
ToC (hr.)	0.03	0.12	0.08	0.09
Open Channel Flow				
Length (ft)	19,310	10,838	9,891	15,975
ToC (hr.)	0.83	0.43	0.28	0.30
Totals				
Length (ft)	20,513	14,489	12,661	18,702
ToC (hr.)	0.90	0.60	0.49	0.80
Lag Time (hr)	0.54	0.36	0.29	0.48
Starting Elevation (NAVD88 ft)	2,925.40	3,042.00	3,171.90	3,191.10
Ending Elevation (NAVD88 ft)	1,584.80	1,531.51	1,229.47	1,445.9

3.7 Climatic Index

The climatic index was applied to account for quick return flow (QRF) during the PSH in accordance with NEH-630.21 (NRCS 2019).

$$C_i = \frac{100P_a}{T_a^2} \quad \text{(Eq. 21-1)}$$

Where:

C_i = climatic index
P_a = average annual precipitation in inches
T_a = average annual temperature in deg. F

The climatic index used for the Jewell Brook Watershed Project was calculated using the above equation (Eq. 21-1). The average annual precipitation was obtained from PRISM Climate Group 1981-2010 precipitation normal. The average annual temperature was obtained from the PRISM Climate Group 1981-2010 normal. The average annual precipitation and average annual temperature were sampled from the basin centroids of each drainage area.

Table 21: Jewell Brook Watershed Project Dam's Climatic Index

	Climatic Index
Site No. 1	2.953
Site No. 2	2.885
Site No. 3	2.802
Site No. 5	3.040

3.8 Base flow

In accordance with TR 210-60 Part 6 "Routing of Principal Spillway Hydrographs", principal spillway hydrograph routing must start no lower than the water surface elevation produced by base flow if 1) the base flow elevation is significantly higher (0.5 feet) than lowest principal spillway inlet or 2) if base flow occupies more than 10 percent of the floodwater storage capacity.

Stream gaging suitable for direct calculation base flow above the retarding pool of the dams was not available. Base flow was determined using regression methods from Journal of Hydrology (2008) "Regional Estimation of base flow for the conterminous United States by hydrologic landscape regions". The baseflow regression equation computes the average annual baseflow value based upon an average annual precipitation and the characteristics of the USGS hydrologic landscape region. The

Jewell Brook Watershed Project dams all lie within the same hydrologic landscape region and therefore shared the same average base flow value of 2.03 cfs per square mile (csm). The average baseflow value is then multiplied by each sites individual drainage area to compute a baseflow discharge in cfs. Table 22 depicts the base flow for each of the four dams.

Table 22: Jewell Brook Watershed Baseflow

	Baseflow (cfs)
Site No. 1	3.9
Site No. 2	3.9
Site No. 3	2.8
Site No. 5	3.7

3.9 Sedimentation

The dams must be designed to account for aerated and submerged sediment over the dams' implementation period and design life. Aerated sediment is sediment that is accumulated within the reservoirs flood storage pool (above the normal pool elevation), and submerged sediment is sediment that is accumulated within the pond of the reservoir (below normal pool elevation).

3.9.1 Submerged & Aerated Sedimentation Rates

Submerged and aerated sedimentation volumes and rates were determined by comparison storage volumes reported on historical documents to current storage volumes. Utilizing the original dam design calculations and the As-Built Plans, flood storage and permanent pool storage volumes were established between known points on the dam, such as the auxiliary spillway crest and the invert of the low-level orifice.

Current storage volumes were generated from bathymetric and topographic data from the DDK surveys, along with topographic data from the State of Vermont 2016 Middle Connecticut River Basin LIDAR model, were blended to create a reservoir Digital Elevation Model (DEM). Within ArcGIS, the DEM was used to generate surface area measurements and volumetric measurements at various elevations within the reservoir.

Comparison of the storage volumes was used to calculate the volume of accumulated submerged sediment (the difference between volumes below the low-level orifice) and the volume of aerated sediment (the difference between the volumes above the low-level orifice and the auxiliary spillway or top of dam).

During the bathymetric survey in May 2020, a dual-frequency sonar probe was used to collect water depth / bottom of pool elevations as well as depth to a restrictive layer. This generated an elevation model for the restrictive layer, presumed to be native pond bottom, which was used to estimate the volume of submerged sediment between the two elevation models. To verify the depths of submerged sediment between the elevation models, measurements were collected during a site visit of the submerged sediment, as well as, a visual review of the stream confluence with the pond where aerated sediment may have accumulated.

Sedimentation rates were calculated by dividing the accumulated sediment volume by the number of years since installation. The following sections elaborate the rationale for determining submerged and aerated sediment volumes and rates for each dam.

3.9.1.1 Jewell Brook Site No. 1 Sediment Accumulation Analysis

Submerged Sediment:

- The original design volume computations (February 1966) indicate a total volume of water, from the retarding pool bottom to the low-level outlet invert of 17.2-acre-feet.

- The current conditions (May 2020) stage-storage curve yields a computed total volume of water, from the retarding pool bottom to the low-level outlet invert, of 12.3-acre-feet.

- The accumulated submerged sediment (below the invert of the low-level outlet) is computed to be 4.9 acre-feet (17.2 ac-ft. – 12.3 ac-ft.).

- The computed average annual rate of accumulated submerged sediment, since the dam was constructed in 1966 (54 years) is 0.09 ac-ft. / year (rounded).

Aerated Sediment:

- The storage volume value indicated on the 1966 As-Built drawings indicate a retarding storage volume between the low-level outlet invert to the auxiliary spillway crest of 410.8 acre-feet.

- The current conditions (May 2020) stage-storage curve yields a computed total volume of water, from the low-level outlet invert to the auxiliary spillway crest of 404.4 acre-feet,
- The loss of storage volume at the auxiliary spillway crest between the May 2020 data and the 1966 data is 6.4 acre-ft. (410.8 ac-ft. – 404.4 ac-ft.).

- Utilizing the loss of storage volume at the auxiliary spillway crest, the computed average annual rate of accumulated aerated sediment, since the dam was constructed in 1967 (53 years) is 0.12 ac-ft. / year (rounded).

Table 23: Jewell Brook Site No. 1 Sediment Data

	Historic Information		2020 Data	
	Surface Area (acres)	Water Storage (ac-ft.)	Surface Area (acres)	Water Storage (ac-ft.)
Low Level Orifice Invert	3.7	17.2	3.1	12.3
Auxiliary Spillway Crest*	29.2	410.8	30.9	404.4

* Does not include permanent pool storage

3.9.1.2 Jewell Brook Site No. 2 Sediment Accumulation Analysis

Submerged Sediments:

- The original design volume computations (December 1966) indicate a total volume of water, from the retarding pool bottom to the low-level outlet invert of 5.1-acre feet.

- The current conditions (May 2020) stage-storage curve yields a computed total volume of water, from the retarding pool bottom to the low-level outlet invert is 2.3-acre-feet.

- The accumulated submerged sediment (below the invert of the low-level outlet) is computed to be 2.8 acre-feet (5.1 ac-ft. – 2.3 ac-ft.).

- The computed average annual rate of accumulated submerged sediment, since the dam was constructed in 1967 (53 years) is 0.05 ac-ft. / year (rounded),

Aerated Sediment:

- The storage volume value indicated on the 1967 As-Built drawings indicate a storage volume between the low-level outlet invert to the riser weir crest of 180.5 acre-feet and between the low-level outlet invert to the auxiliary spillway crest of 278.2 acre-feet.

- The current conditions (May 2020) stage-storage curve yields a computed retarding volume of water, from the low-level outlet invert to the riser weir crest of 154.7 acre-feet and from the low-level outlet invert to the auxiliary spillway crest of 261.4 acre-feet.

- The loss of storage volume at the riser weir crest between the May 2020 data and the 1967 data is 25.8 acre-ft (180.5 ac-ft. – 154.7 ac-ft.). The loss of storage volume at the auxiliary spillway crest is 16.8 acre-feet (278.2 ac-ft. – 261.4 ac-ft.).

- Utilizing the loss of storage volume at the auxiliary spillway crest, the computed average annual rate of accumulated aerated sediment, since the dam was constructed in 1967 (53 years) is 0.32 ac-ft. / year (rounded).

Table 24: Jewell Brook Site No. 2 Sediment Data

	Historic Information		2020 Data	
	Surface Area (acres)	Water Storage (ac-ft.)	Surface Area (acres)	Water Storage (ac-ft.)
Low Level Orifice Invert	1.95	15.1	1.0	2.3
Riser Weir Crest*	13.4	180.5	12.1	154.7
Auxiliary Spillway Crest*	17.9	278.2	18.9	261.4

* Does not include permanent pool storage

3.9.1.3 Jewell Brook Site No. 3 Sediment Accumulation Analysis

Submerged Sediments:

- The original design volume computations (November 1967) indicated a total volume including submerged sediment storage, from the bottom of the permanent pool to the low-level outlet invert of 23.0-acre-feet.

- The current conditions (May 2020) stage-storage curve yields a computed total volume of water, from the retarding pool bottom to the low-level orifice invert (El. 1229.47-ft) of 79.2-acre-feet.

- The original computations were not clear on the volume of sediment estimated to accumulate to produce a total volume of 23.0 acre-feet. The difference between the current volume to the design volume is -56.3 acre-feet (23.0 ac-ft – 70.2 ac-ft). There are no records of the pond being dredged or if modifications were made to the low-level orifice to increase the permanent pool storage.

- During site reconnaissance sediment deposition was observed at Dam Site No. 3. The original design volume computations (November 1967) indicates that the surface area at the invert of the orifice was 9.8 acres. The 2020 topographic and bathymetric information yields a surface area of 9.43 acres at the invert of the orifice.

- The May 2020 bathymetric elevation model was compared to the restrictive soil layer elevation model. The volume between the two elevation models was calculated to be 7.30 ac-ft. Adding the difference to the existing conditions total water volume, the assumed water volume at construction is 86.5 ac-ft. (79.2 ac-ft. + 7.30 ac-ft.).

- This sediment volume estimate (7.30 ac-ft) appears to be consistent other sediment depositions in similar watersheds. The characteristics of the upstream channel geomorphology of Site No. 3 is similar to Site No. 1. The average slope of Site No.3 upstream channel is roughly double that of Site No. 1 which may account for some of the difference in sediment deposition.

- Using the sonar derived sediment deposition, the computed average annual rate of accumulated submerged sediment, since the dam was constructed in 1968 (52 years) is 0.14 ac-ft. / year (rounded).

Aerated Sediment:

- The storage volume value listed on the 1968 As-Built drawings indicate a retarding storage volume between the low-level outlet invert to the auxiliary spillway crest of 336.3 acre-feet.

- The As-Built drawings list a value that does not match the November 1967 design volume computations which show a total storage volume of 209.6 ac-ft. at the crest of the auxiliary spillway. The calculations also indicate that there is a storage volume of 135.5 acre-feet between the lower-level orifice to the riser crest.

- The current conditions (May 2020) stage-storage curve yields a computed retarding volume of water, from the low-level outlet invert to the riser weir crest of 126.1 acre-feet and from the low-level outlet invert to the auxiliary spillway crest of 191.3 acre-feet.

- To determine the accumulated aerated sediment, both the As-Built drawings and the original design calculations were used,
  - The difference between the As-Built drawings and the current for storage from the low-level orifice to the auxiliary spillway crest would be 145 ac-ft. This would equate to a sedimentation rate of 2.79 ac-ft./year,
  - The difference between the original design calculations and the current for storage from the low-level orifice to the auxiliary spillway crest would be 18.3 ac-ft. This would equate to a sedimentation rate of 0.35 ac-ft./year,
  - The difference between the original design calculations and the current for storage from the low-level orifice to the riser crest would be 9.4 ac-ft. This would equate to a sedimentation rate of 0.18 ac-ft./year,

- During the site visit, aerated sediment was observed in the vicinity of the confluence of the brook and the pond. The area was surveyed to be approximately 7.0 acres and when compared between the two elevation models there was approximately 3.9 ac-ft. difference.

- The aerated sedimentation rate between the low-level orifice to the riser crest is closest to the submerged sedimentation rate. The other computed sedimentation rates are 2 to 20 times greater than the submerged sedimentation rate. This analysis will use an aerated sedimentation rate of 0.18 ac-ft./year.

Table 25: Jewell Brook Site No. 3 Sediment Data

	Historic Information		2020 Data	
	Surface Area (acres)	Water Storage (ac-ft.)	Surface Area (acres)	Water Storage (ac-ft.)
Low Level Orifice Invert	9.8	23.0	9.3	79.0
Riser Weir Crest	14.5	135.5	16.0	126.1
Auxiliary Spillway Crest*	17.1	209.6	18.4	191.3

* Does not include permanent pool storage

3.9.1.4 Jewell Brook Site No. 5 Sediment Accumulation Analysis

Jewell Brook Site 5 principal spillway structure was not constructed with a low level orifice but instead with a pond drain regulated by a gate valve. It is not clear if this gate valve was left open or closed. Sediment deposition has buried the pond drain intake structure. Based on the 1967 design calculations, the flood storage volume value reported on the 1969 As-Built plans is the total available storage which includes the anticipated submerged and aerated sediment storage from the riser weir crest to the auxiliary spillway crest. For the purposes of estimating submerged sediments, DDK used the volume between the pond bottom and the top of the riser weir crest.

Submerged Sediments:

- The original design volume computations (December 1967) (beginning at the pond drain invert) indicates that there is approximately 9.65-ac-ft. of storage between the pond drain and the riser weir crest.

- The current conditions (May 2020) stage-storage curve yields a computed total volume of water, from the retarding pool bottom to the riser weir crest (1445.90-ft) of 0.86-acre-feet.

- Local reports claim a significant portion of the change in volume occurred during Tropical Storm Irene (large rain event) eroded logging and other exposed areas within the watershed.

- The accumulated submerged sediment (below the invert of the low-level outlet) is computed to be 8.79 acre-feet (9.65 ac-ft. – 0.9 ac-ft.).

- The computed average annual rate of accumulated submerged sediment, since the dam was constructed in 1969 (51 years) is 0.17 ac-ft. / year (rounded).

Aerated Sediment:

- The original design volume computations (December 1967) (beginning at the pond drain invert) indicates that there is approximately 205.8 ac-ft. (215.4 ac-ft. – 9.65 ac-ft.) from the riser weir crest to the auxiliary spillways' crest.

- Site No. 5 has two auxiliary spillways. For the purpose of estimating the retarding volume the lower crest elevation of the auxiliary spillways will be used (El. 1489.25-ft). The current conditions (May 2020) stage-storage curve yields a computed total volume of water, from the riser weir crest to the auxiliary spillway crest of 206.0 acre-feet. (206.9 ac-ft. – 0.86 ac-ft.).

- There is no computed loss of storage volume at the at the auxiliary spillway crest between the May 2020 data and the 1967 data.

- Site reconnaissance observed that there was aerated sediment in the vicinity of the normal pool. The approximate area of observed aerated sediment was 0.71 acres with an approximate volume of 3.9 ac-ft of earthen material between the two elevation models.

- Using this the estimated aerated volume of 3.9 ac-ft. of aerated sediment since constructed in 1969 (51 years), the computed average annual rate of accumulated submerged sediment is 0.08 ac-ft. / year (rounded).

Table 26: Jewell Brook Site No. 5 Sediment Data

	Historic Information		2020 Data	
	Surface Area (acres)	Water Storage (ac-ft.)	Surface Area (acres)	Water Storage (ac-ft.)
Riser Weir Crest*	1.3	30	0.6	0.9
Auxiliary Spillway Crest*	8.2	205.4	11.5	206.0
Top of Dam	13.2	283.1	13.9	288.6

* Does not include permanent pool storage

3.9.2 Project Evaluation Timeline

Using the submerged sedimentation rates and the remaining storage below the normal pool elevation; the remaining life of the dam (before the low-level outlets would be submerged by sediment) was estimated. The remaining service life of each dam is listed in Table 27 below. NRCS PR & G requires that the design life for the rehabilitated dams to be a minimum of 50-years and a maximum of 100-years, and also that the sediment analysis accounts for the additional time to design, implement and construct the dam. For the purposes of the Jewell Brook sediment analysis, it was assumed that it would take an additional 10 years to design, implement, and construct the preferred rehabilitation alternative of each dam.

Table 27: Remaining Service Life & Selected Rehabilitation Design Life

	Submerged Sedimentation Rate (acre-ft/yr)	Remaining Storage Below Normal Pool Elevation (acre-ft)	Remaining Service Life (years)	Selected Design Life for Rehabilitation + Time for Implementation / Construction (years)	Dredging Required (Y/N)
Site No. 1	0.09	12.3	136	100 + 10 = 110	No
Site No. 2	0.05	2.3	46	50 +10 = 60	Yes
Site No. 3	0.14	79.2	565	100 + 10 = 110	No
Site No. 5	0.17	0.86	5	50 + 10 = 60	Yes

To accommodate additional service life for Site No. 2 and No. 5 dredging of the pond areas is included the preferred alternative. Site No. 2 includes dredging of 5,000 CY (3.1 ac-ft.) of material to expose the pond drain intake structure. The addition storage from dredging along with the remaining storage will provide 108-years of service life (2.3 ac-ft. + 3.1 ac-ft./ 0.05ac-ft/year = 108 years). Due to site constraints Site No. 5 can only be dredged to expose the pond drain structure which was estimated to be 14,200 CY (8.8 ac-ft.). This additional space along with the remaining storage will provide 57-years (0.86 ac-ft. + 8.8 ac-ft. / 0.17 ac-ft./year = 56 years). To match the remaining dam sites' minimum 100-year service life, the Operation and Maintenance costs reflect an additional dredging of 14,500 CY ((110 years – 57 years) x 0.17 ac-ft./year = 9.0 ac-ft.).

With the above additional improvements, the dam sites will have a design life of 100 years with and additional 10 year to implement. At the end of the 110-years, Site No. 2 and No. 5 are projected to require additional dredging dependent upon actual sediment accumulation. Site No. 1 is projected to have an additional 2.4 ac-ft. of storage or an additional 20 years. Site No. 3 is projected to have an additional 62.8 ac-ft. of storage.

3.9.3 Projected Storage Loss

Aerated sediment accumulation over the 110-year period is expected to impact the flood retarding storage. This loss is estimated in Table 28 below. The total volume of accumulated aerated sediment was evenly distributed across the entire existing elevation storage rating curves to generate future elevation storage rating curves for the future watershed condition SITES models. The results of the storage reduction can be seen in 3.10 Storage below.

Table 28: Projected Normal Pool Storage Loss over Selected Rehabilitation Design Life

	Design Life + Implementation (years)	Aerated Sedimentation Rate (acre-ft./yr.)	Projected Aerated Sediment Accumulation (acre-ft.)	2020 Current Flood Retarding Storage (acre-ft.)	Future Flood Retarding Storage (acre-ft.)
Site No. 1	110	0.12	13.2	404.4	391.2
Site No. 2	110	0.32	35.2	261.4	226.2
Site No. 3	110	0.18	19.8	191.3	171.5
Site No. 5	110	0.08	8.8	206.0	197.2

3.10 Storage

The elevation-storage relationships for the Jewell Brook Watershed dams were calculated using the ArcGIS Multi-Volumes tool developed by USGS. The ArcGIS Multi-Volumes tool will compute storage volumes and surface area at a specified interval of the users choosing. This tool was used to calculate volumes from the existing conditions surface which included existing conditions bathymetry within the pond. Storage values were calculated from the lowest bathymetry elevation to an elevation higher than the top of dam in 0.1 ft. elevation increments.

For the purposes of the hydraulic analysis performed using NRCS SITES the storage values below the normal pool elevations of each dam were removed as they provide no flood retarding benefit. SITES limits the number of storage elevation values to 20. The future storage values were obtained by equally distributing the lost storage from aerated sediment as calculated in 3.9.3 Projected Storage Loss above across the entire elevation range via a percent reduction.

Table 29 through Table 32 below represent the storage-elevation curves used in the SITES analysis.

Table 29: Jewell Brook Site No. 1 Storage Values

Elevation (NAVD88 ft.)	Existing Storage (acre-ft)	Future Storage (acre-ft)
1584.8 (PS Low)	0.0	0
1590.0	24.2	21.8
1595.0	64.6	59.8
1600.0	125.5	118.4
1605.2 (PS High)	213.2	203.6
1607.0	249.5	239.1
1608.0	273.5	262.6
1609.0	296.0	284.6
1610.0	322.6	310.8
1611.0	349.2	336.9
1612.0	377.3	364.5
1612.9 (Aux Crest)	404.4	391.2
1614.0	439.6	425.9
1615.0	473.2	459.0
1616.0	508.3	493.6
1617.0	544.7	529.6
1618.0	582.9	567.3
1619.0	622.5	606.4
1620.0	663.7	647.2
1625.0	890.8	871.9

Table 30: Jewell Brook Site No. 2 Storage Values

Elevation (NAVD88 ft)	Existing Storage (acre-ft)	Future Storage (acre-ft)
1531.5 (PS Low)	0.0	0
1535.0	5.3	1.7
1540.0	18.4	9.7
1545.0	39.3	25.4
1550.0	70.7	51.7
1555.0	113.8	89.6
1558.7 (PS High)	154.7	126.7
1560.0	171.0	141.7
1565.0	248.4	213.9
1565.7 (Aux Crest)	261.4	226.2
1567.0	286.7	250.2
1568.0	307.2	269.6
1569.0	328.6	290.0
1570.0	350.9	311.3
1571.0	374.0	333.3
1572.0	398.1	356.4
1573.0	422.9	380.2
1574.0	448.8	405.1
1575.0	475.4	430.6
1579.0	591.1	542.2

Table 31: Jewell Brook Site No. 3 Storage Values

Elevation (NAVD88 ft.)	Existing Storage (acre-ft)	Future Storage (acre-ft)
1229.47 (PS Low)	0.0	0
1232.0	26.1	22.4
1236.0	77.8	68.3
1237.0	92.2	81.2
1238.0	107.3	94.8
1239.16 (PS High)	126.1	112.0
1240.0	139.1	123.7
1241.0	155.9	139.1
1242.0	173.3	155.0
1243.03 (Aux Crest)	191.3	171.5
1244.0	209.9	188.7
1245.0	229.2	206.5
1246.0	249.6	225.5
1247.0	270.7	245.1
1248.0	292.4	265.3
1249.0	314.7	286.2
1250.0	337.6	307.6
1251.0	361.1	329.7
1252.0	385.3	352.4
1256.0	489.8	451.1

Table 32: Jewell Brook Site No. 5 Storage Values

Elevation (NAVD88 ft.)	Existing Storage (acre-ft)	Future Storage (acre-ft)
1445.9 (PS Low)	0.0	0
1451.0	6.0	5.0
1456.0	15.6	13.6
1461.0	28.9	25.9
1466.0	45.9	41.9
1471.0	67.1	62.0
1476.0	93.0	86.9
1481.0	126.1	119.0
1486.0	168.4	160.3
1489.58 (Aux Crest)	206.0	197.2
1490.0	210.7	201.8
1491.0	222.5	213.4
1493.0	247.2	237.7
1494.0	260.1	250.4
1495.0	273.2	263.3
1496.23	289.5	279.4
1497.0	300.7	290.4
1498.0	315.0	304.5
1500.0	344.5	333.6
1505.0	422.6	410.7

4.0 HYDRAULIC ANALYSES

Hydraulic analysis of the Jewell Brook watershed was prepared using NRCS (2007) SITES version 2005.1.8 hydrologic & hydraulic modeling software and the US Army Corps of Engineers (USACE) HEC-RAS versions 5.0.7 & 6.0. Hydraulic evaluation of the dams was completed utilizing the SITES program and the collected hydrologic parameters. Key components of the dams such as the principal spillway or auxiliary spillway were derived from survey data or from the As-Built plans. The geotechnical and geologic investigations provided information related to the auxiliary spillway soil properties. Hydrographs from the SITES models were used to perform the downstream flood routings within HEC-RAS.

4.1 DAM HYDRAULIC MODELING

4.1.1 Existing Principal Spillway

4.1.1.1 Geometry

The principal spillway geometry information was entered into the SITES model based on the 2019/2020 DDK survey and dimensions provided on the As-Built drawings for each dam. Each of the four dams all had a principal spillway structure that could be directly modeled using SITES without the need for a custom discharge rating curve.

4.1.1.2 Principal Spillway Hydrograph

Rainfall-runoff modeling for the PSH was developed in accordance with the procedures described in NEH-630.2102 (NRCS 2019) as incorporated into the SITES computer program.

4.1.1.3 PSH Drawdown Analysis

TR 210-60 requires the principal spillway to have hydraulic capacity to empty the retarding pool within 10 days following the maximum water surface elevation during the PSH storm. This requirement is considered to be met if 15% or less of the maximum volume of retarding storage remains within 10 days. The entire design inflow hydrograph including baseflow, quick return flow, and rainfall runoff must be considered in determining the evacuation time of the retarding storage.

If this criterion is not met the dam must be analyzed with a starting water surface elevation corresponding to the water surface elevation remaining after 10 days. The SITES program determined that the required starting water surface elevation for all four dams to be above their respective low-level orifice/normal pool elevations for both the existing dam/existing sediment/existing curve number and the existing dam/future sediment/future curve number scenarios. The starting water surface elevations computed by the PSH analyses are listed in the table below. Further investigation of the existing dam principal spillways revealed that the low-level orifices on the principal spillway structures were small relative to the baseflow/quick return flow values which was a primary factor in why the dams were unable to meet the 10-day drawdown criteria.

Table 33: PSH Starting Water Surface Elevation

	Low Level Orifice Elevation (NAVD88 FT)	High Stage Crest (NAVD 88 FT)	Current Conditions PSH Starting Water Surface Elevation (NAVD88 FT)	Depth of Water above Low-Level Orifice Invert (feet)	Future Conditions PSH Starting Water Surface Elevation (NAVD88 FT)	Depth of Water above Low-Level Orifice Invert (feet)
Site No. 1	1,584.77	1,605.50	1,600.32	15.52	1,600.29	15.49
Site No. 2	1,531.51	1,558.71	1,555.84	24.34	1,555.73	24.23
Site No. 3	1,229.47	1,229.47	1,236.34	6.87	1,236.22	6.75
Site No. 5*	n/a	1,445.90	1,446.52	0.62	1,446.52	0.62

* Site No. 5 was not designed with a low-level orifice outlet. High stage weir crest elevation listed instead.

4.1.1.4 PSH Auxiliary Spillway Crest Analysis

The NRCS design approach for a vegetated earth or armored auxiliary spillways requires the crest elevation of the auxiliary spillway to contain the maximum water-surface elevation for the PSH (TR 210-60). The maximum elevation of the PSH is controlled by the starting elevation of the reservoir pool, the inflow of the PSH, and the hydraulic performance of the principal spillway.

Table 34 below summarizes the current auxiliary spillway control section compared to the PSH resulting control section for both the existing watershed curve number and the future watershed curve number plus future sediment scenarios. This comparison is to determine if the dam is meeting the maximum frequency of use for an earth auxiliary spillway type outlined in TR 210-60. In addition, the anticipated Vermont Dam Safety requirement is that the auxiliary spillway should not activate in the 100-year flood. The resulting peak water surface during the 100-year flood are included to compare.

Table 34: PSH Auxiliary Spillway Control Section Results

	Existing Auxiliary Spillway Control Section (El.)	Existing Conditions PSH Resulting Auxiliary Spillway Control Section (El.)	Existing Conditions 100-year Flood Peak Water Surface	Future Conditions PSH Resulting Auxiliary Spillway Control Section (El.)	Future Conditions 100-year Flood Peak Water Surface
Site No. 1	1,612.93	1,613.06	1610.09	1,613.88	1611.05
Site No. 2	1,565.67	1,571.63	1566.25	1,572.86	1566.55
Site No. 3	1,243.03	1,247.70	1243.33	1249.32	1243.85
Site No. 5	1,489.58 (Left) & 1,489.68 (Right)	1,489.28	1484.31	1,490.86	1486.13

*This table is a comparison of Existing Watershed Conditions and Future Watershed Conditions at the existing dam and does not reflect improvements proposed under the preferred alternatives.

4.1.2 **Existing Auxiliary Spillway**

The elevation-discharge rating for the auxiliary spillway is generally calculated within SITES using the direct-step water surface profile calculation for supercritical flows downstream of the spillway crest and a standard-step backwater calculation upstream of the crest. Inputs to the calculation include the auxiliary spillway crest elevation, crest length, side-slope, and the longitudinal profile. The profile for each dam was obtained 2019/2020 ground survey. The profile stationing matches the auxiliary spillway stationing depicted on the DDK existing conditions plans.

4.1.2.1 Auxiliary Spillway Hydrograph

In accordance with NRCS requirements in TR 210-60, auxiliary spillway stability analyses were performed using the 24-hour and 6-hour ASH. The results of hydrologic modeling are presented in Table 35 below.

Table 35: Auxiliary Spillway Hydrograph Peak Inflow

	Future Conditions 24-hr ASH Peak Inflow (cfs)	Future Conditions 6-hr ASH Peak Inflow (cfs)
Site No. 1	1,675.9	3,736.6
Site No. 2	1,689.1	4,445.9
Site No. 3	1,238.5	3,626.2
Site No. 5	1,584.9	3,820.8

4.1.2.2 *Spillway Material Definition*

Parameters pertaining to the condition of the vegetal cover were assigned to the auxiliary spillways on the basis of visual inspection. The parameters characterize the ability of the cover to hold soils particles together and resist the shear forces of flows over the spillway. These parameters are summarized below:

- Vegetal Retardance Curve Index………………………4.19
- Vegetal Cover Factor………………………………...…0.5
- Maintenance Code……………………….………….2.0
- Potential Root Depth…………………………….….0.5-ft
- Depth of Topsoil……………………………………..…0.4-ft

In addition to defining vegetal cover within the auxiliary spillway, SITES requires that the user enter underlying soils information in order for the program to evaluate the potential for erosion and head cutting (a potential dam failure mechanism). DDK performed a geotechnical investigation at each dam which included collecting a series of borings in the auxiliary spillway. Soils information entered into the SITES model included plasticity index, dry density, head cut index, percent clay, representative diameter, and profile station and elevation. Table 36 below presents a list of materials found within the auxiliary spillway.

Table 36: Auxiliary Spillway Material Layers

	Site No. 1	Site No. 2	Site No. 3	Site No. 5 (Left)	Site No. 5 (Right)
Material 1	Fill (SM/ML)	Glacial Till (SM)	Fill (SM)	Glacial Till (SM)	Glacial Till (SM)
Material 2	Glaciofluvial (SM)	Glacial Till (SM/ML)	Glacial Till (GM)	Glacial Till (SM)	Glacial Till (SM)
Material 3	Lacustrine (ML)	Glacial Till (ML)	Glacial Till (ML)	Glacial Till (ML)	N/A
Material 4	Glaciofluvial (SM)	Glacial Till (SM)	Schist	N/A	N/A
Material 5	Silty Gravel (GM)	N/A	N/A	N/A	N/A

4.1.2.3 SITES Integrity Analysis

NRCS integrity analyses for the Jewell Brook Watershed Dams were conducted in accordance with NRCS methods in NEH Part 628, Chapter 51 - Earth Spillway Erosion Model (NEH-628.51) (NRCS 1997) as incorporated in the SITES model. The integrity analysis is based on the auxiliary spillways ability to pass the design flood events FBH 24-hr, FBH 6-hr, ASH 24-hr, and ASH 6-hr without head cutting occurring through the auxiliary control section (resulting in dam failure).

The auxiliary spillways of all four dams do not meet current NRCS integrity criteria during both the existing and future watershed condition scenarios. The control sections at each dam are breached during both the FBH 24-hr and FBH 6-hr events. Site No. 1 also breaches during the ASH 24-hr event. None of the dams' breach during the recurrence interval storm events however, significant portions of the auxiliary spillways are eroded.

4.1.2.4 SITES Stability Analysis

NRCS stability analyses for the Jewell Brook Watershed Dams were conducted in accordance with NRCS methods in NEH Part 628, Chapter 51 - Earth Spillway Erosion Model (NEH-628.51) (NRCS 1997) as incorporated in the SITES model. The stability analysis is based on the auxiliary spillways ability to pass the ASH flood events without the vegetal cover surface of the spillway failing.

The auxiliary spillways of all four dams do not meet current NRCS stability criteria. The vegetal cover failures occur very soon after the auxiliary spillway is activated in both the existing and future watershed condition. The rehabilitation alternatives for the dam will need to address this through modification of the auxiliary spillway geometries or armoring the spillways.

4.1.3 Existing Dam Crest

4.1.3.1 Freeboard Hydrograph

In accordance with NRCS requirements in TR 210-60, the integrity analyses for the existing auxiliary spillway were performed for the 6- and 24-hour FBHs with existing and future watershed conditions. The results of hydrologic modeling are presented in Table 37 below.

Table 37: Freeboard Hydrograph Peak Inflow

	Future Conditions 24-hr FBH Peak Inflow (cfs)	Future Conditions 6-hr FBH Peak Inflow (cfs)
Site No. 1	4,766.6	13,329.7
Site No. 2	4,823.8	16,088.0
Site No. 3	3,492.7	12,650.0
Site No. 5	4,522.1	14,070.9

4.1.3.2 Overtopping / Freeboard Analysis

NRCS TR 210-60 requires that the dams are able to pass the FBH 24-hr, FBH 6-hr, and all other design events without the dam overtopping (no minimum freeboard requirement).

Table 38 below depicts the maximum water surfaces experienced by the dam during the largest of the design events during the existing dam existing watershed curve number scenario, and the existing dam future watershed curve number & sediment scenario. VT Dam Safety rules are currently under development for anticipated implementation in 2022. Based on conversations with the Chief VT Dam Safety Engineer the design rules will likely require a high hazard dam to pass the PMP storm with a minimum of 18-inches of freeboard.

The controlling storm which produces the maximum water surface elevation for all four of the dams is the FBH 6-hr. The FBH 6-hr maximum surface elevation is below the surveyed average crest elevation for both existing and future conditions at all the dams. Settled and/or low points in the dam crest that may be subject to overtopping, will be filled in and graded in a suitable manner to provide a uniform top of dam crest. All four dams meet the VT Dam Safety freeboard requirement of one-foot of freeboard during the IDF.

Table 38: Existing Top of Dam - FBH 6-hr Results

	Existing Top of Dam (El.) (Surveyed Average) (NAVD88 FT)	Existing Top of Dam (El.) (Surveyed Low Point) (NAVD88 FT)	Saddle Dike (El.) (Surveyed Low Point) (NAVD88 FT)	Existing Conditions 6-hr FBH Maximum Water Surface (El.)	Future Conditions FBH Maximum Water Surface (El.)
Site No. 1	1,620.00	1,618.25	1,618.75	1,618.99	1,619.12
Site No. 2	1,573.50	1,572.58	N/A	1,572.14	1,572.26
Site No. 3	1,251.80	1,250.37	1,250.37	1,249.88	1250.07
Site No. 5	1,496.80	1,495.62	N/A	1,495.71	1495.76

*This table is a comparison of Existing Watershed Conditions and Future Watershed Conditions at the existing dam and does not reflect improvements proposed under the preferred alternatives.

4.1.4 Frequency Storm Events

Storm events with recurrence intervals of 2-, 5-, 10, 25-, 50-, 100-, 200-, and 500-yrs were modeled in SITES for each dam. The frequency event hydrographs for the future watershed conditions are presented in Table 39 below.

Table 39: Future CN Frequency Storm Event Peak Inflows

	Site No. 1 Future CN (cfs)	Site No. 2 Future CN (cfs)	Site No. 3 Future CN (cfs)	Site No. 5 Future CN (cfs)
2-yr	385.7	451.1	422	367.9
5-yr	649.1	778.7	700.1	646.3
10-yr	884.6	1,076.6	958.6	895.3
25-yr	1,237.8	1,532.3	1,335.3	1,279.3
50-yr	1,511.4	1,879.0	1,621.5	1,576.5
100-yr	1,811.4	2,263.0	1,930.5	1,898.8
200-yr	2,147.8	2,705.6	2,299.6	2,274.2
500-yr	2,637.9	3,352.7	2,820.6	2,763.8

4.2 DOWNSTREAM FLOOD MODELING

Downstream flood routing analysis was performed for the existing dam future watershed condition for the following frequency (recurrence interval) storm events; 500-yr, 200-yr, 100-yr, 50-yr, 25-yr, and 10-yr. These flood routings are performed to evaluate the performance of the dam in reference to its ability to mitigate flood damages.

4.2.1 Analyzed Scenarios

Beyond Dam Site #1, Jewell Brook flows approximately 3 miles prior to the confluence with the Black River. Impacts downstream of the dams are not limited within the Jewell Brook Watershed but extend downstream along the Black River. To accommodate this, a separate model of only the Black River was created to provide a basis of additional impacts beyond the Black River flooding extent. The following scenarios were modeled as part of the downstream flood routing analysis.

1. All four dams in place:
    a. selected rehabilitation alternative for each dam;
    b. future watershed curve number;
    c. future sediment storage values;
    d. unsteady outflow hydrograph from SITES model of each dam;
    e. additional Jewell Brook Watershed drainage area not captured by the flood control dams represented via unsteady flow hydrograph from approximate SITES model (lag method time of concentration, assumes average of the four dams curve number);

    f. steady peak discharge (FEMA) in Black River . As described in Section 4.3.1 of the Plan-EA, two (2) Black River discharge conditions were modeled in HEC-RAS, including 1) coincident peaks (Jewell and Black watershed both modeled with Q100 steady state peak floes) and 2) a 25-year event in the Black River occurring when the Jewell Watershed was at a 100-year peak flow.
    g. and additional drainage area from Black River tributaries represented with USGS StreamStats steady peak discharges.
2. All four dams decommissioned:
    a. inflow hydrographs from the SITES mode of each dam (no flood attenuation);
    b. future watershed curve number;
    c. additional Jewell Brook Watershed drainage area not captured by the flood control dams represented via unsteady flow hydrograph from approximate SITES model (lag method time of concentration, assumes average of the four dams curve number);
    d. steady peak discharge (FEMA) in Black River; As described in Section 4.3.1 of the Plan-EA, two (2) Black River discharge conditions were modeled in HEC-RAS, including 1) coincident peaks (Jewell and Black watershed both modeled with Q100 steady state peak floes) and 2) a 25-year event in the Black River occurring when the Jewell Watershed was at a 100-year peak flow.
    e. additional drainage area from Black River tributaries represented with USGS StreamStats steady peak discharges.
3. No flow from Jewell Brook watershed with steady FEMA peak discharge in Black River:
    a. 0 cfs contribution from Jewell Brook Watershed;
    b. steady peak discharge (FEMA) in Black River;
    c. and additional drainage area from Black River tributaries represented with USGS StreamStats steady peak discharges.

4.2.2 Two-Dimensional HEC-RAS Modeling

Two-dimensional (2-D) hydraulic modeling of the floodplain downstream of the Jewell Brook Watershed Project Flood Control Dams was performed using US Army Corps of Engineers (USACE) HEC-RAS version 6.0. The 2-D hydraulic model uses an implicit finite volume solution scheme and a computational mesh to solve unsteady-state flow routing equations. The mesh is made of polygons or cells, with up to eight sides. Every cell face functions as a cross section and the area inside of the cells functions as a storage area with a rating curve based upon the underlying terrain. 2-D modeling is better suited for modeling the movement of water through complex terrain, such as heavily urbanized areas, than 1-D modeling.

4.2.3 Model Terrain (Elevation Data)

The Vermont Center of Geographic Information (VCGI) Open Data Portal 2013 & 2016 hydro-flattened LIDAR derived DEM's were utilized in combination with DDK 2019/2020 survey surfaces to represent the terrain surface in the 2-D model. The DEM contains values for elevation in 0.7 square meter cells providing a high level of elevation detail. HEC-RAS also uses the terrain as a reference for recording results parameters, such as depth and velocity, over the course of a modeling run.

4.2.4 HEC-RAS Geometry

HEC-RAS 2-D geometry consists of a computational mesh, overland Manning's n-values, connection structures, and boundary conditions. A single model geometry was used to represent the downstream flooding scenarios. A series of plan files were generated to reference the geometry file and carry out runs with different flow condition files by altering boundary conditions. Each separate flow run had its own plan name and file.

4.2.4.1 Computational Mesh

The HEC-RAS 2-D computational mesh uses cells to capture the underlying terrain by building an elevation-volume relationship instead of assigning an average elevation value for the entire cell. Each cell face is assigned elevation versus: wetted perimeter, area, and roughness relationships. The computational mesh created for the floodplain modeling in this study consists of approximately 190,000 cells. The mesh was developed by first generating an unstructured, uniform 40-foot x 40-foot square mesh. The mesh was then refined using break lines and refinement regions to give more definition in areas where the slope of the water surface varies rapidly and where the terrain acts as a barrier to flow. The determination of cell sizes was made based on the need to capture changes in Manning's n-value with minimal averaging and to achieve Courant numbers within tolerance for the shallow water equations. The smallest cell size within the model is 15 ft. x 15 ft.

4.2.4.2 Manning's n-Values

Manning's n-values were defined in the model using the 2016 National Land Cover Database (NLCD) raster obtained from the Multi-Resolution Land Characteristics Consortium (MRLCC). The NLCD raster is made up of 30 m x 30 m cells. In areas where a more detailed cell resolution was desired or where it was determined that the NLCD raster was not accurately representing the current land cover manual modifications were made by drawing in override regions. In general the land cover raster was found to accurately represent land cover observed on recent aerial imagery and during site visits. Modifications were made primarily in and immediately around the stream channels of Jewell Brook and the Black River. The Manning's n-value used in the HEC-RAS model for the Jewell Brook varied from 0.04 to 0.05. The Manning's n-value used to represent the stream channel roughness of the Black River varied from 0.04 to 0.05. Table 40 below depicts the Manning's n-value pairings that were applied to the NLCD raster land cover which align with values found in the HEC-RAS hydraulic reference manual (USACE 2016).

Table 40: 2-D HEC-RAS Model Manning's n-values

NLCD Land Cover	Manning's N-Value
Agricultural, Cultivated Crops	0.035
Agricultural, Pasture/Hay	0.03
Developed, High Density	0.15
Developed, Low Density	0.1
Developed, Medium Density	0.08
Developed, Open Space	0.04
Open Water	0.04
Undeveloped, Barren Land	0.025
Undeveloped, Deciduous Forest	0.16
Undeveloped, Evergreen Forest	0.16
Undeveloped, Grassland	0.035
Undeveloped, Mixed Forest	0.16
Undeveloped, Shrub/Scrub	0.1
Wetlands, Forested	0.12
Wetlands, Non-Forested	0.07

4.2.4.3 Hydraulic Structures within the Downstream Flood Routings

To account for the presence of bridge and culverts in the downstream floodplain the DEM used to represent the model terrain was manually edited to remove bridge decks and roadways to allow for the passage of water to occur between the opening width of the hydraulic structure. While this does not allow for the representation of flow impedance due to bridge decks it is deemed an appropriate representation given the project scope, size of the hydraulic model, and the purpose of the analysis (providing a reasonable basis for evaluating flood benefits provided by the flood control dams).

4.2.5 **Boundary Conditions**

Unsteady flow hydrographs from SITES were utilized as an inflow hydrograph boundary condition with the HEC-RAS model for flows emanating from the Jewell Brook watershed. A flow hydrograph boundary condition for the Black River was defined upstream of the confluence of the Jewell Brook. This hydrograph held a steady peak discharge for its full duration as referenced from the FEMA Flood Insurance Report for Windsor County Vermont (2007). Additional flow hydrograph boundary conditions were defined along the Black River for noticeable tributary streams with drainage areas greater than 0.2 square miles utilizing USGS StreamStats peak discharge data. The downstream boundary condition of the model was located just above the upstream end of the North Springfield Dam.

For each of the Jewell Brook frequency (recurrence interval) storm events analyzed (500-yr, 200-yr, 100-yr, 50-yr, 25-yr, and 10-yr,) an equivalent recurrence interval discharge was modeled in the Black River (i.e. if the 100-yr Jewell Brook flow was analyzed, then the downstream routing was performed with the 100-yr peak discharge occurring within the Black River as well).

4.2.6 **Downstream Flood Routing Results**

Results from HEC-RAS primarily come in the form of GIS raster surfaces and shapefiles. These files include maximum depth, maximum velocity, maximum water surface elevation, and the maximum inundation boundary. The 500-yr, 200-yr, 100-yr, 50-yr, 25-yr, 10-yr events for the modeled scenarios were exported from HEC-RAS to ArcGIS to generate inundation mapping. The maximum depth, velocity, and water surface elevation rasters contain the maximum respective value for each terrain cell that occurred at any point in the model run. The inundation boundary shapefile encompasses the entire inundation area (i.e. any cell that recorded any depth of water). These files were used to create inundation maps shown in **Appendix C**. They were also used in estimating maximum flood depths and maximum velocities affecting property and infrastructure in the economic analysis (Appendix D4).

The all four dams decommissioned scenario (scenario 2) has a noticeably larger inundation extent than the with all four rehabilitated dams' scenario (scenario 1) from the upper end of the Jewell Brook until the confluence with the Black River. After the confluence with the Black River the difference between the with-dams and without-dams scenarios becomes harder to notice but is still apparent. The results from the Black River only scenario (scenario 3) show that a significant number of structures are vulnerable to flood damages regardless of whether or not flood contributions are made from Jewell Brook due to flooding from the Black River.

5.0 BREACH ANALYSIS

Breach analyses are performed to help understand the potential consequences of dam failure and verify hazard classification to the dam. This analysis also assists with determining the scope needed to lower the hazard classification as a flood-proofing alternative. All four dams are currently classified by NRCS and the State of Vermont dam safety program as High Hazard. The analysis, including the downstream flood routing of the failure waves were performed utilizing the SITES model to develop and route the storm hydrographs through the reservoirs, and the HEC-RAS 2-D version 5.0.7 to develop and route the breach discharge hydrographs downstream.

Two separate HEC-RAS models were utilized to perform the breach analysis. The first model was used to generate the breach discharge hydrograph at each dam and were configured to only include the dams and a minimum distance downstream of each dam to allow the water to discharge through the breach opening with a representative energy slope. This model was utilized to adjust empirical Froehlich breach parameters to create a site-specific outflow hydrograph that had a resulting peak discharge that matched the values computed by the NRCS 210-60 peak breach discharge equations described in section 5.2 below.

The second HEC-RAS model was utilized to route the beach hydrographs through the downstream floodplain to gauge the flood impacts and generate inundation mapping (outflow from the breach model was input as an inflow hydrograph to the downstream flood routing model).

5.1 Breach Scenarios

A static breach failure was conducted for each existing condition dam and routed downstream with a 100-yr flood base flow. The breach analysis and associated flood routing for each dam assumes that the three other dam sites do not breach and function as designed.

The SITES model was used to generate and route the 100-year storm event through each reservoir and the breach was initiated at the peak reservoir water level. The Q100 peak water levels ranged from within several inches to about 2-ft of the existing auxiliary spillway crest elevation.

The potential mode of failure for each dam included an internal erosion (piping) for the static breach. HEC-RAS model breach parameters were calculated using the Froehlich dam breach equations (1995) and then were iteratively adjusted within the model to generate an outflow hydrograph meeting TR-210-60 criteria. All breach hydrographs had a peak discharge equal to or greater than the minimum computed value by the TR-210-60 equations and less than the maximum value computed by these equations.

As indicated in the table below, the computed peak breach outflow varies from 22,372 to 53,235 cfs for Dam sites 1-5, respectively. The estimated maximum breach failure wave height varies from 34.5-ft to 48.8-ft (Sites 1-5).

A sensitivity analysis of the breach conditions (hydrologic, static and seismic breach) was completed, and determined the flooding from the static breach generated greater downstream damages compared to flooding in the other scenarios.

The outflow hydrographs from SITES modelling were input as outflow boundary conditions at the downstream toe of each respective dam. The remaining portion of the watershed was modeled with an unsteady flow that was estimated within a SITES model (lag method time of concentration, assumes average of the four dams curve number). Lastly, the FEMA 100-yr peak discharge input as inflow boundary condition upstream of Jewell Brook confluence with Black River.

In addition to analyzing the four Jewell Brook watershed dams breaching, the downstream flooding from a no-breach condition was modeled to quantify the actual damage from the breach above and beyond the normal flooding downstream. The downstream model included additional flow along the Black River from the several tributaries.

5.2 Peak Breach Discharges

Breach hydrographs were developed in accordance with TR 210-60, Part 1 – Peak Breach Discharge Criteria utilizing the following procedure.

1.) For depth of water at the dam at the time of failure where $H_w \geq 103$ ft

$$Q_{max} = 65{H_w}^{1.85}$$

2.) For a depth of water at the dam at the time of failure where

$$H_w \leq 103 \text{ ft} Q_{max} = 1100B_r^{1.35} \text{ where } B_r = \frac{V_s H_w}{A}$$

But not less than $Q_{max} = 3.2{H_W}^{2.5}$ nor more than $Q_{max} = 65{H_w}^{1.85}$

3.) When the width of the valley, L, at the water surface elevation corresponding to the depth, H_w, is less than $T = \frac{65{H_w}^{0.35}}{0.416}$

replace the equation, $Q_{max} = 65{H_w}^{1.85}$, in 1 and 2 above with

$$Q_{max} = 0.416L{H_w}^{1.5}$$

Where:

Q_{max} = peak breach discharge, cubic feet per second Q_{min}
B_r = breach factor, acre
V_s = reservoir storage at the time of failure, acre feet
H_w = depth of water at the dam at the time of failure; however, in the case of dam overtopping, not to exceed depth at the top of the dam, feet
A = cross-sectional area of embankment at the assumed location of breach, usually the template section (normal to the dam longitudinal axis) at the general floodplain location, square feet
T = theoretical breach width at the water surface elevation corresponding to the depth, H_w, feet
L = width of the valley at the water surface elevation corresponding to the depth, H_w, feet

HEC-RAS model breach parameters were initially calculated using the Froehlich dam breach equations (1995) and were iteratively adjusted within the model to generate an outflow hydrograph meeting TR-210-60 criteria. All breach hydrographs had a peak discharge equal to or greater than the minimum computed value by the TR-60 breach equations and less than the maximum value computed by the TR-210-60 equations.

	Site No. 1	**Site No. 2**	**Site No. 3**	**Site No. 5**
Reservoir water surface elevation (ft.)	1610.49	1565.60	1244.50	1489.70
Peak Discharge (Q_{max})	22,372	33,360	27,612	53,235
Depth of Water (H_w)	34.5	40.6	37.53	48.8

5.3 Breach Routing Results

A georeferenced building shapefile (2016) obtained from the Vermont Center of Geographic Information Open Data Portal was utilized to count the number of impacted structures during each analyzed scenario. A structure was considered impacted if it fell within the maximum extent of the inundation area for the given scenario.

Table 41: Summary of Impacted Structures from Static Breach

Jewell Brook Site	Total Number of Impacted Structures during Breach Condition
Site 1	490
Site 2	485
Site 3	421
Site 5	445

In addition to number of structures impacted breach inundation mapping depicting maximum flood depths for each of the analyzed scenarios in included on **Plates H-7 though H-10**. Flooding from the breaches at each dam continued to the North Springfield Dam (USACE flood control dam).

6.0 ALTERNATIVES ANALYSIS

Programmatic alternatives and potential rehabilitation alternatives were analyzed to determine if adequate to meet NRCS and State of Vermont hydraulic criteria. The alternatives utilized future watershed condition parameters and were modeled in the SITES program. The following summarize the alternative analysis completed. Conceptual design drawings are included in Appendix C – Support Maps I the Supplemental Plan /EA. Relevant sheet numbers from the drawings are reference herein.

6.1 FWOFI Alternative

The FOWFI alternative involves rehabilitating the existing Jewell Brook Dams to future State of Vermont Dam Safety design and performance standards. Currently, the State of Vermont recommends federal agency criteria for dam design. VT Dam safety is in the process of updating dam design standards by 2022.

This alternative requires the dam to passing the 100-year 24-hour event for existing watershed conditions without engaging the auxiliary spillway and passing the PMF with 18-inches of freeboard for existing watershed condition parameters through the auxiliary spillway without exceeding top of dam.

Table 42: FWOFI Alternative Details

Alterative	Key Components	Deficiency Addressed
Alternative 1.1 Future without Federal Investment (FWOFI)	Armor auxiliary spillway with ACB system	to prevent head cutting
	Raise dam crest	to achieve 18” of freeboard (required)
	Increase low level orifice size	improve hydraulic performance
	Expose pond inlet structure (dredging)	to remove sediment blocking intake
Alternative 2.1 Future without Federal Investment (FWOFI)	Armor auxiliary spillway with ACB system	to prevent head cutting
	Raise dam crest	to achieve 18” of freeboard (required)
	Increase low level orifice size	improve hydraulic performance
	Expose pond inlet structure (reservoir dredging)	to remove sediment blocking intake and provide sediment storage
Alternative 3.1 Future without Federal Investment (FWOFI)	Armor auxiliary spillway with ACB system	to prevent head cutting
	Raise dam crest	to achieve 18” of freeboard (required)
	Increase low level orifice size	improve hydraulic performance
	Expose pond inlet structure (reservoir dredging)	to remove sediment blocking intake
Alternative 5.1 Future without Federal Investment (FWOFI)	Armor auxiliary spillway with ACB system	to prevent head cutting
	Raise dam crest	to achieve 18” of freeboard (required)
	Expose pond inlet structure (reservoir dredging)	to remove sediment blocking intake and provide sediment storage

6.2 Decommissioning

The Decommissioning Alternative includes removing a portion of the existing dam embankment to restore the landscape and reconnect the stream impounded by the dam. The opening in the embankment will be shaped with a cross section similar to the upstream and downstream channels and sized to provide adequate cross-sectional area for the future conditions, 100-year, 24-hour inflow hydrograph.

The HEC-RAS 2D computational mesh was modified to represent the geometry of the decommissioned embankment and floodplain restoration. The resulting inundation map was used to identify properties and structures that would by impacted without the dam in place as described in Appendix D4.

6.3 Nonstructural, Floodproofing

This alternative includes acquisition and demolition, relocation, or flood protection of existing structures; modifications to prevent overtopping at downstream road crossings; and acquisition or easements to prevent or regulate future development within the flood inundation area. In addition, this alternative leaves the dams vulnerable as they are modeled to breach during the Freeboard Hydrographs. As reported in **Table 41** the dams are expected to impact the following during a static breach.

- Site #1: 490 structures were within inundation limits.
- Site #2: 485 structures were within inundation limits.
- Site #3: 421 structures were within inundation limits.
- Site #5: 509 structures were within inundation limits.

6.4 Rehabilitation Alternatives

Conceptual rehabilitation alternatives were formulated to address hydraulic deficiencies as well as other deficiencies at the dam that were found in other analysis. Rehabilitation alternatives were analyzed in SITES for suitability to the State of Vermont and NRCS criteria. The outflow for the recurrence interval storm events for the selected preferred alternative for each site was modeled through the downstream floodplain in HEC-RAS 2D.

The hydraulic deficiencies are presented in Table 43 below.

Table 43: Existing Dam Hydraulic Deficiencies

	Principal Spillway Hydrograph (PSH), (future CN)	Freeboard Hydrograph (FBH)	Auxiliary Spillway Integrity	Sediment Accumulation
Site 1	• PSH water surface el. 1613.88', auxiliary spillway control section El. 1612.93', existing auxiliary spillway activates with 0.95' through control section • Does not meet NRCS 10-day drawdown criteria (<15% volume in 10-days) due to low level orifice size	• FBH 6-hr water surface El. 1619.12' • Average top of dam el. 1620.00' • Low point on top of dam el. 1618.25' • Low point of saddle dike el. 1618.75' • FBH 6-hr event does not overtop average dam crest. • FBH 6-hr event overtops the low point dam crest el. by 0.87' • FBH 6-hr event overtops the saddle dike by 0.37'	• Auxiliary spillway modeled to breach during: ○ FBH 24-hr ○ FBH 6-hr ○ ASH 24-hr	• Remaining Sediment Storage: 12.3 acre-feet • Project Design Life: 100-yrs • Accumulated Sediment Removal: expose pond inlet drain inlet • Flood Storage: 404.4 acre-feet
Site 2	• PSH water surface el. 1572.89' • Auxiliary spillway control section El. 1565.67', existing auxiliary spillway activates with 7.19' through control section, • Does not meet NRCS 10-day drawdown criteria (<15% volume in 10-days) due to low level orifice size	• Does not overtop dam crest	• Auxiliary spillway modeled to breach during: ○ FBH 24-hr ○ FBH 6-hr	• Remaining Sediment Storage: 2.3 acre-feet • Project Design Life: 100-yrs requires dredging • Accumulated Sediment Removal: 3 acre-feet • Flood Storage: 261.4 acre-feet
Site 3	• PSH water surface El. 1249.32', auxiliary spillway control section El. 1243.03', existing auxiliary spillway activates with 6.29' through control section. • Does not meet NRCS 10-day drawdown criteria (<15% volume in 0-days) due to low level orifice size	• Does not overtop dam crest	• Auxiliary spillway modeled to breach during: ○ FBH 24-hr ○ FBH 6-hr	• Remaining Sediment Storage: 79.2 acre-feet • Project Design Life: 100-yrs • Accumulated Sediment Removal: expose pond inlet drain inlet • Flood Storage: 191.3 acre-feet
Site 5	• PSH water surface el. 1490.86', auxiliary spillway control section El. 1489.68' (right) and 1489.58' (left), existing auxiliary spillway activates with 1.18' (right) and 1.28' (left) through control section • Does not meet NRCS 10-day drawdown criteria (<15% volume in 10-days) due to low level orifice size	• Does not overtop dam crest	• Auxiliary spillway modeled to breach during: • FBH 24-hr (right and left) • FBH 6-hr (right and left) • ASH 24-hr (left)	• Remaining Sediment Storage: 0.86 acre-feet • Project Design Life: 100-yrs requires dredging • Accumulated Sediment Removal: 10.2 acre-feet • Flood Storage: 206.0 acre-feet

Note: values in Table 43 reflect existing dam geometrics and elevations and future watershed CN conditions.

To address these deficiencies several alternatives were formulated for each site. The following summarizes the alternatives proposed for each site.

Table 44: Site No. 1 Rehabilitation Alternatives

Alterative	Key Components	Deficiency Addressed
Alternative 1.2 Structural Rehabilitation (ACB Armoring) Preferred Alternative	Armor auxiliary spillway with ACB system	to prevent head cutting
	Raise dam crest	for overtopping
	Increase low level orifice size	improve hydraulic performance
	Expose pond inlet structure (dredging)	to replace
Alternative 1.3 Structural Rehabilitation (RCC Armoring)	Armor auxiliary spillway with RCC system	to prevent head cutting
	Raise dam crest	for overtopping
	Increase low level orifice size	improve hydraulic performance
	Expose pond inlet structure (dredging)	to replace

Table 45: Site No. 2 Rehabilitation Alternatives

Alterative	Key Components	Deficiency Addressed
Alternative 2.2 Structural Rehabilitation (ACB Armoring)	Armor auxiliary spillway with ACB system	to prevent head cutting
	Raise dam crest	for overtopping
	Increase low level orifice size	improve hydraulic performance
	Expose pond inlet structure (reservoir dredging)	to replace and provide sediment storage (60-year)
Alternative 2.3 Structural Rehabilitation (RCC Armoring)	Armor auxiliary spillway with RCC system	to prevent head cutting
	Raise dam crest	for overtopping
	Increase low level orifice size	improve hydraulic performance
	Expose pond inlet structure	to replace and provide sediment storage (60-year)
Alternative 2.4 Structural Rehabilitation (New Auxiliary Spillway at Principal Spillway with RCC Armoring)	Armor auxiliary spillway with ACB system	to prevent head cutting
	Raise dam crest	for overtopping
	Increase low level orifice size	improve hydraulic performance
	Expose pond inlet structure (reservoir dredging)	to replace and provide sediment storage (60-year)
Alternative 2.5 Structural Rehabilitation (ACB Armoring and Rise Normal Pool Elevation)	Armor auxiliary spillway with ACB system	to prevent head cutting
	Increase size and raise invert of low-level orifice size	improve hydraulic performance and provide sediment storage
	Expose pond inlet structure (reservoir dredging)	to replace
	Armor auxiliary spillway with ACB system	to prevent head cutting
Alternative 2.6 Structural Rehabilitation (ACB Armoring) Preferred Alternative	Raise dam crest	for overtopping
	Increase low level orifice size	improve hydraulic performance
	Expose pond inlet structure (reservoir dredging)	to replace and provide sediment storage (110-year)

Table 46: Site No. 3 Rehabilitation Alternatives

Alterative	Key Components	Deficiency Addressed
Alternative 3.2 Structural Rehabilitation (ACB Armoring) Preferred Alternative	Armor auxiliary spillway with ACB system	to prevent head cutting
	Raise dam crest	for overtopping
	Increase low level orifice size	improve hydraulic performance
	Expose pond inlet structure (reservoir dredging)	to replace
Alternative 3.3 Structural Rehabilitation (RCC Armoring)	Armor auxiliary spillway with RCC system	to prevent head cutting
	Raise dam crest	for overtopping
	Increase low level orifice size	improve hydraulic performance
	Expose pond inlet structure (reservoir dredging)	to replace

Table 47: Site No. 5 Rehabilitation Alternatives

Alterative	Key Components	Deficiency Addressed
Alternative 5.2 Structural Rehabilitation (ACB Armoring)	Armor auxiliary spillway with ACB system	to prevent head cutting
	Raise dam crest	for overtopping
	Expose pond inlet structure (reservoir dredging)	
Alternative 5.3 Structural Rehabilitation (RCC Armoring)	Armor auxiliary spillway with RCC system	to prevent head cutting
	Raise dam crest	for overtopping
	Expose pond inlet structure (reservoir dredging)	
Alternative 5.4 Structural Rehabilitation (Widen, ACB Armoring, Abandon Right Aux.) Preferred Alternative	Armor auxiliary spillway with ACB system	to prevent head cutting
	Widen Left Auxiliary Spillway	Abandon Right Auxiliary Spillway
	Raise dam crest	for overtopping
	Expose pond inlet structure (reservoir dredging)	
Alternative 5.5 Structural Rehabilitation (Labyrinth Spillway, Abandon Right Aux.)	Reconstruct with new labyrinth spillway	to prevent head cutting, Abandon Right Auxiliary Spillway
	Raise dam crest	for overtopping
	Expose pond inlet structure (reservoir dredging)	

Table 48: Preferred Alternative Data

Item	Unit	Jewell Dam Site No. (Preferred Alternative Future Conditions			
		Dam Site #1	**Dam Site #2**	**Dam Site #3**	**Dam Site #5**
Hazard Class of Structure	-	High	High	High	High
Total Drainage Area	Sq. Mi.	1.92	1.94	1.40	1.83
Future Runoff Curve Number	-	71.7	70.2	71.9	69.5
Time of Concentration (T_c)	Hours	0.90	0.60	0.49	0.80
Top of Dam Elevation	Feet	1620.00	1573.50	1252.50	1496.80*
Principal Spillway Riser Crest Elevation	Feet	1605.17	1558.73	1239.16	1449.90
Principal Spillway Low-level Orifice Elevation (Permanent Pool)	Feet	1584.77	1531.51	1229.47	N/A
Principal Spillway Low-level Orifice Opening Size	H x W	2-ft by 1.5-ft	3-ft by 1.5-ft	2-ft by 1.5-ft	N/A
Auxiliary Spillway Crest Elevation	Feet	1613.00	1566.81	1245.51	1489.58
Auxiliary Spillway Type	-	Vegetated Earth (ACB armored)	Vegetated Earth (ACB armored)	Vegetated Earth (ACB armored)	Vegetated Earth (ACB armored)
Auxiliary Spillway Bottom Width	Feet	250	300	200	235
Total Capacity (Below top of dam)	Acre-Feet	647.2	392.65	352.4	276.4
Freeboard Hydrograph					
Freeboard Hydrograph 6-hr event Rainfall	Inches	23.7	23.7	23.7	23.7

Freeboard Hydrograph 24-hr event Rainfall	Inches	29.1	29.1	29.1	29.1
Max. reservoir water surface elevation (6-hr)	Ft	1619.00	1573.31	1252.33	1496.63*
Max. reservoir water surface elevation (24-hr)	Ft	1616.54	1569.93	1248.70	1494.32

- Assumes waiver for site 5

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