Appendix D. Sedimentation

Draft Kansas River Reservoirs Flood and Sediment Study

October 2023

U.S. Army Corps of Engineers
Kansas City District

US Army Corps of Engineers®
Kansas City District

Kansas River Reservoirs Flood and Sediment Study

Appendix D1: U.S. Army Corps of Engineers Lakes Existing Conditions Sedimentation Summary

November 2022

Contents

1.0 INTRODUCTION

The Kansas River Basin contains 18 federal reservoirs, including seven USACE lakes and eleven Bureau of Reclamation lakes (see Figure 1-1). Table 1-1 summarizes key information about each reservoir. Agricultural cropland and grazing land constitute the major land uses in each of the reservoirs.

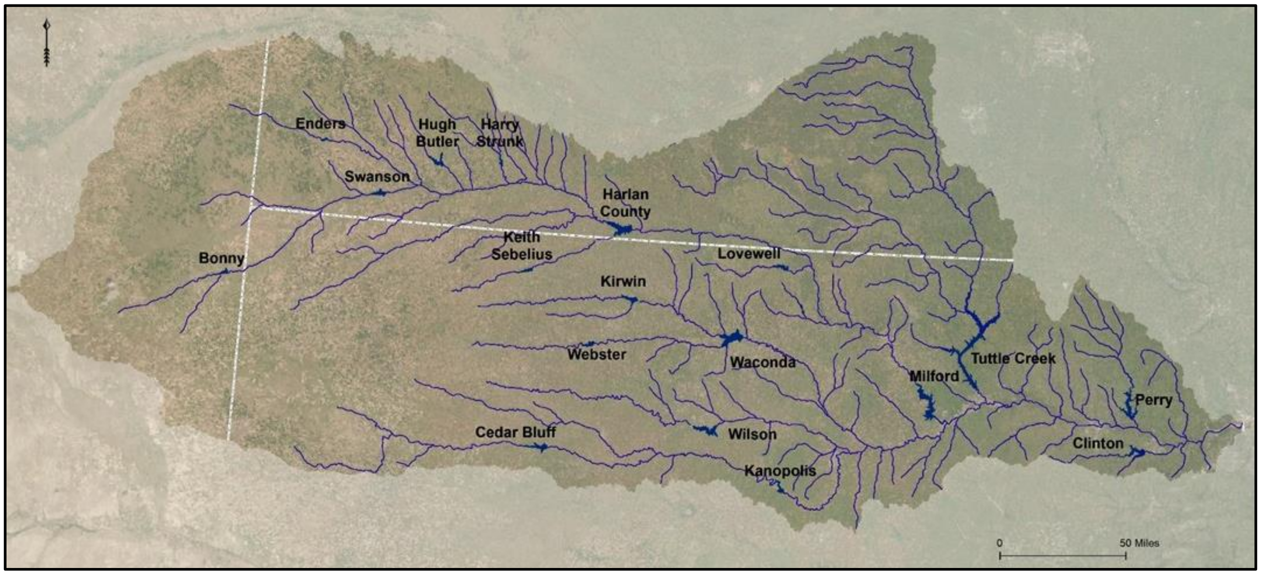


Figure 1-1. Overall Kansas River Basin Map.

Table 1-1. Kansas River Basin USACE Lakes

Name	Date Closed	Total Drainage Area (sq mi)	Unregulated Drainage Area (sq mi)
Clinton	August of 1975	367	367
Perry	August of 1966	1,117	1,117
Tuttle Creek	July 1959	9,556	9,556
Milford	December 1966	24,882	3,733
Kanopolis	March 1948	7,860	2,330
Wilson	October 1964	1,971	1,971
Harlan County	October 1952	20,753	7,296

Note: Lakes are listed from east to west.

In Table 1-1, “unregulated” refers to the portion of the watershed that is not upstream from a USACE or Bureau of Reclamation dam. Smaller state, city, and county dams also impound water, but their sediment effects are small. For example, sediment deposition in Mission Lake upstream of Perry Lake equates to less than 1% of Perry Lake’s sediment accumulation.

Kansas City District, River Engineering Section (NWK-EDH-R) performed existing conditions analyses at each of the USACE lakes (see Appendix D1.1 to D1.7). These separate appendices describe the dam infrastructure, lakeside infrastructure and environmental resources, sedimentation impacts on O&M, current storage/elevation curves, history of sediment accumulation, sediment trapping efficiency, bulk density, chemical constituents, delta progression, and downstream channel geomorphic change and sediment concentrations. In addition, these appendices developed flow/load rating curves calibrated to observed historic reservoir deposition. This document aggregates the key findings from those appendices.

2.0 DAM INFRASTRUCTURE

Each of these dams include service gates at the bottom of the lake, smaller low-flow gates at higher elevations, various water supply intakes, and emergency spillways. The configuration of gates is unique to each lake. Important for sediment management, in each of the Kansas River Basin USACE lakes, the main service gates open and close at the bottom of the lake. Table 2-1 indicates the average number of days these service gates are used. On the days the service gates are not used, water is typically existing the lake via low flow conduits or other means.

Table 2-1. Service Gate Usage

Lake	Service Gates Used When Flows Exceed (cfs)	Service Gate Average Annual Usage (days/year)
Clinton	50	91
Perry	200	91
Tuttle Creek	200	322
Milford	400	227
Kanopolis	varies	167
Wilson	50	61
Harlan County	0	140

Note: Lakes are listed from east to west.

Each of these dams serves multiple authorized purposes. Each of these dams includes a significant volume of empty space for flood control. The multipurpose pool (MPP) provides storage for water supply, recreation, irrigation, environmental and water quality releases, and navigation support. None of the USACE dams in the Kansas River Basin produce hydropower. Table 2-2 summarizes key elevations.

Table 2-2. Key Pool and Infrastructure Elevations.

Lake	Multipurpose Pool Elevation (ft)	Flood Control Pool Elevation (ft)	Spillway Elevation (ft)	Dam Elevation (ft)
Clinton	875.5-NGVD29	903.4-NGVD29	907.4-NAVD88	928-NAVD88
Perry	891.5-NGVD29	920.6-NGVD29	922.3-NAVD88	946.3-NAVD88
Tuttle Creek	1075-NGVD29	1136-NGVD29	1116-NGVD29	1159-NGVD29
Milford	1144.4-NGVD29	1176.2-NGVD29	1176.2-NGVD29	1213.0-NGVD29
Kanopolis	1463.0-NGVD29	1508.0-NGVD29	1507.0-NGVD29	1537.0-NGVD29
Wilson	1516.0-NGVD29	1554.0-NGVD29	1581.5-NGVD29	1591.5-NGVD29
Harlan County	1945.73-NGVD29	1973.5-NGVD29	1943.5-NGVD29	1982.0-NGVD29

Note: Lakes are listed from east to west.

3.0 HISTORIC POOL VOLUMES AND SEDIMENTATION

Table 3-1 lists the sediment deposition volumes in each lake. Table 3-2 provides the average annual rates of sediment deposition.

Table 3-1. Total Sediment Accumulation in Kansas River Basin Lakes.

Lake	MPP Initial Volume, ac-ft (Survey Year)	MPP Recent Volume, ac-ft (Survey Year)	% of MPP Lost to Sediment	FCP Initial Volume, ac-ft (Survey Year)	FCP Recent Volume, ac-ft (Survey Year)	% of FCP Lost to Sediment
Clinton	129,171 (1977)	113,032 (2019)	12	268,367 (1977)	291,570 (2019)	-8.6*
Perry	243,220 (1969)	200,004 (2009)	18	521,880 (1969)	515,519 (2009)	1.2
Tuttle Creek	424,312 (1962)	257,014 (2009)	39	1,942,705 (1962)	1,870,735 (2000)	3.7
Milford	415,352 (1967)	373,152 (2009)	10	757,746 (1967)	757,872 (2009)	-0.02*
Kanopolis	73,200 (1948)	47,170 (2017)	36	447,091 (1948)	413,521 (2007)	2.3
Wilson	247,835 (1964)	236,188 (2008)	4.7	530,710 (1964)	529,289 (2008)	0.5
Harlan County	346,512 (1951)	314,111 (2000)	9.4	503,488 (1951)	500,000 (2000)	0.7

Note: Lakes are listed from east to west.
*Negative deposition is a numerical artifact caused by the shift in survey methods.

Table 3-2. Annual Sediment Accumulation in Kansas River Basin Lakes.

Lake	MPP Deposition Years	MPP Deposition (ac-ft/year)	MPP% Lost to Sediment (%/year)	FCP Deposition Years	FCP Deposition (ac-ft/year)	FCP% Lost to Sediment (%/year)
Clinton	1977-2019	384	0.30%	1977-2019	-552	-0.21%
Perry	1969-2009	1,080	0.44%	1969-2009	159	0.03%
Tuttle Creek	1963-2009	3,794	0.84%	1963-2009	1,242*	0.06%
Milford	1967-1994	1,558	0.38%	1967-1994	178	0.02%
Kanopolis	1948-2017	374	0.51%	1948-2007	147	0.04%
Wilson	1964-1995	459	0.19%	1964-2019	28	0.01%
Harlan County	1952-2000	756	0.22%	1964-2009	69	0.01%

Note: Lakes are listed from east to west.
*May underestimate deposition rate due to “negative deposition” over the shift in survey methods.

Figure 3-1 maps the average annual sediment accumulation per square mile of unregulated drainage area. As seen, the eastern watersheds produce much more sediment per square mile than the western watersheds. This is due to the difference in mean annual precipitation, which quadruples from Western Kansas to Eastern Kansas.

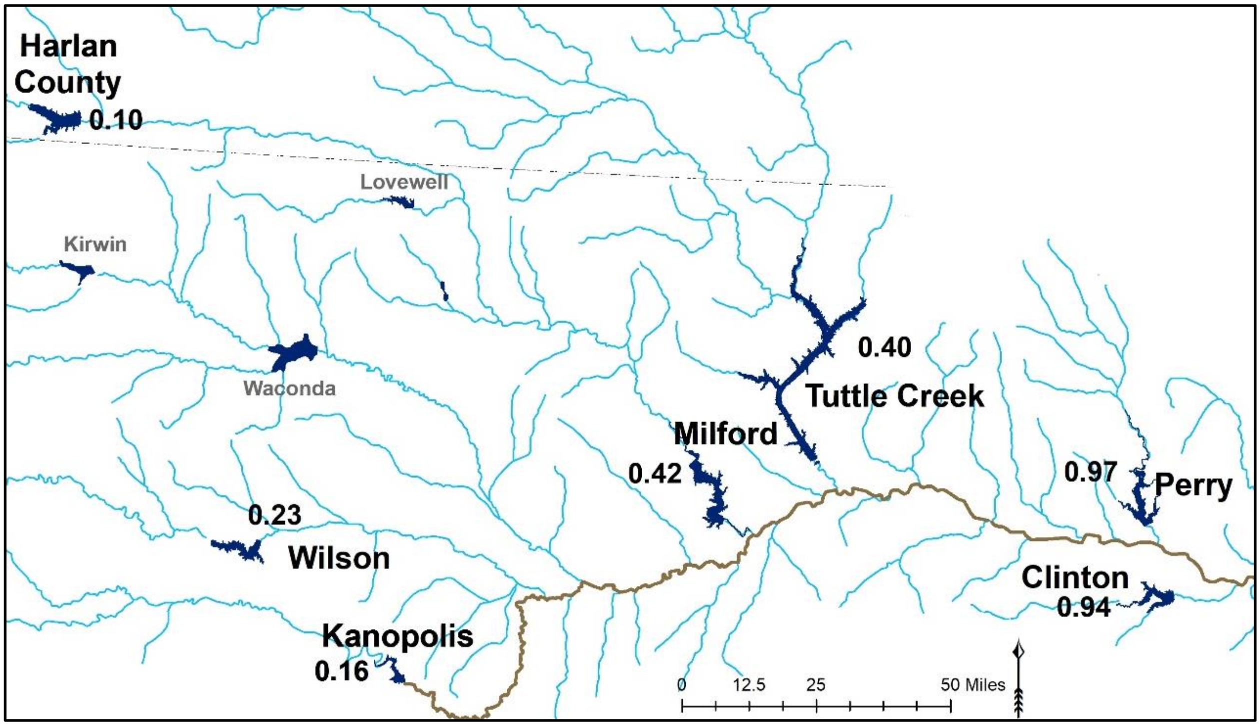


Figure 3-1. Average annual deposition of unregulated drainage areas at USACE lakes. *Average annual deposition per square mile of unregulated drainage area at USACE lakes, ac-ft/y/mi^2. Smaller labels indicate Bureau of Reclamation lakes.*

4.0 SEDIMENTATION EFFECTS ON O&M

Sedimentation has already impacted both O&M costs and recreation facilities at Kansas River Basin lakes. At Kanopolis, two targeted dredging actions have occurred to remove sediment that interfered with the deployment and operation of the emergency bulkheads. At many lakes, boat ramps, docks, and small water intakes for campgrounds have been silted in and rendered unusable. Appendix G documents these impacts in greater detail.

5.0 TRAPPING EFFICIENCY

The trapping efficiency is the portion of incoming sediment that deposits in the lake instead of passing downstream. Several lakes had measured trapping efficiencies, wherein sediment inflows and outflows were measured and trapping efficiency directly computed. For other lakes, the trapping efficiency was estimated using the Brune (1953) curve that estimates trapping efficiency based on the ratio of reservoir capacity to mean annual inflow. The volume of the multipurpose pool was used for this computation. The Brune Curve yields reasonable approximations at lakes with measured trapping efficiencies, which lends confidence to using the Brune Curve for lakes without measured trapping efficiencies. As seen in Table 5-1, trapping efficiencies are very high.

Table 5-1. Trapping Efficiency by Lake.

Lake	Measured TE (%)	TE- Brune Curve (%)
Clinton	97	96.2
Perry	-	96.8
Tuttle Creek	98	96.7
Milford	-	96.3
Kanopolis	95.6	94.6
Wilson	-	97.0
Harlan County	-	96.9

Note: Lakes are listed from east to west.

6.0 BULK DENSITY

The method used to transform mass into volume in the computations required separate bulk densities for both the flood control pool and the multipurpose pool. The availability of measured bulk density varied from lake to lake. Where a measured value was not available, the total bulk density was estimated using the following formula:

$$\gamma = \frac{1.0}{\left(\left(\frac{F}{\gamma}\right)_{clay} + \left(\frac{F}{\gamma}\right)_{silt} + \left(\frac{F}{\gamma}\right)_{sand}\right)}$$

Where γ = the bulk density (combined for both pools)

F = the fraction of clay, silt, or sand in the incoming load

γ = unit weight for clay, silt or sand assumed to be 30 pcf, 65 pcf, and 93 pcf respectively (the defaults from HEC-RAS)

This yields the total bulk density for both pools combined. To differentiate the bulk densities between pools, measured data for a single pool, estimates using the above function with reservoir deposit size fractions, and estimates for nearby lakes were used. Table 6-1 lists the bulk densities for each lake. Figure 6-1 maps the bulk densities used in the creation of the rating curves. As seen, the bulk density of the flood control pool deposits generally increases from east to west.

Table 6-1. Bulk Densities (lb/ft^3)

Lake	Measured MPP Bulk Density	Measured FCP Bulk Density	Measured Combined Bulk Density	Combined Bulk Density from Incoming Load	Estimated MPP Bulk Density	Estimated FCP Bulk Density
Clinton	-	-	-	42.0	39.4	57.4
Perry	39.4	-	-	42.1	-	57.4
Tuttle Creek	37.4	60.9	-	42.9	-	-
Milford	27.2[a]	-	-	44.5	41.1	72.6
Kanopolis	38.7	75.1	43.9	43.8	39.7	-
Wilson	84.7[a]	-	-	40.9	41.3	72.4
Harlan County	44.6	66.0	49.3	49.9	-	-

Note: Lakes are listed from east to west.
a = this measured value not used

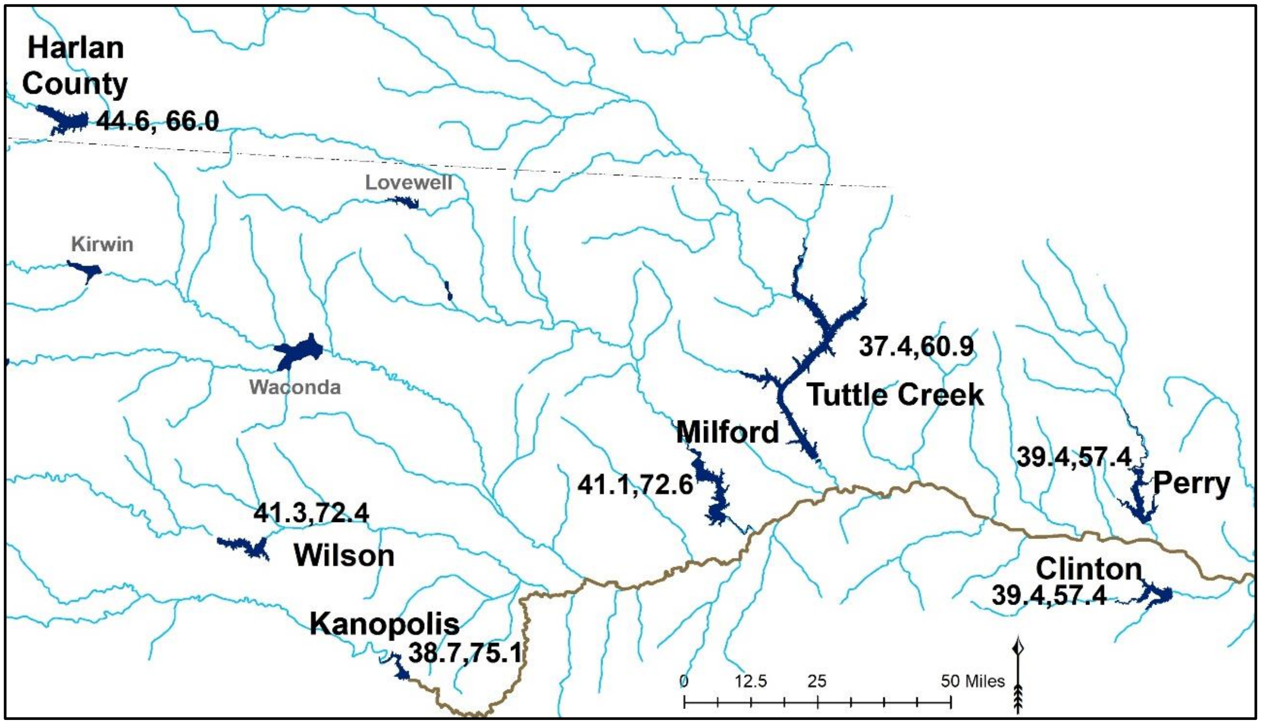


Figure 6-1. Bulk Densities in lb/ft^3. (Multipurpose Pool, Flood Control Pool).

7.0 FLOW/LOAD RATING CURVES

7.1 Shape of Flow/Load Rating Curves

Flow/load rating curves were developed for each lake. These were based on paired flow/sediment concentrations measured at upstream USGS gages. At lakes with sediment data at sufficiently high flows, the flow/load curves exhibited a "flattening out" at approximately the 1.2 to 1.5 year discharge (see examples in Figure 7-1 and Figure 7-2). This range corresponds to the typical range of the geomorphic bankfull flow in Kansas (Shelley 2012). This same behavior was assumed when extrapolating the upper end of rating curves with insufficient data.

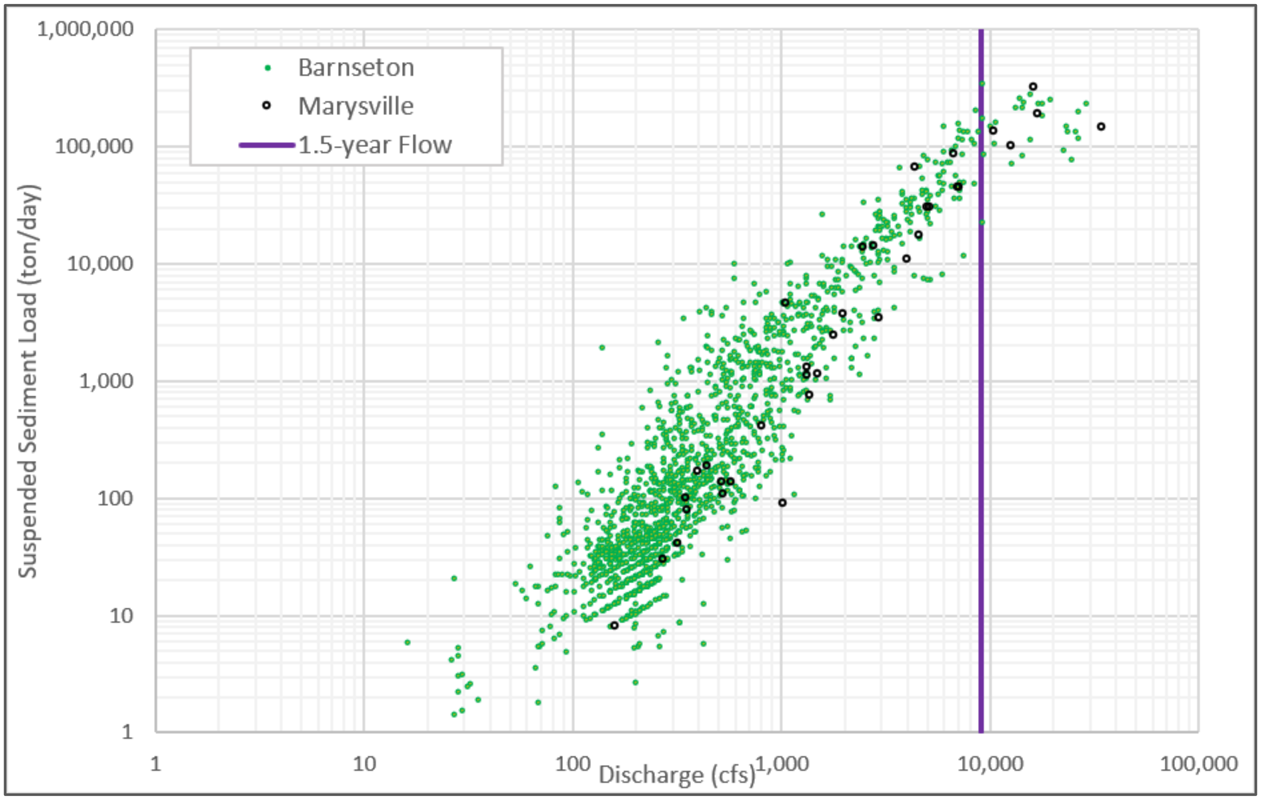


Figure 7-1. Flow/load data on the Big Blue River above Tuttle Creek Lake.

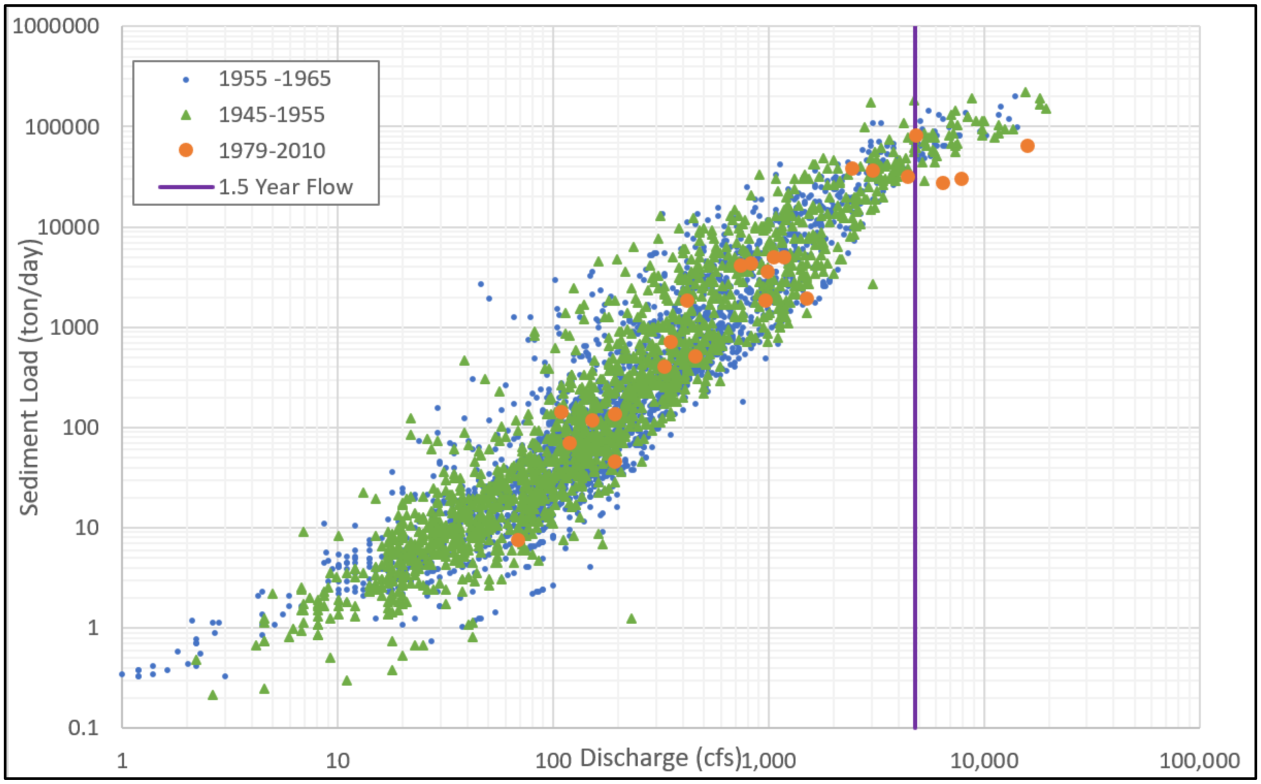


Figure 7-2. Flow/load data for the Smoky Hill River above Kanopolis Lake.

Data from the Smoky Hill at Ellsworth gage above Kanopolis Lake were used to investigate potential reasons for this flattening out. As demonstrated in Figure 7-3 and Figure 7-4, the Smoky Hill River exhibits clockwise hysteresis during flood events.

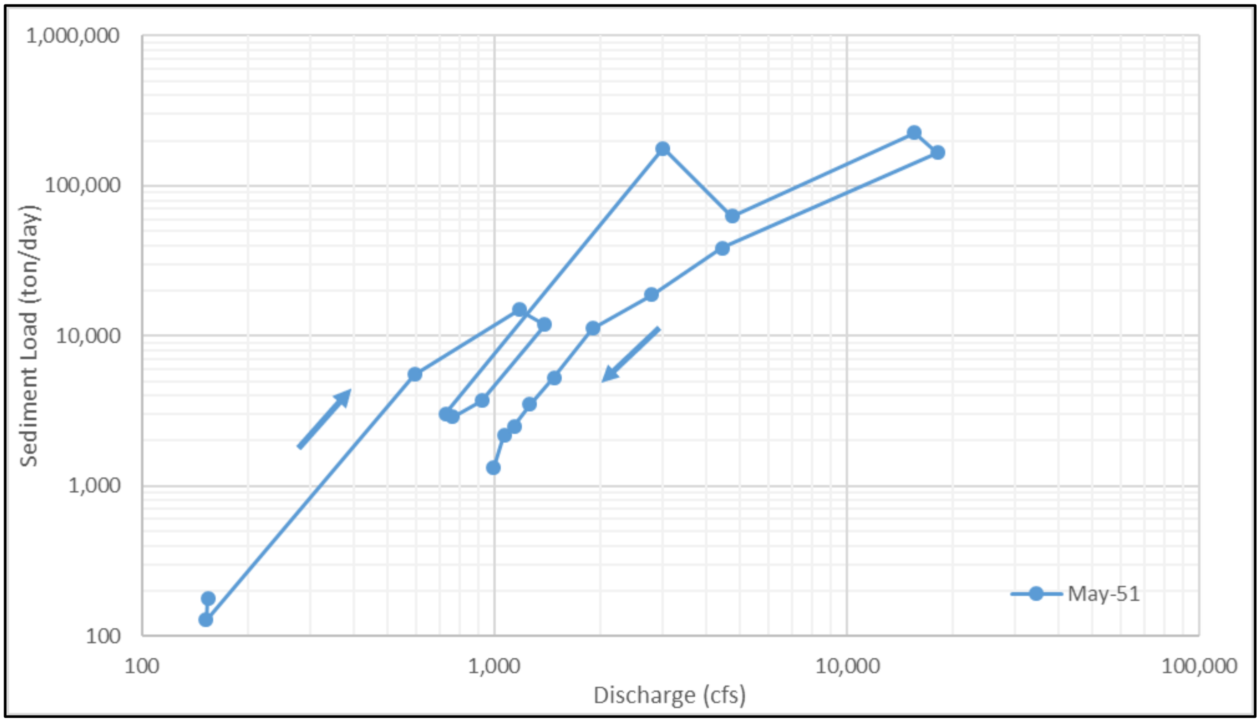


Figure 7-3. Evidence of clockwise hysteresis on the Smoky Hill River during the 1951 Flood.

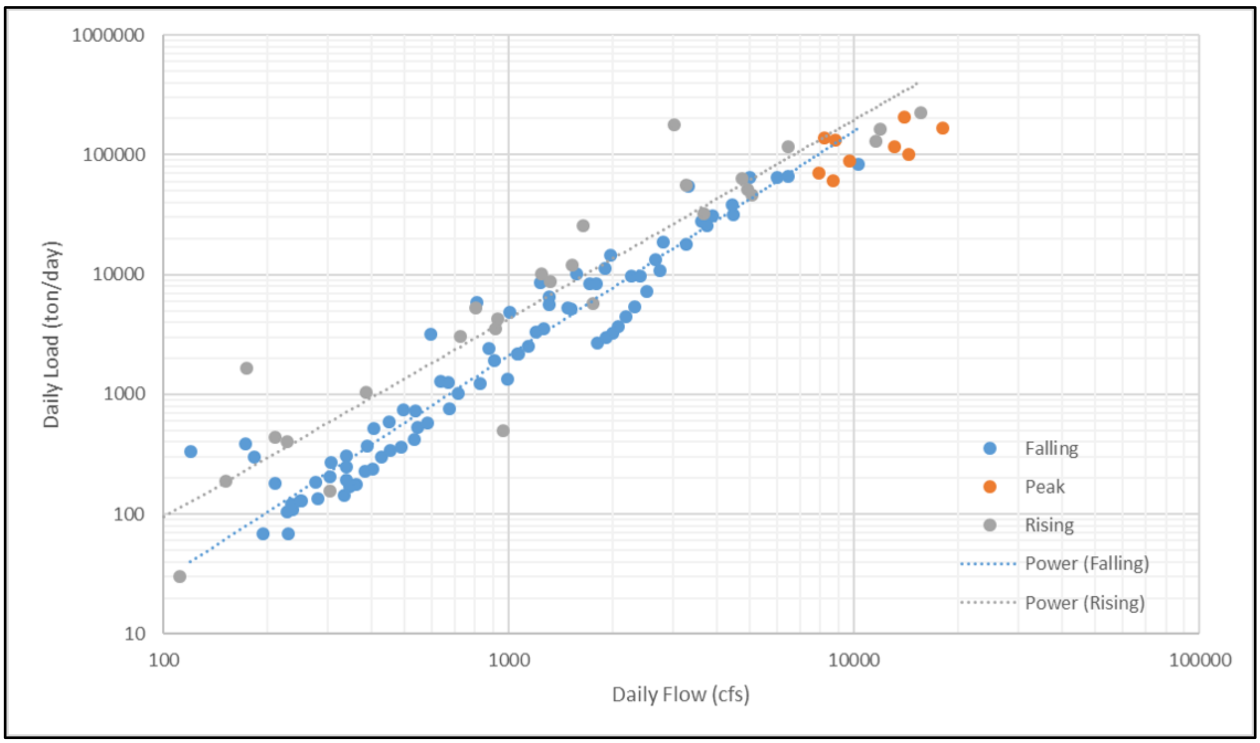


Figure 7-4. A comparison of the sediment loads between the rising and falling limbs of the hydrograph of multiple flood event. The rising limb generally has the greatest sediment load.

This suggests that supply-limitation may be the cause of the flattening out. Sediment sinks in the floodplain may be a contributing factor in the supply limitation, as the bend in the curve occurs for flows above bankfull.

7.2 Development and Calibration of Flow/Load Rating Curves

The rating curve development followed these steps:

1. Estimate the best fit regression line of the form $Q_s = aQ^b$ using log-log linear regression using data up to the point where the curve bends over (typically 1.2 or 1.5-year flow).
2. Correct for bias using the Duan (1983) correction factor. Table 7-1 lists the Duan E values.
3. Add a percentage to account for bed load.
4. At three gages, sufficient data at the lowest flows was available to indicate a flattening of the slope of the flow/load relationship at low flows. A point on the rating curve was chosen to provide a visual fit in log space, i.e., to provide reasonable straight line through the low-flow points. As this portion of the curve is inconsequential for long-term volumetric measurements, extrapolation based on this behavior was not exported to other lakes.
5. Adjust for the ungaged area. This was accomplished in one of three ways (1) by multiplying the load by the ratio of total unregulated drainage area to unregulated gaged drainage area (at Perry, Tuttle, Milford, and Kanopolis), (2) by multiplying by a ratio based on historic correlations in sediment load from discontinued to current gages (at Wilson and Harlan County), or (3) by running a daily time series of total inflows to the lake as computed by Water Control (rather than inflows at the gage) through the rating curve (this was done at Clinton). Data availability dictated which method was used.
6. Using the measured data, estimate the percentages of clay, silt, and sand/gravel. For some lakes, the percentages varied by flow.
7. Apply this rating curve to daily flow rates from dam closure to present to compute the cumulative mass of sediment entering the reservoir.
8. Acquire the bulk density of sediment deposits in the flood control and multipurpose pools, as listed in Table 3-1.
9. Apportion the sediment into either the flood control or the multipurpose pool. The ratio of flood control pool deposition to total deposition was computed based on surveys over time and transformed from a volume to mass using the bulk densities.
10. Multiply by the trapping efficiency, as listed in Table 5-1, to determine deposition in the multipurpose pool.
11. Compare the volume of depositing sediment computed via the rating curve to measured deposition between surveys.
12. Calibrate the rating curves by adjusting the “bent down” extrapolation for high flows. At several lakes, this was sufficient to calibrate. At others, an additional factor was needed to bring the rating curves in closer agreement with the measured surveys.

Figure 7-5 presents the calibrated rating curves for all upstream gages. The color scheme is from hot (west) to cold (east). Some dams feature more than one gaged tributary and are plotted in the same color but different line type. Figure 7-5 does not evince obvious spatial trends, but tributaries to the same lake do exhibit similarities. The three tributaries to Tuttle exhibit flat rating curves at the upper end, and the two tributaries to Harlan County (which is farther upland) produce significantly more sediment per unit of water flow than the other lakes which are further downstream. Table 7-1 summarizes key coefficients in the rating curve calibrations.

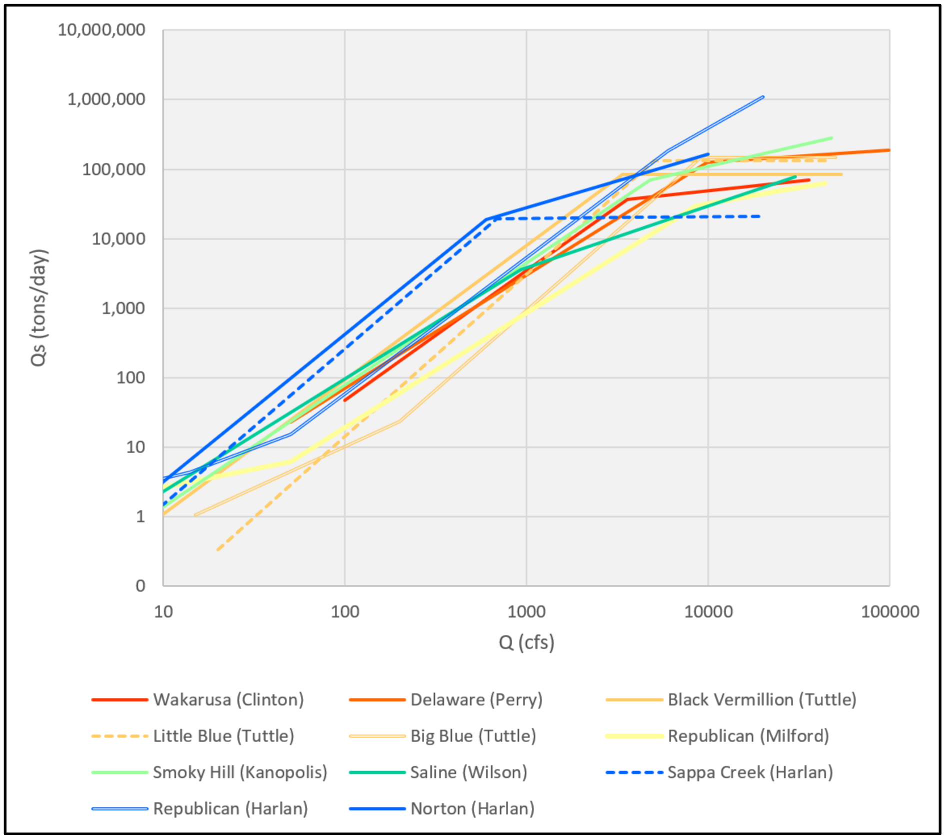


Figure 7-5. Calibrated Sediment Rating Curves.

Table 7-1. Rating Curve Summaries.

Lake	River	Qs=aQb a	Qs=aQb b	Duan E	Bed load %	Ungaged Area Ratio	High Flow Extrapolation Flow (cfs)	High Flow Extrapolation Return Period (Years)	Additional Calibration Factor
Clinton	Wakarusa	0.0068	1.8637	1.24	5%	NA	3,570	1.5	NA
Perry	Delaware	0.0203	1.6357	1.76	5%	2.59	9,820	1.5	NA
Tuttle Creek	Big Blue	0.0000911	2.30	1.27	5%	1.12	9,080	1.2	NA
Tuttle Creek	Little Blue	0.000276	2.31	1.46	5%	1.12	5,000	<1.2	NA
Tuttle Creek	Black Vermillion	0.0078	1.94	1.52	5%	1.12	3,360	1.2	NA
Milford	Republican	0.0054	1.65	1.58	15%	1.1	8,510	1.5	NA
Kanopolis	Smoky Hill	0.017	1.7572	1.23	15%	1.14	4,760	1.5	1.04

Lake	River	Qs=aQb a	Qs=aQb b	Duan E	Bed load %	Ungaged Area Ratio	High Flow Extrapolation Flow (cfs)	High Flow Extrapolation Return Period (Years)	Additional Calibration Factor
Wilson	Saline	0.0376	1.6211	1.27	15%	1.69	930	1.5	5.35
Harlan County	Republican	0.0032	2.404	1.89	17%	1.28	6,000	>1.5*	1.19
Harlan County	Sappa Creek	0.0067	1.961	1.11**	17%	1.28	680	1.2*	1.19

Notes: *Values from pre-1970 flows.
**At Harlan County, an adjustment factor was included so the rating curve matched the sum of the daily sediment loads, which took the place of the Duan E.

Table 7-2 to Table 7-8 demonstrate how well the rating curves were able to calibrate to the observed data over multiple time periods. The ability of the rating curve to match deposition over multiple time frames differs by lake, largely based on data quality and availability.

Clinton Lake was calibrated using only change in the multipurpose pool because the measured flood control pool deposition was inconsistent and unreliable. Many lakes showed “negative deposition” in the flood control pool over the time period where the survey method shifted from sediment rangelines to LIDAR. At Clinton, the survey method for the flood control pool changed with every survey, and the flood control pool deposition amounts were highly unreliable. Moreover, Clinton had only limited flow/load data and no measured bulk densities. While calibration to the most recent time period was usually favored, at Clinton calibration to the 2009 – 2019 deposition would have required an unrealistic shape in the extrapolation of the curve at high flows, therefore, the 1990-2009 time period was used for calibration.

Table 7-2. Clinton Lake: Rating Curve vs. Measured Deposition.

Years	Multipurpose Pool Surveys (ac-ft)	Rating Curve (ac-ft)	Rating Curve / Surveyed
1977-1990	3,837	4,373	1.14
1990-2009	6,635	6,625	1.00
2009-2019	5,667	3,098	0.55

Perry Lake was calibrated using the change in only the multipurpose pool over the most recent two surveys (2001 to 2009). The flood control pool deposition was not reliable because the change from sedimentation rangelines to LIDAR resulted in erroneous “negative deposition.” However, as the older time periods used sedimentation rangelines in both surveys, total deposition over the older time periods served as a verification. From 1969 to 1979, the rating curve underpredicts, while from 1979 to 1989 it overpredicts. Over both time periods (1969 to 1989) it slightly overpredicts.

Table 7-3. Perry Lake: Rating Curve vs. Measured Deposition.

Year	Surveys (ac-ft)	Rating Curve (ac-ft)	Rating Curve / Surveyed
1969-1979	25,063	23,182	0.92
1979-1989	14,729	19,310	1.31
1969-1989	39,792	42,492	1.07
2001-2009 (Multipurpose Pool Only)	6,678	6,677	1.00

Tuttle Creek Lake was calibrated to the most recent set of surveys (2000 to 2009) for just the multipurpose pool. However, the rating curve reproduces the measured deposition combined for both pools very well for all three additional time periods, as well as over the long term (1963 to 2000).

Table 7-4. Tuttle Creek Lake: Rating Curve vs. Measured Deposition.

Years	Surveyed (ac-ft)	Computed (ac-ft)	Rating Curve / Surveyed
1963-1972	40,898	42,907	1.05
1972-1983	68,773	61,557	0.90
1983-2000	124,411	126,015*	1.01
2000-2009**	23,123	23,367	1.01
Total (1963-2000)	234,082	230,479	0.98

Note: *Flood pool (FP) deposition calculated using mean bulk density
**Surveyed and computed are for the MPP. Change in survey method makes FP deposition unreliable.

At Milford Lake, several data issues prevent good calibration. The flow/load rating curve shifts (lowers) over time, which makes calibration to newer data necessary (see Figure 7-6). The 1994 survey presented odd values compared to both earlier and later surveys, which makes it necessary to skip 1994 and calibrate to total deposition from 1980 to 2009. Unfortunately, the largest shift in the rating curves occurs in the middle of the 1980 to 2009 period (likely a result of the 1993 flood), which makes using a single rating curve for the time period unrepresentative. Finally, official estimates for the volume of the flood control pool yielded “negative deposition” from 1980 to 2009 due to a shift in survey method.

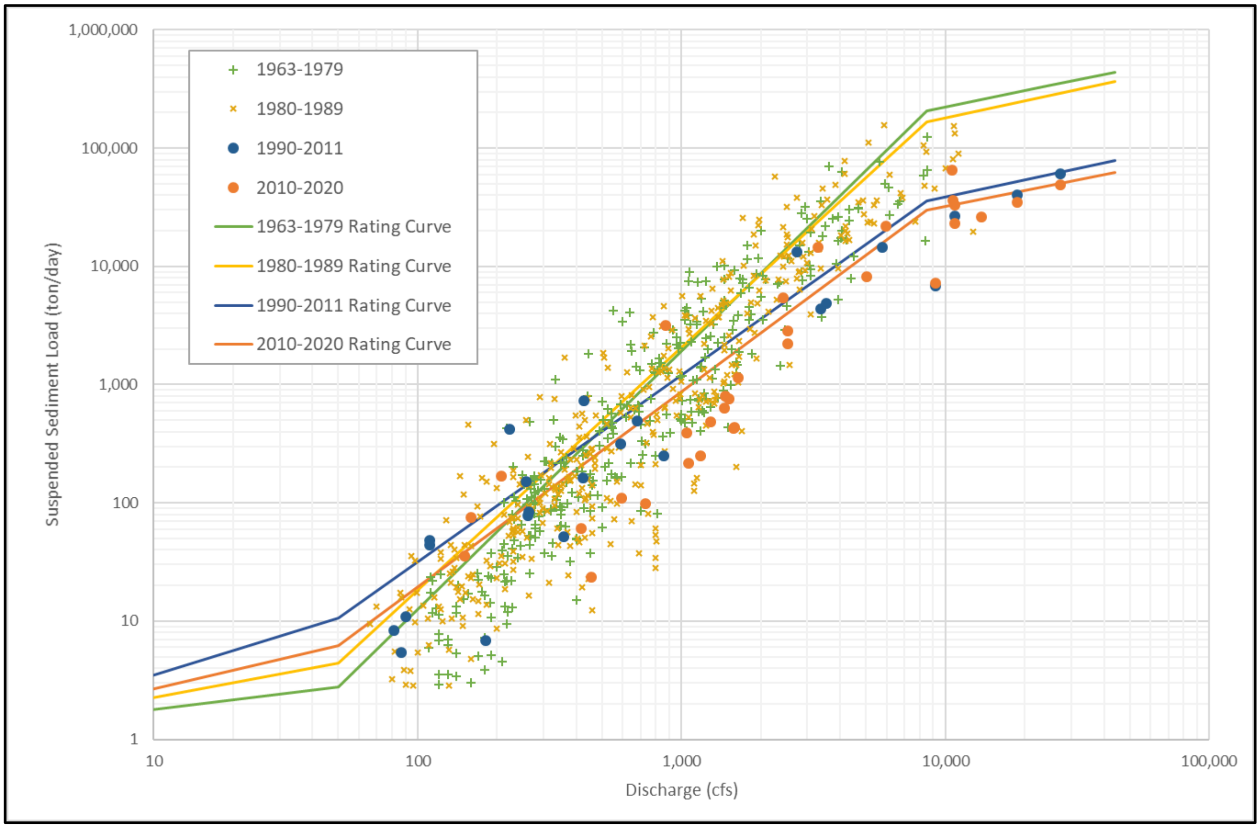


Figure 7-6. Multiple Rating Curves at Milford Lake.

These problems were addressed through various means, as explained below. First, deposition in the lake from 1980 to 2009 was re-computed by sampling from the LIDAR and bathymetry along the sedimentation lines and using the XS Viewer software. This methodology yielded reasonable results in other time periods. Second, flow/load data was sufficient to create three separate rating curves (1963 to 1979, 1980 to 1989, 1990 to 2011), the results of which are shown in Table 11. As seen in Figure 7, the angle on the bent-down portion of the curve was kept constant amongst rating curves, but the location and slope of the $Qs=aQ^b$ portion of the curve changes based on the data. The calibration period from 1980 to 2009 uses two rating curves, first from 1980 to 1989 and then from 1990 to 2011. This is the best calibration that could be accomplished given the data issues. As new Milford Lake surveys and years of sediment data are collected, this calibration should be revisited.

Table 7-5. Milford Lake: Rating Curve vs. Measured Deposition.

Time Period	Rating Curve Time Period	Surveyed Deposition	Rating Curve	Rating Curve / Surveyed
1967-1980	1963 to 1979	28,889	28,933	1.00
1980-1994	1980 to 1989; 1990 to 2011	19,046	27,930	1.47
1994-2009	1990 to 2011	14,272*	5,469	0.38
1980-2009	1980 to 1989; 1990 to 2011	33,287*	33,399	1.00
Total		62,176*	62,332	1.00

Note: *1994-2009 deposition calculated using Cross Section Viewer Software from the 2009 survey resampled at the sedimentation rangelines.

Kanopolis Lake was calibrated to multipurpose pool volume change from 2007 to 2017, with total volume change from other time periods and over the full life of the reservoir as verification. By adjusting the high point on the rating curve, the calibration was able to match the measured deposition from 2007 to 2017 to within 4% (rating curve was 4% high). Adjusting the final point to lower the rating curve volume yielded a curve that did not stay within the available high flow/load data. While the source of the 4% error could not be identified, error in the bedload percentage (which is high compared to other lakes) is a possibility. The final rating curve was divided by 1.04 to achieve a better calibration. This led to a better calibration over the full life of the reservoir as well. The reason for the low surveyed deposition from 1971 to 1982 is not known.

Table 7-6. Kanopolis Lake: Rating Curve vs Measured Deposition.

Period	Surveyed Deposition (ac-ft)	Computed Deposition (ac-ft)	Computed/Surveyed
1946-1961	13,576	13,893	0.99
1961-1971	5,861	6,030	0.83
1971-1982	,495	5,705	3.82
1982-1993	4,834	4,827	0.82
1993-2007	3,686	3,815	0.78
2007-2017*	1,578	1,653	1.00
1946-2017**	35,097	35,922	1.02

Notes: *Computed and Surveyed volumes for this time period are for the multipurpose pool since the flood control pool was not surveyed
**Flood control pool volume for 2017 was estimated based on the average ratio between MPP and FCP volumes from 1946-2007

At Wilson Lake, the rating curves were developed using the same procedure as at the other lakes. However, unlike the other lakes the resulting volumes were significantly lower than the volume of deposition measured in the lake. This could be due to shoreline erosion, underestimation of the bedload, underestimation of the sediment yield from the drainage area below the gages, or underestimation of the suspended sediment load at the Russell gage. With a factor of 5.35 applied, the rating curve matches the deposition volume over the calibration period (1984 to 1995) and reasonably matches from 1964 to 1984. Calibration from 1995 to 2008 was not possible due to negative deposition caused by a change in survey methods.

Table 7-7. Wilson Lake: Rating Curve vs. Measured Deposition

Time Period	Surveyed Deposit (ac-ft)	Computed Calibrated (ac-ft)	Computed / Surveyed
1964-1984	5,813	6,349	1.06
1984-1995	9,281	9,281	1.00
1995-2008	-3,662	7,785	NA

At Harlan County Lake, the flow/load data at the three major tributaries derives from 1947 to 1970. Newer data is not available. The rating curves were calibrated against measured sediment deposition from 1962 to 1972. It was found during calibration that an additional factor of 1.19 was required for the rating curve results to match the measured surveys. Using this factor, the rating curve computation matches the measured deposition from 1952 to 1962, 1962 to 1972, and performs reasonably well from 1972 to 2000 in the multipurpose pool. The 1988 survey reflected higher elevations (additional deposition) than the

2000 survey along the same lines. As deposition ending in the 1988 survey also seems anomalous, a time period skipping the 1988 survey (1972 to 2000) was assessed.

Table 7-8. Harlan County Lake: Rating Curve vs. Measured Deposition.

Period	Surveyed Deposition (ac-ft)	Computed Deposition Calibrated	Computed / Surveyed Calibrated
1952-1962	14,516	16,858	1.00
1962-1972	8,735	8,735	1.00
1972-1988	9,828	4,613	0.44
1972-2000*	7,650	7,030	0.93

*Estimates are for the multipurpose pool

8.0 QUALITY FLOW/LOAD RATING CURVES

The data was sufficient to create and calibrate a rating curve at every lake adequate for recon-level purposes of the Watershed Study. However, the availability, completeness, consistency, and quality of data vary from lake to lake, which necessitated varying levels of approximations and extrapolations. Table 8-1 provides a point score for major aspects of the rating curve development. The total score can be used when qualitatively describing the confidence in the rating curve.

Table 8-1. Quality Matrix for Rating Curve Development.

Lake	Surveys[1]	Flow-load[2]	Sediment Sources[3]	Bulk density[4]	Time Period[5]	Total Score
Clinton	0.5	0.5	1	0.5	0.5	3.0
Perry	0.5	1	1	1	1	4.5
Tuttle Creek	0.5	1	1	1	1	4.5
Milford	0	0	1	0.5	0.5	2.0
Kanopolis	0.5	1	0.5	1	1	3.5
Wilson	0	1	0	0	1	2.0
Harlan County	0	0.5	0.5	0.5	1	2.5

Notes: 1. If two recent bathymetric/LIDAR surveys were available for both the FP and MPP, score = 1. If only available for the MPP, score = 0.5. I no recent, reliable surveys were available, score = 0.
2. If enough flow-load measurements were available to define a reliable rating curve and the points were collected during the same time period as the measured volume change, score = 1. If there were enough flow-load measurements but the points were collected before the measured volume change, or if the points were collected during the same time period as the measured volume change but there were not a significant number of points, score = 0.5. If the points were collected before the measured volume change and there were not a significant number of points, score = 0.
3. If adjusting only the angle of the “bent down” portion, score = 1. If a small factor required, score = 0.5, if large factor required, score = 0.
4. If bulk density was measured, score = 1. If estimated but consistent with other lakes and no conflicting evidence, = 0.5. If estimated and conflicting evidence, score = 0.
5. If calibration matches more than two time periods, score = 1. If calibration matches only two time periods, score = 0.5.

As seen in Table 8-1, the quality is highest for Perry, Tuttle Creek, and Kanopolis. The rating curve quality is moderate for Clinton and Harlan County, and lowest for Milford and Wilson. For Milford, a new bathymetric survey combined with new LIDAR would allow an improved calibration to more recent flow/load data and result in a higher-quality rating curve. At Wilson, an investigation of bulk density and an inventory of additional sediment sources, in addition to new surveys, would be required to significantly improve the quality. As seen in Figure 8-1, there is no geographic trend to the quality score.

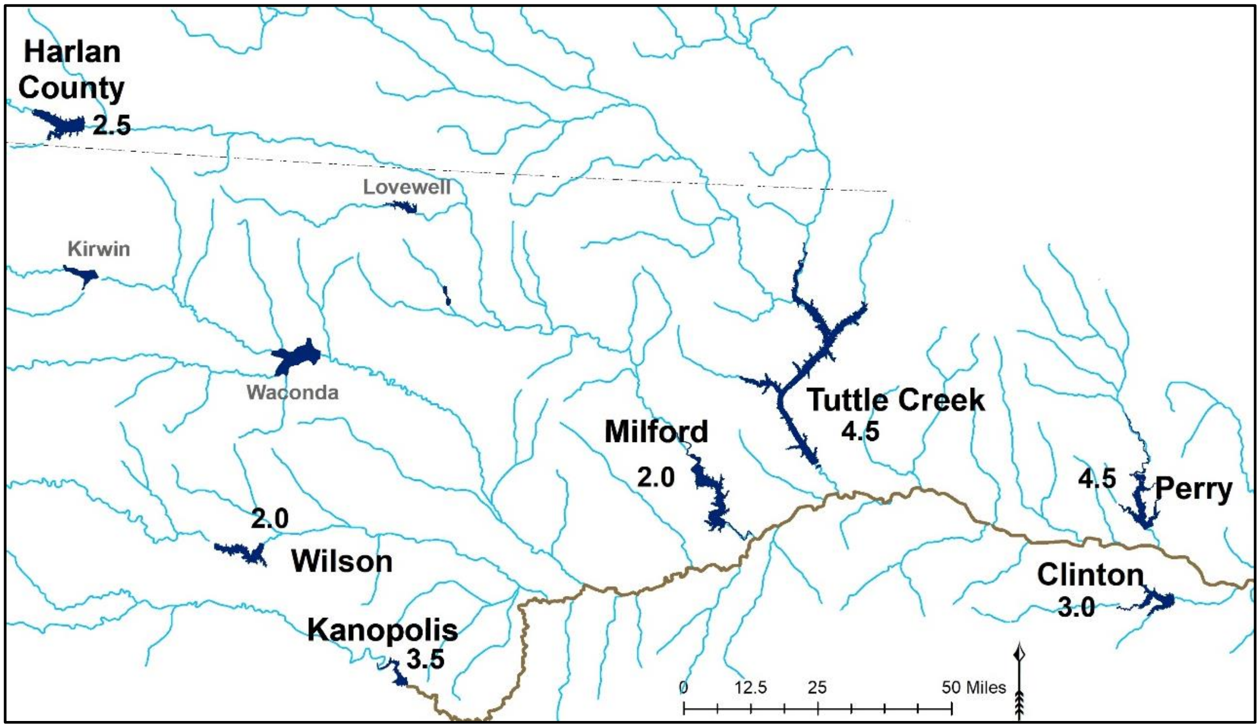


Figure 8-1. Quality Score for Rating Curve Calibration.

9.0 SEDIMENT TRAPPING DURING FLOOD CONTROL VS. MULTIPURPOSE POOL OPERATIONS

As seen in Table 3-1, the multipurpose pools of each of the lakes have accumulated much more sediment than the flood control pools, both in absolute volumes and as a percentage of the original pool volume. Daily flows, daily recorded pool elevations, and the calibrated rating curves from each lake were used to determine whether the sediment enters the lake during multipurpose or during flood control operations. This was defined in two ways: (1) As the daily pool elevation being at or below vs. above the multipurpose pool elevation and (2) as the daily pool elevation being at or below vs. above the Water Level Management Plan (WLMP) elevation. The WLMP reflects the seasonal target water surface elevations which may be higher or lower than the top of the multipurpose pool. Table 9-1 provides these percentages for each lake. The sediment is classified according to when it enters the lake, but represents the volume that eventually deposits (i.e., not including the small percentage that passes through the dam.)

Table 9-1. Incoming Sediment during Flood Control and Multipurpose Operations.

Lake	Deposition Years	Total Deposition (ac-ft)	% At/below multipurpose pool	% Above multipurpose pool	% At/Below WLMP	% Above WLMP
Clinton	1977-2019	23,812	9	91	17	83
Perry	1969-2009	100,864	7	93	24	76
Tuttle Creek	1965-2019	302,110	5	95	5	95
Milford	1967-2019	62,517	10	90	5	95
Kanopolis	1969-2019	21,598	3	97	23	77
Wilson	1973-2019	23,235	18	82	33	67
Harlan County	1957-2019	20,006	30	70	No WLMP	No WLMP

Note: Lakes are listed east to west.

As seen in Table 9-1, nearly all of the sediment that eventually deposits enters the lake while in flood control operations. This is true whether the definition for flood control operations is the lake level above the multipurpose pool or the lake level above the WLMP.

10.0 SEDIMENT CONCENTRATIONS

Figures 10-1 to 10-7 illustrate the concentration of incoming sediment, together with the 80% confidence intervals. These graphs represent the range of natural variability in the sediment concentrations in the river, i.e., what the concentration would be if the dam were not in place. As a first approximation, the lower bound can be thought of as a minimum target for naturalizing downstream sediment levels, and the upper bound can be thought of as a maximum limit to avoid excessive sediment releases.

The flow/concentration relationship is not monotonic or with a consistent slope. Rather the relationships in log space at low flows, moderate flows, and high flows exhibit separate slopes. By observation, at many of the lakes the sediment concentrations actually reverse at higher flows. This behavior translates into flatter flow-low curves as described earlier in the document. While the reasons for this phenomenon are unknown, it could be can be explained by either the supply limitation of easily erodible sediments or by sediment lost to the floodplain during overbank flows. Fourth-order polynomials through log-transformed data were used to reflect the overall trends in the data. These curves are valid fits over the range of observed data but should not be used for extrapolation.

Also evident in the data is a reduction in the variability in concentration at higher flows. While fewer measurements could by itself lead to the perception of less variability, visual inspection of the flow/concentration measurements suggests that physical reasons may drive the lower variability. The first possible reason for the reduction in variability is the supply limitation that drives the reduction in concentration. Higher concentrations are constrained by lack of readily available material. A second explanation for lower variability is that moderate flows can be achieved by a precipitation in only part of the watershed, and different sub-watersheds may have different sediment contributions. On the other hand, very high flows are only achievable when most if not all the entire watershed contributes, which reduces the spatial variability based on storm placement.

The confidence intervals were originally computed based on the statistics of the total sample and departure from the best-fit polynomial. However, the height of the intervals was driven by the high variability at moderate flows, which yielded confidence intervals at the highest flows that exceeded all the measured data points. A more refined approach was taken to consider differences in variability as a function of flow.

The 4th-degree polynomial through log-transformed data was used as a predictor for the mean of a normal distribution in a generalized additive model. To capture the changing variance of concentration with respect to flow, a concave-down function was needed to predict the sigma term in order to produce a local maximum in the middle with decreases in variance at both extremes. A quadratic function was used to ensure $d2\sigma/dx2<0,\forall x$. The result is a six-parameter model (plus two intercepts) in a hierarchical structure.

The interpretation should be looked at as "the conditional distribution of suspended sediment concentration, given an amount of flow." That is, $f(y|x)=N(\mu=b4x4+b3x3+b2x2+b1x+b0, \sigma=a2x2+a1x+a0)$ where y is SSC, x is flow, and N is the normal distribution with parameters μ and σ. Because the conditional distribution is normal, the best estimate of $y|x$ is $E[y|x]$ and is also the median of the conditional distribution, and it can be computed as μ. To obtain percentiles of the conditional distribution, the quantile function (inverse CDF) is used, $F\text{-}1(p;\mu,\sigma)$ where μ and σ are computed for the given value of flow. The result is overall a function that has a central tendency dictated by the 4th-order polynomial, with spread about the mean dictated by a concave-down quadratic.

These analyses were automated using R-scripts.

Clinton Lake possessed insufficient data to describe the variability using just the gage closest to the lake, so data from multiple upstream gages were compiled. As seen in Figure 10-1, this resulted in a flow-concentration curve for Clinton Lake with an anomalous shape. Additional data collection is underway.

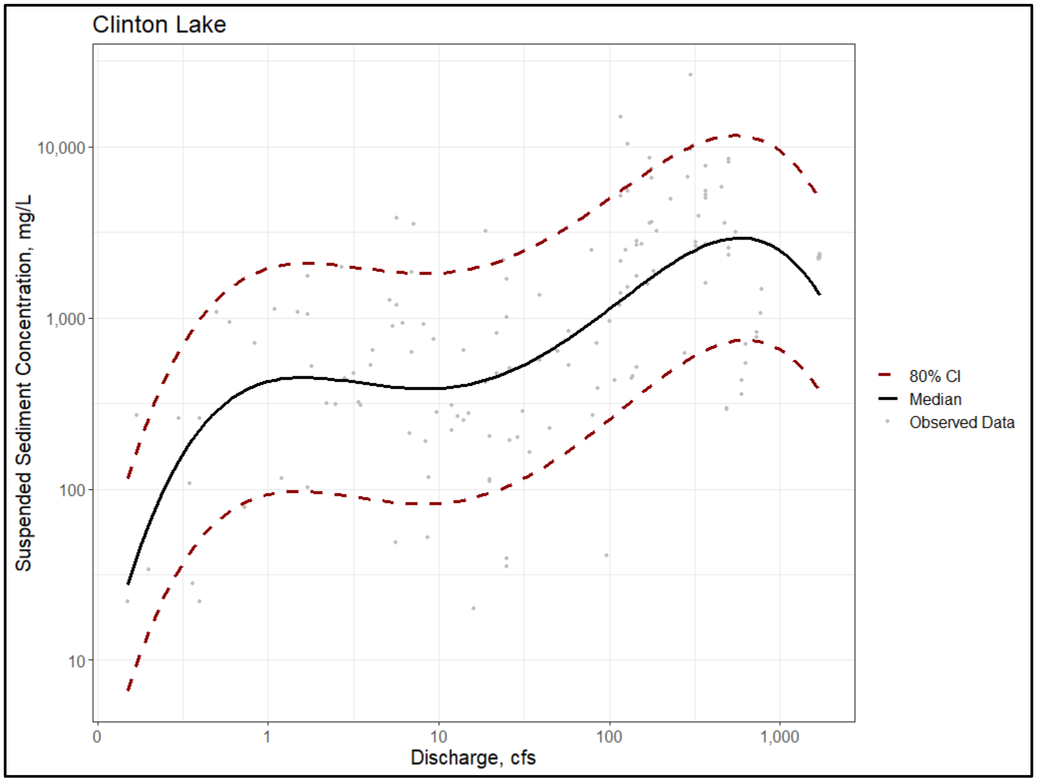


Figure 10-1. Incoming Sediment Concentrations to Clinton Lake.

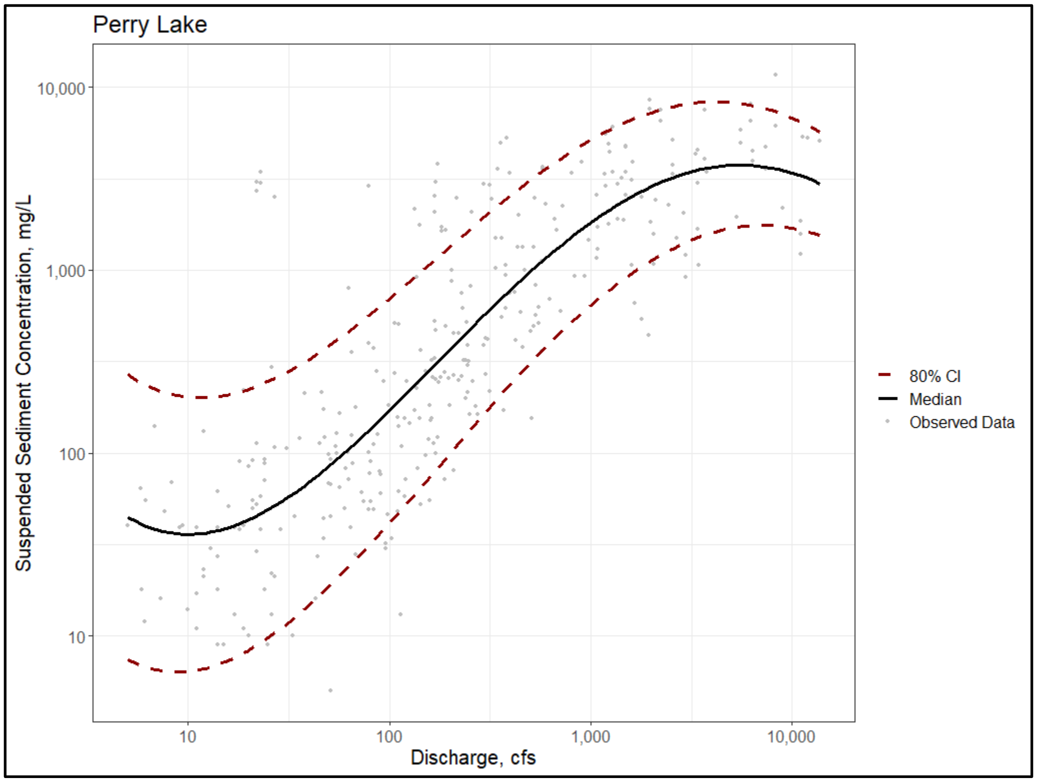


Figure 10-2. Incoming Sediment Concentrations to Perry Lake

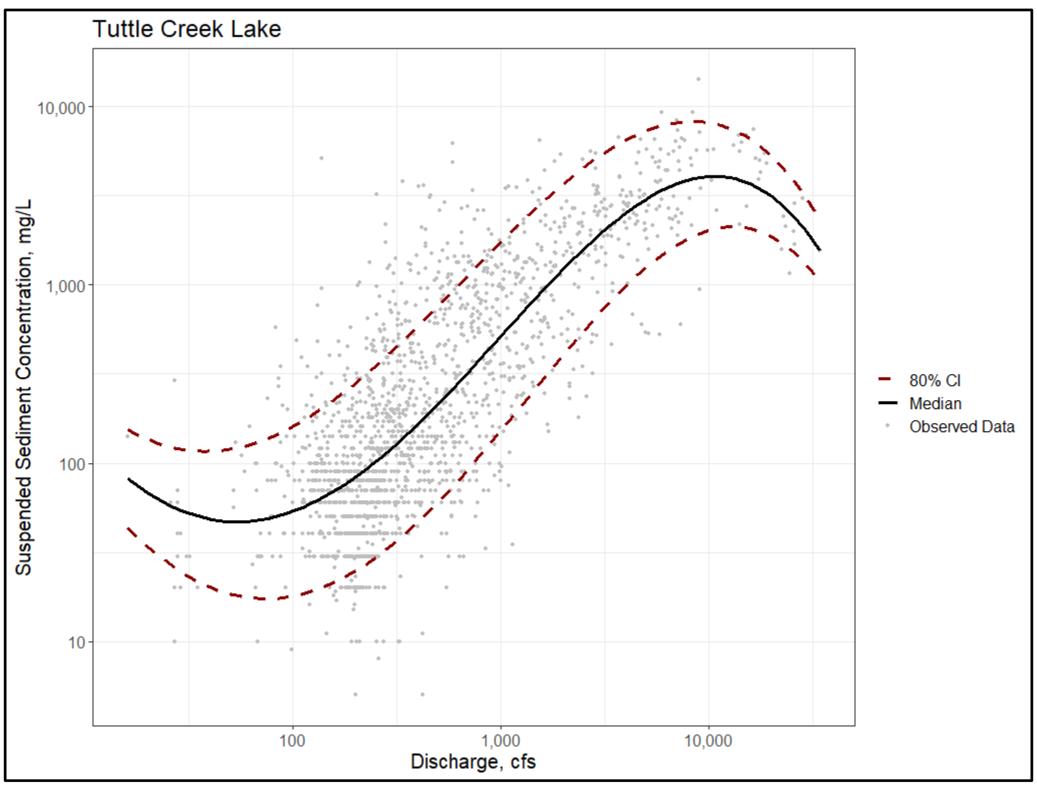


Figure 10-3. Incoming Sediment Concentrations to Tuttle Creek Lake

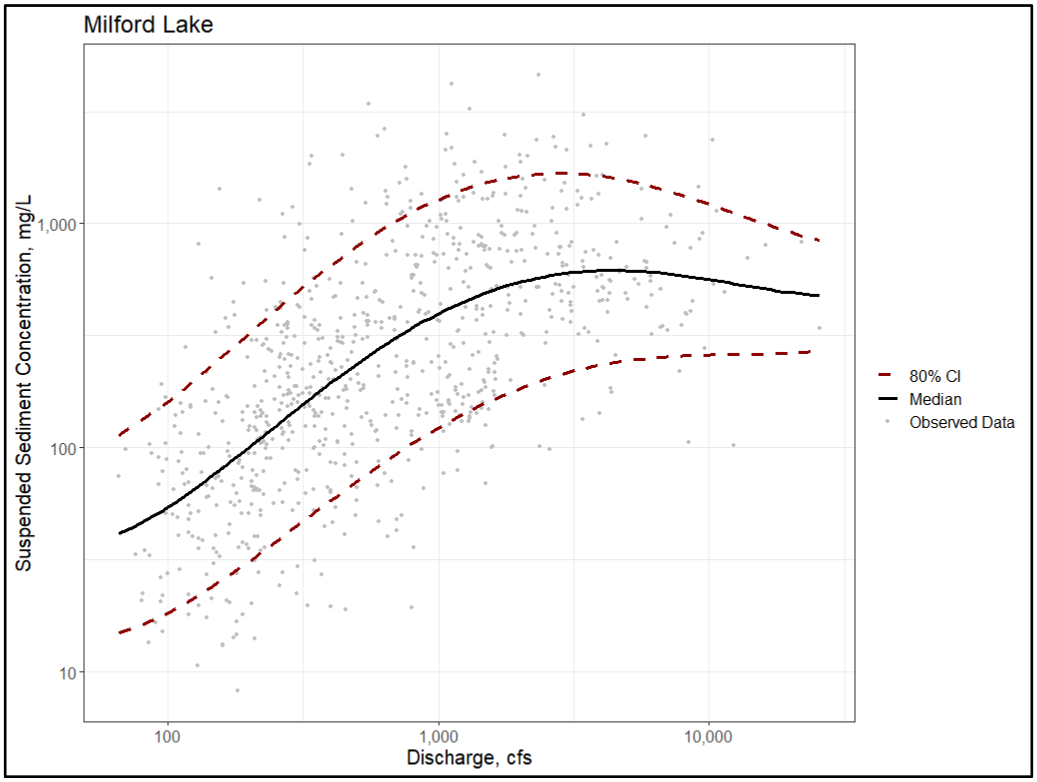


Figure 10-4. Incoming Sediment Concentrations to Milford Lake. *Earlier points are adjusted to account for the lowering of the flow/load rating curve over time.*

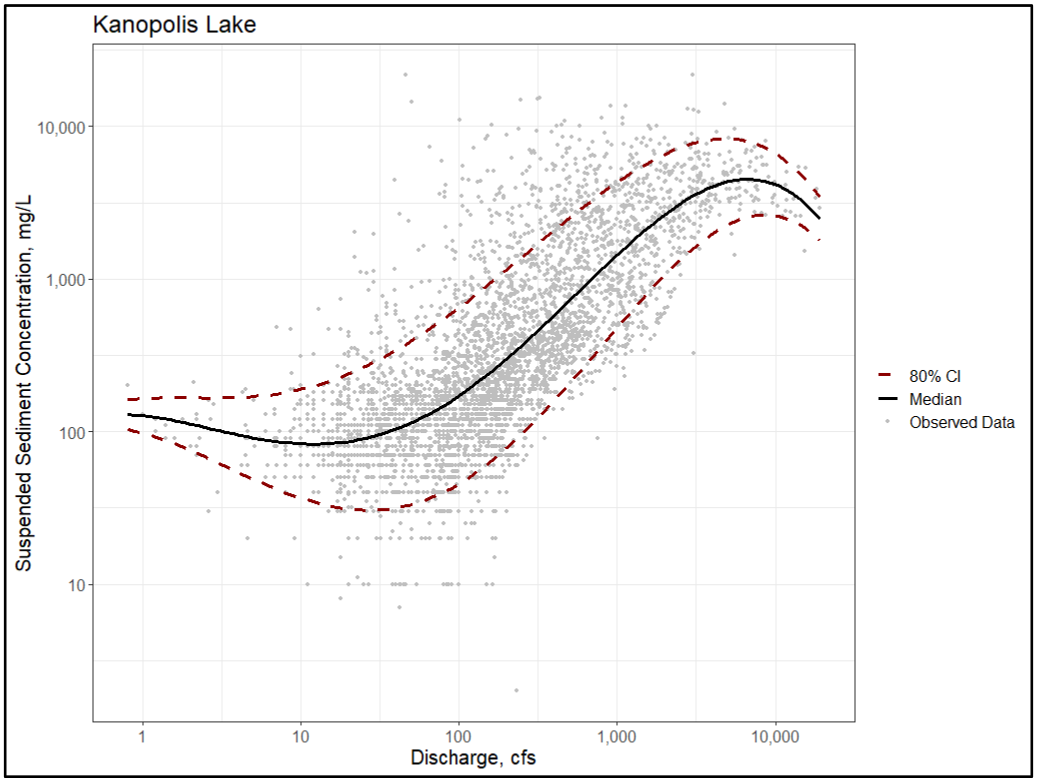


Figure 10-5. Incoming Sediment Concentrations to Kanopolis Lake.

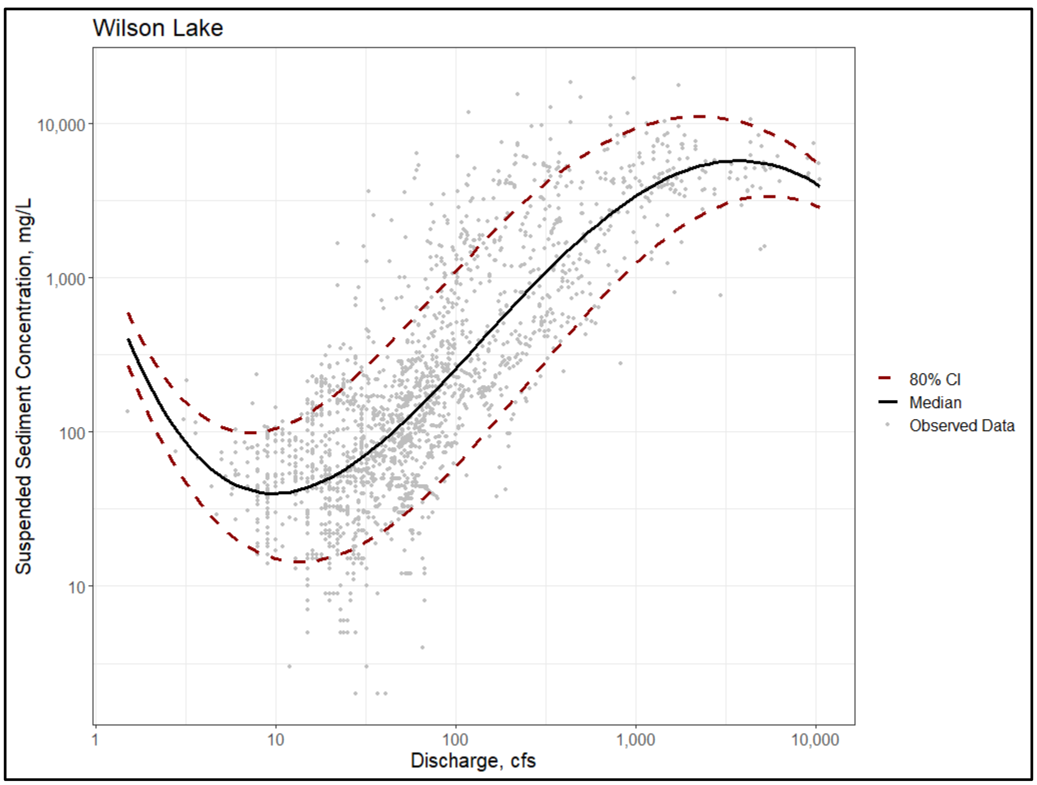


Figure 10-6. Incoming Sediment Concentrations to Wilson Lake.

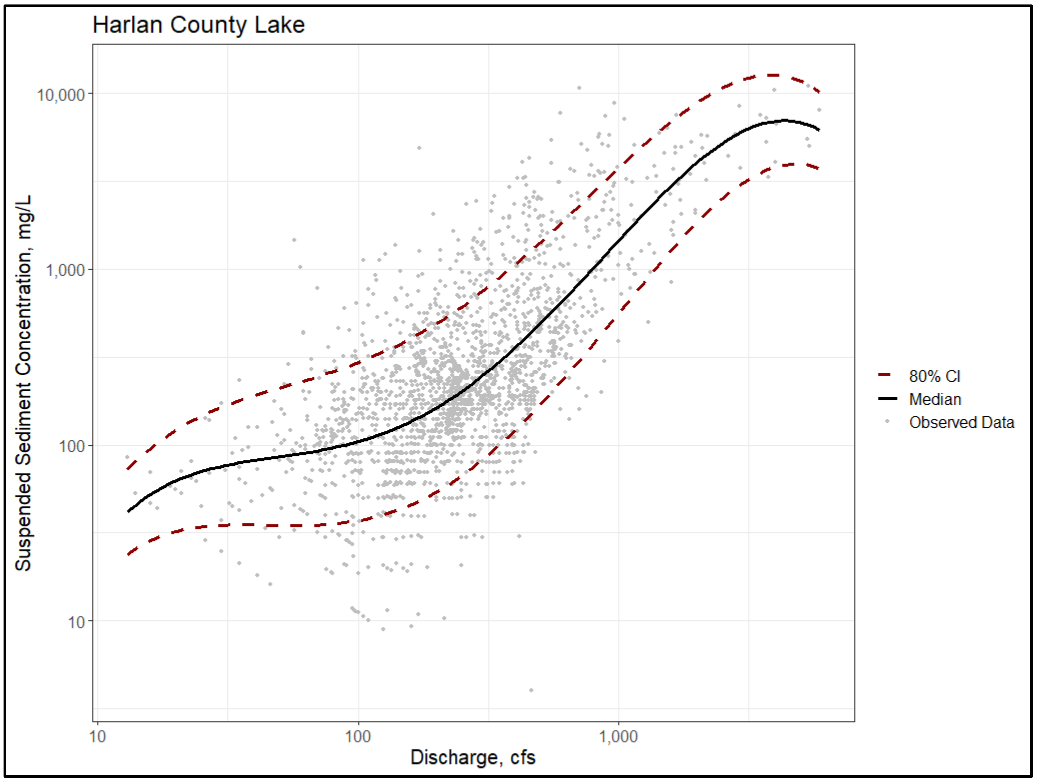


Figure 10-7. Incoming Sediment Concentrations to Harlan County Lake.

11.0 SEDIMENT CHEMICAL CONCENTRATIONS

Rivers and streams in the Kansas River Basin drain predominantly agricultural areas, including significant percentages of cropland. Milford Lake experiences frequent, extensive toxic algae blooms in part due to the high nutrient loading. Nutrient levels in the lake sediment and associated algae bloom issues will be discussed in a separate appendix and are not addressed here.

Metals and trace elements in sediment originate naturally within the sub-watersheds of the Kansas River basin and have been studied in several of the lakes. US Environmental Protection Agency (USEPA) has established two levels of concern in sediment for concentration of heavy metals, trace elements, and organochlorine compounds: the threshold-effects level (TEL) and the probable-effects level (PEL). Levels in the Kansas River Lakes are typical for sediments in the watershed and are generally not concerning.

Clinton Lake:

No reports were found related to the sediment chemical concentrations in Clinton Lake.

Perry Lake:

Tests conducted in 2001 by USGS (Juracek 2003) indicate the presence of 22 metals and trace elements of the 26 tested. Arsenic, chromium and copper exceeded the TELs but were less than the PELs in Perry Lake. For nickel, the concentrations exceeded both the TEL and most the PEL.

Tuttle Creek Lake:

Chemical characteristics of the deposited sediment were investigated by the USGS in a 2002 report (Juracek & Mau, 2002). A total of 44 elements were tested by the USGS at 17 coring sites in Tuttle Creek for a total of 41 sediment samples. Of the nine metals with published guidelines by the USEPA, eight of them exceeded the TEL and three of them exceeded the PEL in at least one of the samples. Organochlorine compounds were either not detected or were detected at concentrations below the TEL.

Milford Lake:

Chemical characteristics of the deposited sediment were investigated by the USGS in a 2000 report (Christensen & Juracek, 2000). The chemical concentration of 17 metal were tested in 20 samples from the deposited sediment in Milford Lake. Of the eight metal having guidelines published by the USEPA, five of them exceeded the TEL in at least one of the samples. However, none of the samples contained any metals with a concentration above the PEL.

Kanopolis Lake:

No previous reports or information were found related to the chemical concentrations of the sediment deposited in Kanopolis.

Wilson Lake:

No previous reports or information were found related to the chemical concentrations of the sediment deposited in Wilson.

Harlan County Lake:

Chemical characteristics of the deposited sediment were investigated by the USGS in a 2001 report (Christensen & Juracek, 2001). A total of 18 elements were tested by the USGS from twenty sample taken out of Harlan County Lake. Of the nine metals with published guidelines by the USEPA, six of them exceeded the TEL while none of them exceeded the PEL.

12.0 DELTA LOCATION AND VOLUME

Sedimentation rangelines allow the ability to quantify the rate of downstream delta progression over time. At each rangeline, the thalweg location was plotted against the distance from the dam to create delta progression curves. Figure 12-1 provides the delta progression curves for Tuttle Creek Lake, which demonstrates typical behavior for most other deltas in the Kansas River Basin. The delta progressed downstream rapidly between the first and second surveys, but the rate slowed considerably over subsequent surveys. This occurs because the narrow upstream reaches of the lake filled quickly with sediment, but the rate slowed as the delta entered the wider body of the lake. Individual sub-appendices contain similar plots for each lake. Note that the rise in the bed level upstream of the multipurpose pool between 2000 and 2009 corresponds with the shift in survey methods from rangelines to LIDAR. As LIDAR cannot penetrate the water surface, the "thalweg" plotted is actually the water surface. Hence the large vertical shift upstream of the multipurpose pool is a numerical artifact.

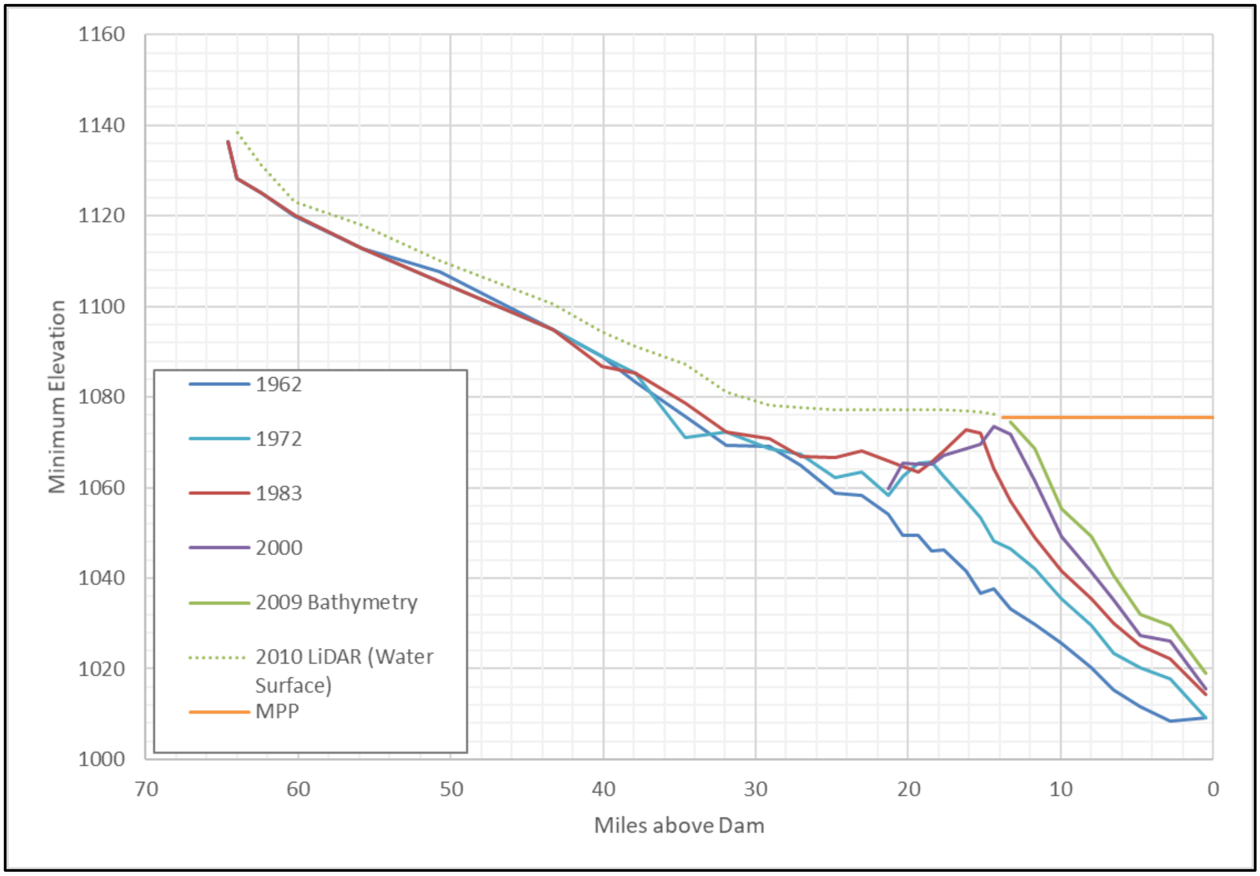


Figure 12-1. Example of delta progression at Tuttle Creek Lake.

At Clinton and Kanopolis, two detailed bathymetric surveys are available, which allows a more precise computation for delta location and progression. In those lakes, the delta progression was based on the elevations of a centerline rather than the thalweg of the cross sections. At lakes with one bathymetric survey, data at the rangeline locations was lifted from the bathymetric data and the thalweg was plotted to allow comparability with the older data. Table 12-1 lists the recent delta progression rate for each lake.

These rates are descriptions valid over the years indicated. As the rate changes over time, the rates in Table 12-1 will not be used to project future delta locations.

Table 12-1. Delta Progression Rates.

Lake	Survey Years	Delta Progression	Data Source
Clinton	2009-2019	None evident	Bathymetric Surveys
Perry	1979-2009	397	Sediment Rangelines/ Rangelines from Bathymetry
Tuttle Creek	1983-2009	581	Sediment Rangelines/ Rangelines from Bathymetry
Milford	1980-2009	228	Sediment Rangelines/ Rangelines from Bathymetry
Kanopolis	2007-2017	472	Bathymetric Surveys
Wilson	1984-1995	168	Sediment Rangelines
Harlan County	1962-2000	99	Sediment Rangelines/ Rangelines from Bathymetry

Note: Lakes are listed East to West.
Delta Progression is measured in feet per year (ft/year).

The deltas also grow in the upstream direction, but this effect was not quantified for this appendix.

13.0 DOWNSTREAM CHANNEL

Sediment trapping by the dams has induced bed degradation and bank erosion in the channels downstream of every lake except Wilson. Degradation rangelines allow monitoring and quantification of these effects. Table 13-1 summarizes the volume of bed degradation and bank erosion in the channels downstream from each dam.

Table 13-1. Downstream Channel Degradation.

Lake	Survey Years	Miles Downstream of the Dam	Degradation Rate (ac-ft/year)	Degradation/Sand Trapped (%)
Clinton	1977-2018	19.74	14.5	0.78
Perry	1967-2012	4.38	14.7	0.20
Tuttle Creek	1961-2019	9.24	93.1	0.24
Milford	1967-2016	6.3	48.7	0.30
Kanopolis	1948-2015	16	4.0	0.05
Harlan County	1988-2015	12	7.2	0.44

Note: Lakes are listed East to West.

14.0 SUMMARY AND CONCLUSIONS

This appendix summarized the existing conditions analysis for the seven USACE lakes in the Kansas River Basin. Multipurpose pools ranged from 4.7% to 39% full of sediment, relative to the original volume. The flood control pools range from a 0.02% gain in volume to a 3.7% loss in volume, relative to the originally specified pool volume. Deposition rates in the flood control pools are higher than these percentages would suggest, but newer LIDAR surveys in the flood control pool compute higher pool volumes than previous sediment rangeline surveys. In both the multipurpose pool and the flood control pool, deposition rates are highest in Tuttle Creek Lake.

Sediment deposition has induced a host of operations and maintenance problems, including access issues at boat ramps and marinas and impeding the function of the emergency gates. Multiple millions of dollars have already been spent on targeted dredging projects to help alleviate these issues.

Rating curves were developed that yield a mass of incoming sediment to each lake as a function of the daily flow rates at upstream gages. This mass was transformed into a volume of deposition using measured and estimated bulk densities, measured, and estimated trapping efficiencies, and the historic distribution of sediment between the flood control and multipurpose pools. These rating curves were calibrated to and verified against historic deposition over multiple time periods. The quality of the rating curves varied among lakes due to data availability and quality. Quality was highest at Perry, Tuttle Creek, and Kanopolis and lowest at Milford, Wilson, and Harlan County.

Downstream delta progression rates over recent surveys range from imperceptible to 581 ft per year.

Sediment chemical concentrations are typical for the watersheds.

Sediment concentrations in the lake inflows reflect much higher concentrations as flow increases. The lakes trap nearly all the incoming sediment, which has resulted in bed degradation and bank erosion in the downstream rivers. The downstream degradation equals 11% to 78% of the sand trapped in the lake over the same time.

Information on how these sedimentation processes will continue can be found in the Future Without Project appendices.

15.0 REFERENCES

Juracek, K. E. (2003, April 1). Sediment Deposition and Occurrence of Selected Nutrients, Other Chemical Constituents, and Diatoms in Bottom Sediment, Perry Lake, Northeast Kansas, 1969 - 2001, Water-Resources Investigations Report 03-4025, p. 20, 28-29

USACE. (1984). Lake Regulation Manual Perry Lake Kansas. Kansas City District: US Army Corps of Engineers. Volume No. 3.

USACE (2018). Perry Dam (NIDI KS 00009) Periodic Inspection No. 14 Periodic Assessment No. 01. Kansas City District.

Brune, G. M. (1953). Trap Efficiency of Reservoirs. American Geophysical Union.

Confidence and Prediction Intervals for Forecasted Values. Real Statistics Using Excel, www.real-statistics.com/regression/confidence-and-prediction-intervals/.

US Army Corps of Engineers ®
Kansas City District

Kansas River Reservoirs Flood and Sediment Study

Appendix D1.1: Clinton Lake Existing Condition Sedimentation

November 2022

Contents

1.0 INTRODUCTION

Clinton Lake Dam is located at river mile 22.2 on the Wakarusa River. The reservoir is located in Douglas County, Kansas. The project was authorized by the Flood Control Act of 1962 (Public Law 874, 87th Congress). The lake provides flood control, water conservation, water supply, recreation, and fish and wildlife enhancement. Clinton Lake has several non-federal Operation and Maintenance Repair, Replacement and Rehabilitation (OMRR&R) project. The City of Lawrence, Kansas is responsible for the maintenance of the toe road of the dam.

Construction of Clinton Lake Dam began in November of 1970, it was completed in August of 1975. Above the dam, the total drainage area is 367 square miles. Figure 1-1 gives the location of Clinton lake with respect to the Kansas River basin, while Figure 1-2 shows the watershed upstream of the lake.

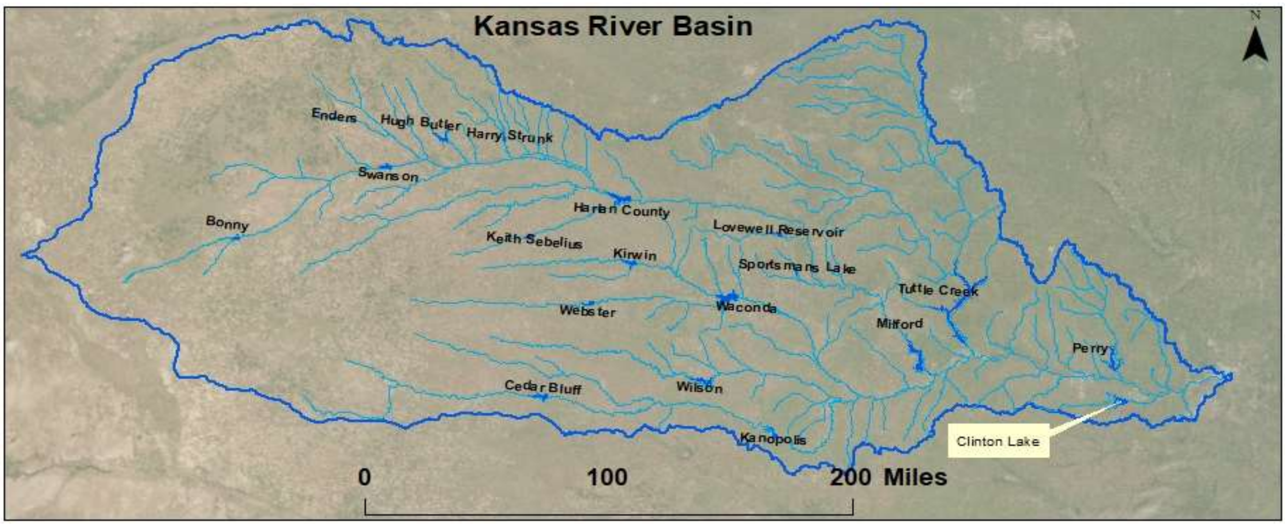


Figure 1-1. Overall Kansas River Basin Map

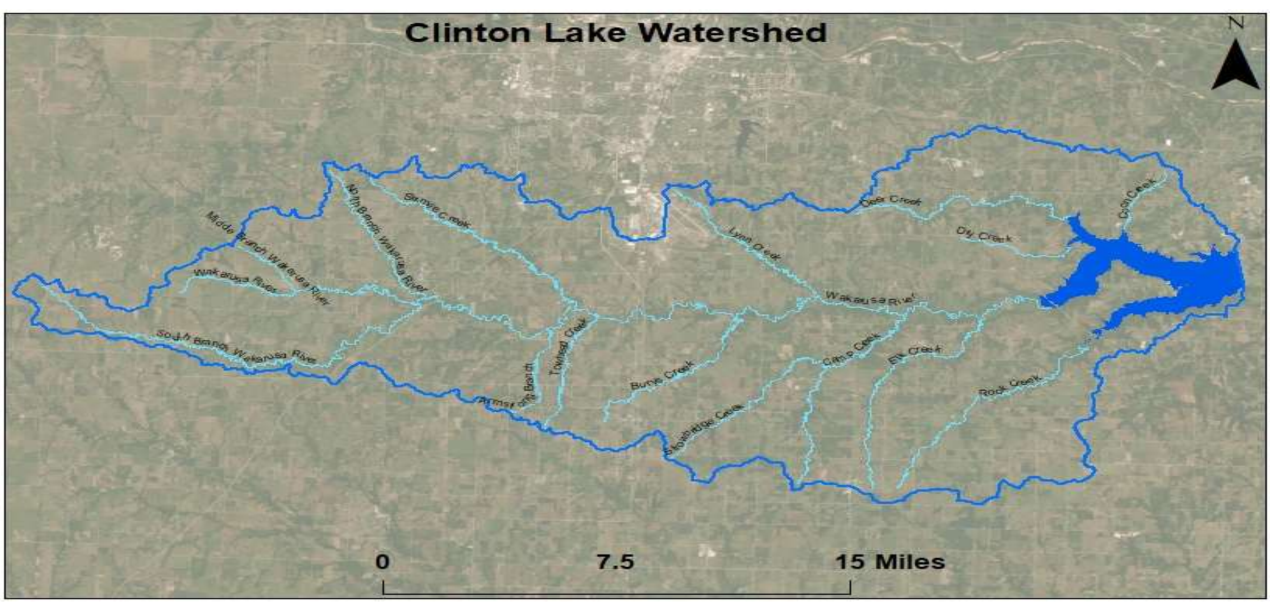


Figure 1-2. Clinton Lake Watershed

2.0 DAM INFRASTRUCTURE

The dam consists of a zoned rolled earthfill embankment of approximately 9,250 feet long with a 30-foot-wide crest and dike, controlled outlet works and an uncontrolled spillway. The intake tower consists of a working platform, trash fenders, streamline inlet, gate passages, a transition from the gate passages to the conduit, multilevel intake for the water supply/low flow within the trash fender structure, two wet wells for a single cable-hoist operated emergency gate, a dry well for two hydraulically operated service gates, an operating room, a service deck, and an entrance house. The water supply inlet is located above the flood control passageway.

There are four gates and two valves at the outlet works. There are two hydraulically operated 6.33 feet by 12.67 feet service gates to control intermediate and regular water releases. There is one emergency cable hoist operated wheel gate provided to block off the service gate passageway when one of the service gates is operative, a one water supply/low sluice gate and a 24-inch low flow hand operated Dezurik knife valve. This valve has a 24-inch discharge line connected to a 36-inch water supply line located in the southwest corner of the city of Lawrence, Kansas pumping plant. Discharge capacity of the outlet works with both gates fully open is 7,570 cfs with the water surface elevation at the top of the multipurpose pool (MPP). Regulation of the low flows are controlled by a butterfly valve located in a manhole attached to the stilling basin.

Table 2-1. Important Information Relating to the Dam Infrastructure

Parameter	Value
Multipurpose Pool Elevation	Capacity at elevation 875.5 feet-NGVD29
Lowest Elevation Outlet	823 ft
Low Flow Gates at This Low Elevation	One 24-in knife/gate valve, hand operated
Releases from Low Level Outlet	Operated when Q < 50 cfs, average of 91 days/year
Spillway Elevation	907.4 feet-NAVD88
Dam Elevation	926 ft (plus 2-ft overbuild for post construction settlement)
Service Gate	Two 6.33 by 12.67-foot hydraulically operated fabricated wheel gate
Emergency Gate	One 6.33 by 12.67-foot cable hoist operated fabricated wheel gate
Emergency Water Supply Gate	One 54 by 54 un standard hydraulically operated slide gate
Typical Tailwater Elevation	815-840 ft for discharge of 1,000cfs
Other Pipes Going Through the Dam or Embankment (e.g., Water Intakes)	42-in diameter water supply line

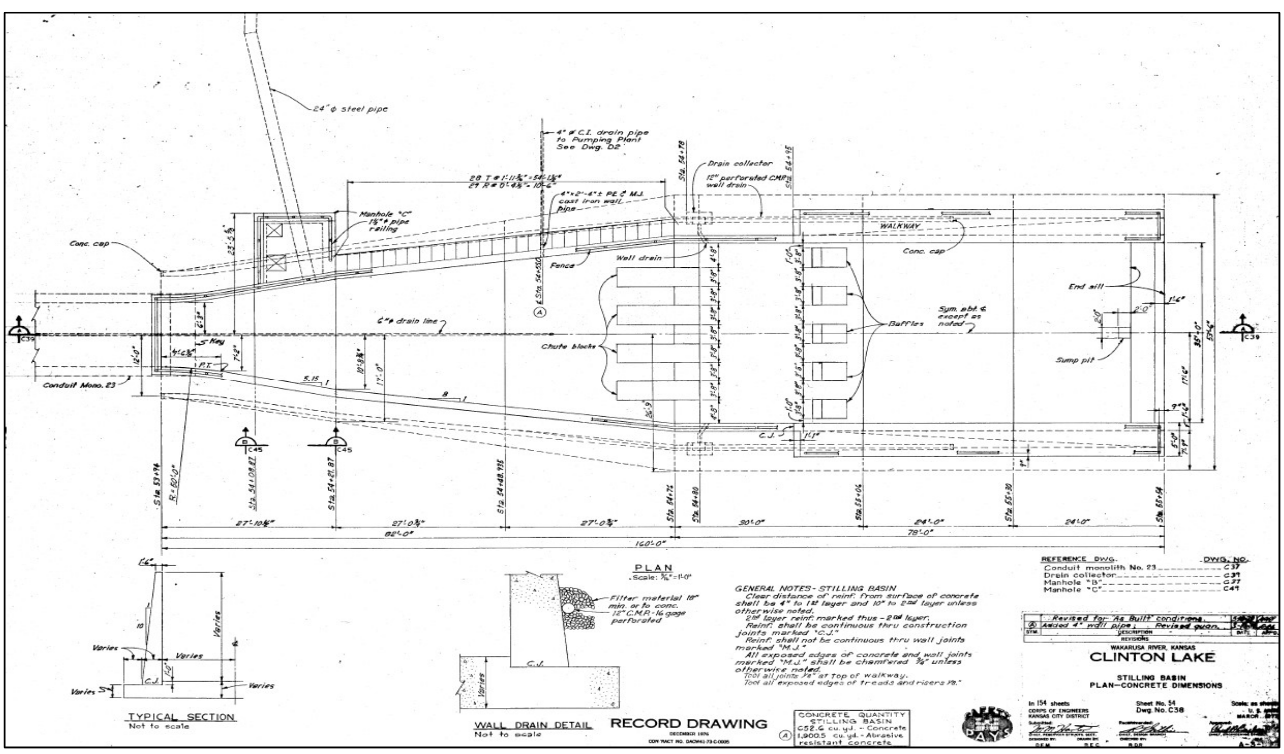


Figure 2-1. Stilling Basin

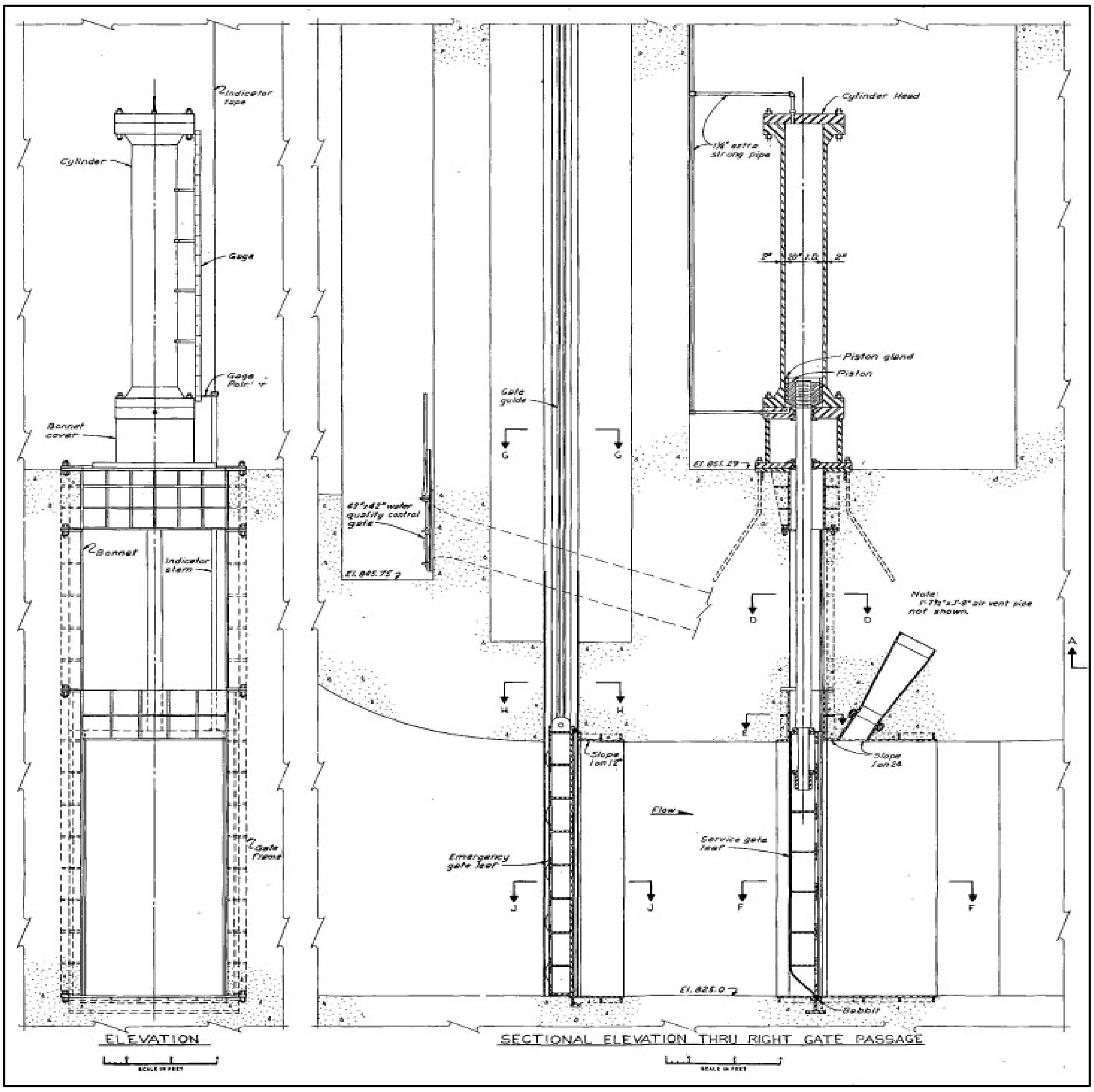


Figure 2-2. Outlet Works Control Gates

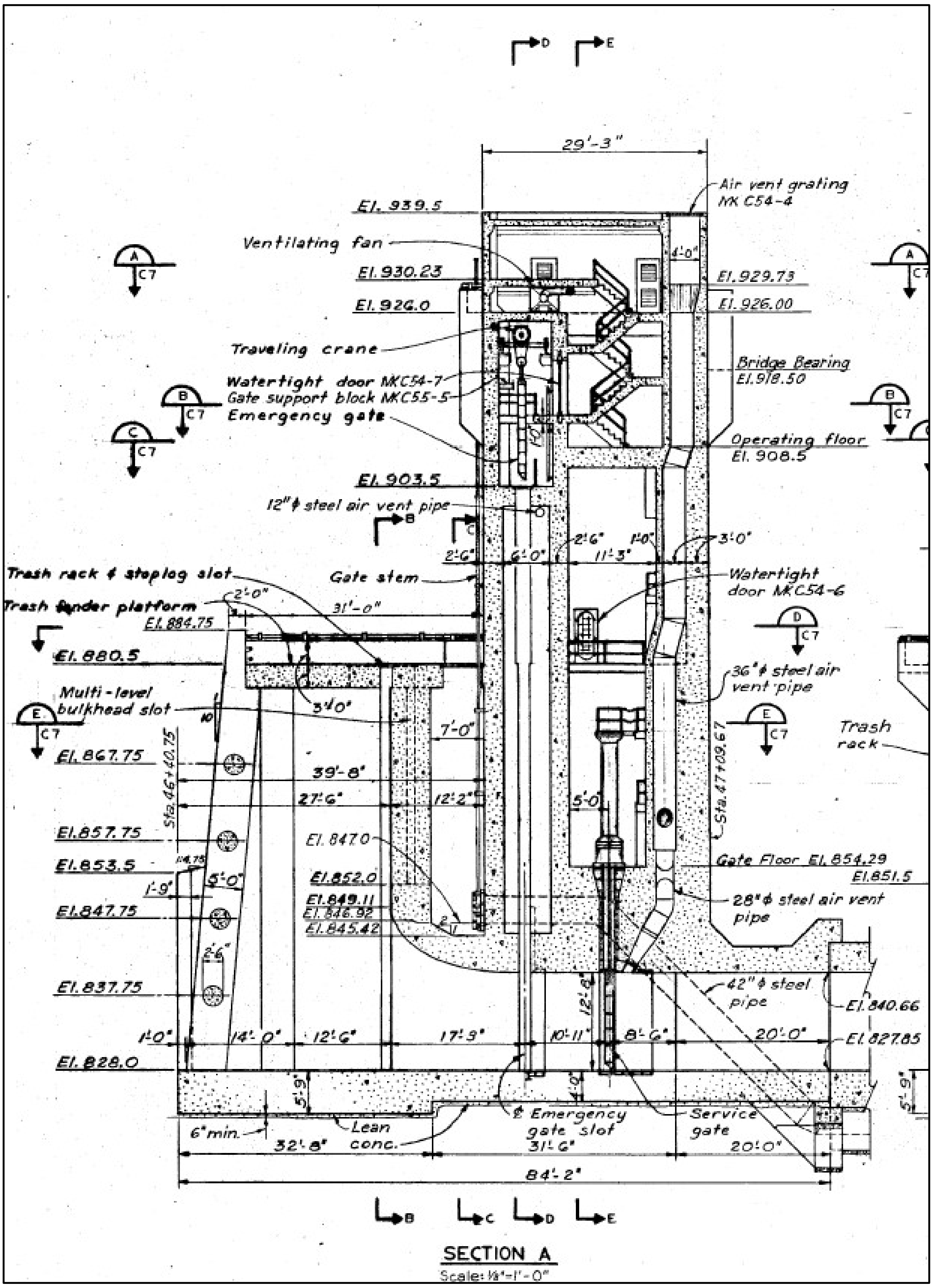


Figure 2-3. Intake Tower

3.0 SEDIMENTATION EFFECTS ON OPERATIONS & MAINTENANCE

Currently there are no sedimentation impacts on the services gates and the lake in general. The first potential boat ramp to be impacted by sedimentation is in the Woodridge Park.

4.0 EXISTING CONDITION STORAGE ELEVATION CURVES

The most current storage elevation table for Clinton Lake has been used operationally from March 2012 to present. The pool volumes were originally computed by Surdex Corporation in NAVD88 using results of the KBS August 2009 bathymetric survey combined with 2009 and 2010 LiDAR.

A new capacity table was calculated using the 2019 bathymetric survey combined with 2015 and 2010 LiDAR. The following table shows the storage elevation for Clinton Lake.

Table 4-1. Storage Elevation Curve for Clinton Lake.

Elev. (ft) NAVD88	Volume (ac-ft)	Elev. (ft) NAVD88	Volume (ac-ft)	Elev. (ft) NAVD88	Volume (ac-ft)	Elev. (ft) NAVD88	Volume (ac-ft)
834	0	**859**	26,412	**883**	174,452	**907**	454,078
835	0	**860**	29,606	**884**	183,643	**908**	468,733
836	0	**861**	33,010	**885**	193,034	**909**	483,656
837	1	**862**	36,658	**886**	202,631	**910**	498,839
838	5	**863**	40,500	**887**	212,438	**911**	514,262
839	14	**864**	44,539	**888**	222,447	**912**	529,937
840	34	**865**	48,873	**889**	232,674	**913**	545,856
841	79	**866**	53,586	**890**	243,104	**914**	562,009
842	179	**867**	58,597	**891**	253,730	**915**	578,389
843	357	**868**	63,919	**892**	264,552	**916**	594,994
844	634	**869**	69,550	**893**	275,565	**917**	611,837
845	1,053	**870**	75,461	**894**	286,773	**918**	628,924
846	1,656	**871**	81,604	**895**	298,177	**919**	646,266
847	2,442	**872**	87,969	**896**	309,819	**920**	663,852
848	3,416	**873**	94,572	**897**	321,711	**921**	681,682
849	4,561	**874**	101,448	**898**	333,826	**922**	699,765
850	5,915	**875**	108,705	**899**	346,155	**923**	718,107
851	7,463	**875.58**	113,032	**900**	358,735	**924**	736,725
852	9,201	**876**	116,218	**901**	371,586	**925**	755,633
853	11,127	**877**	123,935	**902**	384,704	**926**	774,847
854	13,232	**878**	131,804	**903**	398,090	**927**	794,366
855	15,512	**879**	139,869	**903.48**	404,602	**928**	814,182
856	17,958	**880**	148,191	**904**	411,718	**929**	834,117
857	20,566	**881**	156,726	**905**	425,585	**930**	854,067
858	23,392	**882**	165,476	**906**	439,694	**-**	-

Notes: Elevation is in feet NAVD88, Volume is in acre-feet

The following table provides a summary by pool of official storage elevation curves with its respective data type. In Table 3 the Flood Control Pool (FP) does not include the Multipurpose pool (i.e., it lists the volume above the Multipurpose Pool top elevation but below the Flood Control Pool top elevation).

Table 4-2. Pool Volumes Over Time.

Year	Multipurpose Pool Volume (ac-ft)	Flood Control Pool Volume (ac-ft)	Data Type
1977	129,171	268,367	Computed from 1965 topographic maps
1990	125,334	268,783	Survey of sediment ranges
2009	118,699	292,496	June 2009 bathymetric survey combined with 2009 and 2010 LiDAR data, initially computed by Surdex Corporation
2019	113,032	291,570	September 2018 bathymetric survey combined with 2015 and 2010 LiDAR

From 1977 to 2019, the multipurpose pool lost 16,139 ac-ft of storage to sedimentation. This represents 12% of the original multipurpose pool volume. The average annual rate of loss was 384 ac-ft/year or 0.30% of the original volume/year.

From 1977 to 2019, volume computations would indicate that the flood control pool gained 23,203 ac-ft of storage to sedimentation. Moreover, the flood control pool gains volume every year. This erroneous gain in volume is most likely the result of the change in survey methods from digitized topographic maps to sediment range lines to LIDAR of varying resolutions. Similar issues have been noted at other lakes in the basin when the survey method changes. This makes using these volumes to compute the volume of deposition in the flood control pool unreliable.

5.0 TRAPPING EFFICIENCY

The trapping efficiency for Clinton Lake was calculated by the U.S. Geological Survey (USGS) (Juracek 2013) to be 97%. This trapping efficiency was used during the analyses presented in this appendix.

5.1 Depositional Volume

Bathymetric surveys were conducted on Clinton Lake in 2009 and 2019. The 2009 data sources were combined two 2010 LiDAR datasets with differing resolutions: Douglas County LiDAR (2-m resolution) and Osage County LiDAR (1-m resolution). The USGS NED surface with a 1/3 arc-second resolution was used for remaining areas not covered by the two county LIDAR surfaces. These data were processed and combined by Surdex Corporation.

The 2019 data sources consisted of 2015 LiDAR for Douglas and Osage County with a 1-meter resolution. A small portion of the flood control pool that was not covered by the 2015 LIDAR used the 2010 LiDAR, as seen in Figure 6-1. Figure 5-2 shows the 2009 and 2019 bathymetric surveys that were collected on Clinton Lake.

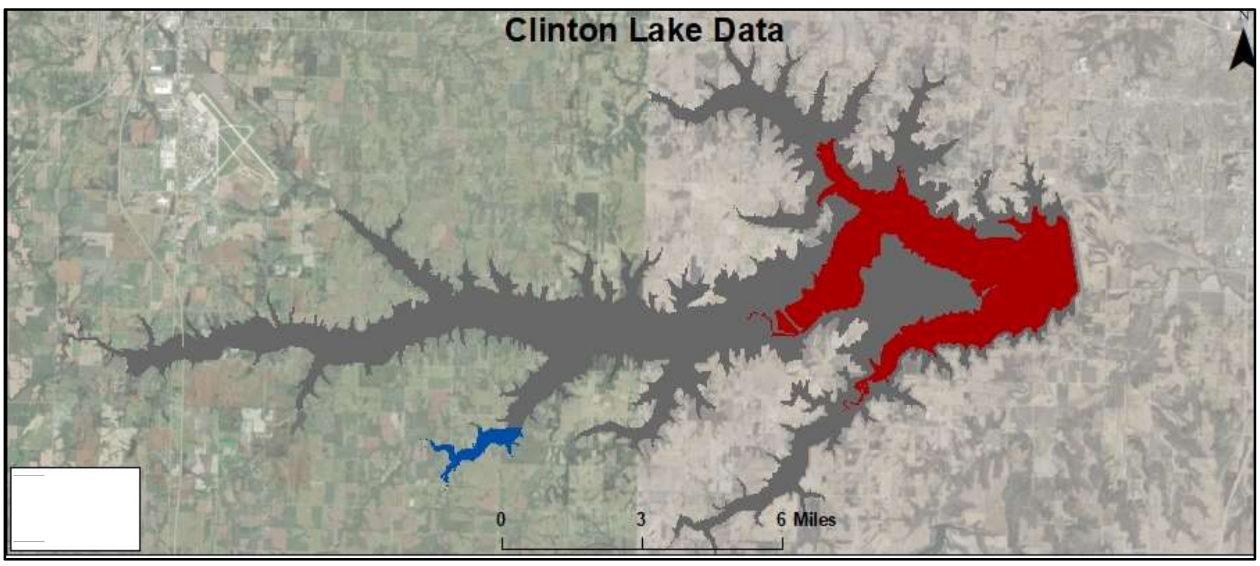


Figure 5-1. Clinton Lake Data Sources.

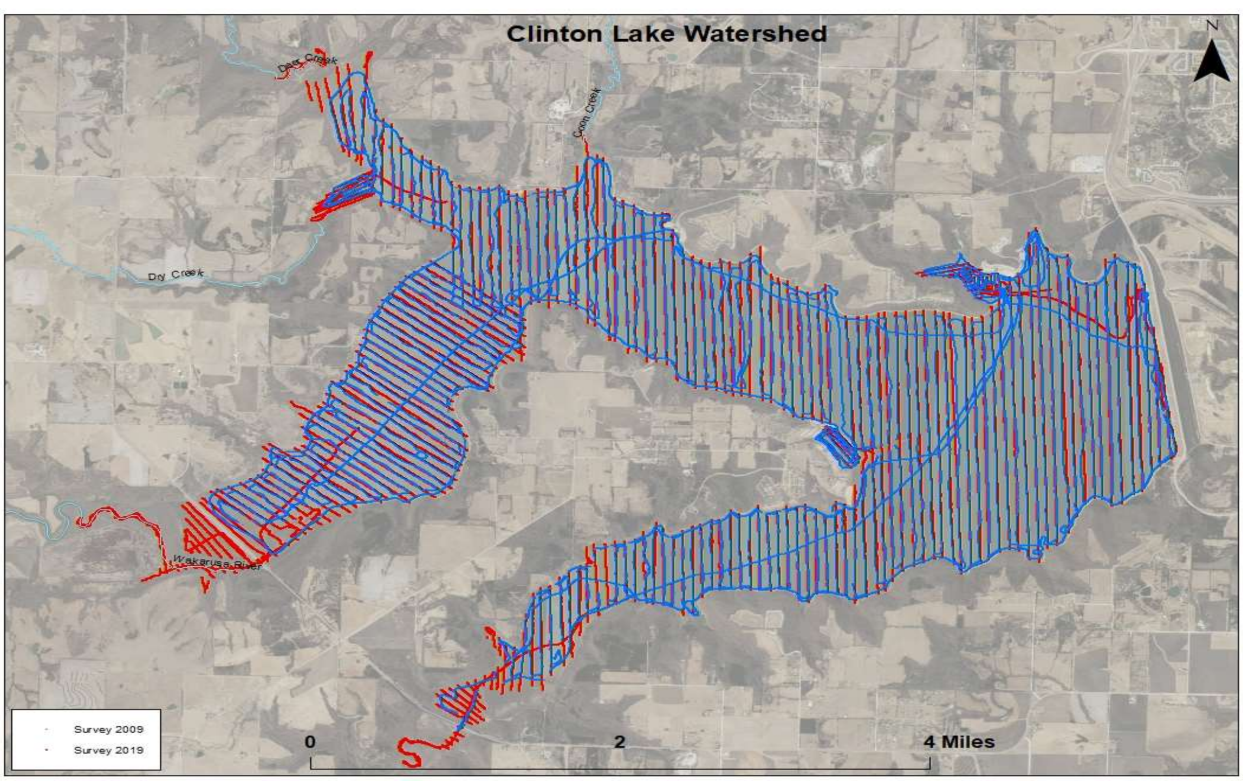


Figure 5-2. Bathymetric Survey Data for Clinton Lake.

Table 6-1 provides the multi-purpose pool and flood control pool deposition from each survey.

Table 5-1. Deposition Between Time.

Year	Deposition-MP (ac-ft)	Deposition-FC (ac-ft)
1977-1990	3,837	-416
1990-2009	6,635	-23,713
2009-2019	5,667	926

As stated before, the computed negative deposition in the flood control pool is not reliable and is most likely the result of a change in survey methods. Similar shifts have been observed at other lakes.

6.0 INCOMING SEDIMENT LOADS

USACE and USGS have collected a limited number of paired flow/sediment concentration measurements from 2011 to 2012 at gage 06891260 on the Wakarusa River near Richland, Kansas. These data were downloaded from the sediment data portal (https://cida.usgs.gov/sediment/). Two additional data points collected in 2019 by USGS, specifically for this study, were also included. Discrete suspended-sediment concentrations were multiplied by the instantaneous flow and a constant of 0.0027 to compute sediment load in tons per day. The following figure plots the flow/load data.

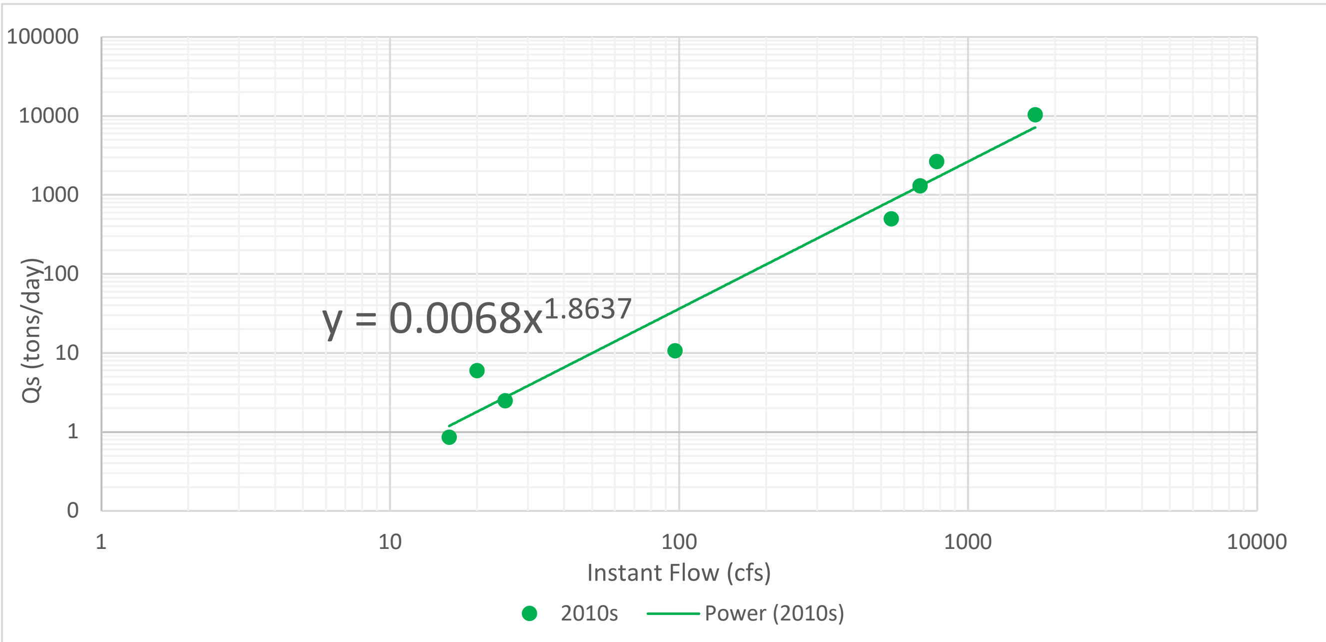


Figure 6-1. Suspended-Sediment Data

Daily historical lake inflows flow from December 1977 to December 2019 was provided by NWK-EDH-C (Water Management). This data was used for developing of a rating curve using the following steps:

1. Estimate the best fit regression line of the form $Q_s = aQ^b$ using log-log linear regression.
2. Correct for bias using the Duan correction factor (Duan 1983). The Duan E value was 1.24.
3. Most lakes in the Kansas River basin with sufficient data exhibit a bending down of the flow-load curve. Similar behavior was assumed for Clinton Lake to extrapolate the upper end of the curve. The flow for the second to the last point of the rating curve was set to the 1.5-year flow with the sediment computed using the rating curve equation with the Duan correction. The flow for the last point was set to be ten times the 1.5-year flood, and the sediment load was determined through calibration, as explained later.
4. Add 5% to account for bed load to create a total load rating curve.
5. Apply this rating curve to daily historical flow from December 1977 to March 2020 to determine the cumulative mass of sediment entering the reservoir.
6. Multiply by the appropriate percentages to determine the cumulative mass of clay, silt, and sand entering the lake.

Figure 6-2 shows that final calibrated rating curve and Table 6-1 gives the flow/load points for the rating curve.

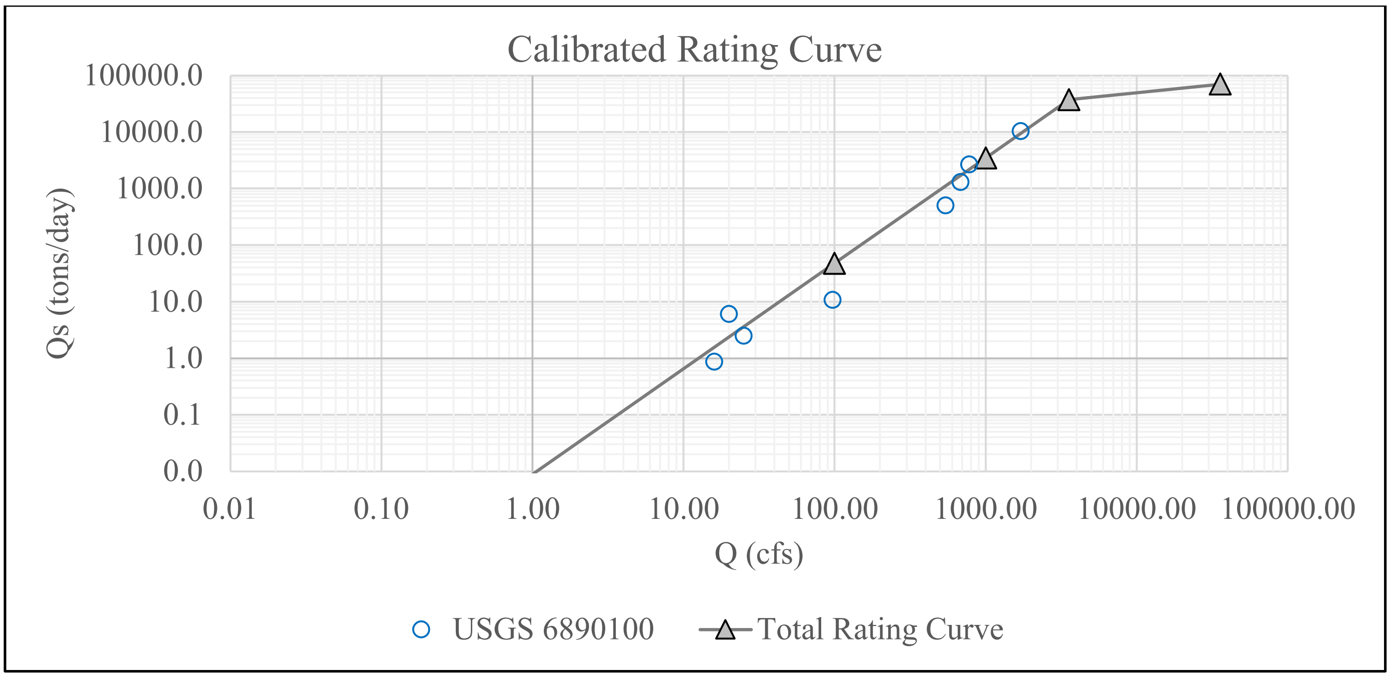


Figure 6-2. Calibrated Rating Curve

Table 6-1. Calibrated Rating Curve Points

Flow (cfs)	Load (T/d)
1	0
100	47
1,000	3,466
3,570	37,142
35,700	70,570

This analysis found the incoming load to be 49.5% clay, 40.5% silt, and 10% sand/gravel.

Table 6-2. Incoming Sediment to Lake Clinton from 1977 to 2019

Years	1977-2019
Total Incoming Sediment (tons)	15,393,289
Total Incoming Clay Fraction	7,619,678
Total Incoming Silt Fraction	6,234,282
Total Incoming Sand Fraction	1,539,329

7.0 BULK DENSITY AND CONSOLIDATION OF SEDIMENT DEPOSITS

The incoming load can provide an estimate of the composite bulk density, via the following equation.

$$\boldsymbol{\gamma_c} = \frac{\mathbf{1.0}}{\left(\left(\frac{\boldsymbol{F}}{\gamma}\right)_{\boldsymbol{clay}} + \left(\frac{\boldsymbol{F}}{\gamma}\right)_{\boldsymbol{silt}} + \left(\frac{\boldsymbol{F}}{\gamma}\right)_{\boldsymbol{sand}}\right)}$$

Where γc is the composite bulk density

F: the fraction of clay, silt, or sand

γ: for clay, silt and sand assumed to be 30 pcf, 65 pcf, and 93 pcf respectively.

The composite bulk density of the incoming load is 42.01 lb/ft^3. This is similar to that computed for nearby Perry Lake.

For Clinton Lake, there was no recorded measurements of bulk density for the multipurpose or flood control pool. As the composite bulk density computed from the incoming load and percent of fines from Clinton Lake is similar to nearby Perry Lake, the bulk densities from Perry Lake were assumed to be valid for Clinton as well. The multipurpose pool bulk density used in these computations is 39.38 lb/ft^3 and the flood control pool bulk density is 57.36 lb/ft^3.

8.0 CALIBRATING LOAD AND DEPOSITIONAL PARAMETERS

To calibrate the rating curve, the deposition computed with the rating curve was compared to the volume of deposition calculated from the lake surveys. Due to the unrealistic "negative deposition" in the floodplain, only the multipurpose pool volume was used for calibration. This was accomplished following these steps:

1. Apply the rating curve to the daily flow while adjusting for bedload.
2. Compute the mass of trapped sediment for the multipurpose pool by applying the trapping efficiency and mass ratio to the day of incoming lake sediment.
3. Compute mass of trapped load sediment for the flood control pool by applying the mass ratio to the day of incoming lake sediment.
4. Compute the deposited volume by dividing the trapped load by the bulk density.
5. Sum each day of deposition to yield a total accumulated volume.
6. Compare the total accumulated volume based on load (VLoad) to the deposition calculated via the lake surveys (Vsurvey).
7. Adjust the sediment load at the highest flow in the rating curve listed in Table 6-1 (i.e., the "bent down" portion of the curve) until the ratio of Vload to Vsurvey = 1.

No time period used for calibration yielded particularly good results for the other time periods. Calibration to the most recent time period (2009 to 2019) required a "bent down" portion that was much steeper than at other lakes in the basin. Calibration to the middle time period (1990 to 2009) yielded a curve that was more consistent with other lakes and so was selected for this analysis. Table 8-1 summarizes these parameters.

Table 8-1. Analysis Summary

Parameter	Value
FP Bulk Density (lb/ft^3)	57.36
MPP Bulk Density (lb/ft^3)	39.38
a in $Q_s = aQ^b$	0.0068
b in $Q_s = aQ^b$	1.8637
Trapping efficiency	97%

Table 8-2. Computed vs. Measured Multipurpose Pool Deposition

Year	Days/Year with Flows above 3,570 cfs	Multipurpose Pool Accumulated Computed (ac-ft)	Multipurpose Pool Survey (ac-ft)	Ratio
1977-1990	3.3	4,373	3,837	1.14
1990-2009	3.6	6,625	6,635	1.00
2009-2019	3.4	3,098	5,667	0.55

Due to the lack of site-specific bulk density measurements, the relatively few flow/load points, and the lack of reliable flood control pool deposition estimates, this calibrated rating curve remains fairly uncertain.

9.0 SEDIMENT TRAPPING DURING FLOOD CONTROL VS. MULTIPURPOSE POOL OPERATIONS

Using the calibrated rating curve computed above, the sediment that enters the lake was apportioned according to whether it enters during flood control or multipurpose pool operations. Table 9-1 indicates the quantity of deposition from sediment entering when the reservoir is in flood control operations (i.e., with a water surface above the multipurpose pool elevation), vs. multipurpose pool operations (i.e., when the water surface is at or below the multipurpose pool elevation).

Table 9-1. Deposition Amounts Entering during Flood Control and Multipurpose Operations

Deposition	Deposition 1977 - 2019 (ac-ft)	% of Total
Total Deposition	23,812	100
Multipurpose Operations (below 891.5 ft)	2,027	9
Flood control Operations (above 891.5 ft)	21,786	91

The deposition was also computed according to the following Water Level Management Plan (WLMP) elevations (see Figure 9-1) used in the Lake Regulation Manual (USACE 1984):

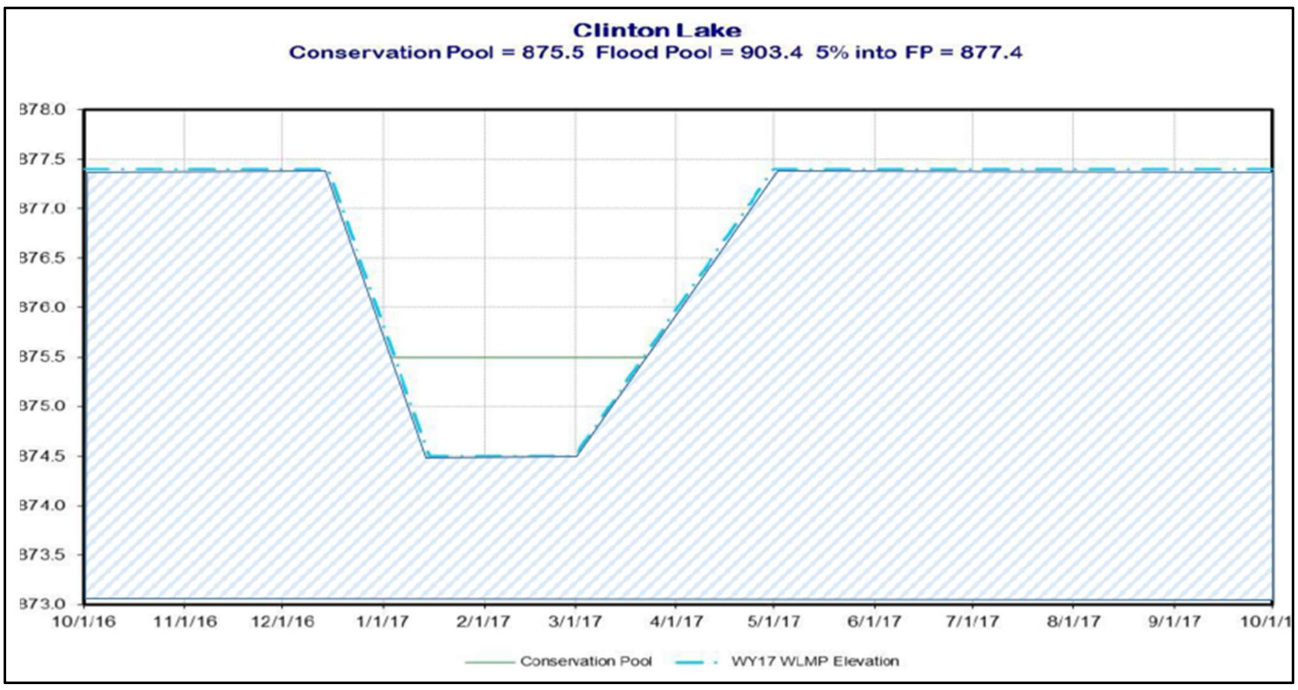


Figure 9-1: Clinton Lake Water Level Management Plan Elevations.

Table 9-2: Deposition Amounts- WLMP

Deposition	Deposition 1977 - 2019 (ac-ft)	% of Total
Total Deposition	23,812	100
Above WLMP	19,824	83
Below WLMP	3,988	17

By either computation, it is clear that most of the sediment enters the lake during flood control operations.

10.0 SEDIMENT CONCENTRATIONS

Gage station 06891260 (Wakarusa River near Richland, Kansas) did not have enough data to determine the range of natural variability in sediment concentration for a given incoming flow. Nearby gage stations 385329095353400 (Wakarusa River near Berryton, Kansas), 385351095542900 (Wakarusa River 5 miles west of Auburn, Kansas), 385350095525300 (Wakarusa River 4 miles west of Auburn, Kansas), 385718095464000 (Sixmile Creek Tributary 5 miles northeast of Auburn, Kansas) and 385632095464000 (Sixmile Creek Tributary 4 miles northeast of Auburn, Kansas) were used to augment the analysis of the natural incoming sediment concentrations. Figure 10-1 shows that there is only a slight correlation between flow and concentration. This may be due to the lack of data for higher flows. See Table 10-1 for a summary of the gauges and Figure 10-2 for their locations.

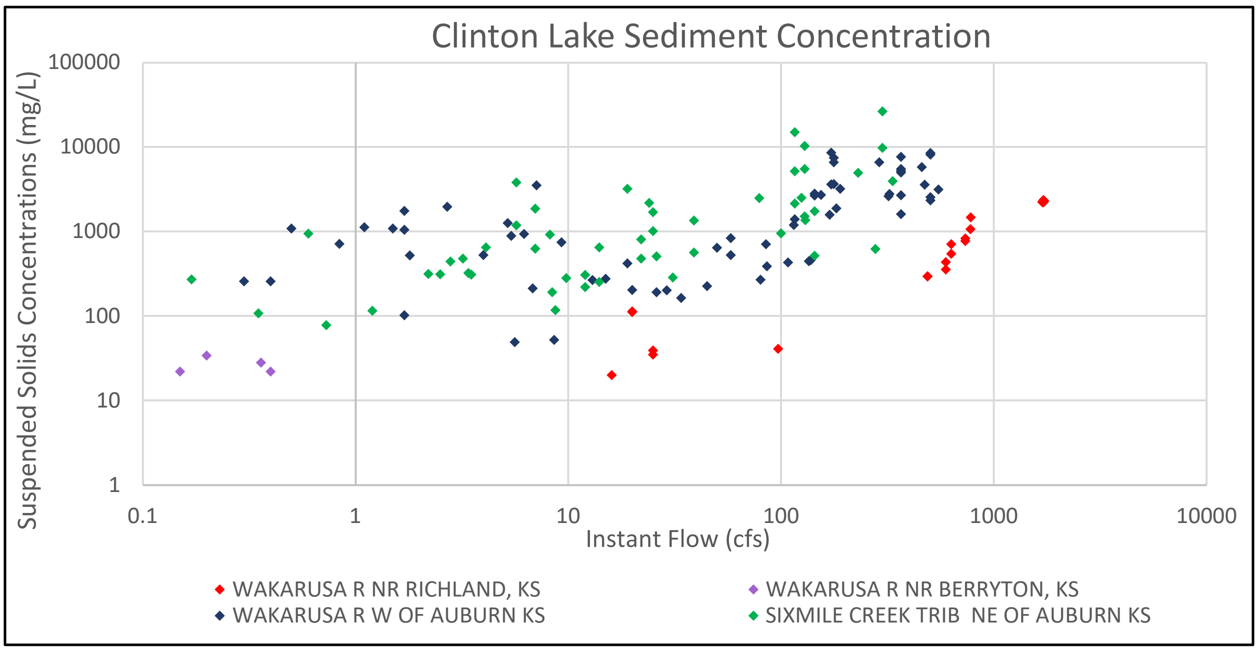


Figure 10-1. Clinton Lake Incoming Sediment Concentration vs Flow Data

Table 10-1. Clinton Lake Gage Station Timelines

Station	Name	Years
06891260	Wakarusa River near Richland, Kansas	2011-2019
385329095353400	Wakarusa River near Berryton, Kansas	1987-1989
385351095542900	Wakarusa River 5 miles west of Auburn, Kansas	1978-1980
385350095525300	Wakarusa River 4 miles west of Auburn, Kansas	1978-1980
385718095464000	Sixmile Creek Tributary 5 miles northeast of Auburn, Kansas	1978-1980
385632095464000	Sixmile Creek Tributary 4 miles northeast of Auburn, Kansas	1978-1980

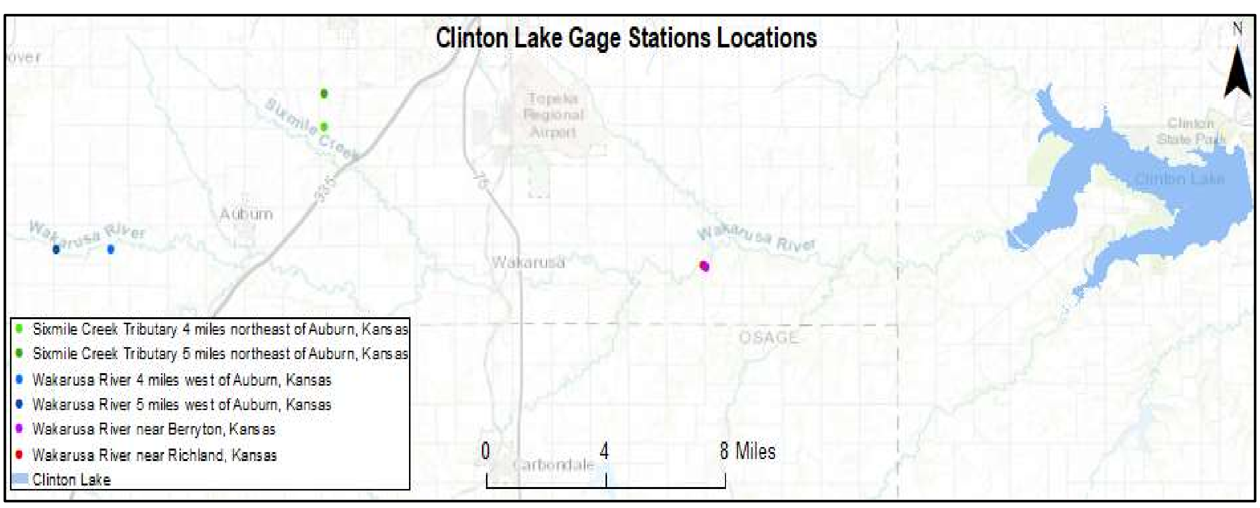


Figure 10-2. Clinton Lake Gage Station Locations

Using the data previously mentioned, there was sufficient data to define the 80% confidence intervals. The confidence intervals were originally computed based on the statistics of the total sample and departure from the best-fit polynomial. However, the height of the intervals was driven by the high variability at moderate flows, which yielded confidence intervals at the highest flows that exceeded all the measured data points. A more refined approach was taken to take into account differences in variability as a function of flow.

The 4th-degree polynomial through log-transformed data was used as a predictor for the mean of a normal distribution in a generalized additive model. The interpretation should be looked at as "the conditional distribution of suspended sediment concentration, given an amount of flow." That is, f(y|x) =N ($\mu=b_4x^4+b_3x^3+b_2x^2+b_1x+b_0$, $\sigma=a_2x^2+a_1x+a_0$) where y is SSC, x is flow, and N is the normal distribution with parameters μ and σ. Because the conditional distribution is normal, the best estimate of y|x is E[y|x] and is also the median of the conditional distribution, and it can be computed as μ. To obtain percentiles of the conditional distribution, the quantile function (inverse CDF) is used, F-1(p; μ, σ) where μ and σ are computed for the given value of flow. The result is overall a function that has a central tendency dictated by the 4th-order polynomial, with spread about the mean dictated by a concave-down quadratic. These analyses were automated using R-scripts. For a more detail on the process refer to the Kansas River Basin Sedimentation Summary.

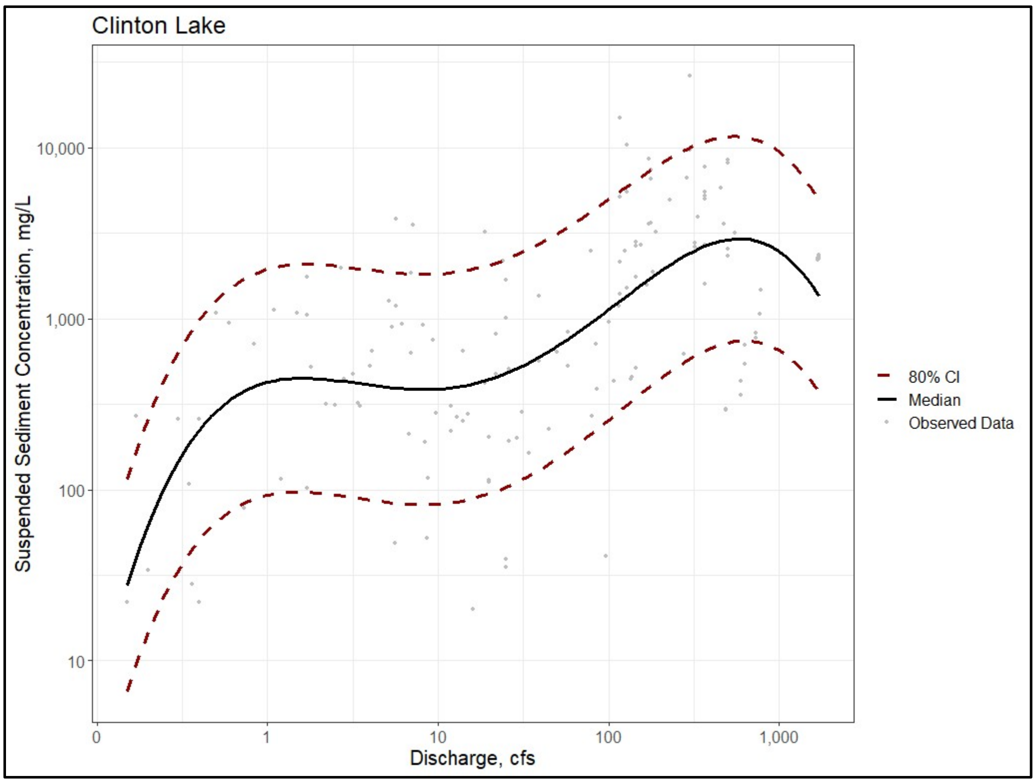


Figure 10-3: Clinton Lake Incoming Sediment Concentration vs Flow.

Gage station 06891483 (Wakarusa River BL Clinton Dam, Kansas) was used to determine the relationship between flow and the downstream sediment concentration. Prediction intervals around a regression between flow and concentration were plotted, so that there was a probability of 90% of the real values within the limits. Figure 10-4 and Figure 10-5 shows that there was sufficient data to define the variability of the downstream load.

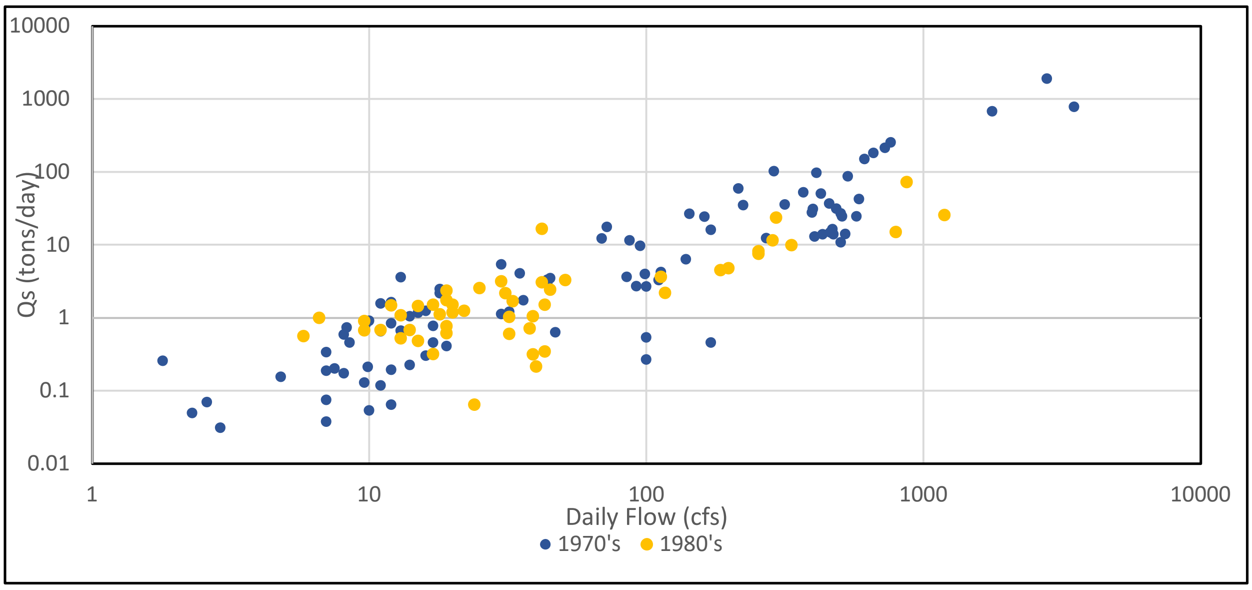


Figure 10-4. Clinton Lake Downstream Suspended-Sediment Data

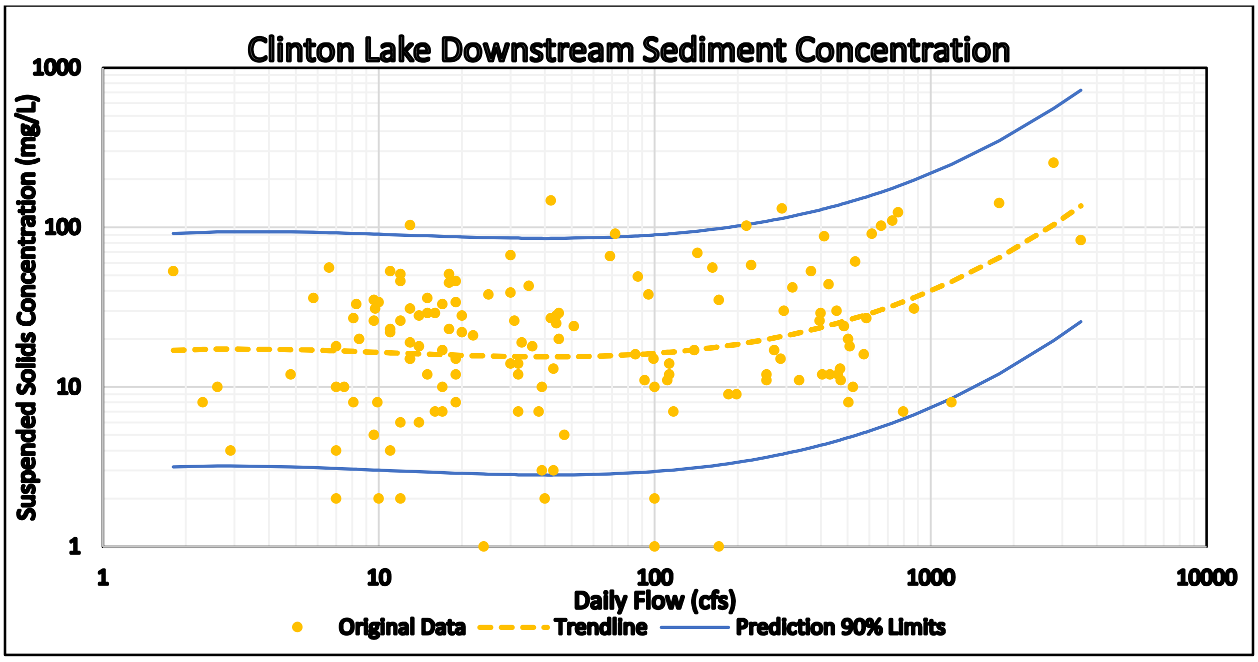


Figure 10-5. Clinton Lake Downstream Sediment Concentration

Sediment size gradations were collected in Clinton Lake in 2009 by the Kansas Biological Survey (2010). Figure 10-6 presents the results. Twelve coring sites were distributed across the reservoir. Texture analysis by the Kansas Biological Survey (2010), indicated that the predominant sediment in the reservoir was silt with a secondary fraction of clay.

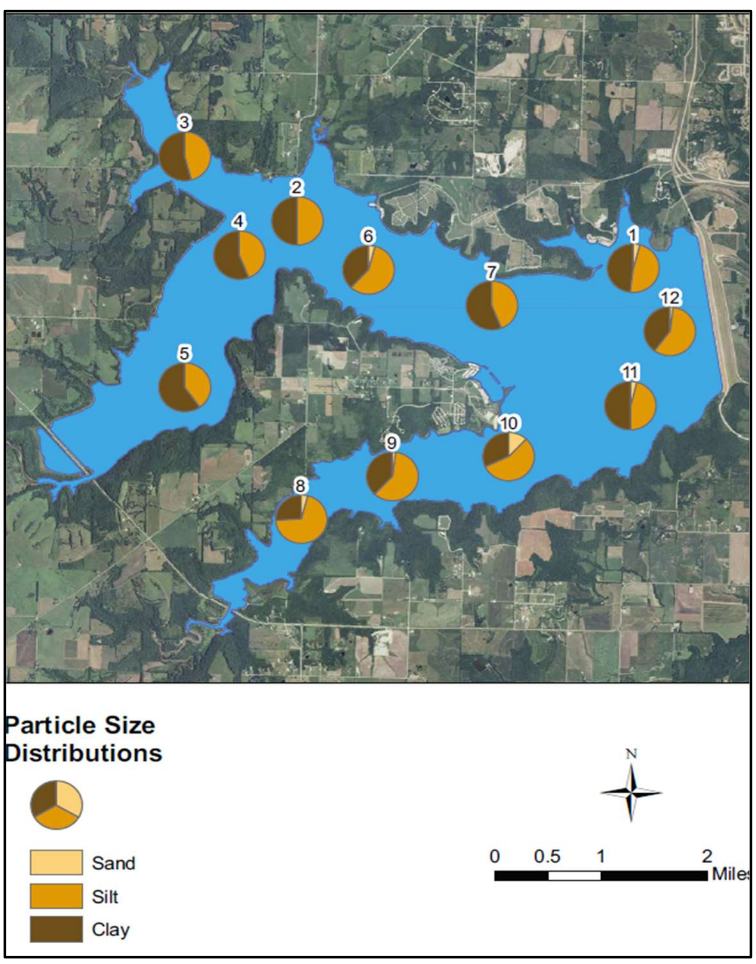


Figure 10-6. Clinton Lake Sediment Size Gradations

11.0 SEDIMENT CHEMICAL CONCENTRATIONS

No previous reports were found related to the sediment chemical concentrations in Clinton Lake.

12.0 DELTA LOCATION AND VOLUME

Rangeline surveys were examined to estimate the rate that the sediment delta is moving downstream in Clinton Lake. Figure 12-1 shows the locations of the rangelines, while Figure 12-2 also includes the centerline used to plot the elevations. Figure 12-3 provides a profile plot of a centerline through the lake. The main branch was selected to obtain surface profiles for the 2009 and 2019 surveys. As seen, no delta progression can be identified. The sedimentation lines have changed locations over time. The original locations are found in Attachment #1.

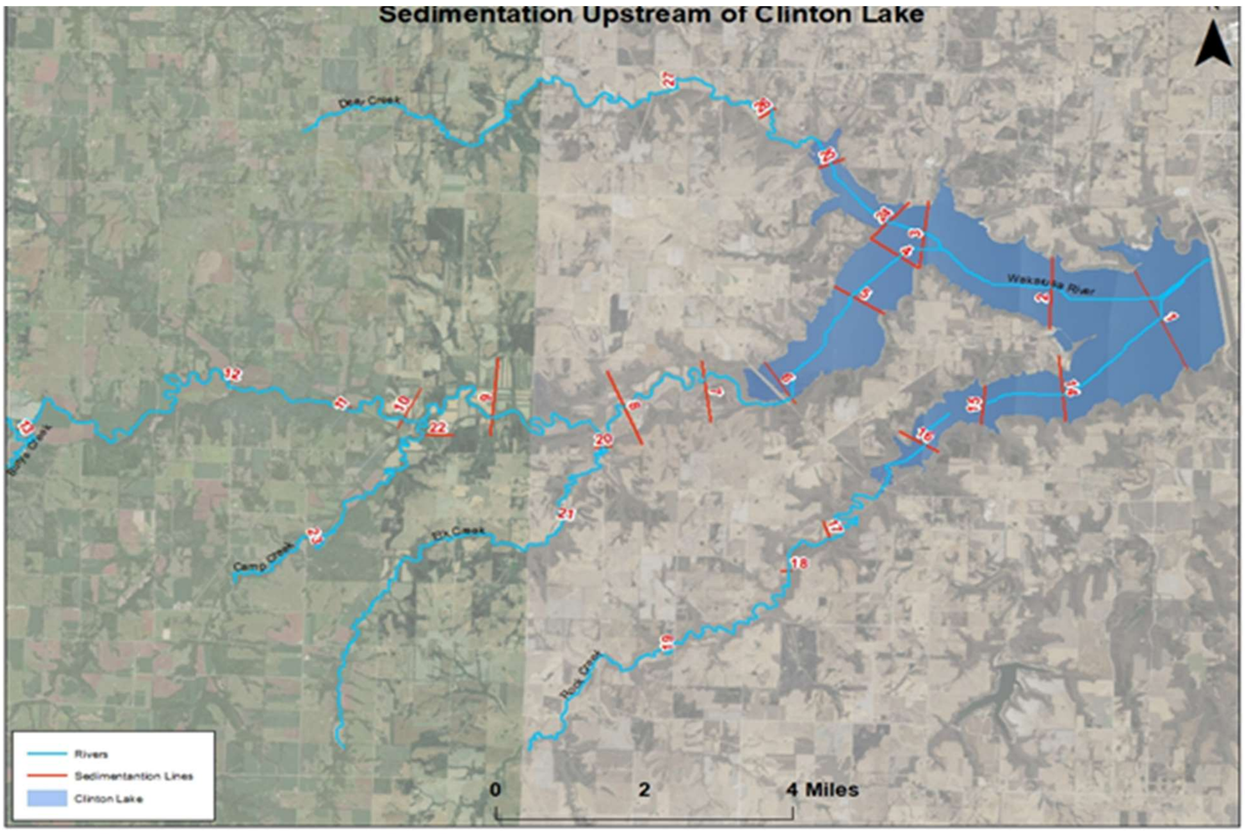


Figure 12-1. Clinton Lake Sedimentation Lines

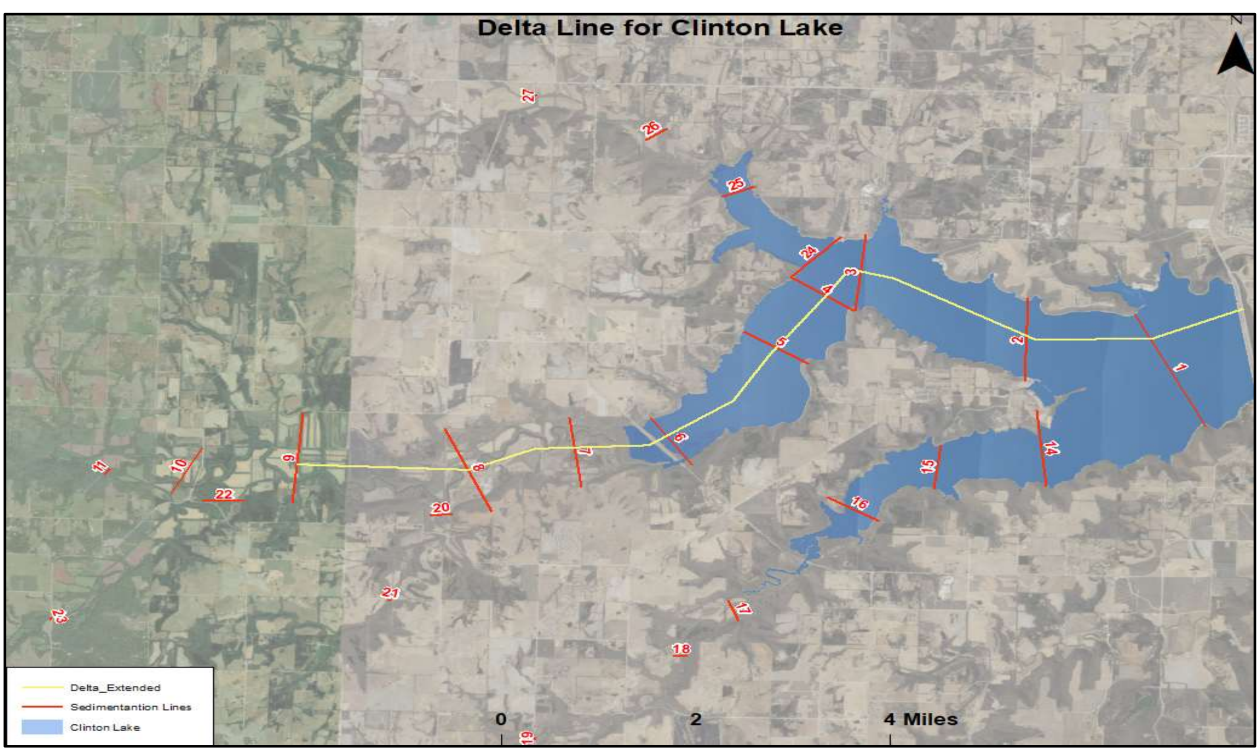


Figure 12-2. Location of the Delta Line used for Clinton Lake

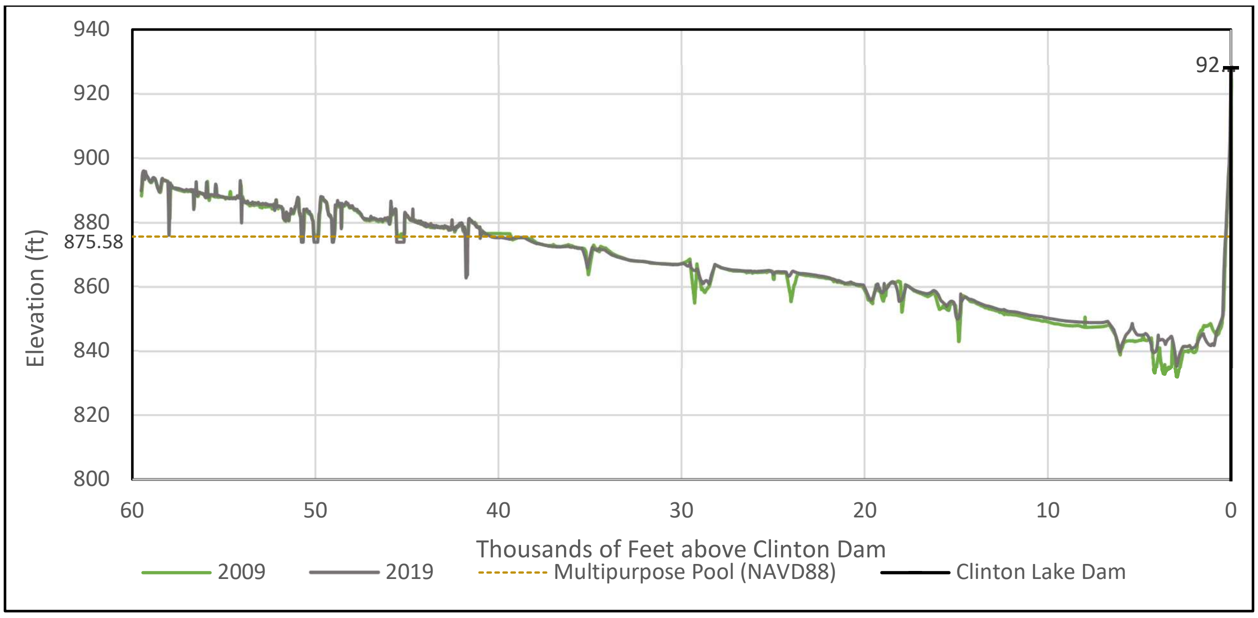


Figure 12-3. Profile for Delta Location

13.0 DOWNSTREAM CHANNEL

Sediment trapping by dams very often induces bed degradation and bank erosion downstream. Degradation rangelines downstream from Clinton Lake allow this effect to be quantified. Data from 1977 was converted from datum NGVD29 to NAVD88 by using the National Vertical Datum Conversion Raster. For each range line an average correction factor was applied. These average factors were determined based on the different quadrants where the lines where located.

Figure 13-1 shows the location of the range lines and the total bed elevation change and channel width change at each location.

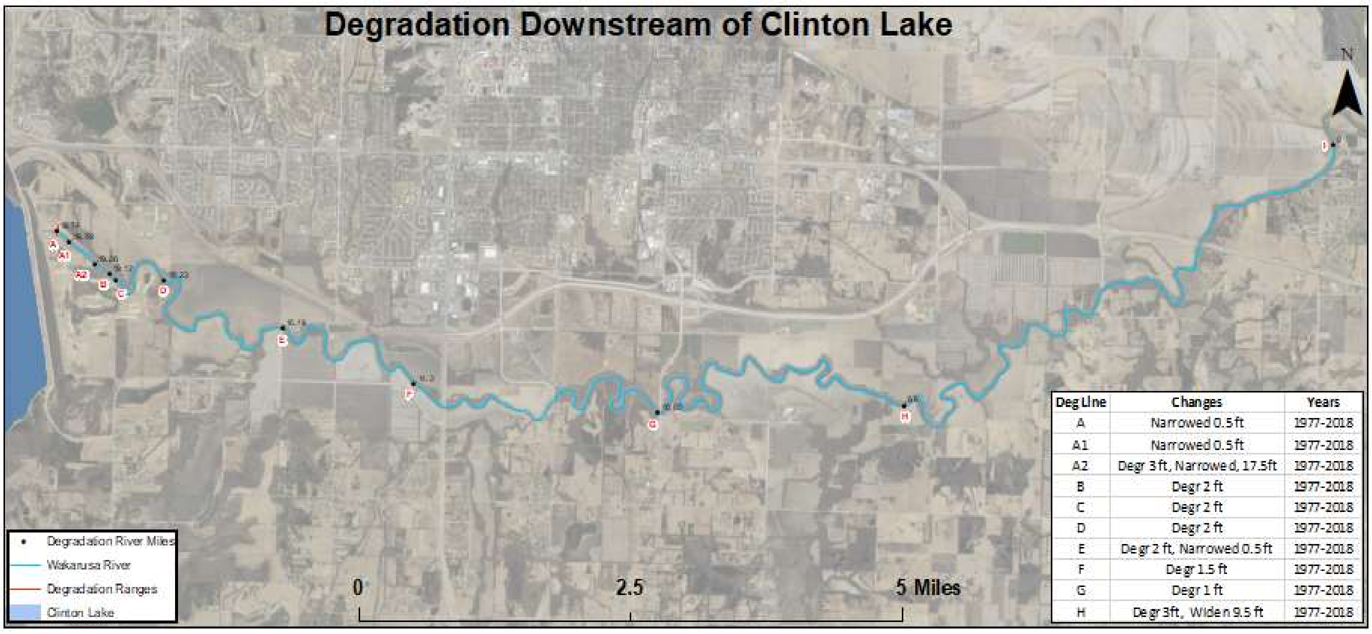


Figure 13-1. Degradation Downstream of Clinton Lake

Figure 13-2 plots the cumulative volume change over time based on the degradation rangelines, while Figure 13-3 plots the average yearly cumulative volume change. If these rangelines are indicative of the whole reach from A to I, the bed and banks have lost 593 acre-ft of material since dam closure. This represents 78% of the amount of the 760 ac-ft of sand estimated to have accumulated in Clinton Lake.

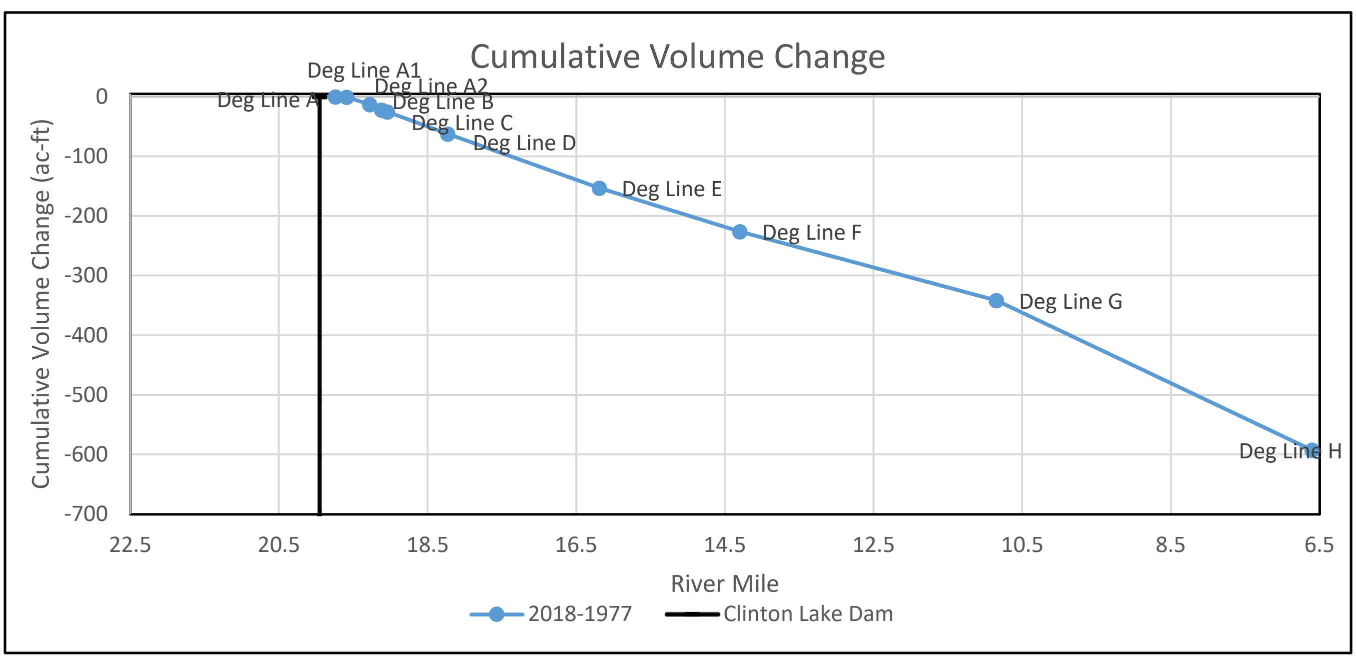


Figure 13-2. Longitudinal Cumulative Volume for Degradation Rangelines

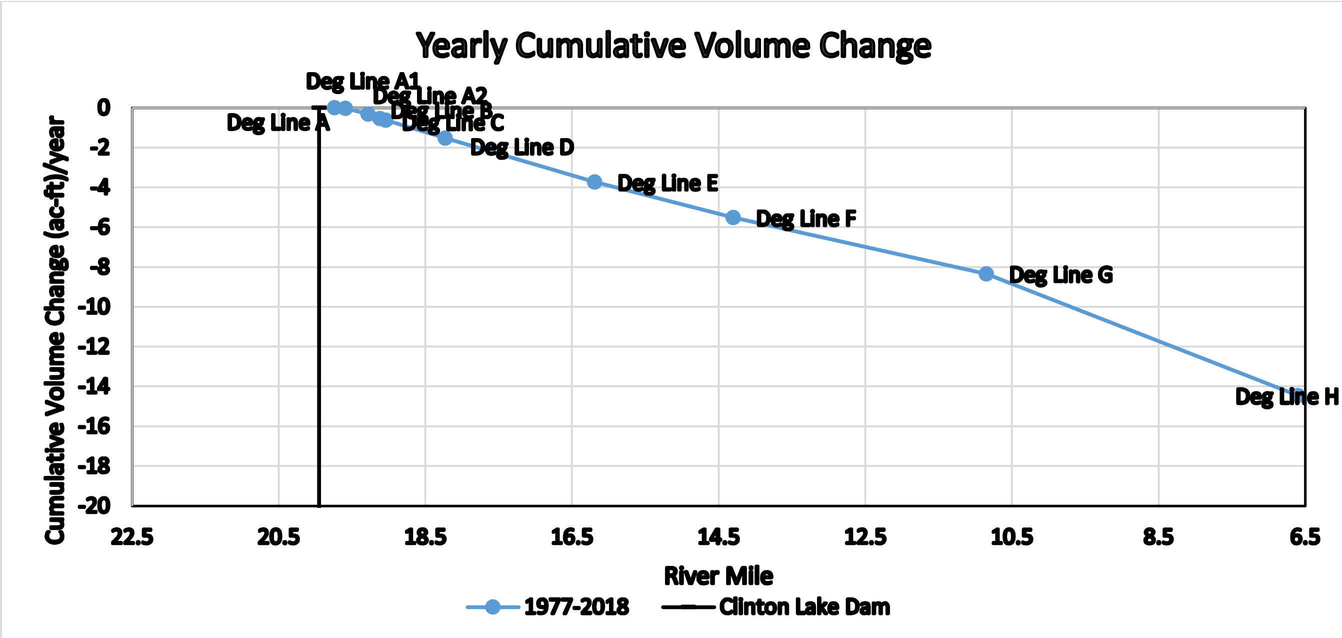


Figure 13-3. Yearly Cumulative Volume Change

These analyses indicate that the Wakarusa River downstream of Clinton Lake is sediment starved. Continued degradation with associated bank erosion is expected if sediment trapping continues.

14.0 SUMMARY AND CONCLUSIONS

Clinton Dam is located on the Wakarusa River. It was completed in August of 1975 with a total drainage area approximately 367 square miles. The dam has a top elevation of 928-ft (NAVD88). The multipurpose pool has an elevation of 875.5 ft (NGVD29) and a flood control pool elevation of 903.4 ft (NGVD29). The most current storage elevation curve has been used operationally from March 2012 to present using a 2009 survey. A new storage elevation curve was calculated using a 2019 survey. From 1977 to 2019, the multipurpose pool lost 16,139 ac-ft of storage to sedimentation. Surveys of the flood control pool indicate a gain in flood control pool volume of 23,203 ac-ft. This is a spurious result of the change in survey methods.

The trapping efficiency was calculated by the USGS to be 97%. The composite bulk density of the incoming load was calculated to be 42.01 lb/ft^3 using the fraction of loads formula. Bulk density values in the multipurpose and flood control pool values from Perry Lake were adopted. The multipurpose pool used was 39.38 lb/ft^3 and the flood control pool bulk density used was 57.36 lb/ft^3.

Instantaneous flow data from 2001 to 2019 at gage #06891260 on the Wakarusa River was used to develop a rating curve. The power function was extrapolated up to the 1.5-year flow, at which point in keeping with the shape of other rating curves in the Kansas River Basin, the flow/load relationship was assumed to flatten. The amount of flattening was the calibration metric. The rating curve was calibrated using the deposition between 1990 and 2009 surveys. The incoming load has 49.5% clay, 40.5% silt, and 10% sand/gravel. The total incoming load was 7,619,678 tons of clay, 6,234,282 tons of silt, and 1,539,329 tons of sand.

Using the calibrated daily deposition, the total sediment that enters the lake during multipurpose pool operations was 2,027 ac-ft and during flood control operations was 23,812 ac-ft. The deposition was also computed according following the WLMP elevations used on the Lake Regulation Manual. The total incoming load when pool levels were above the WLMP was 19,824 ac-ft whereas the total deposition when pool levels were below the WLMP was 3,988 ac-ft. By either metric, virtually all the sediment that eventually deposits enters the lake during flood control operations.

There was insufficient data to define the variability of the sediment concentration in the water during high inflow or low flow events using only the nearest gage to the lake. Prediction intervals based on concentration data gages further upstream were developed. Sediment size gradations were collected in 2009 by the Kansas Biological Survey (2010). The texture analysis indicated that the predominant sediment in the reservoir was silt with clay being the secondary. There was sufficient data to define the variability of the downstream sediment concentration. No previous report for sediment chemical concentrations reports were found for Clinton Lake.

No delta progression was able to be identified between the 2009 and 2019 bathymetric surveys. Degradation rangelines downstream of the dam indicate the bed and banks have lost 593 acre-ft of material since dam closure, which is 78% of the amount of sand trapped in Clinton Lake. These analyses indicate that the Wakarusa River downstream of Clinton Lake is sediment starved. Continued degradation with associated bank erosion is expected if sediment trapping continues.

REFERENCES

Juracek, K. E. (2013). *Suspended-Sediment loads and reservoirs sediment trap efficiency for Clinton Lake, Kansas, 2010 -2012*. Geological Survey Scientific Investigations Reports 2013-5153, http://pubs.usgs.gov/sir/2013/5153/.

USACE. (1984). *Lake Regulation Manual Clinton Lake Kansas*. Kansas City District: US Army Corps of Engineers. Volume No.4.

USACE (2013). *Clinton Dam (KS 00026) Periodic Inspection No. 12 Periodic Assessment No. 01.* Kansas City District.

Kansas Biological Survey (2010). Bathymetric and Sediment Survey of Clinton Lake Reservoir, Douglas County. Applied Science and Technology for Reservoir Assessment (ASTRA) Program Report 2009-09 (February 2010), https://services1.arcgis.com/q2CglofYX6ACNEeu/arcgis/rest/services/Lakes_All/FeatureServer/0/22/attachments/126

Duan, N. (1983). “Smearing estimate: A nonparametric retransformation method.” Journal of the American Statistical Association , 78(383), 605–610.

ATTACHMENT #1- ORIGINAL SEDIMENTATION RANGELINE LOCATIONS

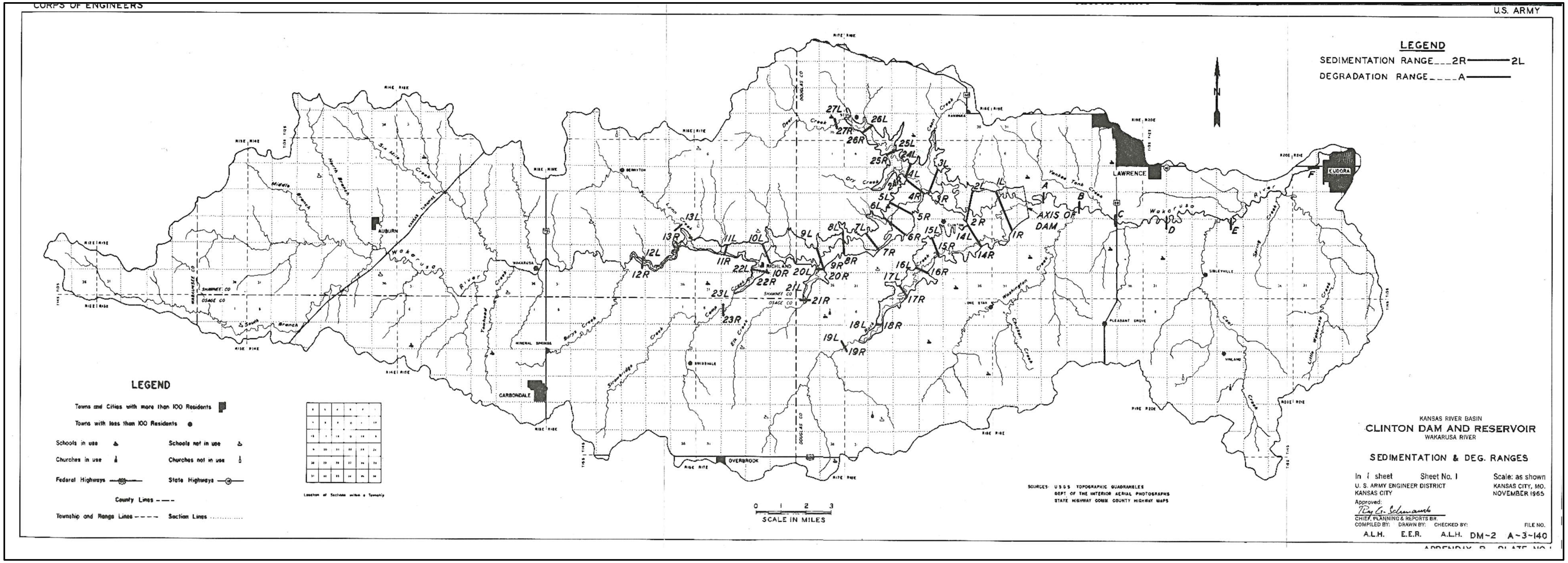


Figure A-1. Range Lines Original Locations

US Army Corps of Engineers ®
Kansas City District

Kansas River Reservoirs Flood and Sediment Study

Appendix D1.2: Perry Lake Existing Condition Sedimentation

November 2022

Contents

1.0 INTRODUCTION

Perry Dam is located at river mile 5.7 on the Delaware River. The reservoir is located in Jefferson County, Kansas, except for the upper 7 miles which extends into Atchison County at full pool. The project was authorized by the Flood Control Act of 1954 (Public Law 85-500). The lake provides flood control for Missouri, Kansas and the Mississippi River Valley as part of the Missouri River Basin comprehensive plan. Under the Water Supply Act of 1958 (Title III, Public Law 85-500) water supply was added as a project purpose. Perry Lake is also used for making minimum releases to supplement water flow and for navigational purposes. It provides 150,000 acre-feet of water storage, recreation, fish and wildlife, and water quality.

Construction of Perry Dam started in July of 1964 and was completed in August of 1966. Above the dam, the total drainage area is approximately 1,117 square miles. The dam is considered a high hazard dam. Figure 1-1 shows Perry Lake with respect to the overall Kansas River Basin and Figure 1-2 shows the Delaware River Basin.

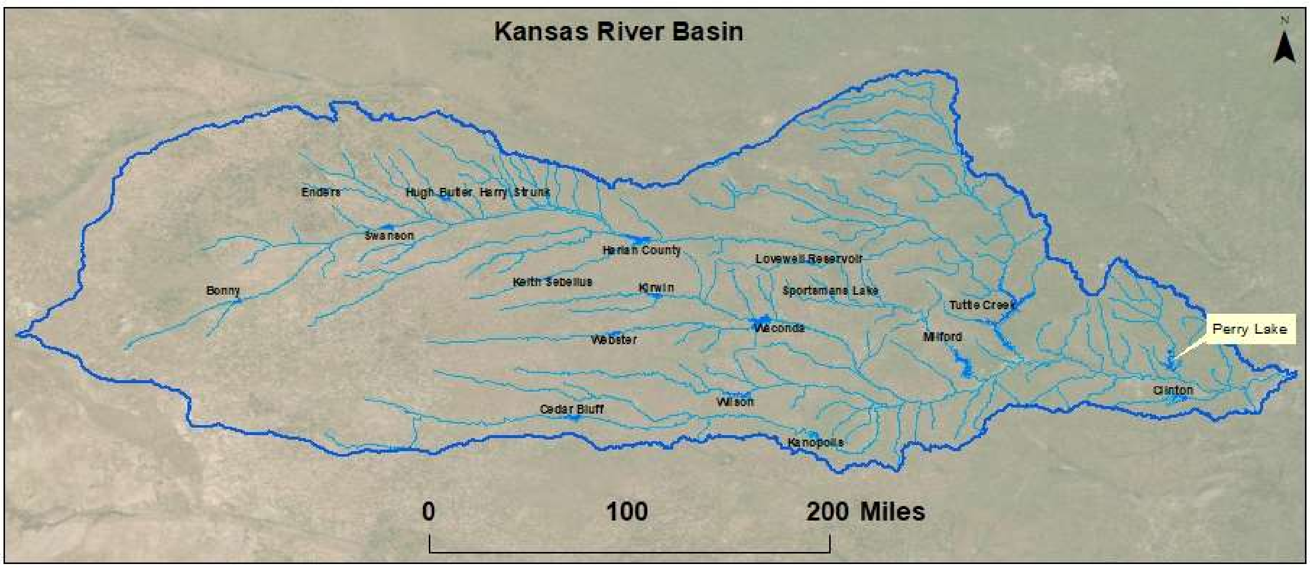


Figure 1-1: Overall Kansas River Basin Map

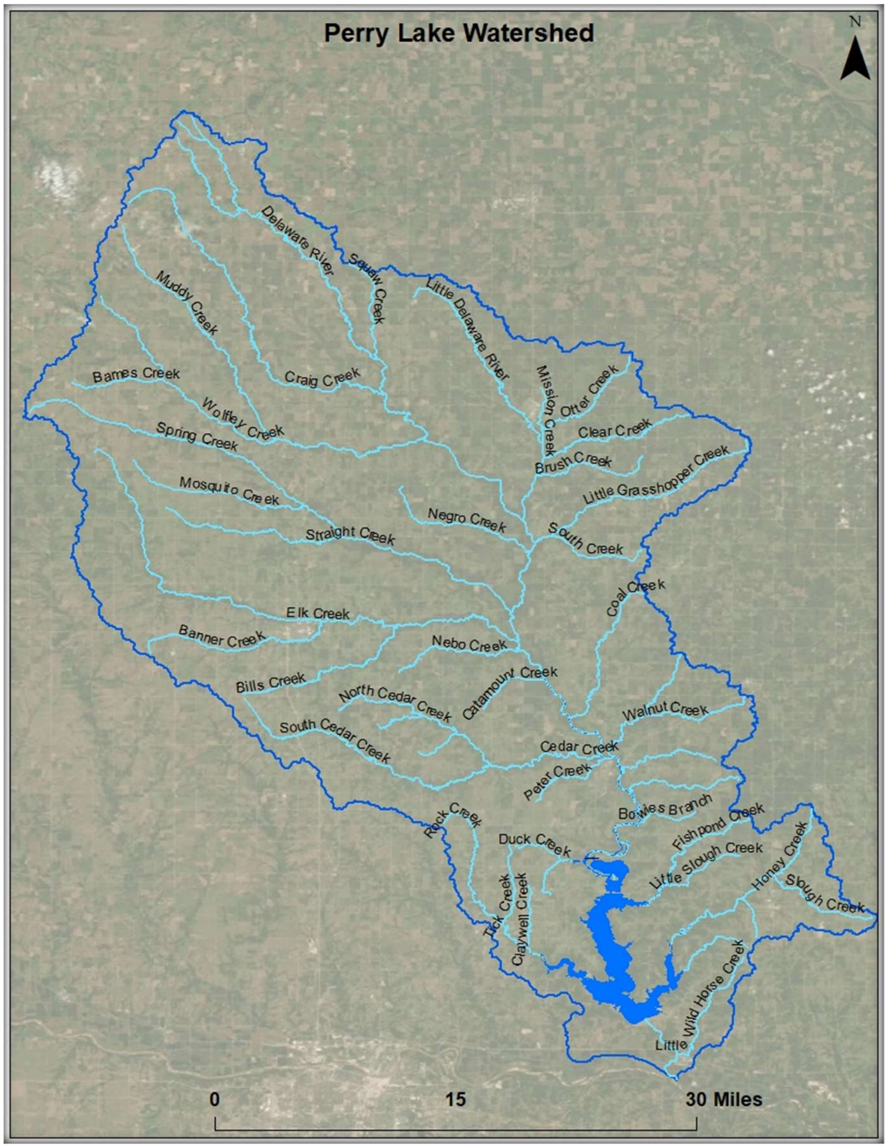


Figure 1-2: Perry Lake Watershed

2.0 DAM INFRASTRUCTURE

The dam consists of a zoned rolled earthfill embankment of approximately 7,750 feet long with a 30-foot-wide crest and dike, controlled outlet works and an uncontrolled spillway. The intake tower consists of a working platform, trash fenders, streamline inlet, gate passages, gate chambers, service deck and an entrance house. The outlet is a single horseshoe 23.5 feet diameter conduit that is located near the center of the embankment. It has 2 hydraulic operated wheeled service gates, and each contains a hydraulic wheeled emergency gate located upstream of the service gate. Only one of the smaller gates is required to meet low flow requirements but two gates provide greater flexibility.

The spillway is an unlined excavated chute with a concrete control sill at elevation 922.3 feet (NAVD88) located approximately 300 feet wide and approximately 2,250 feet long. Table 2-1 summarizes key information for the lake.

Table 2-1: Important Information Relating to the Dam Infrastructure

Multipurpose Pool Elevation	Capacity at elevation 891.5 feet-NGVD29
Lowest Elevation Outlet	831.3 feet (NAVD88)
Number of Gates at This Low Elevation	Two 2’ x 2’ low flow gates hydraulic slide built into two <size of service gates>
Releases from Low Level Outlet	Operated when Q < 200 cfs per low flow gate, average of 91 days/year
Spillway Elevation	Capacity at elevation 922.3 feet-NAVD88
Dam Elevation	946.3 ft (NAVD88)
Typical Tailwater Elevation near Perry Dam	850.5-852.5 (NAVD88)
Other Pipes Going Through the Dam or Embankment (i.e. Water Intakes, etc.)	A rural water district on the west side of the lake has a pump to the lake

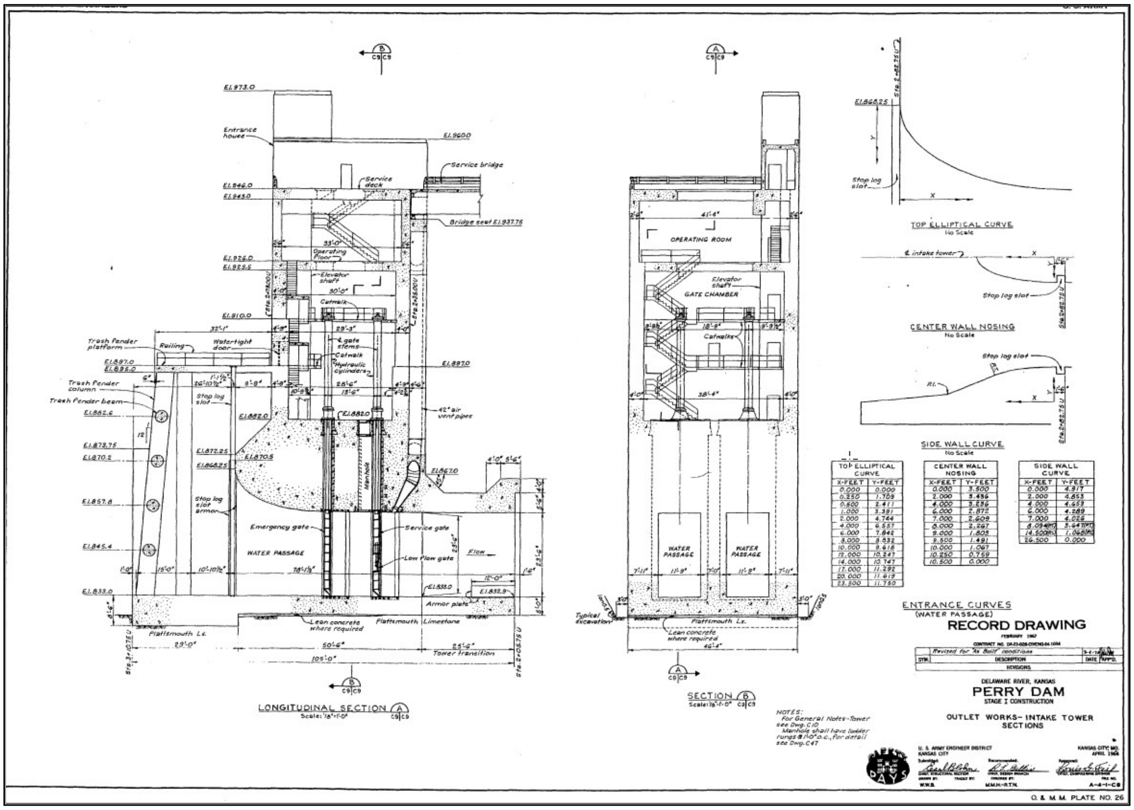


Figure 2-1: Outlet Works- Intake Tower

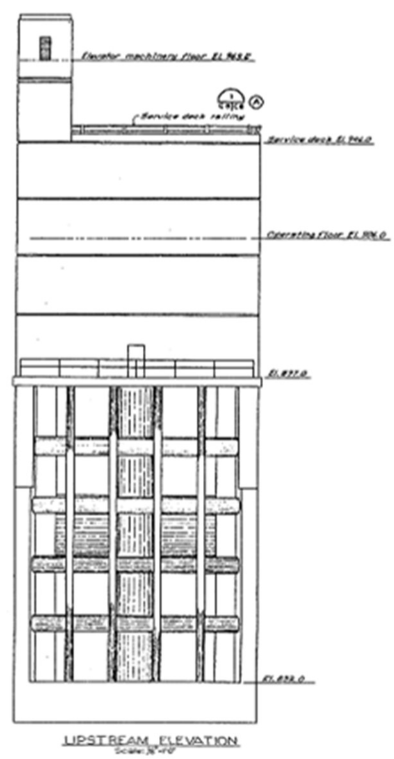


Figure 2-2: Outlet Works-Intake Tower Elevations

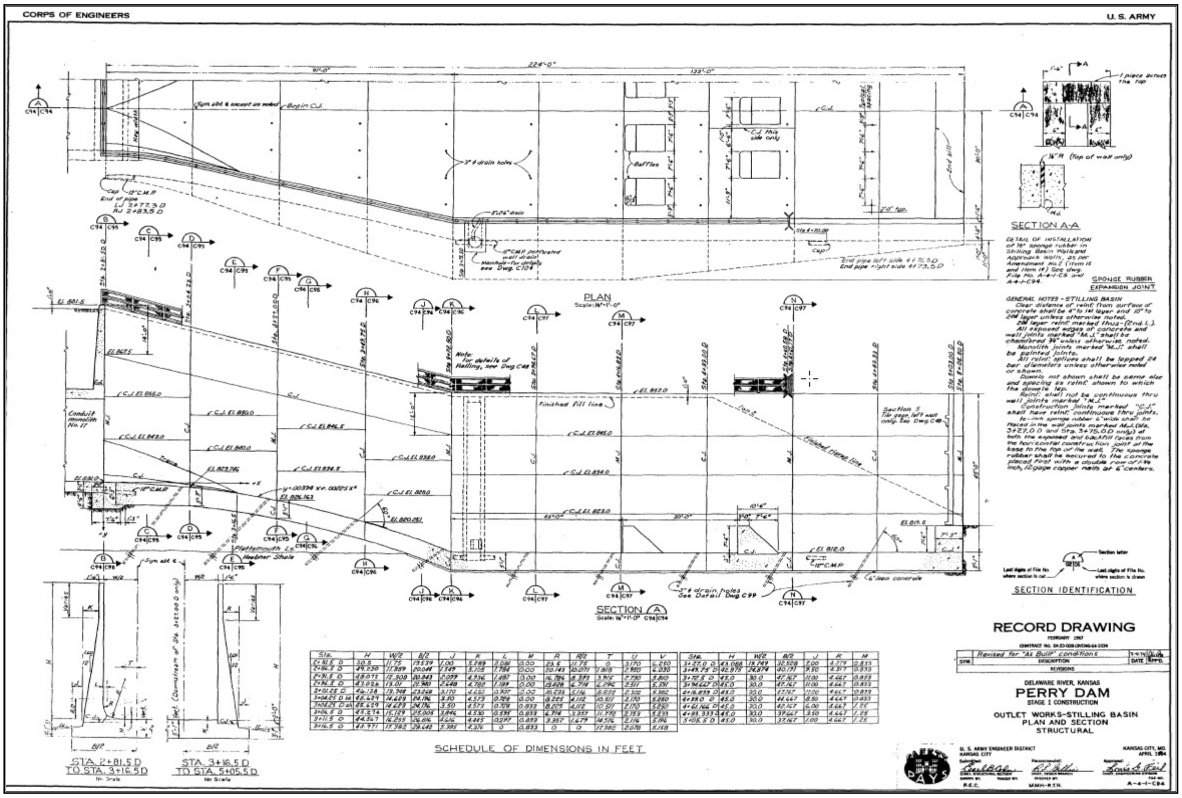


Figure 2-3: Outlet Works- Stilling Basin

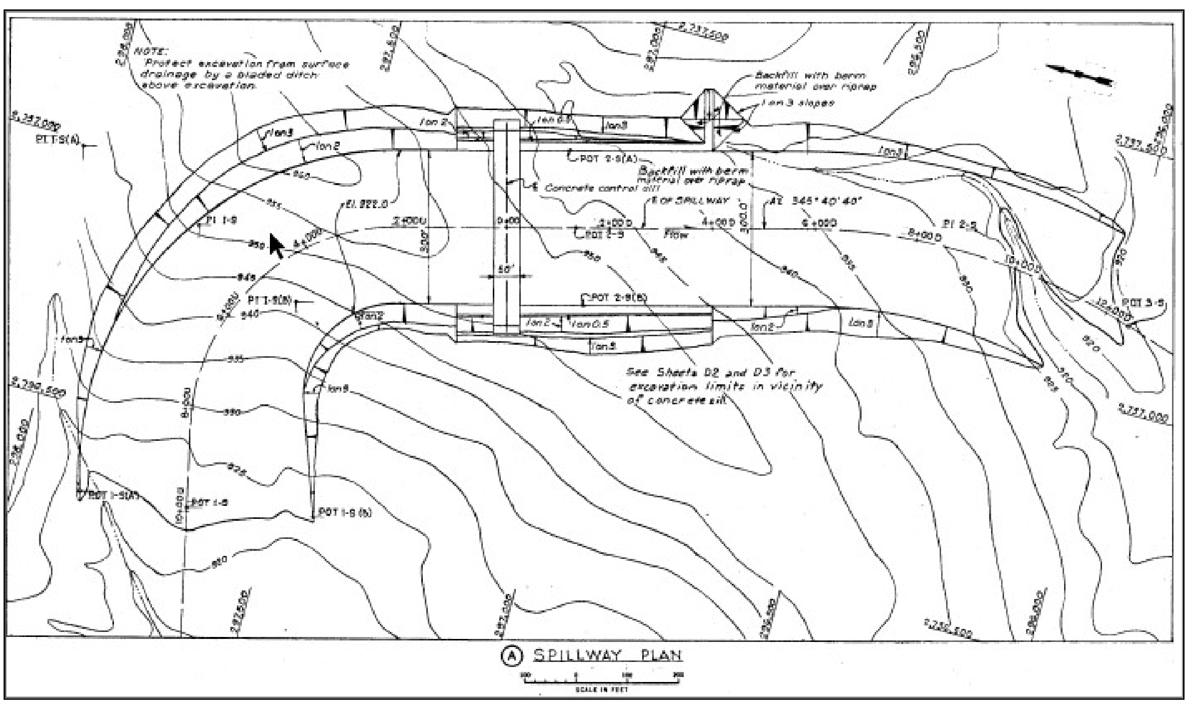


Figure 2-4: Spillway

3.0 SEDIMENTATION EFFECTS ON O&M

Discussions with lake staff indicate that there have been some sedimentation problems north of the 92 Highway Bridge. Two of the areas affected are recreational areas at Paradise Point and Sunset Ridge where they closed boat access ramps in the 1980s. Another area that is starting to experience sedimentation problems is Old Town Public Park south of the 92 Highway Bridge, which is expected to experience worsening sedimentation as the delta progresses downstream over the next 5-10 years.

4.0 EXISTING CONDITION STORAGE ELEVATION CURVES

The most current storage elevation table for Perry Lake has been used operationally from March 2012 to present (2020). The pool volumes were originally computed by Surdex Corporation in NAVD88 using results of the Eisenbraun August 2009 bathymetric survey combined with 2006 and 2010 LiDAR. For Perry Lake, the elevation reference was in NGVD29 using an average conversion factor for NGVD29 = NAVD88 - 0.285 ft using the centroid coordinates. The following table shows the storage elevation for Perry Lake.

Table 4-1: Storage Elevation Curve for Perry Lake

Elevation (ft) NGVD29	Volume (ac-ft)	Elevation (ft) NGVD29	Volume (ac-ft)	Elevation (ft) NGVD29	Volume (ac-ft)	Elevation (ft) NGVD29	Volume (ac-ft)
838	0	**866**	26,176	**893**	215,795	**921**	725,877
839	1	**867**	29,970	**894**	227,324	**922**	752,272
840	3	**868**	34,067	**895**	239,328	**923**	779,428
841	7	**869**	38,482	**896**	251,894	**924**	807,361
842	15	**870**	43,166	**897**	265,097	**925**	836,055
843	26	**871**	48,058	**898**	278,772	**926**	865,482
844	42	**872**	53,100	**900**	307,580	**927**	895,649
845	68	**873**	58,341	**901**	322,631	**928**	926,512
846	116	**874**	63,890	**902**	338,102	**929**	958,075
847	218	**875**	69,691	**903**	354,048	**930**	990,379
848	368	**876**	75,710	**904**	370,446	**931**	1,023,447
849	562	**877**	81,953	**905**	387,242	**932**	1,057,253
850	813	**878**	88,434	**906**	404,416	**933**	1,091,874
851	1,122	**879**	95,192	**907**	422,007	**934**	1,127,393
852	1,502	**880**	102,198	**908**	440,038	**935**	1,163,780
853	1,959	**881**	109,395	**909**	458,602	**936**	1,201,029
854	2,518	**882**	116,750	**910**	477,734	**937**	1,239,174
855	3,178	**883**	124,268	**911**	497,410	**938**	1,278,268
856	3,988	**884**	131,968	**912**	517,580	**939**	1,318,419
857	4,961	**885**	139,870	**913**	538,294	**940**	1,359,687
858	6,104	**886**	148,131	**914**	559,596	**941**	1,402,170
859	7,538	**887**	156,835	**915**	581,473	**942**	1,445,750
860	9,326	**888**	165,904	**916**	603,955	**943**	1,490,373
861	11,473	**889**	175,277	**917**	627,098	**944**	1,535,969
862	13,915	**890**	184,944	**918**	650,837	**945**	1,582,507
863	16,575	**891**	194,917	**919**	675,191	**946**	1,629,992
864	19,494	**891.5**	200,004	**920**	700,200	**947**	1,678,434
865	22,690	**892**	205,146	**920.6**	715,523	-	-

The following table provides a summary by pool of official storage elevation curves with their respective data type.

Table 4-2: Pool Volumes Over Time

Survey Year	Multipurpose Pool Volume (ac-ft)	Flood Control Pool Volume (ac-ft)	Data Type
1969	243,220	765,100	Computed from 1960 topographic maps

Survey Year	Multipurpose Pool Volume (ac-ft)	Flood Control Pool Volume (ac-ft)	Data Type
1979	223,743	740,037	Survey of sediment ranges
1989	209,513	725,308	Survey of sediment ranges
2001	206,682	722,079	Bathymetry survey of sediment ranges, combined with USGS DEM's
2009	200,004	715,523	Eisenbraun August 2009 bathymetric survey combined with 2006 and 2010 LiDAR data, computed by Surdex Corporation

From 1969 to 2009, the multipurpose pool (MPP) lost 43,216 ac-ft of storage to sedimentation. This represents 18% of the original multipurpose pool volume. The average annual rate of loss was 1,080 ac-ft/year or 0.44% of the original volume/year.

From 1969 to 2009, the flood control pool (FP) lost 6,361 ac-ft of storage to sedimentation. This represents 1% of the original flood control pool volume. The average annual rate of loss was 159 ac-ft/year or 0.03% of the original volume/year.

5.0 TRAPPING EFFICIENCY

Trapping efficiency for Perry Lake was estimated using the method developed by Brune (1953). Brune's method, which was based on data from 44 reservoirs across the United States, predicts trapping efficiency as a function of the ratio of the reservoir storage capacity to the mean annual water inflow. Three curves describe the relationship (see Figure 6).

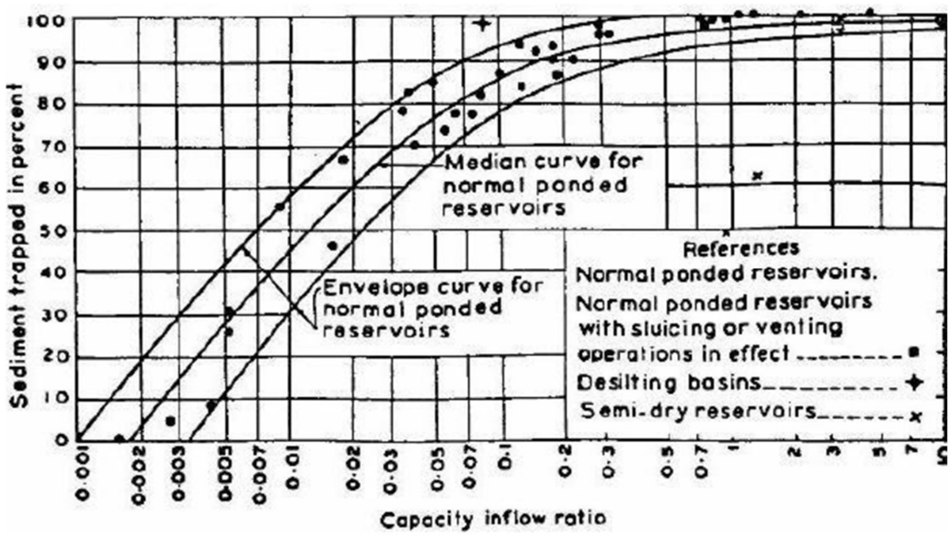


Figure 5-1: Brune Curve

For Perry Lake, the "medium" curve was used, approximated with the following formula:

$$TE = a[1 - 2e^{-bV_*^{0.35}}]$$

Where:

$V_* = \frac{V_{res}}{V_{inflow}}$ = 1.16

V_{res} = volume from the multi-purpose pool for Perry Lake (200,004 ac-ft)

V_{inflow} = average of the daily inflow converted into volume

Table 5-1: Trapping Efficiency Constant Values

Constant	Low	Medium	High
a	95	97	100
b	5.37	6.42	7.71

The volume from the multi-purpose pool for Perry Lake was 200,004 ac-ft and the mean annual water inflow during those years was 187,572.24 ac-ft. The computed sediment trapping efficiency for Perry Lake is 96.8%.

6.0 DEPOSITIONAL VOLUME

Lake surveys were conducted in 2001 and 2009. The 2001 survey was collected on the historic sediment range lines. The 2009 survey was collected via single-beam sonar on transects lines with an average line spacing of 250 ft (see Figure 6-1). Two LiDAR datasets describe the flood control pool: Jefferson County LiDAR (2006) and Atchison County LiDAR (2010). The Jefferson County LiDAR has a 2-meter resolution and the Atchison County LiDAR has a 1-meter resolution. A USGS NED surface with a 1/3 arc-second resolution was used for remaining uncovered areas. Surdex Corporation processed and combined this data to create a single DEM (see Figure 6-2).

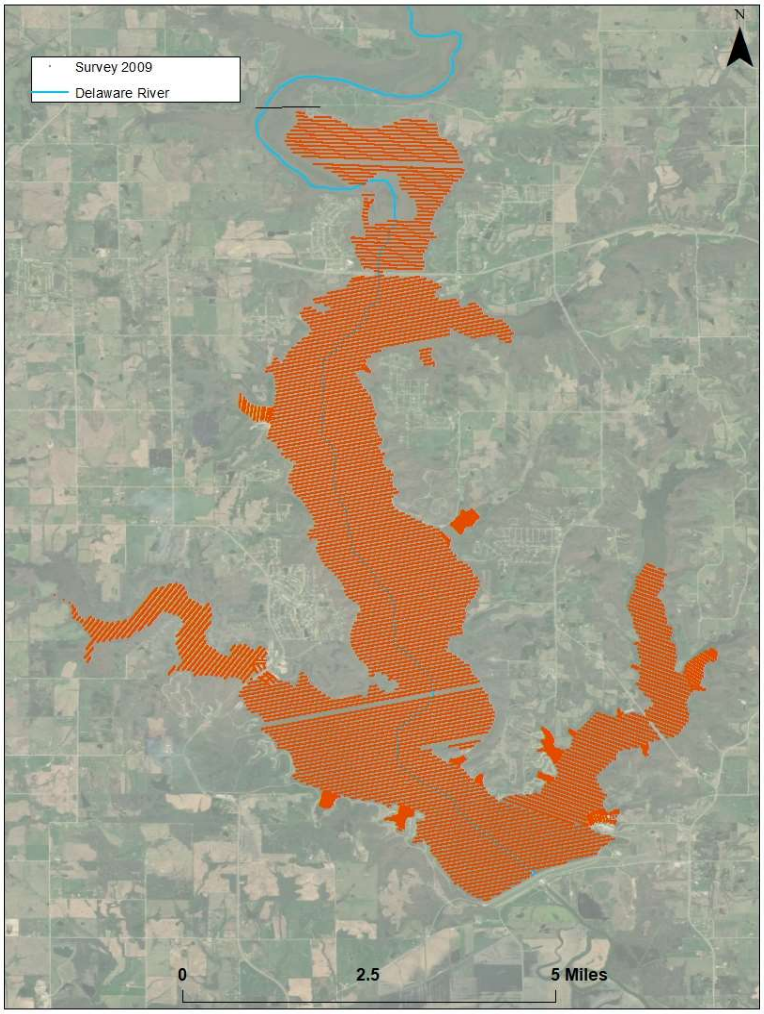


Figure 6-1: 2009 Bathymetric Survey Data for Perry Lake

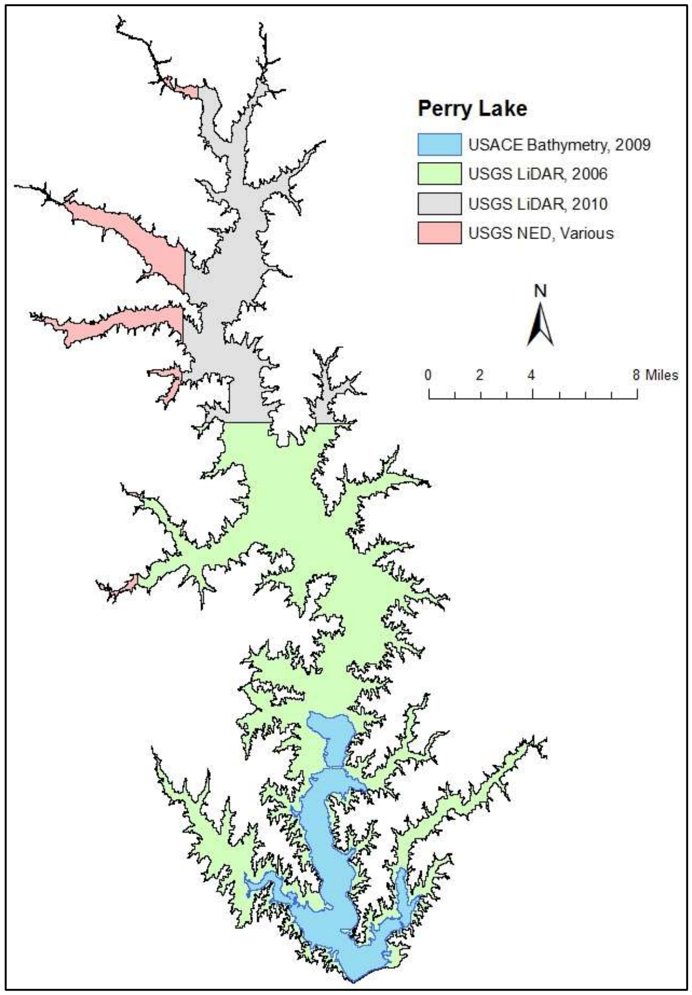


Figure 6-2: Perry Lake Data Sources used by Surdex

Table 6-1 provides the multi-purpose pool and flood control pool deposition between various pairs of surveys.

Table 6-1: Deposition Between Time

Year	Deposition-MP (ac-ft)	Deposition-FC (ac-ft)
1969-1979	19,477	5,586
1979-1989	14,230	499
1989-2001	2,831	398
1989-2009	6,678	-122

The computed negative deposition in the flood control pool from 1989 to 2009 was most likely due to the change in survey methods. Similar shifts have been observed at other lakes.

7.0 INCOMING SEDIMENT LOADS

USACE and USGS have sporadically collected paired flow/sediment concentration measurements on the Delaware River at gage 06890100 near Muscotah, Kansas. Data from 1977 to 2010 USGS was downloaded from the sediment data portal (https://cida.usgs.gov/sediment/). Two additional data points collected in 2019 by USGS, specifically for this study were also included. Discrete suspended-sediment concentrations were multiplied by the instantaneous flow and a constant of 0.0027 to compute sediment load in tons per day. Figure 7-1 plots the flow/load data for separate decades. As seen, the flow/load relationship fluctuates based on data variability but does not appear to trend over time.

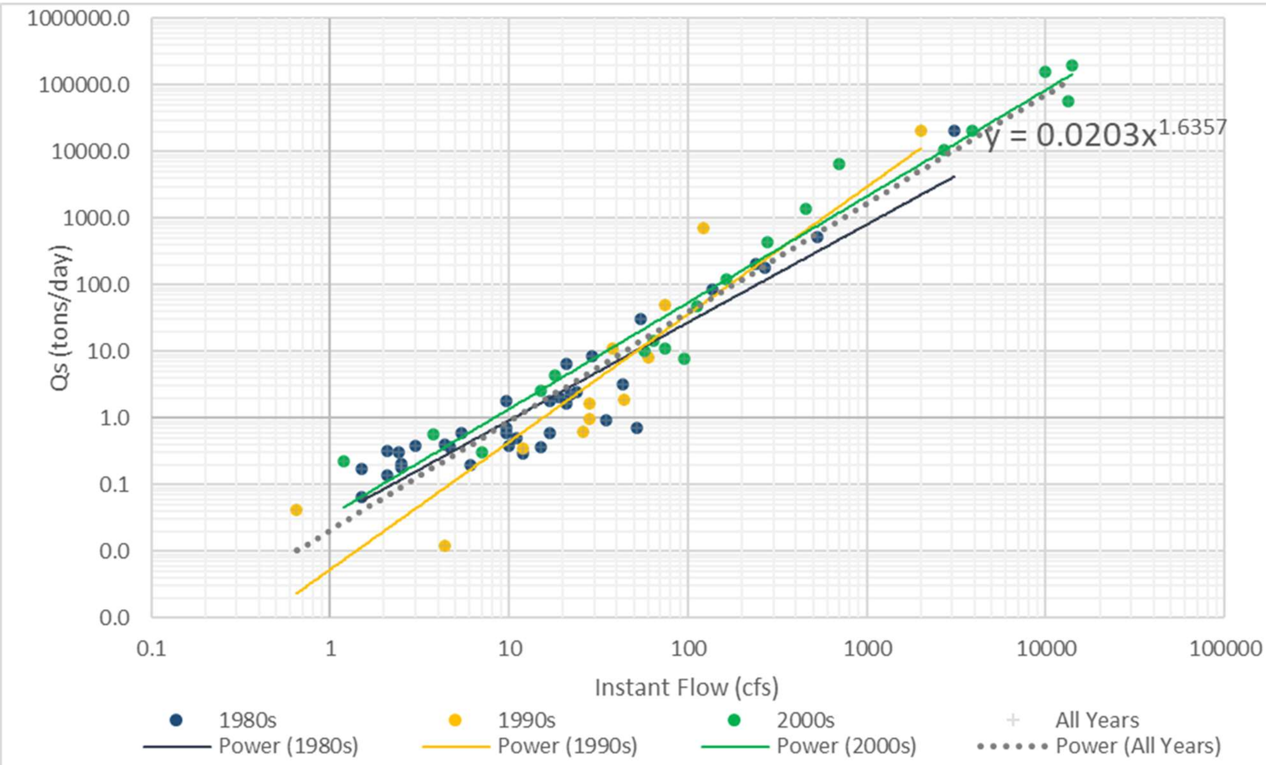


Figure 7-1: Suspended-Sediment Data

A rating curve was developed using the discrete data using the following steps:

1. Estimate the best fit regression line of the form $Q_s = aQ^b$ using log-log linear regression using data up to the 1.5-year flow.
2. Correct for bias using the Duan correction factor (Duan 1983). The Duan E correction was 1.76. Figure 7-2 presents the flow-load rating curve with Duan E correction.
3. Most lakes in the Kansas River basin with sufficient data exhibit a bending down of the flow-load curve. Similar behavior was assumed for Perry Lake to extrapolate the upper end of the curve. The flow for the second to the last point of the rating curve was set to the 1.5-year flow with the sediment computed using the rating curve equation with the Duan correction. The flow for the last point was set to be ten times the 1.5-year flood, and the sediment load was determined through calibration, as explained later.
4. Add 5% to account for bed load to create a total load rating curve.

5. Multiply the load by the ratio of total drainage area to gaged drainage area.
6. Using the measured data, estimate the percentages of clay, silt, and sand/gravel.
7. Apply this rating curve to daily flow rates from 1969 to 2019 to determine the cumulative mass of sediment entering the reservoir.
8. Multiply by the appropriate percentages to determine the cumulative mass of clay, silt, and sand entering the lake.

Figure 7-2 shows the calibrated rating curve and Table 7-1 give the flow/load points for the rating curve.

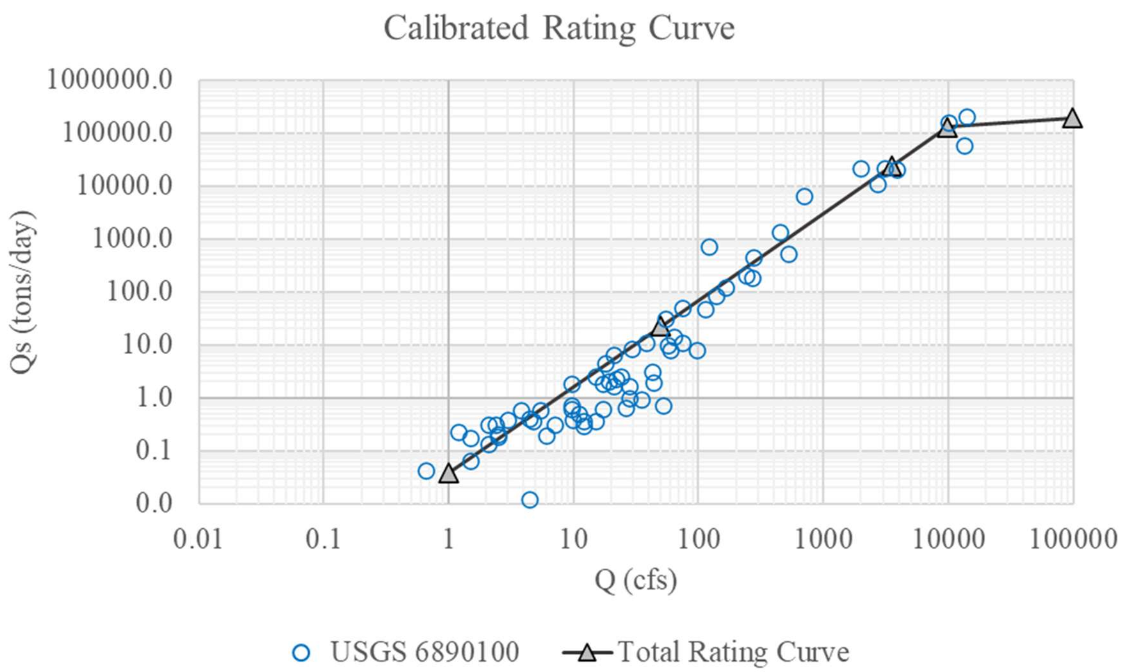


Figure 7-2: Calibrated Rating Curve

Table 7-1: Calibrated Rating Curve Points

Flow (cfs)	Load (T/d)
1	0
50	23
3,500	23,458
9,820	126,809
98,200	190,214

This analysis found the incoming load to be 49.2% clay, 40.5% silt, and 10.3% sand/gravel with the totals given in Table 7-2.

Table 7-2: Incoming Sediment to Lake Perry from 1969 to 2019

Years	1969-2019
Total Incoming Sediment (tons)	65,232,018
Total Incoming Clay Fraction (tons)	32,082,292

Years	1969-2019
Total Incoming Silt Fraction (tons)	26,400,721
Total Incoming Sand Fraction (tons)	6,749,005

8.0 BULK DENSITY AND CONSOLIDATION OF SEDIMENT DEPOSITS

Tests conducted in 2001 by USGS (Juracek 2003) indicate surficial multipurpose pool bulk densities ranged from 18.4 to 46.3 lb/ft^3. Bulk densities increased with depth into the sediment deposits, indicating that older deposits had consolidated over time. The multipurpose pool bulk density was estimated by using the total computed bottom sediment mass and the total estimated bottom sediment volume as reported in Juracek (2003). The volumes and masses from Juracek (2003) are repeated in Table 8-1.

Table 8-1: Multipurpose Pool Sediment Deposits (from Juracek 2003)

Multipurpose Pool In-Channel Lake Component	Estimated Bottom Sediment Volume (ft^3)	Representative Bulk Density (lb/ft^3)	Computed Bottom Sediment Mass (lb)
-	**Delaware River**	-	-
Dam to range line 1	2,570,000	28.8	74,000,000
Range lines 1 to 2	26,500,000	27.1	718,000,000
Range lines 2 to 3	31,100,000	27.0	840,000,000
Range lines 3 to 4	31,300,000	29.7	930,000,000
Range lines 4 to 5	52,200,000	45.4	2,370,000,000
Range lines 5 to 6	84,600,000	52.3	4,420,000,000
Range lines 6 to 7	75,300,000	48.9	3,680,000,000
Range lines 7 to 8	16,300,000	55.9	911,000,000
-	**Rock Creek**	-	-
Confluence with Delaware River to range line 24	2,670,000	34.9	93,200,000
Range lines 24 to 25	9,330,000	34.9	326,000,000
Range lines 25 to 26	7,850,000	34.9	274,000,000
-	**Slough Creek**	-	-
Confluence with Delaware River to range line 28	3,780,000	35.2	133,000,000
Range lines 28 to 29	8,880,000	35.2	313,000,000
Range lines 29 to 30	8,080,000	35.2	284,000,000
-	-	-	-
Total for lake	**360,460,000**	-	**15,400,000,000**

Multipurpose Pool Out-Channel Lake Component	Estimated Bottom Sediment Volume (ft^3)	Representative Bulk Density (lb/ft^3)	Computed Bottom Sediment Mass (lb)
-	**Delaware River**	-	-
Dam to range line 1	13,500,000	18.4	248,000,000
Range lines 1 to 2, 24 to 28	42,700,000	21.1	901,000,000
Range lines 2 to 3	179,000,000	26.1	4,670,000,000
Range lines 3 to 4	158,000,000	27.2	4,300,000,000
Range lines 4 to 5	554,000,000	37.4	20,700,000,000
Range lines 5 to 6	749,000,000	46.3	34,700,000,000
Range lines 6 to 7	197,000,000	46.3	9,120,000,000
Range lines 7 to 8	51,800,000	46.3	2,400,000,000
-	**Rock Creek**	-	-

Multipurpose Pool Out-Channel Lake Component	Estimated Bottom Sediment Volume (ft^3)	Representative Bulk Density (lb/ft^3)	Computed Bottom Sediment Mass (lb)
Range lines 24 to 25	59,500,000	25.8	1,540,000,000
Range lines 25 to 26	36,300,000	32.7	1,190,000,000
-	**Slough Creek**	**-**	-
Range lines 28 to 29	41,800,000	30.7	1,280,000,000
Range lines 29 to 30	25,500,000	30.7	783,000,000
-	-	-	-
Total for lake	**2,108,100,000**	**-**	**81,800,000,000**

The multipurpose pool was calculated by dividing the total computed bottom sediment mass by the estimated bottom sediment volume from Table 8-1.

$$\boldsymbol{\gamma}_{MP} = \frac{(15{,}400{,}000{,}000+81{,}800{,}000{,}000)\ lb}{(360{,}460{,}000+2{,}108{,}100{,}000)\ ft^3}$$

The average unit weight of sediment deposits in the multipurpose pool was thus calculated to be 39.38 lb/ft^3. While no sediment size gradation was found for Perry Lake deposits, the low bulk density suggests mostly clay deposits.

The incoming load can also provide an estimate of the composite bulk density, via the following equation:

$$\boldsymbol{\gamma_c} = \frac{\mathbf{1.0}}{\left(\left(\frac{F}{\gamma}\right)_{clay} + \left(\frac{F}{\gamma}\right)_{silt} + \left(\frac{F}{\gamma}\right)_{sand}\right)}$$

Where γ_c is the composite bulk density

F: the fraction of clay, silt, or sand

γ: for clay, silt and sand assumed to be 30 pcf, 65 pcf, and 93 pcf respectively (the defaults from HEC-RAS)

The composite bulk density of the incoming load was 42.13 lb/ft^3. This represents the bulk density for deposition in both the flood control and multipurpose pools.

Volumetrically, between 1969 and 1989, 84.7% of the total deposited volume deposited in the multipurpose pool and 15.4% of the total deposited in the flood control pool. These volumetric percentages plus the estimates for total bulk density and the multipurpose pool bulk density were used to back-calculate the flood control pool bulk density, as follows:

$$\gamma_c V_c = \gamma_{mp} Vmp + \gamma_{fp} V_{fp}$$

Where $V_{c:}$ composite volume of deposition

γ_c: composite bulk density

$V_{mp:}$ multipurpose pool volume of deposition

γ_{mp}: multipurpose bulk density

$V_{fp:}$ flood control pool volume of deposition

γ_{fp}: flood control bulk density

$V_{fp} = 0.153V_c$ (based on measured pool volumes)

$V_{mp} = 0.847V_c$ (based on measured pool volumes)

$$\gamma_{fp} = \frac{(\gamma_c V_c - \gamma_{mp} V_{mp})}{V_{fp}}$$

$$\gamma_{fp} = \frac{(\gamma_c V_c - \gamma_{mp} * 0.847V_c)}{0.153V_c}$$

$$\gamma_{fp} = \frac{V_c(\gamma_c - \gamma_{mp} * 0.847)}{0.153V_c}$$

$$\gamma_{fp} = \frac{\gamma_c - \gamma_{mp} * 0.847}{0.153}$$

The flood control pool bulk density was thus back calculated to be 57.36 lb/ft^3.

9.0 CALIBRATING LOAD AND DEPOSITIONAL PARAMETERS

To calibrate the rating curve, the deposition computed with the rating curve was compared to the volume of deposition calculated from the lake surveys. Deposition in the multi-purpose pool between 2001 and 2009 was used for this purpose. This was accomplished following these steps:

1. Apply the rating curve to the daily flow, adjusting for bedload and the ungauged areas.
2. Compute the mass of trapped load sediment for the multipurpose pool by applying the trapping efficiency and mass ratio to the day of incoming lake sediment.
3. Compute mass of trapped load sediment for the flood control pool by applying the mass ratio to the day of incoming lake sediment.
4. Compute the deposited volume by dividing the trapped load to the bulk density.
5. Sum each day of deposition to yield a total accumulated volume.
6. Compare the total accumulated volume based on load (V_{Load}) to the deposition calculated via the lake surveys (V_{survey}).
7. Adjust the sediment load for highest flow (i.e., the "bent down" portion of the curve) until the ratio of V_{load} to V_{survey} is 1.

Table 9-1 summarizes this analysis.

Table 9-1: Analysis Summary

Parameter	Value
FP Bulk Density (lb/ft^3)	57.36
MPP Bulk Density (lb/ft^3)	39.38
a in $Q_s = aQ^b$	0.0203
b in $Q_s = aQ^b$	1.6357
Trapping Efficiency	96.77%
Total Accumulated MPP (ac-ft)	6,677

Table 9-2 indicates how well the calibrated rating curve predicts volume change over other time periods. Unfortunately, differences in survey methods plague the most recent several surveys, which leads to unreliable estimates. For example, the flood control pool in 1989 was based on rangeline surveys, in 2011 on a lower quality USGS DEM, and in 2009 on a higher quality LIDAR. The only volume change estimates deemed reliable are 1969 to 1979 (total), 1979 to 1989 (total), and 2001 to 2009 (multipurpose pool only).

Table 9-2: Volume Calibration Results

Year	Days/Year with Flows above 9,280 cfs	Total Accumulated Computed (ac-ft)	Total Accumulated Survey (ac-ft)	Ratio
1969-1979	1.4	23,182	25,063	0.92
1979-1989	0.6	19,310	14,729	1.31
1969-1989	1	42,492	39,792	1.07
2001-2009* (Multipurpose Pool Only)	1	6,677	6,678	1.00

*Shift in the values due to the change in data collection method described in Table 4-2.

As indicated in Table 9-2, calibrating to the 2001 to 2009 multipurpose pool change leads to an overall underprediction from 1969 to 1979 and an overprediction from 1979 to 1989. Including both decades (1969 to 1989) the differences compensate, such that the model overpredicts by only 7%. This is the best calibration possible given the inconsistencies in the data. When a new Perry Lake survey is conducted, the 2009 to 2020 change will be available as a more consistent and representative calibration time period.

10.0 SEDIMENT TRAPPING DURING FLOOD CONTROL VS. MULTIPURPOSE POOL OPERATIONS

Using the daily deposition amounts computed above, the incoming sediment deposition for the flood control and multipurpose pool were computed according to the reservoir pool elevation on that day. Table 10-1 indicates the quantity of depositing sediment that enters the lake when the reservoir is in flood control operations (i.e., with a water surface above the multipurpose pool elevation), vs. multipurpose pool operations (i.e., when the water surface is at or below that specified). Note that the deposition is classified based on lake conditions when the sediment enters the lake, which precedes the actual deposition.

Table 10-1: Deposition Amounts during Flood Control and Multipurpose Operations

Deposition	Deposition 1969 - 2019 (ac-ft)	% of Total
Total Deposition	100,864	100
Multipurpose Operations (below 891.5 ft)	7,338	7
Flood control Operations (above 891.5 ft)	93,526	93

The deposition was also computed according to the following Water Level Management Plan (WLMP) elevations (Figure 10-1) used on the Lake Regulation Manual (USACE 1984):

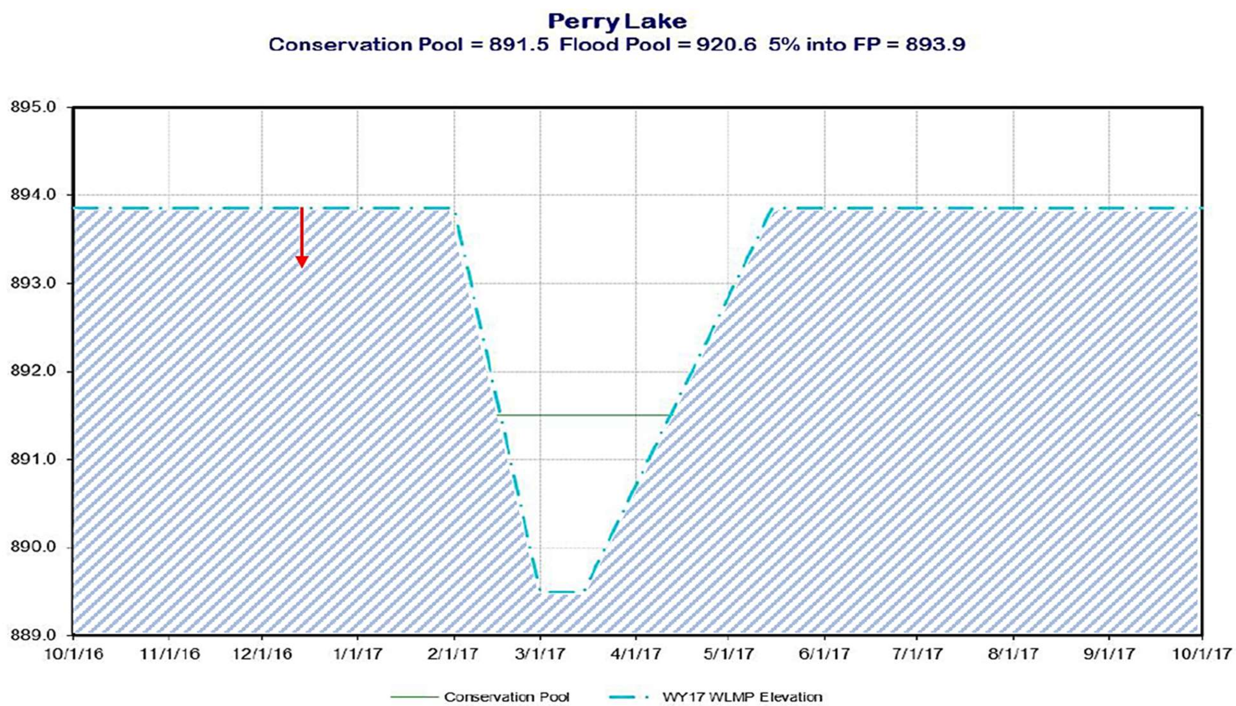


Figure 10-1: Perry Lake WLMP Elevations

Table 10-2: Deposition Amounts- WLMP

Deposition	Deposition 1969 - 2019 (ac-ft)	% of Total
Total Deposition	100,864	100

Deposition	Deposition 1969 - 2019 (ac-ft)	% of Total
Above WLMP	24,105	24
Below WLMP	76,758	76

By either computation, it is clear that most of the sediment enters the lake during flood control operations.

11.0 SEDIMENT CONCENTRATIONS

Figure 11-1 indicates the natural variability concentration of the incoming sediment with the 80% confidence intervals. The confidence intervals were originally computed based on the statistics of the total sample and departure from the best-fit polynomial. However, the height of the intervals was driven by the high variability at moderate flows, which yielded confidence intervals at the highest flows that exceeded all the measured data points. A more refined approach was taken to take into account differences in variability as a function of flow.

The 4th-degree polynomial through log-transformed data was used as a predictor for the mean of a normal distribution in a generalized additive model. The interpretation should be looked at as "the conditional distribution of suspended sediment concentration, given an amount of flow." That is, $f(y|x) = N(\mu=b_4x^4+b_3x^3+b_2x^2+b_1x+b_0, \sigma=a_2x^2+a_1x+a_0)$ where y is SSC, x is flow, and N is the normal distribution with parameters μ and σ. Because the conditional distribution is normal, the best estimate of $y|x$ is $E[y|x]$ and is also the median of the conditional distribution, and it can be computed as μ. To obtain percentiles of the conditional distribution, the quantile function (inverse CDF) is used, $F\text{-}1(p; \mu, \sigma)$ where μ and σ are computed for the given value of flow. The result is overall a function that has a central tendency dictated by the 4th-order polynomial, with spread about the mean dictated by a concave-down quadratic. These analyses were automated using R-scripts. For a more detail on the process refer to the Kansas River Basin Sedimentation Summary.

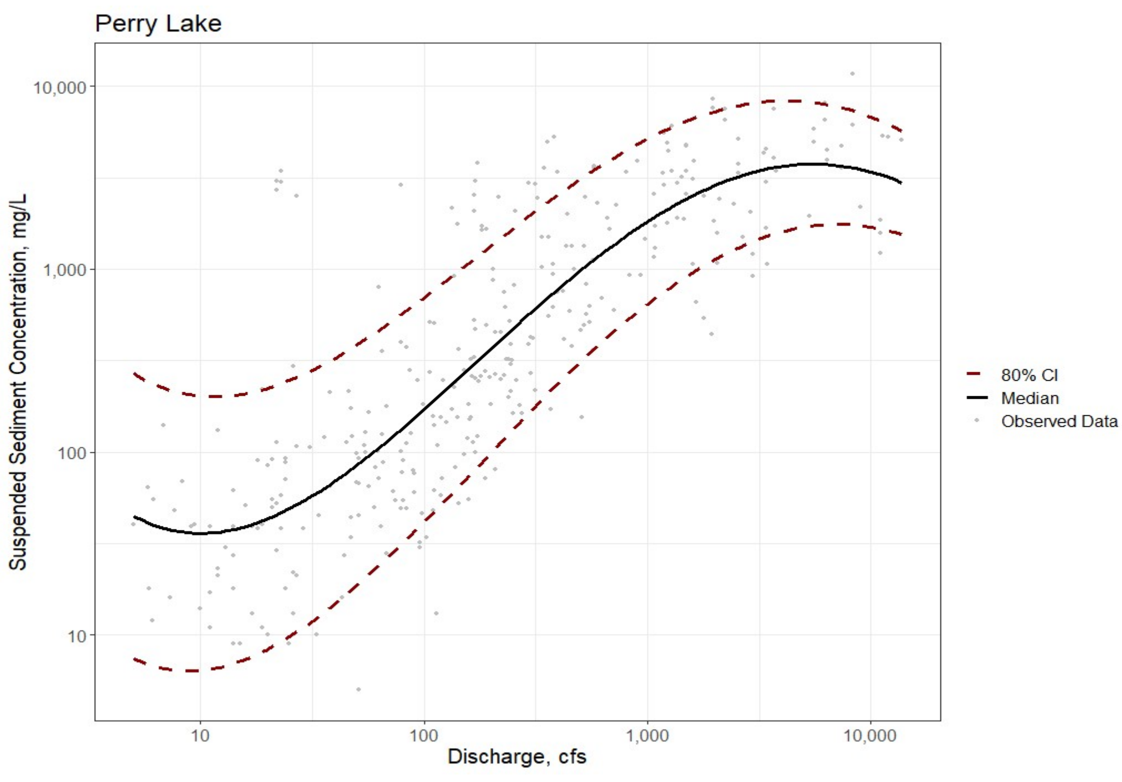


Figure 11-1: Perry Lake Incoming Sediment Concentration vs Flow

Gage station 06890900 (Delaware River at Perry, KS) was used to determine the relationship between flow and the downstream sediment concentration. Figure 11-2 shows that there was insufficient data to define a temporal trend in the downstream sediment loads. Prediction intervals around a regression between flow and concentration were plotted, so that there was a probability of 90% of the real values within the limits (see Figure 11-3).

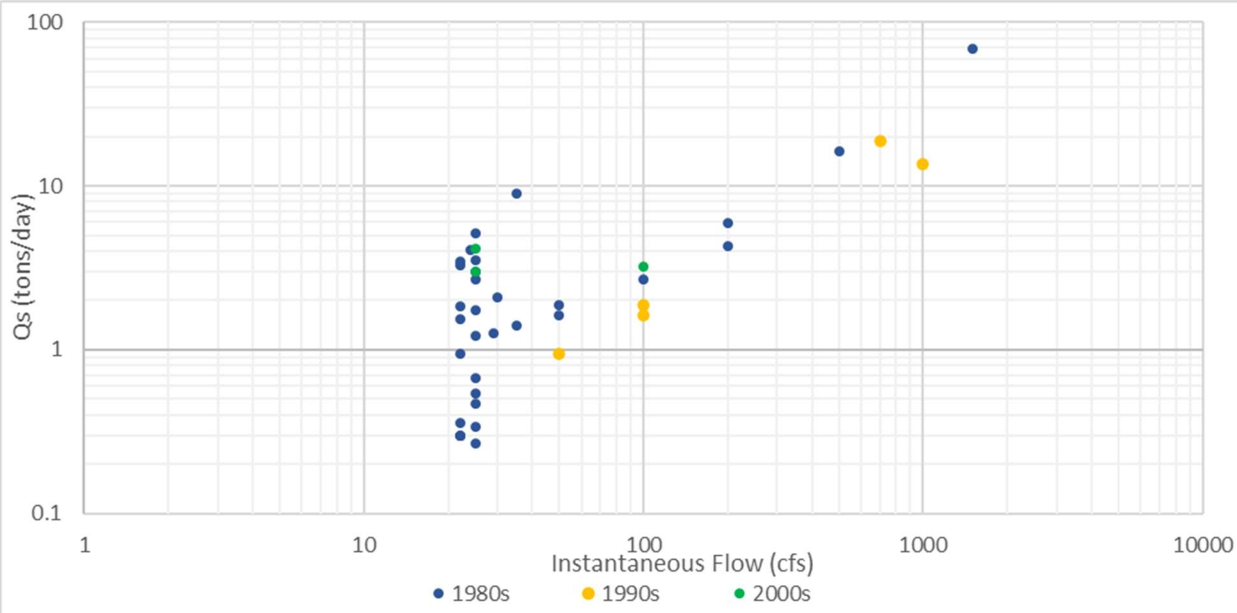


Figure 11-2: Downstream Suspended-Sediment Data

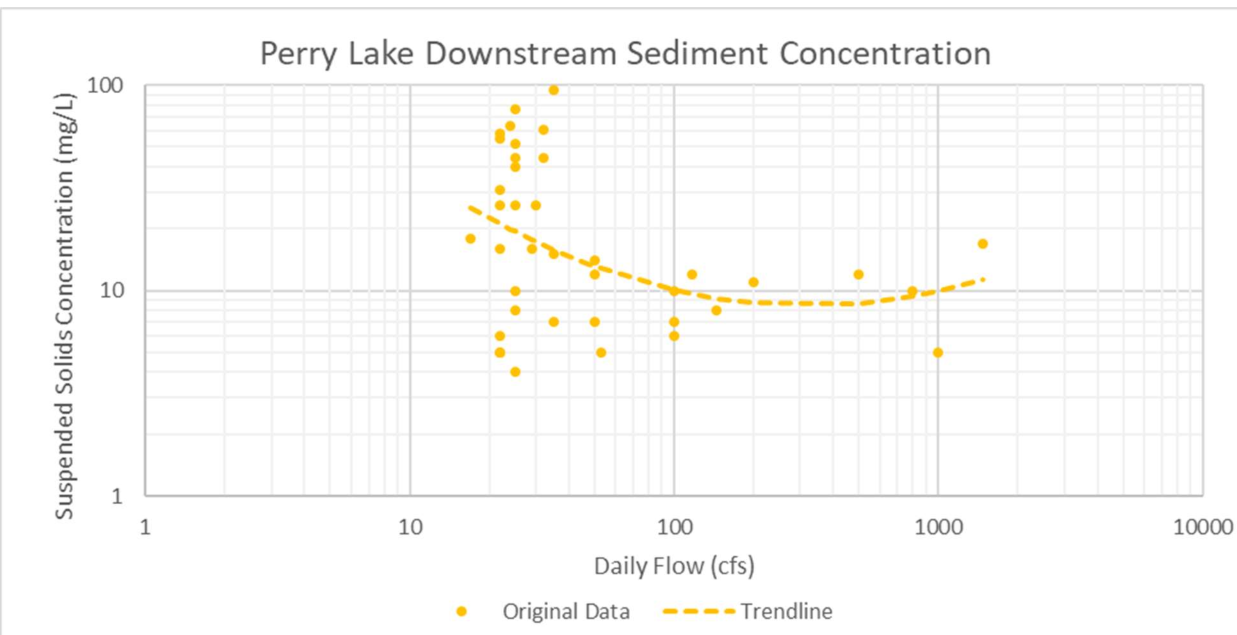


Figure 11-3: Perry Lake Downstream Sediment Concentration vs Flow

The sediment-poor condition of the discharge was evident by comparing the incoming sediment load at 1,000 cfs in the incoming water shown in Figure 11-1 (approx. 2,000 mg/L) to the outgoing water shown in Figure 11-3 (approx. 10 mg/L).

12.0 SEDIMENT CHEMICAL CONCENTRATIONS

Metals and trace elements in sediment originate naturally within the basin. The daily intake of metals and trace elements are classified as deficient, optimal or toxic.

US Environmental Protection Agency (USEPA) has established two levels of concern in sediment for concentration of heavy metals, trace elements, and organochlorine compounds. These two levels are threshold-effects level (TEL) and probable-effects level (PEL). Tests conducted in 2001 by USGS (Juracek 2003) indicate the presence of 22 metals and trace elements of the 26 tested.

Arsenic, chromium and copper exceeded the TELs but were less than the PELs in Perry Lake. For nickel, the concentrations exceeded both the TEL and most the PEL. The following figure represents the chemical concentrations in the bottom sediment of Perry Lake.

[Mean annual net loads and yields have been rounded to three significant figures. mg/kg, milligrams per kilogram; kg/yr, kilograms per year; lb/yr, pounds per year; (kg/ha)/yr, kilograms per hectare per year; (lb/mi²)/yr, pounds per square mile per year; <, less than; --, not calculated or not available]
Kansas, and associate

Constituent	Median concentration (mg/kg)	Mean annual net load[1] (kg/yr)	Mean annual net load[1] (lb/yr)	Mean annual yield[2] [(kg/ha)/yr]	Mean annual yield[2] [(lb/mi²)/yr]	Bioaccumulation index[3]
Nutrients						
Total nitrogen	2,500	3,450,000	7,610,000	12.0	6,850	--
Total phosphorus	1,100	1,520,000	3,350,000	5.29	3,020	--
Carbon						
Carbon (organic, TOC)	19,000	26,200,000	57,800,000	91.2	52,100	--
Carbon (total)	21,000	29,000,000	63,900,000	101	57,700	--
Metals and trace elements						
Aluminum	96,000	132,000,000	291,000,000	460	263,000	--
Antimony	1.3	1,790	3,950	.006	3.43	moderate
Arsenic	19	26,200	57,800	.09	51.4	moderate
Barium	710	980,000	2,160,000	3.41	1,950	low
Beryllium	2.8	3,860	8,510	.01	5.71	low
Cadmium	.5	690	1,520	.002	1.14	moderate
Chromium	99	137,000	302,000	.48	274	moderate
Cobalt	17	23,500	51,800	.08	45.7	high
Copper	33	45,500	100,000	.16	91.4	high
Iron	53,000	73,100,000	161,000,000	254	145,000	low
Lead	28	38,600	85,100	.13	74.2	moderate
Lithium	69	95,200	210,000	.33	188	slight
Manganese	1,400	1,930,000	4,260,000	6.71	3,830	low
Mercury	.05	69	152	.0002	.114	high
Molybdenum	1	1,380	3,040	.005	2.86	high
Nickel	50	69,000	152,000	.24	137	moderate
Selenium	.9	1,240	2,730	.004	2.28	high
Silver	<.5	--	--	--	--	moderate
Strontium	130	179,000	395,000	.63	360	moderate
Sulfur	<1,000	--	--	--	--	--
Thallium	<50	--	--	--	--	low
Tin	3	4,140	9,130	.01	5.71	--
Titanium	4,400	6,070,000	13,400,000	21.1	12,000	moderate
Uranium	<50	--	--	--	--	--
Vanadium	160	221,000	487,000	.77	440	low
Zinc	120	166,000	366,000	.58	331	high
Organochlorine compounds						
Aldrin	<.0002	--	--	--	--	--
Chlordane	<.003	--	--	--	--	--
DDD	<.0005	--	--	--	--	--
DDE	<.0002	--	--	--	--	--
DDT	<.0005	--	--	--	--	--

Figure 12-1: Chemical Concentrations within the Sediment within Perry Lake

Constituent	Median concentration (mg/kg)	Mean annual net load[1] (kg/yr)	Mean annual net load[1] (lb/yr)	Mean annual yield[2] [(kg/ha)/yr]	Mean annual yield[2] [(lb/mi²)/yr]	Bioaccumulation index[3]
		Organochlorine compounds—Continued				
Dieldrin	<0.0002	--	--	--	--	--
Endosulfan	<.0002	--	--	--	--	--
Endrin	<.0002	--	--	--	--	--
Gross polychlorinated biphenyls (PCBs)	<.005	--	--	--	--	--
Heptachlor	<.0002	--	--	--	--	--
Heptachlor epoxide	<.0002	--	--	--	--	--
Lindane	<.0002	--	--	--	--	--
Methoxychlor	<.0025	--	--	--	--	--
Mirex	<.0002	--	--	--	--	--
Toxaphene	<.05	--	--	--	--	--

[1]Mean annual net load in kilograms per year was computed as median concentration multiplied by the mean annual sediment load deposited in Perry Lake (1,380 million kilograms), divided by 1 million. Mean annual net load in pounds per year was computed as mean annual net load in kilograms per year multiplied by 2.205.

[2]Mean annual yield in kilograms per hectare per year was computed as the total mean annual load divided by the area of the Perry Lake Basin (289,303 hectares). Mean annual yield in pounds per square mile per year was computed as the mean annual yield in kilograms per hectare per year multiplied by 571.09.

[3] Bioaccumulation index information for metals and trace elements from Pais and Jones (1997).

Figure 12-2: Chemical Concentrations within the Sediment Within Perry Lake

More information about this study can be found in *Sediment Deposition and Occurrence of Selected Nutrients, Other Chemical Constituents, and Diatoms in Bottom Sediment, Perry Lake, Northeast Kansas, 1969–2001*.

13.0 DELTA LOCATION AND VOLUME

Data from 1979 was converted from datum NGVD29 to NAVD88 by using the National Vertical Datum Conversion Raster. Each range line had multiple correction factors. These factors were determined based on the different quadrants where the lines where located.

Sedimentation rangelines upstream from Perry Lake (Figure 13-1) allow the rate of delta progression to be quantified. Figure 13-2 provides a profile plot of centerline through the lake. At each location, the invert elevation (lowest elevation in a given sedimentation range line) is plotted.

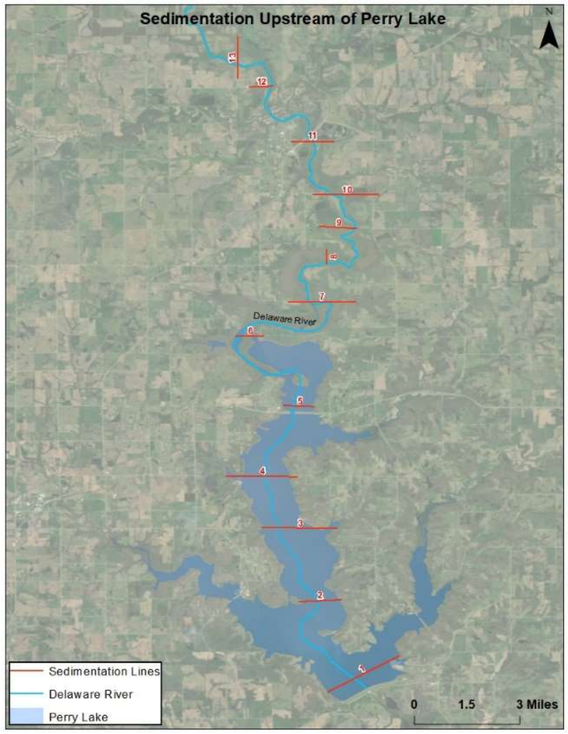


Figure 13-1: Perry Lake Sedimentation Lines

As seen in Figure 13-2, the delta crest grew significantly immediately after dam closure. From 1979 to 2009, the delta progressed downstream at an approximate rate of 397 ft/year. The calibrated rating curve indicates that 6,749,005 tons of sand entered Perry Lake from 1969 to 2019. Using a bulk density of 93 lb/ft^3 for sand, that equals 3,332 ac-ft of sand.

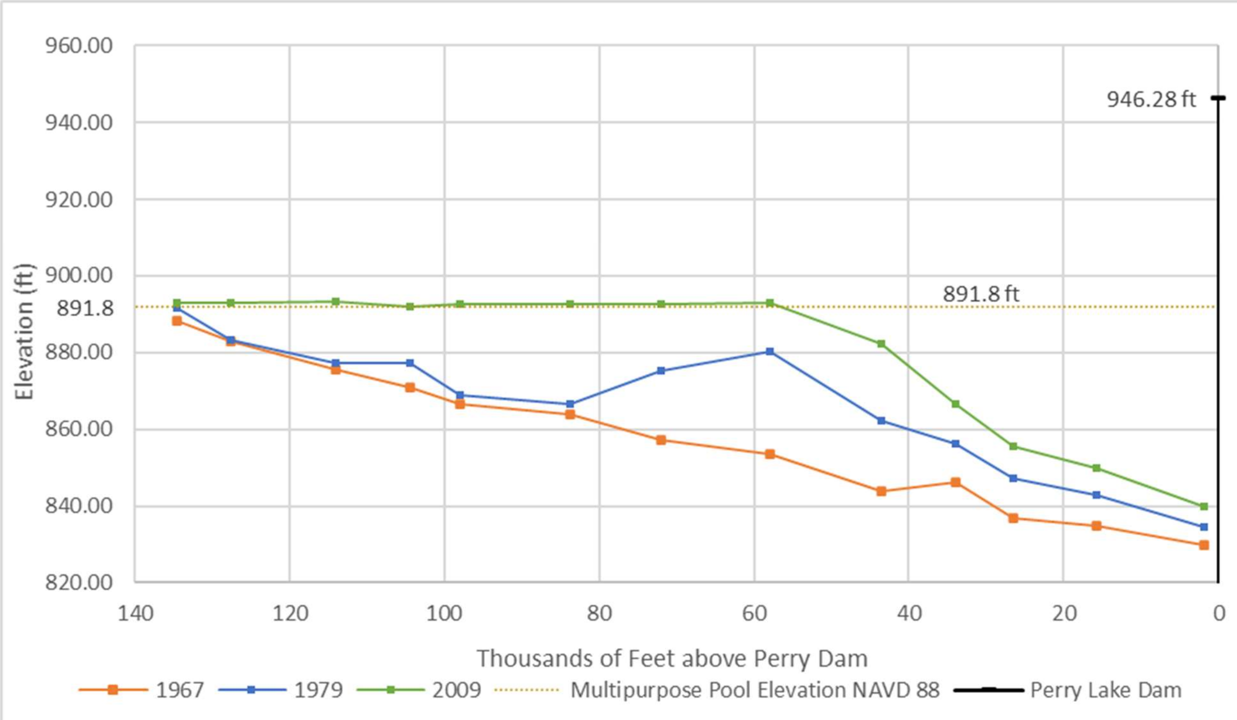


Figure 13-2: Profile of Invert Elevations Indicating Delta Location

14.0 DOWNSTREAM CHANNEL

Data from 1967 to 2002 was converted from datum NGVD29 to NAVD88 by using the National Vertical Datum Conversion Raster. For each range line an average correction factor was applied. These average factors were determined based on the different quadrants where the lines were located.

Sediment trapping by dams very often induces bed degradation and bank erosion downstream. Degradation rangelines downstream from Perry Lake allow this effect to be quantified. For quantification of the downstream geomorphic effects, only the rangelines on the Delaware River (9 to 4) were used. Degradation on the Kansas River rangelines further downstream were not quantified. Figure 14-1 shows the location of the degradation rangelines and the total bed elevation change and channel width change at each.

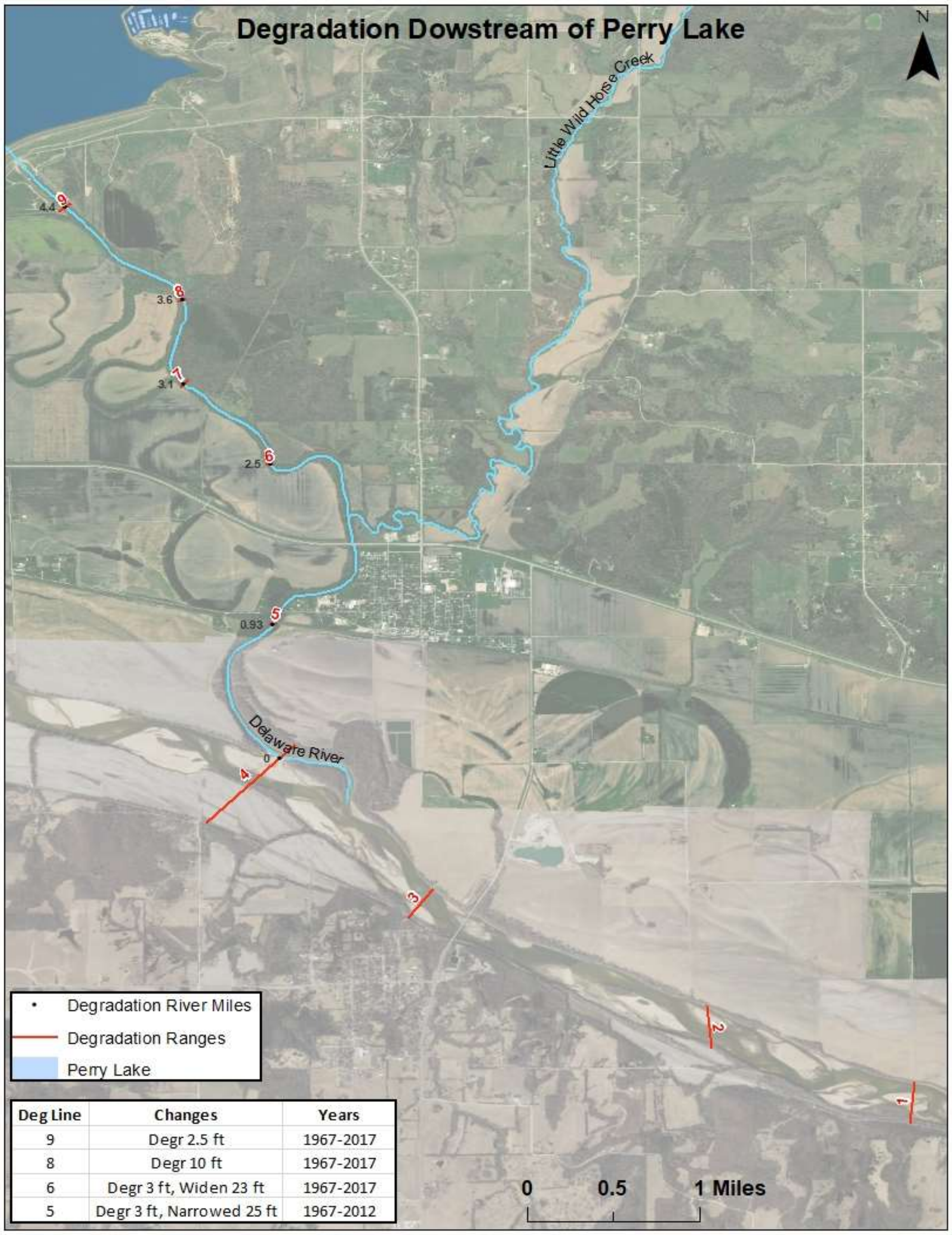


Figure 14-1: Degradation Downstream of Perry Lake

Figure 14-2 plots the cumulative volume change over time based on the degradation rangelines. Based on the rangelines from river mile 0 to 4.4, the bed and banks on the Delaware River downstream from Perry lost 662.8 acre-ft of material from 1967 to 2017. This was significantly less than the 3,332 ac-ft of sand that deposited in Perry Lake. Figure 14-3 shows the yearly cumulative volume change from downstream of the dam.

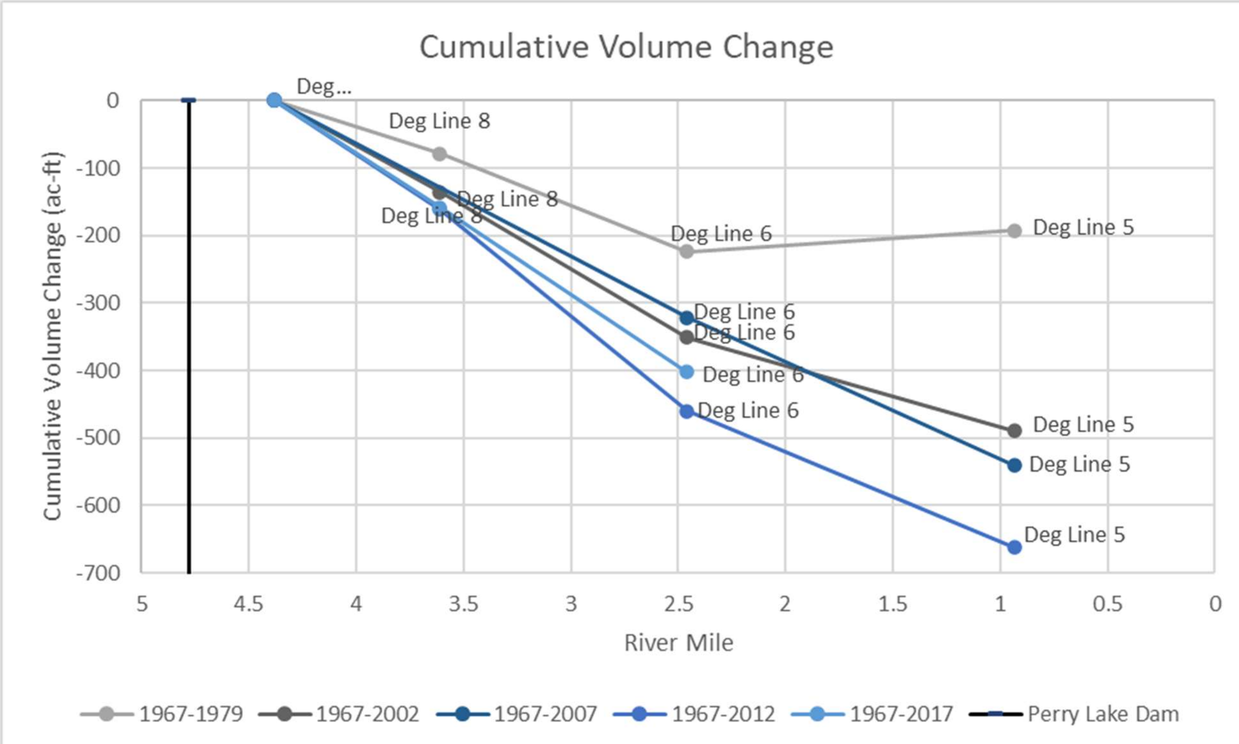


Figure 14-2: Longitudinal Cumulative Volume for Degradation Rangelines

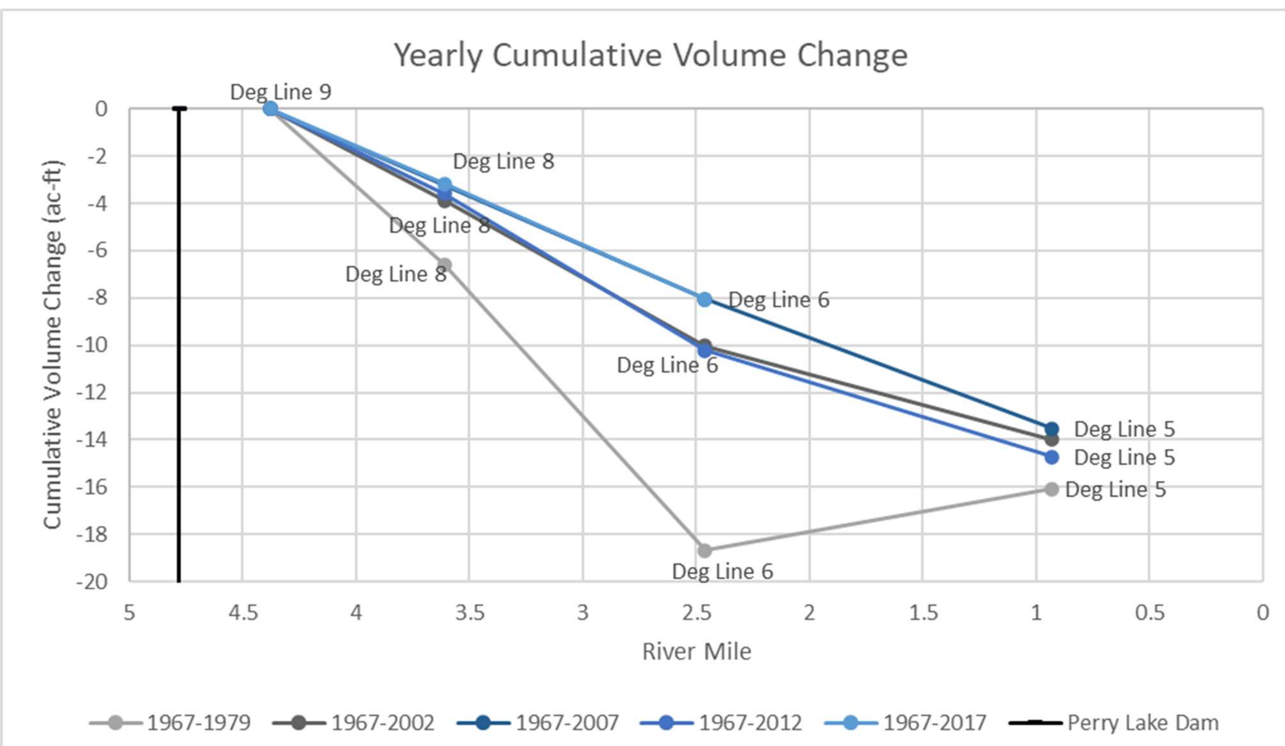


Figure 14-3: Longitudinal Yearly Cumulative Volume for Degradation Rangelines

These analyses indicate that the Delaware River downstream of Perry Lake is sediment starved. Continued degradation with associated bank erosion is expected as sediment trapping continues into the future.

15.0 SUMMARY AND CONCLUSIONS

Perry Dam is located on the Delaware River. It was completed in August of 1966 with a total drainage area approximately 1,117 square miles. The dam has a top elevation of 946.3-ft (NAVD88). The multipurpose pool has an elevation of 891.5 ft (NGVD29) and a flood control pool elevation of 920.6 ft (NGVD29). The most current storage elevation curve has been used operationally from March 2012 to present using a 2009 survey. From 1969 to 2009, the multipurpose pool lost 43,216 ac-ft of storage to sedimentation while the flood control pool lost 6,361 ac-ft.

The trapping efficiency was calculated to be 96.8% using the medium Brune Curve. The bulk density of the incoming load was calculated to be 42.13 lb/ft^3 using the fraction of loads formula. Based on measured data, the multipurpose pool was calculated to be 39.38 lb/ft^3 and the flood control pool bulk density was thus back calculated to be 57.36 lb/ft^3.

Instantaneous flow and sediment data from 1977 to 2019 at USGS #06890100 was used to develop a flow-load rating curve. To be consistent with other lakes in the basin, the upper end of the rating curve was bent down after the 1.5-year flow. The rating curve was calibrated using the deposition in the multipurpose pool between 2001 and 2009 and compared to other. The incoming load is 49.2% clay, 40.5% silt, and 10.3% sand/gravel. The total incoming load for clay was 32,082,292 tons, for silt was 26,400,721 tons and for sand it was 6,749,005 tons.

Using the calibrated daily deposition, the total sediment entering the lake while the water surface was at or below the multipurpose pool elevation was 1,630,800.45 tons/year. 1,512,157.52 tons/year entered the lake while the water surface was in the flood control pool, indicating that 93% of all incoming sediment enters the lake during flood control operations. The incoming sediment was also computed according to the WLMP elevations used on the Lake Regulation Manual. The total incoming sediment while stage was above the WLMP was 15,589,769 tons, whereas the total deposition when stage was below the WLMP was 49,642,249 tons. Thus, 24% of the sediment enters the lake while the pool is above the WLMP elevations.

The 90% prediction limits were calculated to observe the relationship between the flow and the incoming sediment concentration. The sediment concentration during high inflow events is considerably higher than those during low flows. There was insufficient data to define the variability of the downstream sediment.

The USEPA has established two levels of concern guidelines regarding sediment quality for concentration of several metals, trace elements, and organochlorine compounds. These two levels were threshold-effects level (TEL) and probable-effects level (PEL). Arsenic, chromium and copper exceeded the TELs but were less than the PELs in Perry Lake. For nickel, the concentrations exceeded both the TEL and the PEL.

The delta crest grew significantly immediately after dam closure. More recently, from 1979 to 2009, the delta progressed downstream at an approximate rate of 396.54 ft/year. The calibrated rating curve indicates that 6,749,004.93 tons of sand entered Perry Lake from 1969 to 2019. Degradation rangelines on the Delaware River downstream from the dam indicate the downstream bed and banks have lost 662.8 acre-ft of material since dam closure.

These analyses indicate that the Delaware River downstream of Perry Lake is sediment starved. Continued degradation with associated bank erosion is expected if sediment trapping continues.

16.0 REFERENCES

Juracek, K. E. (2003, April 1). Sediment Deposition and Occurrence of Selected Nutrients, Other Chemical Constituents, and Diatoms in Bottom Sediment, Perry Lake, Northeast Kansas, 1969 - 2001, Water-Resources Investigations Report 03-4025, p. 20, 28-29

USACE. (1984). Lake Regulation Manual Perry Lake Kansas. Kansas City District: US Army Corps of Engineers. Volume No. 3.

USACE (2018). Perry Dam (NIDI KS 00009) Periodic Inspection No. 14 Periodic Assessment No. 01. Kansas City District.

Brune, G. M. (1953). Trap Efficiency of Reservoirs. American Geophisical Union.

Confidence and Prediction Intervals for Forecasted Values. Real Statistics Using Excel, www.real-statistics.com/regression/confidence-and-prediction-intervals/.

Duan, N. (1983). “Smearing estimate: A nonparametric retransformation method.” Journal of the American Statistical Association , 78(383), 605–610.

US Army Corps of Engineers ®
Kansas City District

Kansas River Reservoirs Flood and Sediment Study

Appendix D1.3: Tuttle Creek Lake Existing Condition Sedimentation

November 2022

Contents

1.0 INTRODUCTION

Tuttle Creek Dam is located on the Big Blue River 12.3 miles above its confluence with the Kansas River. The Black Vermillion and Little Blue River are major tributaries to the dam. Construction of the dam began in October 1952 and closure was completed in July 1959. The multipurpose pool (MPP) level was first reach in April 1963. Drainage area above the dam is 9,556 square miles, with the predominant land use in the watershed being agriculture and grazing. Authorized purposes of the reservoir include flood control, irrigation, recreation, fish and wildlife, navigation support on the Missouri and Mississippi Rivers, and water quality (USACE, 1973). Figure 1-1 shows Tuttle Creek Lake with respect to the overall Kansas Watershed, while Figure 1-2 shows the lake and the Big Blue Watershed. Also shown in Figure 1-2 are the main U.S. Geological Survey (USGS) gages above the dam that are used in this report.

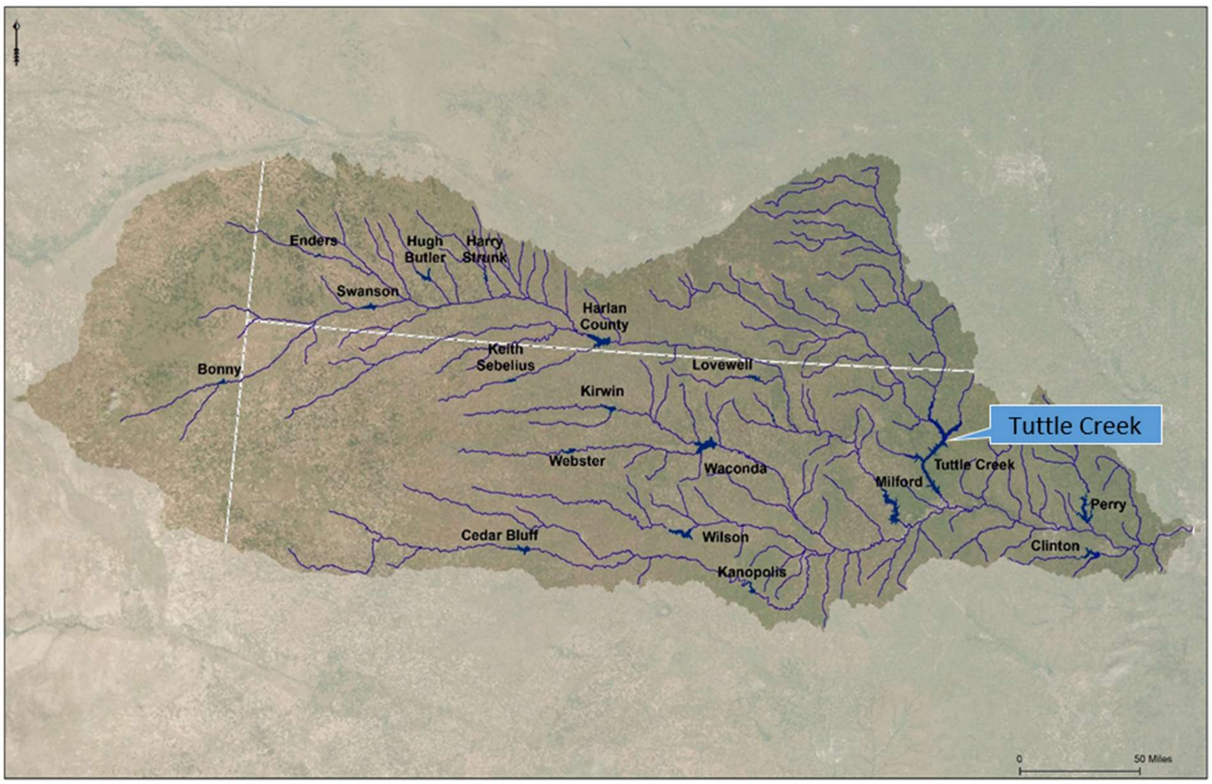


Figure 1-1: Overall Kansas River Basin and Tuttle Creek Reservoir

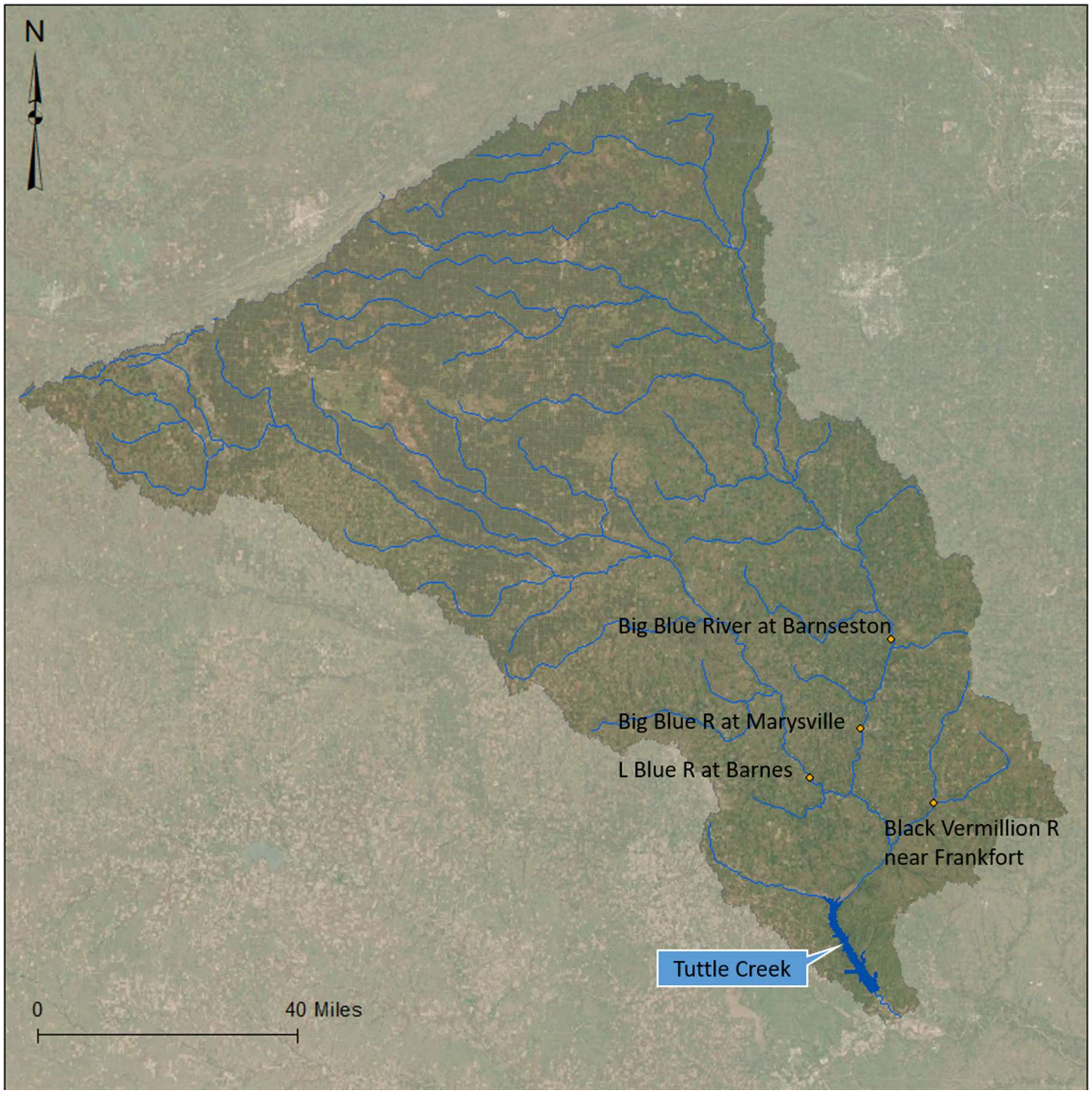


Figure 1-2: Big Blue River Watershed and Tuttle Creek Lake. Dots indicate USGS gages used in this analysis.

2.0 DAM INFRASTRUCTURE

The outlet works consists of an intake tower with four intake gates, two horseshoe conduits, and a stilling basin. Table 2-1 gives pertinent data related to the dam and Figure 2-1 shows the location of key features. Elevations are shown in both NVGD29 and NAVD88. A survey by the Kansas City District (NWK) in 2013 showed a 0.89 ft difference between NVGD29 and NAVD88 (USACE, 2013) at the lake's gage (USGS gage number 06886900). This is higher than the factor of 0.44 feet that was obtained when using the USACE CORPSCON software (Surdex, 2011). The conversion factor obtained from the 2013 survey was used to convert from NVGD29 to NAVD88 in this section as this is considered the official conversion factor for the lake. The four intake gates all have an invert elevation of 1003.16 feet (NVGD29) and a total capacity of 31,300 cfs at multipurpose pool elevation with all gates fully open (USACE, 2015). These gates are not used for discharges below 200 cfs. Additionally, there are two low flow outlets that bypass the main service gates and allow for smaller releases up to 100 cfs each at a lake elevation of 1061.0 (NVGD29). These are mainly operated from April to early November to prevent Zebra Mussels from building up in the pipes. Two concrete, horseshoe conduits carry the water from the intake tower to the baffled stilling basin. The dam also has a gated emergency spillway with an invert elevation of 1116.0 feet (NVGD29) and a capacity of 612,000 cfs during the spillway design flood (USACE, 2015).

Table 2-1: Important Information Relating to the Dam Infrastructure. Elevations in NVGD29 (NAVD 88 in parenthesis)

Parameter	Value
Multipurpose Pool Elevation	1075.0 (1075.89)
Flood Pool (FP) Elevation	1136.0 (1136.89)
Surcharge Elevation	1156.85 (1157.74)
Lowest Elevation Outlet	1003.16 (1004.05)
Number of Gates at This Low Elevation	4
Days per Year Low Level Outlet Operated	322
Spillway Elevation	1116.0 (1116.89)
Dam Elevation	1159.0 (1159.89)
Typical Tailwater Elevation	1015
Other Pipes Going Through the Dam or Embankment (e.g.., Water Intakes, etc.)	None

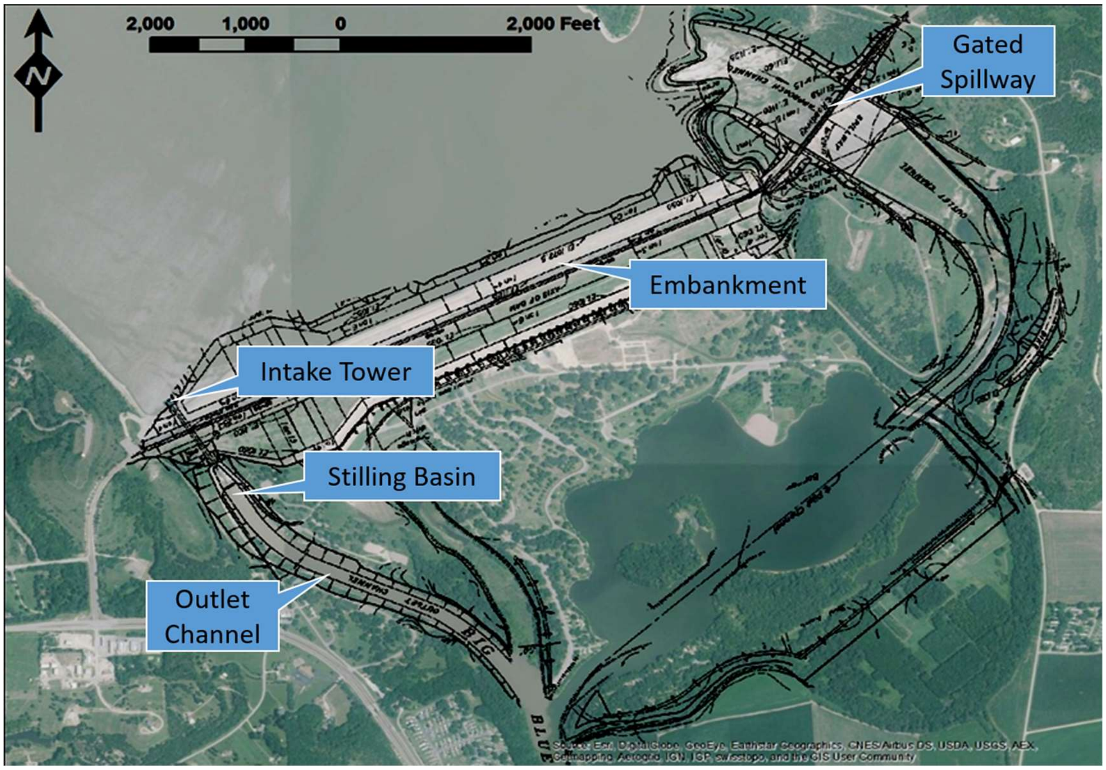


Figure 2-1: Tuttle Dam and Infrastructure (USACE, 2015)

Rocky Ford Dam is located 1.3 miles downstream from the dam at river mile 8.6 and provides a minimum tailwater elevation of 1011.3 NVGD29 (USACE, 1973). The typical tailwater elevation was estimated at roughly 1015 feet (NVGD29) using the tailwater curve shown in Figure 2-2 along with the average discharge from the lake. Figure 2-2 through Figure 2-5 are record drawings of the dam infrastructure.

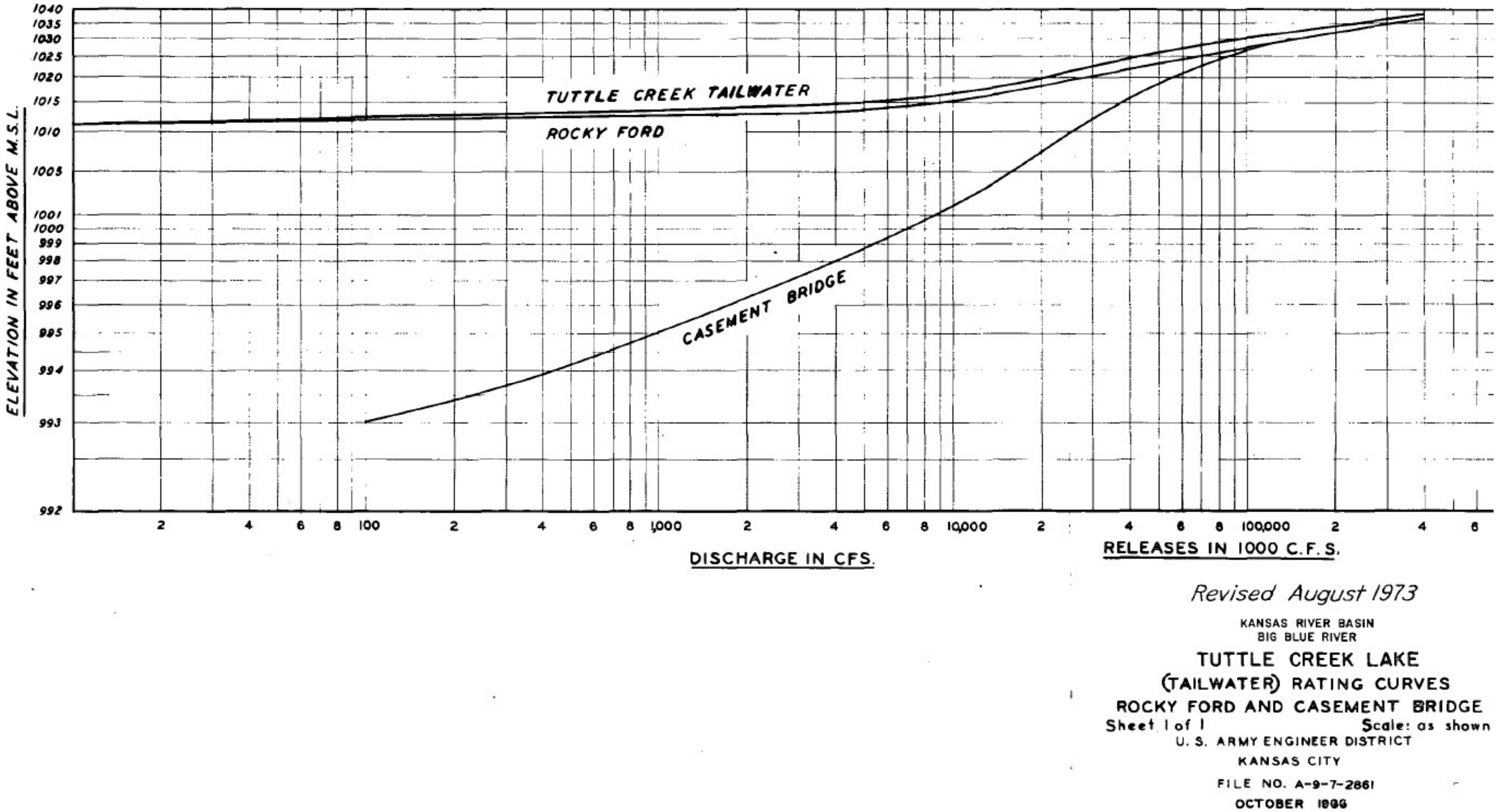


Figure 2-2: Tailwater Rating Curve (USACE, 1973). Elevations in NVGD29.

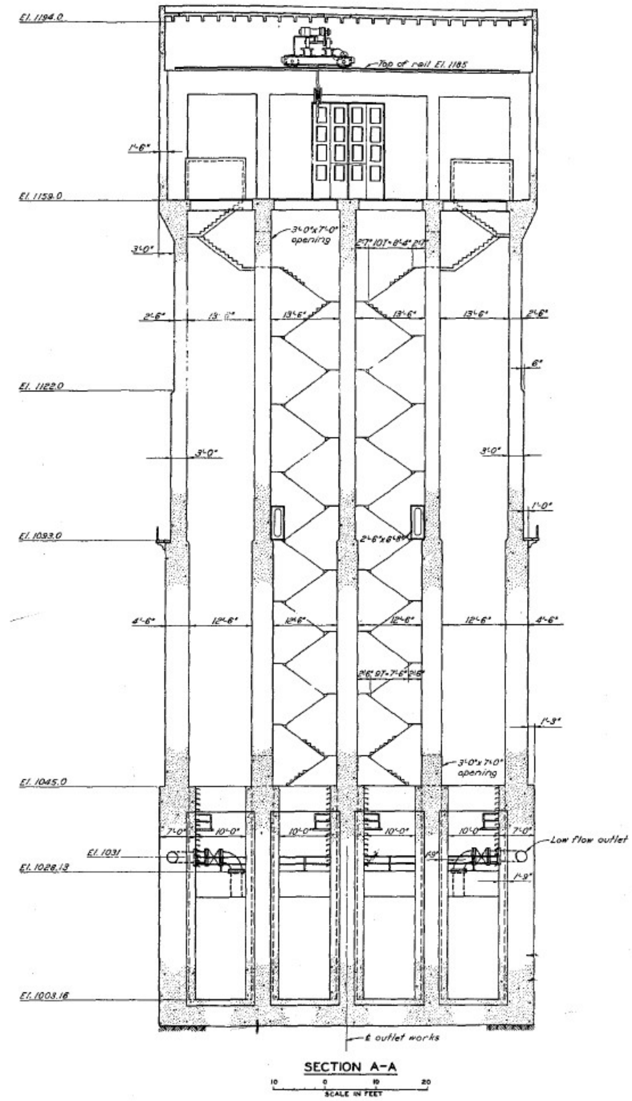


Figure 2-3: Intake Tower and Gates. Elevations in NVGD29

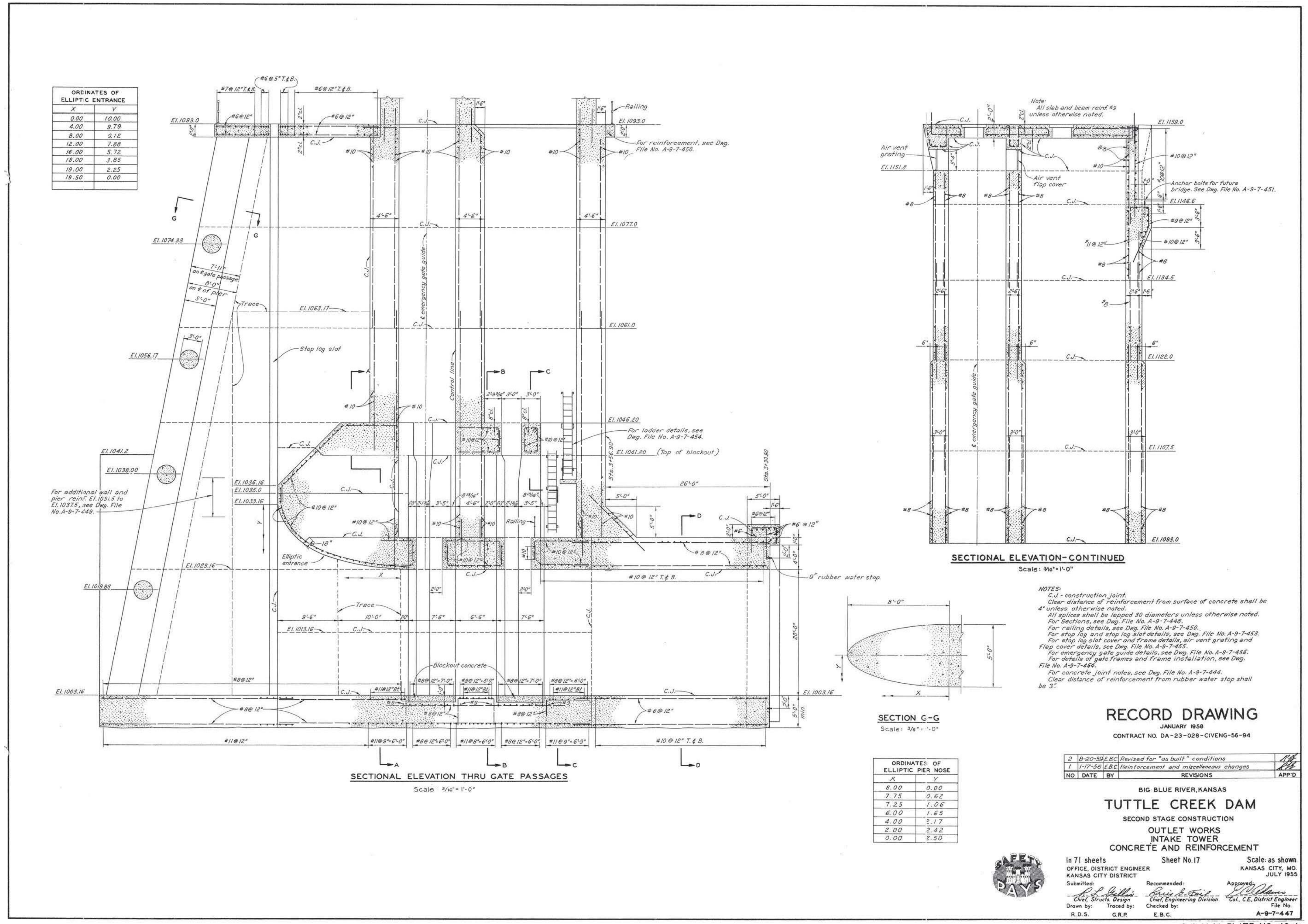


Figure 2-4: Outlet Works and Intake Tower. Elevations in NVGD29.

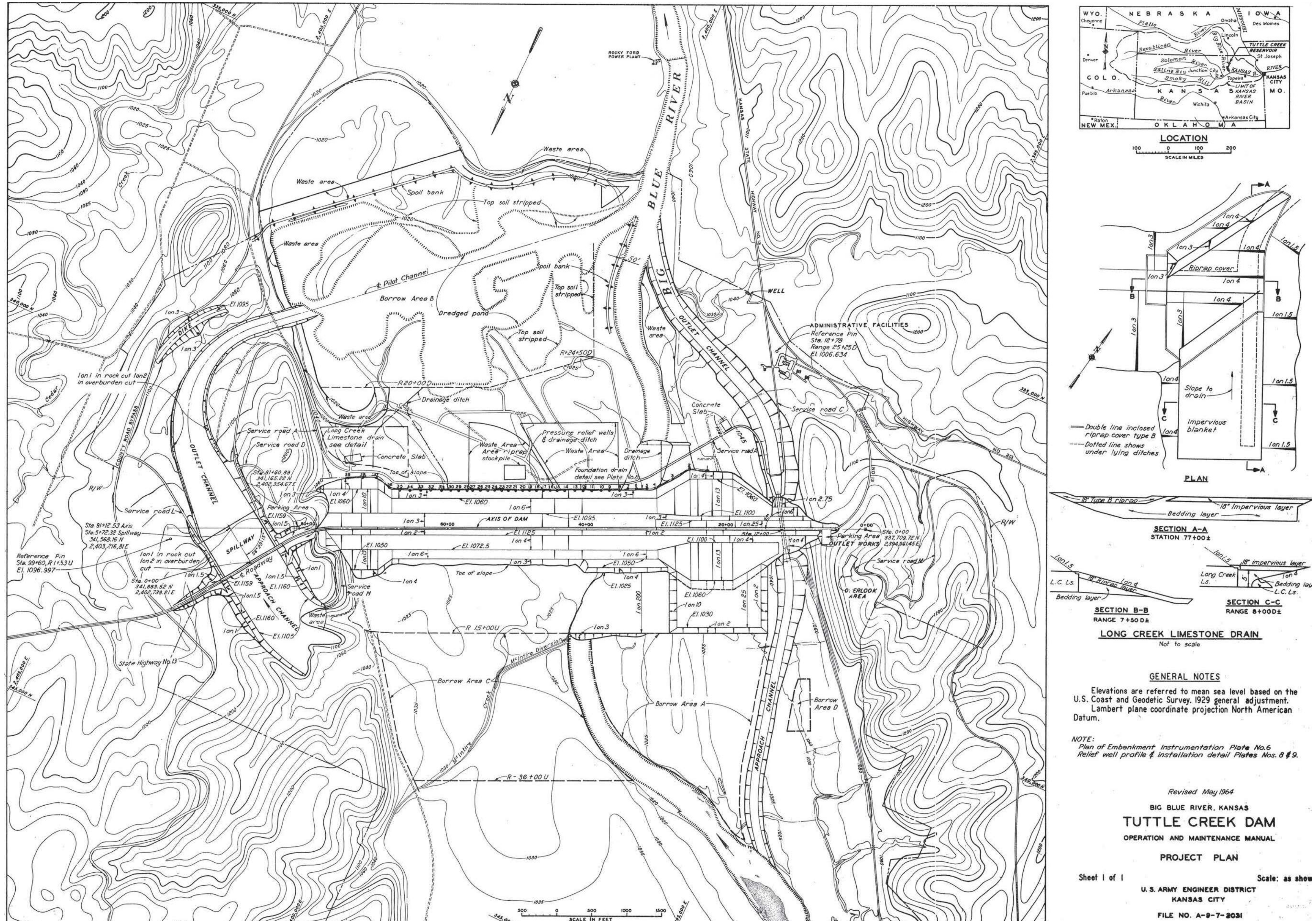


Figure 2-5: Project Plan from the Operation and Maintenance Manual. Elevations in NVGD29.

3.0 SEDIMENTATION EFFECTS ON O&M

Sedimentation has impacted O&M and infrastructure surrounding the lake mainly through the closure of boat ramps and other recreational facilities. Impacts have mostly occurred in the upper portions of the lake as the delta has migrated downstream. A large number of boat ramps have been closed due to insufficient water depths, which generally have to be about three feet to launch a recreational vessel. Maintenance dredging was attempted at several of the boat ramps, which was mostly unsuccessful because of the rapid accumulation of sediment.

Also, a marina was once located in Fancy Creek Cove, which was relocated downstream near the dam once the cove silted in. Campground water supply intakes have also been silted in at several sites.

So far there has not been any impacts to the operation of the service gates from sedimentation; maybe because there is sufficient flushing from their operation to prevent the buildup of sediment near the gates.

4.0 EXISTING CONDITION STORAGE ELEVATION CURVES

The most recent storage elevation curve for Tuttle Creek Lake was created by Surdex from a combination of LiDAR and bathymetry. Bathymetry was collected by Eisenbraun and Associates in June 2009 at a 250 foot transect spacing using single beam sonar. The LiDAR used in the storage curve was obtained by the USGS, mainly in 2010, though data collected in other years was used in the most upstream portions of the flood pool. The 2010 LiDAR had a grid spacing of two meters (Surdex, 2011). Both the LiDAR and bathymetry were combined by Surdex into a digital elevation model (DEM) using Arc-GIS. The Surface Volume in Arc-GIS was then used to calculate the storage elevation tables. Table 4-1 shows the final 2009 storage elevation table.

Table 4-1: 2009 Storage Elevation Curve for Tuttle Creek Lake at 1-ft intervals (NVGD29)

Elevation (ft)	Volume (ac-ft)	Elevation (ft)	Volume (ac-ft)	Elevation (ft)	Volume (ac-ft)	Elevation (ft)	Volume (ac-ft)
1015	0	**1052**	67,028	**1089**	471,719	**1126**	1,645,540
1016	1	**1053**	73,056	**1090**	491,772	**1127**	1,691,366
1017	2	**1054**	79,351	**1091**	512,372	**1128**	1,738,012
1018	5	**1055**	85,901	**1092**	533,530	**1129**	1,785,431
1019	9	**1056**	92,642	**1093**	555,229	**1130**	1,833,632
1020	15	**1057**	99,632	**1094**	577,481	**1131**	1,882,649
1021	24	**1058**	106,831	**1095**	600,277	**1132**	1,932,530
1022	40	**1059**	114,244	**1096**	623,650	**1133**	1,983,328
1023	82	**1060**	121,878	**1097**	647,622	**1134**	2,035,048
1024	142	**1061**	129,683	**1098**	672,176	**1135**	2,087,692
1025	225	**1062**	137,630	**1099**	697,328	**1136.0**	2,141,326
1026	342	**1063**	145,733	**1100**	723,122	**1137**	2,195,960
1027	488	**1064**	153,984	**1101**	749,626	**1138**	2,251,558
1028	667	**1065**	162,369	**1102**	776,862	**1139**	2,308,117
1029	916	**1066**	170,908	**1103**	804,772	**1140**	2,365,649
1030	1,235	**1067**	179,624	**1104**	833,321	**1141**	2,424,220
1031	1,618	**1068**	188,513	**1105**	862,476	**1142**	2,483,874
1032	2,104	**1069**	197,575	**1106**	892,272	**1143**	2,544,617
1033	2,733	**1070**	206,838	**1107**	922,731	**1144**	2,606,539
1034	3,481	**1071**	216,314	**1108**	953,911	**1145**	2,669,648
1035	4,430	**1072**	226,005	**1109**	985,809	**1146**	2,733,922
1036	5,739	**1073**	235,930	**1110**	1,018,444	**1147**	2,799,356
1037	7,321	**1074**	246,166	**1111**	1,051,901	**1148**	2,865,973
1038	9,162	**1075.0**	257,014	**1112**	1,086,172	**1149**	2,933,755
1039	11,301	**1076**	268,008	**1113**	1,121,221	**1150**	3,002,683
1040	13,825	**1077**	279,485	**1114**	1,157,034	**1151**	3,072,760
1041	16,751	**1078**	291,674	**1115**	1,193,617	**1152**	3,144,022
1042	20,000	**1079**	304,259	**1116.0**	1,230,926	**1153**	3,216,501
1043	23,456	**1080**	317,910	**1117**	1,268,947	**1154**	3,290,194
1044	27,179	**1081**	332,409	**1118**	1,307,690	**1155**	3,365,117
1045	31,197	**1082**	347,583	**1119**	1,347,184	**1156**	3,441,291
1046	35,523	**1083**	363,421	**1120**	1,387,456	**1157**	3,518,764
1047	40,174	**1084**	379,947	**1121**	1,428,523	**1158**	3,597,588
1048	45,135	**1085**	397,088	**1122**	1,470,362	**1159.0**	3,677,757
1049	50,319	**1086**	414,834	**1123**	1,512,966	**1160**	3,759,248
1050	55,696	**1087**	433,210	**1124**	1,556,354	**1161**	3,842,070
1051	61,256	**1088**	452,196	**1125**	1,600,544	**-**	-

5.0 TRAPPING EFFICIENCY

From 2009 to 2010, the USGS measured incoming and outgoing sediment loads to Tuttle Creek and computed a sediment trapping efficiency of 98 percent (Juracek, 2011).

From the 2009 survey, the multipurpose pool capacity can be estimated at 257,014 ac-ft. The mean annual water inflow into the lake is 1,558,785 acre-feet, based on stream gage data. Brune offers three curves for estimating trapping efficiency (Brune, 1953), which can be calculated using Equations 1 and 2 and the constants in Table 5-1.

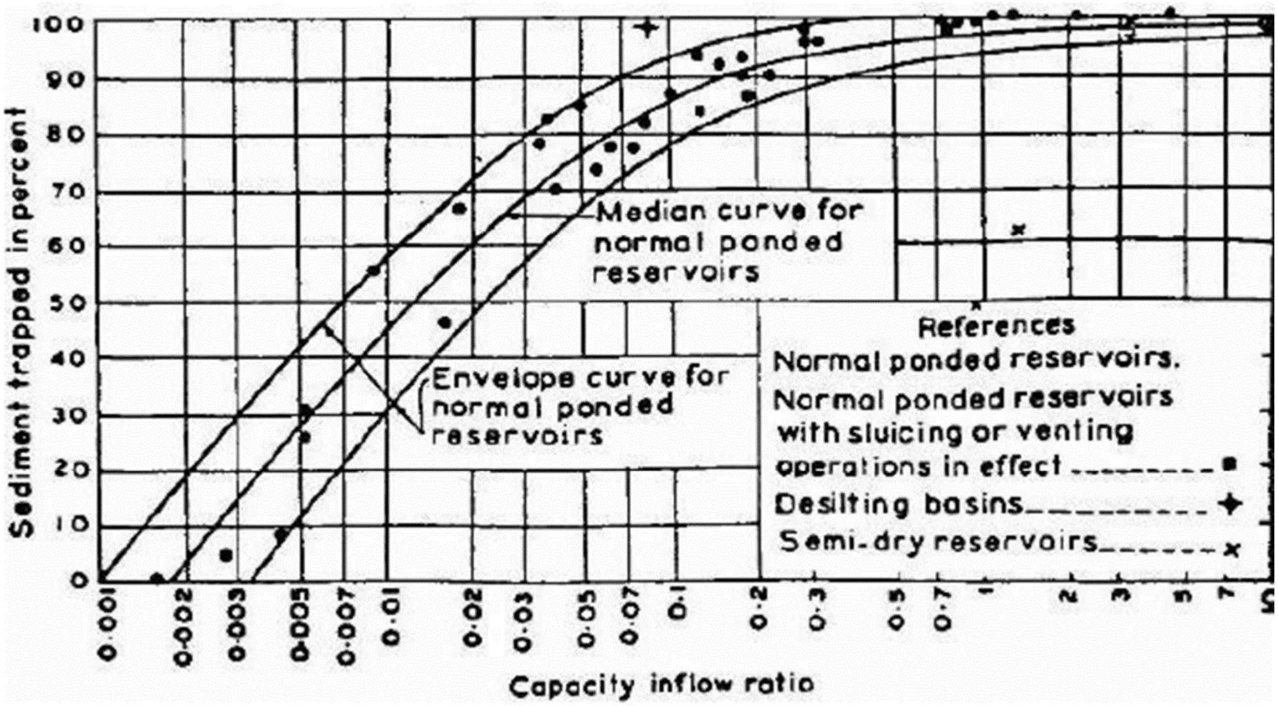


Figure 5-1: Brune Curves, (Brune, 1953)

$$TE = a[1 - 2e^{-bV_*^{0.35}}] \quad (1)$$

$$V_* = \frac{V_{res}}{V_{inflow}} \quad (2)$$

Where:

V_{res} = volume from the multi-purpose pool for Perry Lake (200,004 ac-ft)

V_{inflow} = average of the daily inflow converted into volume

Table 5-1: Constants to be used in Equations 1 and 2

Constant	Low	Medium	High
a	95	97	100
b	5.37	6.42	7.71

Of the three, the high curve best matches the trapping efficiency measured by USGS. The Brune estimate is 96.7%, which is 1.3% lower than the USGS estimate.

6.0 DEPOSITIONAL VOLUME

Historically, sedimentation rangelines have been used for calculating the capacity of the reservoir and determining sediment deposition amounts. Tuttle Creek Lake has a total of 48 rangelines spaced at varying distances as shown in Figure 6-1. Reservoir capacity was calculated from rangeline surveys in 1962, 1973, 1983, and 1993, and 2000.

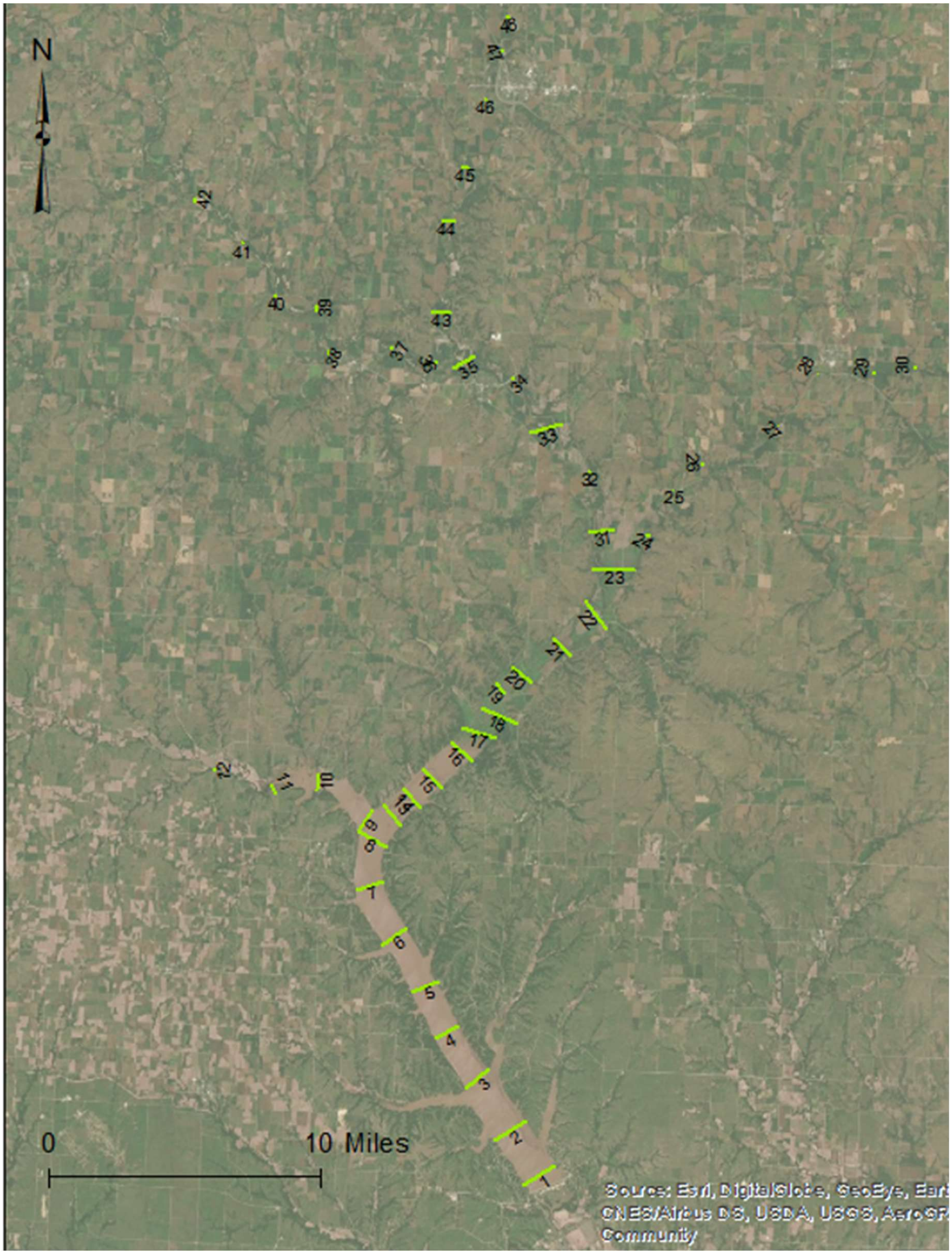


Figure 6-1: Sedimentation Rangelines

As stated previously, the 2009 bathymetric survey was collected using single-beam sonar along transect lines having a spacing of 250 feet. Areas above the multipurpose pool were not surveyed. Figure 6-2 displays the data points.

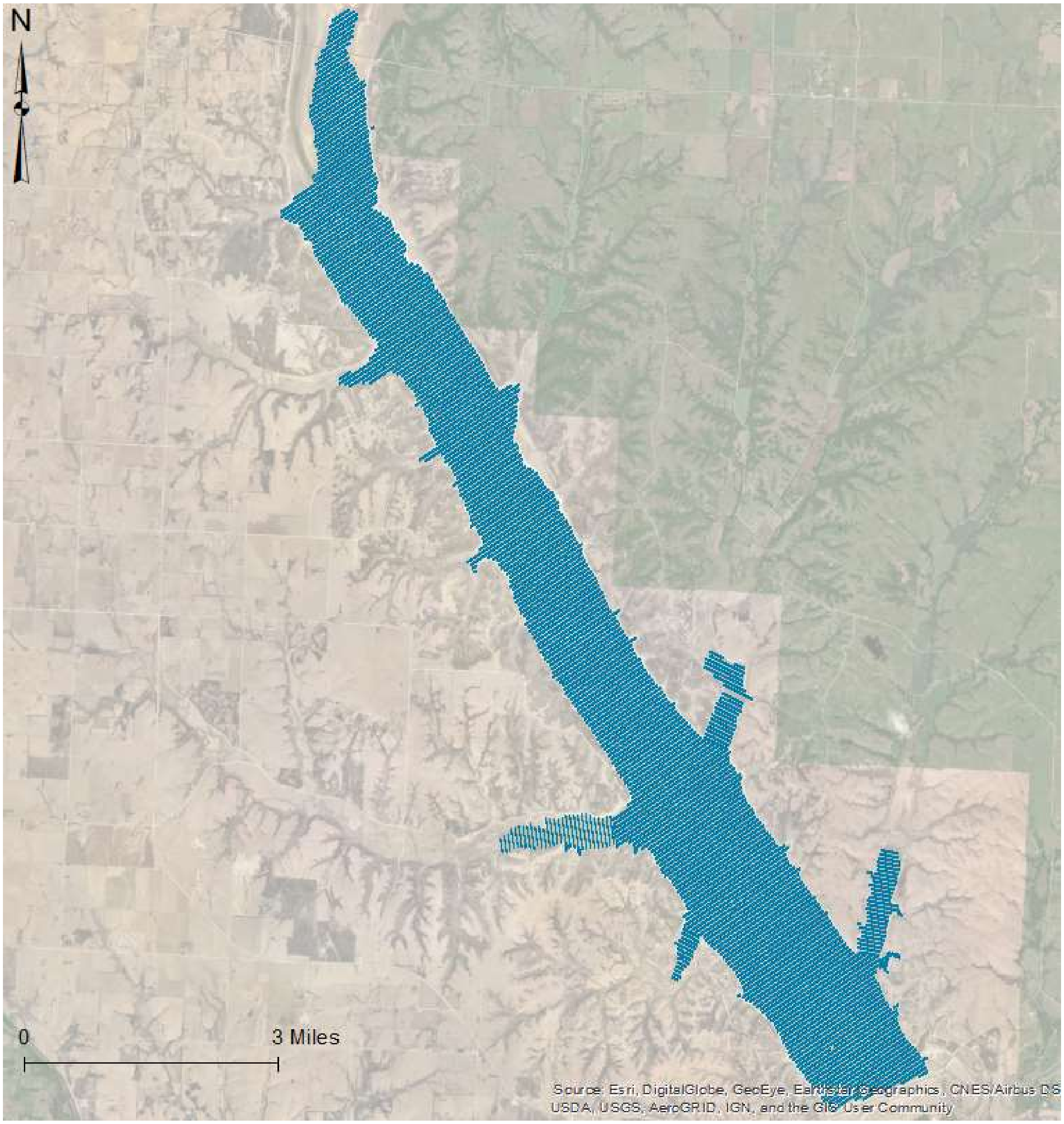


Figure 6-2: Bathymetric Survey Data Points in 2009

The flood control pool was surveyed via LIDAR in 2010 with a grid spacing of 2 meters (Surdex, 2011). Surdex then combined this dataset with the 2009 bathymetry into a DEM having a grid spacing of 12 feet.

Table 6-1 provides the estimated pool volumes for each of the surveys on Tuttle Creek along with the survey methodology. The flood control pool volume does not include the multipurpose pool. The values in this table were taken from the 2011 Surdex report.

Table 6-1: Pool Volumes over Time

Year	Multipurpose Pool Volume (ac-ft)	Flood Control Pool Volume (ac-ft)	Data Type
1962	424,312	1,942,705	Sedimentation Ranglines
1973	388,598	1,937,366	Sedimentation Ranglines
1983	335,100	1,922,085	Sedimentation Ranglines
1993	298,883	-	Sedimentation Ranglines, partial survey
2000	280,137	1,870,735	Sedimentation Ranglines
2009	257,014	1,884,312	Single beam sonar 250 ft spacing, LiDAR

Table 6-2 gives the amount of sediment deposition in the reservoir calculated by subtracting the pool volumes measured from the surveys. Since the 1993 survey was only a partial survey, this survey was skipped when calculating deposition. Also, the survey methodology switched for the 2009 survey, which is likely why there is negative deposition from 2000 to 2009 within the flood pool. Similar “negative deposition” has been observed at other lakes in the basin coinciding with the switch from rangelines to LIDAR. Deposition within the multipurpose pool from 2000 to 2009 appears to be reasonable. In Table 5, FP deposition indicates deposition at elevations higher than the multipurpose pool but lower than the top of flood pool.

Table 6-2: Deposition amounts (ac-ft)

Years	MPP Deposition	MPP Yearly	FP Deposition	FP Yearly
1963-1973	35,714	3,571	5,339	534
1973-1983	53,498	5,350	15,281	1,528
1983-2000	54,963	5,496	51,350	5,135
2000-2009	23,123	2,312	-13,577	-1,358
Total	167,298	3,794	71,970	1,894

From 1962 to 2009, the multipurpose pool lost 167,298 ac-ft of storage to sedimentation. This represents 39.4% of the original multipurpose pool volume. The average annual rate of loss was 3,560 ac-ft/year or 0.84% of the original volume/year.

From 1962 to 2000, the flood control pool lost 71,970 ac-ft of storage to sedimentation. This represents 3.7% of the original flood control pool volume. The average annual rate of loss was 1,894 ac-ft/year or 0.1% of the original volume per year.

7.0 INCOMING SEDIMENT LOADS

USACE and USGS have sporadically collected paired flow/sediment concentration measurements at multiple gages upstream of Tuttle Creek. Table 7-1 shows the gages that were used to estimate sediment inflow into Tuttle Creek along with the period of record when sediment measurements were collected, basin drainage area, and average annual streamflow volume. The Marysville gage does not have daily flow records extending back to closure of the dam since it was installed in 1985. However, there is another stream gage on the Big Blue River that is roughly 20 miles upstream at Barnseton, Nebraska. Yearly streamflow at the Marysville gage is 1.16 times greater than the Barneston gage for the period after 1985. The Barneston daily flows were scaled up by this amount to estimate the daily flows at Marysville prior to 1985. A sensitivity analysis shows that from 2000 to 2009, using a scaled-up Barneston gage would cause a 2.6% decrease in the incoming sediment mass from the Big Blue River and a 1.1% decrease in the overall mass entering Tuttle Creek Lake.

Table 7-1: Stream Gage Information

Gage Name	USGS Gage Number	Sediment Measurements	Drainage Area	Average Annual Streamflow (ac-ft)
Little Blue River near Barnes, KS	06884400	1975-1989, 2008-2010	3,351	483,476
Big Blue River at Marysville, KS	06882510	1986-1989, 2008-2010	4,777	779,949
Big Blue River at Barnseton, NE	06882000	NA	4,370	672,162 (post-1985)
Black Vermillion R near Frankfort, KS	06885500	1976-1990, 2000-2010	410	129,312

The measured sediment concentration was converted into a daily suspended sediment load by multiplying the sediment concentration by the discharge and a conversion factor of 0.0027. The measurements were plotted versus discharge and power trendlines were fitted to them using Microsoft Excel. Duan's (1983) correction for bias introduced by the log transform was applied.

Measurements from various gages within the Kansas River watershed indicate that the sediment rating curves generally begin flattening out at around the 83.3 to 66.7% (1/1.2 to 1/1.5) annual exceedance probability (AEP) discharge. Meaning the rating curve no longer follows the power fit trendlines when the discharge exceeds these values. These flows correspond with the typical range for bankfull flow in Kansas (Shelley 2012). The 1/1.2 and 1/1.5 AEPs are given in Table 7-2 for the gages above Tuttle Creek Lake. Only the data before the rating curves begin flattening was used to create the power fit trendlines. The new slope at the higher end of the rating curve was determined through calibration, as explained later in this appendix.

Table 7-2: 83.3% (1/1.2) and 66.7% (1/1.5) AEP Discharges

Gage	83.3% (1/1.2) AEP Discharge (cfs)	66.7% (1/1.5) AEP Discharge (cfs)
Big Blue River at Marysville, KS	9,080	12,990
Little Blue River at Barnes, KS	7,110	9,940
Black Vermillion River at Frankfort, KS	3,360	5,400

The following figures display the measured suspended sediment loads along with power fit trendlines fitted to the data. The measurements were broken into different time periods to see if the sediment load shifted over time. Figure 7-1 is for the Big Blue River at Marysville, Kansas. There was insufficient data available at this gage to tell if there has been a shift in the rating curve. The flattening in the rating curve appears to begin at the 83.3% (1/1.2) AEP discharge.

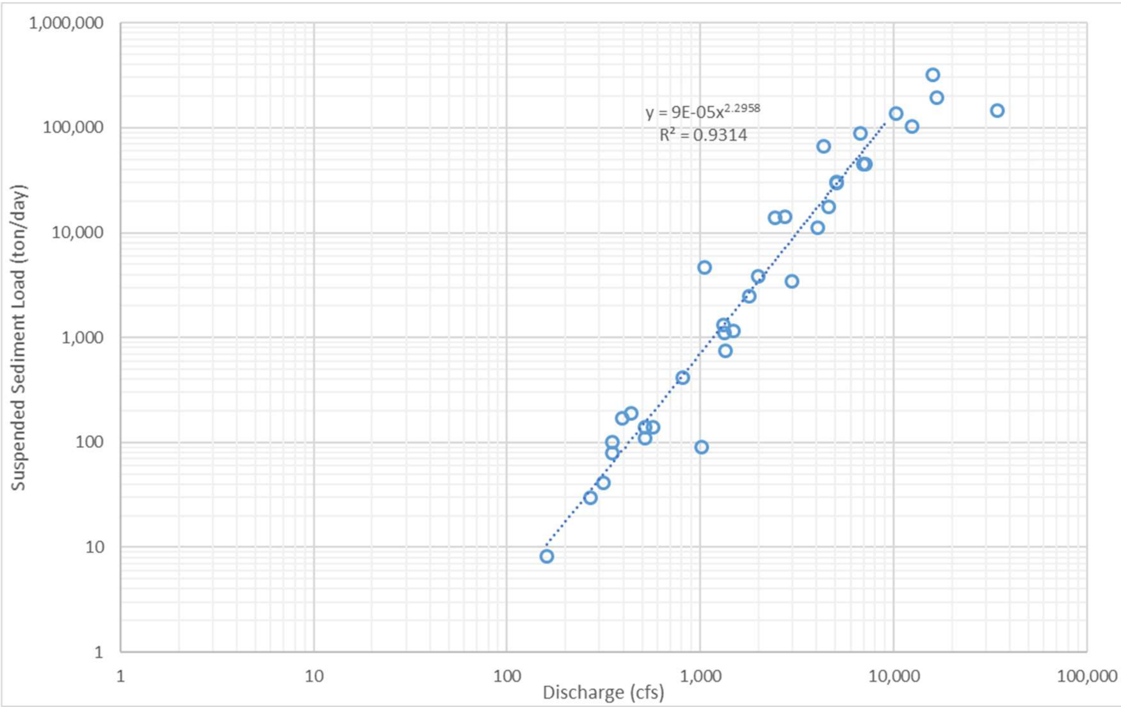


Figure 7-1: Suspended Sediment Rating Curve for the Big Blue River at Marysville, KS

Figure 7-2 is for the Little Blue River at Barnes, Kansas. There appears to have been a small decrease in the rating curve when comparing the 1988-2010 data to 1975-1987. Because of this, only the more recent 1988-2010 measurements were used in developing the sediment rating curves. The rating curve seems to begin flattening at a discharge of approximately 5,000 cfs, which is lower than the 83.3 (1/1.2) AEP event.

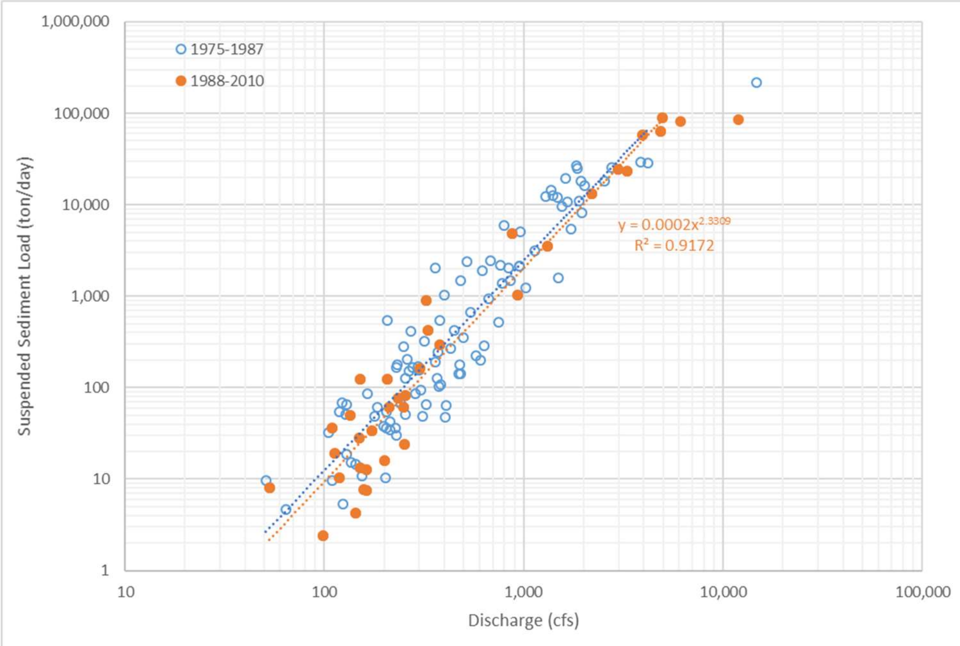


Figure 7-2: Suspended Sediment Rating Curve for the Little Blue River at Barnes, KS

Figure 7-3 is for the Black Vermillion River at Frankfort, Kansas. The rating curve does not seem to have shifted significantly over time. The 1986-2010 data was used to create the sediment rating curve. There have not been many sediment measurements collected at high discharges at this gage. The 83.3 (1/1.2) AEP discharge was used as the starting point of the flatted portion of the rating curve.

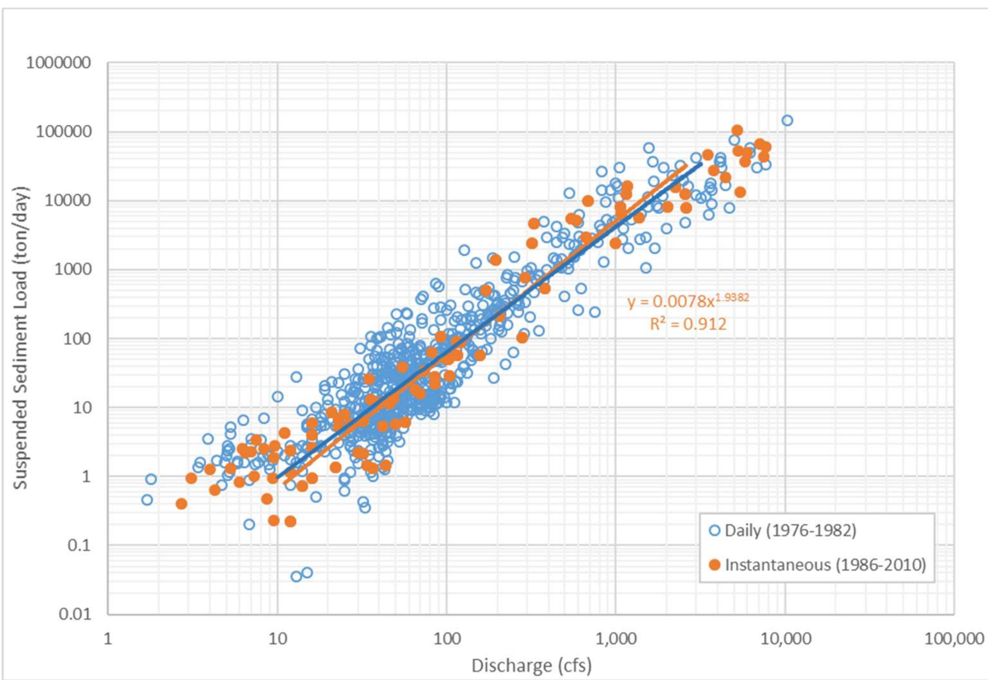


Figure 7-3: Suspended Sediment Rating Curve for the Black Vermillion River at Frankfort, KS

The USGS also periodically measures the suspended sediment gradations of the samples they collect. These were plotted versus the discharge when they were collected to detect trends in the sediment size with discharge. Only measurements with the full gradation measured were used in later analyses.

Sediment gradation appears to trend with discharge at the three gages. Figure 7-4 shows the sediment sizes versus discharge for the Big Blue River at Marysville, KS. Linear trendlines and equations were fitted to the data points up to 5,000 cfs. After this the sizes seem to generally remain constant, and the average of these points was used.

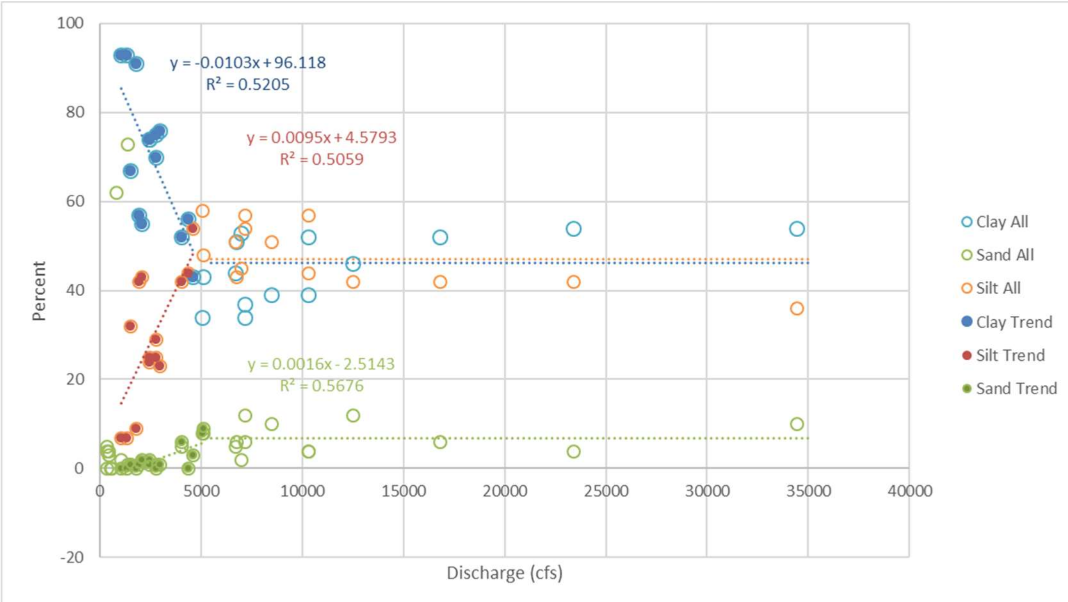


Figure 7-4: Sediment Sizes from the Big Blue River at Marysville with Flowrate

The Little Blue River at Barnes, KS, Figure 7-5, shows similar trends as the Marysville data. The gradations appear to vary up to a discharge of 2,000 cfs before leveling off.

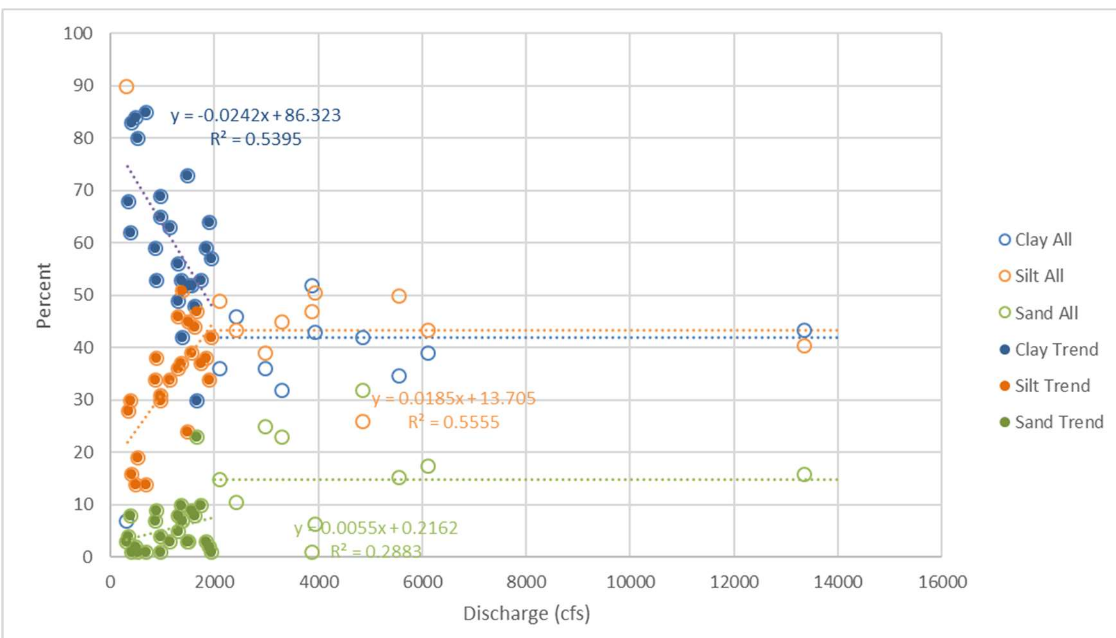


Figure 7-5: Sediment Sizes from the Little Blue River and Barnes, KS

Sediment gradations from the Black Vermillion River at Frankfort, KS, Figure 7-6, show a decrease in the percentage of silt and an increase in clays as discharges increases. The percentage of sand remains basically constant. However, the data exhibits significant and the R-squared values of the trendlines are low.

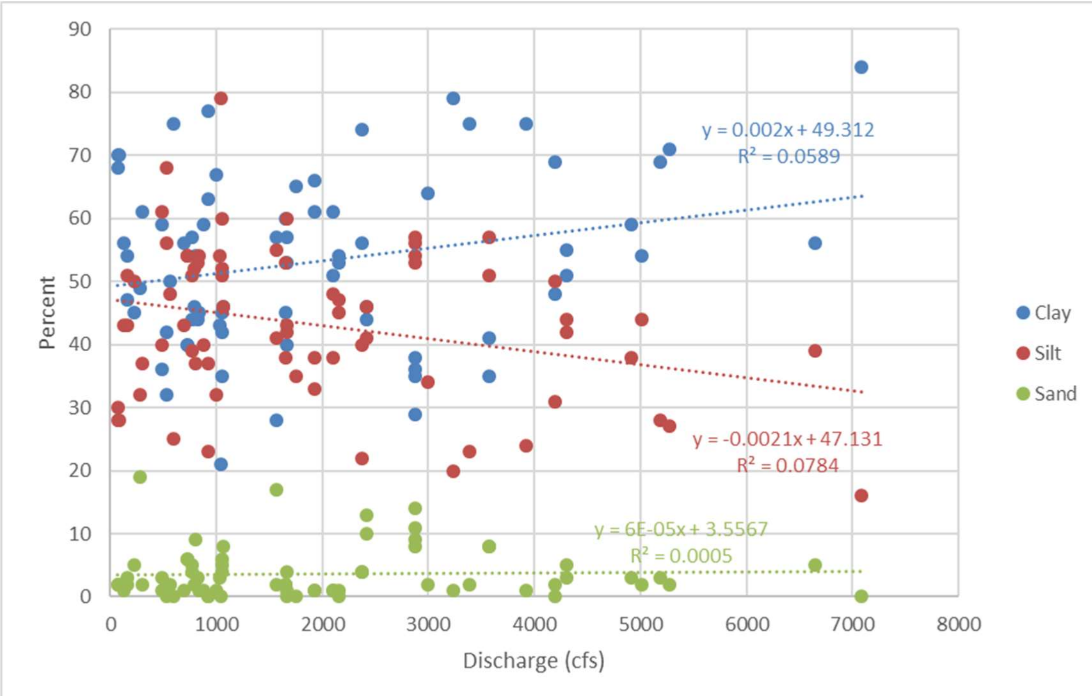


Figure 7-6: Sediment Sizes from the Black Vermillion River at Frankfort, KS

Initially the bedload was assumed to be 5% of the suspended load. USACE (1952) calculates the bedload as being 3.7% of the suspended load based on measurement on the Big Blue River at Randolph, Kansas (USACE, 1952). As the suspended sediment loads have declined slightly overtime, the 5% is an adequate approximation.

A total load rating curve was developed through the following steps:

1. Estimate the best fit regression line of the form $Q_s = aQ^b$ using log-log linear regression (see Table 7-3).
2. Correct for bias using the Duan correction factor (Duan 1983). The Duan E values for the three rating curves are given in Table 7-3.
3. Add 5% to account for bed load to create a total load rating curve.
4. Using the measured data, estimate the percentages of clay, silt, and sand/gravel.
5. Apply this rating curve to daily flow rates from 1962 to 2019 to determine the daily mass of sediment entering the reservoir.
6. Multiply by a factor of 1.12 to account for the ungauged area. This was determined by dividing the total drainage area above the dam (9,556 square miles) by the gauged drainage area above the three sediment gages (8,538 square miles)
7. Multiply by the appropriate percentages to determine the cumulative mass of clay, silt, and sand entering the lake.
8. Sum the daily loads for the three gages to find the total load.

Table 7-3: Sediment Rating Curve Coefficients

Stream	a	b	Duan E

Big Blue River	0.0000911	2.30	1.27
Little Blue River	0.000276	2.31	1.46
Black Vermillion River	0.0078	1.94	1.52

This analysis found the incoming load to be 47.1% clay, 40.8% silt, and 12.1% sand/gravel. Table 7-4 summarizes the results for 1961 to 2019.

Table 7-4: Preliminary Incoming Sediment to Tuttle Creek Lake from 1961 to 2020

Parameter	Value
Years	1962 - 2019
Total Incoming Sediment (tons)	310,192,634
Total Incoming Clay Fraction	47.1 %
Total Incoming Silt Fraction	40.8 %
Total Incoming Sand Fraction	12.1 %

8.0 BULK DENSITY AND CONSOLIDATION OF SEDIMENT DEPOSITS

The bulk density of the deposited sediments has been measured at various times by both the USGS and USACE. The USGS measured bulk density in 1999 and recorded the results in Water-Resources Investigations Report 02–4048. Figure 8-1 shows the location of these bulk density measurements.

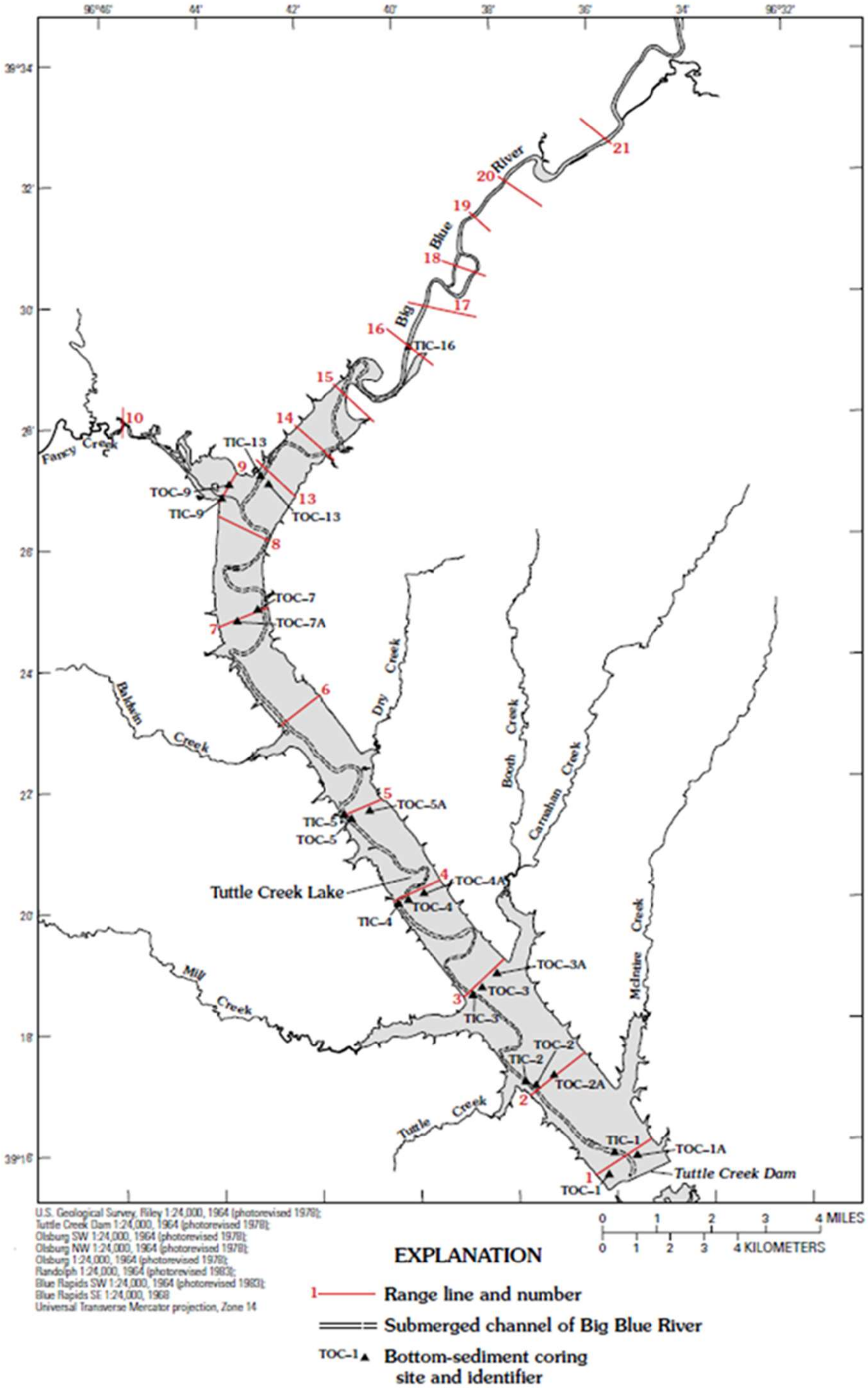


Figure 8-1: Bulk Density Measurements within Tuttle Creek Lake (Juracek & Mau)

Table 8-1 shows the bulk density and sediment mass for different segments of the lake extracted from the USGS report. These values were used to obtain the overall bulk density within the multipurpose pool by weighting the bulk density values between the dam and rangeline 13 by the deposited mass. The mass-weighted bulk density within the multipurpose pool was determined to be 37.4 pcf. The flood pool deposits were assumed to have a bulk density of 60.9 pcf based on bulk density measurements made by USACE in 1988.

Table 8-1: Bulk Densities and Bottom Mass Determined by the USGS

Segment	In Channel Bottom Mass (lbs)	In Channel Bulk Density (pcf)	Out of channel Bottom Mass (lbs)	Out of Channel Bulk Density (pcf)
Dam-1	47,600,000	36.6	610,000,000	30.2
1 to 2	1,310,000,000	37.3	6,160,000,000	30.2
2 to 3	3,890,000,000	39.1	10,700,000,000	26.8
3 to 4	3,330,000,000	36.1	5,240,000,000	26.2
4 to 5	2,850,000,000	34.9	6,850,000,000	30.3
5 to 6	3,620,000,000	37.7	9,130,000,000	31.6
6 to 7	6,990,000,000	46.3	14,600,000,000	33.9
7 to 8	5,740,000,000	46.3	20,300,000,000	36.3
8 to 13	4,310,000,000	46.3	22,800,000,000	48.4
13 to 14	4,570,000,000	54.9	17,900,000,000	64.3
14 to 15	5,090,000,000	57.9	21,100,000,000	64.3
15 to 16	9,210,000,000	57.9	21,900,000,000	64.3
16 to 17	1,900,000,000	60.8	14,500,000,000	64.3
17 to 18	5,440,000,000	60.8	14,500,000,000	64.3
18 to 19	4,370,000,000	60.8	5,120,000,000	64.3
19 to 20	3,050,000,000	60.8	2,940,000,000	64.3
20 to 21	4,560,000,000	60.8	7,460,000,000	64.3

The incoming load can also provide an estimate of bulk density, via Equation 3 along with gradations from suspended sediment samples.

$$\boldsymbol{\gamma_c} = \frac{\mathbf{1.0}}{\left(\left(\frac{F}{\gamma}\right)_{clay} + \left(\frac{F}{\gamma}\right)_{silt} + \left(\frac{F}{\gamma}\right)_{sand}\right)} \tag{3}$$

Where γ_c is the composite bulk density

F is the fraction of clay, silt, or sand

γ for clay, silt and sand is assumed to be 30 pcf, 65 pcf, and 93 pcf respectively.

By Equation 3, the bulk density of the incoming load is 42.9 pcf. This represents the bulk density of both the multipurpose and flood pools.

9.0 CALIBRATING LOAD AND DEPOSITIONAL PARAMETERS

Sufficient data exists to calibrate the rating curve by comparing the deposition computed using the incoming sediment loads to the deposition computed using the surveyed volumes. This was accomplished following these steps:

1. Determine the trapping efficiency of the reservoir (in this case 98%).
2. Apply daily flows at the three gages to their respective rating curves.
3. Apportion the deposition into the multipurpose pool or the flood control pool. Multiply the deposition found in Step 4 by a factor, *m* such that the ratio of MPP deposition to total deposition is correct per the survey analysis. *m* was computed from the surveyed deposition and the bulk densities determined for the MPP and FP.
4. Repeat steps 2 through 3 for each day over a period of time to obtain the cumulative sediment inflow.
5. Compute the mass of trapped sediment in the multipurpose pool by applying the trapping efficiency to the incoming sediment. The percentage of sediment depositing in the flood pool is based on the total incoming load before the trapping efficiency is applied.
6. Transform the trapped mass to a deposited volume by using the bulk densities determined in Section 9 (37.4 pcf in the MPP and 60.9 pcf in the FP.)
7. Compare the total rating-curve-based deposition to the deposition calculated from the surveyed volumes.
8. Adjust the sediment rating curve (described below) to more closely match the surveyed deposition.

Table 9-1 summarizes this analysis.

Table 9-1: Parameters used to Calculate Sediment Deposition in the Reservoir

Parameter	Initial
Bulk Density FP	60.9
Bulk Density MPP	37.4
Bedload % of Suspended	5%
Ungauged Drainage Area Correction	1.12
a in $Q_s = aQ^b$	See Table 7-3
b in $Q_s = aQ^b$	See Table 7-3
Average trapping efficiency	98%

The flattened portion of the rating curves at higher discharges was chosen as the portion of the rating curve to be adjusted for calibration, because it is generally the portion of the rating curve that has the most uncertainty due to lack of measurements. See Table 7-2 for the AEP discharges at the three gages.

Figure 9-1 shows that final total load rating curve for the Big Blue River at Marysville along with the suspended sediment measurements taken at the Barneston and Marysville gages. The Barneston data was not used to create the rating curve but shows that the flatted portions of the rating curve for the higher and lower discharges are reasonable. The rating curve begins flattening at the 1/1.2 AEP discharge, which Shelley (2012) indicates as the median recurrence interval for bankfull flow in Kansas.

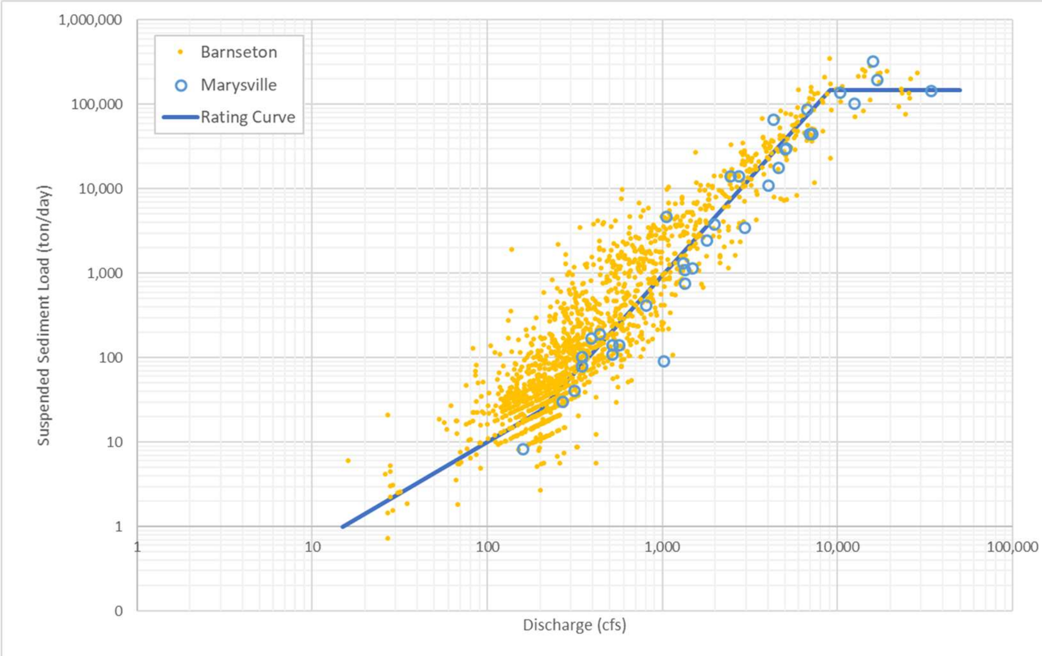


Figure 9-1: Final calibrated total load rating curve for the Big Blue River at Marysville

Figure 9-2 shows that final total load rating curve for the Little Blue River along with suspended sediment measurements taken at Barnes, Kansas. This rating curve begins flattening at around 5,000 cfs, which is lower than the 1/1.2 AEP discharge of 7,110 cfs. However, there is only one measurement in the flattened portion of the curve.

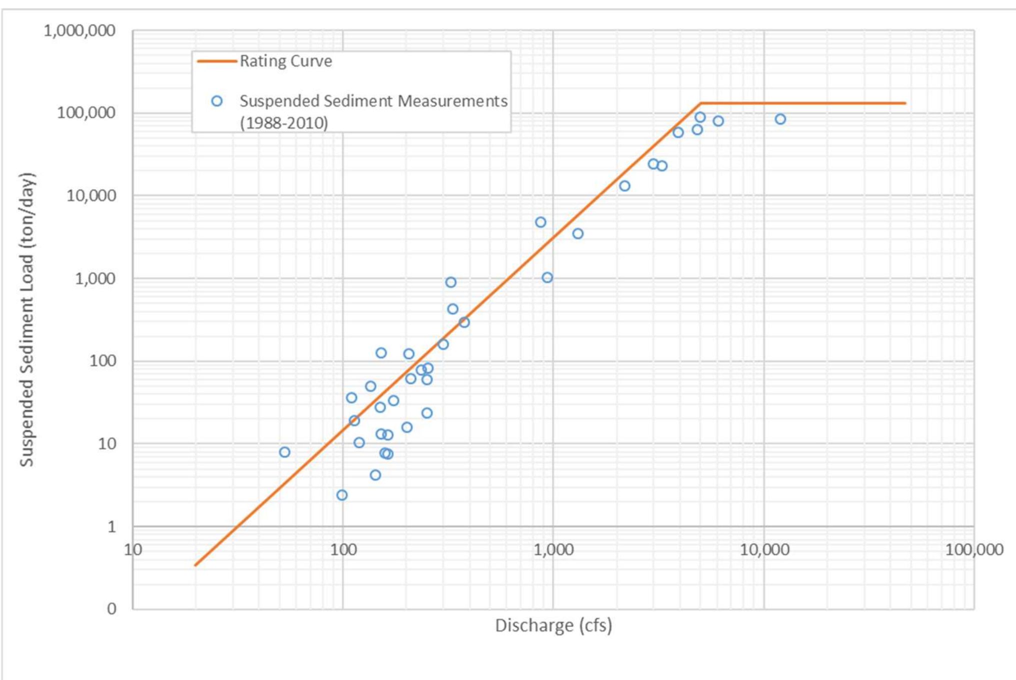


Figure 9-2: Final Total Load Rating Curve for the Little Blue River at Barnes, KS

Figure 9-3 shows the final total load rating curve for the Black Vermillion River and the suspended sediment measurement taken at Frankfort, Kansas. The rating curve begins flattening at the 1.2 AEP event. However, there are few measurements taken in this part of the rating curve.

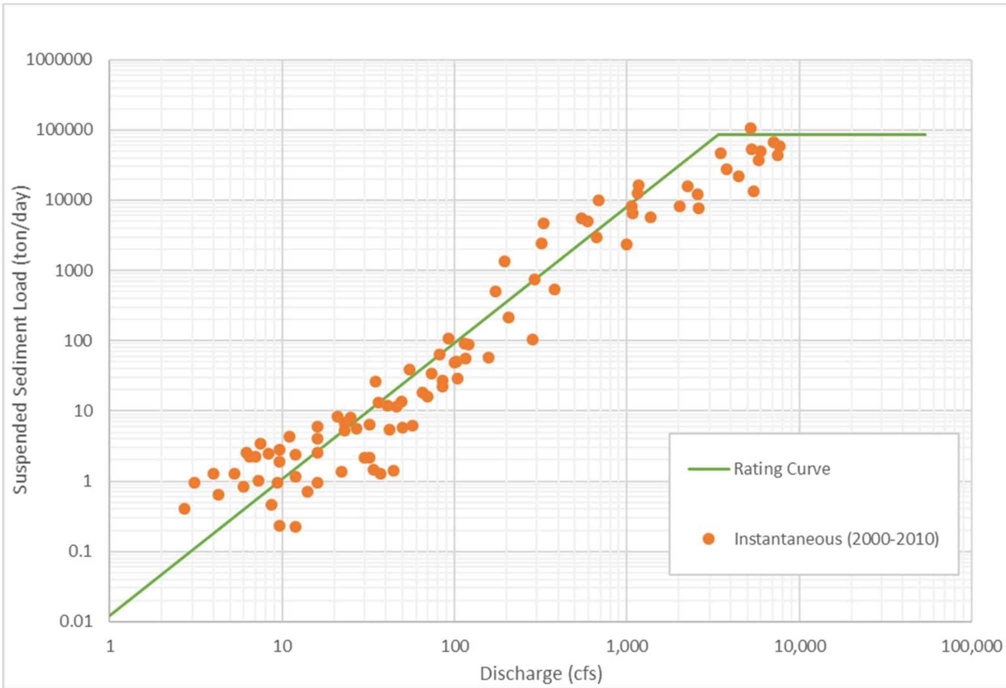


Figure 9-3: Final Total Load Rating Curve for the Black Vermillion River at Frankfort, KS

Table 9-2 gives the final surveyed and computed sediment deposition in the reservoir. The computed deposition generally matches well with the surveyed values. For the time period from between the 1983 and 2000 surveys, the mean bulk density of 42.95 pcf was used to calculate the deposition in the flood pool (FP). This is lower than value of 60.9 pcf used for the other time periods, but it is reasonable considering that most of the sediment deposition in this time period occurred in the flood pool due to the 1993 flood. The 1993 flood set the record for the highest pool elevation at Tuttle Creek and likely caused the sediment, including significant fines, to deposit further upstream in the lake. This would cause the sediment gradation of the flood pool deposits to be finer than during other time periods.

Table 9-2: Surveyed and Computed Sediment Deposition

Time Period	Surveyed (ac-ft)	Computed (ac-ft)	Calculated / Surveyed
1963-1972	40,898	42,907	1.05
1972-1983	68,773	61,557	0.90
1983-2000	124,411	126,015[a]	1.01
2000-2009[b]	23,123	23,367	1.01
Total (1963-2000)	234,082	230,479	0.98

[a]FP deposition calculated using mean bulk density
[b]Surveyed and computed are for the MPP. Change in survey method makes FP deposition unreliable.

10.0 SEDIMENT TRAPPING DURING FLOOD CONTROL VS. MULTIPURPOSE POOL OPERATIONS

The calculated sediment inflows into Tuttle Creek Lake were used to estimate the deposition based on when the lake was in flood control operations (i.e., with a water surface above the multipurpose pool elevation), vs. multipurpose pool operations (i.e., when the water surface is at or below the multipurpose pool elevation). Table 10-1 shows the results of this analysis which indicates that most of the sediment which eventually deposits enters the lake during flood control operations.

Table 10-1: Deposition Amounts during Flood Control and Multipurpose Operations

Deposition	Deposition 1965 - 2019 (ac-ft)	% of Total
Total Deposition	302,110	100 %
Multipurpose Operations	16,039	5.31%
Flood control Operations	286,070	94.69%

Lake levels vary throughout the year depending on a variety of factors. However, lake managers generally try to operate the lake so that pool elevations match the Water Level Management Plan (WLMP) shown in Figure 10-1.

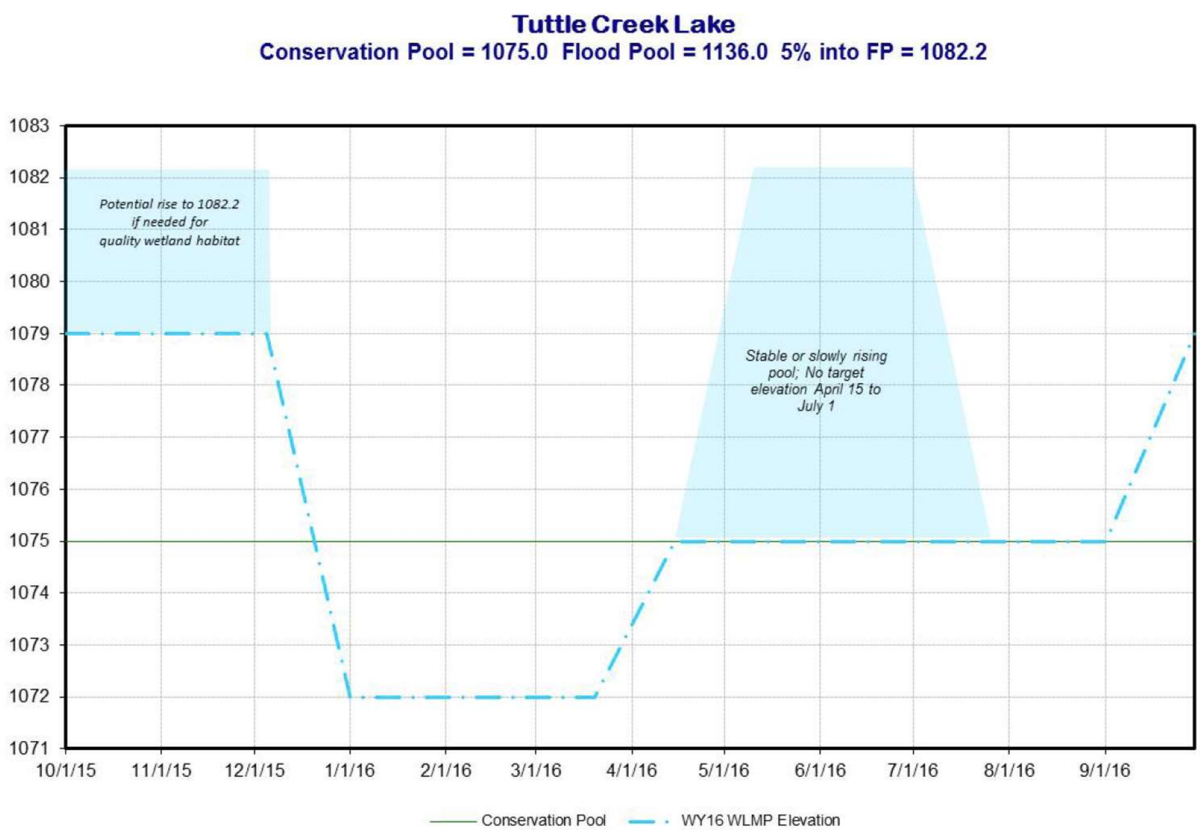


Figure 10-1: Tuttle Creek Water Level Management Plan (USACE, 1973). Elevations in NGVD29.

The quantity of sediment that eventually deposits which enters the lake when the pool elevation is above or below the WLMP line shown in Figure 10-1 is given in Table 10-2. The results are very similar to those from Table 10-1 and show that most of the deposition in the lake occurs when the pool elevation is above the levels given in the WLMP.

Table 10-2: Deposition amounts when Pool Elevation is Below or Above the WLMP

Deposition	Deposition 1965 – 2019 (ac-ft)	% of Total
Total Deposition	302,110	100 %
Below WLMP	14,502	4.8 %
Above WLMP	287,607	95.2 %

By either computation, it is clear that most of the lake deposition derives from sediment entering the lake during flood control operations.

11.0 SEDIMENT CONCENTRATIONS

Figure 11-1 indicates the relationship between flow and sediment concentration for three gages upstream of Tuttle Creek Lake, together with the 80% confidence intervals. The Big and Little Blue Rivers have similar concentrations at a given flow rate, however, the Black Vermillion River concentrations are generally higher and were not included. This graph represents the range of natural variability in the sediment concentrations in the river, i.e., what the concentration would be if the dam were not in place. As a first approximation, the lower bound can be thought of as a minimum target for naturalizing downstream sediment levels, and the upper bound can be thought of as a maximum limit to avoid excessive sediment releases.

The flow/concentration relationship is not monotonic or with a consistent slope. Rather the relationships in log space at low flows, moderate flows, and high flows exhibit separate slopes. By observation, at many of the lakes the sediment concentrations actually reverse at higher flows. This behavior translates into flatter flow-low curves as described earlier in the document. While the reasons for this phenomenon are unknown, it could be can be explained by either the supply limitation of easily erodible sediments or by sediment lost to the floodplain during overbank flows. A fourth-order polynomials through log-transformed data were used to reflect the overall trends in the data. This curve is a valid fit over the range of observed data but should not be used for extrapolation.

Also evident in the data is a reduction in the variability in concentration at higher flows. While fewer measurements could by itself lead to the perception of less variability, visual inspection of the flow/concentration measurements suggests that physical reasons may drive the lower variability. The first possible reason for the reduction in variability is the supply limitation that drives the reduction in concentration. Higher concentrations are constrained by lack of readily available material. A second explanation for lower variability is that moderate flows can be achieved by a precipitation in only part of the watershed, and different sub-watersheds may have different sediment contributions. On the other hand, very high flows are only achievable when most of if not all the entire watershed contributes, which reduces the spatial variability based on storm placement.

The confidence intervals were originally computed based on the statistics of the total sample and departure from the best-fit polynomial. However, the height of the intervals was driven by the high variability at moderate flows, which yielded confidence intervals at the highest flows that exceeded all the measured data points. A more refined approach was taken to take into account differences in variability as a function of flow.

The 4th-degree polynomial through log-transformed data was used as a predictor for the mean of a normal distribution in a generalized additive model. To capture the changing variance of concentration with respect to flow, a concave-down function was needed to predict the sigma term in order to produce a local maximum in the middle with decreases in variance at both extremes. A quadratic function was used to ensure $d2\sigma/dx2<0,\forall x$. The result is a six-parameter model (plus two intercepts) in a hierarchical structure.

The interpretation should be looked at as "the conditional distribution of suspended sediment concentration, given an amount of flow." That is, $f(y|x)=N(\mu=b4x4+b3x3+b2x2+b1x+b0, \sigma=a2x2+a1x+a0)$ where y is SSC, x is flow, and N is the normal distribution with parameters μ and σ. Because the conditional distribution is normal, the best estimate of $y|x$ is $E[y|x]$ and is also the median of the conditional distribution, and it can be computed as μ. To obtain percentiles of the conditional distribution, the quantile function (inverse CDF) is used, $F\text{-}1(p;\mu,\sigma)$ where μ and σ are computed for the given value of flow. The result is overall a function that has a central tendency dictated by the 4th-order polynomial, with spread about the mean dictated by a concave-down quadratic.

These analyses were automated using R-scripts.

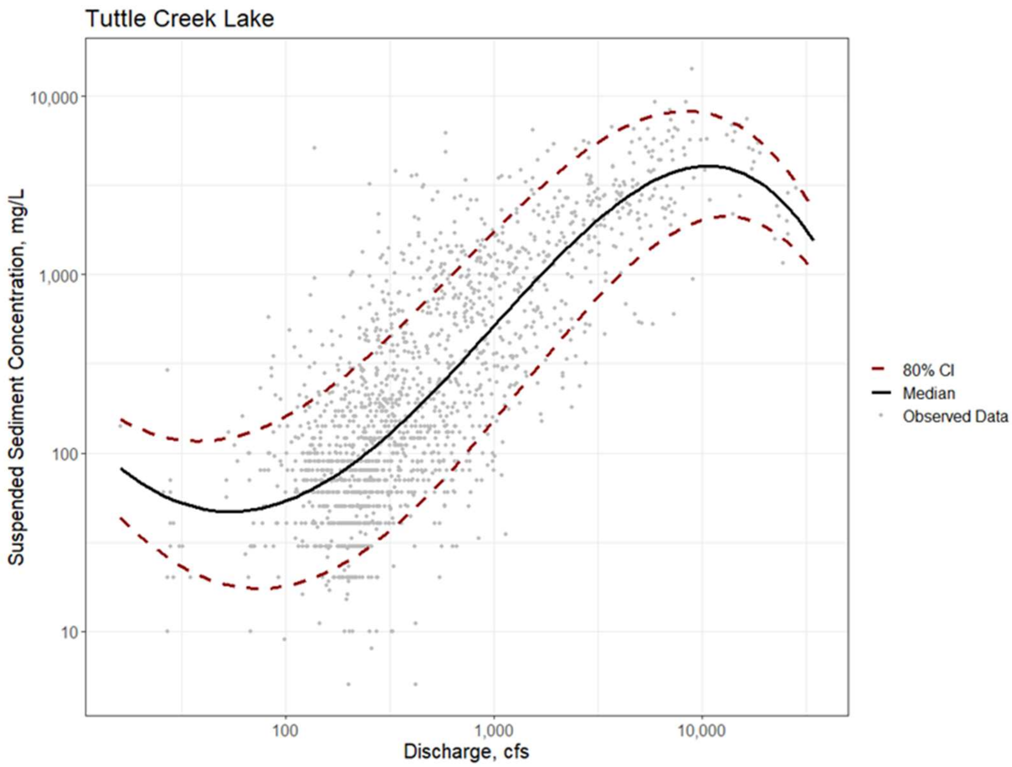


Figure 11-1: Incoming Sediment Concentration to Tuttle Creek Lake

As seen in Figure 11-1, the sediment concentration in the water during high inflow events is considerably higher than those during low flows. This sets a reasonable upper bound for sediment restoration activities to remain within the natural variability of the system. At the highest flows the concentrations decrease, suggesting a supply-limited system.

Downstream sediment concentrations were obtained from the Big Blue River near Manhattan, Kansas (USGS gage 6887000). The results given in **Error! Reference source not found.** are for the concentrations both upstream and downstream of the dam and shows the effect that sediment trapping from the lake has on sediment concentration. The concentration of sediment is noticeably less downstream of the dam then it is upstream, especially for higher flow rates. The Manhattan gage is located 2.4 miles downstream of the dam, so there may be some sediment that enters between it and the dam.

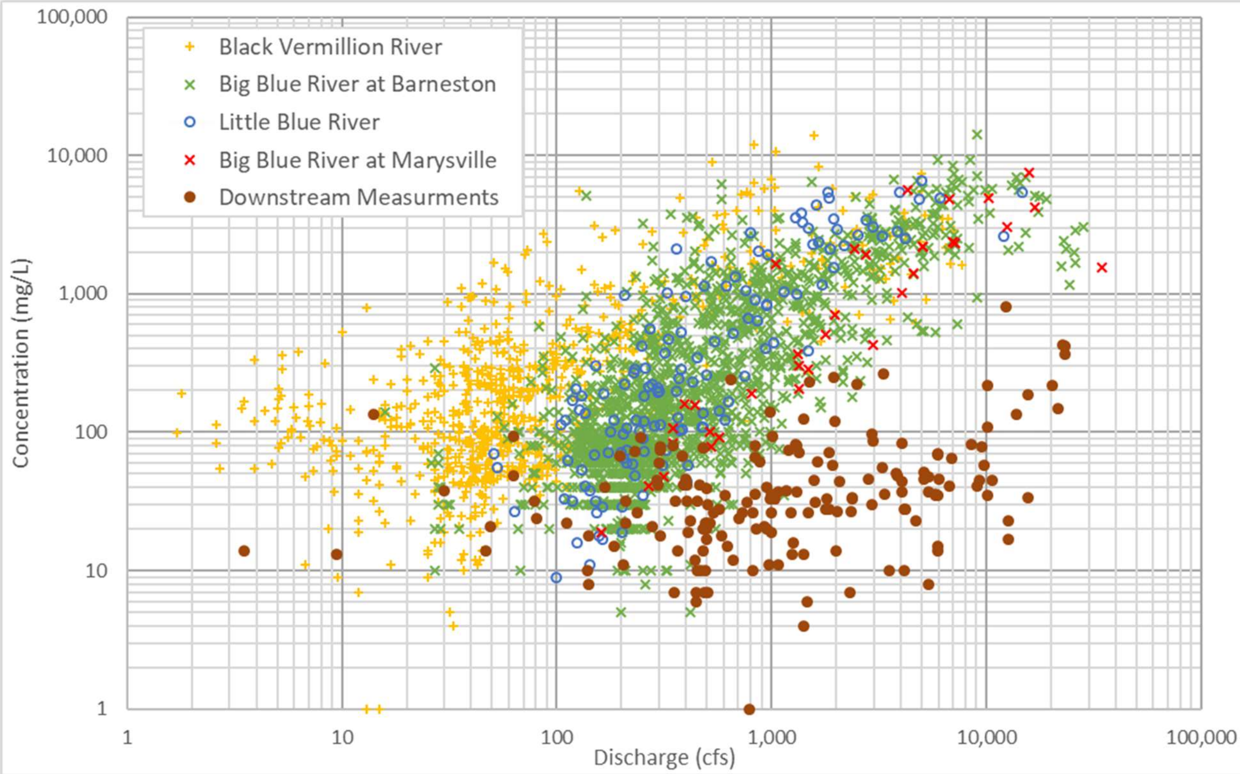


Figure 11-2: Upstream and Downstream Sediment Concentration Measurements and 90% Prediction Intervals

12.0 RESERVOIR BED SEDIMENT COMPOSITION

Measurements of the sediment sizes within the multipurpose pool were collected by the Kansas Biological Survey in 2013. The results are presented in Figure 12-1 and are generally representative of the top six inches of sediment (KBS, 2013). It shows that most of the sediment is clay and that there is a coarsening of the sediment moving upstream. The average of the samples shows that the sediment is 5.9% sand, 16.6% silt, and 77.5% Clay. This is finer than the results from the incoming load calculations, but this could be because the samples were only collected within the multipurpose pool and only represent the surficial deposits.

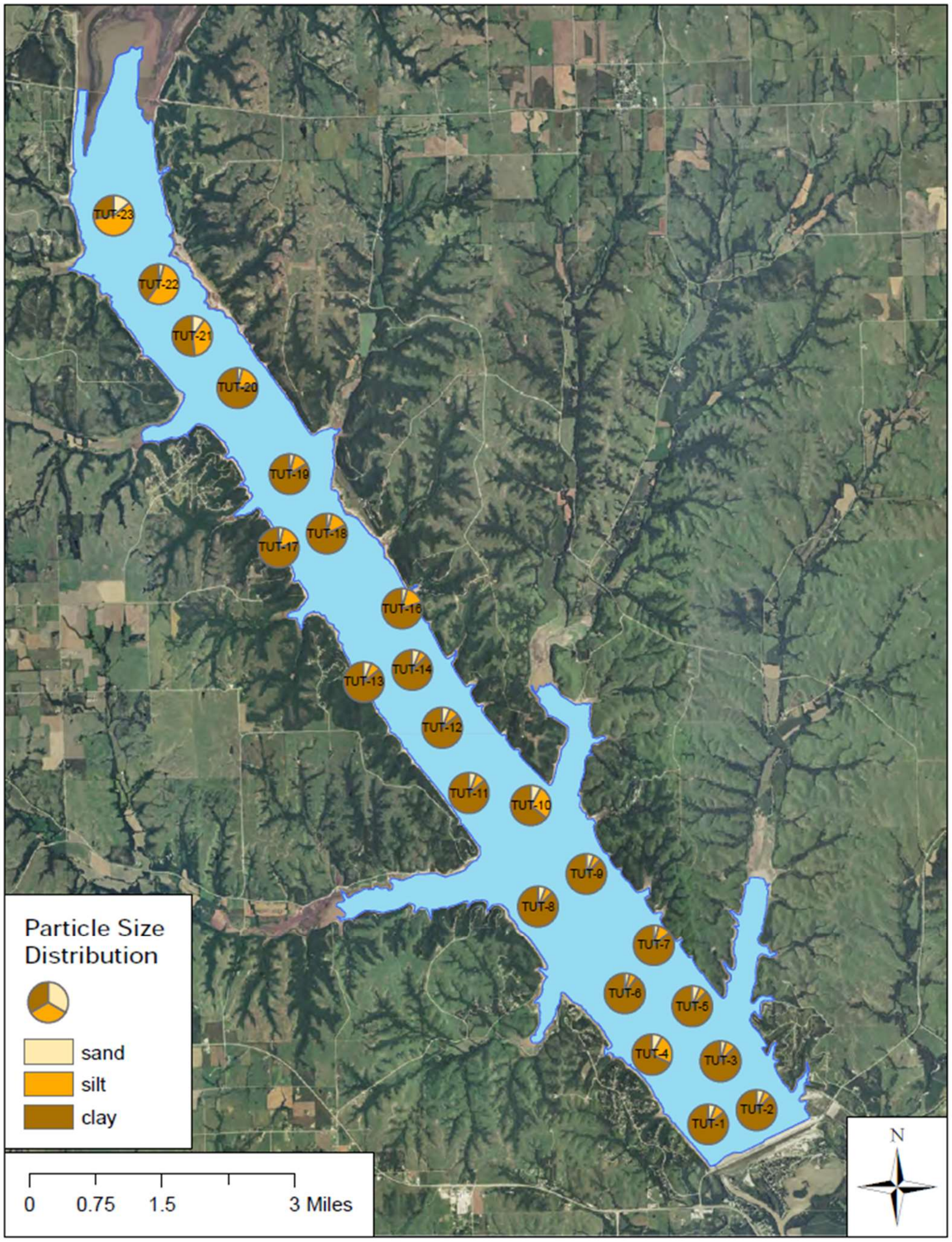


Figure 12-1: Sediment Size Gradation in Multipurpose Pool (KBS, 2013)

13.0 SEDIMENT CHEMICAL CONCENTRATIONS

Chemical characteristics of the deposited sediment were investigated by the USGS in a 2002 report (Juracek & Mau, 2002). The study investigated the concentration of various metals and other chemical compounds in the sediment and compared them to guidelines published by the U.S Environmental Protection Agency (USEPA). The threshold-effect level (TEL) is the concentration below which toxic biological effects seldom occur, while the probable-effect level (PEL) is the level above which toxic effect usually or frequently occur. Between the TEL and PEL toxic effects will occasionally occur. However, these guidelines are screening tools and are not regulatory criteria (Juracek & Mau, 2002). Sediment quality guidelines have been published by the USEPA for nine metals and six organochlorine compounds.

A total of 44 elements were tested by the USGS at 17 coring sites in Tuttle Creek for a total of 41 sediment samples. The minimum, median, and maximum concentrations of chemicals in the samples were published in the report. Of the nine metals with published guidelines by the USEPA, eight of them exceeded the TEL and three of them exceeded the PEL in at least one of the samples. Organochlorine compounds were either not detected or were detected at concentrations below the TEL. Additional information can be found in the USGS report.

14.0 DELTA LOCATION AND VOLUME

Figure 14-1 provides a profile plot of centerline through the lake taken from the sedimentation rangline surveys. At each location, the invert elevation (lowest elevation in a given sedimentation range line) is plotted. The 1962-2000 rangeline surveys were converted from NVGD29 to NAVD88 using values obtained from a conversion raster in Arc-GIS based on the USACE CORPSCON conversion tool. The rangelines for the 2009 survey were pulled from the 2009 combined bathymetry and LiDAR DEM. Because LiDAR cannot penetrate through water and bathymetry data was not collected above the multipurpose pool, the invert elevations are likely overestimated upstream of this for the 2009 survey. From 1983 to 2009 the delta progressed an average of 2.95 miles downstream or 0.11 miles per year.

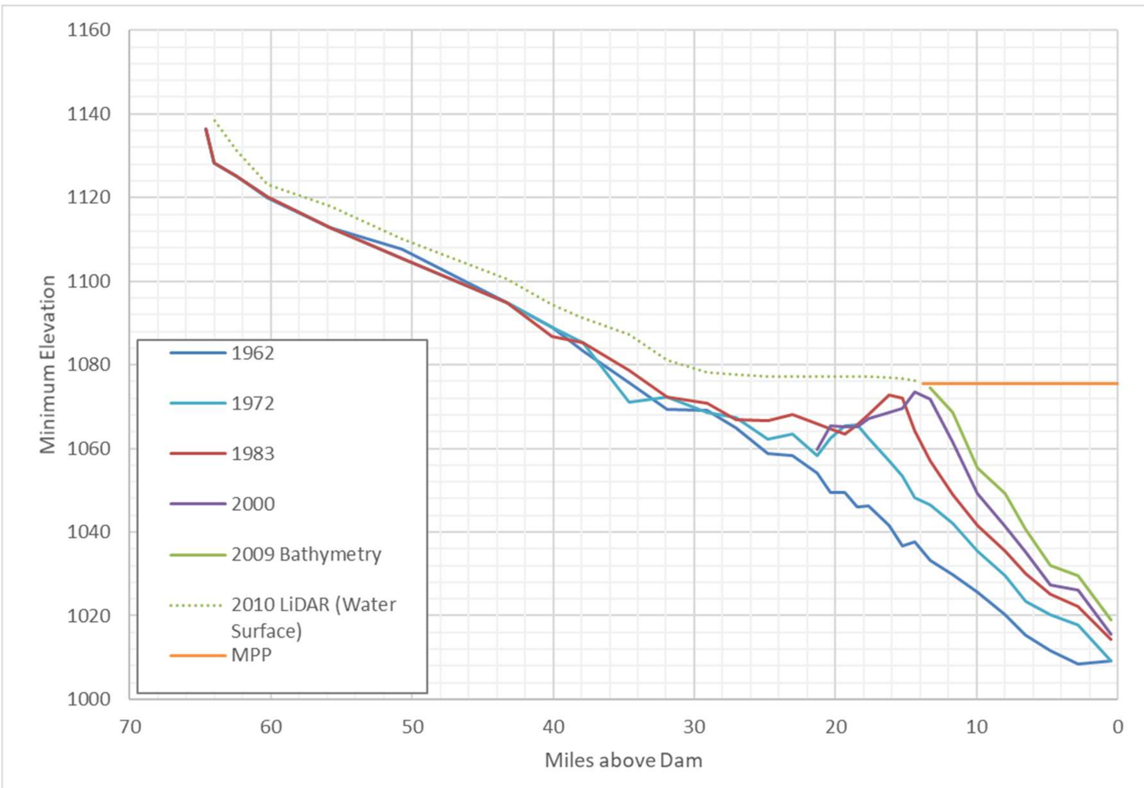


Figure 14-1: Profile of Invert Elevation Indicating Delta Location and Growth

For the 2000 and 2009 surveys, additional data was available to better define the delta crest as shown in Figure 14-2.

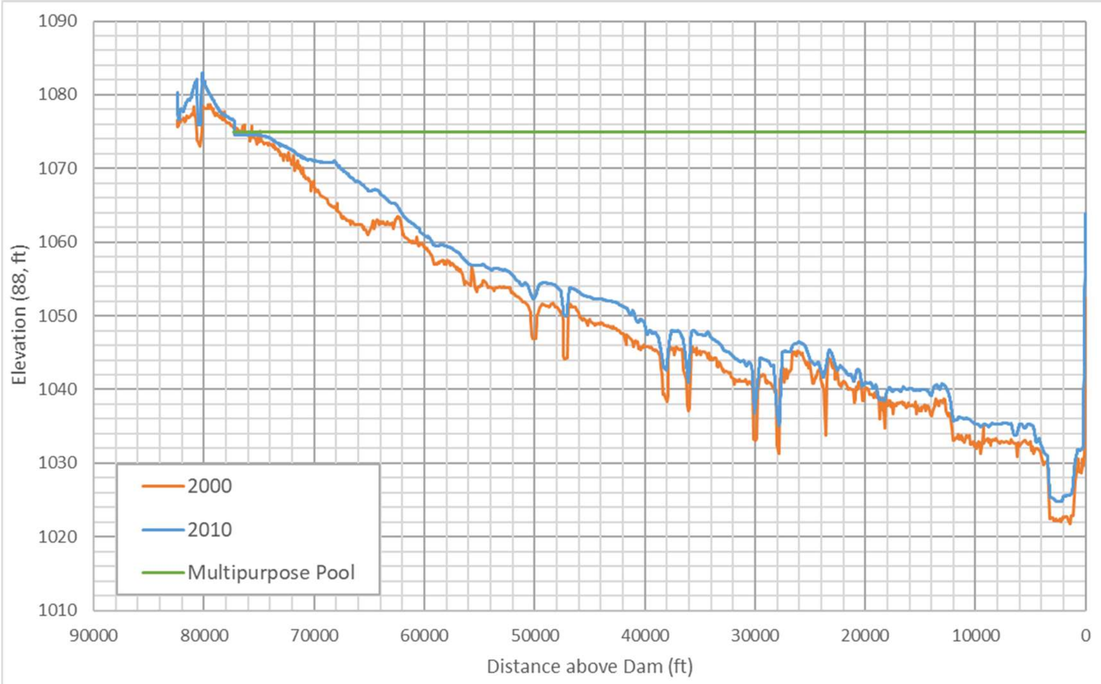
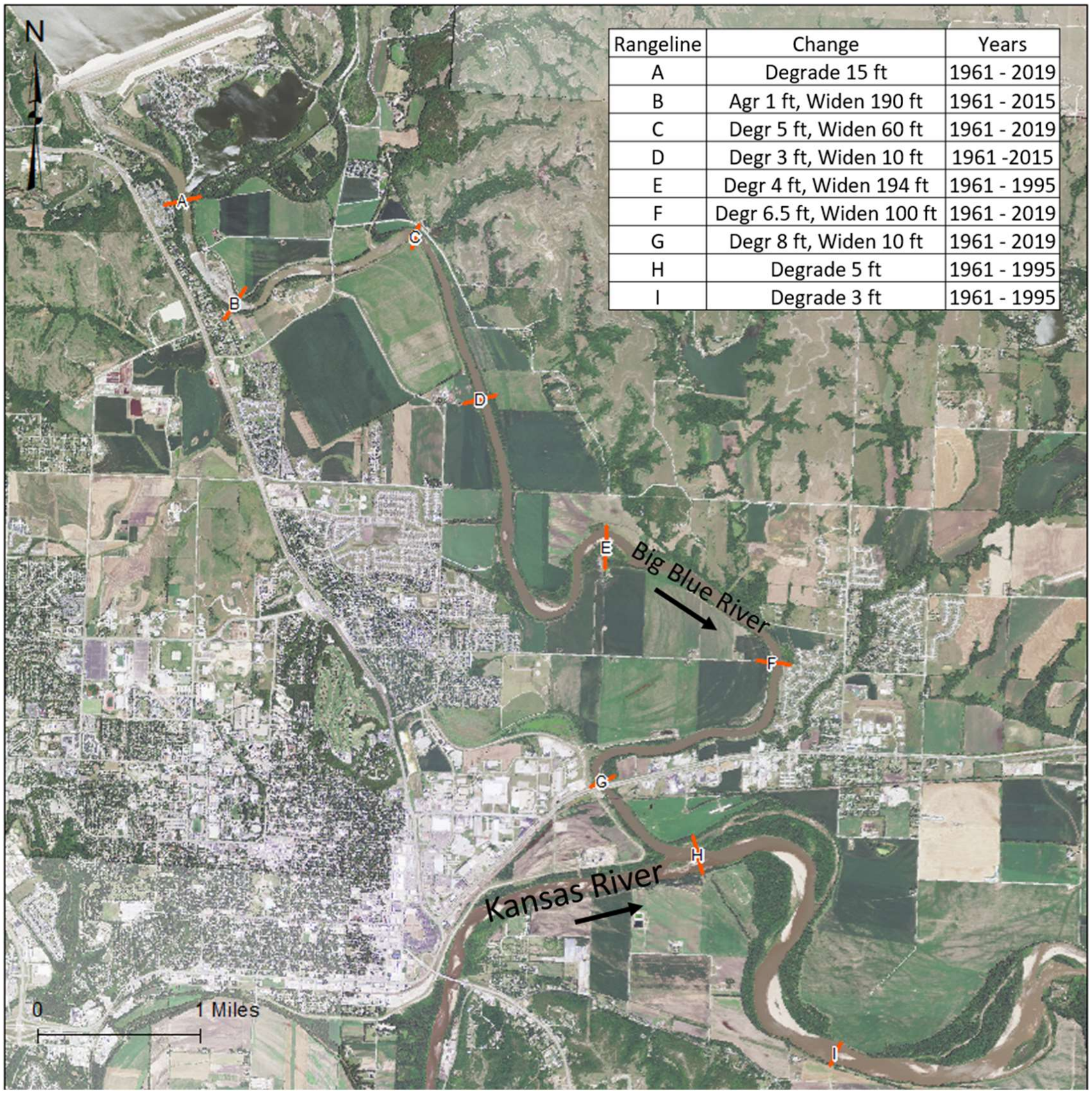


Figure 14-2: Centerline profile of Tuttle Creek Lake for the 2000 and 2009 surveys (NAVD88)

15.0 DOWNSTREAM CHANNEL

Sediment trapping by dams very often induces bed degradation and bank erosion downstream. Degradation rangelines downstream from Tuttle Creek allow this effect to be quantified. Figure 15-1 shows the location of the rangelines, the total bed elevation change, and the channel width change. The 1961-1995 rangeline surveys were converted from NVGD29 to NAVD88 using values obtained from a conversion raster in Arc-GIS based on the USACE CORPSCON conversion tool.



Rangeline	Change	Years
A	Degrade 15 ft	1961 - 2019
B	Agr 1 ft, Widen 190 ft	1961 - 2015
C	Degr 5 ft, Widen 60 ft	1961 - 2019
D	Degr 3 ft, Widen 10 ft	1961 -2015
E	Degr 4 ft, Widen 194 ft	1961 - 1995
F	Degr 6.5 ft, Widen 100 ft	1961 - 2019
G	Degr 8 ft, Widen 10 ft	1961 - 2019
H	Degrade 5 ft	1961 - 1995
I	Degrade 3 ft	1961 - 1995

Figure 15-1: Degradation Downstream of Tuttle Creek

Figure 15-2 plots the cumulative volume change over time based on the degradation rangelines between 1961 and 2019. As seen in Figure 29, the Big Blue River downstream of Tuttle Creek Lake exhibits continued degradation over time. As some of the degradation rangelines are near stabilized bridge locations, this analysis may under-predict the degradation. If these rangelines are indicative of the whole reach from 1961 to 2019, the bed and banks have lost 5,305 ac-ft of material since dam closure. This only includes degradation on the Big Blue River from the dam to its confluence with the Kansas River; approximately nine miles downstream of the dam. Sediment trapping in the reservoir has likely contributed to degradation in the Kansas River as well, but it is difficult to isolate the effects of the dam from other variables affecting the Kansas River. The total degradation on the Big Blue River equates to 29% of the volume of sand deposition within the reservoir, which was estimated to be 18,134 acre-feet.

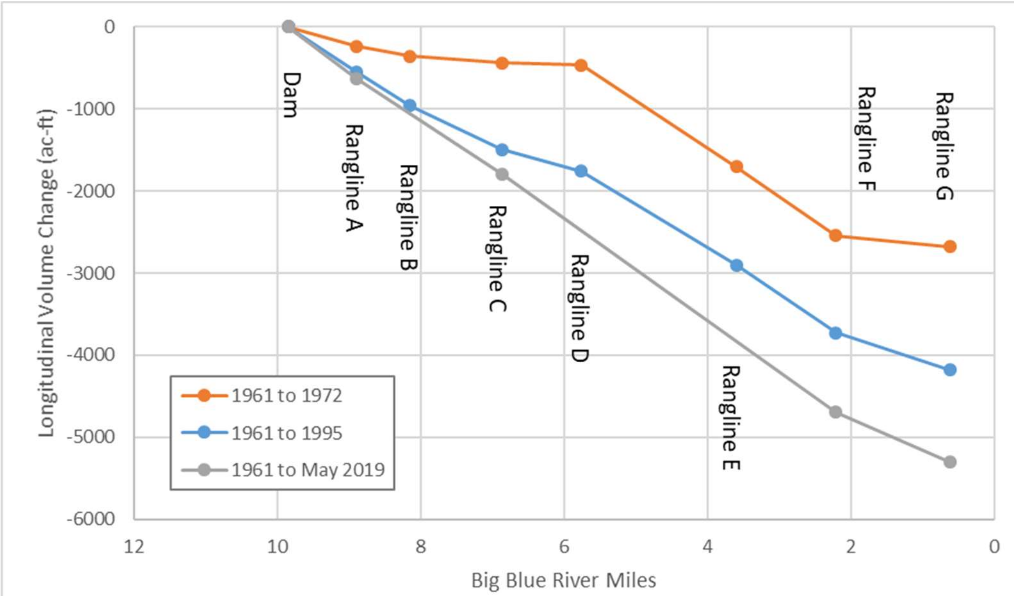


Figure 15-2: Longitudinal Cumulative Volume Change Curves for Degradation Rangelines, 1961 to 2019

These analyses indicate that the Big Blue River downstream of Tuttle Creek is sediment starved. Continued degradation with associated bank erosion is expected if sediment trapping continues.

16.0 SUMMARY AND CONCLUSIONS

Sedimentation has had a significant impact on Tuttle Creek Lake through loss of storage capacity and impacts to infrastructure surrounding the lake. As of June 2009, the multipurpose pool had lost 167,298 ac-ft of storage capacity to sedimentation, or 39.4% of the original volume. Between 1963 and 2000 the flood pool lost 71,970 ac-ft of storage, which is 7.0% of its original volume.

From 2009 to 2010 the USGS estimated the trapping efficiency of the reservoir to be 98.0% based on measured sediment inflows and outflows. Sediment rating curves were created from suspended sediment measurements taken upstream of the lake on its three main tributaries. Bulk density measurements by the USGS indicate a bulk density of 37.4 pcf in the multipurpose pool and 60.9 pcf in the flood pool. The sediment deposition within the reservoir was calculated using the sediment rating curves, bulk density, and trapping efficiency. This was compared to the sediment deposition estimated from the survey data and the upper portion of the rating curves were adjusted to bring them into closer agreement. The final computed deposition values matched well with the surveyed deposition.

Approximately 95% of the incoming sediment enters Tuttle Creek Lake during flood control operations.

A range in the natural sediment concentrations in inflow to Tuttle Creek Lake was estimated from the upstream suspended sediment measurements by fitting 90% prediction intervals to the data. Concentration increases with discharge and peaks at approximately 10,000 cfs. Chemical concentrations within the deposits were measured by the USGS for various metals and other contaminates. Eight of the metals tested exceeded the TEL and three of them exceeded the PEL. Using the sedimentation rangelines, the delta was estimated to have moved towards the dam at a rate of 0.11 miles per year from 1983 to 2009. The estimated degradation on the Big Blue River downstream of the lake was 5,305 ac-ft from 1961 to 2019 based on the degradation rangelines, which is 29% of the sand accumulation in the lake.

17.0 REFERENCES

Brune, G. M. (1953). *Trap Efficiency of Reservoirs.* American Geophisical Union.

Duan, N. (1983). *Smearing estimate: A nonparametric retransormation method.* Journal of the American Statistical Association, 78(383), 605-610.

Juracek, K. E. (2011). *Suspended-Sediment Loads, Reservoir Sediment Trap Efficiency, and Upstream and Downstream Channel stability for Kanopolis and Tuttle Creek Lakes, Kansas, 2008-2010.* U.S. Geological Survey Scientific Investigations Report 2011-5187.

Juracek, K. E., & Mau, D. P. (2002). *Sediment Deposition and Occurrence of Selected Nutrients and other Chemical Constituents in Bottom Sediment, Tuttle Creek Lake, Northeast Kansas, 1962-99.* Lawrence, KS: U.S. Geological Survey.

KBS. (2013). *Sediment Surveys of Milford and Tuttle Creek Reservoirs, Kansas.* Kansas Biological Survey.

Shelley, J. E. (2012). *Geomorphic Equations and Methods for Natural Channel Design.* Lawrence, KS: Doctoral Dissertation, University of Kansas.

Surdex. (2011). *Tuttle Creek Lake, Kansas Lakes LiDAR Mapping, Hydrographic Survey Data Integration & Flood Pool Survey.* Surdex Corporation.

USACE. (1952). *Tuttle Creek Dam & Reservoir, Definite Project Report, Appendix I, Hydrology.* U.S. Army Corps of Engineers, Kansas City District.

USACE. (1973). *Lower Kansas River Basin Lake Regulation Manual in 6 Volumes, Volume No. 2, Tuttle Creek Lake, Kansas.* U.S. Army Corpose of Engineers.

USACE. (2013). *Vertical datum update for Tuttle Creek Lake.* U.S. Army Corps of Engineers, Kansas City District.

USACE. (2015). *Tuttle Creek Dam, Periodic Inspection No. 11.* USACE, Kansas City District, Northwest Division.

Zaiontz, C. (2014). *Confidence/prediction intervals | Real Statistics Using Excel*. Retrieved from real-statistics.com: http://www.real-statistics.com/regression/confidence-and-prediction-intervals/

US Army Corps of Engineers®
Kansas City District

Kansas River Reservoirs Flood and Sediment Study

Appendix D1.4: Milford Lake Existing Condition Sedimentation

November 2022

Contents

1.0 INTRODUCTION

Milford dam is located on the Republican River nine miles above its confluence with the Kansas River. Construction of the dam began in 1962 and was completed in December 1966. The multipurpose pool (MPP) elevation was first reached in July 1967. The authorized purposes of the reservoir include flood control, water supply, low flow supplementation, water quality, navigation supplementation, recreation, and fish and wildlife (USACE 1984). The total drainage area of the Republican River Basin above the dam is 24,882 square miles, although much of it is controlled by other reservoirs. The contributing area above the dam is 17,505 square miles while the uncontrolled area is 3,624 square miles (USACE 2016). Predominant land use in the watershed is agricultural consisting of cropland and grazing. There is one USACE reservoir, Harlan County Lake, and seven USBR reservoirs upstream of the dam (six of which are also above Harlan County Lake). Figure 1-1 shows Milford Reservoir with respect to the overall Kansas River Basin, and Figure 1-2 shows Milford and the Republican River Basin.

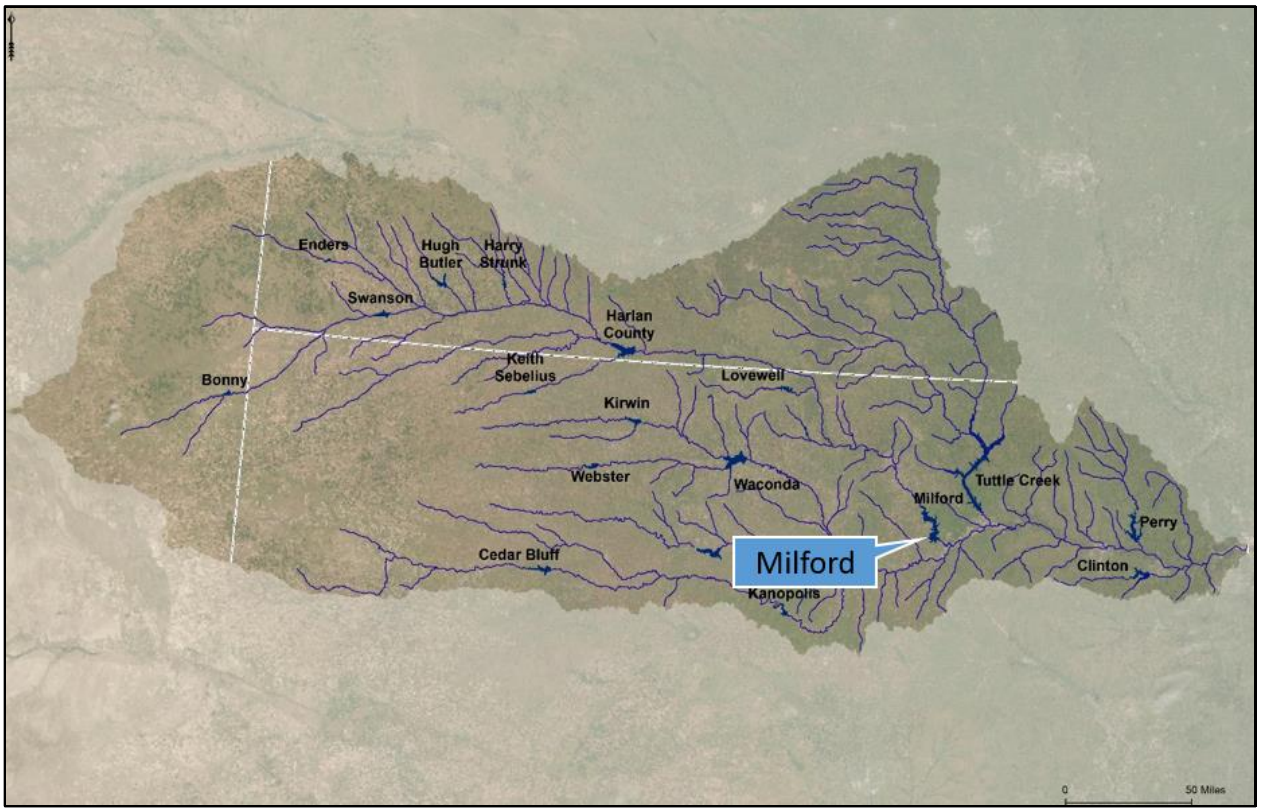


Figure 1-1: Milford Reservoir and the Kansas River Basin.

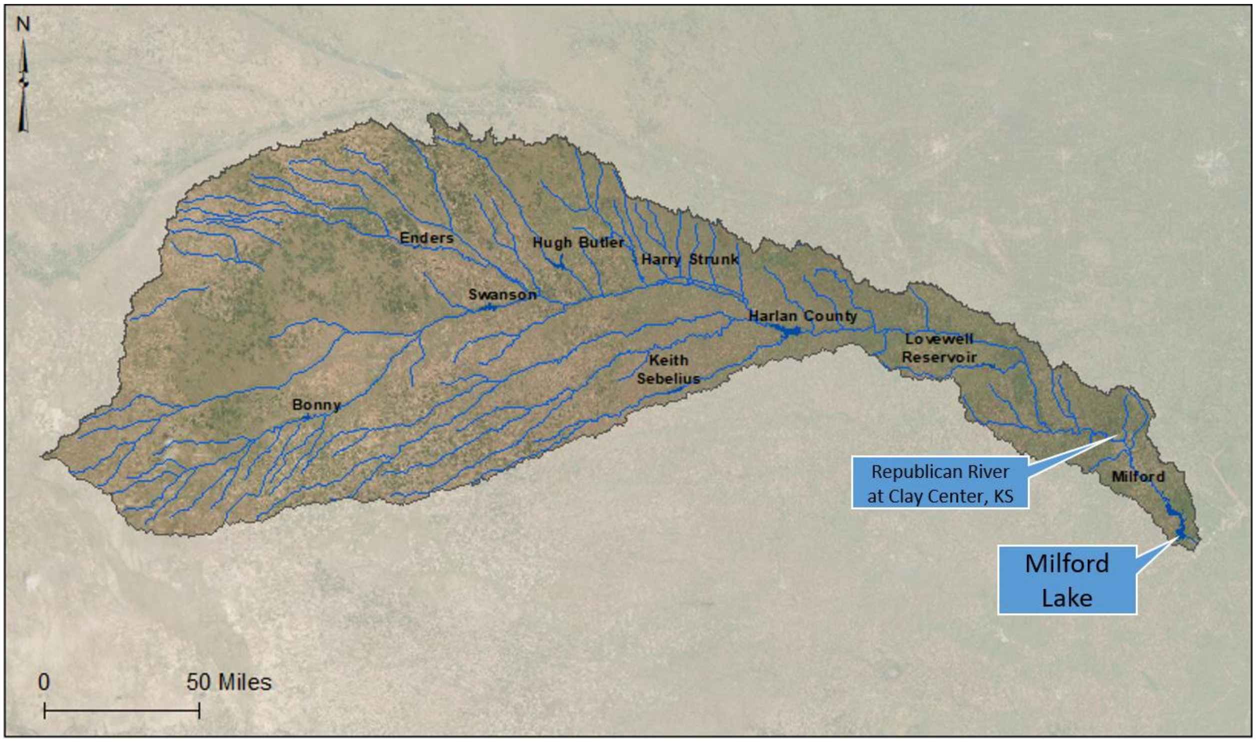


Figure 1-2: Milford Reservoir and the Republican River Basin

2.0 DAM INFRASTRUCTURE

Table 2-1 gives a summary of the dam infrastructure and Figure 2-1 shows the location of each feature. Elevations are shown in both NVGD29 and NAVD88. A survey by the Kansas City District (NWK) in 2013 showed a 0.59-foot difference between NVGD29 and NAVD88 at the lake's gage (USGS gage number 06857050). This is higher than the factor of 0.43 feet that was obtained when using the USACE CORPSCON software. The conversion factor obtained from the 2013 survey was used to convert from NVGD29 to NAVD88 in this section as this is considered the official conversion factor for the lake.

The outlet works consists of two service gates located in an intake tower. Both gates have an invert elevation of 1080.0 feet NVGD29, a 21 ft diameter horseshoe conduit, and a stilling basin. There are also two 2'x2' low flow gates that are used for smaller releases. The dam also has an emergency spillway with an elevation of 1176.2 feet NVGD29.

Table 2-1: Important Information Relating to the Dam Infrastructure NAVD88.

Information type	Elevation in NVGD29 and (NAVD88)
Multipurpose Pool Elevation	1144.4 (1144.99)
Lowest Elevation Outlet	1080.0 (1080.59)
Number of Gates at This Low Elevation	2
Days per Year Low Level Outlet Operated	227
Low Level Outlet Used when Flows Exceed (cfs)	400
Spillway Elevation	1176.2 (1176.79)
Dam Elevation	1213.0 (1213.59)
Typical Tailwater Elevation	1060
Other Pipes Going Through the Dam or Embankment (e.g. Water Intakes, etc.)	None

Note: Elevations in NVGD29 (NAVD 88 in parenthesis).

Figure 2-1: Milford Dam infrastructure

The typical tailwater elevation was estimated from the average discharge from the dam and the tailwater rating curve given in the 2016 Periodic Assessment (Figure 2-2). Figures 2-1 through 2-2 are drawings of the dam infrastructure.

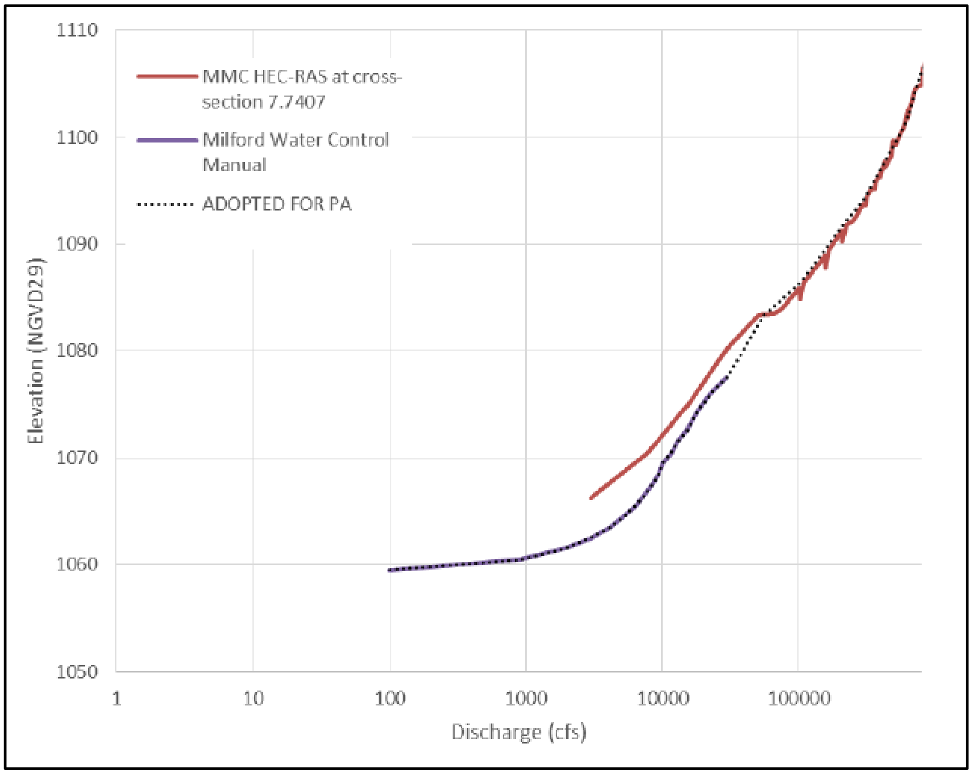


Figure 2-2: Tailwater Rating Curve (USACE, 2016). Elevations in feet NVGD29.

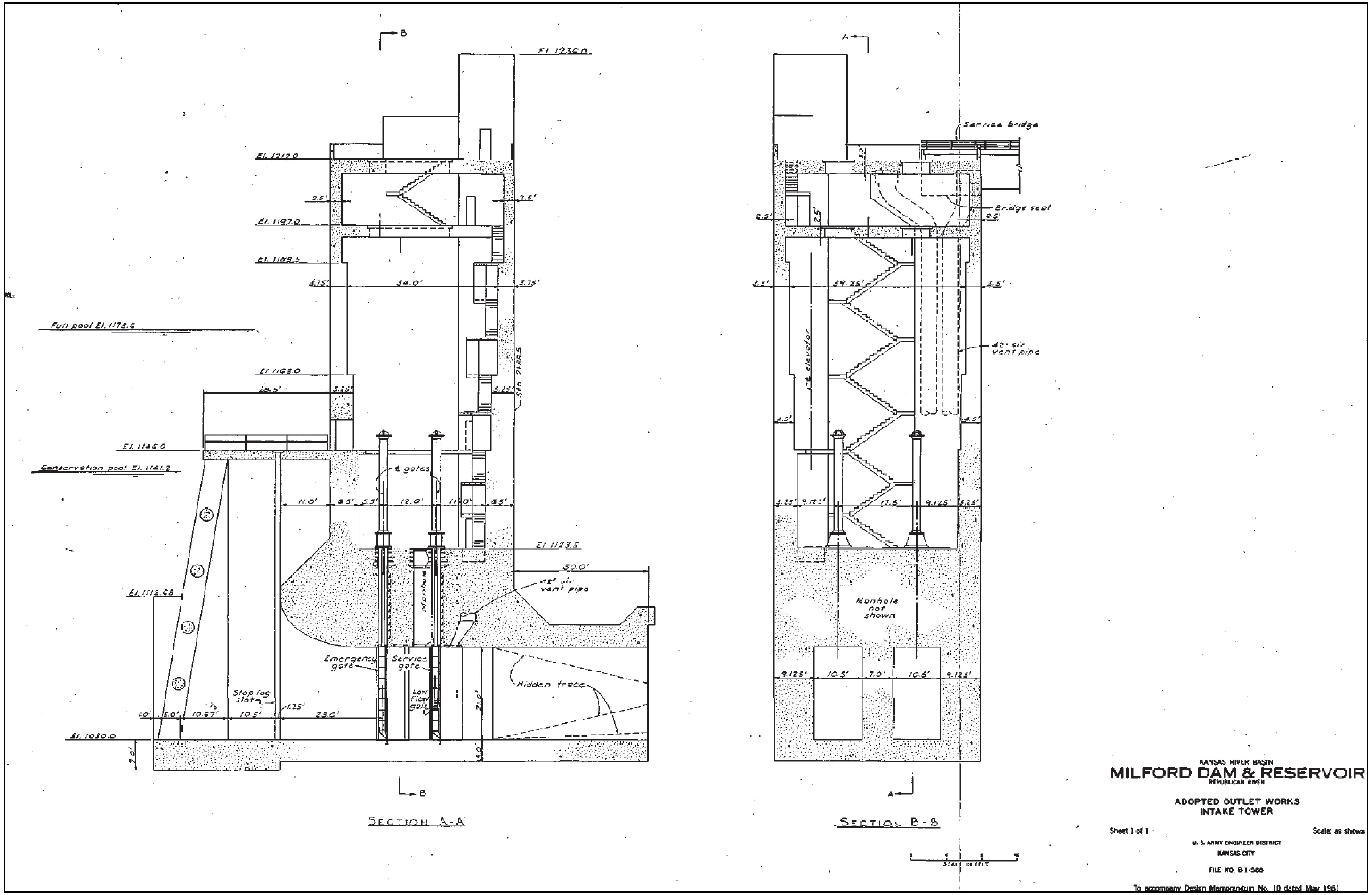


Figure 2-3: Outlet Works Intake Tower (USACE, 1961). Elevations in NVGD29.

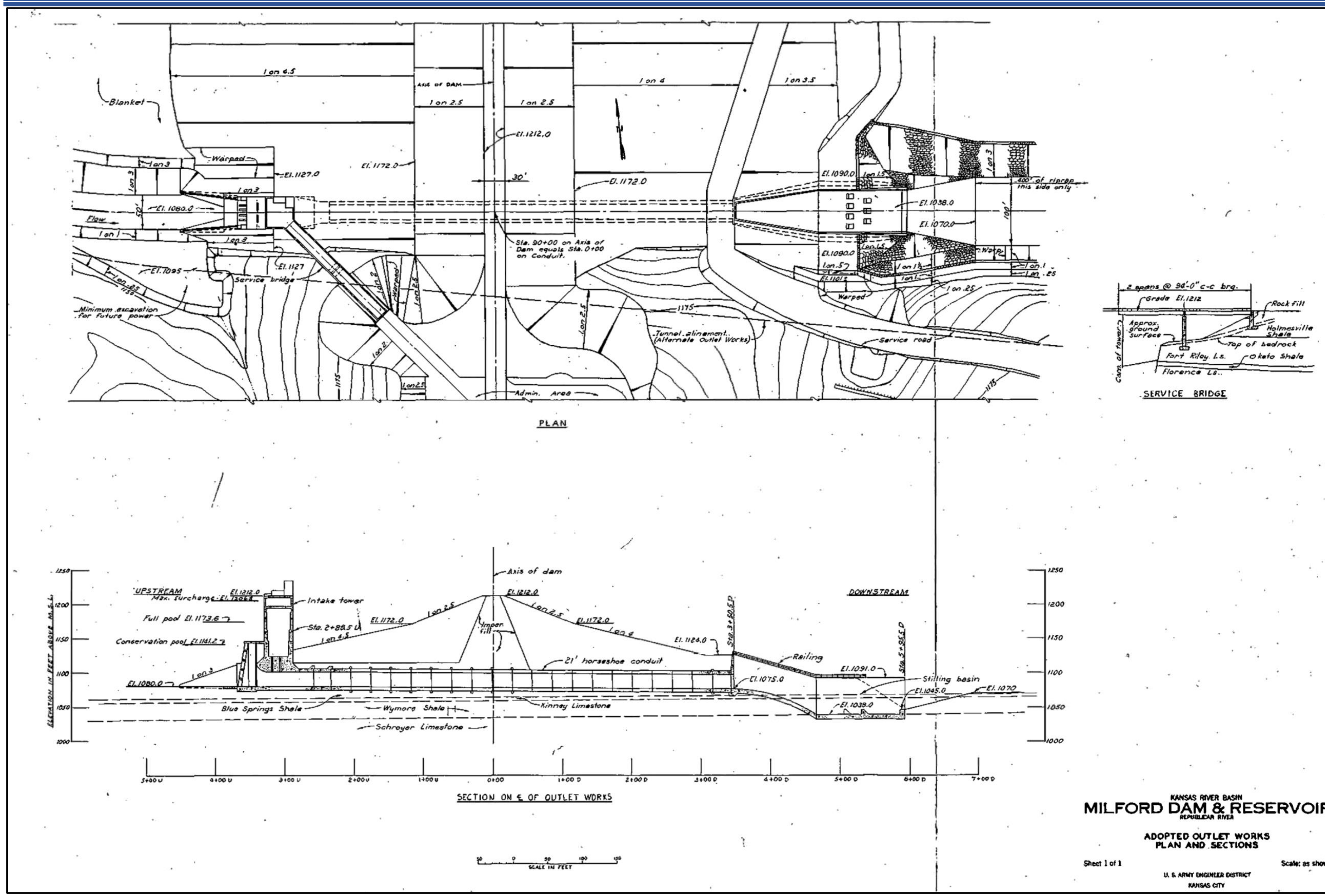


Figure 2-4: Outlet Works Plan and Sections (USACE, 1961). *Elevations in NVGD29.*

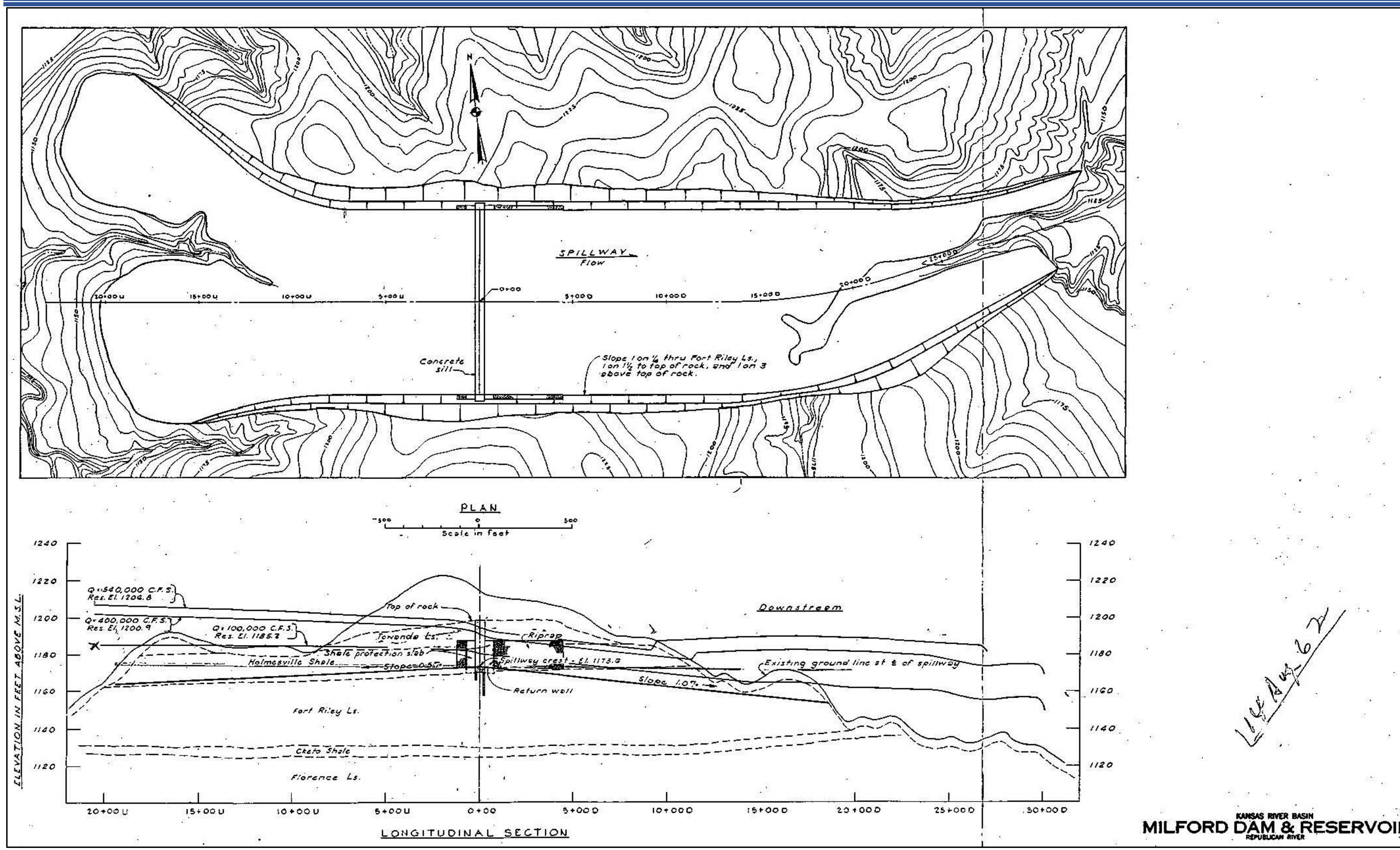


Figure 2-5: Emergency Spillway Plan and Section (USACE, 1961). Elevations in NVGD29.

3.0 SEDIMENTATION EFFECTS ON O&M

Sedimentation has historically not had a significant impact on the operation and maintenance (O&M) at Milford Lake. Sedimentation around the Clay County Park boat ramp has restricted access to the ramp for larger boats, which have had to shift to other access locations on the lake. However, sedimentation O&M issues are expected to increase as the delta progresses downstream in the lake.

4.0 EXISTING CONDITION STORAGE ELEVATION CURVES

The most recent storage elevation curve for Milford Lake was created by Surdex from a combination of LiDAR and bathymetry. Bathymetry was collected by Eisenbraun and Associates in October 2009 at a 250 foot transect spacing using single beam sonar. The LiDAR used in the storage curve was obtained by the USGS, mainly in 2010, though data collected in other years was used in the most upstream portions of the flood pool (FP). The 2010 LiDAR had a grid spacing of 1 meter. Both the LiDAR and bathymetry were combined by Surdex into a digital elevation model (DEM) using Arc-GIS. The Surface Volume in Arc-GIS was then used to calculate the storage elevation tables. Table 4-1shows the final 2009 storage elevation table.

Table 4-1: 2009 Elevation-Volume-Area Table. Elevations are in ft, NVGD29.

Elevation	Volume (ac-ft)	Area (acres)	Elevation	Volume (ac-ft)	Area (acres)	Elevation	Volume (ac-ft)	Area (acres)
1080.0	0	0	1126	157,466	8,733	1171	967,429	29,643
1081	2	9	1127	166,371	9,104	1172	997,460	30,398
1082	20	26	1128	175,694	9,530	1173	1,028,194	31,062
1083	55	46	1129	185,422	9,915	1174	1,059,583	31,722
1084	113	71	1130	195,509	10,242	1175	1,091,651	32,447
1085	202	113	1131	205,887	10,508	1176	1,124,403	33,050
1086	361	222	1132	216,515	10,728	1176.2	1,131,024	33,170
1087	649	354	1133	227,328	10,896	1177	1,157,767	33,697
1088	1,057	472	1134	238,308	11,063	1178	1,191,771	34,314
1089	1,590	584	1135	249,454	11,232	1179	1,226,411	34,977
1090	2,228	700	1136	260,789	11,444	1180	1,261,776	35,922
1091	2,993	836	1137	272,353	11,692	1181	1,298,145	36,766
1092	3,902	986	1138	284,223	12,128	1182	1,335,289	37,528
1093	4,992	1,212	1139	296,648	12,720	1183	1,373,169	38,222
1094	6,337	1,484	1140	309,670	13,319	1184	1,411,734	38,914
1095	7,911	1,640	1141	323,272	13,875	1185	1,451,012	39,694
1096	9,614	1,755	1142	337,390	14,350	1186	1,491,050	40,369
1097	11,418	1,849	1143	351,961	14,792	1187	1,531,743	41,016
1098	13,323	1,966	1144	366,995	15,290	1188	1,573,119	41,725
1099	15,376	2,168	1144.4	373,152	15,498	1189	1,615,200	42,462
1100	17,675	2,426	1145	383,164	17,205	1190	1,658,086	43,423
1101	20,205	2,632	1146	400,484	17,445	1191	1,701,991	44,351
1102	22,939	2,824	1147	418,047	17,705	1192	1,746,767	45,198
1103	25,835	2,967	1148	435,900	18,017	1193	1,792,375	46,022
1104	28,878	3,130	1149	454,130	18,438	1194	1,838,809	46,845
1105	32,124	3,396	1150	472,791	19,020	1195	1,886,093	47,755
1106	35,668	3,676	1151	492,047	19,454	1196	1,934,293	48,611
1107	39,479	3,945	1152	511,688	19,823	1197	1,983,304	49,417
1108	43,562	4,238	1153	531,679	20,157	1198	2,033,131	50,240
1109	47,937	4,504	1154	551,992	20,471	1199	2,083,800	51,108
1110	52,551	4,723	1155	572,626	20,797	1200	2,135,409	52,306
1111	57,392	4,964	1156	593,582	21,115	1201	2,188,465	53,729
1112	62,473	5,194	1157	614,875	21,470	1202	2,242,819	54,976
1113	67,778	5,416	1158	636,534	21,844	1203	2,298,405	56,201
1114	73,284	5,596	1159	658,574	22,240	1204	2,355,204	57,397
1115	78,979	5,794	1160	681,067	22,980	1205	2,413,212	58,672
1116	84,887	6,031	1161	704,485	23,725	1206	2,472,490	59,854
1117	91,027	6,248	1162	728,478	24,262	1207	2,532,890	60,938
1118	97,380	6,450	1163	752,996	24,769	1208	2,594,394	62,085
1119	103,917	6,627	1164	778,003	25,239	1209	2,657,107	63,374
1120	110,651	6,843	1165	803,465	25,687	1210	2,721,274	65,371
1121	117,638	7,152	1166	829,381	26,146	1211	2,787,586	67,111
1122	124,954	7,492	1167	855,766	26,636	1212	2,855,423	68,545
1123	132,620	7,826	1168	882,661	27,158	1213.0	2,924,649	69,905
1124	140,604	8,138	1169	910,103	27,740	1214	2,995,353	71,528

Elevation	Volume (ac-ft)	Area (acres)	Elevation	Volume (ac-ft)	Area (acres)	Elevation	Volume (ac-ft)	Area (acres)
1125	148,881	8,436	1170	938,213	28,700	-	-	-

5.0 TRAPPING EFFICIENCY

In 2009, the multipurpose pool capacity is estimated at 373,152 ac-ft. The mean annual water inflow into the lake is 564,138 acre-feet, based on stream gage data. Brune offers three curves for estimating trapping efficiency, which can be calculated using Equations 1 and 2 and the constants give in Table 5-1 (Brune, 1953). The trapping efficiency was determined to be 96.3 percent in 2009 using the median curve.

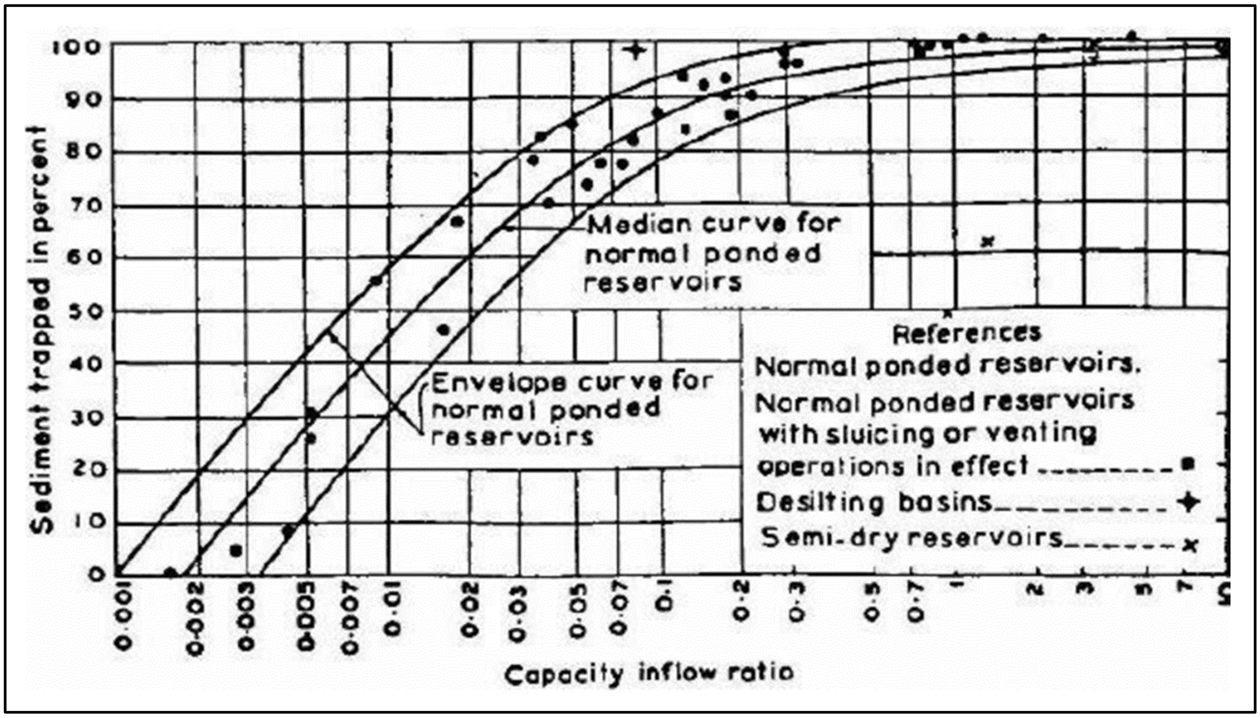


Figure 5-1: Brune Curves

$$TE = a\left[1 - 2e^{-bV_*^{0.35}}\right] \quad (1)$$

$$V_* = \frac{V_{res}}{V_{inflow}} \quad (2)$$

Where:

V_{res} = volume from the multi-purpose pool for Perry Lake (200,004 ac-ft)

V_{inflow} = average of the daily inflow converted into volume

Table 5-1: Constants to be used in Equations 1 and 2.

Constant	Low	Medium	High
a	95	97	100
b	5.37	6.42	7.71

6.0 DEPOSITIONAL VOLUME

Historically, sedimentation rangelines have been used to calculate the capacity of the reservoir and to determine sediment deposition. Milford Lake has a total of seventeen rangelines spaced at varying distances as shown in Figure 6-1. Reservoir capacity was calculated from rangeline surveys in 1967, 1980, and 1994.

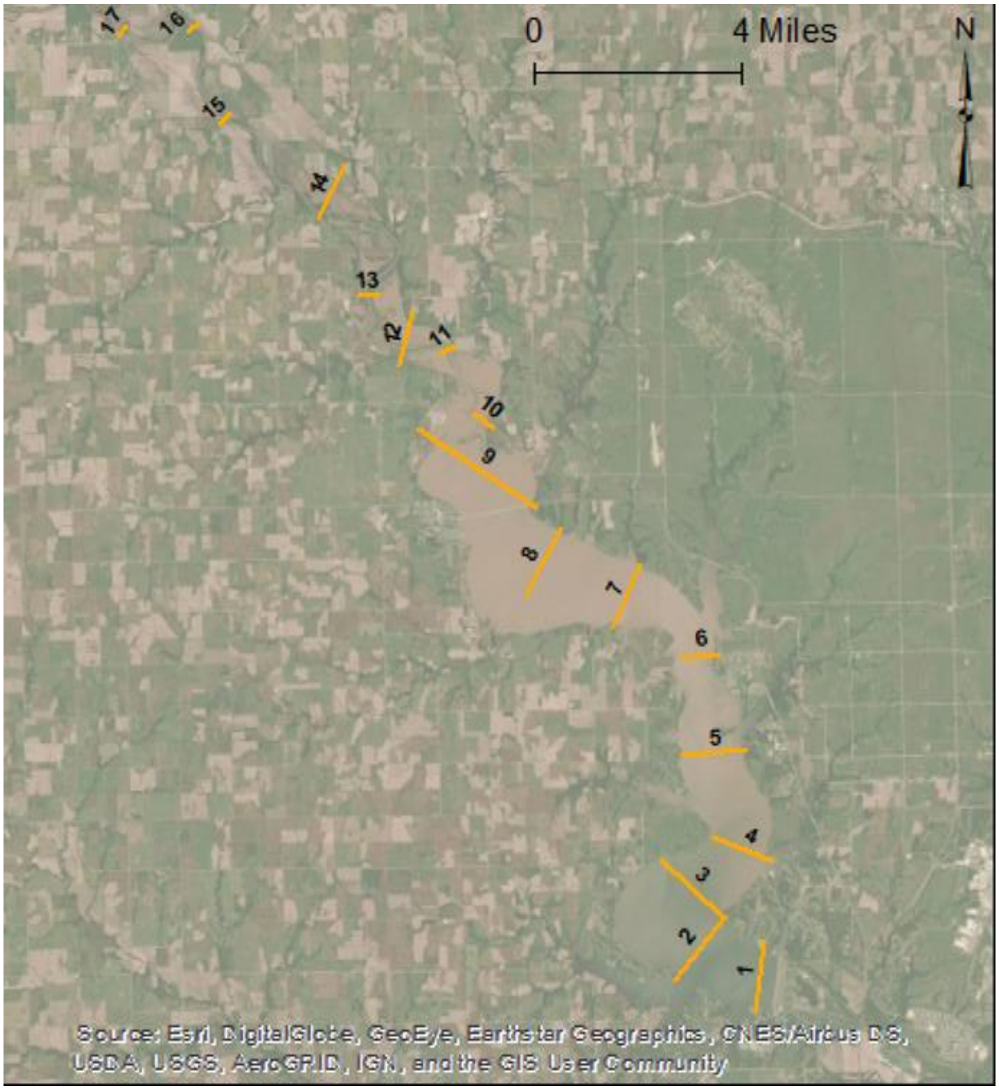


Figure 6-1: Sedimentation Rangelines.

As stated previously, the 2009 bathymetric survey was collected using single-beam sonar along transect lines having a spacing of 250 feet. Areas above the multipurpose pool were not surveyed. Figure 6-2 displays the data points.

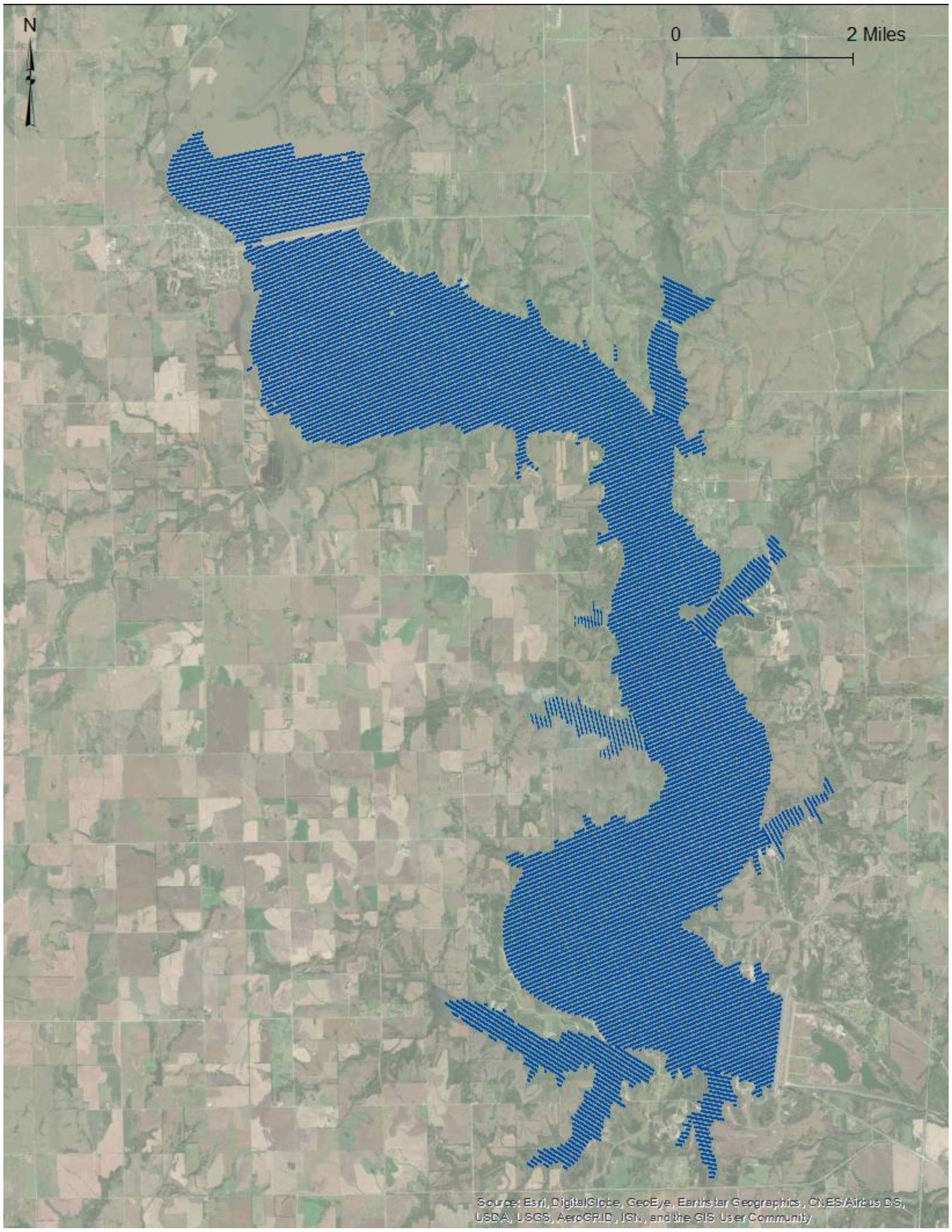


Figure 6-2: Bathymetric Survey Data Points from the 2009 Bathymetric Survey

A portion of the flood control pool was surveyed by the USGS via LIDAR in 2010 with a grid spacing of 1 meter. The USGS National Elevation Dataset (NED) with a grid spacing of 1/3 arc-second was used to fill in the remaining portion of the flood control pool. The LiDAR and bathymetric surveys were combined to create a raster surface having a grid spacing of 12 feet.

Table 6-1 provides the multi-purpose pool volume and flood control pool volume computed from each survey along with the survey methodology. The flood control pool volume does not include the multipurpose pool. The values in this table were taken from the 2011 Surdex report.

Table 6-1: Pool Volumes over Time

Year	MPP	FP	Data Type
1967	415,352	757,746	Sediment Rangelines
1980	388,608	755,601	Sediment Rangelines
1994	372,341	752,822	Sediment Rangelines
2009	373,152	757,872	Single beam sonar, LiDAR

Table 6-2 gives the amount of sediment deposition in the reservoir calculated by subtracting the pool volumes measured from the surveys. FP deposition indicates deposition at elevations higher than the multipurpose pool but lower than the top of flood pool. Table 6-2 shows that there is an apparent increase in the flood pool and multipurpose pool capacity between surveys in 1994 and 2009 which is caused by the change in survey methods. A similar shift was seen at many of the lakes in the Kansas River basin. A second estimation for deposition was developed from the sedimentation rangelines by importing them into the Cross Section Viewer Software. Stationing and elevations were extracted from the 2009 DEM along the rangeline transects using Arc-GIS. This allowed the 2009 survey to be comparable to the older surveys. Volumes computed from Cross Section Viewer for the first two time periods are in fairly close agreement with the older calculations, which lends support for its use in estimating the most recent time period.

Table 6-2: Deposition amounts (ac-ft)

Years	MPP Deposition	MPP Yearly	FP Deposition	FP Yearly	Total Deposition	XS Viewer Total Deposition
1967-1980	26,744	2,057	2,145	165	28,889	26,879
1980-1994	16,267	1,162	2,779	199	19,046	20,290
1994-2009	-811	-54	-5,050	-337	-5,861	14,272

From 1967 to 1994, the multipurpose pool lost 43,011 ac-ft of storage to sedimentation. This represents 10.36% of the original multipurpose pool volume. The average annual rate of loss was 1,558 ac-ft/year or 0.38% of the original volume/year.

From 1967 to 1994, the flood control pool lost 4,924 ac-ft of storage to sedimentation. This represents 0.65% of the original flood control pool volume. The average annual rate of loss was 178 ac-ft/year or 0.02% of the original volume/year.

Based on the Cross Section Viewer estimates, annual deposition from 1994 to 2009 was lower than for the previous two time periods. Total deposition was estimated to be 14,272 ac-ft, or 938 ac-ft per year.

7.0 INCOMING SEDIMENT LOADS

USACE and USGS have sporadically collected paired flow/sediment concentration measurements on the Republican River at Clay Center, Kansas (USGS gage number 0685660) since 1949. The measurements were converted into daily sediment loads in tons per day and were plotted versus discharge for different time periods. Power fit trendlines and equations were then added to the plots as shown in Figure 7-1. It appears that loads have decreased over time. The 1949 to 1959 data show the highest sediment loads, with progressively decreasing loads for each time period since. More recent measurements appear to show that since 1990 the decrease in load has been mainly for higher discharges, while loads at lower discharges have stayed fairly constant. Because the rating curve decreases over time, separate rating curves were created for the different time periods to better estimate sediment inflow into the lake. The decrease in sediment loads could be caused by factors such as land management changes and bank stabilization.

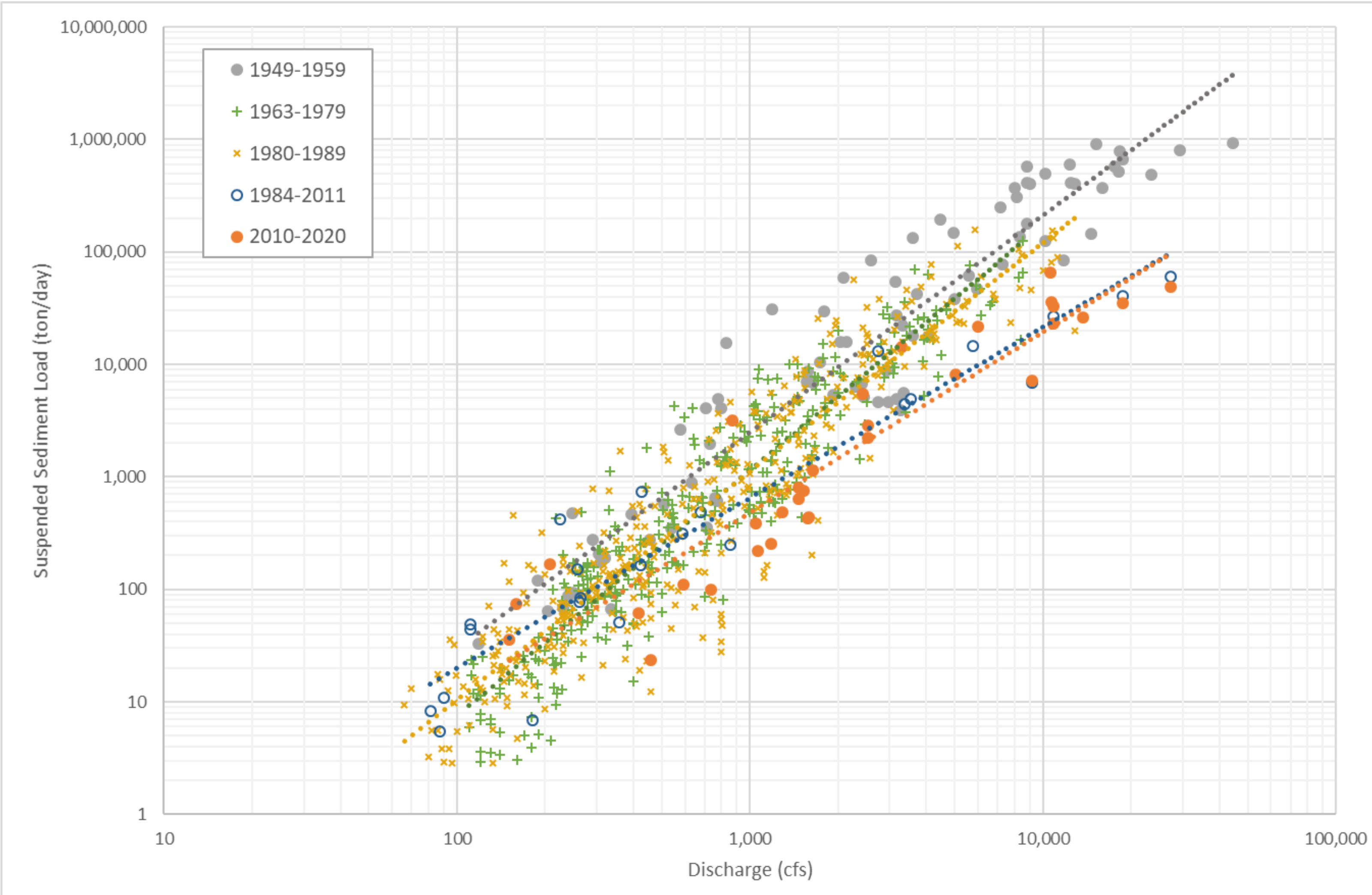


Figure 7-1: Suspended Sediment Load Measurements at the Republican River at Clay Center, KS

Measurements from various gages within the Kansas River watershed indicate that the sediment rating curves generally begin flattening out at around the 83.3 to 66.7% (1/1.2 to 1/1.5) annual exceedance probability (AEP) discharge. Meaning the rating curve no longer follows the power fit trendlines when the discharge exceeds these values. These flows correspond with the typical range for bankfull flow in Kansas (Shelley 2012). The 83.3% and 66.7% AEP flow rates for the Republican River at Clay Center are 6,070 and 8,510 cfs respectively. It was assumed that the flatted portion of the rating curve begins at the 66.7% AEP event. Only the data before the rating curves begins flattening was used to create the power fit trendlines. The new slope at the higher end of the rating curve was determined through calibration, as explained later in this appendix, and was kept constant amongst the time periods. Duan's (1983) correction for bias introduced by the log transform was applied to the portion of the rating curve that follows the power fit trendline.

Table 7-1 shows the coefficients for the power fit trendline portion of the rating curves.

Table 7-1: Rating curve parameters.

Time Period	a	b	Duan E
1963-1979	0.00032	2.18	1.47
1980-1989	0.00079	2.05	1.56
1990-2011	0.013	1.58	1.49
2010-2020	0.0054	1.65	1.58

No site-specific bedload measurements were available. Historic bedload calculations from nearby Kanopolis Lake estimated the bedload to be 15% of the suspended load on the Smoky Hill River at Ellsworth, Kansas. This value was assumed to be the percentage of bedload for Milford Lake as well.

The USGS also periodically measures the suspended sediment gradations of the samples they collect. These were plotted versus the discharge when they were collected to detect trends in the sediment size with discharge. Only measurements with the full gradation measured were used in the sediment size analyses.

Gradations of the suspended sediment indicates that there is some correlation between sediment gradation and discharge. Linear trendlines were fitted to the data as shown in Figure 7-2. The percentage of clay declines as flow increases while the percentage of sand increases. Silts increase slightly with increasing flow. However, there is significant scatter in the data and fewer measurements at the highest discharges, so there is significant uncertainty in these results. Because there are few measurements taken above 25,000 cfs, it was assumed that the sediment gradation remains constant for discharges higher than 25,000 cfs.

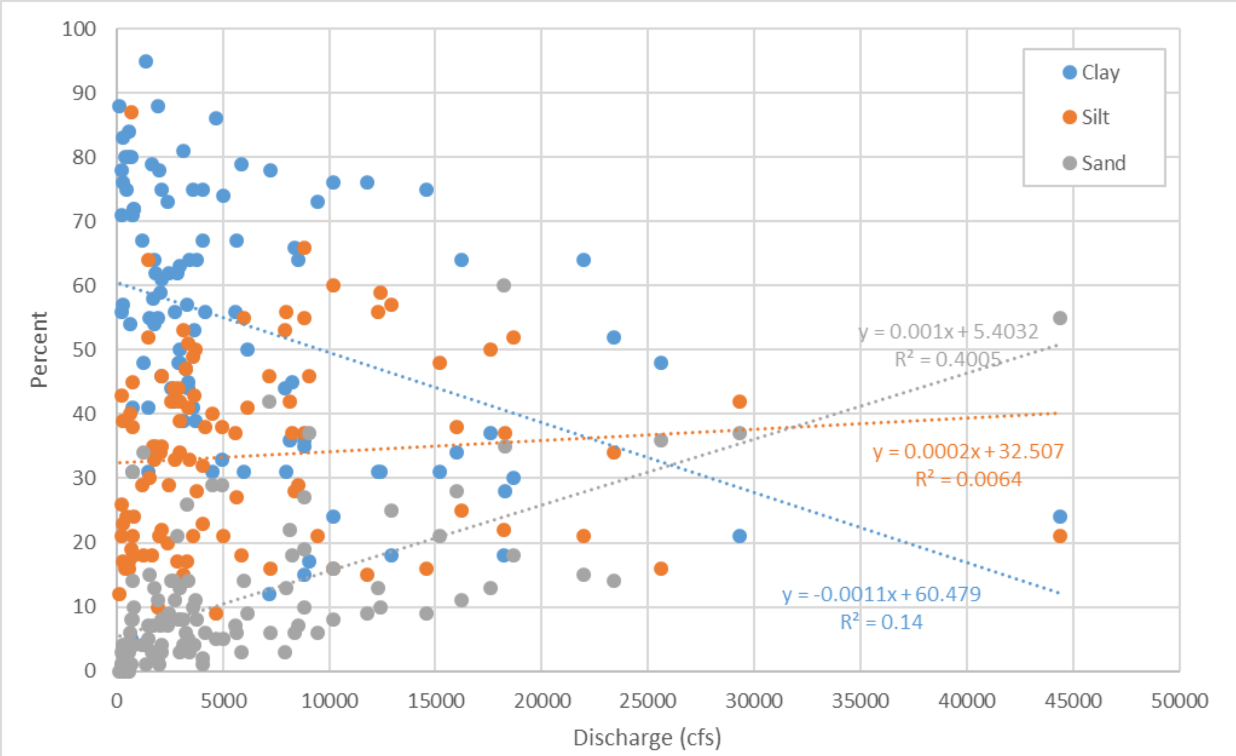


Figure 7-2: Suspended Sediment Gradation vs Discharge

A total load rating curve was developed through the following steps:

1. Estimate the best fit regression line of the form $Q_s = aQ^b$ using log-log linear regression.
2. Correct for bias using the Duan correction factor (Duan 1983).
3. Add 15% to account for bed load to create a total load rating curve.
4. Multiply by 1.1 to account for the ungauged drainage area above the dam.
5. Using the measured data, estimate the percentages of clay, silt, and sand/gravel.
6. Apply this rating curve to daily flow rates from 1964 to 2019 to determine the cumulative mass of sediment entering the reservoir.
7. Multiply by the appropriate percentages to determine the cumulative mass of clay, silt, and sand entering the lake.

This analysis found the incoming load to be 45.5% clay, 29.9% silt, and 24.6% sand/gravel. Table 7-2 summarizes the results for 1964 to 2019.

Table 7-2: Preliminary Incoming Sediment to Milford from 1964 to 2019.

Parameter	Value
Total Incoming Sediment (tons)	66,232,884
Total Incoming Clay Fraction	45.5 %
Total Incoming Silt Fraction	29.9 %
Total Incoming Sand Fraction	24.6 %

8.0 BULK DENSITY AND CONSOLIDATION OF SEDIMENT DEPOSITS

Historical bulk density measurements from Milford Lake were found in file storage at NWK and are shown in Table 8-1. The average bulk density of the deposited sediment within the multipurpose pool was estimated to be 27.2 pcf from these measurements. However, this is significantly lower than has been estimated for other lakes within the Kansas River basin, and seems unlikely for Milford Lake, considering the high percentage of sand and silt that is depositing in the reservoir.

Table 8-1: Bulk Density Measurements at Milford Lake.

Sample #	Sediment Rangeline	Depth (inch)	Dry Density (pcf)
1	1	18.5	23.83
2	1	18.5	29.97
3	2	18.5	20.94
4	2	12.62	24.31
5	2	18.5	21.13
6	3	11.75	20.52
7	3	13.38	19.7
8	4	18.25	25.3
9	5	17.5	49.16
10	5	9.75	27.82
11	5	11.88	18.91
12	5	17.5	20.36
13	6	12.88	42.88
14	6	18.25	31.65
15	6	18.5	23.39
16	7	18.75	41.39
17	7	12.38	31.33
18	7	18.5	21.9
19	8	17.5	26.34
20	8	17	24.05
21	9	18.38	28.59
22	9	18	33.82
23	9	18.62	35.45
24	9	15.25	29.32
25	9	17.38	33.75
26	10	17.75	38.78
27	10	2.5	82.65
28	11	12	32.95
29	11	5	152.15
30	11	7.75	84.06
31	11	6.38	88.64

The incoming load can also provide an estimate of bulk density, via Equation 1.

$$\boldsymbol{\gamma_c} = \frac{\mathbf{1.0}}{\left(\left(\frac{F}{\gamma}\right)_{clay} + \left(\frac{F}{\gamma}\right)_{silt} + \left(\frac{F}{\gamma}\right)_{sand}\right)} \tag{1}$$

Where:

γ_c is the composite bulk density

F is the fraction of clay, silt, or sand

γ for clay, silt and sand is assumed to be 30 pcf, 65 pcf, and 93 pcf respectively.

Three methods were used to estimate the gradation of the sediment within the lake. The first was to use the sediment gradations from the incoming sediment given in the previous section. Also, the Kansas Biological Survey collected surficial and core samples of the sediment within the multipurpose pool, which are given in Section 13 of this report. Each of these methods was used to calculate the bulk density as shown in Table 8-2. Bulk density from the KBS samples would be representative of the multipurpose pool bulk density, while the stream gage data would be representative of the overall bulk density including the flood pool deposits.

Table 8-2: Bulk densities calculated from the gradation of deposited sediment.

Source	Clay %	Silt %	Sand %	Bulk Density
Stream Gage	45.8	29.8	24.5	44.5
KBS Core Samples	54.6	26.8	18.7	41.1
KBS Surficial Samples	45.2	38.1	16.8	44.0

These measurements do not match well with the results from Table 8-1. As there was some uncertainty in the measurements from Table 8-1, and because it matched better with the surveyed deposition, the bulk density calculated from the size gradation of the core samples was used for the sedimentation calculations.

The bulk density of the flood pool deposits was estimated using the following equations.

$$\gamma_C V_c = \gamma_{mp} V_{mp} + \gamma_{fp} V_{fp}$$
$$V_{fp} = 0.103 V_c$$
$$V_{mp} = 0.897 V_c$$
$$\gamma_{fp} = \frac{\gamma_c V_c - \gamma_{mp} V_{mp}}{V_{fp}} = \frac{\gamma_c V_c - \gamma_{mp} 0.897 V_c}{0.103 V_c} = \frac{\gamma_c - 0.897\gamma_{mp}}{0.103}$$

Where:

γ_{fp} = Flood pool bulk density

γ_{mp} = Multipurpose pool bulk density

γ_c = Composite bulk density

V_c = Flood pool + multipurpose volume

V_{mp} = Volume of the multipurpose pool

V_{fp} = Volume of the Flood Pool

The resulting flood control pool bulk density is 72.6 pcf. This is comparable to the flood pool bulk density of 75.1, which was determined from physical samples taken at Kanopolis Lake.

9.0 CALIBRATING LOAD AND DEPOSITIONAL PARAMETERS

Sufficient data exists to calibrate the rating curve by comparing the deposition computed using the incoming sediment loads to the deposition computed using the surveyed volumes. This was accomplished following these steps:

1. Determine the trapping efficiency of the reservoir (in this case 96.3%).
2. Apply daily flows at the Clay Center gage to its sediment rating curve.
3. Apportion the deposition into the multipurpose pool or the flood control pool. Multiply the deposition found in Step 2 by a factor, *m* such that the ratio of MPP deposition to total deposition is correct per the survey analysis. *m* was computed from the surveyed deposition and the bulk densities determined for the MPP and FP.
4. Repeat steps 2 through 3 for each day over a period of time to obtain the cumulative sediment inflow.
5. Compute the mass of trapped sediment in the multipurpose pool by applying the trapping efficiency to the incoming sediment.
6. Transform the trapped mass to a deposited volume by using the bulk densities determined in Section 9 (41.1 pcf in the MPP and 72.6 pcf in the FP.)
7. Compare the total rating-curve-based deposition to the deposition calculated from the surveyed volumes.
8. Adjust the sediment rating curve (described below) to match the surveyed deposition more closely.

Table 9-1 summarizes this analysis.

Table 9-1: Parameters used to Calculate Sediment Deposition.

Parameter	Value
FP Bulk Density	72.6
MPP Bulk Density	41.1
Bedload % of Suspended	15%
Correction for Ungauged Area	1.1
a in $Q_s = aQ^b$	See Table 7-1
b in $Q_s = aQ^b$	See Table 7-1
Average trapping efficiency	96.3%

The flattened portion of the rating curve at higher discharges was chosen as the portion of the rating curve to be adjusted for calibration because it is generally the portion of the rating curve that has the most uncertainty due to lack of measurements. Figure 9-1 shows the final total load rating curve along with the suspended sediment measurements taken at the Clay Center gage for the different time periods. As mentioned earlier, the rating curve begins flattening at the 66.7% (1/1.5) AEP flow rate. The same slope was used for all four rating curves above this point.

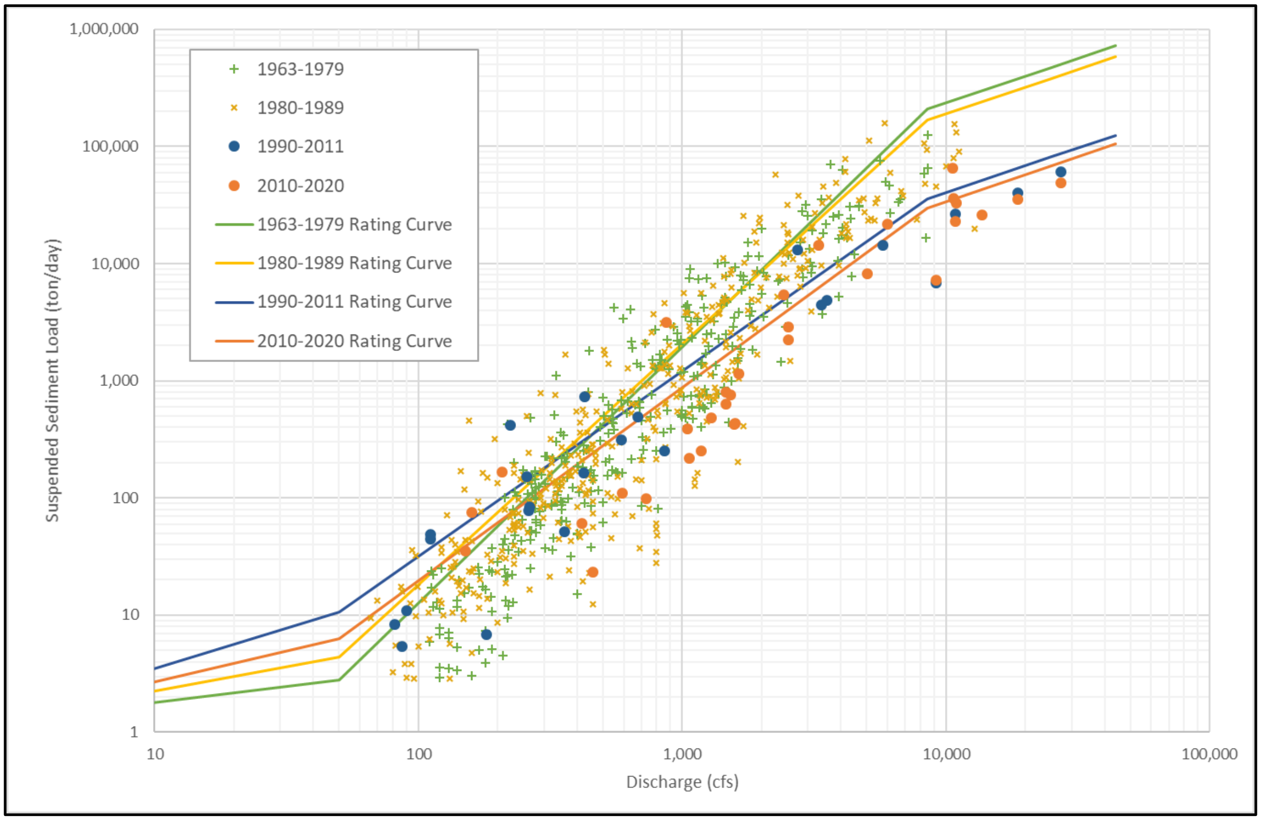


Figure 9-1: Total load rating curves for the Republican River at Clay Center along with suspended sediment measurements.

Table 9-2 summarizes the results of the calculations for deposition between each of the surveys. The flattened portion of the rating curve was adjusted so that the computed deposition matched the total surveyed deposition. The computed deposition does not match as well with the 1980-1994 or 1994-2009 time periods. However, the 1980 to 2009 deposition matches very well, which indicates that there could be errors in the 1994 survey.

Table 9-2: Surveyed and Calculated Reservoir Deposition.

Time Period	Surveyed Deposition	Computed	Computed / Surveyed	# of Day > 8510 cfs	Days per year
1967-1980	28,889	30,628	1.06	35	2.7
1980-1994	19,046	26,329	1.38	58	4.0
1994-2009	14,272*	5,236	0.37	17	1.1
1980-2009	33,287*	31,565	0.95	75	2.5
Total (1967-2009)	62,176*	62,193	1.00	110	2.6

Table Note: *1994-2009 deposition calculated using Cross Section Viewer Software from the 2009 survey resampled at the sedimentation range lines.

10.0 SEDIMENT TRAPPING DURING FLOOD CONTROL VS. MULTIPURPOSE POOL OPERATIONS

The calculated sediment inflows into Milford Lake were used to estimate the deposition based on when the lake was in flood control operations (i.e., with a water surface above the multipurpose pool elevation), vs. multipurpose pool operations (i.e., when the water surface is at or below the multipurpose pool elevation). Table 10-1 shows the results of this analysis which indicates that most of the deposition occurs during flood control operations.

Table 10-1: Deposition Amounts from sediment entering during Flood Control and Multipurpose Operations

	Deposition 1967 - 2019 (ac-ft)	% of Total
Total Deposition	65,680	100.0 %
Multipurpose Operations	6,460	9.8%
Flood control Operations	59,220	90.2 %

Lake levels vary throughout the year depending on a variety of factors. However, lake managers generally try to operate the lake so that pool elevations match the Water Level Management Plan (WLMP) shown in Figure 10-1. The quantity of deposition occurring when the pool elevation is above or below the WLMP line shown in Figure 10-1 is given in Table 10-2.

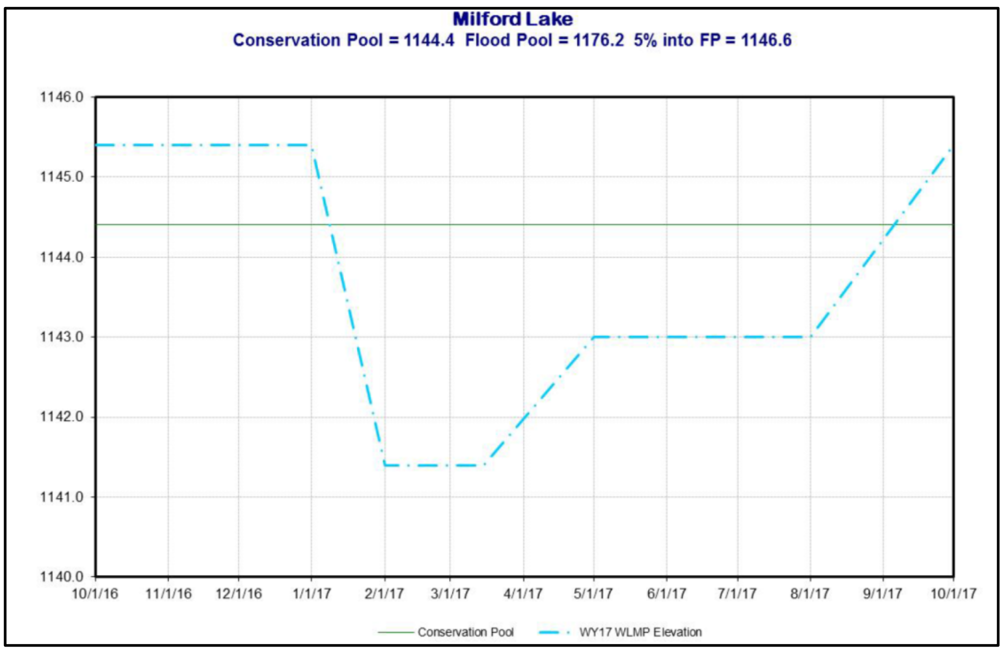


Figure 10-1: Milford Lake Water Level Management Plan (USACE, 1984)

Table 10-2: Deposition Amounts when Lake Elevation is Above or Below the WLMP.

Total Deposition	Deposition 1967 – 2019 (ac-ft)	% Total
Total	65,680	100 %
Below WLMP	2,997	4.6 %
Above WLMP	62,683	95.4 %

11.0 SEDIMENT CONCENTRATIONS

Figure 11-1 indicates the relationship between flow and sediment concentration over time for the Republican River at Clay Center, Kansas. It can be seen that the sediment concentration for the higher discharges has decreased over time.

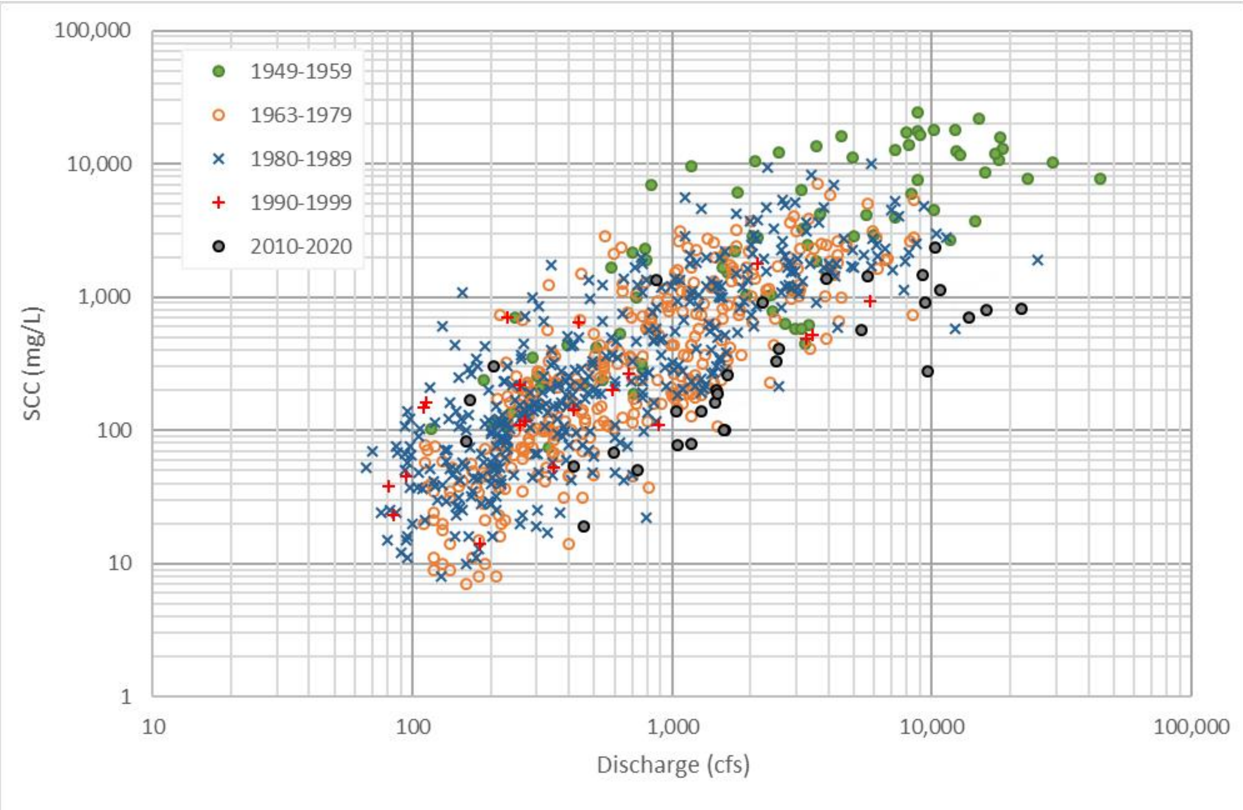


Figure 11-1: Sediment concentration over time

Because the sediment load has trended down over time, the measurements had to be adjusted to account for this decrease. The measurements were adjusted using the four sediment rating curves already discussed. Figure 11-2 illustrates the concentration of incoming sediment, together with the 80% confidence intervals. This graph represents the range of natural variability in the sediment concentrations in the river, i.e., what the concentration would be if the dam were not in place. As a first approximation, the lower bound can be thought of as a minimum target for naturalizing downstream sediment levels, and the upper bound can be thought of as a maximum limit to avoid excessive sediment releases.

The flow/concentration relationship is not monotonic or with a consistent slope. Rather the relationships in log space at low flows, moderate flows, and high flows exhibit separate slopes. By observation, at many of the lakes the sediment concentrations actually reverse at higher flows. This behavior translates into flatter flow-low curves as described earlier in the document. While the reasons for this phenomenon are unknown, it could be can be explained by either the supply limitation of easily erodible sediments or by sediment lost to the floodplain during overbank flows. A fourth-order polynomials through log-transformed data were used to reflect the overall trends in the data. This curve is a valid fit over the range of observed data but should not be used for extrapolation.

Also evident in the data is a reduction in the variability in concentration at higher flows. While fewer measurements could by itself lead to the perception of less variability, visual inspection of the flow/concentration measurements suggests that physical reasons may drive the lower variability. The first possible reason for the reduction in variability is the supply limitation that drives the reduction in concentration. Higher concentrations are constrained by lack of readily available material. A second explanation for lower variability is that moderate flows can be achieved by a precipitation in only part of the watershed, and different sub-watersheds may have different sediment contributions. On the other hand, very high flows are only achievable when most of if not all the entire watershed contributes, which reduces the spatial variability based on storm placement.

The confidence intervals were originally computed based on the statistics of the total sample and departure from the best-fit polynomial. However, the height of the intervals was driven by the high variability at moderate flows, which yielded confidence intervals at the highest flows that exceeded all the measured data points. A more refined approach was taken to take into account differences in variability as a function of flow.

The 4th-degree polynomial through log-transformed data was used as a predictor for the mean of a normal distribution in a generalized additive model. To capture the changing variance of concentration with respect to flow, a concave-down function was needed to predict the sigma term in order to produce a local maximum in the middle with decreases in variance at both extremes. A quadratic function was used to ensure d2σ/dx2<0,∀x. The result is a six-parameter model (plus two intercepts) in a hierarchical structure.

The interpretation should be looked at as "the conditional distribution of suspended sediment concentration, given an amount of flow." That is, f(y|x)=N(μ=b4x4+b3x3+b2x2+b1x+b0, σ=a2x2+a1x+a0) where y is SSC, x is flow, and N is the normal distribution with parameters μ and σ. Because the conditional distribution is normal, the best estimate of y|x is E[y|x] and is also the median of the conditional distribution, and it can be computed as μ. To obtain percentiles of the conditional distribution, the quantile function (inverse CDF) is used, F-1(p;μ,σ) where μ and σ are computed for the given value of flow. The result is overall a function that has a central tendency dictated by the 4th-order polynomial, with spread about the mean dictated by a concave-down quadratic.

These analyses were automated using R-scripts.

As seen in Figure 11-2, the sediment concentration in the water during high inflow events is considerably higher than those during low flows. This sets a reasonable upper bound for sediment restoration activities to remain within the natural variability of the system. At the highest flows the concentrations decrease, suggesting a supply-limited system.

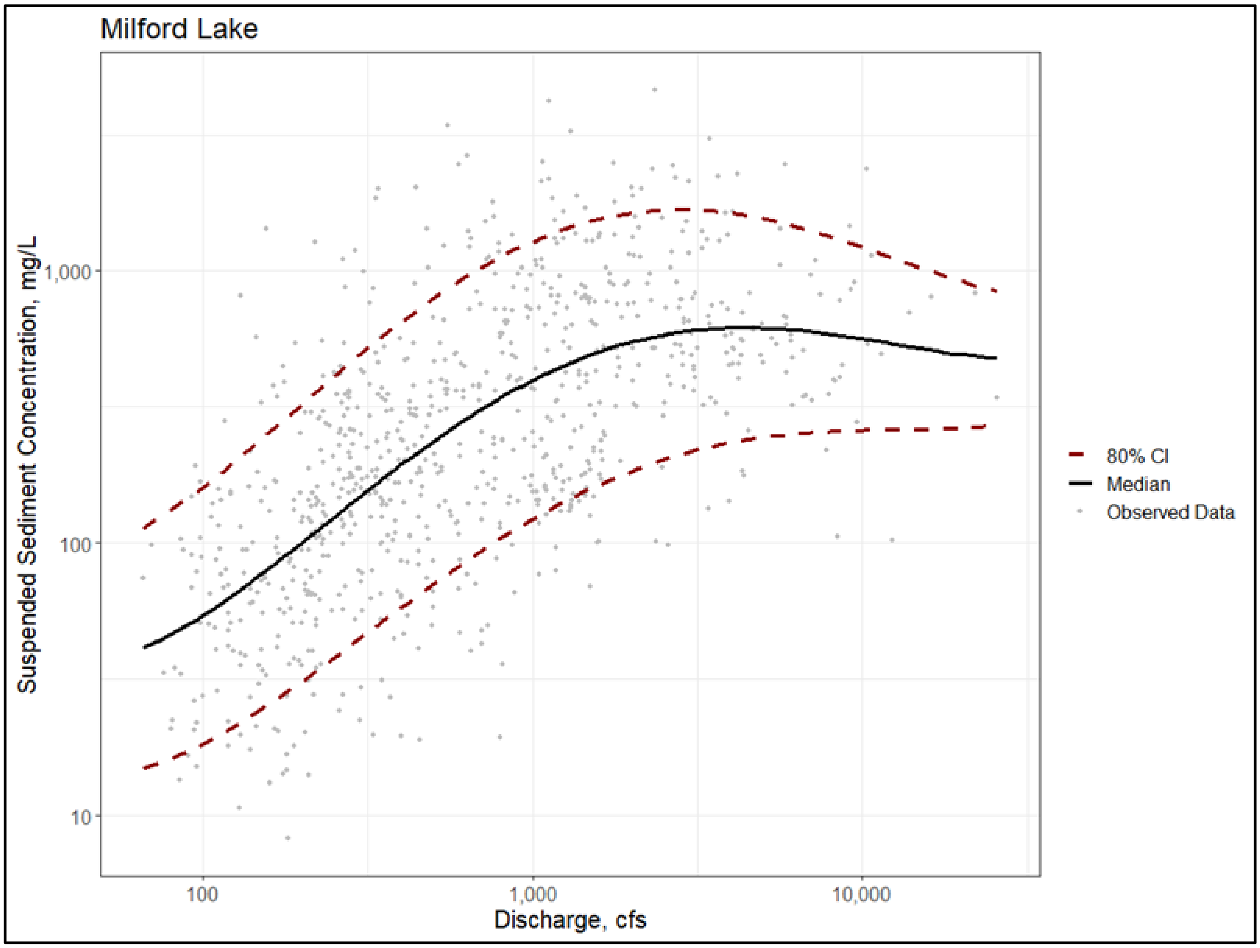


Figure 11-2: Incoming Sediment Concentration to Milford Lake. Earlier points are adjusted to account for the lowering of the flow/load rating curve over time.

12.0 RESERVOIR BED SEDIMENT COMPOSITION

Measurements of the sediment size in the lake were collected by the Kansas Biological in 2013 within the multipurpose pool. The results are presented in Figure 12-1 are of the surface sediments. These results indicate that the deposits within the multipurpose pool are 18.6% sand, 26.8% silt, and 54.6% clay by mass which is similar to the estimate from the stream gage data. The sediment inflow analysis found the incoming load to be 46.1% clay, 29.78% silt, and 24.1% sand/gravel. However, since the KBS measurements are of only the surface sediments and were only collected within the multipurpose pool, they may not be reflective of all the deposits.

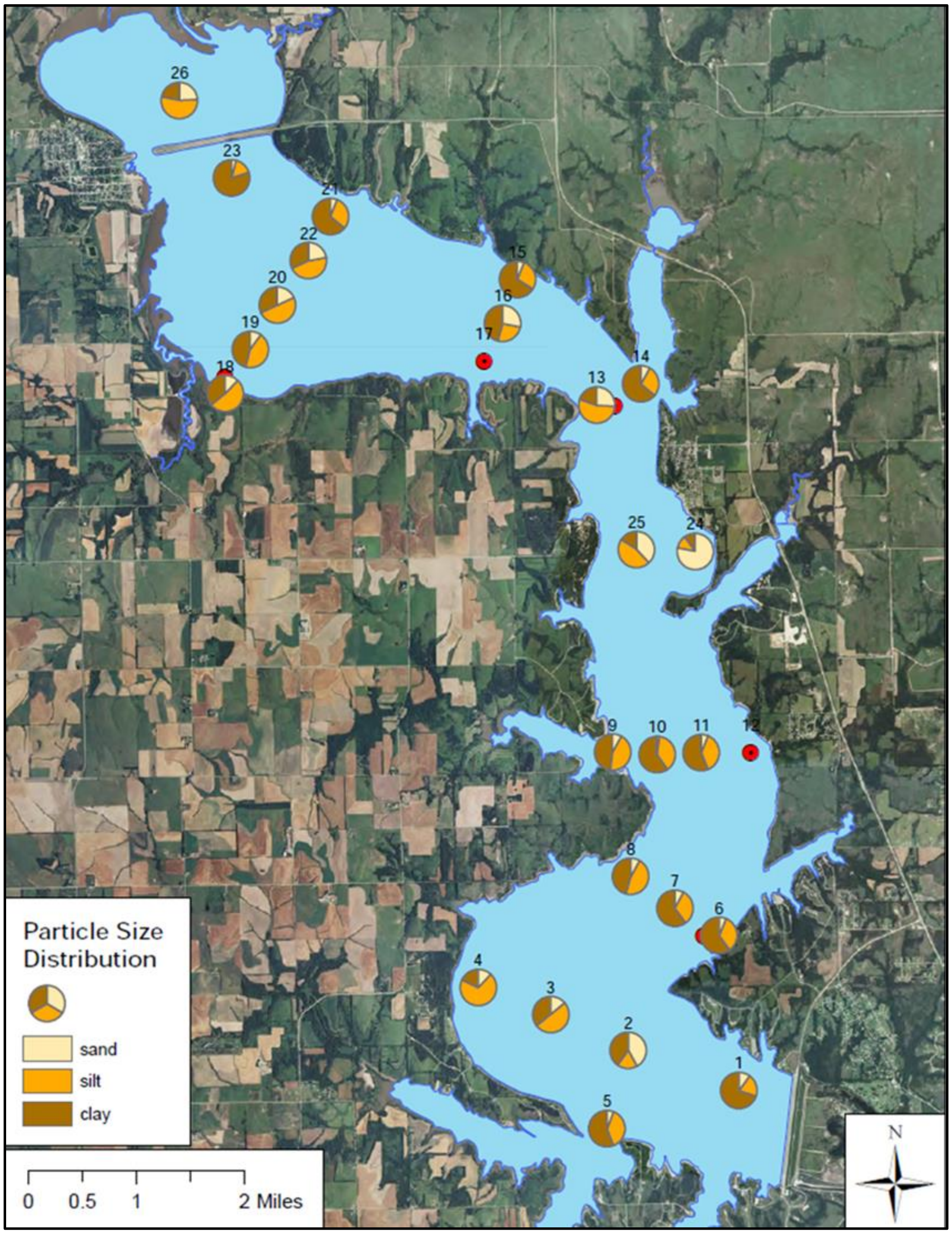


Figure 12-1: Sediment Size Gradations in the Multipurpose Pool. *(KBS 2013)*

Table 12-1 shows the measured percentages from the surficial samples.

Table 12-1: Sediment sizes from surficial measurements.

Site	Sand	Silt	Clay
MIL-1	10	20	70
MIL-2	42	18	40
MIL-3	14	50	36
MIL-4	12	70	18
MIL-5	6	38	56
MIL-6	6	34	60
MIL-7	8	32	60
MIL-8	8	46	46
MIL-9	8	44	48
MIL-10	2	38	60
MIL-11	6	38	56
MIL-13	26	54	20
MIL-14	8	28	64
MIL-15	6	28	66
MIL-16	28	26	46
MIL-18	12	52	36
MIL-19	10	44	46
MIL-20	18	50	32
MIL-21	6	30	64
MIL-22	22	46	32
MIL-23	4	16	80
MIL-24	78	12	10
MIL-25	38	46	16
MIL-26	24	54	22
Average	16.8	38.1	45.2

The Kansas Biological Survey also collected eight sediment cores from the lake and determined the sediment size with depth. Figure 12-2 shows the core collection locations.

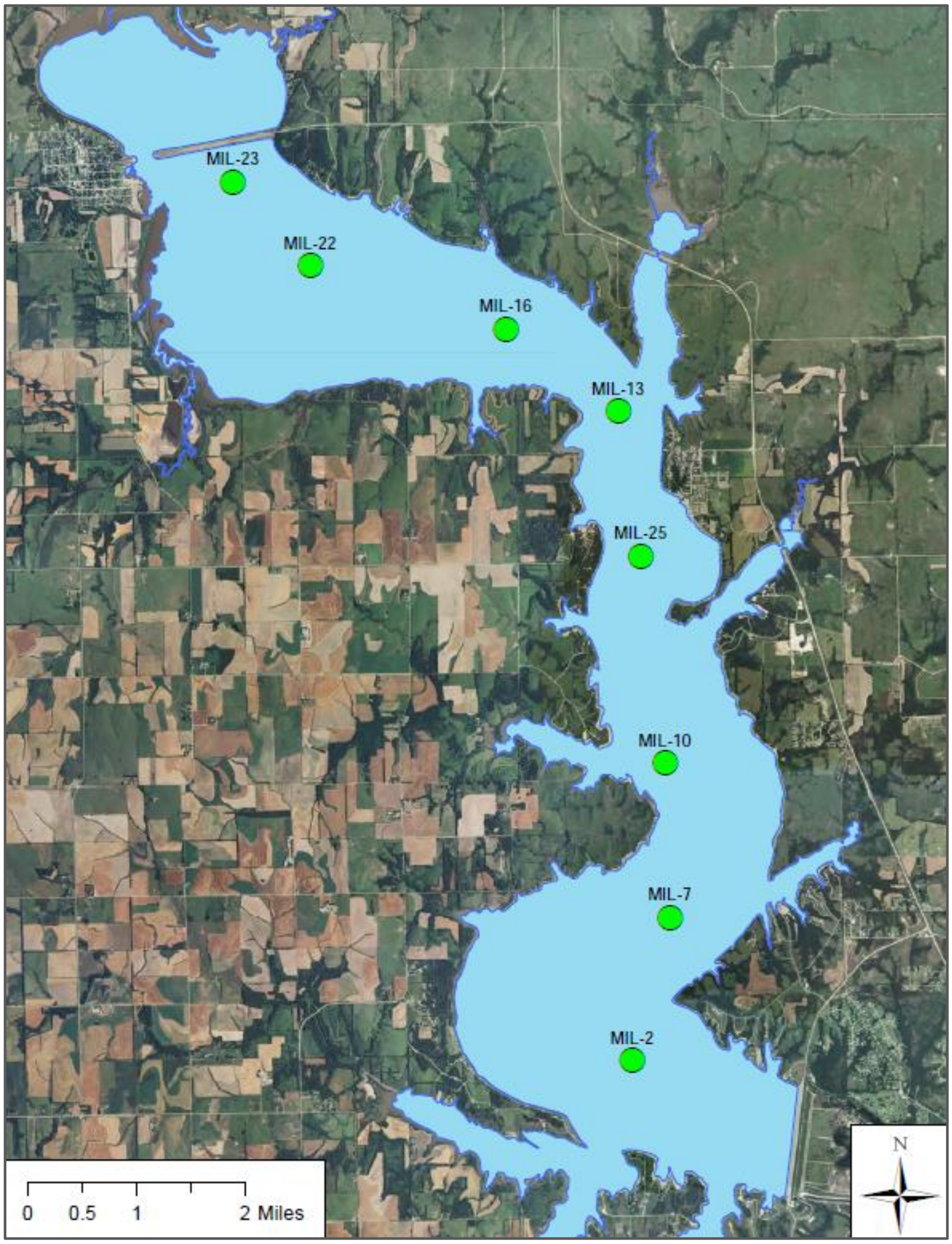


Figure 12-2: Sediment core collection locations (KBS, 2013)

Sediment sizes were measured at depth in increments of 5 cm. Figure 12-3 is an example breakdown of one of the sediment cores.

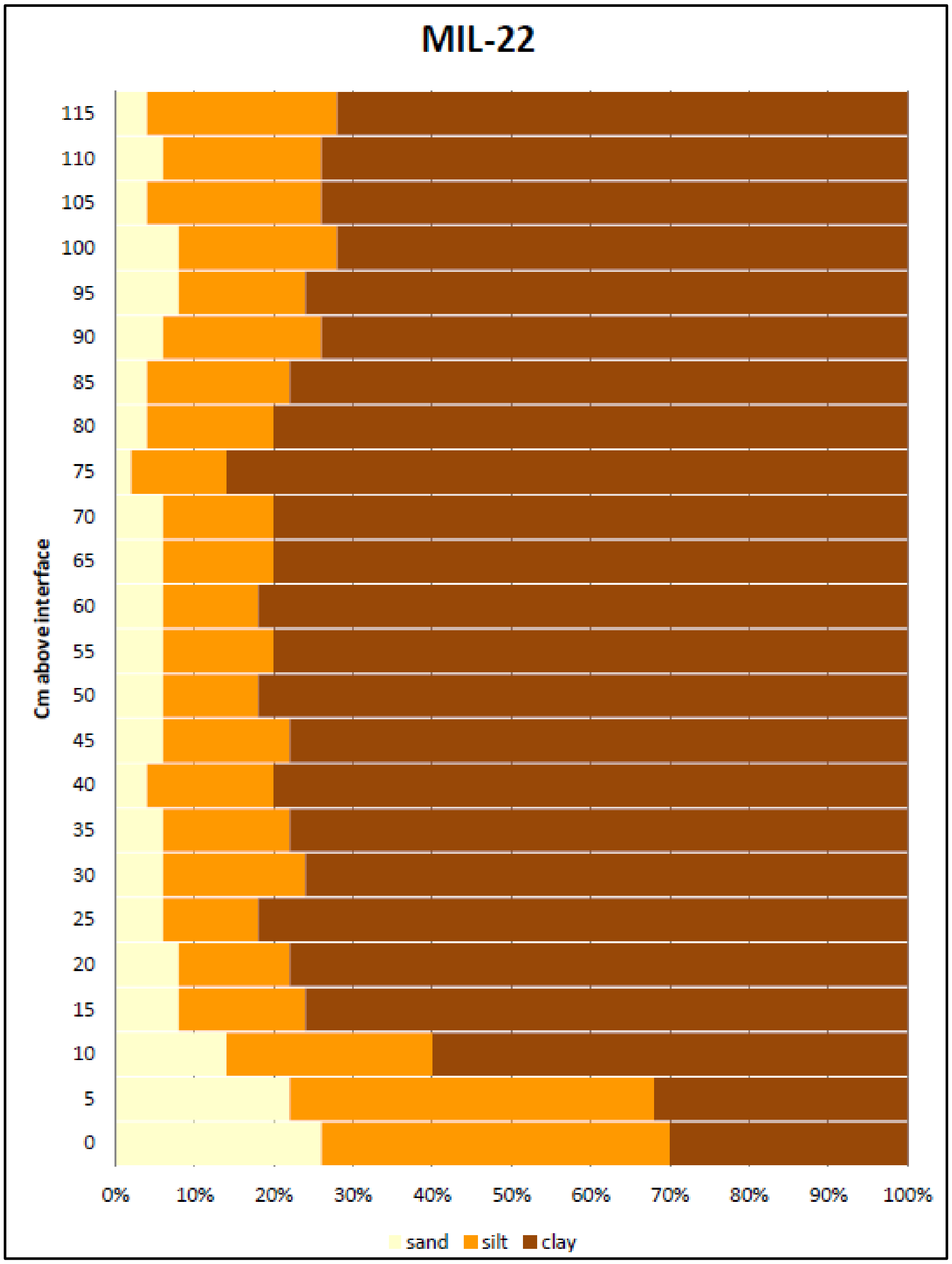


Figure 12-3: Sediment core taken from Milford Lake

Table 12-2 gives the average of the sediment sizes from each of the sediment cores along with the overall average.

Table 12-2: Average sediment size from sediment cores.

Core	Sand	Silt	Clay
2	9.3	24.7	66.0
7	10.0	31.8	58.2
10	44.6	38.7	16.8
13	51.4	23.3	25.4

Core	Sand	Silt	Clay
16	10.8	21.5	67.6
22	7.6	19.1	73.3
23	3.6	22.9	73.5
25	12.0	32.2	55.8
Average	18.7	26.8	54.6

13.0 SEDIMENT CHEMICAL CONCENTRATIONS

Chemical characteristics of the deposited sediment were investigated by the USGS in a 2001 report (Christensen & Juracek, 2000). The study investigated the concentration of various metals and other chemical compounds in the sediment and compared them to guidelines published by the U.S Environmental Protection Agency (USEPA). The threshold-effect level (TEL) is the concentration below which toxic biological effects seldom occur, while the probable-effect level (PEL) is the level above which toxic effect usually or frequently occur. Between the TEL and PEL toxic effect will occasionally occur. However, these guidelines are to be used as screening tools and are not regulatory criteria (Juracek & Mau, 2002). Sediment quality guidelines have been published by the USEPA for nine metals and six organochlorine compounds.

The chemical concentration of 17 metals were tested in 20 samples from the sediment in Milford Lake. Of the eight metals having guidelines published by the USEPA, five of them exceeded the TEL in at least one of the samples. However, none of the samples contained any metals with a concentration above the PEL. Additional information can be found in the report.

14.0 DELTA LOCATION AND VOLUME

Figure 14-1 provides a profile plot of a centerline through the lake. At each location, the invert elevation (lowest elevation in a given sedimentation range line) is plotted. The 1967-1980 rangeline surveys were converted from NVGD29 to NAVD88 using values obtained from a conversion raster in Arc-GIS based on the USACE Corpscon conversion tool. The rangelines for the 2009 survey were pulled from the 2009 combined bathymetry and LiDAR DEM. Because LiDAR cannot penetrate through water and bathymetry data was not collected above the multipurpose pool, the invert elevations and calculated volumes are likely overestimated upstream of this for the 2009 survey. As seen in Figure 14-1, the delta crest grew significantly immediately after dam closure. From 1980 to 2009, the delta progressed downstream at a rate of 228 ft/year.

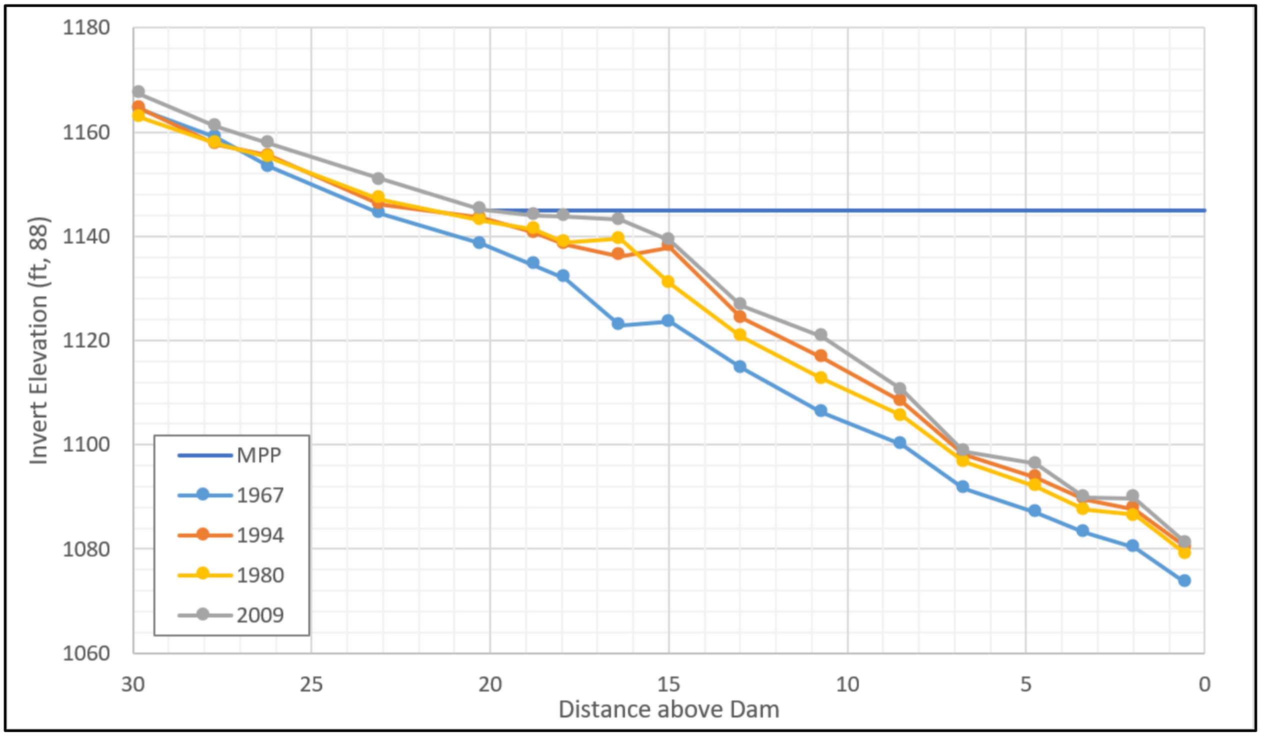
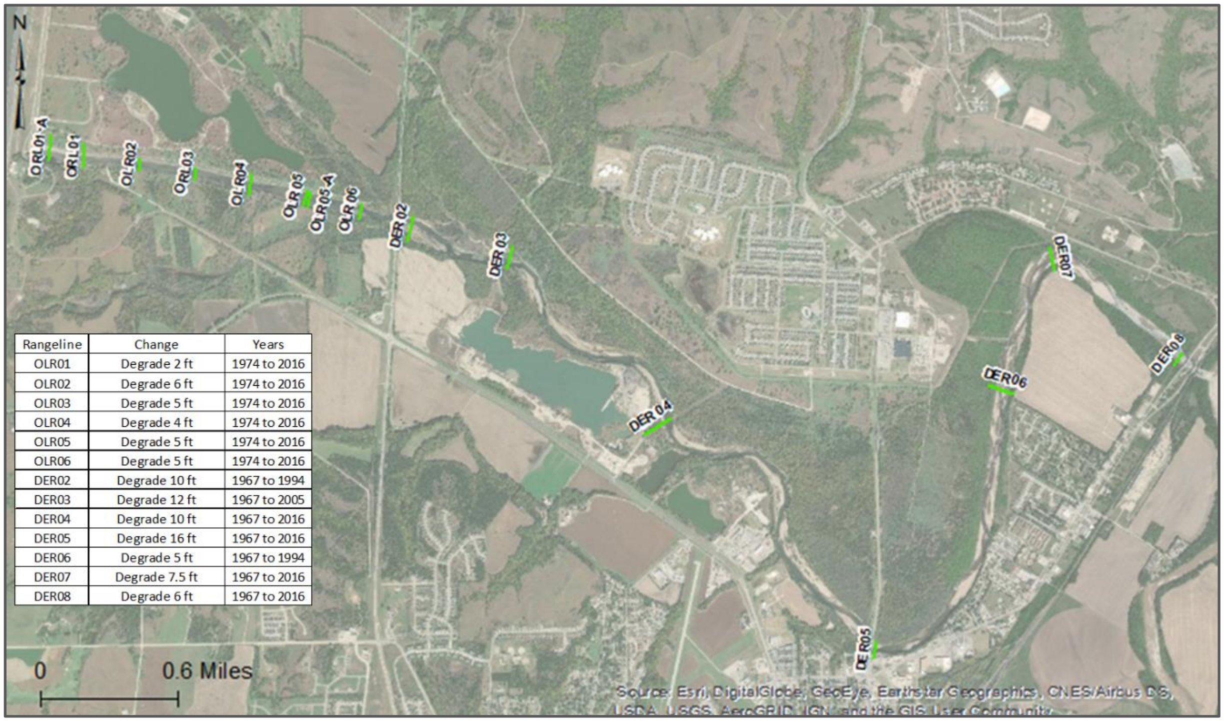


Figure 14-1: Profile of Invert Elevation Indicating Delta Location and Growth.

15.0 DOWNSTREAM CHANNEL

Sediment trapping by dams very often induces bed degradation and bank erosion downstream. Degradation rangelines downstream from Milford Lake allow this effect to be quantified. Figure 15-1 shows the location of the rangelines, the total bed elevation change, and channel width change at each.



Rangeline	Change	Years
OLR01	Degrade 2 ft	1974 to 2016
OLR02	Degrade 6 ft	1974 to 2016
OLR03	Degrade 5 ft	1974 to 2016
OLR04	Degrade 4 ft	1974 to 2016
OLR05	Degrade 5 ft	1974 to 2016
OLR06	Degrade 5 ft	1974 to 2016
DER02	Degrade 10 ft	1967 to 1994
DER03	Degrade 12 ft	1967 to 2005
DER04	Degrade 10 ft	1967 to 2016
DER05	Degrade 16 ft	1967 to 2016
DER06	Degrade 5 ft	1967 to 1994
DER07	Degrade 7.5 ft	1967 to 2016
DER08	Degrade 6 ft	1967 to 2016

Figure 15-1: Degradation Range lines Downstream of Milford

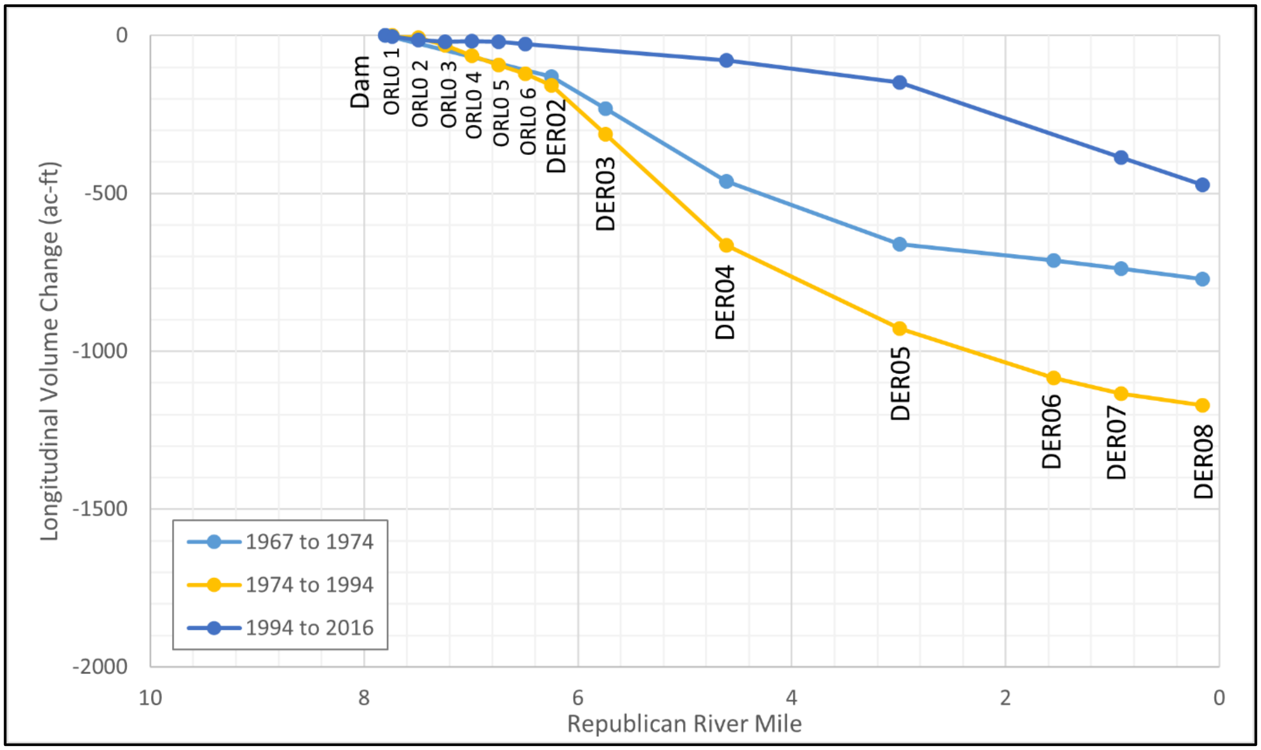


Figure 15-2 plots the cumulative volume change over time based on the degradation range lines.

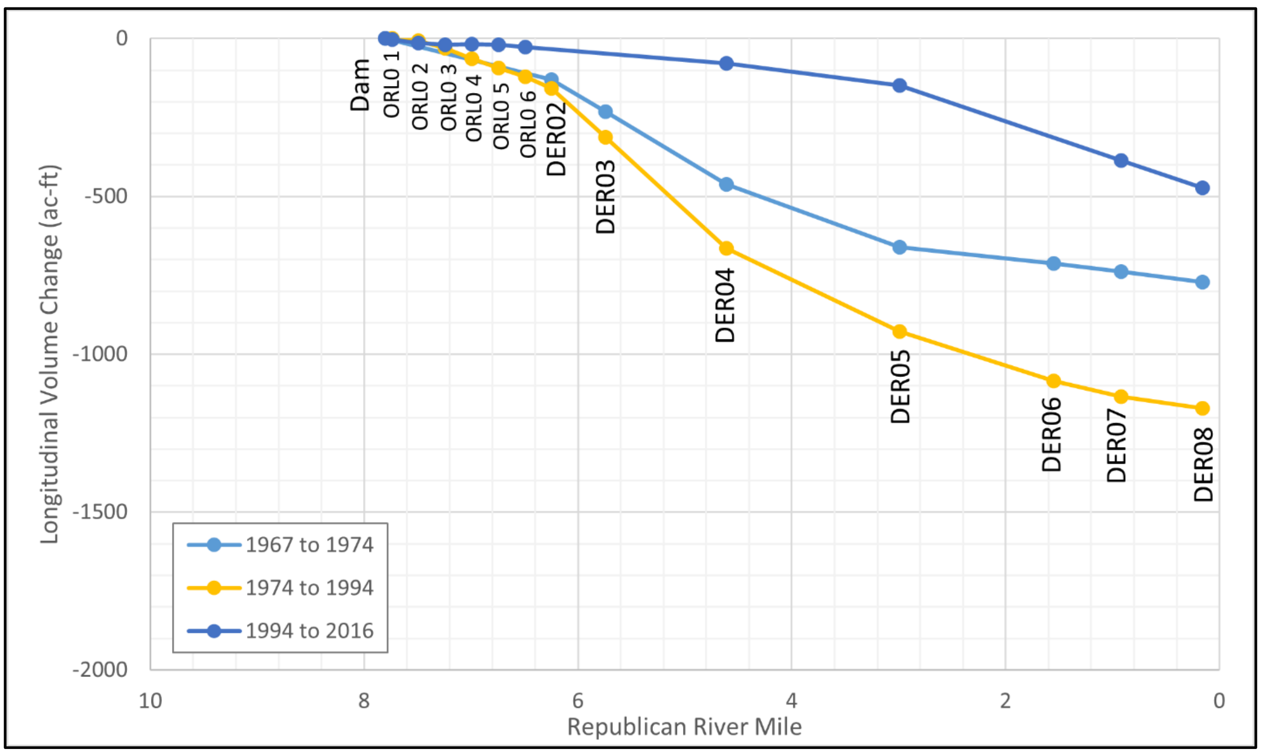


Figure 15-2: Longitudinal Volume Change of Republican River by River Mile 1967-2016.
Longitudinal Cumulative Volume Change Curves for Degradation Rangelines, 1967-2016. Each time period indicates additional degradation.

Extending these rangeline results to the whole reach from 1967 to 2016, suggests that the bed and banks have lost 2,415 acre-feet of material since dam closure as shown in Table 15-1. As some of the degradation rangelines are near stabilized bridge locations, this analysis may under-predict the degradation. The total degradation on the Republican River equates to 35% of the volume of sand deposition within the reservoir, which was estimated to be 6,906 acre-feet.

Table 15-1: Degradation downstream of Milford Lake.

Years	Degradation (ac-ft)
1967-1974	771
1974-1994	1,172
1994-2016	472
Total	2,415

These analyses indicate that the Republican River downstream of Milford Dam is sediment starved. Continued degradation with associated bank erosion is expected if sediment trapping continues.

16.0 SUMMARY AND CONCLUSIONS

Sedimentation has had a moderate impact on Milford Lake through loss of storage capacity and impacts to infrastructure surrounding the lake. As of August 1994, the multipurpose pool had lost 43,011 ac-ft of storage capacity to sedimentation, or 10.4% of the original volume. Between 1967 and 1994 the flood pool lost 4,924 ac-ft of storage, which 0.65% of its original volume. Analysis using the Cross Section Viewer Software showed that the reservoir overall lost an additional 14,241 ac-ft of storage from 1994 to 2009. However, the proportions that deposited in the flood and multipurpose pools were not determined.

The trapping efficiency of the reservoir was estimated to be 96.3% from the Brune Curve. Sediment rating curves were created from suspended sediment measurements taken at the Clay Center gage on the Republican River. Bulk density was estimated to be 41.1 pcf in the multipurpose pool and 72.4 pcf in the flood pool. The sediment deposition within the reservoir was calculated using the sediment rating curves, bulk density, and trapping efficiency. This was compared to the sediment deposition estimated from the survey data and the upper portion of the rating curves were adjusted to bring them into closer agreement. The final computed deposition values matched well with the surveyed deposition.

Approximately 90% of the incoming sediment enters Milford Lake during flood control operations.

A range in the natural sediment concentrations in inflow to Milford Lake was estimated from the upstream suspended sediment measurements by fitting 80% confidence intervals to the data. Concentration increases with discharge and peaks at approximately 11,000 cfs. Using the sedimentation rangelines, the delta was estimated to have moved towards the dam at a rate of 228 feet per year from 1980 to 2009. The estimated degradation on the Republican River downstream of the lake was 2,415 ac-ft from 1967 to 2016, which is 35% of the sand accumulation.

17.0 REFERENCES

Brune, G. M. (1953). *Trap Efficiency of Reservoirs.* American Geophisical Union.

Christensen, V. G., & Juracek, K. E. (2000). Variability of Metals in Reservoir Sediment from two Adjacent Basins in the Central Great Plains. *Enviromental Geology*, 470-481.

Duan, N. (1983). *Smearing estimate: A nonparametric retransormation method.* Journal of the American Statistical Association, 78(383), 605-610.

Juracek, K. E., & Mau, D. P. (2002). *Sediment Deposition and Occurrence of Selected Nutrients and other Chemical Constituents in Bottom Sediment, Tuttle Creek Lake, Northeast Kansas, 1962-99.* Lawrence, KS: U.S. Geological Survey.

KBS. (2013). *Sediment Surveys of Milford and Tuttle Creek Reservoirs, Kansas.* Kansas Biological Survey.

Shelley, J. E. (2012). *Geomorphic Equations and Methods for Natural Channel Design.* Lawrence, KS: Doctoral Dissertation, University of Kansas.

USACE. (1961). *Milford Dam & Reservoir Design Memorandum No. 10, Outlet Works and Spillway.* US Army Corps of Engineers, Kansas City District.

USACE. (1984). *Lake Regulation Manual, Milford Lake, Kansas.* US Army Corps of Engineers, Kansas City District.

USACE. (2016). *Millford Dam Periodic Inspection No. 14, Periodic Assessment No. 1.* US Army Corps of Engineers, Kansas City District.

Zaiontz, C. (2014). *Confidence/prediction intervals | Real Statistics Using Excel*. Retrieved from real-statistics.com: http://www.real-statistics.com/regression/confidence-and-prediction-intervals/

US Army Corps of Engineers ®
Kansas City District

Kansas River Reservoirs Flood and Sediment Study

Appendix D1.5: Kanopolis Lake Existing Condition Sedimentation

November 2022

Contents

1.0 INTRODUCTION

Kanopolis Lake is located in central Kansas on the Smoky Hill River. The dam was closed on 24 March 1948 and the multipurpose pool (MPP) was first filled on 19 July 1948. The multipurpose pool elevation was raised four feet in 1968 to an elevation of 1463 feet NGVD29. Authorized purposes include flood control, silt control, irrigation, water supply, low-flow supplementation, recreation, and fish and wildlife (USACE, 2016). The contributing drainage area above the dam is approximately 7,860 square miles with about 67 percent of this controlled by a U.S. Bureau of Reclamation (USBR) dam named Cedar Bluff. Cedar Bluff dam was closed on 10 September 1950 and the multipurpose pool was first filled on 21 June 1951 (USACE, 2016). Predominant land use in the watershed is agricultural, consisting of cropland and grazing. Average annual precipitation increases from west to east in the basin. Figure 1-1 shows Kanopolis Lake with respect to whole Kansas River Basin, while Figure 1-2 shows the lake and the Smoky Hill River Basin above Kanopolis. Also shown in the figure is the portion of the basin that is controlled by Cedar Bluff Dam and the U.S. Geological Survey (USGS) gage on the Smoky Hill River at Ellsworth, Kansas that was used in this report.

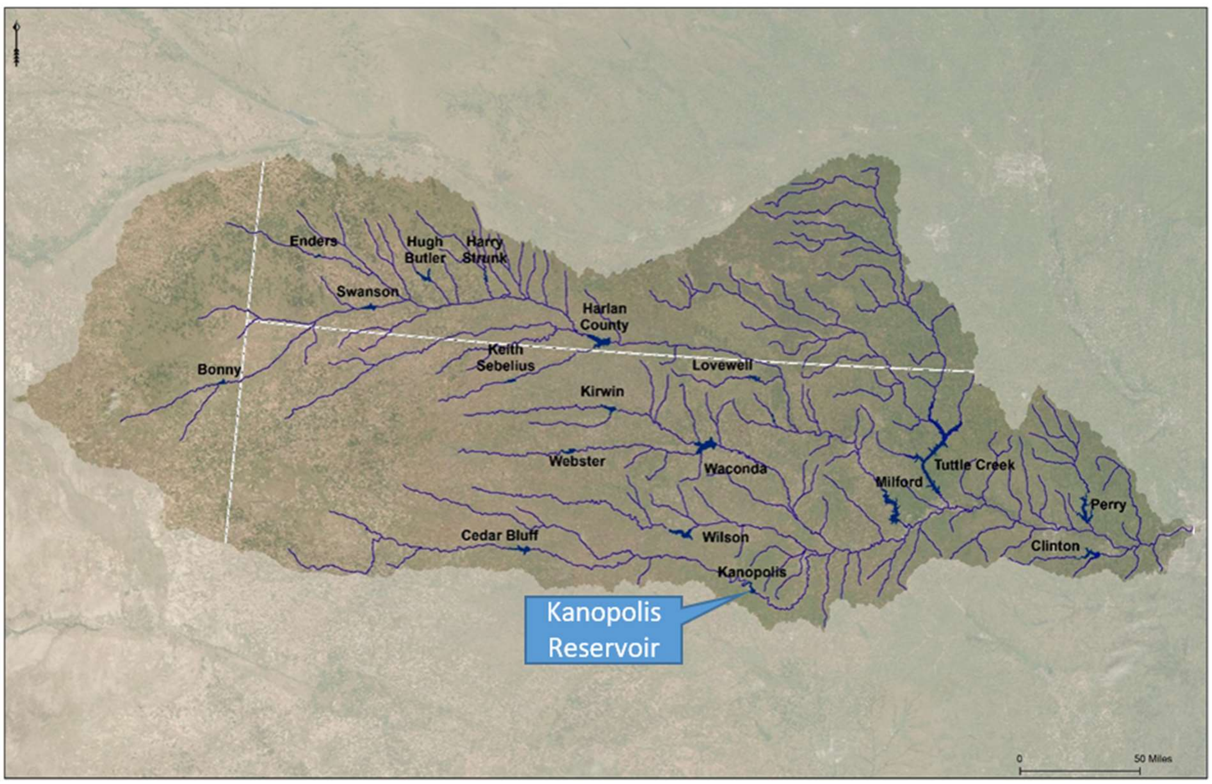


Figure 1-1: Overall Kansas River Basin and Kanopolis Lake

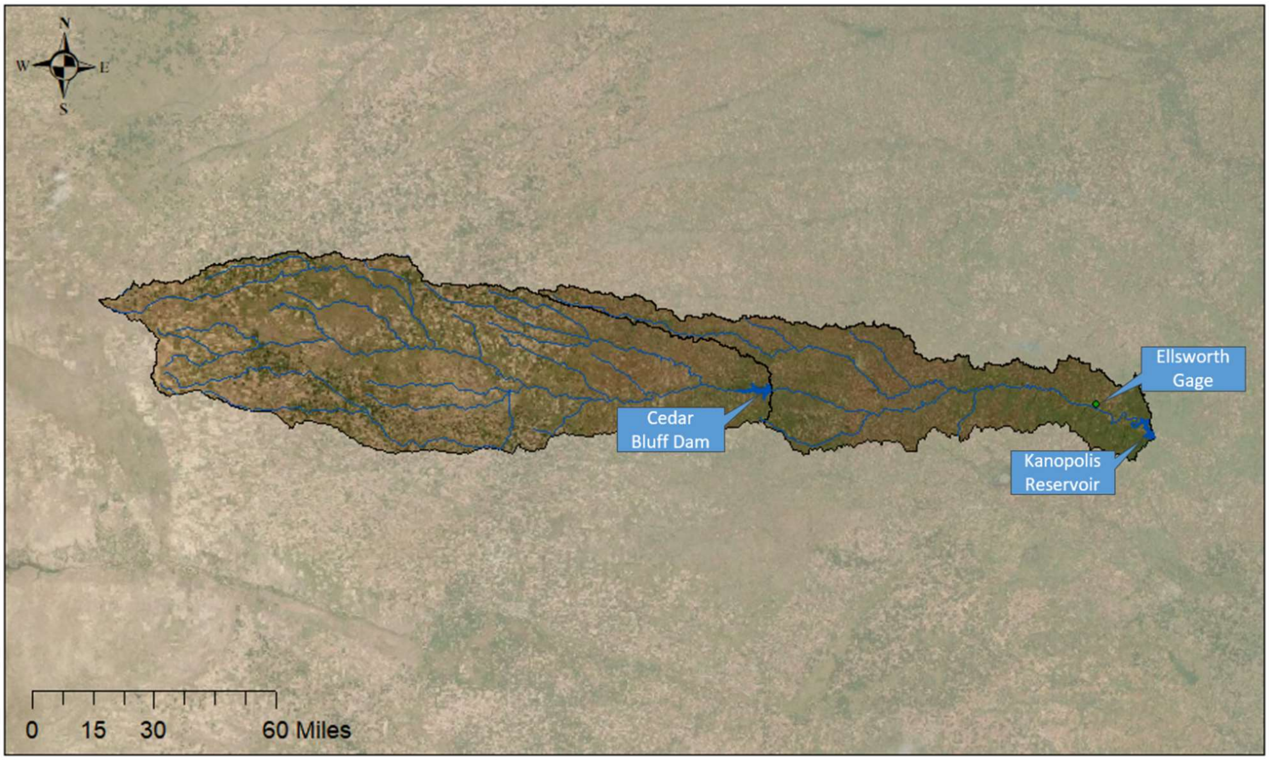


Figure 1-2: Smoky Hill Watershed above Kanopolis Dam

2.0 DAM INFRASTRUCTURE

Table 2-1 gives a summary of the dam infrastructure and Figure 2-1 shows the location of each feature. Elevations are shown in both NVGD29 and NAVD88. A survey by the Kansas City District (NWK) in 2013 showed a 0.36 ft difference between NVGD29 and NAVD88 at the lake's gage (USGS gage number 06865000). This is lower than the factor of 0.44 feet that was obtained when using the USACE CORPSCON software (SURDEX, 2011). The conversion factor obtained from the 2013 survey was used to convert from NVGD29 to NAVD88 in this section as this was considered to the official conversion value used by the Kansas City District.

The outlet works consists of two controlled gates located in the control tower having an invert elevation of 1415.0 feet NVGD29. There is also an uncontrolled port that has an invert elevation 1459.0 feet NVGD29. The dam's emergency spillway has an elevation of 1507.0 NVGD29. The typical tailwater elevation was approximated using the USGS gage near Langley, Kansas (06865500), which is 0.4 miles downstream of the lake. This was determined by adding the average stage of 4.25 feet to the gage datum of 1395.66 feet (NVGD29).

Table 2-1: Important Information Relating to the Dam Infrastructure. Elevations in NVGD29 (NAVD 88 in parenthesis)

Parameter	Value
Multipurpose Pool Elevation	1463.0 (1463.36)
Lowest Elevation Outlet	1415.0 (1415.36)
Number of Gates at This Low Elevation	2
Service Gate Average Annual Usage	167 days/year
Uncontrolled Port Elevation	1463.0 (1463.36)
Spillway Elevation	1507.0 (1507.36)
Dam Elevation	1537.0 (1537.36)
Typical Tailwater Elevation	1400.3 (1400.68)
Other Pipes Going Through the Dam or Embankment	Post Rock Water supply intake

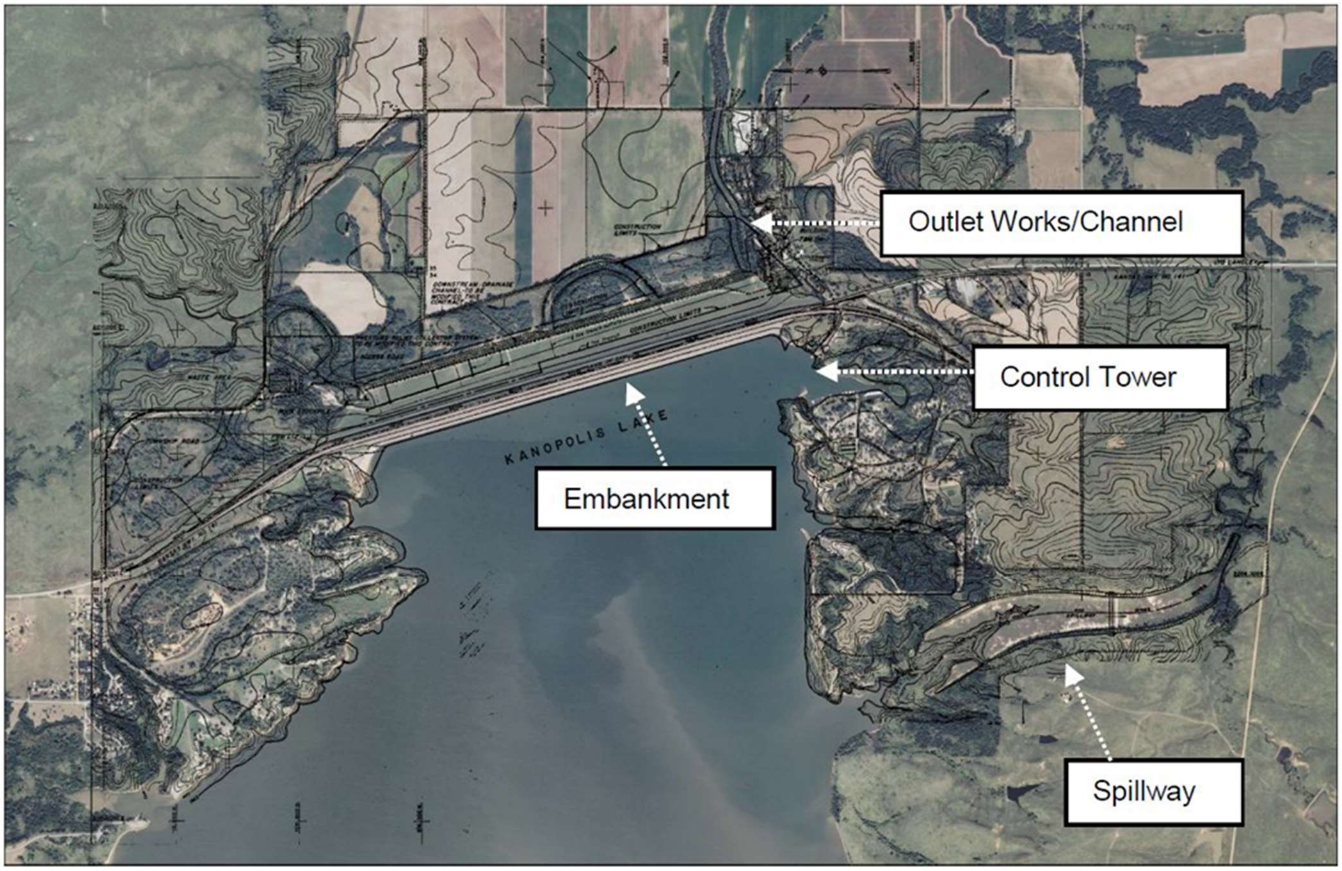


Figure 2-1: Kanopolis dam and infrastructure (USACE, 2016)

Figure 2-2 through Figure 2-4 show drawings of the control tower and other dam infrastructure.

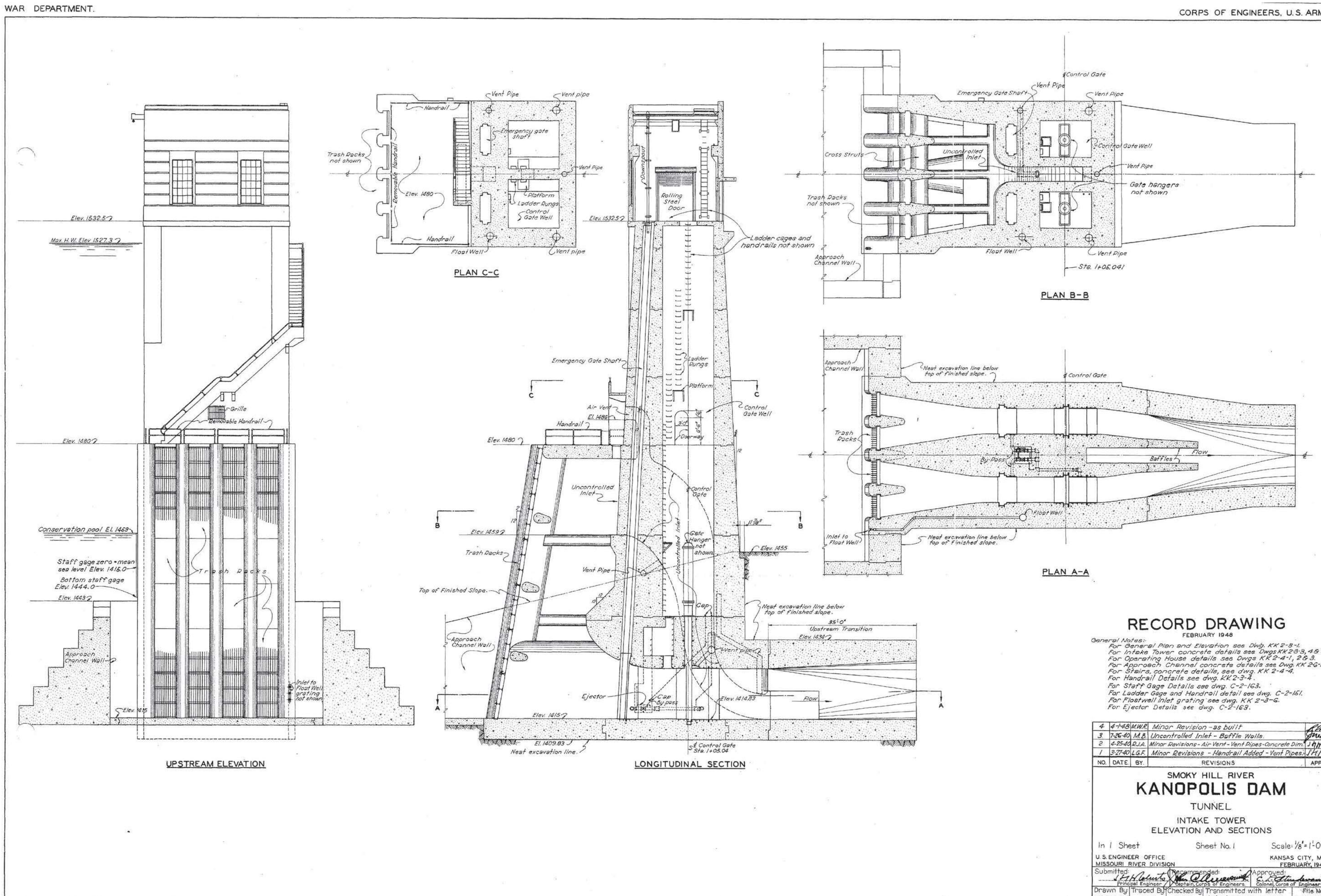


Figure 2-2: Intake tower, elevation and sections. Elevations in NVDG29.

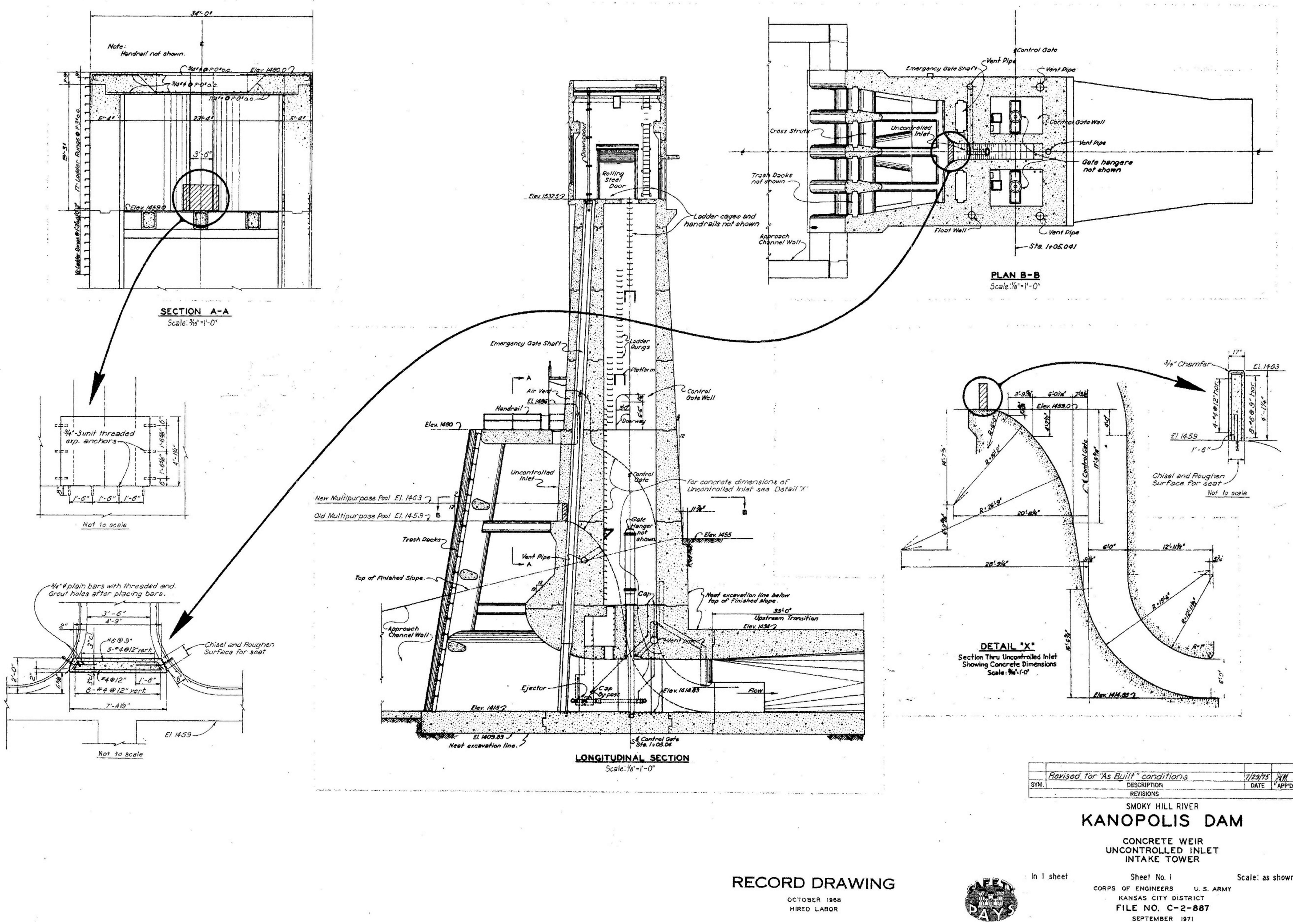


Figure 2-3: Concrete Weir Uncontrolled Inlet. Elevations in NVGD29.

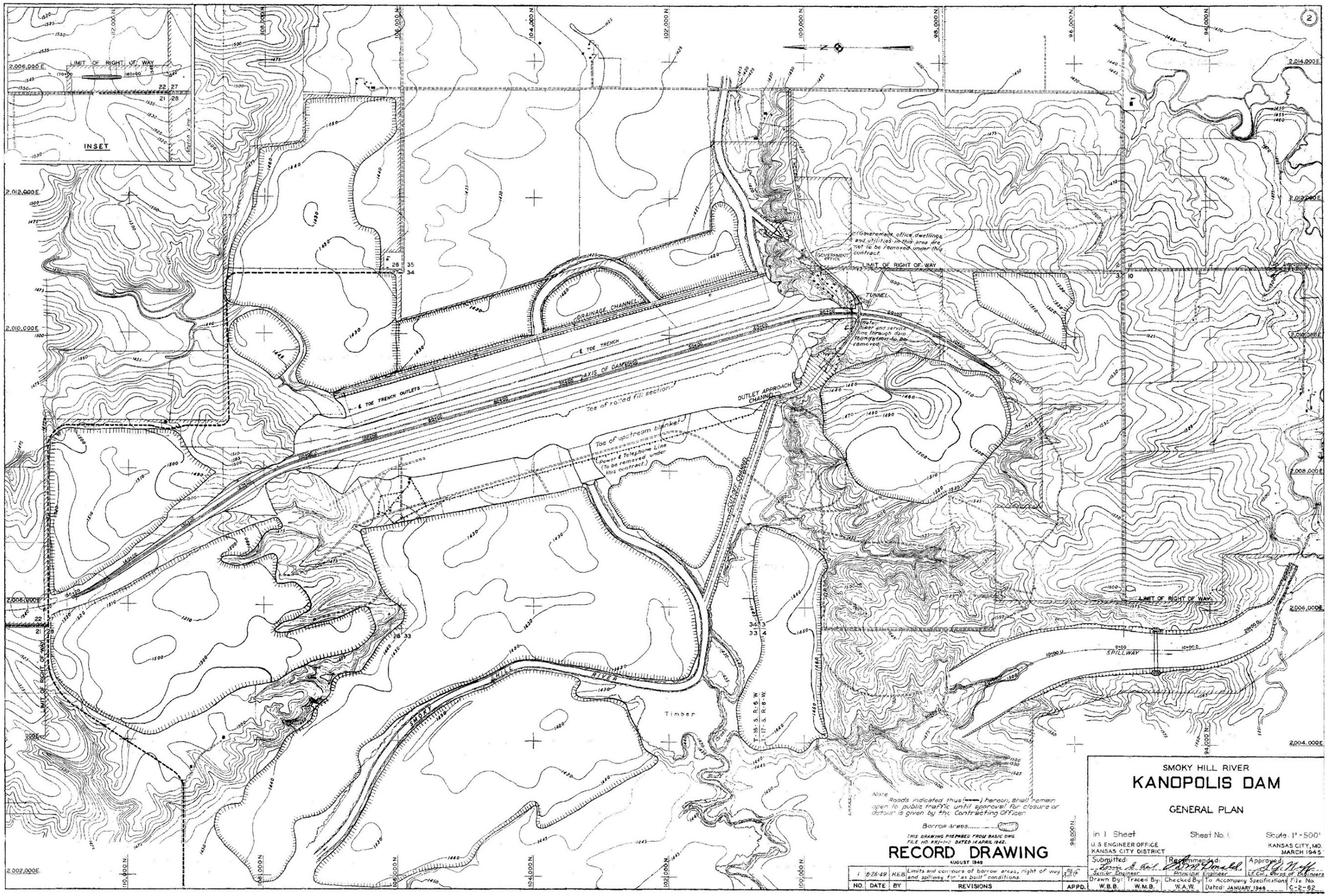


Figure 2-4: Kanopolis General Plan Drawing (USACE, 2016). Elevations in NVGD 29.

3.0 SEDIMENTATION EFFECTS ON O&M

In addition to the lost benefits from shrinking pools, the lake has experienced impacts to the operations and maintenance. Sedimentation has created difficulties in the operation of the gates because of sediment accumulation around the control tower. In 2007 during a routine exercise, the emergency bulkhead gates were unable to be deployed because of sedimentation around the gates (USACE, 2016). A dive inspection revealed 10 to 19 feet of sediment and driftwood near the gate. In September 2009, approximately 5,000 cubic yards of sediment was dredged from around the tower with the assistance of divers. However, sedimentation has continued to be a problem and additional dredging was completed in 2018 through 2020. The sedimentation problems may be caused by eroding banks on either side of the intake tower. The 2018-2020 dredging project also included bank stabilization on the left side of the control tower. However, the right bank has continued to erode. Annual silt flushes have been recommended in past inspections; however, test flushes have shown that this is not effective in removing enough sediment.

The Post Rock water supply intake, which is located in the lake had to be modified to extend three feet farther into the lake because of sedimentation issues. This project was completed in 2017.

Sedimentation has also had a large impact on recreation on the lake depending on the elevation of the pool. When pool elevation is at multipurpose elevation, only one out of five boat ramps is easily accessible. Also, the state of Kansas has dredged around this boat ramp several times in order to maintain access. Sedimentation has also created difficulties at the lakes' marina and docks.

4.0 EXISTING CONDITION STORAGE ELEVATION CURVES

The most recent storage elevation curve for Kanopolis Lake was created from a combination of LiDAR and bathymetry. Bathymetry was collected by USACE in October 2017 at an approximate 200 foot transect spacing using single beam sonar. The LiDAR used in the storage curve was obtained by the state of Kansas, in 2018. The 2018 LiDAR had a grid spacing of one meter. Both the LiDAR and bathymetry were combined into a digital elevation model (DEM) using Arc-GIS. The Surface Volume in Arc-GIS was then used to calculate the storage elevation tables. Table 4-1 shows the final values for the updated storage elevation curve.

Table 4-1: Storage Elevation Curve for Kanopolis Lake at 1-ft intervals (NVGD29) created from 2017 bathymetry and 2018 LiDAR

Elevation (ft)	Volume (ac-ft)	Elevation (ft)	Volume (ac-ft)	Elevation (ft)	Volume (ac-ft)	Elevation (ft)	Volume (ac-ft)
1432	0	**1460**	38,301	**1488**	197,264	**1516**	547,295
1433	1	**1461**	41,090	**1489**	205,974	**1517**	564,948
1434	60	**1462**	44,189	**1490**	214,920	**1518**	583,016
1435	386	**1463**	47,794	**1491**	224,085	**1519**	601,520
1436	916	**1464**	51,522	**1492**	233,474	**1520**	620,490
1437	1,539	**1465**	55,305	**1493**	243,086	**1521**	639,956
1438	2,238	**1466**	59,108	**1494**	252,926	**1522**	659,923
1439	3,001	**1467**	62,953	**1495**	263,010	**1523**	680,399
1440	3,812	**1468**	66,977	**1496**	273,359	**1524**	701,383
1441	4,673	**1469**	71,239	**1497**	283,983	**1525**	722,854
1442	5,578	**1470**	75,882	**1498**	294,886	**1526**	744,809
1443	6,533	**1471**	80,840	**1499**	306,074	**1527**	767,245
1444	7,545	**1472**	86,098	**1500**	317,562	**1528**	790,170
1445	8,648	**1473**	91,597	**1501**	329,339	**1529**	813,597
1446	9,875	**1474**	97,304	**1502**	341,417	**1530**	837,527
1447	11,233	**1475**	103,200	**1503**	353,818	**1531**	861,972
1448	12,724	**1476**	109,285	**1504**	366,517	**1532**	886,946
1449	14,328	**1477**	115,585	**1505**	379,521	**1533**	912,453
1450	16,036	**1478**	122,085	**1506**	392,853	**1534**	938,500
1451	17,833	**1479**	128,769	**1507**	406,535	**1535**	965,129
1452	19,725	**1480**	135,615	**1508**	420,602	**1536**	992,345
1453	21,707	**1481**	142,638	**1509**	435,061	**1537**	1,020,158
1454	23,785	**1482**	149,852	**1510**	449,898	**1538**	1,048,604
1455	25,965	**1483**	157,248	**1511**	465,128	**1539**	1,077,694
1456	28,241	**1484**	164,835	**1512**	480,755	**1540**	1,096,645
1457	30,613	**1485**	172,620	**1513**	496,786	-	-
1458	33,076	**1486**	180,602	**1514**	513,214	-	-
1459	35,633	**1487**	188,809	**1515**	530,048	-	-

5.0 TRAPPING EFFICIENCY

From 2009 to 2010, USGS measured incoming and outgoing sediment loads to Kanopolis and computed a suspended sediment trapping efficiency of 95.0 percent (Juracek, 2011). Once the bedload is factored into this, assuming a 100 percent trapping efficiency, the overall trapping efficiency becomes 95.6%.

In 2017, the multipurpose pool capacity can be estimated at 46,916 ac-ft. The mean annual water inflow for 1970-2019 was 139,271 ac-ft. Brune offers the three curves shown in Figure 5-1 for computing trapping efficiency, which can be estimated using Equations 1 and 2 and the constants given in Table 5-1 (Brune, 1953).

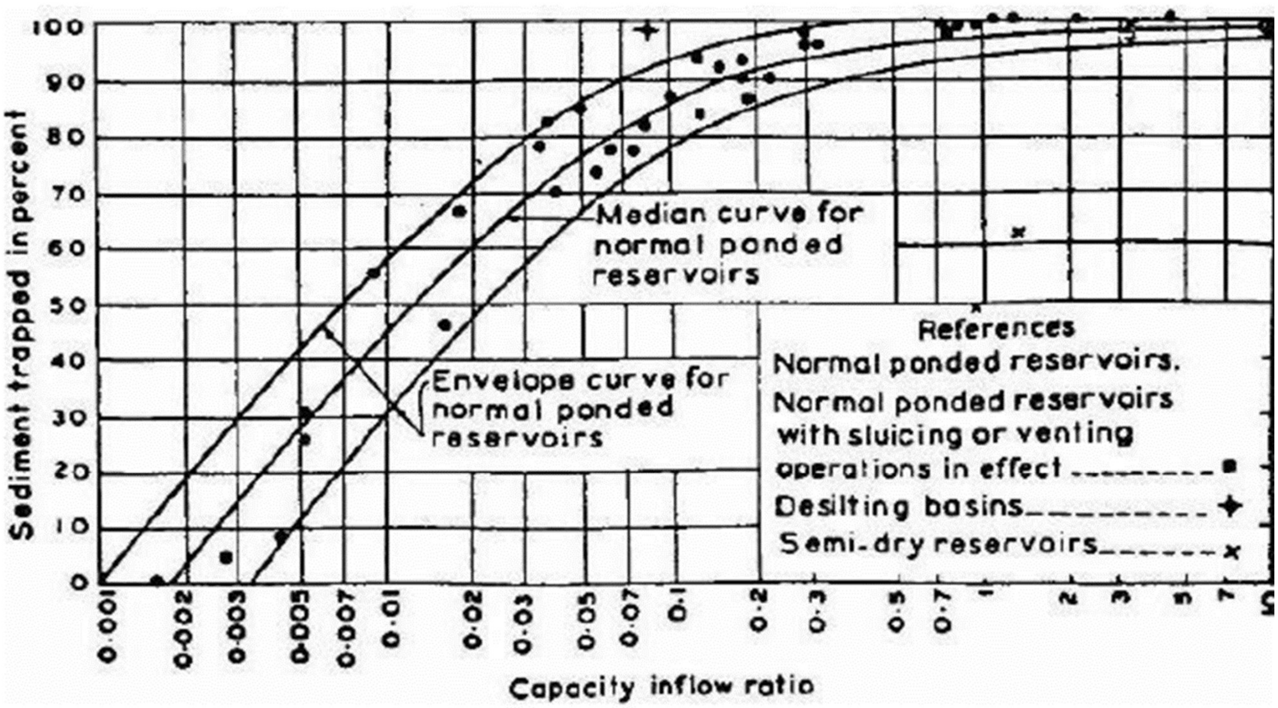


Figure 5-1: Brune Curves (Brune, 1953)

$$TE = a[1 - 2e^{-bV_*^{0.35}}] \quad (1)$$

$$V_* = \frac{V_{res}}{V_{inflow}} \quad (2)$$

Where:

V_{res} = volume from the multi-purpose pool for Perry Lake (200,004 ac-ft)

V_{inflow} = average of the daily inflow converted into volume

Table 5-1: Constants to be used in Equations 1 and 2

Constant	Low	Medium	High
a	95	97	100
b	5.37	6.42	7.71

Of the three, the medium curve estimates the closest trapping efficiency to that measured by USGS. The Brune estimate is 94.07%, or 1.59% lower than the USGS estimate.

6.0 DEPOSITIONAL VOLUME

Historically sedimentation rangelines have been used to calculate the capacity of the reservoir and to determine sediment deposition. Kanopolis Lake has a total of seventeen rangelines spaced at varying distances as shown in Table 6-1. Reservoir capacity was calculated from rangeline surveys in 1948, 1960, 1972, 1982, and 1993.

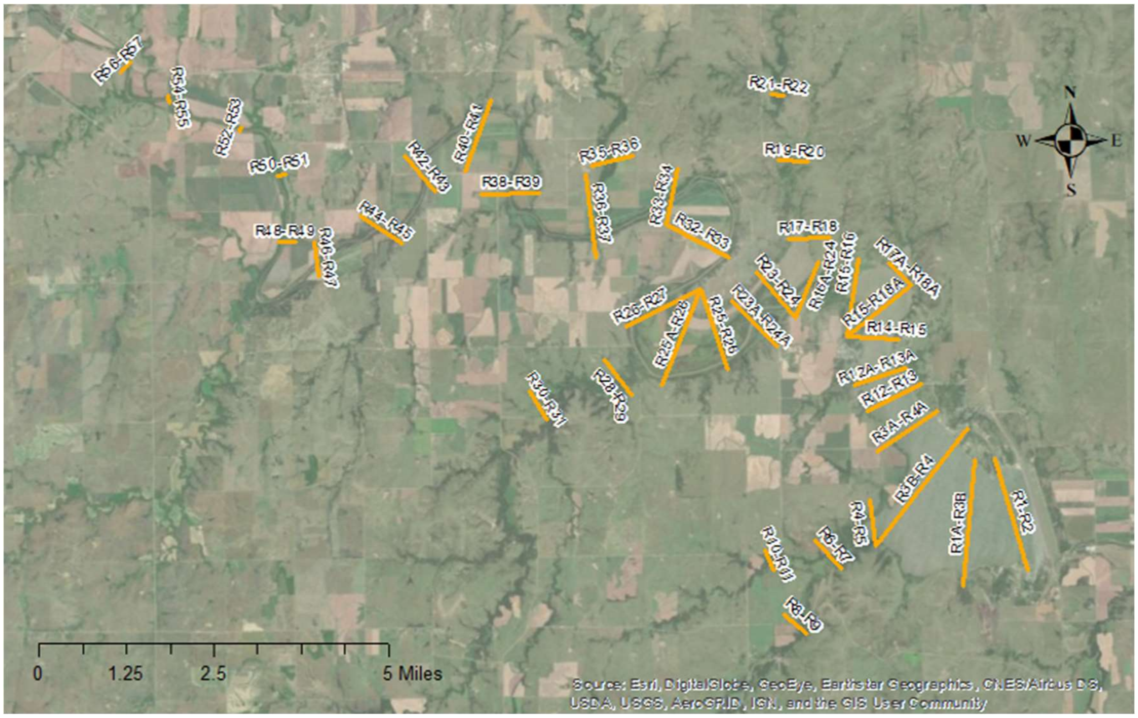


Figure 6-1: Kanopolis Sedimentation Rangelines

Bathymetric surveys were conducted on Kanopolis in 2007 and 2017 (Figure 6-2) using single-beam sonar. The 2017 survey was collected along transect lines spaced at 200 feet while the 2007 data was collected using the pattern shown in Figure 6-2.

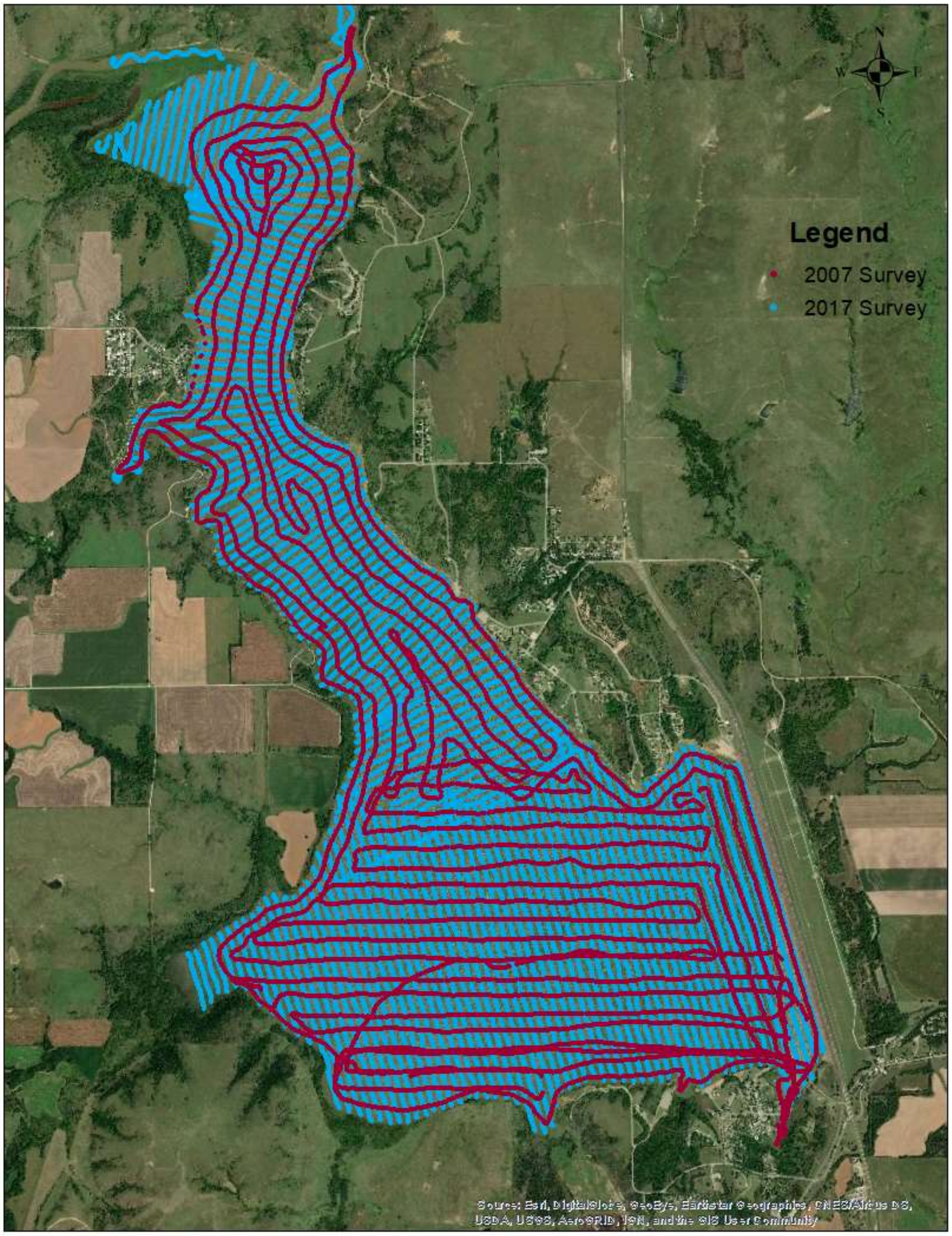


Figure 6-2: Bathymetric survey data points in 2007 and 2017

Table 6-1 provides the multi-purpose and flood control pool volumes computed from each survey along with the survey methodology. The flood control pool volume does not include the multipurpose pool or surcharge volumes. The values in this table were taken from the 2011 Surdex report.

Table 6-1: Pool Volumes over Time

Year	Multipurpose Pool Volume (ac-ft)	Flood Control Pool Volume (ac-ft)	Data Type
1948	73,200	447,091	Sedimentation Rangelines
1960	61,382	432,880	Sedimentation Rangelines
1972	55,784	425,758	Sedimentation Rangelines

1982	54,502	424,263	Sedimentation Rangelines
1993	50,273	418,387	Sedimentation Rangelines
2007	48,378	413,521	2007 Single beam sonar, 2010 LIDAR
2017	46,916	-	2017 Single beam sonar, 2018 LIDAR

Table 6-2 gives the amount of sediment deposition in the reservoir calculated by subtracting the pool volumes measured from the surveys. The yearly deposition varies significantly between the surveys. The flow/load relationship appears to be unchanged with time and the difference in deposition is caused by differing amounts of runoff between the survey years. The 1948-1961 period had the highest amount of deposition. In Table 6-2, Flood pool (FP) deposition indicates deposition at elevations higher than the multipurpose pool but lower than the top of flood pool.

Table 6-2: Deposition amounts (ac-ft)

Years	Deposition	Deposition FP	Average Annual Deposition
1948-1961	11,909	2,136	1,080
1961-1971	5,507	1,781	729
1971-1982	1,282	213	136
1982-1993	4,229	1,647	534
1993-2007	1,895	2,971	348
2007-2017	1,462	470	168

From 1948 to 2017, the multipurpose pool lost 26,284 ac-ft of storage to sedimentation. This represents 35.6% of the original multipurpose pool volume. The average annual rate of loss was 378 ac-ft/year or 0.52% of the original volume/year.

From 1948 to 2007, the flood control pool lost 8,748 ac-ft of storage to sedimentation. This represents 2.34% of the original flood control pool volume. The average annual rate of loss was 147 ac-ft/year or 0.04% of the original volume/year.

7.0 INCOMING SEDIMENT LOADS

USACE and USGS have sporadically collected paired flow/sediment concentration measurements on the Smoky Hill River since 1942 at USGS gage 06864500 at Ellsworth, KS. Most of the data was collected daily from 1945 to 1965 with flow data in the form of mean daily discharge. From 1979 to 2010 the USGS collected additional measurements that also included instantaneous flow. The daily load was calculated by multiplying the flow by the suspended sediment concentration and a factor of 0.0027 to obtain the load in tons per day. These measurements are shown in Figure 7-1 broken into the time periods in which they were collected. There appears to be no clearly defined trend in the data over time. Because it is the most recent data and was collected with instantaneous flow data, the measurements from 1979 to 2010 were used in this analysis.

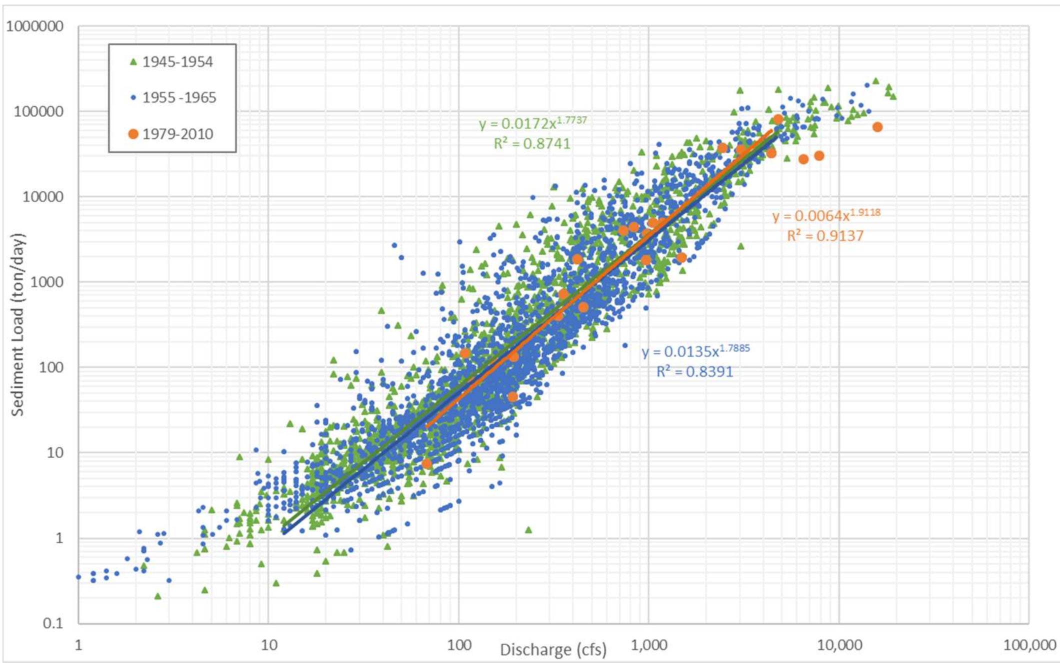


Figure 7-1: Sediment loads computed at the Ellsworth, Gage

Gage data from various gages within the Kansas River watershed indicate that the sediment rating curves generally begin flattening out at around the 83.3 to 66.7% (1/1.2 to 1/1.5) annual exceedance probability (AEP) discharge. The rating curve no longer follows the power fit trendlines when the discharge exceeds these values. These flows correspond with the typical range for bankfull flow in Kansas (Shelley 2012). The 1/1.2 and 1/1.5 AEPs for the Smoky Hill River at Ellsworth are 2,680 and 4,760 cfs respectively. It was assumed that the flatted portion of the rating curve begins at the 66.7% AEP event. Only the data for flows below the rating curves begins flattening was used to create the power fit trendlines. The new slope at the higher end of the rating curve was determined through calibration, as explained later in this appendix. Duan's (1983) correction for bias introduced by the log transform was applied to the portion of the rating curve that follows the power fit trendline.

The USGS also periodically measures the suspended sediment gradations of the samples they collect. These were plotted versus the discharge when they were collected to detect trends in the sediment size with discharge. Only measurements with the full gradation measured were used in these analyses.

Gradations of the suspended sediment indicates that there is some correlation between sediment gradation and discharge. Linear trendlines were fitted to the data as shown in Figure 7-2. The percentage of clay and silt declines as flow increases while the percentage of sand increases. However, there is a significant amount of scatter in the data with low R-squared value. The sediment gradation was assumed to remain constant for flows higher than 10,000 cfs due to the limited data at high flows.

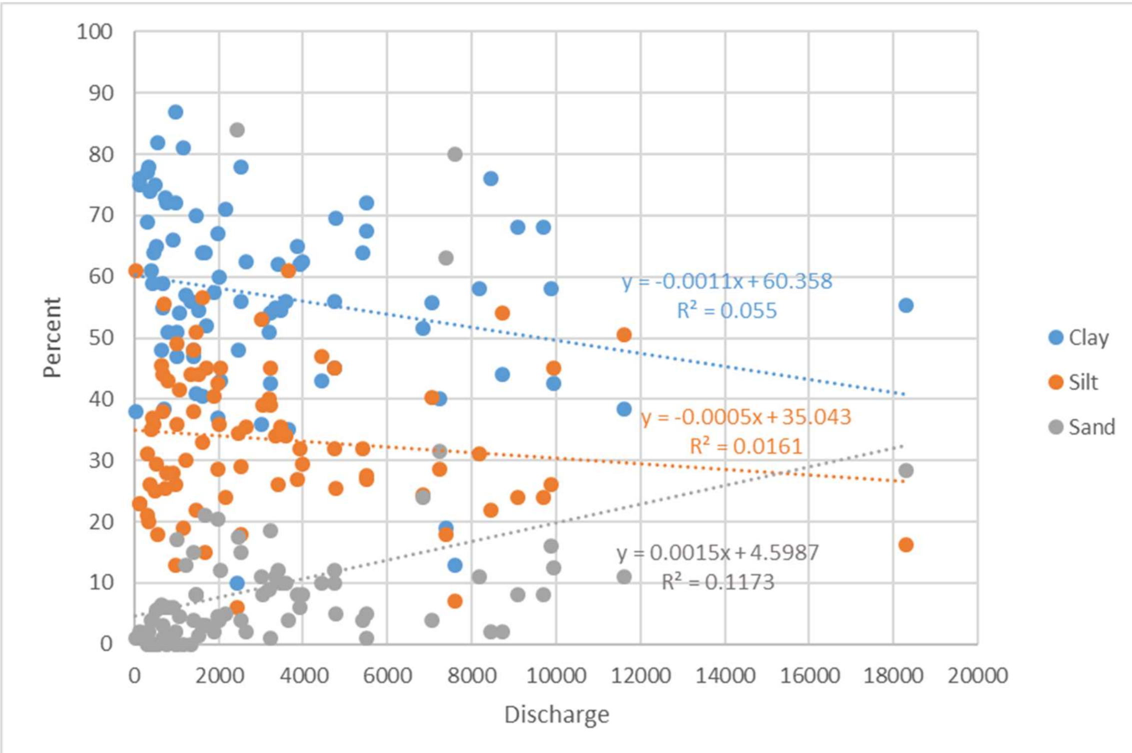


Figure 7-2: Suspended Sediment Size vs Discharge at the Ellsworth Gage with Flowrate

The suspended sediment measurements were investigated to see if there is a difference between the seasons of the year. Figure 7-3 shows the sediment measurements plotted by season and indicates there may be some variability with season. Spring and summer appear to have a noticeably higher sediment load for a given discharge then do fall and winter. Winter appears to have the lowest sediment load of all the seasons.

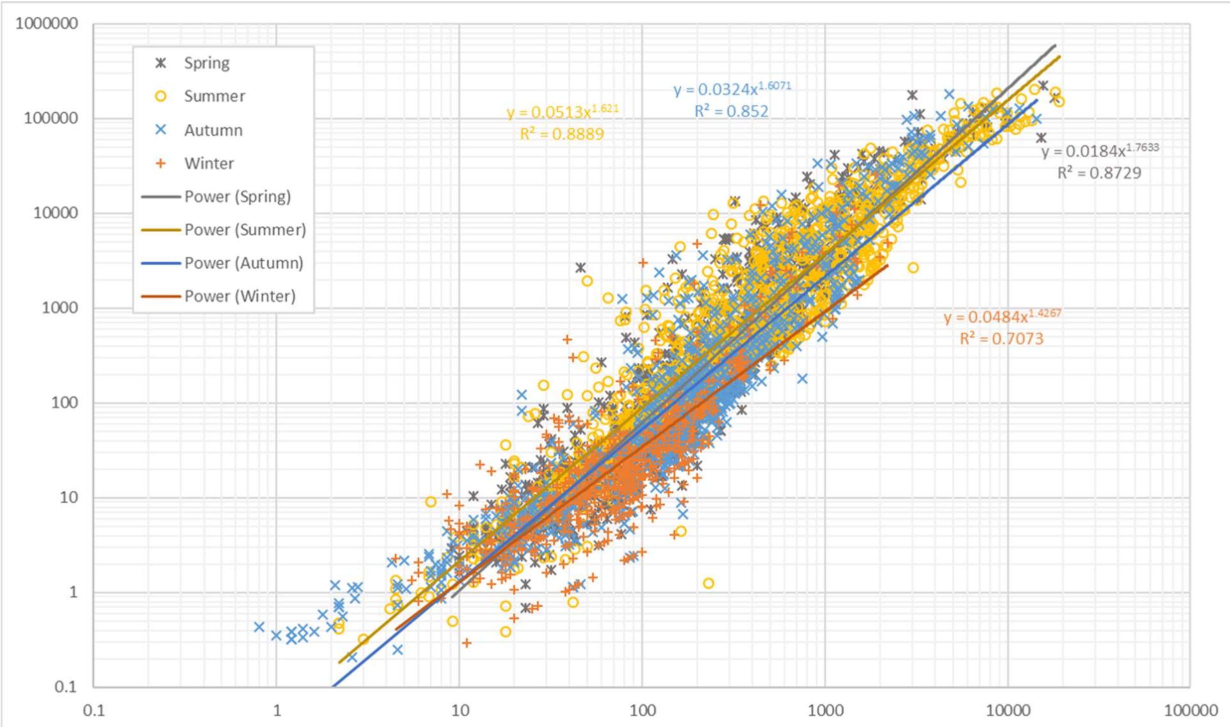


Figure 7-3: Suspended Sediment Load Plotted by Season

There also appears to be some hysteresis in sediment load over the course of a hydrograph. Figure 7-4 shows the flow and load hydrographs for the 1951 flood, which had the fifth highest peak flow rate for the Ellsworth gage with records dating back to 1895. The sediment loads at the beginning of the event appear to be greater than later in the event.

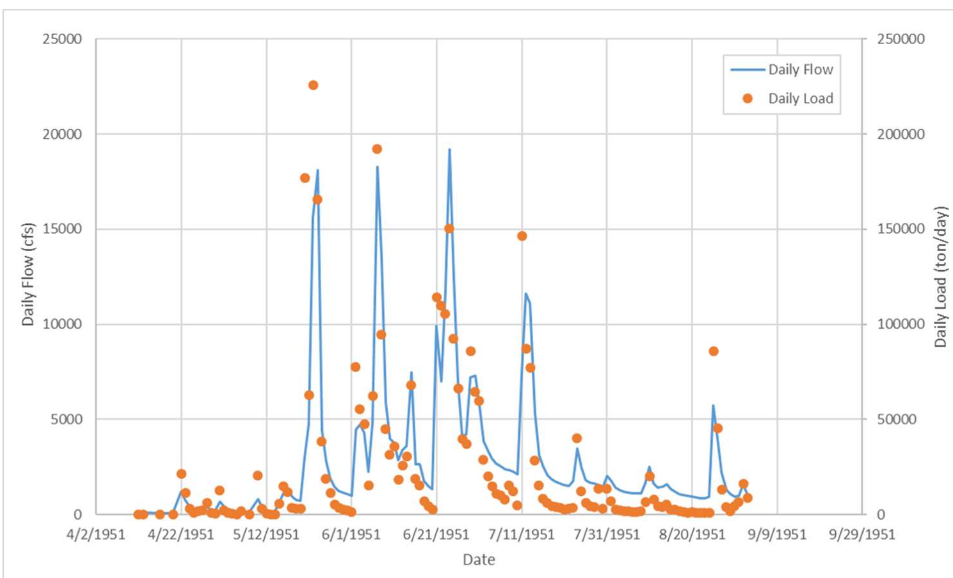


Figure 7-4: Daily flow and sediment loads for the 1951 flood of record

Figure 7-5 shows the daily sediment load plotted versus daily discharge for the May 1951 event. Sediment load on the rising limb of the event is higher than on the receding limb, indicating hysteresis in the suspended sediment load.

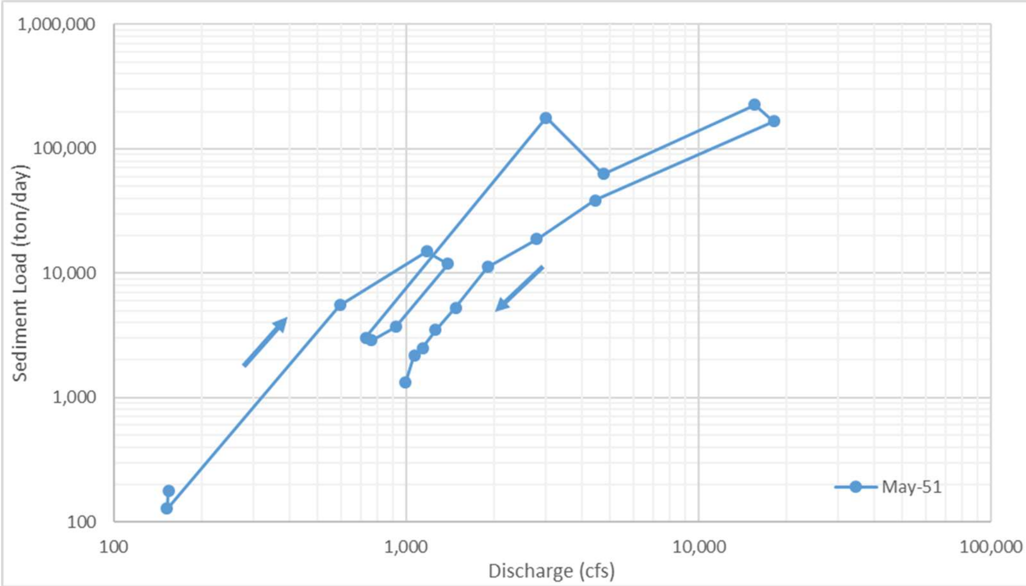


Figure 7-5: Suspended Sediment Load vs. Discharge Showing the Hysteresis at the Ellsworth Gage

Measurement taken from nine major flow events were used to compare sediment loads on the rising limb versus the falling limb of the hydrograph as shown in Figure 7-6. As can be seen, there is a noticeable difference between the rising and falling limbs. Figure 7-3 through Figure 7-6 indicate that the system is supply limited.

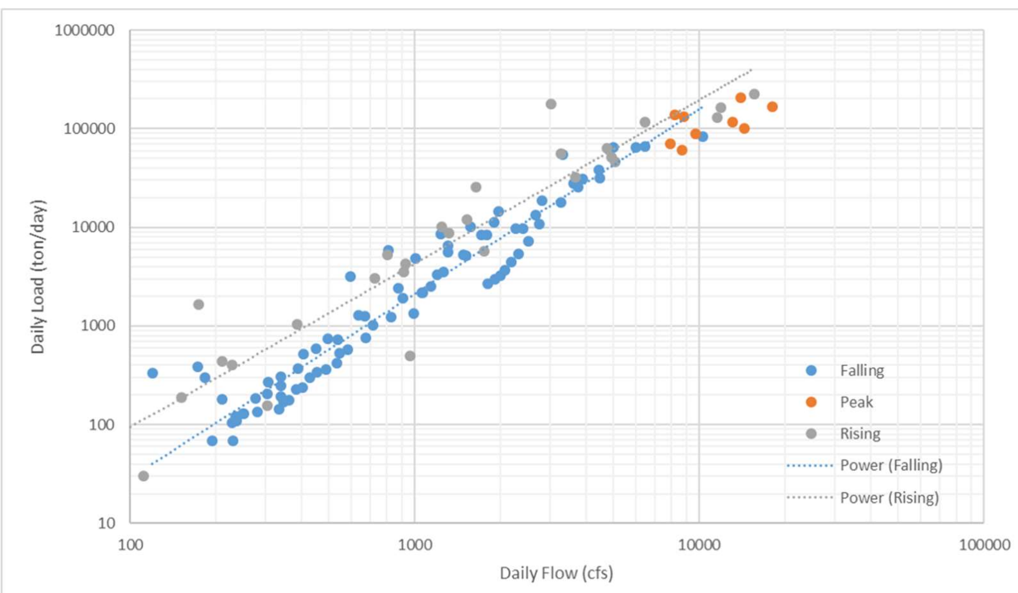


Figure 7-6: Suspended Sediment Measurements on the Rising Limb, Falling Limb, and Peak of Multiple Flood Events

In a 1972 report on sedimentation in Kanapolis Lake, NWK estimated the bed load based on bed material sediment gradation, hydraulic parameters, and the Shield's Bedload Equation (USACE, 1972). Table 7-1 shows the results for water years 1947 through 1971 along with the suspended load. The percentage of the bedload to the suspended load was estimated to be approximately 15.4% from the values in this table.

Table 7-1: Suspended and Bed Loads Estimated in 1972 Sedimentation Report

Water Year	Suspended Load	Bed Load
1947	1,080,000	133,600
1948	397,400	39,200
1949	734,000	138,200
1950	1,902,000	184,200
1951	3,947,000	744,200
1952	154,000	35,600
1953	136,000	15,100
1954	167,000	15,100
1955	478,000	32,000
1956	122,000	2,000
1957	1,816,000	340,000
1958	1,046,000	280,000
1959	724,000	60,000
1960	1,329,000	300,000
1961	1,052,400	280,000
1962	835,000	140,000
1963	161,600	15,000
1964	197,900	10,000
1965	706,000	80,000
1966	550,700	30,000
1967	1,140,000	140,000
1968	215,000	10,000
1969	1,088,000	160,000
1970	530,000	25,000
1971	580,000	30,000

A total load rating curve was developed through the following steps:

1. Estimate the best fit regression line of the form $Q_s = aQ^b$ using log-log linear regression.
2. Correct for bias using the Duan correction factor (Duan, 1983). The Duan E value was 1.35.
3. Add 15.4% to account for bed load to create a total load rating curve.
4. Multiply by 1.14 to account for the ungauged drainage area above the dam. This was determined by dividing the unregulated drainage area above the dam (2,330 square mile) by the unregulated drainage area above the Ellsworth gage (2,050 square miles).
5. Using the measured data, estimate the percentages of clay, silt, and sand/gravel.
6. Apply this rating curve to daily flow rates from 1948 to 2019 to determine the cumulative mass of sediment entering the reservoir.
7. Multiply by the appropriate percentages to determine the cumulative mass of clay, silt, and sand entering the lake.

This analysis found the incoming load to be 48.2% clay, 28.5% silt, and 23.3% sand/gravel. Table 7-2 summarizes the results for 1948 to 2019.

Table 7-2: Incoming Sediment to Kanopolis Lake from 1948 to 2019

Parameter	Value
Years	1948 - 2019
Total Incoming Sediment (tons)	49,430,068
Total Incoming Clay Fraction	48.2 %
Total Incoming Silt Fraction	28.5 %

Total Incoming Sand Fraction	23.3 %

8.0 BULK DENSITY AND CONSOLIDATION OF SEDIMENT DEPOSITS

Tests in 1971 using a density probe indicated bulk densities of 30.5 to 85.1 pounds per cubic foot (USACE, 1972). These measurements are given in Table 8-1. There appears to be a general increase in the density with depth into the sediment deposits, indicating that older deposits had consolidated over time. There also appeared to be an increase in density moving upstream in the reservoir. Samples collected near range lines 14-15 and 18A-15 appear to be located within the 1971 multipurpose pool, while the others are likely in the flood pool. A multipurpose pool bulk density of 38.7 was obtained by averaging the samples in the multipurpose pool, which was determined to be samples from rangelines 15-16, 18A-15, and 14-15. However, because these three rangelines are all in the upper portion of the multipurpose pool, they may not be representative of the overall multipurpose pool bulk density. A flood pool bulk density of 53.8 was estimated by averaging the rest of the samples.

Table 8-1: 1971 bulk density measurements

Sample	Density (lbs/cu ft)	Depth (ft)	Range Line
1	31.1	3.1	15-16
1	50.1	3.6	15-16
1	36.9	5.6	15-16
1	42.7	7.6	15-16
1	55.2	9.6	15-16
1	49.8	10.1	15-16
2	33.1	2.1	16A-24
2	40.6	4.1	16A-24
2	37.9	6.1	16A-24
2	42.4	7.1	16A-24
3	44.3	2	16A-24
3	41.3	4	16A-24
3	42.4	6	16A-24
3	49.6	8	16A-24
4	51.6	2	16A-24
4	55.2	4	16A-24
5	60.4	2	16A-24
6	59.3	2.3	16A-24
6	53.3	3.3	16A-24
7	85.1	2	24-23
7	54.8	4	24-23
8	38.7	3	18A-15
8	30.8	5	18A-15
8	34.2	7	18A-15
8	36.6	9	18A-15
8	38.5	10	18A-15
9	31	2	14-15
9	30.5	4	14-15
9	35.2	6	14-15
9	38.5	8	14-15
9	37.7	10	14-15

Sample	Density (lbs/cu ft)	Depth (ft)	Range Line
9	38.5	12	14-15
9	41	13.5	14-15

Bulk density was also measured in 1971 from physical samples; mainly taken from the flood pool. These have an average bulk density of 75.1 pcf, which is substantially higher than the values from Table 8-1. These measurements are likely more accurate than the density probe measurements, so the value of 75.1 pcf was used for the flood pool bulk density for the rest of this analysis.

The incoming load can also provide an estimate of bulk density, via Equation 1.

$$\gamma_c = \frac{1.0}{\left(\left(\frac{F}{\gamma}\right)_{clay} + \left(\frac{F}{\gamma}\right)_{silt} + \left(\frac{F}{\gamma}\right)_{sand}\right)} \qquad (1)$$

where γ_c is the composite bulk density

F is the fraction of clay, silt, or sand

γ for clay, silt and sand is assumed to be 30 pcf, 65 pcf, and 93 pcf respectively.

By equation 1, the bulk density of the incoming load is 43.8 pounds per cubic foot. This compares well with the overall average of the measurements in Table 8-1, which is 43.9.

The multipurpose pool bulk density was also calculated using Equation 1 and gradation from the sediment cores given Section 8. The resulting bulk density is 39.7 pcf, which is the same determined from the 1971 bulk density measurements.

9.0 CALIBRATING LOAD AND DEPOSITIONAL PARAMETERS

Sufficient data exists to calibrate the rating curve by comparing the deposition computed using the incoming sediment loads to the deposition computed using the surveyed volumes. This was accomplished following these steps:

1. Determine the trapping efficiency of the reservoir (in this case 95.5%).
2. Apply daily flows at the Ellsworth gages to its sediment rating curve.
3. Apportion the deposition into the multipurpose pool or the flood control pool. Multiply the deposition found in Step 2 by a factor, *m* such that the ratio of MPP deposition to total deposition is correct per the survey analysis. *m* was computed from the surveyed deposition and the bulk densities determined for the MPP and FP.
4. Repeat steps 2 through 3 for each day over a period of time to obtain the cumulative sediment inflow.
5. Compute the mass of trapped sediment in the multipurpose pool by applying the trapping efficiency to the incoming sediment. The flood pool deposits were assumed to have a 100% trapping efficiency.
6. Transform the trapped mass to a deposited volume by using the bulk densities determined in Section 9 (37.8 pcf in the MPP and 75.1 pcf in the FP.)
7. Compare the total rating-curve-based deposition to the deposition calculated from the surveyed volumes.
8. Adjust the sediment rating curve (described below) to more closely match the surveyed deposition

Table 9-1 summarizes this analysis.

Table 9-1: Parameters used in estimating deposition into Kanopolis

Parameter	Initial
FP Bulk Density (lb/cu ft)	75.1
MPP Bulk Density (lb/cu ft)	38.7
Correction for Ungauged Area	1.14
Bedload % Suspended	15%
a in $Q_s = aQ^b$	0.017
b in $Q_s = aQ^b$	1.76
Duan E	1.23
Average trapping efficiency	95.7%

Figure 9-1 shows the final total load rating curve along with the suspended sediment measurements taken at the Ellsworth gage. As mentioned earlier, the rating curve begins flattening at the 66.7% (1/1.5) AEP flow rate.

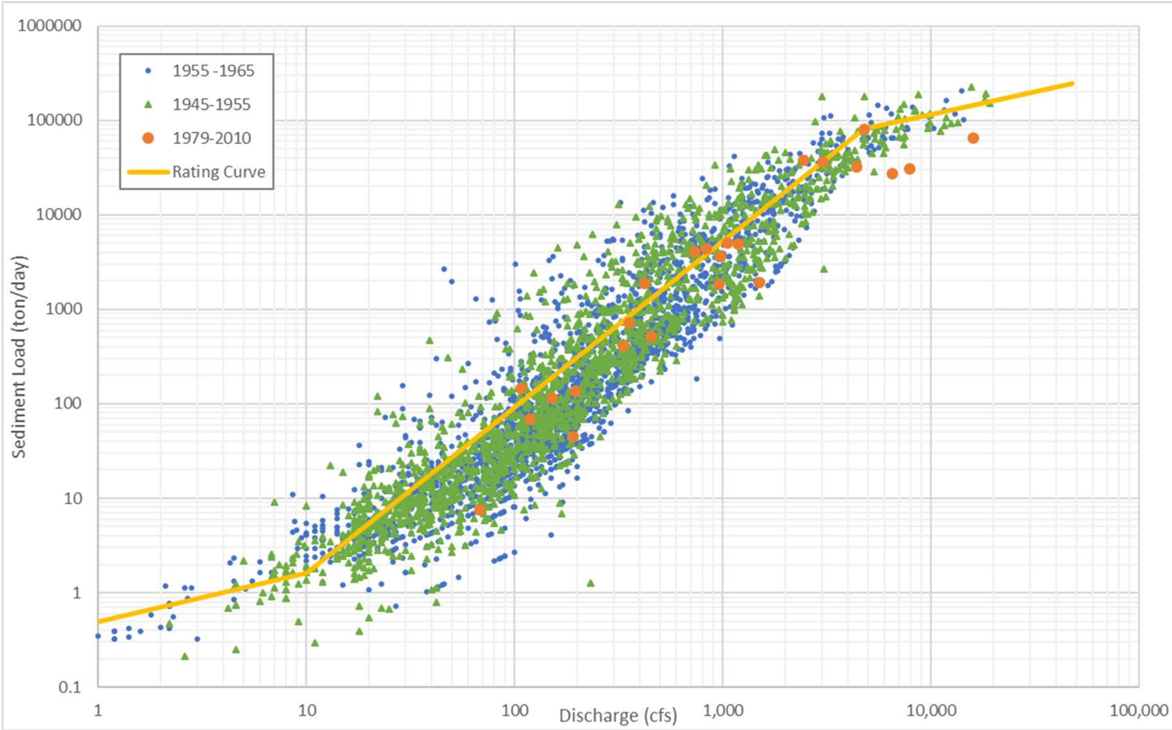


Figure 9-1: Final Total Load Rating Curve and Suspended Sediment Measurements at the Ellsworth Gage

The computed and the surveyed volumes of deposited sediment between each survey are shown in Table 9-2. Also shown in the table is the number of days that the daily flow exceeded the 1/1.5 ACE flow for each time period. The computed deposition generally matches well with the surveyed values. There are several time periods where the surveyed and computed deposition differ significantly, particularly 1971-1982. However, this could be due to errors in the surveyed deposition rather than the rating curve computations. The total deposition was computed by comparing pool volumes in 1946 to pool volumes in 2007, then adding the 2007 to 2017 MPP change and an estimate for the 2007 to 2017 FP change.

Table 9-2: Surveyed and computed volume of sediment deposits

Period	Computed Deposition (ac-ft)	Surveyed Deposition (ac-ft)	Computed / Surveyed	# of Days > 4760 cfs	Days Per Year > 4760 cfs
1946-1961	14,451	13,576	1.03	61	4.9
1961-1971	6,272	5,861	0.86	26	2.4
1971-1982	5,934	1,495	3.97	24	2.1
1982-1993	5,021	4,834	0.85	25	2.3
1993-2007	3,968	3,686	0.82	18	1.3
2007-2017*	1,257	1,462	0.86	0	0.0
1946-2017**	37,365	35,204	1.05	154	2.2

*Computed and Surveyed volumes for this time period are for the multipurpose pool since the flood control pool was not surveyed

**Flood control pool volume for 2017 was estimated based on the average ratio between MPP and FCP volumes from 1946-2007

The 2007-2017 time period was chosen for calibration as this is the most recent time period. For this time period the computed deposition is 14% lower than the surveyed deposition, which could be caused by a number of factors. In the computations, the bedload estimate probably has the greatest uncertainty and is the most likely source of the error. However, other factors that could be causing the computed deposition to be high are the correction for the ungauged drainage area, the bulk density estimates, and the estimate for the percent of mass that deposits in the flood pool. To account for this unknown source of error, the total load rating curve was calibrated to the 2007-2017 deposition by dividing by a factor of 0.86. Calibrated values of deposition can be seen in Table 9-3.

Table 9-3: Surveyed and calibrated computed volume of sediment deposits

Period	Computed Deposition (ac-ft)	Surveyed Deposition (ac-ft)	Computed/Surveyed
1946-1961	16,815	13,576	1.20
1961-1971	7,298	5,861	1.00
1971-1982	6,904	1,495	4.62
1982-1993	5,842	4,834	0.99
1993-2007	4,617	3,686	0.95
2007-2017*	1,462	1,462	1.00
1946-2017**	35,922	35,097	1.22

*Computed and Surveyed volumes for this time period are for the multipurpose pool since the flood control pool was not surveyed

**Flood control pool volume for 2017 was estimated based on the average ratio between MPP and FCP volumes from 1946-2007

10.0 SEDIMENT TRAPPING DURING FLOOD CONTROL VS. MULTIPURPOSE POOL OPERATIONS

The calculated sediment inflows into Kanopolis Lake were used to estimate the incoming sediment based on when the lake was in flood control operations (i.e., with a water surface above the multipurpose pool elevation), vs. multipurpose pool operations (i.e., when the water surface is at or below the multipurpose pool elevation). Table 10-1 shows the results of this analysis, which indicates that most of the sediment enters the lake during flood control operations. The quantities shown are the volumes that eventually deposit (i.e., the trapping efficiency has already been applied.)

Table 10-1: Deposition Amounts during Flood Control and Multipurpose Operations

Deposition	Deposition 1969 - 2019 (ac-ft)	Percent
Flood control Operations	24,364	97.0 %
Multipurpose Operations	766	3.0 %
Total Deposition	25,131	100.0 %

Lake levels vary throughout the year depending on a variety of factors. However, lake managers generally try to operate the lake so that pool elevations match the Water Level Management Plan (WLMP) shown in Figure 10-1.

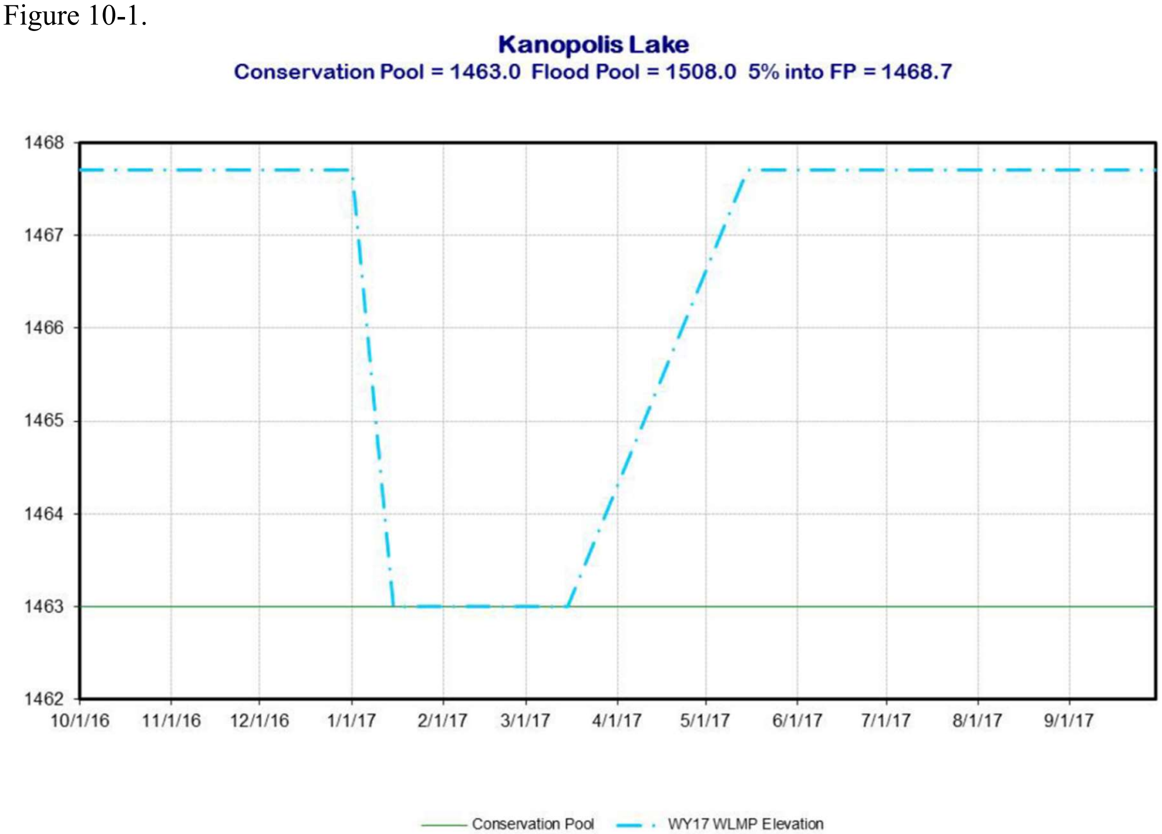


Figure 10-1: Kanopolis Lake Water Level Management Plan. Elevations in NVGD29.

The quantity of incoming sediment occurring when the pool elevation is above or below the WLMP line shown in Figure 10-1 is given in Table 10-2.

Table 10-2: Deposition Amounts when Lake Elevation is Above or Below the WLMP

Deposition	Deposition 1969 - 2019 (ac-ft)	Percent
Deposition While Above WLMP	19,377	77.1%
Deposition While Below WLMP	5,753	22.9 %
Total	25,131	100.0 %

By either computation, it is clear that most of the sediment enters the lake during flood control operations.

11.0 SEDIMENT CONCENTRATIONS

Figure 11-1 illustrates the concentration of incoming sediment, together with the 80% confidence intervals. This graph represents the range of natural variability in the sediment concentrations in the river, i.e., what the concentration would be if the dam were not in place. As a first approximation, the lower bound can be thought of as a minimum target for naturalizing downstream sediment levels, and the upper bound can be thought of as a maximum limit to avoid excessive sediment releases.

The flow/concentration relationship is not monotonic or with a consistent slope. Rather the relationships in log space at low flows, moderate flows, and high flows exhibit separate slopes. By observation, at many of the lakes the sediment concentrations actually reverse at higher flows. This behavior translates into flatter flow-low curves as described earlier in the document. While the reasons for this phenomenon are unknown, it could be can be explained by either the supply limitation of easily erodible sediments or by sediment lost to the floodplain during overbank flows. A fourth-order polynomials through log-transformed data were used to reflect the overall trends in the data. This curve is a valid fit over the range of observed data but should not be used for extrapolation.

Also evident in the data is a reduction in the variability in concentration at higher flows. While fewer measurements could by itself lead to the perception of less variability, visual inspection of the flow/concentration measurements suggests that physical reasons may drive the lower variability. The first possible reason for the reduction in variability is the supply limitation that drives the reduction in concentration. Higher concentrations are constrained by lack of readily available material. A second explanation for lower variability is that moderate flows can be achieved by a precipitation in only part of the watershed, and different subwatersheds may have different sediment contributions. On the other hand, very high flows are only achievable when most of if not all the entire watershed contributes, which reduces the spatial variability based on storm placement.

The confidence intervals were originally computed based on the statistics of the total sample and departure from the best-fit polynomial. However, the height of the intervals was driven by the high variability at moderate flows, which yielded confidence intervals at the highest flows that exceeded all the measured data points. A more refined approach was taken to take into account differences in variability as a function of flow.

The 4th-degree polynomial through log-transformed data was used as a predictor for the mean of a normal distribution in a generalized additive model. To capture the changing variance of concentration with respect to flow, a concave-down function was needed to predict the sigma term in order to produce a local maximum in the middle with decreases in variance at both extremes. A quadratic function was used to ensure $d2\sigma/dx2<0,\forall x$. The result is a six-parameter model (plus two intercepts) in a hierarchical structure.

The interpretation should be looked at as "the conditional distribution of suspended sediment concentration, given an amount of flow." That is, $f(y|x)=N(\mu=b4x4+b3x3+b2x2+b1x+b0, \sigma=a2x2+a1x+a0)$ where y is SSC, x is flow, and N is the normal distribution with parameters μ and σ. Because the conditional distribution is normal, the best estimate of $y|x$ is $E[y|x]$ and is also the median of the conditional distribution, and it can be computed as μ. To obtain percentiles of the conditional distribution, the quantile function (inverse CDF) is used, $F\text{-}1(p;\mu,\sigma)$ where μ and σ are computed for the given value of flow. The result is overall a function that has a central tendency dictated by the 4th-order polynomial, with spread about the mean dictated by a concave-down quadratic.

These analyses were automated using R-scripts.

As seen in the Figure 11-1, the sediment concentration in the water during high inflow events is considerably higher than those during low flows. This sets a reasonable upper bound for sediment restoration activities to remain within the natural variability of the system. At the highest flows the concentrations decrease, suggesting a supply-limited system.

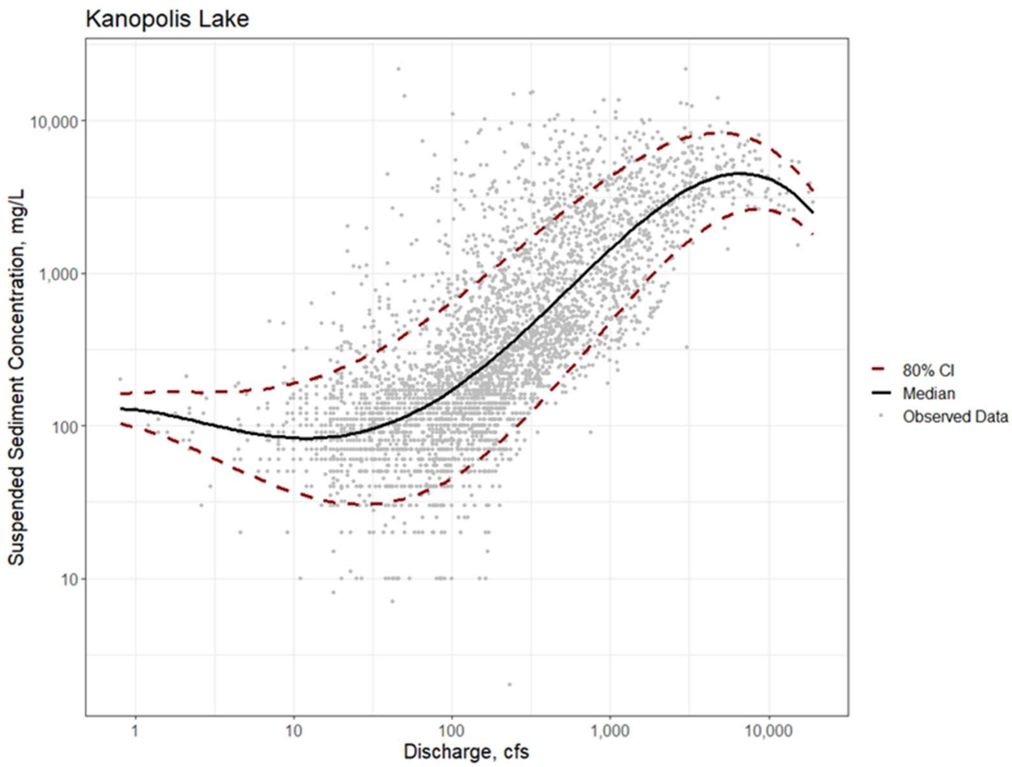


Figure 11-1: Incoming suspended sediment concentrations

Sediment concentration measurements have also been collected downstream of Kanopolis Dam at USGS gage 06865500. There appears to be no discernable trend in the concentration with flow rate. Both the measurements upstream and downstream of Kanopolis Dam can be seen in Figure 11-2.

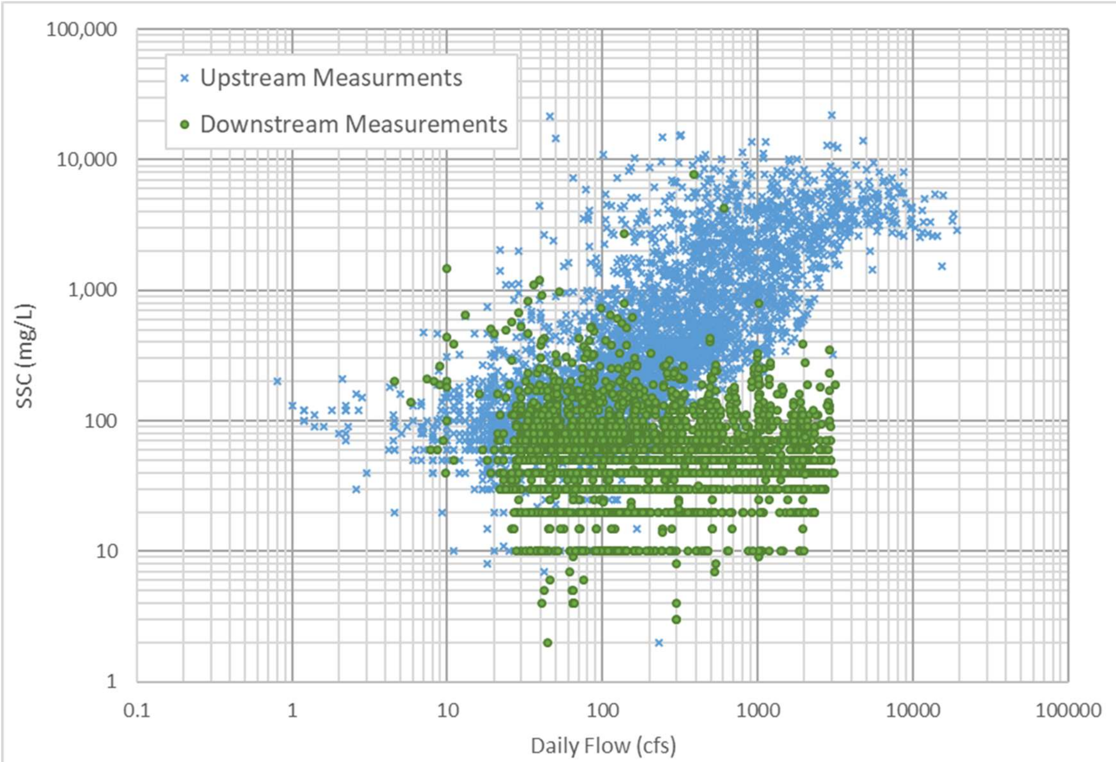


Figure 11-2: Sediment concentration collected upstream and downstream of Kanopolis Lake

12.0 RESERVOIR BED SEDIMENT COMPOSITION

Samples collected by the Kansas Biological Survey in 2008 indicate that the sediment deposits are 56% clay, 31% silt, and 13% sand. Figure 12-1 shows the particle sizes of the samples, which generally represent the top six inches of sediment. These results are somewhat finer than those given in Table 7-2, but this could be caused by the depth and locations where the samples were collected, which may not be representative of the overall deposition. Also, these are representative of the multipurpose pool deposits and not the overall sediment inflow.

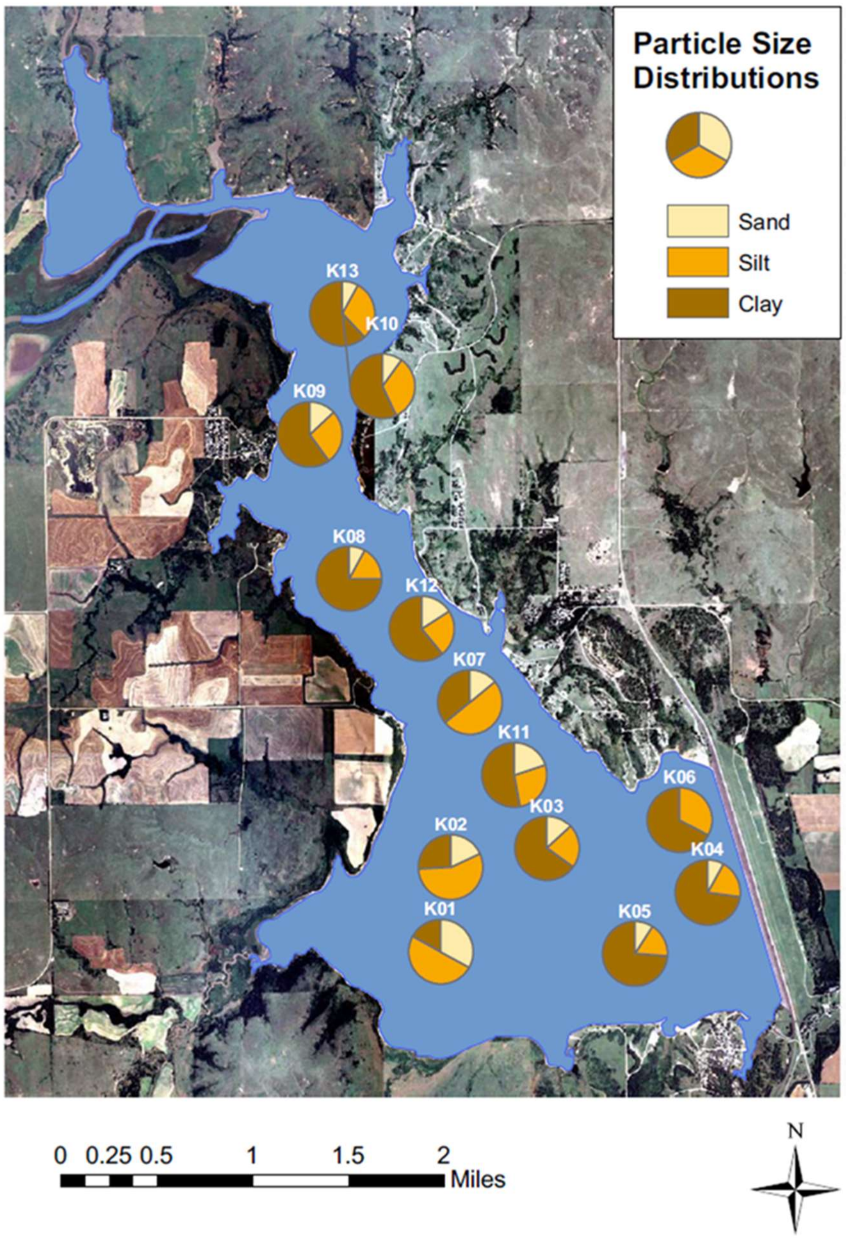


Figure 12-1: Sediment size gradation (Kansas Biological Survey, 2009)

Table 12-1 shows sediment size percentages from the sediment cores.

Table 12-1: Sediment Size Gradations from KBS Sediment Cores

Code	Sand	Silt	Clay
K01	33	50	17
K02	18	56	26
K03	13	22	65
K04	8	19	73
K05	9	17	74
K06	0	33	67
K07	14	50	36
K08	8	17	75
K09	13	27	60
K10	10	33	57
K11	20	27	53
K12	16	23	61
K13	8	30	62
Average	13	31	56

13.0 SEDIMENT CHEMICAL CONCENTRATIONS

No previous reports or information were found related to the chemical concentrations of the sediment deposited in Kanopolis.

14.0 DELTA LOCATION

Figure 14-1 provides a profile plot of centerline through the lake. At each location, the invert elevation (lowest elevation in a given sedimentation range line) is plotted. The locations of the sedimentation rangelines can be found in Figure 10-1. The 1946-1982 rangeline surveys were converted from NVGD29 to NAVD88 using values obtained from a conversion raster in Arc-GIS based on the USACE Corpscon conversion tool. As seen in Figure 14-1, the delta crest grew significantly immediately after dam closure.

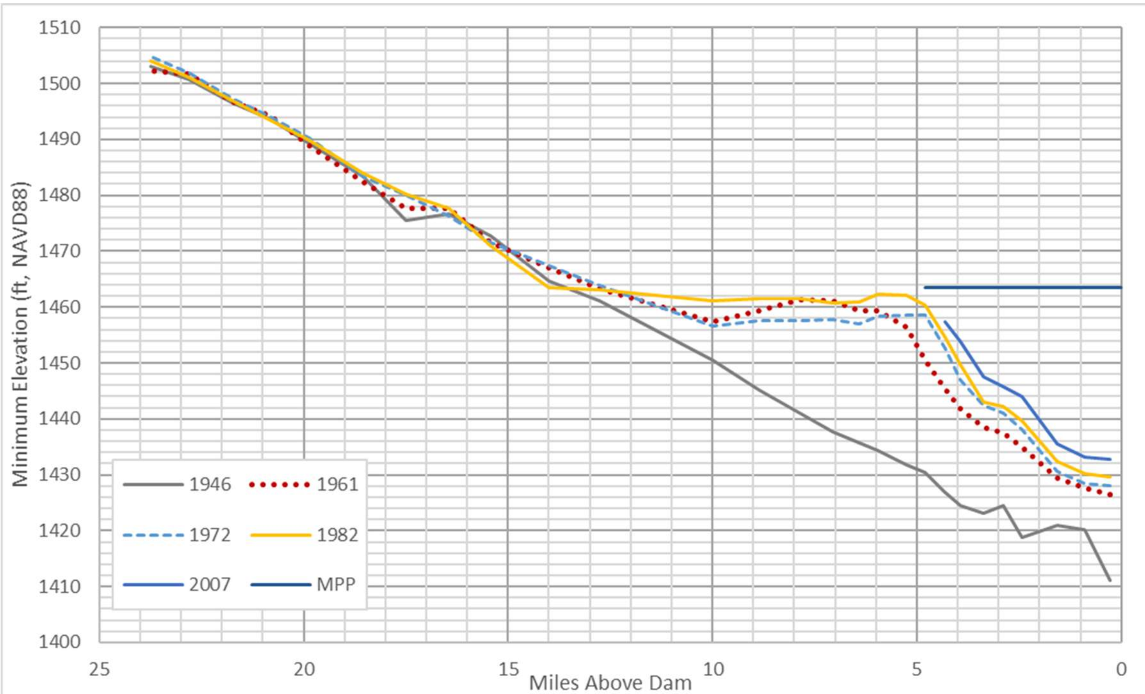


Figure 14-1: Profile of invert elevations indicating delta location and growth

A centerline through the lake was drawn in ArcGIS to compare the recent 2007 and 2017 bathymetric surveys with better resolution. As seen in Figure 14-2, the sediment deposition appears to be fairly uniform throughout the lake.

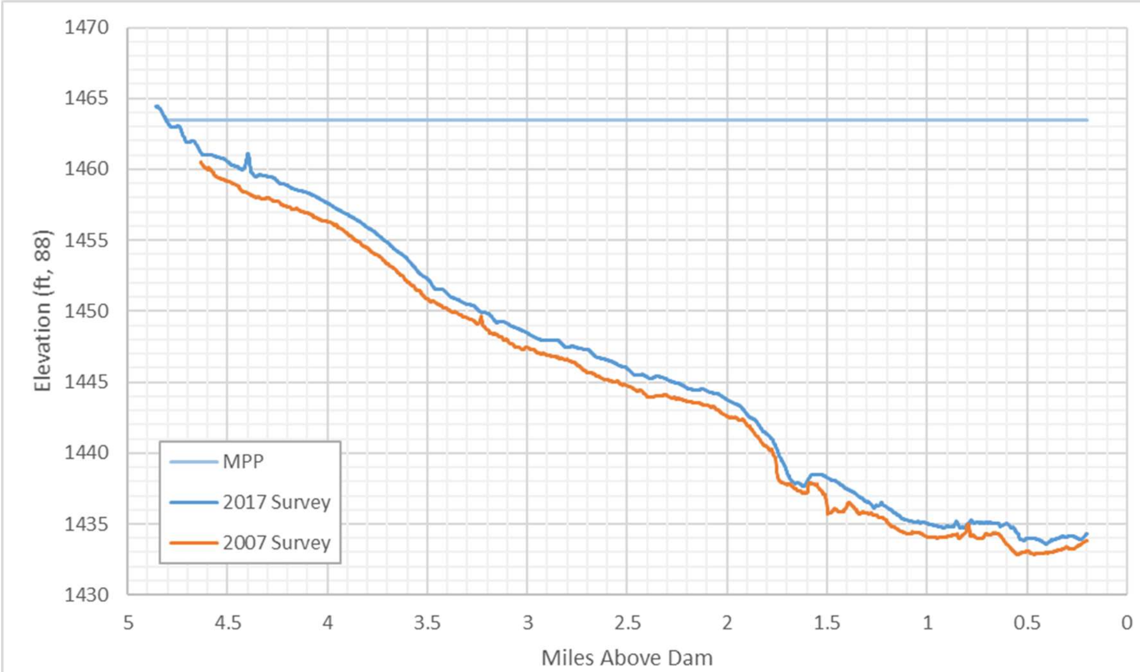


Figure 14-2: Minimum Elevations along Lake Centerline from the 2007 and 2017 Surveys

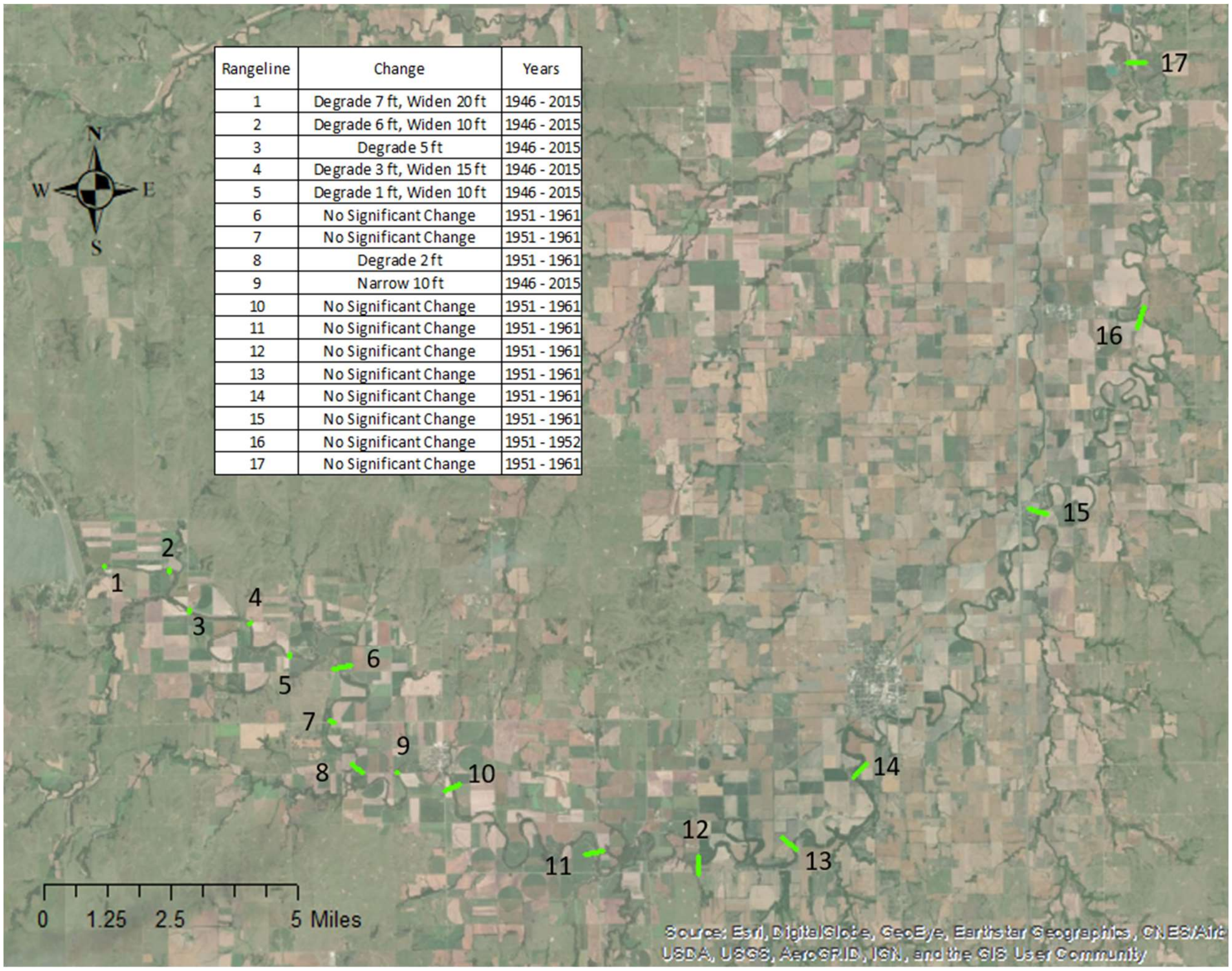
The delta progression rate was calculated between each of the surveys and is shown in Table 14-1. The table shows that there is significant variance in the delta migration rate that is likely caused by differences in survey methods as well as in sediment inflow rates due to flow differences.

Table 14-1: Delta Progression Rate

Year	Rate (ft/year)
1961-1972	316
1972-1982	98
1982-2007	95
2007-2017	472

15.0 DOWNSTREAM CHANNEL

Sediment trapping by dams very often induces bed degradation and bank erosion downstream. Degradation rangelines downstream from Kanopolis allow this effect to be quantified. Figure 15-1 shows the location of the rangelines, the total bed elevation change, and channel width change at each. All the degradation rangelines were available in the NVGD29 datum, so no conversion was necessary to calculate volume change.



Rangeline	Change	Years
1	Degrade 7 ft, Widen 20 ft	1946 - 2015
2	Degrade 6 ft, Widen 10 ft	1946 - 2015
3	Degrade 5 ft	1946 - 2015
4	Degrade 3 ft, Widen 15 ft	1946 - 2015
5	Degrade 1 ft, Widen 10 ft	1946 - 2015
6	No Significant Change	1951 - 1961
7	No Significant Change	1951 - 1961
8	Degrade 2 ft	1951 - 1961
9	Narrow 10 ft	1946 - 2015
10	No Significant Change	1951 - 1961
11	No Significant Change	1951 - 1961
12	No Significant Change	1951 - 1961
13	No Significant Change	1951 - 1961
14	No Significant Change	1951 - 1961
15	No Significant Change	1951 - 1961
16	No Significant Change	1951 - 1952
17	No Significant Change	1951 - 1961

Figure 15-1: Approximate locations of degradation rangelines downstream of Kanopolis

Figure 15-2 plots the cumulative volume change over time based on the degradation rangelines. As seen in Figure 29, the Smoky Hill River downstream of Kanopolis Lake exhibits continued degradation over time. If these rangelines are indicative of the whole reach from 0 to approximately 16 miles downstream of the dam, the bed and banks have lost 271 ac-ft of material from 1946 to 2015. This is 5% of the volume of sand that has deposited in the reservoir, which was estimated to be 5,248 ac-ft.

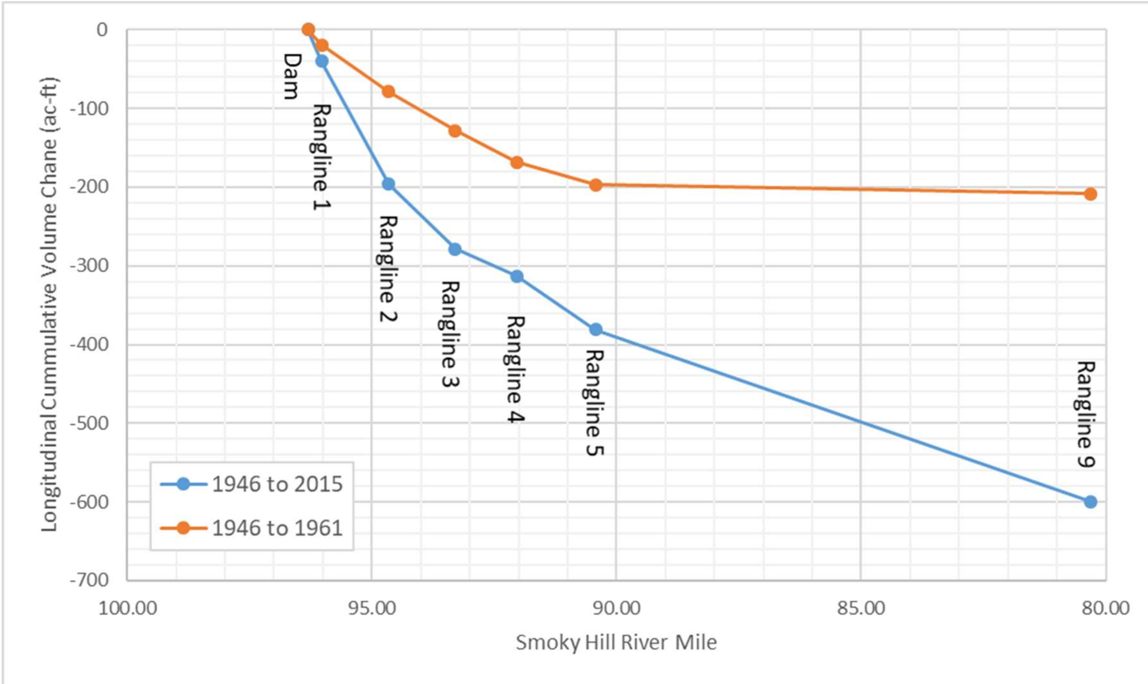


Figure 15-2: Cumulative Longitudinal Volume Change Downstream of Kanopolis

These analyses indicate that the Smoky Hill River downstream of Kanopolis is sediment starved. Continued degradation with associated bank erosion is expected if sediment trapping continues.

16.0 SUMMARY AND CONCLUSIONS

Sedimentation has had a significant impact on Kanopolis Lake through loss of storage capacity and impacts to infrastructure surrounding the lake. As of October 2017, the multipurpose pool had lost 26,030 ac-ft of storage capacity to sedimentation, or 35.6% of the original volume. Between 1948 and 2007 the flood pool lost 8,748 ac-ft of storage, which 2.34% of its original volume.

From 2009 to 2010 the USGS estimated the suspended sediment trapping efficiency of the reservoir to be 95.0% based on measured sediment inflows and outflows. This was adjusted to 95.67% to account for 100% trapping efficiency of the sand load. Sediment rating curves were created from suspended sediment measurements taken at the Ellsworth gage on the Smoky Hill River. Bulk density measurements by USACE indicate a bulk density of 38.7 pcf in the multipurpose pool and 75.1 pcf in the flood pool. The sediment deposition within the reservoir was calculated using the sediment rating curves, bulk density, and trapping efficiency. This was compared to the sediment deposition estimated from the survey data and the upper portion of the rating curves were adjusted to bring them into closer agreement. The final computed deposition values matched well with the surveyed deposition.

Approximately 97% of the incoming sediment enters Kanopolis Lake during flood control operations.

A range in the natural sediment concentrations in inflow to Kanopolis Lake was estimated from the upstream suspended sediment measurements by fitting 80% confidence intervals to the data. Concentration increases with discharge and peaks at approximately 3,000 cfs. Using the sedimentation rangelines, the delta was estimated to have moved towards the dam at a rate of 472 feet per year from 2007 to 2017. The estimated degradation on the Smoky Hill River downstream of the lake was 599 ac-ft from 1946 to 2015 based on the degradation rangelines, which is 13% of the volume of sand accumulation in the lake.

17.0 REFERENCES

Brune, G. M. (1953). *Trap Efficiency of Reservoirs.* American Geophisical Union.

Duan, N. (1983). *Smearing estimate: A nonparametric retransormation method.* Journal of the American Statistical Association, 78(383), 605-610.

Juracek, K. E. (2011). *Suspended-Sediment Loads, Reservoir Sediment Trap Efficiency, and Upstream and Downstream Channel stability for Kanopolis and Tuttle Creek Lakes, Kansas, 2008-2010.* U.S. Geological Survey Scientific Investigations Report 2011-5187.

Kansas Biological Survey. (2009). *Bathymetric and Sediment Survey of Kanopolis Reservoir, Ellsworth County, Kansas.*

Shelley, J. E. (2012). *Geomorphic Equations and Methods for Natural Channel Design.* Lawrence, KS: Doctoral Dissertation, University of Kansas.

SURDEX. (2011). *Kanopolis Reservoir, Kansas Lakes LIDAR Mapping, Hydrographic Survey Data Integration & Flood Pool Survey.* SURDEX Corporation.

USACE. (1972). *Sedimentation in Kanopolis Reservoir.* Kansas City District: US Army Corps of Engineers.

USACE. (2013). *Vertical datum update for Kanopolis Lake.* U.S. Army Corps of Engineers, Kansas City District.

USACE. (2016). *Kanopolis Dam, Smoky Hill River, Kansas, Enbankment and Spillway, Periodic Inspection No. 11, Periodic Assessment No. 01.* Northwest Division, Kansas City District.

Zaiontz, C. (2014). *Confidence/prediction intervals | Real Statistics Using Excel*. Retrieved from real-statistics.com: http://www.real-statistics.com/regression/confidence-and-prediction-intervals/

US Army Corps of Engineers ®
Kansas City District

Kansas River Reservoirs Flood and Sediment Study

Appendix D1.6: Wilson Lake Existing Condition Sedimentation

November 2022

Contents

1.0 INTRODUCTION

Wilson Lake is located on the Saline River in central Kansas. The dam was first closed on 9 October 1964 and the conservation pool was filled by 12 March 1973. Authorized purposes include flood control, silt control, irrigation, recreation, and fish and wildlife (USACE, 1983). Contributing drainage area above the dam is 1971 square miles. Predominant land use in the watershed is agricultural consisting of cropland and grazing. Average annual precipitation ranges from 18 to 26 inches from west to east (USACE, 1983). Figure 1-1 shows Wilson Lake with respect to the entire Kansas River Basin and Figure 1-2 shows the Saline River Basin above the dam. Also shown in the figure are the main USGS gages upstream of the Lake that were used in this report.

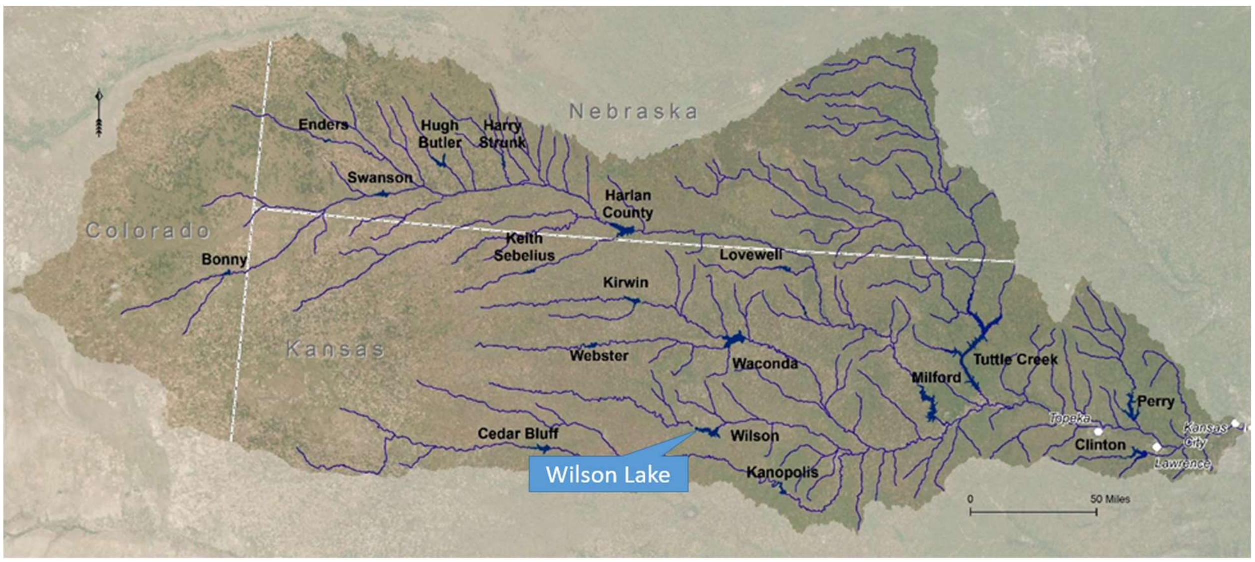


Figure 1-1: Overall Kansas River Basin

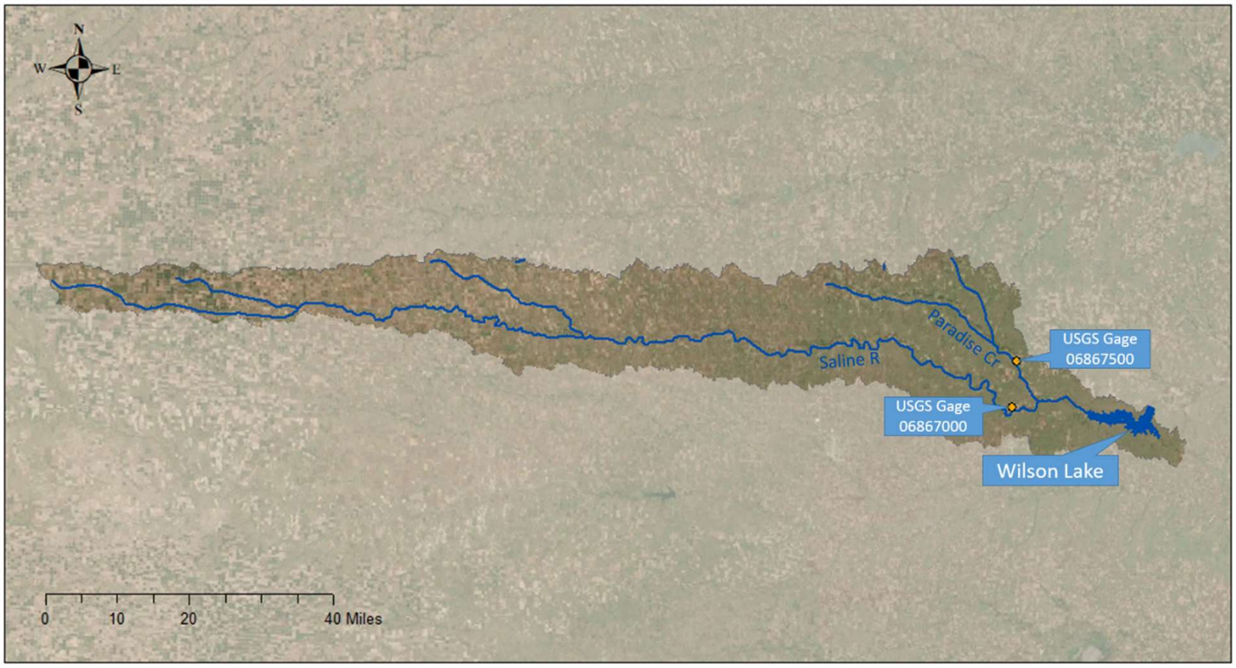


Figure 1-2: Saline River Basin above Wilson Lake. Dots indicate USGS gages used in this analysis.

2.0 DAM INFRASTRUCTURE

Table 2-1 gives pertinent data related to the dam and Figure 2-1 shows the location of key features. Elevations are shown in both NVGD29 and NAVD88. A survey by the Kansas City District (NWK) in 2014 showed a 0.46 ft difference between NVGD29 and NAVD88 (USACE, 2014) at the lake's staff gage. This is lower than the factor of 0.59 feet that was obtained when using the USACE CORPSCON software (Surdex, 2011). The conversion factor obtained from the 2013 survey was used to convert from NVGD29 to NAVD88 for this section as this is considered the official conversion factor for the lake.

The outlet works consists of an intake tower with two intake gates at an invert elevation of 1450.0 ft NVGD29, a circular tunnel, a stilling basin, and an outlet channel (USACE, 1983). There are also two 2'x 2' hydraulic slide low flow gates (USACE, 1983) for making smaller releases. The typical tailwater downstream of the dam was obtained from the USGS gage located approximately half a mile downstream of the dam.

Table 2-1: Important Information Relating to the Dam Infrastructure. Elevations in NVGD29 (NAVD 88 in parenthesis)

Parameter	Value
Multipurpose Pool Elevation	1516.0 (1516.46)
Lowest Elevation Outlet	1450.0 (1450.46)
Number of Gates at This Low Elevation	2
Service Gates Used When Flows Exceed	50 cfs
Service Gates Average Annual Usage	61 days
Spillway Elevation	1581.5 (1581.96)
Dam Elevation	1591.5 (1591.96)
Typical Tailwater Elevation	1431.1
Other Pipes Going Through the Dam or Embankment (i.e., Water Intakes, etc.)	None

Figure 2-1: Wilson Dam and infrastructure

Figure 2-2 through Figure 2-5 show drawings of the control tower and other dam infrastructure.

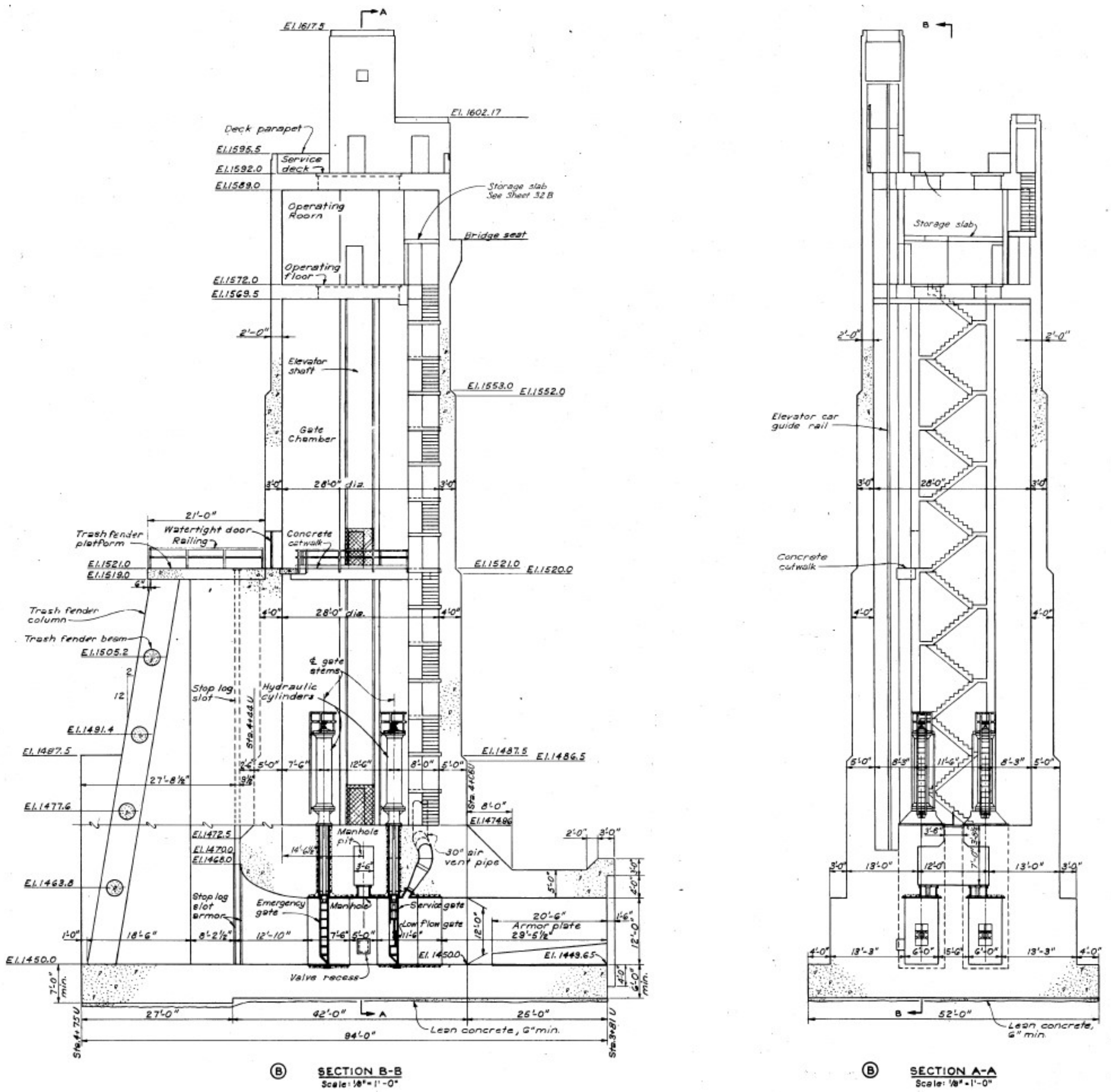


Figure 2-2: Intake tower. Elevations are NVGD29.

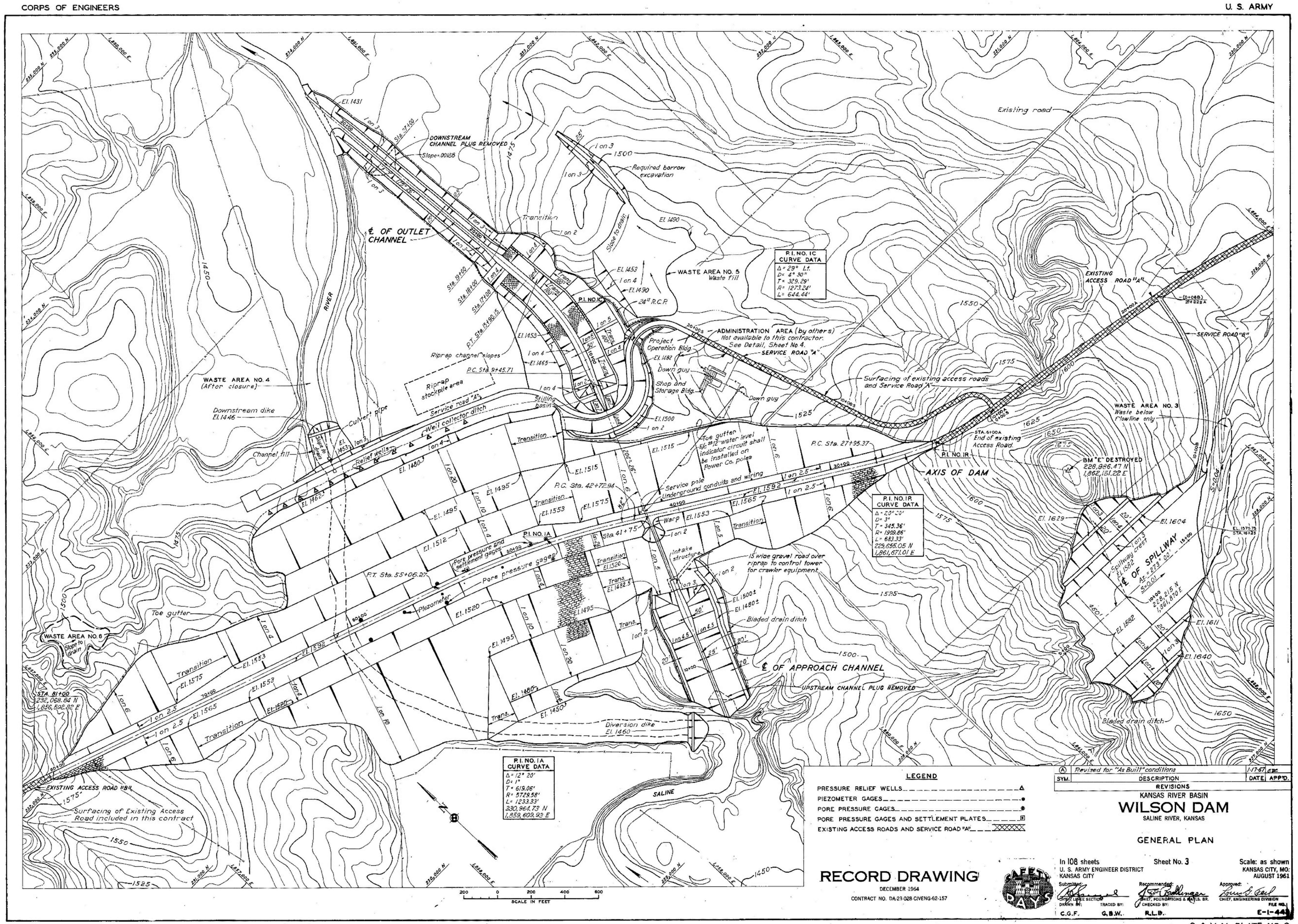


Figure 2-3: Wilson Dam General Plan, Elevations are in NVGD29.

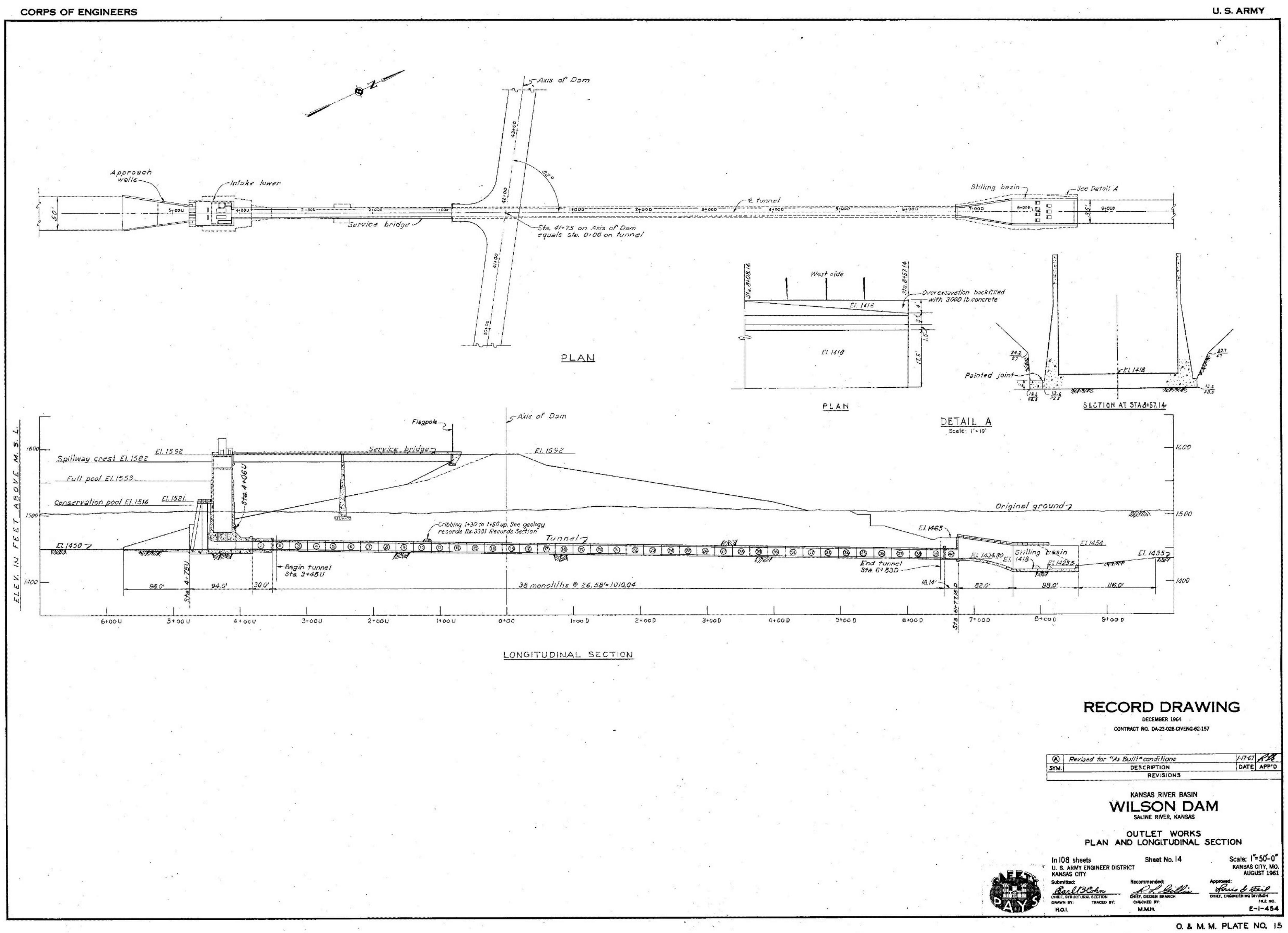


Figure 2-4: Wilson Dam Outlet Works Plan and Longitudinal Section. Elevations are in NVGD29.

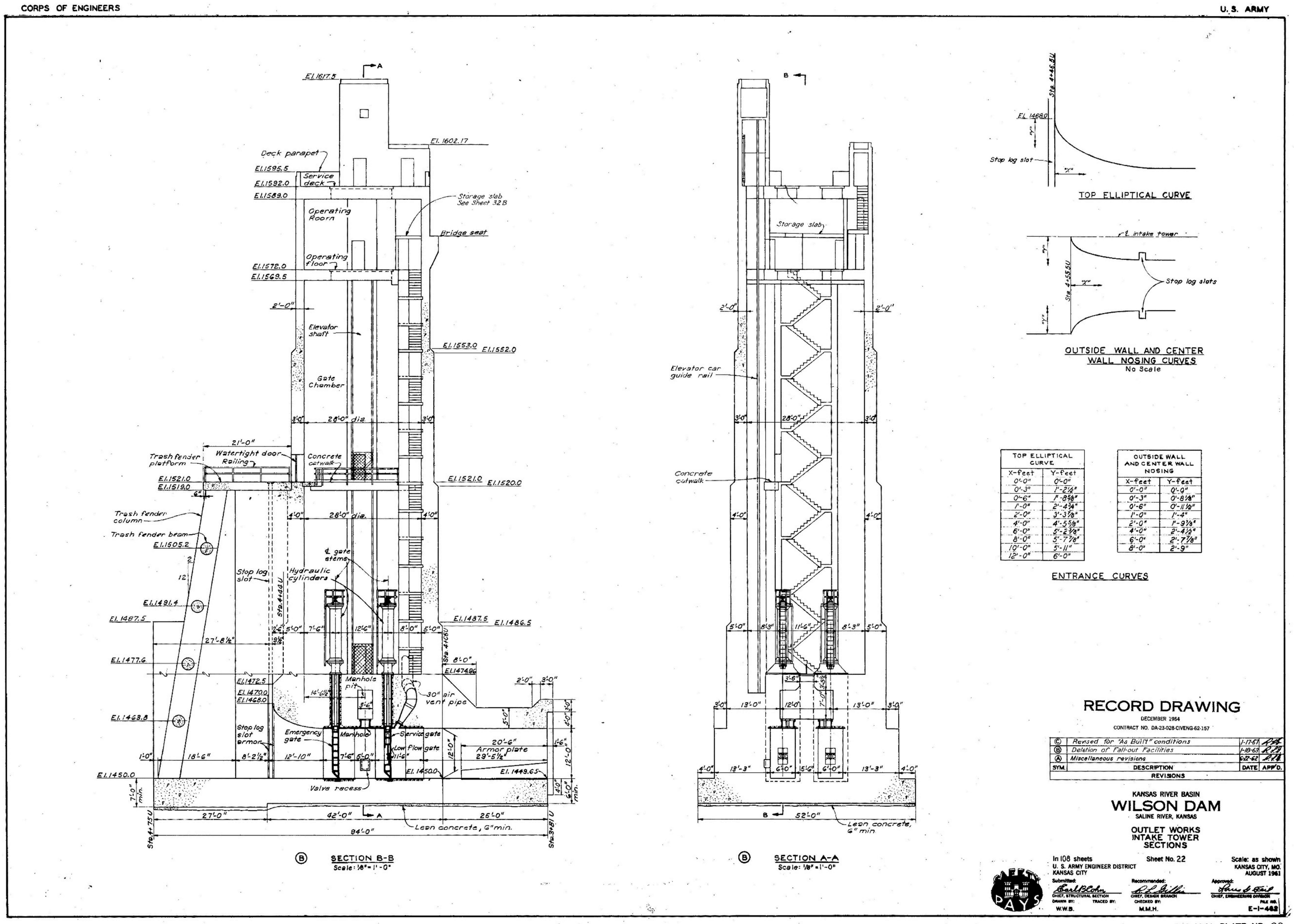


Figure 2-5: Wilson Dam Outlet Works Intake Tower Sections. Elevations are in NVGD29.

3.0 SEDIMENTATION EFFECTS ON O&M

Wilson lake has not had any significant impacts to O&M because of sedimentation. There have been some difficulties in accessing boat ramps when lake levels have been low due to drought conditions. Some dredging has been done to maintain boat ramps usability when the lake levels were too low.

Although there has been no impact to gate operation from sedimentation, there has been some buildup of sediment in the stilling basin. Silt flushes are conducted at Wilson Dam before each periodic inspection to remove silt from the stilling basin. The 1983 water control manual states that silt flushes of 2,000 cfs for an hour were to be performed every month and also identifies the source of the silt as the downstream slopes of the dam (USACE, 1983). It appears that these flushes have deceased in frequency to only occur before each periodic inspection. A recommendation for silt flushes was made in the 2015 Periodic Inspection, which were to be performed twice a year to scour out silt from around the gate slots (USACE, 2015). However, based on communication with the lake managers, this recommendation has not been implemented.

4.0 EXISTING CONDITION STORAGE ELEVATION CURVES

The most recent storage elevation curve for Wilson Lake was created by Surdex from a combination of LiDAR and bathymetry. Bathymetry was collected by the Kansas Biological Survey in July and October 2008 at a 500 foot transect spacing using single beam sonar. The LiDAR used in the storage curve was obtained by Surdex Corporation, mainly in 2010, though USGS data collected in other years was used in areas that were not surveyed in 2010. The 2010 LiDAR had a grid spacing of 2 meters and had a stated accuracy of 18cm RMSE Vertical (Surdex, 2011). Both the LiDAR and bathymetry were combined by Surdex into a digital elevation model (DEM) using Arc-GIS. The Surface Volume in Arc-GIS was then used to calculate the storage elevation tables. Table 4-1 shows the final 2008 storage elevation table.

Table 4-1: 2008 Storage Elevation Curve for Wilson Lake. Elevations in NVGD29

Elevation	Area (acres)	Volume (ac-ft)	Elevation	Area (acres)	Volume (ac-ft)	Elevation	Area (acres)	Volume (ac-ft)
1436	0	0	**1489**	64,089	4,160	**1542**	552,045	15,765
1437	0	1	**1490**	68,314	4,294	**1543**	567,980	16,110
1438	1	2	**1491**	72,680	4,435	**1544**	584,261	16,457
1439	5	6	**1492**	77,190	4,583	**1545**	600,893	16,801
1440	14	12	**1493**	81,844	4,732	**1546**	617,858	17,131
1441	28	17	**1494**	86,661	4,902	**1547**	635,150	17,456
1442	48	22	**1495**	91,661	5,103	**1548**	652,769	17,787
1443	73	28	**1496**	96,846	5,266	**1549**	670,719	18,116
1444	107	41	**1497**	102,202	5,445	**1550**	689,026	18,504
1445	152	51	**1498**	107,728	5,610	**1551**	707,731	18,911
1446	209	63	**1499**	113,434	5,799	**1552**	726,857	19,337
1447	279	79	**1500**	119,315	5,960	**1553**	746,392	19,740
1448	368	97	**1501**	125,342	6,094	**1554.0**	766,340	20,152
1449	472	113	**1502**	131,509	6,246	**1555**	786,693	20,556
1450.0	594	131	**1503**	137,832	6,398	**1556**	807,455	20,971
1451	733	148	**1504**	144,301	6,540	**1557**	828,628	21,370
1452	890	168	**1505**	150,917	6,697	**1558**	850,196	21,772
1453	1,069	193	**1506**	157,699	6,876	**1559**	872,171	22,173
1454	1,276	222	**1507**	164,683	7,090	**1560**	894,539	22,563
1455	1,512	252	**1508**	171,890	7,323	**1561**	917,290	22,943
1456	1,781	291	**1509**	179,323	7,546	**1562**	940,430	23,344
1457	2,103	355	**1510**	186,992	7,791	**1563**	963,979	23,759
1458	2,483	404	**1511**	194,875	7,966	**1564**	987,951	24,188
1459	2,912	456	**1512**	202,905	8,089	**1565**	1,012,344	24,597
1460	3,392	503	**1513**	211,051	8,202	**1566**	1,037,139	24,995
1461	3,917	550	**1514**	219,310	8,317	**1567**	1,062,326	25,376
1462	4,490	597	**1515**	227,685	8,434	**1568**	1,087,895	25,770
1463	5,108	642	**1516.0**	236,188	8,637	**1569**	1,113,868	26,183
1464	5,773	689	**1517**	245,123	9,188	**1570**	1,140,263	26,604
1465	6,489	746	**1518**	254,431	9,433	**1571**	1,167,069	27,005
1466	7,271	817	**1519**	263,965	9,649	**1572**	1,194,269	27,397
1467	8,131	906	**1520**	273,715	9,860	**1573**	1,221,863	27,795
1468	9,105	1,060	**1521**	283,685	10,094	**1574**	1,249,856	28,187
1469	10,248	1,215	**1522**	293,901	10,341	**1575**	1,278,233	28,565
1470	11,531	1,354	**1523**	304,374	10,612	**1576**	1,306,985	28,942
1471	12,955	1,498	**1524**	315,121	10,876	**1577**	1,336,113	29,319
1472	14,524	1,639	**1525**	326,123	11,125	**1578**	1,365,626	29,712
1473	16,233	1,783	**1526**	337,366	11,360	**1579**	1,395,540	30,117
1474	18,095	1,938	**1527**	348,849	11,610	**1580**	1,425,856	30,513
1475	20,102	2,078	**1528**	360,579	11,850	**1581**	1,456,570	30,916
1476	22,263	2,251	**1529**	372,549	12,090	**1582.0**	1,487,684	31,312
1477	24,614	2,444	**1530**	384,757	12,323	**1583**	1,519,194	31,709
1478	27,137	2,595	**1531**	397,194	12,554	**1584**	1,551,100	32,107
1479	29,794	2,719	**1532**	409,869	12,799	**1585**	1,583,407	32,515
1480	32,575	2,850	**1533**	422,798	13,064	**1586**	1,616,131	32,938
1481	35,496	2,988	**1534**	436,002	13,348	**1587**	1,649,290	33,378
1482	38,555	3,132	**1535**	449,486	13,619	**1588**	1,682,888	33,822
1483	41,757	3,268	**1536**	463,241	13,897	**1589**	1,716,926	34,256
1484	45,097	3,427	**1537**	477,280	14,183	**1590**	1,751,391	34,676
1485	48,597	3,574	**1538**	491,609	14,474	**1591**	1,786,278	35,096
1486	52,251	3,730	**1539**	506,240	14,790	**1592.0**	1,821,585	35,516
1487	56,054	3,876	**1540**	521,183	15,100	**1593**	1,857,308	35,930
1488	59,999	4,017	**1541**	536,446	15,431	**1594**	1,893,443	36,345

5.0 TRAPPING EFFICIENCY

In 2008, the multipurpose pool (MPP) capacity was estimated as 236,188 ac-ft. The mean annual water inflow based on gage data is 86,471 acre-ft. Brune offers three curves for estimating trapping efficiency, which can be estimated using Equations 1 and 2 and the constants give in Table 5-1 (Brune, 1953).

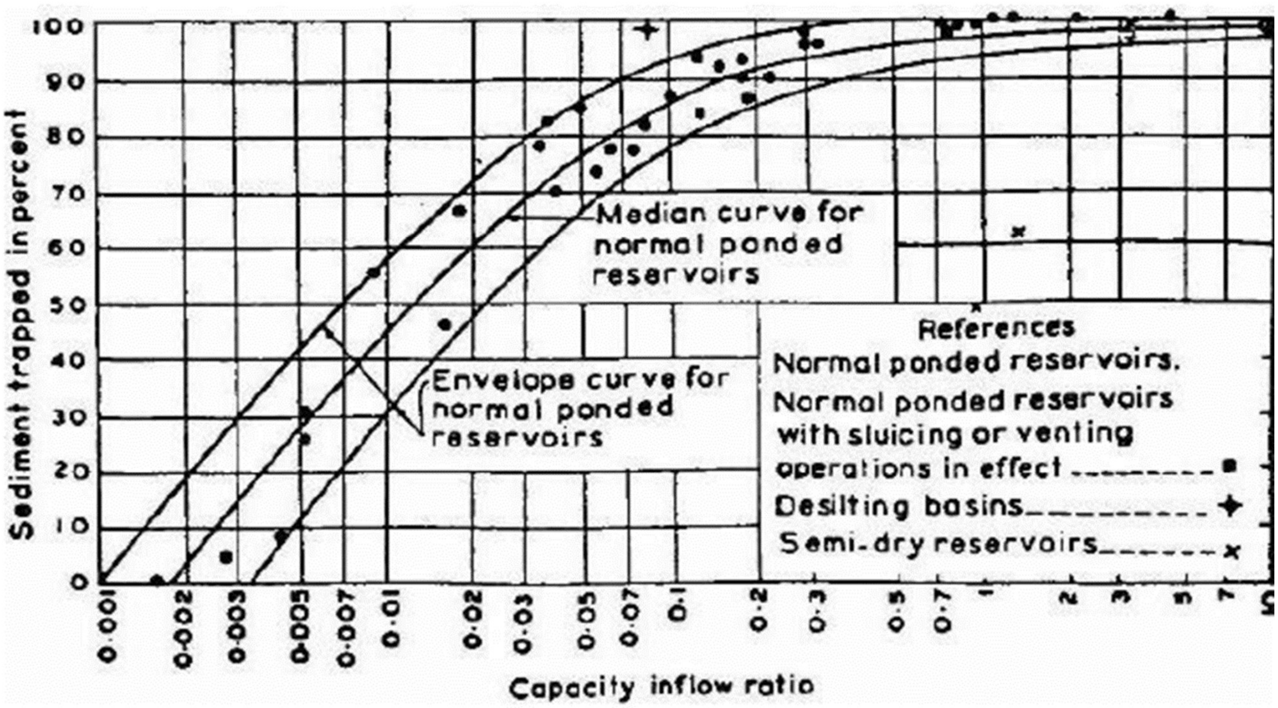


Figure 5-1: Brune Curves for estimating reservoir trapping efficiency (Brune, 1953)

$$TE = a[1 - 2e^{-bV_*^{0.35}}] \qquad (1)$$

$$V_* = \frac{V_{res}}{V_{inflow}} \qquad (2)$$

Table 5-1: Constants to be used in Equations 1 and 2

Constant	Low	Medium	High
a	95	97	100
b	5.37	6.42	7.71

The medium curve was selected for this analysis which results in an average trapping efficiency of 97.0 percent.

6.0 DEPOSITIONAL VOLUME

Historically, sedimentation rangelines have been used for calculating the capacity of the reservoir and determining sediment deposition amounts. Wilson Lake has a total of 19 rangelines spaced at varying distances as shown in Figure 6-1. Reservoir capacity was calculated from rangeline surveys in 1964, 1984, and 1995.

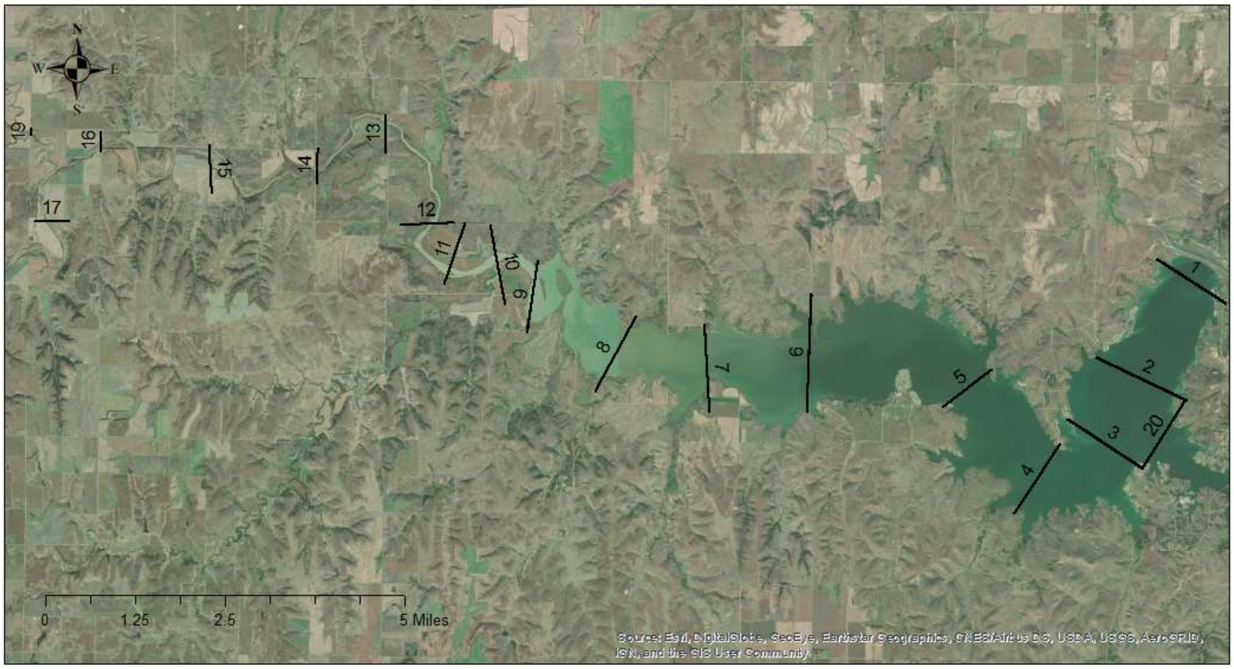


Figure 6-1: Sedimentation rangelines

As stated previously, the 2008 survey was collected using single-beam sonar along transect lines with an approximate transect spacing of 500 feet. Areas above the multipurpose pool were not surveyed. Figure 6-2 displays the data points. Surdex then combined this bathymetry dataset with the 2010 LiDAR into a DEM having a 12 ft grid spacing.

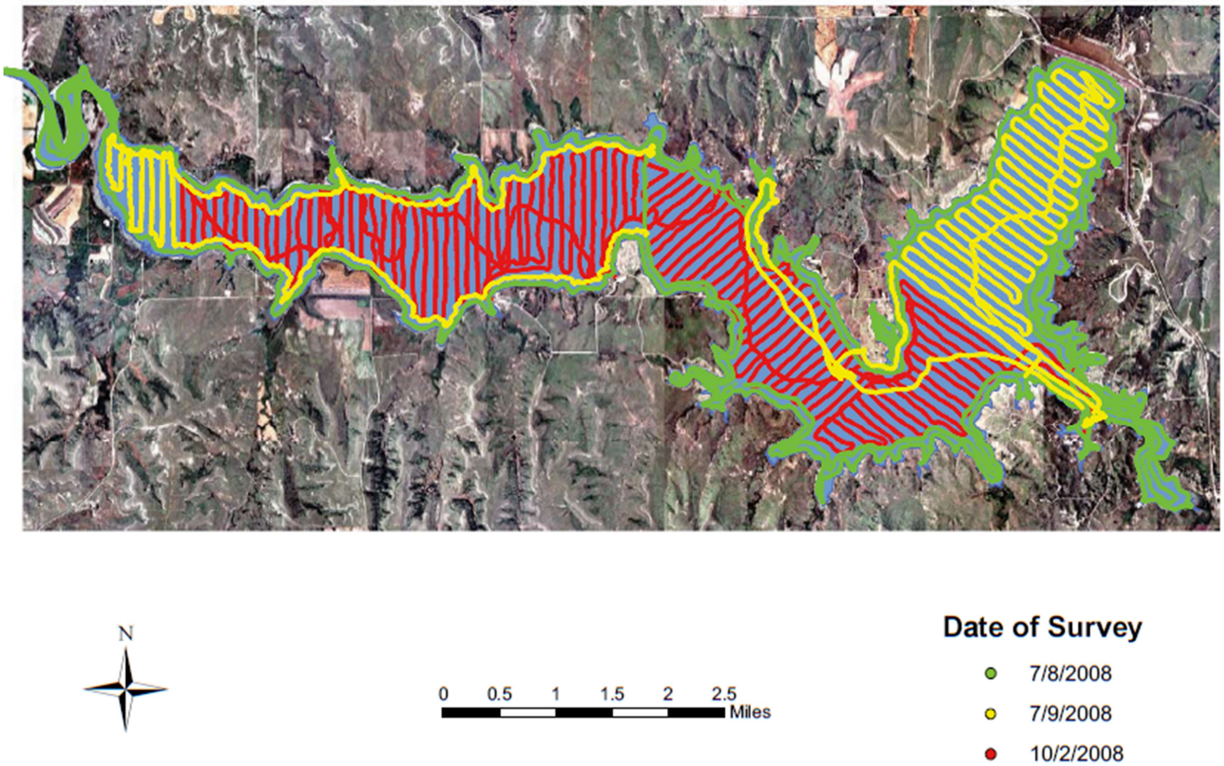


Figure 6-2: 2008 bathymetric survey (KBS, 2009)

Table 6-1 gives the calculated pool volumes for each of the surveys along with the survey methodology. The flood control pool volume does not include the multipurpose pool. The values in this table were taken from the 2011 Surdex report.

Table 6-1: Pool volumes over time

Year	Multipurpose Pool (acre-ft)	Flood Control Pool (acre-ft)	Data Type
1964	247,835	530,710	Sedimentation Rangelines
1984	242,528	530,204	Sedimentation Rangelines
1995	233,605	529,846	Sedimentation Rangelines
2008	236,188	529,289	Single beam sonar, LIDAR

Table 6-2 provides the amount of sediment deposition in the reservoir calculated by subtracting the pool volumes measured from the surveys. The survey methodology was switched for the 2008 survey, which is likely why there is negative deposition from 1995 to 2008. In Table 6-2, flood pool (FP) deposition indicates deposition at elevations higher than the multipurpose pool but lower than the top of flood pool.

Table 6-2: Deposition Amounts

Years	MPP Deposition	FP Deposition	Total Deposition	Yearly Deposition
1964-1984	5,307	506	5,813	291
1984-1995	8,923	358	9,281	844
1995-2008	-3,583	-216	-3,662	NA

From 1964 to 1995, the multipurpose pool lost 14,230 ac-ft of storage to sedimentation. This represents 5.7% of the original multipurpose pool volume. The average annual rate of loss was 459 ac-ft/year or 0.19% of the original volume/year.

From 1964 to 1995, the flood control pool lost 864 ac-ft of storage to sedimentation. This represents 0.16% of the original flood control pool volume. The average annual rate of loss was 27.9 ac-ft/year or 0.01% of the original volume/year.

7.0 INCOMING SEDIMENT LOADS

USACE and USGS have sporadically collected paired flow/sediment concentration and load measurements near Russell, KS on the Saline River since 1946. The USGS number for this gage is 06867000. All the available measurements taken at this gage are shown in Figure 7-1 which plots the suspended sediment load versus discharge. There seems to have been a decrease in the sediment load over time for discharges greater than 200 cfs as indicated by the most recent measurements. For this analysis the more recent measurements made near Russell, KS were used to develop the sediment rating curve for the Lake.

Based on gage data from various gages within the Kansas River watershed it was determined that the sediment rating curves generally begin flattening out at around the 83.3 to 66.7% (1/1.2 to 1/1.5) annual exceedance probability (AEP) discharge. Meaning the rating curve no longer follows the power fit trendlines when the discharge exceeds these values. These flows correspond with the typical range for bankfull flow in Kansas (Shelley 2012). The 1/1.2 and 1/1.5 AEPs are 400 and 930 cfs respectively for the Saline River at Russell. Based on the sediment measurements, the 1/1.5 AEP flow rate was chosen as the point to break rating curve. Only the data before the rating curves begins flattening was used to create the power fit trendline. The new slope at the higher end of the rating curve was determined through calibration, as explained later in this appendix. There is also a flatting of the sediment load for discharges below approximately 10 cfs. The Duan (1983) correction for bias introduced by the log transform was applied per Duan (1983).

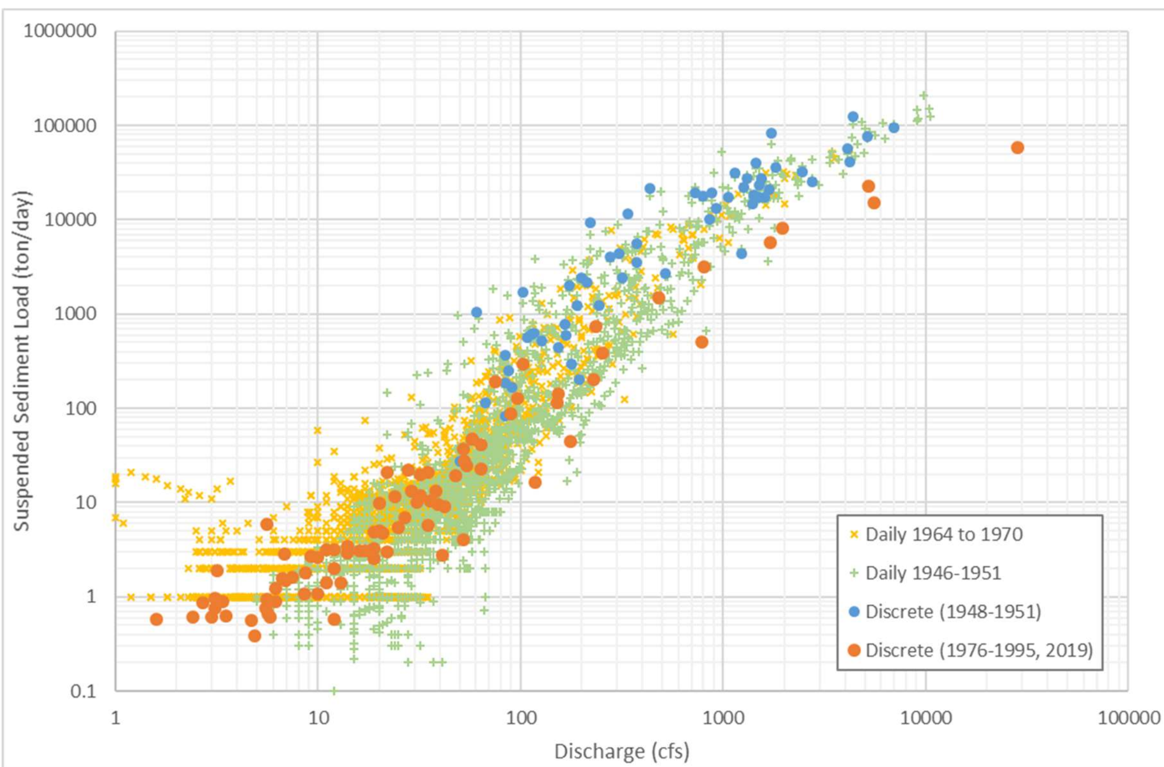


Figure 7-1: Suspended sediment load versus discharge on the Saline River near Russell, KS and Paradise Creek, near Paradise, KS

Suspended sediment measurements have also been collected on Paradise Creek and other gages within the Saline River Basin. Paradise Creek is a tributary to the Saline that enters downstream of the Russell gage.

There has not been any recent sediment measurement made at the Paradise Creek gage, and it does not have a continuous period of record. Analysis by the USGS in 1964 showed that Paradise Creek and areas below the Russell gage contribute substantially more sediment per square mile than areas upstream of the gage. Table 7-1 shows values of the long-term average sediment contribution from different areas in the Saline River basin taken from Table 13 of USGS Water Supply Paper 1641. Using the values from this table the areas above the Russell gage were estimated to have a sediment yield of 271,000 tons per year while the areas below Russell were 187,930. The Drainage area below the Paradise and Russell gages but above the dam is 257 square miles, which was assumed to have a sediment yield of 490 tons per square mile. A ratio of 1.69 was calculated by dividing the total sediment yield of the area above the dam by the sediment yield above the Russell gage. As this data is based on older measurements which do not take into account the shift in the rating curve, it is possible that this ratio could be too small if the areas downstream of Russell have not seen the same decrease in sediment loads as above the gage.

Table 7-1: Long-term average sediment contributions from areas in the Saline River basin (Jordan, Blair, & Lester, 1964)

Area	Drainage Area	Average runoff (in/year)	Tons per year	Tons per square mile per year	Discharge weighted average concentration (parts per million)
Upstream from Sheridan Lake	463	0.4	55,000	120	4100
Between Sheridan Lane and Wakeeney	233	0.7	46,000	200	3900
Between Wakeeney and Russell	806	1.1	170,000	210	2600
Paradise Creek	212	0.9	62,000	290	4600
Between Russell and Tescott Excluding Paradise and Wolf Creek drainage areas	845	1.6	410,000	490	3600

Estimates made using the Shields Bedload Equation in 1971 by USACE indicate that the bedload was 15% of the total load for Kanopolis Lake.

USGS measurements seem to show that there is a relationship between sediment gradation and discharge as shown in Figure 7-2. It appears that the percentage of clays increase with discharge, silt decreases, and sand stays largely constant. However, these trendlines are based on mostly older data, 1948 – 1995, and there are not enough recent measurements to provide a more current estimate. It is possible that the percentages have shifted with the reduction in sediment load. Because it is the only data available, the equations given in Figure 7-2 were used to calculate the sediment gradations with discharge.

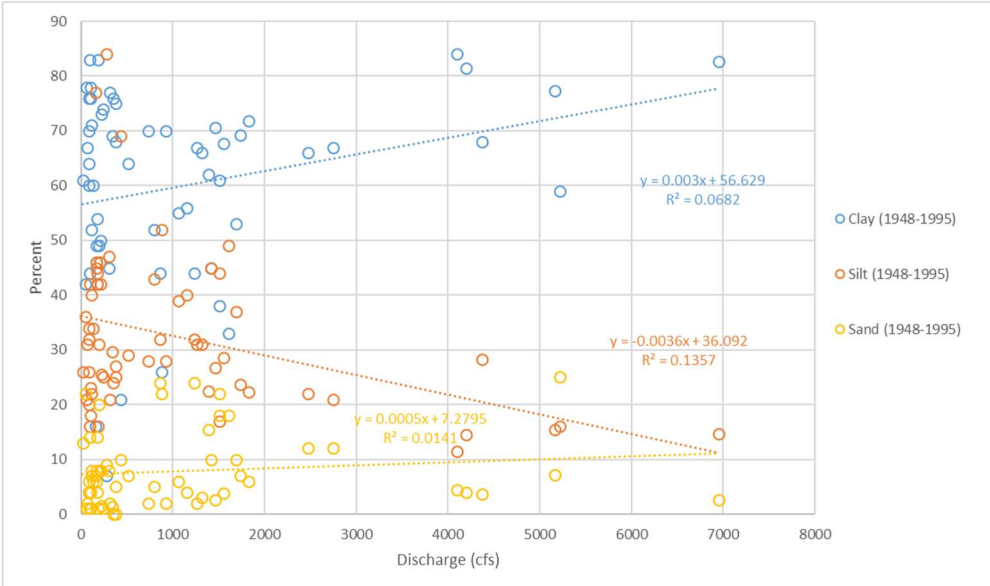


Figure 7-2: Sediment Gradation vs. Discharge

A total load rating curve for sediment inflows into Wilson was developed through the following steps:

1. Estimate the best fit regression line to the sediment measurements of the form $Q_s = aQ^b$ using log-log linear regression.
2. Correct for bias using the Duan correction factor (Duan 1983). The Duan E value was 1.35.
3. Add 15% to account for bed load to create a total load rating curve.
4. Multiply by a ratio 1.69 calculated earlier to account for the ungauged sediment.
5. Using the measured data, estimate the percentages of clay, silt, and sand/gravel.
6. Apply this rating curve to daily flow rates from 1965 to 2019 to determine the cumulative mass of sediment entering the reservoir.
7. Multiply by the appropriate percentages to determine the cumulative mass of clay, silt, and sand entering the lake.

This analysis found the incoming load to be 55.1% clay, 26.9% silt, and 18.0% sand/gravel. Table 7-2 summarizes the results for 1965 to 2019.

Table 7-2: Preliminary Incoming Sediment to Wilson from 1965 to 2019

Parameter	Value
Years	1964 - 2019
Total Incoming Sediment (tons)	4,750,762
Total Incoming Clay Fraction	55.1%
Total Incoming Silt Fraction	26.9%
Total Incoming Sand Fraction	18.0%

8.0 BULK DENSITY AND CONSOLIDATION OF SEDIMENT DEPOSITS

The incoming load can provide an estimate of the bulk density using the suspended sediment gradations from the Russell gage and Equation 1.

$$\boldsymbol{\gamma_c} = \frac{\mathbf{1.0}}{\left(\left(\frac{F}{\gamma}\right)_{clay} + \left(\frac{F}{\gamma}\right)_{silt} + \left(\frac{F}{\gamma}\right)_{sand}\right)} \tag{1}$$

where γ_c is the composite bulk density

F is the fraction of clay, silt, or sand

γ for clay, silt and sand is assumed to be 30 pcf, 65 pcf, and 93 pcf respectively.

By equation 1, the bulk density of the incoming load is 40.9 pcf.

In 2008 Kansas Biological Survey collected measurements of the sediment bulk density and gradation from six coring sites in Wilson Reservoir with surficial bulk densities of 69.4 to 100.6 pcf. Typically, the top six inches of the core are used for this (KBS, 2009), which may not be representative of total sediment deposition. The average thickness of deposition from the cores was close to two feet. Table 8-1 presents the gradations and shows that by mass, sand was predominate in most of the samples; followed by silt then clay. This is significantly different than the gradations from the suspended samples and could indicate that there is a large amount of sand entering the lake which is not being measured at the gages. The average bulk density from the KBS measurements is 84.68 pcf while the weighted bulk density is 83.2 pcf. These values are much higher than the calculations using the gradations from the Russell gage. Part of this difference is likely caused by the higher percentage of sand and silt in the KBS samples. The density of the sediment could also be higher than typical values because of the geology of the watershed.

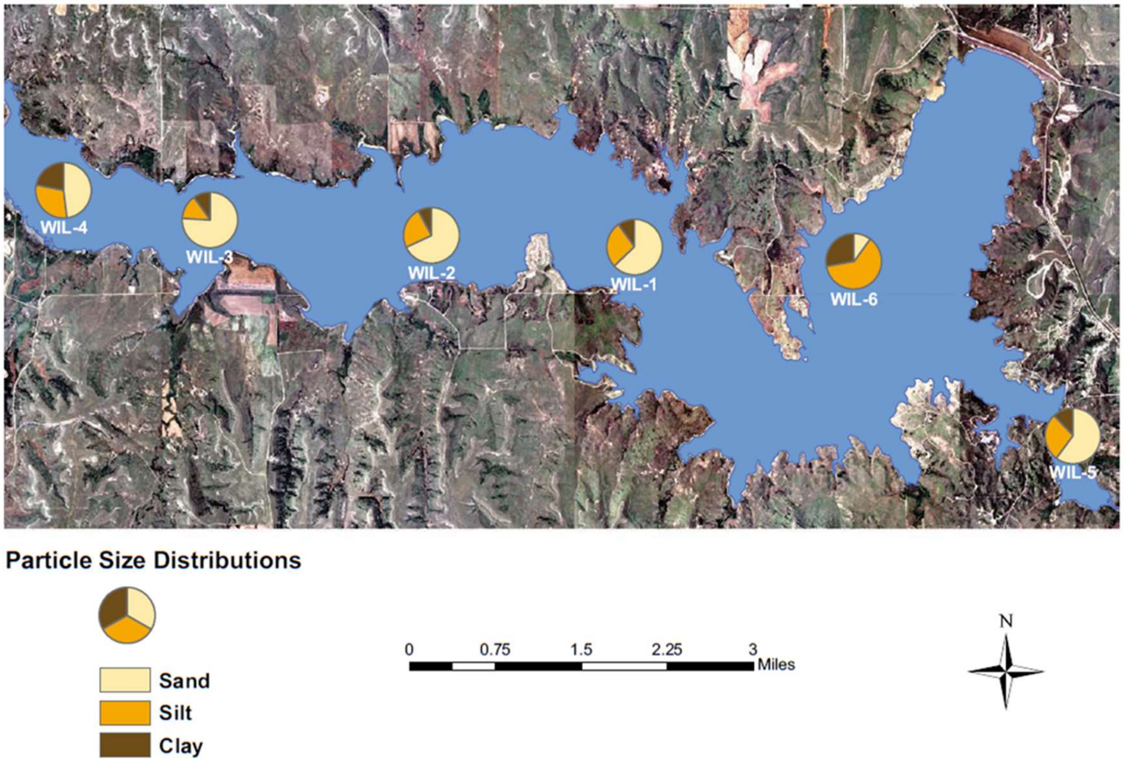


Figure 8-1: Sediment size gradations (KBS, 2009)

Based on this data and the surveyed deposition given in Section 6.0 , the mass of clay, silt, and sand were estimated to be the values given in Table 8-1. Also shown in the table are the masses estimated from the gage analysis. The total mass from the gaged inflow is nearly an order of magnitude lower than the mass estimated from the measured bulk density and surveyed deposition. Most of the difference is in the mass of sand and silt, although the mass of clay is also significantly different.

Table 8-1: Sediment masses from KBS samples and gage analysis 1964-1995

Grain Size	Rangelines/KBS	Inflow
Total	27,838,574	2,834,268
sand	15,079,228	574,171
silt	8,583,560	713,341
clay	4,175,786	1,546,755

These differences could be from various sources including: an overestimation of the bulk density from the surface samples; the collected samples not being representative of the overall deposits; an over estimation of deposition from the rangeline surveys; shoreline erosion; underestimation of the bedload; underestimation of the sediment yield from the drainage area below the gages; and an underestimation of the suspended sediment load at the Russell gage. It could also be caused by a combination of these factors.

Observations by KBS and geological maps of Russel County, located in Appendix A, show that the area surrounding the lake consists largely of Dakota Formation Sandstone. Dakota Sandstone consists of clay, shale, and siltstone with lenses of fine-grained sandstone (Ross, Michael, Crouse, Johnson, & Arbogast, 1996). The USGS conducted a study, *Water Supply Paper 1651*, of the Saline River Basin in the early

1960's; approximately when the dam was being completed. In the report the USGS made extensive observations and calculations of the sediment transport and deposits in the Saline River Basin. Below is an excerpt from the report noting the bed and floodplain material in the vicinity of the dam. Sylvan Grove is located approximately fourteen miles downstream of Wilson Dam while the Wilson gage was located within the area now occupied by the lake.

> *From the Wilson station to about 4 miles west of Sylvan Grove, medium and coarse sand derived from channel sandstone of the Dakota is abundant. Near Sylvan Grove the streambed and banks undergo a radical change in about 20 river miles. The bed changes from predominantly medium and coarse sand to mostly silt and some fine sand. The channel changes from wide and shallow to narrow and deep and from gently sloping sandy banks to steep well-vegetated banks composed mainly of silt. Accompanying the changes are a decrease in gradient and marked increases in valley width and degree of meandering. Likewise, the soils in the valley change from fine sand and sandy loam to silt loam.*

This supports the conclusion that sand could be a significant portion of the sediment load entering the reservoir. However, it is difficult to identify what the exact source of the sand is or how to quantify it. For the purposes of this study, it was decided to use the bulk density of 41.25 pcf calculated from the sediment gradations at the Russell gage, as this is the best estimate of the bulk density of the sediment for the Russell rating curve. The flood pool was assumed to have a bulk density of 72.4 which is the bulk density determined for the flood pool at downstream Milford Lake. Later revisions for Milford adjusted the bulk density to 72.6. As this is close to the 72.4, no adjustment was made to Wilson.

9.0 CALIBRATING LOAD AND DEPOSITIONAL PARAMETERS

Sufficient data exists to calibrate the rating curve by comparing the deposition computed using the incoming sediment loads to the deposition computed using the surveyed volumes. This was accomplished following these steps:

1. Determine the trapping efficiency of the reservoir (in this case 96.97%).
2. Apply daily flows at the Russell gage to its sediment rating curve.
3. Apportion the deposition into the multipurpose pool or the flood control pool. Multiply the deposition found in Step 4 by a factor, *m* such that the ratio of MPP deposition to total deposition is correct per the survey analysis. *m* was computed from the surveyed deposition and the bulk densities determined for the MPP and FP.
4. Repeat steps 2 through 3 for each day over a period of time to obtain the cumulative sediment inflow.
5. Compute the mass of trapped sediment in the multipurpose pool by applying the trapping efficiency to the incoming sediment. The flood pool deposits were assumed to have a 100% trapping efficiency.
6. Transform the trapped mass to a deposited volume by using the bulk densities determined in Section 9 (41.25 pcf in the MPP and 72.4 pcf in the FP).
7. Compare the total rating-curve-based deposition to the deposition calculated from the surveyed volumes.
8. Adjust the sediment rating curve (described below) to more closely match the surveyed deposition

Table 9-1 lists the uncalibrated parameters that were used in calculating the volume of sediment deposited in the reservoir. The computed value of bulk density was used for this analysis rather than the measured bulk density by the KBS.

Table 9-1: Parameters used in calculating reservoir deposition

Parameter	Initial
Bulk Density MPP (pcf)	41.25
Bulk Density FCP (pcf)	72.4
Bedload % of Suspended	15%
Correction for Ungauged Sediment Load	1.69
a in $Q_s = aQ^b$	0.0376
b in $Q_s = aQ^b$	1.6211
Average trapping efficiency	96.97%

Table 9-2 summarizes the results of the calculations for deposition between each of the surveys. As stated previously, the surveys show that there is an increase in the capacity of the reservoir between 1995 and 2008, which is likely caused by the change in survey methods in 2008. The initial rating curve and bulk density produced volumes that were 4-5 times less than the surveyed deposition. As mentioned in the previous section, if the surveyed bulk density measured by KBS is used, the difference becomes even greater.

Table 9-2: Surveyed and (uncalibrated) computed reservoir deposition (ac-ft)

Time Period	Surveyed Deposit (ac-ft)	Computed Initial (ac-ft)	Computed / Surveyed
1964-1984	5,813	1,589	0.20

1984-1995	9,281	2,444	0.19
1995-2008	-3,662	1,146	NA

It is difficult to determine what exactly is causing such a large difference in the surveyed and calculated deposition. However, for the purposes of this study, it was decided to calibrate the Russell sediment rating curve to the 1984-1995 surveyed deposition by multiplying the rating curve by a constant value of 5.35. This was determined by dividing the surveyed deposition by the computed. It can be seen that the calibrated deposition also matches well with the surveyed deposition from 1964 to 1984.

Table 9-3: Surveyed and Calibrated Deposition

Time Period	Surveyed Deposit (ac-ft)	Computed Calibrated (ac-ft)	Computed / Surveyed
1964-1984	5,813	6,349	1.06
1984-1995	9,281	9,281	1.00
1995-2008	-2,583	7,785	NA

Figure 9-1 shows that final total load rating curves along with the suspended sediment measurements at the Russell gage. The calibrated rating curve matches closer to the older sediment measurements than it does with the new ones at higher flow rates. This could indicate that there has not been a shift in the sediment rating curve over time, or it could be caused by other factors already mentioned. At this moment there is significant uncertainty in the sedimentation computations. Additional bathymetric surveys and bulk density measurements to compare to would help reduce this uncertainty.

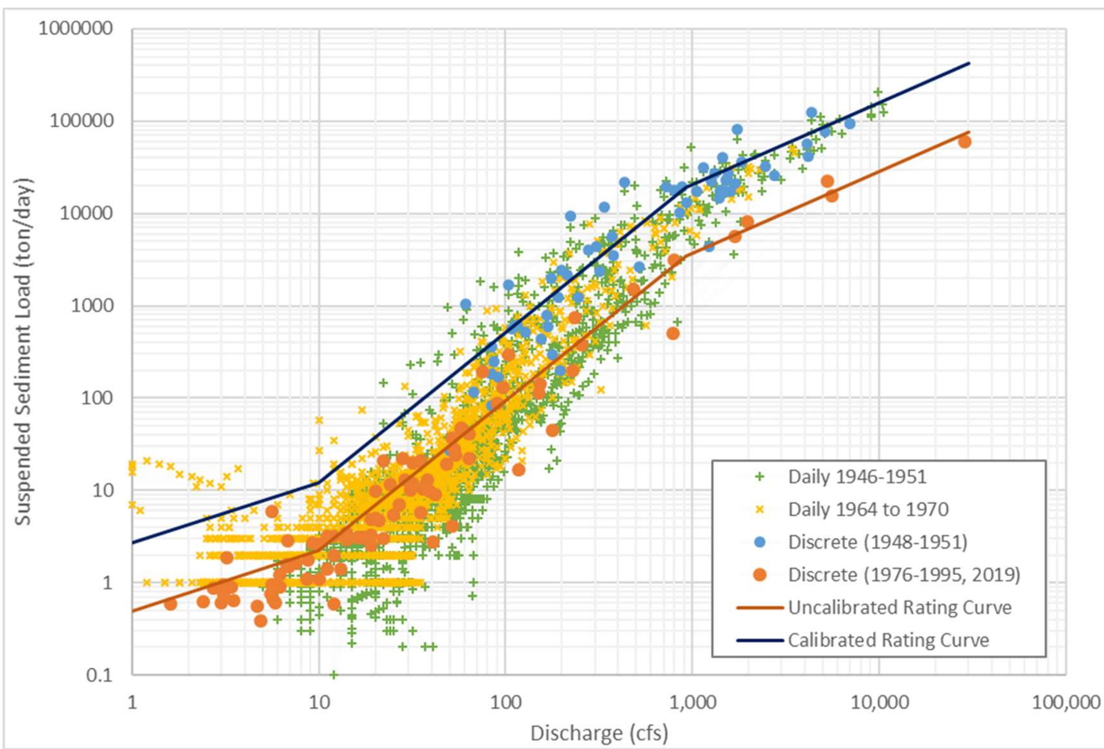


Figure 9-1: Suspended sediment measurement and final total load rating curves

Using the calibrated rating curve, the total deposition in the multipurpose pool from 1964 to the end of 2019 is 29,848 ac-ft, or 12% of the original storage.

10.0 SEDIMENT TRAPPING DURING FLOOD CONTROL VS. MULTIPURPOSE POOL OPERATIONS

Table 10-1 indicates the quantity of sediment that enters the lake when the reservoir is in flood control operations (i.e., with a water surface above the multipurpose pool elevation), vs. multipurpose pool operations (i.e., when the water surface is at or below the multipurpose pool elevation).

Table 10-1: Incoming Sediment Amounts during Flood Control and Multipurpose Operations

Deposition	Deposition 1973 - 2019 (ac-ft)	% of Total
Total Deposition	23,235	100 %
Multipurpose Operations	4,253	18.3%
Flood control Operations	18,982	81.7%

Lake levels vary throughout the year depending on a variety of factors. However, lake managers generally try to operate the lake so that pool elevations match the Water Level Management Plan (WLMP) shown in Figure 10-1. The quantity of deposition occurring when the pool elevation is above or below the WLMP line shown in Figure 10-1 is given in Table 10-2.

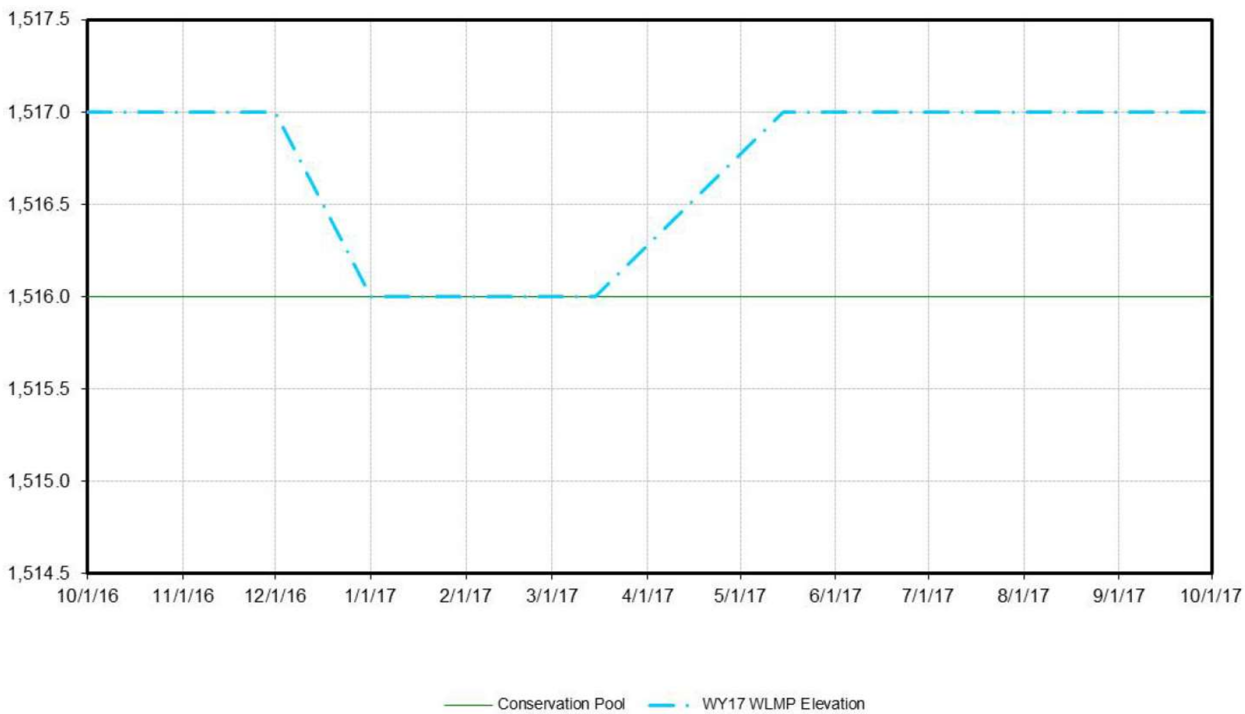


Figure 10-1: Wilson Lake Water Level Management Plan (USACE, 1983)

Table 10-2: Deposition amounts when Pool Elevation is Below or Above the WLMP

Deposition	Deposition 1964 – 2019 (ac-ft)	% of Total
Total Deposition	23,235	100 %
Below WLMP	7,624	32.8 %
Above WLMP	15,610	67.2 %

11.0 SEDIMENT CONCENTRATIONS

Figure 11-1 illustrates the concentration of incoming sediment, together with the 80% confidence intervals. This graph represents the range of natural variability in the sediment concentrations in the river, i.e., what the concentration would be if the dam were not in place. As a first approximation, the lower bound can be thought of as a minimum target for naturalizing downstream sediment levels, and the upper bound can be thought of as a maximum limit to avoid excessive sediment releases.

The flow/concentration relationship is not monotonic or with a consistent slope. Rather the relationships in log space at low flows, moderate flows, and high flows exhibit separate slopes. By observation, at many of the lakes the sediment concentrations actually reverse at higher flows. This behavior translates into flatter flow-low curves as described earlier in the document. While the reasons for this phenomenon are unknown, it could be can be explained by either the supply limitation of easily erodible sediments or by sediment lost to the floodplain during overbank flows. A fourth-order polynomials through log-transformed data were used to reflect the overall trends in the data. This curve is a valid fit over the range of observed data but should not be used for extrapolation.

Also evident in the data is a reduction in the variability in concentration at higher flows. While fewer measurements could by itself lead to the perception of less variability, visual inspection of the flow/concentration measurements suggests that physical reasons may drive the lower variability. The first possible reason for the reduction in variability is the supply limitation that drives the reduction in concentration. Higher concentrations are constrained by lack of readily available material. A second explanation for lower variability is that moderate flows can be achieved by a precipitation in only part of the watershed, and different sub-watersheds may have different sediment contributions. On the other hand, very high flows are only achievable when most of if not all the entire watershed contributes, which reduces the spatial variability based on storm placement.

The confidence intervals were originally computed based on the statistics of the total sample and departure from the best-fit polynomial. However, the height of the intervals was driven by the high variability at moderate flows, which yielded confidence intervals at the highest flows that exceeded all the measured data points. A more refined approach was taken to take into account differences in variability as a function of flow.

The 4th-degree polynomial through log-transformed data was used as a predictor for the mean of a normal distribution in a generalized additive model. To capture the changing variance of concentration with respect to flow, a concave-down function was needed to predict the sigma term in order to produce a local maximum in the middle with decreases in variance at both extremes. A quadratic function was used to ensure $d2\sigma/dx2<0,\forall x$. The result is a six-parameter model (plus two intercepts) in a hierarchical structure.

The interpretation should be looked at as "the conditional distribution of suspended sediment concentration, given an amount of flow." That is, $f(y|x)=N(\mu=b4x4+b3x3+b2x2+b1x+b0, \sigma=a2x2+a1x+a0)$ where y is SSC, x is flow, and N is the normal distribution with parameters μ and σ. Because the conditional distribution is normal, the best estimate of $y|x$ is $E[y|x]$ and is also the median of the conditional distribution, and it can be computed as μ. To obtain percentiles of the conditional distribution, the quantile function (inverse CDF) is used, $F\text{-}1(p;\mu,\sigma)$ where μ and σ are computed for the given value of flow. The result is overall a function that has a central tendency dictated by the 4th-order polynomial, with spread about the mean dictated by a concave-down quadratic.

These analyses were automated using R-scripts.

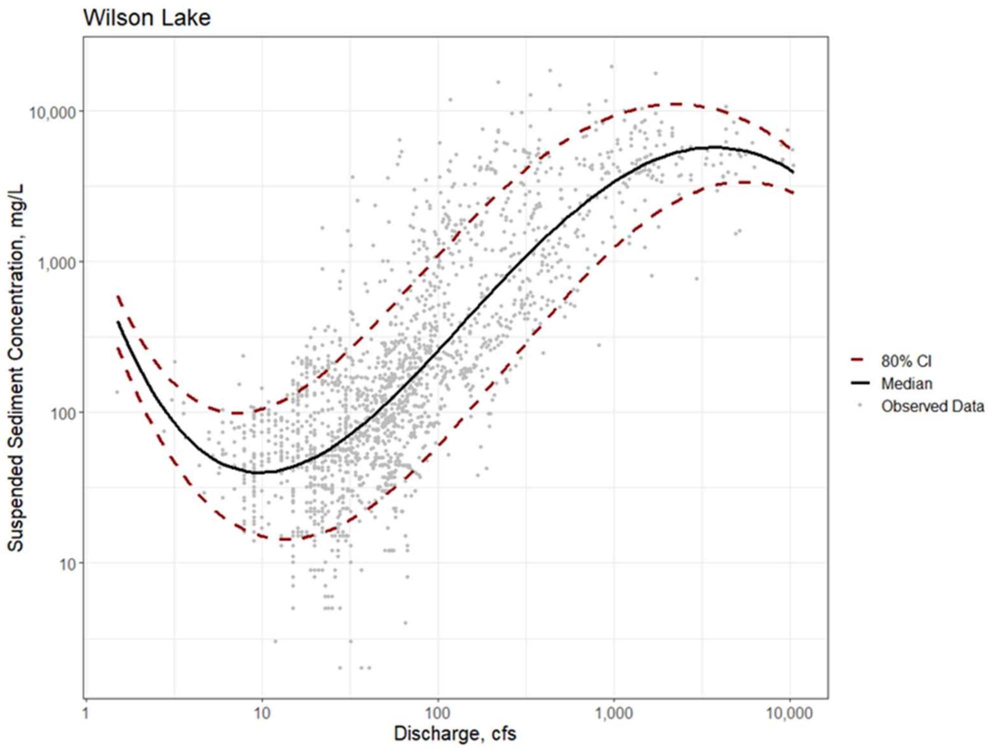


Figure 11-1: Incoming Sediment Concentrations to Wilson Lake

Sediment concentration measurements have also been collected downstream of Wilson dam at USGS gage 06868200. The downstream concentrations are significantly lower than the upstream measurements due to sediment trapping by Wilson Lake. Figure 11-2 shows the upstream and downstream measurements.

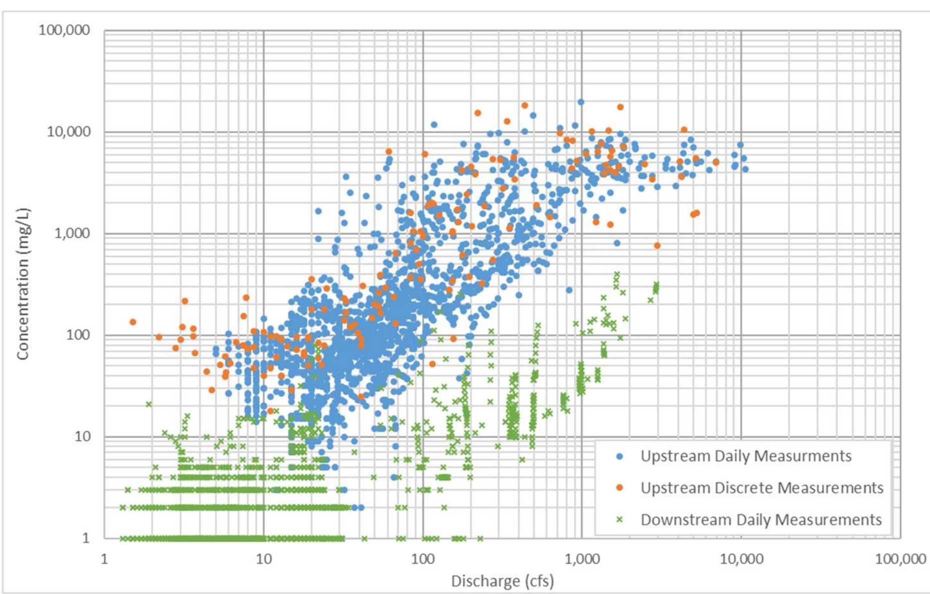


Figure 11-2: Sediment concentration measurements taken upstream and downstream of Wilson Dam

12.0 SEDIMENT CHEMICAL CONCENTRATIONS

No previous reports or information were found related to the chemical concentrations of the sediment deposited in Wilson Lake.

13.0 DELTA LOCATION AND VOLUME

Figure 13-1 provides a profile plot of centerline through the lake. At each location, the invert elevation (lowest elevation in a given sedimentation range line) is plotted. The1963-1995 rangeline surveys were converted from NVGD29 to NAVD88 using values obtained from a conversion raster in Arc-GIS based on the USACE Corpscon conversion tool. The rangelines for the 2008 survey were pulled from the 2008 combined bathymetry and LiDAR DEM. Because LiDAR cannot penetrate through water and bathymetry data was not collected above the multipurpose pool, the invert elevations are likely overestimated upstream of this for the 2008 survey. As seen in Figure 13-1, the delta crest grew significantly immediately after dam closure. From 1984 to 1995 the delta progressed an average of 1.22 miles downstream or 168 feet per year. No downstream migration was evident from 1995 to 2008.

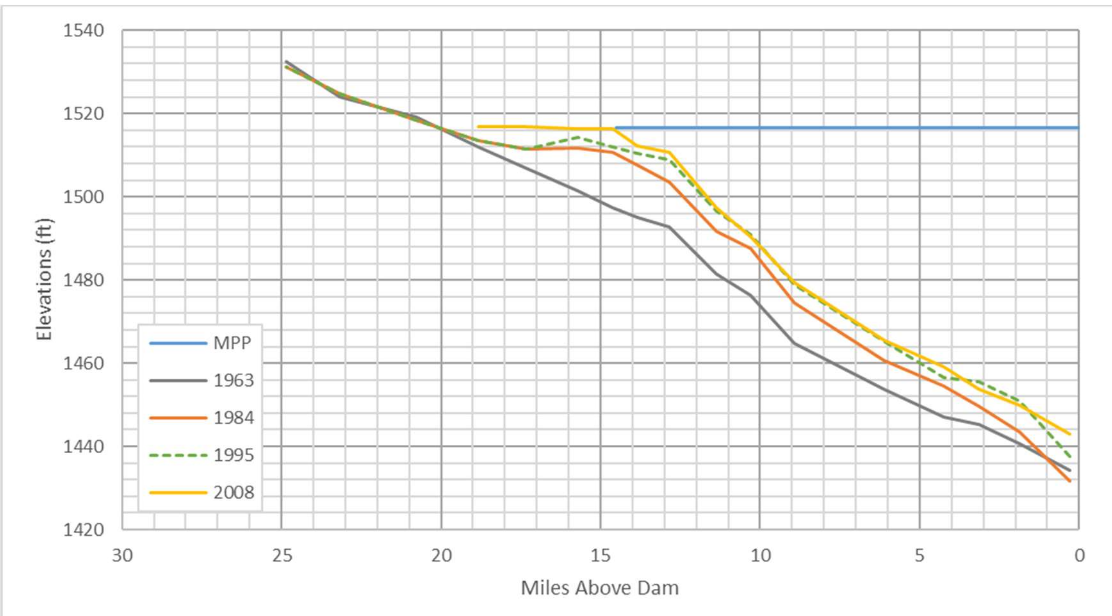


Figure 13-1: Profile of Invert Elevations Indicating Delta Location and Growth

14.0 DOWNSTREAM CHANNEL

Sediment trapping by dams very often induces bed degradation and bank erosion downstream. However, an analysis of the degradation rangelines downstream of Wilson does not indicate any significant degradation. See Figure 14-1 for an example of one of the degradation rangelines downstream of Wilson Lake. This could be caused by the attenuation of high flows caused by the dam resulting in insufficient sediment transport capacity to move the heavily inflows of sediment from small tributaries immediately below the dam. It could also be a function of the location of the rangelines.

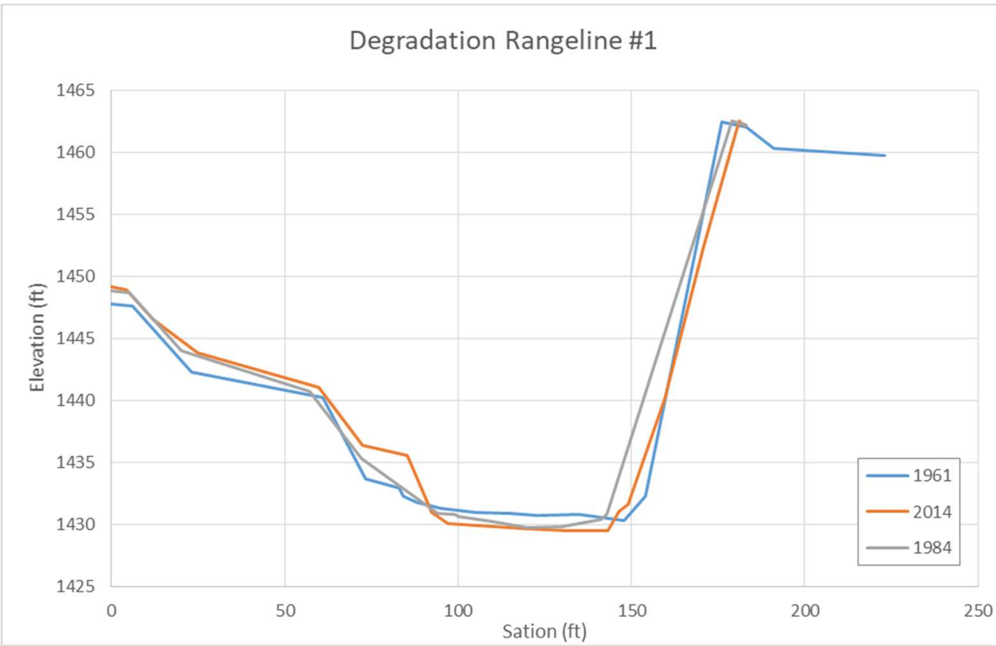


Figure 14-1: Degradation rangeline immediately downstream of Wilson Lake

Figure 14-2 shows the locations of the four degradation rangelines immediately below Wilson Dam.

Figure 14-2:The four degradation rangelines immediately below Wilson Dam

Figure 14-3 shows all twenty-one degradation rangelines that are downstream of Wilson Dam on the Saline River.

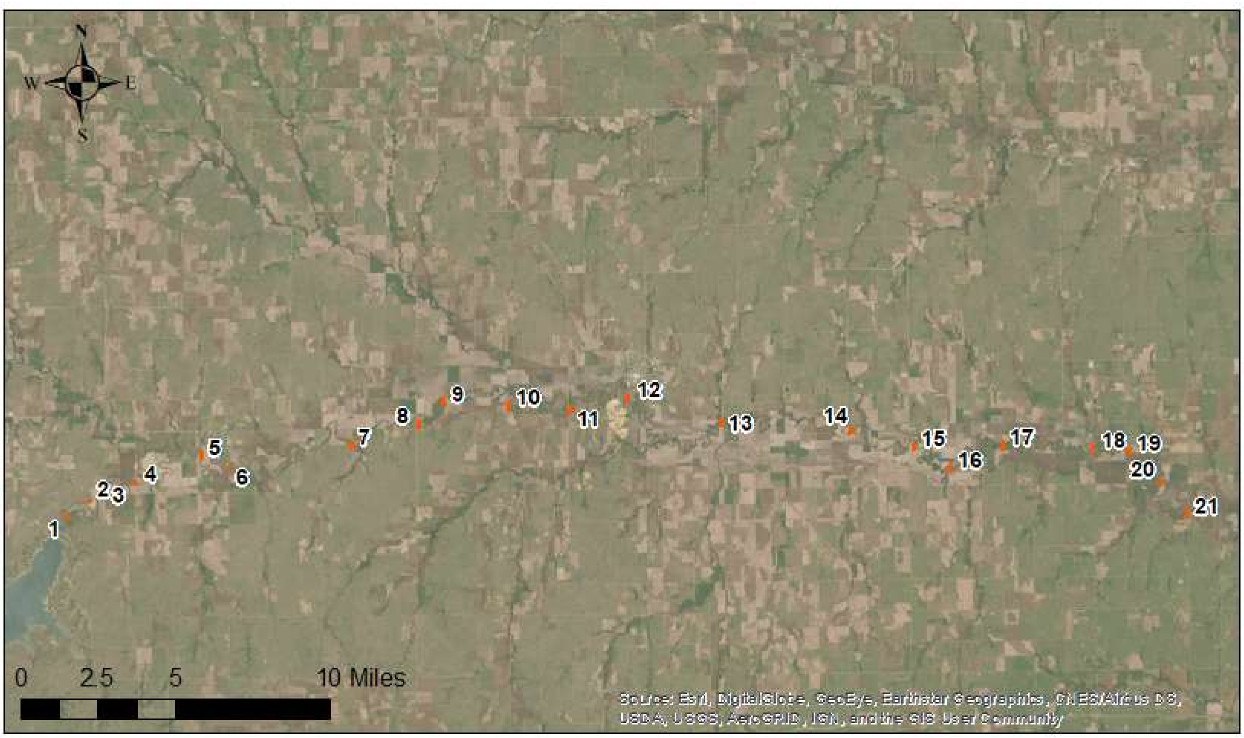


Figure 14-3: All 21 degradation rangelines downstream of Wilson Dam on the Saline River

15.0 SUMMARY AND CONCLUSIONS

Sedimentation has had a minor impact on Wilson Lake through loss of storage capacity and a moderate impact to infrastructure surrounding the lake. As of June 1995, the multipurpose pool had lost 14,230 ac-ft of storage capacity to sedimentation, or 5.74% of the original volume. Between 1964 and 1995 the flood pool lost 864 ac-ft of storage, which 0.16% of its original volume.

The calibrated rating curve indicates a multipurpose pool loss of 29,848 ac-ft from 1964 to 2020, or a rate of 571ac-ft/year or 0.23% per year.

The trapping efficiency of the lake was estimated to be 97% in 2008 based on the Brune Curve. Sediment rating curves were created from suspended sediment measurements taken at the Russell gage on the Saline River. Applying standard bulk density estimates to the size percentages in the deposited material as determined by KBS indicates a bulk density of 84.7 pcf in the multipurpose pool with most of the sediment within the lake being sand. However, this is a much higher bulk density and sand percentage than was estimated from the suspended sediment sizes at the Russell gage. The bulk density 41.3 pcf calculated from these measurements was used as the multipurpose pool bulk density while the flood pool bulk density was assumed to be equal to 72.4 pcf, which was the value determined for nearby Milford Lake.

The sediment deposition within the reservoir was calculated using the sediment rating curves, bulk density, and trapping efficiency. This was compared to the sediment deposition estimated from the survey data and the sediment rating curve was adjusted by multiplying it by a factor of 5.35. This is the largest adjustment needed for any lake in the basin and reflects significant uncertainty in the sediment load measurements or the possibility of additional sediment downstream from the gage locations or through bank erosion. With the 5.35 factor, the rating curve closely approximates the sediment deposition over two separate time periods.

Approximately 82% of the incoming sediment enters Wilson Lake during flood control operations.

A range in the natural sediment concentrations in inflow to Wilson Lake was estimated from the upstream suspended sediment measurements by fitting 80% confidence intervals to the data. Concentration increases with discharge and peaks at approximately 300 cfs. Using the sedimentation rangelines, the delta was estimated to have moved towards the dam at a rate of 168 feet per year from 1984 to 1995. There was little or no degradation observed on the Saline River below Wilson Dam

New survey data, more recent sediment load data, and a survey of additional sources of sediment (including reservoir bank erosion) could provide better estimates of sedimentation rates for use in future projections. However, because the rate of sedimentation is relatively low, the benefit of this additional precision may not justify the expense.

16.0 REFERENCES

Brune, G. M. (1953). *Trap Efficiency of Reservoirs.* American Geophisical Union.

Duan, N. (1983). *Smearing estimate: A nonparametric retransormation method.* Journal of the American Statistical Association, 78(383), 605-610.

Jordan, P. R., Blair, F. J., & Lester, R. P. (1964). *Chemical Quality of Surface Waters and Sedimentation in the Saline River Basin.* Washington, DC: US Geological Survey.

KBS. (2009). *Bathymetric and Sediment Survey of Wilson Lake Reservoir, Russell County, Kansas.* Kansas Biological Survey.

Ross, J. A., Michael, G. E., Crouse, E. C., Johnson, W. C., & Arbogast, A. F. (1996). *KGS - Geological Map - Russell - Large Size.* Retrieved from the Kansas Geological Survey: http://www.kgs.ku.edu/General/Geology/County/rs/russellLarge.html

Shelley, J. E. (2012). *Geomorphic Equations and Methods for Natural Channel Design.* Lawrence, KS: Doctoral Dissertation, University of Kansas.

Surdex. (2011). *Wilson Reservoir, Kansas Lakes LIDAR Mapping, Hydrographic Survey Data Integration & Flood Pool Survey.* Surdex Coorporation.

USACE. (1983). *Smoky Hill River Basin, Lake Regulation Manual, Wilson Lake Kansas.* US Army Corps of Engineers, Kansas City District.

USACE. (2014). *Vertical Datum Update for Wilson Lake.* U.S. Army Corps of Engineers, Kansas City District.

USACE. (2015). *Wilson Dam, Periodic Inspection No. 13, Periodic Assessment No. .* US Army Corps of Engineers, Kansas City District, Northwestern Division.

Zaiontz, C. (2014). *Confidence/prediction intervals | Real Statistics Using Excel*. Retrieved from real-statistics.com: http://www.real-statistics.com/regression/confidence-and-prediction-intervals/

US Army Corps of Engineers ®
Kansas City District

Kansas River Reservoirs Flood and Sediment Study

Appendix D1.7: Harlan County Lake Existing Condition Sedimentation

November 2022

Contents

1.0 INTRODUCTION

Harlan County Lake located on the Republican River in southern Nebraska. The dam was closed in October 1952 and multipurpose pool (MPP) elevation was first reached in June 1957. Authorized purposes include flood risk management, irrigation, recreation, and fish and wildlife (USACE, 2015). The contributing drainage area above the dam is approximately 20,753 square miles, though much of this is either controlled by upstream Bureau of Reclamation dams. Predominant land use in the watershed is agricultural, consisting of cropland and grazing. The climate of the Republican Basin above Harlan County is semi-arid with an average annual precipitation of 16 to 22 inches from west to east (USACE, 1973). Streamflow within the basin has declined significantly since the 1960s due to irrigation and other factors.

Figure 1-1 shows Harlan County Lake in respect the whole Kansas River Basin, while Figure 1-2 shows the lake and the Republican River Basin above Harlan County Dam. Sappa Creek and Prairie Dog Creek are two major tributaries that enter the lake besides the Republican River. The drainage areas of these streams are also shown in Figure 1-2.

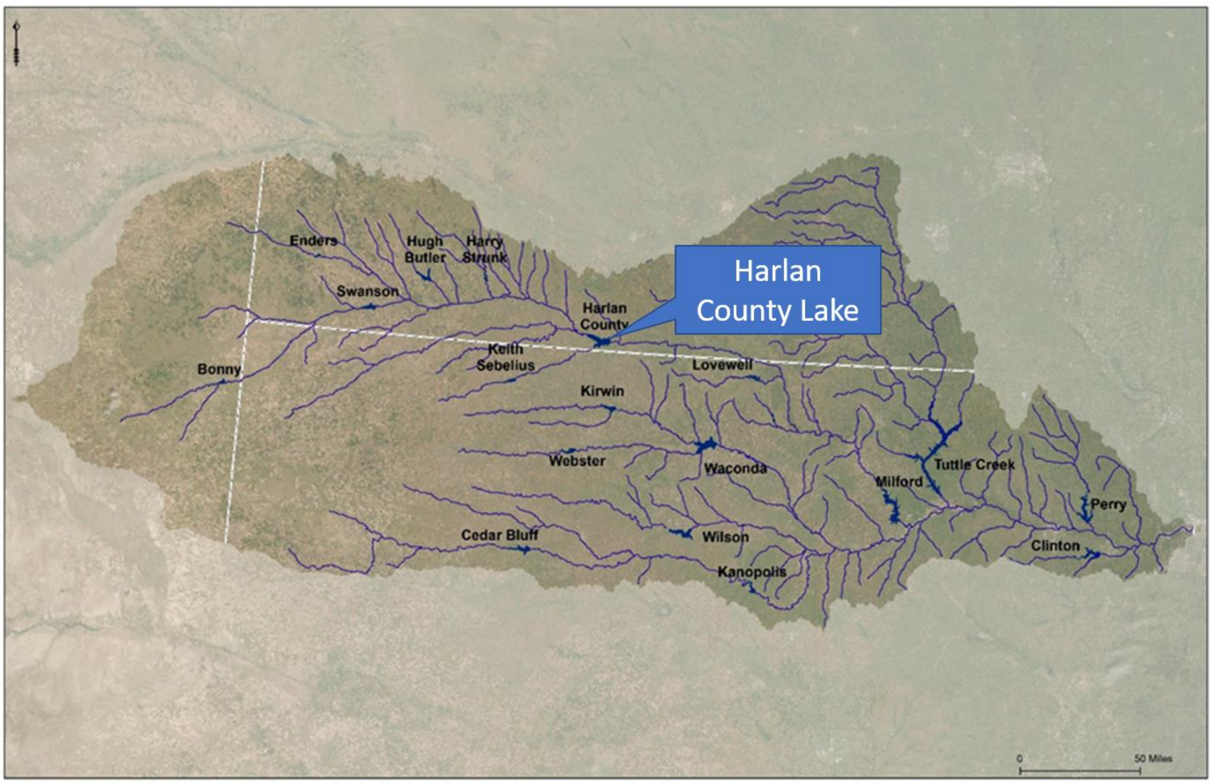


Figure 1-1: Overall Kansas River Basin and Harlan County Lake

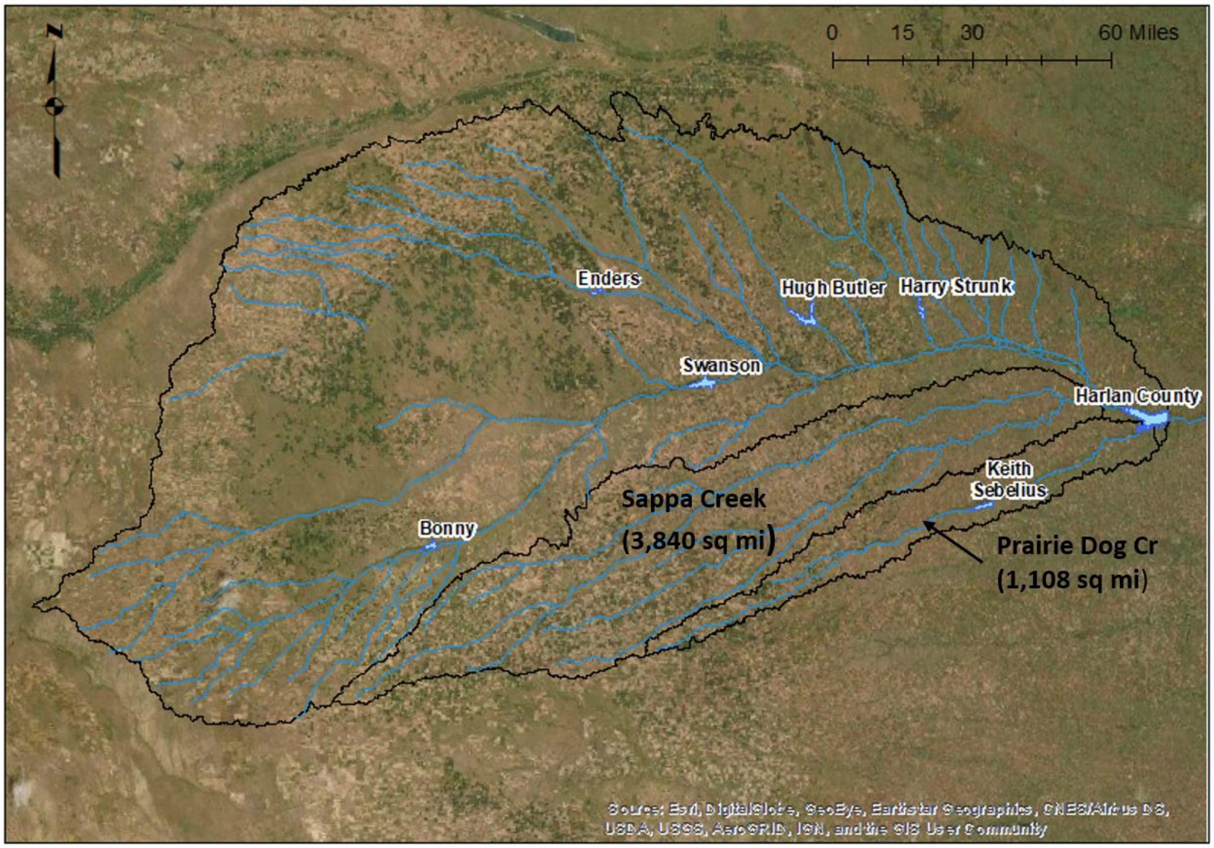


Figure 1-2: Republican River basin above Harlan County Dam

2.0 DAM INFRASTRUCTURE

Table 2-1 gives pertinent data related to the dam and Figure 2-1 shows the location of key features. Elevations are shown in both NVGD29 and NAVD88. A survey by the Kansas City District (NWK) in 2014 showed a 0.64 ft difference between NVGD29 and NAVD88 (USACE, 2014) at the lake's staff gage. This is smaller than the factor of 0.8 feet that was obtained from the USACE Corpscon conversion tool. The conversion factor obtained from the 2014 survey was used to convert from NVGD29 to NAVD88 in this section as this is considered the official conversion factor for the lake.

The outlet works consists of nine 5' x 9' sluice gates located at the center of the dam, each having invert elevations of 1885.0 feet NVGD29 (USACE, 1973). These have a discharge capacity of 17,370 cfs at a pool elevation of 1946.0 NVGD29 and all gates fully open. An ogee spillway controlled by 18 radial gates has an invert elevation of 1943.5 NVGD29 and a discharge capacity of 480,000 cfs at a pool elevation of 1975.5. Both the spillway and sluice ways discharge into a concrete stilling basin. There are also two irrigation outlet works for making releases into Franklin and Naponee Canals. The outlet for Franklin canal has a maximum discharge of 520 cfs at pool elevation 1946, while Naponee canal has a maximum discharge of about 40 cfs. Invert elevations for these outlets are given in Table 2-1. There was also a 12'x12' opening left in the left bulkhead that was plugged for future installation of a 9' steel conduit for potential hydropower. Discharge from the dam can be zero at times when water is being stored for irrigation.

Table 2-1: Important Information Relating to the Dam Infrastructure. Elevations in NVGD29 (NAVD 88 in parenthesis)

Parameter	Value
Multipurpose Pool Elevation	1945.73 (1946.37)
Lowest Elevation Outlet	1885.0 (1885.64)
Number of Gates at This Low Elevation	9
Days per Year Low Level Outlet Operated	140
Service Gates Used When Flows Exceed (cfs)	0
Spillway Elevation	1943.5 (1944.14)
Dam Elevation	1982.0 (1982.64)
Typical Tailwater Elevation	1896
Opening for Future Hydropower	1909.5 (1910.14)
Franklin Canal Invert	1920.0 (1920.64)
Naponee Canal Invert	1921.5 (1922.14)

Figure 2-1: Harlan County Dam and Infrastructure

Figure 2-5 shows the tailwater rating curve for the dam taken from the 2015 Periodic Inspection Report. The curve labeled Future DM was used along with the average release of 210 cfs to obtain the typical tailwater. Figure 2-3 through Figure 2-5 are drawings of the dam infrastructure.

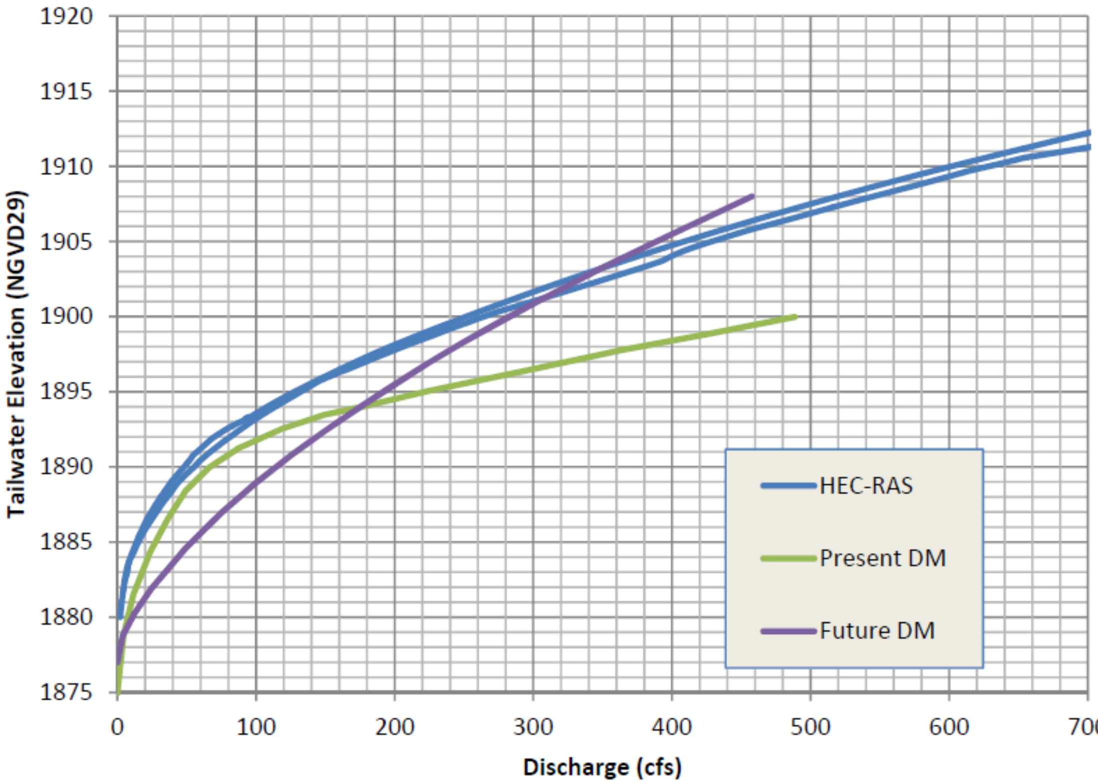


Figure 2-2: Typical Tailwater Elevation Curve (USACE, 2015). Elevations in feet NVGD29.

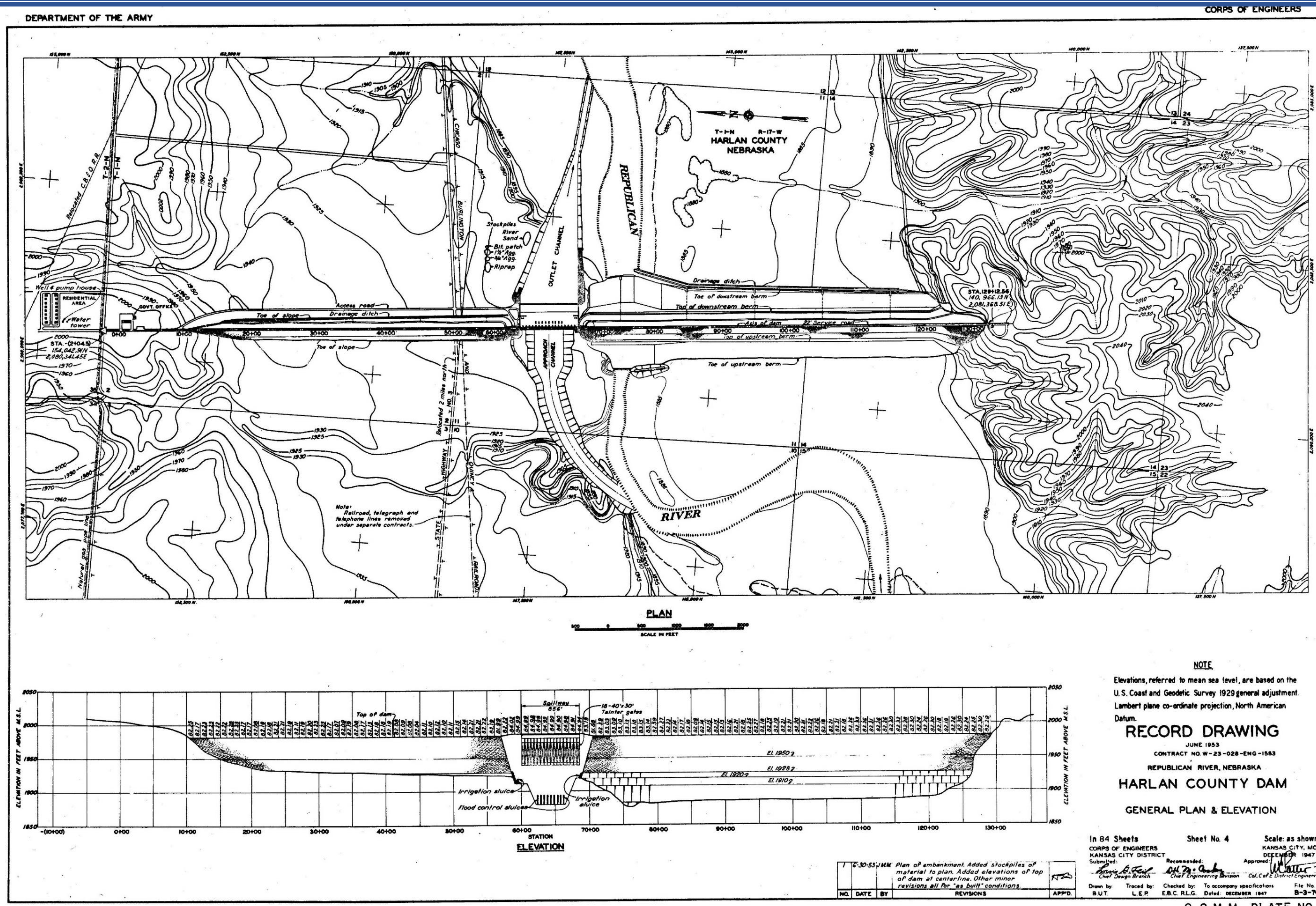


Figure 2-3: Kanapolis General Plan Drawing (USACE, 2015). Elevations in feet NVGD29.

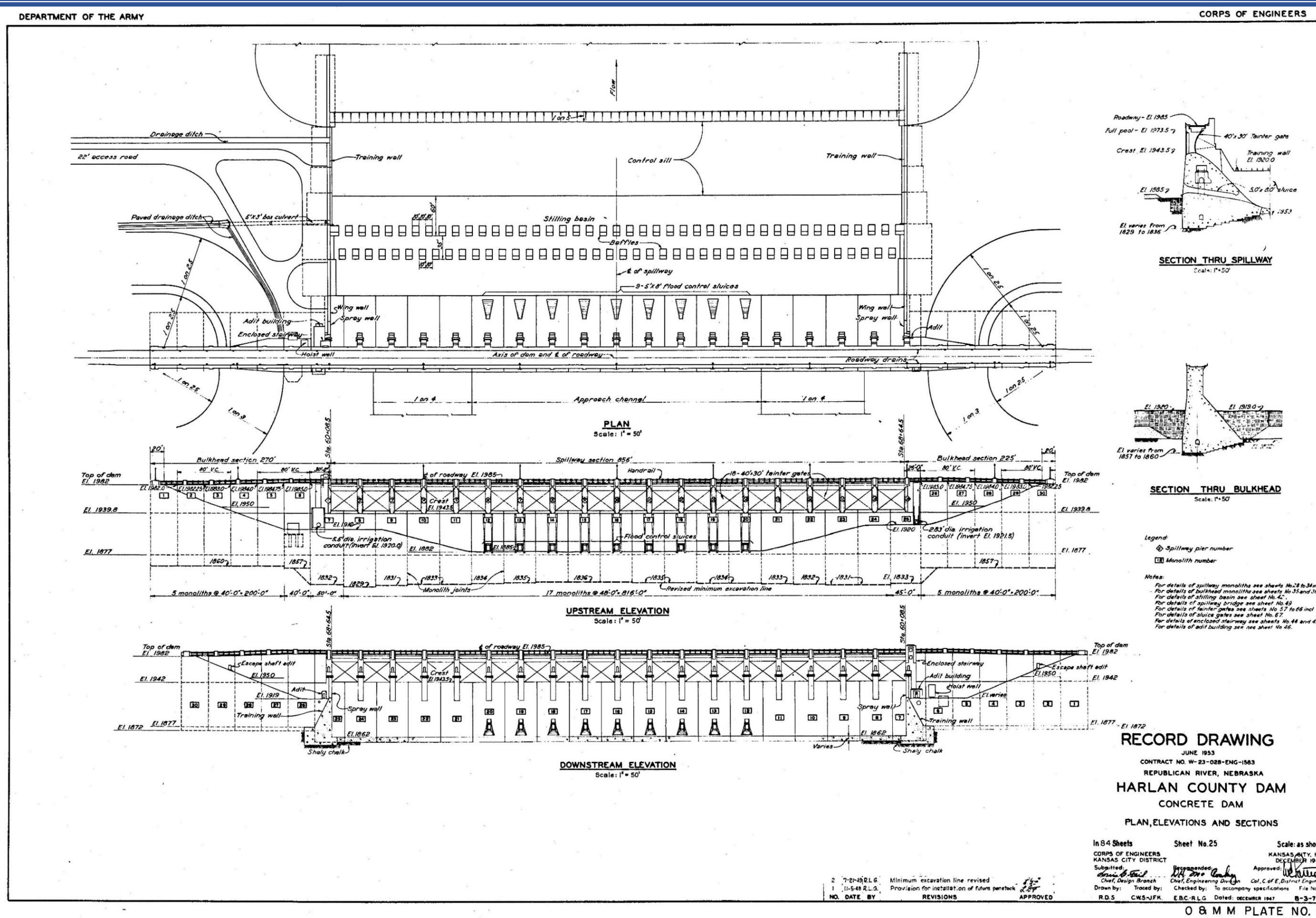


Figure 2-4: Concrete Dam Plan Elevations and Sections (USACE, 2015). Elevations in feet NVGD29.

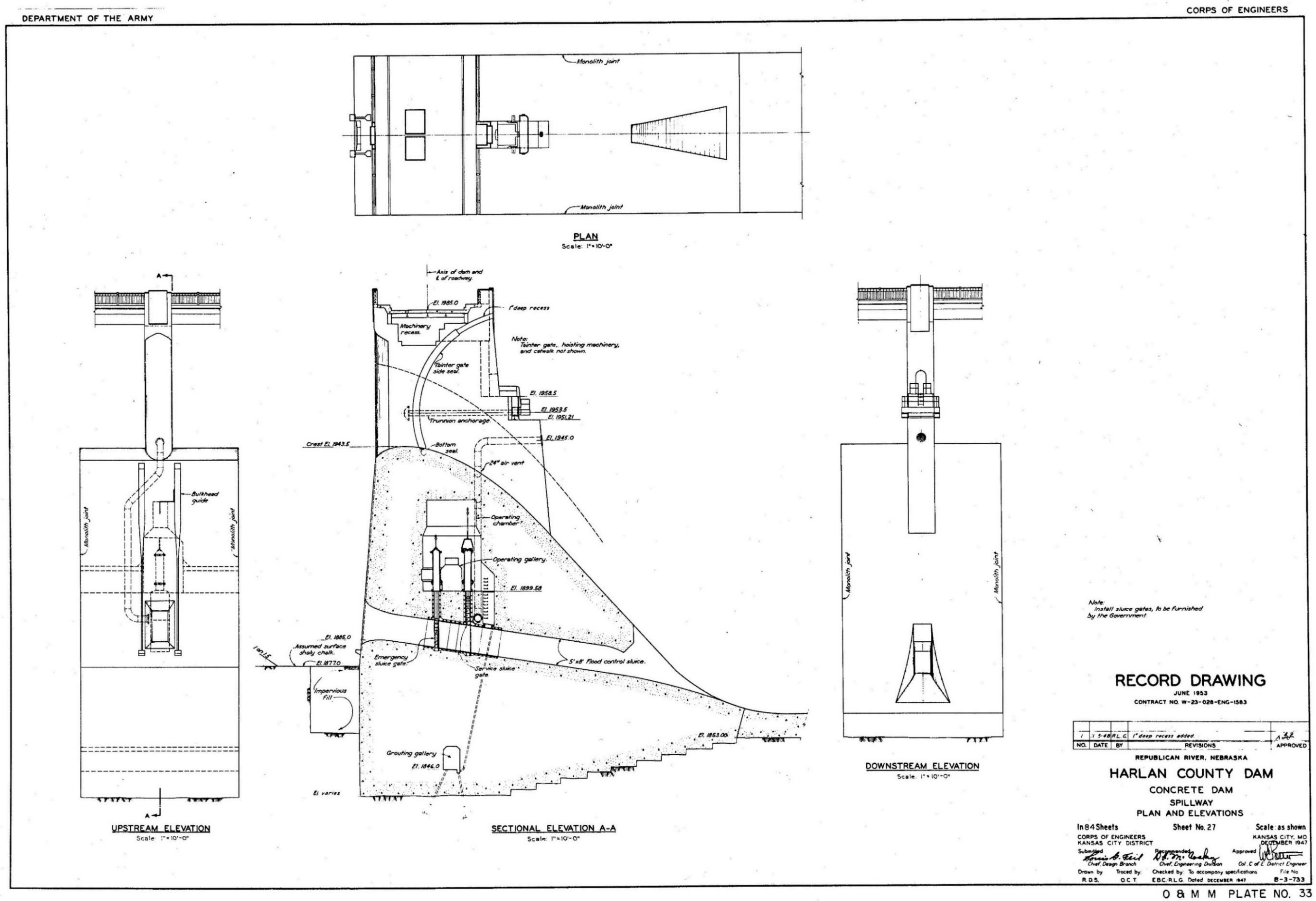


Figure 2-5: Harlan County Dam Spillway (USACE, 2015). Elevations in feet NVGD29.

3.0 SEDIMENTATION EFFECTS ON O&M

Sedimentation has had a significant impact on recreation and the operation and maintenance (O&M) expenses at Harlan County Lake. Sedimentation impacts have mainly been experienced in the lake's coves and marinas, where sediment buildup has restricted access to boaters and destroyed fish habitat. Much of the sediment has likely come from shoreline erosion within the lake due to wave erosion. In the 1970's congress approved a dredge for Harlan County to remove sediment from coves, boat ramps and marinas. The dredge must be operated on average ever 2-3 years to keep access open, and coves that have not been dredged are now completely inaccessible.

A design contract was awarded in 2022 for a cove restoration project at Methodist Cove on the north side of the lake. This project includes dredging to remove sediment that blocks the entrance of the cove, dredging to deepen the cove, building a sediment trap on the cove's incoming tributary, and the construction of baffles to minimize future accumulation at the cove entrance. Construction is anticipated to begin in late 2023.

4.0 EXISTING CONDITION STORAGE ELEVATION CURVES

The most recent storage elevation and area curves for Harlan County were created in 2001 from sedimentation rangelines surveyed in 2000. The location of the sedimentation rangelines can be found in Section 6.0 . Table 4-1 shows the final area capacity table for the 2000 survey.

Table 4-1: 2000 Storage Elevation Curve for Harlan County Lake NVGD29

Elevation	Volume (ac-ft)	Area (acre)	Elevation	Volume (ac-ft)	Area (acre)	Elevation	Volume (ac-ft)	Area (acre)
1895	116	232	**1918**	62,506	5,316	**1941**	254,418	12,076
1896	441	417	**1919**	67,900	5,472	**1942**	266,608	12,303
1897	960	620	**1920**	73,451	5,630	**1943**	279,002	12,484
1898	1,698	856	**1921**	79,162	5,792	**1944**	291,572	12,656
1899	2,672	1,091	**1922**	85,052	5,988	**1945.73**	314,111	13,305
1900	3,882	1,329	**1923**	91,149	6,206	**1946**	317,687	13,383
1901	5,296	1,498	**1924**	97,460	6,415	**1947**	331,217	13,677
1902	6,869	1,647	**1925**	104,026	6,717	**1948**	345,043	13,975
1903	8,597	1,808	**1926**	110,898	7,026	**1949**	359,170	14,279
1904	10,504	2,006	**1927.0**	118,099	7,375	**1950**	373,603	14,587
1905	12,609	2,203	**1928**	125,656	7,739	**1951**	388,347	14,901
1906	14,921	2,420	**1929**	133,576	8,101	**1952**	403,407	15,219
1907	17,499	2,735	**1930**	141,857	8,461	**1953**	418,788	15,543
1908	20,396	3,058	**1931**	150,480	8,785	**1954**	434,496	15,872
1909	23,563	3,276	**1932**	159,435	9,124	**1955**	450,536	16,207
1910	26,945	3,487	**1933**	168,715	9,435	**1956**	466,913	16,547
1911	30,531	3,685	**1934**	178,318	9,771	**1957**	483,633	16,892
1912	34,336	3,924	**1935**	188,253	10,099	**1958**	500,701	17,243
1913	38,397	4,198	**1936**	198,511	10,417	**1959**	518,122	17,599
1914	42,741	4,489	**1937**	209,068	10,696	**1960**	535,902	17,961
1915	47,362	4,753	**1938**	219,910	10,987	**1960.5**	544,975	18,145
1916	52,221	4,965	**1939**	231,030	11,253	**1973.5**	814,111	23,431
1917	57,276	5,144	**1940**	242,518	11,723	**-**	-	-

5.0 TRAPPING EFFICIENCY

In 2000 the multipurpose pool capacity is estimated at 314,111 ac-ft. The mean annual water inflow into Harlan County Lake from 1970 to 2019 was 129,743 ac-ft based on stream gage data. Brune offers the three curves shown in Figure 5-1 for estimating trapping efficiency, which can be calculated using Equations 1 and 2 and the constants given in Table 5-1 (Brune, 1953). Calculations show that the trapping efficiency of the reservoir was approximately 96.9% using the median curve.

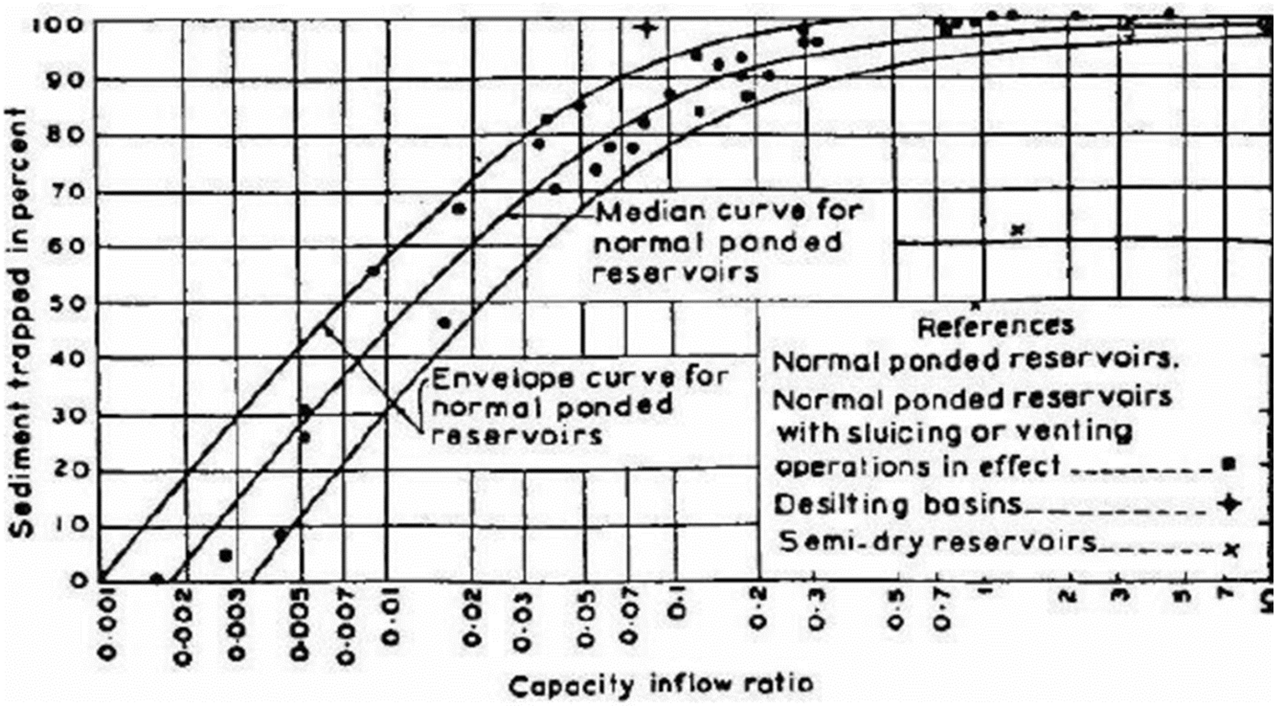


Figure 5-1: Brune Curves (Brune, 1953)

$$TE = a[1 - 2e^{-bV_*^{0.35}}] \quad (1)$$

$$V_* = \frac{V_{res}}{V_{inflow}} \quad (2)$$

Table 5-1: Constants to be used in Equations 1 and 2

Constant	Low	Medium	High
a	95	97	100
b	5.37	6.42	7.71

6.0 DEPOSITIONAL VOLUME

Historically sedimentation rangelines have been used to calculate the capacity of the reservoir and to determine sediment deposition. Harlan County Lake has a total of thirty-seven rangelines spaced at varying distances as shown in Figure 6-1. Reservoir capacity was calculated from rangeline surveys in 1951, 1962, 1972, 1988, and 2000.

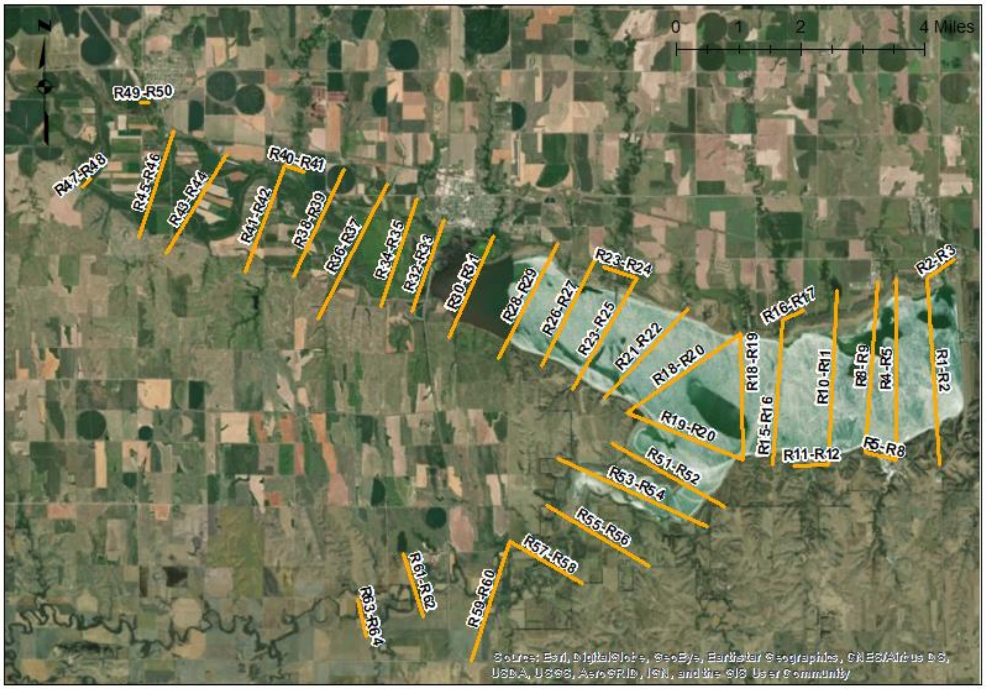


Figure 6-1: Harlan County Lake Sedimentation Rangelines

Bathymetric surveys were conducted on Harlan County Lake in 2000 and 2010 using single-beam sonar. Both surveys were collected by Eisenbraun and Associates with generally a transect spacing of 250 feet. The 2000 survey points are shown in Figure 6-2 while the 2010 points are given in Figure 6-3. The 2000 bathymetric survey with 250 foot transects was collected in addition to the rangeline survey mentioned previously. Digital Elevation Models (DEM) were created from the 2000 and 2010 bathymetric surveys of the multipurpose pool.

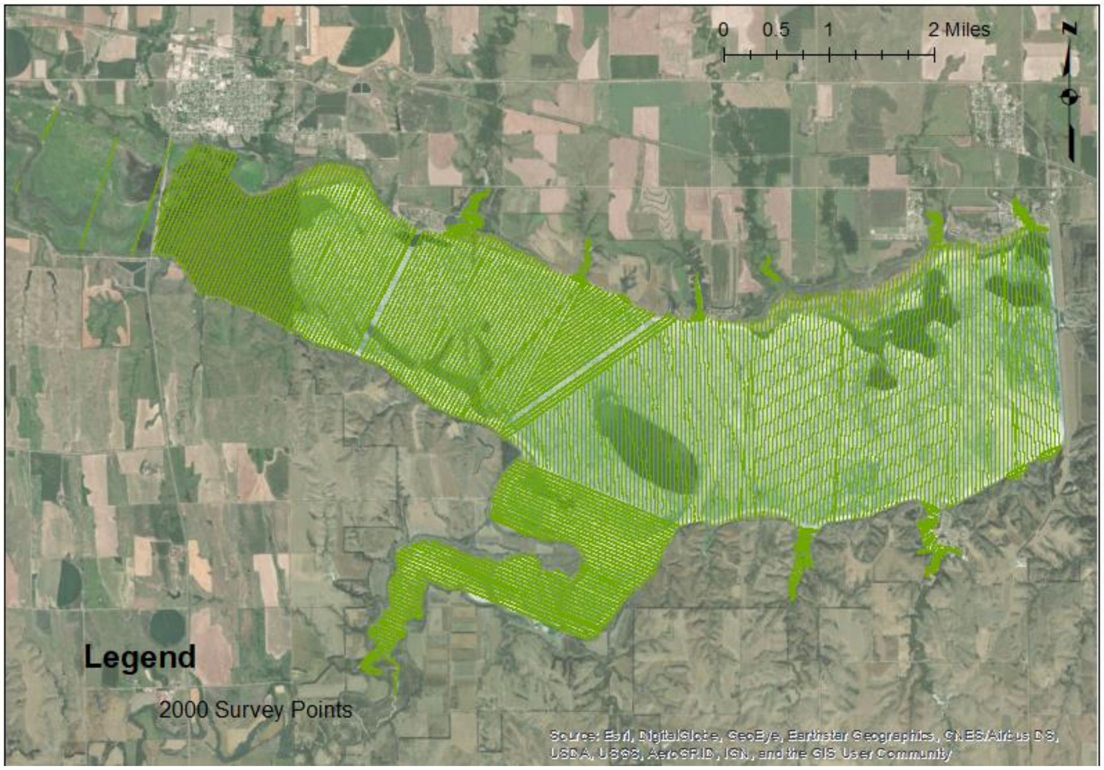


Figure 6-2: 2000 Bathymetric Survey Points

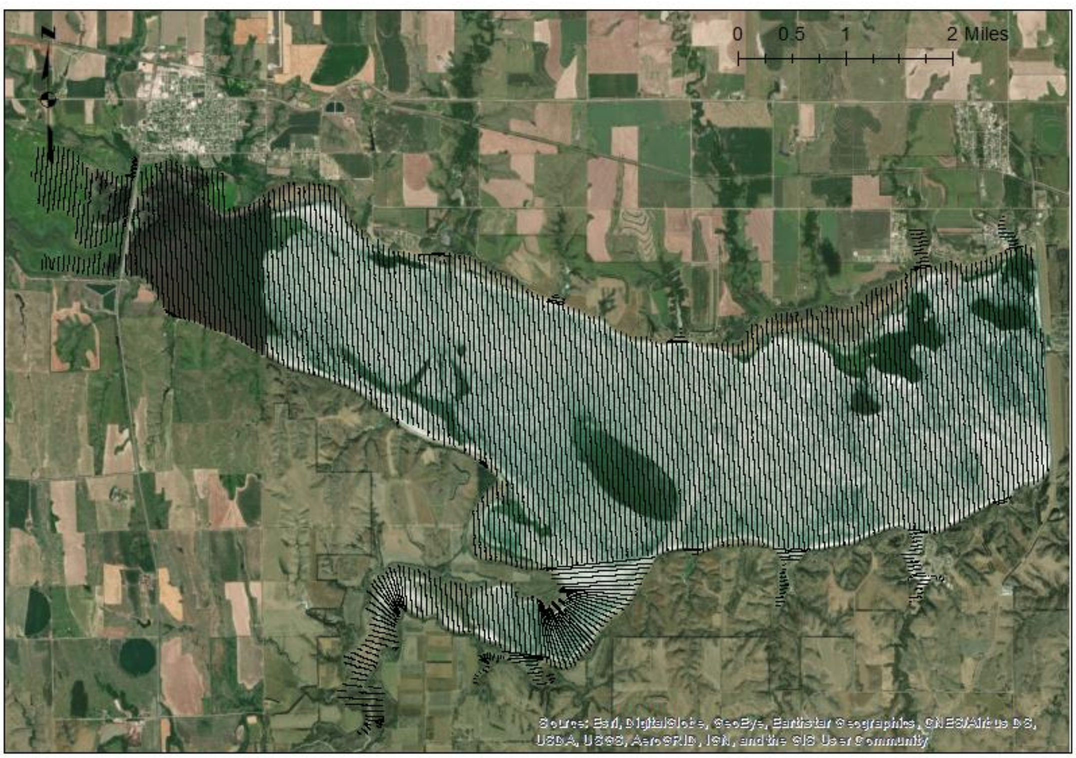


Figure 6-3: 2010 Bathymetric Survey Data Points

The 2010 survey displays inconsistencies in the bottom elevation with a noticeable shift in elevation between some of the transect lines. The shift appears to be generally about 2.5 feet and only occurs in the deepest part of the transect. Figure 6-4 shows the 2010 digital elevation model (DEM) created from the 2010 bathymetric points with the locations of the shifts in elevation labeled.

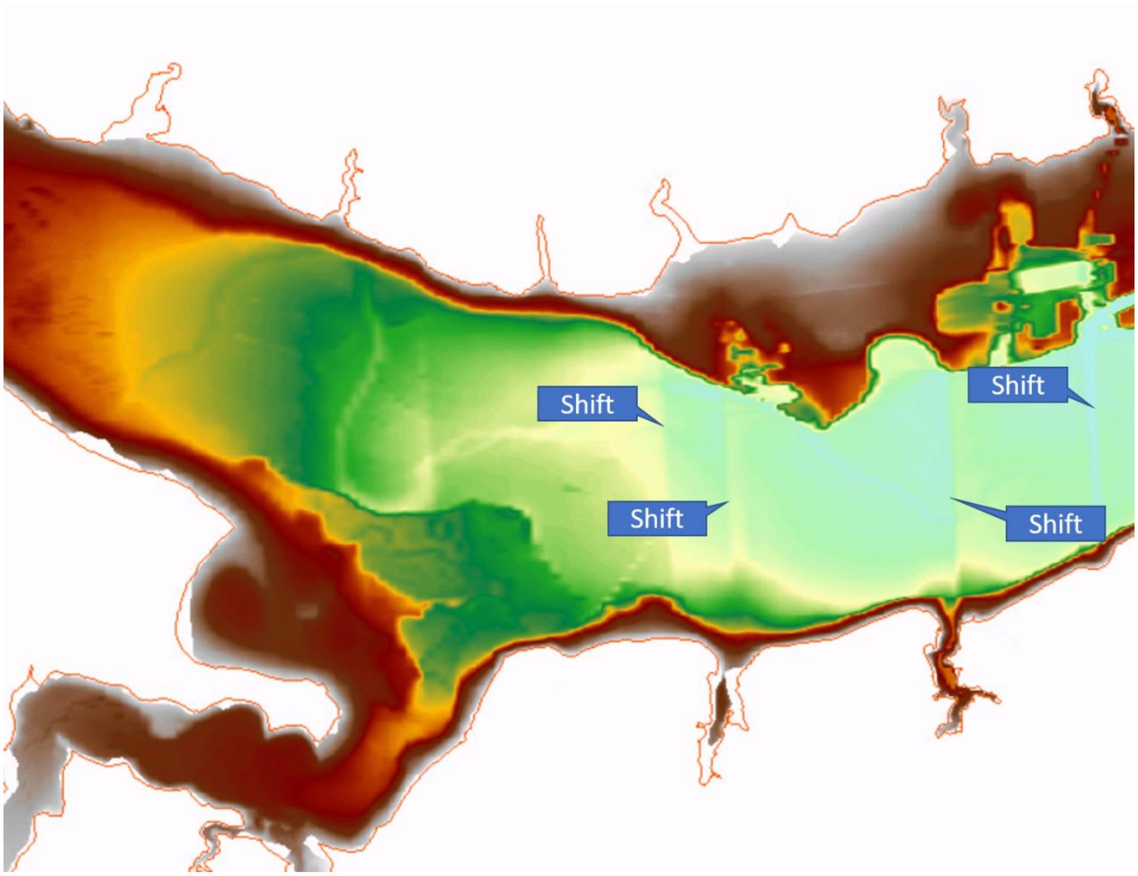


Figure 6-4: 2010 DEM showing inconstancies in bottom elevation

Measured water surface elevations are used to transform measured depths into elevations. These were checked for any errors, but they appeared to be correct. One possible explanation for the numerical artifacts are changes in the single beam sonar power and gain settings. Because the deepest deposits in the reservoir are composed of the lightest sediment, changes in the sounder settings can cause the sound waves to penetrate to different depths. This is explained in the following paragraph taken from Hydrographic Surveying Manual EM 1110-2-1003 (USACE, 2013).

> *In soft material, varied power and gain settings should be tested in the deeper part of the channel to determine if significant depth differences result from small setting changes. If so, then maintaining records of these settings will be critical for survey measurement & payment repeatability in these areas. In these soft bottom areas, obtaining repeatability between two different survey vessels (echo sounders) will be difficult, at best. Different echo sounders with different power/gain settings will yield constant depth biases over the same area, resulting in differing clearance assessment and pay quantities. In these problematic soft sediment channels, it is recommended that the same vessel (echo sounder) be used for all payment and clearance surveys.*

The 2010 survey was not used for this study because of these inconsistencies.

Table 6-1 provides the multi-purpose and flood control pool (FP) volumes over time, taken from the Harlan County area capacity tables along with the survey methodology.

Table 6-1: Pool Volumes over Time from Capacity Tables

Year	Multipurpose Pool Volume (ac-ft)	Flood Control Pool Volume (ac-ft)	Data Type
1951	346,512	503,488	Sedimentation Rangelines
1962	339,010	501,551	Sedimentation Rangelines
1972	324,065	501,716	Sedimentation Rangelines
1988	311,523	500,284	Sedimentation Rangelines
2000	314,111	500,000	Sedimentation Rangelines

There is considerable uncertainty in the amount of deposition that has occurred between each of the surveys due to different methods of calculation and inconsistencies in the surveys. Also, shoreline erosion has been determined to be a significant component of sedimentation, which further complicates estimating exact amounts of deposition. Table 6-2 gives estimates of total volume change, which includes both the flood control and multipurpose pool, computed using three different methods. Details on how these values were obtained are explained below.

Table 6-2: Estimated Total Volume Change (ac-ft)

Years	Change from Capacity Tables	Change from 1988 Study	Change from Cross Section Viewer	Corrected Change from Cross Section Viewer
1951-1962	9,439	17,614	12,739	16,796
1962-1972	14,779	10,308	8,764	11,555
1972-1988	13,974	13,153	9,650	12,723
1988-2000	-2,303	-	-	-
1972-2000	-	-	8,612	11,355

The deposition between each of the surveys can be determined by subtracting the pool capacities obtained from the storage elevation curves. However, this differs significantly from what was estimated by the 1988 sedimentation study. The 1988 study re-estimated the deposition from each of the previous surveys using modified cross sections to better estimate sedimentation. The study also estimated the amount of shoreline erosion using the sedimentation rangelines.

As a check, the modified cross sections from the 1988 study were obtained from data files located on storage discs and were imported into Cross Section Viewer software. The deposition between each of the surveys was calculated using the distances between the rangelines that were given in the 1988 study. Rangelines extracted from the 2000 DEM were also added into Cross Section Viewer since the full 2000 rangelines could not be located. This dataset does not include elevations above the flood pool. However, a USGS National Elevation Dataset (NED) was collected in March 2009 with a grid spacing of 1/9 arc-second, which was combined with the 2000 rangelines.

The Cross Section Viewer estimates are 72-85% lower than the estimates from the 1988 study, which is thought to be caused by a difference in calculation. Computations from the 1988 study included two distances when calculating depositional volume between each of the rangelines; it is thought that one was measured along the old river channel, and another was the straight-line distance between the rangelines. The Cross Section Viewer estimates were divided by a correction factor of 0.76 to correct for this difference, since the 1988 estimates were assumed to be more accurate.

Also, the 2000 rangelines are lower in elevation than the 1988 ranelines at many locations, which is likely why the capacity tables show negative deposition from 1988 to 2000. It is likely that either the 1988 or

2000 surveys are in error, as it is improbable that lake could increase in capacity. The 1962 sediment report does mention subsidence observed for portions of the lake because of consolidation of the loess soils in the area. The report estimates that this caused a gain in capacity of 1,350 ac-ft between 1951 and 1962, which is much smaller than the amount of sediment deposition. All the surveys are in the NVGD29 elevation datum, so the error does not derive from datum shifts. For this study, the 2000 survey was assumed to be correct since this is a more recent survey and agrees better with sedimentation calculations discussed later. However, considerable uncertainty remains because of the differences between the surveys. Another bathymetric survey would give greater confidence.

The estimated volume changes within the multipurpose and flood control pools can be seen in Table 6-3. However, these numbers do not represent the deposition coming from the drainage area above the lake, since shoreline erosion has contributed to the numbers below. Erosion appears to have created a net increase in capacity in the flood control pool between several of the surveys. Erosion estimates and the net sediment deposition are discussed in the next section.

Table 6-3: Flood Control and Multipurpose Pool Change in Volume (ac-ft)

Years	Capacity Tables MPP	Capacity Tables FP	Sediment Calculations MPP	Sediment Calculations FP
1952-1962	7,502	1937	14,225	3,084
1962-1972	14,944	-165	10,712	-407
1972-1988	12,542	1,432	13,333	-587
1972-2000	9,954	1,716	11,332	1,256
1988-2000	-2,587	284	-	-

From 1952 to 2000, the multipurpose pool lost 36,269 ac-ft of storage to sedimentation. This represents 10.5% of the original multipurpose pool volume. The average annual rate of loss was 756 ac-ft/year or 0.22% of the original volume/year.

From 1952 to 2009, the flood control pool lost 3,933 ac-ft of storage to sedimentation. This represents 0.78% of the original flood control pool volume. The average annual rate of loss was 69 ac-ft/year or 0.014% of the original volume/year.

7.0 SHORELINE EROSION AND NET SEDIMENT DEPOSITION

Shoreline erosion, which is a significant contributor to sedimentation within Harlan County Lake, has been estimated at various times using the sedimentation rangelines discussed in the previous section. The 1963 sedimentation report stated that there was 27 miles of shoreline that have been affected by erosion, which was estimated to be as much as 1,000 feet at one location. Total erosion was estimated at 2,700 ac-ft from the sedimentation rangelines.

The 1988 sedimentation study also investigated shoreline erosion around Harlan County Lake and determined the erosion volumes given in Table 7-1. The eroded volume includes erosion from both the multipurpose and flood control pools. It was assumed in the study that the material eroded had a bulk density of 105 pounds per cubic foot (pcf) and would deposit with a bulk density of 45 pcf, resulting in the deposited volumes also shown in Table 7-1.

Table 7-1: Erosion Estimates from 1988 Sedimentation Study

Years	Eroded Volume	Deposited Volume
1952-1972	2,329	5,427
1962-1972	1,183	2,756
1972-1988	2,500	5,825

Erosion volumes from the 2009 LiDAR and 2000 bathymetry were also calculated using Cross Section Viewer. When compared to the 1972 survey, calculations show there was 2,439 ac-ft of erosion from 1972 to 2000/2009, which is slightly less than the eroded volume from 1972 to 1988 shown above. This may be caused by a difference in calculation methods and may also indicate that erosion has slowed significantly over time. The volume eroded from the flood pool was estimated to be 452 ac-ft between 1972 and 1988, and it was 510 acre-feet between 1972 and 2009. Erosion from the multipurpose pool between 1972 and 2000 was assumed to be 2,500 ac-ft minus 452 ac-ft, or 2048 ac-ft. However, this is possibly an underestimation of bank erosion, because erosion from 1988 to 2000 was likely greater than zero.

Net sediment deposition from the drainage basin above the lake was estimated from the erosion volumes and the volume changes from the previous section. This was done using the following equation.

$$V_i = V_t + V_d - V_e \qquad (3)$$

Where:

V_i = sediment inflow from drainage area above the lake

V_t = total volume change given in Table 6-2

V_d = deposition from erosion

V_e = Volume of erosion

The net deposition from sediment inflow is given in Table 7-2.

Table 7-2: Net sediment inflow from the drainage area above the lake (ac-ft)

Years	Total Volume Change	Net Deposition from Erosion	Deposition from Sediment Inflow
1951-1962	17,614	3,098	14,516

1962-1972	10,308	1,573	8,735
1972-1988	13,153	3,325	9,828
1972-2000*	11,355	3,681	7,650

*Estimates are for the multipurpose pool

8.0 INCOMING SEDIMENT LOADS

USACE and USGS have sporadically collected paired flow/sediment concentration and load measurements at multiple gages upstream of Harlan County Lake. Gages that were used in this study are given in Table 8-1. Few sediment concentration measurements have been collected at the Prairie Dog Creek near Norton, Kansas, but this is the gage furthest downstream on this tributary.

Table 8-1: Stream Gages upstream of Harlan County Lake

Gage	USGS Gage No.	Drainage Area (sq mi)	Years Sediment Measured
Republican River near Orleans, NE	06844500	15580	1948-1970
Prairie Dog Creek near Woodruff, KS	6848500	1007	-
Prairie Dog Creek near Norton, KS	6848000	684	1948-1952, 1958, 1976, 1986
Sappa Creek near Stamford, NE	06847500	3370	1948-1953

Gage data from various gages within the Kansas River watershed indicate that the sediment rating curves generally begin flattening out at around the 83.3 to 66.7% (1/1.2 to 1/1.5) annual exceedance probability (AEP) discharge. Meaning the rating curve no longer follows the power fit trendlines when the discharge exceeds these values. These flows correspond with the typical range for bankfull flow in Kansas (Shelley 2012). The 1/1.2 and 1/1.5 AEPs are given in Table 8-2 for the gages upstream of Harlan County Lake for both 1970-2019 and pre-1970. Values for 1970-2019 are significantly lower than the pre 1970 values, which illustrates the decline in streamflow over time. However, the pre 1970 AEP discharges are only based on 23 to 29 peaks depending on the gage. Also, although Prairie Dog Creek has drainage area less than a third as large as Sappa Creek and is also partly controlled by Norton Dam, both the 1979-2019 and pre 1970 AEP discharges for Prairie Dog Creek are greater than those for Sappa Creek. This could be an indicator that the Prairie Dog Creek basin below Norton Dam is a significant contributor of flow and sediment to the lake. Only the data before the rating curves begin flattening was used to create the power fit trendlines.

Table 8-2: 83.3% (1/1.2) and 66.7% (1/1.5) AEP Discharges

Stream Gage	Pre 1970 83.3% AEP (cfs)	Pre 1970 66.7 AEP (cfs)	1970-2019 83.3% AEP (cfs)	1970-2019 66.7 AEP (cfs)
Republican River at Orleans, NE	2,380	3,240	550	920
Sappa Creek near Stanford, NE	680	1,080	100	170
Prairie Dog Creek near Woodruff, KS	850	1,320	180	320

The suspended sediment concentration was converted into a daily load by multiplying by the daily flow and a factor of 0.0027 to obtain the load in tons per day. The measurements for the Republican River at Orleans, Nebraska are shown in Figure 8-1 broken into the time periods in which they were collected. Discrete measurements are those taken sporadically, while daily data is a continuous record of measurements. There appears to have been shifts in the sediment load over time as indicated by the data. The sediment load is highest from 1948 through 1952 and then decreases for 1953 through 1964. For 1965 to 1970 the sediment load appears to have decreased for lower flow rates but increased at the higher flows. Surprisingly, the discrete measurements taken in 1948 appear to match well with the 1965 to 1970 data rather than the daily measurements from 1948-1952.

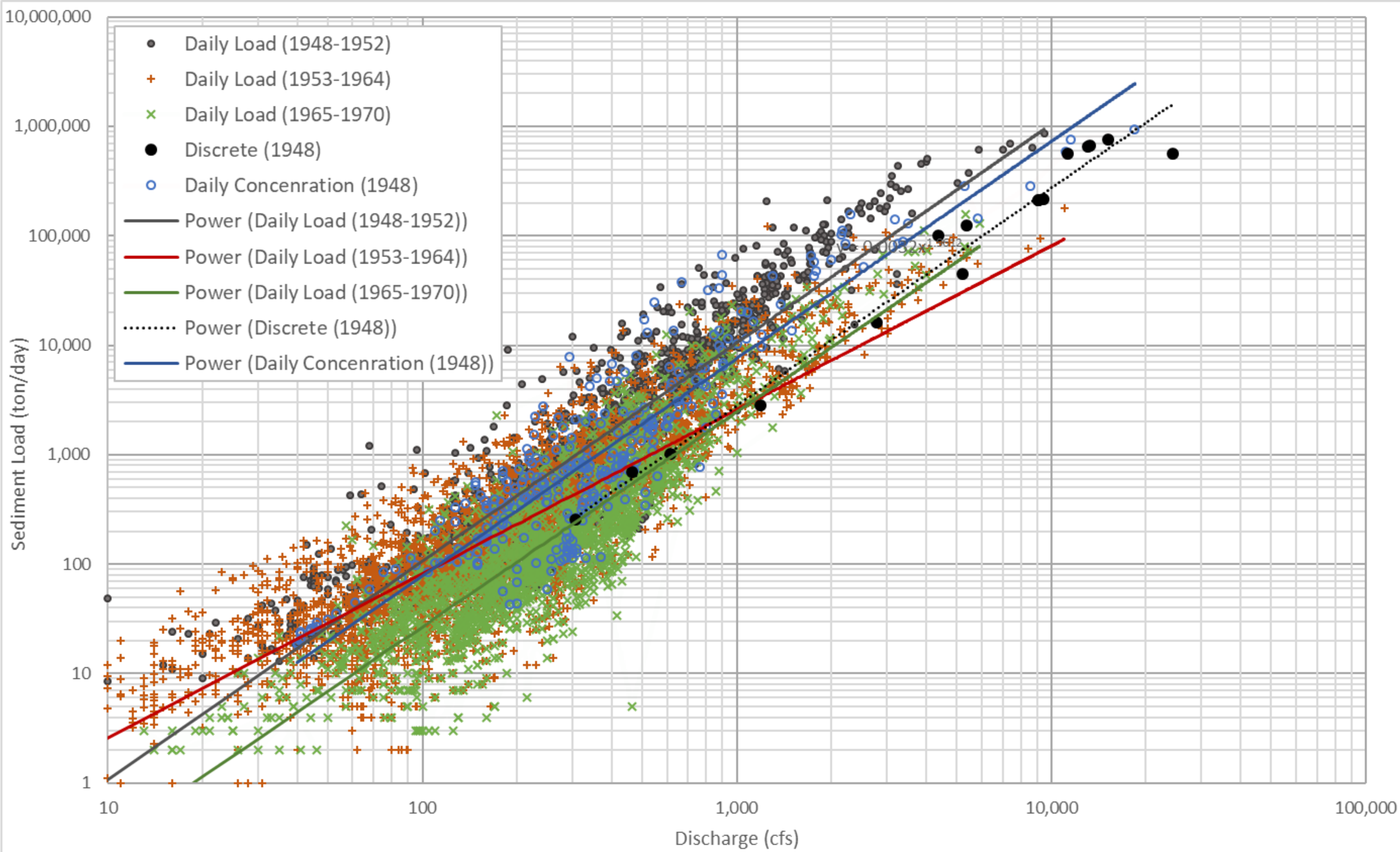


Figure 8-1: Sediment Load Measurements Taken at the Republican River at Orleans, Nebraska Gage

These shifts in rating curve could be caused by the construction of five USBR dams built upstream of Harlan County Lake between 1949 and 1964. Table 8-3 gives information for the USBR dams that have been constructed upstream of Harlan County Lake. Date from the 1965 through 1970 was used to construct the sediment rating curves, because this is after all the upstream USBR dams were constructed.

Table 8-3: Information for USBR Dams Constructed upstream of Harlan County Lake

Dam Name	Lake Name	Stream	Date Completed	Drainage Area (sq mi)
Medicine Creek	Harry Strunk	Medicine Creek	Aug-49	642
Enders	Enders	Frenchman Creek	10/23/1950	2240
Trenton	Swanson	Republican River	May-53	8620
Red Willow	Hugh Butler	Red Willow and Spring Creeks	9/5/1961	880
Norton	Keith Sebelius	Prairie Dog Creek	Dec-64	693

The measurements were also checked to see if there is a trend in the sediment load for different seasons of the year. Figure 8-2 shows the suspended sediment measurements that have been taken on the Republican River at Orleans, Nebraska broken into the seasons in which they were collected. There is a very strong correlation between the season of the year and sediment load at a given discharge. Summer has the highest sediment load followed by Spring and Fall. Winter has the lowest sediment load.

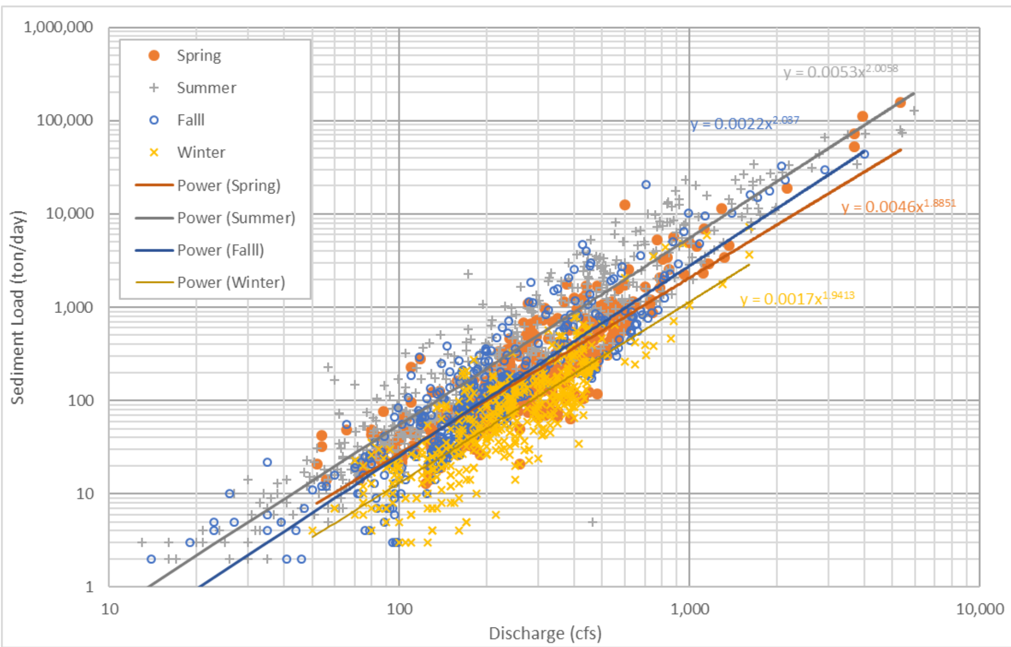


Figure 8-2: Suspended sediment measurements broken into seasons for the Republican River at Orleans, Nebraska (1965-1970)

Figure 8-3 shows the final total load sediment rating curve for the Republican River at Orleans, NE. The sediment rating curve was corrected for bias by comparing the 1965-1970 summed sediment load from rating curve to the summed total from the measurements. The measured sediment load was 1.89 times higher than the rating curve load so the rating curve was multiplied by this factor. It is difficult to tell if the flow/load curve flattens for the higher discharges. However, none of the measurements collected from 1965 ot 1970 were taken at a flow rate exceeding 6,000 cfs, so this adds to the uncertainty of the upper part of the rating curve. The discrete measurements from 1948, which align well with the 1965-1970 measurements, do not appear to show any flattening up to 11,000 cfs. For the 1965-1970 rating curve it was assumed there would be a slight flattening from 6,000 to 11,000 cfs, which would generally align with the 1948 discrete measurements. Six thousand cfs is higher than the pre-1970 66.7% AEP discharge of 3,240 cfs.

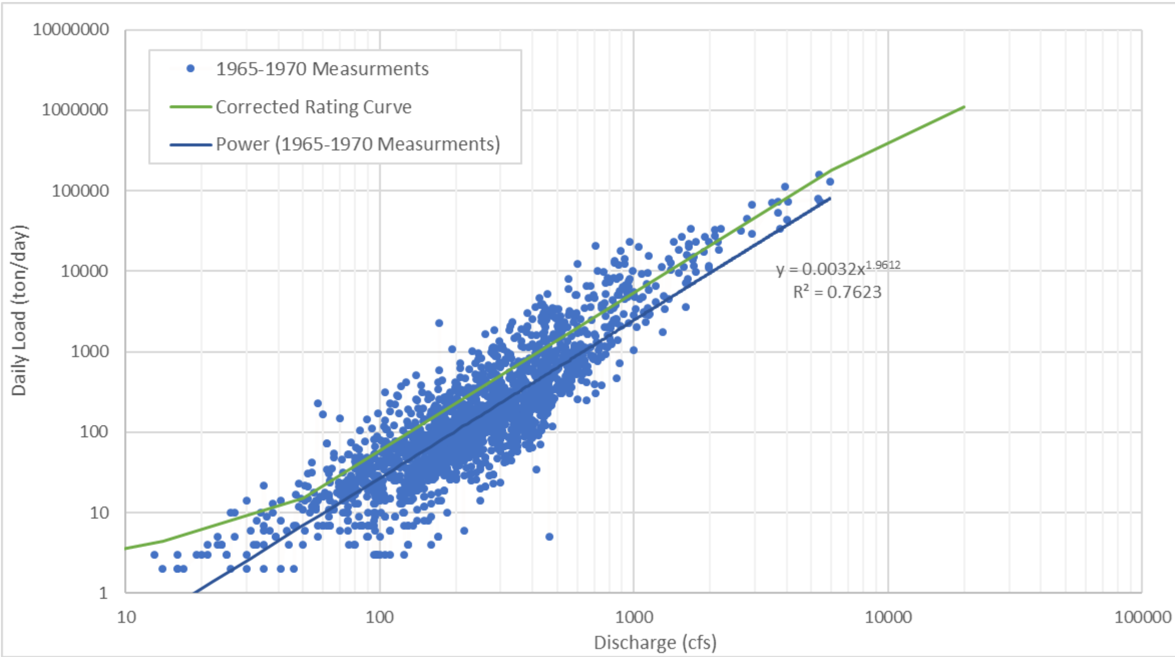


Figure 8-3: Total Load Sediment Rating Curve for the Repbulican River at Orleans, NE along with the suspended sediment measurements

Figure 8-4 shows the sediment measurements that were collected on Sappa Creek near Stamford, Nebraska from 1948 to 1953. The sediment rating curve, which is also shown in the figure, was created from the daily measurments and begins flattening at the pre-1970 83.3% AEP discharge of 680 cfs. However, since the AEP discharges have decreased since these measurements were taken, it is unknown if the break in the rating curve has also shifted. Another break also occurs at 10 cfs when the concentration begins flatting again at the lowest discharges. There have been no major dams constructed in the Sappa Creek watershed, so this will not have affected sediment loads. A bias correction value of 1.11 was determined using the same methodolgy as for the Orleans gage.

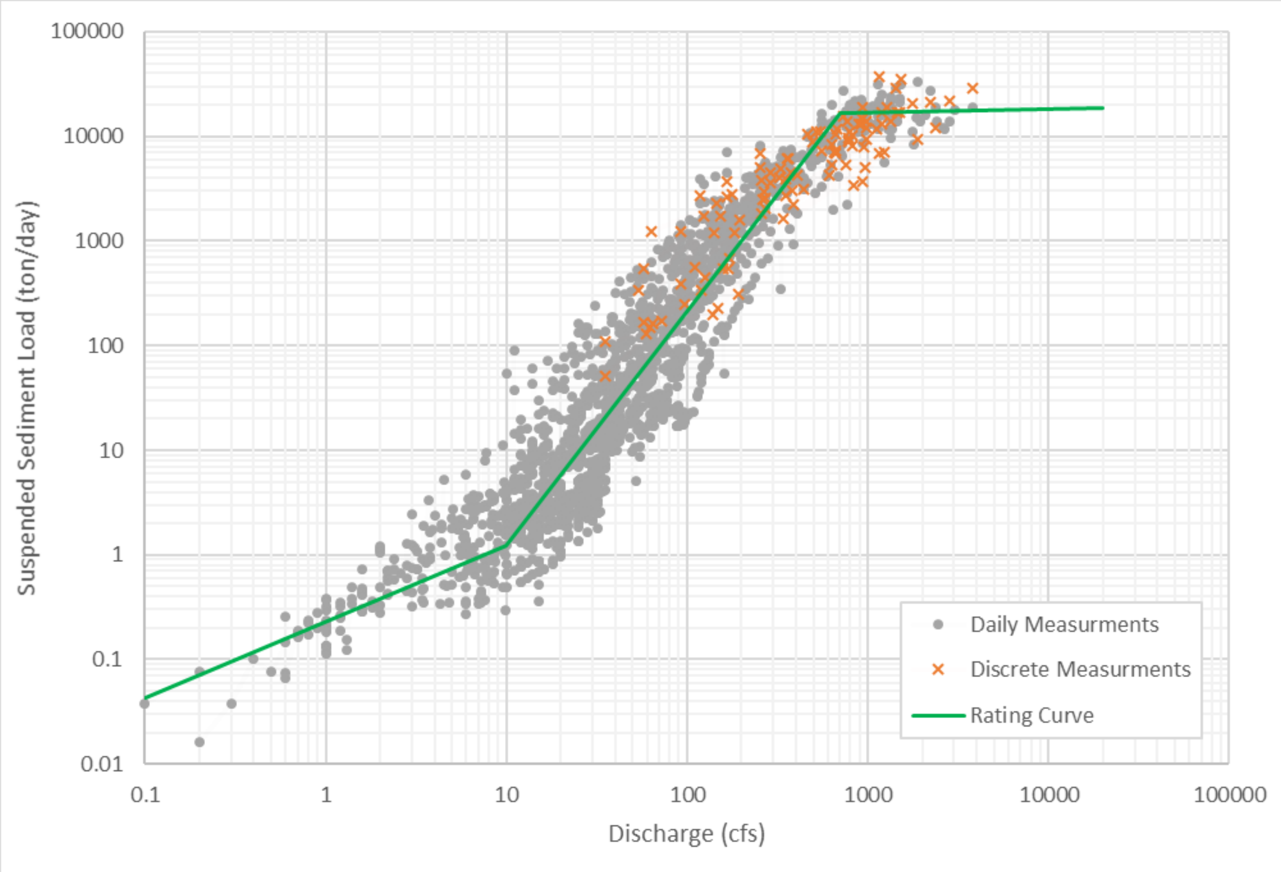


Figure 8-4: Suspended Sediment Measurements collected on Sappa Creek near Stamford, Nebraska 1948-1953 along with the sediment rating curve

Figure 8-5 shows the sediment rating curve and suspended sediment measurements for Prairie Dog Creek at Norton, Kansas. This gage is located upstream of the Woodruff gage. The measurements were collected daily and at discrete times from 1947 to 1952. Norton Dam was constructed just upstream of this gage in 1964, which means that sediment loads have been significantly reduced at this site. However, the sediment load from 1952 to 1962 was used to estimate the sediment runoff for the uncontrolled portion of the basin. Two breaks in the rating curve were used for the higher and lower discharges at 3 and 600 cfs respectively.

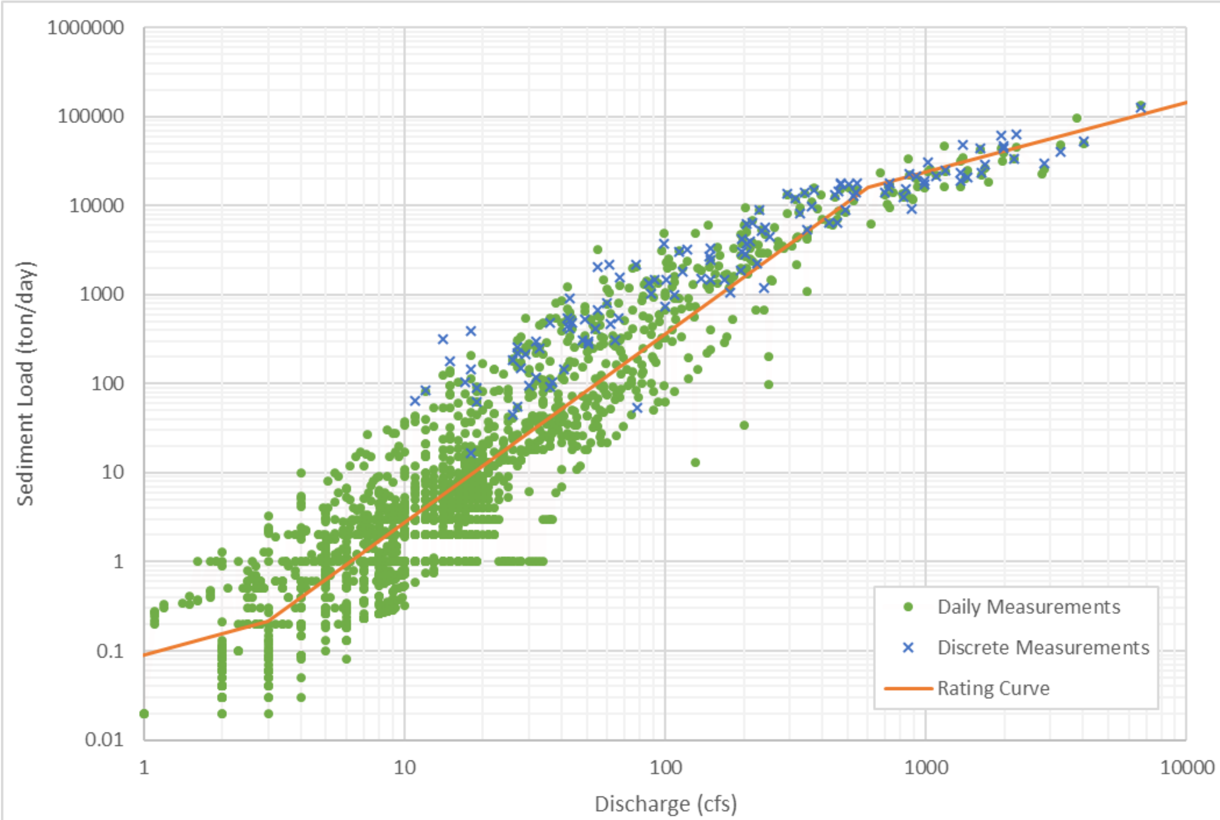


Figure 8-5: Sediment rating curve and suspended Sediment measurements collected on Prairie Dog Creek at Norton, Kansas

Table 8-4 summarizes the coefficients that were used to create the sediment rating curves.

Table 8-4: Coefficient for sediment rating curve above Harlan County Lake

Gage	a	b	Correction Factor
Republican River at Orleans, NE	0.0032	1.96	1.89
Sappa Creek near Stanford, NE	0.0067	2.24	1.11
Prairie Dog Creek near Woodruff, KS	0.018	2.12	1.18

The USGS also periodically measures the suspended sediment gradations of the samples they collect. These were plotted versus the discharge when they were collected to detect trends in the sediment size with discharge.

There have been few measurements of the suspended sediment gradation on the Republican River at Orleans, Nebraska. The limited available measurements do not indicate a correlation between sediment gradation and discharge (see Figure 8-6). Because of the lack of data, it was assumed that the sediment gradation remains constant with discharge.

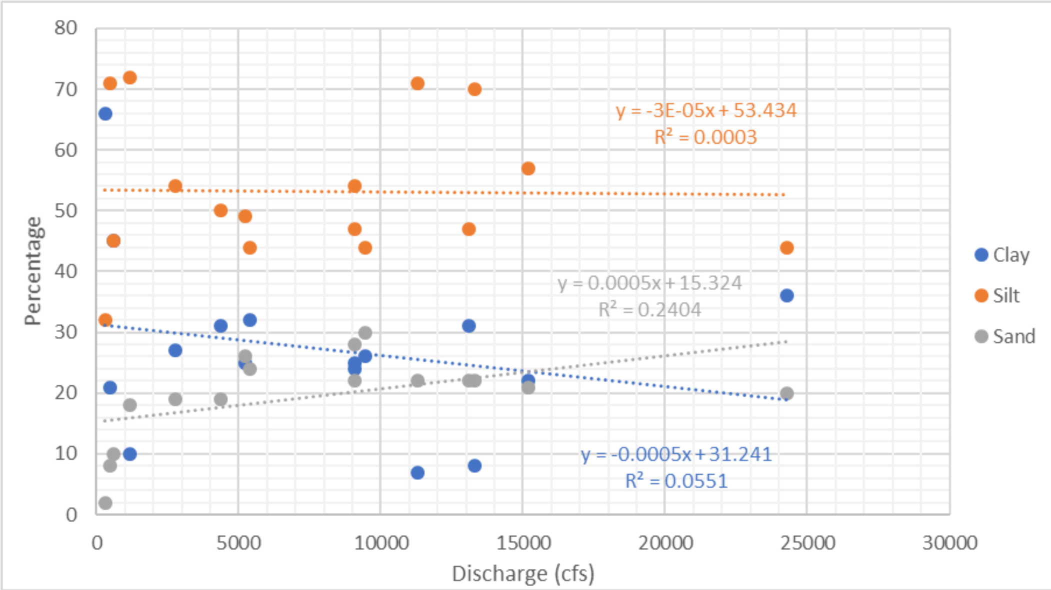


Figure 8-6: Suspended Sediment Size vs Discharge on the Republican River at Orleans, Nebraska

There appears to be a correlation between sediment size and discharge on Sappa Creek near Stamford, Nebraska as shown in Figure 8-7. The percentage of silt declines as flow increases while the percentage of clay increases. However, there is a significant amount of scatter in the data with low R-squared value.

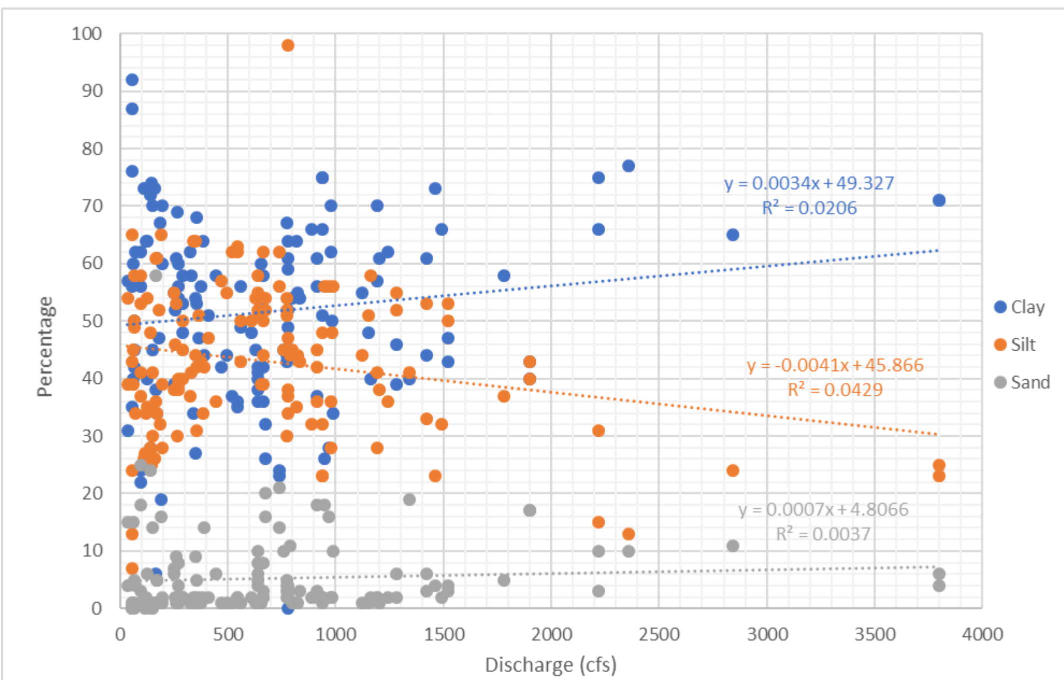


Figure 8-7: Suspended Sediment Size vs Discharge on Sappa Creek near Stamford, Nebraska

Sediment sizes measured on Prairie Dog Creek at the Norton gage do not shown any significant trend in sediment size with discharge (Figure 8-8)

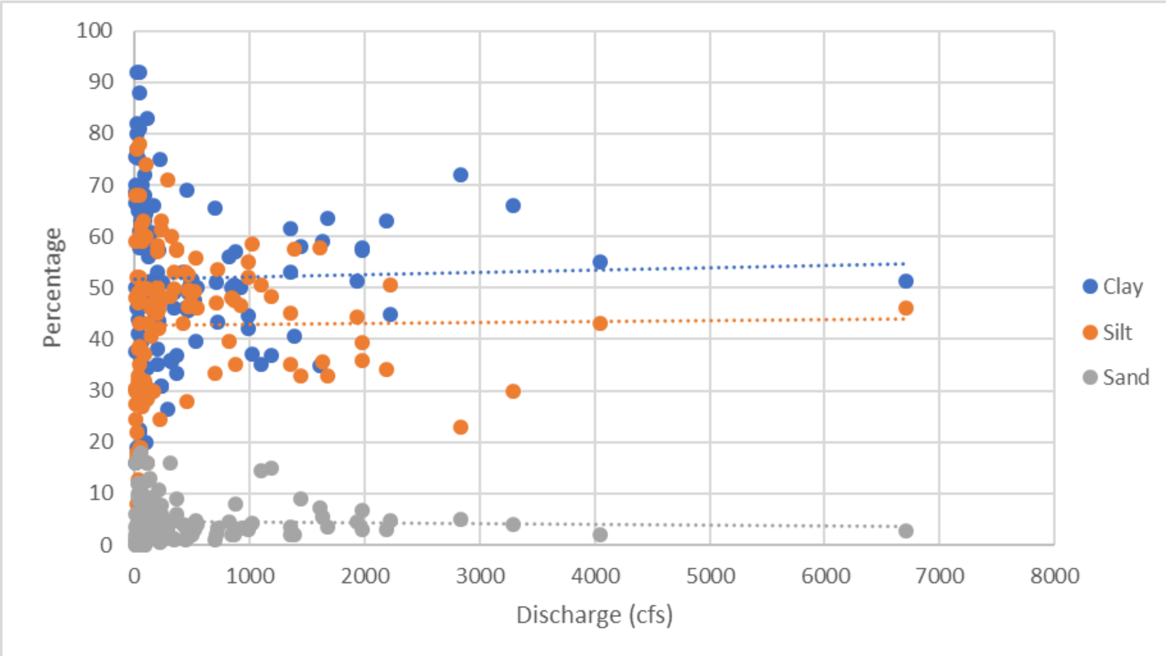


Figure 8-8: Suspended sediment sizes for Prairie Dog Creek at Norton, Kansas

Bedload was computed in the 1988 sedimentation study. Based on these calculations, the bedload was estimated to be 17% of the suspended load.

A total load rating curve was developed through the following steps:

1. Estimate the best fit regression line of the form $Q_s = aQ^b$ using log-log linear regression.
2. Correct for bias using the correction factors determined above.
3. Add 17% to account for bed load to create a total load rating curve.
4. Using the measured data, estimate the percentages of clay, silt, and sand/gravel.
5. Apply this rating curve to daily flow rates from 1952 to 2019 to determine the cumulative mass of sediment entering the reservoir.
6. Multiply by the appropriate percentages to determine the cumulative mass of clay, silt, and sand entering the lake.

Table 8-5 gives the sediment loads for the three main tributaries of Harlan County Lake. Sediment loads were not determined for Prairie Dog Creek at Norton after 1964 since Norton Dam was constructed in December 1964. The trapping efficiency of Norton Dam was estimated to be 97% using the Brune Curves, so the drainage area above the dam is likely an insignificant contributor of sediment to Harlan County Lake.

Table 8-5: Sediment loads from the major tributaries of Harlan County Lake

Time Period	Republican River (ton)	Sappa Creek (ton)	Prairie Dog Creek at Norton (ton)
1952-1962	7,516,321	2,120,598	1,587,978
1962-1972	3,833,985	1,973,187	N/A
1972-1988	2,628,948	301,127	N/A
1988-2000	1,574,499	345,637	N/A

It can be seen from the Table 8-5 that Sappa and Prairie Dog Creeks are significant contributors of sediment to Harlan County Lake. A ratio to correct for the ungauged area was calculated from the 1952-1962 sediment loads shown above and the basin drainage areas. The ungauged area not controlled by the USBR dams was determined to be 635 square miles. For the 1952-1962 period the drainage area above Prairie Dog Creek was determined to have produced 2,322 tons per square mile. This was multiplied by the 635 square miles to obtain a total sediment load of 1,474,220 tons, which was 15% of the Sappa Creek and Republican River loads for this period. For all the time periods, the total load from Sappa Creek and the Republican River was multiplied by 1.15 to account for the ungauged area.

This analysis found the incoming load to be 30.9% clay, 49.6% silt, and 19.5% sand/gravel. Table 4 summarizes the results for 1953 to 2019.

Table 8-6: Incoming Sediment to Harlan County Lake from 1953 to 2019

Years	1953-2020
Total Incoming Sediment (tons)	30,929,338
Total Incoming Clay Fraction	30.9%
Total Incoming Silt Fraction	49.6%
Total Incoming Sand Fraction	19.5%

9.0 BULK DENSITY AND CONSOLIDATION OF SEDIMENT DEPOSITS

Tests in 1988 from sediment cores indicated bulk densities of 20.8 to 87.7 pounds per cubic foot. There appeared to be an increase in density moving upstream in the reservoir. A weighted multipurpose pool bulk density of 44.6 pcf was obtained using the bulk density measurements and the sediment deposition estimates. The flood pool bulk density was estimated to be 66.0 pcf. The weighted bulk density for both the flood control and multipurpose pools was estimated to be 49.3 pcf.

The incoming load can also provide an estimate of bulk density, via equation 1.

$$\boldsymbol{\gamma_c} = \frac{\mathbf{1.0}}{\left(\left(\frac{F}{\gamma}\right)_{clay} + \left(\frac{F}{\gamma}\right)_{silt} + \left(\frac{F}{\gamma}\right)_{sand}\right)} \qquad (1)$$

where γ_c is the composite bulk density

F is the fraction of clay, silt, or sand

γ for clay, silt and sand is assumed to be 30 pcf, 65 pcf, and 93 pcf respectively.

By equation 1, the bulk density of the incoming load is 49.9 pounds per cubic foot. This agrees well with (only 0.6% higher than) the overall bulk density of 49.3 pcf from the core measurements.

10.0 CALIBRATING LOAD AND DEPOSITIONAL PARAMETERS

Sufficient data exists to calibrate the rating curve by comparing the deposition computed using the incoming sediment loads to the deposition computed using the surveyed volumes. This was accomplished following these steps:

1. Determine the trapping efficiency of the reservoir (in this case 97%).
2. Apply daily flows at the upstream gages to their respective rating curves.
3. Apportion the deposition into the multipurpose pool or the flood control pool. Multiply the deposition found in Step 4 by a factor, *m* such that the ratio of MPP deposition to total deposition is correct per the survey analysis. *m* was computed from the surveyed deposition and the bulk densities determined for the MPP and FP.
4. Repeat steps 2 through 3 for each day over a period of time to obtain the cumulative sediment inflow.
5. Compute the mass of trapped sediment in the multipurpose pool by applying the trapping efficiency to the incoming sediment.
6. Transform the trapped mass to a deposited volume by using the bulk densities determined in Section 10 (44.6 pcf in the MPP and 66 pcf in the FP.)
7. Compare the total rating-curve-based deposition to the deposition calculated from the surveyed volumes.
8. Adjust the sediment rating curve (described below) to more closely match the surveyed deposition

Table 10-1 summarizes this analysis.

Table 10-1: Parameters used in estimating deposition into Harlan County Lake

Parameter	Initial
FP Bulk Density (lb/cu ft)	66.0
MPP Bulk Density (lb/cu ft)	44.6
Bedload	17%
Ungauged drainage area correction	1.15
a in $Q_s = aQ^b$	See Table 8-4
b in $Q_s = aQ^b$	See Table 8-4
Average trapping efficiency	97%

The computed and the surveyed volumes of deposited sediment are shown in Table 10-2. The surveyed volumes are those given in Table 7-2.

Table 10-2: Surveyed and initially computed volume of sediment deposits from drainage area above the lake

Period	Surveyed Deposition (ac-ft)	Computed Deposition Initial	Computed / Surveyed Initial
1952-1962	14,516	10,974	0.76
1962-1972	8,735	6,632	0.76
1972-1988	9,828	3,317	0.34
1972-2000*	7,650	5,490	0.70

*Estimated from multipurpose pool survey. Does not include flood pool deposition.

The computed deposition from 1952 to 1962 is 24% lower than the surveyed deposition. However, since the sediment rating curve was higher for 1952-1962, this is not the best time period over which to compare the rating curve computation. It is most appropriate to compare the computed deposition from the rating curves to the 1962-1972 surveyed deposition, as this is when the rating curve measurements were collected. The computed deposition is also 24% lower than the surveyed deposition over this time period.

As mentioned before, the 1972-1988 surveyed deposition has possibly been overestimated because of errors in the 1988 survey (the 2000 surveyed elevations are lower for many areas of the lake). This would explain why the computed deposition is less than half of the surveyed. The sediment computations for 1972 to 2000 are 30% lower than surveyed values, which may be partly caused by an underestimation of the volume of shoreline erosion.

There are many factors that could be causing the rating curve calculations to be lower than the surveyed values. Sediment measurements for Sappa Creek and Prairie Dog Creek were taken in the 1940s and early 1950s, and sediment loads may have increased from these tributaries. Other factors that could be causing the calculations to be low are the bulk densities, the estimated percentage of mass that deposits in the flood pool, and the correction factor for the ungauged drainage area.

The sediment rating curves were calibrated to the 1962-1972 survey by multiplying by a factor of 1.32 (1/0.76). The final deposition calculations are given in Table 10-3. The surveyed and computed deposition matches well for all the time periods except 1972-1988.

Table 10-3: Surveyed and calibrated volumes of sediment deposits from drainage area above lake

Period	Surveyed Deposition (ac-ft)	Computed Deposition Calibrated	Computed / Surveyed Calibrated
1952-1962	14,516	14,454	1.00
1962-1972	8,735	8,735	1.00
1972-1988	9,828	4,369	0.44
1972-2000*	7,650	7,030	0.93

*Values are for the multipurpose pool

11.0 SEDIMENT TRAPPING DURING FLOOD CONTROL VS. MULTIPURPOSE POOL OPERATIONS

The calculated sediment inflows into the Harlan County Lake were used to estimate the deposition based on when the lake was in flood control operations (i.e., with a water surface above the multipurpose pool elevation), vs. multipurpose pool operations (i.e., when the water surface is at or below the multipurpose pool elevation). Table 11-1 shows the results of this analysis which indicates that most of the deposition occurs during flood control operations. Unlike other lakes within the Kansas River Basin, Harlan County Lake does not have a water level management plan.

Table 11-1: Deposition Amounts during Flood Control and Multipurpose Operations

Deposition	Deposition 1957 - 2019 (ac-ft)	Percent
Flood control Operations	23,838	70.4%
Multipurpose Operations	10,039	29.6%
Total Deposition	33,878	100.0%

12.0 SEDIMENT CONCENTRATIONS

Figure 12-1 illustrates the concentration of incoming sediment, together with the 80% confidence intervals. This graph represents the range of natural variability in the sediment concentrations in the river, i.e., what the concentration would be if the dam were not in place. As a first approximation, the lower bound can be thought of as a minimum target for naturalizing downstream sediment levels, and the upper bound can be thought of as a maximum limit to avoid excessive sediment releases.

The flow/concentration relationship is not monotonic or with a consistent slope. Rather the relationships in log space at low flows, moderate flows, and high flows exhibit separate slopes. By observation, at many of the lakes the sediment concentrations actually reverse at higher flows. This behavior translates into flatter flow-low curves as described earlier in the document. While the reasons for this phenomenon are unknown, it could be can be explained by either the supply limitation of easily erodible sediments or by sediment lost to the floodplain during overbank flows. A fourth-order polynomials through log-transformed data were used to reflect the overall trends in the data. This curve is a valid fit over the range of observed data but should not be used for extrapolation.

Also evident in the data is a reduction in the variability in concentration at higher flows. While fewer measurements could by itself lead to the perception of less variability, visual inspection of the flow/concentration measurements suggests that physical reasons may drive the lower variability. The first possible reason for the reduction in variability is the supply limitation that drives the reduction in concentration. Higher concentrations are constrained by lack of readily available material. A second explanation for lower variability is that moderate flows can be achieved by a precipitation in only part of the watershed, and different sub-watersheds may have different sediment contributions. On the other hand, very high flows are only achievable when most of if not all the entire watershed contributes, which reduces the spatial variability based on storm placement.

The confidence intervals were originally computed based on the statistics of the total sample and departure from the best-fit polynomial. However, the height of the intervals was driven by the high variability at moderate flows, which yielded confidence intervals at the highest flows that exceeded all the measured data points. A more refined approach was taken to take into account differences in variability as a function of flow.

The 4th-degree polynomial through log-transformed data was used as a predictor for the mean of a normal distribution in a generalized additive model. To capture the changing variance of concentration with respect to flow, a concave-down function was needed to predict the sigma term in order to produce a local maximum in the middle with decreases in variance at both extremes. A quadratic function was used to ensure d2σ/dx2<0,∀x. The result is a six-parameter model (plus two intercepts) in a hierarchical structure.

The interpretation should be looked at as "the conditional distribution of suspended sediment concentration, given an amount of flow." That is, f(y|x)=N(μ=b4x4+b3x3+b2x2+b1x+b0, σ=a2x2+a1x+a0) where y is SSC, x is flow, and N is the normal distribution with parameters μ and σ. Because the conditional distribution is normal, the best estimate of y|x is E[y|x] and is also the median of the conditional distribution, and it can be computed as μ. To obtain percentiles of the conditional distribution, the quantile function (inverse CDF) is used, F-1(p;μ,σ) where μ and σ are computed for the given value of flow. The result is overall a function that has a central tendency dictated by the 4th-order polynomial, with spread about the mean dictated by a concave-down quadratic.

These analyses were automated using R-scripts.

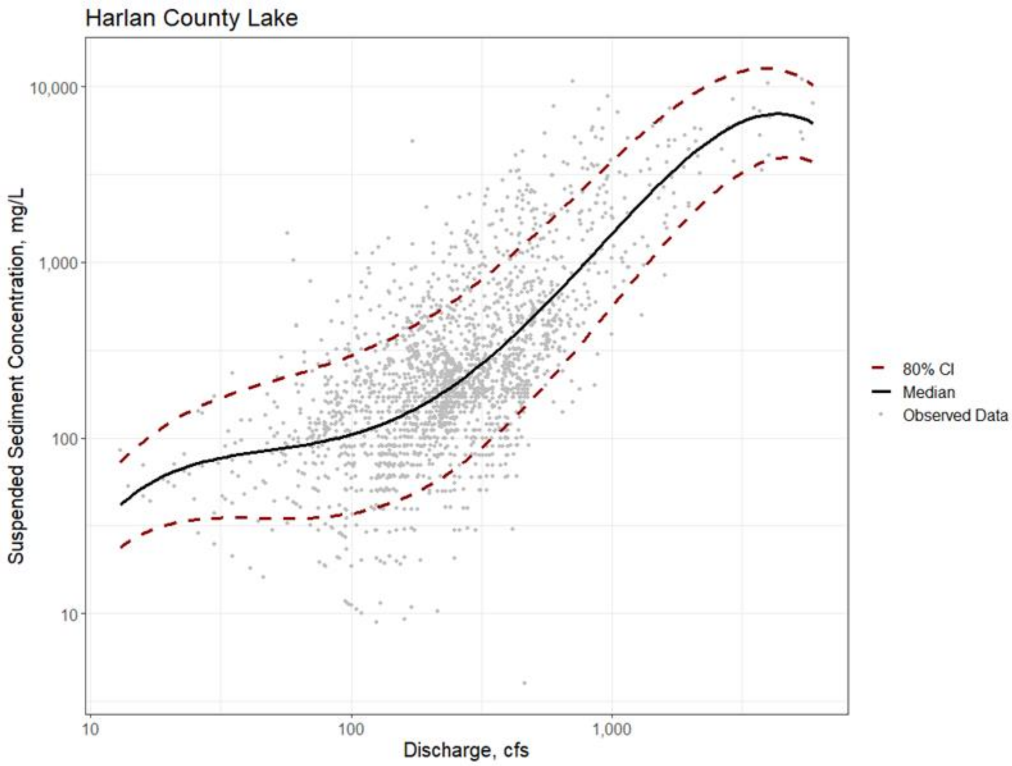


Figure 12-1: Incoming Sediment Concentration to Harlan County Lake

Sediment concentration measurements have also been collected downstream of Harlan County Lake at USGS gage 06849500. Figure 12-2 shows the concentration measurements that have been collected upstream and downstream of Harlan County Dam on the Republican River. The downstream concentrations are significantly less than the upstream concentrations, due to the sediment trapping of the lake.

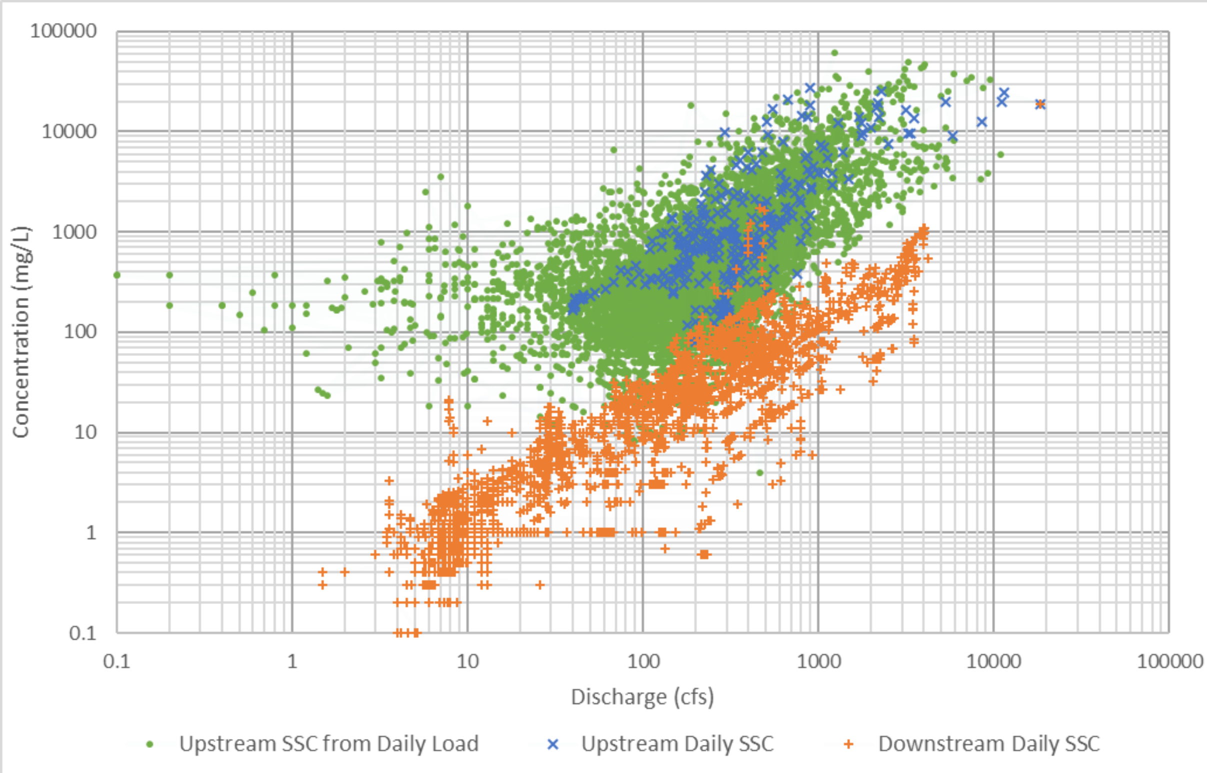


Figure 12-2: Sediment concentration measured upstream and downstream of Harlan County Lake on the Republican River

13.0 RESERVOIR BED SEDIMENT COMPOSITION

In 1988 a total of 27 sediment samples were collected from drill cores within Harlan County Lake at locations near some of the sedimentation rangelines. Figure 13-1 shows the particle sizes of the samples at the sedimentation rangelines, which was obtained by averaging the samples for each rangeline. The location of the pie graph in Figure 13-1 does not indicate the location of the sample along the rangeline. From these measurements and the surveyed volume of deposition, the deposits were determined to be 27.6% clay, 57.1% silt, and 15.3% sand. This matches well with the sediment inflow analysis which found the incoming load to be 30.9% clay, 49.6% silt, and 19.5% sand/gravel. The volume of sand from the inflow is 5.5% higher than from the core measurements, but this is probably because some of the sand is depositing further upstream compared to where the core measurements were taken.

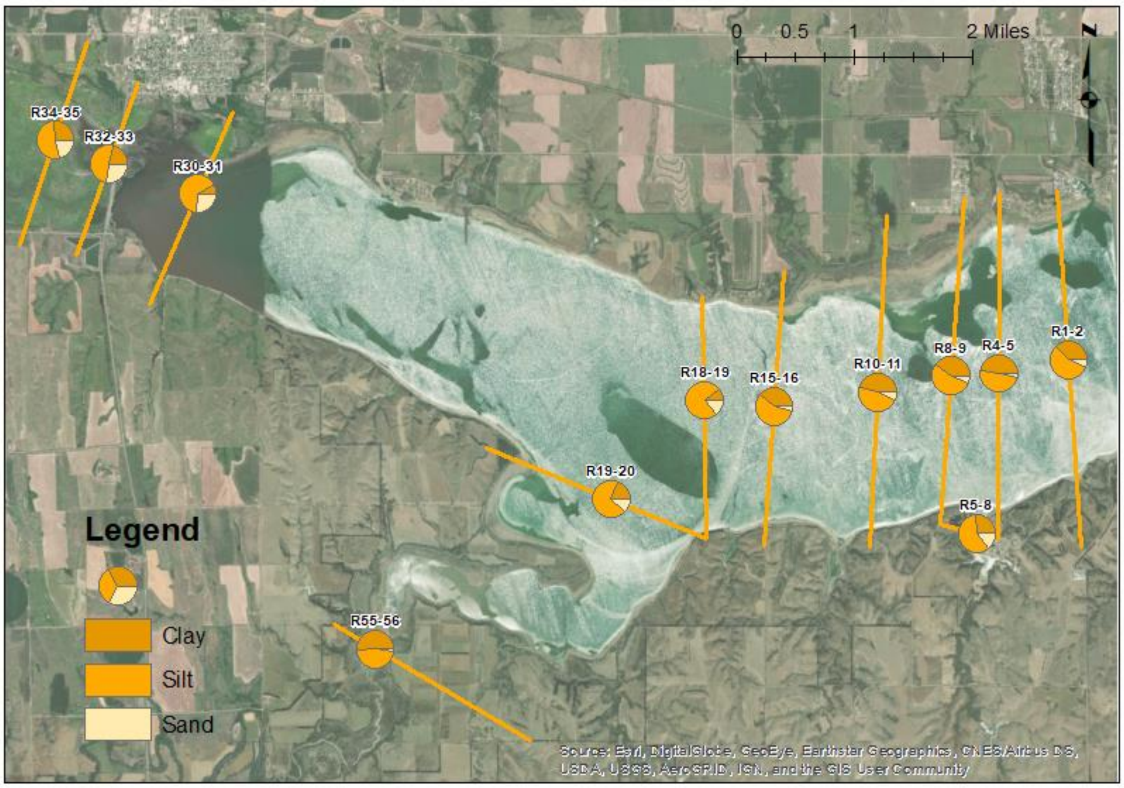


Figure 13-1: Average sediment size distribution, taken from cores located near sedimentation rangelines.

14.0 SEDIMENT CHEMICAL CONCENTRATIONS

Chemical characteristics of the deposited sediment were investigated by the USGS in a 2001 report (Christensen & Juracek, 2001). The study investigated the concentration of various metals and other chemical compounds in the sediment and compared them to guidelines published by the U.S Environmental Protection Agency (USEPA). The threshold-effect level (TEL) is the concentration below which toxic biological effects seldom occur, while the probable-effect level (PEL) is the level above which toxic effect usually or frequently occur. Between the TEL and PEL toxic effect will occasionally occur. However, these guidelines are to be used as screening tools and are not regulatory criteria (Juracek & Mau, 2002). Sediment quality guidelines have been published by the USEPA for nine metals and six organochlorine compounds.

A total of 18 elements were tested by the USGS from twenty sample taken out of Harlan County Lake. The minimum, median, and maximum concentrations of chemical in the samples were published in the report. Of the nine metals with published guidelines by the USEPA, six of them exceeded the TEL while none of them exceeded the PEL. Table 14-1 shows the median and range of the measurements along with the TE and PE levels. Additional information can be found in the USGS report.

Table 14-1: Concentration of trace metals from Harlan County Lake in mg/kg (Christensen & Juracek, 2001)

Trace metal	Median	Range	Threshold effect level	Probably effect level
Al	32,400	19,900-39,100	na	na
As	7.2	5.7-9.0	7.24	41.6
B	10	25-Oct	na	na
Ba	306	262-347	na	na
Be	2	1.5-2.4	na	na
Cd	-	<1.0-1.4	0.676	4.21
Cr	29	18-38	52.3	160
Cu	26	20-32	18.7	108
Fe	25,500	18,000-29,700	na	na
Hg	-	< 0.2	0.13	0.696
Mg	7,220	514-8,600	na	na
Mn	480	407-695	na	na
Ni	25	21-28	15.9	42.8
Pb	-	<23-31	30.2	112
Se	1.6	0.8-2.7	na	na
Sr	162	97-242	na	na
V	63	37-94	na	na
Zn	88	72-1.3	124	271

15.0 DELTA LOCATION AND VOLUME

Figure 15-1 provides a profile plot of centerline through the lake. At each location, the invert elevation (lowest elevation in a given sedimentation range line) is plotted. The locations of the sedimentation rangelines can be found in Figure 6-1. As seen in Figure 15-1, the delta crest grew significantly immediately after dam closure. Also, most of the 2000 survey is lower in elevation than the 1988 survey as discussed earlier. From 1962 to 2000 the delta moved downstream an average of 0.71 miles (98.9 feet per year). However, the majority of that movement appears to be occurring in the lower six miles of the reservoir.

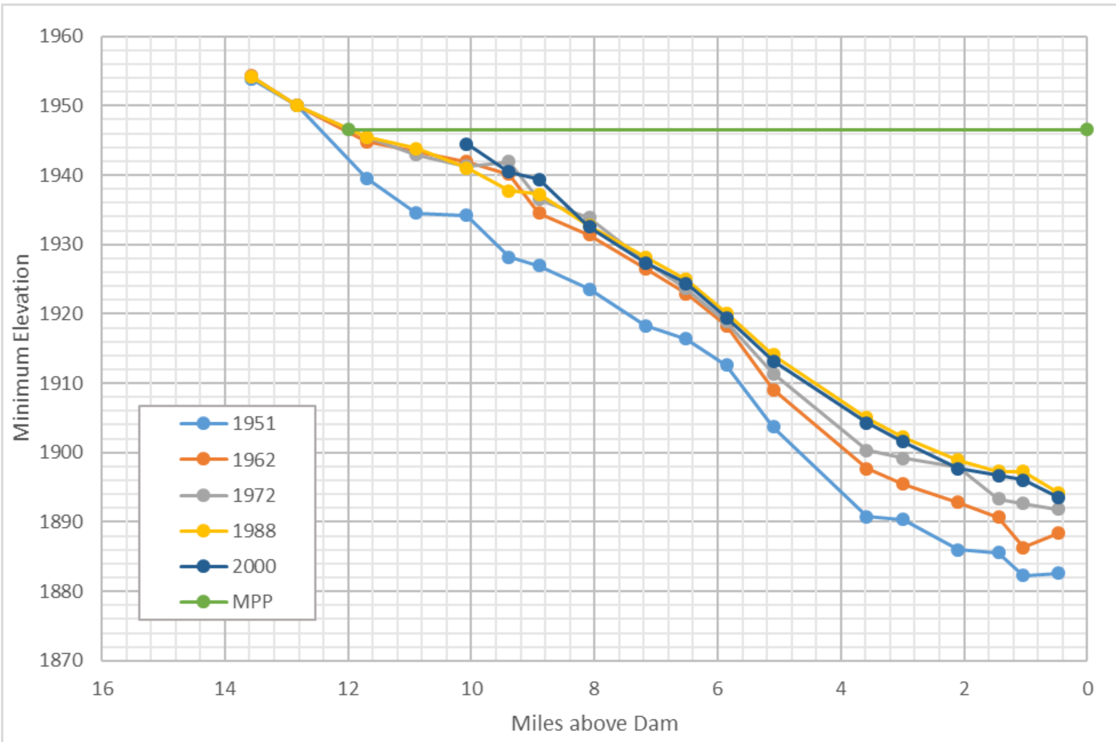
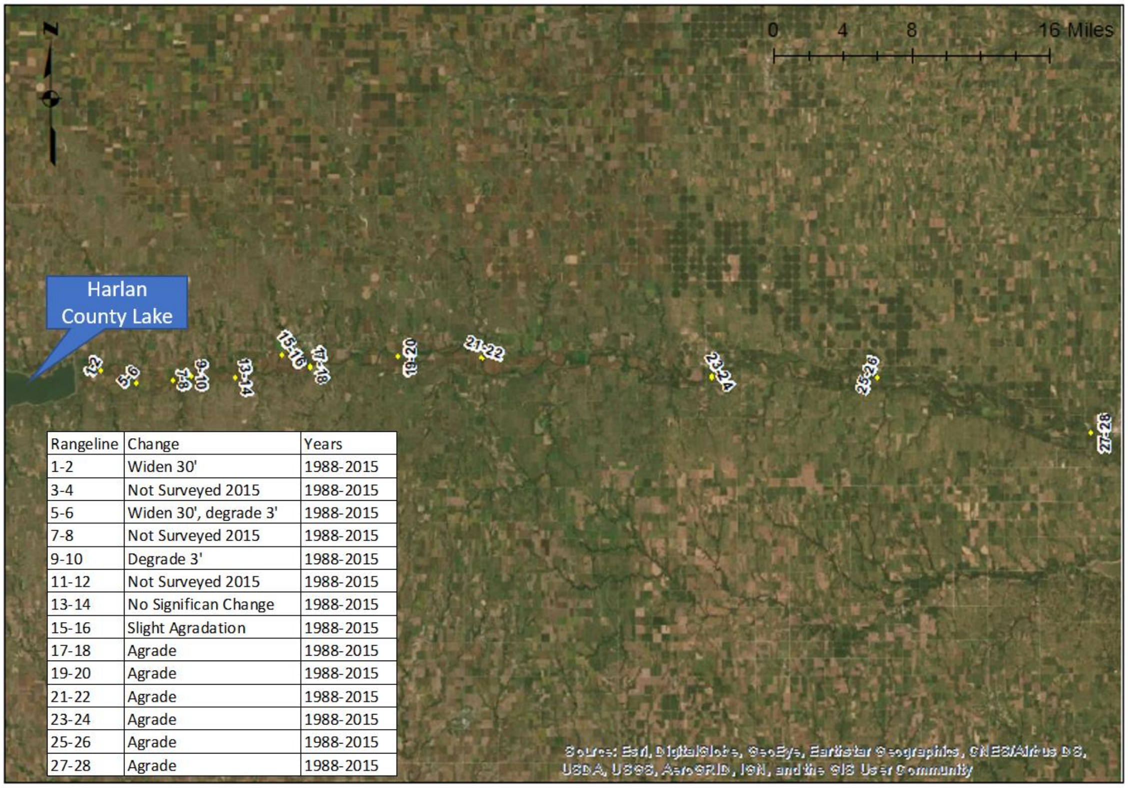


Figure 15-1: Profile of invert elevations indicating delta location and growth

16.0 DOWNSTREAM CHANNEL

Sediment trapping by dams very often induces bed degradation and bank erosion downstream. Degradation rangelines downstream from Harlan County Lake allow this effect to be quantified. All the rangelines were available in the NVGD29 datum, so no conversion was necessary for calculating volume change. Figure 16-1 shows the location of the rangelines, the total bed elevation change, and channel width change at each.



Rangeline	Change	Years
1-2	Widen 30'	1988-2015
3-4	Not Surveyed 2015	1988-2015
5-6	Widen 30', degrade 3'	1988-2015
7-8	Not Surveyed 2015	1988-2015
9-10	Degrade 3'	1988-2015
11-12	Not Surveyed 2015	1988-2015
13-14	No Significan Change	1988-2015
15-16	Slight Agradation	1988-2015
17-18	Agrade	1988-2015
19-20	Agrade	1988-2015
21-22	Agrade	1988-2015
23-24	Agrade	1988-2015
25-26	Agrade	1988-2015
27-28	Agrade	1988-2015

Figure 16-1: Degradation Downstream of Harlan County Lake

Figure 16-2 plots the cumulative volume change from 1988 to 2015 based on the degradation rangelines. Degradation rangelines prior to 1988 are not comparable to the newer surveys because of agricultural encroachment in the channel (USACE, 1973). If these rangelines are indicative of the whole reach from river mile 227-239, the bed and banks have lost 194 ac-ft since 1988. The Republican River is aggrading from river miles 160 to 227 indicating that sediment trapping from the dam does not have a significant impact more than about ten miles downstream. As some of the degradation rangelines are near stabilized bridge locations, this analysis may under predict the degradation. The total degradation on the Republican River in the reach immediately below the dam equates to 42% of the volume of sand deposition within the reservoir, which was estimated to be 464 acre-feet between 1988 and 2015.

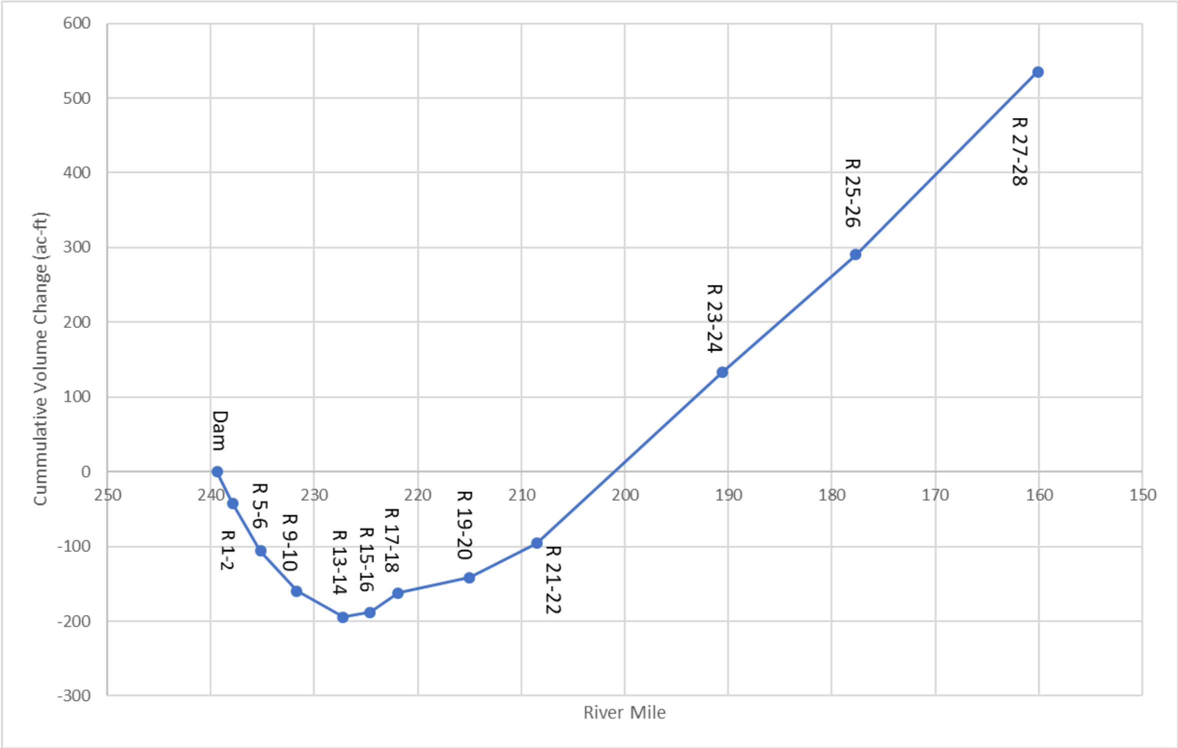


Figure 16-2: Cumulative longitudinal volume change downstream of Harlan County (1988-2015)

These analyses indicate that the Republican River downstream of Harlan County is sediment starved in the reach immediately below the dam. Continued degradation with associated bank erosion is expected if sediment trapping continues. The aggregational reach further downstream may be caused by a reduction in transport capacity due to flow attenuation from the dam.

17.0 SUMMARY AND CONCLUSIONS

Sedimentation has had a moderate impact on Harlan County Lake through loss of storage capacity and moderate impacts to infrastructure surrounding the lake. As of 2000, the multipurpose pool had lost 36,269 ac-ft of storage capacity to sedimentation, or 10.5% of the original volume. Between 1948 and 2009 the flood pool lost 3,933 ac-ft of storage, which is 0.78% of its original volume. Shoreline erosion has been significant at Harlan County Lake but appears to have slowed over time.

Trapping efficiency of Harlan County Lake was estimated to be 97% in 2000 using the Brune Curve method. Sediment rating curves were created from suspended sediment measurements taken at on the Republican River, Sappa Creek, and Prairie Dog Creek. Bulk density measurements by USACE indicate a bulk density of 44.8 pcf in the multipurpose pool and 66.0 pcf in the flood pool. The sediment deposition within the reservoir was calculated using the sediment rating curves, bulk density, and trapping efficiency. This was compared to the sediment deposition estimated from the survey data and the rating curves were adjusted to bring them into closer agreement. The final computed deposition values matched well with the surveyed deposition.

Approximately 70% of the incoming sediment enters Harlan County Lake during flood control operations.

A range in the natural sediment concentrations in inflow to Harlan County Lake was estimated from the Republican River suspended sediment measurements by fitting 90% prediction intervals to the data. Concentration increases with discharge and peaks at approximately 3,000 cfs. Using the sedimentation rangelines, the delta was estimated to have moved towards the dam at a rate of 98.9 feet per year from 1962 to 2000. The estimated degradation on the Republican River downstream of the lake was 194 ac-ft from 1988 to 2015 based on the degradation rangelines, which is 42% of the sand accumulation within the lake.

18.0 REFERENCES

Brune, G. M. (1953). *Trap Efficiency of Reservoirs.* American Geophisical Union.

Christensen, V. G., & Juracek, K. E. (2001). *Variability of metals in reservoir sediment from two adjacent basins in the central Great Plains.* Enviromental Geology.

Duan, N. (1983). *Smearing estimate: A nonparametric retransormation method.* Journal of the American Statistical Association, 78(383), 605-610.

Juracek, K. E., & Mau, D. P. (2002). *Sediment Deposition and Occurrence of Selected Nutrients and other Chemical Constituents in Bottom Sediment, Tuttle Creek Lake, Northeast Kansas, 1962-99.* Lawrence, KS: U.S. Geological Survey.

Shelley, J. E. (2012). *Geomorphic Equations and Methods for Natural Channel Design.* Lawrence, KS: Doctoral Dissertation, University of Kansas.

Surdex. (2011). *Tuttle Creek Lake, Kansas Lakes LiDAR Mapping, Hydrographic Survey Data Integration & Flood Pool Survey.* Surdex Corporation.

USACE. (1963). *Sedimentation in Harlan County Reservoir, Republican River, Nebraska.* U.S. Army Corps of Engineers, Kansas City District.

USACE. (1973). *Republican River Basin Lake Regulation Manual, Volume 2, Harlan County Lake, Nebraska.* U.S. Army Corps of Engineers, Kansas City District.

USACE. (2013). *EM 1110-2-1003; Hydrographic Surveying.* U.S. Army Corps of Engineers.

USACE. (2014). *Vertical Datum Update for Harlan County Lake.* U.S. Army Corpos of Engineers, Kansas City District.

USACE. (2015). *Harlan County Dam Periodic Inpection No. 10 Periodic Assessment No. 1.* U.S. Army Corps of Engineers, Kansas City District, Northwestern Division.

Zaiontz, C. (2014). *Confidence/prediction intervals | Real Statistics Using Excel*. Retrieved from real-statistics.com: http://www.real-statistics.com/regression/confidence-and-prediction-intervals/

US Army Corps
of Engineers ®
Kansas City District

Kansas River Reservoirs Flood and Sediment Study

Appendix D2: U.S. Army Corps of Engineer Lakes Future Without Project Sedimentation Summary

November 2022

Contents

1.0 INTRODUCTION

This appendix documents Future Without Project (FWOP) projections for sedimentation conditions at USACE lakes in the Kansas River Basin in 25, 50, and 100 years.

The Existing Condition Sedimentation Reports (Appendix D1) documented the development and calibration of rating curves for the USGS gages upstream of the USACE lakes in the Kansas River Basin. These rating curves, the bulk densities and trapping efficiencies as described in the Existing Condition Reports, plus 100 years of future daily flows allows the computation of total volume loss over time at the USACE lakes. With the assumption of uniform deposition throughout the lake, the surface area loss can also be approximated.

FWOP computations were first completed in December 2020 but were later updated in May 2022 based a new methodology incorporating sediment modeling results from Tuttle Creek Lake. The new methodology included the effects of additional sediment depositing in the flood pool (FP) over time. See Appendix D4 for additional details on the HEC-RAS sediment modeling.

2.0 VOLUME COMPUTATIONS

Long-term sediment deposition was computed by running 100 years of daily flow through the calibrated rating curves described in Appendix D1. The future flows were determined by:

1. Using measured flows from the date of the most recent survey through 2019
2. Repeating the record of past years of daily flows until 2124

Table 2-1 summarizes the flow records used for each lake. The time periods were found reasonably representative to use for future projections.

Table 2-1. Flow Years Used in Future Projections.

Lake	River	Lake Most Recent Survey Year	Repeated Flow Years	Years of Flow
Clinton	Wakarusa	2019	1978-2019	42
Perry	Delaware	2009	1970-2019	50
Tuttle Creek	Big Blue	2009	1970-2019	50
Tuttle Creek	Little Blue	2009	1970-2019	50
Tuttle Creek	Black Vermillion	2009	1970-2019	50
Milford	Republican	2009	1970-2019	50
Kanopolis	Smoky Hill	2017	1970-2019	50
Wilson	Saline	2008	1970-2019	50
Harlan County	Republican	2000	1970-2019	50
Harlan County	Sappa Creek	2000	1970-2019	50

At each lake, the incoming sediment is transformed into a deposition volume following these steps:

1. Use the rating curve, including Duan E, ungauged area, and other adjustment factors as described in Appendix D1 to convert a daily flow to a daily sediment load (in tons).
2. Compute the ratio of flood control deposition to incoming sediment to compute the mass of sediment that remains in the flood control pool. The original FWOP calculations assumed this ratio remained constant.
3. Apply the bulk density of the flood control pool to convert the flood control mass to a volume of deposition (in cubic ft).
4. Multiply the remaining mass by the trapping efficiency of the multipurpose pool, computed via a curve parallel to the Brune Curve to compute the mass of sediment trapped in the multipurpose pool (in tons). The Brune Curve computes a lower trapping efficiency over time as the lake fills.
5. Transform the trapped mass into a volume of deposition in the multipurpose pool using the multipurpose pool bulk density.
6. Subtract the deposition from the previous days' pool volumes.
7. Repeat these steps for the next day.

Table 2-2 provides the remaining multipurpose pool volumes at the end of 25, 50, and 100 years. Year 0 is 2024, so there are varying number of years of simulation to take each lake from its most recent survey to Year 0. The results for Tuttle Creek were obtained from the HEC-RAS sediment model. Figure 1 maps the percent full for each reservoir.

Table 2-2. Remaining Multipurpose Pool Volumes in Kac-ft and percentage.

Lake	Original Volume kac-ft	Year 0 (2024) kac-ft	Year 0 (2024) %	Year 25 (2049) kac-ft	Year 25 (2049) %	Year 50 (2074)	Year 50 (2074) %	Year 100 (2124) kac-ft	Year 100 (2124) %
Clinton	129.2	112.2	87	103.9	80	96.7	75	83.9	65
Perry	243.2	182.9	75	159.4	66	140.0	58	105.2	43
Tuttle Creek	424.3	216.6	51.0	153.0	36.1	106.5	25.1	29.2	6.9
Milford	415.4	366.5	88.2	355.8	85.7	349.9	84.2	333.0	80.2
Kanopolis	73.2	43.3	59.1	34.8	47.5	30.2	41.3	18.2	24.9
Wilson	247.8	228.1	92.0	218.0	87.9	209.0	84.3	190.1	76.7
Harlan County	346.5	309.4	89.3	304.0	87.7	300.7	86.8	291.0	84

Note: Kac-ft = 1000 acre-ft; % = percent of original pool remaining.

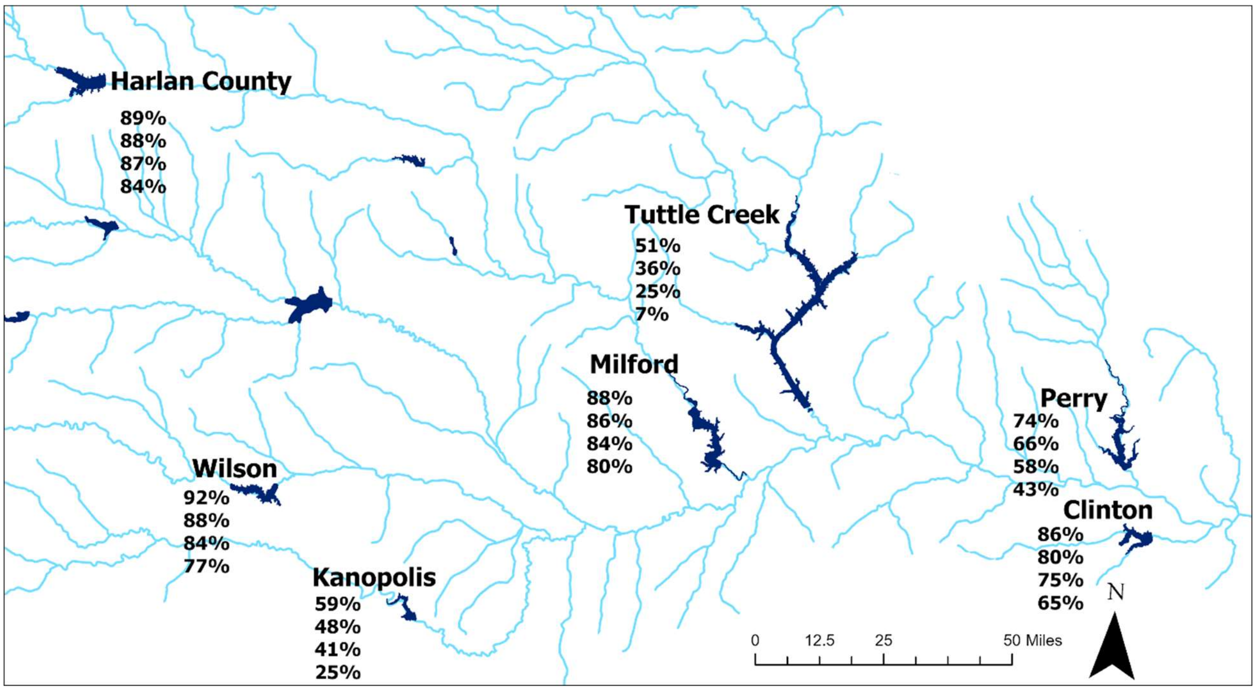


Figure 2-1. Percent remaining of the original multipurpose pool at the end of 0, 25, 50, and 100 years.

Table 2-3 provides the remaining flood control volumes at the end of 25, 50, and 100 years. Three factors lead to significantly less deposition in the flood control pools than the multipurpose pools. (1) Much more sediment deposits in the multipurpose pool, (2) The flood control pools are much larger than the multipurpose pools, so as a percentage the losses are smaller, (3) At many of the lakes, the original volume was underestimated based on the survey method (sediment rangelines). The new LIDAR method reflects larger existing pools than originally thought.

Table 2-3. Remaining Flood Control Pool Volumes in Kac-ft and Percentage.

Lake	Original Volume kac-ft	Year 0 (2024) kac-ft	Year 0 (2024) %	Year 25 (2049) kac-ft	Year 25 (2049) %	Year 50 (2074) kac-ft	Year 50 (2074) %	Year 100 (2124) kac-ft	Year 100 (2124) %
Clinton	268.4	291.4	109	289.4	108	287.0	107	281.0	105

Lake	Original Volume kac-ft	Year 0 (2024) kac-ft	Year 0 (2024) %	Year 25 (2049) kac-ft	Year 25 (2049) %	Year 50 (2074) kac-ft	Year 50 (2074) %	Year 100 (2124) kac-ft	Year 100 (2124) %
Perry	521.9	510.1	98	499.3	96	487.3	93	458.8	88
Tuttle Creek	1942.7	1851.7	95.3	1780.9	91.7	1738.3	89.5	1612.3	83.0
Milford	757.7	756.9	99.9	755.0	99.6	753.9	99.5	749.9	99.0
Kanopolis	373.9	362.9	97.1	358.0	95.7	354.7	94 9	344.8	92.2
Wilson	530.7	528.5	99.6	527.3	99.4	525.8	99.1	521.2	98.2
Harlan County	503.5	499.3	99.2	498.4	99.0	497.8	98.9	495.7	98.5

Note: Kac-ft = 1000 acre-ft; % = percent of pool remaining.

3.0 TRAPPING EFFICIENCY

The trapping efficiency for the multipurpose pool of each lake shrinks according to the Brune Curve (see Figure 3-1). If a lake had a measured trapping efficiency, the difference between the closest Brune Curve and the measured value was added to the Brune Curve at each time step. Thus, the lake trapping efficiency would fall on a line parallel to the Brune Curve. See Appendix D1 for a more in-depth discussion of trapping efficiency.

Table 3-1 lists the trapping efficiency of each lake in Year 0, 25, 50, and 100. As seen, the trapping efficiency for most of the lakes remains high for the entire projection period.

Table 3-1. Trapping Efficiency.

Lake	Year 0 (2024)	Year 25 (2049)	Year 50 (2074)	Year 100 (2124)
Clinton	97.0%	96.9%	96.7%	96.4%
Perry	96.6%	96.5%	96.4%	95.9%
Tuttle Creek	96.8%	91.4%	75.3%	0.0%
Milford	96.2%	96.2	96.2%	96.1%
Kanopolis	95.3%	94.2%	93.3%	89.1%
Wilson	97.0%	97.0%	97.0%	97.0%
Harlan County	96.9 %	96.9 %	96.9 %	96.9 %

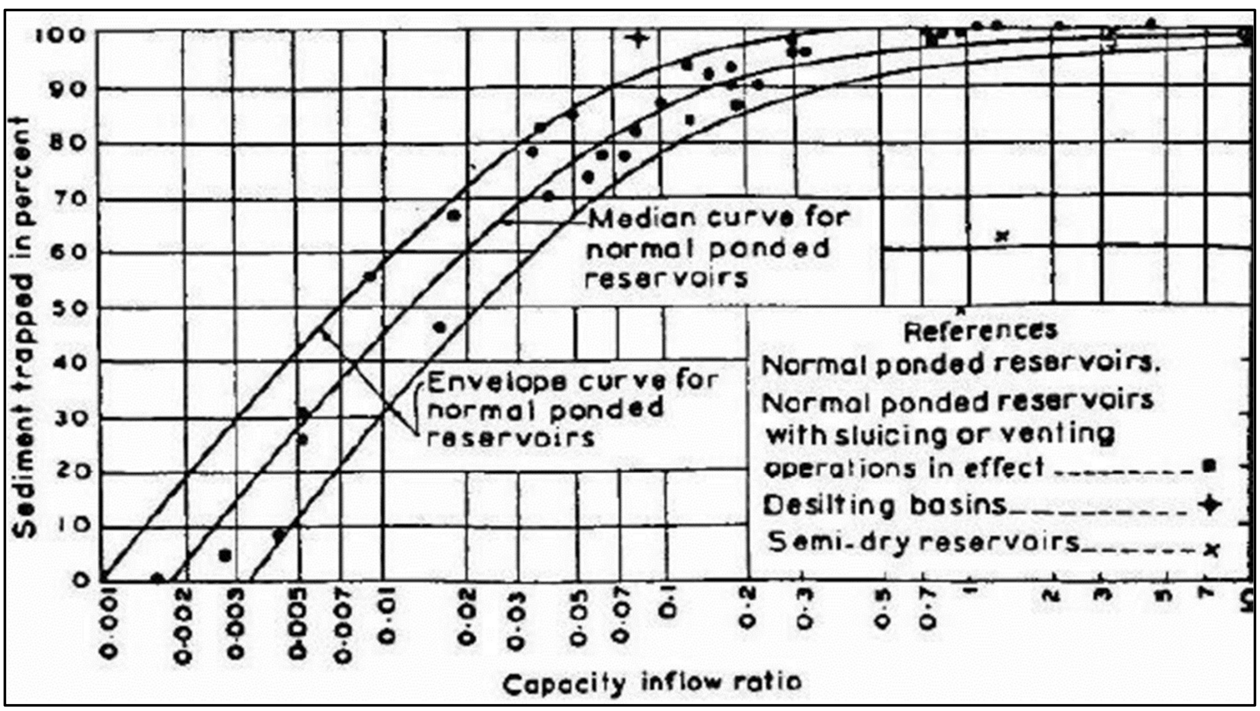


Figure 3-1.Brune Curves.

Over time as the multipurpose pool (MPP) loses capacity, additional mass is deposited in the FP. The percent of total deposition that deposits in the flood control pool was estimated based on a regression equation developed from lake surveys and the HEC-RAS sediment modeling results. Figure 3-2 shows the percentage of total incoming mass that deposits in the FP on the Y-axis and the percent volume lost from the MPP on the X-axis. Both survey data and HEC-RAS sediment model results for Tuttle Creek are included in Figure 3-2. A polynomial regression equation was fitted to the data, until the MPP has lost

80% of its original volume, after which the % mass to the FP is held constant. Additional details on the development of Figure 3-2 can be found in Appendix D4.

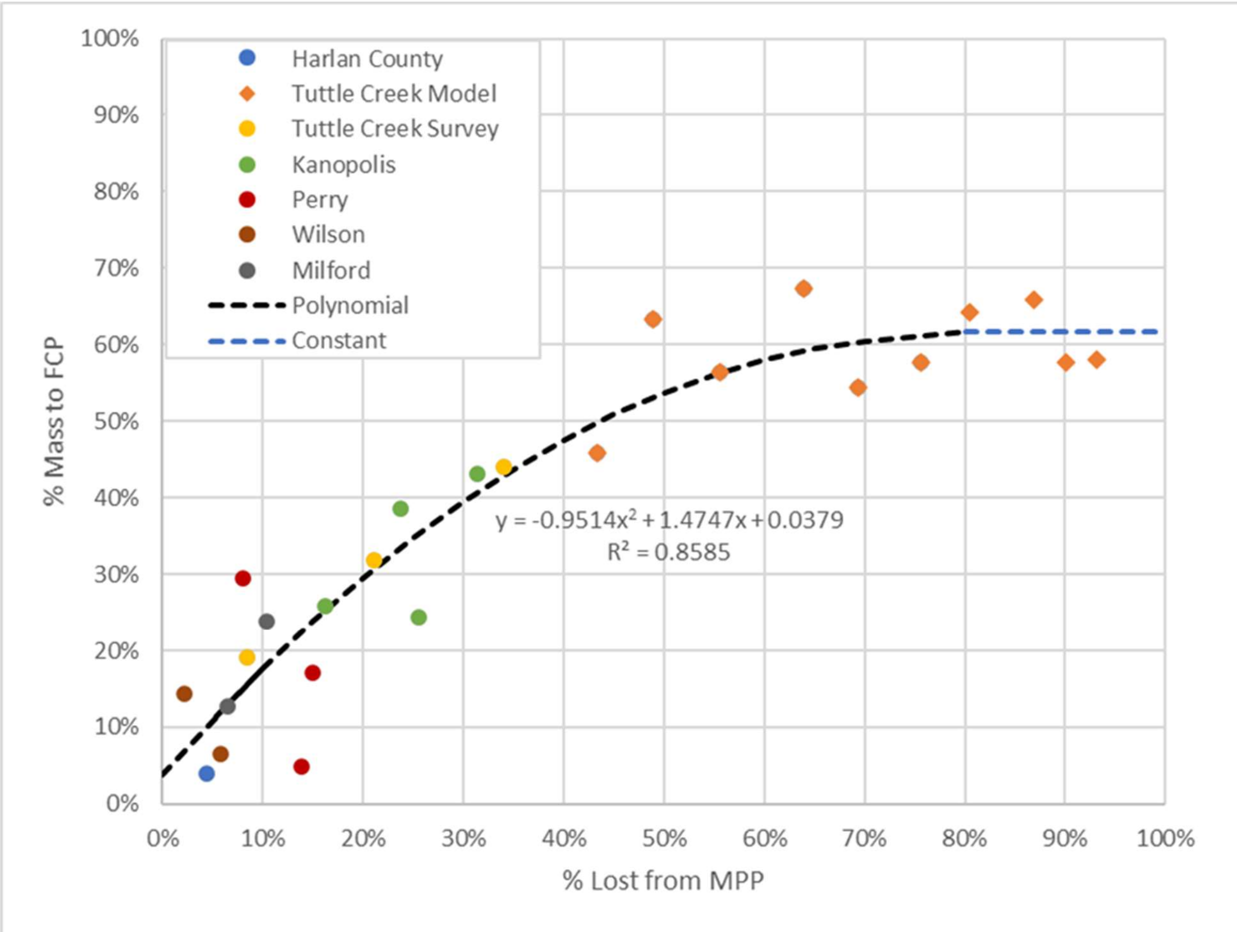


Figure 3-2. The percentage of mass depositing in the FP vs. the percent of volume lost from the MPP

4.0 DOWNSTREAM CONCENTRATIONS

As the lake trapping efficiency decreases, more sediment will be passed through the dam and supplied to the river downstream. For this analysis, the overall trapping efficiency was used, which was calculated by compositing the FP and MPP trapping efficiencies discussed previously. Because additional sediment deposits in the FP over time, as computed using the developed regression equation, several lakes showed a slight increase in overall trapping efficiency over time, since the decrease in MPP trapping efficiency was more than offset by the increase in FP trapping efficiency. This increase in overall trapping efficiency may be a real effect that will occur in the lakes over time, or it could be due to lack of precision in the analysis. The overall trapping efficiency was assumed to remain constant for the purposes of this analysis if it was calculated to increase over time.

Most of the additional sediment that passes downstream from the lake will be fine sediment as most of the coarse sediment (sands and gravels) will continue to deposit in the FP or in the delta at the upstream end of the MPP. The actual concentration in the future for any given flow depends on the timing of inflows and outflows as well as the sediment trapping and therefore cannot be determined without numerical modeling beyond the scope of this study. However, the average annual sediment concentration (total mass divided by total volume) in the water passed to the downstream channel is directly related to the trapping efficiency, and can be computed as follows for any year x:

$$C_x = C_0 * \frac{(1 - TE_x)}{(1 - TE_0)}$$

where C_X , 0 = The average concentration in year x or year 0 (2024)

TE_X, 0 = The average trapping efficiency in year x or year 0 (2024)

Table 4-1 and Figure 4-1 present the ratio of future concentration to Year 0 concentration for each lake. Because additional mass is deposited in the FP over time, which offsets the decreasing MPP trapping efficiency, only Tuttle Creek and Kanopolis will have an increase in downstream sediment concentration.

Table 4-1. Future Sediment Concentration Ratios.

Lake	C_{25}/C_0	C_{50}/C_0	C_{100}/C_0
Clinton	1.00	1.00	1.00
Perry	1.00	1.00	1.00
Tuttle Creek	1.18	1.23	2.78
Milford	1.00	1.00	1.00
Kanopolis	1.08	1.18	1.75
Wilson	1.00	1.00	1.00
Harlan County	1.00	1.00	1.00

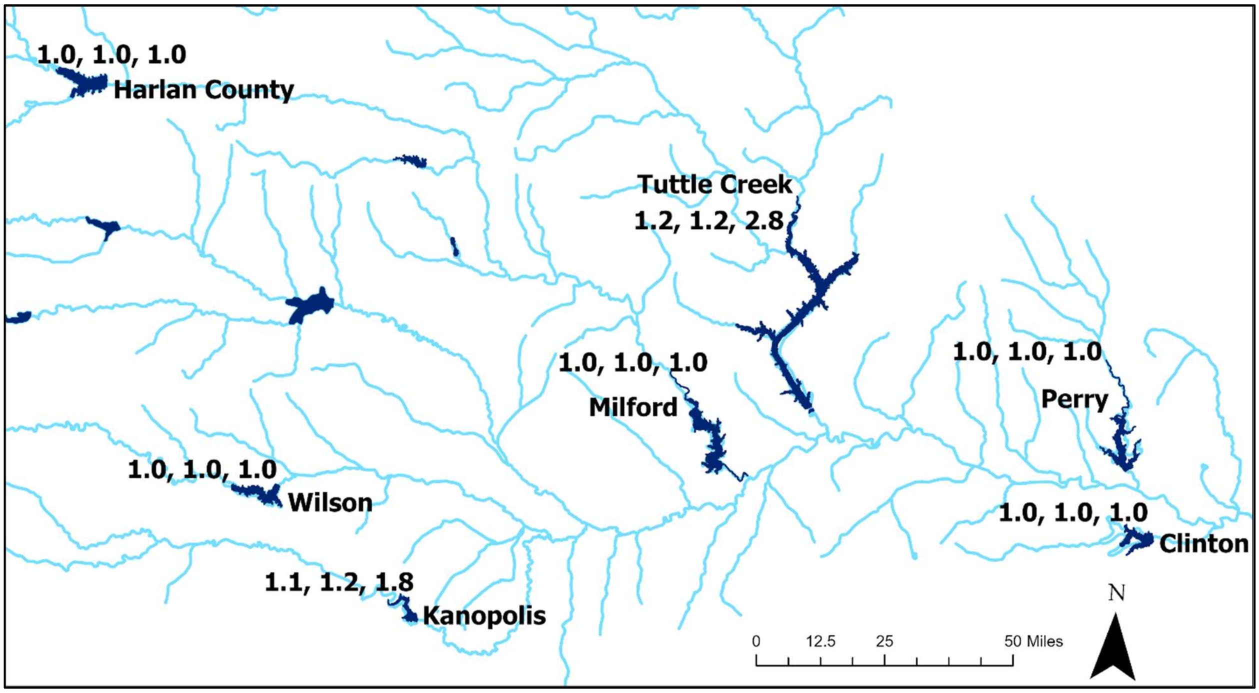


Figure 4-1. Future sediment downstream channel concentration ratios: C_{25}/C_0, C_{50}/C_0, C_{100}/C_0.

While as a ratio to the current sediment-starved conditions the concentrations go up, the downstream concentrations will still be significantly lower than they would be if the dams were not present. The multipurpose pool trapping efficiencies remain high in all lakes except for Tuttle Creek (see Table 4-1).

5.0 DELTA PROGRESSION, MPP SURFACE AREA, AND COVE HABITATS

Appendix D1 indicated that current delta progression rates ranged from undetectable (over the past decade) to 581 ft/year. Future projections cannot simply rely on historic projections, however, because the delta progression slows as it moves into deeper portions of the lake. To estimate future delta progression with associated loss in surface area for the multipurpose pool, the following procedure was employed based on the estimated volumetric deposition amounts. This procedure deposits sediment as an even veneer at all DEM cells that are below the multipurpose pool elevation. If the added elevation raises a cell to the elevation of the top of the MPP, that cell is no longer part of the MPP, which equates to delta progression. If the elevation change equates to a MPP volume that is appreciably different than the computed volume, the excess volume is redistributed among the remaining MPP cells. Here are the steps:

1. In ArcGIS compute the surface area of the multipurpose pool.
2. Divide the projected deposition volume by the surface area of the MPP obtain the elevation change within the MPP.
3. Add this elevation to the most recent digital elevation model (DEM) of the MPP to obtain a first estimate of the projected DEM after the deposition has occurred.
4. Use the surface volume tool to estimate the new volume and area of the MPP.
5. Determine the difference between the predicted MPP volume and the MPP from the modified DEM
6. Divide the volume difference by the new surface area to obtain an additional elevation increase.
7. Add the additional elevation increase to the modified DEM
8. Repeat steps 3-7 until the DEM MPP volume approximately equals the projected MPP volume. A difference less than the average annual deposition was considered to be adequate.

Figures 5-1 through 5-7 depict the shrinking multipurpose pools as sediment fills. For Tuttle Creek, DEMs were developed from the HEC-RAS modeling results and not the procedure outlined above.

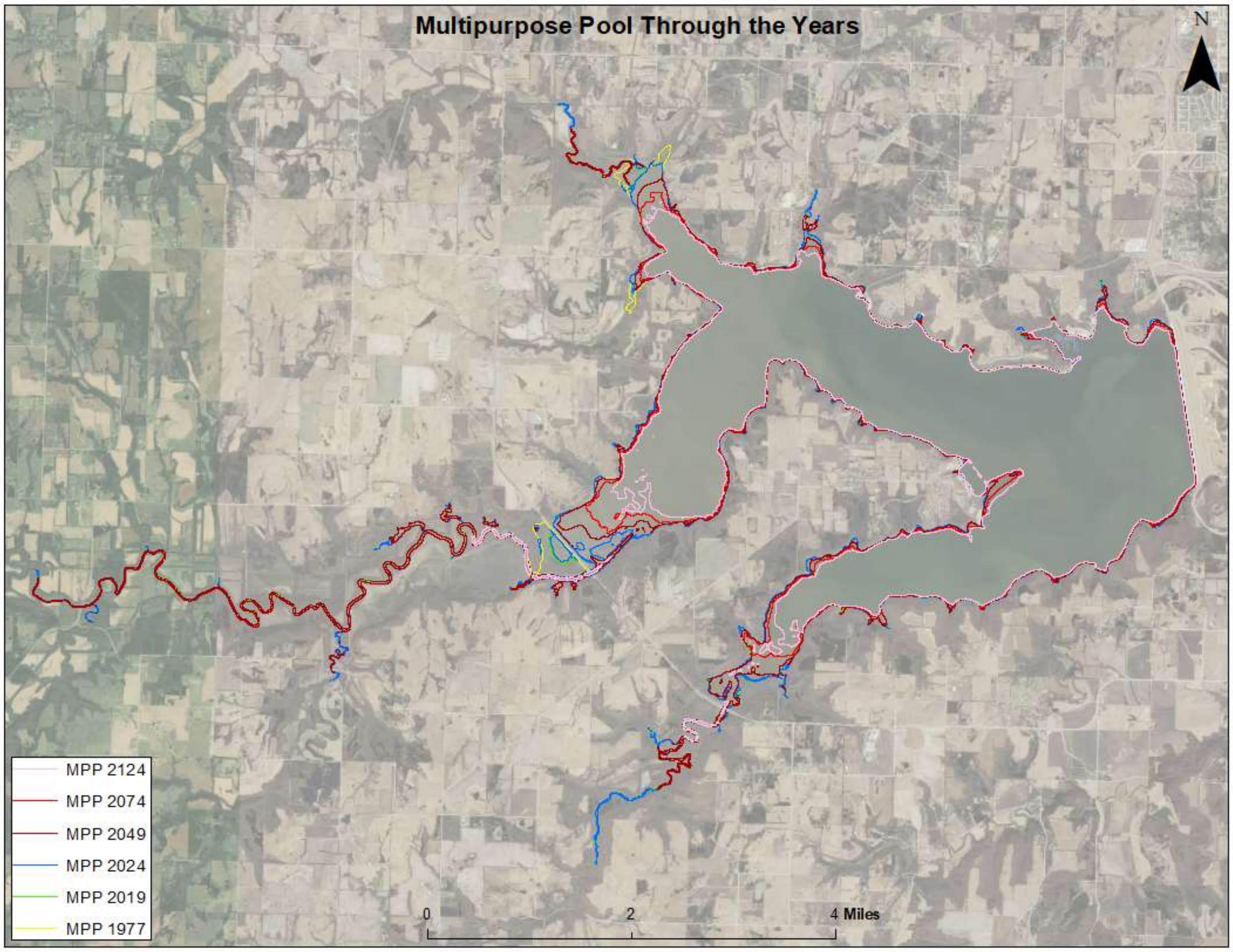


Figure 5-1. Clinton Lake Multipurpose Pool Elevation Contours.

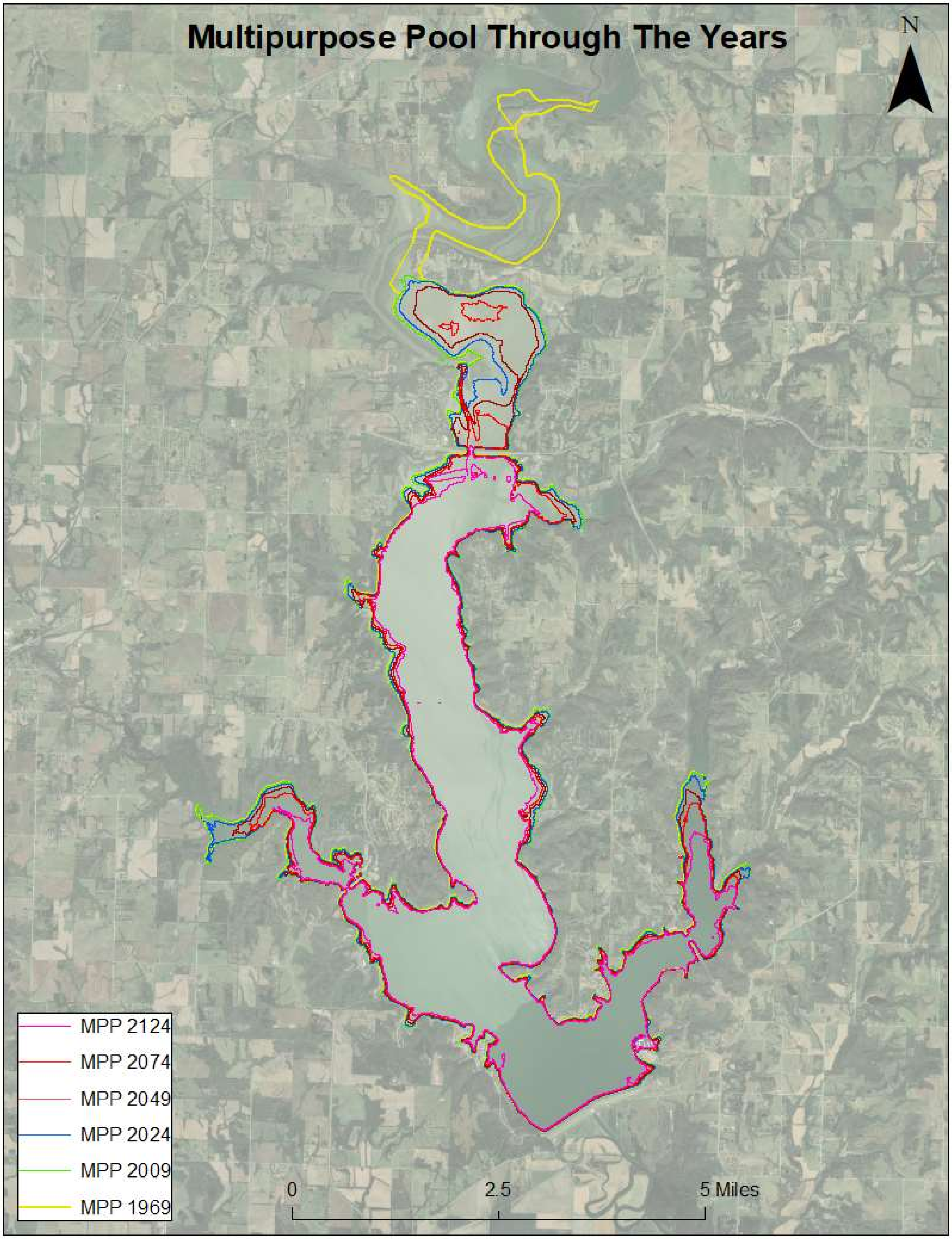


Figure 5-2. Perry Lake Multipurpose Pool Elevations.

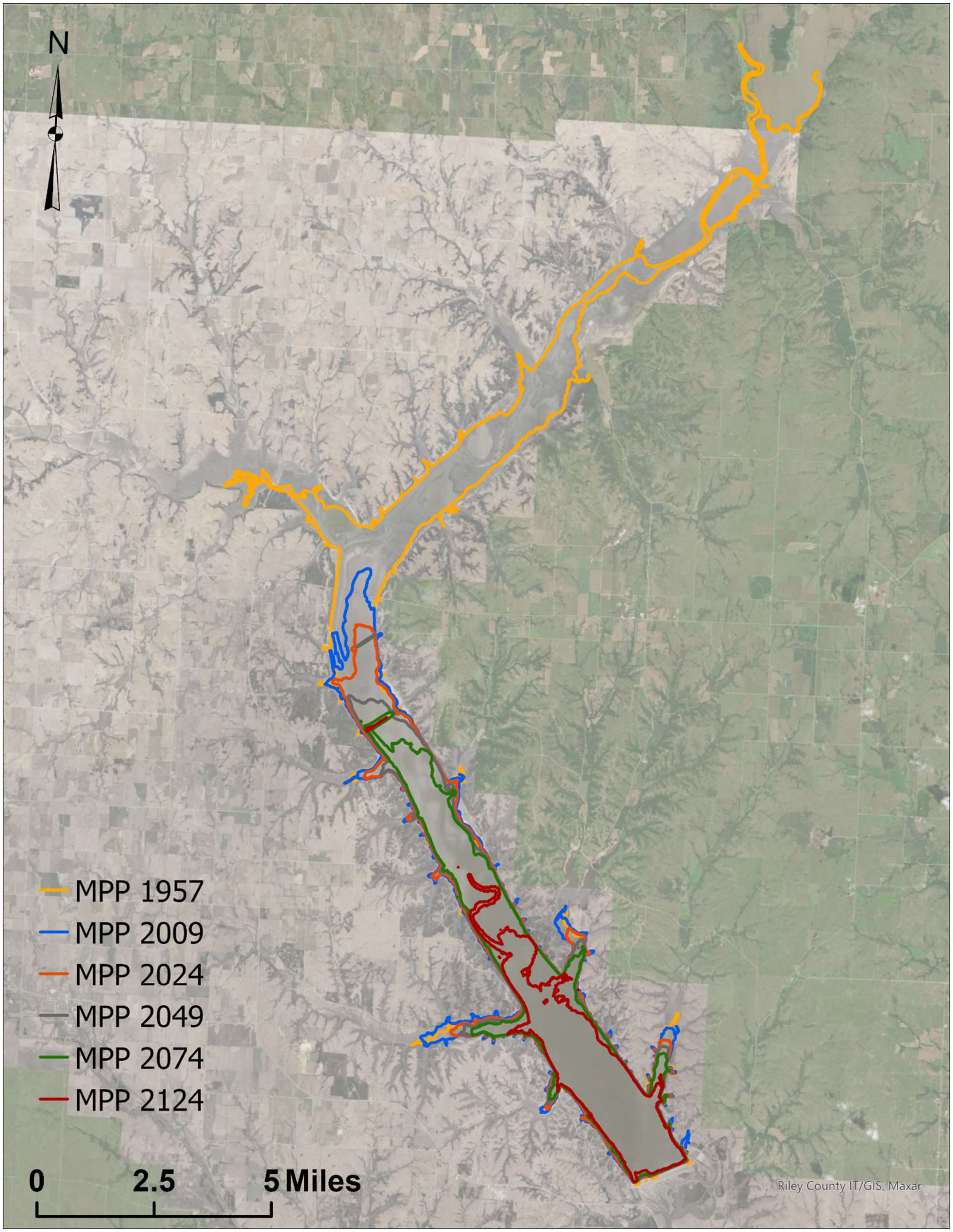


Figure 5-3. Tuttle Creek Lake Multipurpose Pool Elevation Contours.

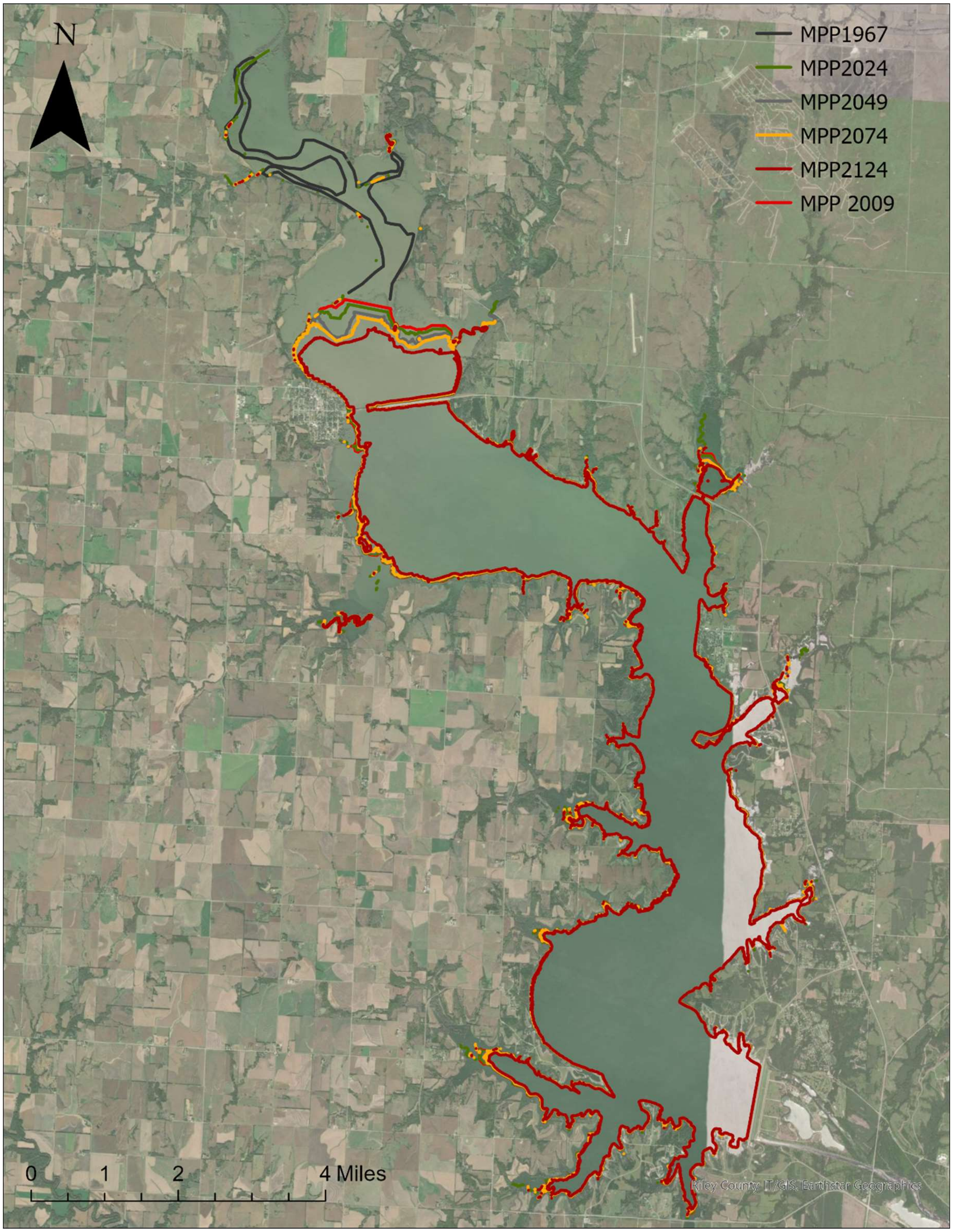


Figure 5-4. Milford Lake Multipurpose Pool Elevation Contours.

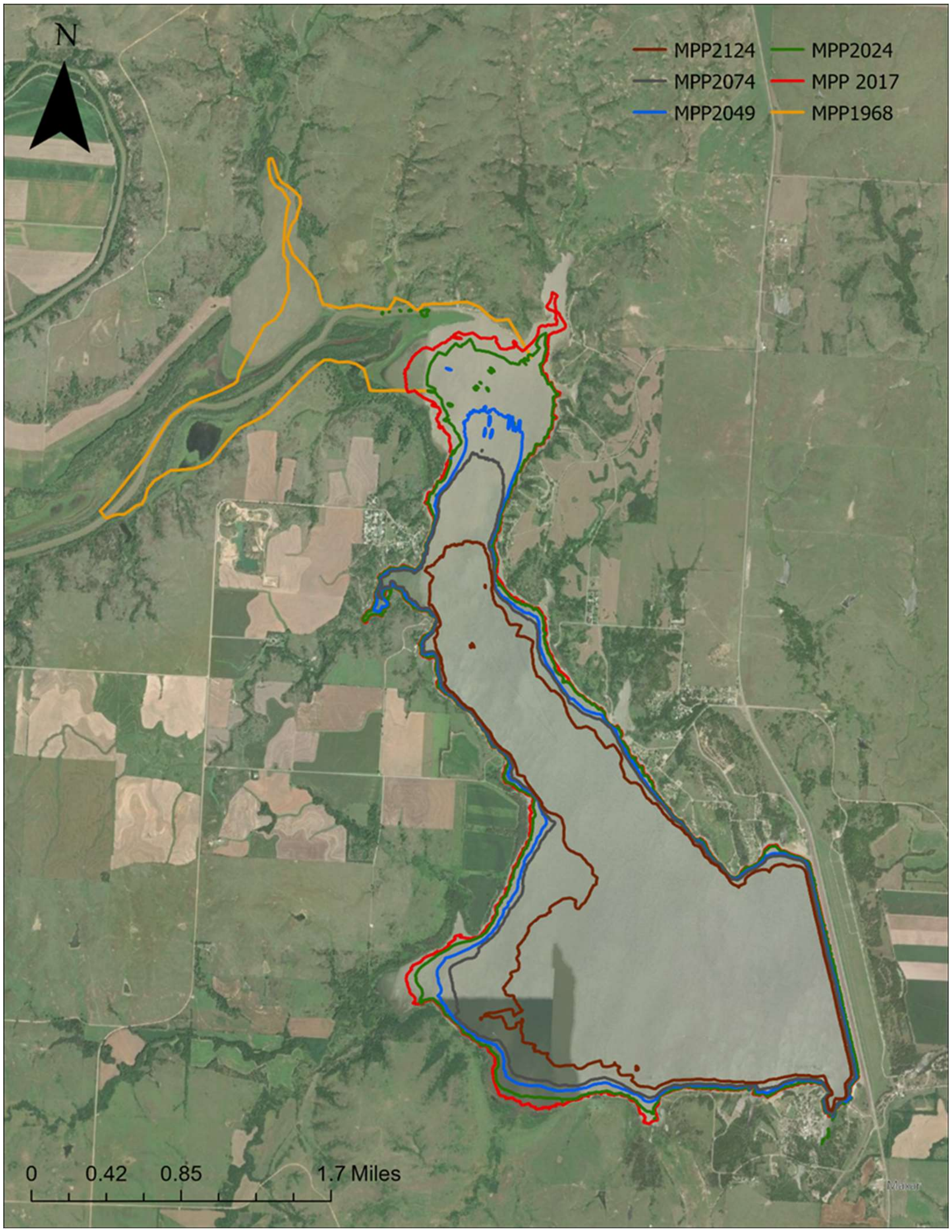


Figure 5-5. Kanopolis Lake Multipurpose Pool Elevation Contours.

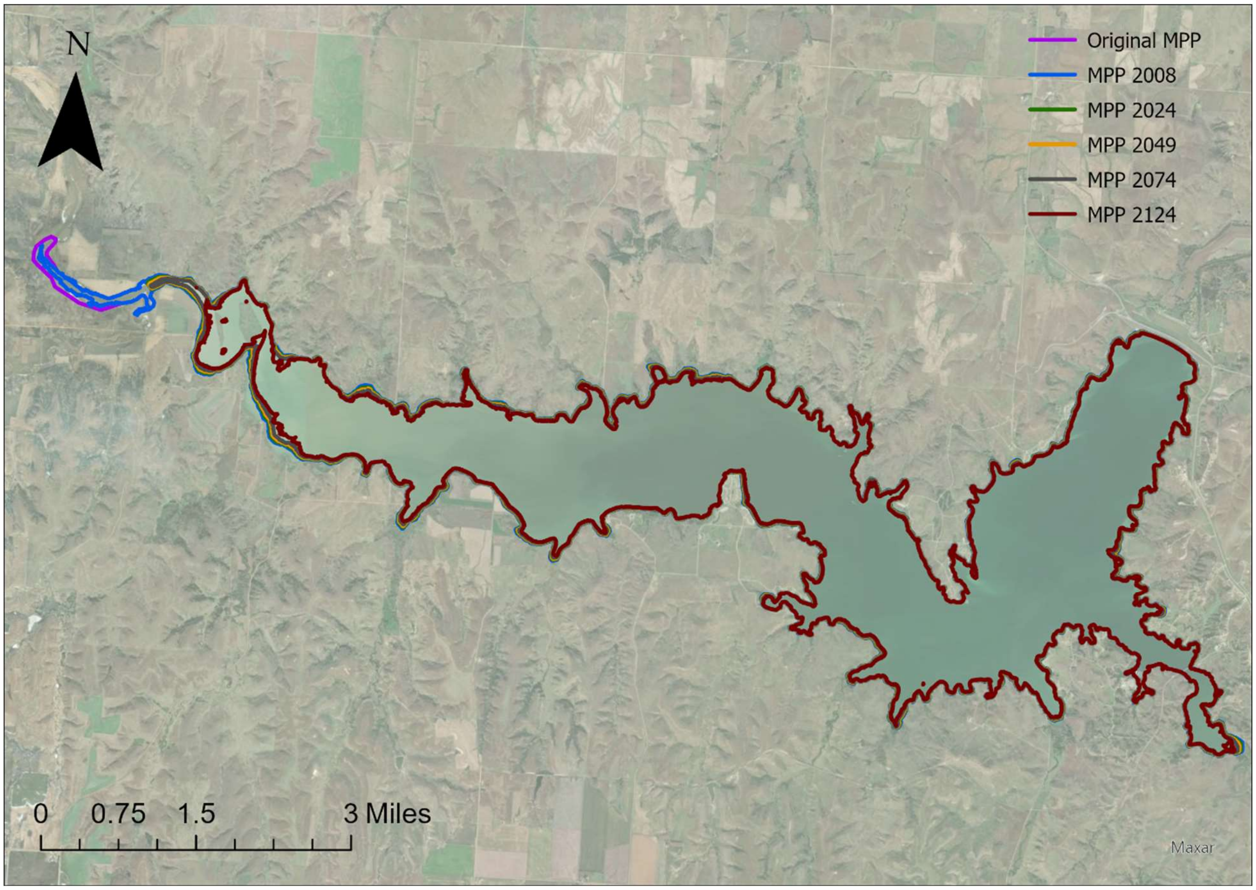


Figure 5-6. Wilson Lake Multipurpose Pool Elevation Contours.

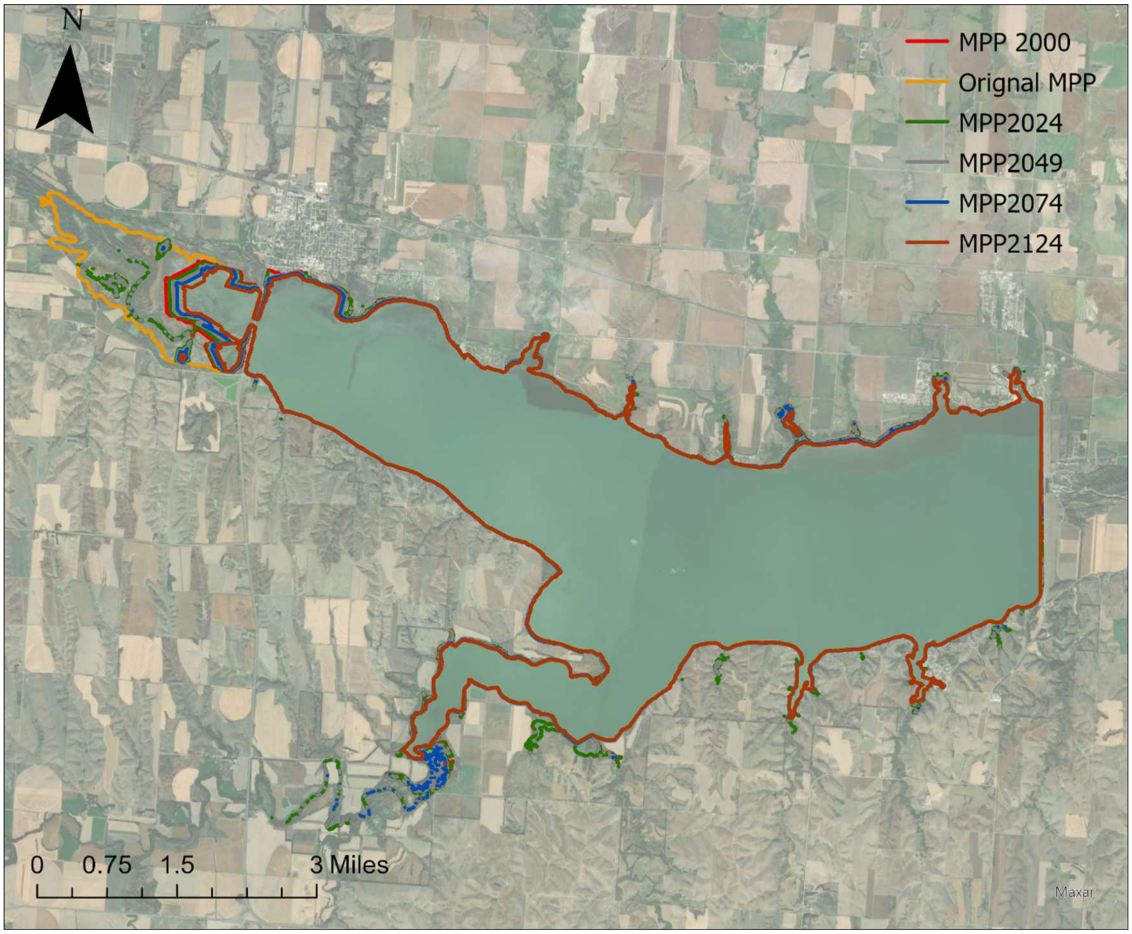


Figure 5-7. Harlan County Lake Multipurpose Pool Elevation Contours.

Table 5-1 summarizes the remaining MPP surface area over time for each lake.

Table 5-1. Multipurpose Pool Surface Area in thousands of acres.

Lake	Original	Year 0 (2024)	Year 25 (2049)	Year 50 (2074)	Year 100 (2124)
Clinton	7.0	7.5	7.2	6.8	6.3
Perry	12.2	9.8	9.0	8.1	7.2
Tuttle Creek	15.8	9.4	7.5	6.4	3.2
Milford	16.6	15.3	14.9	14.7	14.2
Kanopolis	4.1	2.9	2.6	2.4	1.9
Wilson	9.0	8.4	8.3	8.2	7.9
Harlan County	13.8	12.9	12.7	12.6	12.4

Note: Acreage shown in 1,000s.

6.0 DOWNSTREAM CHANNEL DEGRADATION

Appendix D1 indicated that the channels downstream of every lake except Wilson are experiencing bed degradation and bank erosion as a consequence of the sediment trapping in the lakes. The ratio of downstream bed and bank loss to upstream Sand & Gravel trapping ranged from 0.05 to 0.78. This represents just the loss in the immediate downstream receiving river (i.e., not including more dispersed effects on the Kansas River). Table 6-1 indicates the Sand & Gravel trapping and associated downstream bed and bank loss for each lake at the end of 25, 50, and 100 years, assuming the ratio of upstream trapping to downstream loss remains consistent. Figure 6-1 maps the total bed and bank loss. Note that the ratio represents cubic yards of sand trapped in the lake compared to total downstream bed and bank change of any sediment size. While sands and gravels comprise the beds downstream of the dams, silts, clays, and sands comprise the banks.

Table 6-1. Volume eroded from channel bed and banks downstream of lakes.

Lake	Downstream Loss/Lake Trapping	Lake Sand & Gravel Trapping (Year 25)	Lake Sand & Gravel Trapping (Year 50)	Lake Sand & Gravel Trapping (Year 100)	Bed/Bank Loss (Year 25)	Bed/Bank Loss (Year 50)	Bed/Bank Loss (Year 100)
Clinton	0.78	765	1,495	2,967	597	1,166	2,315
Perry	0.2	2,771	5,383	10,777	551	1,071	2,144
Tuttle Creek	0.30	14,310	23,764	49,404	4,338	7,203	14,975
Milford	0.35	2,741	4,133	8,620	965	1,455	3,035
Kanopolis	0.05	2,910	4,596	9,641	150	237	498
Wilson	NA	NA	NA	NA	NA	NA	NA
Harlan County	0.42	1,053	1,689	3,656	441	707	1,530

Notes: All lake sand and gravel trapping, and Bed/Bank loss values are in Thousand Cubic Yards.

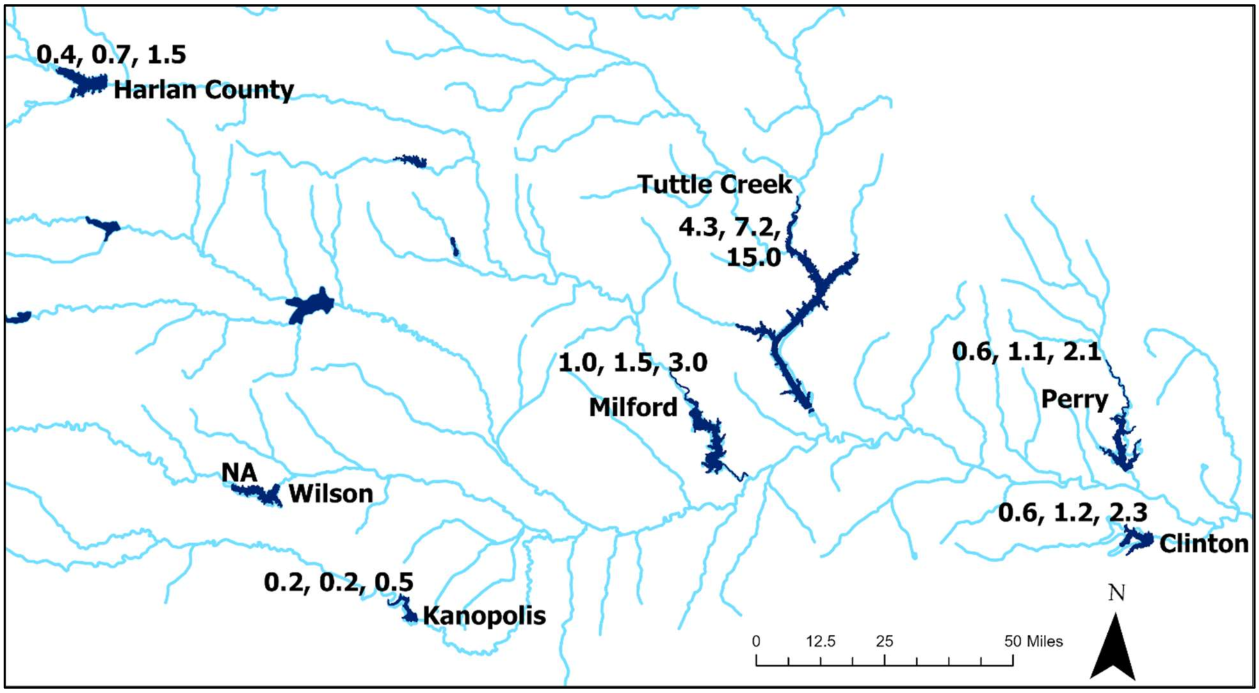


Figure 6-1. Bed and Bank Degradation Downstream from USACE Lakes. *Units in millions of cubic yards at 25, 50, and 100 years.*

7.0 SUMMARY AND CONCLUSIONS

Sedimentation will continue to varying degrees in each lake in the Kansas River Basin, impacting authorized purposes, habitat, operations and maintenance. The projections in this summary appendix and the associated sub-appendices are based on flow-load rating curves calibrated to volumes of deposition between previous surveys. Major assumptions and simplifications for future projections are: (1) The flow-load relationship remains constant in the future, (2) Future inflows resemble the past several decades of inflows, (3) The mass split between the flood control pool and multipurpose pool follows the regression curve given in Figure 3-2 (4) All sediment that deposits in the multipurpose pool deposits as a uniform veneer (only Tuttle Creek was numerically modeled), (5) Deposition near the gates, impacts to recreational facilities, and other location-specific damages are based on this uniform veneer approach. Separate appendices document these impacts.

These are necessary simplifying assumptions that allow the computation of information sufficient for evaluating and ranking recommendations in this Watershed Study. More in-depth analysis is recommended, particularly of Tuttle Creek Lake and Kanopolis Lake, where the projected deposition is most severe. Each lake is briefly summarized here in the order of most to least impacted multipurpose pool.

Within 25, 50, and 100 years, respectively 36.1%, 25.1%, and 6.9% of the original multipurpose pool Tuttle Creek Lake will remain. The flood control pool will shrink to 91.7%, 89.5%, and 83.0% of the original storage. The surface area of the multipurpose pool will shrink to 47.3%, 40.3%, and 20.4% of the original surface area. The downstream channel will degrade 4.3, 7.2, and 15.0 million cubic yards.

Within 25, 50, and 100 years, respectively 47.5%, 41.3%, and 24.9% of the original multipurpose pool Kanopolis Lake will remain. The flood control pool will shrink to 95.7%, 94.9%, and 92.2% of the original storage. The surface area of the multipurpose pool will shrink to 62.8%, 58.8%, and 46.1% of the original surface area. The downstream channel will degrade 0.15, 0.24, and 0.50 million cubic yards.

Within 25, 50, and 100 years, 66%, 58%, and 43% of the original multipurpose pool Perry Lake will remain. The flood control pool will shrink to 96%, 93%, and 88% of the original storage. The surface area of the multipurpose pool will shrink to 74%, 66%, and 59% of the original surface area. The downstream channel will degrade 0.6, 1.1, and 2.1 million cubic yards.

Within 25, 50, and 100 years, respectively 80%, 75%, and 65% of the original multipurpose pool Clinton Lake will remain. The flood control pool will shrink to 108%, 107%, and 105% of the original storage. (Higher than 100% due to additional volume computed using LIDAR vs. the original sedimentation rangelines.) The surface area of the multipurpose pool will shrink to 103%, 97%, and 90% of the original surface area. The downstream channel will degrade 0.6, 1.2, and 2.3 million cubic yards.

Within 25, 50, and 100 years, respectively 87.9%, 84.3%, and 76.7% of the original multipurpose pool Wilson Lake will remain. The flood control pool will shrink to 99.4%, 99.1%, and 98.2% of the original storage. The surface area of the multipurpose pool will shrink to 91.8%, 90.4%, and 87.0% of the original surface area. If the past trends continue, the channel downstream from Wilson Lake will not degrade.

Within 25, 50, and 100 years, respectively 85.7%, 84.2%, and 80.2% of the original multipurpose pool Milford Lake will remain. The flood control pool will shrink to 99.6%, 99.5%, and 99.0% of the original storage. The surface area of the multipurpose pool will shrink to 90.1%, 88.8%, and 85.4% of the original surface area. The downstream channel will degrade 1.0, 1.5, and 3.0 million cubic yards.

Within 25, 50, and 100 years, respectively 87.7%, 86.8%, and 84.0% of the original multipurpose pool Harlan County Lake will remain. The flood control pool will shrink to 99.0%, 98.9%, and 98.5% of the original storage. The surface area of the multipurpose pool will shrink to 91.7%, 90.9%, and 89.3% of the original surface area. The downstream channel will degrade 0.4, 0.7, and 1.5 million cubic yards.

8.0 REFERENCES

Brune, G. M. (1953). Trap Efficiency of Reservoirs. American Geophysical Union.

US Army Corps of Engineers ®
Kansas City District

Kansas River Reservoirs Flood and Sediment Study

Appendix D2.1: Clinton Lake Future Without Project Sedimentation

November 2022

Contents

1.0 INTRODUCTION

This appendix documents the projections for the Future Without Project (FWOP) condition for Clinton Lake. The FWOP condition is the expected conditions of the lake if sediment accumulation continues unabated, i.e., without intentional sediment management or removal. This analysis also assumes no pool raise or reallocation takes place, as such actions would be potential measures for a Future With Project condition. This analysis incorporates the trends documented in the Existing Conditions analysis for Clinton Lake, Appendix D1.1. The location of Clinton Lake with respect to the Kansas River Basin is shown in Figure 1-1

FWOP computations were first completed in December 2020 but were later updated in May 2022 based a new methodology incorporating sediment modeling results from Tuttle Creek Lake. The new methodology included the effects of additional sediment depositing in the flood pool (FP) over time.

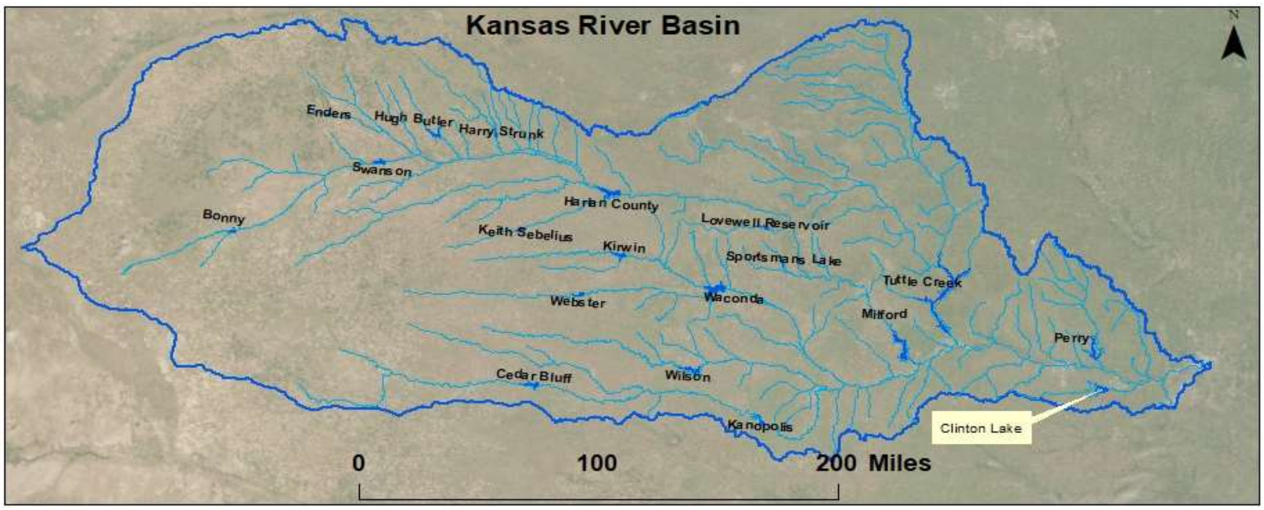


Figure 1-1. Overall Kansas River Basin Map.

2.0 FWOP SEDIMENT DEPOSITION

Long-term sediment deposition was computed by running 100 years of daily flow through the calibrated rating curves described in Appendix D1.1.

A Bulletin 17 analysis was made on the daily lake inflow data and compared to the computed flow frequency curves by the Hydraulics and Hydrology section. Results of the B17C analyses were compared to determine whether the last 50-years adequately represented the period of record. Clinton Lake's flow frequency curve was deemed sufficient to adequately represent the period of record without further manipulation.

The daily flows were developed by using the historic flow from Water Management upstream from Clinton Lake, repeating the years from 1978 to 2019 to obtain a total of 100-years. To obtain total deposition since construction of the lake, calculations were made from the date of the last bathymetric survey in 2009 to the beginning of 2124. The measured flows were used for the period from 2009 to 2019, while data from 1970 to 2019 was used to fill in between 2020 and 2024.

Water use in the watershed has changed over time due to changing demand, which is referred to as depletions. A timeseries of depletion data was obtained from NWK Water Management (see Appendix B) and added to the gauged flows to obtain the FWOP flows. Generally, the depleted flows were less than the gauged flows.

As noted in the Existing Conditions Appendix, 91% of all sediment enters the lake while the lake water surface is in the flood control pool. However, deposition in the flood control pool represents only 20.8% of the total mass. From these numbers, the historic trapping efficiency of the flood control pool is 22.1%. The original FWOP computations assumed that this ratio remained constant over the FWOP. However, additional investigations indicated that additional mass should be deposited in the FP as the multipurpose pool (MPP) fills in.

For the final FWOP calculations, the percentage of the incoming mass that is deposited in the FP was varied over time based on a regression equation developed from lake surveys and numerical modeling at Tuttle Creek Lake. Figure 2-1 shows the percentage of total incoming mass that deposits in the FP on the Y-axis and the percent volume lost from the MPP on the X-axis. Both survey data and HEC-RAS sediment model results for Tuttle Creek are included in Figure 2-1. See Appendix D4 for additional details on the HEC-RAS sediment modeling. A polynomial regression equation was fitted to the data, until the MPP has lost 80% of its original volume, after which the % mass to the FP is held constant.

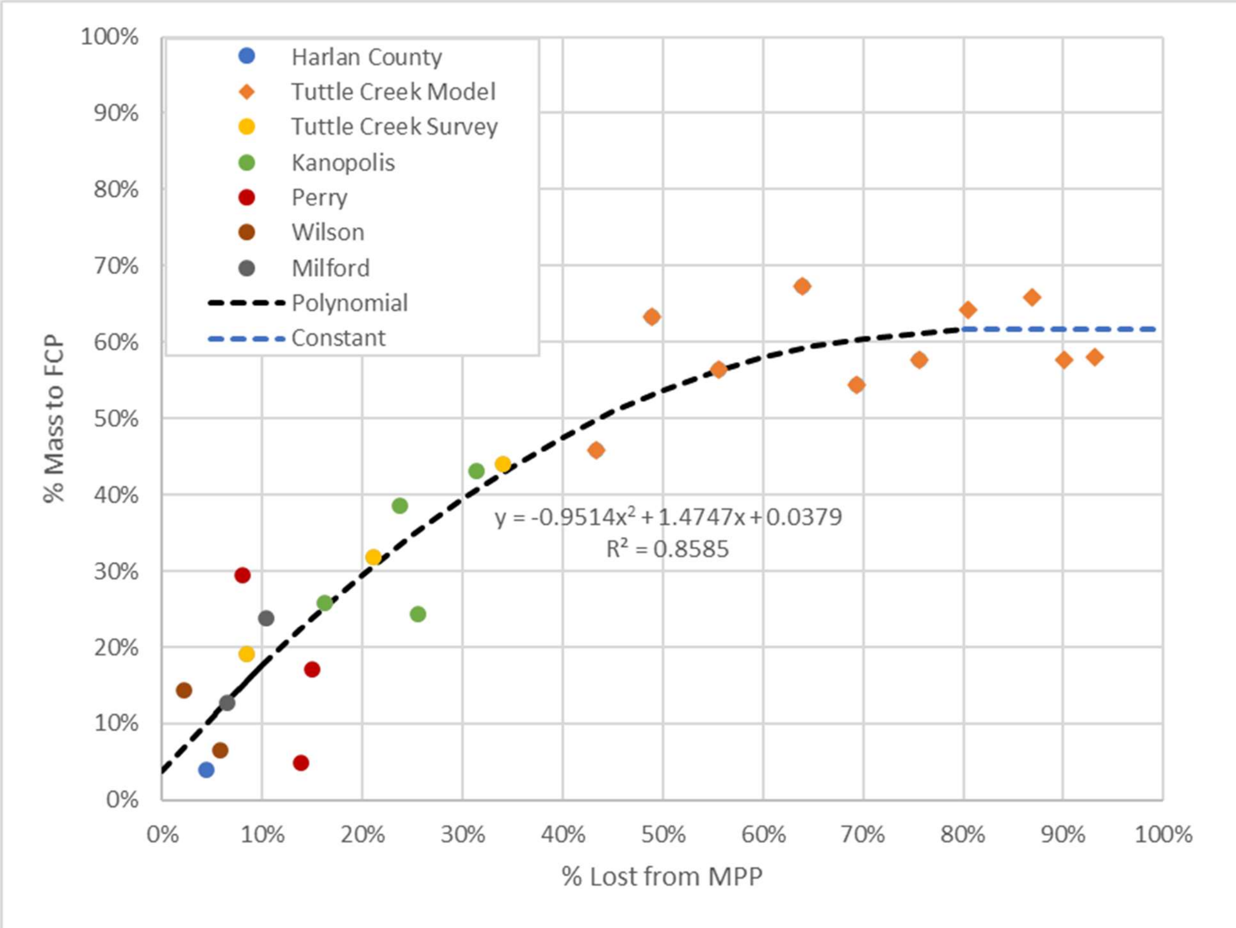


Figure 2-1. The percentage of mass depositing in the FP vs. the percent of volume lost from the MPP

For the multipurpose pool, the trapping efficiency starts at 97%, as measured by USGS (Juracek 2013). The trapping efficiency decreases over time, using the Brune Curve method.

The Brune Curve estimate for Clinton Lake in 2019 is 96.16%. In the FWOP analysis, the trapping efficiency was assumed to start at the USGS value but decrease parallel to the Brune Curve. This was accomplished by adding 0.81 to the Brune Curve estimation at each daily time step. As the lake fills in, the trapping efficiency of the multipurpose pool decreases. Table 2-1 lists the MPP trapping efficiency at year 0, 25, 50, and 100.

Table 2-1: Trapping Efficiency over Time

Year	Cumulative Incoming Load (tons)	Deposited in the FCP (tons)	Passed to the MPP (tons)	MPP Final TE (%)
0	964,496	208,074	756,422	96.99
25	10,574,607	2,702,795	7,871,812	96.86
50	19,738,626	5,704,558	14,034,068	96.72
100	38,218,861	13,191,985	25,026,876	96.40

As the lake shrinks, the trapping efficiency decreases (at Clinton only slightly). The volume that deposits over time is only subdivided into the MPP or the FCP (not into discreet elevations within each pool). To make this scheme workable but still incorporate shrinking pools over time, two simplifying assumptions were incorporated. (1) The volume of the multi-purpose pool is used to compute the trapping efficiency, independent of whether the pool elevation is above or below the MPP level on any particular day. (2) The mass is apportioned based on the ratios given in Figure 2-1

This list summarizes the steps to compute future sedimentation volumes and remaining pool capacities:

1. For each day, compute the mass of incoming sediment using the calibrated rating curve.
2. Compute the percentage of mass that deposits in the FCP. The remaining mass is passed to the MPP. As previously discussed, the original FWOP calculations assumed this ratio remained constant.
3. Use the Brune Curve to compute the trapping efficiency.
4. Apply the trapping efficiency to mass entering the MPP to compute trapped mass.
5. Use the bulk density of the FCP and MPP to covert the mass trapped into a volume trapped.
6. Subtract the volume of deposition in the FCP and MPP from the remaining capacities.
7. Repeat for each subsequent day, re-computing trapping efficiency with the progressively smaller MPP capacity.

Figure 2-2 provides the remaining storage capacities of the multipurpose and flood control pools for both the original and final FWOP computations. Table 2-2 and Table 2-3 summarize the final results.

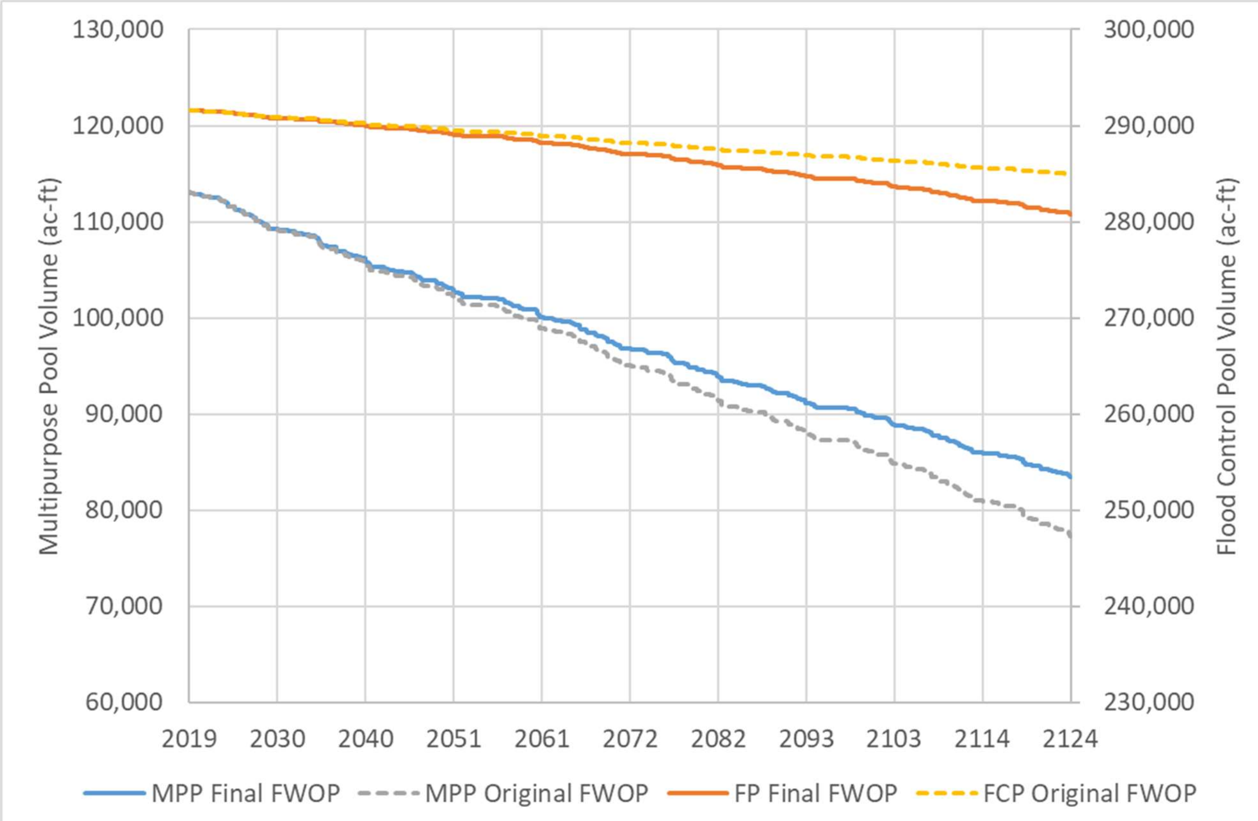


Figure 2-2. Remaining Pool Volumes of the Multipurpose and Flood Control Pools Over Time.

Table 2-2. Summary of Remaining Pool Volumes.

Year	Multi-Purpose Pool (ac-ft)	Multi-Purpose Pool % of Original Volume	Flood Control Pool (ac-ft)	Flood Control Pool % of Original Volume
1977	129,171	100%	268,367	100%
2019	113,032	88%	291,570	109%
2024	112,150	87%	291,403	109%
2049	103,854	80%	289,406	108%
2074	96,669	75%	287,003	107%
2124	83,853	65%	281,010	105%

By 2124, the multipurpose pool has shrunk to 65% of its original capacity. As explained in Appendix D1.1, the shift in survey methods from rangelines to LIDAR revealed a larger flood control pool than originally computed using sedimentation rangelines. Thus, while deposition occurs in the flood control pool, by 2124 the flood control pool volume is still larger than the original estimate.

Table 2-3. Sediment Deposition Volumes.

Year	Cumulative Deposition Multi-Purpose Pool (ac-ft)	Average Annual MPP Deposition (ac-ft/yr)	Average Annual MPP Deposition Since 2024 (ac-ft/yr)	Average Annual FCP Deposition (ac-ft/yr)	Average Annual FCP Deposition Since 2024 (ac-ft/yr)
2019	16,139	-	-	-	-
2024	17,021	209	-	40	-
2049	25,317	332	332	80	80
2074	32,502	287	310	96	88
2124	45,318	256	283	120	104

Table 2-4 gives the storage volumes from the original iteration of the FWOP computations along with the difference from the final results. As seen in Table 2-4, the MPP has greater storage over the FWOP for the final results, while the FP has less storage.

Table 2-4. Pool volumes in acre-feet from the original FWOP computations, and the differences from the final computations

Year	Multi-Purpose Pool Original	Flood Control Pool Original	Difference Multipurpose Pool	Difference Flood Control Pool
2024	112,142	291,404	+8	-1
2049	103,319	289,760	+535	-354
2074	94,870	288,184	+1,800	-1,180
2124	77,965	285,019	+5,888	-4,009

FWOP elevation-storage and elevation-area curves were estimated for use in the Water Management analyses documented in Appendix B. For the MPP, the curves were estimated using the surface volume tool in Arc-GIS along with the digital elevation models (DEMs) discussed in Section 3. However, for the FP portion of the lake, the curves were estimated based on the daily pool elevation from the FWOP HEC-ResSIM modeling (see Appendix B). In this analysis, the FP was broken up into 5-ft increments and for each day in the FWOP, the projected deposition was evenly distributed across all increments at or below the daily pool elevation. The reduction in surface area over that increment was then estimated by assuming it had the same percent reduction as the storage volume. However, because different methods were used to estimate the MPP and FP curves, a discontinuity was observed in the surface area between them. This was resolved replacing the surface area for first point in the FP with the value obtained by interpolating between the final MPP point and the second FP point. This methodology compared well with the 1D HEC-RAS sediment modeling results discussed in Appendix D4. See Appendix B for the final elevation-storage and elevation-area curves.

3.0 DELTA PROGRESSION

As listed in Appendix D1.1, the last two bathymetric surveys do not indicate discernible delta progression rate towards the dam. As deltas progress into wider and deeper sections of the lake, the rate of progression slows. The future rate was estimated by finding the delta progression that would yield the correct volume of sediment deposition as listed in Table 2-3. The delta progression analysis provides approximate results based on applying MPP deposition as an even veneer of deposition over all areas under the multipurpose pool elevation. Projected digital elevation models (DEM), recent surface area, and elevation contours of the MPP were computed using the following steps in ArcGIS.

1. Divide the projected deposition volume by the surface area of the MPP of 2019 to obtain the new elevation change within the MPP.
2. Add this elevation to the 2019 DEM of the MPP to obtain a first estimate of the projected DEM.
3. The surface volume tool was used to estimate the new volume and area of the lake.
4. Determine the difference between the predicted MPP volume and the MPP from the modified DEM
5. Divide the volume difference by the new surface area to obtain an additional elevation increase.
6. Add the additional elevation increase to the modified DEM
7. Repeat steps 3-7 until the DEM MPP volume was approximately equal to the projected MPP volume. A difference less than the average annual deposition was considered to be adequate.

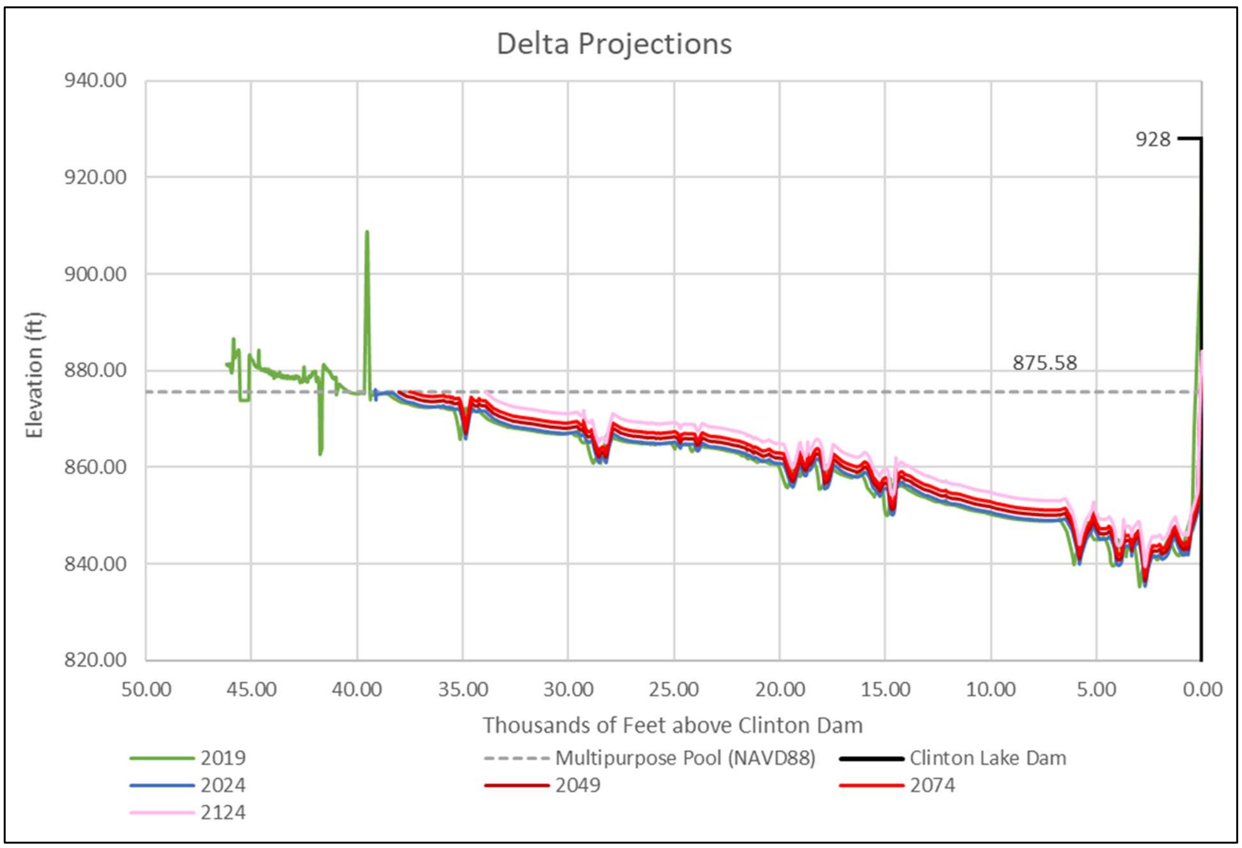


Figure 3-1: Projected Delta

The 25, 50, and 100-year projections indicate delta progression of 354, 876, and 4,476 ft. This equates to a loss in multipurpose pool surface area, as listed in Table 3-1.

Table 3-1. Multipurpose Pool Surface Area.

Year	MPP Surface Area (ac)	Lost Surface Area compared to Original (ac)	Lost Surface Area Compared to 2024 (ac)
1977	7,006	-	-
2019	7,541	-535	-
2024	7512	-506	-
2049	7224	-218	288
2074	6799	207	713
2124	6333	673	1179

Figure 3-2 shows the elevation contour of the MPP for various years. The 1969 contour was digitized from a historic map of the lake.

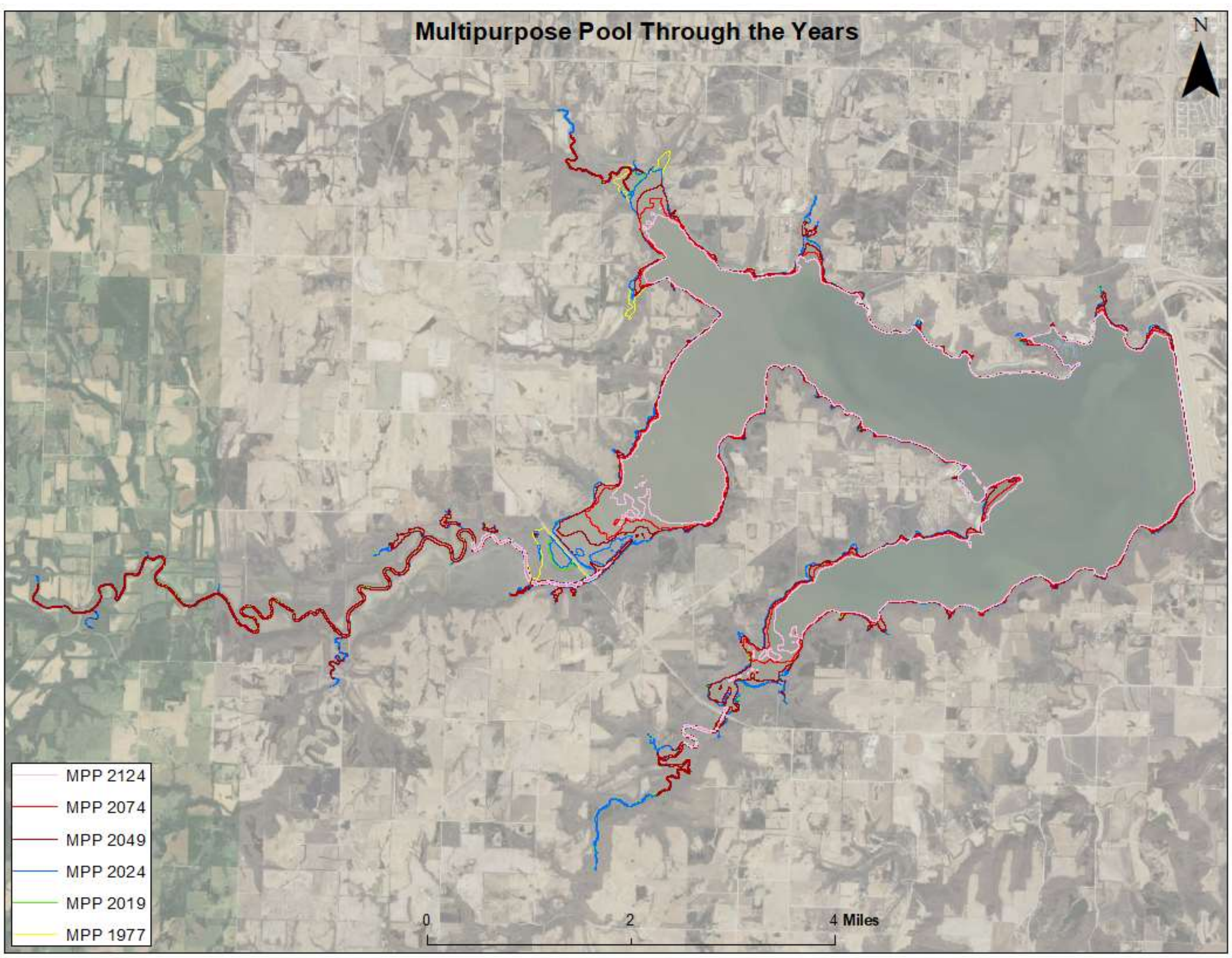


Figure 3-2. Multipurpose Pool Through the Years.

4.0 DOWNSTREAM SEDIMENT CONCENTRATIONS

The downstream sediment concentrations are not expected to change significantly over the FWOP. Although the MPP trapping efficiency declines slightly (Table 2-1), the FP trapping efficiency is expected to increase over time (Figure 2-1), which will offset the declining MPP trapping efficiency.

5.0 DOWNSTREAM CHANNEL

The existing conditions report demonstrated bed degradation and bank erosion downstream from Clinton Lake. Over the next 25, 50, and 100 years an additional 961,011; 1,877,413; and 3,725,437 tons of sand will be trapped in Clinton Lake which will further induce bed degradation and bank erosion on the Wakarusa River. Geomorphic principles suggest that rivers adjust to transport the bed material load supplied to them. Thus, the downstream channel will degrade (which lowers the slope and hence the applied shear stress) and an amour layer will form (which increases the critical shear required to initiate motion) until the flow from the dam is no longer able to transport sediment. Sedimentation modeling beyond the scope of this watershed study is required to compute the final bed elevations and to estimate bank erosion at the end of 25, 50, and 100 years. While applying trendlines to degradation or bank erosion is not ideal, it has been used when detailed modeling is not practical (references). Herein is provided an order-of-magnitude approximation based on assumptions that geomorphic feedbacks do not change the rates of geomorphic change relative to the historic rates of change. This approximation was computed via the following steps:

1. Compute the ratio of sand trapped in the lake to volumetric degradation downstream (ac-ft) over a defined distance. For Clinton Lake, 1,539,329 tons of sand trapped by the dam induced 592.8 ac-ft of bed and bank degradation in the 19.74 miles downstream of the dam, or a ratio of 592.8 ac-ft / 759.96 ac-ft of downstream sand degradation. The ratio value is 0.78.
2. Apply the ratio to the future estimates for trapped sand.

Table 5-1 provides the estimated degradation volume in the 4.4 miles below Clinton Lake Dam at the end of 25, 50, and 100 years. This analysis provides a high-level assessment suitable for relative ranking of alternatives.

Table 5-1. Estimated Degradation for Sand.

Year	Total Clay Accumulation (ac-ft)	Total Silt Accumulation (ac-ft)	Total Sand Accumulation (ac-ft)	Total Accumulated Sediment (ac-ft)	Downstream Degradation as 0.78 Accumulated Sand (ac-ft)
2024	236	193	48	476	37
2049	2,584	2,114	522	5,221	407
2074	4,824	3,947	974	9,745	760
2124	9,340	7,642	1,887	18,868	1472

6.0 SUMMARY, LIMITATIONS, AND CONCLUSIONS

This appendix documented projections for the 25, 50, and 100-year FWOP condition at Clinton Lake. The following list summarizes the main points for the 50-year FWOP. (For the 25 years and 100 years projections, see the body of the report.)

- 16,896,717 tons of fine sediment (silt/clay) and 1,877,413 tons of coarse sediment (sand/gravel) are expected to enter Clinton Lake, of which 5,496,484 tons are trapped in the flood control pool, 14,034,067 tons are trapped in the multi-purpose pool, and 208,075 tons pass downstream.
- The flood control pool (FCP) and multi-purpose pool (MPP) shrinks by an additional 4,400 ac-ft and 15,481ac-ft, respectively.
- At the end of 50 years, the MPP has 75% remaining of its original capacity.
- At the end of 50 years, the FCP has 107% remaining of its original capacity. (Sediment does deposit in and shrink the flood control pool, but the shift from sedimentation rangelines to LIDAR reflected a significant increase in flood control pool volume compared to the original, so by the end of 50 years, more volume is still available than the original estimate.)
- The multipurpose pool surface area shrinks from 7,512 to 6,799 acres.
- The downstream fine sediment concentrations will rise to 109% of existing.
- The downstream channel will experience additional bed degradation and bank erosion, with 723 ac-ft degrading from the lower 19.74 miles.

These findings are based on trendline projections and empirical equations. For more precise estimations, particularly of where in the pools the sediment will deposit and how the downstream channels will adjust over time, numerical modeling is recommended. Moreover, this analysis did not compute damages due to upstream delta migration and only approximated downstream degradation as a total volume, not as bank erosion or bed degradation at specific affected infrastructure.

Notwithstanding the limitations, these projections indicate that sediment accumulation in Clinton Lake will be a serious problem over the next 50 years with serious implications for loss in benefits, environmental harm, infrastructure damage, and increased O&M needs. These trends continue over the 100 years of analysis.

US Army Corps
of Engineers ®
Kansas City District

Kansas River Reservoirs Flood and Sediment Study

Appendix D2.2: Perry Lake Future Without Project Sedimentation

November 2022

Contents

1.0 INTRODUCTION

This appendix documents the projections for the Future Without Project (FWOP) condition for Perry Lake. The FWOP condition is the expected conditions of the lake if sediment accumulation continues unabated, i.e., without intentional sediment management or removal. This analysis also assumes no pool raise or reallocation takes place, as such actions would be potential measures for a Future With Project condition. This analysis incorporates the trends documented in the Existing Conditions analysis for Perry Lake, Appendix D1.2. Figure 1-1 shows the location of Perry Lake with respect to the Kansas River Basin.

FWOP computations were first completed in December 2020 but were later updated in May 2022 based a new methodology incorporating sediment modeling results from Tuttle Creek Lake. The new methodology included the effects of additional sediment depositing in the flood pool (FP) over time.

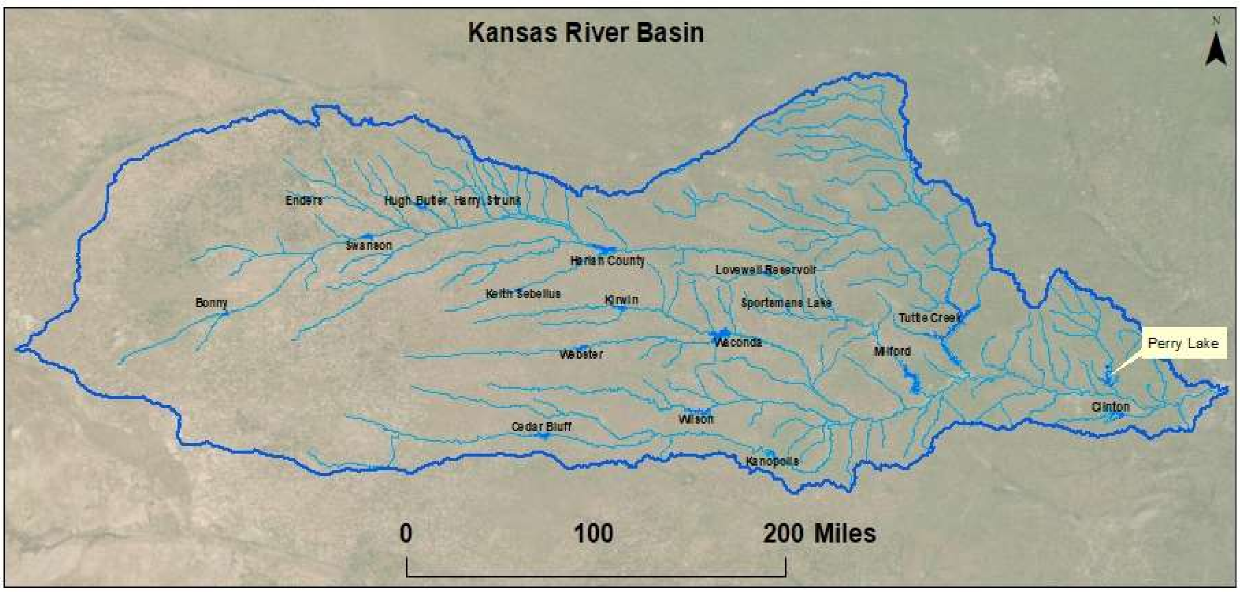


Figure 1-1: Overall Kansas River Basin

2.0 FWOP SEDIMENT DEPOSITION

Long-term sediment deposition was computed by running 100 years of daily flow through the calibrated rating curves described in Appendix D1.2.

A Bulletin 17 analysis was made on the daily lake inflow data and compared to the computed flow frequency curves by the Hydraulics and Hydrology section. Results of the B17C analyses were compared to determine whether the last 50-years adequately represented the period of record. Most of the lakes had flow frequency curves that looked close enough to assume that the last 50 years of data adequately represent the period of record without further analysis.

The daily flows were developed by using the historic flow at the U.S Geological Survey (USGS) gage 06890100 upstream from Perry Lake, repeating the year period from 1970 to 2019 to obtain a total of 100-years. To obtain total deposition since construction of the lake, calculations were made from the date of the last bathymetric survey in 2009 to the beginning of 2124. The measured flows were used for the period from 2009 to 2019, while data from 1970 to 2019 was used to fill in between 2020 and 2024.

Water use in the watershed has changed over time due to changing demand, which is referred to as depletions. A timeseries of depletion data was obtained from NWK Water Management (see Appendix B) and added to the gauged flows to obtain the FWOP flows. Generally, the depleted flows were less than the gauged flows.

As noted in the Existing Conditions Appendix, 93% of all sediment enters the lake while the lake water surface is in the flood control pool. However, deposition in the flood control pool represents 20.8% of the total mass. From these numbers, the historic trapping efficiency of the flood control pool is 21.7%. The original FWOP computations assumed that the FP trapping efficiency remained constant over the FWOP. However, additional investigations indicated that additional mass should be deposited in the FP as the multipurpose pool (MPP) fills in.

For the final FWOP calculations, the percentage of the incoming mass that is deposited in the FP was varied over time based on a regression equation developed from lake surveys and numerical modeling at Tuttle Creek Lake. Figure 2-1 shows the percentage of total incoming mass that deposits in the FP on the Y-axis and the percent volume lost from the MPP on the X-axis. Both survey data and HEC-RAS sediment model results for Tuttle Creek are included in Figure 2-1. See Appendix D4 for additional details on the HEC-RAS sediment modeling. A polynomial regression equation was fitted to the data, until the MPP has lost 80% of its original volume, after which the % mass to the FP is held constant.

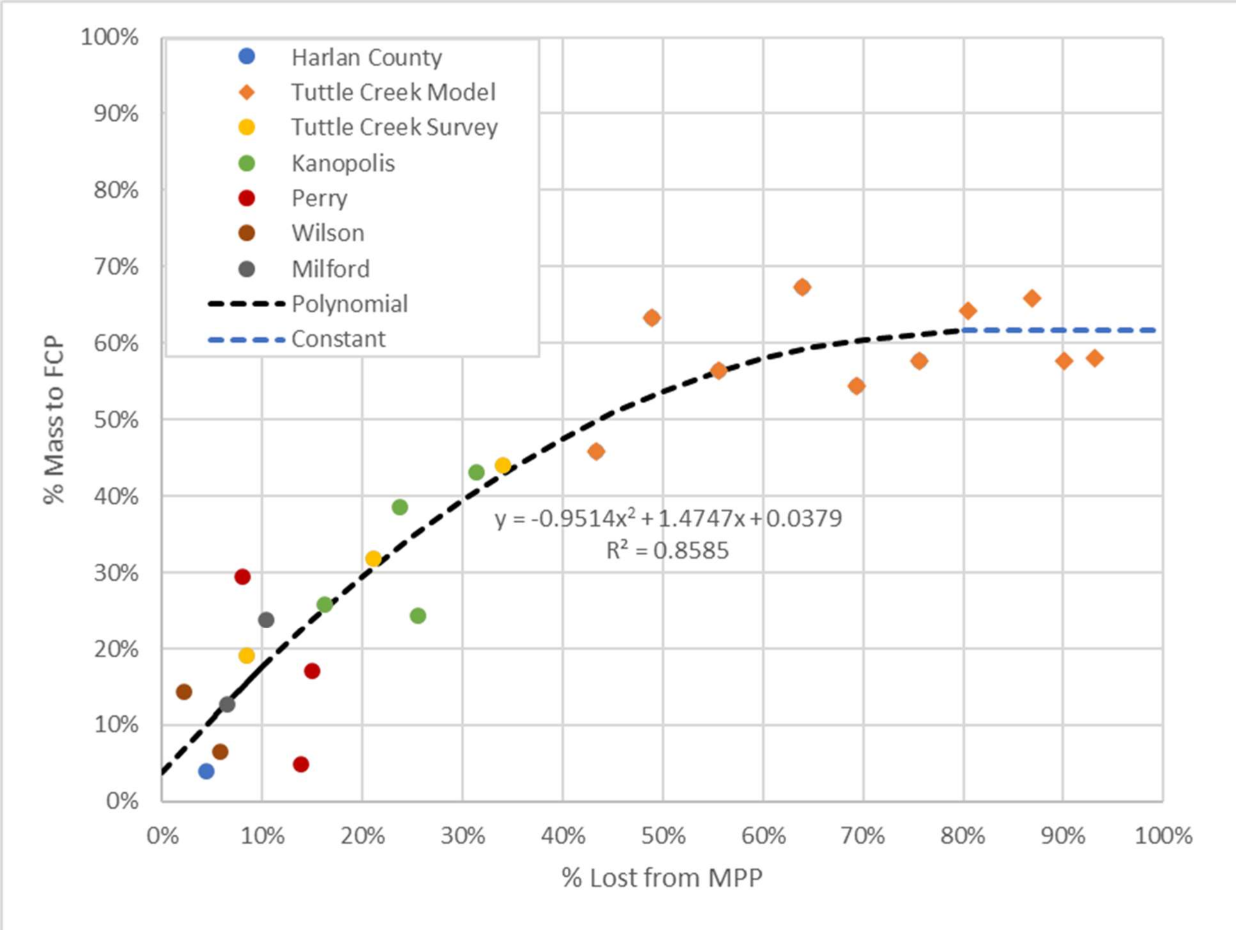


Figure 2-1. The percentage of mass depositing in the FP vs. the percent of volume lost from the MPP

For the multipurpose pool, the trapping efficiency starts at 96.77%, based on sediment inflow and outflow measurements. The trapping efficiency decreases over time, using the Brune Curve method.

The Brune Curve estimate for Perry Lake in 2009 is 96.70%. In the FWOP analysis, the trapping efficiency was assumed to start at the 2009 survey value but decrease parallel to the Brune Curve. This was accomplished by adding 0.0007 to the Brune Curve estimate. As the lake fills in, the trapping efficiency of the multipurpose pool decreases, but only slightly. Table 2-1 lists the MPP trapping efficiency at year 0, 25, 50, and 100.

Table 2-1: Trapping Efficiency over Time

Year	Cumulative Incoming Load (tons)	Deposited in the FCP (tons)	Passed to the MPP (tons)	MPP Final TE (%)
0	21,444,713	6,769,951	14,674,762	96.63
25	55,070,795	13,465,700	41,605,095	96.50
50	86,767,784	15,025,973	71,741,811	96.35
100	152,227,370	35,627,692	116,599,678	95.89

As the lake shrinks, the trapping efficiency decreases slightly. The volume that deposits over time is only subdivided into the MPP or the FCP (not into discreet elevations within each pool). To make this scheme workable but still incorporate shrinking pools over time, two simplifying assumptions were incorporated. (1) The volume of the multi-purpose pool is used to compute the trapping efficiency, independent of whether the pool elevation is above or below the MPP level on any particular day. (2) The mass is apportioned based on the ratios given in Figure 2-1. This list summarizes the steps to compute future sedimentation volumes and remaining pool capacities:

1. For each day, compute the mass of incoming sediment using the calibrated rating curve.
2. Compute the percentage of the mass deposits in the FCP. The remaining mass is passed to the MPP. As previously discussed, the original FWOP calculations assumed this ratio remained constant.
3. Use the Brune Curve plus 0.0007 to compute the trapping efficiency.
4. Apply the trapping efficiency to mass entering the MPP to compute trapped mass.
5. Use the bulk density of the FCP and MPP to covert the mass trapped into a volume trapped.
6. Subtract the volume of deposition in the FCP and MPP from the remaining capacities.
7. Repeat for each subsequent day, re-computing trapping efficiency with the progressively smaller MPP capacity.

Figure 2-1 provides the remaining storage capacities of the multipurpose and flood control pools for both the original and final FWOP computations. Table 2-2 and Table 2-3 summarize the final results.

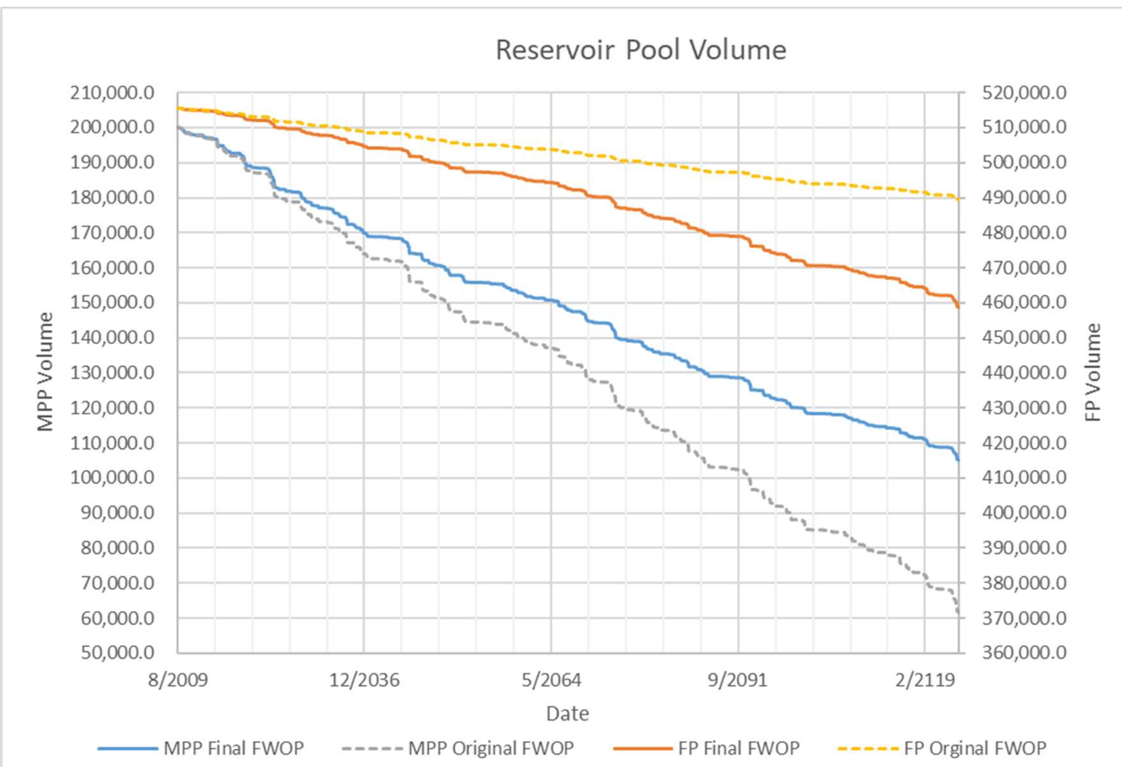


Figure 2-2: Remaining Pool Volumes of the Multipurpose and Flood Control Pools Over Time

Table 2-2: Summary of Remaining Pool Volumes

Year	Multi-Purpose Pool (ac-ft)	Multi-Purpose Pool % of Original Volume	Flood Control Pool (ac-ft)	Flood Control Pool % of Original Volume
1969	243,220	100%	521,880	100%
2009	200,004	82%	515,519	99%
2024	182,893	75%	510,100	98%
2049	159,384	66%	499,323	96%
2074	139,945	58%	487,296	93%
2124	105,160	43%	458,780	88%

Table 2-3: Sediment Deposition Volumes

Year	Cumulative Deposition Multi-Purpose Pool (ac-ft)	Average Annual MPP Deposition (ac-ft/yr)	Average Annual MPP Deposition Since 2024 (ac-ft/yr)	Cumulative Deposition Flood Control Pool (ac-ft)	Average Annual FCP Deposition (ac-ft/yr)	Average Annual FCP Deposition Since 2024 (ac-ft/yr)
2009	43,216	1,085	-	6,361	160	-
2024	60,327	1,187	-	11,780	376	-
2049	83,836	940	940	22,557	431	431
2074	103,275	778	859	34,584	481	481
2124	138,060	696	777	63,100	570	570

Table 2-4 gives the storage volumes from the original iteration of the FWOP computations along with the difference from the final results. As seen in Table 2-4, the MPP has greater storage over the FWOP for the final results, while the FP has less storage.

Table 2-4: Pool volumes in acre-feet from the original FWOP computations, and the differences from the final computations

Year	Multi-Purpose Pool Original	Flood Control Pool Original	Difference Multipurpose Pool	Difference Flood Control Pool
2024	180,366	511,851	+2,527	-1751
2049	149,573	506,093	+9,812	-6,770
2074	120,752	500,690	+19,194	-13,394
2124	61,732	489,529	+43,428	-30,749

FWOP elevation-storage and elevation-area curves were estimated for use in the Water Management analyses documented in Appendix B. For the MPP, the curves were estimated using the surface volume tool in Arc-GIS along with the digital elevation models (DEMs) discussed in Section 3. However, for the FP portion of the lake, the curves were estimated based on the daily pool elevation from the FWOP HEC-ResSIM modeling (see Appendix B). In this analysis, the FP was broken up into 5-ft increments and for each day in the FWOP, the projected deposition was evenly distributed across all increments at or below the daily pool elevation. The reduction in surface area over that increment was then estimated by assuming it had the same percent reduction as the storage volume. However, because different methods were used to estimate the MPP and FP curves, a discontinuity was observed in the surface area between them. This was resolved replacing the surface area for first point in the FP with the value obtained by interpolating between the final MPP point and the second FP point. This methodology compared well with the 1D HEC-RAS sediment modeling results discussed in Appendix D4. See Appendix B for the final elevation-storage and elevation-area curves.

3.0 DELTA PROGRESSION

As listed in Appendix D1.2, the delta progression rate towards the dam from 1979 to 2009 was 397 ft per year. As deltas progress into wider and deeper sections of the lake, the rate of progression is expected to slow. The future rate was estimated by finding the delta progression that would yield the correct volume of sediment deposition as listed in Table 2-3. The delta progression analysis provides approximate results based on applying MPP deposition as an even veneer of deposition over all areas under the multipurpose pool elevation. This was accomplished using ArcGIS to add the elevation change, determined by dividing projected volume change by lake surface area, to the 2009 bathymetric surface. Projected digital elevation models (DEM), recent surface area, and elevation contours of the MPP were computed using the following steps in ArcGIS.

1. Divide the projected deposition volume by the surface area of the MPP of 2009 to obtain the new elevation change within the MPP.
2. Add this elevation to the 2009 DEM of the MPP to obtain a first estimate of the projected DEM.
3. Use the surface volume tool to estimate the new MPP volume and surface area.
4. Determine the difference between the predicted MPP volume and the MPP from the modified DEM
5. Divide the volume difference by the new surface area to obtain an additional elevation increase.
6. Add the additional elevation increase to the modified DEM
7. Repeat steps 3-7 until the DEM MPP volume approximately equal to the projected MPP volume. A difference less than the average annual deposition was considered to be adequate.

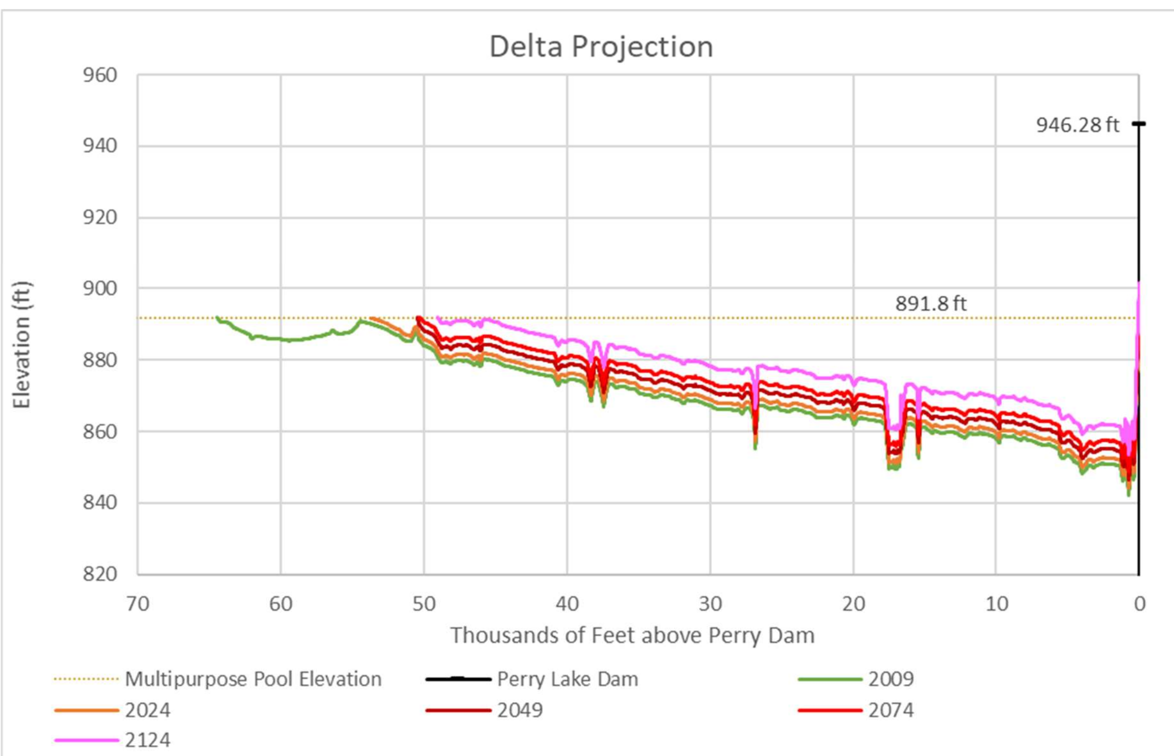


Figure 3-1: Projected Delta Location

The 25, 50, and 100-year projections indicate a horizontal delta progression of 3,189, 3,399, and 4,661 ft. This equates to a loss in multipurpose pool surface area, as listed in Table 3-1.

Table 3-1: Multipurpose Pool Surface Area Predictions

Year	MPP Surface Area (ac)	Lost Surface Area compared to Original (ac)	Lost Surface Area Compared to 2024 (ac)
1969	12,202	-	-
2009	10,227	1,975	-
2024	9771	2,431	-
2049	9007	3,195	764
2074	8055	4,147	1716
2124	7191	5,011	2580

Figure 3-2 shows the elevation contour of the MPP for various years. The 1969 contour was digitized from a historic map of the lake and is approximate.

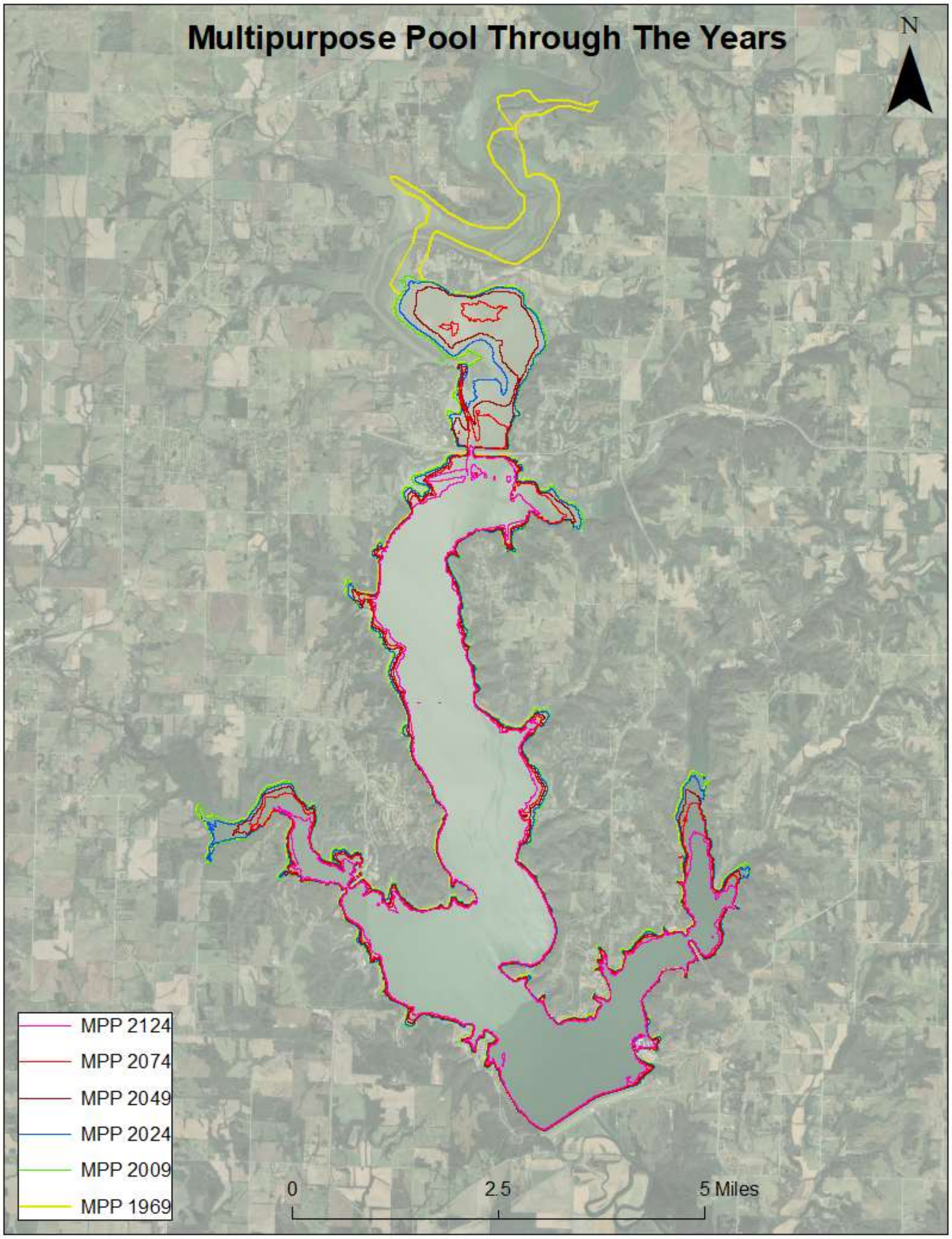


Figure 3-2: Multipurpose Pool Through the Years

Observations about deposition relative to important lakeside infrastructure will be assessed in other appendices. This delta progression analysis provides approximate results based on applying MPP deposition as an even veneer of deposition over all areas under the multipurpose pool elevation. When the elevation reaches the top elevation of the MPP, no further deposition is assumed for that location and the rest of the volume is spread over the remaining nodes.

4.0 DOWNSTREAM SEDIMENT CONCENTRATIONS

The downstream sediment concentrations are not expected to change significantly over the FWOP. Although the MPP trapping efficiency declines slightly (Table 2-1), the FP trapping efficiency is expected to increase over time (Figure 2-1), which will offset the declining MPP trapping efficiency.

5.0 DOWNSTREAM CHANNEL

The existing conditions report demonstrated bed degradation and bank erosion downstream from Perry Lake. Over the next 25,50 and100 years, an additional 3,479,006, 6,758,425 and 13,530,975 tons of sand will be trapped in Perry Lake which will further induce bed degradation and bank erosion on the Delaware River. Geomorphic principles suggest that rivers adjust to transport the bed material load supplied to them. Thus, the downstream channel will degrade (which lowers the slope and hence the applied shear stress) and an amour layer will form (which increases the critical shear required to initiate motion) until the flow from the dam is no longer able to transport sediment. Sedimentation modeling beyond the scope of this watershed study is required to compute the final bed elevations and to estimate bank erosion at the end of 25, 50, and 100 years. While applying trendlines to degradation or bank erosion is not ideal, it can provide a first approximation. Herein is provided an order-of-magnitude approximation based on assumptions that geomorphic feedbacks do not change the rates of geomorphic change. This approximation was computed via the following steps:

1. Compute the ratio of sand trapped in the lake to volumetric degradation downstream (ac-ft) over a defined distance. For Perry Lake, 6,749,005 tons of sand trapped by the dam induced 662.8 ac-ft of bed and bank degradation in the 4.4 miles downstream of the dam, or a ratio of 662.8 ac-ft / 3,332 ac-ft of downstream sand degradation. The ratio value is 0.20.
2. Apply the ratio to the future estimates for trapped sand.

Table 5-1 provides the estimated degradation volume in the 4.4 miles below Perry Lake dam at the end of 0, 25, 50, and 100 years. This analysis provides a high-level assessment suitable for relative ranking of alternatives.

Table 5-1: Estimated Degradation for Sand

Year	Total Accumulation Clay (ac-ft)	Total Accumulation Silt (ac-ft)	Total Accumulation Sand (ac-ft)	Total Accumulated Sediment (ac-ft)	Downstream Degradation as 0.2 Accumulated Sand (ac-ft)
2024	5,207	4,285	1,095	10,587	218
2049	13,372	11,004	2,813	27,188	560
2074	21,068	17,337	4,432	42,837	882
2124	36,962	30,416	7,776	75,154	1,547

6.0 SUMMARY, LIMITATIONS, AND CONCLUSIONS

This appendix documented projections for the 50-year FWOP condition at Perry Lake. (For the 25 years and 100 years projections, see the body of the report.) The list summarizes the main points:

- 58,564,645.04 tons of fine sediment (silt/clay) and 6,758,425 tons of coarse sediment (sand/gravel) are expected to enter Perry Lake, of which 18,671,557 tons are trapped in the flood control pool, 68,494,019 tons are trapped in the multi-purpose pool, and 2,500,342 tons pass downstream.
- The flood control pool (FCP) and multi-purpose pool (MPP) shrinks by an additional 22,805 ac-ft and 42,947 ac-ft, respectively.
- At the end of 50 years, the MPP has 58% remaining of its original capacity.
- At the end of 50 years, the FCP has 93% remaining of its original capacity.
- At the end of 50 years, the top of the delta progresses downstream an additional 14,350 ft.
- The downstream fine sediment concentrations will rise to 108% of existing.
- The downstream channel will experience additional bed degradation and bank erosion, with 664 ac-ft sand degrading from the lower 4.4 miles.

These findings are based on trendline projections and empirical equations. For more precise estimations, particularly of where in the pools the sediment will deposit and how the downstream channels will adjust over time, numerical modeling is recommended. Moreover, this analysis did not compute damages due to upstream delta migration and only approximated downstream degradation as a total volume, not as bank erosion or bed degradation at specific affected infrastructure.

Notwithstanding the limitations, these projections indicate that sediment accumulation in Perry Lake will be a serious problem over the next 50 years, with the trends continuing through the 100 years analyzed. This will have serious implications for loss in benefits, environmental harm, infrastructure damage, and increased O&M needs that will be discussed in other appendices.

US Army Corps of Engineers®
Kansas City District

Kansas River Reservoirs Flood and Sediment Study

Appendix D2.3: Tuttle Creek Lake Future Without Project Sedimentation

November 2022

Contents

1.0 INTRODUCTION

This appendix documents the projections for the Future Without Project (FWOP) condition for Tuttle Creek Lake. The FWOP condition is the expected conditions of the lake if sediment accumulation continues unabated, i.e., without intentional sediment management or removal. This analysis also assumes no pool raise or reallocation takes place, as such actions would be potential measures for a Future With Project condition. This analysis incorporates the trends documented in the Existing Conditions analysis for Tuttle Creek Lake, Appendix D1.3. Figure 1-1 shows the lake with respect to the Kansas River Basin.

FWOP computations were first completed in December 2020 but were later updated in May 2022 based on HEC-RAS 1D sediment modeling results. Results from the sediment modeling showed that additional sediment would deposit in the flood pool (FP) over time.

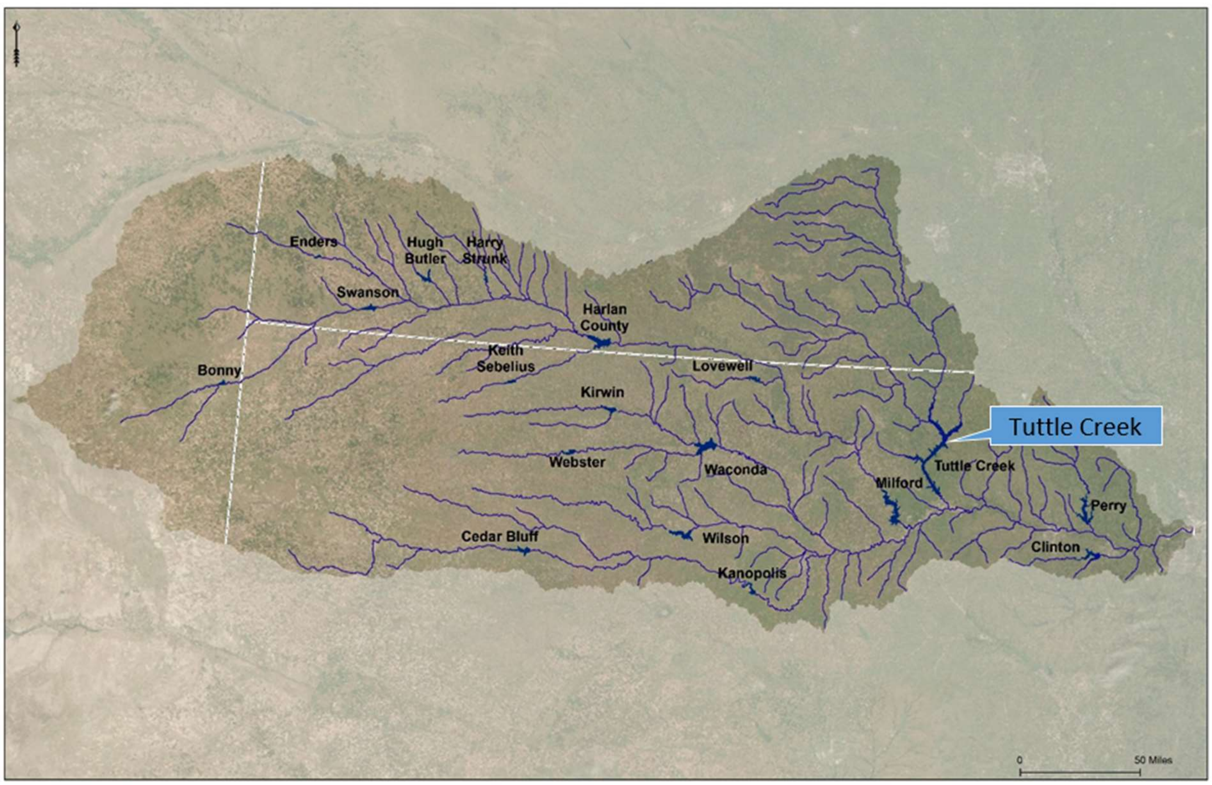


Figure 1-1: Overall Kansas River Basin Map and Tuttle Creek Lake

2.0 FWOP SEDIMENT DEPOSITION

Long-term sediment deposition was computed by running 100 years of daily flow through the calibrated rating curves described in Appendix D1.3. The flows were obtained for the three U.S. Geological Survey (USGS) gages upstream of the lake used in the calibration, repeating the 50-year period from 1970 to 2019 to obtain a total of 100-years. The beginning date for the simulation was 31 December 2024, the assumed date this project will be completed. To obtain total deposition since construction of the lake, calculations were made from the date of the last bathymetric survey in 2009 to the end of 2124. The measured flows were used for the period from 2009 to 2019, while data from 1970 to 1975 was used to fill in between 2020 and 2024. Since the Big Blue River at Marysville, Kansas gage did not have a 50-year period of record, the flow record was extended using the Big Blue River near Barnseton, Nebraska gage as described in Appendix D1.3.

The lake inflow data, which is calculated by NWK Water Management Section, was used to check for stationarity and climate change. This was done by determining how well the flow frequency statistics based on the previous 50-years of data match with the entire available period of record for the lakes. A Bulletin 17 analysis was performed on the daily lake inflow data (annual maxima) and the computed flow frequency curves were compared. The curves matched closely enough to use the last 50 years of data (1970-2019) for a period of analysis. Also, it appears that the data is stationary based on results of running the inflow annual maxima data through the Non-Stationarity Detection Analysis and Trend Analysis in the Time Series Toolbox application developed by the Climate Preparedness and Resilience (CPR) CoP. These are the standard tools used in Qualitative Climate Change analyses to evaluate the stationarity of stream flow data.

Although lake inflow data was used to check for stationarity, the USGS gauged inflows were used for the sedimentation analysis, since this is what was used to develop the flow/load rating curves. However, water use in the watershed has changed over time due to changing demand, which is referred to as depletions. A timeseries of depletion data was obtained from NWK Water Management (see Appendix B) and added to the gauged flows to obtain the FWOP flows. Generally, the depleted flows were less than the gauged flows.

As noted in Appendix D1.3, 95% of all sediment enters the lake while the lake water surface is in the flood control pool. However, deposition in the flood control pool represents 44% of the total mass. From these numbers, the historic trapping efficiency of the flood control pool is 45.4%. The original FWOP computations assumed that this ratio remained constant over the FWOP. However, additional investigations and the HEC-RAS modeling indicated that additional mass should be deposited in the FP as the multipurpose pool (MPP) fills in.

The original FWOP procedure was modified to account for the additional sediment depositing in the FP. Figure 2-1 shows the percentage of total incoming mass that deposits in the FP on the Y-axis and the percent volume lost from the MPP on the X-axis. Both survey data and HEC-RAS sediment model results for Tuttle Creek are included in Figure 2-1. See Appendix D4 for additional details on the HEC-RAS sediment modeling. A polynomial regression equation was fitted to the data, until the MPP has lost 80% of its original volume, after which the % mass to the FP is held constant. Although the final pool volume for Tuttle Creek Lake were derived from the HEC-RAS model results, the modified spreadsheet procedure was also completed to compare to the model results.

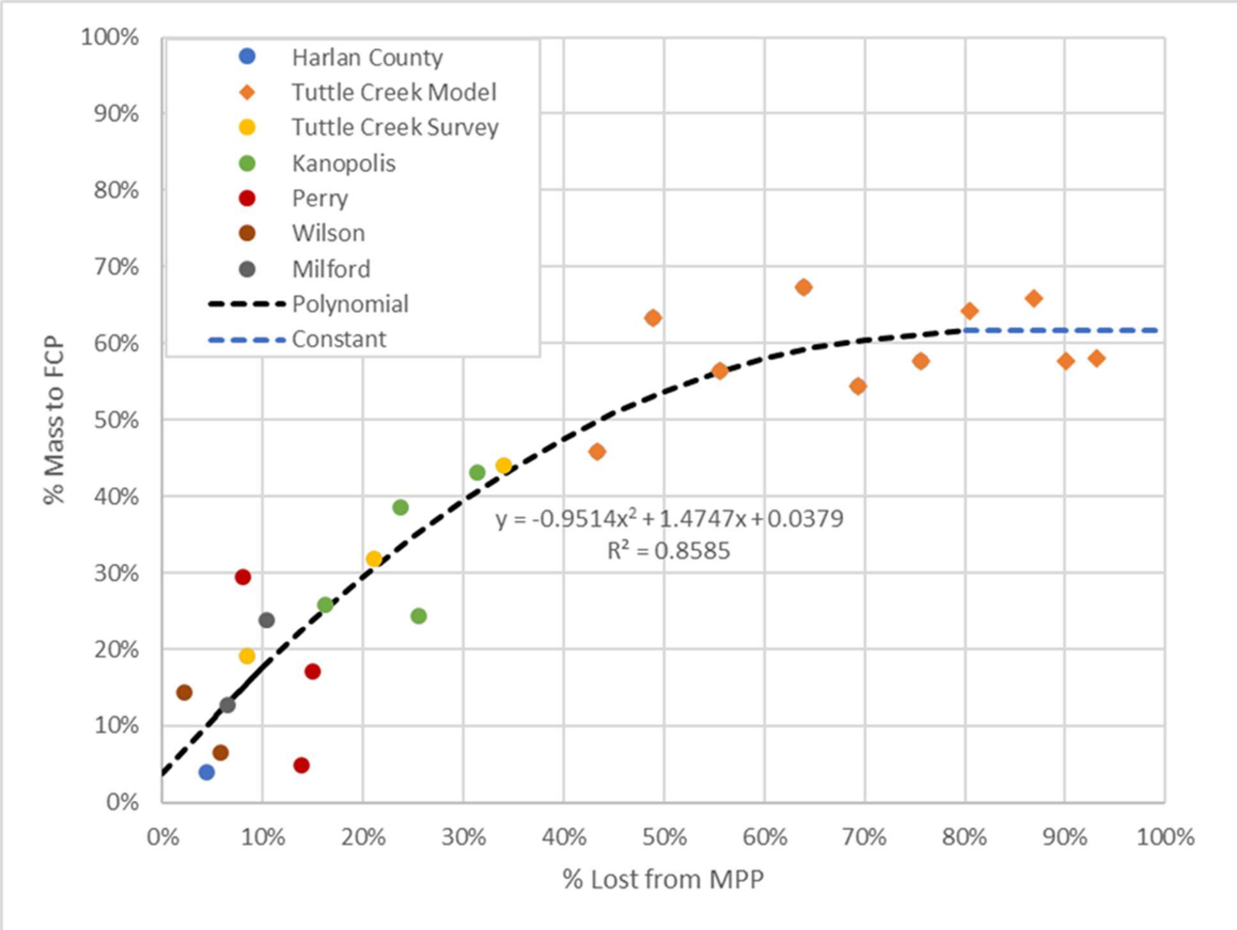


Figure 2-1. The percentage of mass depositing in the FP vs. the percent of volume lost from the MPP

For the multipurpose pool, the trapping efficiency starts at 98%, as measured by USGS (Juracek, 2011). The trapping efficiency decreases over time, using the Brune Curve method.

From the 2009 survey, the multipurpose pool capacity can be estimated at 257,014 ac-ft. The mean annual water inflow into the lake is 1,558,785 acre-feet, based on stream gage data. Brune offers three curves shown in Figure 2-2 for estimating trapping efficiency (Brune, 1953). These can be estimated using Equations 1 and 2 and the constants in Table 2-1.

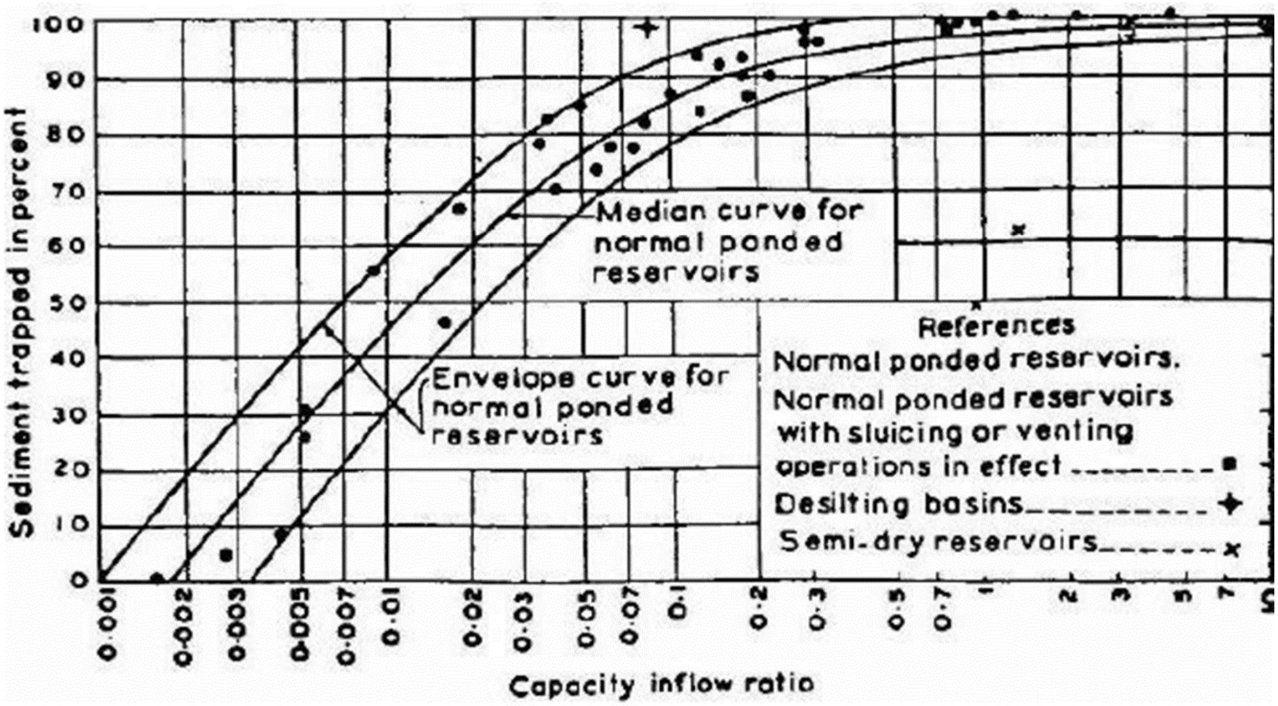


Figure 2-2: Brune Curves, (Brune, 1953)

$$TE = a[1 - 2e^{-bV_*^{0.35}}] \quad (1)$$

$$V_* = \frac{V_{res}}{V_{inflow}} \quad (2)$$

Table 2-1: Constants to be used in Equations 1 and 2

Constant	Low	Medium	High
a	95	97	100
b	5.37	6.42	7.71

The Brune Curve estimate for Tuttle Creek in 2009 is 96.7%, which is 1.3% lower than the USGS estimate. In the FWOP analysis, the trapping efficiency was assumed to start at the USGS value but decrease parallel to the Brune Curve. This was accomplished by adding 1.3% to the Brune Curve estimation at each daily time step. As the lake fills in, the trapping efficiency of the multipurpose pool decreases. Table 2-2 lists the MPP trapping efficiency at year 0, 25, 50, and 100. The mass deposited in the FP and MPP in Table 2-2 were derived from the HEC-RAS model results, while the Brune trapping efficiency was estimated using the projected MPP from the HEC-RAS model.

Table 2-2: Trapping Efficiency over Time

Year	Cumulative Incoming Load (tons)	Deposited in the FCP (tons)	Passed to the MPP (tons)	MPP Brune Final TE (%)
0	78,346,952	43,299,067	35,047,885	89.2%
25	226,095,958	137,103,543	88,992,415	85.8%
50	326,828,843	193,641,428	133,187,415	81.2%
100	594,547,866	360,734,920	233,812,946	57.7%

As the lake shrinks, the trapping efficiency decreases. For the spreadsheet calculations, volume that deposits over time is only subdivided into the MPP or the FCP (not into discreet elevations within each pool). To make this scheme workable but still incorporate shrinking pools over time, two simplifying assumptions were incorporated. (1) The volume of the multi-purpose pool is used to compute the trapping efficiency, independent of whether the pool elevation is above or below the MPP level on any particular day. (2) The mass is apportioned based on the regression equations shown in Figure 2-1.

This list summarizes the steps to compute future sedimentation volumes and remaining pool capacities using the spreadsheet method:

1. For each day, compute the mass of incoming sediment using the calibrated rating curve.
2. Compute the percentage of the mass that deposits in the FCP. The remaining mass is passed to the MPP. As previously discussed, the original FWOP calculations assumed this ratio remained constant.
3. Use the Brune Curve plus 1.3% to compute MPP the trapping efficiency.
4. Apply the trapping efficiency to mass entering the MPP to compute trapped mass.
5. Use the bulk density of the FCP and MPP to covert the mass trapped into a volume trapped.
6. Subtract the volume of deposition in the FCP and MPP from the remaining capacities.
7. Repeat for each subsequent day, re-computing trapping efficiency with the progressively smaller MPP capacity.

As previously discussed, the original spreadsheet method assumed a constant percentage of the sediment that would deposit in the FP would remain constant over time. However, this assumption led to an overestimation of deposition within the MPP and an overestimation within the FP, based on the HEC-RAS model results. Figure 2-3 provides the remaining storage capacities of the multipurpose and flood control pools from the HEC-RAS model, the original spreadsheet method, and the modified spreadsheet method. As can be seen in Figure 2-3, the modified spreadsheet method agress relatively well with the HEC-RAS model results. Table 2-3 and Table 2-4 summarize the results.

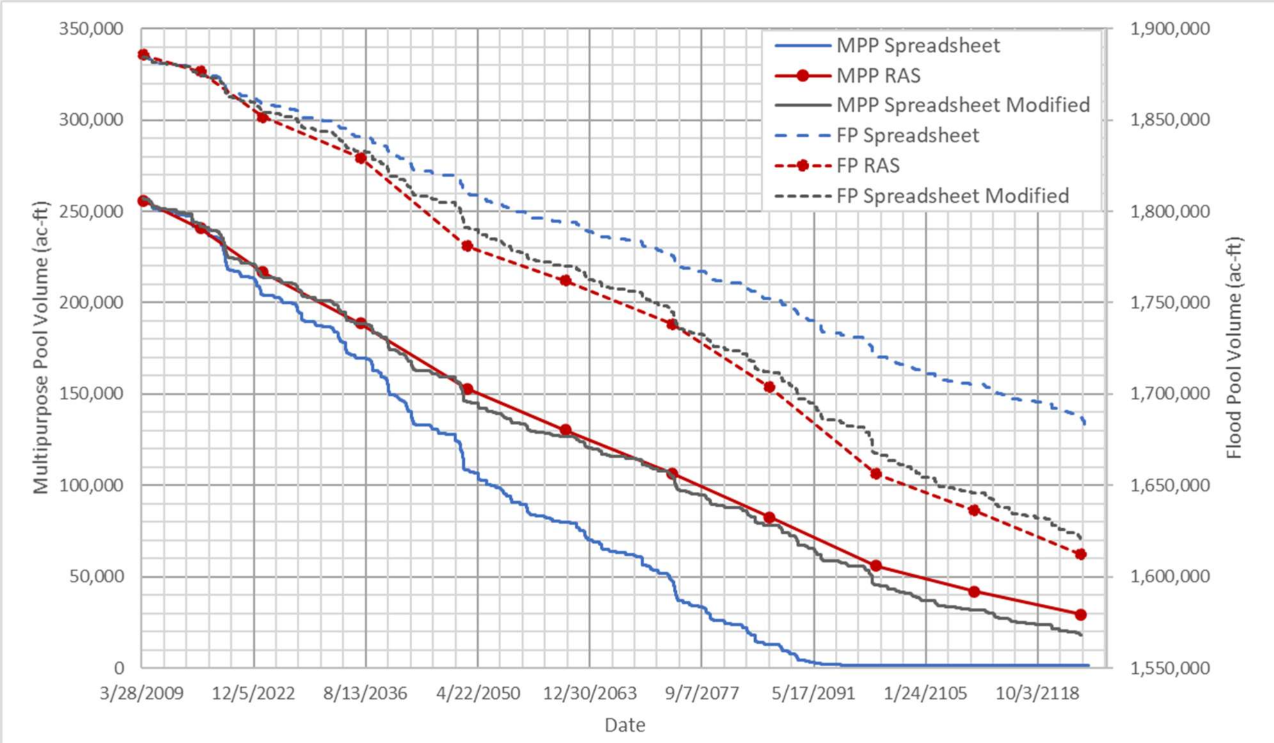


Figure 2-3: Remaining Pool Volumes Over Time

Table 2-3: Summary of Remaining Pool Volumes from HEC-RAS Model Results

Year	Multi-Purpose Pool (ac-ft)	Multi-Purpose Pool % of Original Volume	Flood Control Pool(ac-ft)	Flood Control Pool % of Original Volume
1962	424,312	100.0%	1,942,705	100.0%
2024	216,570	51.0%	1,851,668	95.3%
2049	153,046	36.1%	1,780,947	91.7%
2074	106,499	25.1%	1,738,322	89.5%
2124	29,247	6.9%	1,612,347	83.0%

Table 2-4: Sediment Deposition Volumes from the HEC-RAS Model Results

Year	2009	2024	2049	2074	2124
Cumulative Deposition Multi-Purpose Pool (ac-ft)	167,298	207,742	271,266	317,813	395,065
Average Annual MPP Deposition Since 2024 (ac-ft/yr)	3,351	2,773	2,541	1,862	1,545
Average Annual MPP Deposition over Increment (ac-ft/yr)	-	-	2,541	2,201	1,873
Cumulative Deposition Flood Control Pool (ac-ft)	58,393	91,037	161,758	204,383	330,358
Average Annual FCP Deposition Since 2024 (ac-ft/yr)	1,170	2,238	2,829	1,705	2,520
Average Annual FCP Deposition over Increment (ac-ft/yr)	-	-	2,829	2,267	2,393

FWOP elevation-storage and elevation-area curves were estimated for use in the Water Management analyses documented in Appendix B and were obtained from the HEC-RAS model results discussed in Appendix D4. See Appendix B for the final elevation-storage and elevation-area curves.

3.0 DELTA PROGRESSION

As listed in Appendix D1.3, the delta progression rate towards the dam from 1983 to 2009 was 0.11 miles per year. As deltas progress into wider and deeper sections of the lake, the rate of progression is expected to slow. The future rate was estimated by finding the delta progression that would yield the correct volume of sediment deposition as listed in Table 2-4. This delta progression analysis provides approximate results based on applying MPP deposition as an even veneer of deposition over all areas under the multipurpose pool elevation. This was accomplished using ArcGIS to add the elevation change, determined by dividing projected volume change by lake surface area, to the 2009 bathymetric surface. Multiple iterations were needed since areas of the lake would rise above the MPP, and the volume above the MPP had to be added again.

The 25, 50, and 100 year projections indicate delta progression of 1.8, 3.1, and 7.1 miles. This equates to a loss in multipurpose pool surface area, as listed in Table 3-1.

Table 3-1: Multipurpose Pool Surface Area

Year	MPP Surface Area (ac)	Lost Surface Area compared to Original (ac)	Lost Surface Area Compared to 2024 (ac)
1962	15,830	-	-
2009	10,900	4,930	-
2024	9,415	6,415	-
2049	7,493	8,337	1,922
2074	6,386	9,444	3,029
2124	3,237	12,593	6,178

Figure 3-1 shows the historic and projected elevation contours of the MPP. The 1957 MPP contour was created from a 1957 DEM produced by the KBS from historic topographic maps.

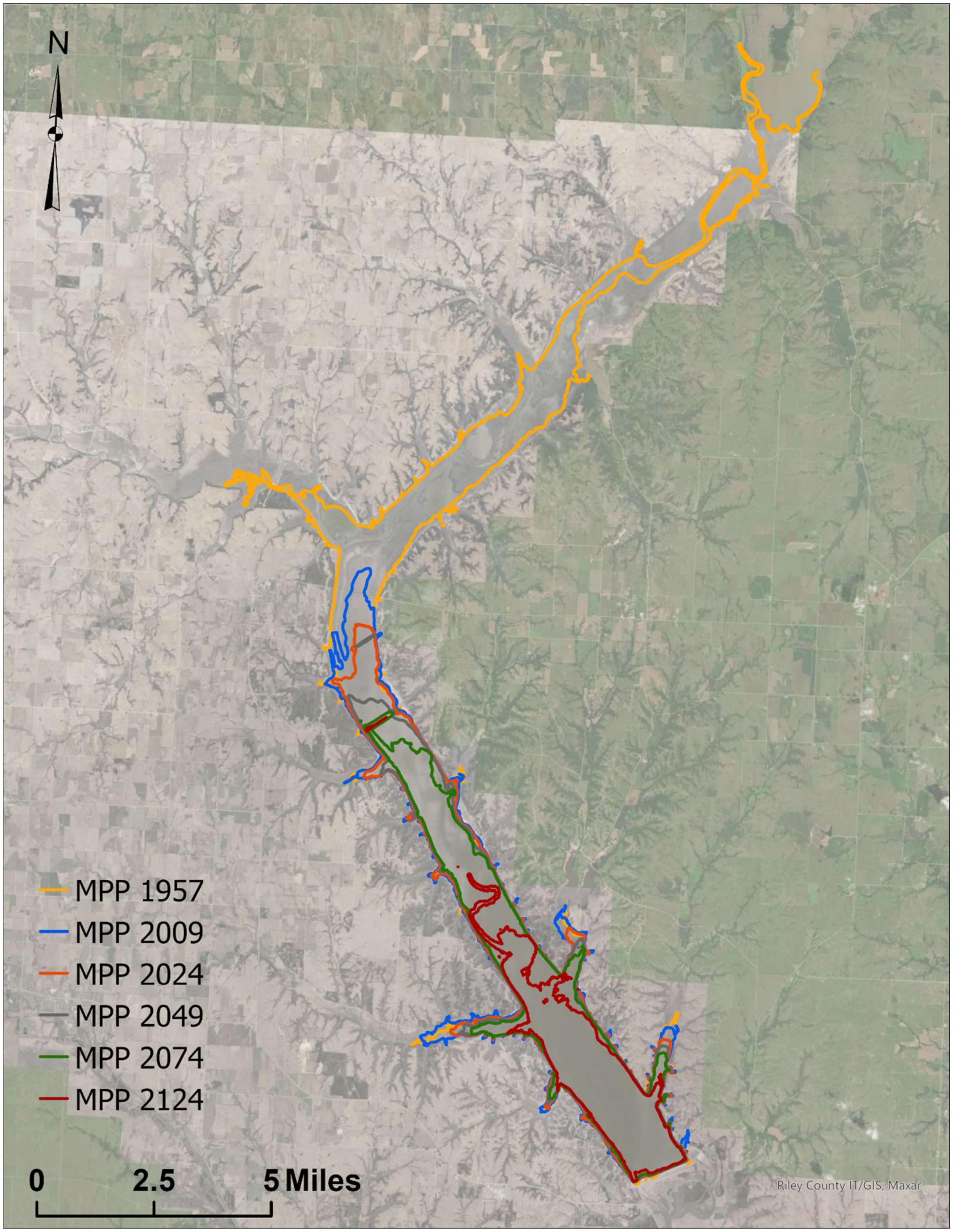


Figure 3-1: MPP Elevation Contours and depth below MPP

Figure 3-2 depicts the historic, current, and projected delta locations. The effect this deposition will have on recreational infrastructure will be discussed in Appendix G.

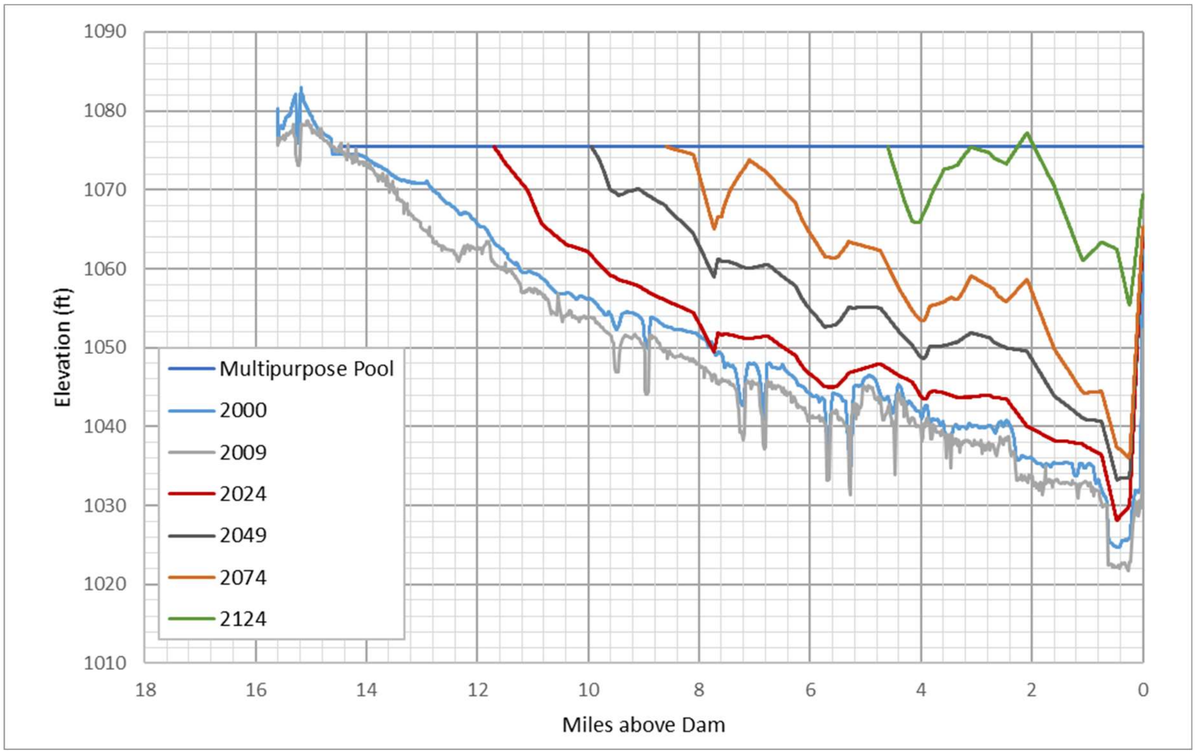


Figure 3-2: Projected Delta Location. Include lines for locations of important features.

4.0 DOWNSTREAM SEDIMENT CONCENTRATIONS

As the multipurpose trapping efficiency of the Tuttle Creek Lake decreases from 95% to 80% as explained in Section 2, more sediment will be supplied to the river downstream. Most of this additional sediment will be fine sediment since it is assumed that most of the coarse sediment (sands and gravels) will continue to deposit in the FCP. It was determined in Appendix D1.3 that 95% of the sediment enters the lake during flood control operations.

As most of the coarse sediment will continue to be trapped, this will continue to induce degradation and bank erosion, as indicated in the next section. The fine sediment loading to the river, however, will increase as the lake trapping efficiency decreases from 95% to 80%. Table 4-1 indicates how the sediment concentration of the water released from Tuttle Creek Dam increases over time. These are approximate estimates obtained by comparing the mass trapped by the dam, estimated using the Brune curve, to the total sediment inflow. The increase in sediment concentration will vary with flow rate and other factors, which would require more detailed modeling in order to predict.

Table 4-1: Increase in the sediment concentration of releases from Tuttle Creek Dam

Year	MPP Trapping Efficiency	MPP Passing Percentage	% of 2024 Concentration
2024	94.94%	5.1%	100%
2049	94.02%	6.0%	118%
2074	92.63%	7.4%	146%
2124	79.52%	20.5%	405%

5.0 DOWNSTREAM CHANNEL

The existing conditions report demonstrated bed degradation and bank erosion downstream from Tuttle Creek Lake. Over the next 100 years, an additional 62,026,182 tons of sand will be trapped in Tuttle Creek Lake which will further induce bed degradation and bank erosion on the Big Blue River. Geomorphic principles suggest that rivers adjust to transport the bed material load supplied to them. Thus, the downstream channel will degrade (which lowers the slope and hence the applied shear stress) and an amour layer will form (which increases the critical shear required to initiate motion) until the flow from the dam is no longer able to transport sediment. Sedimentation modeling beyond the scope of this watershed study is required to compute the final bed elevations and to estimate bank erosion at the end of 25, 50, and 100 years. While applying trendlines to degradation or bank erosion is not ideal, it can provide a first approximation when detailed modeling is not practical. Herein is provided an order-of-magnitude approximation based on assumptions that geomorphic feedbacks do not change the rates of geomorphic change. This approximation was computed via the following steps:

1. Compute the ratio of sand trapped in the lake (tons) to volumetric degradation downstream (CY) over a defined distance. For Tuttle Creek Lake, 28,235,184 CY of sand trapped by the dam induced 8,558,733 CY of bed and bank erosion in the 9.24 miles downstream of the dam, or a ratio of 0.3 CY downstream degradation/tons trapped.
2. Apply the ratio to the future estimates for trapped sand.

Table 5-1 provides the estimated degradation volume in the 9.24 miles below Tuttle Creek Dam at the end of 0, 25, 50, and 100 years. This analysis provides a high-level assessment suitable for relative ranking of alternatives.

Table 5-1: Degradation Downstream of the Tuttle Creek Dam Compared to latest degradation survey in May 2019

Year	Trapped Sand (CY)	Degradation Since Last Survey (ac-ft)	Degradation Since 2024 (ac-ft)
0	3,507,261	659	-
25	17,817,656	3,348	2,689
50	27,270,886	5,124	4,465
100	52,910,831	9,941	9,282

6.0 SUMMARY, LIMITATIONS, AND CONCLUSIONS

The following list summarizes the 50-year FWOP condition at Tuttle Creek Lake. (For the 25 years and 100 years projections, see the body of the report.)

- 218,646,659 tons of fine sediment (silt/clay) and 29,835,231 tons of coarse sediment (sand/gravel) are expected to enter Tuttle Creek Lake, of which 150,342,361 tons are trapped in the flood control pool, 89,660,755 tons are trapped in the multi-purpose pool, and 8,478,775 tons pass downstream.
- The flood control pool (FCP) and multi-purpose pool (MPP) shrinks by an additional 113,346 ac-ft and 110,071 ac-ft, respectively.
- At the end of 50 years, the MPP has 25.1% remaining of its original capacity.
- At the end of 50 years, the FCP has 89.5% remaining of its original capacity.
- At the end of 50 years, the coarse sediment delta progresses downstream an additional 3.1 miles, which shrinks the surface area of the lake to 40% of the original.
- At the end of 50 years, the fine sediment concentrations of releases from Tuttle Creek Dam will rise to 405% of existing but will still be less than the incoming sediment concentrations.
- The downstream channel will experience additional bed degradation and bank erosion, with 4,465 ac-ft degrading from the lower 9.2 miles.

These findings are based on trendline projections and empirical equations. For more precise estimations, particularly of where in the pools the sediment will deposit and how the downstream channels will adjust over time, numerical modeling is recommended. Moreover, this analysis did not compute damages due to upstream delta migration and only approximated downstream degradation as a total volume, not as bank erosion or bed degradation at specific affected infrastructure.

Notwithstanding the limitations, these projections indicate that sediment accumulation in Tuttle Creek Lake will be a serious problem over the next 50 years with serious implications for loss in benefits, environmental harm, infrastructure damage, and increased O&M needs (these impacts will be addressed in separate appendices). From years 50 to 100, sediment trapping will continue, but will slow within the multipurpose pool. Deposition will continue within the flood pool throughout the 100 years.

US Army Corps of Engineers ®
Kansas City District

Kansas River Reservoirs Flood and Sediment Study

Appendix D2.4: Milford Lake Future Without Project Sedimentation

November 2022

Contents

1.0 INTRODUCTION

This appendix documents the projections for the Future Without Project (FWOP) condition for Milford Lake. The FWOP condition is the expected conditions of the lake if sediment accumulation continues unabated, i.e., without intentional sediment management or removal. This analysis also assumes no pool raise or reallocation takes place, as such actions would be potential measures for a Future With Project condition. This analysis incorporates the trends documented in the Existing Conditions analysis for Milford Lake, Appendix D1.4. Figure 1-1 shows the lake with respect to the Kansas River Basin.

FWOP computations were first completed in December 2020 but were later updated in May 2022 based a new methodology incorporating sediment modeling results from Tuttle Creek Lake. The new methodology included the effects of additional sediment depositing in the flood pool (FP) over time.

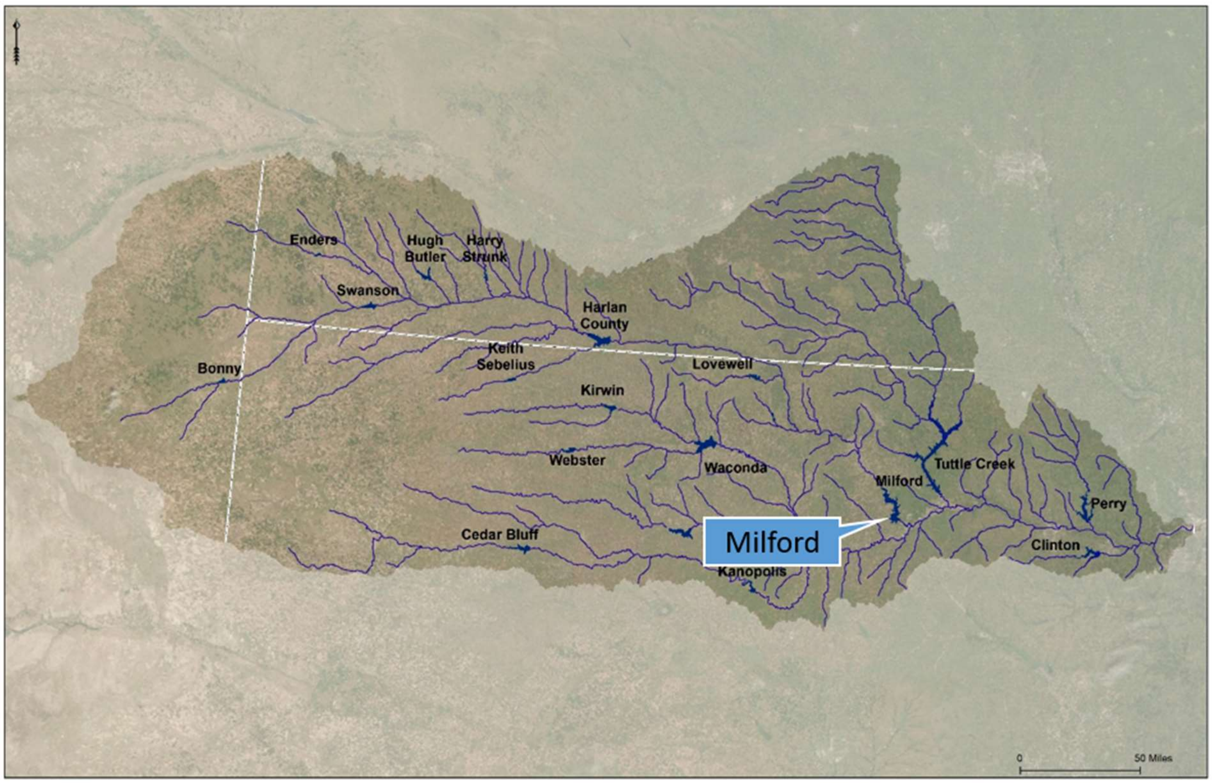


Figure 1-1: Overall Kansas River Basin Map and Milford Lake

2.0 FWOP SEDIMENT DEPOSITION

Long-term sediment deposition was computed by running 100 years of daily flow through the calibrated rating curves described in Appendix D1.4. The flows were obtained for the U.S. Geological Survey (USGS) gage upstream of the lake used in the calibration, repeating the 50-year period from 1970 to 2019 to obtain a total of 100-years. The beginning date for the simulation was 01 January 2024, the assumed date this project will be completed. To obtain total deposition since construction of the lake, calculations were made from the date of the last bathymetric survey in 2009 to the beginning of 2124.

The lake inflow data, which is calculated by NWK Water Management Section, was used to check for stationarity and climate change. This was done by determining how well the flow frequency statistics based on the previous 50-years of data match with the entire available period of record for the lakes. A Bulletin 17 analysis was performed on the daily lake inflow data (annual maxima) and the computed flow frequency curves were compared. The curves matched closely enough to use the last 50 years of data (1970-2019) for a period of analysis. Also, it appears that the data is stationary based on results of running the inflow annual maxima data through the Non-Stationarity Detection Analysis and Trend Analysis in the Time Series Toolbox application developed by the Climate Preparedness and Resilience (CPR) CoP. These are the standard tools used in Qualitative Climate Change analyses to evaluate the stationarity of stream flow data.

Although lake inflow data was used to check for stationarity, the USGS gauged inflows were used for the sedimentation analysis, since this is what was used to develop the flow/load rating curves. However, water use in the watershed has changed over time due to changing demand, which is referred to as depletions. A timeseries of depletion data was obtained from NWK Water Management (see Appendix B) and added to the gauged flows to obtain the FWOP flows. Generally, the depleted flows were less than the gauged flows.

As noted in the Existing Conditions Appendix, 90.2% of all sediment enters the lake while the lake water surface is in the flood control pool. However, deposition in the flood control pool represents 16.8% of the total mass. From these numbers, the historic trapping efficiency of the flood control pool is 17.9%. The original FWOP computations assumed that this ratio remained constant over the FWOP. However, additional investigations indicated that additional mass should be deposited in the FP as the multipurpose (MPP) fills in.

For the final FWOP calculations, the percentage of the incoming mass that is deposited in the FP was varied over time based on a regression equation developed from lake surveys and numerical modeling at Tuttle Creek Lake. Figure 2-1 shows the percentage of total incoming mass that deposits in the FP on the Y-axis and the percent volume lost from the MPP on the X-axis. Both survey data and HEC-RAS sediment model results for Tuttle Creek are included in Figure 2-1. See Appendix D4 for additional details on the HEC-RAS sediment modeling. A polynomial regression equation was fitted to the data, until the MPP has lost 80% of its original volume, after which the % mass to the FP is held constant.

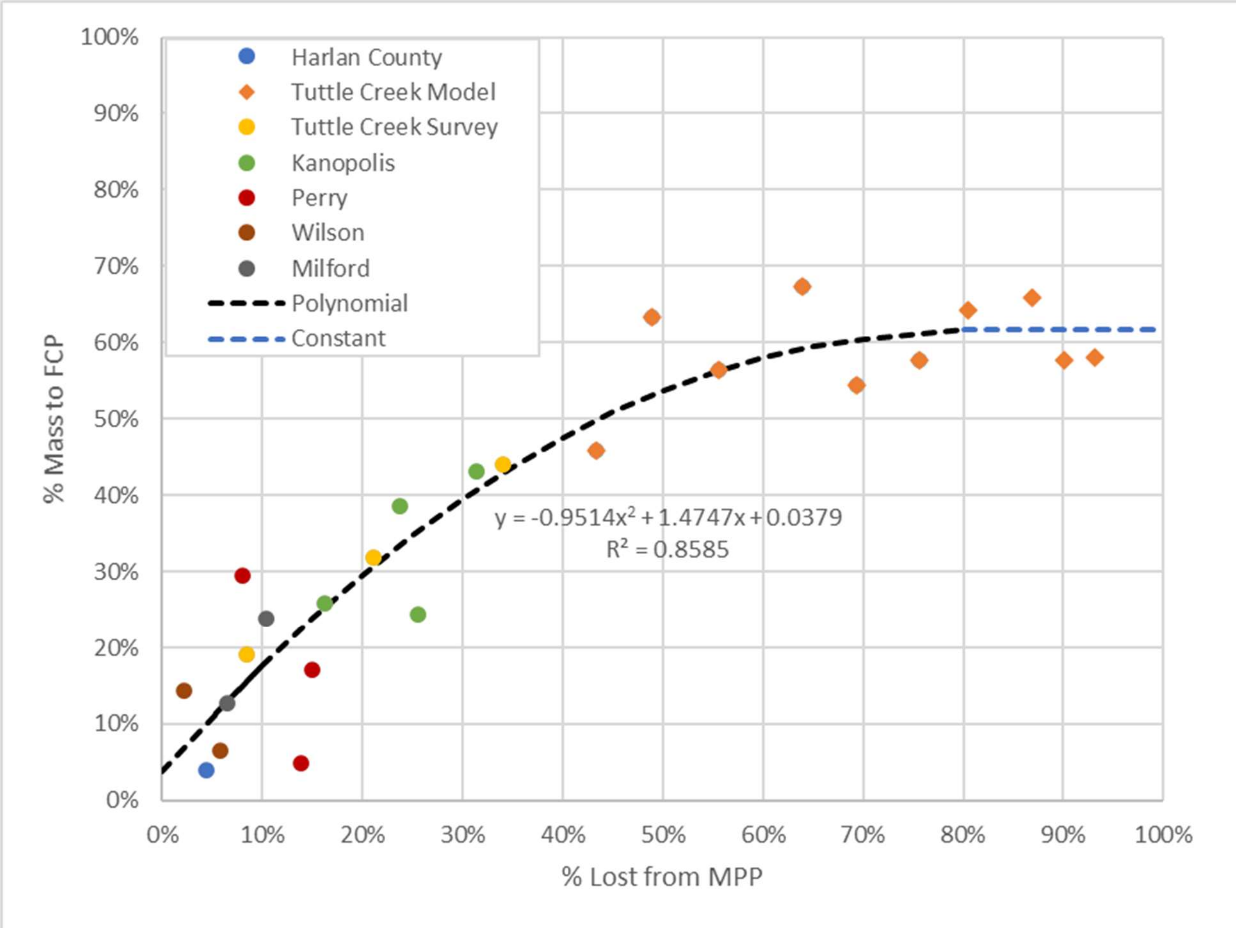


Figure 2-1. The percentage of mass depositing in the FP vs. the percent of volume lost from the MPP

From the 2009 survey, the multipurpose pool capacity can be estimated at 373,152 ac-ft. The mean annual water inflow into the lake is 564,138 acre-feet, based on stream gage data. Brune offers three curves shown in Figure 2-2 for estimating trapping efficiency (Brune, 1953). These can be estimated using Equations 1 and 2 and the constants in Table 2-1. The medium curve was used for this analysis.

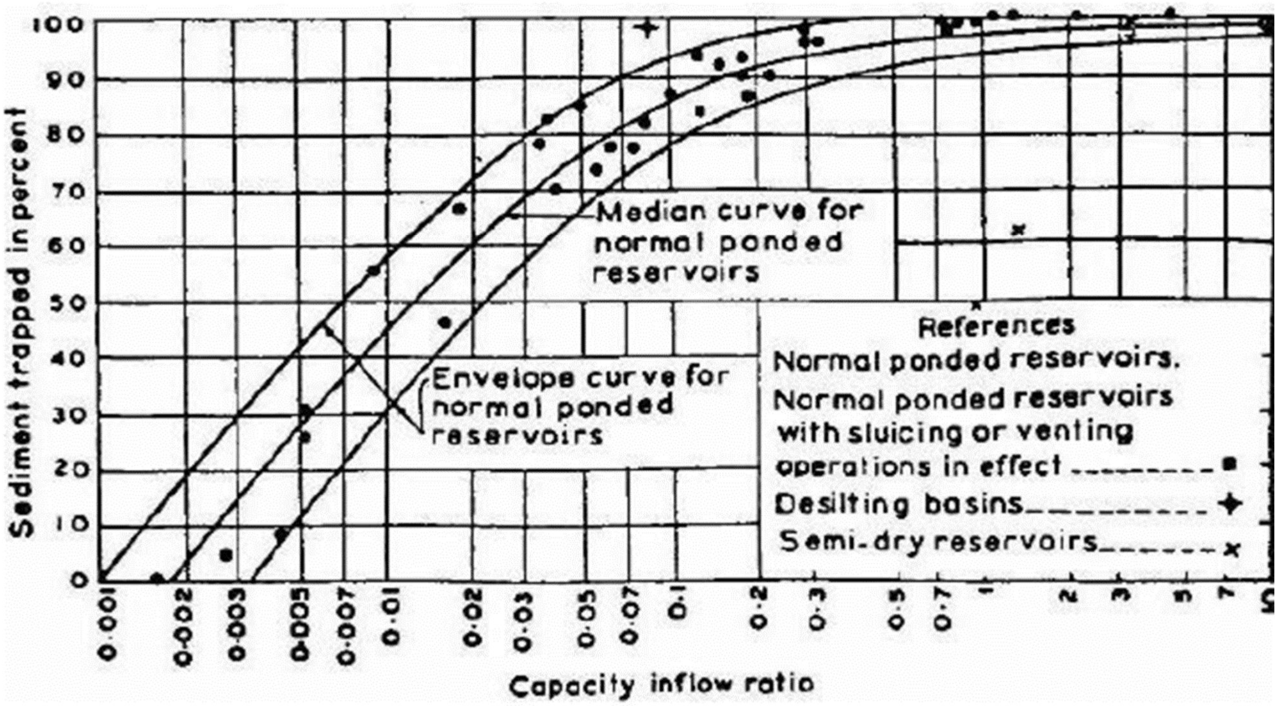


Figure 2-2: Brune Curves, (Brune, 1953)

$$TE = a[1 - 2e^{-bV_*^{0.35}}] \quad (1)$$

$$V_* = \frac{V_{res}}{V_{inflow}} \quad (2)$$

Table 2-1: Constants to be used in Equations 1 and 2

Constant	Low	Medium	High
a	95	97	100
b	5.37	6.42	7.71

The Brune Curve estimate for Milford Lake in 2009 is 96.25% and as the lake fills in, the trapping efficiency of the multipurpose pool decreases. Table 2-2 lists the MPP trapping efficiency at year 0, 25, 50, and 100. Because the capacity of the multipurpose pool is so large compared to the sediment deposition, trapping efficiency does not decrease significantly.

Table 2-2: Trapping Efficiency over Time

Year	Cumulative Incoming Load (tons)	Deposited in the FCP (tons)	Passed to the MPP (tons)	MPP Final TE (%)
0	8,243,412	1,544,415	6,698,997	96.22%
25	21,869,198	4,453,058	17,416,140	96.18%
50	29,666,616	6,306,318	23,360,298	96.15%
100	52,940,940	12,589,365	40,351,575	96.07%

As the lake shrinks, the trapping efficiency decreases. The volume that deposits over time is only subdivided into the MPP or the FCP (not into discreet elevations within each pool). To make this scheme workable but still incorporate shrinking pools over time, two simplifying assumptions were incorporated. (1) The volume of the multi-purpose pool is used to compute the trapping efficiency, independent of

whether the pool elevation is above or below the MPP level on any particular day. (2) The mass is apportioned based on the ratios given in Figure 2-1

This list summarizes the steps to compute future sedimentation volumes and remaining pool capacities:

1. For each day, compute the mass of incoming sediment using the calibrated rating curve.
2. Compute the percentage of the mass that deposits in the FCP. The remaining mass is passed to the MPP. As previously discussed, the original FWOP calculations assumed this ratio remained constant.
3. Use the Brune Curve to compute MPP the trapping efficiency.
4. Apply the trapping efficiency to mass entering the MPP to compute trapped mass.
5. Use the bulk density of the FCP and MPP to covert the mass trapped into a volume trapped.
6. Subtract the volume of deposition in the FCP and MPP from the remaining capacities.
7. Repeat for each subsequent day, re-computing trapping efficiency with the progressively smaller MPP capacity.

Figure 2-3 provides the remaining storage capacities of the multipurpose and flood control pools for both the original and final FWOP computations. Table 2-3 and Table 2-4 summarize the final results. Because the survey methodology is different for the 1962 and 2009 surveys, the change in capacity between the surveys is likely underpredicted as discussed in Appendix D1.4.

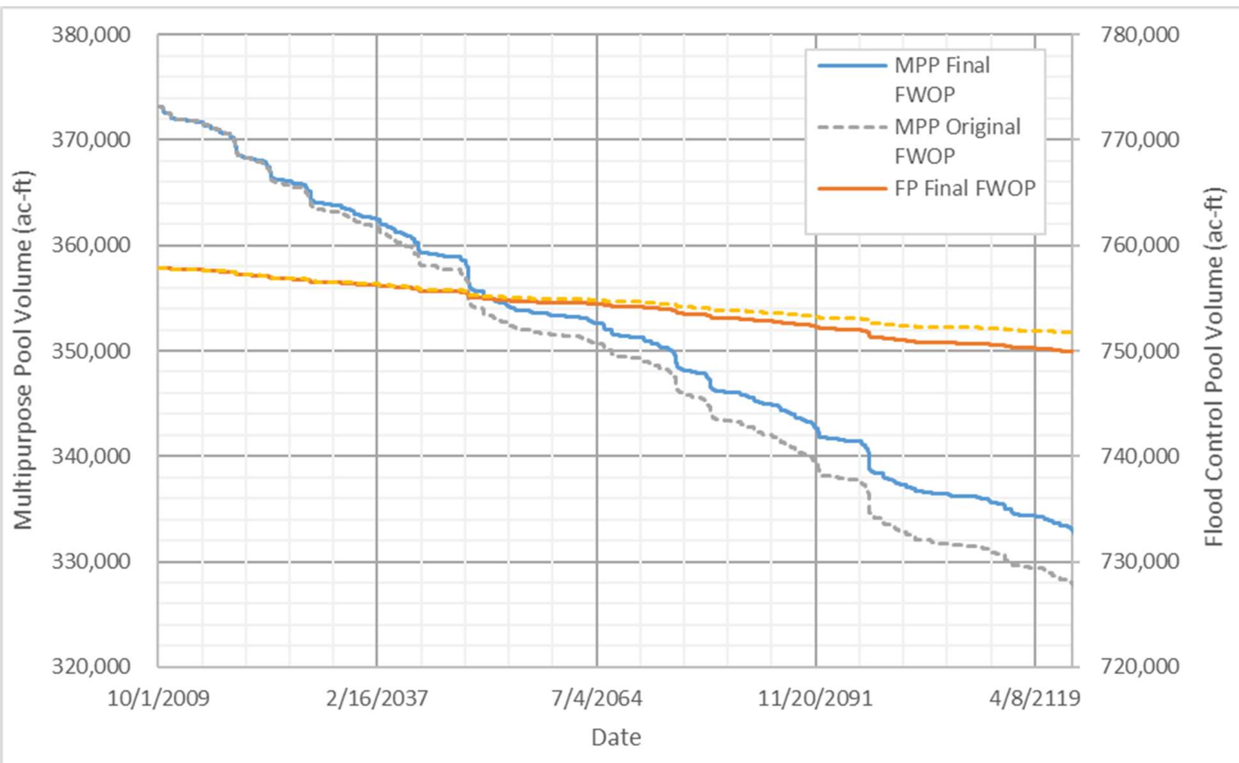


Figure 2-3: Remaining Pool Volumes Over Time

As explained in the Existing Conditions report, the flow/sediment relationship has dramatically decreased in recent years over previous levels. This equates to less deposition in the future than would otherwise occur.

Table 2-3: Summary of Remaining Pool Volumes

Year	Multi-Purpose Pool (ac-ft)	Multi-Purpose Pool % of Original Volume	Flood Control Pool Volume, (ac-ft)	Flood Control Pool % of Original Volume
1962	415,352	100.0%	757,746	100.0%
2009	373,152	89.8%	757,872	100.0%
2024	366,476	88.2%	756,892	99.9%
2049	355,800	85.7%	755,048	99.6%
2074	349,881	84.2%	753,872	99.5%
2124	332,971	80.2%	749,887	99.0%

Table 2-4: Sediment Deposition Volumes since 2009

Year	2024	2049	2074	2124
Cumulative Deposition Multi-Purpose Pool (ac-ft)	6,676	17,352	23,271	40,181
Average Annual MPP Deposition Since 2024 (ac-ft/yr)	-	427	332	335
Average Annual MPP Deposition over Increment (ac-ft/yr)	468	427	237	338
Cumulative Deposition Flood Control Pool (ac-ft)	980	2,824	4,000	7,985
Average Annual FCP Deposition Since 2024 (ac-ft/yr)	-	74	121	280
Average Annual FCP Deposition over Increment (ac-ft/yr)	69	74	47	80

Table 2-5 gives the storage volumes from the original iteration of the FWOP computations along with the difference from the final results. As seen in Table 2-5, the MPP has greater storage over the FWOP for the final results, while the FP has less storage.

Table 2-5. Pool volumes in acre-feet from the original FWOP computations, and the differences from the final computations

Year	Multi-Purpose Pool Original	Flood Control Pool Original	Difference Multipurpose Pool	Difference Flood Control Pool
2024	366,220	756,929	+257	-36
2049	354,235	755,298	+1,566	-250
2074	347,772	754,418	+2,110	-546
2124	327,847	751,703	+5,124	-1,816

FWOP elevation-storage and elevation-area curves were estimated for use in the Water Management analyses documented in Appendix B. For the MPP, the curves were estimated using the surface volume tool in Arc-GIS along with the digital elevation models (DEMs) discussed in Section 3. However, for the FP portion of the lake, the curves were estimated based on the daily pool elevation from the FWOP HEC-ResSIM modeling (see Appendix B). In this analysis, the FP was broken up into 5-ft increments and for each day in the FWOP, the projected deposition was evenly distributed across all increments at or below the daily pool elevation. The reduction in surface area over that increment was then estimated by assuming it had the same percent reduction as the storage volume. However, because different methods were used to estimate the MPP and FP curves, a discontinuity was observed in the surface area between them. This was resolved replacing the surface area for first point in the FP with the value obtained by interpolating between the final MPP point and the second FP point. This methodology compared well with the 1D HEC-RAS sediment modeling results discussed in Appendix D4. See Appendix B for the final elevation-storage and elevation-area curves.

3.0 DELTA PROGRESSION

As listed in Appendix D1.4, the delta progression rate towards the dam from 1980 to 2009 was 228 feet per year. As deltas progress into wider and deeper sections of the lake, the rate of progression is expected to slow. The future rate was estimated by finding the delta progression that would yield the correct volume of sediment deposition as listed in Table 2-4. This delta progression analysis provides approximate results based on applying MPP deposition as an even veneer of deposition over all areas under the multipurpose pool elevation. Projected digital elevation models (DEM), surface area, and elevation contours of the MPP were computed using the following steps in ArcGIS.

1. Divide the projected deposition volume by the surface area of the MPP in 2009 to obtain the elevation change within the MPP.
2. Add this elevation to the 2009 DEM of the MPP to obtain a first estimate of the projected DEM.
3. Use the surface volume tool to estimate the new MPP volume and surface area.
4. Determine the difference between the predicted MPP volume and the MPP from the modified DEM
5. Divide the volume difference by the new surface area to obtain an additional elevation increase.
6. Add the additional elevation increase to the modified DEM
7. Repeat steps 3-6 until the extra volume above the MPP is less than the average annual deposition.

The 25, 50, and 100 year projections indicate delta progression of 452, 739, and 2,243 ft. This equates to a loss in multipurpose pool surface area, as listed in Table 3-1.

Table 3-1: Multipurpose Pool Surface Area

Year	MPP Surface Area (ac)	Lost Surface Area compared to Original (ac)	Lost Surface Area Compared to 2024 (ac)
1967	16,617	-	-
2009	15,498	1,119	-
2024	15,270	1,347	-
2049	14,918	1,699	352
2074	14,727	1,890	543
2124	14,211	2,406	1,059

Figure 3-1 shows the elevation contour of the MPP for various years. The 1967 MPP contour was digitized from a historic map of the lake.

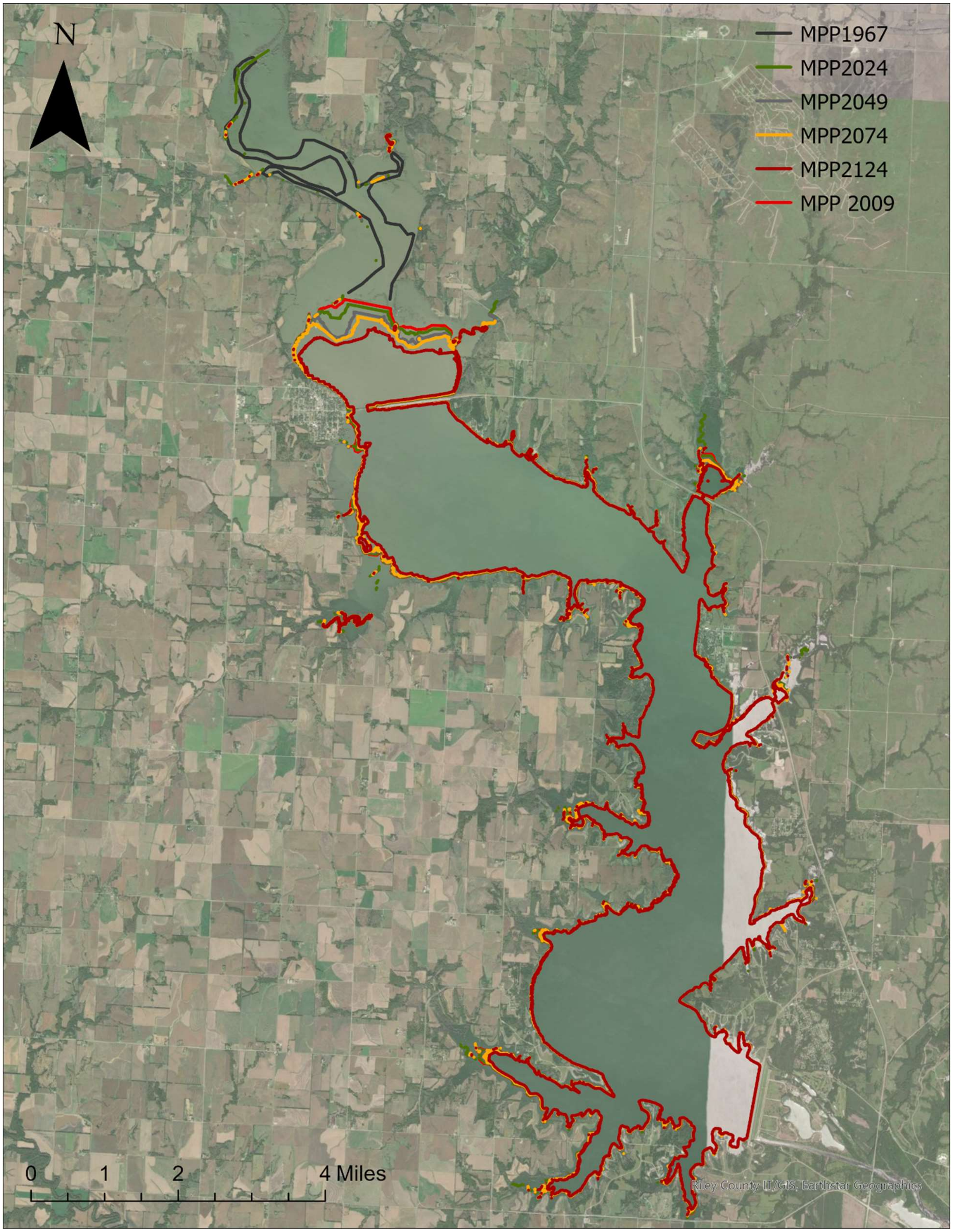


Figure 3-1: Milford MPP elevation contour (1144.83 NAVD 88)

Figure 3-2 gives the bottom elevations along a centerline of the lake. The projected profiles were extracted from the projected DEMs.

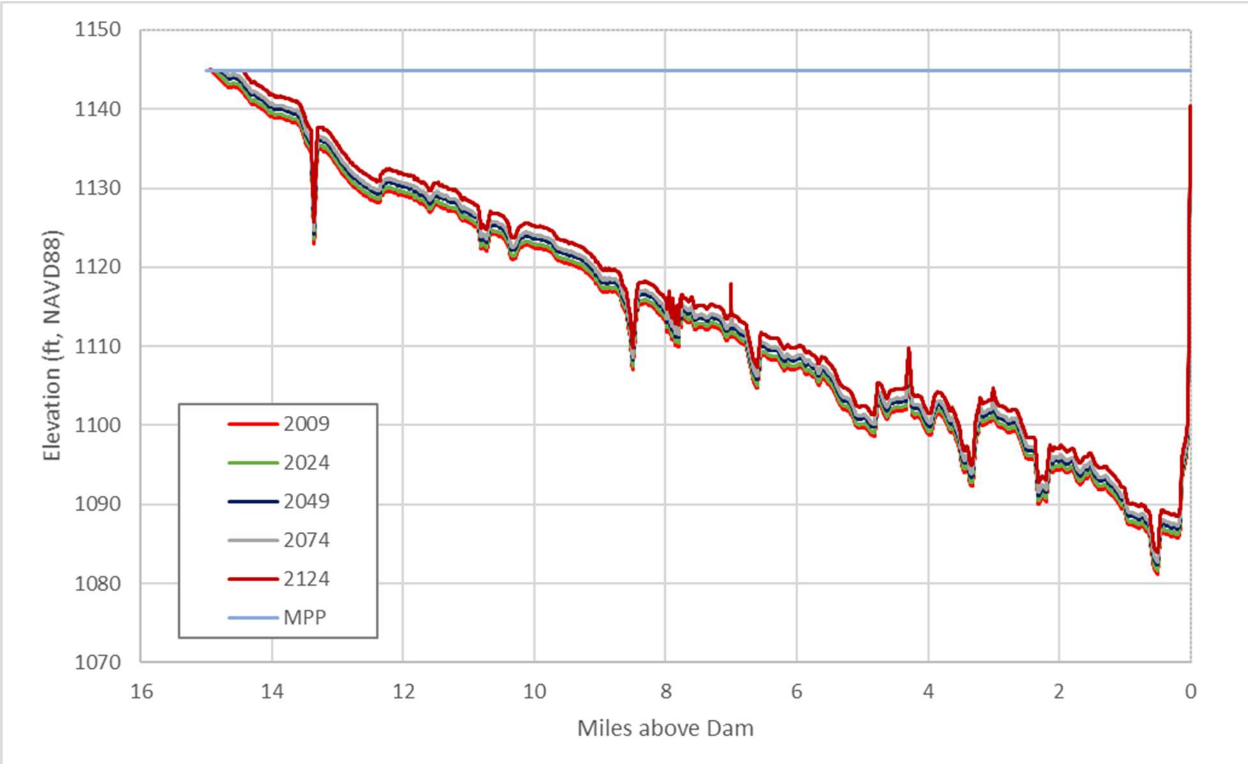


Figure 3-2: Projected Delta Location. Include lines for locations of important features.

4.0 DOWNSTREAM SEDIMENT CONCENTRATIONS

The downstream sediment concentrations are not expected to change significantly over the FWOP. Although the MPP trapping efficiency declines slightly (Table 2-2), the FP trapping efficiency is expected to increase over time (Figure 2-1), which will offset the declining MPP trapping efficiency.

5.0 DOWNSTREAM CHANNEL

The existing conditions report demonstrated bed degradation and bank erosion downstream from Milford Lake. Over the next 100 years, an additional 12,833,439 tons of sand will be trapped in Milford Lake which will further induce bed degradation and bank erosion on the Republican River. Geomorphic principles suggest that rivers adjust to transport the bed material load supplied to them. Thus, the downstream channel will degrade (which lowers the slope and hence the applied shear stress) and an amour layer will form (which increases the critical shear required to initiate motion) until the flow from the dam is no longer able to transport sediment. Sedimentation modeling beyond the scope of this watershed study is required to compute the final bed elevations and to estimate bank erosion at the end of 25, 50, and 100 years. While applying trendlines to degradation or bank erosion is not ideal, it can provide a first approximation. Herein is provided an order-of-magnitude approximation based on assumptions that geomorphic feedbacks do not change the rates of geomorphic change. This approximation was computed via the following steps:

1. Compute the ratio of sand trapped in the lake (tons) to volumetric degradation downstream (CY) over a defined distance. For Milford Lake, 7,533,087 tons of sand trapped by the dam induced 2,652,481 CY of bed and bank erosion in the 6.3 miles downstream of the dam, or a ratio of 0.35 CY downstream degradation/tons trapped.
2. Apply the ratio to the future estimates for trapped sand.

Table 5-1 provides the estimated degradation volume in the 6.3 miles below Milford Dam at the end of 0, 25, 50, and 100 years. This analysis provides a high-level assessment suitable for relative ranking of alternatives.

Table 5-1: Degradation Downstream of the Milford Dam Compared to latest degradation survey in July 2016

Year	Trapped Sand (CY)	Degradation Since Last Survey (ac-ft)	Degradation since 2024 (act)
2024	1,305,913	285	-
2049	4,047,211	883	598
2074	5,439,147	1,187	902
2124	9,926,225	2,166	1,881

6.0 SUMMARY, LIMITATIONS, AND CONCLUSIONS

The following list summarizes the 50-year FWOP condition at Milford Lake. (For the 25 years and 100 years projections, see the body of the report.) The list summarizes the main points:

- 16,233,928 tons of fine sediment (silt/clay) and 5,189,275 tons of coarse sediment (sand/gravel) are expected to enter Milford Lake, of which 4,761,903 tons are trapped in the flood control pool, 16,026,140 tons are trapped in the multi-purpose pool, and 635,161 tons pass downstream.
- The flood control pool (FCP) and multi-purpose pool (MPP) shrinks by an additional 3,020 ac-ft and 16,595 ac-ft, respectively.
- At the end of 50 years, the MPP has 84.2% remaining of its original capacity.
- At the end of 50 years, the FCP has 99.5% remaining of its original capacity.
- At the end of 50 years, the coarse sediment delta progresses downstream an additional 739 feet, which shrinks the surface area of the lake to 89% of the original.
- The fine sediment concentrations of releases from Milford Dam will increase by 4% from existing.
- The downstream channel will experience additional bed degradation and bank erosion, with 902 ac-ft degrading from the lower 6.3 miles.

These findings are based on trendline projections and empirical equations. For more precise estimations, particularly of where in the pools the sediment will deposit and how the downstream channels will adjust over time, numerical modeling is recommended. Moreover, this analysis did not compute damages due to upstream delta migration and only approximated downstream degradation as a total volume, not as bank erosion or bed degradation at specific affected infrastructure.

Notwithstanding the limitations, these projections indicate that sediment accumulation in Milford Lake will be a slight problem over the next 50 years with some implications for loss in benefits, environmental harm, infrastructure damage, and increased O&M needs (quantified in other appendices). From years 50 to 100, sediment trapping and O&M problems will continue at similar rates as years 0 to 50.

US Army Corps of Engineers ®
Kansas City District

Kansas River Reservoirs Flood and Sediment Study

Appendix D2.5: Kanopolis Lake Future Without Project Sedimentation

November 2022

Contents

1.0 INTRODUCTION

This appendix documents the projections for the Future Without Project (FWOP) condition for Kanopolis Lake. The FWOP condition is the expected conditions of the lake if sediment accumulation continues unabated, i.e., without intentional sediment management or removal. This analysis also assumes no pool raise or reallocation takes place, as such actions would be potential measures for a Future With Project condition. This analysis incorporates the trends documented in the Existing Conditions analysis for Kanopolis Lake, Appendix D1.5. Figure 1-1 shows the lake with respect to the Kansas River Basin.

FWOP computations were first completed in December 2020 but were later updated in May 2022 based a new methodology incorporating sediment modeling results from Tuttle Creek Lake. The new methodology included the effects of additional sediment depositing in the flood pool (FP) over time.

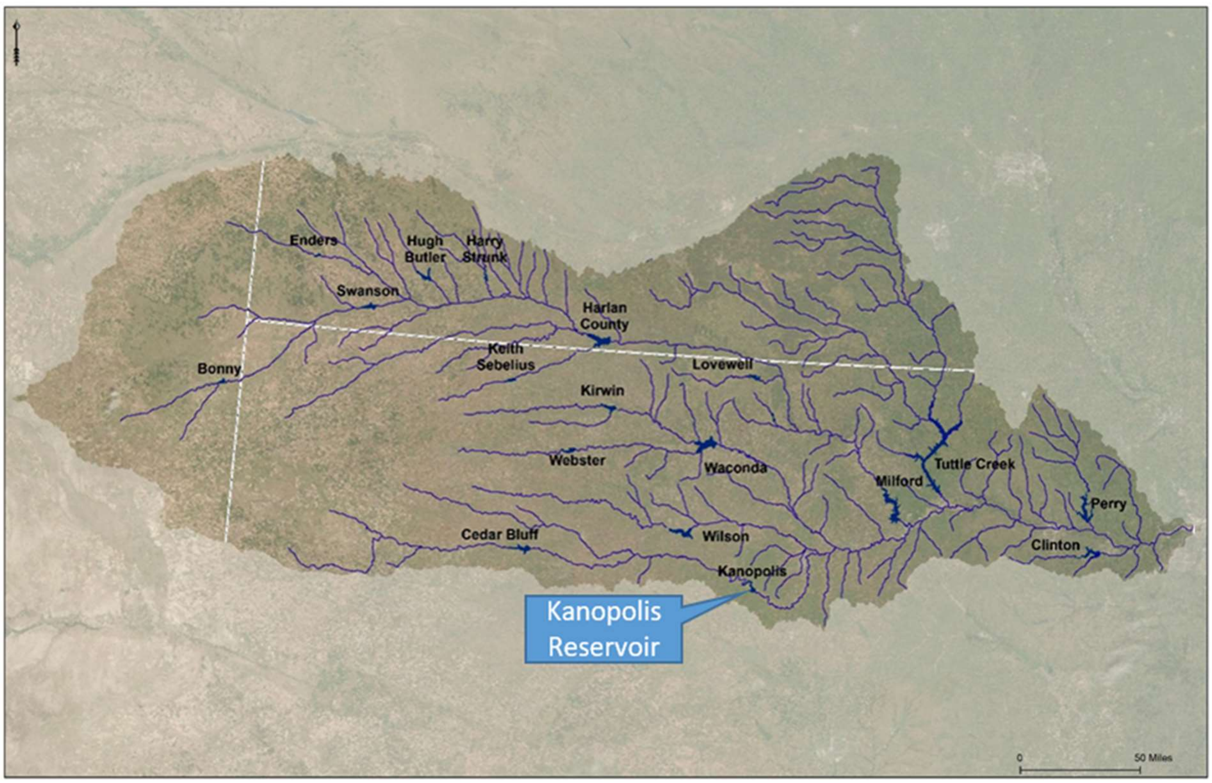


Figure 1-1: Overall Kansas River Basin Map and Kanopolis Lake

2.0 FWOP SEDIMENT DEPOSITION

Long-term sediment deposition was computed by running 100 years of daily flow through the calibrated rating curve described in Appendix D1.5. The flows were obtained for the U.S. Geological Survey (USGS) gage upstream of the lake used in the calibration, repeating the 50-year period from 1970 to 2019 to obtain a total of 100-years. The beginning date for the simulation was 1 January 2024, the assumed date this project will be completed. To obtain total deposition since construction of the lake, calculations were made from the date of the last bathymetric survey in 2017 to the beginning of 2124.

The lake inflow data, which is calculated by NWK Water Management Section, was used to check for stationarity and climate change. This was done by determining how well the flow frequency statistics based on the previous 50-years of data match with the entire available period of record for the lakes. A Bulletin 17 analysis was performed on the daily lake inflow data (annual maxima) and the computed flow frequency curves were compared. The curves matched closely enough to use the last 50 years of data (1970-2019) for a period of analysis. Also, it appears that the data is stationary based on results of running the inflow annual maxima data through the Non-Stationarity Detection Analysis and Trend Analysis in the Time Series Toolbox application developed by the Climate Preparedness and Resilience (CPR) CoP. These are the standard tools used in Qualitative Climate Change analyses to evaluate the stationarity of stream flow data.

Although lake inflow data was used to check for stationarity, the USGS gauged inflows were used for the sedimentation analysis, since this is what was used to develop the flow/load rating curves. However, water use in the watershed has changed over time due to changing demand, which is referred to as depletions. A timeseries of depletion data was obtained from NWK Water Management (see Appendix B) and added to the gauged flows to obtain the FWOP flows. Generally, the depleted flows were less than the gauged flows.

As noted in the Existing Conditions Appendix, 97% of all sediment enters the lake while the lake water surface is in the flood control pool. However, deposition in the flood control pool represents 40.6% of the total mass. From these numbers, the historic trapping efficiency of the flood control pool is 40.1%. The original FWOP computations assumed that this ratio remained constant over the FWOP. However, additional investigations indicated that additional mass should be deposited in the FP as the MPP fills in.

For the final FWOP, the percentage of the incoming mass that is deposited in the flood control pool FP was varied over time based on a regression equation developed from lake surveys and numerical modeling at Tuttle Creek Lake. Figure 2-1 shows the percentage of total incoming mass that deposits in the FP on the Y-axis and the percent volume lost from the MPP on the X-axis. Both survey data and HEC-RAS sediment model results for Tuttle Creek are included in Figure 2-1. See Appendix D4 for additional details on the HEC-RAS sediment modeling. A polynomial regression equation was fitted to the data, until the MPP has lost 80% of its original volume, after which the % mass to the FP is held constant.

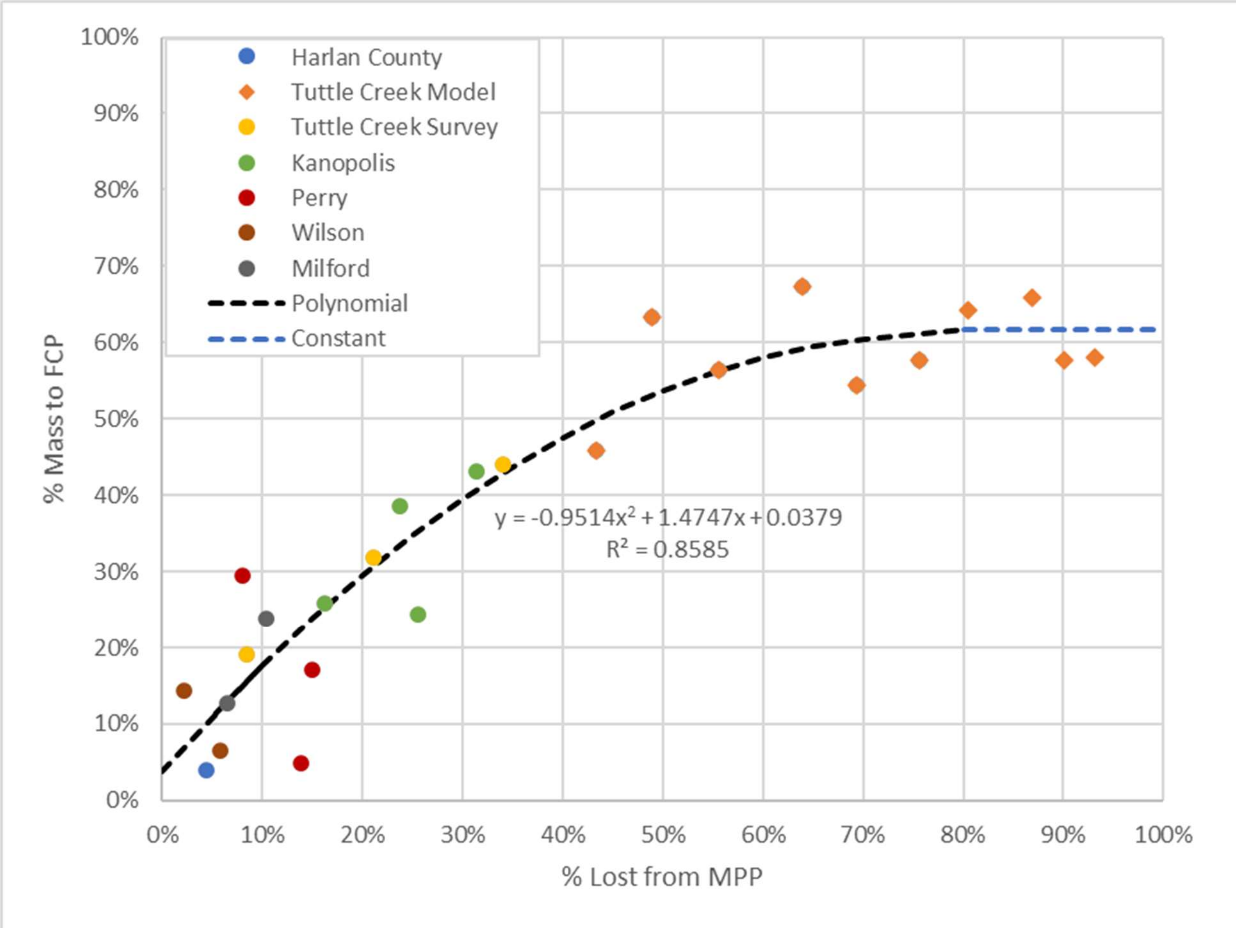


Figure 2-1. The percentage of mass depositing in the FP vs. the percent of volume lost from the MPP

For the multipurpose pool (MPP), the trapping efficiency starts at 95.7%, as measured by USGS (Juracek, 2011). The trapping efficiency decreases over time, using the Brune Curve method.

From the 2017 survey, the multipurpose pool capacity can be estimated at 47,170 ac-ft. The mean annual water inflow into the lake is 158,294 acre-feet, based on stream gage data and a correction for the ungauged area. Brune offers three curves shown in Figure 2-2 for estimating trapping efficiency (Brune, 1953). These can be estimated using Equations 1 and 2 and the constants in Table 2-1. The medium curve produced a trapping efficiency that matched closest with the USGS estimate.

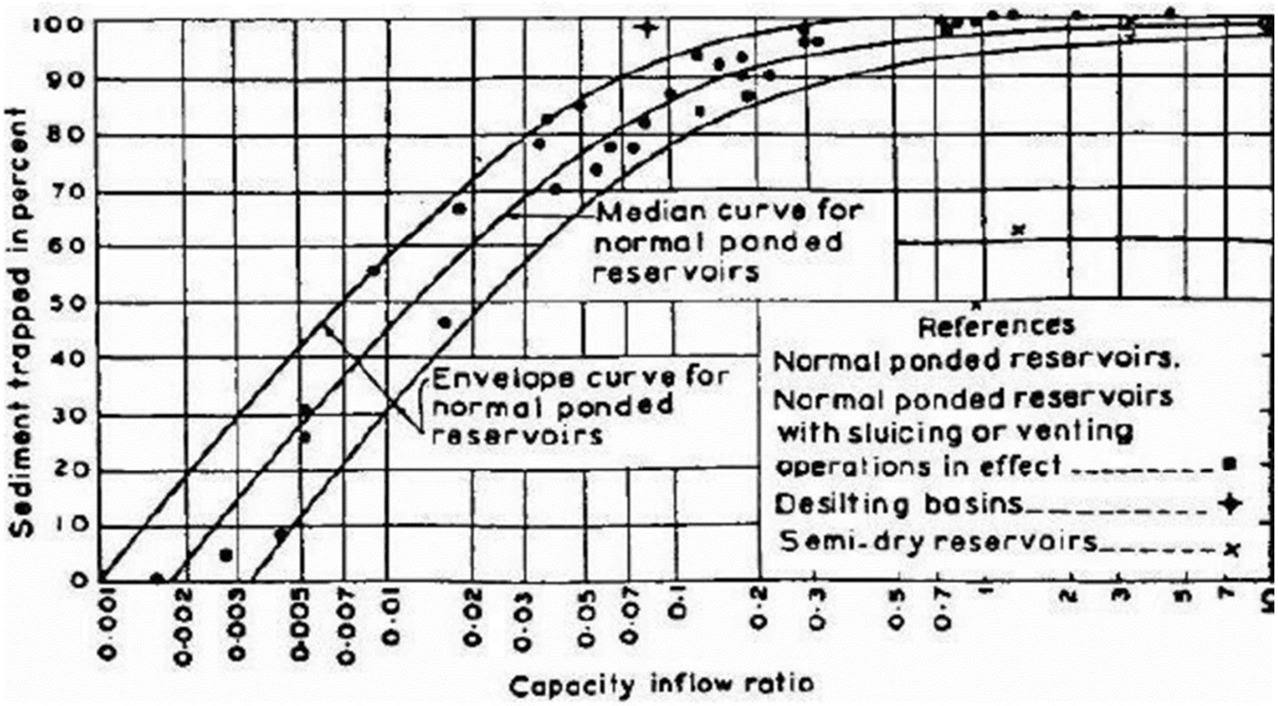


Figure 2-2: Brune Curves, (Brune, 1953)

$$TE = a[1 - 2e^{-bV_*^{0.35}}] \quad (1)$$

$$V_* = \frac{V_{res}}{V_{inflow}} \quad (2)$$

Table 2-1: Constants to be used in Equations 1 and 2

Constant	Low	Medium	High
a	95	97	100
b	5.37	6.42	7.71

The Brune Curve estimate for Kanopolis Lake in 2017 is 94.1%, which is 1.6% lower than the USGS estimate. In the FWOP analysis, the trapping efficiency was assumed to start at the USGS value but decrease parallel to the Brune Curve. This was accomplished by adding 1.6% to the Brune Curve estimation at each daily time step. As the lake fills in, the trapping efficiency of the multipurpose pool decreases. Table 2-2 lists the MPP trapping efficiency at year 0, 25, 50, and 100.

Table 2-2: Trapping Efficiency over Time

Year	Cumulative Incoming Load (tons)	Deposited in the FCP (tons)	Passed to the MPP (tons)	MPP Final TE (%)
0	6,032,463	2,803,013	3,229,450	95.31%
25	21,664,127	10,914,072	10,750,054	94.16%
50	31,092,205	16,219,635	14,872,570	93.28%
100	58,401,432	32,515,750	25,885,682	89.08%

As the lake shrinks, the trapping efficiency decreases. The volume that deposits over time is only subdivided into the MPP or the FCP (not into discreet elevations within each pool). To make this scheme workable but still incorporate shrinking pools over time, two simplifying assumptions were incorporated. (1) The volume of the multi-purpose pool is used to compute the trapping efficiency, independent of

whether the pool elevation is above or below the MPP level on any particular day. (2) The mass is apportioned based on the ratios given in Figure 2-1.

This list summarizes the steps to compute future sedimentation volumes and remaining pool capacities:

1. For each day, compute the mass of incoming sediment using the calibrated rating curve.
2. Compute the percentage of the mass deposits that in the FCP. The remaining mass is passed to the MPP. As previously discussed, the original FWOP calculations assumed this ratio remained constant.
3. Use the Brune Curve plus 1.6% to compute the MPP trapping efficiency.
4. Apply the trapping efficiency to mass entering the MPP to compute trapped mass.
5. Use the bulk density of the FCP and MPP to covert the mass trapped into a volume trapped.
6. Subtract the volume of deposition in the FCP and MPP from the remaining capacities.
7. Repeat for each subsequent day, re-computing trapping efficiency with the progressively smaller MPP capacity.

Figure 2-3 provides the remaining storage capacities of the multipurpose and flood control pools for both the original and final FWOP computations. Table 2-3 and Table 2-4 summarize the final results.

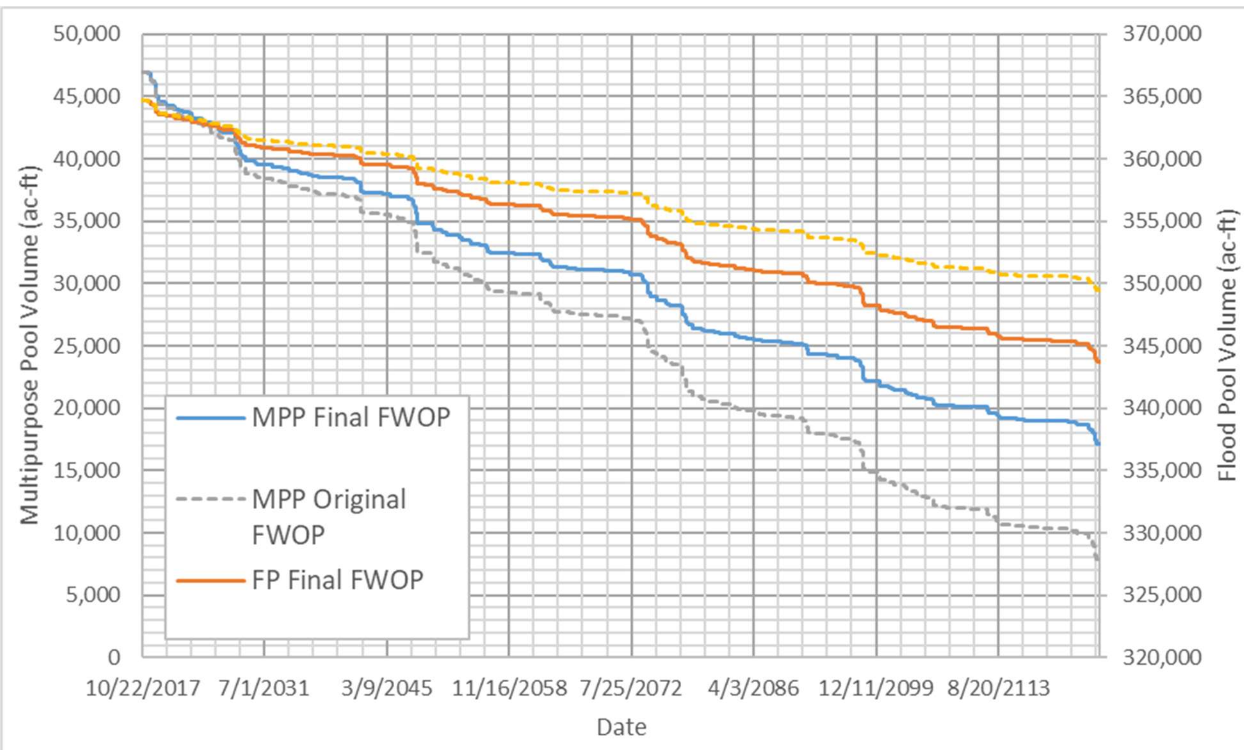


Figure 2-3: Remaining Pool Volumes Over Time

Table 2-3: Summary of Remaining Pool Volumes

Year	Multi-Purpose Pool (ac-ft)	Multi-Purpose Pool % of Original Volume	Flood Control Pool (ac-ft)	Flood Control Pool % of Original Volume
1948	73,200	100.0%	373,891	100.0%
2024	43,255	59.1%	362,915	97.1%

2049	34,794	47.5%	357,956	95.7%
2074	30,207	41.3%	354,713	94.9%
2124	18,248	24.9%	344,752	92.2%

Table 2-4: Sediment Deposition Volumes

Year	2017	2024	2049	2074	2124
Cumulative Deposition Multi-Purpose Pool (ac-ft)	26284	29945	38406	42993	54952
Average Annual MPP Deposition Since 2024 (ac-ft/yr)	-	-	338	261	250
Average Annual MPP Deposition over Increment (ac-ft/yr)	378	590	365	183	239
Cumulative Deposition Flood Control Pool (ac-ft)	9263	10976	15935	19178	29139
Average Annual FCP Deposition Since 2024 (ac-ft/yr)	-	-	198	164	182
Average Annual FCP Deposition over Increment (ac-ft/yr)	133	276	198	130	199

Table 2-5 gives the storage volumes from the original iteration of the FWOP computations along with the difference from the final results. As seen in Table 2-5, the MPP has greater storage over the FWOP for the final results, while the FP has less storage.

Table 2-5: Pool volumes in acre-feet from the original FWOP computations, and the differences from the final computations

Year	Multi-Purpose Pool Original	Flood Control Pool Original	Difference Multipurpose Pool	Difference Flood Control Pool
2024	42,858	363,130	+397	-215
2049	32,444	359,248	+2,350	-1,291
2074	26,263	356,906	+3,944	-2,193
2124	9,334	350,123	+8,914	-5,372

FWOP elevation-storage and elevation-area curves were estimated for use in the Water Management analyses documented in Appendix B. For the MPP, the curves were estimated using the surface volume tool in Arc-GIS along with the digital elevation models (DEMs) discussed in Section 3. However, for the FP portion of the lake, the curves were estimated based on the daily pool elevation from the FWOP HEC-ResSIM modeling (see Appendix B). In this analysis, the FP was broken up into 5-ft increments and for each day in the FWOP, the projected deposition was evenly distributed across all increments at or below the daily pool elevation. The reduction in surface area over that increment was then estimated by assuming it had the same percent reduction as the storage volume. However, because different methods were used to estimate the MPP and FP curves, a discontinuity was observed in the surface area between them. This was resolved replacing the surface area for first point in the FP with the value obtained by interpolating between the final MPP point and the second FP point. This methodology compared well with the 1D HEC-RAS sediment modeling results discussed in Appendix D4. See Appendix B for the final elevation-storage and elevation-area curves.

3.0 DELTA PROGRESSION

As listed in Appendix D1.5, the delta progression rate towards the dam from 2007 to 2017 was 472 feet per year. As deltas progress into wider and deeper sections of the lake, the rate of progression is expected to slow. The future rate was estimated by finding the delta progression that would yield the correct volume of sediment deposition as listed in Table 2-4. This delta progression analysis provides approximate results based on applying MPP deposition as an even veneer of deposition over all areas under the multipurpose pool elevation. This was accomplished using ArcGIS to add the elevation change, determined by dividing projected volume change by lake surface area, to the 2009 bathymetric surface. Multiple iterations were needed since areas of the lake would rise above the MPP, and the volume above the MPP had to be added again.

Figure 3-2 depicts the historic, current, and projected delta locations. The 25, 50, and 100 year projections indicate delta progression of 0.47, 0.76, and 1.25 miles. This equates to a loss in multipurpose pool surface area, as listed in Table 3-1.

Table 3-1: Multipurpose Pool Surface Area

Year	MPP Surface Area (ac)	Lost Surface Area compared to Original (ac)	Lost Surface Area Compared to 2024 (ac)
1948	4,078	-	-
2017	3,056	1,022	-
2024	2,911	1,167	-
2049	2,563	1,515	348
2074	2,399	1,679	512
2124	1,879	2,199	1,032

Figure 3-1 shows the elevation contour of the MPP for various years. The 1968 contour was digitized from a historic map of the lake.

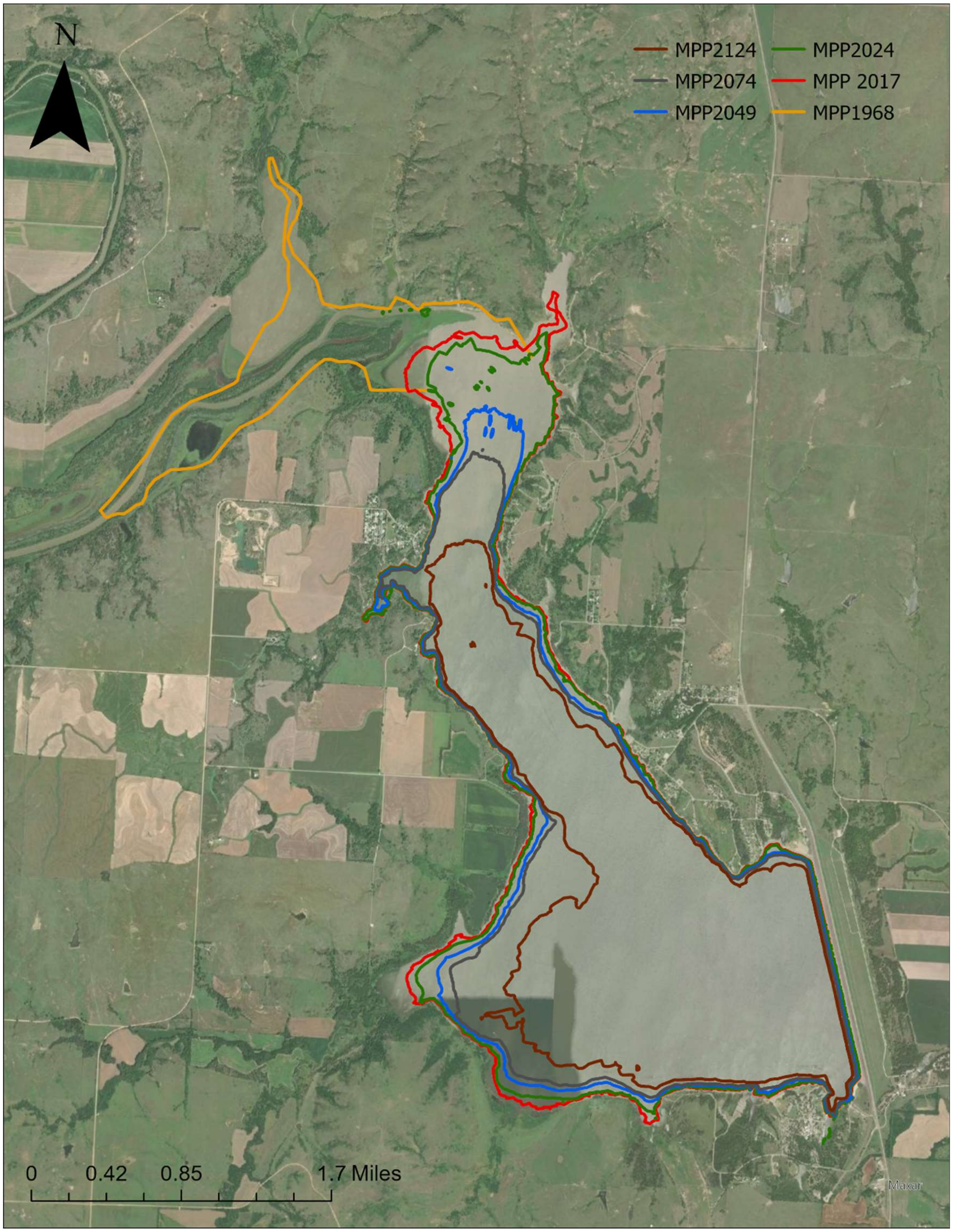


Figure 3-1: Multipurpose Pool Elevation Contours (1463.36 feet NAVD88)

Figure 3-2 gives the projected elevation of a centerline of the lake. The projected elevations were extracted from the projected DEMs.

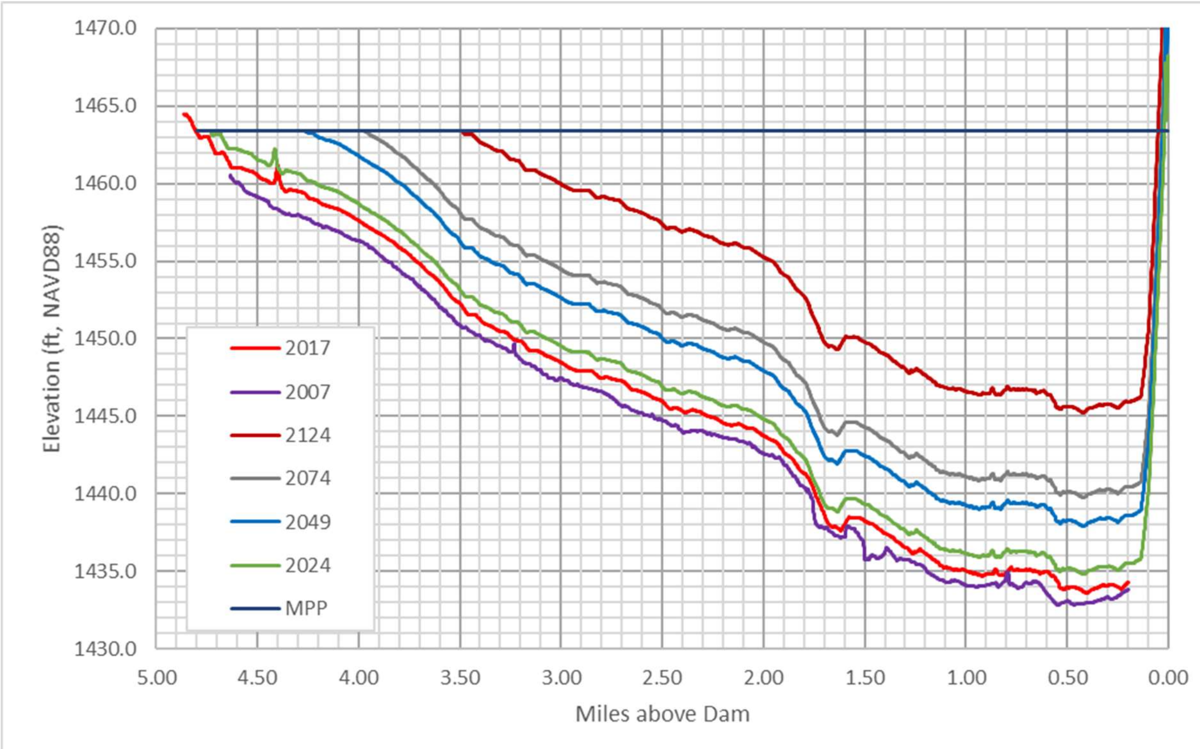


Figure 3-2: Projected Delta Location. Include lines for locations of important features.

4.0 DOWNSTREAM SEDIMENT CONCENTRATIONS

As the multipurpose trapping efficiency of the Kanopolis Lake decreases from 95% to 89% as explained in Section 2, more sediment will be supplied to the river downstream. Most of this additional sediment will be fine sediment since it is assumed that most of the coarse sediment (sands and gravels) will continue to deposit in the FCP. It was determined in Appendix D1.5 that 97% of the sediment enters the lake during flood control operations.

As most of the coarse sediment will continue to be trapped, this will continue to induce degradation and bank erosion, as indicated in the next section. The fine sediment loading to the river, however, will increase as the MPP trapping efficiency decreases from 95% to 89%. Table 4-1 indicates how the sediment concentration of the water released from Kanopolis Dam increases over time. These are approximate estimates obtained by comparing the mass trapped by the dam to the total sediment inflow. The increase in sediment concentration will vary with flow rate and other factors, which would require more detailed modeling in order to predict.

Table 4-1: Increase in the sediment concentration of releases from Kanopolis Dam

Year	Passing Percentage	% of 2024 Concentration
2024	2.43%	100%
2049	2.63%	108%
2074	2.86%	118%
2124	4.25%	175%

5.0 DOWNSTREAM CHANNEL

The existing conditions report demonstrated bed degradation and bank erosion downstream from Kanopolis Lake. Over the next 100 years, an additional 9,641,494 CY of sand will be trapped in Kanopolis Lake which will further induce bed degradation and bank erosion on the Smoky Hill River. Geomorphic principles suggest that rivers adjust to transport the bed material load supplied to them. Thus, the downstream channel will degrade (which lowers the slope and hence the applied shear stress) and an amour layer will form (which increases the critical shear required to initiate motion) until the flow from the dam is no longer able to transport sediment. Sedimentation modeling beyond the scope of this watershed study is required to compute the final bed elevations and to estimate bank erosion at the end of 25, 50, and 100 years. While applying trendlines to degradation or bank erosion is not ideal, it has been used when detailed modeling is not practical (references). Herein is provided an order-of-magnitude approximation based on assumptions that geomorphic feedbacks do not change the rates of geomorphic change. This approximation was computed via the following steps:

1. Compute the ratio of sand trapped in the lake (tons) to volumetric degradation downstream (CY) over a defined distance. For Kanopolis Lake 8,466,455 CY of sand trapped by the dam induced 437,213 CY of bed and bank erosion in the 16 miles downstream of the dam, or a ratio of 0.05 CY downstream degradation/CY trapped.
2. Apply the ratio to the future estimates for trapped sand.

Table 5-1 provides the estimated degradation volume in the 16 miles below Kanopolis Dam at the end of 25, 50, and 100 years. This analysis provides a high-level assessment suitable for relative ranking of alternatives.

Table 5-1: Degradation Downstream of the Kanopolis Dam compared to latest degradation survey in 2015

Year	Trapped Sand (CY)	Degradation Since Last Survey (CY)	Degradation since 2024 (CY)
2024	1,135,822	58,655	-
2049	4,045,536	208,914	150,260
2074	5,731,742	295,991	237,336
2124	10,777,316	556,548	497,893

6.0 SUMMARY, LIMITATIONS, AND CONCLUSIONS

The following list summarizes the 50-year FWOP condition at Kanopolis Lake. (For the 25 years and 100 years projections, see the body of the report.)

- 19,289,564 tons of fine sediment (silt/clay) and 5,770,177 of coarse sediment (sand/gravel) are expected to enter Kanopolis Lake, of which 13,416,622 tons are trapped in the flood control pool, 10,992,540 tons are trapped in the multi-purpose pool, and 650,580 tons pass downstream.
- The flood control pool (FCP) and multi-purpose pool (MPP) shrinks by an additional 8,201 ac-ft and 13,048 ac-ft, respectively.
- At the end of 50 years, the MPP has 41.3% remaining of its original capacity.
- At the end of 50 years, the FCP has 94.9% remaining of its original capacity.
- At the end of 50 years, the coarse sediment delta progresses downstream an additional 0.76 miles, which shrinks the surface area of the lake to 59% of the original.
- The downstream fine sediment concentrations will rise to 143% of existing.
- The downstream channel will experience additional bed degradation and bank erosion, with 237,336 CY degrading from the lower 16 miles.

These findings are based on trendline projections and empirical equations. For more precise estimations, particularly of where in the pools the sediment will deposit and how the downstream channels will adjust over time, numerical modeling is recommended. Moreover, this analysis did not compute damages due to upstream delta migration and only approximated downstream degradation as a total volume, not as bank erosion or bed degradation at specific affected infrastructure.

Notwithstanding the limitations, these projections indicate that sediment accumulation in Kanopolis Lake will be a serious problem over the next 50 years with serious implications for loss in benefits, environmental harm, infrastructure damage, and increased O&M needs. From years 50 to 100, sediment trapping and O&M problems will continue at similar rates as years 0 to 50.

US Army Corps
of Engineers ®
Kansas City District

Kansas River Reservoirs Flood and Sediment Study

Appendix D2.6: Wilson Lake Future Without Project Sedimentation

November 2022

Contents

1.0 WILSON LAKE INTRODUCTION

This appendix documents the projections for the Future Without Project (FWOP) condition for Wilson Lake. The FWOP condition is the expected conditions of the lake if sediment accumulation continues unabated, i.e., without intentional sediment management or removal. This analysis also assumes no pool raise or reallocation takes place, as such actions would be potential measures for a Future With Project condition. This analysis incorporates the trends documented in the Existing Conditions analysis for Wilson Lake, Appendix D1.6. Figure 1-1 shows the lake with respect to the Kansas River Basin.

FWOP computations were first completed in December 2020 but were later updated in May 2022 based a new methodology incorporating sediment modeling results from Tuttle Creek Lake. The new methodology included the effects of additional sediment depositing in the flood pool (FP) over time.

After the FWOP calculations were completed, it was noticed that the wrong multipurpose (MPP) volume was used for 2008. The correct value should be 236,188 ac-ft rather than 237,051. However, since this error is only 0.4% of the total MPP volume, it was decided this would have an inconsequential impact on the results, and the error was not corrected.

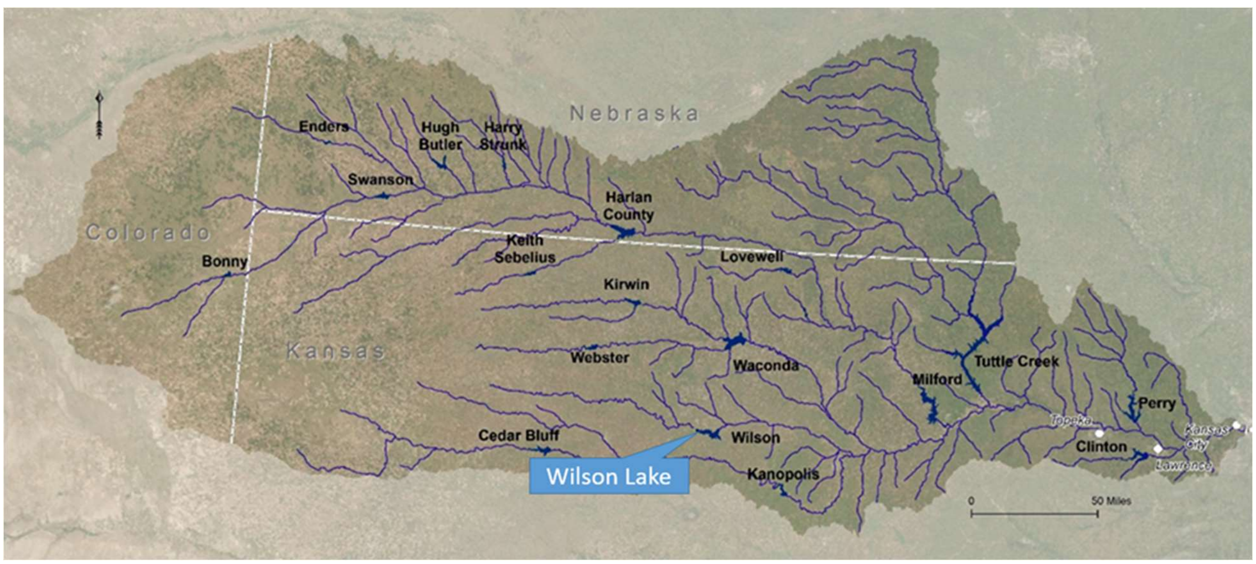


Figure 1-1: Overall Kansas River Basin Map and Wilson Lake

2.0 WILSON LAKE FWOP SEDIMENT DEPOSITION

Long-term sediment deposition was computed by running 100 years of daily flow through the calibrated rating curves described in Appendix D1.6. The flows were obtained from the U.S. Geological Survey (USGS) gage on the Saline River at Russell, Kansas, which is upstream of the Wilson Lake, repeating the 50-year period from 1970 to 2019 to obtain a total of 100-years. The beginning date for the simulation was 01 January 2024, the assumed date this project will be completed. To obtain total deposition since construction of the lake, calculations were made from the date of the last bathymetric survey in 2008 to the beginning of 2124.

The lake inflow data, which is calculated by NWK Water Management Section, was used to check for stationarity and climate change. This was done by determining how well the flow frequency statistics based on the previous 50-years of data match with the entire available period of record for the lakes. A Bulletin 17 analysis was performed on the daily lake inflow data (annual maxima) and the computed flow frequency curves were compared. The curves matched closely enough to use the last 50 years of data (1970-2019) for a period of analysis. Also, it appears that the data is stationary based on results of running the inflow annual maxima data through the Non-Stationarity Detection Analysis and Trend Analysis in the Time Series Toolbox application developed by the Climate Preparedness and Resilience (CPR) CoP. These are the standard tools used in Qualitative Climate Change analyses to evaluate the stationarity of stream flow data.

Although lake inflow data was used to check for stationarity, the USGS gauged inflows were used for the sedimentation analysis, since this is what was used to develop the flow/load rating curves. However, water use in the watershed has changed over time due to changing demand, which is referred to as depletions. A timeseries of depletion data was obtained from NWK Water Management (see Appendix B) and added to the gauged flows to obtain the FWOP flows. Generally, the depleted flows were less than the gauged flows.

As noted in the Existing Conditions Appendix, 82% of all sediment enters the lake while the lake water surface is in the flood control pool. However, deposition in the flood control pool represents 9.6% of the total mass. From these numbers, the historic trapping efficiency of the flood control pool is 11.4%. The original FWOP computations assumed that this ratio remained constant over the FWOP. However, additional investigations indicated that additional mass should be deposited in the FP as the MPP fills in.

For the final FWOP calculations, the percentage of the incoming mass that is deposited in the FP was varied over time based on a regression equation developed from lake surveys and numerical modeling at Tuttle Creek Lake. Figure 2-1 shows the percentage of total incoming mass that deposits in the FP on the Y-axis and the percent volume lost from the MPP on the X-axis. Both survey data and HEC-RAS sediment model results for Tuttle Creek are included in Figure 2-1. See Appendix D4 for additional details on the HEC-RAS sediment modeling. A polynomial regression equation was fitted to the data, until the MPP has lost 80% of its original volume, after which the % mass to the FP is held constant.

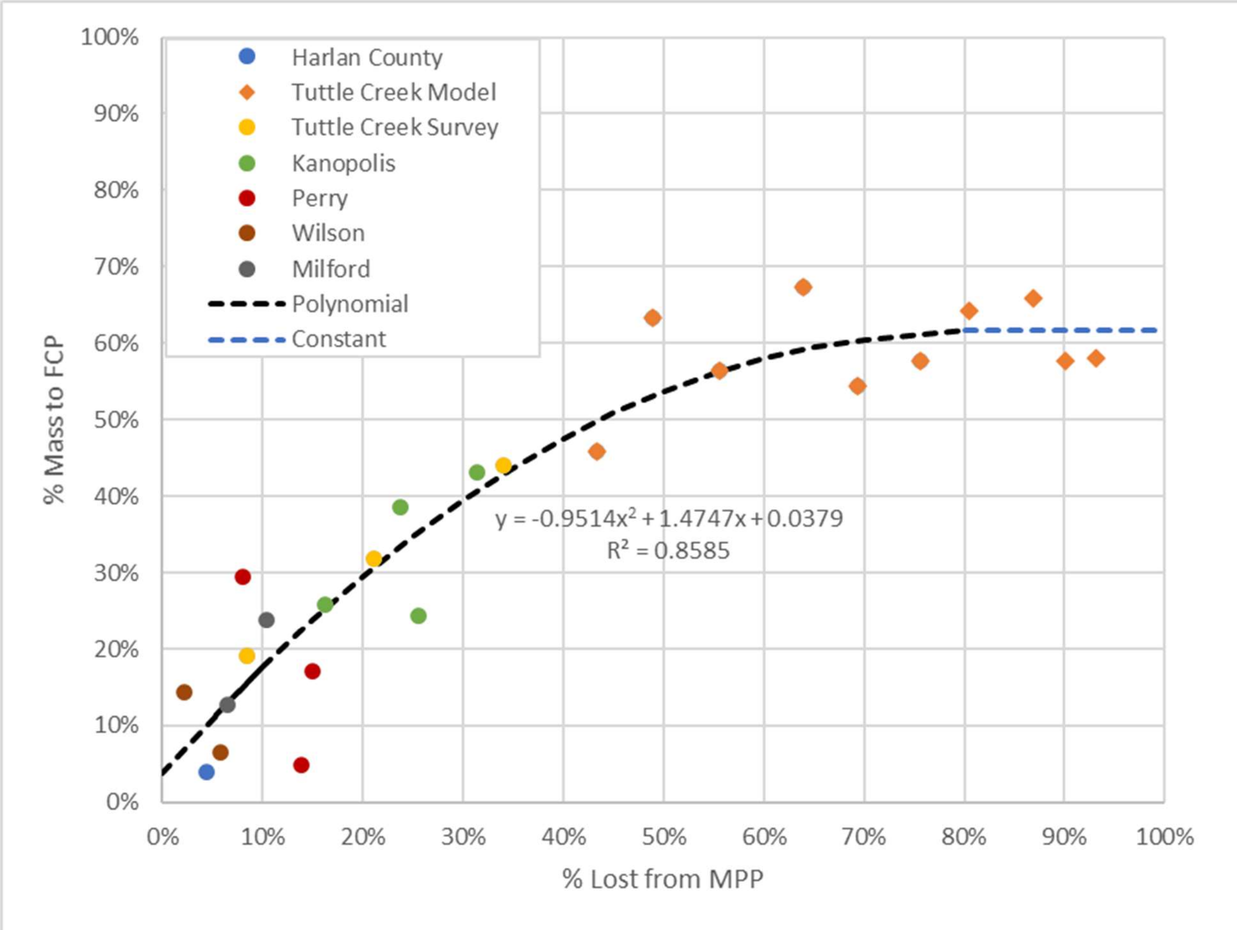


Figure 2-1. The percentage of mass depositing in the FP vs. the percent of volume lost from the MPP

The trapping efficiency in the MPP decreases over time, using the Brune Curve method. From the 2008 survey, the multipurpose pool capacity can be estimated at 237,051 ac-ft. The mean annual water inflow into the lake is 98,019 acre-feet, based on stream gage data. Brune offers three curves shown in Figure 2-2 for estimating trapping efficiency (Brune, 1953). These can be estimated using Equations 1 and 2 and the constants in Table 2-1. The medium curve was used for this analysis.

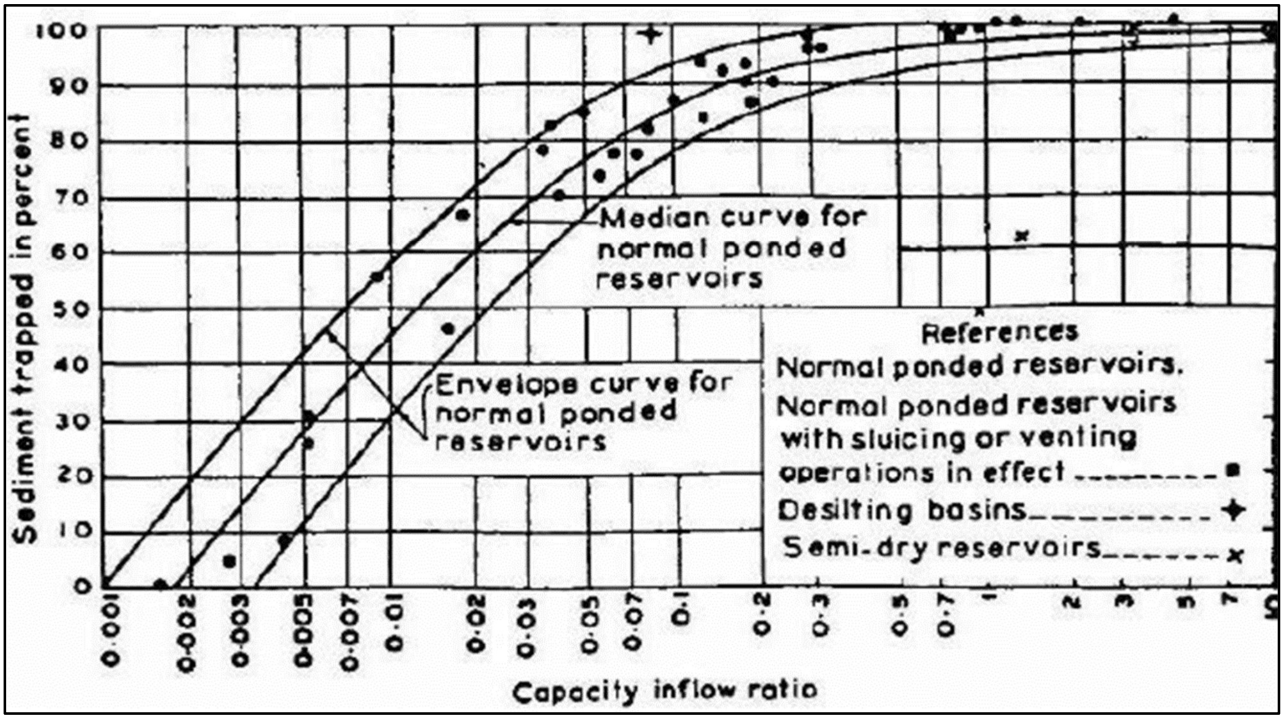


Figure 2-2. Brune Curves (Brune, 1953).

$$TE = a\left[1 - 2e^{-bV_*^{0.35}}\right] \quad (1)$$

$$V_* = \frac{V_{res}}{V_{inflow}} \quad (2)$$

Table 2-1: Wilson Lake Constants to be used in Equations 1 and 2.

Constant	Low	Medium	High
a	95	97	100
b	5.37	6.42	7.71

The Brune Curve estimate for Wilson in 2008 is 97% and as the lake fills in, the trapping efficiency of the multipurpose pool decreases slightly. Table 2-2 lists the MPP trapping efficiency at year 0, 25, 50, and 100. Because the capacity of the multipurpose pool is so large compared to the sediment deposition, trapping efficiency does not decrease significantly.

Table 2-2. Wilson Lake Trapping Efficiency over Time.

Year	Cumulative Incoming Load (tons)	Deposited in the FCP (tons)	Passed to the MPP (tons)	MPP Final TE (%)
2024	9,947,364	1,183,166	8,764,198	96.97%
2049	21,178,130	3,176,480	18,001,651	96.96%
2074	34,601,370	5,556,498	29,044,872	96.95%
2124	59,255,375	12,680,335	46,575,040	96.94%

As the lake shrinks, the trapping efficiency decreases. The volume that deposits over time is only subdivided into the MPP or the FCP (not into discreet elevations within each pool). To make this scheme workable but still incorporate shrinking pools over time, two simplifying assumptions were incorporated. (1) The volume of the multi-purpose pool is used to compute the trapping efficiency, independent of

whether the pool elevation is above or below the MPP level on any particular day. (2) The mass is apportioned based on the ratios given in Figure 2-1.

This list summarizes the steps to compute future sedimentation volumes and remaining pool capacities:

1. For each day, compute the mass of incoming sediment using the calibrated rating curve.
2. Compute the percentage of the mass that deposits in the FCP. The remaining mass is passed to the MPP. As previously discussed, the original FWOP calculations assumed this ratio remained constant.
3. Use the Brune Curve to compute the MPP trapping efficiency.
4. Apply the trapping efficiency to mass entering the MPP to compute trapped mass.
5. Use the bulk density of the FCP and MPP to covert the mass trapped into a volume trapped.
6. Subtract the volume of deposition in the FCP and MPP from the remaining capacities.
7. Repeat for each subsequent day, re-computing trapping efficiency with the progressively smaller MPP capacity.

Figure 2-3 provides the remaining storage capacities of the multipurpose and flood control pools for both the original and final FWOP computations. Table 2-3 and

Table 2-4 summarize the final results. Because the survey methodology is different for the 1964 and 2008 surveys, the change in capacity between the surveys is likely underpredicted as discussed in Appendix D1.6.

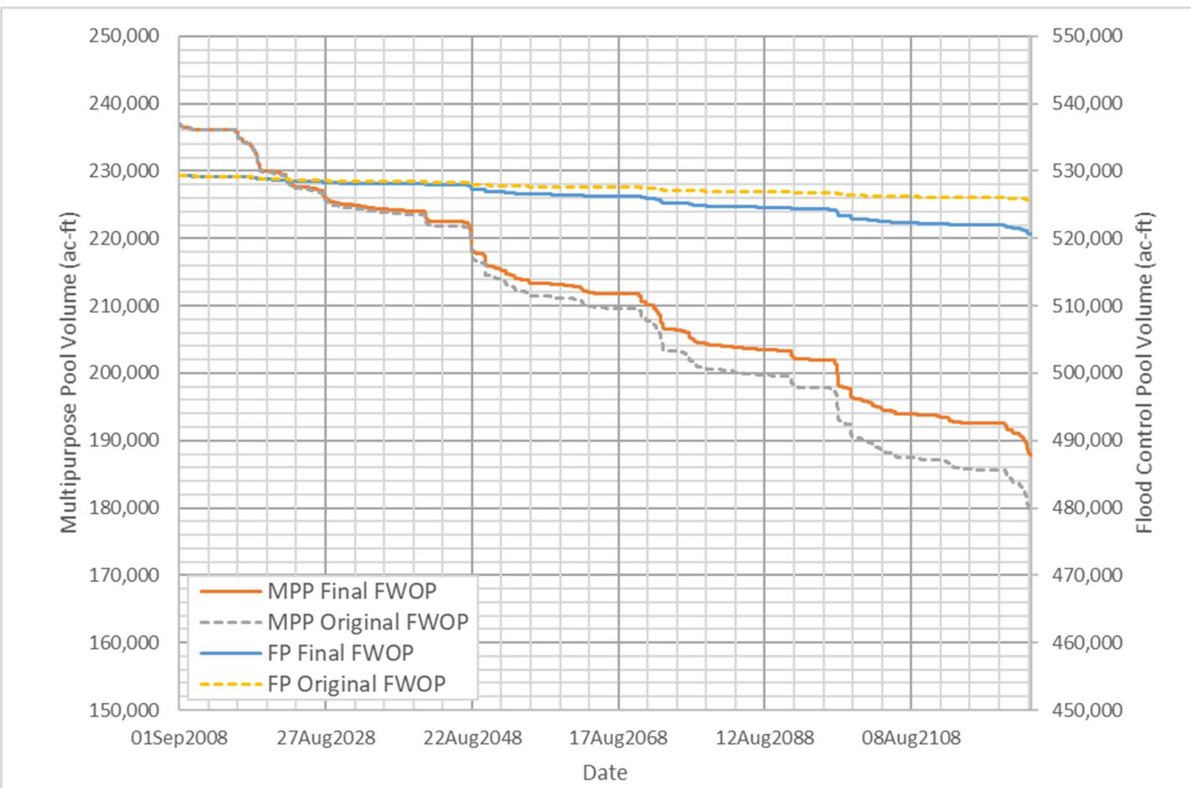


Figure 2-3: Wilson Lake Remaining Pool Volumes Over Time

Table 2-3: Summary of Remaining Pool Volumes in acre-feet over time.

Year	Multi-Purpose Pool (ac-ft)	Multi-Purpose Pool % of Original Volume	Flood Control Pool Volume, (ac-ft)	Flood Control Pool % of Original Volume
1964	247,835	100%	530,710	100%
2008	237,051	95.7%	529,289	99.7%
2024	228,085	92.0%	528,539	99.6%
2049	217,956	87.9%	527,275	99.4%
2074	209,022	84.3%	525,765	99.1%
2124	190,117	76.7%	521,248	98.2%

Table 2-4: Sediment Deposition Volumes

Year	2024	2049	2074	2124
Cumulative Deposition Multi-Purpose Pool (ac-ft)	19750	29879	38813	57718
Average Annual MPP Deposition Since 2024 (ac-ft/yr)	-	405	381	380
Average Annual MPP Deposition over Increment (ac-ft/yr)	549	405	357	378
Cumulative Deposition Flood Control Pool (ac-ft)	2171	3435	4945	9462
Average Annual FCP Deposition Since 2024 (ac-ft/yr)	-	51	55	73
Average Annual FCP Deposition over Increment (ac-ft/yr)	46	51	60	90

Table 2-5 gives the storage volumes from the original iteration of the FWOP computations along with the difference from the final results. As seen in Table 2-5, the MPP has greater storage over the FWOP for the final results, while the FP has less storage.

Table 2-5: Pool volumes in acre-feet from the original FWOP computations, and the differences from the final computations

Year	Multi-Purpose Pool Original	Flood Control Pool Original	Difference Multipurpose Pool	Difference Flood Control Pool
2024	227,372	528,683	+270	-172
2049	216,457	528,000	+1,196	-744
2074	203,404	527,182	+2,620	-1,605
2124	179,440	525,681	+7,670	-4,622

FWOP elevation-storage and elevation-area curves were estimated for use in the Water Management analyses documented in Appendix B. For the MPP, the curves were estimated using the surface volume tool in Arc-GIS along with the digital elevation models (DEMs) discussed in Section 3. However, for the FP portion of the lake, the curves were estimated based on the daily pool elevation from the FWOP HEC-ResSIM modeling (see Appendix B). In this analysis, the FP was broken up into 5-ft increments and for each day in the FWOP, the projected deposition was evenly distributed across all increments at or below the daily pool elevation. The reduction in surface area over that increment was then estimated by assuming it had the same percent reduction as the storage volume. However, because different methods were used to estimate the MPP and FP curves, a discontinuity was observed in the surface area between them. This was resolved replacing the surface area for first point in the FP with the value obtained by interpolating between the final MPP point and the second FP point. This methodology compared well with the 1D HEC-RAS sediment modeling results discussed in Appendix D4. See Appendix B for the final elevation-storage and elevation-area curves.

3.0 DELTA PROGRESSION

As listed in Appendix D1.6, the delta progression rate towards the dam from 1984 to 1995 was 168 feet per year. As deltas progress into wider and deeper sections of the lake, the rate of progression is expected to slow. The future rate was estimated by finding the delta progression that would yield the correct volume of sediment deposition as listed in Table 2-4. This delta progression analysis provides approximate results based on applying MPP deposition as an even veneer of deposition over all areas under the multipurpose pool elevation. Projected digital elevation models (DEM), surface area, and elevation contours of the MPP were computed using the following steps in ArcGIS.

1. Divide the projected deposition volume by the surface area of the MPP in 2009 to obtain the elevation change within the MPP.
2. Add this elevation to the 2009 DEM of the MPP to obtain a first estimate of the projected DEM.
3. Use the surface volume tool to estimate the new MPP volume and surface area.
4. Determine the difference between the predicted MPP volume and the MPP from the modified DEM
5. Divide the volume difference by the new surface area to obtain an additional elevation increase.
6. Add the additional elevation increase to the modified DEM
7. Repeat steps 3-6 until the DEM MPP volume approximately equal to the projected MPP volume. A difference less than the average annual deposition was considered to be adequate

The 25, 50, and 100 year projections indicate delta progression of 36, 72, and 490 ft. This equates to a loss in multipurpose pool surface area, as listed in Table 3-1.

Table 3-1: Multipurpose Pool Surface Area.

Year	MPP Surface Area (ac)	Lost Surface Area compared to Original (ac)	Lost Surface Area Compared to 2024 (ac)
1964	9,040	-	-
2008	8,637	403	-
2024	8,433	607	-
2049	8,303	737	131
2074	8,173	867	260
2124	7,868	1,172	565

Figure 3-1 shows the historical and projected elevation contours of the MPP. The original MPP contour was digitized from a historic map of the lake.

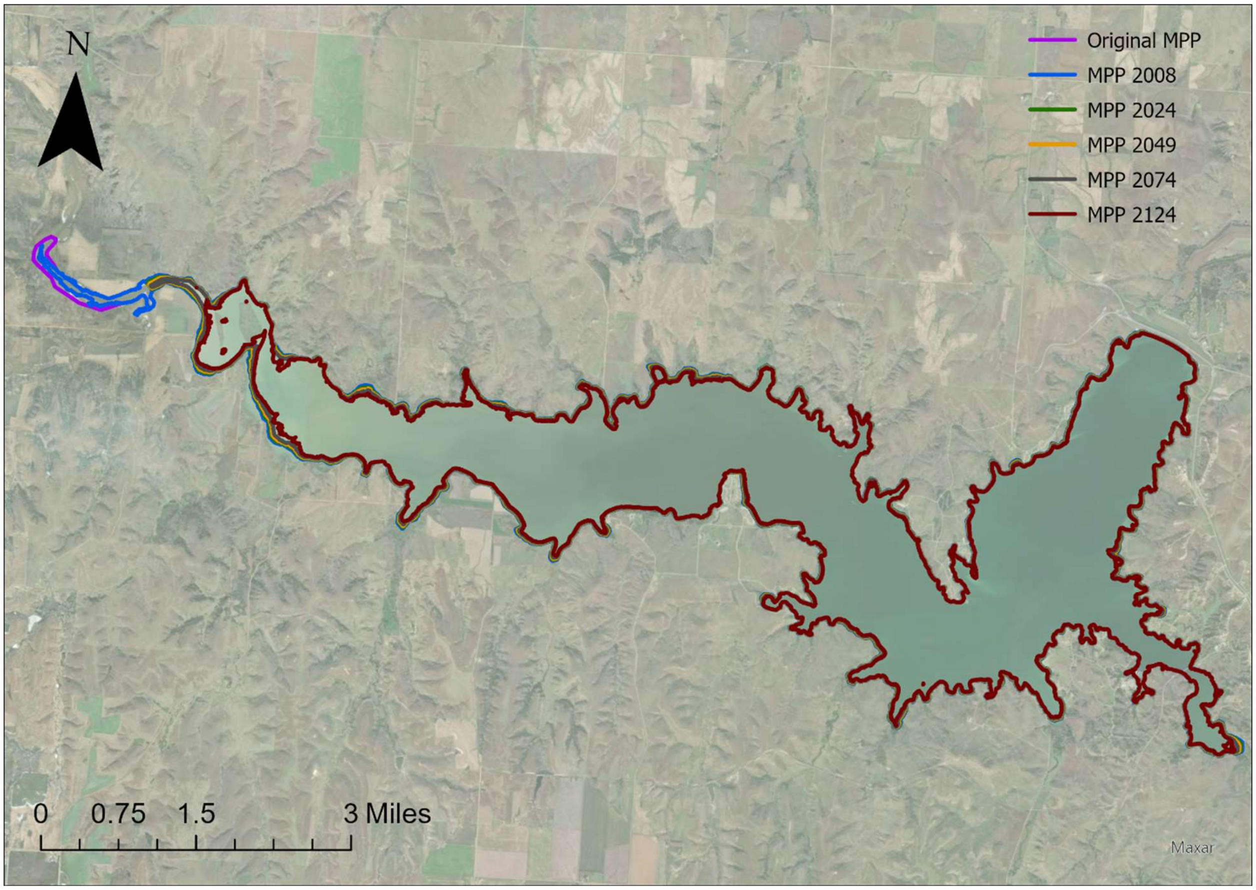


Figure 3-1: Historical and Projected MPP Elevation Contours (1516.59 ft NAVD88)

Figure 3-2 depicts the historic and projected bottom elevation along a centerline of the lake. The projected elevations were taken from the projected DEMs.

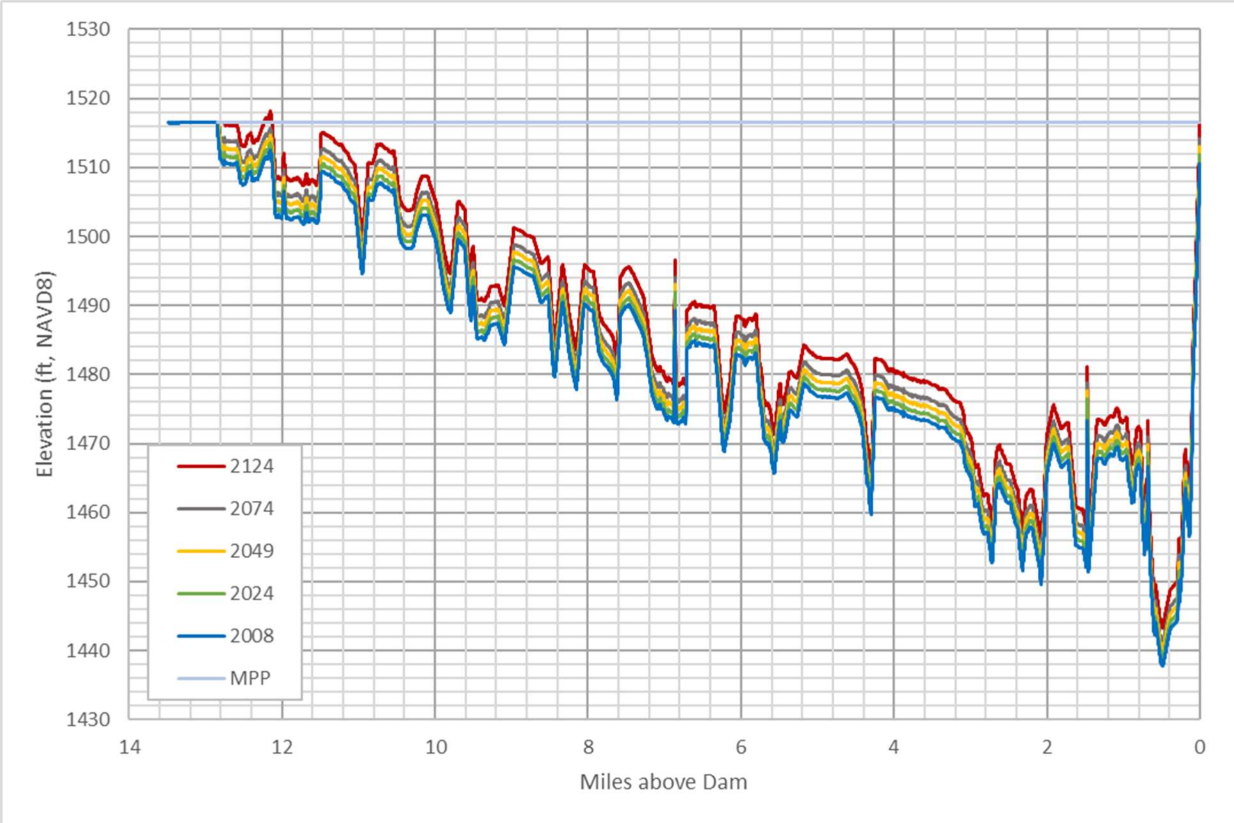


Figure 3-2: Projected Delta Location. Include lines for locations of important features.

4.0 DOWNSTREAM SEDIMENT CONCENTRATIONS

Any decline in the trapping efficiency over the next 100 years is expected to be negligible (0.03%). Because of this, the downstream sediment concentrations are not expected to change significantly.

5.0 DOWNSTREAM CHANNEL

The existing conditions report demonstrated that there has been little bed degradation downstream from Wilson Creek Lake. It assumed that over the next 100 years, the channel will continue to not degrade.

6.0 SUMMARY, LIMITATIONS, AND CONCLUSIONS

This appendix documented projections for the 50-year FWOP condition at Wilson Lake. (For the 25 years and 100 years projections, see the body of the report.) The list summarizes the main points:

- 24,757,815 tons of sediment are expected to enter Wilson Lake, of which 4,373,332 tons are trapped in the flood control pool, 17,126,229 tons are trapped in the multi-purpose pool, and 536,886 tons pass downstream.
- The flood control pool (FCP) and multi-purpose pool (MPP) shrinks by an additional 2,773 ac-ft and 19,063 ac-ft, respectively.
- At the end of 50 years, the MPP has 84.3% remaining of its original capacity if no pool rises are enacted.
- At the end of 50 years, the FCP has 99.1% remaining of its original capacity if no pool rises are enacted.
- At the end of 50 years, the coarse sediment delta progresses downstream an additional 72 feet, which shrinks the surface area of the lake to 90% of the original.
- The downstream fine sediment concentrations will not change.
- The downstream channel will continue to not experience degradation.

These findings are based on trendline projections and empirical equations. For more precise estimations, particularly of where in the pools the sediment will deposit and how the downstream channels will adjust over time, numerical modeling is recommended. Moreover, this analysis did not compute damages due to upstream delta migration and only approximated downstream degradation as a total volume, not as bank erosion or bed degradation at specific affected infrastructure.

Notwithstanding the limitations, these projections indicate that sediment accumulation in Wilson Lake will be a moderate problem over the next 50 years with some implications for loss in benefits, environmental harm, infrastructure damage, and increased O&M needs. From years 50 to 100, sediment trapping and loss of benefits will likely continue at similar rates as years 0 to 50.

US Army Corps of Engineers®
Kansas City District

Kansas River Reservoirs Flood and Sediment Study

Appendix D2.7: Harlan County Lake Future Without Project Sedimentation

November 2022

Contents

1.0 INTRODUCTION

This appendix documents the projections for the Future Without Project (FWOP) condition for Harlan County Lake. The FWOP condition is the expected conditions of the lake if sediment accumulation continues unabated, i.e. without intentional sediment management or removal. This analysis also assumes no pool raise or reallocation takes place, as such actions would be potential measures for a Future With Project condition. This analysis incorporates the trends documented in the Existing Conditions analysis for Harlan County Lake, Appendix D1.7. Figure 1-1 shows the lake with respect to the Kansas River Basin.

FWOP computations were first completed in December 2020 but were later updated in May 2022 based a new methodology incorporating sediment modeling results from Tuttle Creek Lake. The new methodology included the effects of additional sediment depositing in the flood pool (FP) over time.

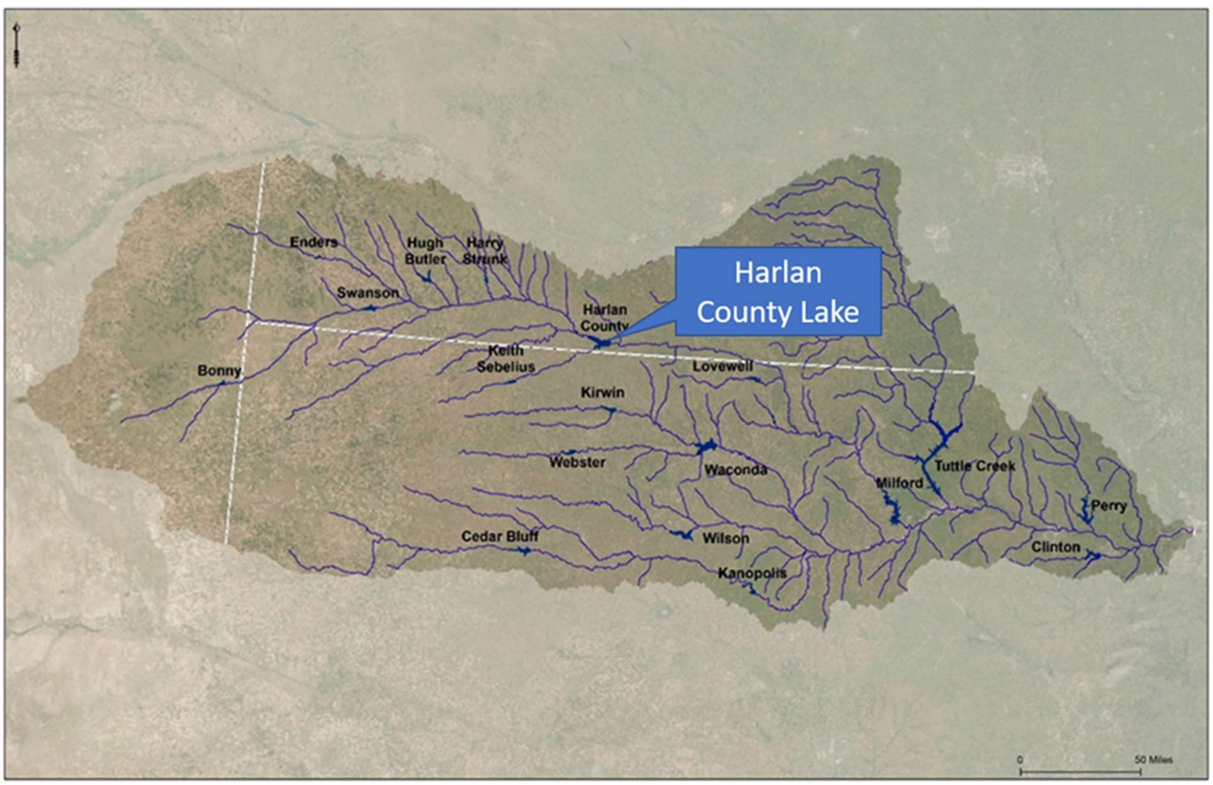


Figure 1-1: Overall Kansas River Basin Map and Harlan County Lake

2.0 FWOP SEDIMENT DEPOSITION

Long-term sediment deposition was computed by running 100 years of daily flow through the calibrated rating curves described in Appendix D1.7. The flows were obtained for the U.S. Geological Survey (USGS) gages upstream of the lake used in the calibration, repeating the 50-year period from 1970 to 2019 to obtain a total of 100-years. The beginning date for the simulation was 01 January 2024, the assumed date this project will be completed. To obtain total deposition since construction of the lake, calculations were made from the date of the last bathymetric survey in 2000 to the beginning of 2124. The measured flows were used for the period from 2000 to 2019, while data from 1970 to 1975 was used to fill in between 2020 and 2024.

The lake inflow data, which is calculated by NWK Water Management Section, was used to check for stationarity and climate change. This was done by determining how well the flow frequency statistics based on the previous 50-years of data match with the entire available period of record for the lakes. A Bulletin 17 analysis was performed on the daily lake inflow data (annual maxima) and the computed flow frequency curves were compared. The curves matched closely enough to use the last 50 years of data (1970-2019) for a period of analysis. Also, it appears that the data is stationary based on results of running the inflow annual maxima data through the Non-Stationarity Detection Analysis and Trend Analysis in the Time Series Toolbox application developed by the Climate Preparedness and Resilience (CPR) CoP. These are the standard tools used in Qualitative Climate Change analyses to evaluate the stationarity of stream flow data. Although lake inflow data was used to check for stationarity, the USGS gauged inflows were used for the sedimentation analysis, since this is what was used to develop the flow/load rating curves.

As noted in Appendix D1.7, 70.3% of all sediment enters the lake while the lake water surface is in the flood control pool. However, deposition in the flood control pool represents 4% of the total mass from 1962 to 1972. From these numbers, the historic trapping efficiency of the flood control pool is 5.5%. The original FWOP computations assumed that this ratio remained constant over the FWOP. However, additional investigations indicated that additional mass should be deposited in the FP as the multipurpose pool (MPP) fills in.

For the final FWOP, the percentage of the incoming mass that is deposited in the FP was varied over time based on a regression equation developed from lake surveys and numerical modeling at Tuttle Creek Lake. Figure 2-1 shows the percentage of total incoming mass that deposits in the FP on the Y-axis and the percent volume lost from the MPP on the X-axis. Both survey data and HEC-RAS sediment model results for Tuttle Creek are included in Figure 2-1. See Appendix D4 for additional details on the HEC-RAS sediment modeling. A polynomial regression equation was fitted to the data, until the MPP has lost 80% of its original volume, after which the % mass to the FP is held constant.

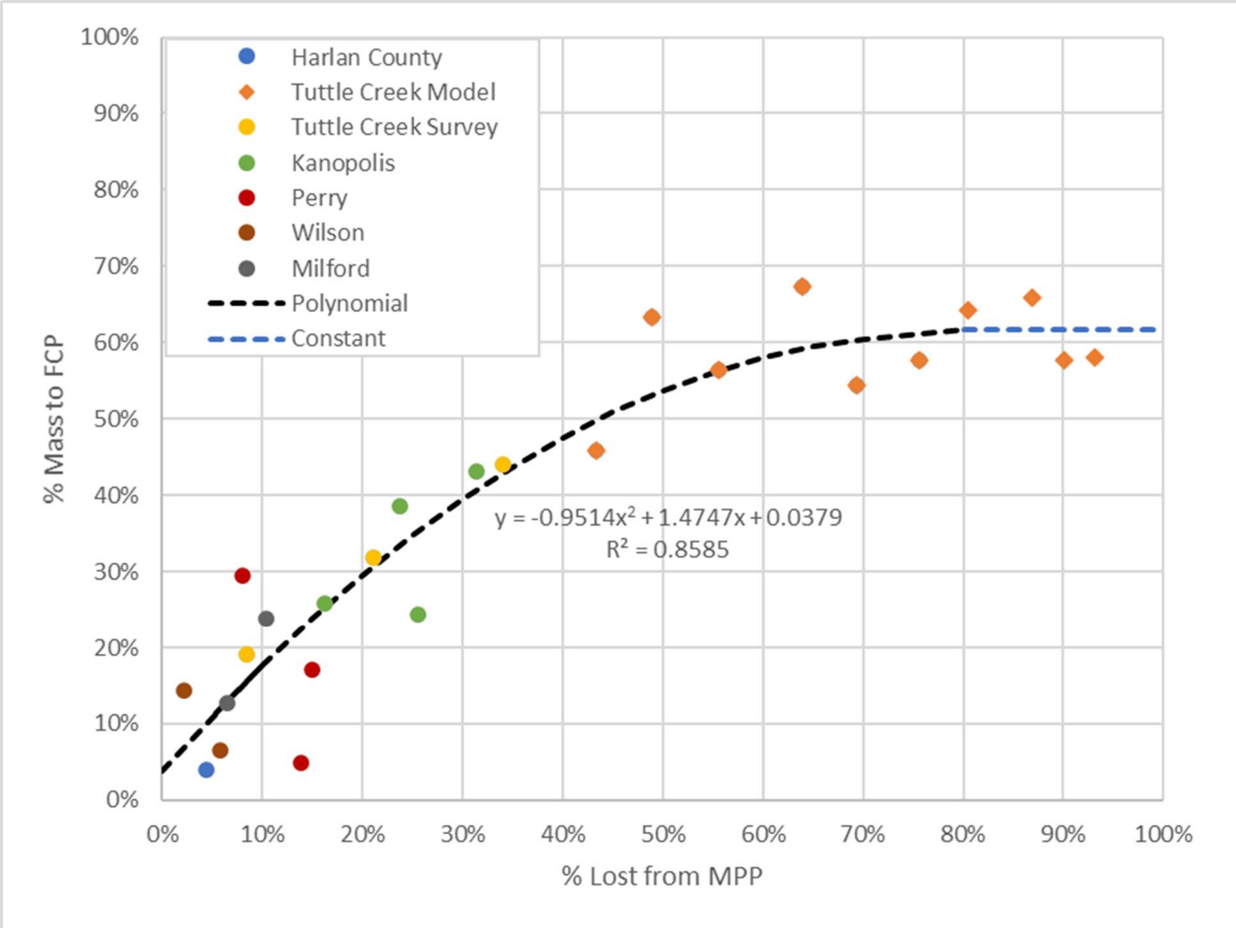


Figure 2-1. The percentage of mass depositing in the FP vs. the percent of volume lost from the MPP

From the 2000 survey, the multipurpose pool capacity can be estimated at 314,111 ac-ft. The mean annual water inflow into the lake is 129,743 acre-feet, based on stream gage data from 1970 to 2019. Brune offers three curves shown in Figure 2-2 for estimating trapping efficiency (Brune, 1953). These can be estimated using Equations 1 and 2 and the constants in Table 2-1. The medium curve was used for this analysis.

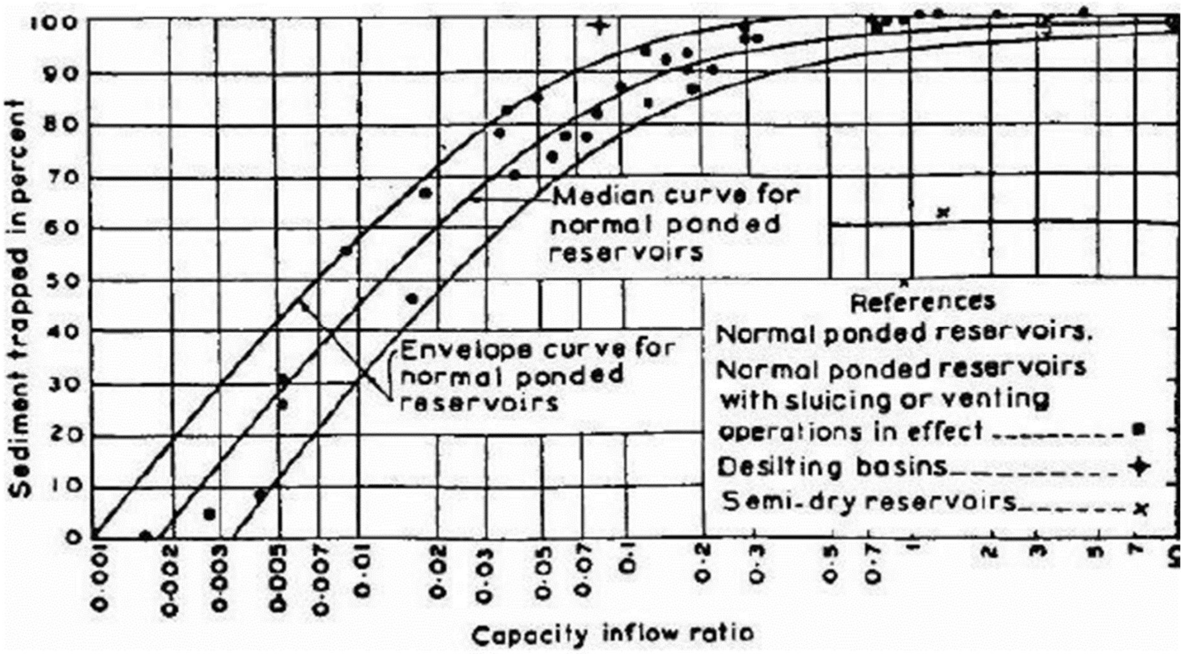


Figure 2-2: Brune Curves, (Brune, 1953)

$$TE = a[1 - 2e^{-bV_*^{0.35}}] \quad (1)$$

$$V_* = \frac{V_{res}}{V_{inflow}} \quad (2)$$

Table 2-1: Constants to be used in Equations 1 and 2

Constant	Low	Medium	High
a	95	97	100
b	5.37	6.42	7.71

The Brune Curve estimate for Harlan County Lake in 2000 is 96.97%, but as the lake fills in, the trapping efficiency of the multipurpose pool decreases. Table 2-2 lists the MPP trapping efficiency at year 0, 25, 50, and 100. Because the capacity of the multipurpose pool is so large compared to the sediment deposition, trapping efficiency does not decrease significantly.

Table 2-2: Trapping Efficiency over Time

Year	Cumulative Incoming Load (tons)	Deposited in the FCP (tons)	Passed to the MPP (tons)	MPP Final TE (%)
0	5,789,606	1,014,847	4,774,760	96.89%
25	12,566,272	2,330,203	10,236,069	96.88%
50	16,667,928	3,189,741	13,478,188	96.88%
100	29,339,265	6,132,834	23,206,431	96.87%

As the lake shrinks, the trapping efficiency decreases. The volume that deposits over time is only subdivided into the MPP or the FCP (not into discreet elevations within each pool). To make this scheme workable but still incorporate shrinking pools over time, two simplifying assumptions were incorporated. (1) The volume of the multi-purpose pool is used to compute the trapping efficiency, independent of whether the pool elevation is above or below the MPP level on any particular day. (2) The mass is apportioned based on the ratios given in Figure 2-1.

This list summarizes the steps to compute future sedimentation volumes and remaining pool capacities:

1. For each day, compute the mass of incoming sediment using the calibrated rating curve.
2. Compute the percentage of the mass that deposits in the FCP. The remaining mass is passed to the MPP. As previously discussed, the original FWOP calculations assumed this ratio remained constant.
3. Use the Brune Curve to compute the MPP trapping efficiency.
4. Apply the trapping efficiency to mass entering the MPP to compute trapped mass.
5. Use the bulk density of the FCP and MPP to covert the mass trapped into a volume trapped.
6. Subtract the volume of deposition in the FCP and MPP from the remaining capacities.
7. Repeat for each subsequent day, re-computing trapping efficiency with the progressively smaller MPP capacity.

Figure 2-3 provides the remaining storage capacities of the multipurpose and flood control pools for both the original and final FWOP computations. Table 2-3 and Table 2-4 summarize the final results. As discussed in Appendix D1.7, the official pool volumes from the capacity tables likely do not reflect the actual sediment deposition and volume change within the lake. However, for the FWOP it was assumed that the pool volumes given in the capacity tables are correct. This likely underestimates the deposition from the original survey to the most recent.

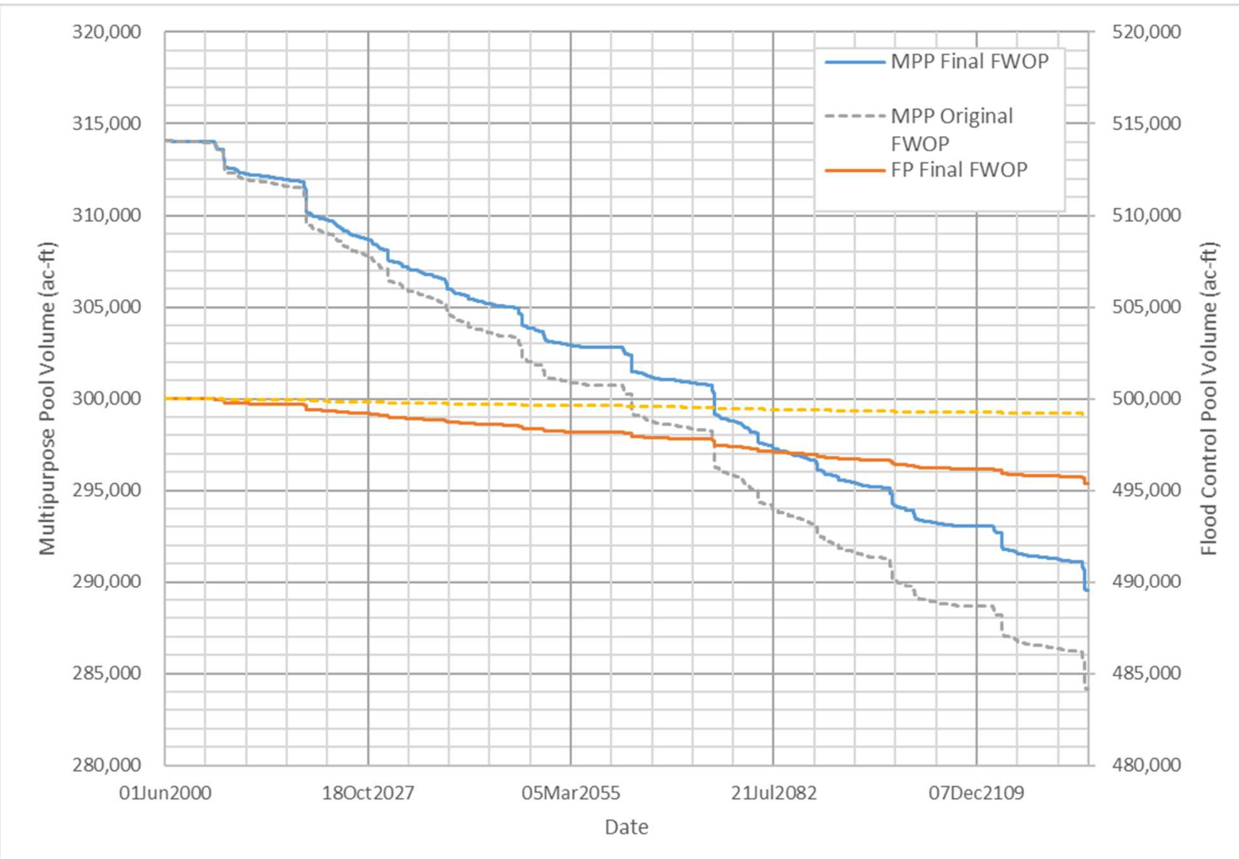


Figure 2-3: Remaining Pool Volumes Over Time

Table 2-3: Summary of Remaining Pool Volumes

Year	Multi-Purpose Pool (ac-ft)	Multi-Purpose Pool % of Original Volume	Flood Control Pool Volume, (ac-ft)	Flood Control Pool % of Original Volume
1952	346,512	100.00%	503,488	100.00%
2024	309,372	89.28%	499,294	99.17%
2049	303,952	87.72%	498,379	98.99%
2074	300,735	86.79%	497,781	98.87%
2124	291,081	84.00%	495,734	98.46%

Table 2-4: Sediment Deposition Volumes

Year	2000	2024	2049	2074	2124
Cumulative Deposition Multi-Purpose Pool (ac-ft)	32,401	37,140	42,560	45,777	55,431
Average Annual MPP Deposition over Increment (ac-ft/yr)	680	201	217	129	193
Average Annual MPP Deposition since 2024 (ac-ft/yr)	-	-	217	173	183
Cumulative Deposition Flood Control Pool (ac-ft)	3,488	4,194	5,109	5,707	7,754
Average Annual FCP Deposition over Increment (ac-ft/yr)	73	30	37	24	41
Average Annual FCP Deposition since 2024 (ac-ft/yr)	-	-	37	30	36

Table 2-5 gives the storage volumes from the original iteration of the FWOP computations along with the difference from the final results. As seen in Table 2-5, the MPP has greater storage over the FWOP for the final results, while the FP has less storage.

Table 2-5: Pool volumes in acre-feet from the original FWOP computations, and the differences from the final computations

Year	Multi-Purpose Pool Original	Flood Control Pool Original	Difference Multipurpose Pool	Difference Flood Control Pool
2024	308,593	499,840	+779	-546
2049	302,136	499,652	+1,816	-1,273
2074	298,227	499,539	+2,507	-1,758
2124	286,154	499,188	+4,927	-3,455

3.0 DELTA PROGRESSION

As listed in Appendix D1.7, the delta progression rate towards the dam from 1962 to 2000 was 98.9 feet per year. As deltas progress into wider and deeper sections of the lake, the rate of progression is expected to slow. The future rate was estimated by finding the delta progression that would yield the correct volume of sediment deposition as listed in Table 2-4. This delta progression analysis provides approximate results based on applying MPP deposition as an even veneer of deposition over all areas under the multipurpose pool elevation. Projected digital elevation models (DEM), surface area, and elevation contours of the MPP were computed using the following steps in ArcGIS.

1. Divide the projected deposition volume by the surface area of the MPP in 2009 to obtain the elevation change within the MPP.
2. Add this elevation to the 2009 DEM of the MPP to obtain a first estimate of the projected DEM.
3. Use the surface volume tool to estimate the new MPP volume and surface area.
4. Determine the difference between the predicted MPP volume and the MPP from the modified DEM
5. Divide the volume difference by the new surface area to obtain an additional elevation increase.
6. Add the additional elevation increase to the modified DEM
7. Repeat steps 3-7 until the extra volume above the MPP is less than the average annual deposition.

The 25, 50, and 100 year projections indicate delta progression of 230, 440, and 879 ft. This equates to a loss in multipurpose pool surface area, as listed in Table 3-1.

Table 3-1: Multipurpose Pool Surface Area

Year	MPP Surface Area (ac)	Lost Surface Area compared to Original (ac)	Lost Surface Area Compared to 2024 (ac)
1952	13,758	-	-
2000	12,980	778	-
2024	12,934	824	-
2049	12,681	1,077	253
2074	12,567	1,191	367
2124	12,392	1,366	542

Figure 3-1 shows the historic and projected MPP elevation contours. The original MPP contour was digitized from a historic map of the lake.

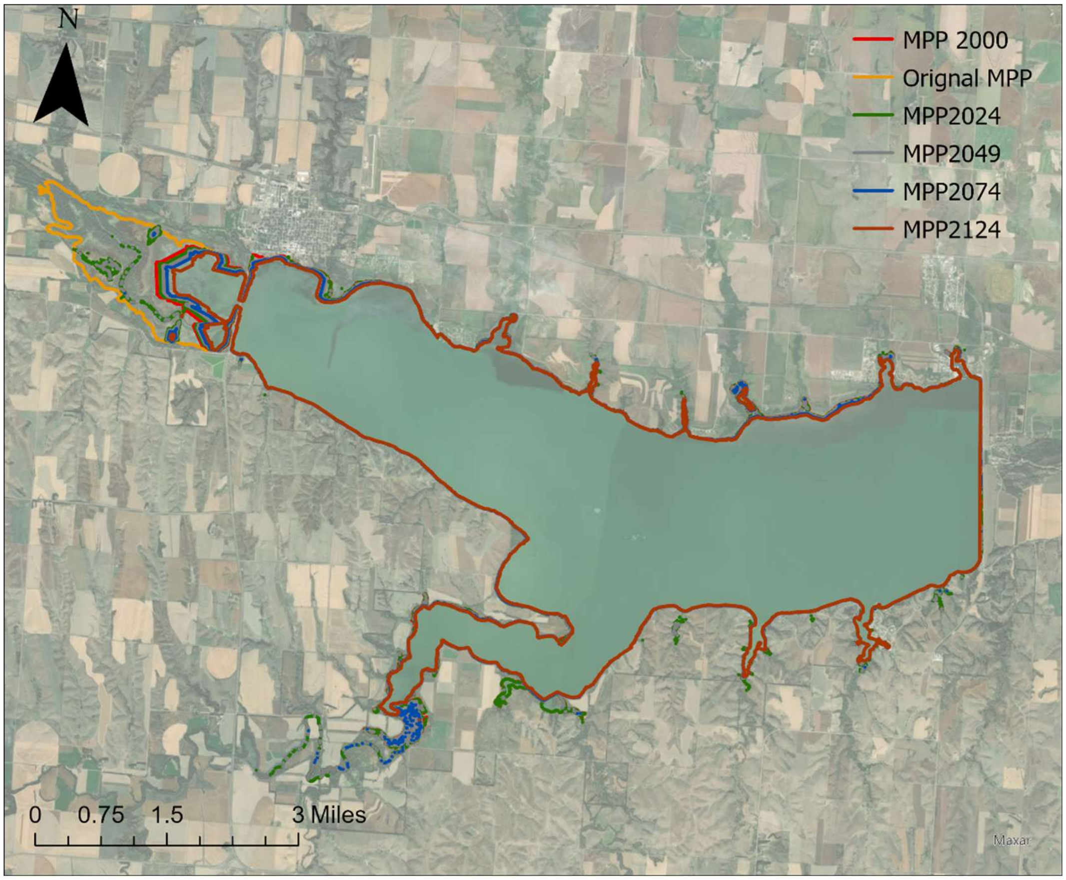


Figure 3-1: Historic and Projected MPP elevation contour

Figure 3-2 depicts the historic, current bottom elevations along a centerline of the lake. The projected profiles were extracted from the projected DEMs.

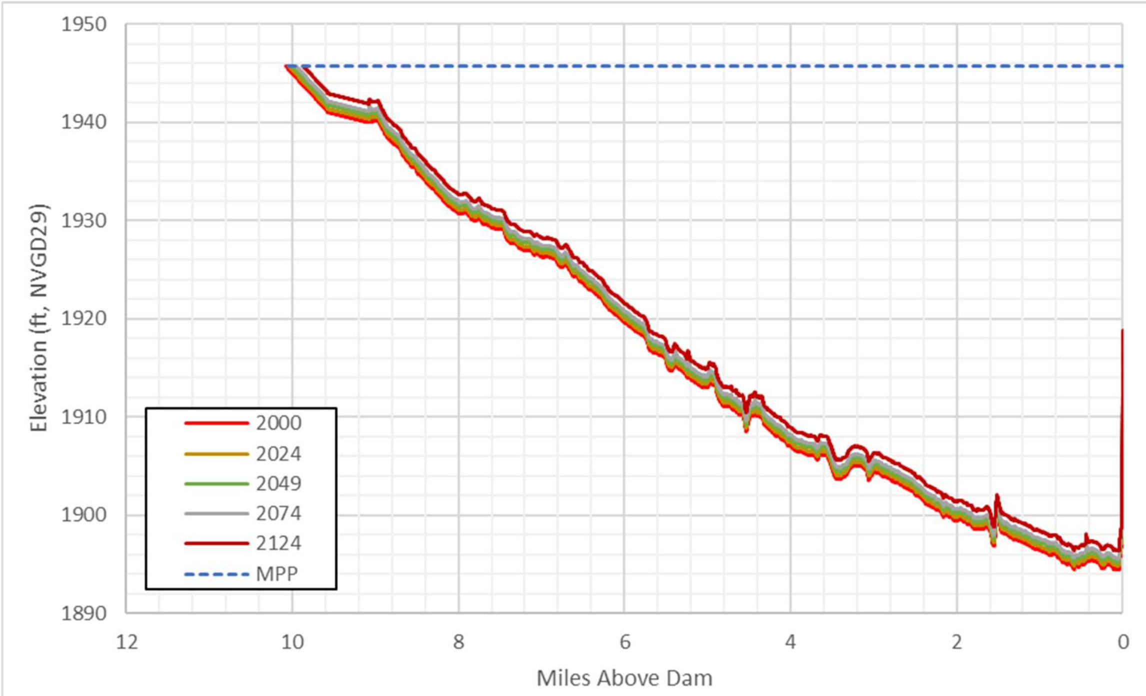


Figure 3-2: Projected Delta Location. Include lines for locations of important features.

4.0 DOWNSTREAM SEDIMENT CONCENTRATIONS

Because the trapping efficiency of Harlan County Lake only decreases slightly (0.02%), the sediment concentration of the water leaving the dam is expected to remain essentially the same.

5.0 DOWNSTREAM CHANNEL

The existing conditions report demonstrated bed degradation and bank erosion downstream from Harlan County Lake. Over the next 100 years, an additional 4,590,498 tons of sand will be trapped in Harlan Lake which will further induce bed degradation and bank erosion on the Republican River. Geomorphic principles suggest that rivers adjust to transport the bed material load supplied to them. Thus, the downstream channel will degrade (which lowers the slope and hence the applied shear stress) and an amour layer will form (which increases the critical shear required to initiate motion) until the flow from the dam is no longer able to transport sediment. Sedimentation modeling beyond the scope of this watershed study is required to compute the final bed elevations and to estimate bank erosion at the end of 25, 50, and 100 years. While applying trendlines to degradation or bank erosion is not ideal, it can be used for a first approximation. Herein is provided an order-of-magnitude approximation based on assumptions that geomorphic feedbacks do not change the rates of geomorphic change. This approximation was computed via the following steps:

1. Compute the ratio of sand trapped in the lake (tons) to volumetric degradation downstream (CY) over a defined distance. For Harlan County Lake, 748,135 CY of sand trapped by the dam induced 312,987 CY of bed and bank erosion in the 12 miles downstream of the dam, or a ratio of 0.42 CY downstream degradation/CY trapped.
2. Apply the ratio to the future estimates for trapped sand.

Table 5-1 provides the estimated degradation volume in the 12 miles below Harlan County Dam at the end of 0, 25, 50, and 100 years. This analysis provides a high-level assessment suitable for relative ranking of alternatives.

Table 5-1: Degradation Downstream of the Harlan County Dam Compared to latest degradation survey in April 2015

Year	Trapped Sand (CY)	Degradation Since Last Survey (ac-ft)	Degradation Since 2024 (ac-ft)
0	509,761	132	-
25	1,563,096	405	273
50	2,198,774	570	438
100	4,166,072	1,080	948

6.0 SUMMARY, LIMITATIONS, AND CONCLUSIONS

The following list summarizes the 50-year FWOP condition at Harlan County Lake. (For the 25 years and 100 years projections, see the body of the report.) The list summarizes the main points:

- 8,757,767 tons of fine sediment (silt/clay) and 2,120,555tons of coarse sediment (sand/gravel) are expected to enter Harlan County Lake, of which 2,174,894tons are trapped in the flood control pool,8,431,991tons are trapped in the multi-purpose pool, and 271,437tons pass downstream.
- The flood control pool (FCP) and multi-purpose pool (MPP) shrinks by an additional 1,513 ac-ft and 8,637 ac-ft, respectively.
- At the end of 50 years, the MPP has 86.79% remaining of its original capacity if no pool rises are enacted.
- At the end of 50 years, the FCP has 98.87% remaining of its original capacity if no pool rises are enacted.
- At the end of 50 years, the coarse sediment delta progresses downstream an additional 440 feet, which shrinks the surface area of the lake to 91% of the original.
- The fine sediment concentrations of releases from Harlan County Dam will remain essentially unchanged.
- The downstream channel will experience additional bed degradation and bank erosion, with 438 ac-ft degrading from the lower 12 miles.

These findings are based on trendline projections and empirical equations. For more precise estimations, particularly of where in the pools the sediment will deposit and how the downstream channels will adjust over time, numerical modeling is recommended. Moreover, this analysis did not compute damages due to upstream delta migration and only approximated downstream degradation as a total volume, not as bank erosion or bed degradation at specific affected infrastructure.

Notwithstanding the limitations, these projections indicate that overall sediment accumulation in Harlan County Lake will be a minor problem over the next 50 years with some implications for loss in benefits, environmental harm, infrastructure damage, and increased O&M needs. O&M issues related to localized sediment sources and deposition were not addressed. From years 50 to 100, sediment trapping and loss of benefits will likely continue at similar rates as years 0 to 50.

US Army Corps
of Engineers ®
Kansas City District

Kansas River Reservoirs Flood and Sediment Study

Appendix D3: U.S. Bureau of Reclamation Lakes Existing Conditions and Future Without Project Sedimentation

November 2022

Contents

1.0 USBR LAKES INTRODUCTION

Eleven Bureau of Reclamation (USBR) lakes are located within the Kansas River Basin as shown in Figure 1-1. Table 1-1 summarizes key information about each reservoir. Agricultural cropland and grazing land constitute the major land uses in each of the reservoirs' watersheds. Also, groundwater pumping and irrigation is significant within many of the lakes' watersheds and has contributed to declining inflows to several of the lakes. This has led to many of the lakes being below the multipurpose pool (MPP) elevation for the majority of the time. Bonny Lake's MPP was permanently drained in 2011, so its sedimentation was not estimated in this report.

Many of the watersheds contain a significant amount of non-contributing area that likely does not contribute any water or sediment to the lakes. Approximate estimates of these areas have been made by other studies in the Kansas City District (NWK) and are shown in Figure 1-1. The majority of the non-contributing area is in the far northwestern portion of the basin where rainfall is lowest. In Table 1-1, "unregulated" refers to the portion of the watershed that is not upstream from a USACE or Bureau of Reclamation dam. Smaller state, city, and county dams also impound water, but their sediment effects are small.

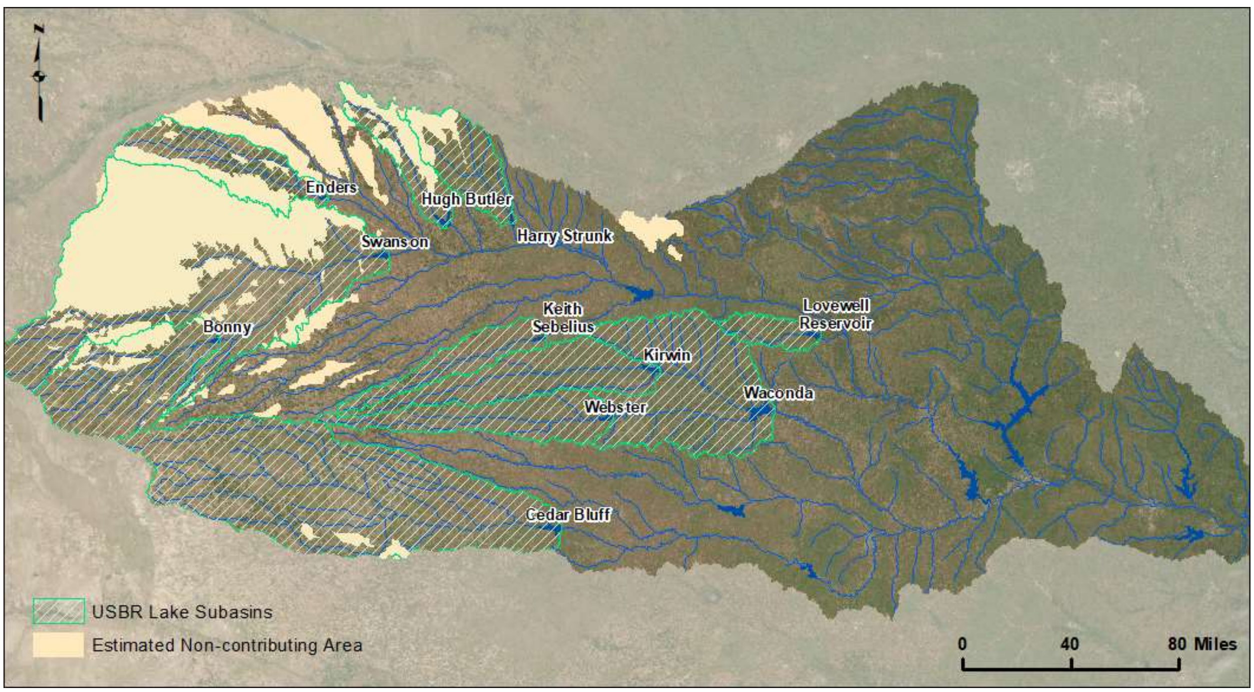


Figure 1-1. Overall Kansas River Basin Map with USBR Lakes.

Table 1-1. Kansas River Basin USBR Lakes (Listed from East to West).

Lake Name	Main Tributary	Date Closed	Total Drainage Area (sq mi)	Unregulated Drainage Area (sq mi)
Lovewell	White Rock Cr	5/29/1957	345	345
Waconda	NF & SF Solomon R	10/1967	5076	2559
Kirwin	NF Solomon R	3/7/1955	1367	1367
Webster	SF Solomon R	5/3/1956	1150	1150
Cedar Bluff	Smoky Hill R	11/1950	5530	5530
Keith Sebelius	Prairie Dog Cr	10/1964	683	683
Harry Strunk	Medicine Cr	8/1949	880	642
Hugh Butler	Red Willow Cr	9/5/1961	730	730
Swanson	Republican R	5/4/1953	8620	2112
Enders	Frenchman Cr	10/23/1950	950	950
Bonny	SF Republican R	1951	1820	-

Each of the USBR dams serves multiple authorized purposes, including a significant capacity for flood control. Depending on the dam, pool capacity is also allotted for water supply, joint use, or conservation. These pool levels will be referred to as the multipurpose pool for the rest of this report. None of the USBR dams in the Kansas River Basin produce hydropower. Table 1-2 summarizes key elevations for the lakes.

Table 1-2. Key Pool and Infrastructure Elevations

Lake	Multipurpose Pool Elevation (ft)	Flood Control Pool Elevation (ft)	Spillway Elevation (ft)	Dam Elevation (ft)
Lovewell	1582.6	1595.3	1575.3	1616
Waconda	1455.6	1488.3	1467.4	1500
Kirwin	1729.25	1757.3	1757.3	1779
Webster	1892.45	1923.7	1884.6	1944
Cedar Bluff	2144	2166	2166	2198
Keith Sebelius	2304.3	2331.4	2296	2347
Harry Strunk	2366.1	2,386.2	2386.2	2415
Hugh Butler	2581.8	2,604.9	2604.9	2634
Swanson	2752	2773	2743	2793
Enders	3112.3	3127	3097	3137.5

Note: All elevations except Waconda Lake are in NGVD29. Waconda Lake Elevations are in a local datum which is 1.2 ft higher than NVGD29.

2.0 HISTORIC POOL VOLUMES AND SEDIMENTATION

Past sedimentation rates were estimated for each of the lakes by comparing the computed pool volume from multiple surveys. However, it was not possible to estimate the deposition in the flood pool with this method since the bathymetric surveys did not reach above the MPP. Another method discussed later in this report was used to estimate flood pool deposition for Waconda Lake; for the other lakes only the MPP sedimentation was estimated. Table 2-1 lists the MPP of each lake with the survey year, total deposition, and percentage lost. Table 2-2 provides the average annual rates of sediment deposition.

Table 2-1. Total Sediment Accumulation in Kansas River Basin Lakes between survey years.

Lake	MPP Initial Volume, (ac-ft)	Initial Survey Year	MPP Recent Volume, ac-ft Survey Year	Recent Survey Year	Volume Lost from MPP (ac-ft)	% of MPP Lost to Sediment
Lovewell	41,687	1957	34,888	2020	6,799	16.3%
Waconda	242,017	1967	219,420	2001	22,597	9.3%
Kirwin	99,432	1955	98,154	1996	1,278	1.3%
Webster	77,370	1956	76,103	1996	1,267	1.6%
Cedar Bluff	185,496	1950	172,452	2000	13,044	7.0%
Keith Sebelius	36,127	1964	34,510	2000	1,617	4.5%
Harry Strunk	41,120	1951	34,647	2006	6,473	15.7%
Hugh Butler	37,840	1997	36,224	1997	1,616	4.3%
Swanson	120,366	1953	110,175	2011	0,191	8.5%
Enders	44,482	1950	42,910	1997	1,572	3.5%

Table 2-2. Annual Sediment Accumulation in Kansas River Basin USBR Lakes

Lake	MPP Deposition Years	MPP Deposition (ac-ft/year)	% of MPP Lost to Sediment (%/year)
Lovewell	1995-2020	31	0.07%
Waconda	1967-2001	670	0.28%
Kirwin	1955-1996	31	0.03%
Webster	1956-1996	32	0.04%
Cedar Bluff	1950-2000	262	0.14%
Keith Sebelius	1964-2000	45	0.12%
Harry Strunk	1951-2006	118	0.29%
Hugh Butler	1961-1997	45	0.12%
Swanson	1953-2011	176	0.15%
Enders	1950-1997	34	0.08%

Figure 2-1-1 maps the average annual sediment accumulation per square mile of unregulated drainage area. As seen, the eastern watersheds generally produce much more sediment per square mile than the western watersheds, following the same trend as for the USACE lakes (Appendix D1). This is likely due to the difference in mean annual precipitation, which quadruples from Western Kansas to Eastern Kansas.

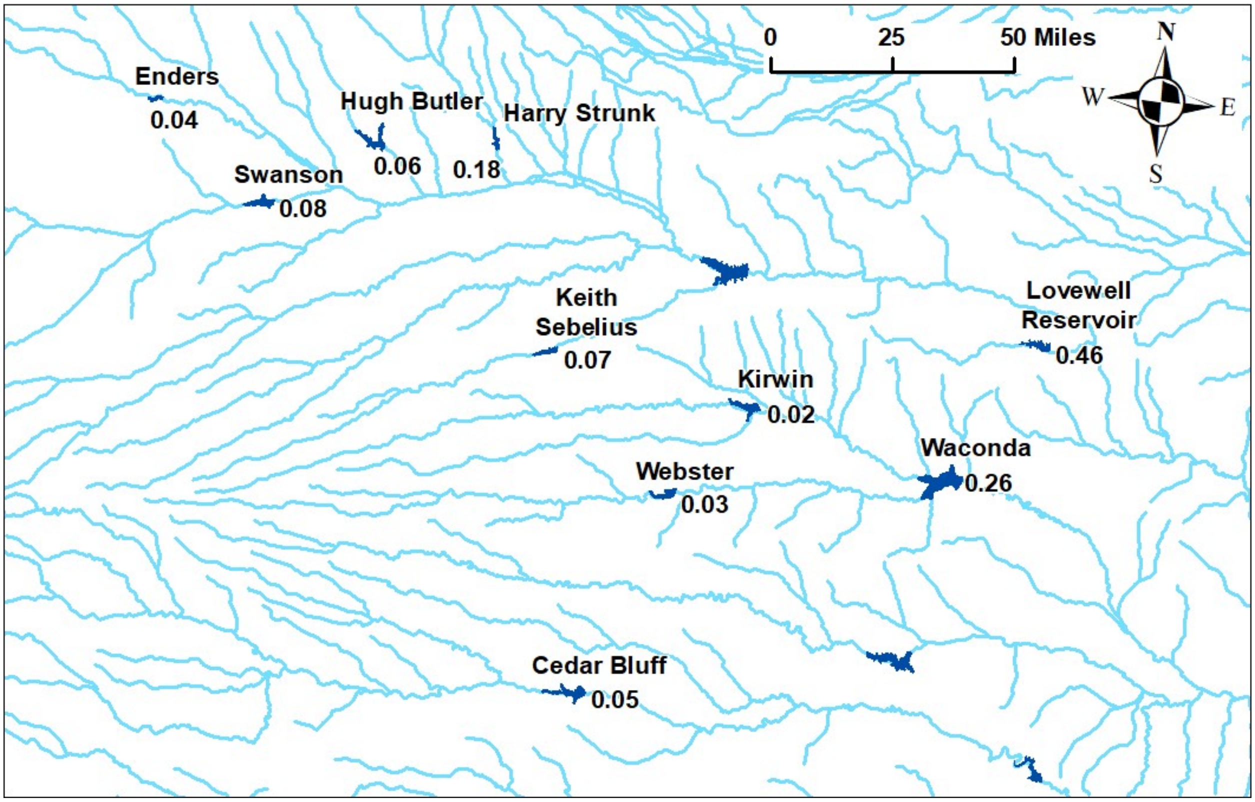


Figure 2-1. Average annual deposition per square mile of unregulated drainage area at USBR lakes. *Data in ac-ft/yr/mi^2.*

3.0 HISTORIC INFLOW TRENDS

Runoff of water and sediment has decreased for many areas within the Kansas River Basin in response to factors such as groundwater pumping and irrigation. This decrease was observed in the water inflow records for five of the USBR lakes, which are Cedar Bluff, Harry Strunk, Hugh Butler and Swanson.

Figures 3-1 through 3-7 show both the annual water inflow (acre-ft) and the annual peak flow (cfs). Annual inflow was either obtained from USBR reports or records in NWK databases. NWK has historically calculated daily inflows from the observed storage in the lake, outflows, and estimates for evaporation. Discharge data from USGS gages was also obtained if the daily inflow records did not cover the entire period of record. This data generally showed lower inflows than the NWK data, possibly because it does not account for inflows below the gages.

Figure 3-1 shows inflow into Cedar Bluff since construction of the dam. Both peak flow and annual inflow have declined significantly over time, mostly between 1951 and 1975.

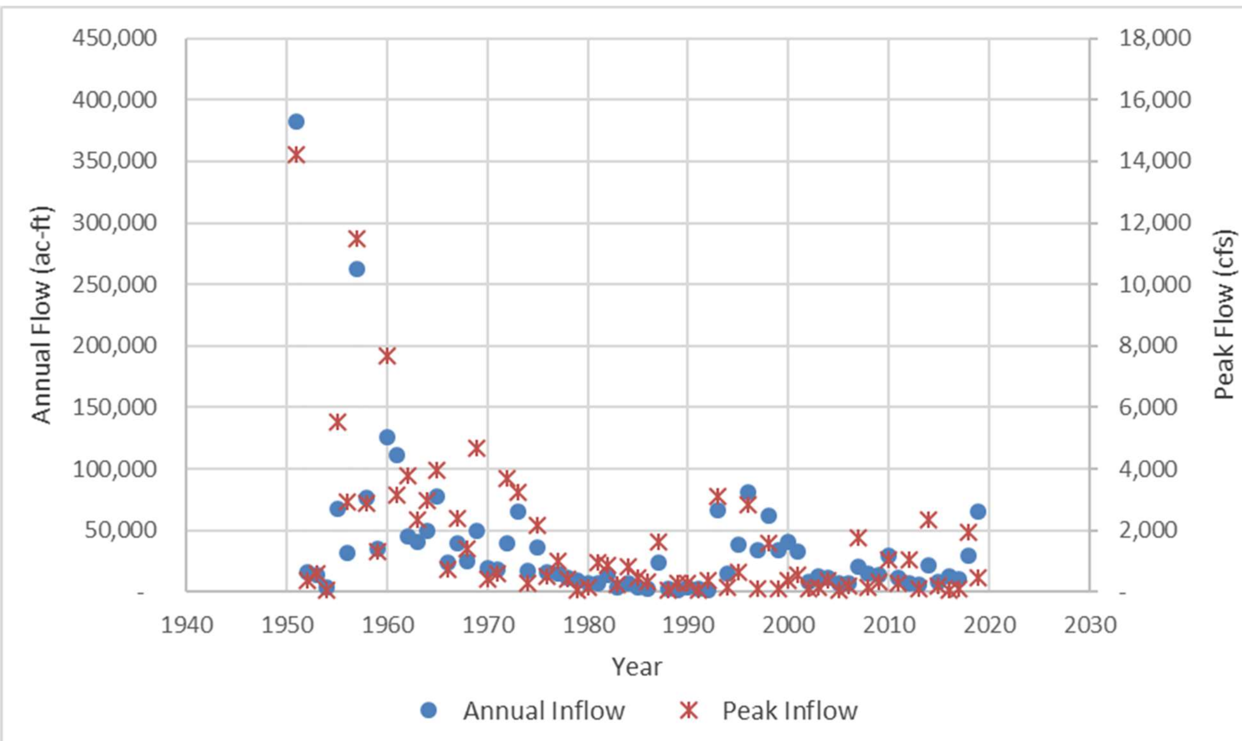


Figure 3-1: Cedar Bluff annual water inflow

Figure 3-2 shows the inflow for Harry Strunk Lake. Inflows appear to have decreased somewhat over time. However, recent years, beginning in 2007, have seen some reversal of this trend. USGS gage data is from the Medicine Creek gage above Harry Strunk Lake, Nebraska (06841000)

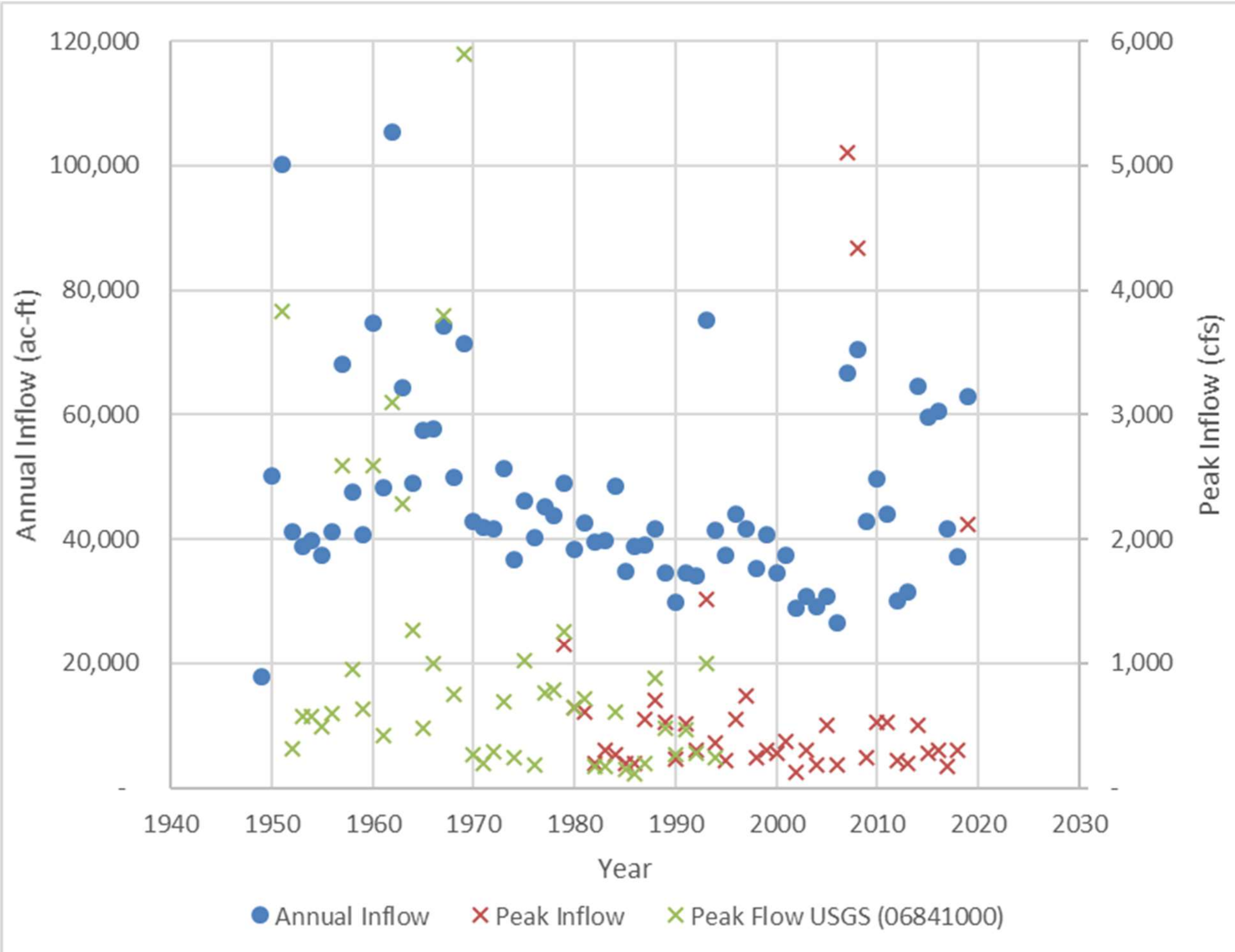


Figure 3-2: Harry Strunk annual water inflow

Figure 3-3 gives the water inflow into Hugh Butler Lake. There is a significant decrease of annual inflow over time. However, peak inflow has not decreased as significantly, as demonstrated by the 2007 event which greatly exceeded any event since construction of the dam. USGS gage data is from the Red Willow Creek gage above Hugh Butler Lake, Nebraska (06837300)

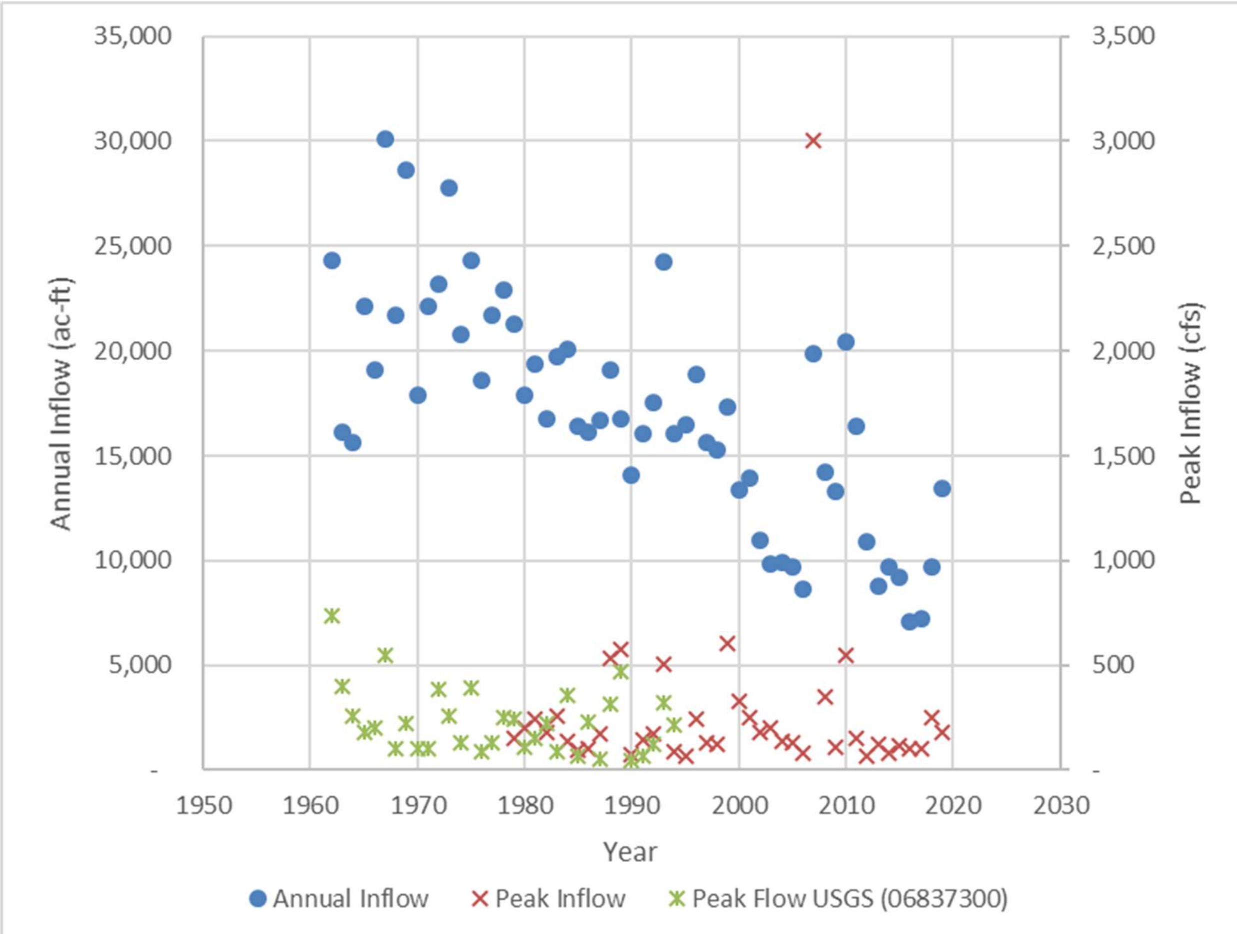


Figure 3-3: Hugh Butler annual water inflow.

Figure 3-4 shows the historic inflows for Swanson Lake. Inflows have decreased over time, reaching a minimum between 2002 and 2008. USGS gage data is from the Republican River gage at Stratton, Nebraska (06828500)

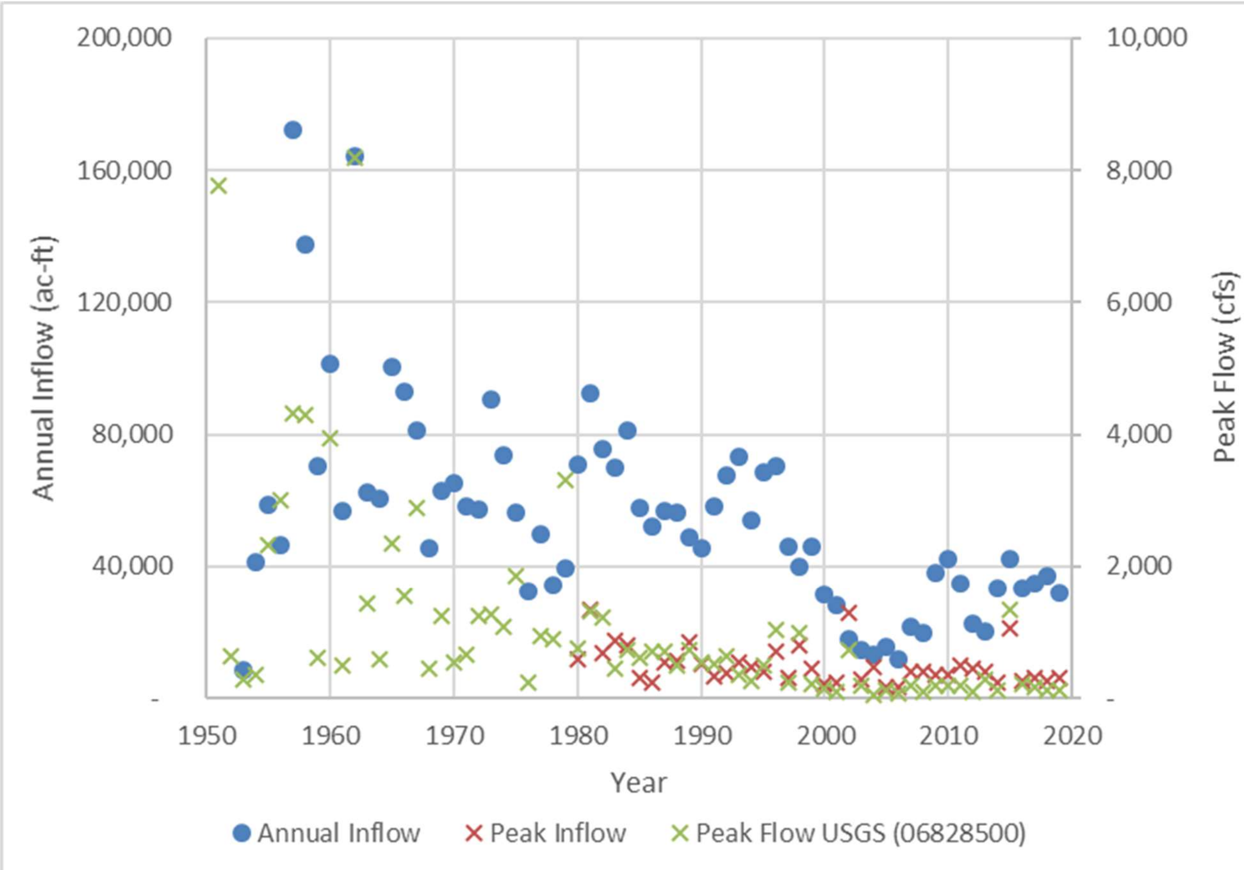


Figure 3-4: Swanson annual water inflow

Figure 3-5 gives the available inflow data for Enders Dam. Annual inflow into Enders has decreased the most for any of the USBR lakes, although it appears to have reached a steady state since about 2002. Peak inflows have also decreased over time, although extreme events can still occur as demonstrated by the 2007 event. USGS gage data is from the Frenchman Creek gage near Imperial, Nebraska (06831500).

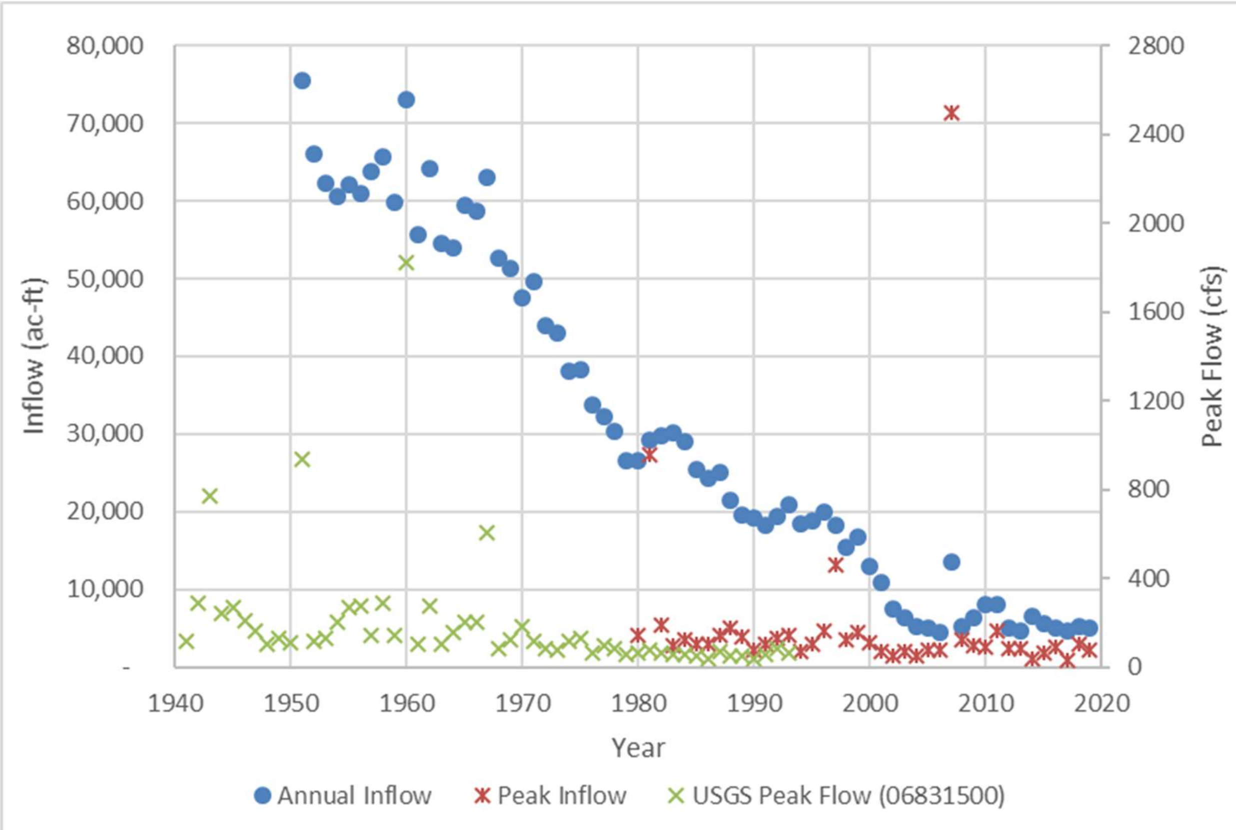


Figure 3-5: Enders annual water inflow.

4.0 FWOP PROJECTIONS

The following sections document the projections of the Future Without Project (FWOP) condition for the USBR lakes. The FWOP condition is the expected conditions of the lake if sediment accumulation continues unabated, i.e., without intentional sediment management or removal. This analysis also assumes no pool raise or reallocation takes place, as such actions would be potential measures for a Future With Project condition. This analysis incorporates the trends documented in the Existing Conditions analysis discussed in the previous sections. This was generally done by assuming that the observed annual deposition rate remained constant over the study period with a correction factor applied to account for declining inflows. The correction factor is based on the ratio of average annual sediment inflows during historic vs. current conditions computed using flow/load rating curves. A second correction factor was applied at three of the lakes to account for declining trapping efficiency in the lake.

4.1 Project MPP Deposition

The FWOP multipurpose pool volumes were computed assuming a constant yearly deposition throughout the 100-year study period. This yearly deposition was generally determined by subtracting the most recent surveyed volume from the initial volume. However, if a lake had multiple surveys and experienced a decline of inflow, a later survey than the initial was used. Also, correction factors for declining trapping efficiency and inflows, discussed in the following sections, were applied to reduce the yearly deposition. The computations were begun at the most recent survey and continued until the year 2124.

The volume of the MPP reported by the USBR survey reports was used as the starting volume for all the lakes except for Waconda and Harry Strunk. For these lakes, the volume of the MPP was computed in GIS using surfaces created from the bathymetric survey points combined with LiDAR collected in 2010. These surfaces showed a lower MPP volume than given in the USBR reports. This is because the USBR volume computations assumed the original survey elevation for areas above the most recent survey. This likely biased the MPP volume and annual sediment deposition low. To maintain consistency with the lakes and because the LiDAR was collected at a later date than the surveys, the USBR values were used for determining the annual deposition. However, the final MPP volumes for these two lakes were shifted so that the volumes would match the GIS surfaces.

4.2 Correction for Declining Sediment Inflows

Declining sediment loads have likely accompanied the declining inflows previously discussed for five of the USBR lakes. Several of the lakes have three or more bathymetric surveys which demonstrate declining sediment loads. Also, paired discharge and concentration measurements have been collected upstream of Cedar Bluff, Swanson, and Harry Strunk Lakes, from which flow load rating curves were created. These rating curves were then used to compute the daily inflow of sediment and ratio representing the reduction in sediment load.

Figures 4-1 through 4-3 show the sediment rating curves developed for Cedar Bluff, Swanson, and Harry Strunk Lakes. Sediment measurements were not available for either Enders or Hugh Butler, so the Harry Strunk rating curve was used for these lakes. The drainage areas for these three lakes are similar, as shown in Table 1-1, and are located near one other. While this introduces additional uncertainty into the calculations, it is likely an adequate assumption since the actual magnitude of sediment is not needed, only the relative load over time.

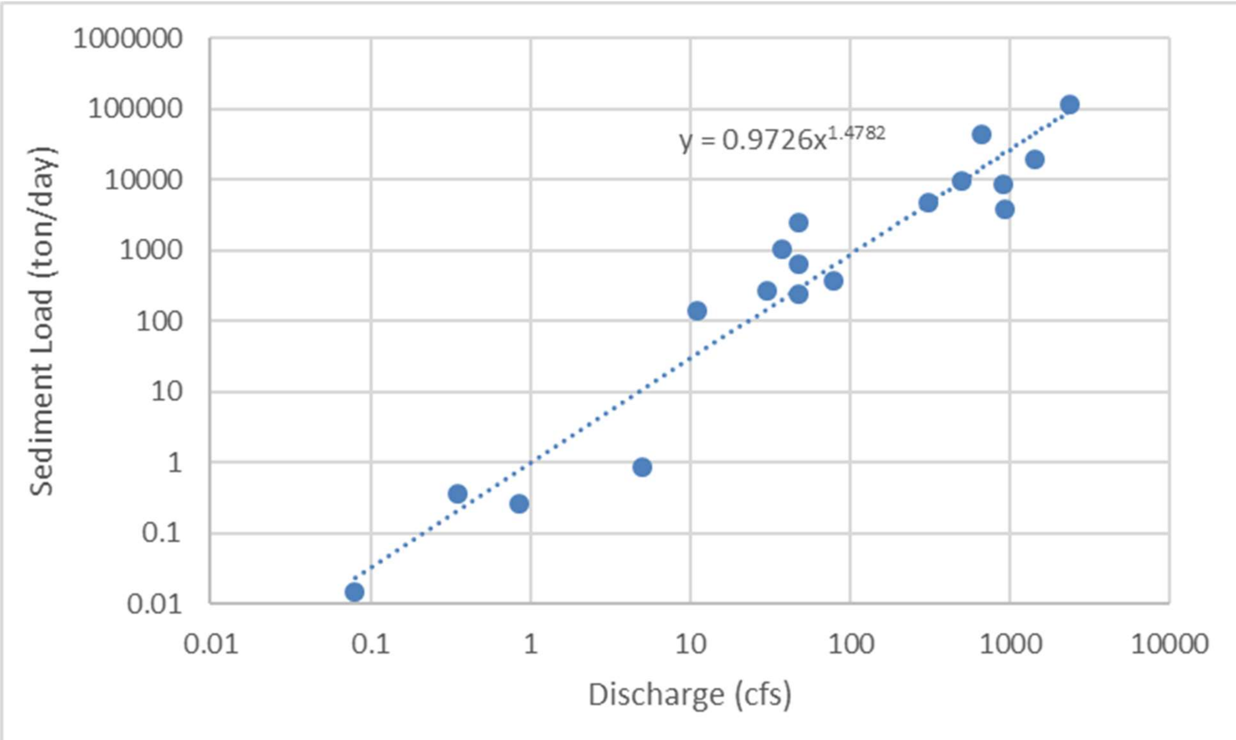


Figure 4-1: Cedar Bluff sediment rating curve.

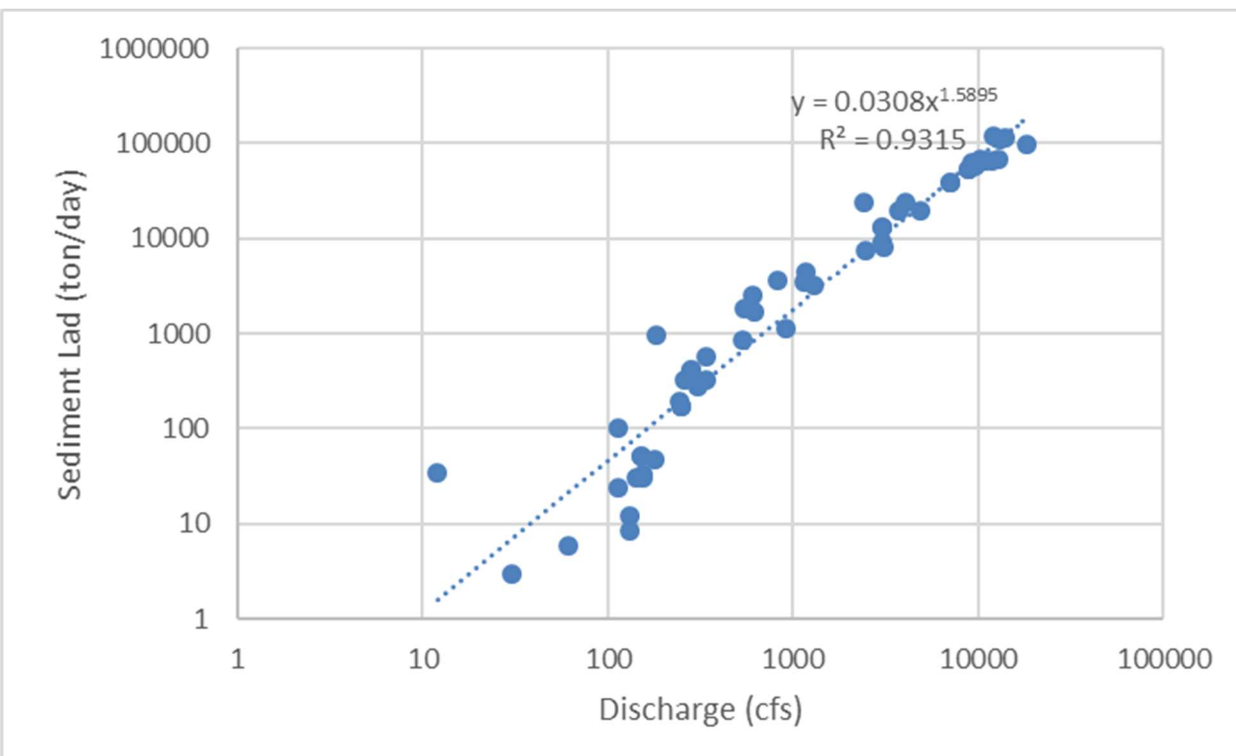


Figure 4-2: Swanson sediment rating curve.

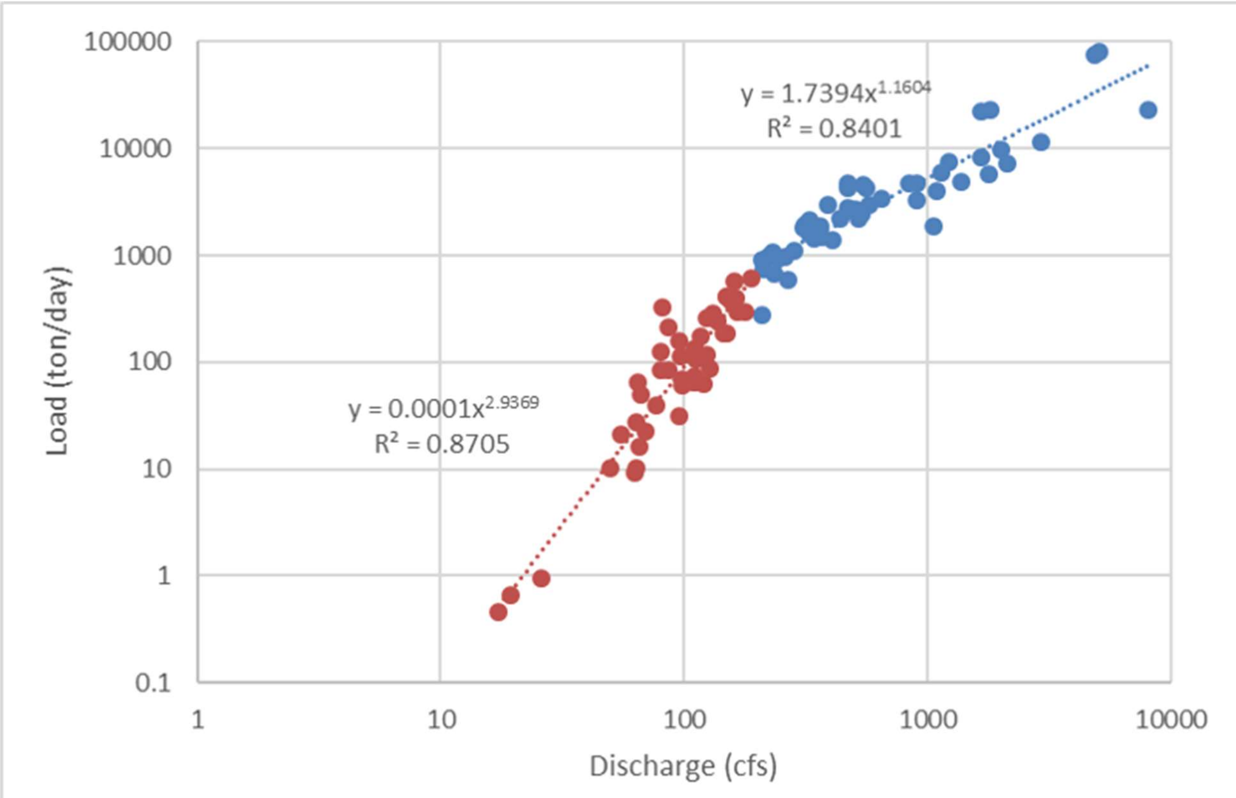


Figure 4-3: Harry Strunk sediment rating curve.

Daily sediment load was calculated using the above rating curves and daily inflow. The daily load was then summed to obtain the annual sediment load. Figures 11 through 15 show the annual sediment load calculated for the five lakes with declining inflows. For Harry Strunk, Hugh Butler, and Enders, daily inflow was only available from the upstream USGS gages for portions of the period of record, while the NWK daily inflows made up the rest of the period of record. Since the datasets overlapped for many of the years, a correction factor consisting of the ratio of the total load computed from the two datasets was determined to make the two datasets comparable.

Figure 4-4 gives the annual sediment load for Cedar Bluff. A large decline in sediment load was observed from the 1950s to about 1975; however, the sediment load appears to have remained fairly steady since. Bathymetric surveys were collected in November 1950 and September 2000. The average annual sediment load between the surveys was estimated to be 845,173 tons from the rating curve. Between 1980 and 2019, after the load was assumed to be no longer declining, the average annual load was 166,758 tons. A correction factor of 0.20 was determined by dividing the 1951-200 average by the 1980-2019 average. A correction factor of 0.12 was determined for the 2000-2019 period using the same method.

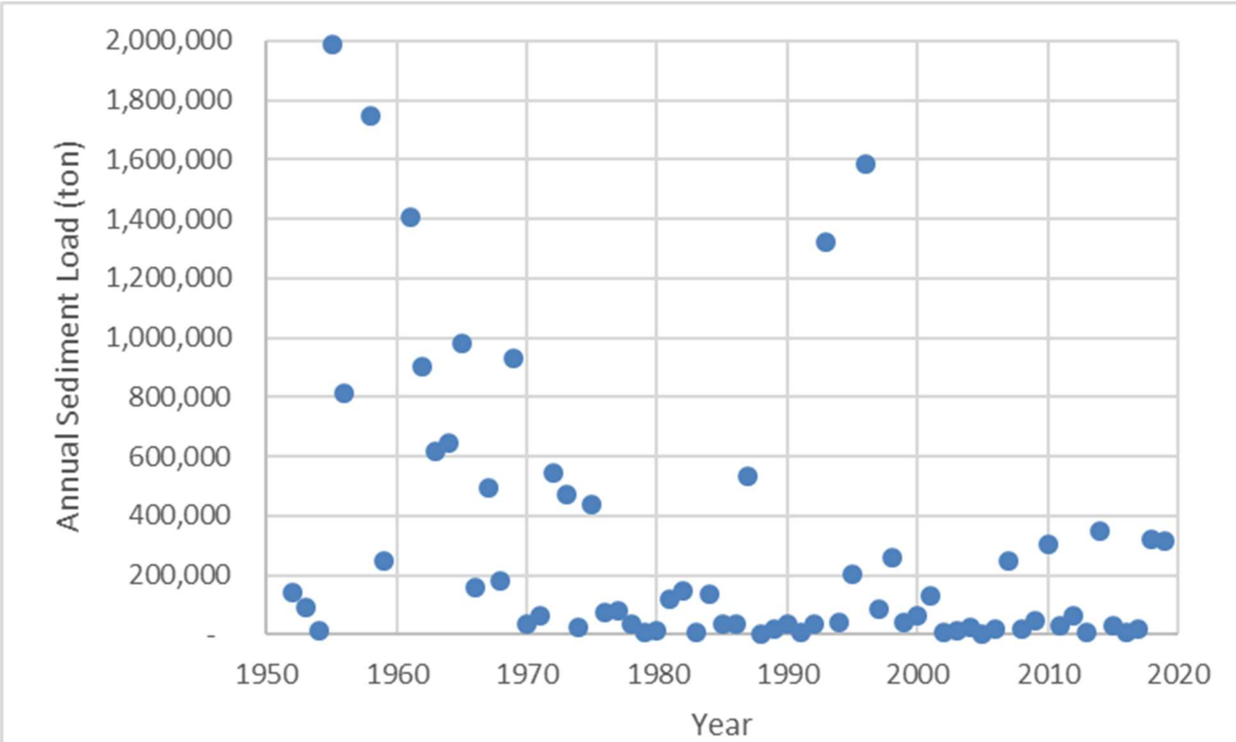


Figure 4-4: Cedar Bluff annual sediment load.

Figure 4-5 shows the annual sediment load for Harry Strunk Lake, with sediment loads gradually declining between the 1960s and 2007. However, recent years show an increased sediment load. A total of five surveys have been collected at Harry Strunk between 1949 and 2006. An annual deposition of 69 ac-ft/year, was calculated from the 1963 and 2006 surveys. This was increased by a factor of 1.46 to make projections for the period between 2007 and 2019, based on the higher sediment loads through this period. It was also assumed the 50-year time period between 1970 and 2019 would be more representative of future conditions, so annual deposition was decreased by a factor of 0.93 for the rest of the simulation period.

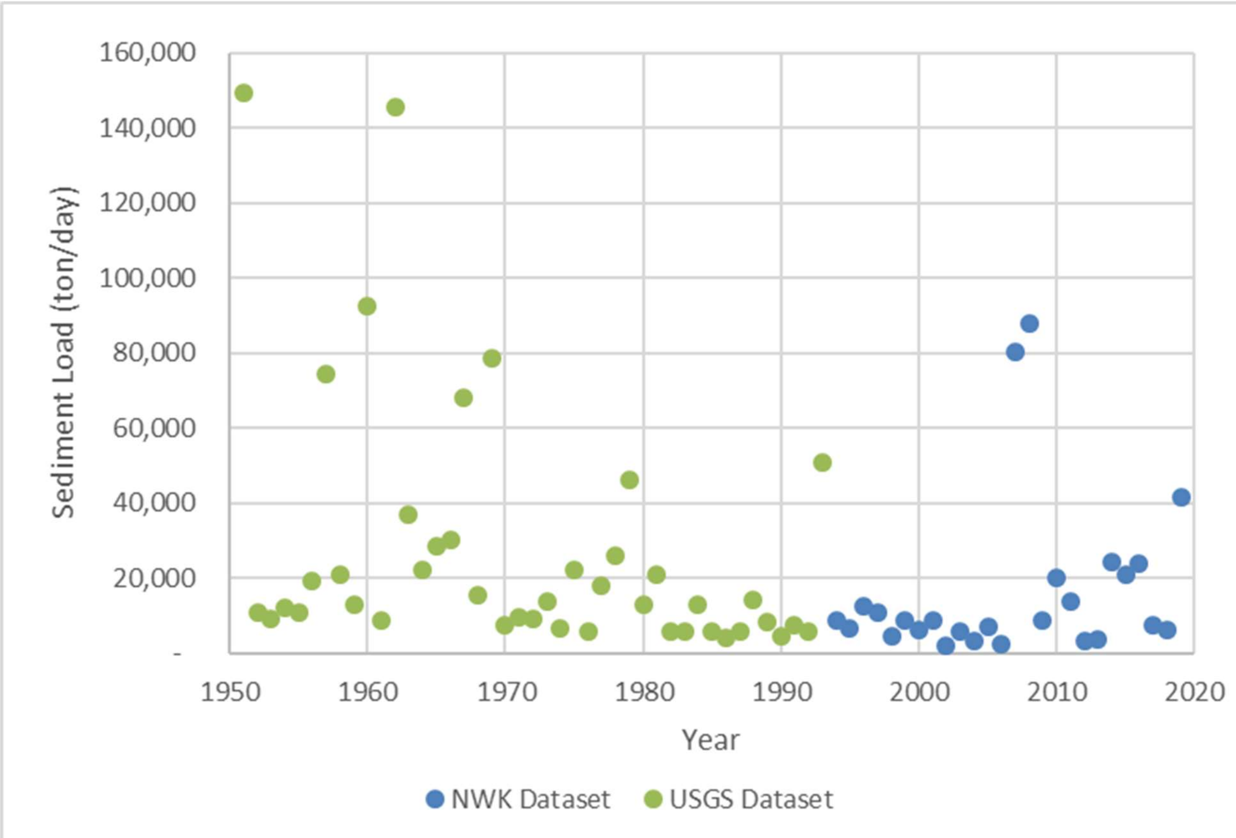


Figure 4-5: Harry Strunk annual sediment load.

The annual sediment load for Hugh Butler is given in Figure 4-6. Sediment loads have generally declined over time, except for 2007 due to the extreme flood event. Relative sediment runoff for this event is highly uncertain because this lake uses the Harry Strunk rating curve. However, it is possible that this event brought in as much sediment as the rest of the period from 2000-2019. Projections of sediment deposition in the lake will be very dependent on the reoccurrence frequency of such an event. For the purposes of this study, it was assumed that a similar event would reoccur once every 50 years through the 100-year projection. The period from 2000-2019 without 2007 was assumed to most representative of future conditions. Bathymetric surveys were collected in September 1961 and May 1997. Average annual sediment load was 5,307 tons from 1962 to 1997 and 5,307 tons and 2,206 from 2000 to 2019. Annual sediment load for 2007 was estimated to be 48,488 tons. Correction factors determined from these loads are shown in Table 4-1.

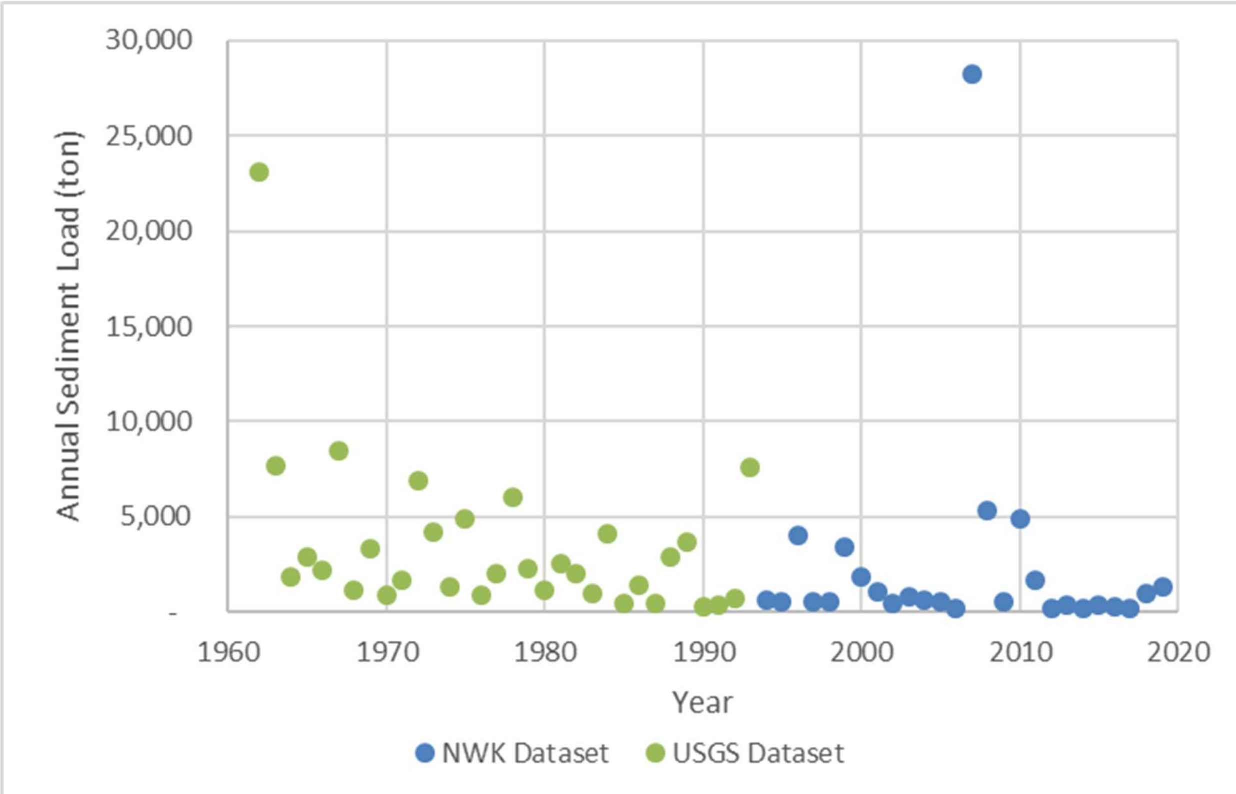


Figure 4-6: Hugh Butler annual sediment load.

Figure 4-7 gives the annual sediment load into Swanson Lake. Bathymetric surveys were collected at the lake in May 1953, May 1982, and May 2011. The 2011 and 1982 surveys were used to compute an average annual sediment load of 69 ac-ft per year. The average annual sediment load between 1982 and 2011 was computed to be 71,492 tons per year from the sediment rating curve. Annual sediment load between 1998 and 2019 was computed to be 9,107 tons and was assumed to be more representative for making the sediment projections. Correction factors determined from these loads are shown in Table 4-1.

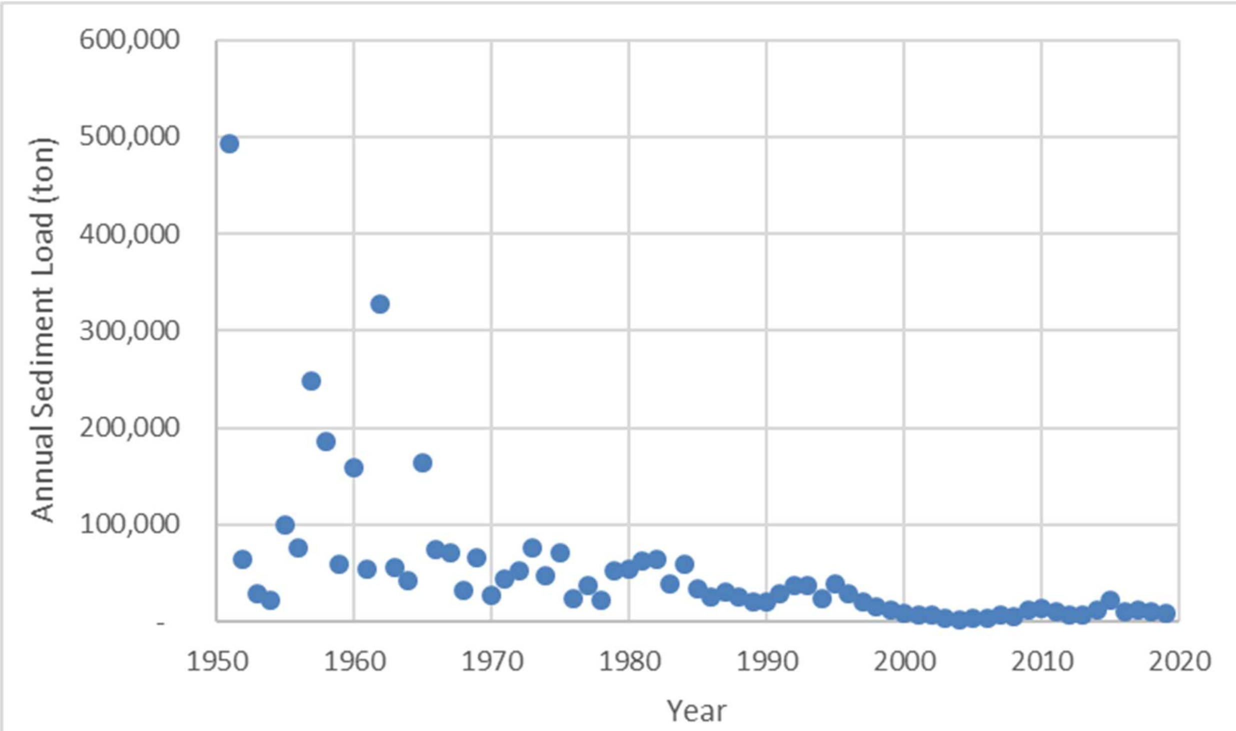


Figure 4-7: Swanson annual sediment load.

Figure 4-8 shows the annual sediment load into Enders Lake. Sediment loads have decreased significantly over time, most notably in the 1970s. The 2007 event was an extreme event that likely brought in a significant amount of sediment. Like the projections for Hugh Butler, the event was assumed to have a return period of 50-years. For the future projections, it was assumed that inflows between 2000 and 2019 would be most representative of future conditions. Bathymetric surveys have been collected in October 1950 and May 1997 and show an annual deposition of 34 ac-ft. Average annual sediment load was 14,220 tons from 1951 to 1997 and 577 tons from 2000-2019 excluding 2007. The annual sediment load for 2007 was 36,545 tons. Correction factors determined from these loads are shown in Table 4-1.

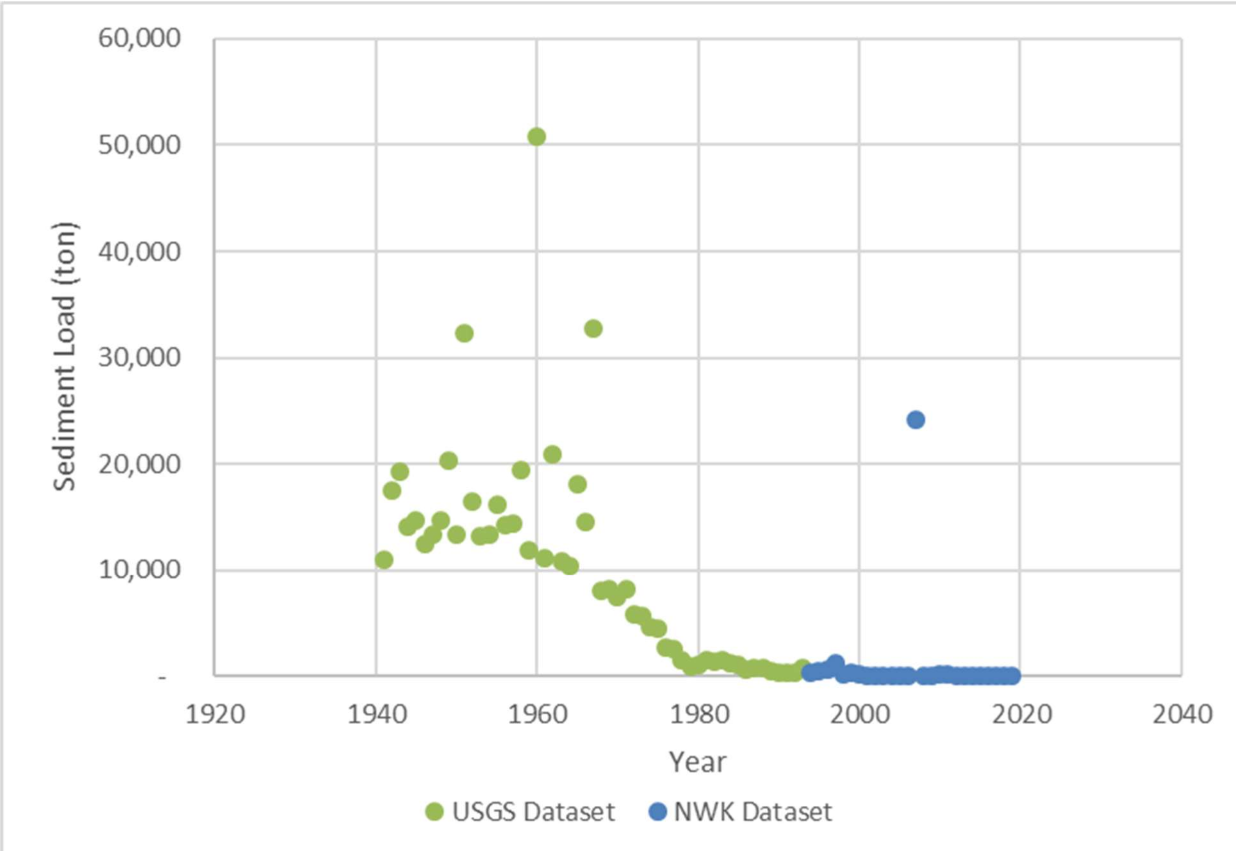


Figure 4-8: Enders annual sediment load

Table 4-1 gives the computed sediment reduction factors for the different projection time periods. Also shown in Table 4-1 are the years which were assumed to be most reflective of future conditions and were used to determine the factors from 2019-2124.

Table 4-1: Sediment correction factors for five USBR lakes

Lake	Repeated Period for Projections	Correction Most Recent Survey to 2019	Correction 2019-2024	Correction 2024-2049	Correction 2049-2074	Correction 2074- 2124
Cedar Bluff	1980-2019	0.12	0.20	0.20	0.20	0.20
Harry Strunk	1970-2019	1.65	0.96	0.96	0.96	0.96
Hugh Butler	2000-2019	0.76	0.36	0.36	0.69	0.52
Swanson	1998-2019	0.13	0.15	0.15	0.15	0.15
Enders	2000-2019	0.14	0.01	0.01	0.12	0.07

Given the number of assumptions that had to be made to make up for insufficient data, there is considerable uncertainty in projected sediment deposition for Enders and Hugh Butler. However, these assumptions likely do not have a high impact on the results of the study since sedimentation has historically been low for these two reservoirs (Table 2-1). Also, not enough water to fill the pools will likely be a much greater problem then sedimentation.

4.3 Correction for Declining Trapping Efficiency

Trapping efficiency declines as reservoir capacity decreases over time. The trapping efficiency for each of the lakes was estimated using the Brune trapping efficiency curves shown in Figure 4-9, which can be estimated using Equations 1 and 2 and the constants given in Table 4-2 (Brune, 1953).

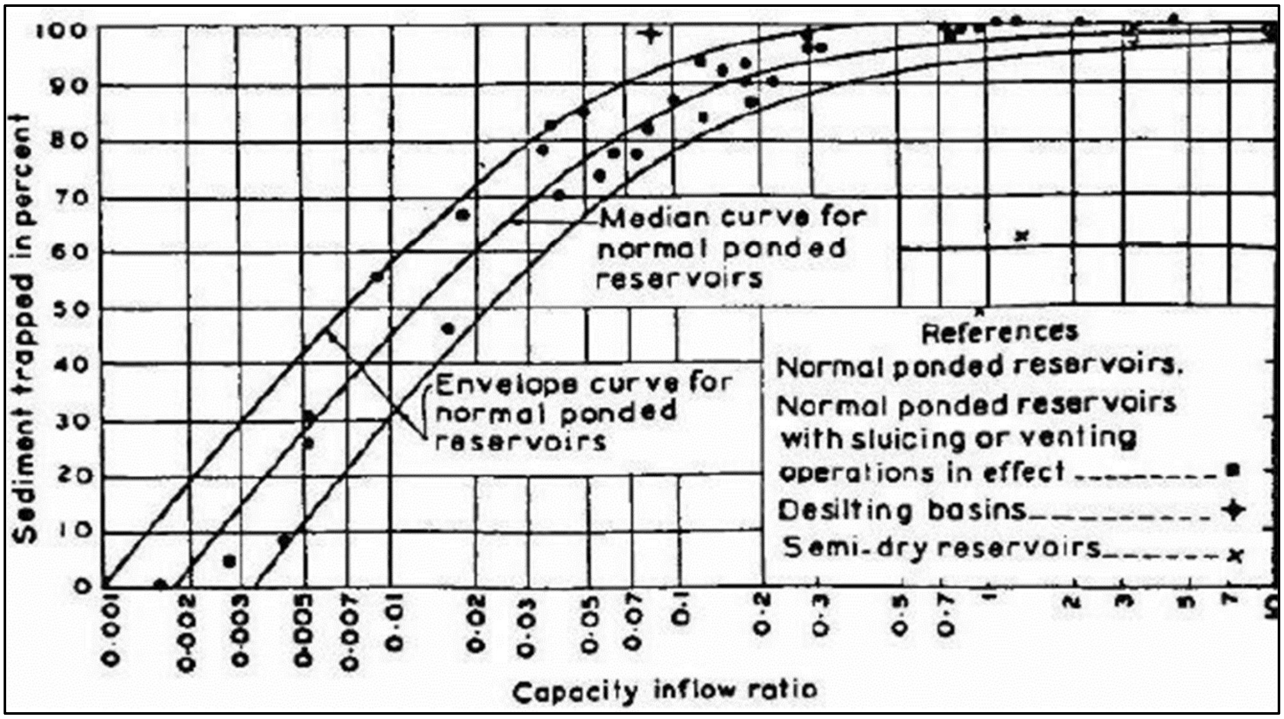


Figure 4-9: Brune Curves (Brune, 1953)

$$TE = a[1 - 2e^{-bV_*^{0.35}}] \quad (1)$$

$$V_* = \frac{V_{res}}{V_{inflow}} \qquad (2)$$

Table 4-2: Constants to be used in Equations 1 and 2

Constant	Low	Medium	High
a	95	97	100
b	5.37	6.42	7.71

The FWOP projections were initially done assuming a constant trapping efficiency over the 100-year time period. Trapping efficiency was then computed for the beginning and ending MPP volumes. All the USBR lakes except for Waconda and Harry Strunk showed inconsequential declines in trapping efficiency through year 2124. For the lakes where trapping efficiency has an impact, the trapping efficiency was computed for each day through 2124 and the daily deposition was then reduced by the ratio of the current trapping efficiency divided by the starting trapping efficiency.

4.4 FWOP Multipurpose Pool Volume

Table 4-3 provides the remaining multipurpose pool volumes at the end of 25, 50, and 100 years for the future without project (FWOP). Year 0 is 2024, so there are varying number of years of simulation to take each lake from its most recent survey to Year 0. Waconda, and Harry Strunk experience significantly more reduction in pool volume than the other lakes.

Table 4-3: Remaining Multipurpose Pool Volumes in Acre Feet (ac-ft).

Lake	Original Volume (ac-ft)	2024ac-ft	2024%	2049ac-ft	2049%	2074ac-ft	2074%	2124ac-ft	2124%
Lovewell	41,687	34,746	83.3	33,967	81.5	33,189	79.6	31,633	75.9
Waconda	242,017	197,740	84.2	181,016	77.3	164,307	70.3	130,949	56.6
Kirwin	99,432	97,267	97.8	96,492	97.0	95,716	96.3	94,166	94.7
Webster	77,370	75,197	97.2	74,407	96.2	73,616	95.1	72,034	93.1
Cedar Bluff	185,496	171,590	92.5	170,299	91.8	169,008	91.1	166,426	89.7
Keith Sebelius	36,127	33,415	92.5	32,289	89.4	31,164	86.3	28,913	80.0
Harry Strunk	41,120	33,270	80.9	32,061	78.0	30,853	75.0	28,439	69.2
Hugh Butler	37,840	35,363	93.5	34,959	92.4	34,176	90.3	32,989	87.2
Swanson	120,366	110,040	91.4	109,819	91.2	109,598	91.1	109,157	90.7
Enders	44,482	42,801	96.2	42,795	96.2	42,691	96.0	42,582	95.7

Table 4-4 lists the trapping efficiency of each lake in Year 0, 25, 50, and 100. As seen, the trapping efficiency for all the lakes remains high for the entire projection period and only declines moderately at Harry Strunk and Waconda.

Table 4-4: Trapping Efficiency.

Name	Recent Survey	Year 2124
Lovewell	96.3	96.2
Waconda	96.7	96.2
Kirwin	97.0	97.0
Webster	97.0	96.9
Cedar Bluff	97.0	97.0
Keith Sebelius	97.0	97.0
Harry Strunk	96.4	96.1
Hugh Butler	97.0	97.0
Swanson	97.0	97.0
Enders	97.0	97.0

4.5 FWOP Flood Pool Volume

The decline in flood control pool storage (FP) for the USBR lakes in the Kansas River Basin could not be easily estimated since the bathymetric surveys did not reach high enough into the flood pool. However, the percentage of incoming sediment mass that deposits in the flood pool was estimated for six of the USACE lakes in the basin and from HEC-RAS sediment modeling results for Tuttle Creek (see Appendix D4). This was correlated to the percent of the volume lost from the MPP as shown in the figure below. The correlation was good for the six USACE lakes and HEC-RAS modeling results. Using the linear regression shown in the Figure 4-10, the percentage of mass depositing in the flood pool at Waconda was determined to initially be 16.65% with final value being 44.29% in 2124. The flood pool at Waconda was estimated due to its importance and inclusion in the Kansas River Basin ResSim modeling. The flood pool deposition for the other USBR lakes was not estimated.

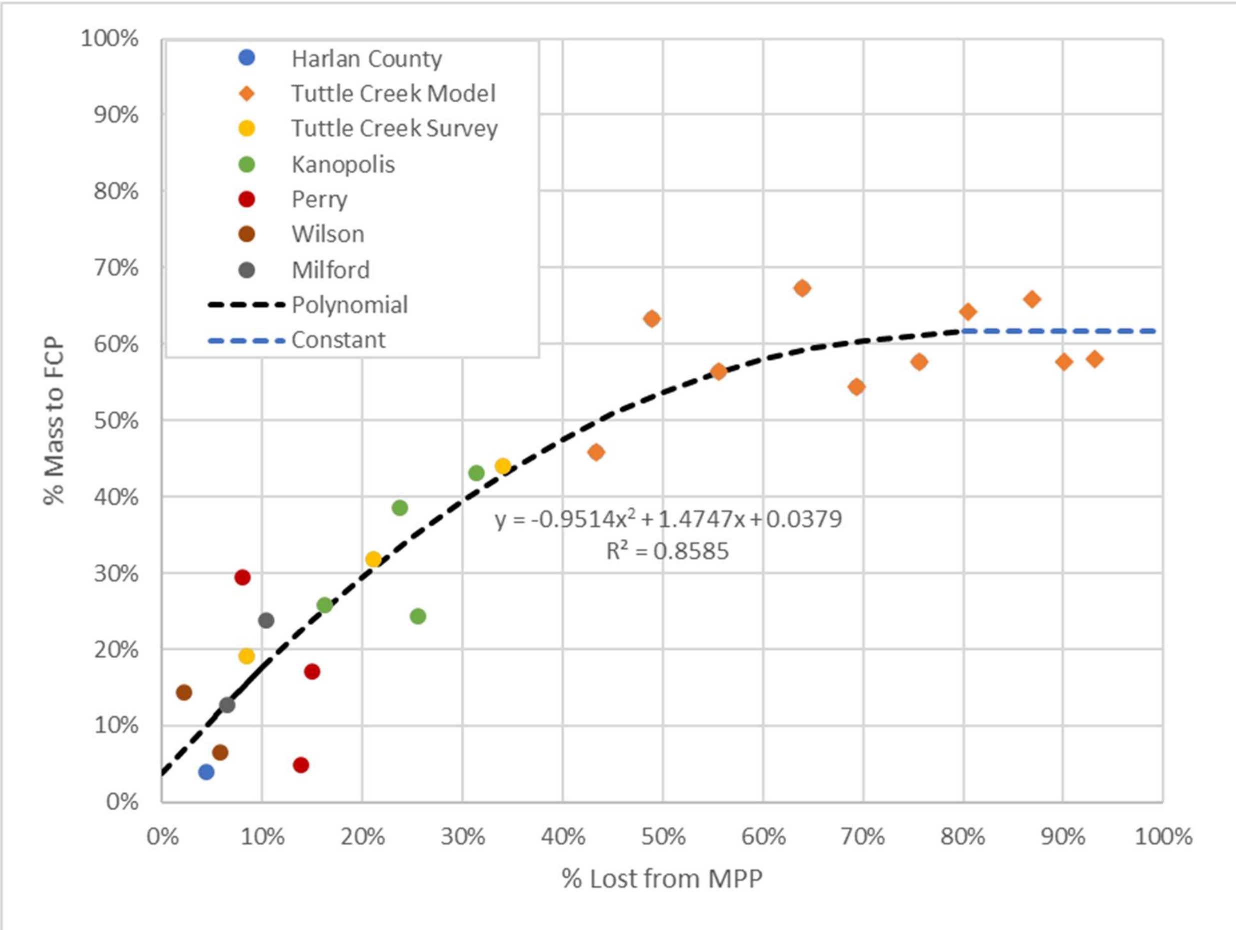


Figure 4-10: Correlation between the percentage lost from the MPP and the percentage of mass that deposits in the flood pool.

A bulk density estimate was needed for both the MPP and flood pools to calculate the FP deposition. This was estimated to be 42.3 and 72.2 lb/ft^3 for the MPP and FP respectively by averaging the bulk densities of nearby Wilson, Kanopolis, Harlan County, and Milford lakes.

Using the parameters estimated above, the FP storage was calculated to be the volumes given in Table 4-5. The change in storage for the FP is much lower than for the MPP given in Table 4-3.

Table 4-5: Waconda FP storage volumes over time

Year	Flood Pool Volume
1967 (surveyed)	722,988
2001	720,592
2024	718,462
2049	715,297
2074	711,429
2124	702,030

4.6 FWOP MPP Surfaces and Area

Lake surface area decreases as sediment deposits within the lakes; especially in the upper portion of the lake known as the delta. As deltas progress into wider and deeper sections of the lake, the rate of progression is expected to slow. The future rate was estimated by finding the delta progression that would yield the correct volume of sediment deposition as listed in Table 4-3. This delta progression analysis provides approximate results based on applying MPP deposition as an even veneer of deposition over all areas under the multipurpose pool elevation. This was accomplished using ArcGIS to add the projected elevation change, determined by dividing projected volume change by lake surface area, to the starting bathymetric surface. Where MPP areas of the lake deposited to elevations above the top of MPP, the volume above the MPP was computed and added again to the remaining MPP areas. This required several iterations.

This process was only done for Harry Strunk, Keith Sebelius, and Waconda lakes since bathymetric survey points were available for these lakes and the projected deposition was higher. Table 4-6 gives the FWOP surface areas at MPP for these three lakes. The areas were computed in ArcGIS by creating a contour of the MPP elevation from the projected surfaces, then converting the contour to a polygon. The calculate geometry tool was then used to determine the area in acres.

Table 4-6: FWOP MPP Surface Areas.

Year	Waconda (ac)	Keith Sebelius (ac)	Harry Strunk (ac)
Initial	11,668	2,174	1,664
2024	10,444	2,126	1,563
2049	10,165	2,078	1,535
2074	9,912	2,029	1,508
2124	9,393	1,926	1,453

Figures 4-11 through 4-13 show the FWOP MPP elevation contours for the three lakes.

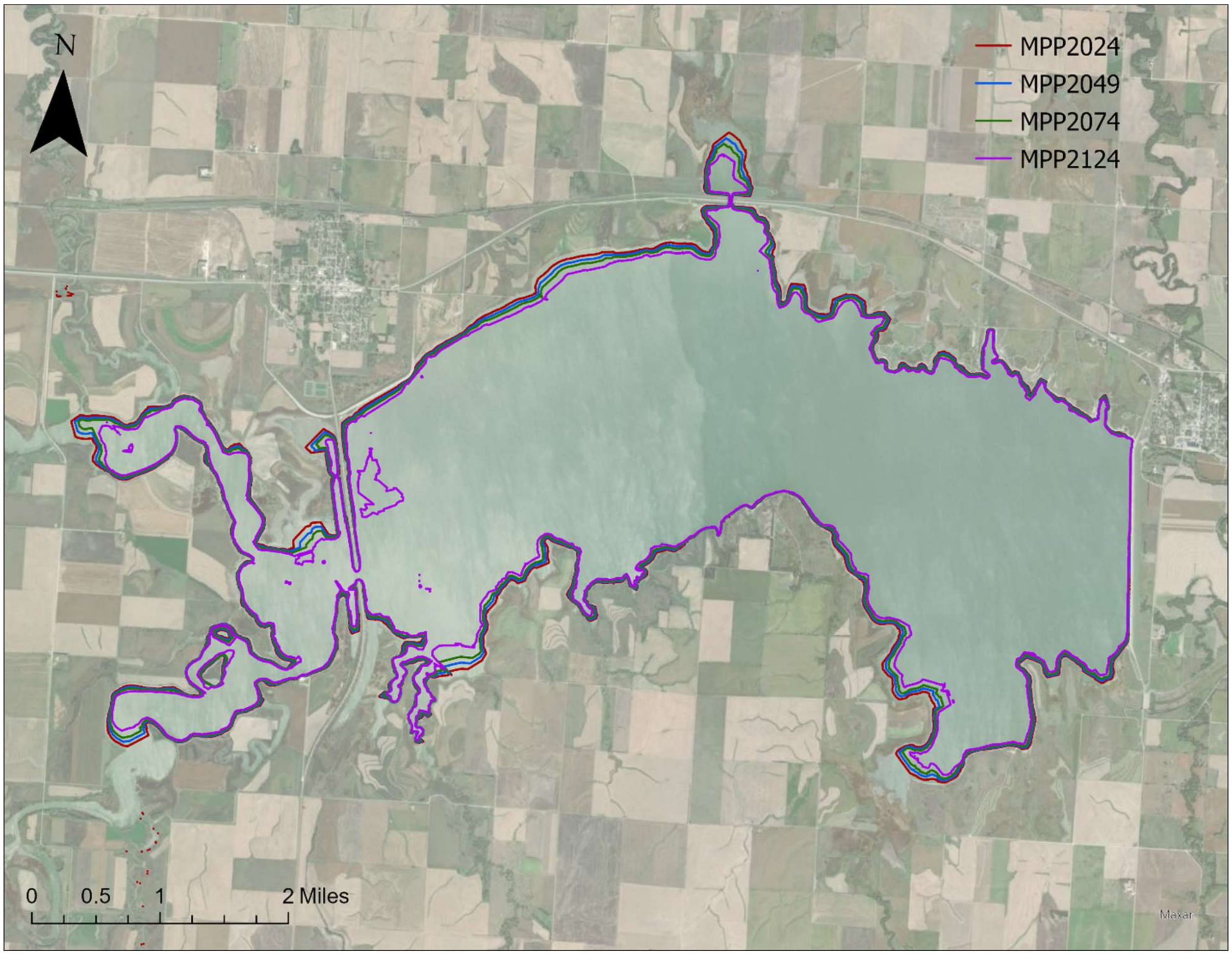


Figure 4-11: FWOP contours of MPP elevation for Waconda Lake.

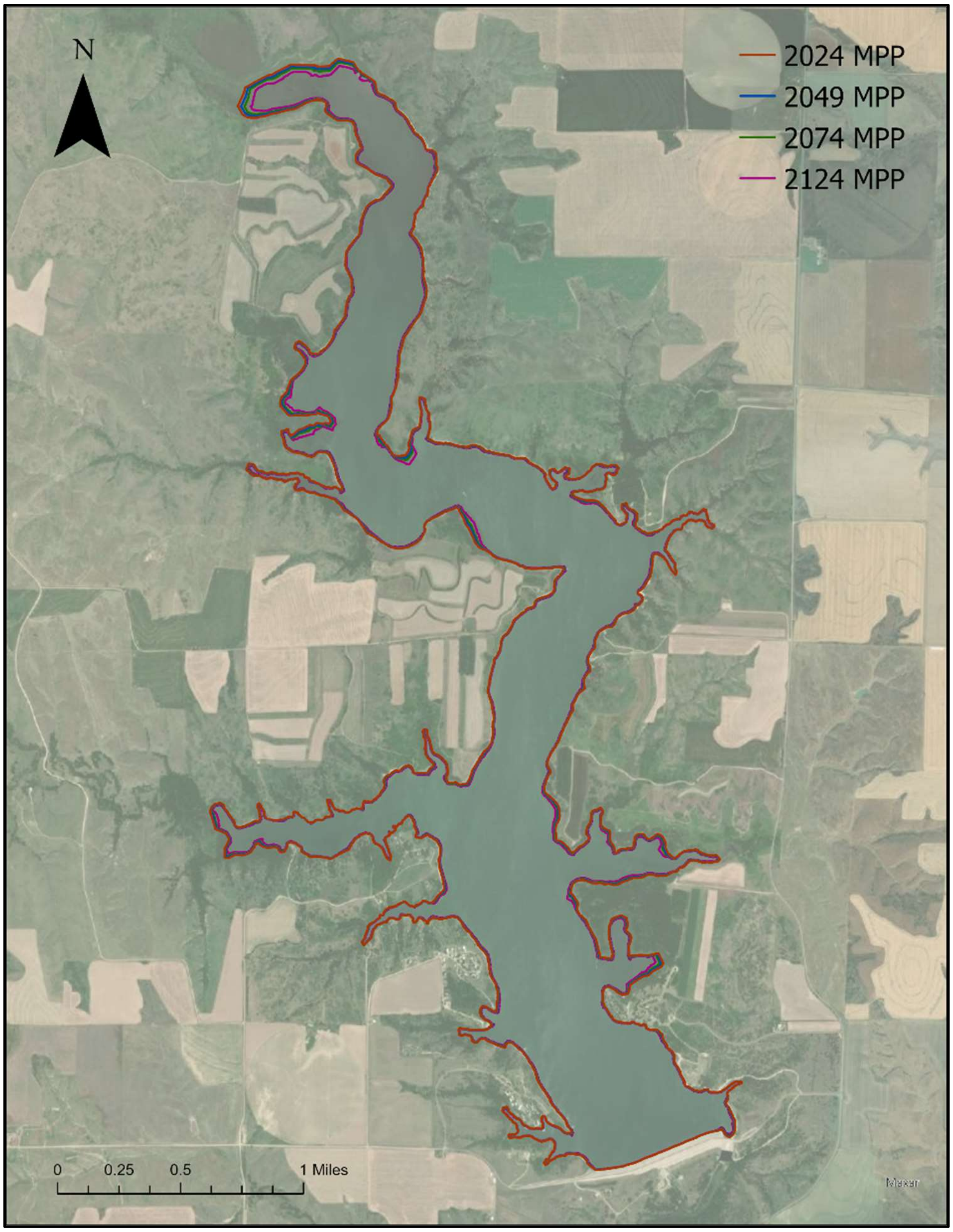


Figure 4-12: FWOP contours of MPP for Harry Strunk Lake

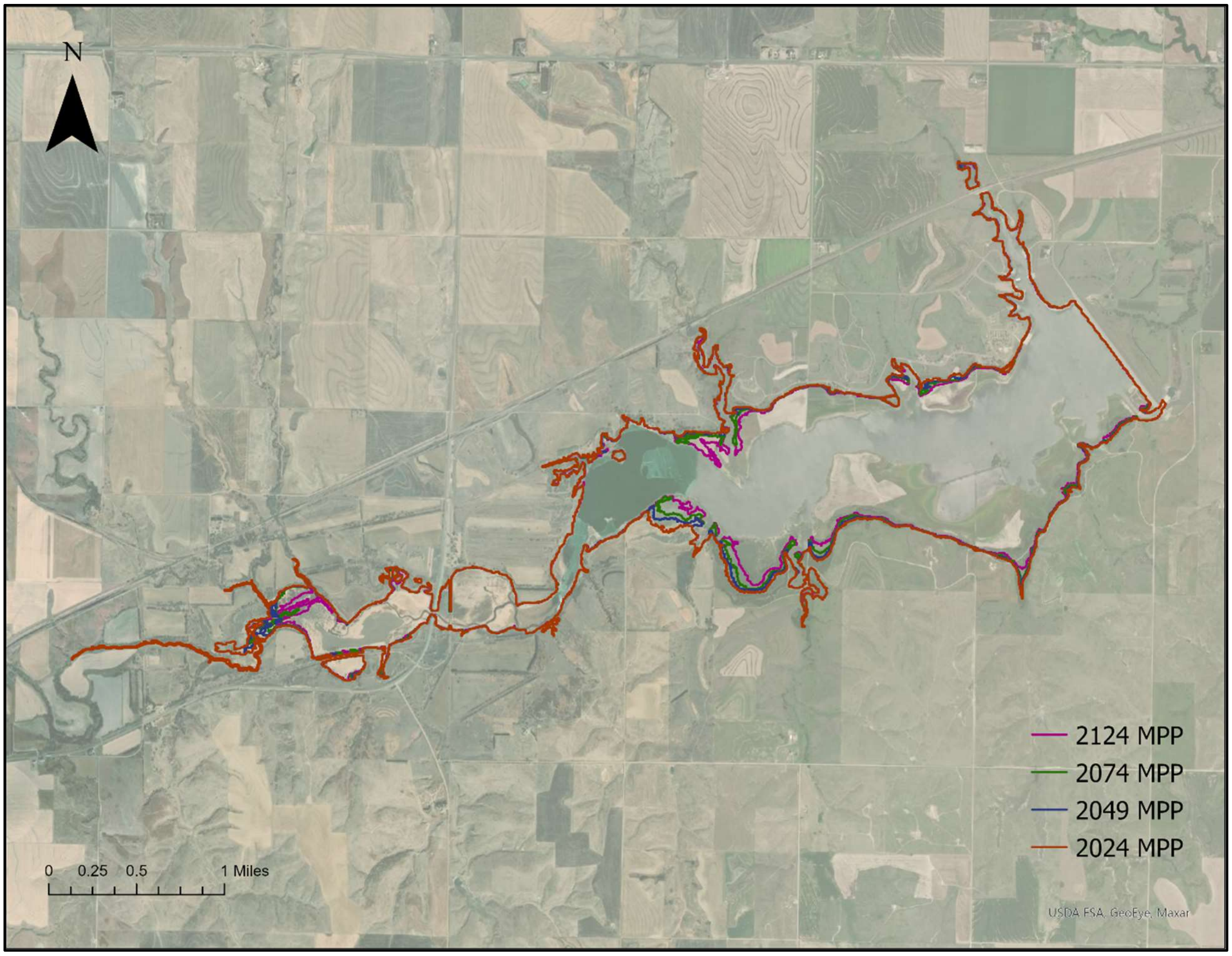


Figure 4-13: FWOP contours of MPP elevation for Keith Sebelius Lake.

4.7 FWOP Waconda Area Capacity Curve

The FWOP area capacity curves were determined for Waconda to input into a HEC-ResSIM model developed by Kansas City Water Management Section (EDH-C). The projected MPP surfaces discussed in the previous section were used to estimate the portion of the curve within the MPP. This was done using the Surface Volume tool in ArcGIS to compute the surface area and volume at 5-foot intervals.

For the portion of the curve in the flood pool, the volumes were determined at five-foot intervals using the existing capacity curve and factors computed for distributing the deposition. These factors were computed using the daily pool elevation and sediment rating curves, which were created from measurement taken at the two upstream gages (USGS gages 06872500 and 06874000).

The factors were determined so that the deposition would be evenly distributed across all elevations below the daily pool elevation. For example, if the daily pool elevation was 1464, half the sediment inflow would be distributed between elevations 1455.6-1460, while the other half would be distributed to elevations 1460-1465.

The daily deposition within these intervals were then summed and divided by the total deposition to get the percentage of total deposition for each 5-ft elevation band. See Table 4-7 for the computed sediment distribution. The reduction in surface area over that increment was then estimated by assuming it had the same percent reduction as the storage volume. However, because different methods were used to estimate the MPP and FP curves, a discontinuity was observed in the surface area between them. This was resolved replacing the surface area for first point in the FP with the value obtained by interpolating between the final MPP point and the second FP point. This methodology compared well with the 1D HEC-RAS sediment modeling results discussed in Appendix D4. See Appendix B for the final elevation-storage and elevation-area curves.

Table 4-7: Sediment Distribution within the Flood Pool for Waconda.

Elevation in feet, local datum	Sediment %
1455.6 to 1460	72.03%
1460 to 1465	14.54%
1465 to 1470	6.24%
1470 to 1475	3.17%
1475 to 1480	2.45%
1480 to 1485	1.15%
1485 to 1488.3	0.41%

5.0 SUMMARY AND CONCLUSIONS

This appendix summarized the existing conditions analysis for the ten USBR lakes in the Kansas River Basin currently used for water supply. The historic loss in MPP capacity ranged from 1.3% to 15.7% according to bathymetric surveys.

Sedimentation will continue to varying degrees in each lake in the Kansas River Basin, impacting authorized purposes, habitat, and operations and maintenance. FWOP projections over a 100-year period were made starting with the assumption that sedimentation continued at the historic rate and correcting for declining sediment loads into five of the lakes and additionally for decreasing trapping efficiencies at three of the lakes. Results indicate the total deposition volume will be highest at Waconda lake.

The FWOP flood pool deposition was estimated for Waconda Lake using a linear regression created from USACE lakes within the Kansas River Basin. These calculations showed a much lower deposition amount in the flood pool than in the MPP.

A GIS method assuming an even veneer of deposition was used to create surfaces of the FWOP elevations below MPP. These were then used to estimate the surface area of the lake at MPP for FWOP.

A FWOP area capacity curve was created for Waconda lake to input into a HEC-ResSIM model of the Kansas River Basin. Arc-GIS and the FWOP surfaces were used to compute the area and volume of the MPP portion of the curve. The existing area capacity curves, daily pool elevation, and daily sediment load was then used to compute the flood pool portion of the curve.

Many simplifying assumptions were needed to conduct this analysis due to insufficient data and the broad scope of this study. This allowed the computation of information sufficient for evaluating and ranking recommendations. More in-depth analysis and data is needed if additional confidence is needed for future studies; particularly for Harry Strunk, and Waconda Lakes where deposition is most severe.

6.0 REFERENCES

Brune, G. M. (1953). *Trap Efficiency of Reservoirs.* American Geophisical Union.

Ferrari, R. (1996). *Lovewell Reservoir 1995 Sedimentation Survey.* U.S. Bureau of Reclamation. Denver, Colorado.

Ferrari, R. (1997). *Kirwin Reservoir 1996 Sedimentation Survey.* U.S. Bureau of Reclamation. Denver, Colorado.

Ferrari, R. (1997). *Webster Reservoir 1996 Sedimentation Survey.* U.S. Bureau of Reclamation. Denver, Colorado.

Ferrari, R. (1998). *Enders Reservoir 1997 Sedimentation Survey.* U.S. Bureau of Reclamation. Denver, Colorado.

Ferrari, R. (1998). *Hugh Butler Lake 1997 Sedimentation Survey.* U.S. Bureau of Reclamation. Denver, Colorado.

Ferrari, R. (2001). *Cedar Bluff Reservoir 2000 Reservoir Survey.* U.S. Bureau of Reclamation. Denver, Colorado.

Ferrari, R. (2001). *Keith Sebelius 2000 Lake Survey.* U.S. Bureau of Reclamation. Denver, Colorado.

Ferrari, R. (2003). *Waconda Lake 2001 Sedimentation Survey.* U.S. Bureau of Reclamation. Denver, Colorado.

Ferrari, R. (2006). *Harry Strunk Lake 2006 Sedimentation Survey.* U.S. Bureau of Reclamation. Denver, Colorado.

Ferrari, R. (2010). *Bonny Reservoir Bathymetric Survey.* U.S. Bureau of Reclamation. Denver, Colorado.

Ferrari, R. (2012). *Swanson Lake – Trenton Dam 2011 Bathymetric Survey.* U.S. Bureau of Reclamation. Denver, Colorado.

US Army Corps of Engineers ®
Kansas City District

Kansas River Reservoirs Flood and Sediment Study

Appendix D4: Tuttle Creek HEC-RAS Sediment Modeling

November 2022

Contents

1.0 INTRODUCTION

Sediment deposition trends and projections were made for seven U.S. Army Corps of Engineers (USACE) and eleven U.S. Bureau of Reclamation (USBR) lakes as part of the Kansas River Watershed Study. Appendix D1 documents the Existing Conditions analysis, which was based on observed deposition trends and a sediment rating curve analysis from upstream USGS gauges. The Future Without Project (FWOP) analysis is documented in Appendix D2 and incorporated the trends presented in Appendix D1. A major assumption in the FWOP project analysis was that the Existing Condition trends would continue as the lake filled in with sediment; particularly that the proportion of incoming sediment that deposited in the flood pool would remain constant.

The lakes analyzed in the Kansas River Watershed Study can be generally divided into two main pools: the multipurpose pool (MPP) and the flood control pool (FP). The FP is utilized to store flood waters to prevent flooding downstream of the lake, while the MPP is the typical operating level of the lake and is used for purposes such as water supply and recreation. Repeated bathymetric surveys show that the mass of sediment deposited in the MPP is greater than in the FP and can vary significantly based on the pool elevation and other factors. Also, based on available measurements, the sediment deposited in the FP has a higher bulk density then the MPP. For the initial spreadsheet computations documented in Appendix D2, it was assumed that a constant fraction of the incoming load would deposit in the FP. The remaining fraction would either deposit in the MPP or pass downstream. Over time, the fraction that deposits in the FP remained constant, the fraction depositing in the MPP decreased, and the fraction passing downstream increased according to the Brune trapping efficiency equation. The assumption that the percentage of incoming sediment depositing in the FP remains constant has a significant impact on the FWOP projections. In reality, sediment deposition would induce backwater effects, which would increase the percentage of sediment that deposits in the FP. A one-dimensional (1D) HEC-RAS sediment model was developed for Tuttle Creek Lake to model these geomorphic feedbacks and quantify this increase in FP sedimentation over time.

Tuttle Creek Dam and Lake are located on the Big Blue River 12.3 miles above its confluence with the Kansas River. Major tributaries to the lake include Fancy Creek, the Black Vermillion River, and the Little Blue River. Construction of the dam began in October 1952 and closure was completed in July 1959. The multipurpose pool (MP) level was first reach in April 1963. Drainage area above the dam is 9,556 square miles, with the predominant land use in the watershed being agriculture and grazing. Authorized purposes of the reservoir include flood control, irrigation, recreation, fish and wildlife, navigation support on the Missouri River, and water quality. Figure 1-1 shows Tuttle Creek Lake with respect to the overall Kansas Watershed, while Figure 1-1 shows the lake and the Big Blue Watershed. Also shown in Figure 1-1 are the main U.S. Geological Survey (USGS) gages above the dam that are used in this report. The spreadsheet computations documented in Appendix D2 predicted that Tuttle Creek would fill faster than any of the other lakes in the Kansas River Watershed. It was estimated that only 34.8% of the MPP would remain in the year 2074 and 1.5% in year 2124. For the FP, it was estimated that 91% of the original volume would remain in 2074 and 86.4% in 2124.

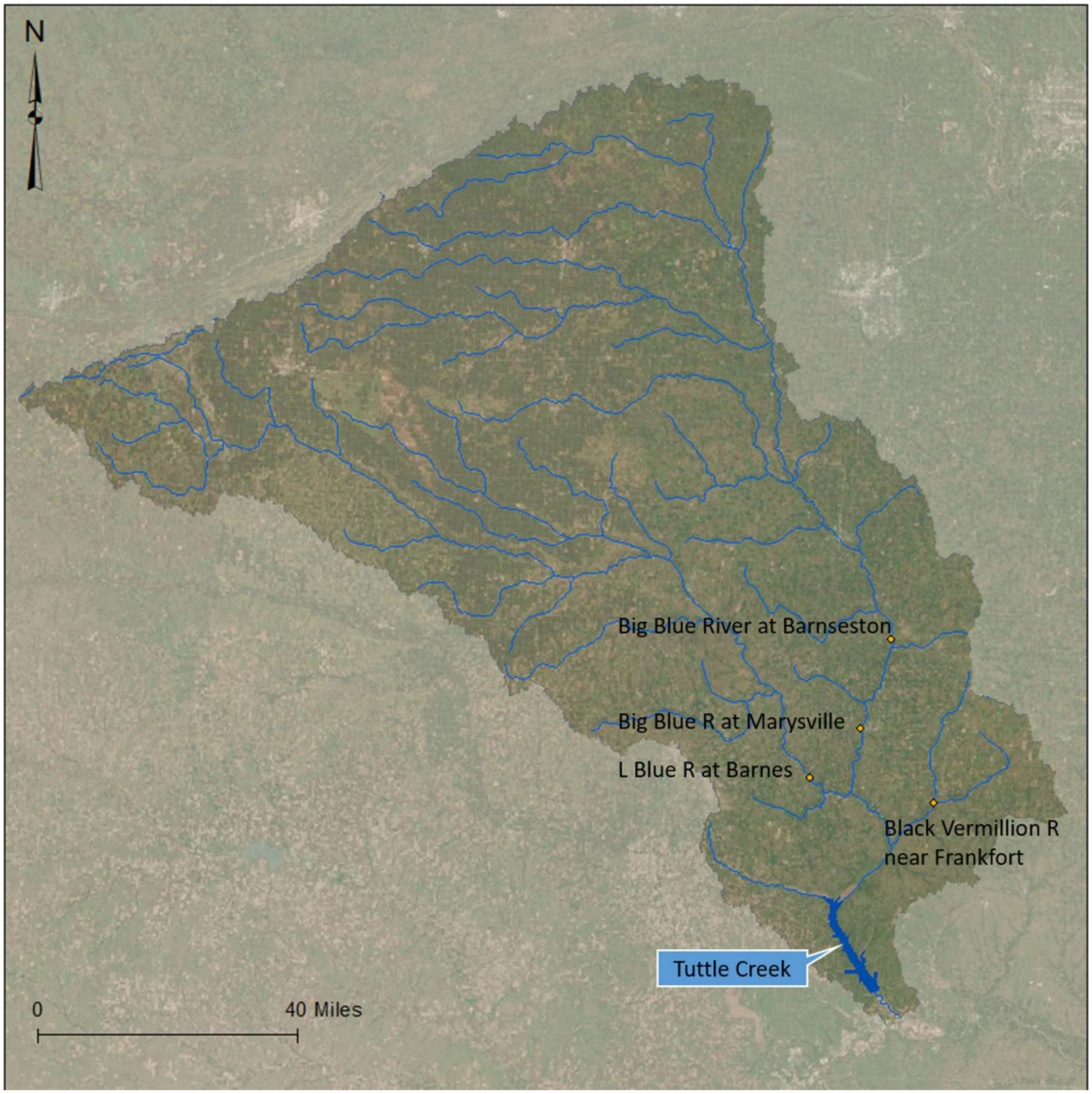


Figure 1-1: Big Blue River Watershed and Tuttle Creek Lake. Points indicate USGS gages used in this analysis.

2.0 MODEL DEVELOPMENT

Sediment modeling of Tuttle Creek Lake began in 2013 and was initially completed using HEC-RAS 5.0 Beta (Shelley, J, Gibson, S, & Williams, A, 2015). Model cross section elevation were initially derived from a bathymetric survey collected in 2000, and the model was calibrated to bathymetric data collected in 2009. However, due to uncertainties in the 2000 survey, and a new survey being available from 2020, the model was updated so that the 2009 survey was the starting condition, and the 2020 survey was then used for calibration. The model was also switched from an unsteady flow model to quasi-unsteady flow.

2.1 Model Geometry

The updated sediment model includes the mainstem of the Big Blue River, as well as the major tributaries: the Little Blue River, the Black Vermillion River, and Fancy Creek. Smaller tributary arms were modeled using long cross sections, branching from the mainstem of the reservoir. Figure 2-1 illustrates how the longer cross sections were used to model both the mainstem and two tributaries. Bounding cross sections were added upstream and downstream at approximately the level of the FP. Over the full 100-year simulation, these cross sections cause the profile of the stream to become irregular. However, not including them caused the model to underpredict the MPP storage volume and sediment deposition.

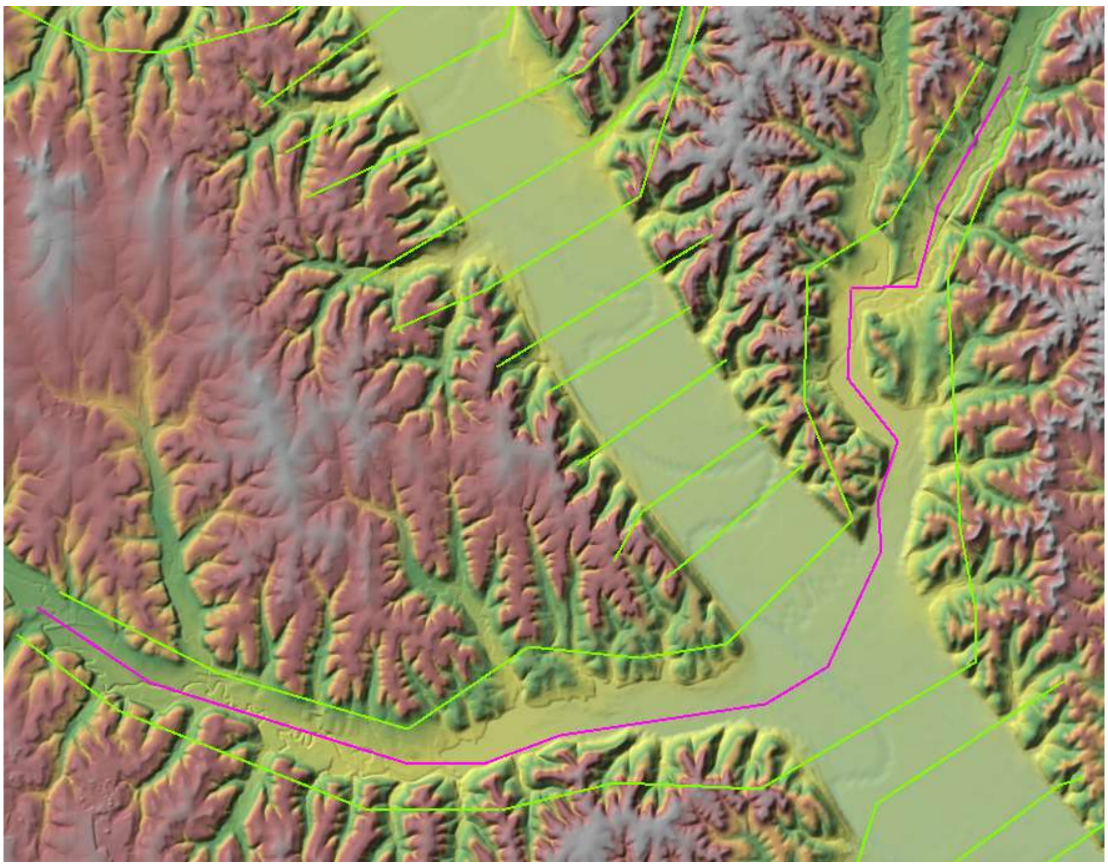


Figure 2-1: Cross sections used to model the mainstem and smaller tributaries

The HEC-RAS model extends far enough upstream of Tuttle Creek dam to include the entire flood control pool and portions of the surcharge pool. Downstream of the dam, the model extends to the confluence and includes the Kansas River from Junction City, KS to Wamego, KS. See Figure 2-2 for a map of the sediment model extents.

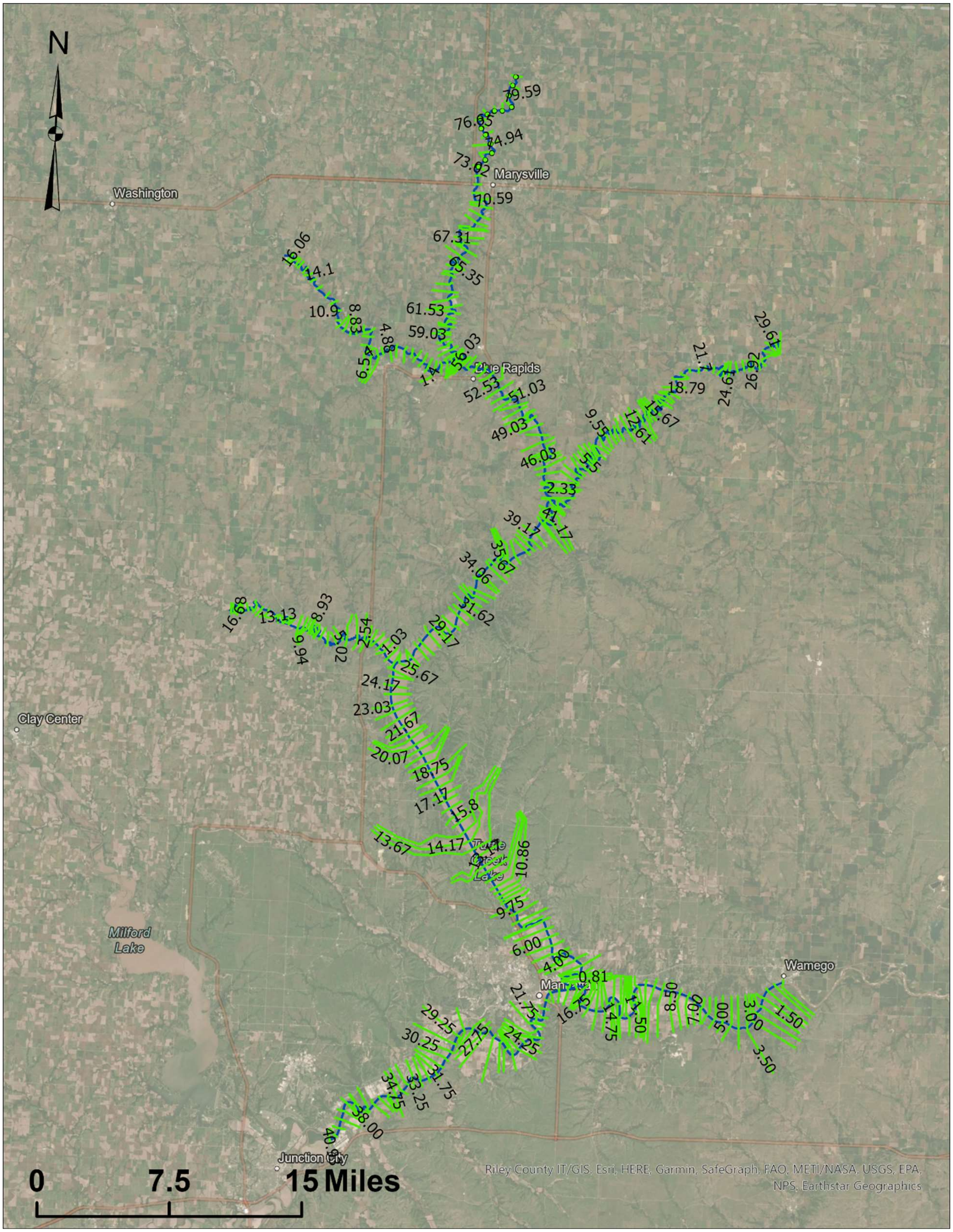


Figure 2-2: Model Extents

Cross section elevation data was derived from bathymetric data, collected in 2009 using single beam sonar, combined with LiDAR from various sources. See Figure 2-3 for a map of the extents of the various datasets. Extents of the bathymetry data only included the MPP, so the LiDAR was used for the rest of the reservoir. The Cross sections downstream of the dam were left unchanged from the original model and were set to be pass through nodes since this was outside the purposes of the study. It was determined by comparing 2010 LiDAR to LiDAR collected in 2018 along with gauge data from the Winkler gauge, that the entire tile of the 2010 LiDAR that included the Fancy Creek Tributary was approximately 4 feet low. Because of this, the Fancy Creek cross sections were corrected by shifting the entire cross section up 4 feet.

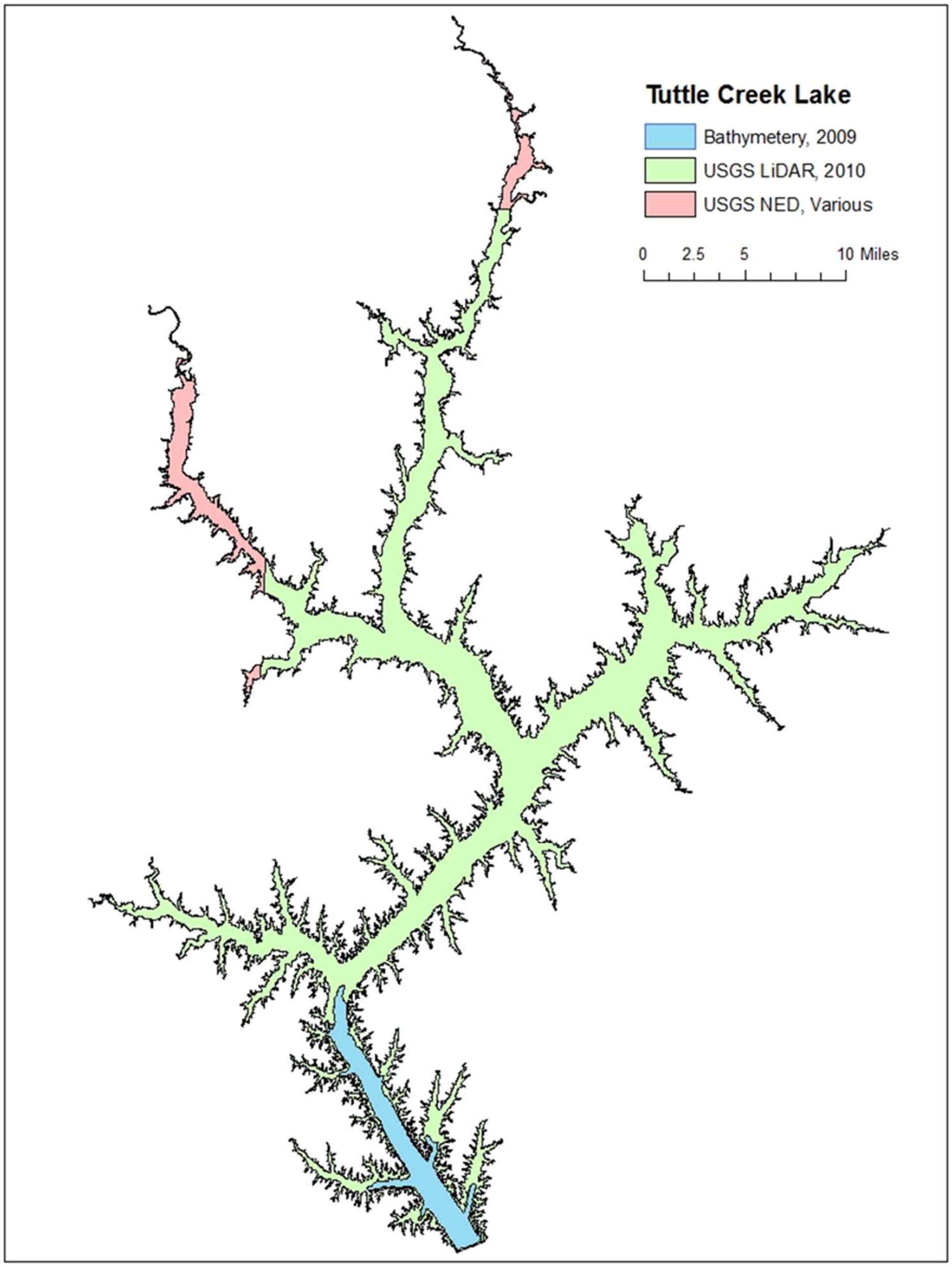


Figure 2-3: Elevation sources for creating the starting model geometry (Surdex, 2011)

Because LiDAR cannot penetrate water, additional area was added to some cross sections to account for water in the channel. On the Big Blue River, the depth of the water on the day of the survey was estimated to be five feet using gauge records from the Marysville gauge. The additional depth was added by projecting the side slopes down an additional five feet. Adding this additional depth to the cross section improved calibration of the model on the Big Blue River. However, on the other tributaries adding additional depth worsened calibration, so those cross sections were not adjusted.

The storage volume of the lake measured in HEC-RAS was compared to the official values from the 2009 storage curve (see Figure 2-4). Storage volume from the HEC-RAS model matches well with what was measured in GIS. Both the original HEC-RAS storage curve and the HEC-RAS storage curve corrected for the error in the 2010 LiDAR along Fancy Creek are included in Figure 2-4.

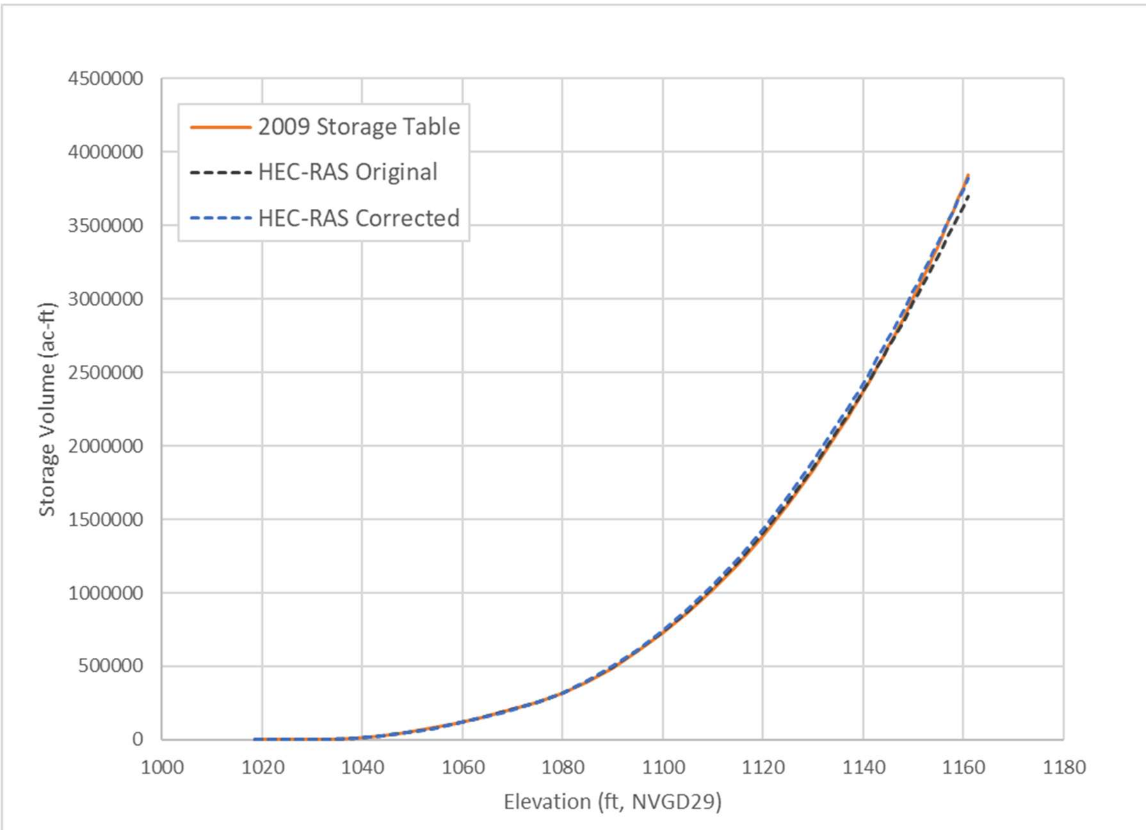


Figure 2-4: Storage volume vs. Elevation from the 2009 storage volume tables and the HEC-RAS model

2.2 Hydraulic Boundary Conditions

Four inflow locations were included upstream of the dam on the four largest tributaries of Tuttle Creek Lake, Fancy Creek, the Big Blue R, the Little Blue R, and the Black Vermillion R, which all have operating USGS gages. However, the period of record for Fancy Creek did not extend long enough so was estimated as part of the ungauged flow. Time series data was inputted into the model as the daily average value taken from the USGS gauges. The model extended some distance upstream of the USGS gauges to reach beyond the region affected by the lake backwater.

The first step in estimating the ungauged inflow was to assume the flow for Fancy Creek was equal to flow on the Black Vermillion River multiplied by a ratio of 0.424, which was determined by dividing

Fancy Creek's drainage area by the Black Vermillion's drainage area. The remaining ungauged inflow was determined by balancing the sum of the inflow, evaporation, outflow, and change in lake storage. Because of the time it takes water to route through the system, and other inaccuracies in the data, the balancing was done over a monthly time period. Also, despite balancing over a monthly period, ungauged inflow was estimated to be negative over some of the time periods. These periods were assumed to have zero ungauged inflow, and the negative inflows were carried over until the ungauged inflow exceeded the deficit. See Figure 2-5 for the computed ungauged inflows. The ungauged inflow was added as a uniform lateral inflow to the Big Blue Above Black Vermillion Reach between cross sections 13.76 and 0.76.

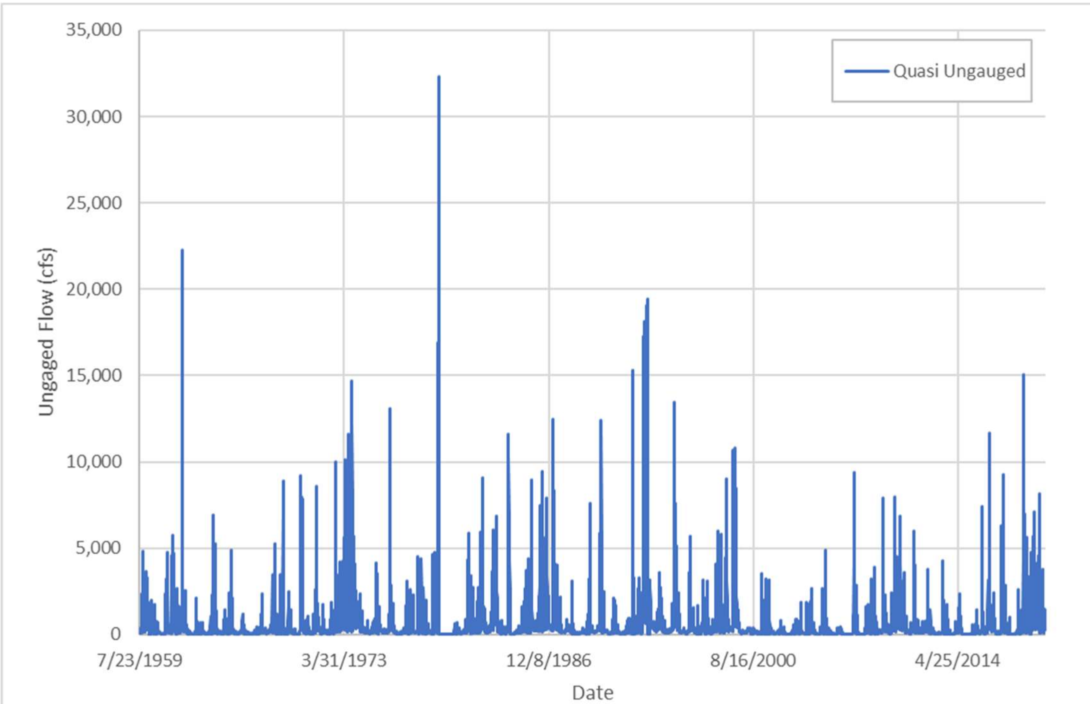


Figure 2-5: Ungauged inflows for the unsteady and quasi-unsteady HEC-RAS models

Outflows from the Tuttle Creek Dam were obtained from Water Management records. Because quasi-unsteady flow does not store water like the unsteady method, it cannot account for the variation of flow through the lake pool. Unsteady flow calculations show that lake discharge transitions through the lake pool from upstream to downstream. At the upstream end of the lake pool, flow is approximately equal to the total gauged flow; while at the downstream end of the lake, flow is equal to the lake release. Figure 2-6 illustrates this and shows how the flow varies through the lake pool within the unsteady model. There appears to be a break in the slope near mile 15, with an increased slope closer to the dam. Also shown in Figure 2-6 is how this effect was modeled with the quasi-unsteady model. A uniform lateral flow was added from cross section 14.76 to 10.17, and flow was either added or removed to transition to the lake release. A more accurate method may have been to use two separate uniform lateral inflows over the full length of the MPP. However, as the lake fills in over the FWOP, this would have been more difficult to implement, and the applied method was determined to provide the best results. Figure 2-6 also illustrates how routing effects the timing and peak of an event, with the peak of the quasi-unsteady model being higher than the unsteady model. Also, the quasi-unsteady model peaked on 2 June 2010 at 24:00, while the unsteady model peaked on 3 June 2010 at 18:00; a difference of 18 hours.

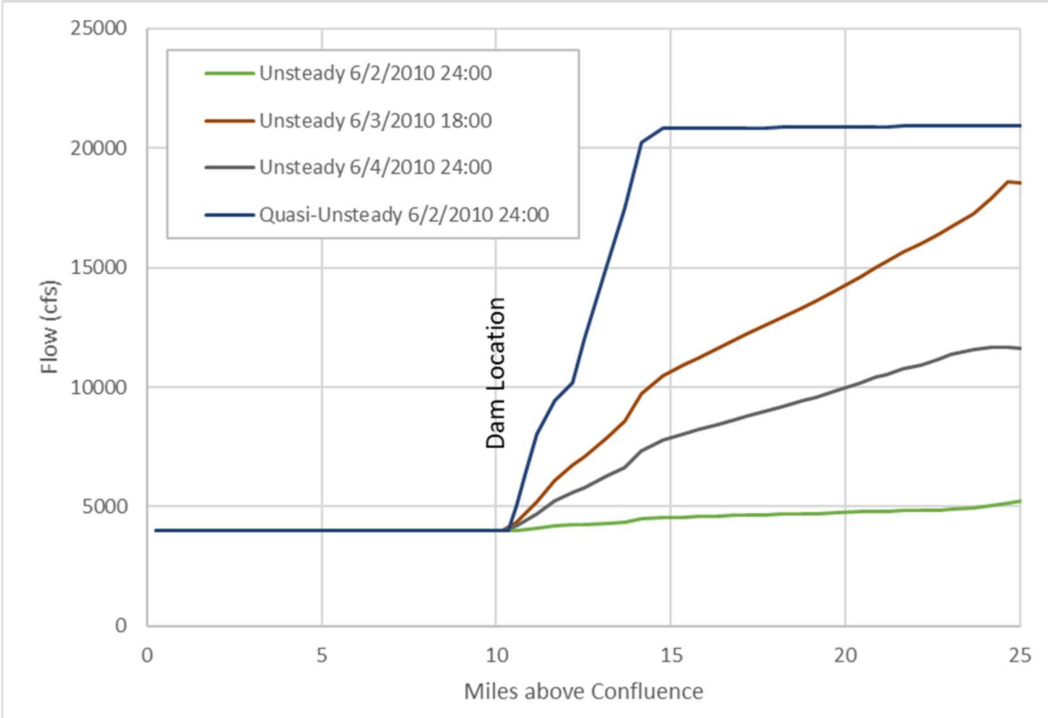


Figure 2-6: Variation in flow through for the quasi-unsteady and unsteady models

An internal stage boundary condition was used at the location of Tuttle Creek Dam and a time series dataset of lake stage was obtained from the lake gauge over the calibration period. A rating curve was used for the downstream boundary condition on the Kansas River.

2.3 Water Temperature

EDH-C has maintained daily records of water temperature within the lake since 2014. Although, this dataset was insufficient to run the full simulation, it was used to calculate the average temperature for each calendar day. A time series of the calendar day average temperature was then created for inputting into the model.

2.4 Sediment Load

Sediment rating curves were used as the upstream sediment boundary for the model and were developed for the spreadsheet calculations documented in Appendix D1. The fraction of each gradation was estimated from measured gradations. Suspended sediment gradations collected by the USGS were used to estimate the suspended load fractions, while bed gradations collected by USACE were used to estimate the bedload fraction. Sediment rating curves were not available for Fancy Creek or the ungauged inflows. The Black Vermillion rating curve was used for both Fancy Creek and the ungauged inflows, and since there was significant uncertainty in this assumption, the rating curve for the ungauged inflows was used as a calibration parameter. Equilibrium load was used for the boundary of the Kansas River. However, all cross section on the Kansas River were modeled as pass through nodes, so this will not have any effect on the results. Table 2-1 through Table 2-4 show the final sediment rating curves for the inflow locations.

Table 2-1: Sediment rating curve for the Big Blue River

Flow (cfs)	15	200	5,000	10,000	40,000
Load (ton/day)	1.05	23.3	37,792	148,687	148,687

Clay	0.864	0.856	0.418	0.428	0.428
VFM	0.011	0.013	0.067	0.053	0.053
FM	0.009	0.009	0.128	0.083	0.083
MM	0.047	0.052	0.175	0.169	0.169
CM	0.021	0.021	0.121	0.151	0.151
VFS	0.006	0.007	0.016	0.034	0.034
FS	0.008	0.008	0.016	0.023	0.023
MS	0.009	0.009	0.025	0.025	0.025
CS	0.005	0.005	0.013	0.013	0.013
VCS	0.003	0.003	0.003	0.003	0.003
VFG	0.002	0.002	0.002	0.002	0.002
FG	0.002	0.002	0.002	0.002	0.002
MG	0.001	0.001	0.001	0.001	0.001
CG	0.005	0.005	0.005	0.005	0.005
VCG	0.008	0.008	0.008	0.008	0.008

Table 2-2: Sediment rating curve for Little Blue River

Flow (cfs)	20	2,000	5,000	47,000
Load (ton/day)	0	15,633	132,309	132,328
Clay	0.690	0.502	0.319	0.319
VFM	0.061	0.059	0.057	0.057
FM	0.065	0.076	0.087	0.087
MM	0.050	0.129	0.209	0.209
CM	0.063	0.112	0.161	0.161
VFS	0.024	0.031	0.038	0.039
FS	0.002	0.019	0.031	0.031
MS	0.008	0.023	0.035	0.035
CS	0.012	0.023	0.037	0.037
VCS	0.011	0.011	0.011	0.011
VFG	0.009	0.009	0.009	0.009
FG	0.005	0.005	0.005	0.005
MG	0.001	0.001	0.001	0.001

Table 2-3: Sediment rating curve for the Black Vermillion and Fancy Creek

Flow (cfs)	1	10	3,360	7,000	54,000
Load (ton/day)	0.01	1.1	84,922	85,007	84,922
Clay	0.478	0.478	0.513	0.557	0.557
VFM	0.080	0.080	0.056	0.032	0.032
FM	0.093	0.093	0.094	0.096	0.096
MM	0.152	0.152	0.137	0.123	0.123
CM	0.119	0.119	0.111	0.103	0.104
VFS	0.016	0.016	0.015	0.016	0.015

FS	0.017	0.017	0.017	0.017	0.017
MS	0.020	0.020	0.020	0.021	0.021
CS	0.005	0.005	0.014	0.012	0.012
VCS	0.003	0.003	0.004	0.004	0.004
VFG	0.002	0.002	0.002	0.002	0.002
FG	0.002	0.002	0.002	0.002	0.002
MG	0.001	0.001	0.001	0.001	0.001
CG	0.005	0.005	0.005	0.005	0.005
VCG	0.008	0.008	0.008	0.008	0.008

Table 2-4: Modified Black Vermillion sediment rating curve used for the ungauged inflows

Flow (cfs)	1	10	3,360	7,000	54,000
Load (ton/day)	0.0	1.1	46754	46754	46754
Clay	0.228	0.228	0.263	0.307	0.307
VFM	0.080	0.080	0.056	0.032	0.032
FM	0.093	0.093	0.094	0.096	0.096
MM	0.252	0.252	0.237	0.223	0.223
CM	0.119	0.119	0.111	0.103	0.104
VFS	0.116	0.116	0.115	0.116	0.115
FS	0.067	0.067	0.067	0.067	0.067
MS	0.020	0.020	0.020	0.021	0.021
CS	0.005	0.005	0.014	0.012	0.012
VCS	0.003	0.003	0.004	0.004	0.004
VFG	0.002	0.002	0.002	0.002	0.002
FG	0.002	0.002	0.002	0.002	0.002
MG	0.001	0.001	0.001	0.001	0.001
CG	0.005	0.005	0.005	0.005	0.005
VCG	0.008	0.008	0.008	0.008	0.008

Table 2-5 gives the overall gradations from the three gauges, along with the overall gradation when the ungauged load is included.

Table 2-5: Overall gradations from the model boundary conditions

Grain Size	Gauged	Gauged + Ungauged
Clay	47.1%	40.9%
Silt	40.8%	45.4%
Sand	12.1%	13.8%

Bulk density was calculated from the gradations given in Table 2-5 using Equation 1. The bulk density was determined to be 42.9 pcf for just the gauged load, while the gauged load plus ungauged load bulk density was determined to be slightly higher at 45.3 pcf. Measurements and calculations from the USGS (Juracek, 2011) were averaged to obtain an estimated bulk density of 46.4 pcf, which is higher than either estimate. The difference could be due to consolidation of the sediment deposits over time, or from too few measurements being collected in the FP.

$$\gamma_c = \frac{1.0}{\left(\left(\frac{F}{\gamma}\right)_{clay} + \left(\frac{F}{\gamma}\right)_{silt} + \left(\frac{F}{\gamma}\right)_{sand}\right)} \tag{1}$$

Where: γ_c is the composite bulk density

F is the fraction of clay, silt, or sand

γ for clay, silt and sand is assumed to be 30 pcf, 65 pcf, and 93 pcf respectively.

Bed gradation measurements were collected on the Little Blue R and the Big Blue R in 2015 as part of a Regional Sediment Management study (Williams & Shelley, 2020). The Big Blue River measurements were used as the starting bed gradation for the Fancy Creek Reach, the Black Vermillion R, and the Big Blue R upstream of cross section 21.17. The reservoir gradation from the original model was used for cross sections downstream of cross section 21.17. See Table 2-6 for the initial bed gradations used in the model. Although, the applied gradations are likely not representative for all model locations, the deposition within the reservoir is mainly dominated by the clay and silt sizes, so bed gradation will likely have a small effect on overall results. A hotstart file was created at the end of the first calibration run, which was then used to hotstart the calibration for a final run.

Table 2-6: Initial model bed gradations

Grain Class	% Finer Big Blue	% Finer Little Blue	% Finer Reservoir
Clay	-	-	67.9
VFM	-	-	76.1
FM	-	-	85.2
MM	0.0	0.0	87.7
CM	6.3	0.5	92.9
VFS	10.5	0.8	97.9
FS	27.0	5.3	98.8
MS	46.5	21.5	99.4
CS	56.0	45.8	99.8
VCS	62.0	68.3	100.0
VFG	65.8	87.8	-
FG	69.0	98.0	-
MG	71.5	100.0	-
CG	83.0	-	-
VCG	100.0	-	-

2.5 Sediment Transport Parameters

Sediment transport parameters in HEC-RAS were developed from measured data and through model calibration. The majority of sediment that deposits in Tuttle Creek Lake are fine particles in the silt and clay ranges. In order to accurately model the fine particles, the cohesive sediment options were used in HEC-RAS for particles in the clay to medium silt grain sizes. All other grain classes were modeled using the Engelund-Hanson transport formula, which was chosen because it produced the most reasonable results.

Field samples were collected in September 2015 within the Tuttle Creek Lake to test the erodibility characteristics of the sediment deposited within the MPP (Shelley & Wells, 2019). A total of eight six-inch diameter core samples were collected at the locations shown in Figure 2-7. Parameters that were estimated from this effort included the critical shear stress, erodibility, and bulk density.

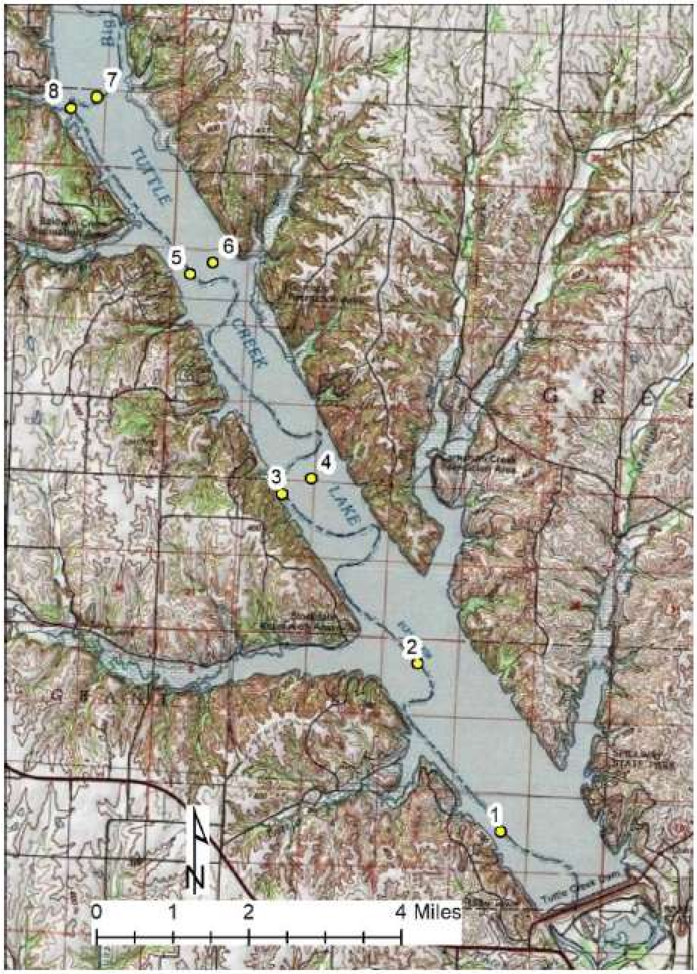


Figure 2-7: Sediment sampling locations in 2015 (Shelley & Wells, 2019)

A total of four cohesive parameters are needed for using the Krone Partheniades method in HEC-RAS, which were estimated from the collected samples. For the critical shear threshold and the slope of the erosion rate curve, the median value from the samples used in HEC-RAS. However, the mass wasting parameters were not estimated in the original study and were determined by fitting a trendline to the upper points on the erodibility curve (see Figure 2-8 for an example).

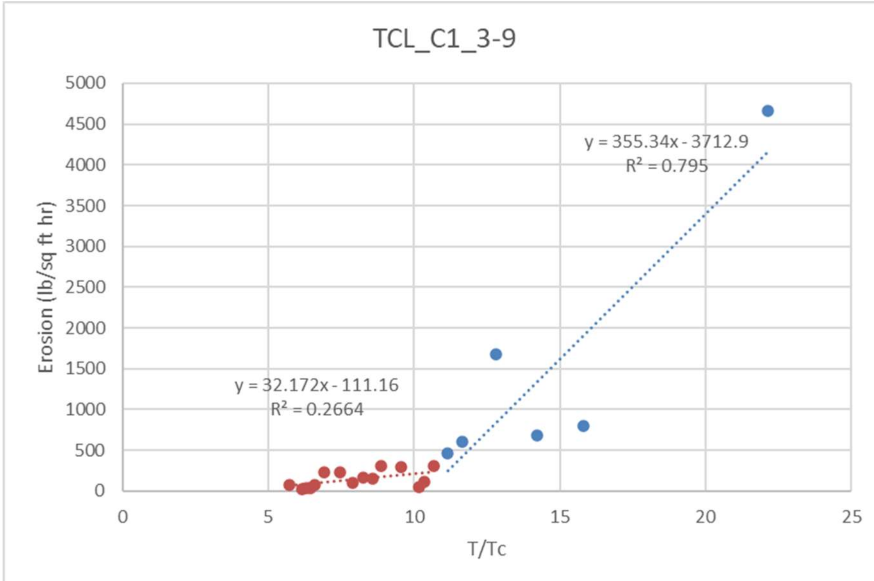


Figure 2-8: Example of the erosion measurements taken at Tuttle Creek showing the slope of the mass wasting line

Table 2-7 gives the measured cohesive parameters from the top layer of each core. Only the top layer of the core was used since it is the least affected by consolidation and was assumed to be more representative of the cohesive transport. The median values, also given in Table 2-7, were the values inputted into the model. Since the purpose of this model is only for estimating deposition and evaluating flushing scenarios, the selected cohesive options are likely adequate for the purposes of this study. Additional work would be needed to vary the cohesive parameters throughout the lake if the model was to be used to estimate reservoir flushing.

Table 2-7: Measured and calibrated cohesive parameters

Core	t_c (lb/ft^2)	M (lb/ft^2/hr)	t_{mw} (lb/ft^2)	M_{mw} (lb/ft^2/hr)
1	0.008	29.0	0.08	355
2	0.005	9.1	0.10	30
3	0.003	15.3	-	-
4	0.005	8.1	0.08	35
5	0.005	3.4	0.08	80
6	0.007	3.4	-	-
7	0.007	8.6	0.07	54
8	0.008	5.4	0.08	49
Median	0.006	8.4	0.08	52

To obtain a better calibration, the mean diameter for clay sized sediment was increased from 0.003 mm to 0.0036 mm. Doing this caused the clay particles to settle sooner within the reservoir, which likely offsets two of the limitations within HEC-RAS. The first limitation is that HEC-RAS fully mixes the sediment at the beginning of each time-step, which resets the center of mass to the center of the water column (USACE, 2022). This can cause the model to underestimate deposition for reservoirs with high residence times such as Tuttle Creek. The second limitation is that HEC-RAS does not account for flocculation. Cohesive particles often form aggregates or flocs, which increases their fall velocity (USACE, 2022). This limitation also has the effect of underestimating deposition.

Additional sediment parameters were used in the model and were selected through calibration. The Thomas (Exner 5) bed mixing algorithm was chosen since it produced the most reasonable results. The

Rubey fall velocity method was chosen as it was the default option in RAS 5.0.7. Testing determined that the model is not very sensitive to the fall velocity method. Bed change options were set to allow deposition outside the movable bed limits. The "limit to water velocity" option was selected in the sediment routing method options, since not selecting this option causes the sediment to travel faster than the water and for sediment deposition to be under-predicted within the reservoir. The computation increments were varied between 24 and 4 hours based on the discharge. All other sediment parameters not already discussed in this report were left as the default options within HEC-RAS.

3.0 CALIBRATION/VERIFICATION

3.1 Hydraulic Calibration

The HEC-RAS sediment model was compared and calibrated to USGS field discharge measurements taken at the four USGS gauges upstream of Tuttle Creek dam. Calibration was accomplished by adjusting the Manning's n values so that the model results were comparable to the measured WSE vs. discharge curve. Table 3-1 gives the final, calibrated Manning's n values, while Figure 3-1 through Figure 3-4 show the modeled and measured WSE vs. discharge. Overbank areas of the model were set to have a roughness of 0.05. Model results generally matched well with the measured values. It is apparent from the figures that backwater from the dam can have a significant impact to the hydraulics at the gauges. However, the model appears capture the backwater affects well. Model results shown below are from the quasi-unsteady sediment model.

Table 3-1: Calibrated main channel Manning's n values

Reach	Channel Manning's n
Big Blue River	0.028
Little Blue River	0.042
Black Vermillion	0.05
Fancy Creek	0.032

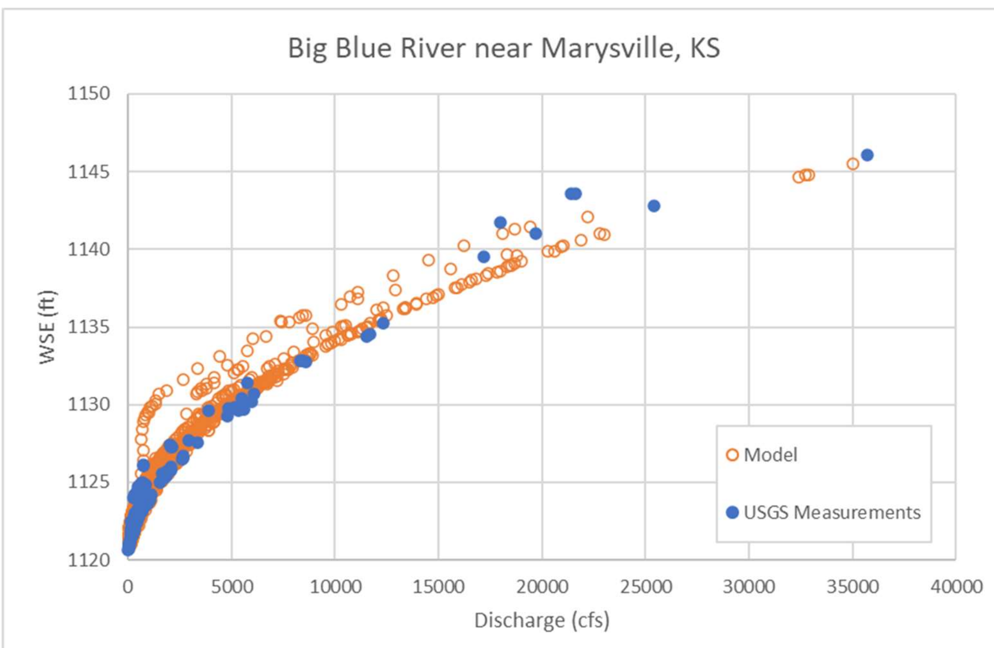


Figure 3-1: Model results compared to USGS field measurements at Marysville, KS

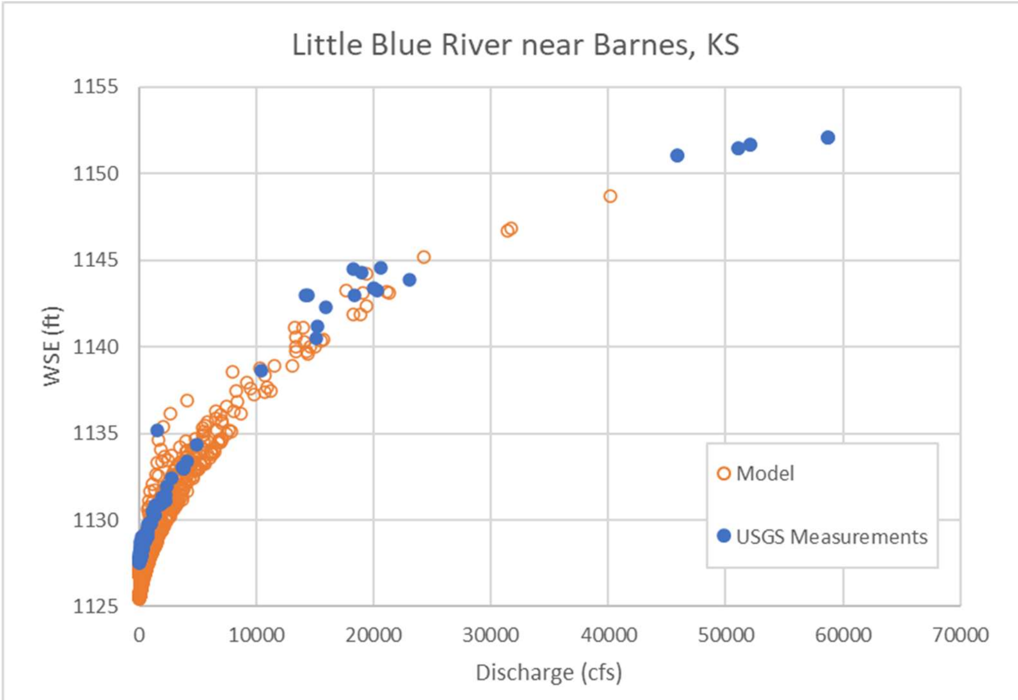


Figure 3-2: Model results compared to USGS field measurements near Barnes, KS

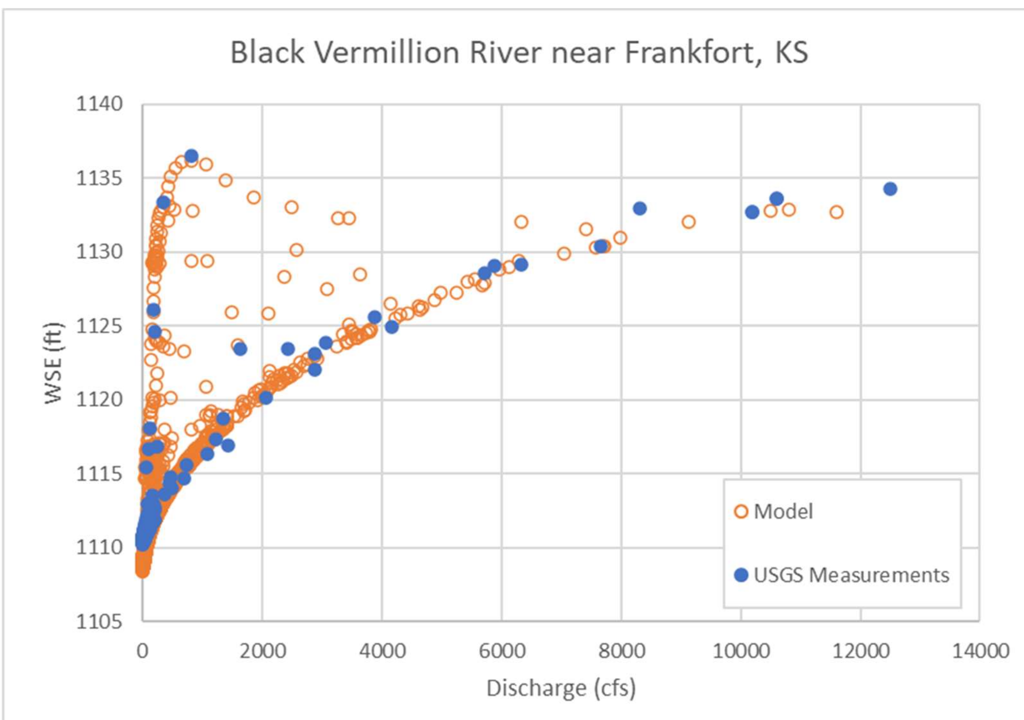


Figure 3-3: Model results compared to USGS field measurments near Frankfort, KS

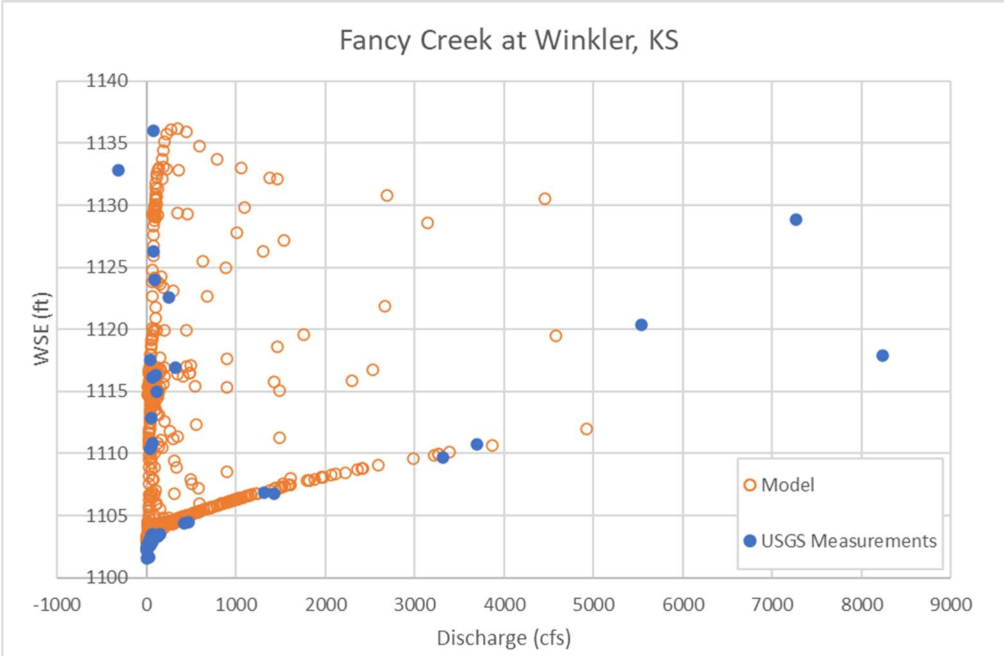


Figure 3-4: Model results compared to USGS field measurements at Winkler, KS

3.2 Hydraulic Verification

A water surface profile (WSP) and cross sections were collected on March 22nd, 2022, between RM 73 and 80 on the Big Blue River at a discharge of 436 cfs. The WSP was used to verify the hydraulic calibration of the model. The main channel bathymetry in the area was updated using the new survey and the model was run using flows from the Marysville gauge. The results given in Figure 3-5 show that there are some discrepancies between the measured and model water surface elevations, which could be because the discharge on the day of the survey was low, representing a baseflow condition. To improve the hydraulic calibration, a WSP collected at higher discharges would likely be needed.

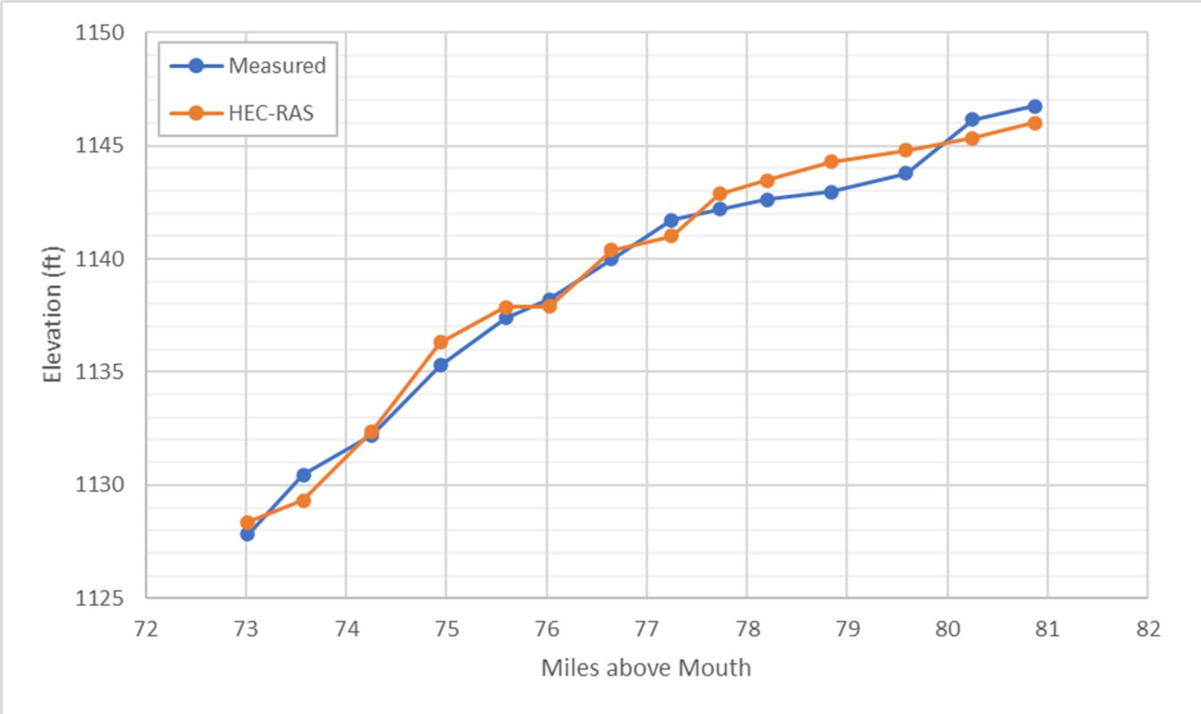


Figure 3-5: Measured and modeled water surface profiles, March 22nd, 2022

Table 3-2 gives the values for the modeled and measured water surface profile, along with the difference between them. The difference ranges between -1.1 to 1.3 feet, with a mean difference of 0.2. The results from the hydraulic verification were considered to be adequate for the purposes of this study.

Table 3-2: Measured and modeled water surface elevations, March 22nd, 2022

RM	Modeled	Measured	Difference
73.02	1128.4	1127.8	0.5
73.57	1129.3	1130.5	-1.1
74.26	1132.4	1132.2	0.2
74.94	1136.3	1135.3	1.0
75.59	1137.9	1137.4	0.5
76.03	1137.9	1138.2	-0.3
76.65	1140.4	1140.0	0.4
77.25	1141.0	1141.7	-0.7
77.73	1142.9	1142.2	0.7
78.2	1143.5	1142.6	0.8
78.84	1144.3	1143.0	1.3
79.59	1144.8	1143.8	1.0
80.25	1145.3	1146.1	-0.8
80.87	1146.0	1146.8	-0.7

3.3 Sediment Calibration

The Tuttle Creek sediment model was calibrated to the volume change observed between the 2009 and 2020 bathymetric surveys in the MPP (see Figure 3-6). Calibration was mainly accomplished through adjusting the sediment parameters discussed in Section 2. The model compares well with the observed data and the calibration was considered adequate for the purposes of this study. The measured and modeled longitudinal cumulative volume change were both computed using the Cross Section Viewer Software (Shelley & Bailey, 2018) in order to allow for a comparison over the same cross section extents.

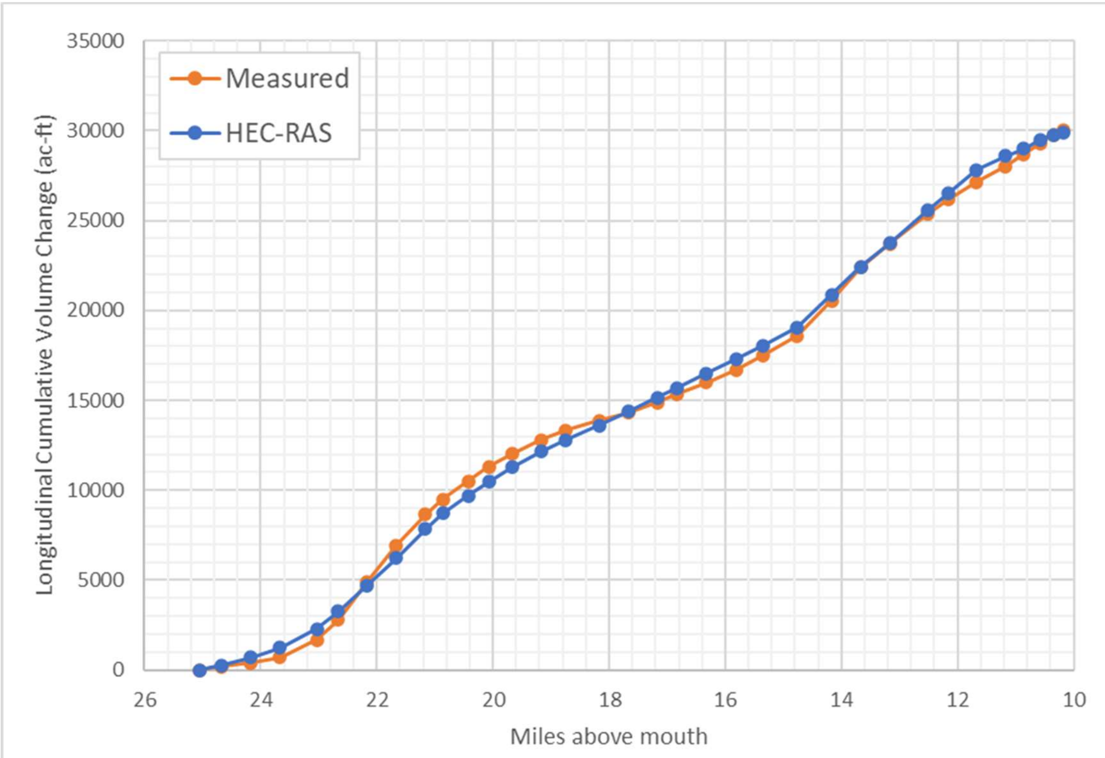


Figure 3-6: Observed and modeled longitudinal cumulative volume change in Tuttle Creek's multipurpose pool, 2009-2020

Measured and modeled volume change from 2009-2020 were compared for each cross section in the MPP (see Figure 3-7).

Resuts are

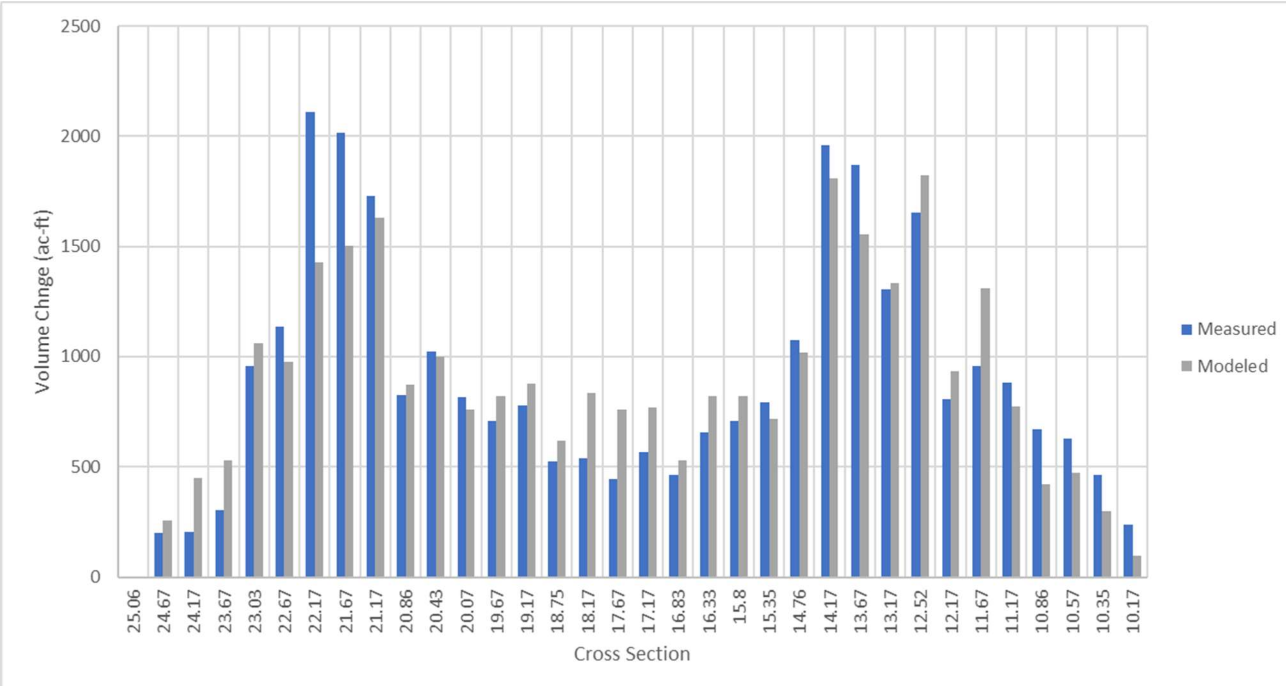


Figure 3-7: Observed and modeled volume change within the MPP by cross section

Sediment deposition within portions of the flood control pool was measured using LiDAR collected 2018 and 2009. The measured volume change is compared to the modeled change in Figure 3-8, which includes the mainstem of the Big Blue River down to cross section 25.67 but none of the modeled tributaries. Modeled volume change agrees relatively well with the measured volume change. However, there is likely uncertainty in the measured volume change because of inaccuracies in LiDAR data and differences in WSE on the dates of collection.

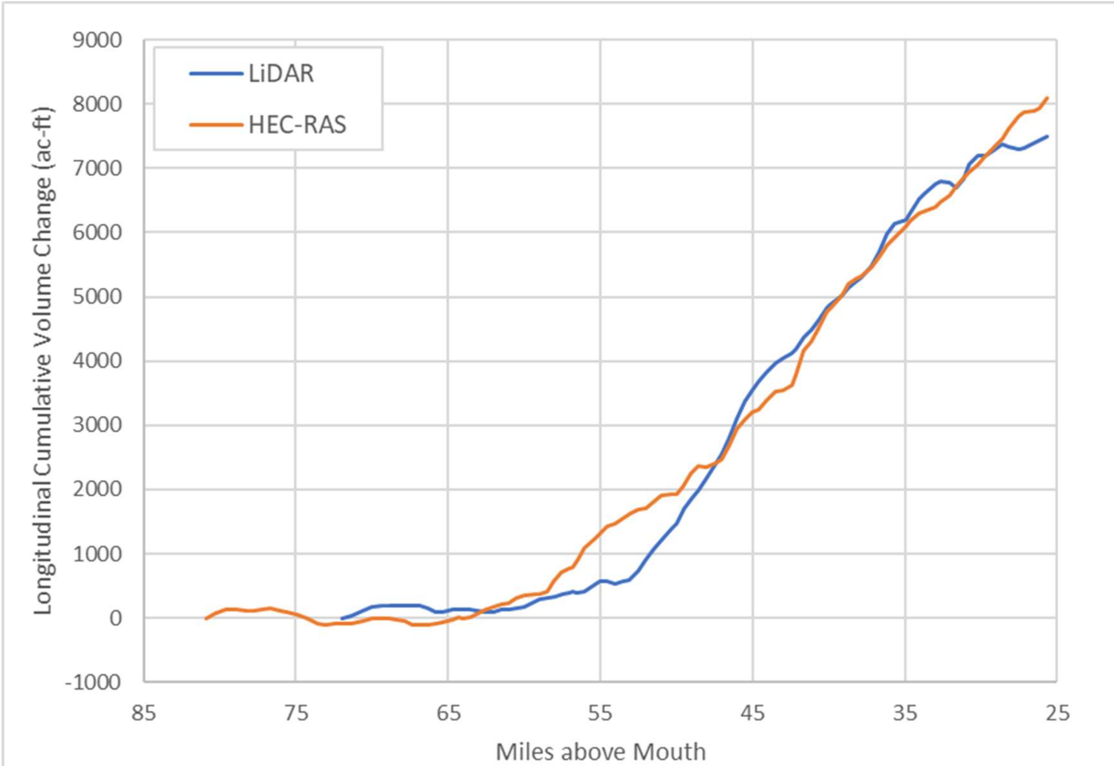


Figure 3-8: Longitudinal cumulative volume change 2009-2018 from LiDAR and HEC-RAS along the Big Blue River

The model was also compared to the trapping efficiency of 98% estimated by the USGS (Juracek, 2011). The estimated trapping efficiency from the HEC-RAS model was 95.6% over this period, which is slightly lower than the measured value.

Sediment concentration downstream of the dam was compared to measured data from the USGS as shown in Figure 3-9. The results are similar for higher discharges but do not match well at lower discharges, which could be caused by the model not accounting for local inflows of sediment downstream of the dam.

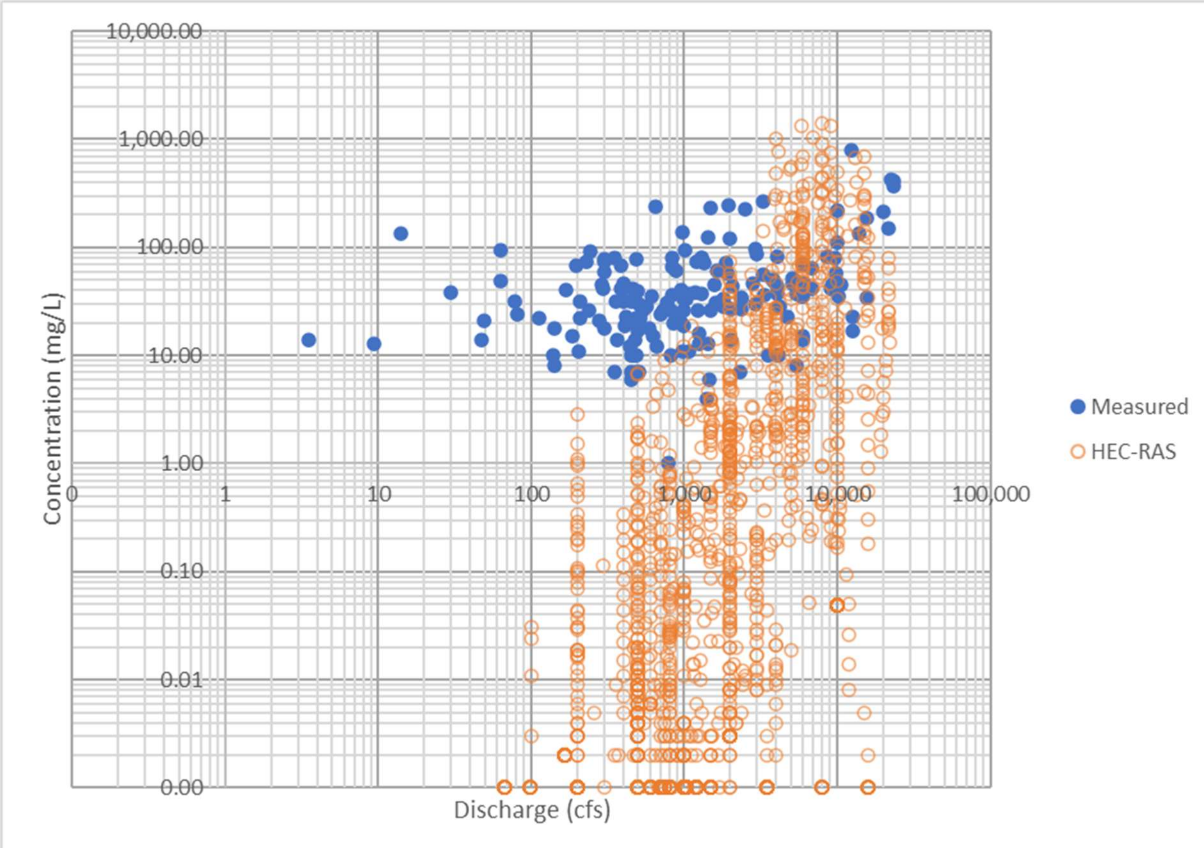


Figure 3-9: Downstream sediment concentration

During the calibration it was noticed that the inactive layer thickness for several cross sections declined over time even though the cross sections were depositing. Stanford Gibson and Steven Piper from HEC were contacted regarding the issue, but the issue has not yet been resolved. However, given that the area where this is occurring is a very small part of the overall model, it is not expected that the issue will have a significant impact on the study results. This issue was observed in both versions 5.0.7 and 6.2 of HEC-RAS.

4.0 FWOP SIMULATION

The FWOP simulation was run between June 2009 and December 2124. The inflows were the same as those given in Appendix D2, except for additions for Fancy Creek and the ungauged inflow. The stage boundary condition at the dam and dam outflow were taken from the HEC-ResSIM FWOP model documented in Appendix D2. Four simulations were run in HEC-ResSIM, each with a different elevation-capacity curve representing the years 2024, 2049, 2074, 2124. These were initially computed using the estimated deposition in Appendix D2. Because HEC-ResSIM is not able to vary the capacity of the lake over time to account for deposition, interpolation was used between the four different capacity curves to obtain a timeseries dataset that would represent a continuous reduction in lake storage. See Figure 4-1 for an example of the HEC-ResSIM pool elevations and interpolated pool elevation.

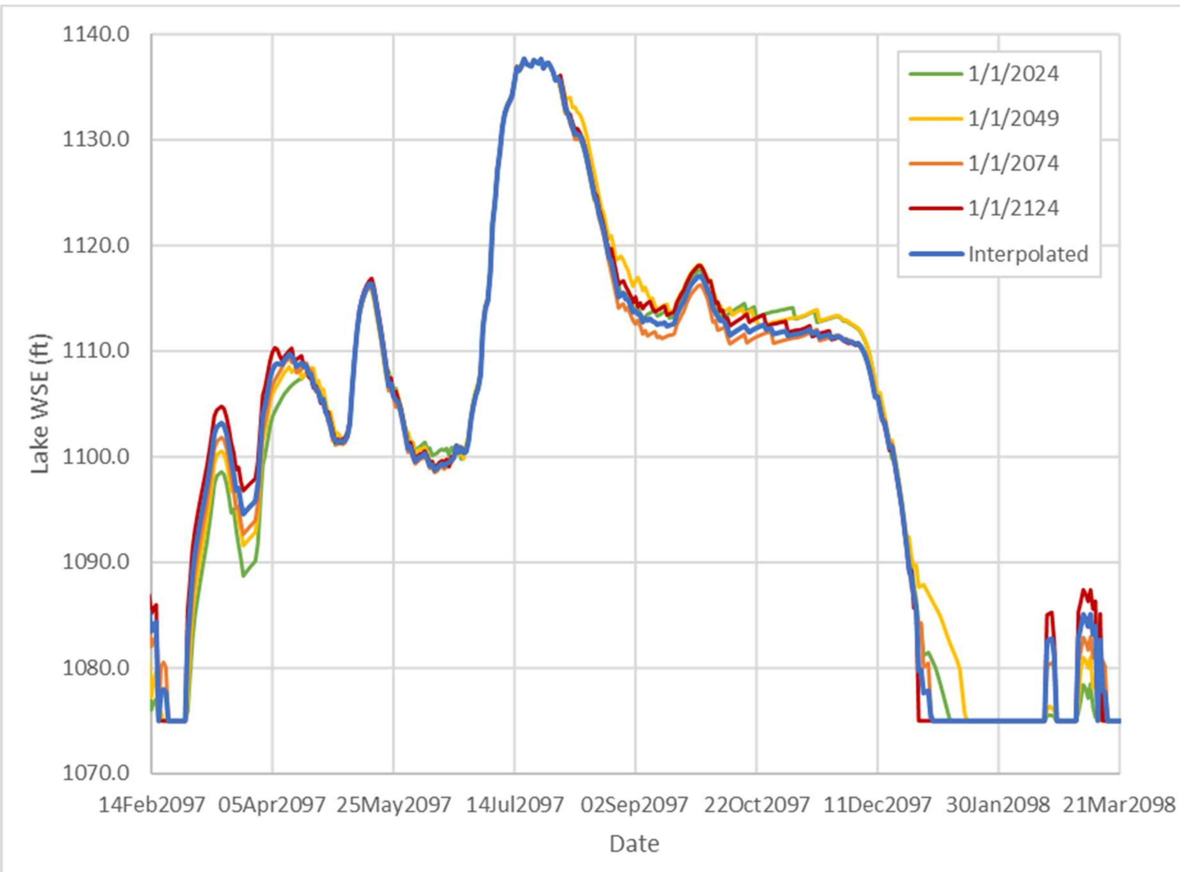


Figure 4-1: Example FWOP lake WSE from HEC-ResSim

The model run was broken into two separate runs because of a limitation in the number of time series points that could be used in the quasi-unsteady editor. A hotstart file for beginning the second half of the FWOP run was created from the gradations at the end of the first FWOP run. All the sediment model parameters discussed in Section 3 were used for the FWOP simulation.

Initially, the HEC-ResSIM Model used FWOP storage elevation curves generated using the Brune spreadsheet method in year 0, 25, 50, and 100 in order to compute future pool elevations. Because RAS depends on HEC-ResSIM for the pool elevation internal boundary condition and HEC-ResSIM depends on HEC-RAS for the storage-elevation curves, additional iterations of each were run. Figure 22 illustrates this process. only two iterations were made.

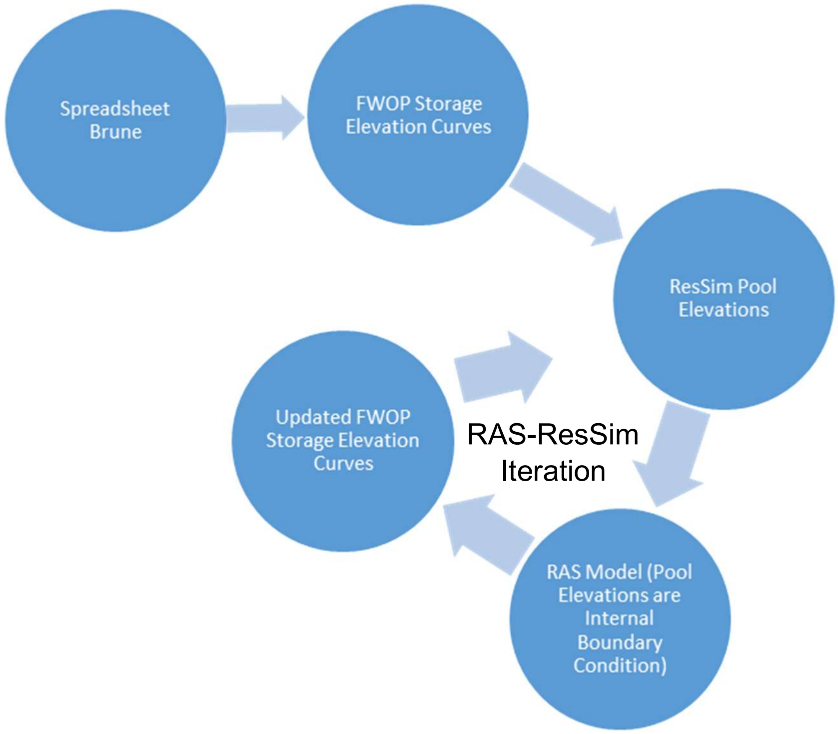


Figure 4-2: Process used to develop the final storage-elevation curves

5.0 FWOP RESULTS

As discussed in the previous section, total of two iterations between HEC-RAS and ResSIM, were performed. Table 5-1 provides the cumulative deposition (100 years) in the FP for each iteration. As, shown in Table 5-1, the second iteration actually produced less deposition in the FP than the first, indicating that additional deposition in the flood pool actually led to lower deposition in the second iteration. This could be caused by the operational rules in HEC-ResSIM, with the flood pool being evacuated sooner due to less storage.

Table 5-1: Cumulative deposition (ac-ft) over the FWOP

Date	Iteration 1	Iteration 2
1/1/2049	71,509	70,721
1/1/2074	131,390	113,346
1/1/2124	251,928	239,321

Results from the FWOP sediment model was used to determine the remaining volume in the MPP and FP over the FWOP and were compared to spreadsheet calculations from Appendix D2 (see Figure 5-1). A comparison of the spreadsheet and HEC-RAS results shows that the spreadsheet calculations overpredicted the rate of deposition within the MPP and under predicted the rate of deposition within the FP compared to the sediment model. These results would suggest then that over time more sediment will begin depositing in the FP as the MPP fill with sediment. There are multiple possible explanations for why this could happen. First, as the MPP fills with sediment, the surface area of the MPP shrinks, causing the surface area of the FP that can experience sediment deposition to increase. Second, deposition within both the MPP and FP causes the pool elevation to be higher over time because there is less capacity within the reservoir. This increased pool elevation could cause greater amounts of sediment to be deposited in the FP.

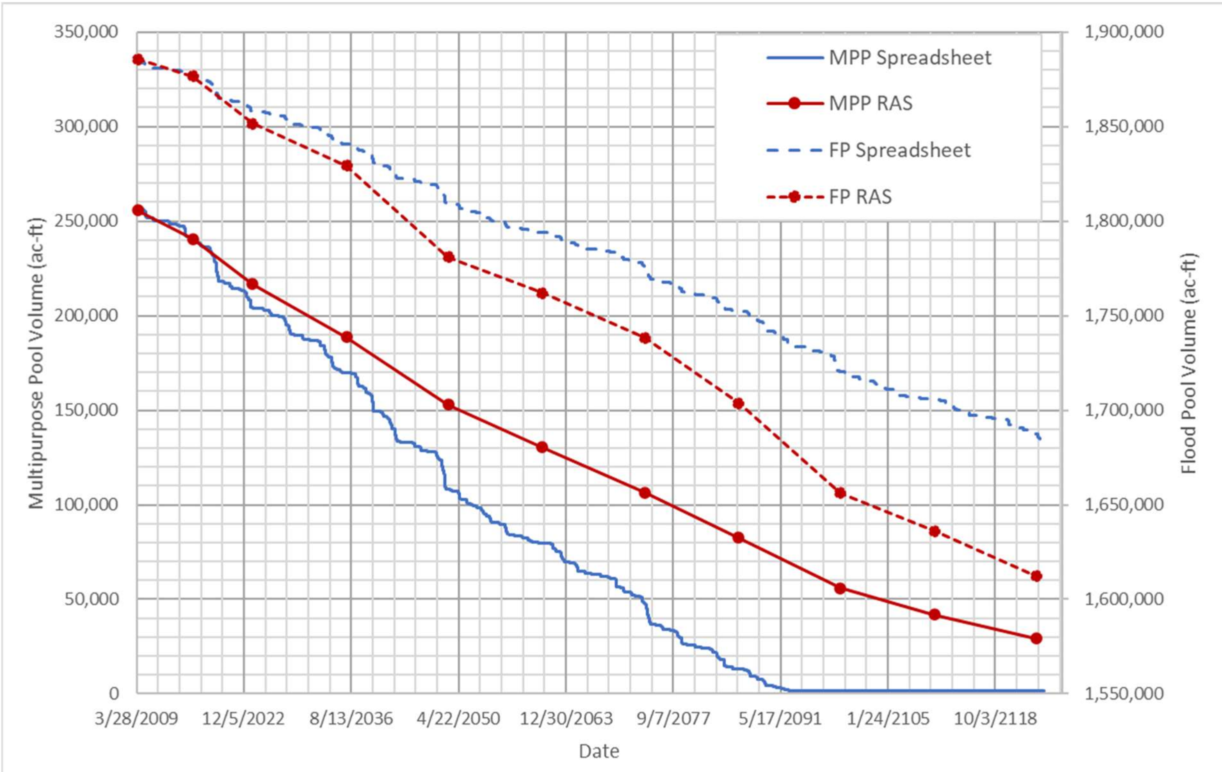


Figure 5-1: FWOP volume remaining in the FP and MPP from spreadsheet calculations and HEC-RAS model results.

Figure 5-2 shows the percentage of mass depositing in the FP versus the percentage of volume lost from the MPP for both the HEC-RAS model results and lake survey data. The lake survey data was obtained from the USACE reservoirs within the Kansas River Watershed. Also, plotted in Figure 5-2 is a polynomial regression equation fitted to the data in Excel. The percentages shown in Figure 5-2 were computed incrementally, meaning the percentage of mass to the FP was computed from successive surveys.

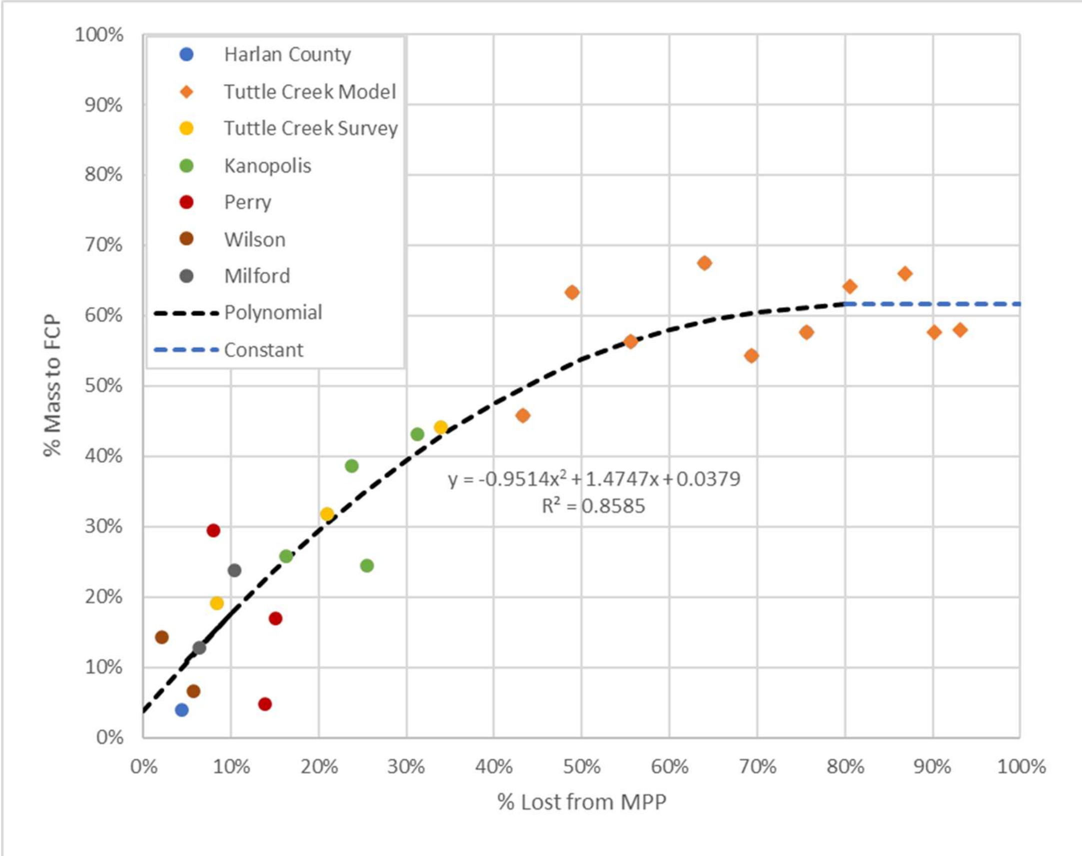


Figure 5-2: Percent of mass to the FP vs percentage volume loss in the MPP computed incrementally

Although there is a fair amount of scatter in the data in Figure 5-2, there is a discernable trend as indicated by the relatively high R squared value. The HEC-RAS model results show that the percent of mass depositing in the FP reaches a steady state at approximately 60%. Currently, it is not certain what is causing this effect. It could be that as the volume of the FP declines, it's trapping efficiency also declines, which could eventually counteract the reducing MPP volume. Another possibility is that the smaller FP volume allows the FP to be evacuated sooner after a flood event.

The spreadsheet computation procedure documented in Appendix D2 was modified using the polynomial regression equation shown in Figure 5-2 to account for greater amounts of sediment being deposited within the FP over time. This modified spreadsheet procedure produced results closer to the HEC-RAS model results as shown in Figure 5-3. The spreadsheet method still overpredicts deposition within the MPP, which could be caused by differences in the Brune and HEC-RAS trapping efficiencies.

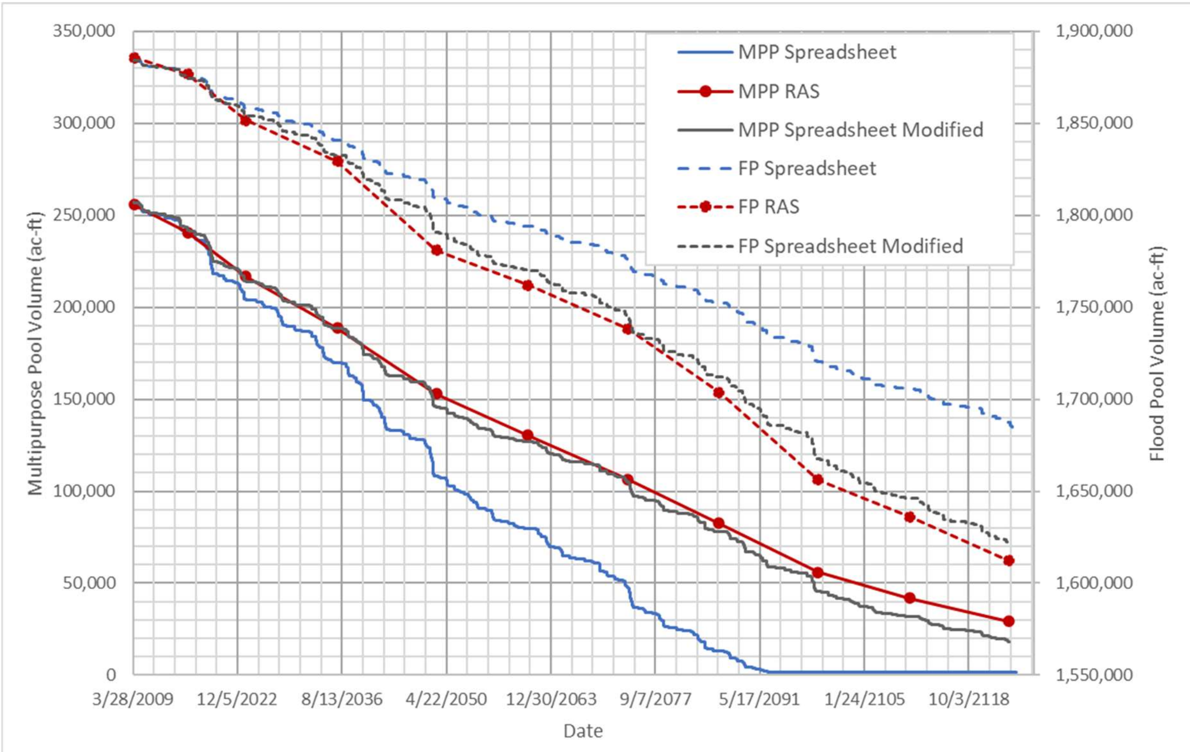


Figure 5-3: FWOP volume remaining in the FP and MPP from the original spreadsheet calculation, the modified spreadsheet calculations, and HEC-RAS model results.

The trapping efficiency of the reservoir was compared to the estimated trapping efficiency from the spreadsheet calculations (see Figure 5-4). Several conclusions can be drawn from Figure 5-4. First, the trapping efficiency from the HEC-RAS model does not drop as quickly as the results from the original spreadsheet calculations. This is likely caused by more sediment being deposited in the FP over time as previously discussed. Also, the trapping efficiency of the HEC-RAS model varies widely from year to year and exhibits increasing variability overtime. This is likely due to the shrinking of the MPP and variability in water inflows and lake operation from year to year. For example, in the year 2116, which is represented by 2011 in the flow data, computed trapping efficiency was nearly 100%. This high trapping efficiency is likely the result of water being held in Tuttle Creek Lake due to flooding on the Missouri River in 2011, which would have caused significantly longer detention times in the reservoir. As seen in Figure 5-3 and Figure 5-4, the Brune Modified Spreadsheet (which increases the % of deposition in the flood pool over time) provides a reasonable approximation for this phenomenon in the absence of modeling. For Tuttle Creek Lake, the FWOP was taken to be the model output. For the remaining reservoirs in the Kansas River Basin, the Brune Modified Spreadsheet (i.e., applying the equation from Figure 5-2 to change the % deposition in the flood pool), provides the FWOP.

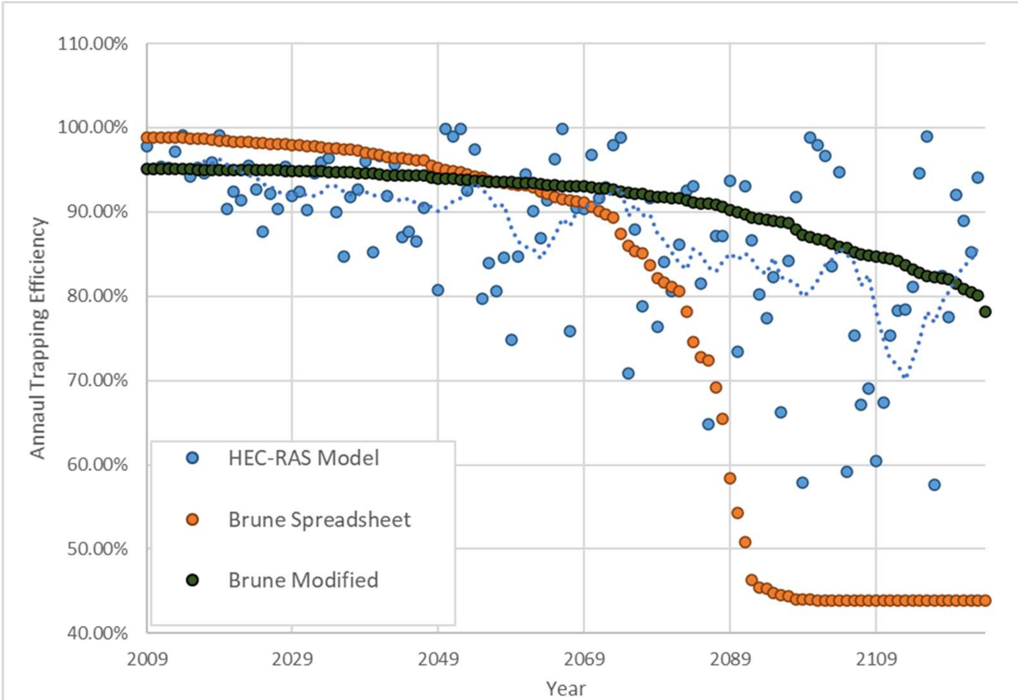


Figure 5-4: Annual trapping efficiency over time

The updated spreadsheet and ArcGIS methodology was compared to the results from the HEC-RAS modeling for both storage volume and surface area. See Figure 5-5 for a comparison of the surface area and Figure 5-6 for a comparison of the storage volume. As the figures show, the spreadsheet/GIS method compares well with the HEC-RAS results. For the spreadsheet calculations, the new area in the flood pool was estimated by multiplying the percent reduction in volume for the increment by the surface area at that increment. This is different than the original methodology, which assumed that the new area would be equal to the 2009 surface area for that volume.

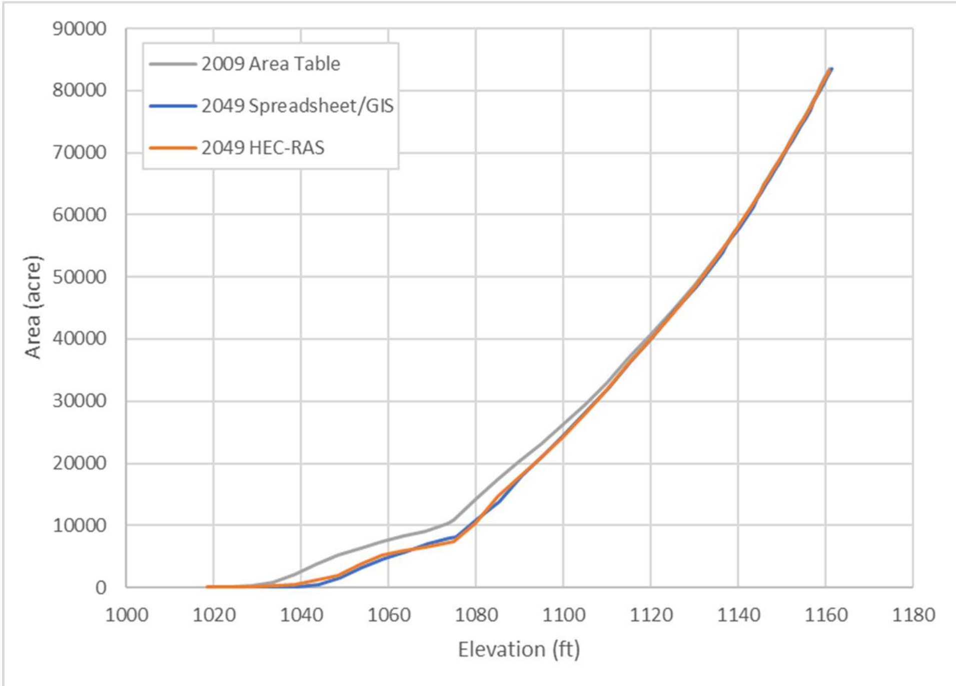


Figure 5-5: Surface area elevation curve from the HEC-RAS model and GIS/Excel computations

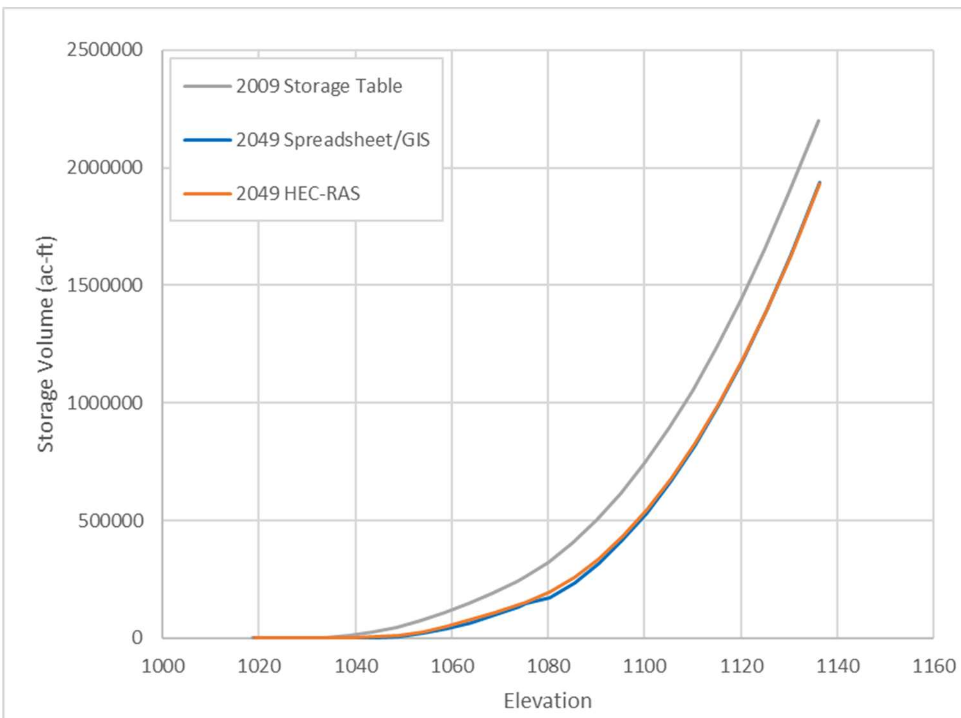


Figure 5-6: Storage elevation curve from the HEC-RAS model and GIS/Excel computations

6.0 CONCLUSIONS

Sediment transport modeling was completed for Tuttle Creek Lake to simulate FWOP sediment deposition within the reservoir. The model was calibrated to measured bed change computed from the 2009 and 2020 bathymetric surveys and was verified using measured volume change in the flood pool computed from 2010 and 2018 LiDAR. The model results generally agreed well with the measured data. The FWOP was simulated using boundary conditions produced from HEC-ResSIM modeling. FWOP model results showed that the spreadsheet calculations overpredicted the sediment deposition within the MPP and underpredicted it in the FP. This indicates that as the MPP fills in with sediment, greater amounts of sediment are deposited within the FP. Survey data from various lakes within the Kansas River Watershed also supports this conclusion.

7.0 REFERENCES

Juracek, K. E. (2011). *Suspended-Sediment Loads, Reservoir Sediment Trap Efficiency, and Upstream and Downstream Channel stability for Kanopolis and Tuttle Creek Lakes, Kansas, 2008-2010.* U.S. Geological Survey Scientific Investigations Report 2011-5187.

Juracek, K. E., & Mau, D. P. (2002). *Sediment Deposition and Occurrence of Selected Nutrients and other Chemical Constituents in Bottom Sediment, Tuttle Creek Lake, Northeast Kansas, 1962-99.* Lawrence, KS: U.S. Geological Survey.

Shelley, J., & Bailey, P. (2018). *The Cross Section Viewer: A Tool for Automating Geomorphic Analysis Using Riverine Cross-Section Data ERDC/TN RSM-18-3.* Vicksburg, MS: U.S. Army Corps of Engineers, Engineer Research and Development Center.

Shelley, J., & Wells, R. R. (2019). *Erodibility Characteristics of Cohesive Sediment Deposits in a Large Midwestern Reservoir.* Conference: Federal Interagency Sedimentation and Hydrologic Modeling ConferenceAt: Reno, NV.

Shelley, J., Gibson, S., & Williams, A. (2015). *Unsteady Flow and Sediment Modeling in a Large Reservoir Using HEC-RAS 5.0.* USACE.

Surdex. (2011). *Tuttle Creek Lake, Kansas Lakes LiDAR Mapping, Hydrographic Survey Data Integration & Flood Pool Survey.* Surdex Corporation.

USACE. (1973). *Lower Kansas River Basin Lake Regulation Manual in 6 Volumes, Volume No. 2, Tuttle Creek Lake, Kansas.* U.S. Army Corpose of Engineers.

USACE. (2022, March 8). *HEC-RAS 1D Sediment Transport*. Retrieved from HEC-RAS Documentation: https://www.hec.usace.army.mil/confluence/rasdocs

Williams, A., & Shelley, J. (2020). *Effects of Bank Stabilization on Regional Sediment Management: Lessons Learned from the Kansas River and Grand River Basins.* U.S. Army Corps of Engineers Engineer Research & Development Center.

US Army Corps of Engineers ®
Kansas City District

Kansas River Reservoirs Flood and Sediment Study

Appendix D5: Measures and Strategies to Reduce Reservoir Sediment Accumulation

June 2023

Contents

1.0 INTRODUCTION

The Existing Conditions and Future Without Project (FWOP) analyses indicate very significant impacts from reservoir sedimentation. This appendix summarizes analyses on how to reduce reservoir sediment accumulation. Two types of measures are considered: reducing the incoming sediment load and increasing the outgoing sediment load. Sediment load can be reduced through various means, including the installation of bank protection structures and cedar tree revetments, preventing failures of low-head dams, building grade control, and implementing stream/wetland complexes. Sediment removal can be accomplished via flushing, hydrosuction, traditional dredging, and water injection dredging. Sub-appendices provide more detail. Optimal long-term reservoir sustainability and watershed health will likely require some combination of both watershed sediment reduction and lake sediment removal.

2.0 REDUCING INCOMING SEDIMENT LOAD

Sediment that flows into Kansas River Basin lakes derives from eroded land and eroded channels. The U.S. Department of Agrigulture (USDA) and others focus significant efforts on soil conservation which translates into reduced sediment loads to downstream reservoirs.

Research in Kansas, however, has confirmed analyses around the country showing that channel sources constitute a significant, and in many watersheds the dominant, source of sediment. Juracek and Ziegler (2009) find that channel sources dominate the sediment load to Perry Lake. The same is likely true for other large reservoirs in the Kansas River Basin.

The following sub-sections describe ways to reduce channel sources of sediment.

2.1 Stabilize Bank Erosion Hot-Spots

The Kansas Water Office has published reports on erosional hotspots, i.e. areas where aerial photos indicate the banks have eroded at least 2,000 square feet. As of 2017, 176 un-stabilized hot-spots remained upstream from Tuttle Creek Lake in the State of Kansas. Stabilizing all 176 streambanks would prevent approximately 2,839 acre-ft of fine sediment (silts and clays) from entering Tuttle Creek Lake over 30 years (see Appendix D5.1). Note that the Tuttle Creek Lake watershed predominantly resides, in Nebraska. So several hundred erosion hotspots may be present beyond those delineated by the Kansas Water Office.

The State of Kansas has implemented dozens of stabilization projects in the Kansas River Basin using mostly a combination of bendway weirs, rock vanes, and longitudinal peaked stone toe (Figure 1).

Figure 1: Typical Bank Stabilization Project using Rock Toe and Bendway Weirs (Big Blue River)

Taking historic costs (without escalation) and normalizing for both linear feet and vertical feet of protection, the costs for the remaining hotspots would range from \$0.3/CY to \$2.2/CY of sediment reduction. This indicates that some amount of bank stabilization is almost certainly cost effective compared to sediment removal methods. However, the full 2,839 acre-ft of sediment reduction over 30 years is less than even a single year of accumulation in the multi-purpose pool. Moreover, some portion of the bank-derived sediment would settle out in the flood control pool and a small fraction would pass through to the downstream channel. This indicates that while stabilizing hotspots could be cost effective in some locations, it represents only a small part of the total sediment management solution.

2.2 Locate Erosion Hotspots in the Nebraska Portion of the Tuttle Creek Lake Watershed

Most of the Tuttle Creek Lake watershed is located in Nebraska. This upper watershed may feature numerous additional erosion hotspots. An aerial photo analysis similar to that undertaken in Kansas should be conducted for the Nebraska portion as well.

2.3 Bank Stabilization via Cedar Tree Revetments

Greater cost-effectiveness could be achieved by implementing low-tech bank stabilization using cedar tree revetments. Cedar tree revetments are comprised of longitudinally placed cedar trees intended to protect the toe of the eroding streambank. Traditionally, earth anchors secure the trees in place, though wooden stakes could also be used. Where working correctly, fine branches increase channel roughness on the bank toe, reduce velocity, and induce sediment deposition. If sediment can be captured, new vegetation will establish and further stabilize the toe, providing a stable lower bank for the mid and upper banks to revegetate.

Figure 2: Cedar Tree Revetment on McConnel Creek near Wichita, KS

Shelley et al (2022a) report high failure rates among cedar tree revetments in Kansas, with failure rates increasing with age according to Figure 3. However, Shelley et al (2022b) found that a cedar tree revetment at the Locust Creek Conservation Area in Missouri reduced erosion by 50 to 64% for the approximately 14 years it remained intact.

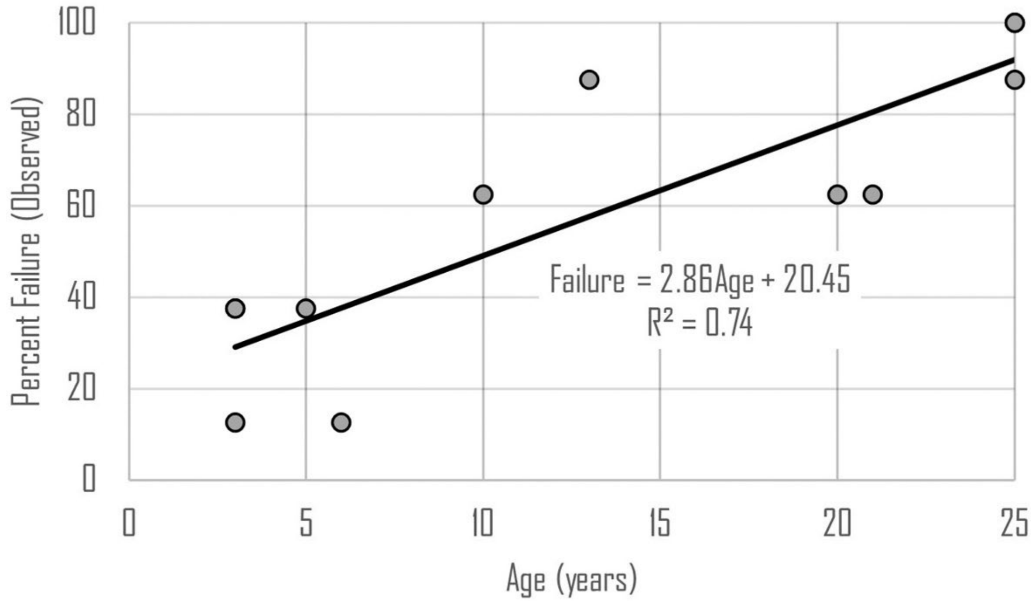


Figure 3: Failure Rates Among Cedar Tree Revetments in Kansas as a Function of Age

Notwithstanding the high failure rates, the low cost for cedar tree revetments may equate to a high cost-effectiveness for using cedar trees to reduce sediment loads to downstream lakes. As each project would have a miniscule impact on sediment load, hundreds to thousands of projects would be needed. These projects would be implemented on smaller streams, not large tributaries.

2.4 Stabilize Low-Head Dams

On May 4, 2018, the Big Blue River failed a low-head dam near Marysville, KS. This failure induced an upstream-migrating headcut which has contributed 696 ac-ft of sediment to Tuttle Creek Lake (see Appendix D5.2). At least 155 ac-ft additional sediment is expected to erode from just the Big Blue River and more is expected from tributaries.

Figure 3: Dam on the Big Blue River which Failed May 4, 2018.

A series of grade control structures could stabilize river beds upstream of this and other imperiled low-head dams. A comprehensive list of these dams and their structural and geomorphic condition is needed.

2.5 Locate and Stabilize Headcuts

Streams throughout the Kansas River Basin have been channelized which has resulted in bed incision and bank erosion that migrates upstream in the form of headcuts (see Figure 4). These headcuts destabilize the bed and induce bank erosion, which erodes land, reduces floodplain interaction, reduces floodplain deposition of sediment, and increases sediment and nutrient loads transported to downstream reservoirs.

Figure 4. Headcut in the Tuttle Creek Watershed

The number and locations of headcuts in the Kansas River Basin remain unknown. A cursory review of aerial photographs upstream of Tuttle Creek Lake indicates a significant number of incised locations.

Figure 5. Incising stream (Tributary to Booth Creek) in the Tuttle Creek Lake Watershed

Locating headcuts throughout the basin could be facilitated by developing a tool that uses readily available remotely-sensed data such a satellite imagery or LIDAR. These locations will then need to be field-truthed via drone, helicopter, or in-field site reconnaissance.

Engineered rock riffles act as grade control, arrest headcuts and reduce bank erosion upstream. They also provide the stability needed for riparian vegetation to establish, as well as riffle and deep pool habitats for aquatic species (Dodd and Wahl). The downstream slope of these structures is set to 1:15 to 1:20, which provides for fish passage. Figure 6 shows an example of an engineered rock riffle.

Figure 6. Engineered Rock Riffle, IL. Source: Dodd and Wahl (2006).

Significant reductions in downstream sediment loads have been achieved in other watersheds with widespread implementation of engineered rock riffles. For example, the Demonstration Erosion Control Project in the Yazoo River basin in northern Mississippi included 192 low drop grade control structures in addition to bank stabilization and other erosion control measures (USDA 2020). This project resulted in significant reductions to sediment deposition and the need for dredging in downstream flood control channels. Effort on a similar scale (hundreds) is needed to significantly reduce sediment loading to the lakes.

2.6 Build Stream/Wetland Complexes

As noted, streams throughout the Kansas River Basin have been channelized which has resulted in bed incision and bank erosion. These degraded channels confine high flows to within the channel banks, which increases the sediment yield to downstream reservoirs. In addition, these areas lack floodplain interaction or riparian wetlands. In some places, degraded streams lower the water table.

Downstream sediment loads could be reduced by creating riparian wetland/stream complexes (Figure 7), which restore channel/floodplain/wetland connectivity. These can be created either by combinations of engineered rock riffles, post-assisted log structures (Figure 8), or other means to induce inundation and saturation of the floodplain. The environmental benefits are similar to Stage Zero channels which include a sizeable increase in acreage of aquatic habitat, reduced sediment transport, and reduced nutrient loading to downstream lakes (see http://stagezeroriverrestoration.com/benefits.html). These areas also provide habitat for waterfowl.

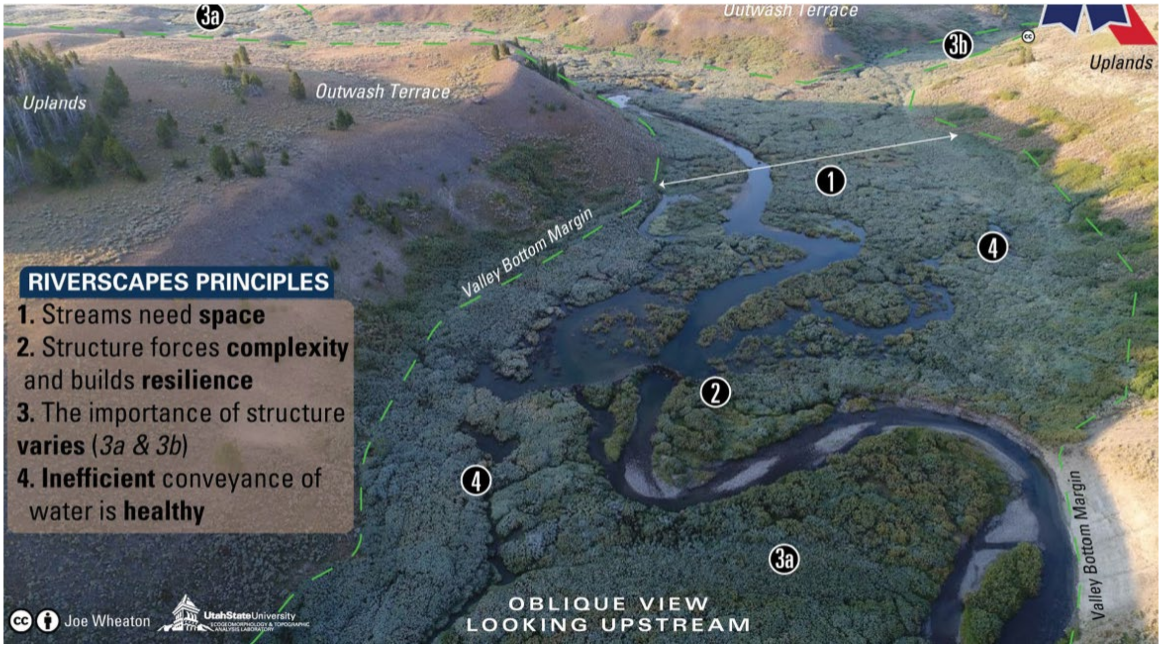


Figure 6. Stream/Wetland Complex Example (from JoeWheaton.org)

Figure 7. Post-Assisted Log Structure, aka Beaver Dam Analog, Installed by NRCS in Kansas

Figure 8 provides a conceptual illustration on Mill Creek, a tributary to Tuttle Creek Lake. This is presented for illustration purposes only; no project has been specifically planned for this area. Mill Creek is straight, incised, has little floodplain connection, and very efficiently transports sediment downstream. A stream/wetland complex in this location would consist in a series of wood structures to slow water and encourage overbank flow, plus an engineered rock riffle at the downstream end to prevent headcutting. In this complex, water would flow much more slowly through multiple small channels and wetland areas, and during even moderate events would flood the adjacent land, depositing sediment. The complex would be very productive habitat for fish and waterfowl and would also help remove nutrients from the water.

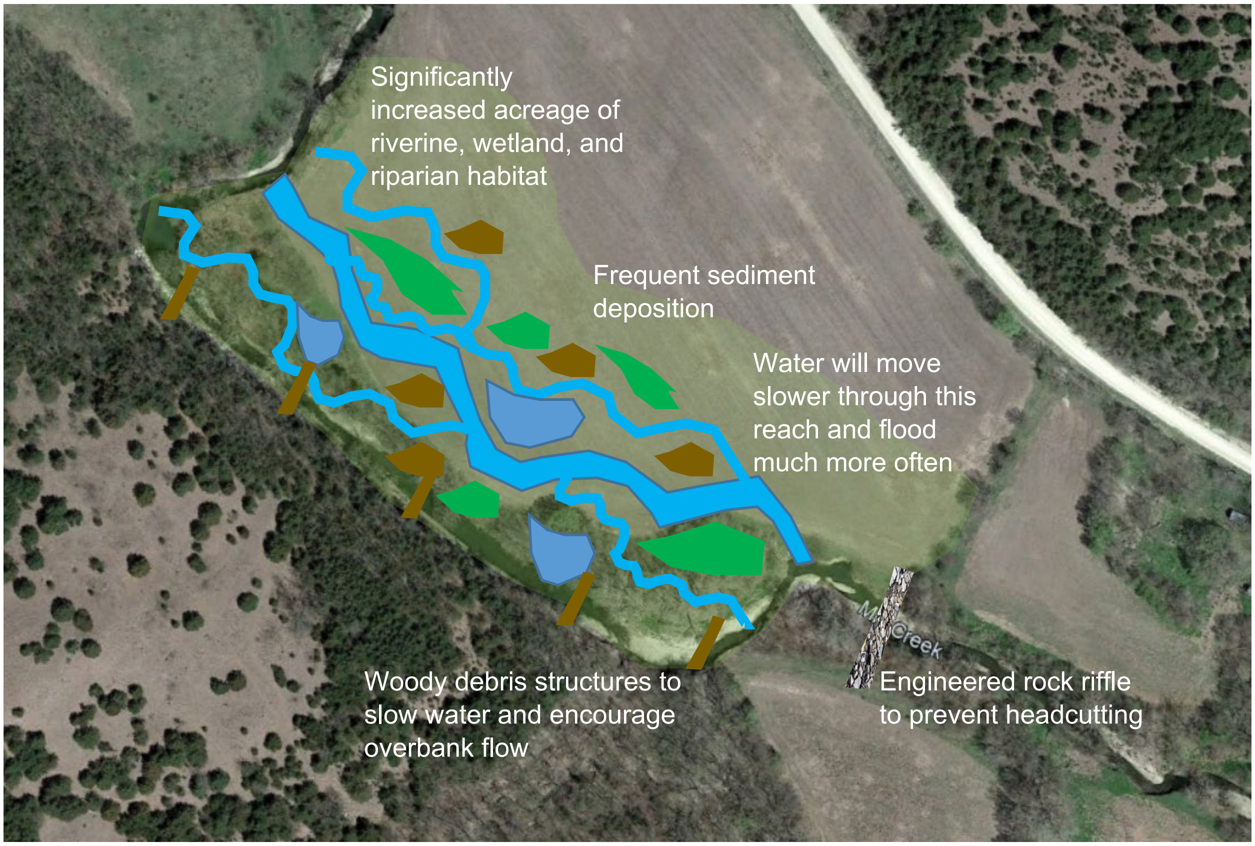


Figure 8. Conceptual Sketch for Riparian Stream/Wetland Complex

2.7 Policy/Authority Changes to Facilitate Sediment Load Reduction

USACE policies, authorities, and funding procedures are generally suited for large projects, wherein the land is acquired with fee title, that can be justified through benefits to a single business line. In contrast, the recommendations detailed above will need to feature hundreds of small projects, dispersed on the landscape. Moreover, they will realize a small amount (per structure) of mostly ecosystem benefits where they are implemented and have a cumulative benefit for the authorized purposes of the lakes (mostly water supply and recreation).

Policy changes that would facilitate implementation of actions summarized in this report are:

> Allow USACE to work on private land implementing low-tech options such as cedar trees and beaver dam analogs without acquiring fee title.
>
> Combine ecosystem and monetary benefits into a single quantitative metric so these types of “multi-benefit” projects can compete with single-benefit projects.

Recognize the Federal Interest in water supply and recreation uses of Federal lakes.

2.8 Impacts from Sediment Load Reduction

Figures 9ab, 9b, and 9c show the impact that sediment reduction from 5% to 20% would have on future multipurpose pool volumes at Tuttle Creek, Perry, and Kanopolis.

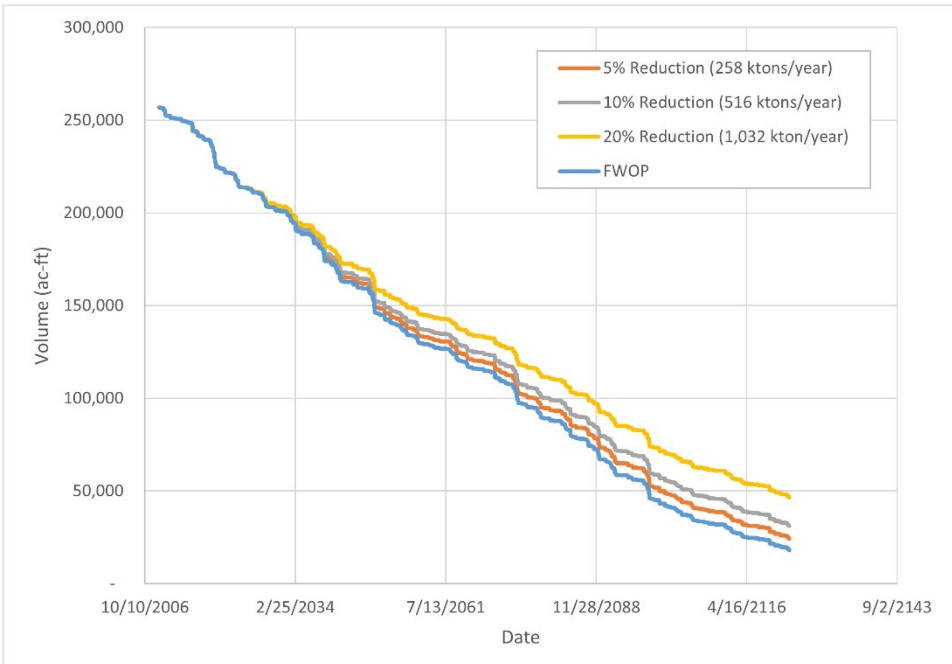


Figure 9a. Effect of Reduction in Sediment Loads on the Tuttle Creek Lake Multipurpose Pool

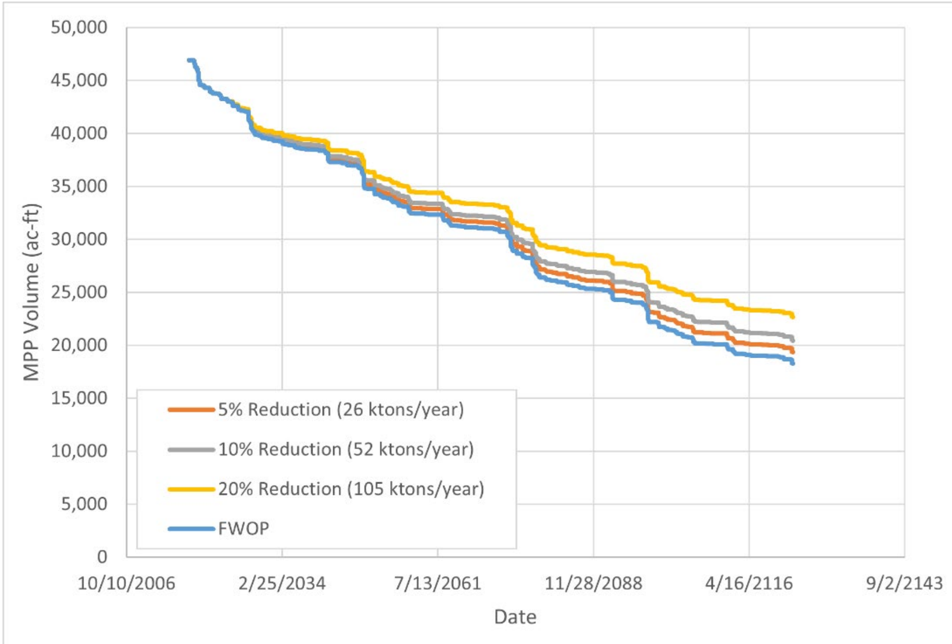


Figure 9b. Effect of Reduction in Sediment Loads on the Kanopolis Lake Multipurpose Pool

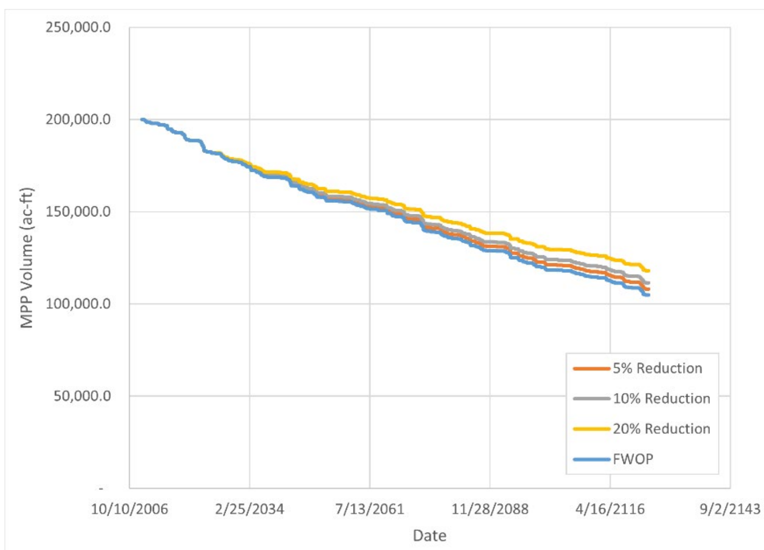


Figure 9c. Effect of Reduction in Sediment Loads on the Perry Lake Multipurpose Pool

As seen, very large reductions are required for only marginal improvements in projected volume. Thus, watershed practices can be best thought of as ways to reduce the volume of sediment that must be removed, rather than practices that will significantly maintain reservoir storage on their own.

3.0 INCREASE SEDIMENT DISCHARGE FROM LAKES

Even aggressive sediment reduction measures in the watershed only modestly improve future storage availability. Reservoir sediment management that meaningfully increases the useful life of the lakes will require intentional, ongoing sediment removal. This section summarizes methods to achieve sediment removal. All options restore sediment continuity, i.e. pass sediment to the downstream channel.

3.1 Dredging with Downstream Discharge

At least 50% of the cost of a lake dredging operation derives from the disposal of sediments into land-based confined disposal facilities. Traditional dredging methods, e.g. cutter-head suction dredges, could be more affordably employed by passing the sediment downstream. Dredging provides the maximum control over timing, discharge rate, location of removal, and grain size transported. Dredging could be combined with other, less expensive options as part of a comprehensive plan.

Feasibility-level analysis is recommended to compare the costs and benefits of traditional dredging vs. other sediment removal methods.

3.2 Hydrosuction

Hydrosuction is a variation of lake dredging that further reduces costs by using the head difference in the lake to power the sediment removal, transport, and discharge downstream. Hydrosuction is essentially a siphon. At the Kansas River Basin lakes, new hydrosuction conduits would need to be built to allow a pipeline to cross from the lake to the downstream channel; gate configurations preclude using the existing conduits.

Appendix D5.3 provides analysis for each of the Kansas River Basin lakes. Figure 9 depicts the configuration analyzed at Tuttle Creek Lake. Figure 10 provides the increased lake storage volumes with this hydrosuction system in place compared to the Future Without Project (FWOP). As seen, the sediment removal is significant even though hydrosuction cannot pass enough sediment to achieve perpetual sustainability.

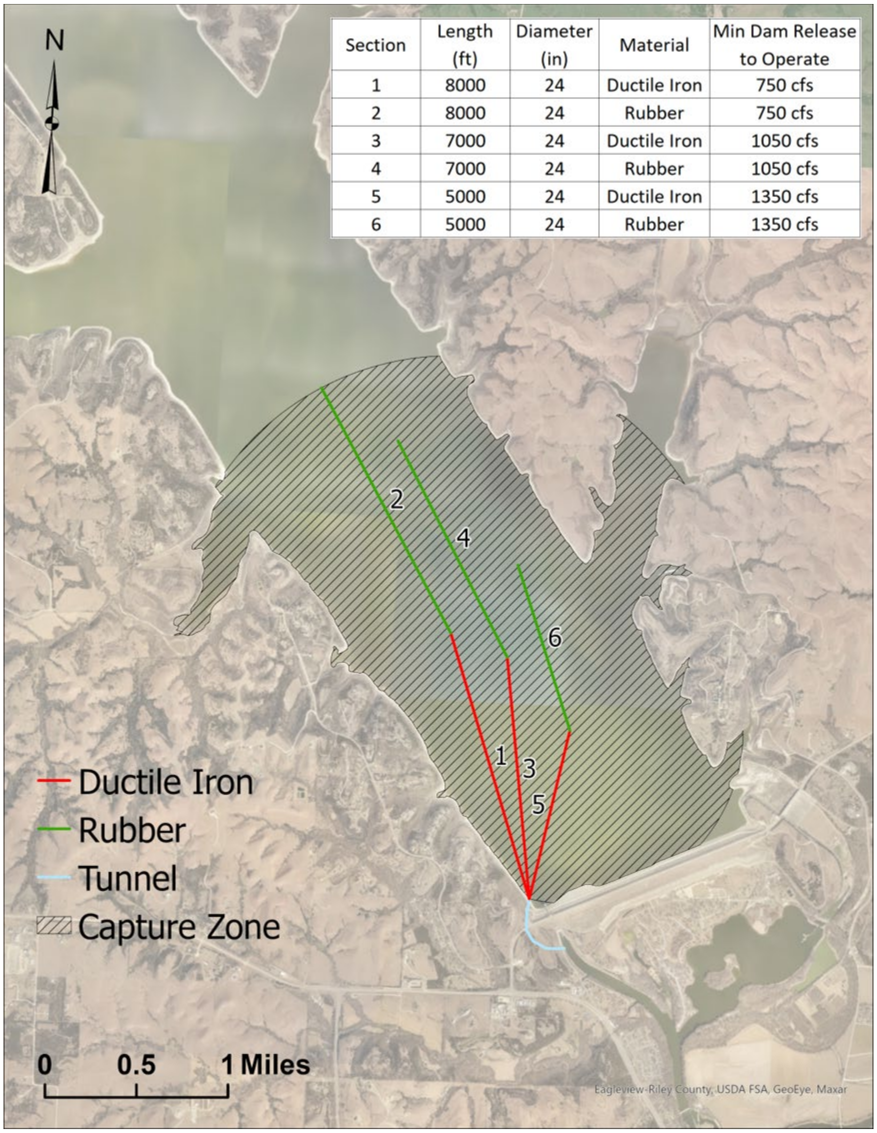


Figure 9. Hydrosuction System Analyzed for Tuttle Creek Lake

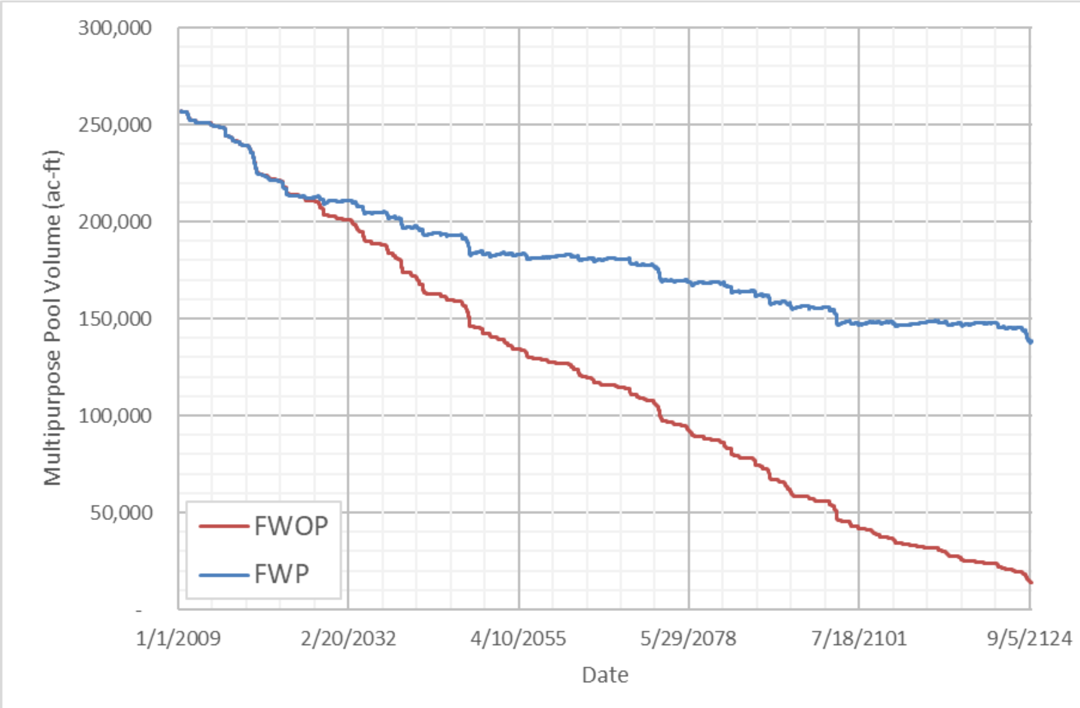


Figure 10. Projected Multipurpose Pool Volume with Hydrosuction (FWP) vs. without Hydrosuction (FWOP) at Tuttle Creek Lake

Table 1 presents the multipurpose pool storage at each lake with operating hydrosuction.

Table 1. Additional Storage (Compared to FWOP) With Hydrosuction Implemented. See Appendix D5.3 for details.

Year	Clinton	Perry	Tuttle Creek	Milford	Kanopolis	Wilson	Harlan County
2049	+7,231	+13,140	+40,311	+11,614	+7,130	+3,338	+3,934
2074	+13,492	+20,915	+58,170	+19,780	+10,497	+7,053	+5,934
2125	+21,531	+30,405	+98,252	+34,303	+19,561	+11,170	+11,742

3.3 Water Injection Dredging

Water Injection Dredging (WID) reduces dredging costs even further by eliminating the need for a new conduit and eliminating all the pipes. Rather than pumping a sediment slurry thousands of feet as with traditional dredging, WID pumps clear water tens of feet down to the reservoir bed. The jets of water fluidize the bed, inducing a density current which flows downslope. Kansas River Basin lakes have service gates on the lake bottom which can be opened to allow sediment to pass downstream.

A WID demonstration project is taking place at Tuttle Creek Lake which will provide information on WID effectiveness in lake settings.

3.4 Drawdown Flushes

A very inexpensive way to remove sediment from a lake is through a drawdown flush. Drawdown flushes are the most common reservoir sediment management method in the world.

A drawdown flush consists in a complete lowering of the reservoir pool down to riverine conditions. Over a short period of time (typically one to two weeks), the river scours and transports downstream a significant quantity of deposited sediment. Flushing also moves coarser sediments from the delta towards the dam.

Flushes work best for hydrologically small reservoirs, i.e. where the storage to be maintained is small relative to the ability of the watershed to refill. Dahl and Ramos-Villanueva (2019) summarize:

> The primary screening criteria for successful flushing of reservoirs in current guidance are based on three factors: the total capacity of the reservoir (CAP), the mean annual runoff to the reservoir (MAR), and the mean annual inflow of sediment to the reservoir (MAS). The lower the ratios of CAP/MAR and CAP/MAS, the more likely it is that flushing can be a successful method for maintaining reservoir capacity.

Several authors have compiled the CAP/MAR and CAP/MAS ratios of reservoirs where flushing has occurred in order determine the range of ratios over which flushing is most likely to be fully successful at maintaining the reservoir capacity. Figure 11 includes data from all the Kansas River Basin lakes, plus projected Future Without Project conditions for Perry Lake, Tuttle Creek Lake, and Kanopolis Lake, on a plot modified from Annandale (2013). The original data in the plot, shown in gray, indicates reservoirs where various sediment management strategies have been successfully employed. The green dots indicate existing conditions for all the federal lakes in Kansas River Basin. The red, blue, and purple dots indicate projections for Tuttle Creek Lake, Kanopolis Lake, and Perry Lake, respectively. The dashed boxes encapsulate clusters of dams where a given management strategy has been successfully employed to achieve sustainability, described in Annandale (2013) as at least a 300 year life.

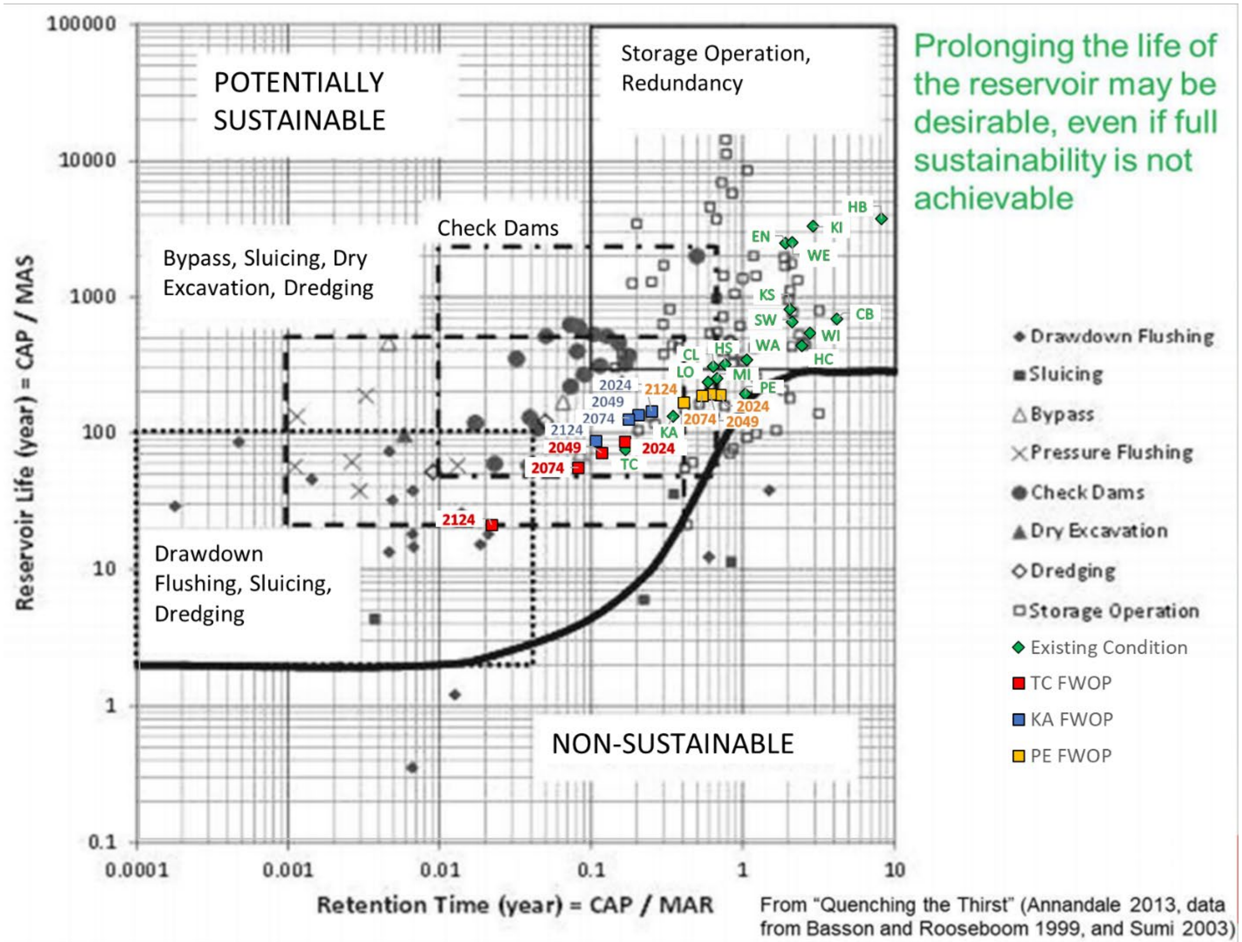


Figure 11. Sediment Management Alternatives Screening

As seen, none of the Kansas River Basin lakes currently fall within the ratios where flushing has been successful at achieving full sustainability. Flushing could be more effective in the future; as the reservoirs shrink due to sedimentation, the CAP/MAR and CAP/MAS ratios decrease, making flushing more effective. According to Figure 11, flushing becomes feasible for Tuttle Creek Lake in between 50 and 100 years.

Figure 11 is limited for several reasons. It does not include all options such as hydrosuction, water injection dredging, intensive sediment reduction in the watershed, or combinations of measures. Nor does it indicate which strategies could prolong the life of a dam even if full sustainability is not achievable.

On flushing specifically, it does not take sediment type into account or allow an assessment of longer drawdown flushes than the typical 1 to 2 weeks. An empirical analysis for drawdown flushing at Tuttle Creek Lake (detailed in Appendix D5.4 and summarized below) overcomes these limitations.

A 4-month drawdown flush at Tuttle Creek Lake could remove 15,800 ac-ft during an average flush. This would return the submerged channel to pre-impoundment conditions after a single flush. However, as is common during flushes, the submerged floodplain would become stranded as the lake draws down. Thus, flushing would have to be combined with other sediment removal means to achieve full sustainability.

Flushing would release sediment loads well in excess of natural levels, which could harm aquatic species in downstream channels. Moreover, draining the lake would decimate the sport fishery. Finally, a lake in its drained condition cannot supply water during drought periods. Thus, flushing represents the method with the least expense but the highest impacts.

3.5 Policy/Authority Changes to Facilitate Sediment Removal

Policy changes that would facilitate implementation of sediment removal actions summarized in this report include:

Combine ecosystem and monetary benefits into a single quantitative metric so these types of "multi-benefit" projects can compete with single-benefit projects.

Recognize the Federal Interest in water supply and recreation uses of Federal lakes.

Update Regulatory Guidance Letter 05-04 to allow a streamlined and predictable permitting process that recognizes the importance of and facilitates restoring sediments to historically turbid systems.

Recognize restoring sediments to sediment-starved systems as a beneficial use.

Implement new economic analyses for reservoirs that account for long-term sustainability.

Implement the recommendations by the USACE Environmental Advisory Board on reservoir sediment management (attached as Appendix D5.5).

3.6 Sediment Removal Conclusion

Each sediment removal method has different effectiveness, costs, and impacts. The tradeoffs between methods, or combinations of methods, require feasibility-level comparisons.

Feasibility studies for reservoir sediment management are recommended for Tuttle Creek Lake, Kanopolis Lake, and Perry Lake. Continued monitoring is recommended for the other lakes.

4.0 CONCLUSIONS

Sediment management at lakes in the Kansas River Basin is needed to reduce the many impacts listed earlier in the report. The two ways to reduce sediment accumulation are to reduce the sediment coming into the lakes and to increase the sediment leaving the lakes. Long-term sustainability and watershed health will require a combination of the two strategies.

Recommendations for reducing sediment load include:

1. Implement bank stabilization at cost-effective hot-spots.
2. Implement cedar tree revetments
3. Create a comprehensive list of low-head dams in Kansas, along with their geomorphic condition
4. Locate headcuts
5. Build engineered rock riffles
6. Develop stream/wetland complexes using rock riffles and/or beaver dam analogs

Even with aggressive watershed sediment reduction measures, sediment removal that passes the sediment downstream will be necessary. Recommendations for sediment removal include:

1. Assess the effectiveness of Water Injection Dredging at Tuttle Creek Lake with a demonstration project
2. Conduct feasibility studies at Tuttle Creek Lake, Kanopolis Lake, and Perry Lake. Assess trade offs between dredging, hydrosuction, water injection dredging, and drawdown flushing.

Policy/Authority Changes that could facilitate these actions include:

1. Allow USACE to work on private land implementing low-tech options such as cedar trees and beaver dam analogs without acquiring fee title.

2. Combine ecosystem and monetary benefits into a single quantitative metric so these types of “multi-benefit” projects can compete with single-benefit projects.

3. Recognize the Federal Interest in water supply and recreation uses of Federal lakes.

4. Update Regulatory Guidance Letter 05-04 to allow a streamlined and predictable permitting process that recognizes the importance of all sediments in historically turbid systems.

5. Recognize restoring sediments to sediment-starved systems as a beneficial use.

6. Implement new economic analyses for reservoirs that account for long-term sustainability.

7. Implement the recommendations by the USACE Environmental Advisory Board on reservoir sediment management.

5.0 REFERENCES

Annandale, G. 2013. Quenching the Thirst: Sustainable Water Supply and Climate Change. CreateSpace Independent Publishing Platform. ISBN: 1480265152, 9781480265158

Dahl, T. and Ramos-Villanueva, M. "Overview of historical reservoir flushing events and screening guidance." ERDC/CHL CHETN-VII-21. U.S. Army Corps of Engineers, Vicksburg, MS. https://hdl.handle.net/11681/33003

Juracek, K., and Ziegler, A. 2009. "Estimation of sediment sources using selected chemical tracers in the Perry lake basin, Kansas, USA." International Journal of Sediment Research, 24(1), pp 108-125. https://doi.org/10.1016/S1001-6279(09)60020-2.

KWO (Kansas Water Office). 2017. Tuttle Creek Watershed Streambank Erosion Assessment. https://kwo.ks.gov/docs/default-source/streambank-erosion-assessments/tuttlecreek_sbassessment_041717_rdr.pdf?sfvrsn=5cfa8614_2

Shelley, J., C. Haring, and N. Chrisman. 2022a. "Failure Modes in Cedar Tree Revetments: Observations on Rivers and Streams in Eastern Kansas, USA." River Research and Applications. Wiley Online Library. https://doi.org/10.1002/rra.3997 .

Shelley, J., Haring, C., and Chrisman, N. 2022b. Evaluation of Cedar Tree Revetments for Bank Stabilization at the Locust Creek Conservation Area, Missouri. ERDC/CHL Technical Report 22-22. U.S. Army Corps of Engineers, Vicksburg, MS.

USDA. 2020. Demonstration Erosion Control Project. Website: https://www.ars.usda.gov/southeast-area/oxford-ms/national-sedimentation-laboratory/watershed-physical-processes-research/docs/demonstration-erosion-control-project/ . Last modified 20 May 2020. US Department of Agriculture, Agricultural Research Service.

Wheaton, J. 2023. Retrieved from JoeWheaton.org.

US Army Corps of Engineers ®
Kansas City District

Kansas River Reservoirs Flood and Sediment Study

Appendix D5.1: Cost Effectiveness of Stabilizing Bank Erosion Hotspots in the Tuttle Creek Watershed

June 2023

Contents

1.0 INTRODUCTION

The purpose of this analysis is to estimate the amount of sediment that could be prevented from entering Tuttle Creek Lake if eroding streambanks in the upstream watershed were stabilized. Tuttle Creek Lake has been filling with sediment and is losing storage capacity. Some of the sediment depositing in the lake is a result of bank erosion at upstream locations. The Kansas Water Office (KWO) performed a bank stabilization analysis in 2017 that analyzed the most severe bank erosion locations (KWO, 2017). The KWO analysis quantifies the erosion rate at each of 176 un-stabilized bends based on the surface area of erosion delineated between two aerial photos. KWO (KWO, 2017) assumes the sediment reduction of the bank stabilization is equal to the pre-project volumetric erosion rate. The analysis presented in this memo refines the KWO analysis by analyzing two additional components: deposition on the bank opposite from the erosion and the percent washload.

The rivers upstream from Tuttle Creek are migrating, i.e. widening on one side while depositing on the other. Thus, the net export of sediment from a bend is the erosion on the outside bend minus the deposition on the inside bend. Conceivably, if the erosion were stopped, the deposition would also be stopped, so the actual benefit of stabilization should be the net erosion (erosion minus deposition).

Because sands from eroding banks supply bed material to the rivers, reducing the sand load through bank stabilization likely induces bed degradation. In the short term, degrading beds supply additional sand to downstream reaches, so Tuttle Creek Lake will not see a reduction in incoming sand. On geomorphic time scales, the reduction in sand will lower slopes throughout the basin and lead to a decrease in sand transport into the lake, but this process could take many decades. On the other hand, the wash load in the banks, consisting mostly of fine sands, very fine sands, silts, and clays (Williams and Shelley, 2020), would very quickly transport downstream and deposit in Tuttle Creek Lake without significant geomorphic interaction with the bed. This represents the sediment reduction benefit over a project time scale, which is of most interest for decreasing sedimentation in Tuttle Creek Lake.

This memo presents a refinement factor that can be applied to the KWO erosion estimates in order to subtract (1) the deposition on the inside bend and (2) the bed load fraction of the bank material. This refinement factor is then applied to all 176 erosion hot spots in order to determine the potential sediment reduction from bank stabilization. The sites are then prioritized and approximate costs included in order to determine the cost per cubic yard of sediment reduction. This allows bank stabilization to be evaluated for cost effectiveness against other sediment management methods at Tuttle Creek Lake.

2.0 METHODS

KWO provided a shapefile with the "erosion hot spots" within the Tuttle Creek watershed. Erosion hotspots were classified as locations with at least 2,000 ft^2 of erosion. This shapefile contained 176 sites that had not been stabilized. Of these sites, 16 sites, shown in Figure 1, were selected to perform deposition analysis in order to formulate a refinement factor.

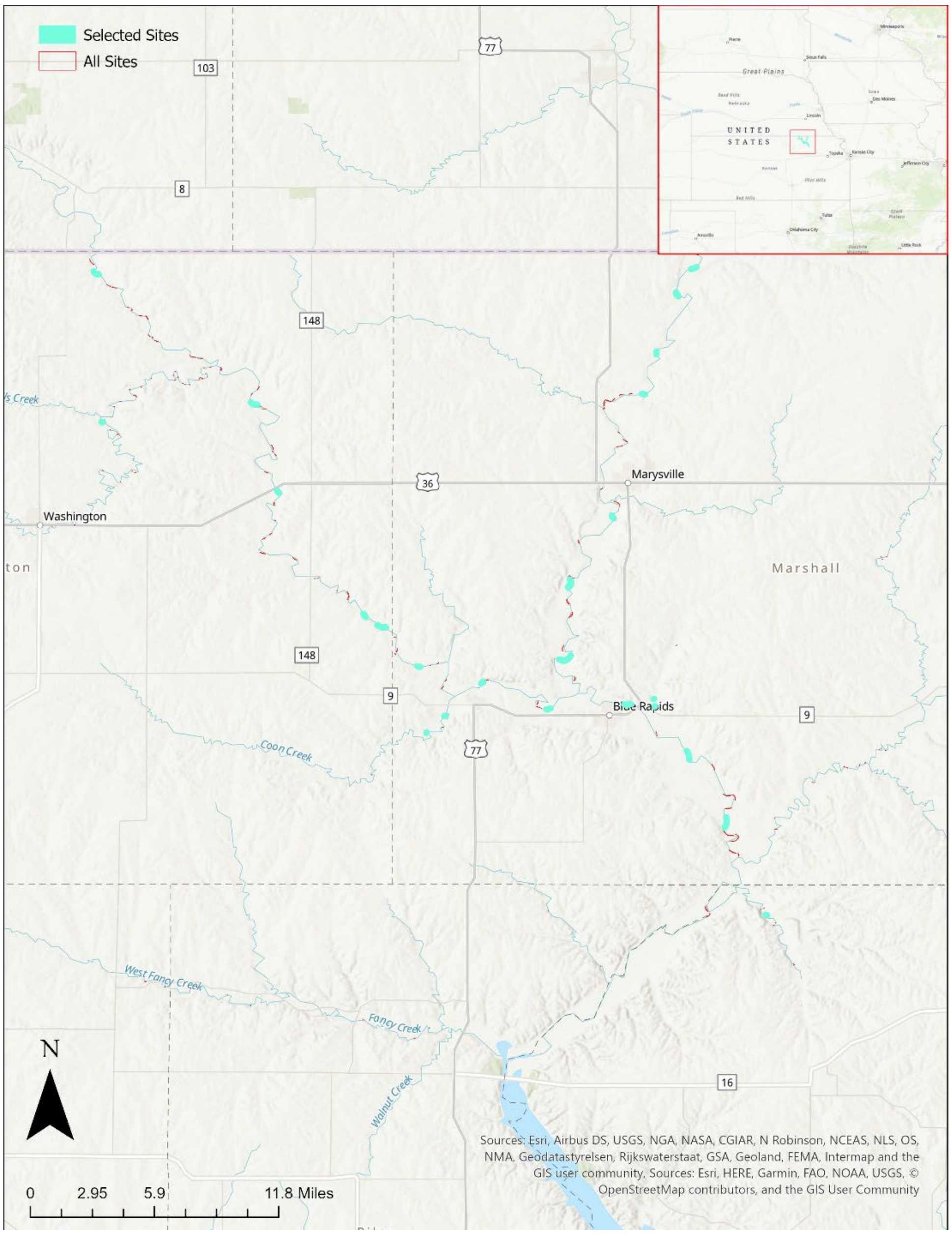


Figure 1: Map of Selected Sites

These 16 sites were selected to represent the watershed in terms of size and geographic location. The un-stabilized sites had bank heights ranging from 5 ft to 30 ft and areas ranging from 0.03

acres to 5.32 acres. The selected sites had bank heights ranging from 12 ft to 24 ft and areas ranging from 0.15 acres to 4.40 acres. Sites were also selected on several tributaries and at upstream and downstream locations. On the opposite bank of the selected sites, polygons were constructed at a 1:2500 scale to represent the deposition of sediment between the same years that were used to delineate the erosion. NAIP imagery was used to delineate these sites. Only deposition with vegetation was considered for this analysis because deposition without vegetation could still be considered bedload rather than permanent deposition (see Figure 2).

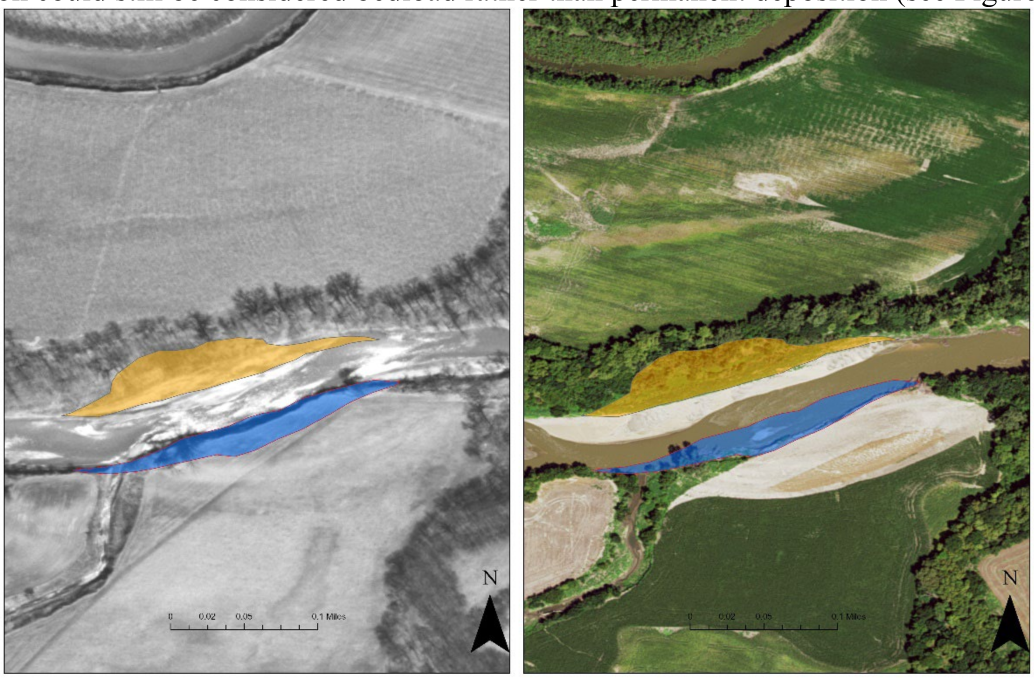


Figure 2: Deposition (orange) compared to Erosion (blue) between 2002 (left) and 2015 (right)

Once the selected sites were delineated for deposition, the bank height was estimated using LiDAR from 2009 and 2010, provided by the Kansas Data Access and Support Center (DASC, 2020). The volume of deposition was then calculated and a ratio of deposition to erosion was calculated for each site.

The percent wash load was estimated based on samples taken along the Big Blue River and the Little Blue River (Williams and Shelley, 2020). These samples were used to form gradation (by mass) of the sediment along several banks. According to Einstein (1950), the wash load could be estimated as the grain size for which 10% of the bed mixture is finer. Williams and Shelley (2020) calculated the percent wash load by mass at 4 sites along the Little Blue River and 3 sites along the Big Blue River. This percent wash load by mass was converted to a percent wash load by volume by assuming the following bulk densities: 30 pcf for clay, 65 pcf for silt, and 93 pcf for sand. The data presented by Williams and Shelley (2020) did not differentiate between clay and silt. For this analysis, half of the sediment smaller than 0.0625 mm was assumed to be silt and the other half was assumed to be clay. The percent wash load by volume was then averaged across the sites along each river to obtain a representative percent wash load by volume for each river. The volumetric percent wash load was estimated as 75% for the Big Blue River and 68% for the Little Blue River. For sites that were not on the Big Blue River or the Little Blue River, the wash load percentage was assumed to be the same as that for the closest river (either the Big Blue River or the Little Blue River).

The reduction in erosion at each site was then calculated using the percent fines and deposition. The reduction equation was given as:

$$R = \frac{E - D}{E} * \%wash\ load$$

Where:

- R=Reduction Factor
- E=Erosion (ft^3)
- D=Deposition (ft^3)
- %wash load= percentage (volume) of the bank that is finer than 90% (by mass) of the bed mixture (Einstein, 1950)

This factor was averaged over the 16 sites to form a representative reduction factor for the erosion.

Once the reduction factor was calculated, the factor was applied to the KWO erosion estimates at all 176 sites across the watershed. This yields the expected reduction in sediment input to Tuttle Creek Lake that could be achieved by stabilizing each site.

3.0 RESULTS

Table 1 below shows the net erosion of the selected sites. Accounting for deposition reduces the erosion estimates by 39%. Accounting for deposition *and* considering only wash load reduces the erosion by 57%, i.e. the adjustment factor to account for these two processes is 0.43.

Table 1: Net Erosion Reduction Accounting for Deposition (D) and Percent Wash load (WL)

Site ID	Stream Name	Area	Percent WL	$E-D$	$\frac{E-D}{E}$	$(E-D)*\%WL$	$R = \frac{(E-D)}{E} * \%WL$
		acre	%	ft^3	-	ft^3	%
BBR57	Big Blue R	0.76	75	729566	0.63	547175	47%
BBR13	Big Blue R	2.27	75	-245566	-0.18	-184174	-13%
LBR1	Little Blue R	1.76	68	-26297	-0.02	-17882	-1%
BBR7	Big Blue R	1.73	75	1749290	0.52	1311967	39%
BBR41	Big Blue R	0.86	75	1076354	0.73	807266	55%
BBR16	Big Blue R	1.09	75	547735	0.46	410801	35%
LBR16	Little Blue R	0.31	68	552565	0.83	375745	56%
LBR10	Little Blue R	0.44	68	525911	0.75	357619	51%
CC5	Coon CR	0.14	68	140297	0.83	95402	56%
CC7	Coon CR	0.04	68	90501	0.92	61541	62%
SPC4	Spring CR	0.05	75	126117	0.87	94588	65%
LBR24	Little Blue R	0.45	68	1019211	0.73	693064	50%
LBR78	Little Blue R	0.69	68	1654845	0.73	1125295	49%
LBR62	Little Blue R	0.38	68	1558364	0.92	1059687	62%
LBR50	Little Blue R	0.50	68	337946	0.49	229804	33%
MCN26	Mill CR	0.14	68	87159	0.54	59268	37%
	Average	0.72	70.63	620250	0.61	439198	43%

The net erosion of washload at each site was then calculated with the following equation:

$$F = A * BH * R$$

Where:

- F=Net erosion of washload (ft^3)
- A=Surface area of erosion (ft^2)
- BH=Bank height (ft)
- R=Average reduction factor to account for deposition and fines (0.43)

This yielded the net erosion between the two years of the aerial photos. The net erosion over the lifetime of a bank stabilization project (assumed to be 30 years) was calculated for each site. The erosion rate was then calculated on a per year, per linear foot, per vertical foot basis. The sites were then ranked from the greatest erosion rate to the smallest erosion rate. This would rank the sites from most cost-effective to least cost-effective with regards to volume of sediment saved and cost of stabilization. The total erosion over the life of the project was then totaled for a cumulative sediment volume.

Once the net erosion had been determined at every site that had not been stabilized, the cost of stabilizing sites was calculated. In previous studies, stabilization costs were estimated at $71.50 per linear foot (KWO, 2017). This value does not account for differences in the vertical height of the bank, which would impact the stabilization cost. Therefore, the stabilization cost was divided by the average bank height of all eroding banks, yielding a stabilization cost of $4.02 per linear foot per vertical foot. This cost estimate was applied to each site. The cumulative cost for stabilizing sites (sorted from most cost-effective to least cost-effective) and the cumulative volume of sediment can be seen in Figure 3 below. The cumulative unit cost was also calculated for each site, which would be useful when comparing bank stabilization to other sediment management options. Figure 3 therefore shows the relationship between the cumulative sediment reduction, the cumulative cost for stabilization, and the average unit cost for stabilization. For example, if 2,000 acre-ft is the sediment reduction goal, it would cost an estimated $5 million dollars and the cost for the last increment of stabilization was $1.50 per cubic yard.

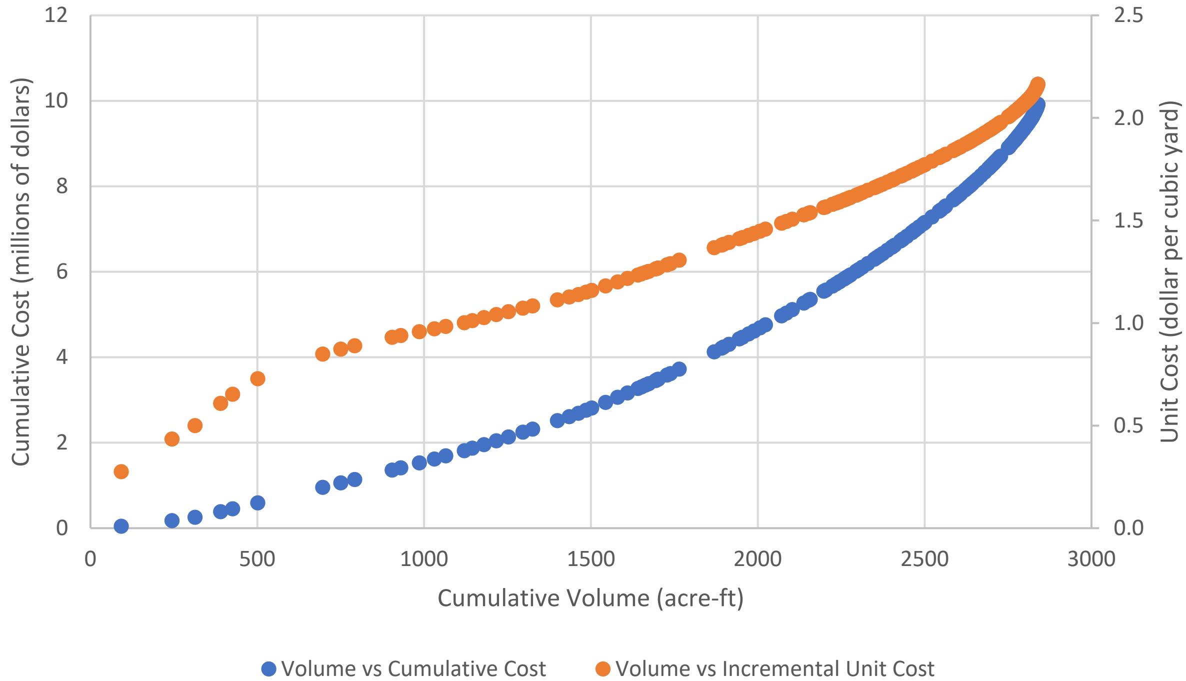


Figure 3: Cumulative Cost and Volume Reduction

According to Figure 3, bank stabilization reduces sedimentation at a cost of \$0.3/CY to \$2.2/CY, indicating that some amount of stabilization is almost certainly cost effective compared to other sediment management options. (For comparison, dredging at John Redmond Lake in 2015 cost \$6.7/CY of sediment removal.) However, stabilizing all 176 streambanks would prevent only approximately 2,839 acre-ft of sediment from entering Tuttle Creek Lake over 30 years, which is less than even one year of accumulation in the multi-purpose pool. Moreover, some portion of the bank-derived sediment would settle out in the flood control pool, so the impact on multipurpose pool storage is further reduced. Finally, not all sediment that enters the multipurpose pool will deposit; a small fraction will pass through. The percentage that passes through depends on the trapping efficiency, which depends on the lake volume and therefore varies over time.

3.0 HYDROLOGIC SIMILARITY

KWO's original estimates for bank erosion compared aerial photographs from 1991, 2002, and 2003 with aerial photographs from 2015. The erosion rate experienced between those photographs depends on the hydrology. To obtain a better understanding of the hydrologic similarity between these years, the average daily flows were analyzed at three gages. The gages used were Gage 06884400 at Little Blue River near Barnes, KS, Gage 06885500 at Black Vermillion River near Frankfort, KS, and Gage 06882510 at the Big Blue River near Marysville, KS. The number of days with average daily flows greater than the 1.2-year return period were counted for each time period between photographs. The 1.2-year return period was used because it has been shown as a reasonable estimate for bankfull flows in Kansas streams (Shelley, 2012). The flows for a 1.2-year return period ($Q_{1.2}$) at the specified gages can be seen in Table 2 below.

Table 2: 1.2-Year Return Period Flows (From Appendix D1)

Gage	83.3% (1/1.2) AEP Discharge (cfs)
Big Blue River at Marysville, KS	9,080
Little Blue River at Barnes, KS	7,110
Black Vermillion River at Frankfort, KS	3,360

The average daily flows above $Q_{1.2}$ were counted in each time period. The total days were then converted to an average number of days per year by dividing by the number of years in the analyzed period. This average number of days per year above $Q_{1.2}$ can be compared to the average annual number of days above $Q_{1.2}$ for the entire period of record. If the specified time period averaged more days above $Q_{1.2}$ than the period of record, higher flows were experienced, and erosion was most likely overpredicted. However, if the specified period averaged fewer days above $Q_{1.2}$ than the period of record, lower flows were experienced, and erosion was most likely underpredicted. These results can be seen in Table 3 below. Table 4 also shows the number of sites analyzed by KWO in each time period.

Table 3: Days above $Q_{1.2}$

	Little Blue	Black Vermillion	Big Blue
1.2 Year Flow (cfs)	7110	3360	9080
1991-2015 Days >$Q_{1.2}$	93	96	126

Average Number of Days above $Q_{1.2}$ per year	3.72	3.84	5.04
2002-2015 Days >$Q_{1.2}$	31	32	49
Average Number of Days above $Q_{1.2}$ per year	2.21	2.28	3.50
2003-2015 Days >$Q_{1.2}$	31	32	48
Average Number of Days above $Q_{1.2}$ per year	2.38	2.46	3.69
Period of Record >$Q_{1.2}$	270	240	223
Average Number of Days above $Q_{1.2}$ per year	4.30	3.57	6.13
Period of Record	5/1/1958-2/6/2021	10/1/1953-2/5/2021	10/1/1984-2/5/2021

Table 4: Number of Sites Analyzed in each Time Period

Period of Record	**Number of Sites**
1991-2015	70
2002-2015	24
2003-2015	66

Table 3 indicates the analyzed periods averaged fewer days above $Q_{1.2}$ than the period of record, indicating lower flows than typical. Thus, erosion may have been underpredicted, and the bank stabilization measures presented in this report may be slightly more effective than predicted.

Analysis over a longer interval could provide a more representative erosion rate. However, given the relatively small contribution of the bank erosion hotspots to the overall sediment accumulation, additional detail is not needed.

4.0 CONCLUSION

This analysis approximates the cost effectiveness of bank stabilization as a measure for reducing sedimentation in Tuttle Creek Lake. The Kansas Water Office estimates for bank erosion should be multiplied by 0.43 to only include washload sediments and to account for deposition on the inside bend. The results indicate bank stabilization may be a cost effective option for reducing a small amount of sedimentation. However, stabilizing all 176 sites would reduce the sedimentation by less than 1 year's worth of accumulation.

Two additional analyses could refine this analysis in the future: (1) measure the bulk density of the in-situ bank material in order to compute an accurate mass of material. And (2) apply the sediment reduction over an implementation timeline in order to reduce the accumulation by the temporally-trending trap efficiency of the lake. Given the relatively small contribution of erosion hotspots to total sediment loading, further refinement is not needed during the Watershed Study.

5.0 REFERENCES

DASC. 2020. Kansas Data Access and Support Center-Your resource for geospatial data from the State of Kansas. Retrieved September 10, 2020, from https://www.kansasgis.org/

Einstein, H. A. 1950. The Bed-Load Function for Sediment Transportation in Open-Channel Flows. Washington, DC: US Department of Agriculture, Soil Conservation Service.

KWO (Kansas Water Office). 2017. Tuttle Creek Watershed Streambank Erosion Assessment. https://kwo.ks.gov/docs/default-source/streambank-erosion-assessments/tuttlecreek_sbassessment_041717_rdr.pdf?sfvrsn=5cfa8614_2

Shelley, 2012. Geomorphic Equations and Methods for Natural Channel Design. https://kuscholarworks.ku.edu/handle/1808/9836?show=full

Williams, A., and J. Shelley. 2020. Effects of Bank Stabilization on Regional Sediment Management: Lessons Learned from the Kansas River and Grand River Basins. ERDC/TN RSM-20-1. Vicksburg, MS: US Army Engineer Research and Development Center. http://dx.doi.org/10.21079/11681/35313

US Army Corps
of Engineers ®
Kansas City District

Kansas River Reservoirs Flood and Sediment Study

Appendix D5.2: Marysville Low Head Dam

June 2023

Contents

1. Introduction

On May 4th, 2018, a low-head dam on the Big Blue River near the city of Marysville, KS failed. The dam was originally built for hydro-electric purposes, but had not been operational for several decades. Deposition upstream of the dam had nearly filled to the top of the dam. As a result of the dam failure, headcutting was observed upstream of the dam, which resulted in increased sediment loads to Tuttle Creek Lake. This analysis estimates the sediment loads contributed to Tuttle Creek Lake so far, the projected sediment loads the headcut will continue to supply, and a cost comparison for stabilizing the headcut.

2. Methods

LiDAR was obtained from the Kansas Data Access and Support Center (DASC) website to determine the pre-failure conditions of the Big Blue River. The pre-failure LiDAR was flown on April 12th, 2018, less than 1 month before the dam failed. Post-failure LiDAR was then flown by the United States Army Corps of Engineers (USACE) Engineering Research and Development Center (ERDC) on August 1st, 2022. These two LiDAR datasets were compared to determine the total amount of sediment that had eroded from the Big Blue River and the future erosion potential. The study reach extends approximately 8.5 kilometers upstream of the dam (see Figure 1).

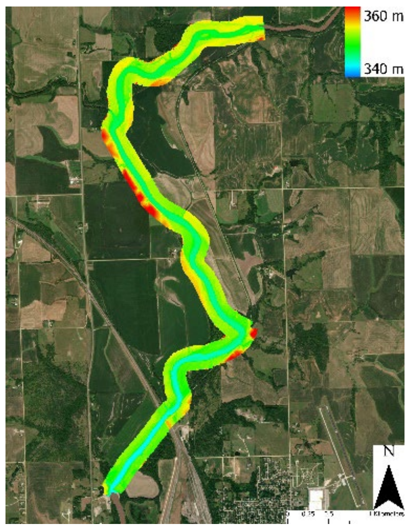


Figure 1: LiDAR extents from August 1st, 2022

Because LiDAR does not penetrate the water surface, it was necessary to collect the LiDAR at approximately the same discharge for each dataset. On April 12th, 2018, the discharge of the Big Blue River at Marysville was approximately 283 cfs. On August 1st, 2022, the discharge of the Big Blue River at Marysville was approximately 185 cfs. Stage and discharge field measurements obtained by the USGS can be seen in Figure 2 below. At these discharges, the gage height at Marysville was approximately equal, meaning the error introduced by the water surface elevation was roughly equal in both LiDAR datasets. Therefore, the change in water surface elevation can be used as a surrogate for the change in bed elevation. However, this may over-estimate sediment that was present pre-dam in the backwater area.

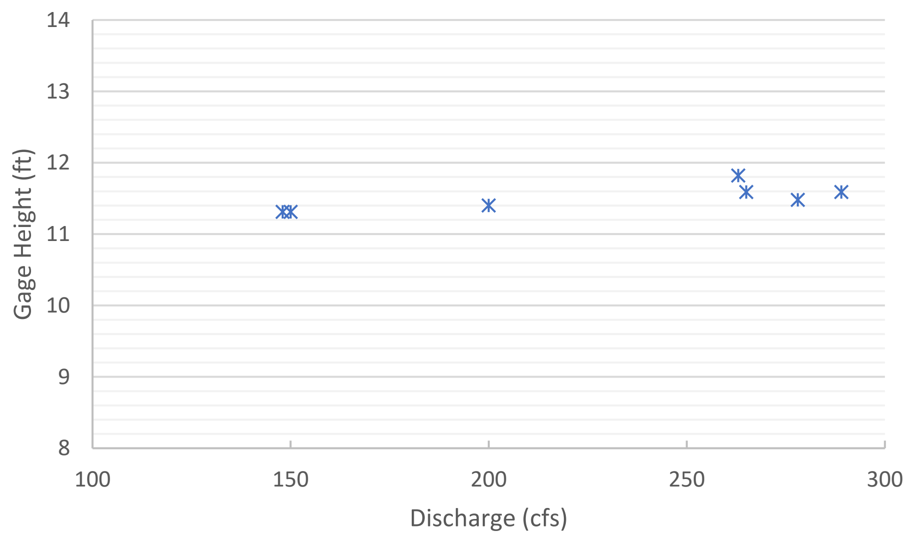


Figure 2: Stage-Discharge Rating Curve at Marysville. Field observations obtained between 8/15/2017 and 4/24/2018.

2.1. Past Erosion

To determine the erosion that occurred between 2018 and 2022, the two LiDAR datasets were subtracted from one another in ArcGIS Pro. The Surface Volume tool was then used to calculate the total difference between the two datasets. The net erosion within the study reach was calculated by subtracting the deposition from the erosion. The net erosion was used because some of the sediment from erosional areas may have re-deposited within the reach, which would not contribute to the sediment loads entering Tuttle Creek Lake.

2.2. Future Erosion

Estimates for future erosion were calculated by identifying the headcut from the centerline profile of the 2022 LiDAR and projecting the volume up to the point at which erosion is expected to stop. Cross sections were cut along the stream at locations with abrupt slope changes (see Figure 3). Two cross-sections were cut downstream of the headcut (at locations which are expected to have stabilized) and seven cross-sections were cut at locations that are still experiencing degradation.

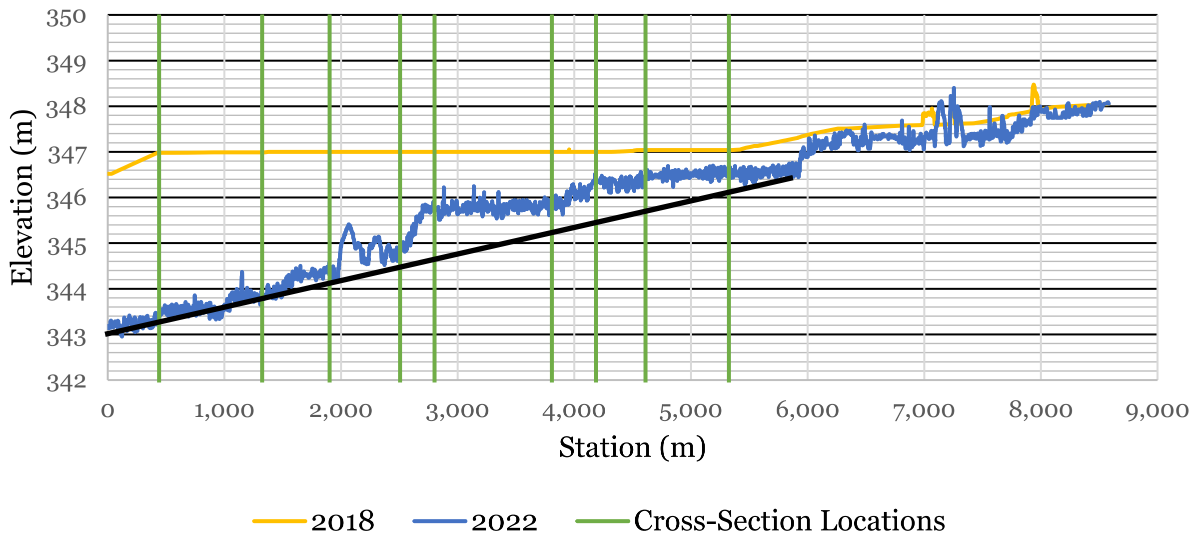


Figure 3: Longitudinal Profiles form 2018 and 2022, as well as cross section locations and projected steady-state bed profile. The headcut appears to be between station 2,000 and 3,000, with the main headcut appearing at approximately station 2,800. The drawdown near station 6,000 is indicative of erosion-resistant material, which indicates the headcut will most likely stabilize at station 6,000. The black line is the estimated steady-state bed profile.

Pre-failure vs. post-failure cross sections were compared at the two most downstream cross sections to produce an “area-adjustment factor.” The area-adjustment factor was calculated by dividing the change in area between the 2018 cross-section and the 2022 cross-section by the change in bed elevation (see Equation 1). This adjustment factor was applied because as the channel invert lowers as a result of headcutting, channel banks steepen until the critical slope is reached. At this point, the banks fail and the channel widens, thus contributing additional sediment. Therefore, this analysis accounts for both the increased sediment contributions from the bed and the banks.

$$AAF = \frac{A_2 - A_1}{d_2 - d_1} \qquad \text{Equation 1}$$

where A_2 is the final cross-sectional area (2022)

A_1 is the beginning cross-sectional area (2018)

d_2 is the final invert elevation of the cross-section, and

d_1 is the beginning invert elevation of the cross-section.

The invert of the cross section was defined as the elevation of the profile displayed in Figure 3. Cross sections 1, 2, and 9 can be seen in Figure 4-Figure 6.

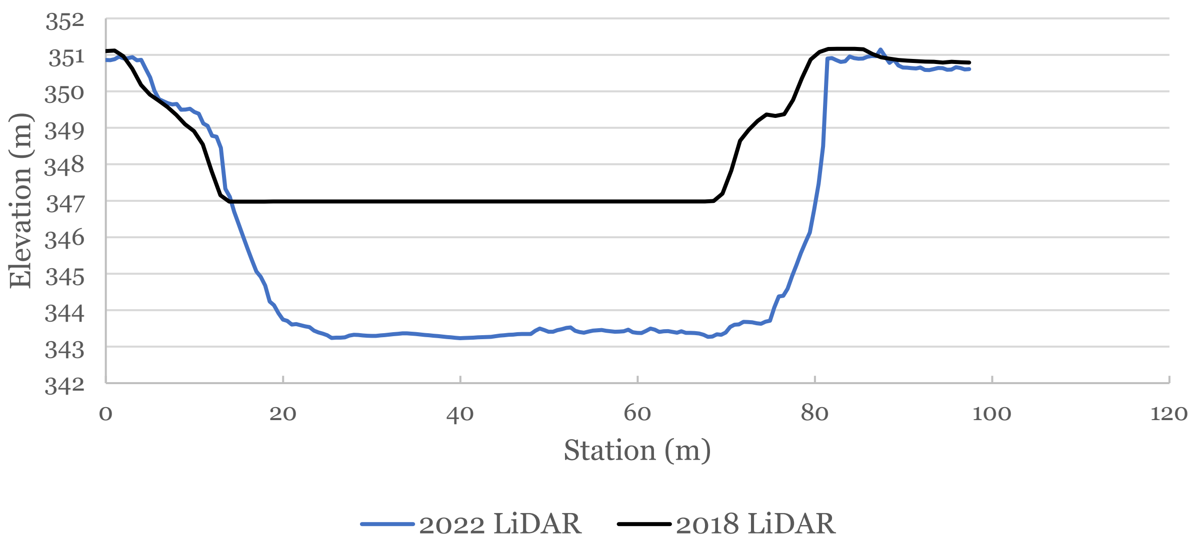


Figure 4: Cross Section 1. Assumed to have stabilized.

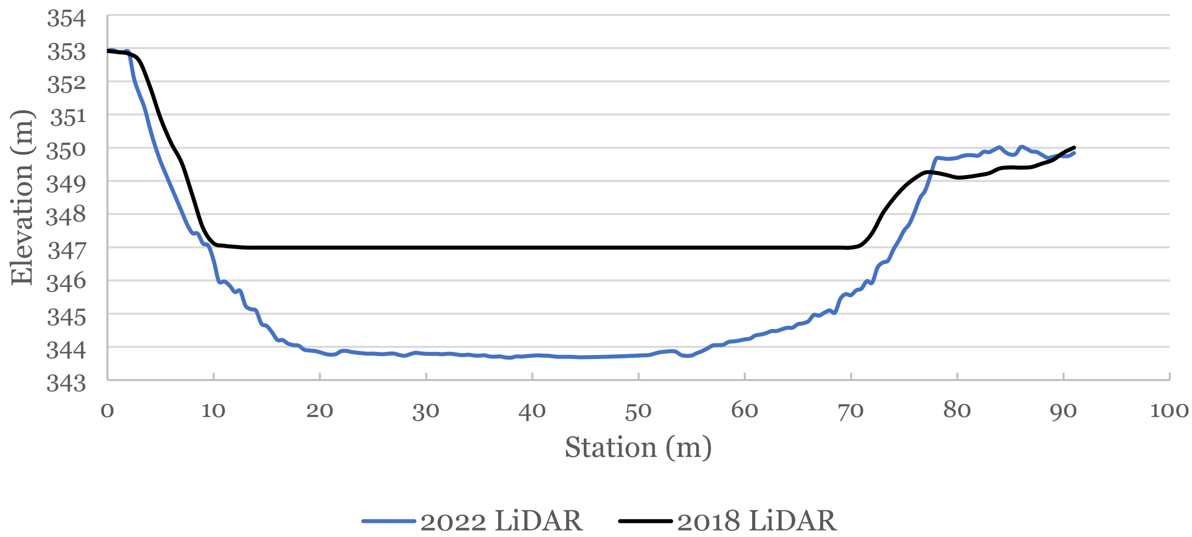


Figure 5: Cross Section 2. Assumed to have stabilized.

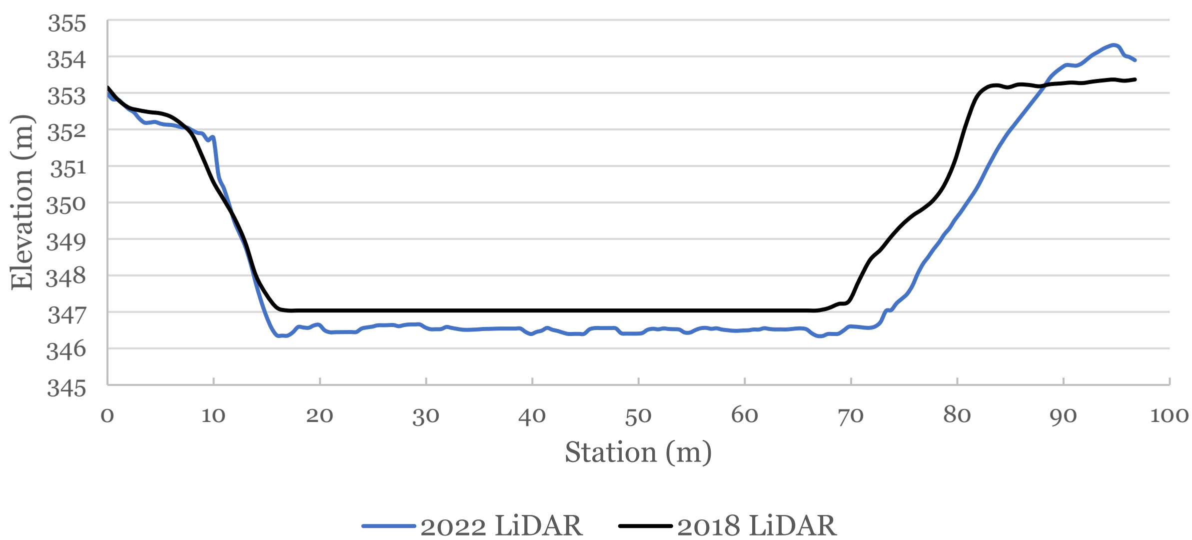


Figure 6: Cross Section 9. Has yet to fully erode.

The arithmetic mean of the area-adjustment factors for the first two cross sections was then calculated and applied to the 7 cross sections that have not yet stabilized to determine the stabilized cross-sectional area. The A_1 and d_1 were obtained from the 2022 data, as this portion of the analysis was interested in the future sediment projections. The d_2 was obtained from the steady-state bed profile shown in Figure 3. The changes in area between A_2 and A_1 were then averaged between the specified cross-section and the next cross-section upstream. The average change in area of cross-sections over a reach was then multiplied by the length of the reach to determine the volume of sediment that eroded.

3. Results

3.1. Past Erosion

A map showing the areas of degradation and aggradation can be seen in Figure 7 below. Most of the erosion occurred near the dam, which is expected due to the localized increase in the energy grade line as a result of the dam failure. Near the dam, nearly 3 meters of degradation can be seen. Towards the upstream end of the study reach, the degradation attenuates, indicating that the headcut has not progressed entirely through the study reach.

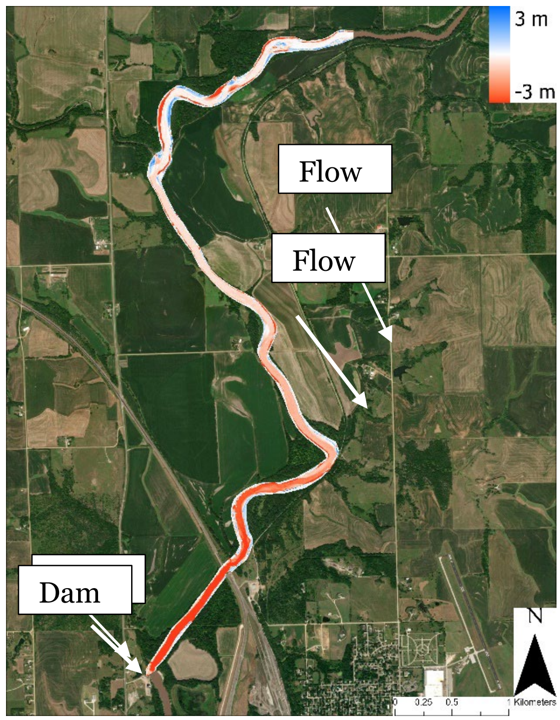


Figure 7: Elevation differences between 2018 and 2022 LiDAR (red indicates erosion, blue indicates deposition). Erosion was most severe near the dam and attenuates upstream.

The net sediment eroded from the study reach can be seen in Table 1. Approximately 860,000 m³ eroded from the site between 2018-2022, equating to approximately 696 acre-ft. Over a span of four years, this equates to an annual load of 174 acre-ft per year. Because the Tuttle Creek Lake flood-control pool extends to Marysville, all of this sediment reached Tuttle, although approximately 40% is expected to have deposited within the flood-control pool and the remaining 60% would have deposited in the multi-purpose pool.

Table 1 Net Sediment Eroded between 2018-2022. Approximately 174 acre-ft eroded from the study reach each year between 2018-2022.

Volume Eroded	Volume Deposited	Net Volume change	Net Volume change per km
m³	m³	m³	m³/km

961,037	102,331	858,706	100,434

3.2. Future Erosion

The estimates for the future sediment loads can be seen in Table 2 below. These values were obtained by averaging the area change between the two bounding cross-sections in a reach and then multiplying by a length to obtain a volume.

Table 2:Projected Future Sediment Loads. Column 1 and 2 represent the bounding cross-sections within a reach. d_1 and d_2 represent the pre-failure elevation and the post-failure elevation of the invert of the upstream cross-section, respectively. The Average Area Difference was obtained by first calculating the are difference at each cross-section for pre-failure and post-failure cross-sections and then averaging the differences between the two bounding cross-sections, similar to the Average-End Area method.

Downstream Cross-Section	Upstream Cross-Section	d1	d2	Length	Average Area Difference	Volume	Volume
		m	m	m	m^2	m^3	acre-ft
2	3	344.4	344	578	10.7	6,171	5.0
3	4	345.0	344.4	604	27.2	16,445	13.3
4	5	345.8	344.5	295	55.5	16,402	13.3
5	6	345.8	345.2	1,005	58.4	58,695	47.6
6	7	346.5	345.4	380	38.9	14,775	12.0
7	8	346.5	345.6	424	51.6	21,890	17.7
8	9	346.5	346	714	58.3	41,617	33.7
9	Hard Point	346.4	346.4	596	26.1	15,556	12.6
					Total	191,549	155.3

4. Discussion

4.1. Contributions to Tuttle Creek Lake

The net volume eroded from the study reach was approximately 696 acre-ft. Averaged over four years, the average net erosion rate was approximately 174 acre-ft per year. Not all of this sediment will end up in the Tuttle Creek Lake Multi-Purpose Pool. According to USACE (2022a), the average deposition rate within the multi-purpose pool is approximately 60%, with the other 40% depositing in the flood-control pool. Therefore, it is estimated that approximately 104 acre-ft/yr deposited in the multi-purpose pool as a result of this headcut. If the average annual deposition rate within the multi-purpose pool is 3,560 acre-ft per year (USACE, 2022b), then this headcut accounted for roughly 2.9% of the total multipurpose pool deposition over the past four years.

Using the same floodpool/multipurpose pool percentages for the future sediment loads, the amount of sediment that could be saved from entering the multi-purpose pool is roughly 0.7% each year over the next four years. It appears that the headcut is halfway to a hard-point, at which point further degradation is not expected. Therefore, the headcut should stabilize by about 2026.

The estimates presented in this analysis are expected to be lower than the actual volumes of sediment eroded. Field investigations confirmed that there is substantial headcutting occurring on several tributaries. As the Big Blue degrades, tributaries that enter the Big Blue will have steeper slopes, resulting in tributary headcuts. The difference between the 2022 LiDAR and the 2018 LiDAR can be seen in Figure 8 below. On the right descending bank, there is a tributary between stations 50 and 100. This tributary had evidence of both degradation and bank erosion. The field photograph in Figure 9 shows steep banks and abrupt changes in bed elevation, which are both indicative of headcutting. Furthermore, the future sediment load estimates rely on the headcut stabilizing at station 6,000 (see Figure 3). If this hard point does not stop the headcut, degradation will move upstream until erosion-resistant material is found.

Figure 8: Difference between LiDAR sets from 2022 and 2018.Flow is moving from the top of the picture to the bottom. Red indicates erosion, blue indicates deposition. The first tributary upstream of the dam on the right descending bank indicates substantial degradation and bank erosion.

Figure 9: Field Photo of first tributary upstream of dam

4.2. Cost Comparison

A cost comparison was performed to determine if it would be more cost-effective to dredge the sediment once it deposited in Tuttle or if it would be more cost-effective to stabilize the headcut and prevent the sediment from entering the lake. The cost comparison can be seen in Table 3 and Table 4. For dredging costs, the cost per cubic yard was estimated based on the dredging that occurred at John Redmond Reservoir (Kansas) in 2015, which costed approximately $6.70 per cubic yard (KWO, 2014). The cost for grade control was estimated by assuming three grade control structures would be needed, one just downstream of the headcut, one just upstream of the headcut, and one farther upstream from the headcut. The design for the grade control structures was assumed to be that of an engineered rock riffle. The estimated tonnage per engineered rock riffle was 2,054 tons. With three riffles and $60 per cubic yard of riprap, the total estimated cost for grade controls is $370,000.

Table 3 shows the cost comparison between dredging and grade control if the headcut were to be stabilized now, while Table 4 shows the cost comparison between dredging and grade control if the headcut had been stabilized prior to the dam failure. The amount of sediment that could have been saved by the grade control structure was much greater prior to failure, which means that the same grade control structures would have been much more effective in 2018 than they would have been in 2022.

Table 3: Cost comparison for stabilizing the headcut as it was in 2022.

	Grade Control now	**Dredging**

Sedimentation Prevented/Removed (CY)	150,291	
Total Cost	$370,000	$1 Million
Cost per cubic yard	$2.46/CY	$6.70*

*Cost per cubic yard set equal to 2016 dredging costs at John Redmond, without escalation.

Table 4: Cost Comparison for stabilizing the headcut prior to dam failure

	Grade Control before dam failure	**Dredging before dam failure**
Sedimentation Prevented/Removed (CY)	823,888	
Total Cost	$370,000	$5.5 Million
Cost per cubic yard	$0.45/CY	$6.70*

*Cost per cubic yard set equal to 2016 dredging costs at John Redmond, without escalation.

5. Conclusions

This analysis indicates that the failure of the low-head dam at Marysville resulted in a large amount of sediment depositing within Tuttle Creek Lake. Approximately 2.9% of the multi-purpose pool deposition from 2018 to 2022 can be attributed to the dam failure. Future sediment projections indicate that the headcut will cause at least an additional 0.7% of the deposition within the multi-purpose pool over the next four years. This volume includes only sediment eroded from the mainstem Big Blue River; headcutting up tributaries will contribute additional sediment.

6. References

KWO (Kansas Water Office). 2014. Final programmatic environmental impact statement—Removal and disposal of sediment and restoration of water storage at John Redmond Reservoir, Kansas. https://www.swt.usace.army.mil/Portals/41/docs/library/john_redmond/fpeis_remove_dispose_sediment_jrr_ks.pdf

USACE. 2022a. Kansas River Reservoirs Flood and Sediment Study. Appendix D1.3: Tuttle Creek Lake Existing Conditions Sediment (DRAFT)

USACE. 2022b. Kansas River Reservoirs Flood and Sediment Study. Appendix D2.3: Tuttle Creek Lake Future without Project Sedimentation (DRAFT)

US Army Corps of Engineers ®
Kansas City District

Kansas River Reservoirs Flood and Sediment Study

Appendix D5.3: Hydrosuction

June 2023

Contents

1.0 INTRODUCTION

This appendix documents Future With Project (FWP) projections for sedimentation conditions at USACE lakes in the Kansas River Basin in 25, 50, and 100 years using hydrosuction as a means to remove sediment. The Kansas River Basin along with the major lakes are shown in Figure 1. USACE lakes ordered from east to west are Clinton, Perry, Tuttle Creek, Milford, Kanopolis, Wilson, and Harlan County.

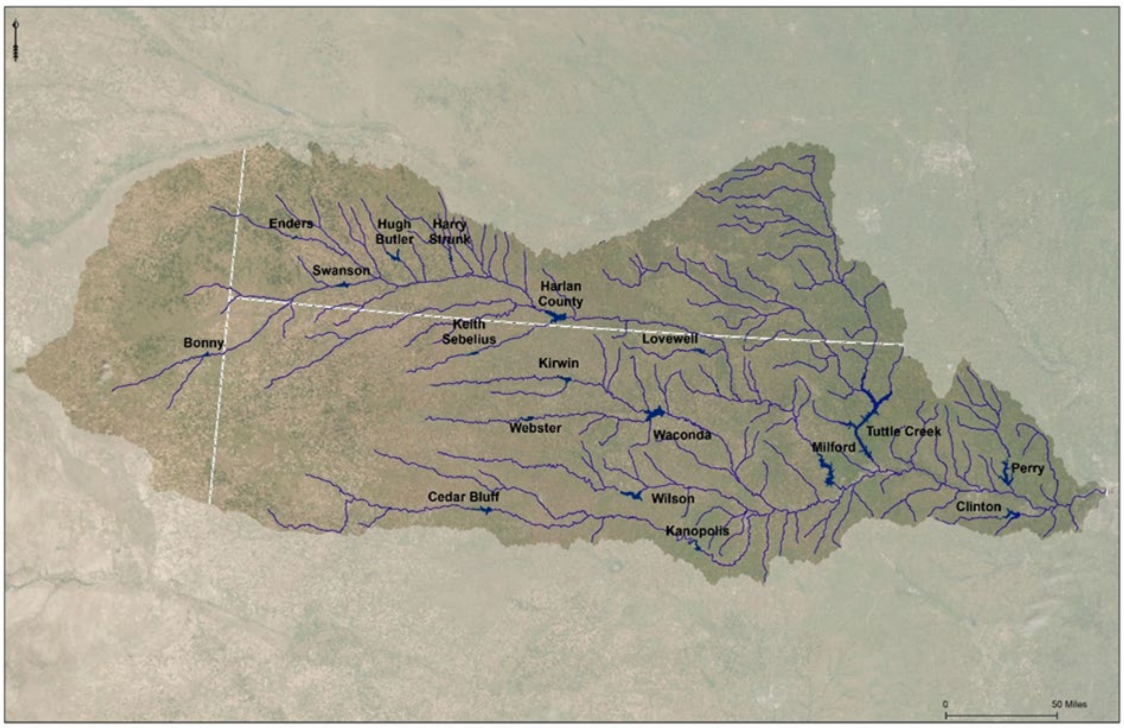


Figure 1: Kansas River Basin

Given the large volume of accumulating sediment and the high cost for traditional dredging with land disposal, the Kansas Water Office and the U.S. Army Corps of Engineers have been looking for less expensive, more sustainable ways to manage sediment. At a workshop held August 2013 (Shelley 2015), state and federal participants identified hydrosuction as a promising option for passing sediment. Hydrosuction utilizes the head of water in the reservoir to power the removal, transport, and discharge of sediment to the downstream channel (Hotchkiss and Huang 1995). Hydrosuction is significantly less expensive than traditional dredging with land disposal because it eliminates the costs for external power, which typically comprise 30% of the cost of dredging operation, as well as the costs for land disposal of the sediments, which can comprise over 50% of the total cost of a dredging operation (McFall and Welp 2015). Hydrosuction can be used with success to take sediment up and over small dams. However, for tall dams like the USACE dams in the Kansas River Basin, the water would cavitate in the pipe before reaching the top of the dam. To prevent cavitation, the pipe must travel through the dam or abutment rather than over it.

2.0 FLUID AND PIPE PROPERTIES

McFall and Welp (2015) assumed 6% solids in the pipe when analyzing hydrosuction for Tuttle Creek Lake. A hydrosuction project in El Canada reservoir in Guatemala reported values that varied from 8% to 12% solids in the pipe (Jimenez, Figueroa, and Jacobsen 2015) using a proprietary water injection system to agitate sediments and increase the sediment removal.

It was assumed for this study that the sediment concentration in the pipes would be 6% by weight. The concentration of solids by weight can be converted to the concentration by volume using Equation 1. The density of the slurry is computed with Equation 2, and the dynamic viscosity of the slurry is computed from Equation 3. Table 1 presents fluid properties assuming 6% solids. The water temperature is assumed to be 59°F.

$$C_v = \frac{C_w \rho_m}{\rho_s} = \frac{100C_w/\rho_s}{\frac{C_w}{\rho_s} + (100 - C_w)/\rho_L} \qquad \text{(Equation 1)}$$

where C_v = concentration of sediment by volume
C_w = concentration of sediment by weight
ρ_m = density of the slurry
ρ_s = density of solids in the slurry
ρ_L = density of the liquid in the slurry

$$\rho_m = [(1 - \varphi) + 2.64\varphi](62.4)(0.031081) \qquad \text{(Equation 2)}$$

$$\mu = \mu_w(1 + 2.5\varphi + 14.1\varphi^2) \qquad \text{(Equation 3)}$$

where φ = the solids fraction by volume
μ_w = the dynamic viscosity of the water
μ = the dynamic viscosity of the slurry in lb-s/ft^2

Table 1: Fluid Properties of the Discharge Slurry at 6%

Solids Fraction by Weight	0.06	
Density of Slurry	2.02	slugs/ft^3
Dynamic Viscosity of Water (μ_w)	2.38E-05	lb s/ ft^2
Median Grain Size	0.002	mm
Dynamic viscosity of slurry (μ)	2.86E-05	lb s/ ft^2

Pipe materials and roughness are presented in Table 2. The length of the pipes varies by lake. Components contributing to minor losses are included in Table 3 and are assumed equivalent across lakes.

Table 2: Pipe Properties

	Type	Diameter (ft)	Pipe Relative Roughness
Suction Pipe	Rubber Tubing	2	0.00007
Discharge Pipe	Ductile Iron	2	0.000075

Table 3: Minor Losses

Component	Loss Coefficient (K)
Entrance	1.5
90° elbow	0.3
45° elbow	0.2
Valve	2
Exit	1
Total	5.5

The Bernoulli equation with the Swamee-Jain approximation for friction, using the fluid and pipe properties given in Tables 1, 2, and 3 were used to calculate slurry discharge for a range of pool elevation and a constant tailwater elevation. The Bernoulli equation is a commonly used equation for determining flow rates through a system based on the principle of the conservation of energy, whereas the Swamee-Jain friction equation is used to estimate the headloss through pipes by approximating the Moody friction factor for different material types.

3.0 EFFECTIVENESS GIVEN POOL FLUCTUATIONS AND DOWNSTREAM TAILWATER

The efficiency of any hydrosuction alternative will depend on the pool level, which fluctuates during the year in response to flood inflows, seasonal operating pool levels, and downstream flow targets and constraints. The daily pool elevation was available from records for each of the lakes.

Tailwater downstream of the dam fluctuates with the discharge from the dam and can be determined using tailwater rating curves. Pipe invert elevations at the downstream end were selected so that they exceeded the tailwater elevation for the majority of the time.

The tailwater rating curve for Clinton Lake is shown in Figure 2 (note, at Clinton Lake, add 0.08 feet to convert from NGVD29 to NAVD88). The maximum discharge from Clinton Dam from 1980 to 2019 was 4,000 cfs. A tailwater elevation of 832 feet NGVD29 (832.08 NAVD88) was selected for the invert elevation of the hydrosuction pipes.

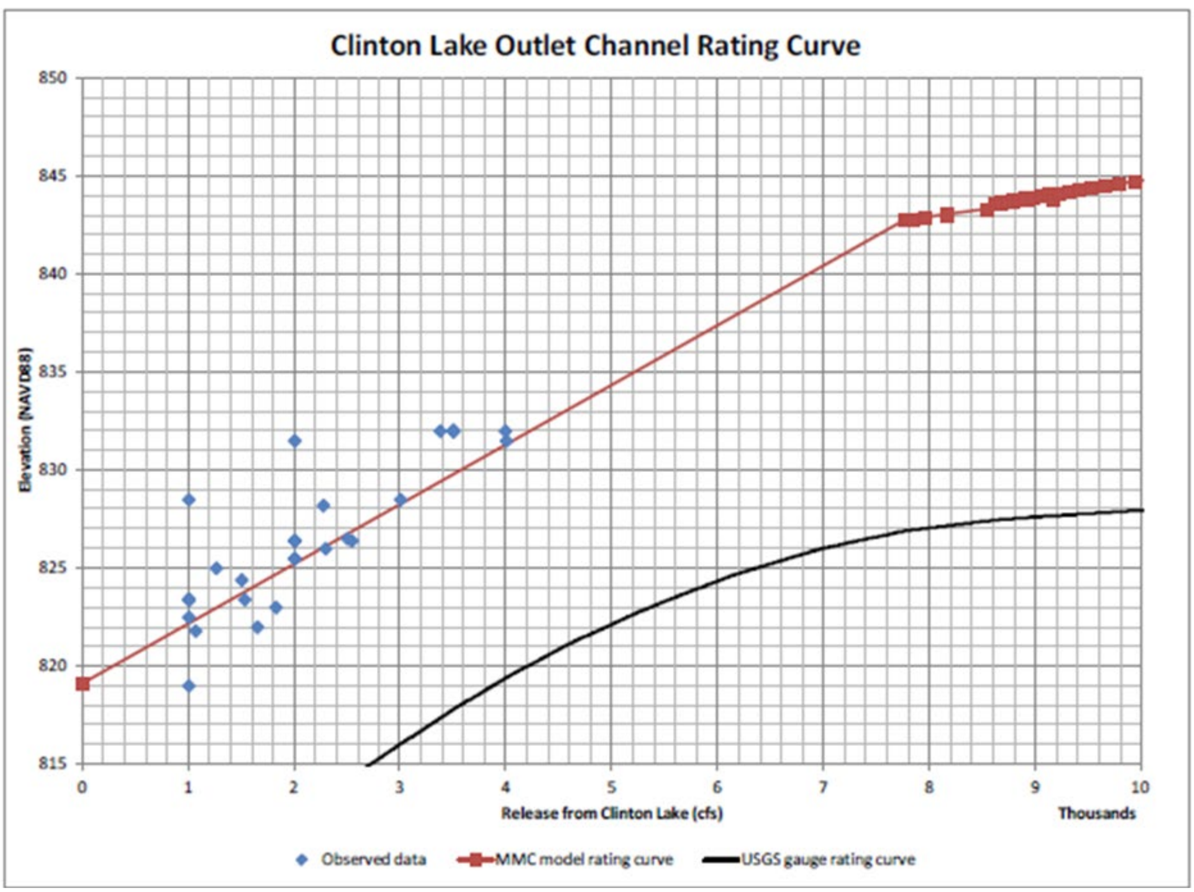


Figure 2: Tailwater Rating Curve Below Clinton Dam taken from the 2013 Periodic Assessment

The tailwater rating curve for Perry Lake is shown in Figure 3 and gives the tailwater for four discharge scenarios on the Kansas River, since tailwater below Perry can be significantly effected by backwater from the Kansas River. The maximum discharge from Perry Dam from 1970 to 2019 was 13,479 cfs. A tailwater elevation of 853 feet NGVD29 was selected for the invert elevation of the hydrosuction pipes, which corresponds to an elevation of 853.3 NAVD88 for reference in Figure 3.

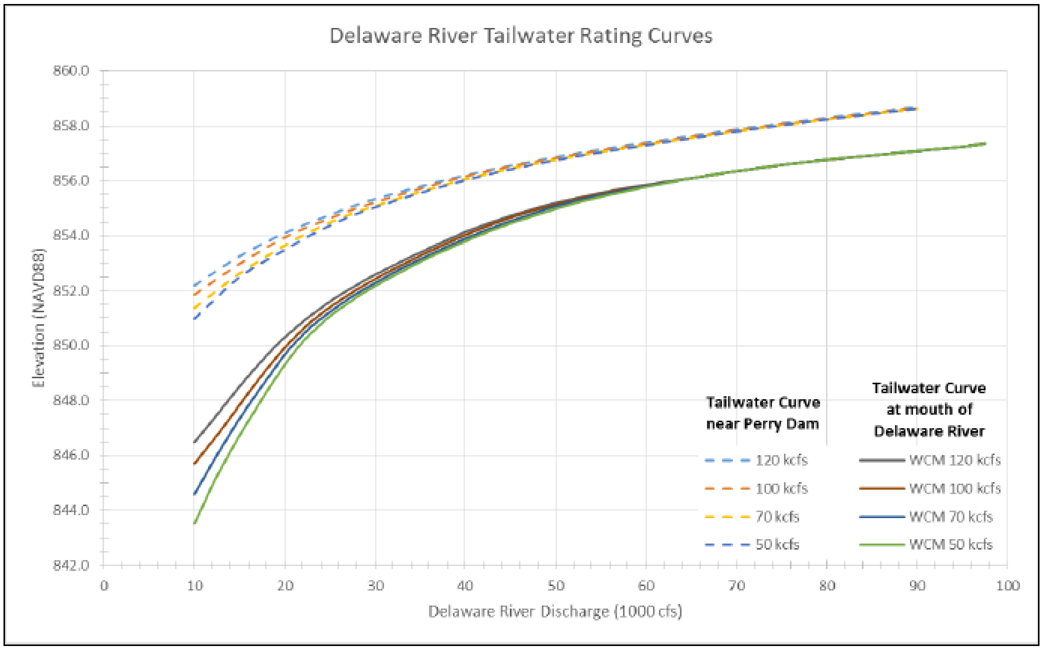


Figure 3: Tailwater Rating Curve below Perry Dam taken from the 2018 Periodic Inspection

The tailwater rating curve for Tuttle Creek Lake is shown in Figure 4. The maximum discharge from Tuttle Creek Dam from 1970 to 2019 was 60,000 cfs. A tailwater elevation of 1028.13 feet NGVD29 was selected for the invert elevation of the hydrosuction pipes.

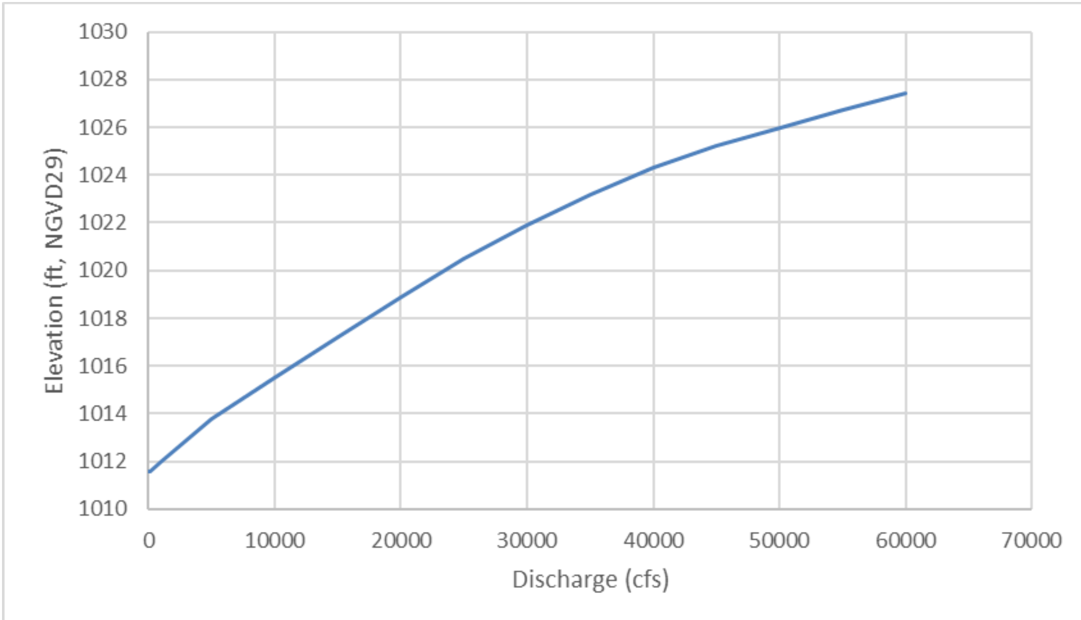


Figure 4: Tailwater Rating Curve below Tuttle Creek Dam taken from the 1973 Water Control Manual

The tailwater rating curve for Milford Lake is shown in Figure 5. The maximum discharge from Milford Dam from 1970 to 2019 was 34,000 cfs. A tailwater elevation of 1075 feet NGVD29 was selected for the invert elevation of the hydrosuction pipes.

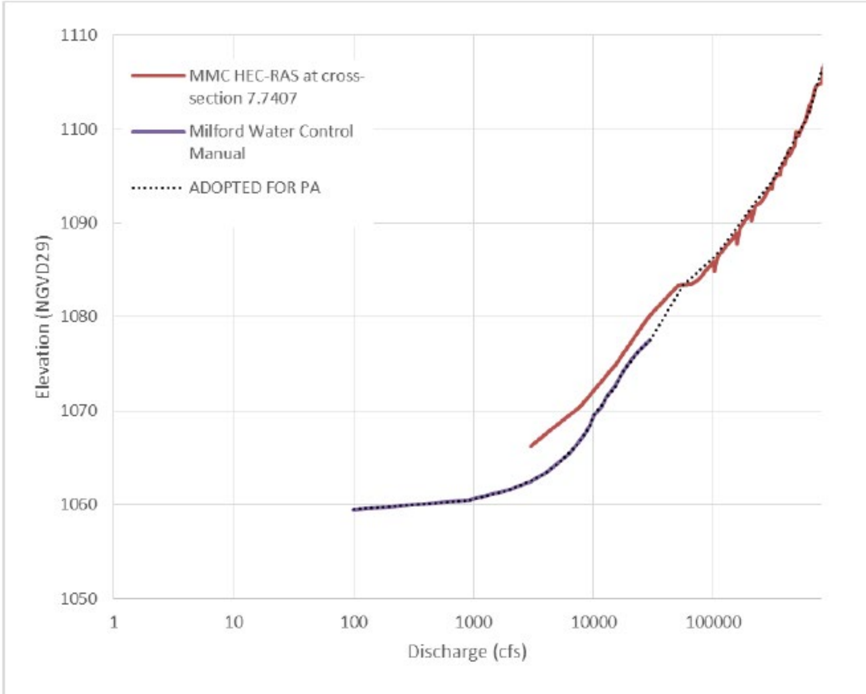


Figure 5: Tailwater Rating Curve below Milford Dam taken from the 2016 Periodic Assessment

The tailwater rating curve for Kanopolis Lake is shown in Figure 6. The maximum discharge from Kanopolis Dam from 1970 to 2019 was 6,500 cfs. A tailwater elevation of 1416 feet NGVD29 was selected for the invert elevation of the hydrosuction pipes.

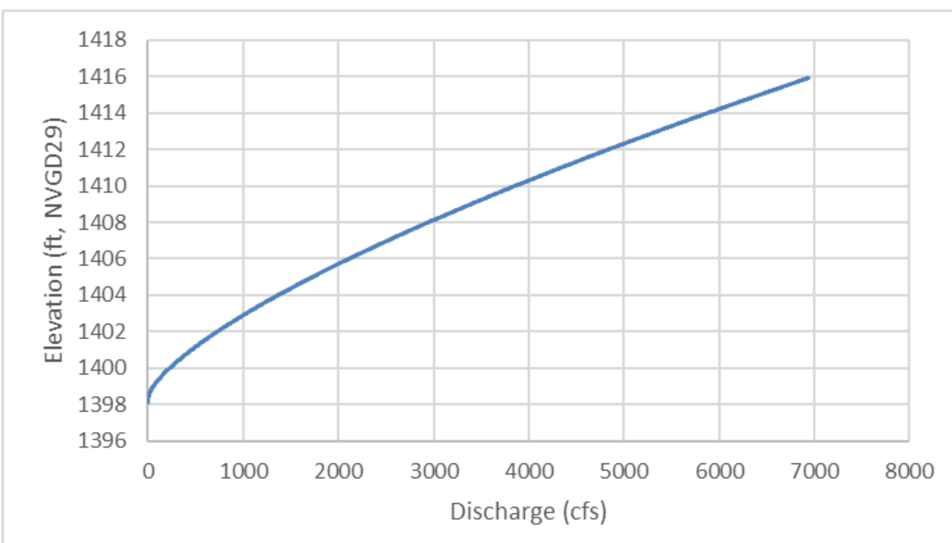


Figure 6: Tailwater Rating Curve below Kanopolis Dam Taken from the USGS gage 06865500

The tailwater rating curve for Wilson Lake is shown in Figure 7. The maximum discharge from Wilson Dam from 1970 to 2019 was 2,775 cfs. A tailwater elevation of 1447 feet NGVD29 was selected for the invert elevation of the hydrosuction pipes.

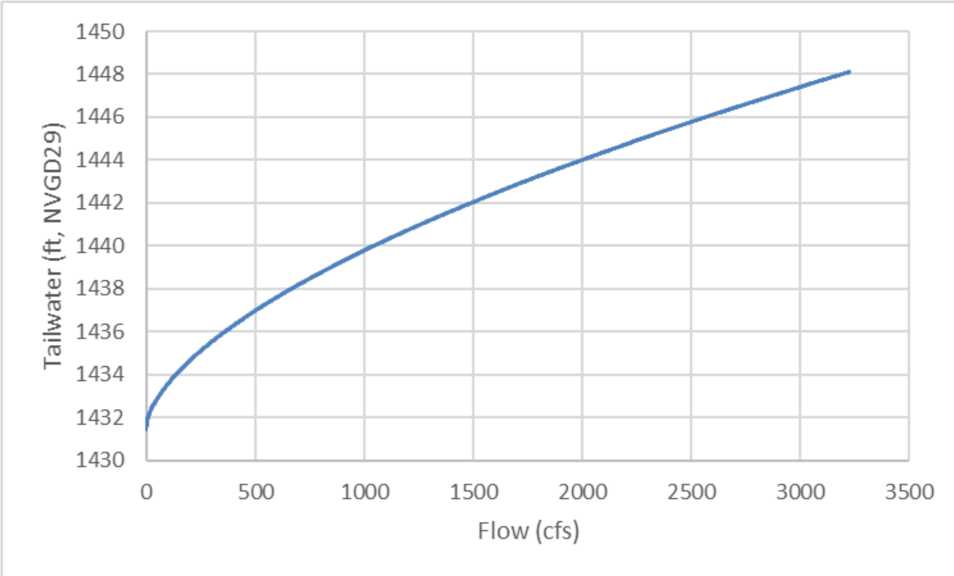


Figure 7: Tailwater Rating Curve Below Wilson Dam Taken from the USGS gage 06868200

The tailwater rating curve for Harlan Lake is shown in Figure 8. The maximum discharge from Harlan County Dam from 1970 to 2019 was 2,000 cfs. A tailwater elevation of 1879 feet NGVD29 was selected for the invert elevation of the hydrosuction pipes.

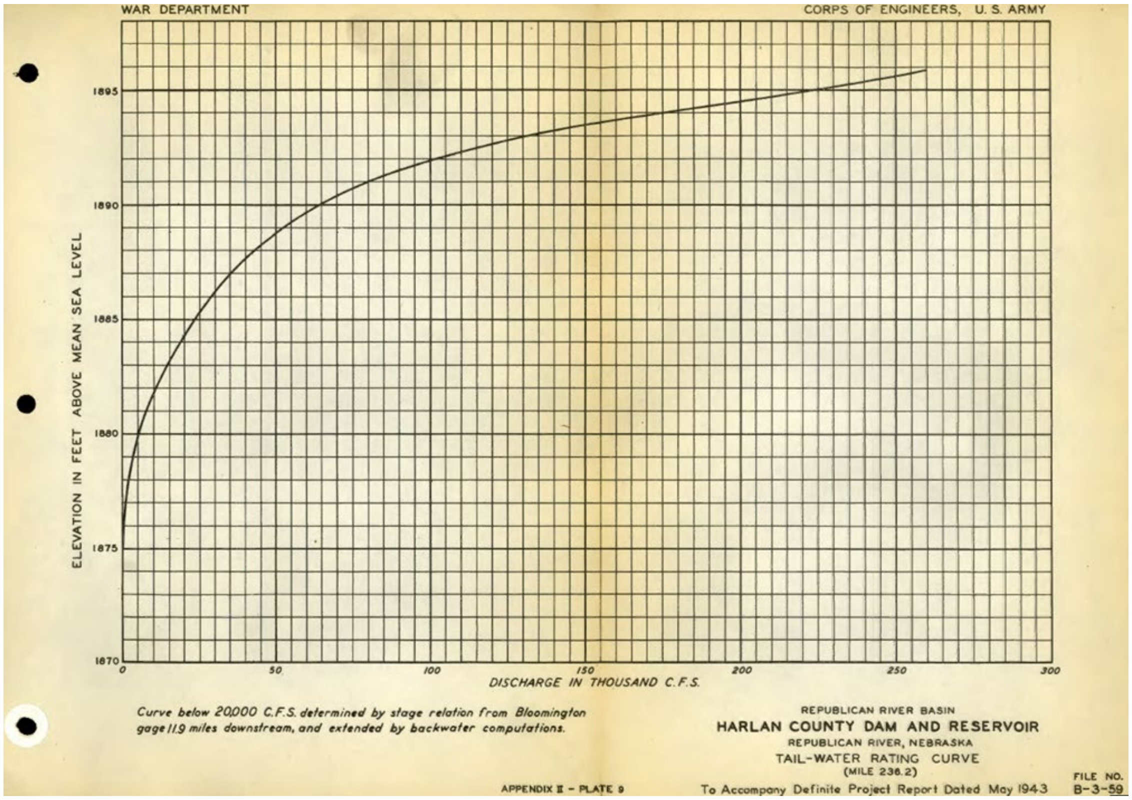


Figure 8: Tailwater Rating Curve Below Harlan County Dam Taken from Definite Project Report

A second order polynomial equation was fitted to the computed slurry discharge versus pool elevation at each of the lakes. This is illustrated in Figure 9, which is for Kanopolis Lake. The amount of sediment removed from the lakes through hydrosuction was then determined using the following steps:

(1) Compute the daily slurry discharge from the fitted equation and the recorded daily pool elevation.
(2) Calculate the solids discharge as the 6% solids by weight times the slurry discharge.
(3) Translate solids discharge to the volume the sediment would occupy in the lake using bulk density values reported in Appendix A.

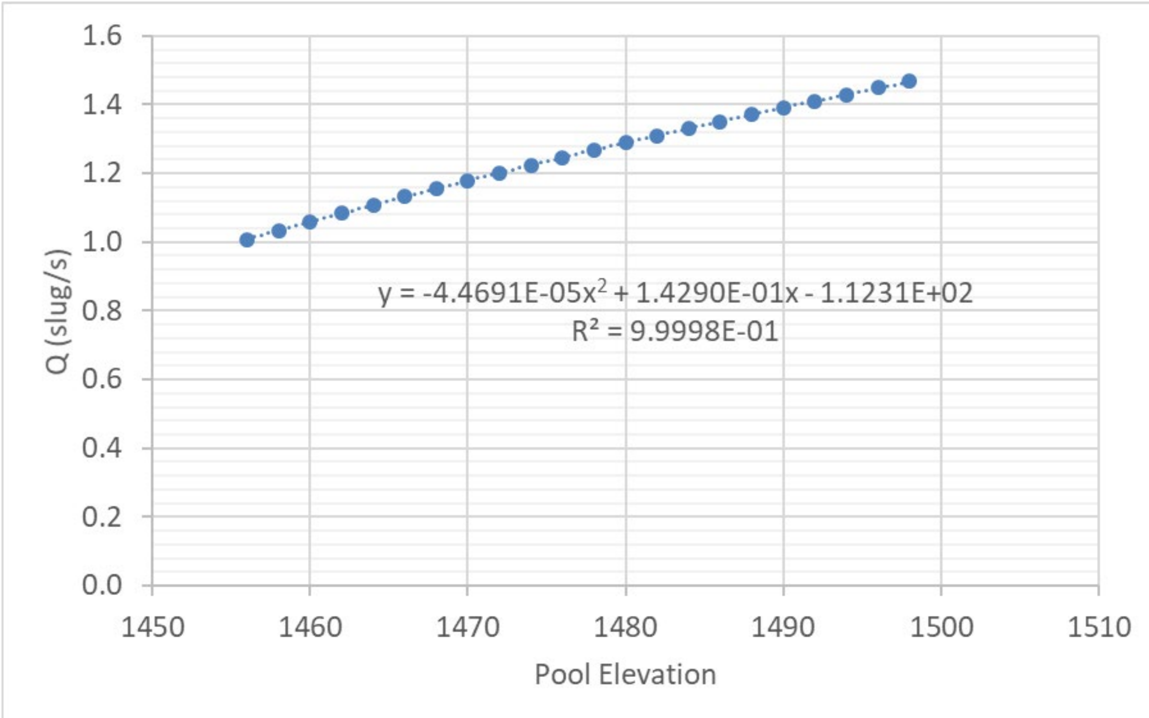


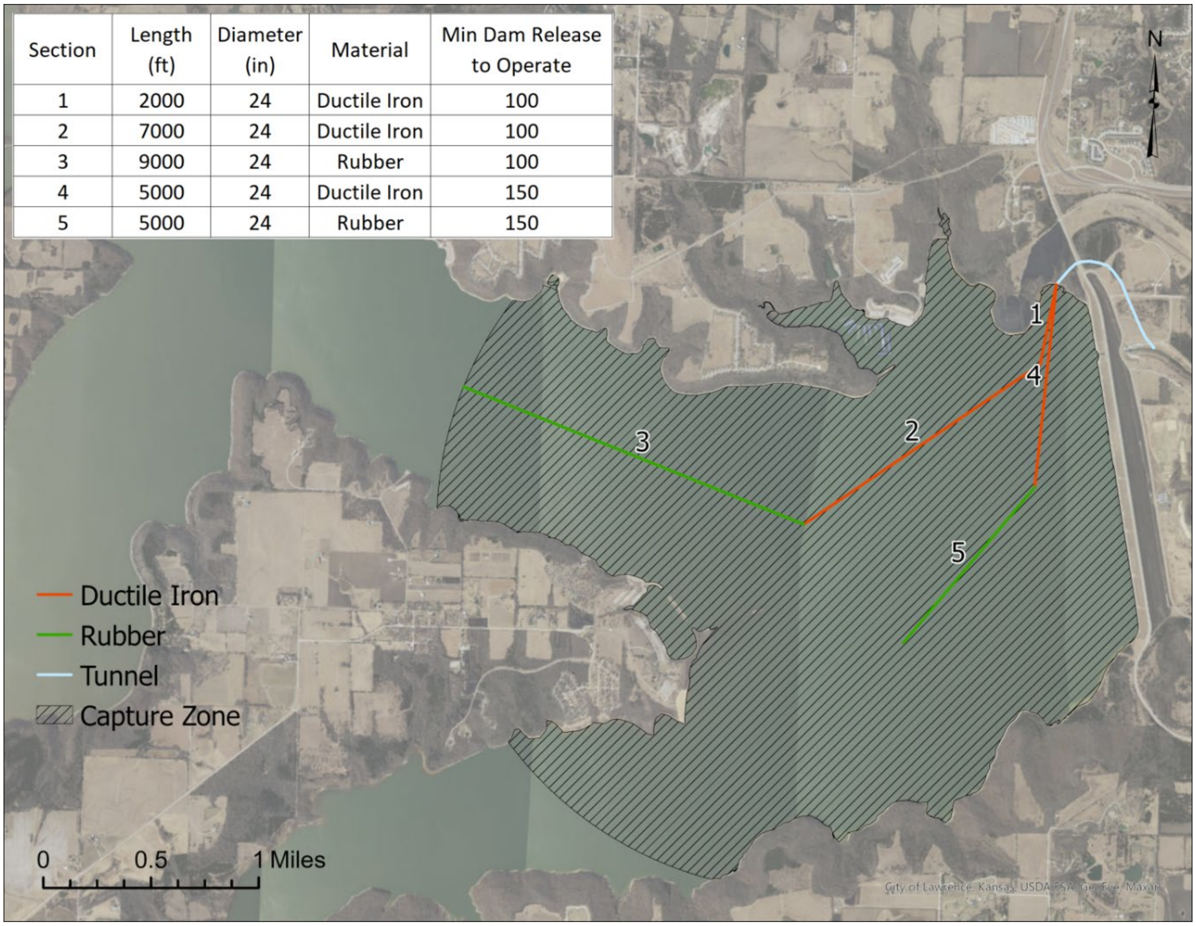
Figure 9: Slurry Discharge versus Pool Elevation at Kanopolis Lake

4.0 PHYSICAL LAYOUT AND OPERATING THRESHOLDS

Multiple options for suction pipe configurations and types exist that could influence effectiveness and cost. The configurations used to determine the effectiveness of hydrosuction at each of the lakes are given in the following sections. For purposes of computing effectiveness, two to three 2-ft diameter pipes were assumed. The first sections of each pipe were assumed to be ductile iron with a fixed location, while the final section of each pipe is assumed to be rubber hose, which could be moved to different areas of the lake. A new conduit would be drilled through the abutment at each of the lakes, through which the two pipes will run. The pipes would be operated according to lake releases so that the downstream sediment concentration would not exceed natural levels. Threshold discharge levels were chosen so that the resulting sediment concentrations would not exceed the upper 80% confidence interval of the natural concentration.

4.1. Clinton Lake

The configuration used to determine the effectiveness of hydrosuction at Clinton Lake is given in Figure 10. An area of 3,696 acres within the lake can be reached using this configuration. The thresholds at which each pipe begins operating are also given in Figure 10.



Section	Length (ft)	Diameter (in)	Material	Min Dam Release to Operate
1	2000	24	Ductile Iron	100
2	7000	24	Ductile Iron	100
3	9000	24	Rubber	100
4	5000	24	Ductile Iron	150
5	5000	24	Rubber	150

Figure 10: Hydrosuction Configuration for Effectiveness Calculations, Clinton Lake

Figure 11 plots the upstream concentration measurements and 80% confidence intervals which were presented in Appendix D1.1, along with the resulting sediment concentrations when the hydrosuction pipes are operating below the discharge thresholds.

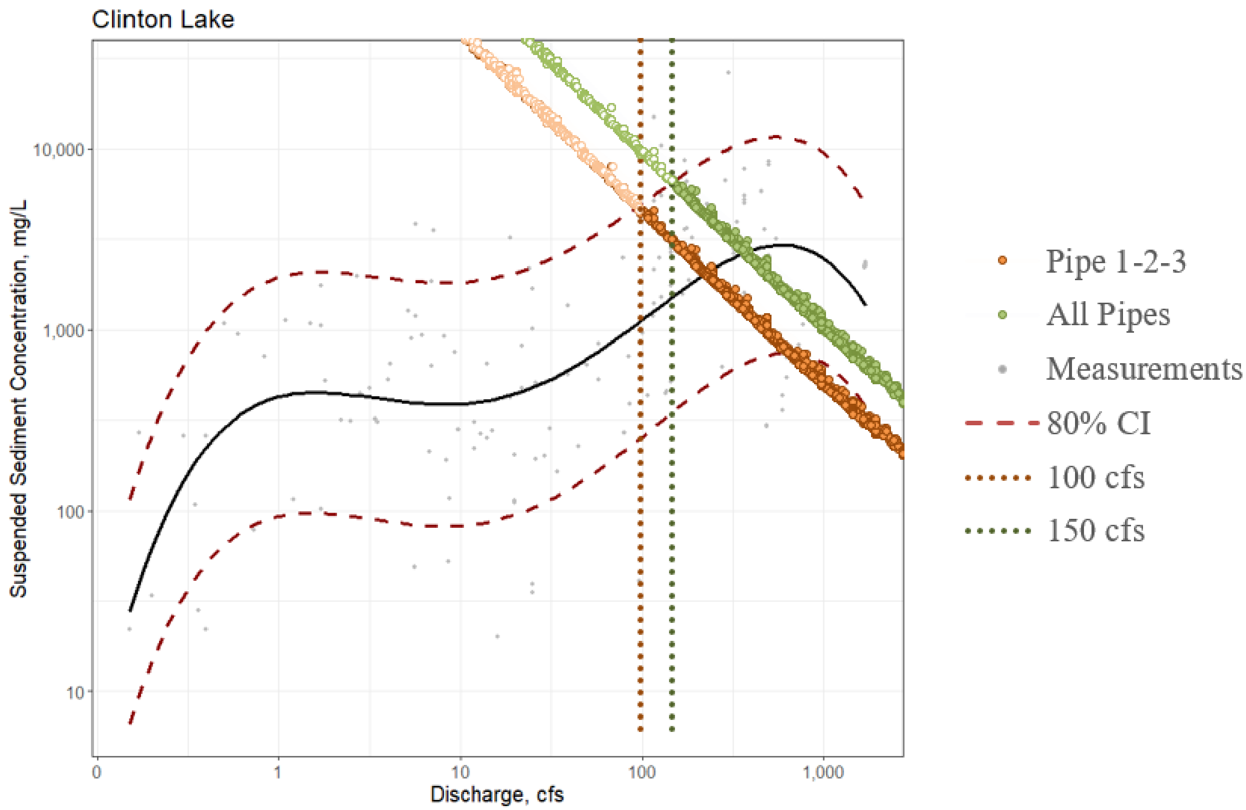
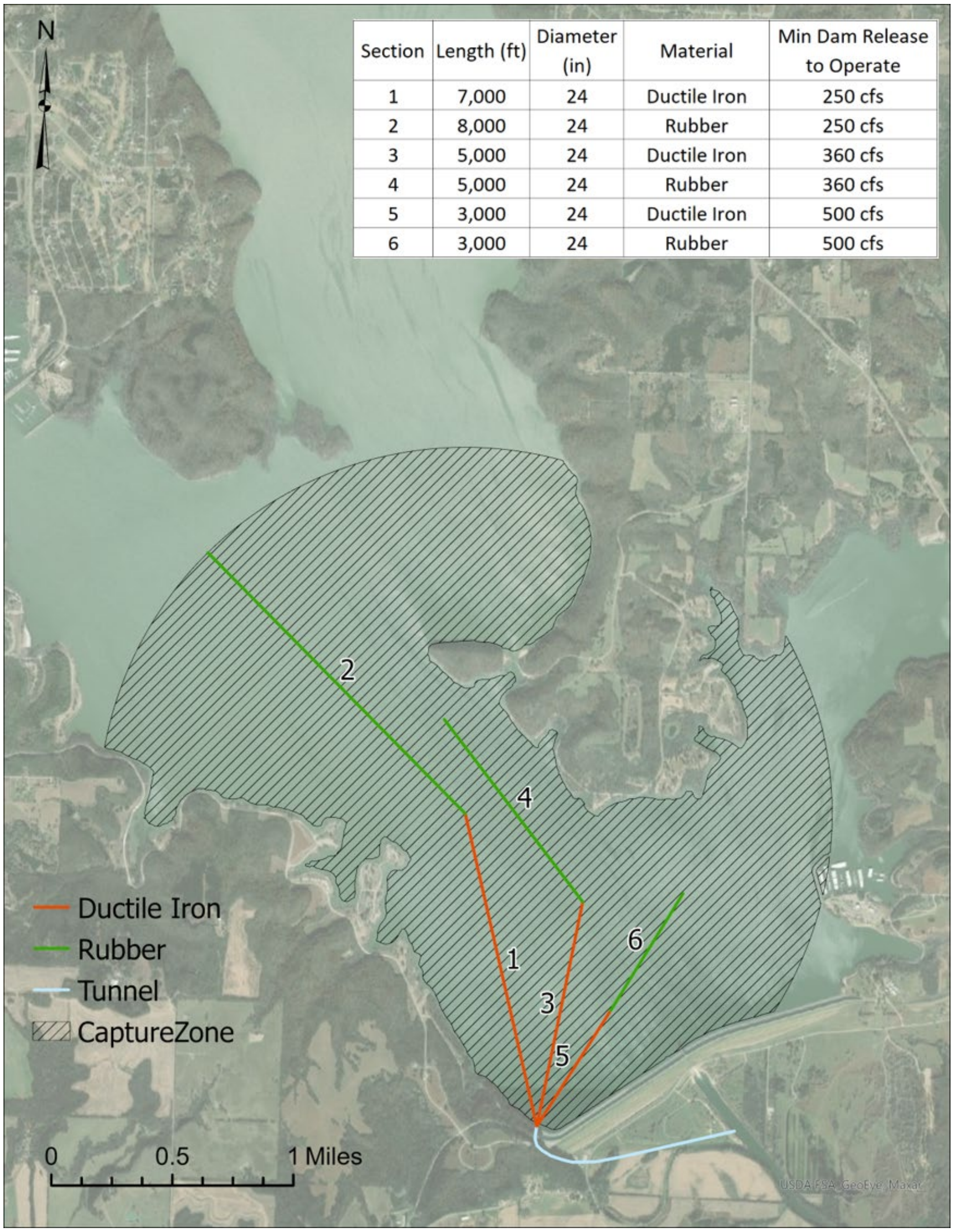


Figure 11: Sediment Concentration Measurements Upstream of Clinton Lake, 80% Confidence Intervals, Sediment Concentration Resulting from Hydrosuction Pipes, and the Flow Thresholds when the Pipes begin Operating

4.2. Perry Lake

The configuration used to determine the effectiveness of hydrosuction at Perry Lake is given in Figure 12. An area of 2,770 acres within the lake can be reached using this configuration.



Section	Length (ft)	Diameter (in)	Material	Min Dam Release to Operate
1	7,000	24	Ductile Iron	250 cfs
2	8,000	24	Rubber	250 cfs
3	5,000	24	Ductile Iron	360 cfs
4	5,000	24	Rubber	360 cfs
5	3,000	24	Ductile Iron	500 cfs
6	3,000	24	Rubber	500 cfs

Figure 12: Hydrosuction Configuration for Effectiveness Calculations, Perry Lake

Figure 13 plots the upstream concentration measurements and 80% confidence intervals which were presented in Appendix D1.2, along with the resulting sediment concentrations when the hydrosuction pipes are operating below the discharge thresholds.

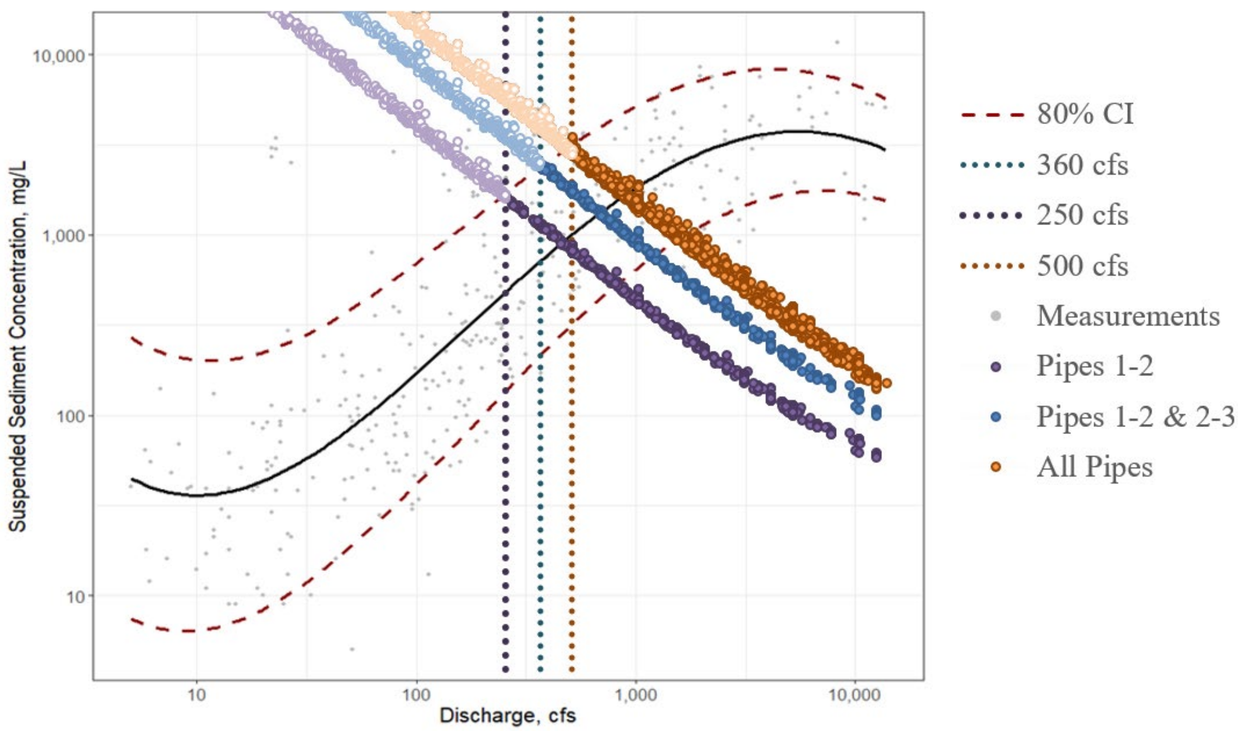
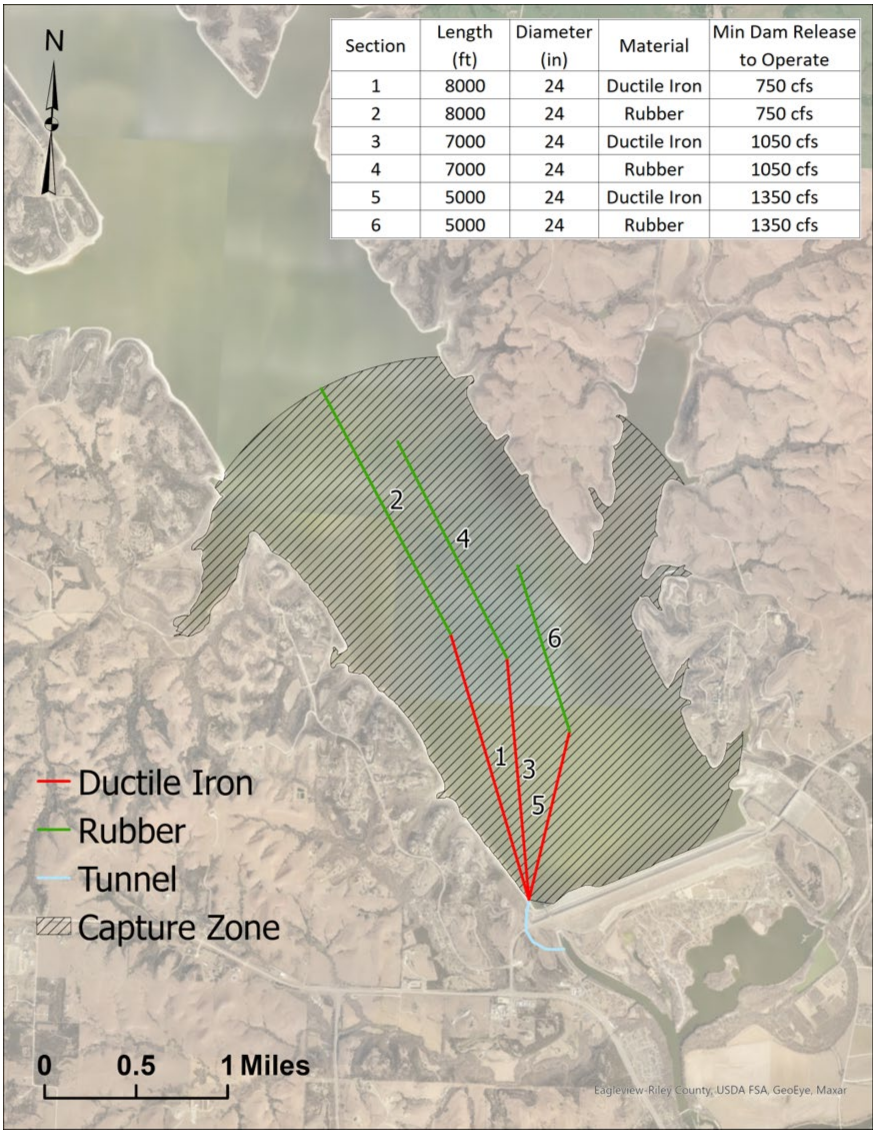


Figure 13: Sediment Concentration Measurements Upstream of Perry Lake, 80% Confidence Intervals, Sediment Concentration Resulting from Hydrosuction Pipes, and the Flow Thresholds when the Pipes begin Operating

4.3. Tuttle Creek Lake

The configuration used to determine the effectiveness of hydrosuction at Tuttle Creek Lake is given in Figure 14. An area of 2,877 acres within the lake can be reached using this configuration.



Section	Length (ft)	Diameter (in)	Material	Min Dam Release to Operate
1	8000	24	Ductile Iron	750 cfs
2	8000	24	Rubber	750 cfs
3	7000	24	Ductile Iron	1050 cfs
4	7000	24	Rubber	1050 cfs
5	5000	24	Ductile Iron	1350 cfs
6	5000	24	Rubber	1350 cfs

Figure 14: Hydrosuction Configuration for Effectiveness Calculations, Tuttle Creek Lake

Figure 15 plots the upstream concentration measurements and 80% confidence intervals which were presented in Appendix D1.3, along with the resulting sediment concentrations when the hydrosuction pipes are operating below the discharge thresholds.

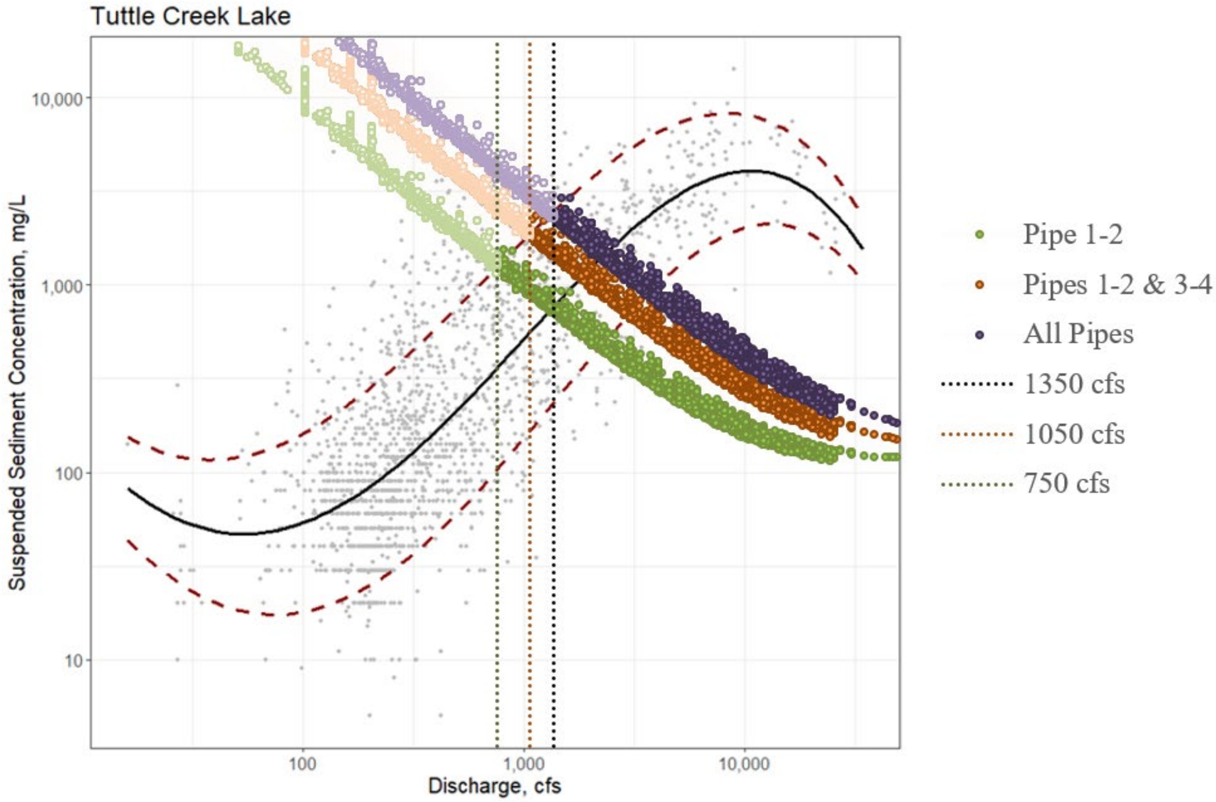
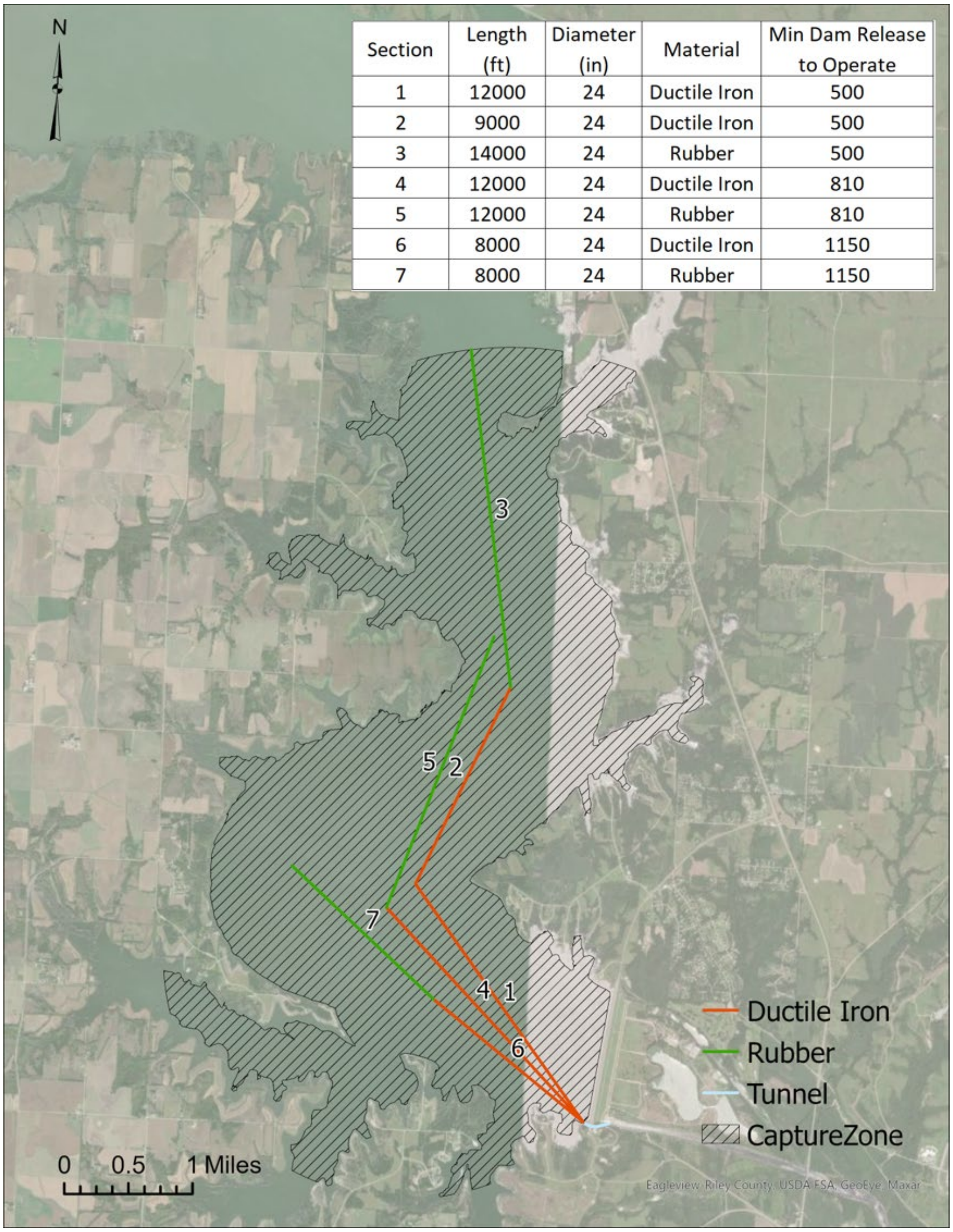


Figure 15: Sediment Concentration Measurements Upstream of Tuttle Creek Lake, 80% Confidence Intervals, Sediment Concentration Resulting from Hydrosuction Pipes, and the Flow Thresholds when the Pipes begin Operating

4.4. Milford Lake

The configuration used to determine the effectiveness of hydrosuction at Milford Lake is given in Figure 16. An area of 7,827 acres within the lake can be reached using this configuration.



Section	Length (ft)	Diameter (in)	Material	Min Dam Release to Operate
1	12000	24	Ductile Iron	500
2	9000	24	Ductile Iron	500
3	14000	24	Rubber	500
4	12000	24	Ductile Iron	810
5	12000	24	Rubber	810
6	8000	24	Ductile Iron	1150
7	8000	24	Rubber	1150

Figure 16: Hydrosuction Configuration for Effectiveness Calculations, Milford Lake

Figure 17 plots the upstream concentration measurements and 80% confidence intervals which were presented in Appendix D1.4, along with the resulting sediment concentrations when the hydrosuction pipes are operating below the discharge thresholds.

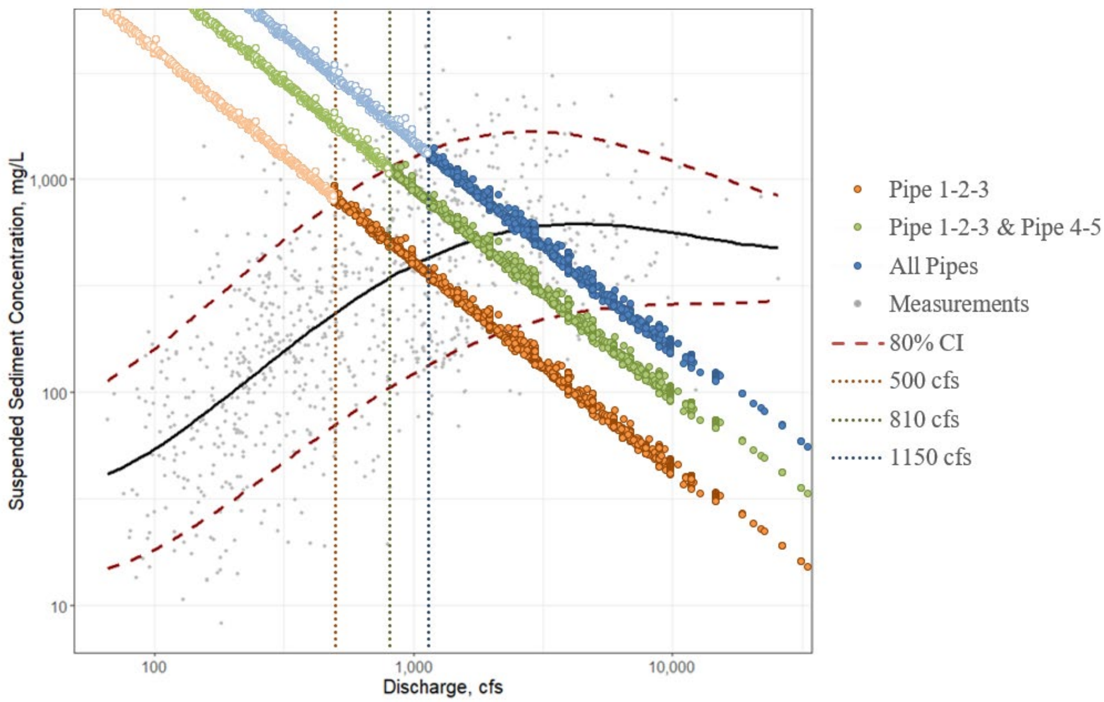
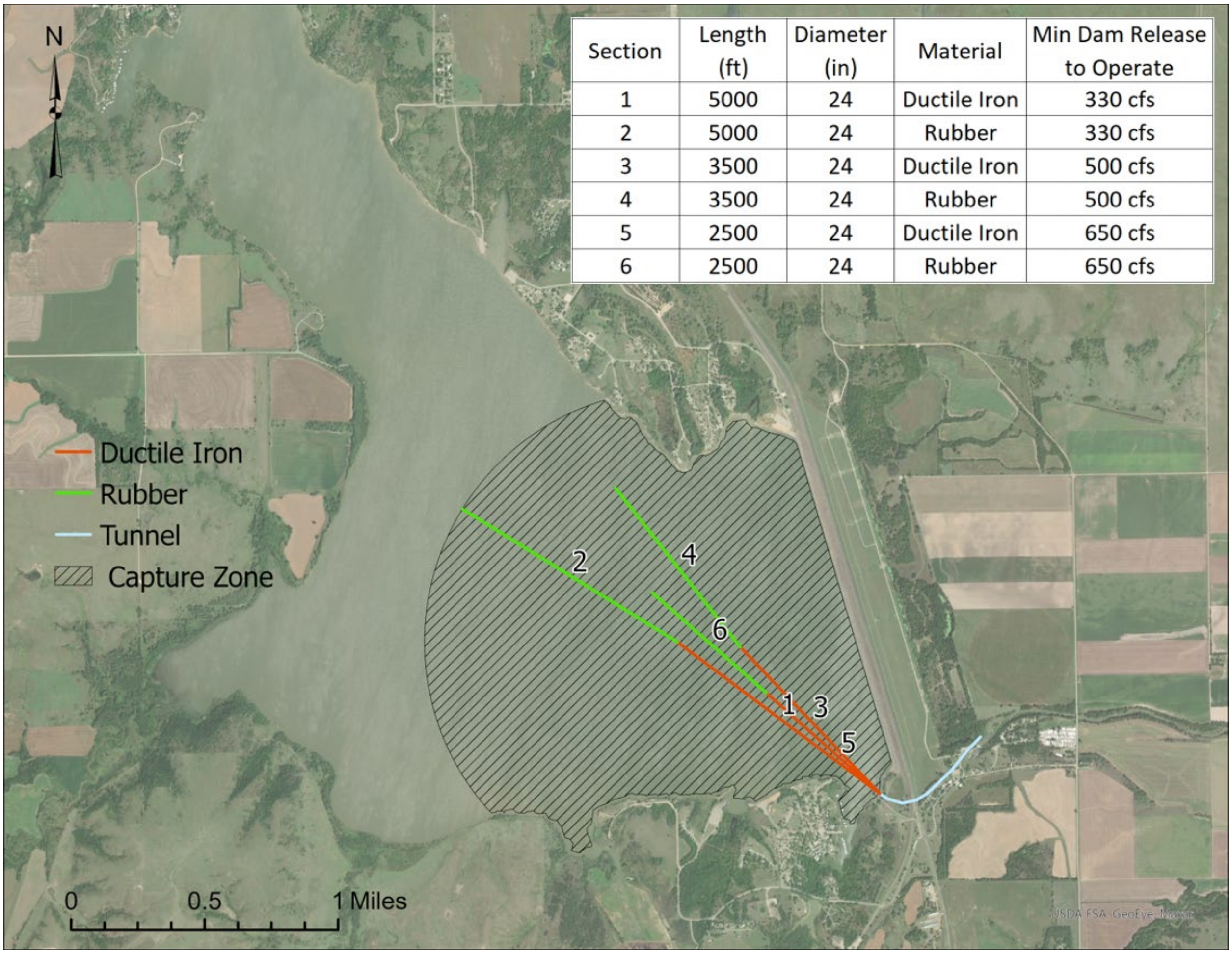


Figure 17: Sediment Concentration Measurements Upstream of Milford Lake, 80% Confidence Intervals, Sediment Concentration Resulting from Hydrosuction Pipes, and the Flow Thresholds when the Pipes begin Operating

4.5. Kanopolis Lake

The configuration used to determine the effectiveness of hydrosuction at Kanopolis Lake is given in Figure 18. An area of 1,310 acres within the lake can be reached using this configuration.



Section	Length (ft)	Diameter (in)	Material	Min Dam Release to Operate
1	5000	24	Ductile Iron	330 cfs
2	5000	24	Rubber	330 cfs
3	3500	24	Ductile Iron	500 cfs
4	3500	24	Rubber	500 cfs
5	2500	24	Ductile Iron	650 cfs
6	2500	24	Rubber	650 cfs

Figure 18: Hydrosuction Configuration for Effectiveness Calculations, Kanopolis Lake

Figure 19 plots the upstream concentration measurements and 80% confidence intervals which were presented in Appendix D1.5, along with the resulting sediment concentrations when the hydrosuction pipes are operating below the discharge thresholds.

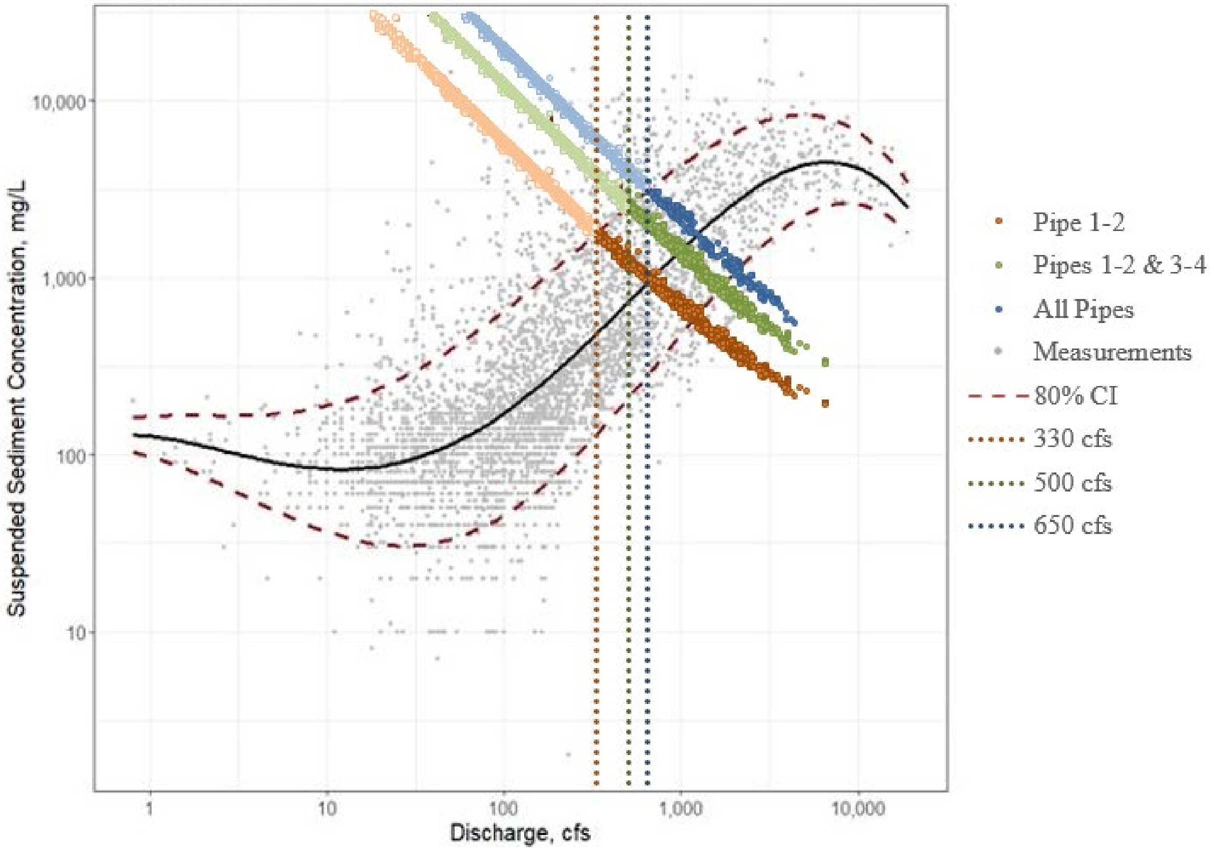
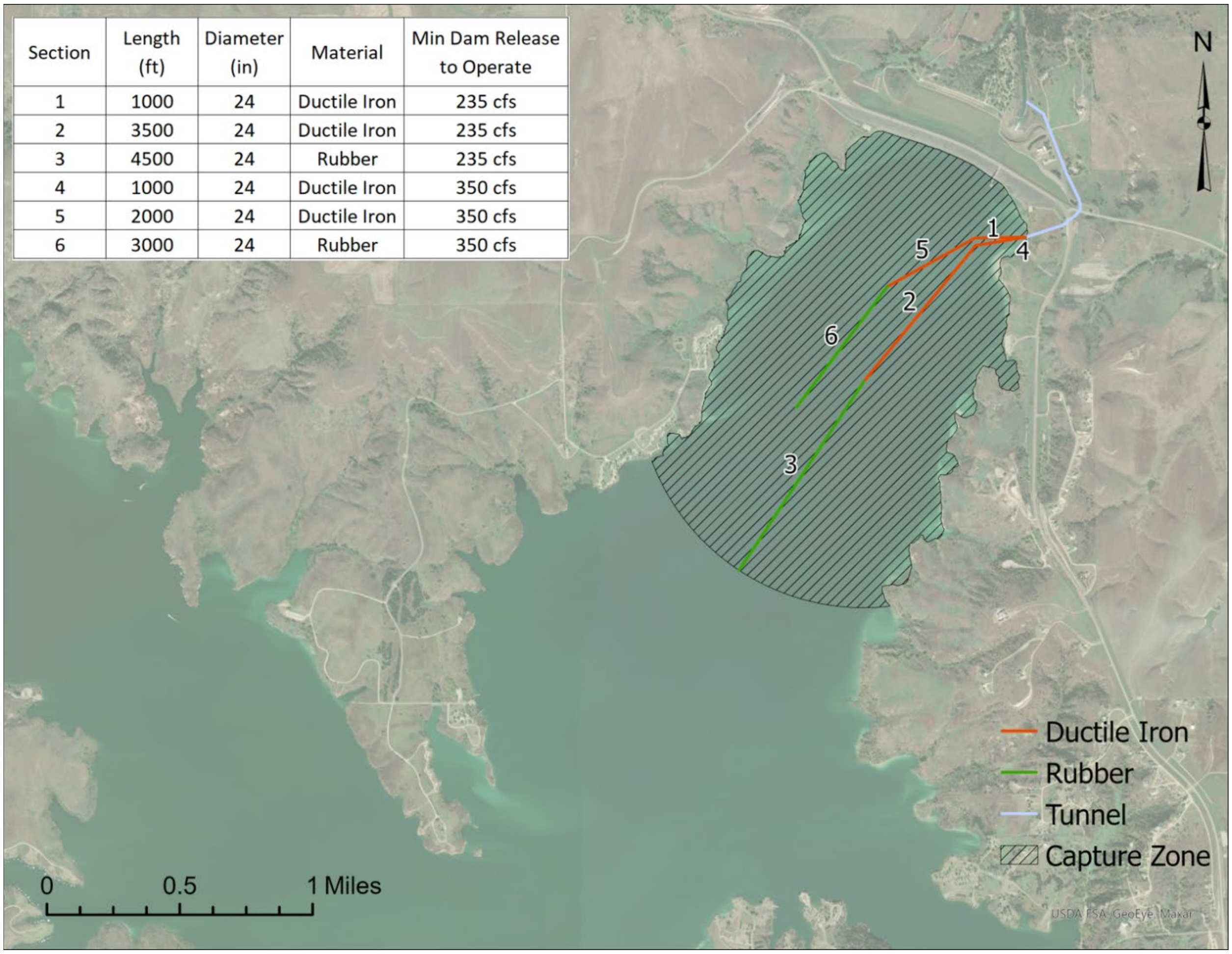


Figure 19: Sediment Concentration Measurements Upstream of Kanopolis Lake, 80% Confidence Intervals, Sediment Concentration Resulting from Hydrosuction Pipes, and the Flow Thresholds when the Pipes begin Operating

Further refinement of the design could be done to either capture more sediment or reach farther into the lake. However, this configuration was considered sufficient for the purposes of this study.

4.6. Wilson Lake

The configuration used to determine the effectiveness of hydrosuction at Wilson Lake is given in Figure 20. An area of 999 acres within the lake can be reached using this configuration



Section	Length (ft)	Diameter (in)	Material	Min Dam Release to Operate
1	1000	24	Ductile Iron	235 cfs
2	3500	24	Ductile Iron	235 cfs
3	4500	24	Rubber	235 cfs
4	1000	24	Ductile Iron	350 cfs
5	2000	24	Ductile Iron	350 cfs
6	3000	24	Rubber	350 cfs

Figure 20: Hydrosuction Configuration for Effectiveness Calculations, Wilson Lake

Figure 21 plots the upstream concentration measurements and 80% confidence intervals which were presented in Appendix D1.6, along with the resulting sediment concentrations when the hydrosuction pipes are operating below the discharge thresholds.

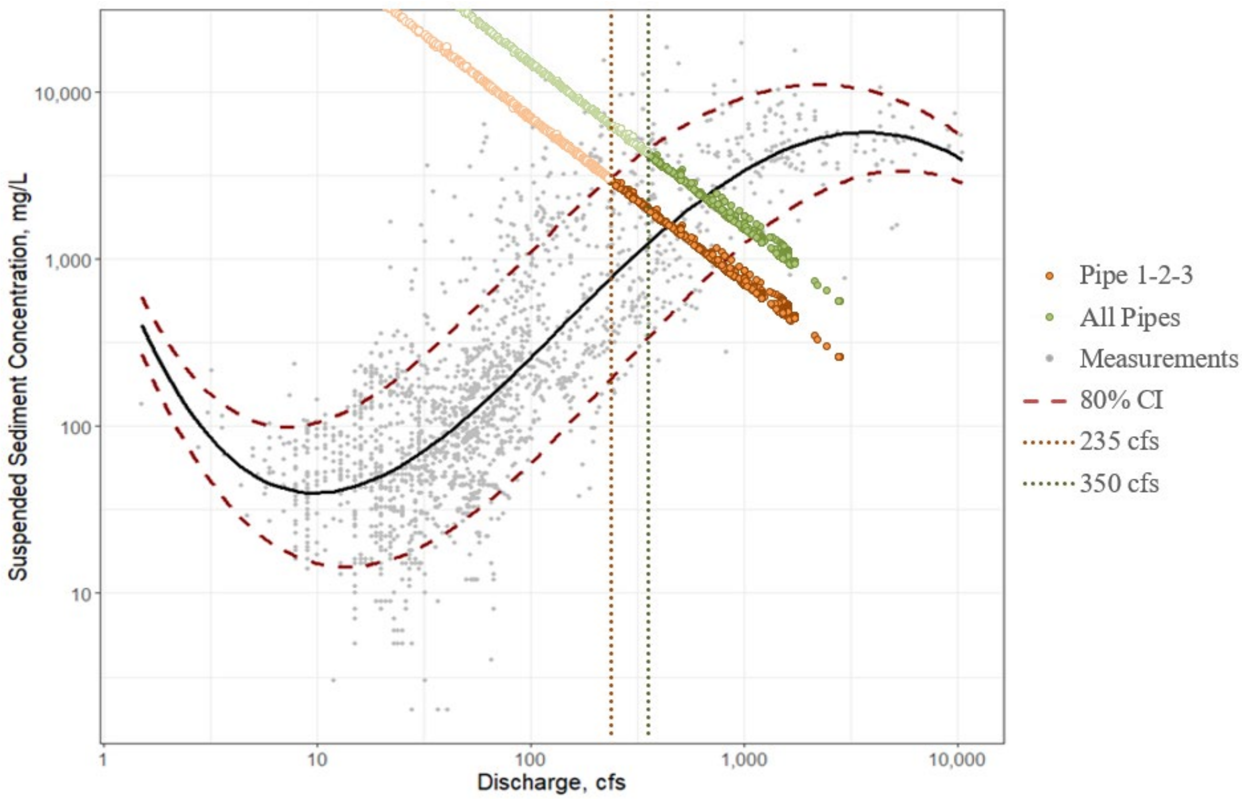
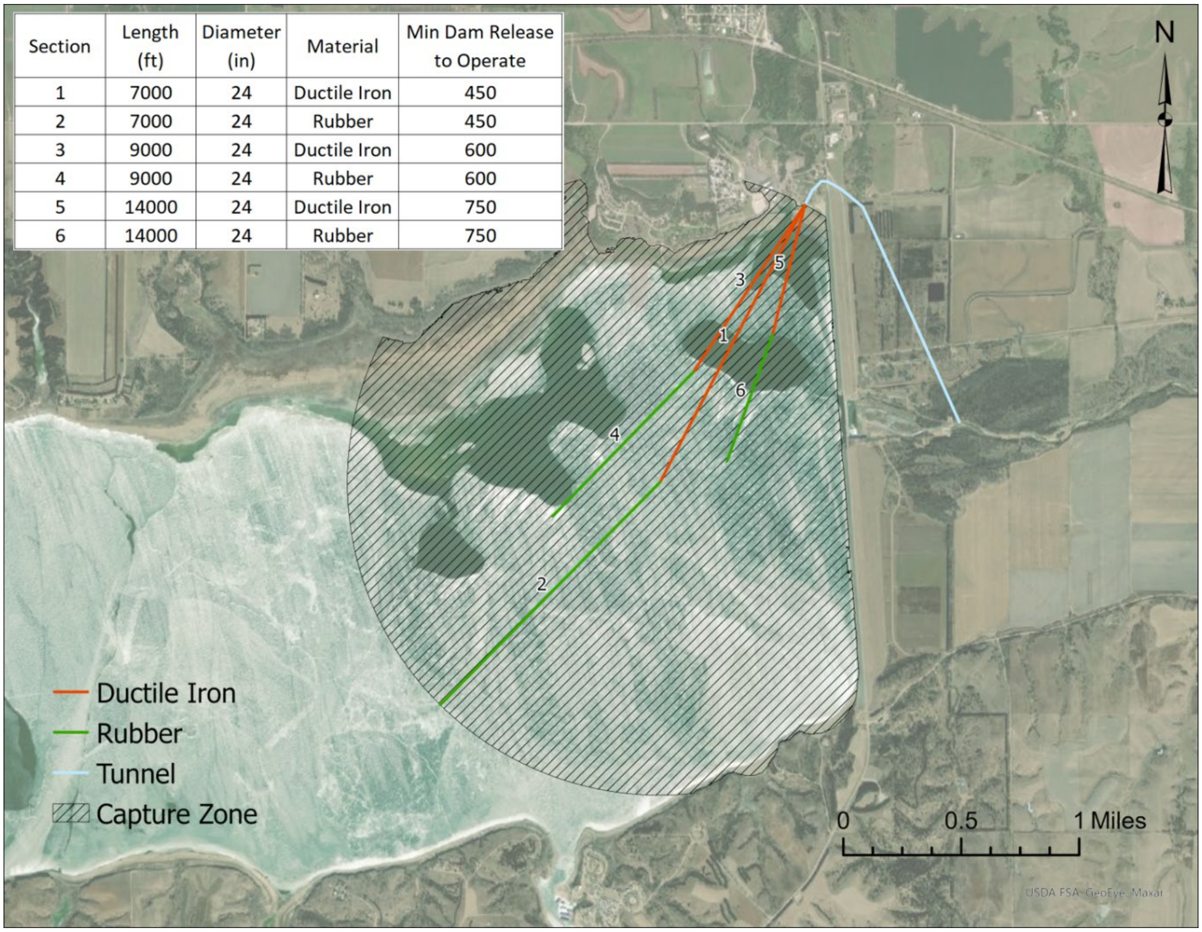


Figure 21: Sediment Concentration Measurements Upstream of Wilson Lake, 80% Confidence Intervals, Sediment Concentration Resulting from Hydrosuction Pipes, and the Flow Thresholds when the Pipes begin Operating

4.7. Harlan County Lake

The configuration used to determine the effectiveness of hydrosuction at Harlan County Lake is given in Figure 22. An area of 2,727 acres within the lake can be reached using this configuration.



Section	Length (ft)	Diameter (in)	Material	Min Dam Release to Operate
1	7000	24	Ductile Iron	450
2	7000	24	Rubber	450
3	9000	24	Ductile Iron	600
4	9000	24	Rubber	600
5	14000	24	Ductile Iron	750
6	14000	24	Rubber	750

Figure 22: Hydrosuction Configuration for Effectiveness Calculations, Harlan County Lake

Figure 23 plots the upstream concentration measurements and 80% confidence intervals which were presented in Appendix D1.7 for Harlan County Lake, along with the resulting sediment concentrations when the hydrosuction pipes are operating below the discharge thresholds.

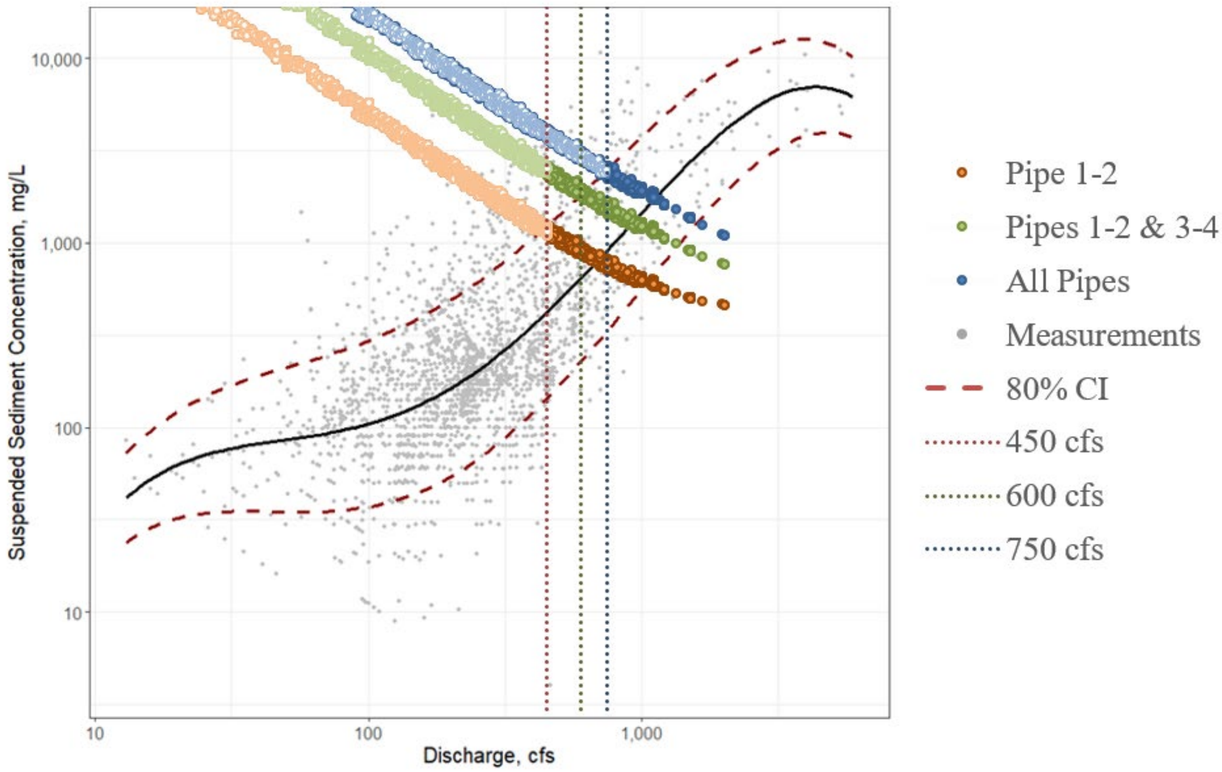


Figure 23: Sediment Concentration Measurements Upstream of Harlan County Lake, 80% Confidence Intervals, Sediment Concentration Resulting from Hydrosuction Pipes, and the Flow Thresholds when the Pipes begin Operating

5.0 PROJECTIONS

Long-term projections of the MPP volume were computed by running 100 years of daily flow and pool elevations through the calibrated rating curves using the same process described in Appendix B. However, the daily volume removed through hydrosuction was also included in the computations. The future flows and pool elevations were determined by

(1) Using measured flows from the date of the most recent survey through 2019
(2) Repeating the record of past years of daily flows until 2174

Table 4 summarizes the flow records used for each lake. The time periods were found reasonably representative to use for future projections.

Table 4: Flow Years and Pool Elevation Used in Future Projections

Lake	River	Lake Most Recent Survey Year	Repeated Flow Years	Repeated Pool Elevation Years	Years of Flow	Years of Pool Elevation
Clinton	Wakarusa	2019	1978-2019	1980-2019	42	40
Perry	Delaware	2009	1970-2019	1970-2019	50	50
Tuttle Creek	Big Blue	2009	1970-2019	1970-2019	50	50
Tuttle Creek	Little Blue	2009	1970-2019	1970-2019	50	50
Tuttle Creek	Black Vermillion	2009	1970-2019	1970-2019	50	50
Milford	Republican	2009	1970-2019	1970-2019	50	50
Kanopolis	Smoky Hill	2017	1970-2019	1970-2019	50	50
Wilson	Saline	2008	1970-2019	1970-2019	50	50
Harlan County	Republican	2000	1970-2019	1970-2019	50	50
Harlan County	Sappa Creek	2000	1970-2019	1970-2019	50	50

The volume available for hydrosuction within the capture zones of the pipes was computed over the 100 years to ensure that lakebed elevation would not go below the pre-impoundment surface. This was done by assuming that the lake MPP area would shrink by the values given in Appendix B, and by assuming that the amount of sediment deposition within the capture zone would be proportional to the capture zone area divided by the total MPP area; determined daily by linearly interpolating from the values given in Appendix B. Figure 24 gives an example for Milford Lake, which shows the sediment available within the capture zone being depleted near the end of the study period. After this point the amount of sediment that can be removed from the lake is limited by how much deposits within the capture zone. The initial volume of sediment within the capture zones was estimated using the sedimentation rangeline surveys and the Cross Section Viewer Software (Shelley and Bailey, 2018).

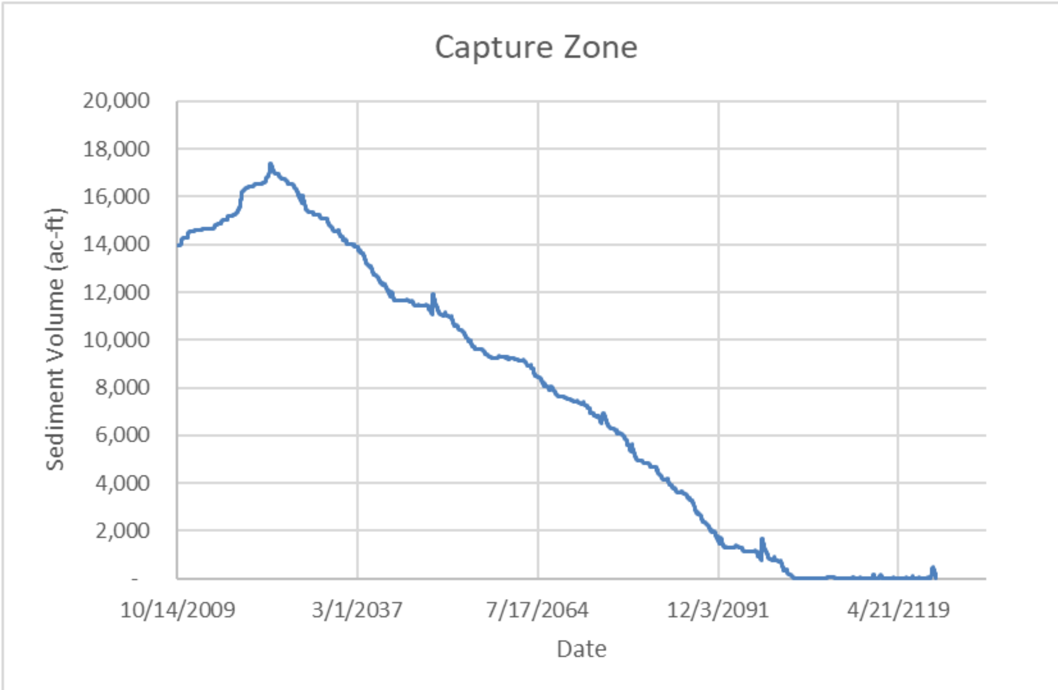


Figure 24: Available Sediment Volume within the Capture Zones of the Hydrosuction Pipes at Milford Lake

The projected volume of the MPP for the FWOP and FWP are shown in Figures 25 through 31.

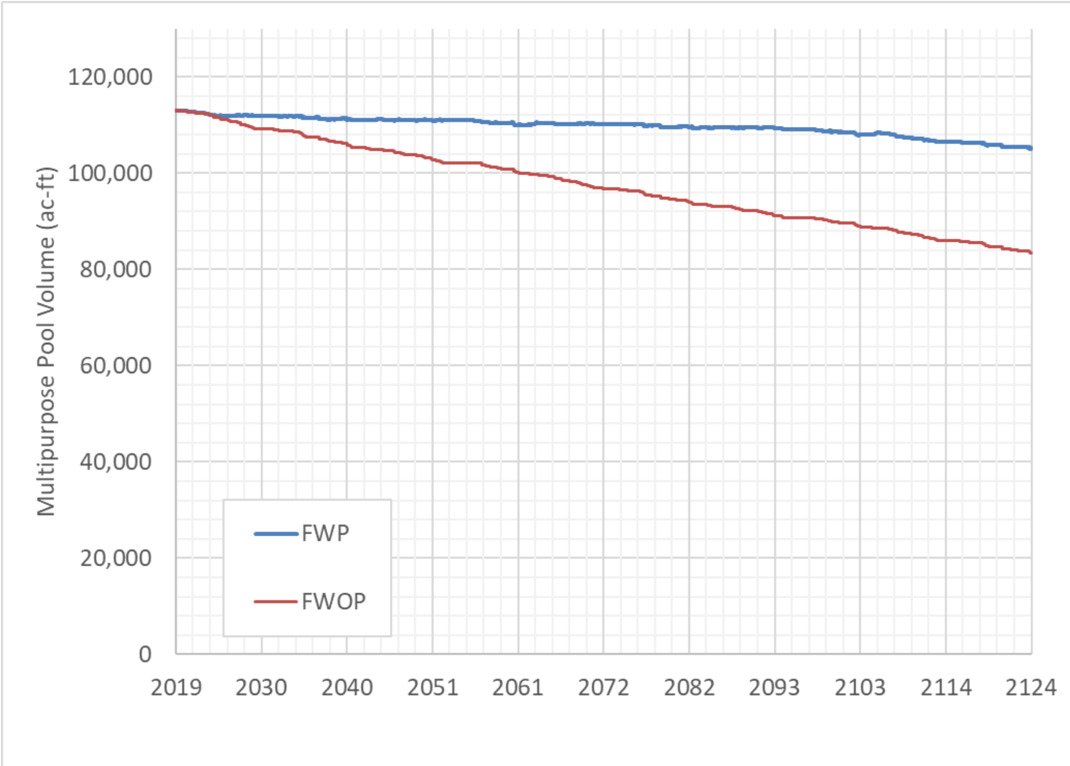


Figure 25: Clinton Lake Multipurpose Volume FWOP and FWP

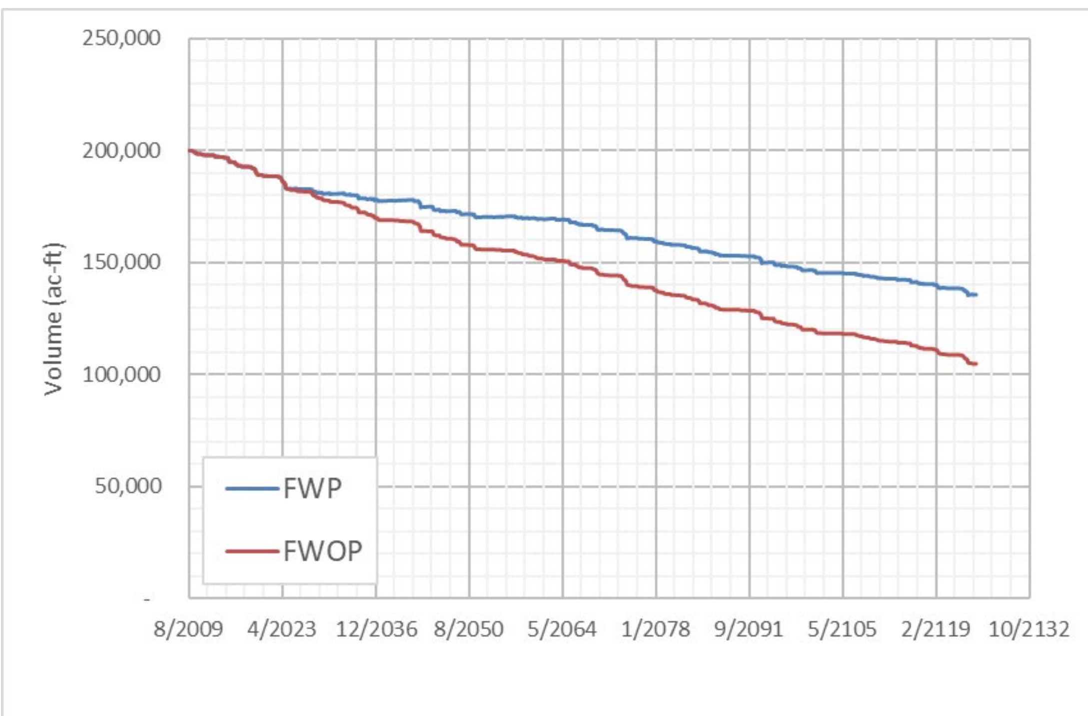


Figure 26: Perry Lake Multipurpose Pool Volume FWP and FWOP

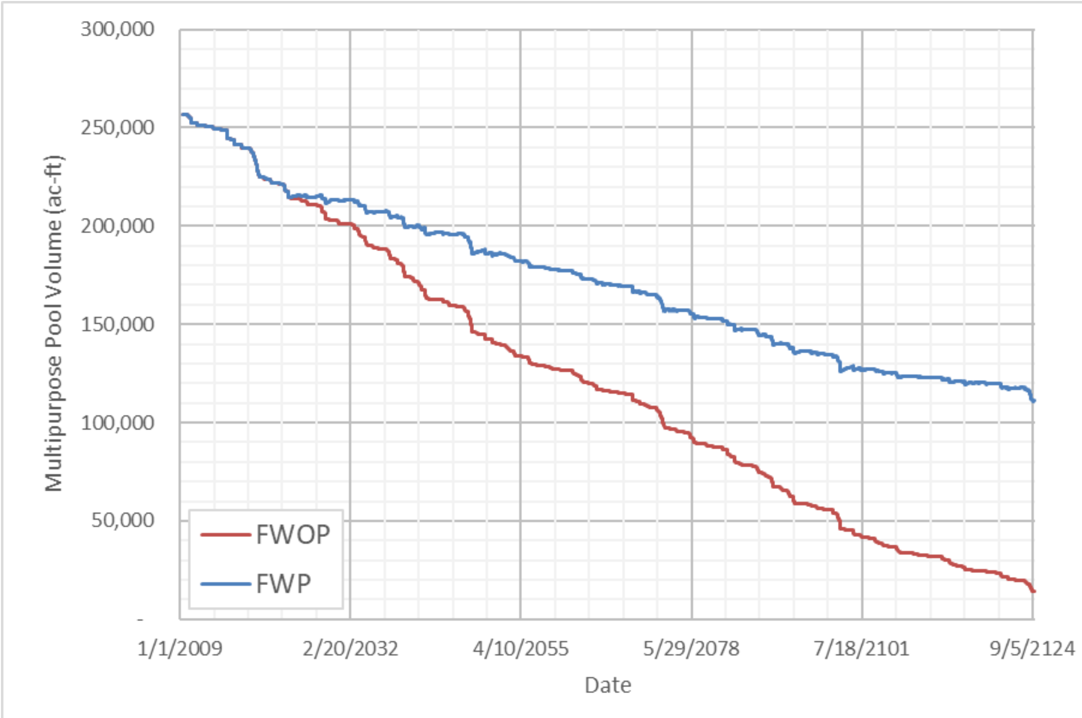


Figure 27: Tuttle Creek Multipurpose Pool Volume FWOP and FWP

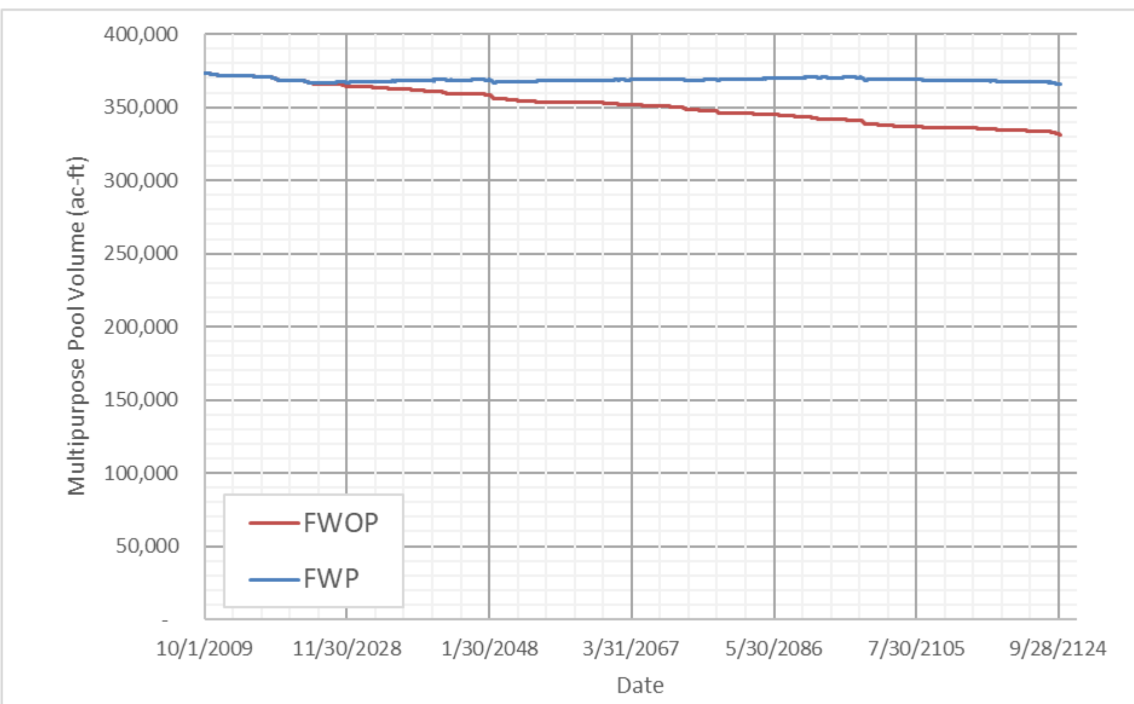


Figure 28: Milford Multipurpose Pool Volume FWP and FWOP

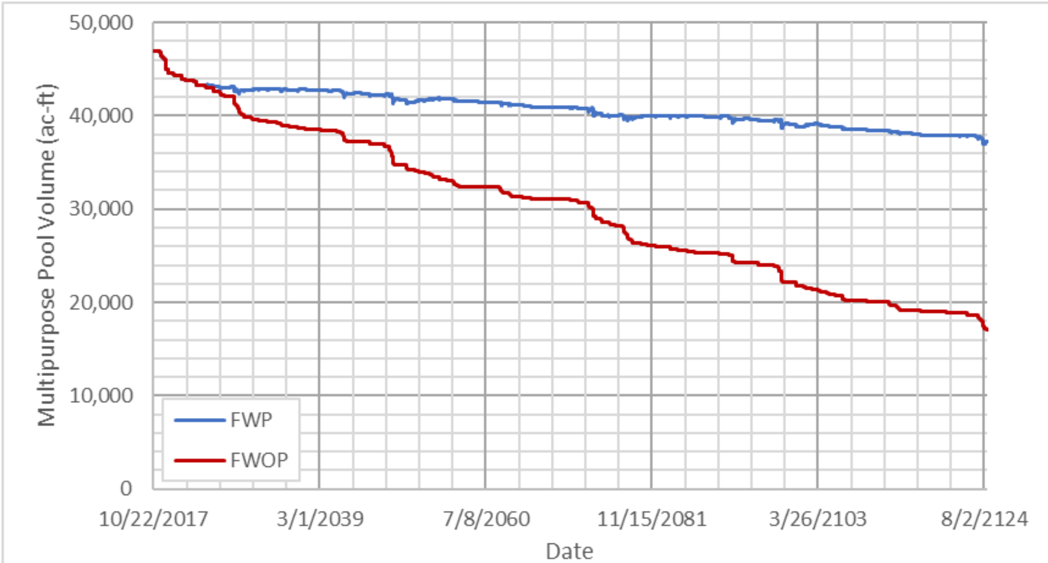


Figure 29: Kanopolis Multipurpose Pool Volume FWP and FWOP

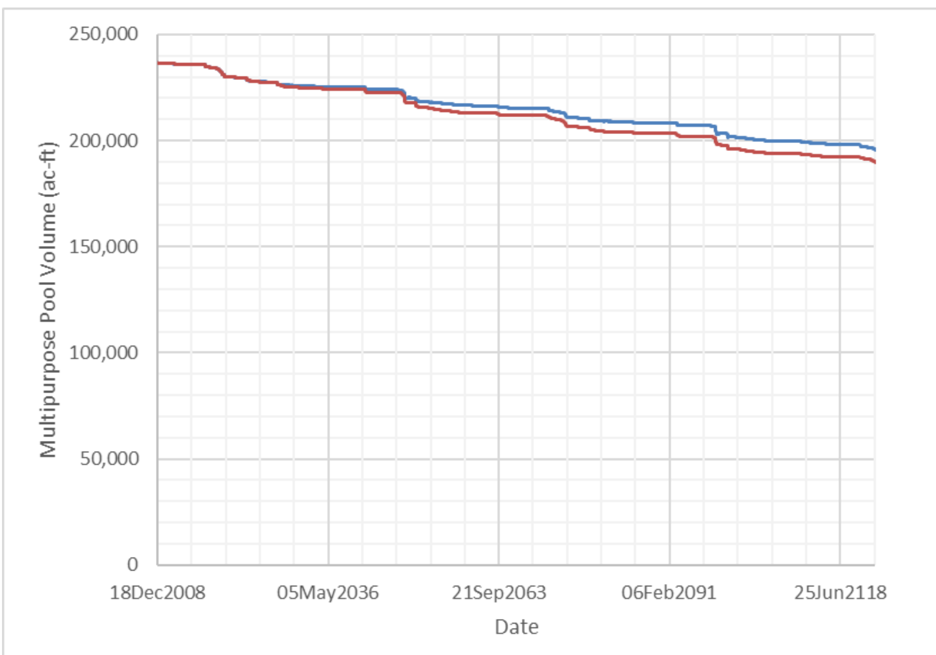


Figure 30: Wilson Multipurpose Pool Volume FWP and FWOP

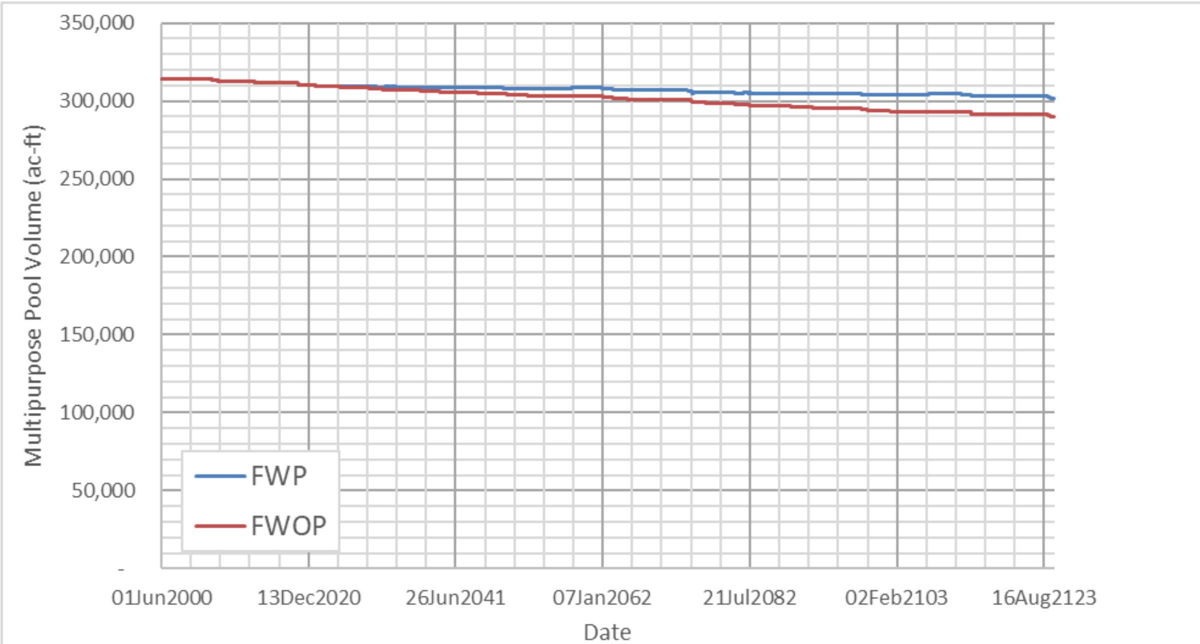


Figure 31: Harlan County Multipurpose Pool Volume FWP and FWOP

Table 4 shows the MPP capacity over time for the FWOP and FWP at the seven lakes, the percentage of the 2024 capacity, and the ratio of FWP over FWOP.

Table 5: Multipurpose Pool capacity for the FWOP and FWP. Volume estimates in ac-ft.

	Clinton	**Perry**	**Tuttle Creek**	**Milford**	**Kanopolis**	**Wilson**	**Harlan County**
2024 Volume	112,150	182,890	214,284	366,481	43,256	227,833	309,372
2049 Volume FWOP	103,854	159,384	145,904	355,800	34,794	217,956	303,952
2049 Volume FWP	111,085	172,524	186,215	367,196	41,767	220,122	307,886
2049 % of 2024 FWOP	93%	87%	68%	97%	80%	96%	98%
2049 % of 2024 FWP	99%	94%	87%	100%	97%	97%	100%
2049 Ratio FWP/FWOP	1.07	1.08	1.28	1.03	1.20	1.01	1.01
2074 Volume FWOP	96,669	139,945	105,054	349,881	30,207	209,022	300,735
2074 Volume FWP	110,161	160,914	163,225	369,158	40,704	212,832	306,669
2074 % of 2024 FWOP	86%	77%	49%	95%	70%	92%	97%
2074 % of 2024 FWP	98%	88%	76%	101%	94%	93%	99%
2074 Ratio FWP/FWOP	1.14	1.15	1.55	1.06	1.35	1.02	1.02
2124 Volume FWOP	83,853	105,160	18,081	332,971	18,248	190,117	291,081
2124 Volume FWP	105,384	135,644	116,333	366,922	37,809	195,949	302,824
2124 % of 2024 FWOP	75%	57%	8%	91%	42%	83%	94%
2124 % of 2024 FWP	94%	74%	54%	100%	87%	86%	98%
2124 Ratio FWP/FWoP	1.26	1.29	6.43	1.10	2.07	1.03	1.04

In 2016, 3 million yd^3 of material was dredged from John Redmond Reservoir at a cost of $20 million or $6.67 / yd^3 (KWO 2016). These costs included the dredging contract price and the price for some of the land used for sediment disposal. A portion of the land used for sediment disposal was donated by USACE. The total volume of sediment removed through hydrosuction and the equivalent cost it would take to remove that volume via traditional dredging, assuming $6.67 / yd^3 is given in Table 5. The $6.67 / yd^3 is a low estimate, as it does not account for inflation.

Table 6: Total Volume of Sediment Removed through Hydrosuction and Cost if Removed Using Traditional Dredging Techniques

Lake	Total Removed (ac-ft)	Total Removed (million yd^3)	Equivalent Cost in Millions at $6.67 / yd^3
Clinton	26,352	42.5	$283.6
Perry	39,274	63.4	$422.6
Tuttle Creek	134,978	217.8	$1,452.5
Milford	37,435	60.4	$402.8
Kanopolis	24,676	39.8	$265.5
Wilson	14,437	23.3	$155.4
Harlan County	12,705	20.5	$136.7

A feasibility level analysis would be needed to provide detailed cost estimates to install and operate a hydrosuction system at the lakes. However, an order of magnitude estimate predicted that $80M would be needed to install a hydrosuction system at Tuttle Creek (USACE, 2017), with most of the cost being the construction of two tunnels through the right abutment. This cost would be for installation of the capital improvements and would not include the cost of operating the system. Further analysis of the cost through a feasibility study would be needed before determining the cost vs. benefits of hydrosuction, since the cost will vary by site and the cost estimate was only intended to be a very rough order of magnitude estimate. Assumptions such as wether or not the reservoir is drained for construction would have a significant impact on the cost.

6.0 DISCUSSION AND CONCLUSIONS

Hydrosuction as analyzed in this report can pass significant quantities of sediment, but is not able to fully maintain the existing multipurpose pool storage volumes for the majority of the lakes. The reasons are (1) hydrosuction is not able to pass a sufficient quantity of sediment during the highest releases and (2) the pipes do not extend throughout the entire lake.

Of the seven lakes, Milford is the only one where the 2024 MPP capacity is maintained through the end of the study period. However, the ratio of FWP/FWOP volumes is lower for this lake than many of the others, because the capacity loss is not as significant overall. Tuttle Creek has the highest ratio of FWP/FWOP since the MPP is essentially filled by the year 2091 for the FWOP condition. Because of this, the greatest benefit from hydrosuction occurs in year 2074 rather than 2124, as the pool continues to fill between 2091 and 2124 for the FWP. The difference between the 2124 FWOP and FWP 2124 as percentage of 2024 capacity is greatest for Kanopolis Lake at 55%.

The price to remove the same amount of sediment using traditional dredging techniques was estimated using a cost of $\$6.67/yd^3$ from a dredging project at nearby John Redmond Lake. However, this does not account for the likelihood that the cost would increase as the distance to disposal sites would increase as land becomes less available near the dam. This report did not estimate the cost to construct, operate, and maintain the hydrosuction system. This will likely vary significantly between the lakes based on the length of the pipelines and other factors.

While hydrosuction by itself cannot achieve full sustainability, it could be used in combination with other means. For example, full sustainability could be achieved with the addition of booster pumps to allow longer pipelines and to allow higher sediment discharge rates during high water releases. Likewise, drawdown flushes that pass significant quantities of sediment could redistribute sediment from upper reaches to the lower reaches accessible to the hydrosuction pipes.

REFERENCES

Hotchkiss, R., and Xi Huang. 1995. "Hydrosuction removal systems (HSTS): principles and field test." *Journal of Hydraulic Engineering* (Journal of Hydraulic Engineering) 4479-489.

Jimenez, A., and T. Jacobsen. 2015. *Dredging of Cohesive Sediments with SedCon Dredge in El Canada Hydropower Plant in Guatemala.* Commision Internationale Des Grads Barrages. Congress Des Grads Barrages.

KWO. 2016. *John Redmond Reservoir Dredging Fact Sheet.* Kansas Water Office.

McFall, Brian, and Tim Welp. 2015. *Tuttle Creek Dam Siphon Dredging Investigation.* Vicksburg, MS: U.S. Army Engineer Research and Development Center.

Shelley, John. 2015. *Reservoir Sediment Management Workshop for Tuttle Creek Lake and Perry Lake Reservoirs in the Kansas River Basin.* Vicksburg, MS: U.S. Army Engineer Research and Development Center.

Shelley, John, and Philip Bailey. 2018. *The Cross Section Viewer: A Tool for Automating Geomorphic Analysis Using Riverine Cross-Section Data ERDC/TN RSM-18-3.* Vicksburg, MS: U.S. Army Corps of Engineers, Engineer Research and Development Center.

USACE. 2013. *Clinton Dam Periodic Inspection No. 12 Periodic Assessment No. 1.* U.S. Army Corps of Engineers, Kansas City District.

USACE. 1943. *Harlan County Dam and Reservoir Definite Project Report.* Kansas City, Missouri: U.S. Army Corps of Engineers. Kansas City District.

USACE. 1973. *Lower Kansas River Basin Lake Regulation Manual in 6 Volumes, Volume No. 2, Tuttle Creek Lake, Kansas.* U.S. Army Corpose of Engineers.

USACE. 2016. *Milford Dam Periodic Inspection No. 14 Periodic Assesment No. 1.* U.S. Army Corps of Engineers, Northwestern Division, Kansas City District.

USACE. 2018. *Perry Dam Periodic Inspection No. 14 Periodic Assessment No. 1.* U.S. Army Corps of Engineers, NWD Division, NWK District.

USACE. 2017. *Tuttle Creek Lake Hydrosuction Assessment.* Kansas City: U.S. Army Corps of Engineers.

US Army Corps of Engineers ®
Kansas City District

Kansas River Reservoirs Flood and Sediment Study

Appendix D5.4: Flushing

September 2023

Contents

1. INTRODUCTION

This appendix presents analysis on the effectiveness of drawdown flushing for managing sediment and preserving storage in Kansas River lakes. Two analyses are presented, (1) a screening-level analysis for all federal lakes in the basin and (2) an empirical equation to analyze flushing at Tuttle Creek Lake specifically.

A drawdown flush consists in a complete lowering of the reservoir pool down to riverine conditions. Over a short period of time (typically one to two weeks), the river scours and transports downstream a significant quantity of deposited sediment. Flushing also moves coarser sediments from the delta towards the dam.

Flushes work best for hydrologically small reservoirs, i.e. where the storage to be maintained is small relative to the ability of the watershed to refill. Dahl and Ramos-Villanueva (2019) summarize:

> The primary screening criteria for successful flushing of reservoirs in current guidance are based on three factors: the total capacity of the reservoir (CAP), the mean annual runoff to the reservoir (MAR), and the mean annual inflow of sediment to the reservoir (MAS). The lower the ratios of CAP/MAR and CAP/MAS, the more likely it is that flushing can be a successful method for maintaining reservoir capacity (Atkinson 1996; ICOLD 1999; Kondolf et al. 2014; Sumi 2008).

Several authors have compiled the CAP/MAR and CAP/MAS ratios of reservoirs where flushing has occurred in order determine which range of ratios over which flushing is most likely to be fully successful at maintaining the reservoir capacity. Figure 1 includes data from all the Kansas River Basin lakes, plus projected Future Without Project conditions for Perry Lake, Tuttle Creek Lake, and Kanopolis Lake, on a plot modified from Annandale (2013). The original data in the plot, shown in gray, indicates reservoirs where various sediment management strategies have been successfully employed. The green dots indicate existing conditions for all the federal lakes in the Kansas River Basin. The red, blue, and purple dots indicate projections for Tuttle Creek Lake, Kanopolis Lake, and Perry Lake, respectively. The dashed boxes encapsulate clusters of dams where a given management strategy has been successfully employed to achieve sustainability, described in Annandale (2013) as at least a 300 year life.

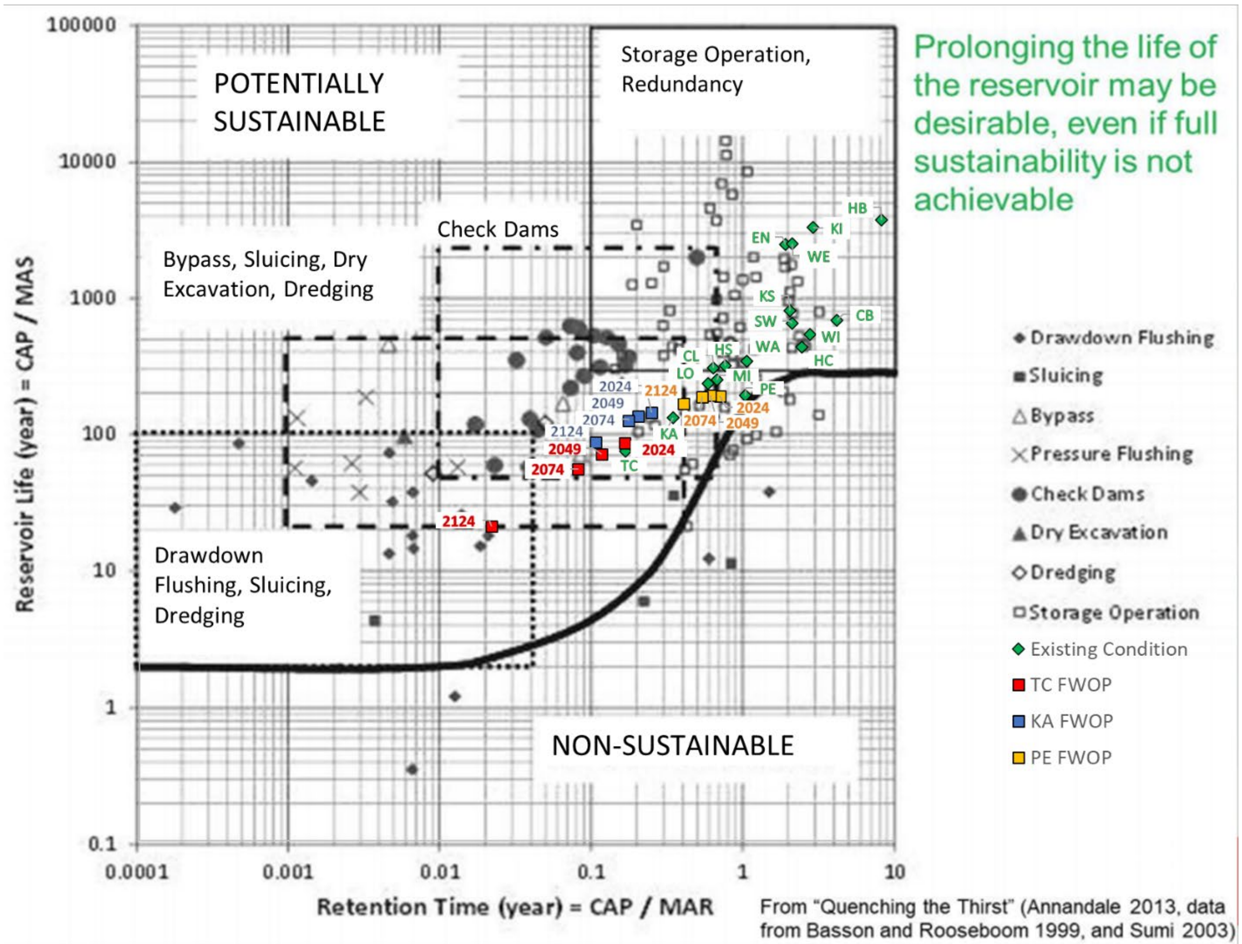


Figure 1. Sediment Management Alternatives Screening

As seen, none of the Kansas River Basin lakes currently fall within the ratios where flushing has been successful at achieving full sustainability. Flushing could be more effective in the future; as the reservoirs shrink due to sediment, the CAP/MAR and CAP/MAS ratios decrease, making flushing more effective. According to Figure 1, flushing becomes feasible for Tuttle Creek Lake in between 50 and 100 years.

Figure 1 is limited for several reasons. On flushing specifically, it does not take sediment type into account or allow an assessment of longer drawdown flushes than the typical 1 to 2 weeks. The following flushing analysis overcomes these limitations.

Moreover, it does not include all options such as hydrosuction, water injection dredging, intensive sediment reduction in the watershed, or combinations of measures. Nor does it indicate which strategies could prolong the life of a dam even if full sustainability isn't achievable.

2. FLUSHING COMPUTATION AT TUTTLE CREEK LAKE

The International Research and Training Centre on Erosion and Sedimentation (IRTCES) (1985) developed an equation for estimating the sediment removed by a drawdown flush. This equation was derived empirically from multiple flushing events at multiple Chinese reservoirs (see Figure 2).

$$Q_s = \psi \frac{Q_f^{1.6} S^{1.2}}{W^{0.6}} \qquad \text{(Eq 1)}$$

Where Qs = the sediment transport capacity (metric tons/day)

Q_f = the flushing discharge, m^3/s

S = the bed slope

W = the channel width, m

ψ = a constant to account for sediment type

1600 for loess sediments

650 for other sediments, $d_{50} < 0.1$ mm

300 for $d_{50} > 0.1$ mm

180 for flushing with a low discharge

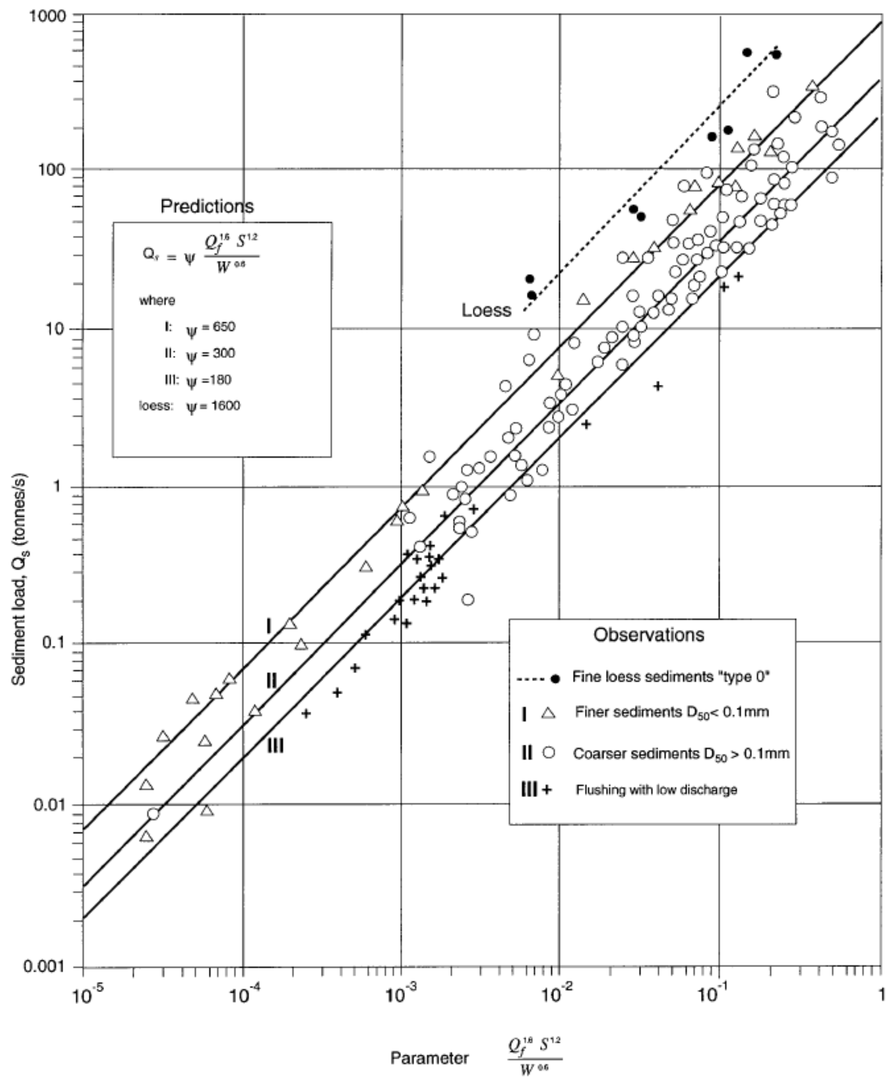


Figure 2. IRTCES Method for Predicting Flushing Efficiency. From Atkinson (1996).

This equation was applied to Tuttle Creek Lake to estimate the effectiveness of flushing. As draining Tuttle Creek Lake for any duration of time would destroy the lake fishery, the preference is for longer flushes that occur less often, rather than an annual or semi-annual flush for 1 to 2 weeks.

Erodibility testing at Tuttle Creek Lake (Shelley and Wells 2019) indicated that the upper layers of sediment are very erodible, with critical shear stresses ranging from 0.1 to 1.6 Pa for the top 44 inches of deposition. To be conservative, a value of 650 was assumed for ψ. The width of the existing channel, 152 m, was used for W.

The flushing transport is an exponential function of the flushing discharge. As there are no large lakes upstream of Tuttle to supply a constant flow for flushing, the flushing discharge varies daily according to natural hydrologic variability. The ResSim model was used to develop the hydrologic time series for the flushing. The following steps were used:

1. A target lake drawdown was set to an elevation of 1020.1 set to begin on March 1st of each year. Drawdown releases remained within downstream flood control targets, meaning if inflows exceeded downstream flood control targets, releases were reduced and water stored.
2. The target lake level was kept at this elevation until June 30th of each year.
3. After June 30th, the target elevation was set to 1075 and the lake allowed to refill.
4. For each day that the lake was in its drawn-down condition, Equation 1 was used to compute a daily sediment flushing mass. No sediment was assumed to leave the lake unless the lake was fully drawn down.
5. The mass of flushed sediment was summed for the year.

Figure 3 shows the flush effectiveness for each of 99 inflow year scenarios. The mean flushing effectiveness is 15,800 ac-ft of sediment removed for the 4-month flush, which is approximately 4 times the multipurpose pool annual sediment accumulation rate.

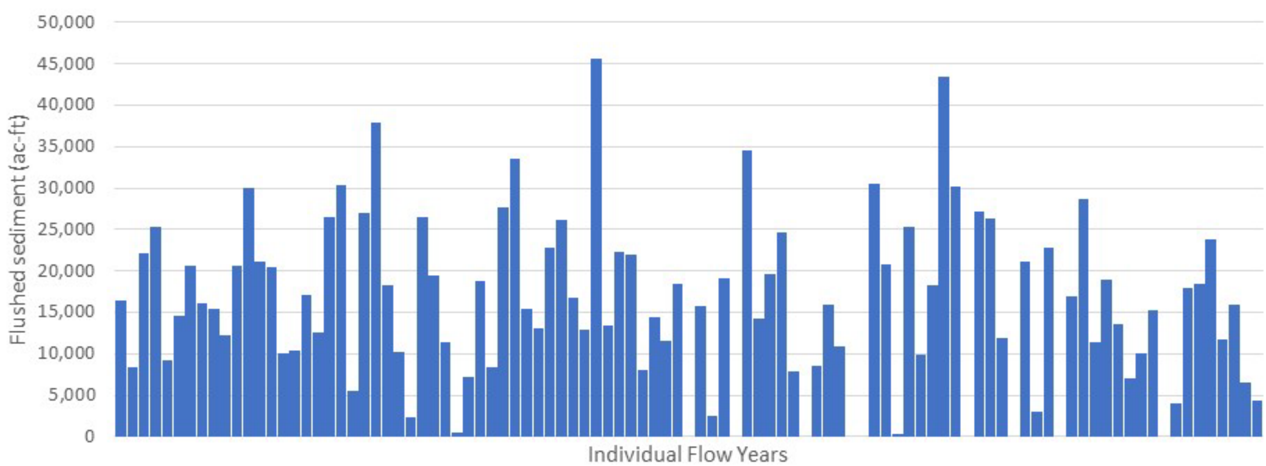


Figure 3. Theoretical Maximum Flushing Effectiveness

Figure 3 represents the theoretical maximum flushing effectiveness. One constraining factor is that flushing channels tend to incise and disconnect from the floodplain, which limits the area available for sediment removal. From 1963 to 2020, 14,820 ac-ft of sediment deposited within the boundaries of the submerged channel, which represents approximately 7% of the total multipurpose pool deposition over that time frame. Thus, a single 4-month flush would be sufficient to return the channel to pre-impoundment capacity, offsetting 3.9 years of overall multipurpose pool accumulation. At $6.7/CY (the 2015 cost of dredging John Redmond Lake), this would equate to over $160M of avoided dredging costs from a single, 4-month drawdown flush. Future flushes could be scheduled at intervals depending on how quickly the channel refills. The flushing channel would most likely remain within the banks of the existing channel, leaving 97% of the existing lake sediment accumulations intact. Widening beyond the original channel banks is unlikely.

Ways to improve the effectiveness of flushing include:

1. Storing extra water in adjacent reservoirs to offset risk due during droughts
2. Partial drawdowns to speed channel refilling with sediment
3. Water injection dredging to speed channel refilling with sediment

Flushing would likely have a short-term negative impact on the downstream channel due to sediment concentrations in excess of natural levels. Numerical modeling is suggested to better estimate release concentrations and inform ways to minimize impacts.

3. CONCLUSIONS

Drawdown flushing works best at hydrologically small lakes. None of the Kansas River Basin lakes are currently as small as lakes which are sustainably managed by drawdown flushing alone. If nothing is done, Tuttle Creek Lake will be similarly small between 50 and 100 years in the future. A drawdown flush at Tuttle Lake during a median year could theoretically remove 15,800 ac-ft of sediment, but because the flushing scour would be confined to the historic channel, a practical maximum for an initial flush would be 14,820 ac-ft. Traditional dredging costs to remove that volume of sediment (using non-escalated costs from John Redmond of $6.7/CY) would exceed $160 M. Such a flush would remove approximately 7% of the multipurpose pool deposition, leaving 93% of the deposited sediment perched on the floodplain. Thus, while a drawdown flush may be cost effective, it would not be a complete solution to maintaining a large multipurpose pool. Moreover, flushing would likely have a short-term negative impacts on the downstream channel due to sediment concentrations in excess of natural levels as well as completely decimating the lake fishery.

4. REFERENCES

Annandale, G. (2013). Quenching the Thirst: Sustainable Water Supply and Climate Change. CreateSpace Independent Publishing Platform. ISBN: 1480265152, 9781480265158

Brune, G. M. (1953). *Trap Efficiency of Reservoirs.* American Geophisical Union.

Dahl, T. and Ramos-Villanueva, M. (2019). "Overview of historical reservoir flushing events and screening guidance." ERDC/CHL CHETN-VII-21. U.S. Army Corps of Engineers, Vicksburg, MS. https://hdl.handle.net/11681/33003

IRTCES. (1985). *Lecture notes of the training course on reservoir sedimentation.* International Research and Tranining Centre on Erosion and Sedimentation.

Shelley, J., & Wells, R. R. (2019). *Erodibility Characteristics of Cohesive Sediment Deposits in a Large Midwestern Reservoir.* Conference: Federal Interagency Sedimentation and Hydrologic Modeling ConferenceAt: Reno, NV.

US Army Corps
of Engineers ®
Kansas City District

Kansas River Reservoirs Flood and Sediment Study

Appendix D5.5: EAB Letter

September 2023

CHIEF OF ENGINEERS
ENVIRONMENTAL ADVISORY BOARD
Army Science Board Subcommittee

Lieutenant General Todd T. Semonite 17 AUG 2020
Commanding General and Chief of Engineers
U.S. Army Corps of Engineers
441 G Street NW
Washington, DC 20314-1000

Subject: Sustainable Sediment Management at U.S. Army Corps of Engineers Reservoirs

Dear LTG Semonite:

The Chief of Engineers' Environmental Advisory Board (EAB), a Subcommittee of the Army Science Board, evaluated sediment management issues at Corps of Engineers' (Corps) reservoirs from a national perspective. The EAB recommends that long-term ecosystem health and the sustainability of reservoir project benefits could be improved by re-establishing sediment continuity through reservoirs and the whole riverine system. Sediment continuity is achieved when the sediment generated from the watershed and upstream river channels is allowed to pass through or around a dam to the downstream river corridor. The EAB recommends the following specific actions. The Corps should:

1. Recognize the downstream channel system and receiving coastal systems as preferred beneficial uses for reservoir and dredging sediments, subject to the principles discussed in the attached document.

2. Expand the footprint for assessing cost-benefits of reservoir sediment management measures to include both downstream and upstream river corridors.

3. Analyze storage lost to sedimentation as a reallocation, with an assessment of lost benefits and associated increased costs both upstream and downstream of the project footprint.

4. Highlight existing sediment passage pilot projects and implement new projects that demonstrate different management options.

5. Hold reoccurring reservoir sediment management training courses. The Regional Sediment Management (RSM) program has produced and held two such courses, one for regulators, managers, and planners (held in 2017) and one for engineers (held in 2018).

Taking these steps can help to enhance upstream and downstream ecosystems and encourage the adoption of cost-effective, environmentally-sound reservoir sediment management practices which will extend the useful life of Corps reservoir projects. These steps are congruent with and build upon current Corps initiatives such as the Environmental Operating Principles (EOPs), Environmental Flows, Sustainable Rivers, and the Corps' RSM and Engineering with Nature programs.

The attached document provides general background and additional justification for each of the five recommendations. This EAB study was led by Dr. Rollin Hotchkiss with significant contributions by Dr. Melinda Daniels, who are available to answer any questions. We hope the recommendations will be useful and look forward to working with your staff on implementation.

Sincerely,

Mary C. Barber, PhD
Chair, Environmental Advisory Board
Subcommittee of Army Science Board

Attachment

CF:
Chief, Planning & Policy Division
Chief, Environmental Division
Chief, Operations & Regulatory Division
Chief, Engineering & Construction Division
Director, ERDC Environmental Laboratory

ATTACHMENT.

Sustainable Sediment Management at U.S. Army Corps of Engineers Reservoirs

Introduction & Background

The Corps is the largest operator of dams in the United States. Each Corps dam was planned, designed and built to provide specific benefits to the American public, including navigation, flood risk reduction, hydropower generation, recreation, and water supply. Most Corps dams have operated for more than 50 years, with some approaching 100 years of operation. Corps dams and reservoirs provide crucial benefits to the nation, especially during times of flooding or drought.

Corps dams and reservoirs were designed with an understanding of dynamic river processes and accounted for a certain amount of sedimentation over their economic design life (much shorter than the actual life of the structure). Planners and Designers have accounted for a rate of sedimentation by identifying a volume of future sediment storage, called the inactive pool or dead pool. Sedimentation occurs throughout the pool, however, and thus can impact conservation pool storage or flood control storage, depending on reservoir operations and incoming sediment timing and magnitude. Once deposited, sediment can also move throughout the pool. As this sediment accumulates over the design life of 100 years or more, it can result in a "reduction in the reliability of water supply, burial of dam outlets and intakes for water supply and power production, damage to hydropower and pumping equipment, burial of boat ramps or marinas, navigation impairment, reduction in the surface area for lake recreation, and increased flood stages upstream" (Randle et. al 2019). Deposition upstream from the reservoir and scour downstream from the dam can also cause well-documented ecosystem and infrastructure damage far from the dam location (George et al. 2016, Kondolf et al. 2014).

Even planned for, the Corps recognizes that dams interrupt the downstream movement of sediment. For example, Regulatory Guidance Letter (RGL) 18-01 (USACE 2018) states:

> Dams and other obstructions disrupt the sediment transport that is critical to sustaining the habitat of riverine and riparian species, including the variations in sediment sizes that are important for habitat heterogeneity for different life stages of aquatic organisms. Stream reaches immediately downstream of a dam or other obstruction become starved of sediment which can lead to stream bank erosion or channel incision. In coastal areas, disruption of sediment transport by dams can contribute to the loss of shoreline habitats because of reduced sediment deposition in those areas.

On the other hand, elevated soil erosion and sediment production due to human activities and extreme natural events can impair surface waters. The U.S. Environmental Protection Agency 2016 National Summary of Impaired Waters and TMDL Information, lists sediment as the sixth most frequent cause of impacted waters for Section 303(d) listed waters. The causes of sediment impairment in order of frequency (high to low) are: siltation (62% of sediment-related impairment), a combination of sedimentation and siltation (33% of sediment-related impairment), and for the remaining 5%, sediment, sedimentation, solids (suspended and bedload) fine sediment, bottom deposits, and particle distribution (embeddedness). Sediment is not inherently a pollutant, but extremes in sediment concentrations, whether too low or too high, lead to less desirable environmental outcomes. In this sense, natural levels of sediment discharge from dams could therefore be a desired environmental outcome. At the same time, because sediment sources can be contaminated, the quality and level of contamination of the sediment must be evaluated whenever a management intervention is considered.

Some watersheds have been disturbed by human activities leading to increased sediment loading to rivers and streams. In these situations the reservoir may have been a source of "free" water quality treatment for decades, i.e. the sediment trapping in the reservoir may help offset increases in sediment loading by other human activities. Unless sediment trapping for downstream water clarity is a specifically-authorized purpose for the project, the reservoir should not be expected to sacrifice long-term sustainability of the actual authorized purposes for temporary water clarity improvements downstream. Rather, the reservoir should be allowed and expected to pass sediment downstream at the annual rate it enters the reservoir under appropriate circumstances.

Recommendation 1 - Recognize the Downstream Channel as a Preferred Beneficial Use

Large watersheds with many dams have a documented history of wide-spread impacts from sediment starvation. Such is the case on the Missouri River (NRC 2011), the Colorado River (Ward et al. 2016), the Kansas River (Shelley et al. 2016), and the Mississippi River (Kondolf et al. 2014; Kesel 2003, 1989, 1988). On the flip side, restoring natural sediment loads to a river system may result in an ecological uplift (Martin et al. 2017, Sumi et al. 2012) or negative impacts if not managed carefully (Espa et al. 2019).

The Corps encourages the beneficial use of dredged sediment where possible, and districts are "encouraged to consider options that provide opportunities for aquatic ecosystem restoration" in ER 1105-2-100 (USACE 2000). The most straight-forward beneficial use for the sediment is to replenish the sediment deficit in the downstream channel. Section 1179(a) of the Water Resources Development Act of 2016, as amended, states that sediment management plans for the Missouri River Basin reservoirs, for example, should "identify beneficial uses for sediment, including discharging to the downstream channel."

The EAB recommends that the downstream channel system and receiving coastal systems be explicitly recognized and evaluated as beneficial uses for the reservoir material. In most cases, discharging sediment to the downstream channel will be the least expensive option as well.

The following principles should be considered when making sediment releases to maximize the environmental benefits and minimize or prevent negative environmental impacts:

- Match the timing (seasonality, discharge level, duration) of sediment passage measures with pre-damming natural sediment regimes as much as possible. Analyze the modern reservoir release and downstream ecosystem sediment regime to identify deviations from the natural sediment regime.
- Sediment releases/bypassing should not have long-term negative impacts on the native ecosystem downstream.
- Do not release sediments with appreciably elevated levels of contamination relative to the sediments downstream. (In these cases, the federal interest will likely favor continued sediment trapping.)
- Attempt to match the concentration of sediment discharges to more natural conditions to benefit the ecological needs of the system.
- The most downstream reservoir in a closely-spaced series can satisfy the sediment deficit in the downstream river by accounting for the trapping by proximal upstream reservoirs.

Recommendation 2 - Expand the Footprint for Reservoir Sedimentation Analyses

Decades of experience have demonstrated that the effects of sediment trapping in reservoirs can extend far upstream and downstream from the original project boundary. Sediment tends to deposit near the confluence of the reservoir multi-purpose pool level and the upstream channel, which forms the familiar reservoir delta. Delta progression proceeds both upstream (new Harrison, 1983) and into the reservoir and effects can impact upstream land use and infrastructure, and have within-reservoir environmental and recreational impacts in addition to degrading primary project purposes. Increased sediment may also increase invasive plants issues in turn increasing project O&M and exacerbating issues related to the delta formation as vegetation slows waters and traps sediment. These deltas can have negative impacts on recreation fishery recruitment and in turn reduce recreational use as the fishery declines. Increased sediment storage often facilitates increased nutrient levels and may encourage harmful algal blooms (new Utah Division of Water Resources, 2010).

Reservoir sediment trapping of sediments can cause sediment starvation for miles downstream (new National Research Council, 2011). Trapping of sediments has led to bed degradation with associated damage to bridge piers, levee toes, water intake structures, and other river-side infrastructure. In addition, a lack of coarse sediments degrades spawning habitats and sand-bar habitats for birds. Trapping fine sediments causes unnaturally clear water releases downstream. Lack of turbidity allows non-native fish to outcompete and prey upon native species and leads to wetland loss (new Kondolf et al. 2014).

The EAB recommends that as sediment management alternatives are evaluated, the footprint for analysis be expanded to include the damages (and prevented damages) upstream and downstream in addition to the lost benefits in the reservoir itself. A comprehensive accounting for the real impacts of reservoir sedimentation will allow for better decision making and more accurate placement of reservoir sediment management among other federal priorities.

Recommendation 3 - Analyze Storage Lost to Sedimentation as a Reallocation

Without intentional action, reservoir pools will by default be reallocated to sediment storage over time. Formal pool reallocation between other authorized purposes are executed only after a pool reallocation study. We recommend that a similar study be done for Corps reservoirs to assess the economic and environmental effects of reallocating storage away from existing authorized purposes to sediment. The economic and environmental benefits that will be lost should be quantified so that either (1) sediment sustainability actions can be justified and implemented, or (2) a plan to optimally redistribute remaining benefits until a final project decommissioning is implemented.

In addition to the value of lost benefits, the economic and environmental consequences of reservoir sediment trapping should include the increase in O&M costs related to acute sediment deposition, the costs of downstream sediment starvation, the costs of upstream delta progression, and the costs of project decommissioning.

The EAB recommends that such a “reallocation” style assessment be performed at Corps reservoirs. Where indicated to be in the federal interest by the results of this sustainability/ reallocation study, maintenance for the lake project should include maintenance of the reservoir pools with appropriate cost sharing of maintenance costs.

Cases may exist where continued sediment trapping with associated decay of storage capacity is deemed as the best option. Reservoirs trapping contaminated sediments or reducing dredging needs in downstream navigation channels may fall into this category. The full costs of continued sediment trapping must be included in such a determination, including the increased O&M the project will see under continued sedimentation, upstream and downstream ecological and geomorphological effects of the sediment trapping, and the cost to eventually decommission the project, as well as the loss in benefits from authorized purposes. In these cases, a reallocation schedule should be created to optimally redistribute remaining storage amongst the authorized purposes.

Recommendation 4 - Implement Reservoir Sediment Sustainability Pilot Projects

International examples attest that reservoirs can pass sediments downstream through a variety of means such as drawdown flushes, hydrosuction, dredging with downstream discharge, bypass tunnels, and turbidity current venting (Morris and Fan 1998). Unfortunately, very few reservoirs in the United States have adopted these management strategies. This lack of adoption reinforces the current misconception that dams must trap sediment and cannot release it downstream.

In December 2014, the multi-agency Advisory Committee and Water Information (ACWI) passed a resolution on reservoir sediment management encouraging “all Federal agencies to develop long-term reservoir sediment-management plans for the reservoirs that they own or manage by 2030.” (ACWI 2014). ACWI also recommends starting with one or two such plans per year in order to work through the methodologies and inform the larger effort. The EAB supports this recommendation by ACWI and recommends that the Corps implement pilot projects which demonstrate sediment management strategies that pass sediment downstream. These projects could be implemented with federal funding through various authorities such as Section 204 or 1146 on a short-term basis. Following a fully federal pilot phase and the creation of a reservoir sustainability plan, long-term implementation could be accomplished with appropriate cost-sharing by non-federal sponsors.

As a practical matter, the Corps may want to perform a high-level initial screening and prioritization. We suggest that such a prioritization include an estimation for upstream and downstream impacts on ecosystems and infrastructure, with an initial focus on known existing impacts to priority species and ecosystems (e.g. T&E species and rapidly degrading receiving coastal wetland systems). We advise against a prioritization based solely on percent loss in reservoir storage capacity, which could be misleading as to where the needs and opportunities are greatest.

Recommendation 5 - Hold Reoccurring Reservoir Sediment Management Training

Currently, no regularly offered Corps training classes cover reservoir sediment management. As a result, many regulators are unsure how to permit these actions and engineers are unsure how to design cost-effective, environmentally-acceptable solutions.

In 2017 the Regional Sediment Management (RSM) program hosted a three-day training workshop titled: “Reservoir Sediment Management Workshop for Regulators, Planners, and Managers” (Shelley et al. 2018). In 2018, RSM hosted a five-day training workshop titled: “Reservoir Sediment Management and Analysis for Engineers Workshop” (Shelley et al. 2019). The EAB recommends that both of these workshops be repeated on a regular basis (perhaps alternating years).

Conclusion

Reservoir sediment trapping has led to undesirable geomorphological and ecological conditions in river and coastal systems and has negatively impacted river-related infrastructure. Downstream river corridors and receiving coastal ecosystems are being deprived of the sediments essential to proper ecosystem functioning and shoreline defense, upstream river corridors are experiencing aggradation, groundwater rise, and flooding, and the reservoirs themselves are losing water storage capacity.

The Environmental Advisory Board recommends the Corps take the following steps:

1. Expand the footprint for assessing cost-benefits of reservoir sediment management to include both downstream and upstream river corridors.

2. Analyze storage lost to sedimentation as a reallocation, with an assessment of lost benefits and associated increased costs both upstream and downstream of the project footprint.

3. Implement reservoir sediment sustainability pilot projects that demonstrate different management options.

4. Recognize the downstream river and coastal systems as preferred beneficial uses for reservoir sediments, subject to the principles discussed in this document.

5. Hold reoccurring reservoir sediment management training courses.

Taking these steps will encourage the adoption of cost-effective, environmentally-sound reservoir sediment management practices that will improve the ecosystems upstream and downstream of reservoirs and facilitate long-term reservoir sustainability.

References

Advisory Committee on Water Information (ACWI). 2014. https://acwi.gov/sos/index.html

Espa, Paolo, Ramon J. Batalla, Maria Laura Brignoli, Giuseppe Crosa, Gaetano Gentili, Silvia Quadroni. 2019. Tackling reservoir siltation by controlled sediment flushing: impact on downstream fauna and related management issues. PLoS ONE 14(6): e0218822.

George, Matthew W., Rollin H. Hotchkiss, and Ray Huffaker. 2016. Reservoir Sustainability and Sediment Management. Journal of Water Resources Planning and Management, DOI: 10.1061/(ASCE)WR.1943-5452.0000720.

Harrison, Alfred S. 1983. Deposition at the heads of reservoirs. U.S. Army Corps of Engineers Missouri River Division (MRD) Sediment Series No. 31, December. Originally part of Proceedings of the Fifth Hydraulics Conference, pp 199-225, Iowa City, Iowa, June 9-11, 1952.

Kesel, R.H. (1988). “The Decline In The Suspended-Load Of The Lower Mississippi River And Its Influence On Adjacent Wetlands.” Environmental Geology And Water Sciences, Springer Verlag, 11(3), 271-281.

Kesel, R.H. (1989). “The Role Of The Mississippi River In Wetland Loss In Southeastern Louisiana, USA.” Environmental Geology And Water Sciences, Springer Verlag, 13(3), 183-193.

Kesel, R.H. (2003). “Human modifications to the sediment regime of the Lower Mississippi River flood plain.” Geomorphology, Elsevier Science BV, 56(3-4), 325-334.

Kondolf, G. Mathias, Yongxuan Gao, George W. Annandale, Gregory L. Morris, Enhui Jiang, Junhua Zhang, Yongtao Cao, Paul Carling, Kaidao Fu, Qingchao Guo, Rollin Hotchkiss, Christopher Peteuil, Tetsuya Sumi, Hsiao-Wen Wang, Zhongmei Wang, Zhilin Wei, Baosheng Wu, Caiping Wu, Chih and Ted Yang. 2014. Sustainable Sediment Management in Reservoirs and Regulated Rivers: Experiences from Five Continents. American Geophysical Union Earth’s Future, 2, pp. 256 – 280. doi: 10.1002/2013EF000184.

Martin, E. J., Doering, M., and Robinson, C.T. 2017. Ecological Assessment of Sediment Bypass Tunnel on Receiving Stream in Switzerland. River Res. Applic. 33: 925–936.

National Research Council. 2011. Missouri River Planning: Recognizing and Incorporating Sediment Management. Washington, DC: The National Academies Press.

Morris, G. L., and Fan, J. (1998). Reservoir sedimentation handbook, McGraw-Hill, New York.

Randle, T, G. Morris, M. Whelan, B. Baker, G. Annandale, R. Hotchkiss, P. Boyd, J. T. 10 Minear, S. Ekren, K. Collins, M. Altinakar, J. Fripp, M. Jonas, K. Juracek, S. Kimbrel, M. Kondolf, D. Raitt, F. Weirich, D. Eidson, J. Shelley, R. Vermeeren, D. Wegner, P. Nelson, K. Jensen, D. Tullos. 2019. “Reservoir Sediment Management: Building a Legacy of Sustainable Water Storage Reservoirs.”

Shelley, J., M. Boyer, J. Granet, and A. Williams. 2016. Environmental benefits of restoring sediment continuity to the Kansas River. ERDC/CHL CHETN-XIV-50. Vicksburg, MS: U.S. Army Engineer Research and Development Center.

Shelley, J., P. Boyd, T. Dahl, I. Floyd, and M. Ramos-Villanueva. 2018. Reservoir Sediment Management Workshop for Regulators, Planners, and Managers. ERDC/TN RSM-18-7. Vicksburg, MS: U.S. Army Engineer Research and Development Center.

Shelley, J., P. Boyd, S. Gibson, I. Floyd, and M. Ramos-Villanueva, and T. Dahl. 2019. Reservoir Sediment Management and Analysis Workshop for Engineers. Tech note under review.

Sumi, Tetsuya, Sameh A. Kantoush, and Shoji Suzuki. 2012. Performance of Miwa dam sediment bypass tunnel: evaluation of upstream and downstream state and bypassing efficiency. In Proceedings, 24th Congress of the International Commission on Large Dams, pp. 576-596, Kyoto, Japan.

US Army Corps of Engineers (USACE). 2018. Regulatory Guidance Letter No. 18-01. Determination of Compensatory Mitigation Credits for the Removal of Obsolete Dams and Other Structures from Rivers and Streams.

USACE. 2000. Engineering Regulation (ER) 1105-2-100, as amended. Planning Guidance Notebook.

Utah Division of Water Resources. 2010. Managing sediment in Utah's reservoirs.

Ward D.L., Morton-Starner R., Vaage B. 2016. Effects of turbidity on predation vulnerability of juvenile humpback chub to rainbow trout and brown trout. Journal of Fish and Wildlife Management 7(1):205-212; e1944-687X. doi: 10.3996/102015-JFWM-101.