LOWERLOUP
NATURAL RESOURCES DISTRICT

Final Report

Lillian Creek Dam

Feasibility Study

Lower Loup Natural Resources District

June 2018

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06/27/18

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Executive Summary

A feasibility study on Lillian Creek in north-central Custer County, Nebraska, was performed for the Lower Loup Natural Resources District (LLNRD). The primary objective of the study was to determine the feasibility and potential conceptual designs for an earthen dam along Lillian Creek (Project). The Study Area is shown in Figure 1. Topographical and soil property data was collected within the Lillian Creek basin to inform dam evaluations and the potential for water retention. In addition, water availability, geotechnical considerations, water right and land acquisition, and potential funding options were evaluated for a future dam structure located along Lillian Creek. Benefits of this future dam structure include capturing excess flows to provide water to current and future downstream users, increasing baseflow through groundwater recharge or reservoir releases, providing mitigation for groundwater depletions, and providing additional recreation opportunities to the area.

Water Supply: Considering downstream surface water appropriations (or demands), water supply from the watershed would not be sufficient to fill the reservoir, thus necessitating the need to capture flow from sources outside of the watershed. There is sufficient water available from the Middle Loup River to fill the reservoir to a normal pool elevation of 2,460 feet. A pump station would be required to convey the water to the proposed dam.

Dam Embankment: Slope stability, seepage, and settlement analyses were performed to evaluate the dam embankment. The subsurface condition is primarily loess/eolian deposits, which are generally soft, fine grained materials that are highly pervious and are susceptible to collapse. The primary geotechnical concerns include dam stability, subsidence of the abutments upon reservoir filling, and seepage and piping. Several measures have been identified for evaluation as potential mitigation measures to address these concerns. These mitigation measures include pre-wetting of the in situ material or removal and replacement of the in situ material, constructing stability berms, staging the construction activities, and constructing an internal drainage system including trench drains and relief wells.

These mitigation measures have been employed to construct other dams in the Project area with similar subsurface conditions. It is possible that through implementation of the mitigation measures, or a combination thereof, that a stable embankment can be constructed at the Project location; however, long-term maintenance and post construction mitigation may also be required. Given the nature of the existing conditions, additional geotechnical testing would be necessary to evaluate and design the dam embankment and mitigation measures. Geotechnical expertise in design and construction of high head dams in pervious and collapse-susceptible soils will be required. Additional geotechnical testing and evaluation would be necessary to evaluate and design the dam embankment and mitigation measures.

Reservoir Operation: The geotechnical investigation indicates the potential for significant reservoir seepage (leakage) above approximate elevation 2,448 feet, relative to the upstream toe of the dam. The seepage estimates determined from the collected geotechnical data are greater by an order of magnitude than the historic seepage observed at Sherman Reservoir and Davis Creek Reservoir. It is likely that the seepage rate at Lillian Creek Reservoir would trend toward the observed values at Sherman Creek; however, additional geotechnical testing and evaluation should be done to confirm this. Preliminary analysis indicates that under certain flow

demand scenarios and using the historic observed seepage rates that the Lillian Creek Reservoir elevation would fluctuate between normal pool elevation and elevation 2,448 feet.

Federal Regulatory Permitting: A wetland and other waters of the United States determination was performed for the Project. Potential jurisdictional waters of the United States were identified and would likely be affected by the proposed Project. Therefore, a Section 404 of the Clean Water Act (CWA) Permit would be needed. The Project would require an Individual Permit issued from the United States Army Corps of Engineers (USACE). USACE is responsible for compliance with Section 7 of the Endangered Species Act and Section 106 of the Historic Preservation Act. USACE often requests assistance from the applicant to assist in demonstrating compliance. This would entail coordination with the United States Fish and Wildlife Service (USFWS) and the Nebraska State Historic Preservation Office (NESHPO), respectively. It is not anticipated that listed threatened and endangered species would be adversely affected. Therefore formal consultation with USFWS will not likely be required. However, it is expected that NESHPO would recommend on-site surveys for cultural resources. Given the Project site location, there is a potential to discover archaeological resources that would need to be evaluated for eligibility for listing on the National Register of Historic Places. If potentially eligible sites are found, additional survey would likely be needed. Further, USACE would likely coordinate with Native American Tribes to ensure that Section 106 of the Historic Preservation Act has been complied with from a tribal perspective.

State Regulatory Permitting: A Section 401 Water Quality Certification would be required from the Nebraska Department of Environmental Quality. The request for this certification would be done concurrently with the Individual Permit for Section 404 of the CWA compliance. In addition, an Application for Approval of Plans for Dams would be required from the Dam Safety Section of the Nebraska Department of Natural Resources (NeDNR).

Water Appropriation: A dam located on Lillian Creek would require a permit to store and use water from the facility as required by NeDNR. The existing water supply of the Lower Platte Basin would suggest that application for diversion and storage of water out of the Middle Loup River and the Lillian Creek are feasible. The application process to divert, store, and use water from these sources would be through NeDNR and while there is no guarantee that they would be granted, the Project concept does not present any obvious impediments.

Economics: A benefit cost analysis was performed for several water demand scenarios, seepage rates, and operating conditions. The two main benefits of the Project are irrigation and recreation, with increased irrigation being the largest contributor to Project benefits. The largest cost components of the Project are the dam embankment materials, seepage and stability mitigation, and the pump station construction. Assuming the historic seepage rate can be achieved, operating the reservoir to maintain a pool sufficient for recreation, while releasing a portion of the flow for irrigation, results in an economically viable alternative. Operating the reservoir to only maximize recreation is not an economically feasible alternative.

Recommendations: The biggest unknowns remaining after the feasibility level study are the design constraints relative to mitigation for embankment stability, and seepage rate for reservoir management. Additional geotechnical evaluation is recommended to inform seepage mitigation for dam embankment stability and reservoir management, and reduce uncertainty associated with the development of Project benefits.

1.0 Background Information

Lillian Creek is a small stream that flows north through north-central Custer County before it becomes a tributary of the Middle Loup River west of the town of Sargent, Nebraska (Figure 1). The drainage is under the Hydrology Unit 12 (HU-12) name of Lillian Creek numbered 102100030401, a subsidiary of the HU-10, Sand Creek-Middle Loup River. The drainage starts northeast of the town of Merna, Nebraska, and drains in a northeasterly direction, encompassing nearly 60 square miles of grassland, primarily used for pasture and grazing. There are 52 certified fields in the basin, totaling approximately 6,835 certified irrigated acres. Lillian Creek has limited stream flow measurements. The only accessible stream flow readings available to LLNRD are the six readings conducted by the United States Geological Survey (USGS) throughout the basin. These flow readings were sampled in 2006 as part of the development of the Elkhorn-Loup modeling project. There are 88 registered wells within the basin, with an average static water level of 139 feet below ground surface. Soil composition is primarily silty loams to loamy sands and the topography throughout the basin is considered rough. Light Detection and Ranging (LiDAR) was obtained by LLNRD in December 2016 and covers the valley section of the Lillian Creek upstream of the approximate centerline of the proposed earthen dam (that is, the Project). A digital elevation model (DEM) in the form of 10-meter statewide data was available from NeDNR, which was outside of the limit of the LiDAR data.

In 1959, a canal was approved as part of the Milburn Diversion Dam/Sargent Canal project that extended from Milburn Diversion Dam, southeasterly 15.5 miles to Lillian Creek (Figure 2). For purposes of this feasibility study, the canal will be referred to as the Lillian Creek Canal or Lillian Canal. This authorized canal was designed as part of the United States Bureau of Reclamation (USBR), Missouri River Basin Project; along with the Middle Loup Division, classified as the Sargent Unit. Though this canal was never developed, the authorized canal, titled Lillian Canal is still viewed as a potential water source for the area. At the time of development, the Lillian Canal and lateral system were deferred features of the Sargent Unit project. During the research phase of development, the Lillian Canal diversion or delivery was expected to average about 9,000 acre-feet per year.

Land rights throughout the majority of the stream basin corridor are encompassed by two landowners. The owner with the larger amount of land of these two landowners and the likely site of the Project has already signed a 5-year temporary access easement with LLNRD to allow access to conduct the on-site elements of this feasibility study. LLNRD is working with the other landowner to obtain a similar access agreement prior to the progression of the on-site research.

LLNRD's *Master Plan 2012–2022* states that, "the purposes of a natural resources district shall be to develop and execute…plans, facilities, works, and programs relating to:"

- Flood prevention and control
- Water supply for beneficial uses
- Conservation of groundwater and surface water
- Development of recreational facilities
- Development of fish and wildlife habitat

Each of these purposes of the LLNRD could be fully realized with the development of this future dam structure (that is, the Project). Researching these elements fulfills LLNRD's duties to its constituent population, and potentially, to additional downstream populations in the Loup Basin. The purposes of LLNRD are priorities for developing research of this nature. The LLNRD Board of Directors has prioritized developing research and development of large-scale water projects, and it is imperative that a thorough investigation of Lillian Creek be conducted to satisfy this objective.

2.0 Data Collection

A topographic survey and geotechnical investigation was conducted, and environmental data was collected, at the outset of the project to inform the feasibility study and dam design consideration.

2.1 Topographic Survey

A topographic survey was conducted on April 17, 2017. Four cross sections were obtained: 1) the approximate dam centerline; 2) the approximate downstream toe; 3) the approximate upstream toe; and 4) approximately 250 feet upstream of the upstream toe. In addition, elevations along the flow line were surveyed for approximately 3,000 feet upstream of the approximate dam centerline. Finally, boring locations for the geotechnical analysis were surveyed. The cross section locations, stream flowline extent, and boring locations, are shown in Figure 3. The raw survey data was provided under separate cover.

2.2 Subsurface Investigation

The field work for the Project consisted of drilling five exploratory test borings along the proposed dam alignment and five exploratory borings upstream of the proposed dam alignment in possible borrow areas. The boring locations are shown in Figure 3. The boring depths along the proposed dam alignment ranged from 50 to 100 feet below existing grade. The boring depths in the possible borrow areas ranged from 10 to 25 feet below existing grades. The schedule of borings/groundwater data and the boring logs, as well as additional information on equipment and sampling techniques are provided in Appendix A.

2.2.1 Subsurface Condition

Based on the subsurface investigations, subsurface profile in the uplands generally consist of deep deposits of silty sand (SM) to sandy silt (ML) loess/eolian deposits; and subsurface profile in the valley generally consists of fine-grained alluvial deposits overlying coarse-grained alluvial deposits. Both the uplands and the valley soils were underlain by dense sandy silt (ML) and silty sand (SM) alluvial deposits. A subsurface profile along the proposed dam centerline is provided as Figure 4. Bedrock was not encountered in any of the borings. Bedrock of the Ogallala Group consisting of sandstones and siltstones was encountered at a depth of about 185 feet below the Middle Loup River valley based on a nearby water well drilling log provided by LLNRD. More detailed information on the stratigraphy is provided in Appendix A.

Groundwater was encountered at the depths and times noted on the boring logs. A summary of recorded groundwater data at the boring locations at the time of our investigation is provided in

Appendix A. Groundwater was encountered at the approximate centerline of the proposed dam in the valley at approximate elevations 2,416 feet and 2,417 feet at the time of the field investigation during March 2017. These groundwater elevations correspond to depths of 14.5 to 27.5 feet below existing ground, respectively.

Upstream of the proposed dam, groundwater was encountered between elevations 2,419.5 feet and 2,431.2 feet at the time of the field investigation during March 2017. These groundwater elevations correspond to depths of 9 and 10 feet below existing ground, respectively. Fluctuations in the level of the groundwater may occur due to seasonal variations in local and regional precipitation and other factors not evident at the time of measurement.

2.2.2 Geologic Investigation

The Project site is located within the dissected plains of central Nebraska, south of the Middle Loup River. This area is steeply rolling and hilly with irregularly shaped tabular remnants left to mark the level of the former plain (Elder 1969). Trees are located within the valley and some of the drainage ways. The loess plains have been dissected locally by the Lillian Creek valley where the site is located. Alluvial deposits are present within the Lillian Creek valley and are formed by deposition in flowing water. The loess deposits in the uplands are wind-blown (eolian) deposits. The alluvial deposits in the valley and loess deposits in the uplands extend down to an older alluvial deposit.

2.2.2.1 Seismic Assessment

According to the Seismic Zone Map (Figure 4-1 in Natural Resources Conservation Service [NRCS] Technical Release Number 60 [TR-60]), the Project site is located near Seismic Zone 1, which corresponds to a low seismic exposure. This designation indicates that the Project would not require special investigations to assess the potential for liquefaction or faulting at the site. Based on Figure 4-1 in NRCS TR-60, the corresponding seismic coefficient for the site is 0.05g.

2.2.2.2 Subsidence

The geologic area is not known to have been mined or contain any karstic areas. The collapse potential of the loess/eolian deposits was evaluated using the criteria developed by USBR (1987) that is based on dry densities and liquid limits. The results of this evaluation indicate that the loess/eolian deposits at the site do exhibit the potential for collapse upon wetting.

The Sherman Dam, located about 45 miles southeast of this Project site, required mitigation for collapse potential of the loess/eolian deposit located along the dam abutments. The mitigation consisted of irrigating the ground for about 11 weeks to saturate the foundation prior to construction of the embankment (USBR 1996). There is no information on the effectiveness or performance of this mitigation measure.

Mitigation for the potential collapse at the abutments would likely be necessary. Possible mitigation measures would include over excavation and re-compaction of the collapse susceptible soils or pre-wetting of the collapse susceptible soils similar to what was completed at Sherman Dam.

2.2.2.3 Dispersive Soils

The Project site is not known to have dispersive soils.

2.2.2.4 Mass Movements

The existing uplands display no observable indications of shallow or deep-seated slope movements in the Study Area. In general, natural slopes of loess/eolian deposits have eroded over geologic time to a stable inclination and do not exhibit a potential for mass movements. Steep soil slopes that are wetted by the reservoir, would likely become unstable and cave.

2.2.2.5 Leakage

The uplands soils in the area of the dam abutments consist of a combination of silty sand and sandy silt, are considered to be well-drained, and would likely be prone to significant reservoir leakage.

Constant head permeability tests were performed on remolded samples of the upland and valley soil deposits. The B-104 sample was compacted in the laboratory to near in situ dry density and moisture content. The B-401, B-402, and B-404 samples (see Figure 3 for location) were compacted in the laboratory to about 95 percent of the maximum dry density and a moisture content between 2 percent below and 2 percent above optimum moisture content as determined by the standard Proctor test (ASTM D698). Table 1 provides the results of the tests.

Table 1. Constant Head Permeability Laboratory Test Results

Boring Number	Depth (feet)	Soil Type	Location	Remolded Dry Density (pcf)	Remolded Moisture Content (%)	Permeability (feet/sec)
B-104	2 to 7	Sandy Silt	Uplands	85	10	2×10^{-6}
B-401	2 to 7	Silty Sand	Uplands	106	11	2×10^{-5}
B-402	2 to 7	Lean Clay	Valley	95	19	1×10^{-7}
B-404	2 to 7	Lean Clay	Valley	94	21	1×10^{-7}

2.3 Environmental Data

A wetland determination, desk top evaluation for threatened and endangered species, and cultural resources critical issues analysis was performed to evaluate possible design constraints associated with the Project. Environmental data was collected within the Project Area (Figure 1), which is the area within the proposed maximum pool of the dam (Figure 5) and along the approximate canal alignment (Figure 6).

2.3.1 Wetland Determination

Wetland and other waters of the United States determinations were performed within the Project Area. The wetland determinations involved a two-step approach. In the first step, a desktop survey was completed to identify areas that have been previously mapped by the National Wetland Inventory (NWI) and areas that display wetland signatures (such as bright green vegetation and areas of visible saturation or inundation) on aerial photography. Streams, ditches, and other waterways were identified from aerial photography and the National Hydrography Dataset (NHD).

The second step was the completion of field wetland determinations. Field wetland determinations were performed on May 8, 2017. No soil samples were collected as part of the field wetland determinations; however, site photographs were taken and are located in Appendix B, Attachment 1. Any area identified with more than 50 percent aerial coverage of hydrophytic vegetation (based on the 2016 National Wetland Plant List [Great Plains Region] [Lichvar et al. 2016]) and adequate surface hydrology was determined to be wetland. Determined wetlands were classified using the Cowardin wetland classification system (Cowardin et al. 1979). Stream channels were identified and documented. Stream channels are defined as having an identifiable bed and bank and ordinary high water mark.

The field wetland determination identified 90 individual areas that had greater than 50 percent aerial coverage of hydrophytic vegetation and apparent surface hydrology within the Study Area. The wetland locations within the Project Area are shown in Appendix B, Attachment 2 and Attachment 3.

Table 2 provides an overview of the wetlands identified in the Project Area and lists the wetlands by Cowardin wetland classification and total area by location (dam or canal).

Table 2. Determined Wetlands

Cowardin Wetland Type[1]	Area within Dam Project Area (acres)	Area within Canal Project Area (acres)	Total Area (acres)
PEMA/PEMC	30.89	113.99	144.88
PEMF	0.80	1.48	2.28
PFOA	0.00	23.68	23.68
PSSA	0.00	3.85	3.85
PUB	2.58	10.14	12.72
Total Area (acre)	34.27	153.14	187.41

[1] PEMA = Palustrine Emergent Temporarily Flooded; PEMC = Palustrine Emergent Seasonally Flooded; PEMF = Palustrine Emergent Semi-permanently Flooded; PFOA = Palustrine Forested Temporarily Flooded; PSSA = Palustrine Scrub-Shrub Temporarily Flooded; PUB = Palustrine Unconsolidated Bottom

No stream channels with a definable bed and bank and observable high water mark were observed in the Project Area. No tributaries of Lillian Creek identified from the NHD were discernable during the field survey. Lillian Creek's former alignment was only discernable in some areas. These areas consisted of either wholly vegetated wetland swales, or standing surface water up to depths of approximately 3 feet and were reminiscent of oxbow depressions.

Five stream channels were observed in the Project Area, including the Middle Loup River. Table 3 provides an overview of the streams identified in the Project Area and lists them by flow regime.

Table 3. Identified Stream Channels

Stream ID	Flow Regime[1]	Linear Feet in Project Area	Name
WUS-1	Perennial	8,549	Victoria Creek
WUS-2	Intermittent	6,820	N/A
WUS-3	Intermittent	308	N/A
WUS-4	Ephemeral[2]	1,143	N/A
WUS-5	Perennial	5,863	Middle Loup River

1 The NHD was referenced to obtain the stream flow regime
2 WUS-4 is an artificially constructed discharge channel of the Milburn Diversion Dam

2.3.2 Threatened and Endangered Species

The potential effects of the Project on federally and state-listed threatened and endangered (T&E) species, migratory birds, and bald eagles were evaluated. Because the proposed dam is located in Custer County and a potential canal is located in both Blaine and Custer Counties, Nebraska, both counties were evaluated.

Species lists from Blaine and Custer Counties were obtained from USFWS (May 2015) and NGPC (January 2017) and range maps for each species were obtained from NGPC (April 2017). Of the seven species listed in Blaine County and eleven species listed in Custer County, seven have ranges that include the Project Area (see Table 4). No designated critical habitat for any of the species listed in Table 4 occurs in Blaine or Custer County. Aerial photography on Google Earth™ mapping service was reviewed for the presence of habitat.

Table 4. Threatened or Endangered Species Listed in Blaine and Custer Counties, Nebraska

Common Name[1]	Scientific Name	Federal and/or State Listing[2]	Habitat	Project in Range
American burying beetle	*Nicrophorus americanus*	FE, SE	Grasslands and open understory oak hickory forests with carrion availability	Yes
Blowout penstemon	*Penstemon haydenii*	FE, SE	Open sand and wind excavated depressions on sand dune tops	Yes
Finescale dace	*Phoxinus neogaeus*	ST	Small, slow-moving steams with clear water and sand or gravel bottoms	No
Interior least tern[3]	*Sternula antillarum athalassos*	FE, SE	Barren to sparsely vegetated sandbars along rivers, sand and gravel pits, lake and reservoir shorelines	Yes
Northern long-eared bat	*Myotis septentrionalis*	FT, ST	Winter – hibernate in caves and mines; Summer – roost and forage in woodlands; Autumn – swarm in surrounding woodlands near caves and mines	Yes

Common Name[1]	Scientific Name	Federal and/or State Listing[2]	Habitat	Project in Range
Northern redbelly dace	*Phoxinus eos*	ST	Small, slow-moving steams with clear water and sand or gravel bottoms	No
Piping plover[3]	*Charadrius melodus*	FT, ST	Barren to sparsely vegetated sandbars along rivers, sand and gravel pits, lake and reservoir shorelines	Yes
River otter[3]	*Charadrius melodus*	FT, ST	Lakes, rivers, inland wetlands	No
Small white lady's slipper[3]	*Cypripedium candidum*	ST	Wet prairies, sedge meadow	No
Western prairie fringed orchid	*Platanthera praeclara*	FT, ST	Wet prairies and sedge meadows	Yes
Whooping crane	*Grus americana*	FE, SE	Large silty rivers with a diversity of depths and velocities formed by braided channels, sand bars, sand flats and gravel bars	Yes

Sources: Nebraska Game and Parks Commission 2015; United States Fish and Wildlife Service 2015 and 2016, IPaC 2017

1 Pallid sturgeon is listed on the IPaC resource exploration of the Study Area. Because the pallid sturgeon is not listed on the more current Nebraska Game and Parks Commission or the United States Fish and Wildlife Service current species range county lists, it has been omitted from the table.

2 FE – federally endangered, FT – federally threatened, SE – state endangered, ST – state threatened

3 Interior least tern, piping plover, river otter, and small white lady's slipper are listed in Custer County only; all other species are listed in both Blaine and Custer Counties.

A survey of the Project Area (Figures 1, 5, and 6) was conducted on May 8, 2017. The canal begins at the Middle Loup River northwest of Milburn, Nebraska, extends south of the river. The land surrounding the proposed reservoir and canal is predominantly used for row crop agriculture or grazing. The results of the field survey are as follows:

- **American Burying Beetle**
  American burying beetle is a habitat generalist that can successfully survive in a wide array of habitats. The beetles have a slight preference for grasslands and open understory oak hickory forests with the potential for carrion availability. Because the canal is located within the range of American burying beetle and there are pasture lands present, there is potential for the beetle to occur within the Project Area.
- **Blowout Penstemon and Western Prairie Fringed Orchid**
  Blowout penstemon is found only in open sand habitats where wind has excavated depressions on dune tops, often on northwestern exposures. Based on the desktop survey and site visit, there are no open sand areas located within the Project Area. Western prairie fringed orchid is found in wet or sedge meadows. Based on the May 8, 2017, site visit, there are no wet or sedge meadows in the Project Area.

- **Interior Least Tern and Piping Plover**
  Interior least terns and piping plovers prefer sparsely vegetated shorelines of shallow water bodies and river channels. Shorelines with bare sand, and sandy or pebbly mud are preferred. Terns and plovers use open sand for feeding and nesting. Based on the desktop survey and site visit, there are no mudflats or sandbars within the Project Area.
- **Northern Long-Eared Bat**
  Suitable summer habitat for northern long-eared bats consists of trees possessing exfoliating bark, cracks, crevices, and/or hollows. There is potential habitat for northern long-eared bats in the trees located within the Project Area. To prevent adverse effects on northern long-eared bats, trees would not be removed during pup season from June 1 to July 31.
- **Whooping Crane**
  The Project Area is not located within the designated critical habitat for whooping cranes, but is located in the primary occurrence area. Whooping cranes migrate through the primary occurrence area during two periods: March 6 through April 29 and October 9 through November 15. While migrating through Nebraska, whooping cranes prefer shallow braided riverine habitats with accompanying wetlands, mudflats, wet prairies, and fields for roosting and feeding. Due to the proximity of Middle Loup River and the surrounding agricultural fields, there is potential for whooping crane to occur in the Project Area.

2.3.2.1 Field Habitat Survey Summary and Recommendations

Based on the desktop and field habitat surveys, the Project would have no effect on blowout penstemon, western prairie fringed orchid, interior least tern, or piping plover. Desktop survey and a field habitat survey determined American burying beetle, northern long-eared bat, and whooping crane have the potential to occur in the Project Area.

The Project may affect, but is not likely to adversely affect the American burying beetle. LLNRD would coordinate with USFWS and Nebraska Game and Parks Commission (NGPC) to determine if any mitigation measures should be implemented during construction. Commonly, mitigation requires mowing, vegetation removal, and/or carrion removal.

Due to the potential tree removal, the Project may affect, but is not likely to adversely affect the northern long-eared bat. The LLNRD would remove trees outside of the pup season from June 1 to July 31.

The Project may affect, but is not likely to adversely affect the whooping crane. LLNRD would coordinate with the lead federal agency, USFWS and NGPC to determine if any mitigation measures should be implemented during construction during the migration periods. Commonly, mitigation requires either no construction during the migration periods, or a daily whooping crane survey during the migration periods. Reseeding should contain native grass or forb species and should not reach more than 4 feet in height at maturity.

2.3.2.2 Bald and Golden Eagle Protection Act

Bald eagles and their active and inactive nests are protected under the Bald and Golden Eagle Protection Act (Note: golden eagles are found in the western quarter of the state of Nebraska, but may be observed in central Nebraska during the spring, fall, and winter. Golden eagles nest on cliffs and rock outcroppings, which are not found in or near the Study Area. There was one active bald eagle nest in Blaine County and two active nests in Custer County in 2016 (Jorgensen and Dinan 2017). Six active nests were located along the Middle Loup River, with one nest in proximity to the Project Area.

There is suitable habitat for bald eagle nesting along the Middle Loup River, but the field site visit confirmed there are no active nests or suitable trees within the Project Area. Eagles may fly over or temporarily roost in areas near the Project, but construction activities would not likely disturb roosting or nesting. A bald eagle nest survey should be completed prior to construction to confirm that no nests are present in the Project Area.

2.3.2.3 Migratory Bird Treaty Act

The Migratory Bird Treaty Act (MBTA) protects all birds native to North America, with the exception of non-migratory upland game birds (for example, quail, grouse, pheasant, and turkey); upland game birds are instead protected by state game laws. Introduced species that are not protected by MBTA include house sparrow, European starling, feral rock pigeon, and Eurasian collared doves. LLNRD would remove trees to the extent possible outside of the migratory bird season (April 1 to July 15) and conduct nest surveys if trees are removed within migratory bird season.[1] The Project would not disturb, harm, or harass migratory birds.

2.3.3 Cultural Resources

A cultural resources Critical Issues Analysis (CIA) was completed for the proposed Project, which includes a database search for historic resources as well as recommendations for future Project cultural resource needs. The extent of the CIA Area is shown in Figure 7.

Electronic files (geographic information system [GIS] shapefiles) from NESHPO were requested on April 6, 2017. Research focused on previously identified archaeological sites, architectural properties, and previously conducted archaeological surveys within the CIA Area. In addition to the background research requested from NESHPO, HDR reviewed General Land Office (GLO) maps accessed online through the Nebraska State Surveyors Office at http://www.sso.nebraska.gov/maps/glo.asp.

The purpose of a CIA is to reveal potential cultural issues and resources that may be encountered that could affect the Project. This section presents the results of a cursory desktop survey that was completed using files obtained from NESHPO and select web sources. The section provides a brief high-level overview and is not meant to replace a literature search or field survey.

2.3.3.1 NEBRASKA REGULATORY FRAMEWORK

State laws and regulations that may potentially apply to the Project include: the Nebraska Archaeological Resource Preservation Act (Nebraska Revised Statutes 82-501 through 82-510); the Unmarked Human Burial Sites and Skeletal Remains Protection Act (Statutes of Nebraska

[1] Note: Trees may not be removed from June 1 to July 31 due to the northern long-eared bat pup season.

Article 12-1201 through 12-1212), and the Human Skeletal Remains or Burial Goods Act (Statutes of Nebraska Article 28-1301).

> *Nebraska Archaeological Resource Preservation Act (Nebraska Revised Statutes 82-501 through 82-510)*. The Act establishes that any state agency having jurisdiction over a proposed state or state-funded undertaking, which has potential to affect archaeological resources or sites, shall, prior to the approval of the expenditure of any state funds on the undertaking, notify the State Archaeological Office of the undertaking and cooperate with the office to identify and develop measures to mitigate the effects of the undertaking on any archaeological site or resource that is included in or eligible for inclusion in the National Register for Historic Places.

> *Unmarked Human Burial Sites and Skeletal Remains Protection Act (Statutes of Nebraska Article 12-1201 through 12-1212)*. This Act protects and regulates the excavation of unmarked burial sites and human skeletal remains. If an unmarked grave should be encountered within the project boundary as a part of pre-construction, construction, or operation the guidance supplied in this law should be followed to resolve the resource issue.

> *Human Skeletal Remains or Burial Goods Act (Statutes of Nebraska Article 28-1301)*. This Act establishes that the removal of items from a burial or the willful disturbance of a burial to conceal, purchase, sell, transport, trade, or dispose of the remains or goods is a violation of state law and shall be charged as a Class IV felony.

The National Historic Preservation Act of 1966 (NHPA) may also apply because the Project would require a Section 404 of the CWA permit through the United States Army Corps of Engineers (USACE) and the Environmental Protection Agency (EPA) may contribute grant money.

> *Section 106 of the National Historic Preservation Act (NHPA) and its implementing regulations (36 CFR Part 800).*Section 106 requires federal agencies to take into account the effects of their undertakings on historic properties (any prehistoric or historic district, site, building, structure, or object listed on or eligible for listing on the National Register of Historic Places [NRHP]).

2.3.3.2 BACKGROUND RESEARCH RESULTS

Previously Recorded Archaeological Sites

No previously recorded archaeological sites were identified within the Study Area.

Previously Recorded Architectural Properties

Eight previously recorded architectural properties were located within the CIA Area (see Table 5, and Appendix B, Attachment 4 and Attachment 5). Properties consist of one ranch, one house, two farmsteads, two sod houses, and two cemeteries. The Washington Rankin Ranch (BL00-019) is eligible for listing on the NRHP and the Isadore Haumont House (CU00-047) is listed on the NRHP. The NRHP eligibility of the remaining architectural properties is unknown.

Architectural property CU00-047 is noted as non-extant and does not intersect the CIA Area. All remaining architectural properties within the Study Area are noted as extant and none intersect the CIA Area.

Table 5. Previously Recorded Architectural Properties within the CIA Area

Property Number	County	Township	Range	Section	Property Type	NRHP Eligibility
BL00-019	Blaine	21N	21W	33	Washington Rankin Ranch	Eligible
CU00-047	Custer	17N	19W	11	Isadore Haumont House	Listed
CU00-063	Custer	18N	20W	1	Farmstead	Unknown
CU00-120	Custer	19N	19W	31	Farmstead	Unknown
CU00-138	Custer	19N	20W	5	Gates Cemetery	Unknown
CU00-154	Custer	20N	21W	9	Sod House	Unknown
CU00-155	Custer	19N	20W	29	Cemetery	Unknown
CU00-269	Custer	19N	20W	27	Bates-Myers Sod House	Unknown

Previous Archaeological Surveys

Two archaeological surveys have been conducted within the CIA Area (Table 6 and Appendix B, Attachment 4 and Attachment 5). Both archaeological surveys intersect the CIA Area.

Table 6. Previous Archaeological Surveys within the CIA Area

Report Number	Title	Author	Year
94-0123	NHAP-PSS STPE-1680(4), Sargent West Borrow Pits.	Trisha Nelson	1994
99-0035	Class III Cultural Resource Survey of the Federal Lands Surrounding Milburn Diversion Dam, and Selected Areas on the Sargent, Farwell and Sherman Feeder Canals, Blaine, Custer, Howard, Sherman, and Valley Counties, Nebraska.	Bob Blasing	1998

General Land Office Maps

No cultural features were identified within the Study Area on GLO maps dating from 1873 to 1874 (Appendix B, Attachment 6). Multiple natural features are depicted on the GLO, including the Middle Loup River, Lillian Creek, unnamed creeks and drainages, sloughs, ravines, and river islands.

2.3.3.3 RECOMMENDATIONS

The proposed canal (Lillian Creek Canal) development mostly falls within the floodplain of the Middle Loup River. The proposed 15-mile-long canal crosses multiple drainages and creeks before terminating near Lillian Creek, northeast of the proposed reservoir. Based on available aerial photography, land use appears to consist of pasture and cultivated fields (Google Earth™ mapping service 2017). The proposed reservoir area is concentrated around Lillian Creek and its drainages within rugged topography. The proposed reservoir area appears to include mostly pasture areas with portions of cultivated land concentrated along Lillian Creek. The area is dissected by several roads and a few structures are also within the proposed reservoir boundary.

A review of the CIA Area identified no previously identified archaeological sites, two previous archaeological surveys, and no cultural GLO map features. Portions of both previous surveys intersect very small areas within the Study Area, specifically within the proposed canal (Lillian Creek Canal) development area. No portions of the proposed reservoir have been surveyed. The lack of archaeological sites found near the CIA Area may be attributable to the low number and small size of the archeological surveys conducted in the vicinity and may not reflect actual site density.

A review of the CIA Area identified eight previously identified architectural properties. Of the eight previously identified architectural properties, one is eligible for listing on the NRHP (BL00-019) and one is listed on the NRHP (CU00-047), although it is also noted as non-extant. The NRHP eligibility of the remaining six properties is unknown. None of the previously identified architectural properties intersect the CIA Area. In addition, a review of aerial photographs revealed the presence of several undocumented structures within the Study Area (Google Earth™ mapping service 2017).

Based on the results of the literature review, the topography of the area, the proximity of water sources, and the lack of formal survey, undiscovered archaeological sites and architectural properties may be present within the current Project Area. Any ground disturbing activity within the Project Area has the potential to affect unknown cultural resources.

For compliance purposes, the lead federal agency would determine the Project Area of Potential Effects (APE) and the next steps required to fulfill permit requirements as they relate to Section 106 of the NHPA responsibilities.

3.0 Reservoir Analysis

3.1 Geotechnical Evaluation

Slope stability, seepage, and settlement analyses were performed to evaluate feasibility of constructing an earth-fill dam along the proposed dam alignment (that is, the Project). A subsurface profile along the proposed dam alignment is provided as Figure 4. Initial analyses were performed on a conventional embankment section for the following:

- A thickness of 69 feet above the valley;
- 3H:1V (Horizontal to Vertical) side slopes on the upstream and downstream sloped at a height of 36 feet above the valley;
- Transitioning to 6H:1V side slopes on the upstream and downstream slopes at a height of 29 feet above the valley;
- A 20-foot-wide bench on the upstream slope at a height of 36 feet above the valley; and
- A 20-foot-wide crest.

Slope stability analyses were performed first to determine mitigation measures needed to construct the embankment and meet the minimum factor of safety requirements. Seepage analyses were then performed on the embankment section, including any slope stability mitigation measures determined from the slope stability analyses, to evaluate the need for seepage mitigation measures. The slope stability and seepage analyses are an iterative process used to determine an appropriate embankment section and potential mitigation measures. Figure 8 provides the embankment section determined as a result of the iterative process. The complete preliminary subsurface investigation and geotechnical evaluation report is provided in Appendix A.

3.1.1 Slope Stability Analysis

Slope stability was evaluated for rapid drawdown, steady seepage, steady seepage with earthquake, and end of construction. The primary assumptions made for the slope stability analyses are presented below.

- The dam would be constructed from soil borrowed from the lean clay (CL) alluvial soils within the valley upstream of the dam and sandy silt (ML) to silty sand (SM) loess/eolian deposits excavated from the auxiliary spillway.
- The embankment section consists of lean clay (CL) upstream of centerline and sandy silt (ML) to silty sand (SM) downstream of centerline.
- The embankment includes an internal drainage system consisting of a vertical chimney drain at the centerline and a series of finger drains extending to the downstream toe of the embankment section.
- The clay blanket is continuous upstream and downstream of the dam.

Evaluation of the factor of safety for each loading case was based on the criteria presented in TR-60. The minimum required factors of safety are provided in Table 7.

Table 7 Minimum Required Factors of Safety for Stability

Loading Case	Minimum Factor of Safety
End of Construction	1.4
Rapid Drawdown	1.2
Steady Seepage	1.5
Steady Seepage w/ Seismic	1.1

The results of the slope stability analyses are presented in Table 8 and Table 9. Bold numbers indicate that the minimum factor of safety was not obtained. Supporting documentation, including calculations, for the stability analyses are provided in Appendix A.

Table 8. Summary of Slope Stability Analyses (Conventional Embankment)

Embankment Section	Slope	Loading Case Minimum Factor of Safety			
		End-of-Construction	Rapid Drawdown	Steady Seepage	Steady Seepage with Seismic
Initial Embankment Section	Upstream	0.77	1.32	N/A	N/A
	Downstream	0.72	N/A	1.43	1.11

N/A = Not Applicable

Based on these results, mitigation is needed for the construction and long-term performance of the proposed dam embankment. Possible mitigation measures for the proposed dam embankment slope stability would consist of stability berms, staged construction, and ground improvement.

Stability berms would consist of embankment placed near the toe of the dam embankment slope. The weight of the berms provides additional resistance as a counterbalance to a slope stability failure. The use of stability berms on the upstream and downstream slopes was evaluated. It was determined that the length of the stability berms was excessive and not a practical solution. Shorter stability berms are possible by staging the construction of the embankment.

Staged construction could consist of building the embankment to the maximum height that can maintain the minimum required factor of safety, then allowing the embankment to sit for approximately 6 to 8 months, so that excess pore pressures in the foundation soils can dissipate as the composite soils settle. The dissipation of excess pore pressures and resulting consolidation increases the effective stress and shear strength of the soil, which allows additional embankment to be placed without falling below the minimum required factor of safety.

Table 9 provides a summary of the slope stability analyses for the embankment section with a combination of stability berms and staged construction.

Table 9. Summary of Slope Stability Analyses (with Stability Berms and Staged Construction)

Embankment Section	Slope	Loading Case Minimum Factor of Safety			
		End-of-Construction	Rapid Drawdown	Steady Seepage	Steady Seepage with Seismic
Stability Berms with Staged Construction Stage 1	Upstream	1.57	N/A	N/A	N/A
	Downstream	1.48	N/A	N/A	N/A
Stability Berms with Staged Construction Stage 2	Upstream	1.47	1.54	N/A	N/A
	Downstream	1.42	N/A	2.03	1.45

N/A = Not Applicable

The results of the stability analyses demonstrated that the minimum factors of safety could be obtained for a dam embankment with the following:

- 3.5H:1V upstream and downstream slopes;
- A 10-foot-wide upstream bench at elevation 2,466 feet;
- A 10H:1V upstream stability berm at elevation 2,466 feet;
- A 10H:1V downstream stability berm at elevation 2,460 Feet; and
- Staged construction with the first stage ending at elevation 2,470 feet and the second stage ending at elevation 2,500 feet.

Ground improvement would consist of either removal of the soft soils and replacement with a higher strength material or installation of high strength elements that can transfer a portion of the embankment load to more competent soil deposits beneath the soft soil.

Due to the relatively thick soft soil beneath the dam embankment (more than 40 feet) and the depth to groundwater (14 feet), ground improvement, consisting of over excavation of the soft soil beneath the proposed dam embankment and re-compacting it, was determined to not be a feasible alternative. Other ground improvement measures such as deep soil mixing or rigid inclusions were not evaluated as a part of this feasibility study because they are typically cost prohibitive relative to stability berms and staged construction.

3.1.2 Seepage Analysis

Dams must be designed and maintained to meet the requirements of TR-60. According to TR-60, seepage analyses made for anticipated seepage rates and pressures through the embankment, foundation, abutments, and reservoir perimeter must show that that the proposed dam can accomplish the intended reservoir function, provide a safely operating structure, and prevent damage to downstream property. For the purpose of this Project, the downstream vertical exit gradient, the factor of safety for piping, and the factor of safety for uplift were evaluated.

TR-60 does not provide specific guidance on acceptable vertical exit gradient and factor of safety for piping and uplift. Based on review of USACE literature seepage at dams, the design of the dam should be based on a maximum vertical exit gradient of 0.2, a minimum factor of safety for piping of 4, and a minimum factor of safety for uplift of 1.5, evaluated at the downstream toe of the dam.

Underseepage was evaluated through the alluvium deposits at the downstream toe of the proposed dam embankment. In addition, underseepage was also evaluated through the loess/eolian deposits at the downstream toe of the abutment groin area, due to the sandy nature of the loess/eolian deposits.

A summary of the underseepage analyses are provided in Table 10. Bold numbers indicate that the allowable average vertical exit gradient is exceeded. Supporting documentation and calculations for the underseepage analyses are provided in Appendix A.

Table 10. Summary of Underseepage Analyses

Embankment Location	Water Surface Elevation (feet)	Average Vertical Exit Gradient at D/S Toe		Factor of Safety at D/S Toe		Comments
		Entire Blanket[1]	Upper Blanket[2]	Piping	Uplift	
Downstream Toe of Stability Berm	2460 (Normal Pool)	0.15	0.28	2.7	4.9	i > 0.2 and Piping Factor of Safety <4; NG
Downstream Toe of Abutment Groin	2460 (Normal Pool)	0.13[3]		3.5	N/A	Piping Factor of Safety <4; NG

N/A = Not Applicable; NG = Not Good; D/S = Downstream

1 Entire blanket consists of all blanket material classifying as fat clay (CH), lean clay (CL), silty clay (CL-ML), and silt (ML).

2 Upper blanket consists of the upper layer classifying as lean clay and fat clay (CL/CH) when it overlies more permeable blanket layers classifying as lean clay (CL), silty clay (CL-ML), or silt (ML).

3 No blanket is present at the downstream toe of the abutment. The vertical exit gradient is average over the top 3 feet.

Based on these results, mitigation is needed due to a high vertical exit gradient at the downstream toe of the stability berm and due to a low factor of safety for piping at both the downstream toe of the stability berm and the downstream toe of the abutment groin. Possible mitigation measures for the high vertical exit gradient and low factor of safety for piping at the downstream toe of the stability berm would include the installation of pressure relief wells or sheet pile cutoff walls. Possible mitigation measures for the low factor of safety for piping at the downstream toe of the abutment groin would consist of trench drains or drainage blankets. Seepage analyses were not completed for the different mitigation measures.

3.1.3 Settlement Analyses

Settlement analyses were performed to estimate the magnitude and time-rate of settlement of the dam and foundation due to consolidation of the fine-grained alluvial deposits in the valley. A summary of the settlement analyses are provided in Table 11. The consolidation parameters and the results of the settlement analyses are presented in Appendix A.

Table 11. Summary of Settlement Analyses

Embankment Section	Location	Embankment Height (feet)	Settlement (inches)	t_{90} (days)
Stability Berms with Staged Construction	Valley	69	28	260

t_{90} = time for 90% consolidation.

Typically, tall earth-fill dam embankments are overbuilt a few feet to compensate for the long-term settlement. Design of the principal spillway would need to account for the anticipated settlement.

3.1.4 Borrow Soil

Potential on-site soils for construction of the Project include the valley soils and the upland soils. The auxiliary spillway would likely be excavated into the upland soils resulting in a substantial quantity of this material. The upland soils consist of sandy silt (ML) to silty sand (SM) and have a relatively high permeability. These soils could be used downstream of the chimney drain and as upstream or downstream stability berms. These soils are generally at a water content below optimum and would need water added to facilitate compaction.

The valley soils consist of primarily lean clay (CL) and some fat clay (CH) near the surface and have a relatively low permeability. The lean clay (CL) soils would be suitable for use anywhere in the embankment and could be used to construct a liner along the upland pool area. The fat clay soils could be used anywhere in the embankment at a depth of 3 feet or greater from the surface. Removal of the valley soils near the dam could result in the need for additional stability and seepage mitigation measures.

3.1.5 Summary of Geotechnical Findings

The results of the analyses indicate that significant mitigation measures will be necessary to construct and operate an earth-fill dam at this site to the evaluated height of approximately 70 feet above the valley. The primary concerns for construction of an earth-fill include:

- Subsidence of the dam abutments, due to the collapse potential of the loess/eolian deposits upon wetting,
- Stability of the dam, due to the presence of thick, soft, fine-grained alluvial deposits within the valley,
- Seepage and piping causing erosion beneath the dam, due to the significant head acting on the dam, and
- Seepage and piping causing erosion along the abutment groin areas, due to the highly erosive nature of the loess/eolian deposits and head acting on the dam.

Potential mitigation measures to address the previously mentioned primary concerns include:

- Pre-wetting or removal and replacement of the collapse susceptible loess/eolian deposits along the abutments,
- Constructing a zoned embankment consisting of lean clay upstream of centerline and silty sand downstream of centerline,
- Constructing an internal drainage system consisting of a vertical chimney drain at the centerline and a series of finger drains extending to the downstream toe of the embankment section to allow for a controlled collection and discharge of seepage through the embankment,
- Constructing upstream and downstream stability berms consisting of silty sand as a counterbalance to slope stability failure,
- Staging construction of the embankment to allow for dissipation of excess pore pressures during construction,

- Constructing downstream pressure relief wells or constructing a sheet pile cutoff wall to reduce seepage pressures at the downstream toe of the embankment, and installing trench drains or blanket drains along the downstream abutments to allow for a controlled collection and discharge of seepage through the pervious loess/eolian deposits.

These mitigation measures have been employed to construct other dams in the Project area with similar subsurface conditions. It is possible that through implementation of the mitigation measures, or a combination thereof, that a stable embankment can be constructed at the Project location; however, long-term maintenance and post construction mitigation may also be required. Given the nature of the existing conditions, additional geotechnical testing would be necessary to evaluate and design the dam embankment and mitigation measures. Geotechnical expertise in design and construction of high head dams in pervious and collapse-susceptible soils will required. Additional geotechnical testing and evaluation would be necessary to evaluate and design the dam embankment and mitigation measures.

3.2 Water Supply Alternatives

3.2.1 Watershed Sources

Potential sources of water evaluated to fill and maintain Lillian Creek reservoir levels include:

- Runoff from the contributing watershed;
- Middle Loup River flows delivered via gravity canal flow from Milburn Diversion Dam;
- Middle Loup River flows captured by alluvial well field adjacent to the Middle Loup River and delivered via pipeline to the reservoir; and
- Groundwater augmentation from existing or new wells located in the upper portion of the watershed.

However, water can only be captured when there are flows available in excess of downstream appropriations and demands. Therefore, an excess runoff analysis was performed to determine when and how much flow would be available to capture and store.

3.2.1.1 Excess Flow Analysis

A new dam along Lillian Creek including its resultant reservoir would require a natural flow appropriation from NeDNR to capture and store natural flows. This new appropriation would be junior to current natural flow appropriations and subject to administration in times of shortage. Therefore, natural flow can only be stored when existing priority appropriations are being fully met. This condition would apply to both runoff and Middle Loup River sources.

Excess Flow Analysis Background

A study was conducted to evaluate the historic excess flows in the Platte River. The study is titled *Evaluation of Historic Platte River Streamflow in Excess of State Protected Flows and Target Flows* (HDR 2010). The purpose of this study was to:

- Evaluate the historic quantity of excess flow in the Platte River;

- Develop a planning tool to estimate the rate of flow and duration and frequency of water in excess of state protected flows by reach; and,
- Determine the quantity of water in excess of target flows based on wet, dry, and normal hydrologic classification.

The 2010 study included the area from the North Platte River just below Lake McConaughy and the South Platte River at Julesburg, Colorado, to the Platte River near Louisville, Nebraska. The 2010 study compared the amount of natural flow available in various specified reaches and then compared those flows to the computed demands for natural flow in the same specified reaches. The following builds on this 2010 study by carrying this analysis upstream into the Middle Loup River Basin.

This analysis did not address future conditions and is not intended to provide a potential applicant with an analysis sufficient to establish whether excess flow is available for a specific project (new use). Any applicant seeking a surface water permit would need to provide to NeDNR a comprehensive package at the time the application or variance is filed.

The results summarize excess flow and the number of days that excess flow has been available. It should be noted that the number of days with excess flow may or may not be consecutive, and operational constraints that limit the ability to divert short-duration occurrences of excess flows were not considered.

Data Inputs

Data used to compute excess flows include the mean daily discharge recorded by Platte River and Loup River gages for the period from January 1, 1988, through December 31, 2011, and the Platte River instream flow appropriations and irrigation requirements (2017 NeDNR Biennial Report).[2] The period from 1988 to 2011 was selected because it represents naturally occurring wet and dry cycles to avoid bias between wet and dry periods and to accommodate non-stationarity in climate patterns. NeDNR has used this period for its annual fully appropriated basin evaluations (NeDNR, 2015). Suitability of the selected climatic period was evaluated by performing an auto-covariance and Kendall Tau statistical analysis of the data.[3] NeDNR uses the periods of record 1988 to 2011.

The Platte River instream flow appropriations are water appropriations granted for recreational use or the needs of existing fish and wildlife, and vary through specific stream reaches and time of year.

Historical gage data were obtained from USGS. Table 12 lists the gages used in this evaluation. The Platte River instream flow appropriations for North Bend, Nebraska, and Louisville are shown in Table 13. The irrigation demands for the Middle Loup River are provided in Table 14. Figure 9 shows the locations of each gage and points of interest.

[2] https://nednr.nebraska.gov/dynamic/waterrights/SelectSearchOptions.aspx
[3] NeDNR performed the statistical analysis internally.

Table 12. USGS Gages Used in Analysis

Gage Number	Gage
06805500	Platte River at Louisville, Nebr.
06796000	Platte River at North Bend, Nebr.
06792500	Loup River Power Canal near Genoa, Nebr.
06793000	Loup River near Genoa, Nebr.
06785000	Middle Loup River at Saint Paul, Nebr.
06784000	South Loup River at Saint Michael, Nebr.[1]

1 The South Loup River at Saint Michael is subtracted from the Middle Loup River at Saint Paul to isolate the volume of flow from the Middle Loup subbasin

Table 13. Platte River Instream Flow Appropriations

Period	North Bend (cfs)	Louisville (cfs)
January 1 – January 31	1,800	3,100
February 1 – July 31	1,800	3,700
August 1 – August 31	1,800	3,500
September 1 – September 30	1,800	3,200
October 1 – December 31	1,800	3,700

cfs = cubic feet per second

Table 14. Middle Loup Basin Demands

Demand	Owner	Appropriation (cfs)
Loup Hydropower	Loup Public Power District	1,260[1]
Sherman Feeder Canal	Loup Basin Reclamation District	730[2]
Sargent Canal	Loup Basin Reclamation District	204[3]
Middle Loup Canals #1, #2, #3, #4	Middle Loup Public Power & Irrigation District	520[3]

1 Only 36% of the Loup hydropower demand was applied in the Middle Loup Basin (3,500 cfs x 0.36 = 1260 cfs). This is consistent with the basin accounting methodology developed for the Lower Platte River Basin Coalition Water Management Plan (https://lprbc.nebraska.gov/MtgMaterials/LPRBC_BWMP_AppendixC_TM_Basin_Accounting_20170911.pdf).

2 Demands for the Sherman Feeder Canal were applied between the months of April and December.

3 Demands for Middle Loup Canals #1, #2, #3, and #4 and Sargent Canal were applied between the months of April and September.

Methodology for Determining Excess Flow

When evaluating whether excess flow in an upstream reach is available, the downstream reach must first be evaluated. If flows in a downstream reach are insufficient to satisfy state protected flows in the Platte River on any given day, then not only is there no excess flow in the downstream reach for that day but the upstream reach, similarly, would not have any available excess flow on that day. The excess flow determination for the Project begins at the Platte River at the Louisville gage, and proceeds upstream to the Platte River at North Bend, the Loup River at Genoa, and finally the Middle Loup River at St. Paul and Sargent. This process is described in more detail in the following paragraphs, and is shown in Figure 10.

STEP 1: If the daily gage flow at Louisville exceeds both the Platte River instream flow appropriation at Louisville, then excess flow is available for the Lower Platte River from North Bend to Louisville subbasin on that day. The Lower Platte River is defined as the Platte River from the Platte River/Loup River confluence to the Platte River/Missouri River confluence. The excess flow amount is equal to the gage flow less the instream flow demand. If the daily gage flow does not satisfy the Platte River instream flow appropriation, then no excess flow is available the Lower Platte River from North Bend to Louisville subbasin, nor is it available for the entirety of the Platte River Basin downstream of the Loup River confluence (including the Loup and Elkhorn Basins) on that day.

STEP 2: If the daily gage flow at North Bend exceeds the instream flow demand at North Bend and excess flow was available from STEP 1, then excess flow is available for the Lower Platte upstream of North Bend on that day. The excess flow amount is equal to the gage flow at North Bend less the instream flow demand at North Bend. If the daily gage flow does not satisfy the instream flow demand at North Bend, then no excess flow is available for not only the Lower Platte Above North Bend subbasin, nor is it available for the Lower Platte River Basin upstream of this analysis point (including the Loup Basin) on that day either.

STEP 3: The Loup River near Genoa excess flow check was performed above the Loup River Public Power District's (LPD's) hydropower appropriation (3,500 cubic feet per second [cfs]). The daily gage flow for this analysis point was calculated as the sum of the daily gage flow for the Loup River near Genoa gage and the Loup River Power Canal near Genoa gage. For the Loup Basin above the hydropower diversion, if excess flow is available from STEP 1 and STEP 2, and the daily gage flow for the Loup River near Genoa plus the Loup River Power Canal near Genoa exceeds the Loup hydropower demand, then excess flow is available in the Loup Basin. The excess flow amount is equal to the gage flow less the LPD hydropower demand. If the daily gage flow does not satisfy the LPD hydropower demand, then no excess flow is available for the Loup Basin on that day. If excess flow is available in the Loup River near Genoa, then excess flow is also available in upstream Loup subbasins in the amount equal to the gage flow at the upstream analysis points on that day capped to the excess flow available at Genoa.

STEP 4: For the Middle Loup River at Sargent, the excess flow check was performed above Middle Loup Public Power and Irrigation District (520 cfs demand), the Sherman Feeder Canal (730 cfs demand), and 36 percent of the Loup River's hydrological demand (the assumed

Middle Loup contribution—see Table 14). The daily gage flow for this analysis point was calculated as the daily gage flow for the Middle Loup River at St. Paul less the South Loup River at St. Michael less the calculated reach gain between Sargent and St. Paul plus the historic canal diversions.

For the Middle Loup Basin at Sargent, if excess flow is available from STEPS 1, 2, and 3, and the calculated daily gage flow for the Middle Loup River at Sargent exceeds the canal demands, then excess flow is available in the Middle Loup Basin at Sargent. The excess flow amount is equal to the calculated gage flow less the canal demands (Canals #1, #2, #3, #4 and Sherman Feeder). If the daily gage flow does not satisfy the canal demands, then no excess flow is available for the Middle Loup Basin at Sargent on that day.

Excess Flow Scenarios

Six demand scenarios were considered. They are as follows:

- Downstream instream flow demands only.
- Downstream instream flow demands and full LPD hydropower appropriation only (no irrigation).
- Downstream instream flow demands, Loup Basin Reclamation District (LBRD) irrigation demands, and Middle Loup Public Power and Irrigation District (MLPPID) irrigation demands (no hydropower appropriation).
- Downstream instream flow demands, full LPD hydropower appropriation, Loup Basin Reclamation District (LBRD) irrigation demands, and Middle Loup Public Power and Irrigation District (MLPPID) irrigation demands (Table 15).
- Downstream instream flow demands, historic LPD hydropower diversions, Loup Basin Reclamation District (LBRD) irrigation demands, and Middle Loup Public Power and Irrigation District (MLPPID) irrigation demands.
- Downstream instream flow demands, historic LPD hydropower diversions, and Loup Basin Reclamation District (LBRD) irrigation demands. The Middle Loup Public Power and Irrigation District (MLPPID) irrigation demands are not included.

3.2.1.2 Watershed Runoff

The Cropsim model was used to estimate the Lillian Creek watershed runoff. Based on the amount of flow available from the watershed, the amount available for Project capture was determined using the results of the excess flow analysis.

CropSim was developed specifically for Nebraska and is an integral component of groundwater modeling efforts throughout the state. CropSim incorporates weather station data, land use, soil parameters, and crop water demands to estimate recharge, runoff, evapotranspiration (ET), and pumping demands through time. For this evaluation, the Cropsim output was used to estimate monthly volumes of watershed runoff from precipitation events for the period of analysis.

The monthly Cropsim watershed runoff volumes were distributed linearly to derive a daily runoff volume estimate. The daily runoff estimate was then multiplied by the number of days each month that excess flows were available. Watershed runoff available for capture was then determined. Figure 11 through Figure 16 show the amount of available watershed runoff as well as the number of days and the amount available for capture based on the excess flow evaluation for each of the six demand scenarios. Table 15 summarizes the number days each month the new appropriation would be in priority and able to store water. In addition, Table 15 summarizes the volume of runoff available for capture in the proposed reservoir from the watershed (column A) for the six demand scenarios.

Table 15. Water Available for Capture

		AVG # Days with Excess Flow	(A) Runoff Volume Available for Capture in Lillian Reservoir (acre-feet)	(B) Middle Loup River Natural Flow Available for Capture (acre-feet)	(C) Average Annual Import using 100 cfs pump (acre-feet)	(D) Average Annual Import using 250 cfs pump (acre-feet)
Middle Loup River at Sargent	Instream Flow Only	254	2,547	484,344	52,888	128,229
	Instream Flow + Full Hydropower	42	894	57,234	9,118	20,341
	Instream Flow + Irrigation	180	1,405	236,115	36,812	87,668
	Instream Flow + Full Hydropower + Irrigation	18	287	34,020	3,864	8,901
	Instream Flow + Historic Loup Diversion + Irrigation[1]	94	565	86,528	19,070	44,244
	Instream Flow + Historic Loup Diversion + Irrigation[2]	135	1,354	117,954	27,948	64,562

1 Includes Middle Loup irrigation demands
2 Middle Loup irrigation demands not applied

When instream flows are the only demand considered, watershed runoff is available for capture on average 254 days. The volume of water available for capture under that scenario is 2,547 acre-feet, on average. Considering instream flows, the full hydropower appropriation, and the full irrigation demand, the watershed runoff is only available for capture 18 days on average each year, resulting in 287 acre-feet available for capture. The volume of water available for capture from the watershed is not sufficient to support the Project. Flows from a different source will be required to make the Project viable.

A qualitative analysis was performed to evaluate the runoff from the Lillian watershed determined from CENEB as compared to watershed yields from adjacent gaged watersheds that have similar hydrologic characteristics as the Lillian Creek watershed. This was done by calculating the annual runoff of each watershed, and subtracting the approximated baseflow based on baseflow indices provided by USGS (USGS SIR 2010 – 5149) to determine the average annual runoff. Baseflow indices are commonly used to estimate the portion of total streamflow that is attributable to baseflow. Baseflow indices for the watersheds were based on *Simulation of Groundwater Flow and Effects of Groundwater Irrigation on Stream Base Flow in the Elkhorn and Loup River Basins, Nebraska, 1895-2055 Phase Two* (USGS 2010). Stream discharge is multiplied by the baseflow index to estimate the baseflow contribution, with the remaining gaged discharge considered as being the estimated runoff.

The Lillian Creek contributing watershed area to the Project is 50 square miles. Several watersheds were identified for comparison and are noted in Table 16.

Table 16. Watersheds adjacent to Lillian Creek watershed

Station	Station Number	Drainage Area (square miles)	Ratio of base flow to total streamflow[1]	Average Annual Runoff (in)
North Branch Verdigre Creek near Verdigre	06465680	137	87%	0.32
Bazile Creek at Center	06466400	265	63%	1.60
Willow Creek near Foster	06799080	137	64%	0.54
Beaver Creek at Loretto	06793500	311	70%	1.37

[1] Baseflow indices obtained from USGS SIR 2010 – 5149

Daily flow data for the above-listed streams were obtained from the NeDNR database or USGS.

The direct runoff for the Lillian Creek Watershed was estimated using the Central Nebraska Model. The estimated average annual runoff is 1.26 inches. This compares favorably with the average annual runoff values listed in table 16.

The average annual runoff from the Lillian Creek watershed was also compared to a published value. The published value in the USGS Hydraulic Atlas 710: Average Annual Runoff in the United States, 1951-1980 (Figure 17) is closer to 2.0 inches annually.

The qualitative analysis of adjacent basin runoff, as well as the published USGS runoff, indicate that the values determined using CENEB and considered reasonable for this feasibility study.

3.2.1.3 Middle Loup River Diversion

Diversion of water from the Middle Loup River was also investigated as a potential source for the Project. The amount of available Middle Loup River water was determined based on the excess flow analysis described in Section 3.2.1.1 near Sargent. Two pumping scenarios were evaluated to convey the water from the Middle Loup River—one that conveys 100 cfs to the Project and one that conveys 250 cfs to the Project. Figure 18 through Figure 23 show the amount of available Middle Loup water based on the excess flow evaluation for each of the six demand scenarios, as well as amount that can be pumped to the proposed reservoir based on two pumping conditions. The amount of available Middle Loup River for each demand scenario is also represented in Table 15. Column B represents the Middle Loup River excess flow available for capture while Columns C and D represent the amount of flow that can be pumped with 100 cfs and 250 cfs pumps, respectively. When instream flows are the only demand considered, the volume of Middle Loup River water available for capture is approximately 484,344 acre-feet. The amount of water that could be pumped to the Project is 52,888 acre-feet and 128,229 acre-feet for the 100 cfs and 250 cfs pumping scenarios, respectively. Considering instream flows, the full hydropower appropriation, and the full irrigation demand, the volume of Middle Loup River water available for capture is approximately 34,060 acre-feet. The amount of water that could be pumped to the Project is 3,864 acre-feet and 8,901 acre-feet for the 100 cfs and 250 cfs pumping scenarios, respectively. Depending on the demand scenario, there appears to be sufficient Middle Loup River water to sustain the Project in conjunction with capturing watershed runoff as part of the Project.

3.2.2 Imported Water

The feasibility of importing water to the proposed reservoir from the Middle Loup River water by way of the Milburn Diversion Dam was evaluated. In the 1950s, USBR preliminarily designed the Lillian Creek Canal. The USBR design reports and current GIS datasets were reviewed to determine if the Lillian Creek Canal could be used to import surface water.

The Lillian Creek Canal was identified in 1954 as a potential component of the Missouri River Basin Project, Sargent Unit by the USBR (USBR Plan). The Sargent Unit Definite Plan report provides a description of the canal's preliminary design components and cost benefit analysis, a summary of which is provided as a bulleted list.

- The Sargent Unit is located primarily in Custer County along the Middle Loup River between the towns of Milburn and Comstock, Nebraska.
- The lands are generally located within the semiarid Loess Hills Region of Nebraska, which is characterized by rolling terrain sloping gently from northwest to the southeast.
- Site geology was formed by glacial deposits of debris, sand, and gravel. Weathering of these deposits along with later aeolian deposition of loess formed the present dune sand and loess topography. Deeper pre-glacial strata are comprised of Pierre Shale bedrock at depths of about 450 feet below the Middle Loup River valley, overlain by largely unconsolidated silty sand, calcareous sandstone, and siltstone.
- Three stages were planned for development in the Sargent Unit. Only the first stage was constructed.
    - Stage 1 – Milburn Diversion Works, Sargent Canal, Sargent Distribution and Surface Drainage Systems.
    - Stage 2 – Milburn Diversion Works enlargement, Lillian Canal, Lillian Hydroelectric Plant, Lillian Distribution and Surface Drainage System.
    - Stage 3 – No new features. Stage three considered the curtailment of divertable flows due to upstream diversions of the Merna Unit, Grand Island Division. Merna Unit features consisted of the Dismal River diversion works and the Mullen Dam and Reservoir.

- The Lillian Creek Canal was designed to operate continuously year-round to serve the planned Hydroelectric Plant. Lillian Distribution systems would only operate during the irrigation season. The Hydroelectric Plant was planned for roughly the same location as the Project's proposed reservoir. This plant would operate with 91 feet of constant head, and discharge to a tailrace adjacent to Lillian Creek.
    - Design Capacities:
        - Initial = 780 cfs at the diversion headworks
        - Terminal = 700 cfs at the hydroelectric plant
        - Distribution System = 70 cfs for irrigation
    - Conveyance losses were estimated to be 10 percent, proportionally distributed along the canal length. The first 10 miles were planned to pass through dune and river sand, requiring a compacted earth lining 2 feet thick at the base and 3 feet thick normal to sideslopes.
    - Milburn Diversion design elevations are listed below. Normal pool elevation was similar to the diversion model elevation reported in USBR Report No. HYD-385 (2484.5 feet).
        - Normal Pool = 2,486.88 feet
        - Maximum Pool = 2,488.08 feet
        - Gate Seat = 2,480.48 feet
    - Lillian Canal design parameters:
        - Slope = 0.00008 foot per foot (ft/ft)
        - Base Width = 18 feet
        - Sideslopes = 2h:1v
        - Kutter's Coefficient 'n' of Roughness = 0.0225
        - Maximum Velocity = 2 feet per second

A benefit-cost analysis was computed for each stage of development. The analysis was updated and revised to reflect costs in January 1953.

A spatial analysis was performed to evaluate the feasibility of importing water to the proposed reservoir by canal. Two canal alignments were considered, the historic Lillian Creek Canal alignment and an alternative alignment (Figure 24). Both alignments divert water from the Middle Loup River at the Milburn diversion. Design elevations, canal slope, soil permeability, and proximity to irrigated and dryland farms were considered in this assessment.

Spatial information for the historic Lillian Creek Canal alignment was generated based on a USBR flyer circa 1959. This alignment shows the canal running along the southern edge of the Middle Loup River valley. ArcGIS was used to generate a profile for this alignment based on 10-meter DEM data (Figure 25). The planned conservation pool for the proposed reservoir is between 2,450 feet and 2,460 feet. Figure 25 shows that to feasibly construct the canal along the historic Lillian Creek Canal alignment would require extensive siphons or 10 to 30 feet of fill over more than 7 miles of canal length.

An alternative alignment the moves the canal further to the south, to higher elevations, was generated. The alternative alignment increases the length of Lillian Creek Canal from 15.6 miles to 17.1 miles from the Milburn Diversion to Lillian Creek when compared to the historic Lillian Creek Canal alignment. To deliver water at an elevation of 2,460.0 feet an average canal bed slope of 0.00022 ft/ft is calculated based on a diversion elevation of 2,480.5 feet per the USBR Plan. This average slope allows for upwards of 12 feet in head losses for siphons and other hydraulic structures when compared to the USBR Plan design slope of 0.0008 ft/ft. Figure 26 shows that the alternative alignment can accommodate this design slope with reduced cut and fill operations when compared to the historic alignment.

To deliver water from the canal to the proposed reservoir, a siphon or extensive cut would be necessary through the adjacent hill slope. Two potential routes (A and B) were identified based on 10-meter DEM data, and confirmed with USGS LiDAR data (Figure 27). Each route is approximately 3,000 feet in length and would require a peak cut of 50 feet.

Soil permeability was evaluated along both the historic and alternative alignments (Figure 28). Based on NRCS soil surveys saturated hydraulic conductivity exceeds 3 inches per hour nearly throughout both alignments, with a median value exceeding 10 inches per hour. The alternative alignment operates at higher elevations relative to the water table, which would further increase potential conveyance losses. This observation underlines the need for canal lining for both the historic and alternative alignments. The USBR Plan assumed a 2- to 3-foot-thick compacted earth liner constructed with clayey loess soils obtained from a borrow site about a mile east of Milburn.

Construction of the Lillian Creek Canal has potential benefits for local agricultural. Figure 29 depicts irrigated (green) and 3,820 acres of irrigable (yellow) lands that were identified in the USBR Plan. Current irrigated and irrigable lands were delineated using 2016 National Agriculture Imagery Program (NAIP) aerial imagery (Figure 30). For this delineation, it was assumed that center pivots were irrigated (4,600 acres) and all other agricultural lands were irrigable (4,140 acres).

The USBR Plan completed a detailed benefit-cost analysis for the Sargent Unit. Construction of the Lillian Creek Canal, Milburn diversion works, laterals, and surface and subsurface drains was estimated at $6,201,000 in January 1953. Adjusted for inflation, this amount equates to $50,650,000 in 2017 dollars. Lillian Creek Canal and the Lillian Hydroelectric plant comprised the second stage of planned development for the Sargent Unit. USBR computed annual irrigation, power, and flood control benefits for a 100-year analysis period and found a benefit-cost ratio of 1.40:1.00. Eliminating the power component from the analysis reduces the ratio to roughly 0.20:1.00. Given the construction cost in 2017 dollars, and a benefit cost ratio less than one, the Lillian Canal option was eliminated from further consideration, based on correspondence with LLNRD staff.

3.2.3 Other Sources

Hydrogeologic information from a group of wells near the upstream portion of the proposed reservoir were assessed as a possible supply source for the Project. The intended purpose of the assessment was to ascertain the general water yield characteristics of the aquifer materials for use as a water source alternative to supply water to the proposed reservoir.

The assessment was performed using available irrigation well registration records from nine select wells near the proposed reservoir. Figure 31 shows the location of the wells assessed, permitted acres of irrigation wells, irrigated lands, and water level changes (1982 to 2017). The irrigation well records include complete geologic logs and well construction information, pumping rates, and static and pumping groundwater levels (Table 17).

Table 17. Information Obtained from Well Registration Records

Well Registration Number*	Diameter of Casing (feet)	Total Well Depth (feet)	Pumping Rate (gallons per minute [gpm])	Depth to Static Water Level (feet)	Drawdown During Pumping (feet)
G-172438	0.497	180	227	99.6	7.4
G-172489	0.497	172	227	96	12.0
G-172490	0.497	380	185	277	13.0
G-172439	0.497	250	185	169	9.0
G-147535	0.339	285	25	210	10.0
G-172445	0.497	404	173	304	20.0
G-097650	1.333	188	900	97	13.0
G-018237	1.500	194	1200	96	24.0
G-045560	1.333	95	900	3	9.0

Information from each well was interpreted or calculated based on the available information from the registration records. These include thickness of permeable materials, aquifer type, specific capacity, bulk aquifer transmissivity, and hydraulic conductivity. Thickness of permeable materials was determined by summing the thickness of generally high-conductivity geologic formations found on the geologic logs beneath the water table (saturated). Similarly, the aquifer type was determined from the geologic logs and static water level, and reflects whether saturated low-permeability geologic formations are located above permeable zones. Confined/semi-confined aquifers have low-permeability strata lying on top of high-permeability zones and are generally disconnected or have a delayed connection to high-permeable units lying above the confining/semi-confining formations. Specific capacity is calculated by dividing the pumping rate by the difference between static and pumping water levels (drawdown). Aquifer transmissivity was determined based on the recorded pumping rate and drawdown, but was calculated using the Cooper-Jacob solution (Cooper and Jacob 1946) for pumping tests in confined aquifers. It applies here to unconfined aquifers as well since the drawdown is less than 25 percent of the thickness of permeable materials. Aquifer hydraulic conductivity is then calculated by dividing the transmissivity by the pre-pumping thickness of permeable zone materials (saturated).

Hydrogeologic characteristics, aquifer transmissivity, and hydraulic conductivity have been determined for nine wells (Table 18). These wells indicate a moderate to high production potential for water yields from the aquifer underlying the upgradient portion of the site, with pumping rates recorded ranging from 25 to 1,200 gallons per minute (gpm), and thicknesses of the permeable zones ranging from 35 to 97 feet. Drawdown under recorded pumping conditions at time of well development were generally less than 25 percent of permeable zone thickness. However, depth to the water table is about 100–300 feet for all wells assessed, except for well G-045560, located west of the proposed reservoir, and the furthest north. This well also has the largest calculated transmissivity and hydraulic conductivity.

Table 18. Interpreted and Calculated Aquifer Properties from Well Registration Records

Well Registration Number*	Permeable Zone Thickness (feet)	Calculated Specific Capacity (gallons per minute [gpm] per foot)	Calculated Transmissivity (square feet per day [ft2/d])	Calculated Hydraulic Conductivity (feet per day [ft/d])	Aquifer Type
G-172438	69	30.7	7627	111	Confined or Semi-Confined
G-172489	48	18.9	4554	95	Confined or Semi-Confined
G-172490	97	14.2	3917	40	Unconfined
G-172439	47	20.6	4976	106	Confined or Semi-Confined
G-147535	66	2.5	657	10	Unconfined
G-172445	35	8.7	1971	56	Confined or Semi-Confined
G-097650	60	69.2	15898	265	Confined or Semi-Confined
G-018237	89	50.0	11020	124	Confined or Semi-Confined
G-045560	95	100.0	26412	278	Unconfined

Based on the hydrogeologic analysis, there is sufficient capacity to develop a groundwater well field to supplement flows into the reservoir. However, potential third-party effects in the form of stream depletions and interference also need to be considered. Since 1987, there has been a decline in the groundwater table in the area ranging from 9 feet to 2 feet (Figure 31). The development of a groundwater wellfield to supply the reservoir would likely increase the rate of decline, and create potential interference with adjacent irrigators. For these reasons, development of a groundwater wellfield to supplement supplies to the reservoir was not considered further.

3.2.4 Water Supply Summary

Based on the water supply analysis, runoff from the watershed alone is not sufficient to create a normal pool. Supplementing runoff flows with Middle Loup River excess flows is more than sufficient to establish a normal pool. However, doing so will require a capital investment in the form of a well field. Seepage losses could be significant and require mitigation. For purposes of this study, three water supply demand scenarios were carried forward to evaluate the viability of the Project when taking seepage losses into consideration. They are:

- Downstream instream flow demands only.
- Downstream instream flow demands, **full** LPD hydropower appropriation, Loup Basin Reclamation District (LBRD) irrigation demands, and Middle Loup Public Power and Irrigation District (MLPPID) irrigation demands (Table 15).
- Downstream instream flow demands, **historic** LPD hydropower diversions, Loup Basin Reclamation District (LBRD) irrigation demands, and Middle Loup Public Power and Irrigation District (MLPPID) irrigation demands (Table 15).

These conditions allow for an evaluation for the best case water supply scenario (instream demands only), a worst case water supply scenario (full appropriations for instream, hydropower and irrigation), and a potential water supply scenario (historic hydro diversions and modified irrigation). These three demand scenarios were carried forward for additional analysis.

3.2.5 Losses

Reservoir losses are a critical component of the water budget in determining the water potentially available for management. Based on the potential losses, an analysis was performed to determine the fate of delivered water to the reservoir taking into consideration seepage and evaporation losses, thus determining the potential net storage.

3.2.5.1 Seepage Losses

A limited subsurface investigation was completed to assist with determining the seepage losses. Based on the subsurface investigation, subsurface profile in the uplands generally consists of deep deposits of silty sand (SM) to sandy silt (ML) loess/eolian deposits, are considered to be well drained, and would likely be prone to significant seepage losses. The subsurface profile in the valley generally consists of fine-grained relatively impervious alluvial deposits of lean clay (CL) overlying coarse-grained alluvial deposits. Both the uplands and the valley soils were underlain by dense sandy silt (ML) and silty sand (SM) alluvial deposits.

At the upstream toe of the dam, the approximate elevation of the break between the relatively impervious alluvial soil deposits in the valley and the relatively pervious loess/eolian deposits in the uplands is at about elevation 2,448 feet.

Seepage rates were determined for the north and south slopes using Darcy's law. Calculated seepage rates using Darcy's law are a function of gradient of flow, and permeability of soil. Gradient of flow is a function of total depth of water and thickness of the soil layer. The assumed permeabilities listed in Table 19. The evaporation was based on historic Grand Island pan evaporation data multiplied by the surface area.

Table 19. Constant Head Permeability Test Results

Location	Permeability (feet/sec)
North Slope	2.43×10^{-5}
South Slope	1.80×10^{-6}
Valley	1.31×10^{-7}

The calculated seepage rates using Darcy's law were considerably high. A qualitative analysis was performed comparing the calculated seepage rates, and observed values for a reservoir near the proposed Project. Sherman Reservoir is located approximately 45 to the southeast in Sherman County. End-of-month volumes of Sherman Reservoir were obtained from the NeDNR annual hydrographic reports, and were used to estimate average annual seepage rates. Based on the analysis, Sherman Reservoir typically seeps at a rate of 10 inches per month during the winter. The results of the calculated seepage rates using Darcy's law and the calculated historic seepage rates are compared in Table 20.

Table 20. Stage-Storage-Area-Seepage

Elevation (NAVD88)	Surface Area (Acres)	Storage (Acre-feet)	Calculated Seepage Using Darcy's Law (acre-feet/day)	Observed Seepage assuming 10" per month[1] (acre-feet/day)
2418	5	0	0	0.13
2420	8	10	0	0.21
2430	41	240	6	1.15
2440	90	880	23	2.50
2448	159	1,860	56	4.42
2450	184	2,200	69	5.10
2460	325	4,710	410	9.02
2470	518	8,880	1,509	14.37
2480	786	15,350	4,204	21.82
2490	1116	24,810	8,948	30.99
2500	1535	38,010	16,897	42.64

NAVD88 = North American Vertical Datum of 1988

[1] 10 inches per month is consistent with the seepage rate observed at Sherman during the winter drawdown.

The calculated seepage rates using Darcy's law were considerably higher than those observed at Sherman Reservoir (Table 20). It is likely that over time, the seepage rates at the proposed dam would decrease as fine materials settle out in the reservoir and approach the seepage rates observed at Sherman Reservoir. However, the duration required to achieve that rate is unknown. In addition, the field permeability tests that were performed are relatively general in nature. It is recommended that more extensive geotechnical analysis be performed to reduce the uncertainty of the potential seepage at the proposed Project location. The importance of obtaining a better understanding of seepage differences is evident in the following section.

3.2.5.2 Reservoir Net Storage

For each of the three demand scenarios carried forward, the net storage volume change over time (assuming no releases) was estimated based on runoff available for capture less evaporation and seepage losses for the period of analysis. Time series plots were developed which provide a representation of the amount of flow that can be captured based on the excess flow analysis, as well as how much of the captured volume is lost to seepage and evaporation. Figures 32 through 37 show the net storage change over time for capturing available water from the watershed, as well as capturing available runoff and excess Middle Loup River flows. The potential net reservoir storage was evaluated for both the calculated and observed seepage, as well as two pumping scenarios to convey Middle Loup River water to the reservoir.

The seepage assumption and the demand scenario has a large effect on the results as shown in Figures 32 through 37. For watershed runoff capture, the reservoir never reaches normal pool elevation for any of the demand scenarios evaluated when using the calculated seepage rate (Figure 32). However, the storage volume reaches or approaches normal pool for the instream only flow demand scenario (Figure 33). Similar effects are seen when supplementing watershed runoff with Middle Loup River excess flows. Using the calculated seepage rate and a conveyance rate of 100 cfs, the reservoir never reaches normal pool elevation under any of the demand scenarios (Figure 34). This is due primarily to the seepage rate being greater than the pumping rate. However, if the observed seepage rate is used, the normal pool elevation is reached for the three demand scenarios carried forward (Figure 35). For these scenarios, the pool is drawn down during the irrigation season when excess flows are not available. Increasing the amount of excess flow pumping to 250 cfs shows that the normal pool is reached under either seepage condition, with the only change being the amount of drawdown during the irrigation season (Figures 36 and 37).

These results suggest that when water is being diverted from the Middle Loup River, agreements will need to be made with other water users in the basin. Additionally, further subsurface investigations should be conducted to reduce uncertainty of seepage rates.

3.2.6 Groundwater Impacts

A steady-state numerical groundwater model was developed to evaluate the groundwater/surface water interactions because of the Project. This evaluation provides insight into long-term potential changes to the groundwater system prior to and after constructing the proposed dam. Two model simulations were performed, one in steady state without the reservoir (Pre-Dam Condition), and another, also in steady state, with the reservoir in place and assumed to be at normal pool elevation of 2,460 feet (Post-Dam Condition). For the Post-Dam simulation, the steady-state model run provides a state in which the hydraulic heads have fully equilibrated to the inflows from the Lillian Creek Reservoir, and do not provide information about the time to reach equilibrium. The modeling and study purposes include: (1) characterize effects on depth to groundwater; (2) estimate possible reservoir volume lost via seepage (or infiltration); and (3) estimate possible effects on groundwater discharge (baseflow to Lillian Creek and the modeled reach of the Middle Loup River).

3.2.6.1 Groundwater Model Development and Simulations

The groundwater flow equations were solved numerically using the MODFLOW-NWT engine and associated packages, with setup and post-processing via Groundwater Vistas (v. 6.96), and ArcGIS (v. 10.3). The model domain is 31,600 feet (6.0 miles) wide—extending from Gates County Road on the west to Road 446 on the northeast—and 33,600 feet (6.4 miles) long—extending from the southern edge of the normal pool to the Middle Loup River in the north. Model grid cells are inactive north of the Middle Loup River. Finite difference grid cells are a uniform 100 feet by 100 feet in horizontal dimension, and the grid is aligned north to south. There are 373,572 active grid cells in the Pre-Dam Condition simulation and 374,739 active grid cells in the Post-Dam Condition simulation.

Five model layers (continuous across the domain) were included. Each layer has a different thickness, with some layers varying in thickness. Thicknesses and elevation of model layer boundaries are a function of using both topography and contacts from a University of Nebraska at Lincoln Conservation and Survey Division (UNL-CSD) test-hole (12-A-66, located at 41.625759 N lat., 99.565049 E lon.) as a basis for model construction. Elevations of the land surface assigned to the model grid are derived ~10-meter DEM data from the National Elevation Dataset. Elevations were resampled to 100-foot resolution and snapped to the grid.

Model layer 1 (uppermost), extends above land surface by 0.5 foot and is only active during the Post-Dam Condition simulation to accommodate the boundary condition. This layer is inactive during Pre-Dam Condition simulation. Model layer 2 extends from land surface and approximately half the distance down to elevation 2,354 feet, and model layer 3 makes up the rest of that lower extent. Model layer 2 has an approximate thickness of 30 feet near the dam, but varies from about 0.3 foot near the Middle Loup River to 160 feet in the interfluves and uplands. Model layer 3 has an approximate thickness of 60 feet near the dam, but varies from about 1.5 to 330 feet. Model layer 4 extends from elevation 2,354 feet to 2,209 feet, and has a constant thickness of 145 feet. Similarly, model layer 5 has constant thickness, in this case equaling 340 feet, extending from elevation 2,209 feet to 1,869 feet. The Pierre Shale is considered to have substantially lower permeability than the overlying units of the High Plains Aquifer does; therefore, the model domain does not extend below the High Plains Aquifer, and the base of the model domain is no-flow boundary.

Boundary conditions for the model include those on the lateral edges of the domain and those affecting the surface (or interior) parts of the domain. The model grid and boundary conditions are displayed on Figure 38 for Pre-Dam Condition and on Figure 39 for the Post-Dam Condition. The lateral boundary conditions are defined as general head boundaries, allowing specification of a head and for flows into and out of the domain depending on hydraulic gradient directions and a conductance (C) parameter. The C for the general head boundaries was set using the initial hydraulic conductivity (K) of each model layer but was subsequently adjusted to match estimated discharge to the Middle Loup River. The conductance parameter is associated with all types of head-dependent boundary conditions used by MODFLOW, and is calculated with the following expression, $C = K * L * W / B$; in this equation, C is in units of length2/time (L2/T), K is in units L/T, L is length in units of L, W is width in units of L, and B is thickness of bed sediments in units of L. Head values for these boundaries were set by interpolating the regional hydraulic head contours from the spring 1995 water-table contours GIS dataset published by the UNL-CSD. Doing this provides the basic flow-field for the simulated water table.

Interior boundary conditions include those assigned to allow groundwater and surface water interactions to and from the Middle Loup River, Lillian Creek, and the Lillian Creek Reservoir (under normal pool conditions). The Middle Loup River and Lillian Creek Reservoir were simulated using river boundaries, and Lillian Creek was simulated using stream boundaries. Cells assigned to Middle Loup River and Lillian Creek boundaries were selected based on review of the National Hydrography Dataset, and recent aerial imagery (August 27, 2017, obtained from Google Earth™ mapping service). Water surface stages on the Middle Loup River and reservoir bottom elevations (on the Lillian Creek Reservoir) were assigned using the 100-foot resolution DEM (derived from the 10-meter DEM), and on Lillian Creek using either 1-meter LiDAR (where available) and 10-meter DEM elevations. Water stage on the Lillian Creek Reservoir was set to the normal pool elevation of 2,460 feet. Water bottom elevations for the Middle Loup River were set equal to the stage elevation minus 2 feet, and minus 1 foot on Lillian Creek. Stream boundaries (Lillian Creek) require the input parameters of slope and channel roughness that controls the one-dimensional surface water routing (based on Manning's Equation) performed by the stream package. Manning's n (roughness) was assigned a value of 0.05 for the entire channel, and the slope varied from 0.0024, for the section of stream above the dam, to 0.0022 for the section downstream of the dam. Stream length was determined based on the lengths of the NHD polyline associated with each MODFLOW grid cell. The surface flow rate is monitored during the simulation, and the stream infiltration is not allowed to exceed the flow in the stream. Refer to Table 21 for the boundary condition parameters assigned in the MODFLOW simulations.

Table 21. Boundary Condition Parameters

Boundary Condition	Number of Cells (-)	Stage/Head (ft NAVD88)	Bed Elevation (ft NAVD88)	Hydraulic Conductivity (ft/d)	Conductance (ft^2/d)
River Boundaries (Middle Loup)	1334	2360.5–2406	2358.5–2404	0.1	1,000
Stream Boundaries (Lillian Creek; Above Dam)	185	2414.75–2451.54	2413.75–2450.54	1.0	1.24–255.58
Stream Boundaries (Lillian Creek; Below Dam)	335	2367.89–2426.41	2366.89–2425.41	1.0	2.42–270.64
River Boundaries (Lower Reservoir; Reach 1)	572	2460	2417.87–2447.95	0.0086	86

Boundary Condition	Number of Cells (-)	Stage/Head (ft NAVD88)	Bed Elevation (ft NAVD88)	Hydraulic Conductivity (ft/d)	Conductance (ft^2/d)
River Boundaries (Upper Reservoir; Reach 2)	595	2460	2448.01–2459.97	0.4	4,000
Head Dependent Boundaries	2877	2364.38–2464.16	N/A	0.1–5	10–500

NAVD88 = North American Vertical Datum of 1988

Recharge to the model from precipitation was applied at two rates following some adjustments (along with general head boundary conductance and Middle Loup River boundary conductance) to match estimated discharge to the Middle Loup River. The negative rate of -3.5 inches per year to agricultural lands and over the Middle Loup River (identified based on August 27, 2017 Google Earth™ mapping service imagery, and cells selected by an on-screen comparison within Groundwater Vistas) was assigned due to the limited rainfall and use of center-pivot irrigation. Other lands were assigned a recharge rate of 1.14 inches per year, a rate half that of the statewide average based on the satellite-derived estimates published by Szilagyi and Jozsa (2013). These values compare favorably in terms of magnitude and spatial distribution shown by Szilagyi and Jozsa (2013) as well.

Hydraulic conductivity, and the storage parameters, specific yield (Sy in units of L3/L3) and specific storage (Ss in units of 1/L), for each model layer was assigned uniform values as follows:

- Layer 1 – Kh = 0.4 ft/d; Kv = 0.1 ft/d; Sy = 0.15; Ss = 0.3000 1/ft
- Layer 2 – Kh = 0.4 ft/d; Kv = 0.1 ft/d; Sy = 0.15; Ss = 0.0050 1/ft
- Layer 3 – Kh = 2.0 ft/d; Kv = 0.5 ft/d; Sy = 0.15; Ss = 0.0025 1/ft
- Layer 4 – Kh = 30 ft/d; Kv = 7.5 ft/d; Sy = 0.15; Ss = 0.0010 1/ft
- Layer 5 – Kh = 10 ft/d; Kv = 5.0 ft/d; Sy = 0.15; Ss = 0.0004 1/ft

Where Kh and Kz are the horizontal and vertical hydraulic conductivity, respectively. These parameters are based on review of the lithology described on the test-hole log from the site (12-A-66), the geotechnical report prepared in support of the *Dam Feasibility Study* (Appendix A) and the lithology/soils from dam and valley borings, and the Central Nebraska Groundwater Flow Model (CENEB) documentation (Brown and Caldwell 2013).

Estimated groundwater discharge (baseflow) to the Middle Loup River from within the model domain was used as a calibration target. This target was developed based on a baseflow estimate to the Middle Loup River of 165 cfs between the Dunning and St. Paul USGS gages for 2005 (Stanton et al. 2010; Table 7). The rate per mile was estimated by dividing 165 cfs by 118 miles (distance measured between Dunning and St. Paul gages along the Middle Loup River) and dividing by a factor of two. Division by two is required to develop the target because flows from the model domain to the Middle Loup River only account for approximately one-half of the flows because the model domain only encompasses one-half of the river valley. Multiplication of this net gain rate of 0.699 cfs per mile by the 6.5 mile of river in the model domain provided the target baseflow gain of 4.54 cfs (392,000 cubic feet per day [ft^3/d]).

Results were evaluated using water budget outputs and exported water table elevations from each MODFLOW grid cell. Depth to groundwater was assessed by subtracting water table elevations from model top elevations (derived from the 10-meter DEM). Changes in the water table position caused by construction of the dam and subsequent filling to the normal pool elevation (2,460 feet) were evaluated by subtracting Pre-Dam Condition from Post-Dam Condition water table elevations.

A transient model run was setup spanning 323 months (26.92 years), for which the initial condition (heads) was set equal to the heads from the steady-state Pre-Dam model run and the reservoir was simulated as active (Post-Dam condition; 2,460-foot pool elevation), allowing for the dynamics of the seepage from the reservoir to be simulated as affected through time as the seepage water causes mounding of the water table. Time steps were set initially equal to 1e-05 days and then increased with a 1.3 multiplier not to exceed increments of 1 year, which was used for the last 23 time steps. Other time step setups were initially tested, but this one allowed the desired temporal aspects from the model outputs to be captured, and ended with seepage rates and aquifer heads closely approximating those obtained from the steady-state Post-Dam simulation.

3.2.6.2 Assessment of Model Performance

The groundwater model performance was assessed by comparing simulated discharge to the Middle Loup River with the target values described above, the latter of which equal 393,000 ft^3/d. Simulated net discharge to and from the Middle Loup River equals 384,000 ft^3/d, only 2.2 percent lower than the target value. As another check on the overall simulation, there were two water table contours from the spring 1995 dataset (2,400 feet and 2,450 feet), which were compared against those from the model. Generally, the difference between simulated and observed water table contours was less than approximately 5 feet, and in some areas very near zero.

The model performance checks indicate that the groundwater model performs well and is capable of providing quantitative estimates of flows and hydraulic heads with a reasonable certainty. However, it is important to note that calibration efforts were limited and the spatially distributed absolute depths to groundwater are potentially inaccurate. Reducing uncertainty through improvements to model calibration would be possible if water levels from wells were assessed, and if further refinements to recharge/evapotranspiration and hydraulic conductivity (of aquifer materials and of boundary conditions) were undertaken. Furthermore, there was no formal assessment of the sensitivity of results to the final selection of model parameters or the distance between the lateral boundaries and Lillian Creek Reservoir. The lateral boundaries do not coincide with any impermeable geologic boundaries. In general, however, the model domain extents are judged as reasonable, and any influence of the boundaries on the overall results are expected to be small.

3.2.6.3 Impacts to Water Table Position and Depth

One of the purposes of the developing the model simulations is to characterize the effects of constructing the dam to the water table elevations and depth to groundwater. The change between Post-Dam and Pre-Dam Condition simulated water table elevations were mapped across the model domain (Figure 40). The increased height of the water table has a magnitude ranging from 0 to 47 feet across the domain, and has an average of 4.1 feet. The largest increases are focused around the reservoir and changes diminish toward the lateral boundaries of the model. As shown on Figure 40, downstream of the proposed dam (approximate northern half of the model domain), the changes in water table elevation are generally 1 to 10 feet.

Pre-Dam Condition depth to groundwater (Figure 41) varies across the model domain from less than 0 (heads greater than land surface) to 286 feet, and has an average equal to 83 feet. Downstream of the proposed dam depth to groundwater (Pre-Dam) is generally between 0 and 30 feet. There is only a small portion of the domain (4.6 percent, 977 acres) with simulated water table within 5 feet of land surface (excluding the Middle Loup River and Lillian Creek Reservoir boundaries).

Post-Dam Condition depth to groundwater (Figure 42) varies across the model domain from less than 0 to 286 feet, and has an average depth equal to 78 feet. Downstream of the proposed dam depth to groundwater (Post-Dam) is generally between 0 and 40 feet. There is only a small portion of the domain (5.9 percent, 1,266 acres) with simulated water table within 5 feet of land surface (excluding the Middle Loup River and Lillian Creek Reservoir boundaries). This is an increase of 289 acres over the Pre-Dam Condition simulation.

3.2.6.4 Effects on Groundwater Discharge (Baseflow)

Evaluation of the water budget output from the Pre-Dam and Post-Dam simulations allows for the quantification of effects on groundwater discharge (baseflow to Lillian Creek and the modeled reach of the Middle Loup River). The simulated water budget flows by boundary condition (and recharge) are shown in Table 22. In addition to displaying the water budget components, Table 22 also shows that the mass balance error was negligibly small. The net flows per boundary condition are calculated as the sum of the inflows to the groundwater system (positive values) and outflows from the groundwater system (negative values).

Table 22. Model-Computed Water Budget for Pre-Dam and Post-Dam Conditions

Volumetric Exchange Rates for the Pre-Dam Condition				
Water Budget Component	IN (ft^3/d)	OUT (ft^3/d)	NET (IN – OUT) (ft^3/d)	NET (IN – OUT) (acre-ft/month)
River Boundaries (Middle Loup)	9,261	393,308	(384,047)	(268.4)
Stream Boundaries (Lillian Creek)	13,309	25,227	(11,918)	(8.3)
General Head Boundaries	793,274	278,140	515,135	360.0
Recharge	87,942	207,116	(119,174)	(83.3)
Total	903,787	903,791	(4)	(0.0)

Volumetric Exchange Rates for the Post-Dam Condition				
Water Budget Component	IN (ft³/d)	OUT (ft³/d)	NET (IN – OUT) (ft³/d)	NET (IN – OUT) (acre-ft/month)
River Boundaries (Middle Loup)	5,354	458,923	(453,569)	(316.9)
River Boundaries (Reservoir)	463,825	-	463,825	324.1
Stream Boundaries	13,063	63,418	(50,355)	(35.2)
General Head Boundaries	478,395	319,137	159,257	111.3
Recharge	87,942	207,116	(119,174)	(83.3)
Total	1,048,579	1,048,594	(14)	(0.0)

Table 23 lists the net flows from the Middle Loup River and Lillian Creek for both the Pre-Dam and Post-Dam condition, and the differences. The differences represent the simulated accretions to surface water caused by developing the dam. Total accretions equal 75.4 acre feet per month (acre-ft/month), 26.9 acre-ft/month to Lillian Creek, and 48.6 acre-ft/month to the Middle Loup River.

Table 23. Model-Computed Changes to Net Groundwater Discharge (Baseflow)*

Volumetric Exchange Rates in Units of Cubic Feet per Day			
River/Stream	NET (Pre-Dam) (ft³/d)	NET (Post-Dam) (ft³/d)	Overall Change (ft³/d)
Middle Loup River	384,047	453,569	69,522
Lillian Creek	11,918	50,355	38,436
Total	395,965	503,924	395,965
Volumetric Exchange Rates in Units of Acre-Feet per Month			
River/Stream	NET (Pre-Dam) (acre-ft/month)	NET (Post-Dam) (acre-ft/month)	Overall Change (acre-ft/month)
Middle Loup River	268.4	316.9	48.6
Lillian Creek	8.3	35.2	26.9
Total	276.7	352.1	75.4

* Net groundwater discharge (baseflow) is reported here as the difference between inflows to groundwater from the river/stream and outflows from groundwater to the river/stream

3.2.6.5 Reservoir Volume Lost via Seepage (or Infiltration)

Reservoir volume lost via seepage and infiltration according to the summation of model-computed flow rates from all reservoir boundary cells equals 463,800 ft3/d. Converting cubic feet to acre-ft and assuming an average month has 30.4375 days, the flow rate equals 324.1 acre-ft/month. A breakout of the total infiltration losses, those incurred in the lower part of the reservoir (with bottom elevations below 2,448 feet), and those incurred in the upper part of the reservoir (with bottom elevation between 2,448 feet and 2,460 feet) are listed in Table 24. Of the seepage losses, approximately 23.3% of the water returns to the downstream river/stream (75.4 acre-ft/month out of 324.1 acre-ft/month) based on the steady-state (long-term) differences between the Pre-Dam and Post-Dam conditions simulated. The other differences in flows are not explicitly accounted for in this evaluation, but the excess water seeping from the reservoir would cause the increased hydraulic heads shown in Figure 40 and would cause net flows out of the domain through the lateral general head boundaries.

Table 24. Dam Seepage (Infiltration) Losses

Reservoir Pool	Area[1] (acres)	Volumetric Seepage Rate (ft^3/d)	Volumetric Seepage Rate (acre-ft/month)	Percent of Total Dam Seepage (%)
Reach 1 (Lower Reservoir; Elevation <2448')	131.31	227,400	158.9	49.0
Reach 2 (Upper Reservoir; Elevation >2448')	133.84	236,400	165.2	51.0
Entire Reservoir	265.15	463,800	324.1	100

[1] Area of the reservoir is determined by multiplying the number of cells by 10,000 ft^2 and converting to acres by dividing by 43,560 acres/ft^2.

The transient simulation, in which Pre-Dam steady-conditions occur at first, followed by immediate filling of the reservoir to the normal pool (2,460 feet; Post-Dam conditions), provides a quantitative evaluation of seepage dynamics. Figure 43 illustrates the temporal response of the overall reservoir seepage losses for this transient simulation over a 26.92-year period (323 months). The ratio of seepage losses over time to steady-state (Post-Dam) seepage losses is also displayed on the secondary y-axis. Early in the simulation, seepage losses are about 10,700 acre-ft/month, but decline quickly to rates much closer to the steady-state rate—90 percent reduction between initial and steady rates occurs in 2.4 days, at which time the seepage rate is 1,300 acre-ft/month, four times the steady rate. After 70 days, the seepage losses decrease further, by a factor of two, equaling twice the steady rate (648 acre-ft/month). Over the rest of the simulated period, approximately 26.73 years, the seepage losses continue to gradually decline and approach the steady-state Post-Dam seepage rate of 324.1 acre-ft/month.

Figure 44 depicts the change in hydraulic head at a centrally located observation point in the aquifer beneath the reservoir. Only about a 1-foot change in hydraulic heads occurs during the time it takes for a 90 percent reduction of the seepage rate (from initial to steady), and a 13.6-foot rise in heads is associated with the reduction of the seepage rate from over 30 times to two times the steady rate. This 13.6-foot increase in head equals about half of the total rise from initial to steady. After the 26.92 years simulated, the hydraulic head is within 0.2 foot of steady-state head, and the change in heads is less than 0.5 percent of the difference between initial and steady head.

3.3 Reservoir Routing

A hydrologic analysis was performed to develop the principal and auxiliary spillway configurations, as well as the corresponding top of dam (TOD). USACE Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) Version 4.2.1 was used to model the proposed reservoir (USACE 2016). The evaluation used the National Oceanic and Atmospheric Administration (NOAA) Atlas 14 point precipitation rainfall data and the Nebraska Site Specific probable maximum precipitation (PMP) rainfall depths.

The SCS unit hydrograph method was used to produce the runoff hydrographs in the HEC-HMS model. The drainage area upstream of Lillian Creek dam was modeled as one subbasin. The subbasin parameters included drainage area (49.6 square miles), curve number (65), percent impervious (0), and lag time (650 minutes).

3.3.1 Hydrology

According to the Soil Conservation Service Technical Release No. 60 (TR-60), high hazard dams are located where failure may cause loss of life, serious damage to homes, buildings, important public utilities, main highways, or railroads. For purposes of the feasibility study, the Lillian Creek reservoir is classified as a high hazard dam given the dam location to downstream property.

3.3.1.1 Normal Pool

The drainage area into the Lillian Creek reservoir is approximately 32,000 acres (50 square miles). A normal pool elevation at 2,460 feet was analyzed and corresponds to a surface area of 325 acres.

3.3.1.2 Precipitation Amounts

In 2013, NOAA published a Precipitation-Frequency Atlas of the United States (NOAA Atlas 14). Volume 8 of Atlas 14 covers the Midwestern states, which includes the state of Nebraska. Precipitation frequency estimates were computed for a range of frequencies and durations using a regional frequency analysis approach. Table 25 shows the 10-, 50-, 100-, and 500-year precipitation depths for storm durations between 5 minutes to 24 hours for the Lillian Creek Watershed.

Table 25. Point Precipitation Depths

Point Precipitation Depths for Given Durations (inches)								
Precipitation Event	5-min	15-min	1-hr	2-hr	3-hr	6-hr	12-hr	24-hr
10-year	0.61	1.08	1.86	2.23	2.44	2.83	3.29	3.77
50-year	0.85	1.52	2.59	3.10	3.37	3.90	4.54	5.18
100-year	0.96	1.73	2.93	3.50	3.80	4.39	5.11	5.83
500-year	1.26	2.25	3.80	4.51	4.87	5.58	6.51	7.43

Source: NOAA, Atlas 14, http://hdsc.nws.noaa.gov/hdsc/pfds/index.html,
Coordinates: 41.6257, -99.5808

3.3.1.3 Probable Maximum Precipitation

The precipitation data to evaluate the auxiliary spillway hydrograph (ASH) and freeboard hydrograph (FBH) are a function of the PMP. PMP depths are available for varying drainage areas including 10 square miles and 200 square miles. Because the drainage area to the Lillian Creek reservoir is approximately 50 square miles, the 10-square-mile drainage area maps were used. The PMP depths for 10-square-mile drainage areas were developed using *HMR-52 and Site-Specific Probable Maximum Precipitation (PMP) Study for Nebraska* (December 2008) and are shown in Table 26.

Table 26. Probable Maximum Precipitation Depths (Inches) for Various Storm Durations

Precipitation Event	Precipitation Depths (Inches) for Various Storm Durations							
	5-min [1]	15-min [1]	1-hr [2]	2-hr [1]	3-hr [1]	6-hr [3]	12-hr [3]	24-hr [3]
PMP (10 mi²)	3.4	5.5	14.5	16.5	17.5	18.7	21.5	21.9

[1] Depths computed utilizing data for other storm durations and procedures provided in National Weather Service Hydrometeorological Report No. 52 (HMR-52).

[2] Data acquired from National Weather Service Hydrometeorological Report No. 52 (HMR-52).

[3] Data acquired from Site-Specific Probable Maximum Precipitation (PMP) Study for Nebraska (Dec 2008).

3.3.1.4 Precipitation for Reservoir Routing

Combinations of the 100-year, 500-year, and PMP base rainfall data were required for use in analysis of the potential regional detention basin. Design hydrographs were generated from base rainfall data according to the NRCS's TR-60 Dams and Reservoirs criteria.

3.3.1.5 Design Hydrograph Precipitation Depths

One variation from TR-60 criteria used in the analysis was the use of a 500-year event for principal spillway design rather than the standard 100-year event. Precipitation depths for each specified duration were computed by the following equation to create a high hazard dam principal spillway hydrograph (PSH):

$$P_{PSH} = P_{500}$$

where:

P_{PSH} = Precipitation depth for principal spillway hydrograph, inches
P_{500} = Precipitation depth for 500-year return period, inches

The precipitation data to evaluate the ASH for each specified duration are computed by the following equation:

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

where:

P_{ASH} = Precipitation depth for auxiliary spillway hydrograph, inches
P_{100} = Precipitation depth for 100-year return period, inches
PMP = Probable Maximum Precipitation, inches

The precipitation data to evaluate the FBH for each specified duration are computed by the following equation:

$$P_{FBH} = PMP$$

where:

P_{FBH} = Precipitation depth for freeboard hydrograph, inches
PMP = Probable Maximum Precipitation, inches

Table 27 summarizes the PSH, ASH, and FBH precipitation depths.

Table 27. Precipitation Depths for PSH, ASH, and FBH Design Events (Inches)

Design Event	Precipitation Depths for Various Storm Durations (Inches)							
	5-min	15-min	1-hr	2-hr	3-hr	6-hr	12-hr	24-hr
PSH [1]	1.26	2.25	3.80	4.51	4.87	5.58	6.51	7.43
ASH [2] (10 mi^2)	1.59	2.71	5.94	6.88	7.36	8.11	9.37	10.01
FBH [3] (10 mi^2)	3.4	5.5	14.5	16.5	17.5	18.7	21.5	21.9

1 Depths equal to point precipitation depths for 500-year event.

2 Combination of 100-year event and PMP. Point precipitation depths for 100-year event must be adjusted for respective storm area before being combined with PMP depths.

3 Depths equal to PMP depths for 10 mi^2.

3.3.1.6 Areal Rainfall Adjustments

The storm area in HEC-HMS was used to automatically compute the depth-area reduction factor. In most cases, the specified storm area should be equal to the watershed drainage area at the point of evaluation. For the Lillian Creek dam analysis, a storm area of 50 square miles was used in the model.

3.3.2 Reservoir Routing and Design Considerations

Reservoir routings of the design events were performed to determine the size of the outlet works and obtain expected reservoir pool elevations for each of the design hydrographs. The HEC-HMS model used for reservoir routing uses the continuity equation to develop an outflow rate as a function of the reservoir stage-storage relationship and the inflow rate.

The methodology for routing the design hydrographs to determine dam design parameters was based on TR-60 criteria. First, the PSH event was routed using the normal pool elevations. Auxiliary spillway crest elevations were established by rounding the peak stage obtained from the respective PSH event to the nearest whole foot. After establishing the auxiliary spillway crest elevation, the ASH event was routed. Adjustments were made to the auxiliary spillway width according to the peak stage obtained from the respective ASH events. Finally the FBH event was routed, and the TOD elevation was established by rounding the peak stage obtained from the respective FBH event up to the nearest whole foot.

3.3.2.1 Principal Spillway Parameters

A stage volume relationship was developed based on LiDAR data and elevation-storage relationships for Lillian Creek reservoir were developed based on LIDAR data provided by LLNRD. These rating curves represent the pre-project condition. Table 28 presents the relationship among the elevation/stage, storage, and surface area based on an approximate alignment.

Table 28. Stage-Storage-Area Relationship

Elevation/Stage 1 (feet)	Storage (acre-feet)	Surface Area (Acres)
2418	0	5
2420	10	8
2430	240	41
2440	880	90
2448	1,860	159
2450	2,200	184
2460	4,710	325
2470	8,880	518
2472	10,170	572
2476	12,760	679
2480	15,350	786
2484	19,134	918
2489	23,864	1083
2490	24,810	1116
2500	38,010	1535

Elevations based on North American Vertical Datum of 1988.

The PSH event for the respective storm area was routed using the normal pool elevation. A minimum standard principal spillway scenario including a 16-foot by 6-foot riser with trash rack intake structure and a 500-foot-long, 48-inch-diameter reinforced concrete cylinder pipe (RCCP) discharge conduit was evaluated at the proposed Lillian reservoir site. The estimated pipe size and estimated length of pipe were based on similar conceptual dam designs as well as anticipated embankment heights (minimum 40 feet) and embankment slopes (3H:1V). Rating curves for the principal spillway were developed by checking each of the possible controls: weir flow at the intake, orifice flow through the riser cap, orifice flow through the riser at the intake, orifice flow at the conduit, and pipe flow control.

3.3.2.2 Auxiliary Spillway Location and Parameters

The 500-year event was used to establish the height of the auxiliary spillway crest. An earth cut, vegetated spillway was used as the auxiliary spillway type. The standard section through the auxiliary spillway was assumed to have a 2 percent approach slope of at least 100 feet in length, a 50-foot flat approach section to the control section, and a supercritical 3 percent slope downstream of the control section. The rating curve for the auxiliary spillway was generated based on the guidelines of NRCS Technical Release 39, “Hydraulics of Broad-Crested Spillways” (TR-39). The location of the auxiliary spillway was placed on the right or east overbank based on topography, site effects, downstream effects, and constructability.

The auxiliary spillway crest elevation was established by rounding the peak stage obtained from the PSH event up to the nearest whole foot. After establishing the auxiliary spillway crest elevation, the ASH event for the respective storm area was routed. Adjustments were made to the auxiliary spillway width according to the peak stages obtained from the respective ASH events. A bottom width of 400 feet was evaluated for the Lillian Creek reservoir site. The bottom width was adjusted as required according to maximum permissible velocities set forth in TR-60 for vegetated earthen spillways. Key elevations and design parameters for the Lillian Creek reservoir preliminary design are summarized in Table 29.

An analysis of the auxiliary spillway stability and integrity was done using NRCS SITES Version 2005.1.3. The analysis was done to determine the effects of routing the ASH and FBH through the site.

Table 29. Key Elevations and Design Parameters for Normal Pool = 2,460 ft

Normal Pool Scenario		Normal Pool	Auxiliary Spillway (AS) Crest	ASH Event[1]	Top of Dam (TOD)[2]
Lillian Creek Reservoir	6-ft x 16-ft riser and 48-IN RCCP		400 ft-wide AS		
	Elevation, ft	2,460	2,476	2,478.6	2,485
	Surface Area, acres	325	679	750	950
	Storage Volume, acre-feet	4,710	12,760	14,440	20,080

1 Key elevations and design parameters for ASH event.

2 Key elevations and design parameters for corresponding TOD elevation. TOD elevations were established by rounding up the peak stage obtained from the FBH event to the nearest whole foot.

For the proposed Lillian Creek reservoir site, the normal pool elevation of 2,460 feet provides a pool area of approximately 325 acres and a storage volume of 4,710 acre-feet. A 400-foot-wide auxiliary spillway results in a TOD elevation of 2,485 feet, corresponding to maximum pool area of 950 acres and 20,080 acre-feet of total storage volume.

3.3.2.3 Top of Dam

The TOD elevation was established by rounding the peak stage obtained from the respective FBH event up to the nearest whole foot with a 400-foot-wide auxiliary spillway. The TOD elevation for a normal pool at 2,460 feet is 2,485 feet.

3.3.3 Pertinent Dam Data

3.3.3.1 Main Dam

With a normal pool elevation of 2,460 feet and a 48-inch RCPP, the 500-year event produced a reservoir stage of 2,475.3 feet, and the auxiliary spillway crest elevation was set at 2,476 feet. The P_{ASH} produced a stage of 2,478.6 feet, with the same normal pool. The P_{FBH} produced an elevation of 2,484.2 feet, and the TOD was set at 2,485 feet. Table 30 summarizes the dam design data, including embankment, spillway, and reservoir operations data for the proposed Lillian Creek Reservoir.

TR-60 criteria require a minimum difference in elevation between the crest of the auxiliary spillway and the settled TOD of 3 feet, which would be met.

Table 30. Dam Data Summary for Lillian Creek Reservoir (Normal Pool = 2,460 feet)

Analysis criteria	NRCS Technical Release 60 (TR-60)			
Drainage area	Approx. 50 mi^2 (32,000 acres)			
Normal pool surface area	325 acres			
Dam classification	High hazard			
Embankment				
Crest length	Approx. 1,200 ft			
Crest elevation	Approx. 2,485.0 ft (msl),			
Height	Approx. 65 ft above valley floor			
Type of fill	Rolled earth			
Auxiliary Spillway				
Type	Earth cut, vegetated			
Location	Right abutment			
Crest elevation	2,485 ft (msl)			
Bottom width	400 ft			
Crest length	50 ft			
Side slopes	Approx. 3H:1V			
Approach slope	2%			
Downstream slope	3%			
Principal Spillway				
Inlet type	6-ft x 16-ft concrete riser			
Elev. of principal outlet	2,460 ft (msl)			
Conduit type	Reinforced concrete pipe			
Conduit diameter	48-IN			
Stilling basin type	Saint Anthony Falls or United States Bureau of Reclamation (USBR) Impact Basin			
Reservoir – Operating at Normal Pool of 2,460 ft			Peak Discharge	
Type of Storage	***Peak Storage Vol. (acre-feet)***	***Elevation (ft, msl)***	***Inflow (cfs)***	***Outflow (cfs)***
Valley floor		Approx. 2,420		
Normal (multipurpose)	4,710	2,460.0		
PSH (500-year)	12,300	2,475.3	6,110	270
ASH	14,470	2,478.6	10,470	4,230
FBH (PMP)	19,290	2,484.2	32,550	28,330

3.3.4 Conclusion

The LLNRD Lillian Creek reservoir feasibility study included this hydrologic analysis. A HEC-HMS (Version 4.2.1) model was developed and used to design the proposed reservoir. The evaluation used NOAA Atlas 14 point precipitation rainfall data and the Nebraska Site Specific PMP rainfall depths.

The drainage area into the Lillian Creek reservoir is approximately 32,000 acres (50 square miles). The normal pool elevation was set at 2,460 feet for the analysis.

With a normal pool elevation of 2,460 feet and a 48-inch RCPP, the auxiliary spillway crest elevation was set at 2,476 feet and the TOD was set at 2,485 feet.

4.0 Feasibility

A benefit cost analysis (BCA) was performed to evaluate the benefits of the Lillian Creek Project Alternatives.

4.1 Economics, Costs, and Benefits – Cost Benefit Analysis

4.1.1 Benefit Cost Analysis Overview

The BCA for Lillian Creek used guidelines for conducting BCA consistent with best practices for water resources projects.

BCA is a logical, systematic approach to finding the optimum use of the society's scarce resources (measured in dollar terms whenever possible), involving comparison of two or more alternatives in achieving a specific objective under the given assumptions and constraints. It explicitly considers the value of resources employed and attempts to measure the private and social costs and benefits of a project to the community or economy. BCA takes a broad perspective, including, in principle, all benefits and costs to whomsoever they accrue, whenever they accrue (now or in the future), and wherever they accrue from the completion of a project.

The BCA conceptual framework quantifies the costs and benefits of each alternative in monetary terms. Benefits represent the extent to which society and economies affected by a project are made better off through lower costs, fewer damages, or enhancements. In principle, any net increase in well-being (as measured by the summation of individual and society well-being changes) is a good thing, even if some groups within society are made worse off. A project or proposal would pass the efficiency test if the benefits to some are large enough to compensate the losses of others. Finally, BCA is a forward-looking exercise, seeking to anticipate the well-being effects of a project or proposal over its entire life-cycle. Future well-being changes are weighed against today's changes through discounting, which is meant to reflect society's general preference for the present, as well as broader inter-generational concerns.

4.1.2 Discounting and Present Value

An inherent problem in any evaluation or decision analysis is the difficulty of making value comparisons among projects that are not measured in common units. For example, dollars spent today are not equal to dollars projected to be spent in 20 years. To account for this, all future costs are converted to present value costs through a process known as discounting, which shows what a dollar received in 20 years, for example, is worth today. Discounting is accomplished using a discount rate selected to represent the time value of money. For the Lillian Creek BCA, the recommended rate is the annual discount rate published in USACE Economic Guidance Memorandum (EGM) Federal Discount Rate, table: Federal Discount Rates for Project Formulation and Evaluation.[4] The EGM is updated annually; the current federal rates should be used. For 2018, the federally approved discount rate is 2.75 percent.

Benefits and costs are converted to present value using the following calculation:

$$PV = \frac{FV}{(1 + r)^n}$$

Where:
PV = present value of the cost or benefit
FV = the future value of the cost or benefit
r = the discount rate
n = the current time period in years

In a BCA framework where benefits and costs occur over the life of the project, total present value costs are obtained by summing the present value of each annual cost.

Finally, all present value benefits and costs are then converted to an annual average (equivalent annual) by using the discount factor to amortize the projects costs and benefits over a specified period of analysis.

4.1.3 Benefit Cost Analysis Metrics

Several metrics resulting from economic analysis are useful for decision-making and may be used to help select the best of many projects, or to prioritize several, from the LLNRD's perspective.

- **Benefit to Cost Ratio (BCR)**: Annualized benefits and costs are presented as a ratio with benefits as the numerator and costs as the denominator. A ratio greater than one (>1.0) indicates benefits exceed costs and the project is economically justifiable.
- **Net Benefits/Net Present Value (NPV)**: The net benefits, or net present value (NPV), is the difference between the annualized values of total benefits and costs of a project. If the NPV is positive, the benefits of the project exceed its expected costs and the alternative is desirable relative to the baseline condition. A project is economically justified if the average annual benefits exceeds the average annual costs over the life of the project.

[4] https://planning.erdc.dren.mil/toolbox/library/EGMs/EGM18-01.pdf

4.1.4 Lillian Creek Benefit Cost Analysis

The following assumptions were used for the BCA:

- All economic damages, benefits and costs use the base year 2017 dollars.
- A period of analysis of 50 years.
- Benefits and costs have been discounted to present value, following a rate of 2.75 percent
- All present value benefits and costs are converted to average annual using the discount rate and period of analysis.
- Construction of alternatives would be begin in the first year of the period of analysis and would follow a 1-year construction phase.
- Project benefits would accrue following the first year of construction and extend for 49 years until the end of the period of analysis.

The BCA considers three water supply alternatives:

- Lillian Creek Dam Only – The dam is filled through natural runoff.
- Lillian Creek Dam plus Middle Loup River water via a 100 cfs Pump Station
- Lillian Creek Dam plus Middle Loup River water via a 250 cfs Pump Station

Each of the water supply alternatives was evaluated under two operational objectives.

- Maximize recreation. This objective would fill the reservoir to the normal pool elevation (2,460 feet) with available water. An ancillary benefit would be water seepage increasing baseflow, which could be used for increased irrigated acres or to offset stream depletions (no summer irrigation/depletion releases)
- Maximize irrigation. This objective would release all stored water on June 1 of every year to increase the base flow in the Middle Loup River for the benefit of irrigation or offsetting potential future stream depletions. Because all water would be released, there would be no recreation benefits.

The economic evaluation was performed using the water demand scenario that included the consideration of instream flow, full hydro diversion, and full irrigation demand. This scenario represents the least amount of available water for the Project based on downstream demands. Utilizing this in the economic analysis yields a conservative estimate of project benefits.

The combination of water supply alternatives and operational objectives yields six alternatives for consideration. Each of the alternatives was evaluated for the calculated and observed seepage condition.

4.1.5 Alternative Costs

The Lillian Creek Dam is estimated to cost approximately $25.0 million, in 2017 dollars, and is shown Table 31 below. The cost includes dam construction, embankment mitigation for stability and seepage, and 25,000 per year operations and maintenance costs. The 100 cfs and 250 cfs pump stations are estimated to cost $9.2 million and $18.6 million, respectively, and includes the cost for pipes and pumps, and 50 years of operations and maintenance costs. The conceptual level costs for the dam and pump station are shown in Appendix C. The alternative costs are assembled by combining the dam and pump station costs. The costs for the three alternatives are shown in Table 32. The costs were converted to an annualized average cost by amortizing the costs over the 50-year period of analysis with the economic discount factor of 2.75 percent.

Table 31. Lillian Creek Cost Estimates (2017 $)

Item	Dam	100 CFS Pump Station	250 CFS Pump Station
First Costs			
Construction	$16,864,000	$6,178,000	$12,433,000
Contingency	$4,216,000	$1,854,000	$3,730,000
Planning, Engineering, and Design	$2,338,000	$964,000	$1,939,000
Construction Management	$1,558,000	$241,000	$485,000
Total First Costs	$24,976,000	$9,237,000	$18,587,000
Operations, Maintenance, Repair, Replacement Costs			
Annual Operations and Maintenance	$25,000	$26,000	$67,000
Repair/Replacement1	$0	$900,000	$2,300,000

1 Pumps have an operational life of 20 years. The economic analysis evaluation period will include 2 rounds of pump replacement at year 20 and year 40. The base cost estimate of $100,000 per pump was used as an estimate of future replacement costs.

Table 32. Lillian Creek Alternative Costs (2017 $)

Alternative	Total Present Value Cost	Annual Average Cost
Dam Only (Natural Flow)	$25,669,000	$950,800
Dam + 100 CFS Pump Station	$36,433,000	$1,349,500
Dam + 250 CFS Pump Station	$48,159,000	$1,783,900

4.1.6 Alternative Benefits

Benefits of the Project include both consumptive and non-consumptive uses including the availability of water to support additional irrigated acres downstream of the reservoir, non-motorized boating and fishing (reservoir) and the value of instream flows (downstream of the dam). There is limited information on the value of instream flows in Nebraska. As such, the BCA focuses on the benefits associated with recreation and the value of irrigated acres in the region.

Recreation benefits are estimated using a benefits transfer approach. Benefits transfer, is a method whereby existing estimates for demand and user value are applied to a new context. In this case, recreation values for boating and fishing as well as demand for water based recreation from comparable sites are applied to the Lillian Creek Project.

The value of irrigated acres is estimated using a per-acre value of irrigation and an estimate of irrigated acres supported by the Project. A per-acre value of irrigation can be estimated following the basic principles of capitalization of an asset; where in the value of productive agricultural land will capture the value of the available irrigation.

4.1.6.1 Value of Recreation

Economic value is the difference between the maximum amount a recreationist would be willing to pay to participate in a recreational activity and the actual cost of participating in that activity. This is often referred to by economists as consumer surplus or net economic value. Put simply, this is the value of a trip to an individual after all expenses have been paid. Typical benefit measurement techniques measure these values in ($/user day) for recreation day activities. For this study a value of consumptive recreation was used of $113/user day based on a review of regional recreation studies.

4.1.6.2 Recreation Demand

The second component of recreation benefits is an estimate of the net number of user days the project will produce. Once the project is built, the recreation enhancement benefits occur annually. A regional review was conducted to identify lakes of comparable size. Recreation visitation was then scaled according to pool size in acres. For this analysis, Wagon Train Lake (surface area 315 Acres, annual visitation 11,200) was selected. Table 33 lists the recreation benefits. There are essentially no recreation benefits for the irrigation operating condition because all stored water is released on June 1 of each year for this alternative.

Table 33. Recreation Benefits by Operating Objective (2017 $)

Alternative	Pool Size (Acres)	Annual Recreation Demand	Annualized Benefits
Maximize Recreation Objective			
Calculated Seepage Condition			
Dam Only (Natural Flow)	11.4	104	$11,700
Dam + 100 CFS Pump Station	44.5	506	$56,700
Dam + 250 CFS Pump Station	60.4	687	$76,900
Observed Seepage Condition			
Dam Only (Natural Flow)	32.3	294	$32,900
Dam + 100 CFS Pump Station	213.9	2434	$272,400
Dam + 250 CFS Pump Station	250.0	2845	$318,400

4.1.6.3 Value of Irrigation

The per-acre value of irrigation can be estimated using a measure of the annual return on production from land. This is done using cash rents for both dry land and irrigated farmland. The Department of Agricultural Economics at UNL publishes regional trends in agricultural land prices for the State of Nebraska. Using the 2017 report, the average cash rents per-acre for the central region is $88 and $230 for dryland and irrigated land, respectively. The difference between the two values ($142 per acre) is the per-acre value of irrigation. Based on this, the BCA assumes that each additional acre of cropland that could be irrigated would generate $142 in annual economic returns.

4.1.6.4 Irrigated Acres

Total annual irrigated acres were estimated using information from water budget calculations. Total acre-feet of allowable development were converted to following the equation:

> Allowable Irrigated Acres of Development = (Acre-feet of Allowable Development)/ (Net Irrigation Requirement)/ (Stream Depletion Factor at location of use)/(% of Depletions occurring during the peak season)

Acre-feet of allowable development are assumed to be the acre-feet of water estimated from annual seepage volumes. The net irrigation requirement was assumed to be 12 inches based on review of UNL irrigation requirements for corn. Stream depletion factor at location of use is assumed 50 percent because most of the well locations are farther away from the stream. Finally, the percentage of depletions occurring during the peak season are 30 percent because the analysis focuses on June, July, and August.

Irrigated acres were estimated for calculated and observed seepage conditions and following the most likely excess flow scenario. The resulting annual irrigated acres and benefits are shown in Table 34.

Table 34. Irrigation Benefits by Operating Objective (2017 $)

Alternative	Annual Irrigated Acres	Annualized Benefits
Maximize Recreation Objective		
Calculated Seepage Condition		
Dam Only (Natural Flow)	170	$23,800
Dam + 100 CFS Pump Station	1,340	$188,500
Dam + 250 CFS Pump Station	2,630	$369,900
Observed Seepage Condition		
Dam Only (Natural Flow)	130	$18,300
Dam + 100 CFS Pump Station	780	$109,800
Dam + 250 CFS Pump Station	920	$129,300
Maximize Irrigation Objective		
Calculated Seepage Condition		
Dam Only (Natural Flow)	570	$80,100
Dam + 100 CFS Pump Station	3,230	$454,300
Dam + 250 CFS Pump Station	5,360	$753,800
Observed Seepage Condition		
Dam Only (Natural Flow)	1,810	$254,500
Dam + 100 CFS Pump Station	20,290	$2,853,700
Dam + 250 CFS Pump Station	24,270	$3,413,400

The benefits are estimated by combining the value of per-acre value of irrigation with the total annual irrigated acres. The benefits accrue over 49 years following an assumed 1-year implementation phase. The annual stream of benefits are converted to present value and then amortized over 50 years to arrive at an annual average benefit.

4.1.7 Comparison of Alternatives

A summary of the costs and combined benefits—recreation and irrigation—associated with the alternatives are shown in Table 35.

Table 35. Benefit Cost Summary

Project Evaluation Metric	Dam Only (Natural Flow)	Dam + 100 CFS Pump Station	Dam + 250 CFS Pump Station
Maximize Recreation Objective			
Calculated Seepage Condition			
Annualized Benefits	$35,500	$245,200	$446,800
Annualized Costs	$950,800	$1,349,500	$1,783,900
Net Present Value	($915,300)	($1,104,300)	($1,337,100)
Benefit / Cost Ratio	0.0	0.2	0.3
Observed Seepage Condition			
Annualized Benefits	$51,200	$382,200	$447,700
Annualized Costs	$950,800	$1,349,500	$1,783,900
Net Present Value	($899,600)	($967,300)	($1,336,200)
Benefit / Cost Ratio	0.1	0.3	0.3
Maximize Irrigation Objective			
Calculated Seepage Condition			
Annualized Benefits	$80,100	$454,300	$753,800
Annualized Costs	$950,800	$1,349,500	$1,783,900
Net Present Value	($870,700)	($895,200)	($1,030,100)
Benefit / Cost Ratio	0.1	0.3	0.4
Observed Seepage Condition			
Annualized Benefits	$254,500	$2,853,700	$3,413,400
Annualized Costs	$950,800	$1,349,500	$1,783,900
Net Present Value	($696,300)	$1,504,200	$1,629,500
Benefit / Cost Ratio	0.3	2.1	1.9

The economic analysis revealed the following:

- The natural flow alternative is not cost effective (benefit cost ratio < 1.0).
- Operating the reservoir to maximize recreation benefits is not cost effective (benefit cost ratio < 1.0).
- Both operating alternatives involving pump stations are cost effective under the maximize irrigation operational alternative with observed seepage.
- Under the observed seepage condition the 100 cfs Pump Station operating alternative has the highest BCR and net benefits.

4.2 Optimization

In the evaluation of alternatives, the two operational alternatives (maximized recreation and maximized irrigation) are bookends of the potential operational ranges. Following the comparison of alternatives, operating to maximize recreation use was not economically efficient but operating to maximize irrigation would be. A subsequent analysis was conducted to identify an optimal range where the Project could deliver both recreation benefits and irrigation benefits. To do this, benefits were recomputed with the following assumptions:

- Benefits are evaluated under the low seepage condition only
- The dam would collect flows between October and June with summer peak season releases occurring for irrigation
- The irrigation releases would draw down the pool while taking into account seepage losses to a predefined elevation to maintain a specified pool surface area for summer recreation purposes.
- Irrigation benefits result from seepage and summer releases.
- Recreation benefits are evaluated with the optimized pool surface area.

Two pool elevations were evaluated, 2,440 feet and 2,448 feet. The resulting net storage time series plots for a poll elevation of 2,448 for each water supply alternative are shown in Figures 45-47. Similarly, the resulting net storage time series plots for a pool elevation of 2,440 feet for each water supply alternative are shown in Figures 48-50. The results of the optimization are shown in Table 36, Table 37. A summary table of all operational objectives is provided in Table 38.

Table 36. Recreation Benefits by Operating Objective (2017 $)

Alternative	Pool Size (Acres)	Annual Recreation Demand	Annualized Benefits
Optimized 2448 Elevation Pool Objective ~ 159 acres			
Dam Only (Natural Flow)	35	321	$35,900
Dam + 100 CFS Pump Station	165	1874	$209,800
Dam + 250 CFS Pump Station	190	2167	$242,500
Optimized 2440 Elevation Pool Objective ~ 90 acres			
Dam Only (Natural Flow)	35	321	$35,900
Dam + 100 CFS Pump Station	113	1283	$143,500
Dam + 250 CFS Pump Station	133	1512	$169,200

Table 37. Irrigation Benefits by Operating Objective (2017 $)

Alternative	Annual Irrigated Acres	Annualized Benefits
Optimized 2448 Elevation Pool Objective ~ 159 acres		
Dam Only (Natural Flow)	130	$18,300
Dam + 100 CFS Pump Station	6,030	$848,100
Dam + 250 CFS Pump Station	9,500	$1,336,100
Optimized 2440 Elevation Pool Objective ~ 90 acres		
Dam Only (Natural Flow)	130	$18,300
Dam + 100 CFS Pump Station	10210	$1,436,000
Dam + 250 CFS Pump Station	15430	$2,170,200

Table 38. Benefit Cost Summary

Project Evaluation Metric	Dam Only (Natural Flow)	Dam + 100 CFS Pump Station	Dam + 250 CFS Pump Station
Maximize Recreation Objective: Normal Pool at Elevation 2460 ~ 325 acres			
Annualized Benefits	$51,200	$382,200	$447,700
Annualized Costs	$950,800	$1,349,500	$1,783,900
Net Present Value	($899,600)	($967,300)	($1,336,200)
Benefit / Cost Ratio	0.1	0.3	0.3
Optimized Pool at Elevation 2448 Objective ~ 159 acres			
Annualized Benefits	$54,200	$1,057,900	$1,578,600
Annualized Costs	$950,800	$1,349,500	$1,783,900
Net Present Value	($896,600)	($291,600)	($205,300)
Benefit / Cost Ratio	0.1	0.8	0.9
Optimized Pool at Elevation 2440 Objective ~ 90 acres			
Annualized Benefits	$54,200	$1,579,500	$2,339,400
Annualized Costs	$950,800	$1,349,500	$1,783,900
Net Present Value	($896,600)	$230,000	$555,500
Benefit / Cost Ratio	0.1	1.2	1.3
Maximize Irrigation Objective			
Annualized Benefits	$254,500	$2,853,700	$3,413,400
Annualized Costs	$950,800	$1,349,500	$1,783,900
Net Present Value	($696,300)	$1,504,200	$1,629,500
Benefit / Cost Ratio	0.3	2.1	1.9

The optimization analysis revealed the following

- Increasing baseflow (for irrigation or to offset depletions) is the primary driver of economic benefit.
- Operating the reservoir to maximize recreation benefits is not cost effective (benefit cost ratio < 1.0).
- Balancing recreation benefits with increasing baseflow results in a cost effective project (BCR>1.0) for the water supply operation alternatives that include pumping excess Middle Loup River water. When balancing recreation benefits with increasing baseflow the operating alternatives that include the 100 and 250 CFS pump stations produce similar benefit cost ratios.
- When balancing recreation benefits with increasing baseflow, the operating alternative that includes the 250 CFS pump station has the greatest net benefits but achieves this with significantly greater cost.

4.3 Sensitivity Analysis

The economic evaluation presents a conservative analysis using the water demand scenario that included the consideration of instream flow, full hydro diversion, and full irrigation demand. A sensitivity analysis was conducted on the benefit cost analysis using the water demand scenario with instream flows historic hydro diversions, and irrigation. The results of the sensitivity analysis are shown in Table 39.

Table 39. Benefit Cost Summary – Sensitivity Analysis Results

Project Evaluation Metric	Dam Only (Natural Flow)	Dam + 100 CFS Pump Station	Dam + 250 CFS Pump Station
Maximize Recreation Objective ~ 325 acres			
Annualized Benefits	$110,900	$527,700	$533,000
Annualized Costs	$950,800	$1,349,500	$1,783,900
Net Present Value	($839,900)	($821,800)	($1,250,900)
Benefit / Cost Ratio	0.1	0.4	0.3
Optimized 2448 Elevation Pool Objective ~ 159 acres			
Annualized Benefits	$82,800	$2,357,300	$2,441,000
Annualized Costs	$950,800	$1,349,500	$1,783,900
Net Present Value	($868,000)	$1,007,800	$657,100
Benefit / Cost Ratio	0.1	1.7	1.4
Optimized 2440 Elevation Pool Objective ~ 90 acres			
Annualized Benefits	$82,800	$3,376,100	$3,436,500
Annualized Costs	$950,800	$1,349,500	$1,783,900
Net Present Value	($868,000)	$2,026,600	$1,652,600
Benefit / Cost Ratio	0.1	2.5	1.9

Project Evaluation Metric	Dam Only (Natural Flow)	Dam + 100 CFS Pump Station	Dam + 250 CFS Pump Station
Maximize Irrigation Objective			
Annualized Benefits	$1,168,700	$4,371,300	$4,371,300
Annualized Costs	$950,800	$1,349,500	$1,783,900
Net Present Value	$217,900	$3,021,800	$2,587,400
Benefit / Cost Ratio	1.2	3.2	2.5

As expected, the demand scenario that provides more water for maintaining a pool that provides recreation while also increasing baseflow provides an even greater benefits relative to cost.

Using the historic hydro demand scenario results in a BCR for the operational condition involving pumps being nearly two times that the demand scenario using full hydro operation.

5.0 Permitting

5.1 Environmental Permitting

The Project would require the collaboration with federal, state, and local agencies.

5.1.1 Federal Permitting

The Project would require a CWA Section 404 individual permit, issued by USACE with oversight by EPA, due to wetland impacts associated with the construction of the Project and the area of inundation behind the Project. A requirement of Section 404 permitting in adherence to the Section 404(b)(1) Guidelines (Guidelines), and specifically, an alternatives analysis. USACE, potentially the lead federal agency, is also required to comply with the National Environmental Policy Act (NEPA), Section 7 of the Endangered Species Act, and Section 106 of the Historic Preservation Act as part of their decision making.

Purpose and need are inter-dependent terms that are critical to an alternatives analysis. The need of a project is typically the problem or opportunity that is trying to be addressed with the project. The elements of project need are used to frame a project purpose. Several purposes have been defined for the Project. A critical first step is determining if a single Project purpose exists, or if multiple Project purposes are realistic. An alternatives analysis needs to be performed on alternatives that address each Project purpose independently. Multiple Project purposes add to the complexity of the alternatives analysis.

All alternatives that meet the Project purpose and need must then be analyzed for practicability. Three practicability tests are applied in accordance with the Guidelines. They are cost, logistics, and technology. An alternative needs only to fail one of these tests to be considered not practicable.

All practicable alternatives are then reviewed for effects on waters of the United States and other significant environmental effects. Only the practicable alternative that has the least impacts on waters of the United States or other significant environmental effects can be permitted by USACE. This is referred to as the Least Environmentally Damaging Practicable Alternative (LEDPA).

If the Project is determined to be the LEDPA, a plan to mitigate for unavoidable effects on waters of the United States would be required. Unavoidable effects on wetlands and stream channels would be required. Wetland and stream impacts include those that would be filled by the proposed reservoir. Wetland impacts would also include wetlands that are inundated by the resulting normal pool of the proposed reservoir. Mitigation requirements for effects on waters of the United States, as defined by the CWA, can be substantial in terms of both duration and cost.

As previously mentioned, as the potential lead federal agency, USACE is also responsible for compliance with other federal laws (in additional to NEPA). This includes Section 7 of the Endangered Species Act (ESA), Section 106 of the NHPA, the MBTA, and the Bald and Golden Eagle Protection Act.

Pending the level of potential effects on federally listed T&E species, the analysis required can vary. USACE may require LLNRD to develop a Biological Assessment (BA). The BA would be used to identify the effects on T&E species. If impacts are minimal, informal consultation with USFWS can be completed. If not, formal consultation would be required. Formal Section 7 consultation process can be very time-consuming (9 to 12 months) and includes a Biological Opinion prepared by USFWS.

At this time, seven species are in the range of the Project (T&E Species Habitat Review, Technical Memorandum, May 16, 2017). Some of these species are present specifically at the Project site (such as American burying beetle), and others are included due to the potential for downstream effects on Loup River flows. Additional protected species, such as the pallid sturgeon, may also require evaluation if it is determined that downstream effects on Loup River flows result in water depletions to the lower Platte River.

Relative to Section 106 of the NHPA, USACE must evaluate the effect of the Project on historic properties. USACE would review the Project and may coordinate with NESHPO relative to the potential for Project related effects on historic properties. HDR has performed a review of existing records (Cultural Resources Critical Issues Analysis, Technical Memorandum, May 12, 2017) and determined that there is a potential for undiscovered archaeological site and architectural properties to existing with the Study Area. USACE, through coordination with NESHPO, would determine if field surveys are needed to comply with Section 106 of the NHPA. Based on recent coordination with NESHPO on other projects, it should be expected that NESHPO would recommend surveys.

Informal consultation with USFWS would be recommended to ensure compliance with the MBTA and Bald and Golden Eagle Protection Act, Consultation would include contacting the Nebraska Ecological Services Field Office of USFWS to determine applicable measures to reduce effects on migratory birds and eagles, including whether incidental take permits are necessary and available under the MBTA and Bald and Golden Eagle Protection Act for a particular activity. The migratory bird species protected by the Act are listed in 50 CFR 10.13.

5.1.2 State Permitting

State-level permitting and approval components would be required for the execution of the Project. Responsibility for two of the permitting components, the CWA Section 401 Certification and the National Pollutant Discharge Elimination System (NPDES) Construction Storm Water General Permit, are delegated to the State of Nebraska by EPA. The remaining state-level permits and approvals are directly related to the allocation and use of waters of Nebraska, dam safety, and development within a floodplain. Processing time for the required state-level permits ranges from 4 week to 25 weeks.

The requirement for surface water appropriation are listed in Section 5.2.

5.1.3 Local Permitting

A Conditional Use Permit would need to be obtained from Custer County to authorize the Project prior to construction activities. Conditional Use Permits are obtained by presenting the Project to the Custer County Commission for review and, if the Project is approved by the County Commission, it would subsequently be forwarded to the Board of Supervisors for review and approval. State and federal permitting does not need to be completed prior to presentation of the Project to the Commission; however, Custer County Conditional Use Permits typically require that construction activity begin within 90 days of the issuance of the permit.

5.1.4 Permitting Approach

The Lillian Creek Canal was approved as part of the Milburn Diversion Dam/Sargent Canal Project in 1959. However, the approval of the Milburn Diversion Dam/Sargent Canal predates the establishment of the federal, and many of the State, environmental regulatory obligations that would need to be adhered to for project execution. Therefore the Project is considered a single and complete action within this section.

Permit sequencing is an important component for efficient Project execution. The proposed order of permit pursuit initiation and a brief rationale for the sequencing is provided below, where relevant. Refer to Table 40 for details on each permitting requirement.

Planning Phase

1) **NeDNR Permit to Appropriate Water & Change of Appropriated Water Application**: Acquiring guaranteed water for the Project should be initiated first as a key element of Project implementation.

2) **Clean Water Act Section 404 Permit:** This permit and associated obligatory reviews and consultations (NEPA, ESA Section 7 and NHPA Section 106) would be the most time consuming to obtain and should therefore be the first to be initiated. This permitting process would evolve into early stages of the design phase

3) **Clean Water Act Section 401 – Water Quality Certification**: Although the CWA Section 401 Certification is a state run program, it would be required for Section 404 permitting. Additionally, Project design and planning would need to comply with the Nebraska Surface Water Quality Standards to qualify for this permit, which may affect

Project design components. This certification is typically process concurrently with the Section 404 permit process, typically including a joint public notice.

Design Phase

1) **NeDNR Application for Approval of Plans for Dams**: The dam design would need to have certified adherence to the requirements of the Safety of Dams and Reservoirs Act in order to receive a Permit to Impound, the next permitting item in the sequence.

2) **NeDNR Permit to Impound Water – See Table 38**

3) **NeDNR Permit to Appropriate Water (Use Permit) – See Section 5.2**

4) **NeDNR Floodplain Development Permit – See Table 38**

5) **Custer County Planning and Zoning Commission - Conditional Use Permit**: Construction activity must begin within 30 days of issuance of this permit so, although coordination with the Planning and Zoning Commission would be critical early in the permitting process, the Conditional Use Permit should not be issued until all of the other project planning components have been completed.

6) **NPDES Construction Storm Water General Permit**: Some design features associated with the SWPPP required for the Project may be integrated into the Section 401 Certification (above) so concurrent development of these permit packages is acceptable.

5.1.5 Incidental Permitting

Additional state laws and regulations that may apply to the Project if specific cultural resources are discovered during construction include: The Nebraska Archaeological Resource Preservation Act (Nebraska Revised Statutes 82-501 through 82-510); the Unmarked Human Burial Sites and Skeletal Remains Protection Act (Statutes of Nebraska Article 12-1201 through 12- 1212), and the Human Skeletal Remains or Burial Goods Act (Statutes of Nebraska Article 28-1301). Refer to the Cultural Resources Critical Issues Analysis, in Section 2.3.3 for details on the applicability of these permit requirements.

5.2 Surface Water Appropriation

Water resources in Nebraska play a major role in the state's economy and government. Nebraska's state constitution specifies that Nebraska surface waters be governed by the Appropriative First-in-Time, First-in-Right Rule. This doctrine allows diversion of water from the surface waters of the state based on the date a water right was obtained. In Nebraska, surface water rights entitle land owners or entities to withdraw a set amount of water from a specific location for a specified use, for example irrigation.

The doctrine of prior appropriation protects those who received their water rights first during periods when the overall water supply is not sufficient to meet all appropriated water rights. This seniority system entitles an entity with a more senior right (that is, an earlier priority date, First-in-Time) to the water supply before an entity with a later priority date receives any water. A water right (or appropriation) is issued by NeDNR.

A surface water permit is a general term for a water appropriation that has been granted by the State of Nebraska for the diversion of surface water for irrigation, hydropower, industrial use, municipal use, domestic use, storage, and other uses as approved by NeDNR. Authority of the NeDNR also includes instream uses for recreation, fish and wildlife, induced ground water recharge for public water suppliers, and diversions by ground water irrigation wells located within 50 feet of the bank of the channel. Each permit has certain conditions, which may include limitations to the amount of use, location of use, time of use, or other conditions specified by NeDNR. Surface water permits are commonly referred to as appropriations or water rights and are typically attached to the land.

Any diversion of surface water from a natural stream, channel, canal, ditch, or reservoir most likely requires a surface water permit. Permits may be granted on a temporary basis if the diversion is to occur for no more than one calendar year. If you are storing water in a reservoir, a permit to impound surface water is required except when storing less than 15 acre-feet of surface water per annum and the impounded water is not diverted for any irrigation or any other purpose. Further exceptions are made for waste lagoons and described in Nebraska Revised Statutes 46-241.

A dam located on Lillian Creek would require a permit to store and use water from the facility. Also, if a diversion was sought for a Lillian Creek Canal, a permit to divert from the Middle Loup River would be necessary. Based on the preliminary dam sizing, an Approval of Dam Plans application would also be required by NeDNR. An application to obtain a surface water permit may be made using the forms prescribed and furnished by NeDNR. Permits that would be required from NeDNR include:

- Permit to Appropriate Water
- Permit to Impound Water
- Dam Safety form: Application for Approval of Plans for Dams

Other permits or considerations for the Project could include:

- Permit to Conduct Water in Stream Channels
- Permit to Appropriate Natural Flows for Induced Ground Water Recharge

An application for a new surface water permit cannot be filed in a river basin that has been declared fully or over appropriated by NeDNR until a variance petition is granted. The most recent Fully Appropriated Basin (FAB) analysis conducted by NeDNR as of December 2016, determined the Lower Platte Basin was not considered fully appropriated. The Lower Platte comprises the Loup River and its tributaries, including Lillian Creek. Approval of new surface water irrigated acres may be further limited by controls adopted though integrated water management planning in cooperation with local Natural Resources Districts.

The FAB reports notes that,

> There are no interstate compacts or decrees, or other formal state contracts or agreements in the Lower Platte River Basin that could be affected by reduced streamflows. There are state and federally endangered and threatened species in the Lower Platte River Basin. The requirements of the Nebraska Nongame and Endangered Species Conservation Act (NNESCA) and the federal Endangered Species Act (ESA) prevent actions that could cause harmful stream flow reductions. At this time, there is sufficient water supply in the basin to comply with NNESCA and the ESA. Because future development will be limited so as to continue compliance with NNESCA, the long-term surface water supply in the basin is sufficient. (NeDNR 2016)

The existing water supply of the Lower Platte Basin would suggest that application for diversion and storage of water out of the Middle Loup and the Lillian Creek are feasible. The application process to divert, store, and use water from these sources would be through NeDNR and while there is no guarantee that they would be granted, the Project concept does not present any obvious impediments.

Once the preliminary study is complete and the Project is more fully developed, the process for applying for permits can begin. That process would generally include the following:

- Meet with NeDNR to inform them of the Project location and general amounts and times of diverted water. At that time, any concerns or expected information that is be part of an application can be sought.
- Develop requirements that will be necessary as part of the permit.
- Submit application for water appropriation and storage to NeDNR. The date of the application will be the date of the appropriation if approved by NeDNR.
- Provide any supplemental information requested by NeDNR.