APPENDIX D

INVESTIGATION AND ANALYSIS REPORT

This Investigation and Analysis Report documents the evaluation of alternatives performed in support of the Pine Lake Watershed Plan and Environmental Assessment (EA). A variety of alternatives has been considered and evaluated for compatibility with the purpose and need for the project. Viable alternatives are identified and discussed in detail. The viable alternatives evaluated in the EA involve adding a new upstream retention basin to the Pine Lake watershed and replacing the existing Pine Lake outlet structure.

This report is organized as follows:

- Hydrologic and Hydraulic Modeling
- Red River Basin Flood Mitigation Strategies
- Identification of Alternatives
- Evaluated Alternatives
- Site Geology
- Sensitivity Analysis for Dam Height
- Breach Analysis and Hazard Classification
- Sedimentation
- Opinion of Probable Construction Costs
- Economics Evaluation
- Environmental Evaluation
- References

Additional information relevant to each of the sections provided in this report is available as part of the administrative record for the project.

1 Hydrologic and Hydraulic Modeling

Hydrologic and hydraulic models were developed to obtain an understanding of baseline existing conditions and to provide a means for evaluating and comparing potential alternatives for achieving the project goal of flood damage reduction.

1.1 Hydrologic Evaluation

Several Hydraulic Engineering Center-Hydrologic Modeling System (HEC-HMS) models and associated datasets have been developed by the U.S. Army Corps of Engineers (USACE) for the Red River Basin. The models have been used in the Red Lake River watershed for assessing the effectiveness of various flood mitigation projects. Modeling for the Pine Lake subwatershed was based on the Clearwater watershed HEC-HMS model. Modifications were incorporated into the Clearwater model for modeling at a smaller watershed scale. Modifications included subdividing subbasins into smaller drainages, updating times of concentration to match adjusted basin sizes, adding existing lake features, and making methodology changes as described in the following sections.

1.1.1 Rainfall Depths

The Minnesota NRCS field office has published National Engineering Handbook supplement MN650.290, which specifies the use of Atlas 14 Volume 8 rainfall data as a replacement for the previous Technical Publication 40. Hydrometeorological Report (HMR) No. 48 and Technical Release TR-60 were used to determine a probable maximum precipitation (PMP) in support of developing spillway hydrographs (for retention basin alternatives). Atlas 14 rainfall data were used for non-PMP scenarios.

1.1.2 Rainfall Distributions

National Engineering Handbook supplement MN650.290 specifies that the Midwest-Southeast (MSE) Distribution 3 is to be applied for hyetographs of 24 hours or less. As indicated in Merkel and Moody (2015), the MSE distributions are regionalized nested hyetographs developed from Atlas 14 data. The recommended MSE 3 is more intense than the previously used Soil Conservation Service (SCS) Type II distribution.

In accordance with NEH recommendations, the MSE 3 distribution was applied for scenarios of 24 hours or less. The hyetographs from TR-60 were applied for longer-duration scenarios. The TR-60 hyetographs for 10-day distributions have been a standard practice for flood management studies in the basin conducted by the Red River Basin Commission and regional watershed districts.

1.1.3 Hydrologic Scenarios

The following hydrologic scenarios were modeled for the Pine Lake watershed:

- MSE distributions have varying durations up to 24 hours. An assessment of critical duration for the Pine Lake watershed indicated that 24-hour duration storms resulted in the highest peak flow rates to Pine Lake.
- The 2-, 5-, 10-, 25-, and 50-year 24-hour events were used to determine inundation limits and to allow economic evaluation of alternatives.
- The 100-year (1% chance annual exceedance) 24-hour event was used in accordance with the National Watershed Program Handbook, Part 601.12.A.13.
- The 500-year (0.2% chance annual exceedance) 24-hour event was used in accordance with National Watershed Program Handbook, Part 601.12.A.13

- The 100-year 10-day event is a standard scenario used regionally in detention basin analysis. It is the TR-60 100-year 10-day runoff staged by the typical time of snow melt occurring in various watersheds.
- PMP 24-hour or spillway hydrographs were used in accordance with TR-60 and HMR 48.

1.2 Hydraulic Evaluation

Hydraulic modeling was performed to accomplish the following objectives:

- Evaluate peak flow rates and inundation limits associated with existing conditions.
- Evaluate peak flow rate and inundation limit changes attributable to implementation of alternatives.
- Evaluate breach inundation areas (and associated risks) from retention basin alternatives.

Hydraulic Engineering Center-River Analysis System (HEC-RAS) 2D version 5.03 was used as the basis for hydraulic modeling. HEC-HMS model hydrographs were used as input for the HEC-RAS 2D model.

1.2.1 Terrain Data

A combination of Light Detection and Ranging (LiDAR) data, aerial photographs, and survey data were used as the basis for terrain data as follows:

- Culvert/bridge crossings – opening sizes were estimated from aerial photographs using Pictometry software.
- Ground surface – LiDAR data was used to define the terrain used in the HEC-RAS 2D model
- Channel bathymetry – Survey was not conducted to evaluate the elevation of channel bottoms. It was assumed that the conveyance capacity of existing channels that is not represented in the LiDAR data set is not significant enough to warrant additional survey.
- Pine Lake survey – Existing structures (inverts of culverts and outlet channel cross section data) were surveyed in the Pine Lake system. Published bathymetry data (water depth information) was used to estimate lake-bottom elevations using the normal pool lake elevation as a basis.

1.2.2 Modeling Conditions

The downstream extent of modeling in the Pine Lake watershed was set at Anderson Lake, which is near the intersection of 139th Avenue and 530th Street. Anderson Lake is approximately 2.7 miles north of Gonvick, Minnesota, and approximately 5 miles north of Pine Lake. The termination of modeling at Anderson Lake was selected based on the limited impacts that Pine Lake watershed retention basins have on flow at this location. The additional drainage areas that enter the Lost River between the Pine Lake outlet structure and Anderson Lake have peak flow rates that exceed the peak flow rates leaving Pine Lake. As such, inundation limits downstream of Anderson Lake are controlled by the downstream watershed flows.

Pine Lake was modeled with normal water level as a starting condition. Lake elevations were determined by using the HEC-RAS 2D model, with verification for consistency through the HEC-HMS model. Channel water surface elevations or inundation areas were determined using the HEC-RAS 2D model.

2 Red River Basin Flood Mitigation Strategies

The Technical and Scientific Advisory Committee of the Red River Basin Flood Damage Reduction Work Group developed Technical Paper (TP) No. 11, which provides recommendations for locations and types of flood damage reduction measures that could be effective for meeting flood damage reduction goals in the greater Red River watershed. The technical paper indicates that the location of flood reduction measures in the watershed is especially critical in determining effectiveness because of influences on timing of flows entering the Red River. The committee evaluated topography and proximity to determine “Timing Zones” in the watershed. The Pine Lake watershed falls in the “Middle Zone,” but is close to the line between the “Middle” and “Late Zones.” Table 1 lists the flood reduction measures evaluated in the paper, potential application within the Pine Lake watershed, and potential effectiveness given that the Pine Lake watershed is in an area on the border of Middle and Late Timing Zones.

Table 1. Flood Reduction Measures Evaluated

Reduction Measure Type	Potential Applications within Pine Lake Basin	Appropriateness/Ranking (according to TP-11)
Reduction of flood volumes	Conversion of upland areas to alternative land use, creation of wetlands, or cropland best management practices (BMPs) to reduce downstream runoff volumes and rates	Substantial positive effects on downstream flooding
Increases to conveyance capacity	Creation of a flow bypass or increase in the flow capacity of the existing Pine Lake outlet channel to reduce flooding damages on Pine Lake	Likely negative impact on downstream flooding
Protection/avoidance	Evacuation of the floodplain or flood-proofing of structures	No impact or likely negative impact on downstream flooding
Temporary flood storage	Creation of detention areas to reduce downstream peak flow rates	Likely or substantial positive effects on downstream flooding

3 Identification of Alternatives

The following text discusses each of the flood mitigation strategies discussed in TP 11 and the potential application of these strategies within the Pine Lake watershed.

3.1 Wetland and Native Vegetation Restoration

TP 11 indicates that substantial positive effects on flood mitigation for the Red River could result from use of volume-reducing measures in the Pine Lake subwatershed. In the Pine Lake watershed, these measures could include approaches such as construction of wetlands, conversion of existing cropland and pasture land to forest, or implementation of cropland BMPs.

One method for evaluating land use changes was to determine the hydrologic and hydraulic impacts of reverting present-day land uses to 1939 conditions. Aerial imagery (USDA 1939) was obtained and georeferenced for the Pine Lake watershed. Based on the difficulty in interpreting wetland areas from the resolution of the photographs, all areas of native vegetation were digitized at a 1:3,000 scale. The native vegetation included both wetlands and forested areas. A similar digitization effort was performed from the U.S. Department of Agriculture (USDA) 2015 aerial photographs. The native vegetation areas from the 2015 photographs were removed from the 1939 dataset, leaving 3.1-square-miles of area that has been cultivated between 1939 and 2015. Figure 1 shows the location of these post-1939 modified areas within the watershed.

The potential impacts of changing these areas from cropland or pasture to wetlands were evaluated by adjusting curve numbers and times of concentration within the hydrologic model of the watershed. The results of the evaluation are shown in Table 2 and Table 3. Restoring historic vegetation based on land uses from 1939 would affect approximately 7% of the Pine Lake watershed. Peak flows and stages at Pine Lake would change by 1% or less. Because the flow reduction would be extremely small, and because it would be very disruptive to convert land use for 3.1 square miles within the watershed, this alternative was deemed to not meet the purpose and need of the project and was dismissed from further evaluation.

Another means for evaluating land use changes was to determine the potential impact of converting existing cropland and pasture to forest. While no specific conversion areas were identified in the Pine Lake watershed, a general conversion of cropland and pastures to forest was estimated with the hydrologic model of the watershed. Results are shown in Table 4. Modeling shows that converting 10% of the Pine Lake drainage area cropland and pastures to forest results in less than a 1% decrease in peak 100-year inflows and less than 0.01-foot change in peak Pine Lake water surface elevation. Converting all cropland and pasture to forest could result in a 5% decrease in peak 100-year inflows and 0.08-foot change in peak water surface elevations. The conversion of all cropland or pasture to forest would be extremely disruptive and was deemed to not meet the purpose and need for the project and was dismissed from further consideration. Implementation of BMPs on farmland would have a lesser impact on peak flow rates or Pine Lake water surface elevations than restoration of wetlands or conversion of cropland and pasture land to forest. Because those other alternatives were not sufficient for making an appreciable difference to downstream peak flow rates or lake elevations, implementation of BMPs was similarly dismissed from further consideration.

Approximately 64% of the watershed area above Pine Lake (28.6 square miles) consists of land uses other than pasture or farmland. The majority of this non-farmed/non-pasture area (26.6 square miles) is undeveloped. These undeveloped lands are predominately undeveloped and consist mostly of deciduous forest, wetlands, open water, evergreen forest, shrub, or grasslands. The deciduous forest, wetland, and open waters areas are already relatively small contributors of runoff in the watershed (these areas either have low curve numbers or are poorly draining). The shrub and grassland areas make up approximately 5% (2.3 square miles) of the area upstream of Pine Lake. Conversion of these undeveloped lands to a modified land use (such as conversion to forest) would

not have an appreciable impact to flow rates. Furthermore, it is assumed that changing these areas that are already in a relatively unmodified natural state would be environmentally disruptive. The conversion of non-farmed/non-pasture undeveloped areas was dismissed from further consideration.

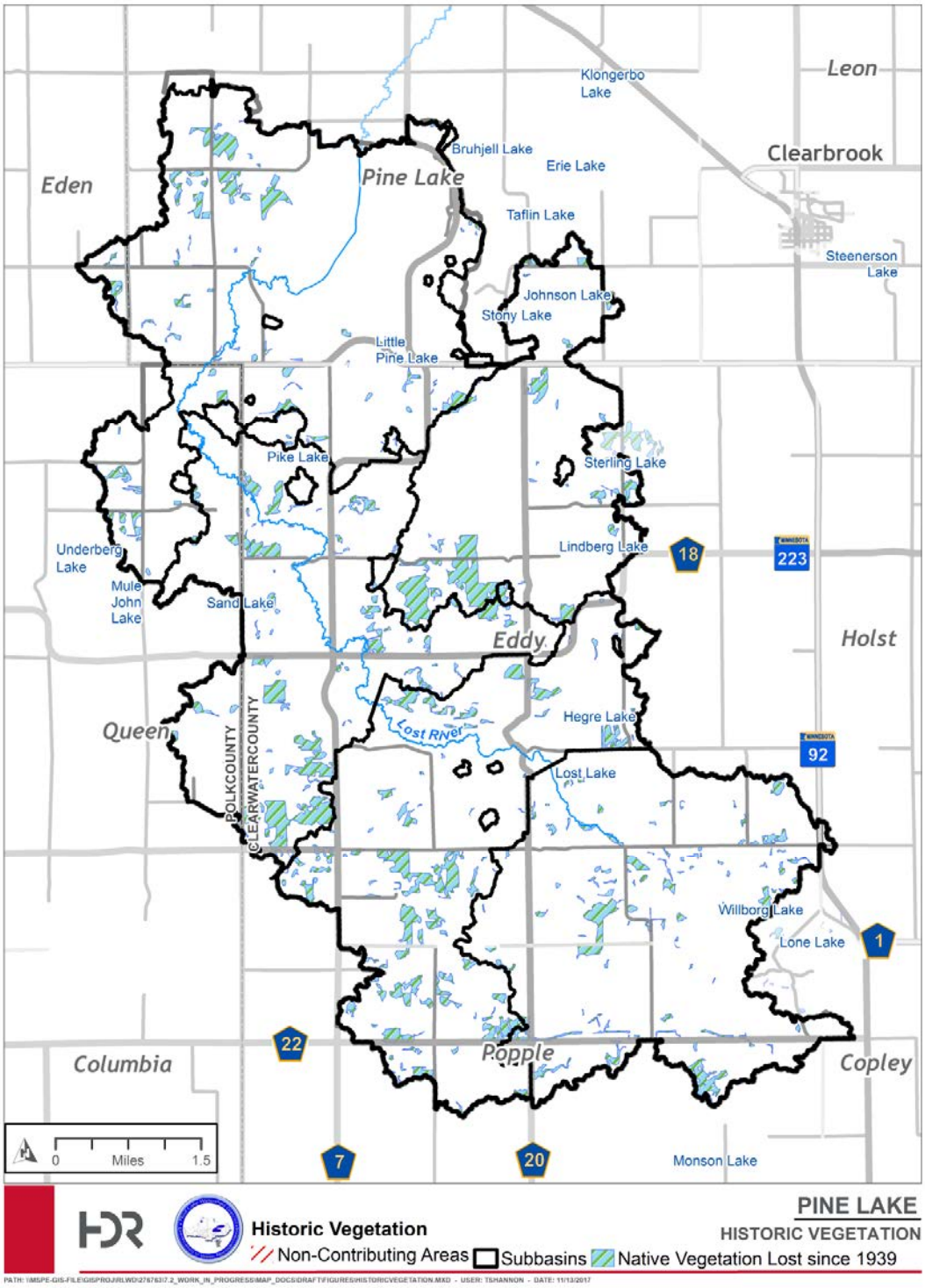

Figure 1. Assumed Wetland Conversion Areas

Table 2. Flow Rate Changes Resulting from Construction of Wetlands

Location	Existing Conditions 100-year flow rates (cubic feet per second)	Proposed Conditions 100-year flow rates (cubic feet per second)	Change (%)
Inflow to Pine Lake	832	825	0.8%
Flow out of Pine Lake to downstream reach	613	607	1.0%

Table 3. Pine Lake Water Surface Elevation Changes Resulting from Construction of Wetlands

Location	Existing Conditions (feet NGVD 29[a])	Proposed Conditions (feet NGVD 29)	Change (feet)
100-year Pine Lake water surface elevations	1,285.70	1,285.69	0.01

[a] National Geodetic Vertical Datum of 1929

Table 4. Pine Lake Impacts Attributable to Conversion of Cropland and Pastures to Forest

BMP and Farmland Conversion	Change in Pine Lake Peak Inflows (100-year 10-day) (%)	Change in Pine Lake Peak Water Surface Elevation (100-year 10-day) (feet)
10%	<1%	<0.01
50%	2%	0.04
100%	5%	0.08

3.2 Increases to Conveyance Capacity

Increases to Pine Lake outlet channel capacity were evaluated for the ability to reduce flooding elevations in Pine Lake. Potential means for increasing capacity could include:

- Performing channel maintenance (such as clearing and grubbing), planting, and managing vegetation within the downstream channel banks to reduce resistance to flow (reduce Manning's n values within the channel)
- Increasing the size of downstream culverts to allow water to flow more freely in the downstream channel without backing up at crossing locations
- Widening the channel to allow increased flows out of Pine Lake

Modeling results show that channel maintenance and vegetation management would have a limited ability to affect water surface elevations within Pine Lake (reductions in water levels of less than 0.1 feet). Modeling also showed that doubling the size of culverts in the channel downstream from Pine Lake would also result in reductions in water levels in Pine Lake of less than 0.1 feet. This modeling indicates that the downstream channel conveyance capacity would need to be substantially increased through channel widening to substantially affect upstream Pine Lake water levels.

Widening of the downstream channel would result in the following negative impacts:

- The existing channel width has been constant for many years, indicating that it is in equilibrium with natural conditions. Widening would likely encourage sedimentation and require continual maintenance to maintain the widened cross section.
- Increases in the downstream channel width would increase downstream peak flow rates and would likely require acquisition of easements or property and floodplain mapping.

According to the zoned mapping in TP 11, “likely negative impacts to downstream flooding” would occur in the Red River as a result of increasing conveyance capacities in the Pine Lake watershed. For the above-listed reasons, channel conveyance increases were deemed to not support the project purpose and need and were dismissed from further consideration.

3.3 Protection/Avoidance

The approach of protection or avoidance relies on removing assets from harm’s way, building barriers to floodwaters to protect assets, or protecting structures and other assets from floodwaters through elevation or protective measures. The application for this type of strategy in the Pine Lake watershed could consist of the following options:

- Buy-out of flood-threatened homes around Pine Lake (and subsequent conversion to unoccupied natural land)
- Construction of barriers between homes and the Pine Lake floodplain
- Raising of homes above the Pine Lake floodplain elevation
- Flood-proofing of structures at their current locations and elevations

Buy-out of homes would result in a fundamental change to property and lake use and would be very disruptive to residents. Construction of barriers between homes and Pine Lake would affect views of the lake, affect local drainage patterns, and fundamentally change the recreation landscape and the community’s access to the lake. Elevation or flood-proofing of homes could be technically accomplished, but would require use of specialty contractors. The cost of elevating or flood-proofing the existing houses could exceed the values of these homes, and this type of work would likely displace many long-time residents. The protection/avoidance strategy would not meet the purpose and need of the project because so many properties would need to be acquired and removed; the fundamental long-term uses of properties and the lake would be forever changed. Furthermore, protection/avoidance would not allow realization of secondary project benefits. As such, this alternative has been dismissed from further consideration.

3.4 Temporary Flood Storage

TP 11 indicates that temporary flood storage within the Pine Lake watershed would have potential to provide flow reduction benefits to the Red River. Temporary flood storage could be managed (capture and releases could be timed) to strategically contribute to downstream Red River flow reduction goals. Temporary flood storage could also provide flow reduction benefits within the Pine Lake watershed such as reducing inundation areas within the Lost River floodplain or reducing water levels within Pine Lake. The “Feasibility Report – Pine Lake Subwatershed Plan and Environmental Assessment,” dated August 2016, determined that temporary flood storage would be effective in providing flood reduction benefits in the Pine Lake watershed. The Feasibility Report documents an evaluation of seven potential retention basin locations and improvements to the Pine Lake outlet structure. The temporary storage options deemed to be viable for fur further evaluation included the following:

- Retention Basins D and E (new basin locations)
- Pine Lake outlet modification

As documented in the Feasibility Report, the Pine Lake outlet structure's ability to modify upstream water surface elevations is limited because of the downstream Lost River channel capacity. Although construction of a new outlet structure would allow realization of some of the secondary benefits identified in this project purpose and need (such as allowing water level management for wildlife benefits and maintenance of summer and fall water levels for recreational purposes), the outlet would not address the primary purpose and need for action (providing flood protection benefits at Pine Lake).

For these reasons, modification of the Pine Lake outlet structure will be evaluated only in combination with Retention Basins D and E. Temporary flood storage, through implementation of Retention Basins D and E, with and without Pine Lake outlet structure modifications, were carried forward for detailed evaluation.

Subsequent to completion of the Feasibility Report, an additional alternative of removing the Pine Lake outlet structure was brought forward for consideration. Elimination of the weir across the outlet channel would result in a permanently lowered normal lake level. This would have the effect of lowering peak flood elevations in the lake and providing additional storage volume for attenuating flows downstream from the lake. Results of the modeling to evaluate this alternative are summarized in Table 5. As shown in Table 5, the reductions to water surface elevations in Pine Lake would be limited to 0.1 feet for a 100-year 10-day event and 0.2 feet for a 100-year 24-hour event. Water surface elevation decreases are slightly larger for smaller more frequent events.

Although some flood protection benefits could be realized through this alternative, those benefits would come at a cost of fundamentally changing the use and value of the lake for all of the home owners around the lake. For these reasons, the alternative of removing the Pine Lake outlet structure was eliminated from further consideration.

Table 5 Effects of Removing the Pine Lake Outlet Structure

Event	Peak Flow Rate Downstream of Pine Lake			Pine Lake Water Surface Elevation (NGVD 29)			Structures Removed from Floodplain
	Existing	No Weir	Difference	Existing	No Weir	Difference	
100-yr 10-hr	539	436	-103	1287.9	1287.8	-0.1	5
100-yr 24-hr	319	175	-144	1286.8	1286.6	-0.2	0
50-yr 24-hr	237	106	-132	1286.4	1286.2	-0.2	17
25-yr 24-hr	175	58	-117	1286.0	1285.6	-0.4	8

4 Evaluated Alternatives

The following alternatives were advanced for detailed evaluation:

- No Action/Future Without Project (FWoP)
- Retention Basin D
- Retention Basin D and Pine Lake Outlet Structure
- Retention Basin E
- Retention Basin E and Pine Lake Outlet Structure

Figure 2 shows the location of the alternative features within the Pine Lake watershed.

4.1 No Action/Future Without Project

Under the No-Action/ FWoP scenario, no retention basins would be installed in the upper portions of the Pine Lake watershed and no modifications would be performed at the Pine Lake outlet. Flooding damages would continue to occur in the area of concern. Operation of the existing Pine Lake outlet structure would continue to require wading into the stream to install and remove stop logs. At some point in the future, the existing Pine Lake outlet structure would need to be replaced because of degradation of the steel sheet piles. Sediment would continue to move through the watershed as it does under existing conditions with continued accumulation within Pine Lake.

There is a Minnesota Department of Natural Resources (DNR)-managed wetland that stores stormwater runoff within the Pine Lake watershed. This storage area is referred to as Site F in the previously completed Feasibility Study. Modification of the Site F outlet structure is being evaluated as part of a separate project. An operation plan for the new Site F structure has already been adopted by the Minnesota DNR. Given the planned implementation of Site F improvements and progress made to date on design and operational agreements, Site F improvements are included in the FWoP scenario.

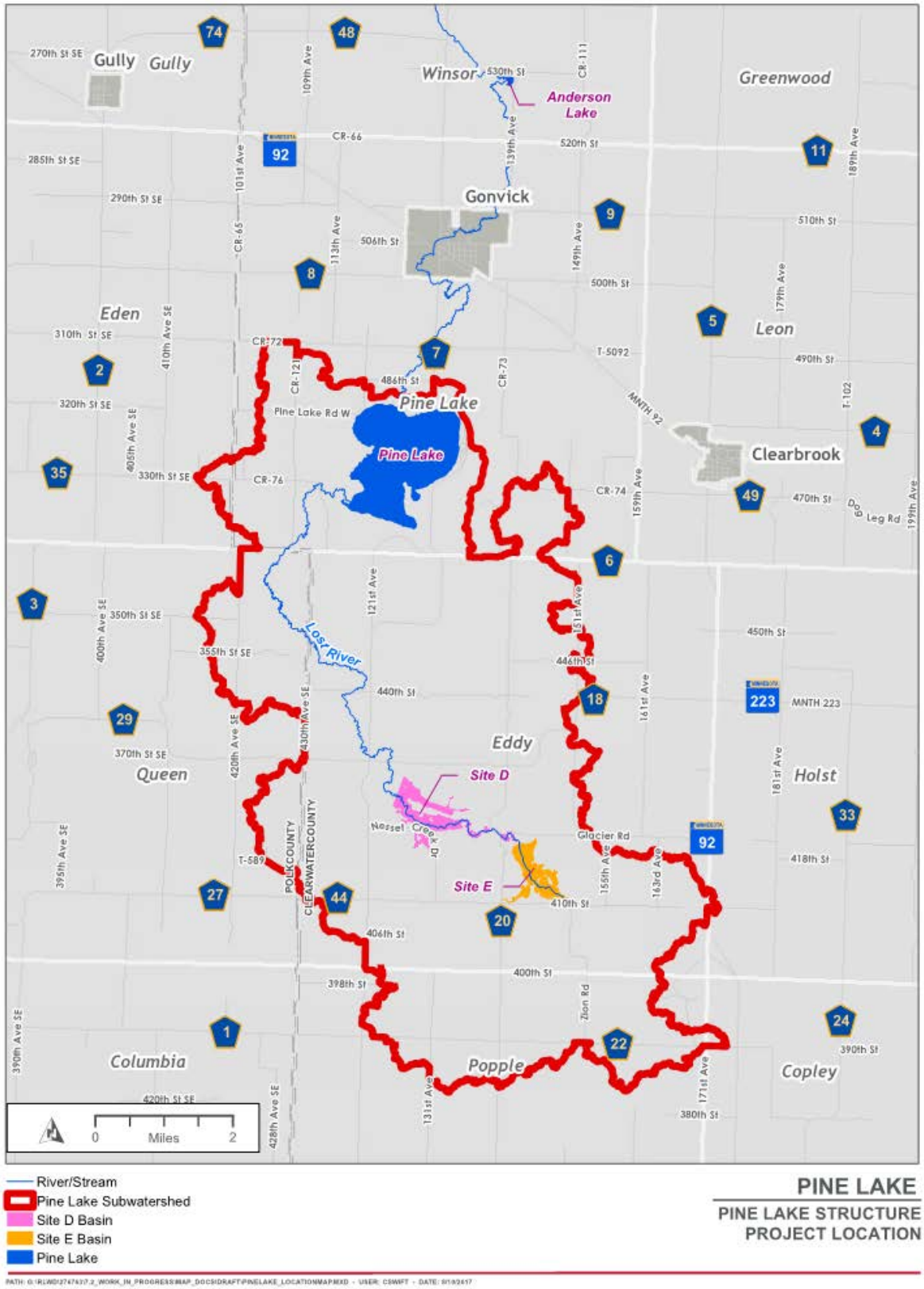

Figure 2. Retention Basin Locations

4.2 Retention Basin D

4.2.1 Layout of Dam

This alternative consists of creating approximately 2,511 acre-feet of storage upstream of an earthen embankment constructed across the floodplain. Figure 2 shows the location of the dam. Table 6 summarizes the dam characteristics. The total drainage area flowing to Pine Lake is approximately 45 square miles. The Retention Basin D structure would capture water from approximately 18.5 square miles (or 41% of the total area draining to Pine Lake).

Table 6. Summary of Retention Basin D Characteristics

Description	Value[a]
Drainage area	18.5 square miles
Top of dam elevation	1,363.0 feet
Top width of dam	20 feet
Invert of channel at base of dam	1,328.6 feet
Height of dam	34.4 feet
Side slopes	5H:1V (wet side) 4H:1V (dry side)
Impounded volume to top of principal spillway	2,142 acre-feet
Hazard class	significant
Principal spillway data: two-stage inlet, circular conduit	
Diameter of conduit	84 inches
Length of conduit	350 feet
Orifice invert elevation	1,328.6 feet
Riser weir crest elevation	1,357.6 feet
Auxiliary spillway data	
Crest elevation	1,360.0feet
Side slopes	3H:1V
Bottom width	100 feet
Maximum water surface elevations (24-hour storm)	
Principal spillway hydrograph	1,359.4 feet
Stability design hydrograph	1,360.0 feet
Freeboard hydrograph	1,361.0 feet

[a] Elevations are in NGVD 29 vertical datum.

The design concept for this structure is based on NRCS criteria as described in TR-60. Based on a breaching evaluation (as discussed in detail later in the report), the dam would meet “significant hazard” conditions. Under “normal” non-flood conditions throughout most of each year, the dam would allow continuous passage of flows through a low-flow opening in the principal outlet structure. This water would be conveyed from the principal outlet structure to the downstream side of the embankment by a pipe. A manually operated slide gate would allow closure of the low-flow opening to begin impoundment of watershed runoff. The principal outlet structure would consist of a concrete riser built into the embankment with a weir overflow and trash rack that would generally be in

accordance with standard NRCS dam outlet structures. An earthen spillway, protected against scour with vegetation and riprap, would be built in the hillside north of the dam embankment. Conceptual drawings for Retention Basin D are provided in Appendix C.

Modeling to define the principal outlet structure and auxiliary overflow spillway was conducted using HEC-HMS. The dam features for Retention Basin D were designed for the following conditions:

- Low-flow opening: Size to allow drawdown of water from principal spillway hydrograph to dry condition within less than 10 days.
- Principal spillway: Position elevation of weir to maximize storage potential in the basin. Size weir capacity to accommodate passage of flows up to the 100-year event with no flow overtopping the auxiliary spillway.
- Auxiliary spillway: Size the spillway so that it is stable and maintains integrity for an “auxiliary spillway hydrograph” and a “freeboard hydrograph” while maintaining at least 3 feet of freeboard to the top of dam.

4.2.2 Downstream Inundation Impacts

Construction of Retention Basin D would reduce downstream peak flow rates and decrease Pine Lake water surface elevations. Table 7 and Table 8 describe the impacts.

Table 7. Peak Flow Rate Changes with Retention Basin D

Location	Peak Flow Rates (cubic feet per second)							
	24-hour Event						10-day Event	
	10-year		50-year		100-year		100-year	
	Existing Condition	With Dam	Existing Condition	With Dam	Existing Condition	With Dam	Existing Condition	With Dam
Downstream face of dam								
Peak Flow Rate	270	150	583	302	731	391	761	594
Difference	–120		–281		–340		–167	
Difference %	–44%		–48%		–46%		–22%	
Entering Pine Lake								
Peak Flow Rate	242	141	454	268	584	331	695	471
Difference	–101		–186		–253		–224	
Difference %	–42%		–41%		–43%		–32%	
Leaving Pine Lake								
Peak Flow Rate	125	78	252	153	330	239	529	453
Difference	–47		–99		–91		–76	
Difference %	–38%		–39%		–28%		–14%	

Table 8. Pine Lake Water Surface Elevation Changes with Retention Basin D

	24-hour Event						10-day Event	
	10-year		50-year		100-year		100-year	
	Existing Condition	With Dam	Existing Condition	With Dam	Existing Condition	With Dam	Existing Condition	With Dam
Lake elevation (feet NGVD29)	1,284.4	1,283.9	1,285.1	1,284.5	1,285.5	1,285.0	1,286.7	1,286.6
Difference (feet)	–0.5		–0.6		–0.5		–0.1	

4.3 Retention Basin D and Pine Lake Outlet Structure

This alternative includes a combination of an upstream Retention Basin D and a new Pine Lake outlet structure. The retention basin would remain unchanged from the basin that is part of the stand-alone Retention Basin D alternative.

The existing Pine Lake outlet control structure consists of a sheet pile weir with two adjustable stop log bays. The top of weir varies between 1,283.5 and 1,284.5 feet. A small notch with an invert elevation of 1,282.6 feet is located in the center of the weir to allow for continuous flow downstream when there are lower lake elevations. The outlet structure is located in the outlet channel approximately 700 feet north of the lake.

The proposed replacement structure would be a concrete weir with one stop log bay and a gated opening. An elevated walkway would allow dry access to the control structure. Conceptual drawings for the outlet structure are provided in Appendix C.

Under normal summer low-flow conditions, the existing ponded water elevation is consistent across all basins at an elevation of approximately 1,283.5 feet. For the purposes of conservative analysis, it is assumed that the proposed outlet configuration will result in a water surface elevation of 1,284.0 feet. Minor adjustments could be warranted during design or be undertaken in the field through gate operations. Evaluation of alternatives for a Pine Lake outlet structure concluded the following:

- Changes to the width of the weir structure would not have an appreciable impact on water surface elevations because the conveyance capacity of the lake system is controlled by the capacity of the downstream river reach.
- A slide gate width of 5 feet was selected as being able to provide a reasonable balance between the cost of the gate and operational flexibility for allowing drawdown of water surface elevations.
- Varying top of weir elevations results in corresponding variations in lake water surface elevations without appreciably changing downstream peak flow rates.

The combination of constructing Retention Basin D and a new Pine Lake outlet structure would reduce downstream peak flow rates and decrease Pine Lake water surface elevations. Table 9 and Table 10 describe the impacts.

Table 9. Peak Flow Rate Changes with Retention Basin D and Pine Lake Outlet Structure

Location	Peak Flow Rates (cubic feet per second)							
	24-hour Event						10-day Event	
	10-year		50-year		100-year		100-year	
	Existing Condition	With Basin D and Outlet Structure	Existing Condition	With Basin D and Outlet Structure	Existing Condition	With Basin D and Outlet Structure	Existing Condition	With Basin D and Outlet Structure
Downstream face of dam								
Peak Flow Rate	270	150	583	302	731	391	761	594
Difference	–120		–281		–340		–167	
Difference %	–44%		–48%		–46%		–22%	
Entering Pine Lake								
Peak Flow Rate	242	141	454	268	584	331	695	471
Difference	–101		–186		–253		–224	
Difference %	–42%		–41%		–43%		–32%	
Leaving Pine Lake								
Peak Flow Rate	125	72	252	141	330	191	529	436
Difference	–53		–111		–139		–93	
Difference %	–42%		–44%		–42%		–18%	

Table 10. Pine Lake Water Surface Elevation Changes with Retention Basin D and Pine Lake Outlet Structure

	24-hour Event						10-day Event	
	10-year		50-year		100-year		100-year	
	Existing Condition	With Basin D and Outlet Structure	Existing Condition	With Basin D and Outlet Structure	Existing Condition	With Basin D and Outlet Structure	Existing Condition	With Basin D and Outlet Structure
Lake elevation (feet NGVD29)	1,284.4	1,284.5	1,285.1	1,285.0	1,285.5	1,285.2	1,286.7	1,286.4
Difference (feet)	0.1		–0.1		–0.3		–0.3	

4.4 Retention Basin E

4.4.1 Layout of Dam

This alternative consists of creating approximately 2,515 acre-feet of storage upstream of an earthen embankment constructed across the floodplain. Figure 2 shows the location of the dam. Table 11 summarizes the dam characteristics. The Retention Basin E structure would capture water from approximately 9.5 square miles (or 21% of the 45-square-mile area draining to Pine Lake).

Table 11. Summary of Retention Basin E Characteristics

Description	Value[a]
Drainage area	9.5 square miles
Top of dam elevation	1,393.2 feet
Invert of channel at base of dam	1,359.6 feet
Height of dam	33.6 feet
Side slopes	5H:1V (wet side) 4H:1V (dry side)
Impounded volume to top of principal spillway	2,515 acre-feet
Hazard class	significant
Principal spillway data: two-stage inlet, circular conduit	
Diameter of conduit	84 inches
Length of conduit	353 feet
Orifice invert elevation	1,359.6 feet
Riser weir crest elevation	1,389.5 feet
Auxiliary spillway data	
Crest elevation	1,390.7 feet
Side slopes	3H:1V
Bottom width	100 feet
Maximum water surface elevations (24-hour storm event)	
Principal spillway hydrograph	1,389.9 feet
Stability design hydrograph	1,390.7 feet
Freeboard hydrograph[b]	1,391.2 feet

[a] Elevations are in NGVD29 vertical datum.
[b] Freeboard hydrograph maximum water surface elevation set based on local roadway flooding; 410th Street floods at 1,391.2 feet NGVD29.

The design concept for this structure is based on NRCS criteria as described in TR-60. Based on a breaching evaluation (as discussed in detail later in the report), the dam would meet “significant hazard” conditions. Under “normal” non-flood conditions throughout most of each year, the dam would allow continuous passage of flows through a low-flow opening in the principal outlet structure. This water would be conveyed from the principal outlet structure to the downstream side of the embankment by a pipe. A manually operated slide gate would allow closure of the low-flow opening to begin impoundment of watershed runoff. The principal outlet structure would consist of a concrete

riser built into the embankment with a weir overflow and trash rack that is generally in accordance with standard NRCS dam outlet structures. An earthen spillway, protected against scour with vegetation and riprap, would be built in the hillside north of the dam embankment. Conceptual drawings for Retention Basin E are provided in Appendix C.

Modeling to define the principal outlet structure and auxiliary overflow spillway was conducted using HEC-HMS. The dam features for Basin E are designed for the following conditions:

- Low-flow opening: Size to allow drawdown of water from principal spillway hydrograph to dry condition in less than 10 days.
- Principal spillway: Position elevation of weir to maximize storage potential in the basin. Size weir capacity to accommodate passage of flows up to the 100-year event with no flow overtopping the auxiliary spillway.
- Auxiliary spillway: Size the spillway so that it is stable and maintains integrity for an “auxiliary spillway hydrograph” and a “freeboard hydrograph” while maintaining at least 3 feet of freeboard to the top of dam.

4.4.2 Downstream Impacts

Construction of Retention Basin E would reduce downstream peak flow rates and decrease Pine Lake water surface elevations. Table 12 and Table 13 describe the impacts.

Table 12. Peak Flow Rate Changes with Retention Basin E

Location	Peak Flow Rates (cubic feet per second)							
	24-hour Event						10-day Event	
	10-year		50-year		100-year		100-year	
	Existing Condition	With Dam	Existing Condition	With Dam	Existing Condition	With Dam	Existing Condition	With Dam
Downstream face of dam								
Peak Flow Rate	270	216	583	447	731	595	761	595
Difference	–54		–136		–136		–166	
Difference %	–20%		–23%		–19%		–22%	
Entering Pine Lake								
Peak Flow Rate	242	179	454	376	584	482	695	540
Difference	–63		–78		–102		–155	
Difference %	–26%		–17%		–17%		–22%	
Leaving Pine Lake								
Peak Flow Rate	125	114	252	211	330	274	529	427
Difference	–11		–41		–56		–102	
Difference %	–9%		–16%		–17%		–19%	

Table 13. Pine Lake Water Surface Elevation Changes with Retention Basin E

	24-hour Event						10-day Event	
	10-year		50-year		100-year		100-year	
	Existing Condition	With Dam	Existing Condition	With Dam	Existing Condition	With Dam	Existing Condition	With Dam
Lake elevation (feet NGVD29)	1,284.4	1,284.2	1,285.1	1,284.8	1,285.5	1,285.1	1,286.7	1,286.1
Difference (feet)	–0.2		–0.3		–0.6		–0.6	

4.5 Retention Basin E and Pine Lake Outlet Structure

This alternative includes a combination of an upstream Retention Basin E and a new Pine Lake outlet structure. The retention basin would remain unchanged from the basin that is part of the stand-alone Retention Basin E alternative.

The Pine Lake outlet replacement structure would be the same as the structure proposed for the alternative that combines Retention Basin D and a Pine Lake outlet structure. The combination of constructing Retention Basin E and a new Pine Lake outlet structure would reduce downstream peak flow rates and decrease Pine Lake water surface elevations. Table 14 and Table 15 describe the impacts.

Table 14. Peak Flow Rate Changes with Retention Basin E and Pine Lake Outlet Structure

Location	Peak Flow Rates (cubic feet per second)							
	24-hour Event						10-day Event	
	10-year		50-year		100-year		100-year	
	Existing Condition	With Basin E and Outlet Structure	Existing Condition	With Basin E and Outlet Structure	Existing Condition	With Basin E and Outlet Structure	Existing Condition	With Basin E and Outlet Structure
Downstream face of dam								
Peak Flow Rate	270	216	583	447	731	595	761	595
Difference	–54		–136		–136		–166	
Difference %	–20%		–23%		–19%		–22%	
Entering Pine Lake								
Peak Flow Rate	242	179	454	376	584	482	695	540
Difference	–63		–78		–102		–155	
Difference %	–26%		–17%		–17%		–22%	
Leaving Pine Lake								
Peak Flow Rate	125	94	252	192	330	248	529	425
Difference	–31		–111		–82		–104	
Difference %	–25%		–44%		–25%		–20%	

Table 15. Pine Lake Water Surface Elevation Changes with Retention Basin E and Pine Lake Outlet Structure

	24-hour Event						10-day Event	
	10-year		50-year		100-year		100-year	
	Existing Condition	With Basin D and Outlet Structure	Existing Condition	With Basin D and Outlet Structure	Existing Condition	With Basin D and Outlet Structure	Existing Condition	With Basin D and Outlet Structure
Lake elevation (feet NGVD29)	1,284.4	1,284.6	1,285.1	1,285.2	1,285.5	1,285.5	1,286.7	1,286.3
Difference (feet)	0.2		0.1		0.0		–0.4	

5 Site Geology

A desktop study of site geology and preliminary geologic field investigation was conducted for the proposed retention basins within the upper Pine Lake watershed and the proposed control structures at the outlet of Pine Lake to allow planning-level design evaluation of alternatives. A supplemental geologic field investigation will be required during the final design phase.

Based on a review of USDA data and the Fosston Quad geologic map, the site stratigraphy within the Pine Lake watershed generally appears to be glacial clay (generally low-plasticity clay) with variable zones of sand and gravel. Additionally, isolated geological zones with greater than 3 feet of surficial organic soils (described as thick recent bog sediment) were noted within the watersheds, particularly at Sites D and E. Bedrock is anticipated to be at a significant depth (> 200 ft) below the ground surface.

The Pine Lake subwatershed is located in the Moraine geomorphic province. The area is underlain by glacial deposits from the Wisconsin Glacial Episode between 85,000 and 11,000 years ago (Harris 2007). The Wisconsin Glacial Episode was the last major advance of continental glaciers in the North American Laurentide ice sheet. Bedrock in this area is granitic rock of Precambrian age.

The northern portion of the subwatershed is part of the Des Moines Lobe – Erskine Moraine of Late Wisconsinan age. It is composed of calcareous till and clayey in texture (Leverett 1932) (see Figure 3). The moraine generally consists of small sharp hummocks, from 10 feet or less to 25 feet or more in height. A few knolls reach heights of 50 to 60 feet. There are also some water-laid portions of this moraine, which are nearly free of hummocks, with smooth ridges and gentle slopes.

The central and southern portion of the subwatershed (upper portion of the Lost River headwaters) is within the Big Stone moraine of the Des Moines Lobe. The lithology of the Big Stone is primarily calcareous till with common glacial stones and also clayey in nature. The topography of the moraine is made up of hills and depressions with local relief up to 150 feet.

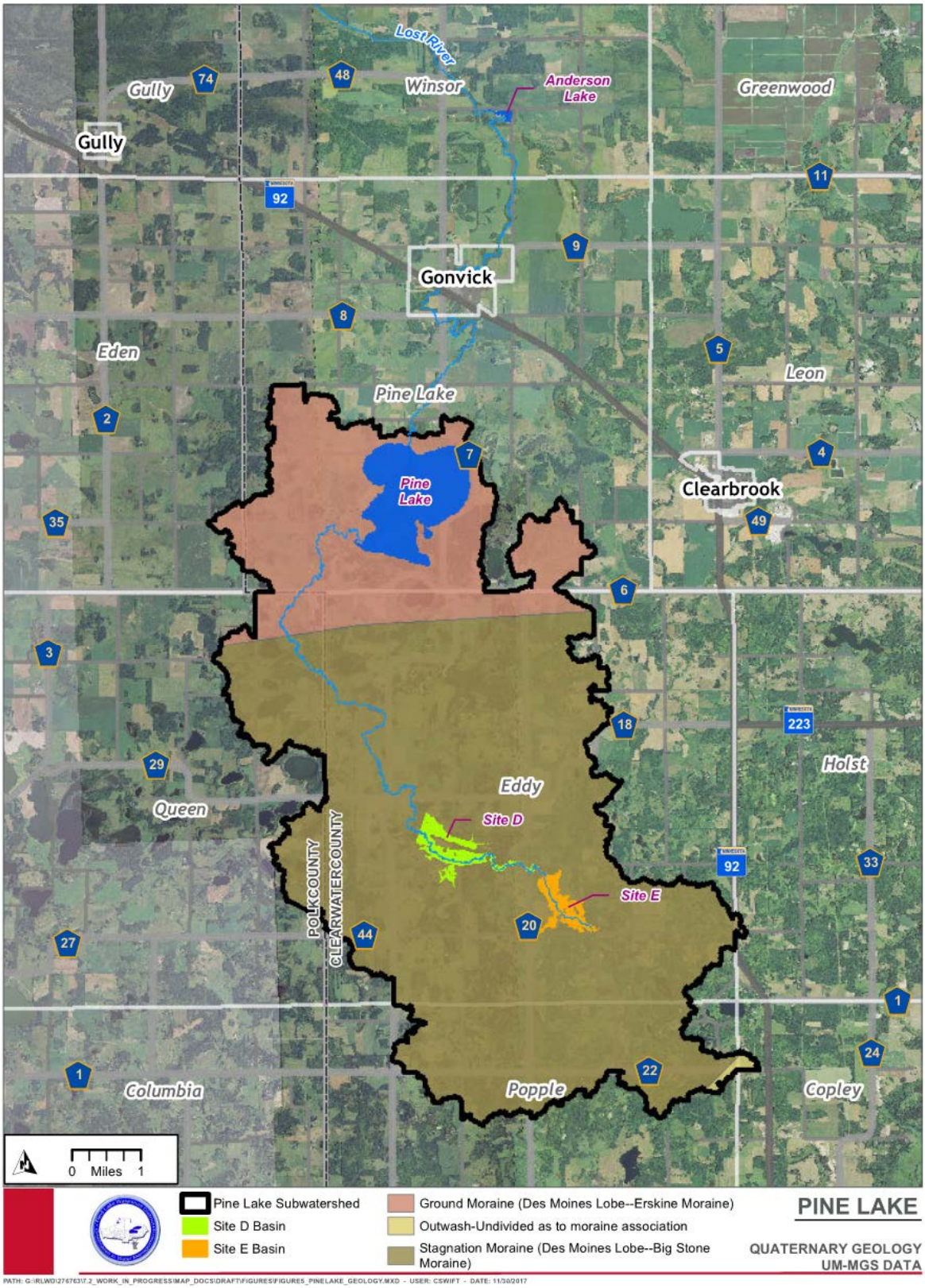

Figure 3. Quaternary Geology

5.1 Site Characterization

Standard methods for undertaking a preliminary geologic investigation in accordance with the National Engineering Manual Part 531 – Geology were used in this planning-level evaluation. A summary of sampling and tests conducted is provided in Table 16.

Table 16. Geotechnical Tests

Watershed Basin	Location	Type	Number	Depth (ft)
Retention Basin D	Embankment	Borings	2	70
		Bulk samples	2	2 to 6
		Shelby tube samples	1	19
	Upstream borrow area	Borings	1	15
		Bulk samples	—	—
Retention Basin E	Embankment	Borings	2	70
		Bulk samples	1	2 to 6
		Shelby tube samples	—	—
	Upstream borrow area	Borings	1	15
		Bulk samples	—	—
Pine Lake Outlet	Structure location	Borings	1	30
		Bulk samples	—	—
		Shelby tube samples	1	9

5.1.1 Retention Basin D

The locations of borings performed for evaluating Retention Basin D are shown on Figure 4. Fence diagrams for those borings are provided in Figure 5. Three borings (17BH-01, 17BH-02, and 17BH-03) were drilled to characterize the site and potential borrow area. Generally, the soils observed in the borings were variable and consisted of firm to very stiff lean clay (CL) interbedded with medium dense to dense sand (SP, SM). The soils near the bottom of the embankment at El. 1330 ft are predominately clay, but transition to sand below El. 1320 ft. While organic soils were not observed in any of the borings for Site D, the Fosston Quad geologic map indicates a significant portion of the project alignment (~75%) is within the geological zone characterized by surficial organic materials with thicknesses greater than 3 ft. The absence of organic soils in the borings at Site D is likely due the location of the borings, which were generally not within the geological zone characterized by organic soils. At Site E, one of the borings (17BH-05) within Site E was drilled within the geological zone characterized by organic soils and 10 to 15-foot of organic soils were observed.

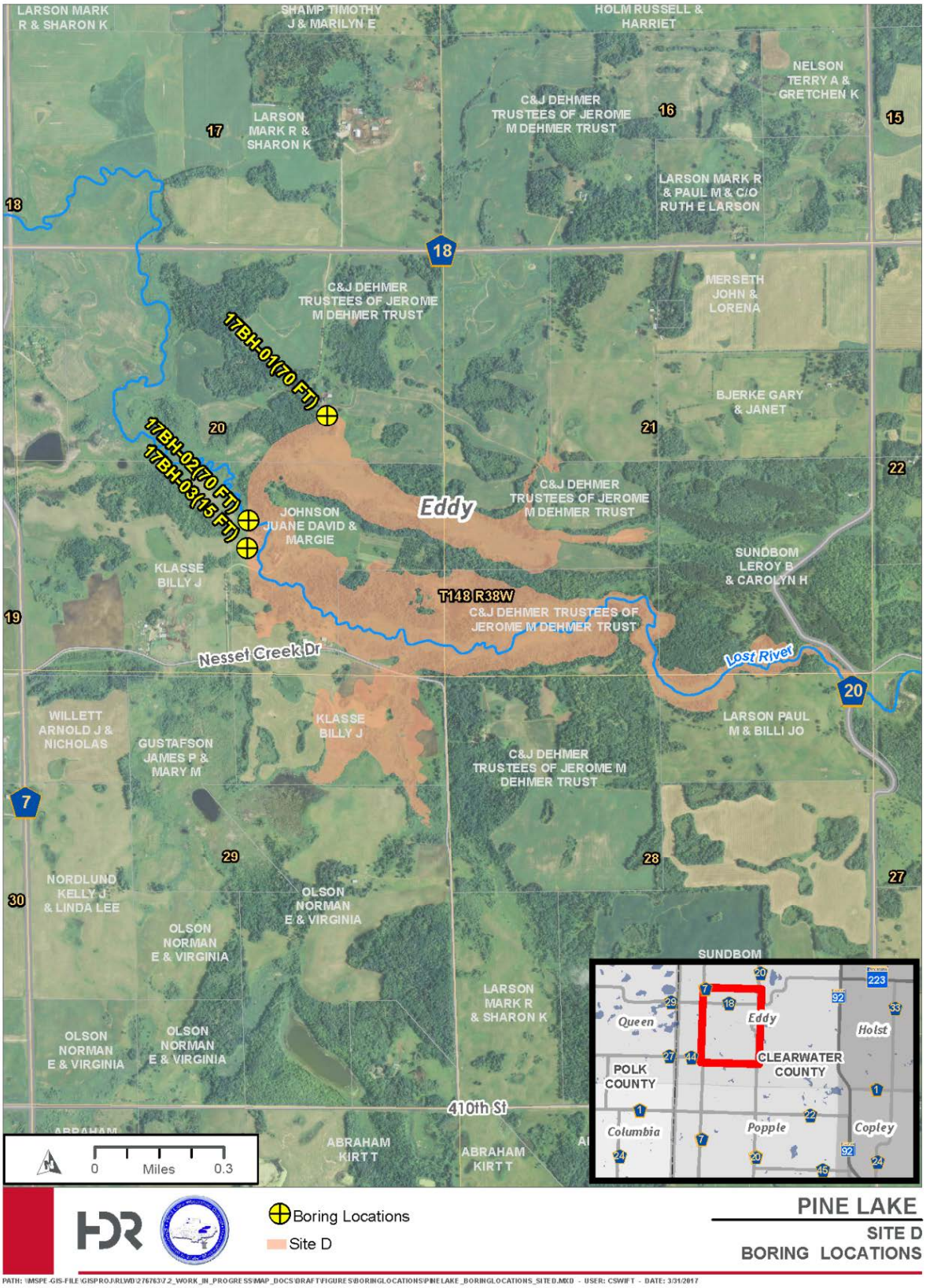

Figure 4. Boring Location Plan for Site D

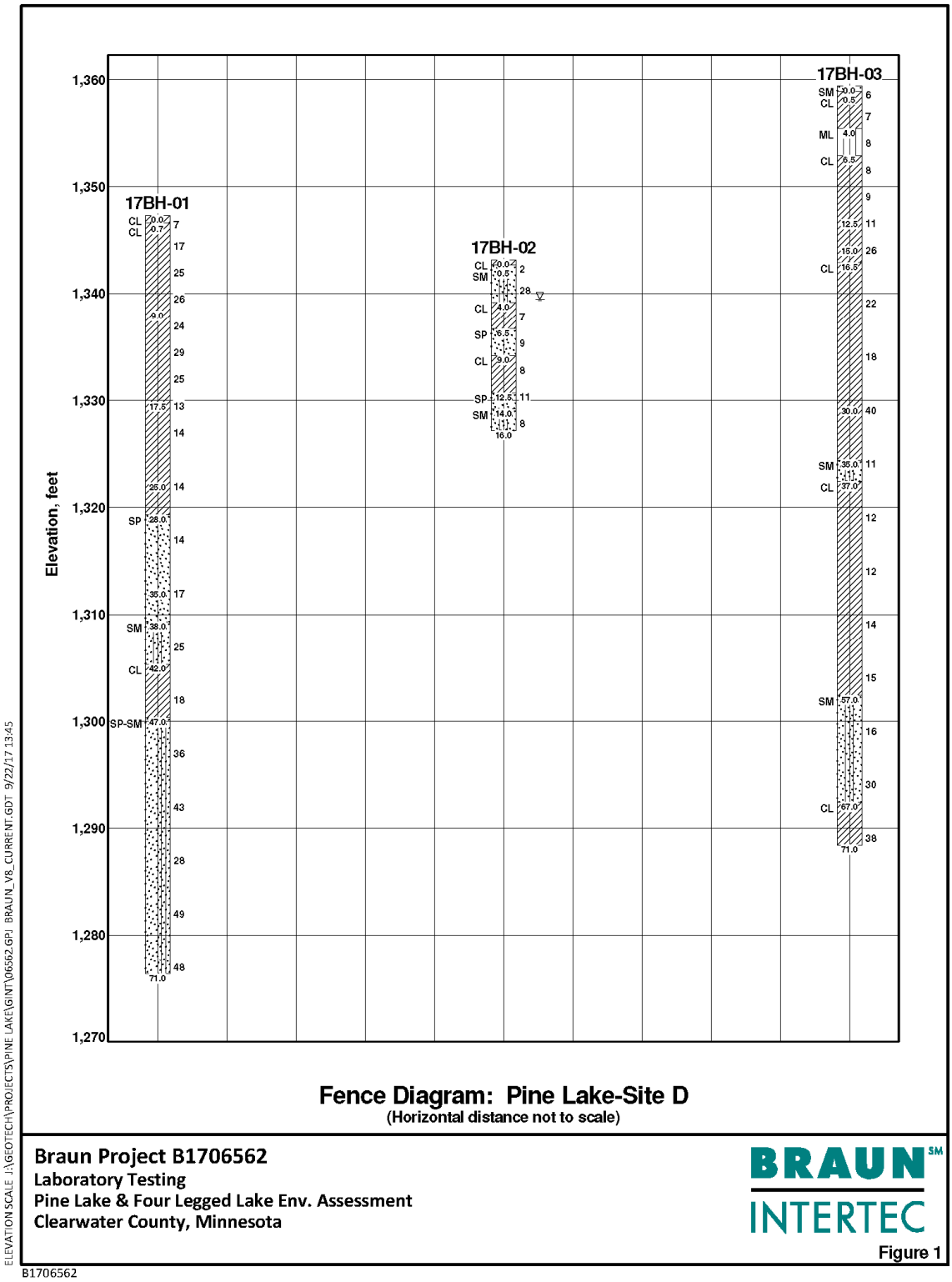

Figure 5. Fence Diagram for Site D

5.1.2 Retention Basin E

Three borings (17BH-04, 17BH-05, and 17BH-06) were drilled within Retention Basin E to characterize the site and borrow area as shown in the boring location plan and the fence diagram (see Figure 6 and Figure 7). Although not included herein, the borings and associated laboratory data can be provided separately upon request.

The soils observed in the borings for Site E were variable and generally consisted of soft to very stiff lean clay and silt (CL, ML) interbedded with loose to dense sand (SP, SM). A 10 to 15-foot thick layer of organic soils was observed in Boring 17BH-05, and the underlying sand and clay soils at this location had lower blow counts relative to other locations. Organic soils were not observed in Borings 17BH-04 and 17BH-06, which were not drilled in the geological zone associated with organic soils based on a review of the Fosston Quad geology map. Overall, the Fosston Quad geology map shows a portion of the project alignment (~25%) is within a geological zone characterized by surficial organic materials with thicknesses greater than 3 ft.

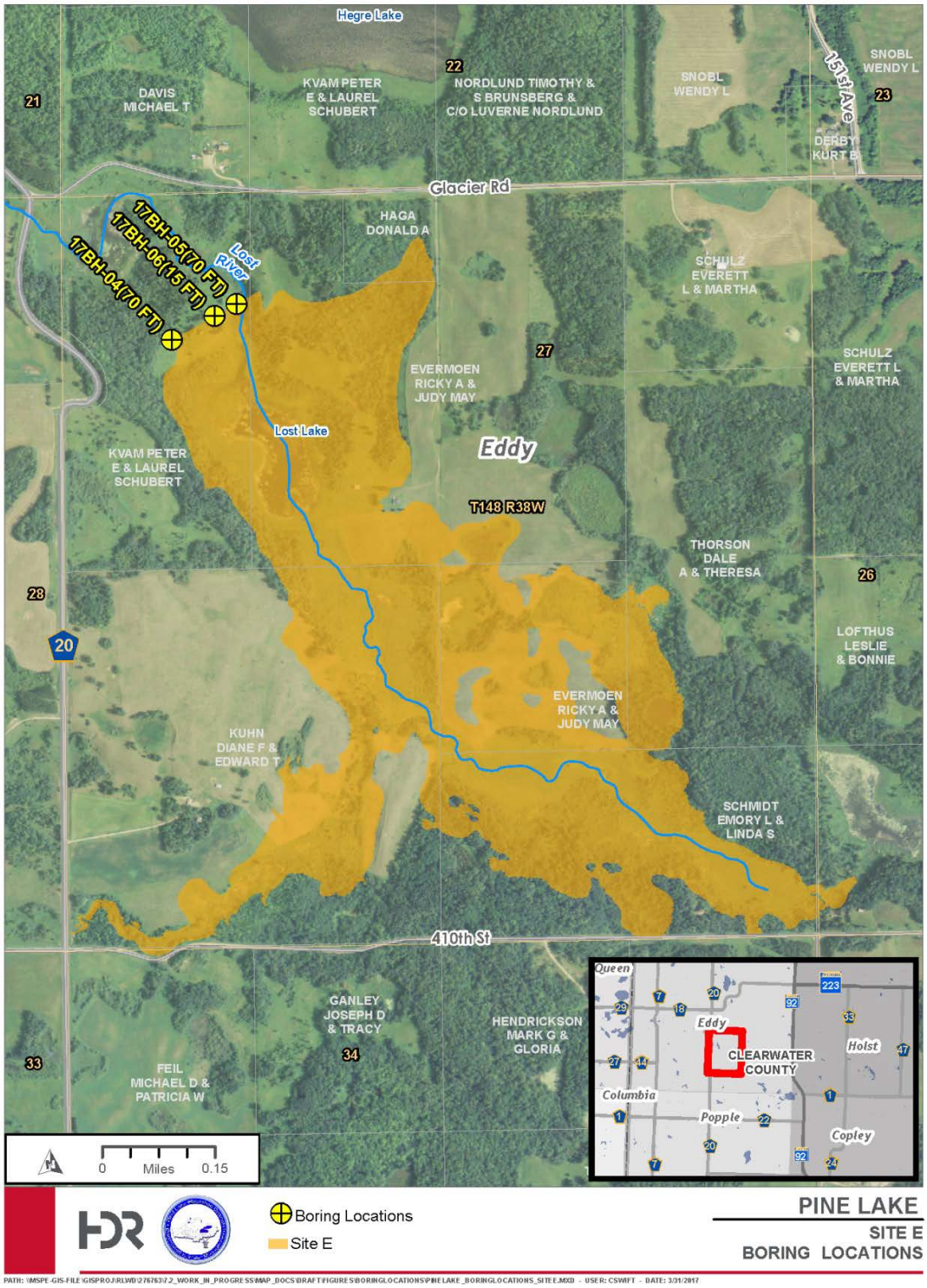

Figure 6. Boring Location Plan for Site E

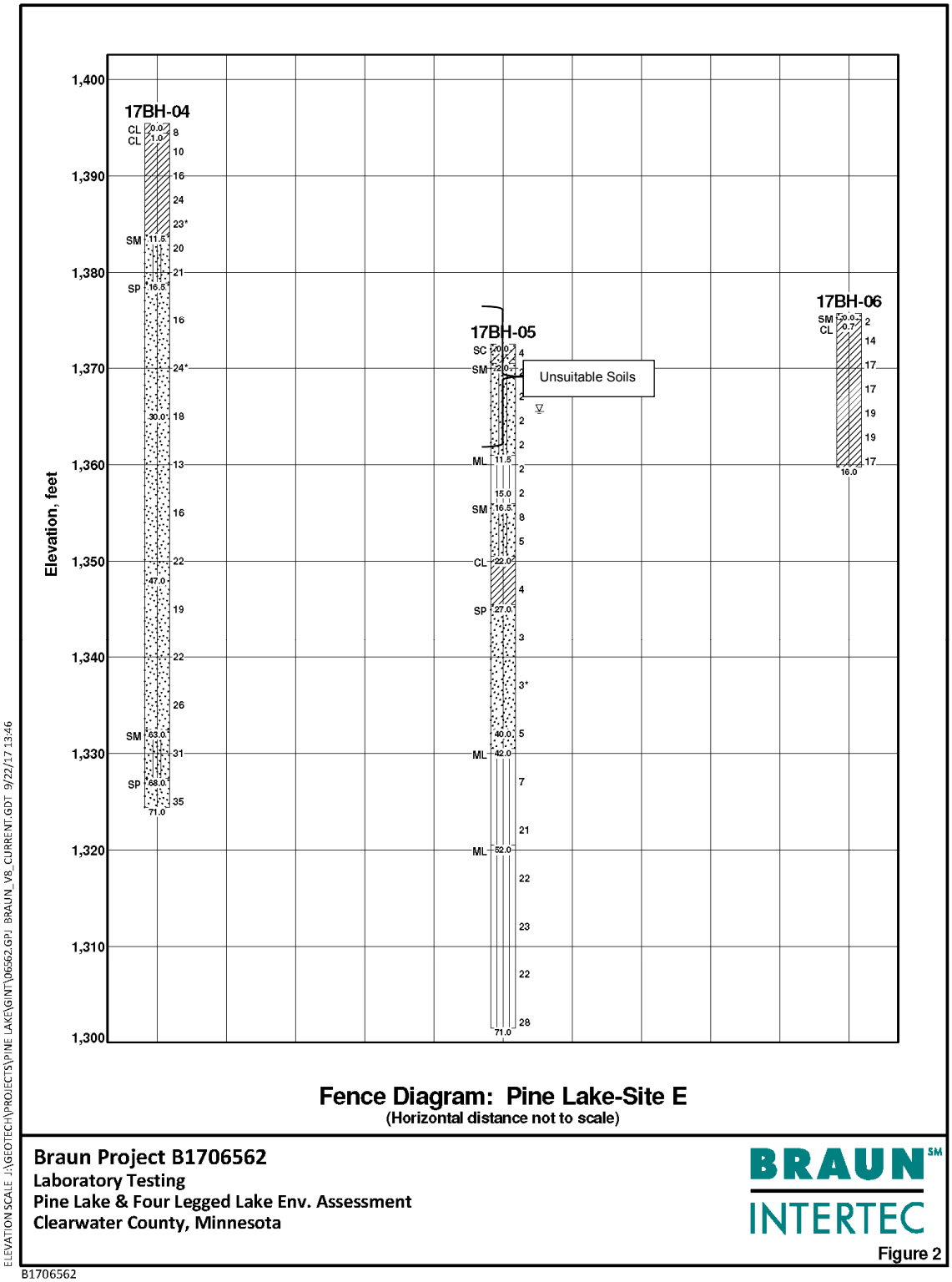

Figure 7. Fence Diagram for Site E

5.1.3 Pine Lake Outlet Structure

One boring (17BH-07) was drilled to characterize the site as shown in the boring location plan and the fence diagram (see Figure 8 and Figure 9). Although not included herein, the borings and associated laboratory data can be provided separately upon request.

The soils observed in the Boring 17BH-07 were generally consistent and classified as firm to stiff lean clay (CL). The approximate location of the project site and boring were Fosston Quad geology map and the geology was generally described as glacial sediment, which is consistent with the stratigraphy observed at this location.

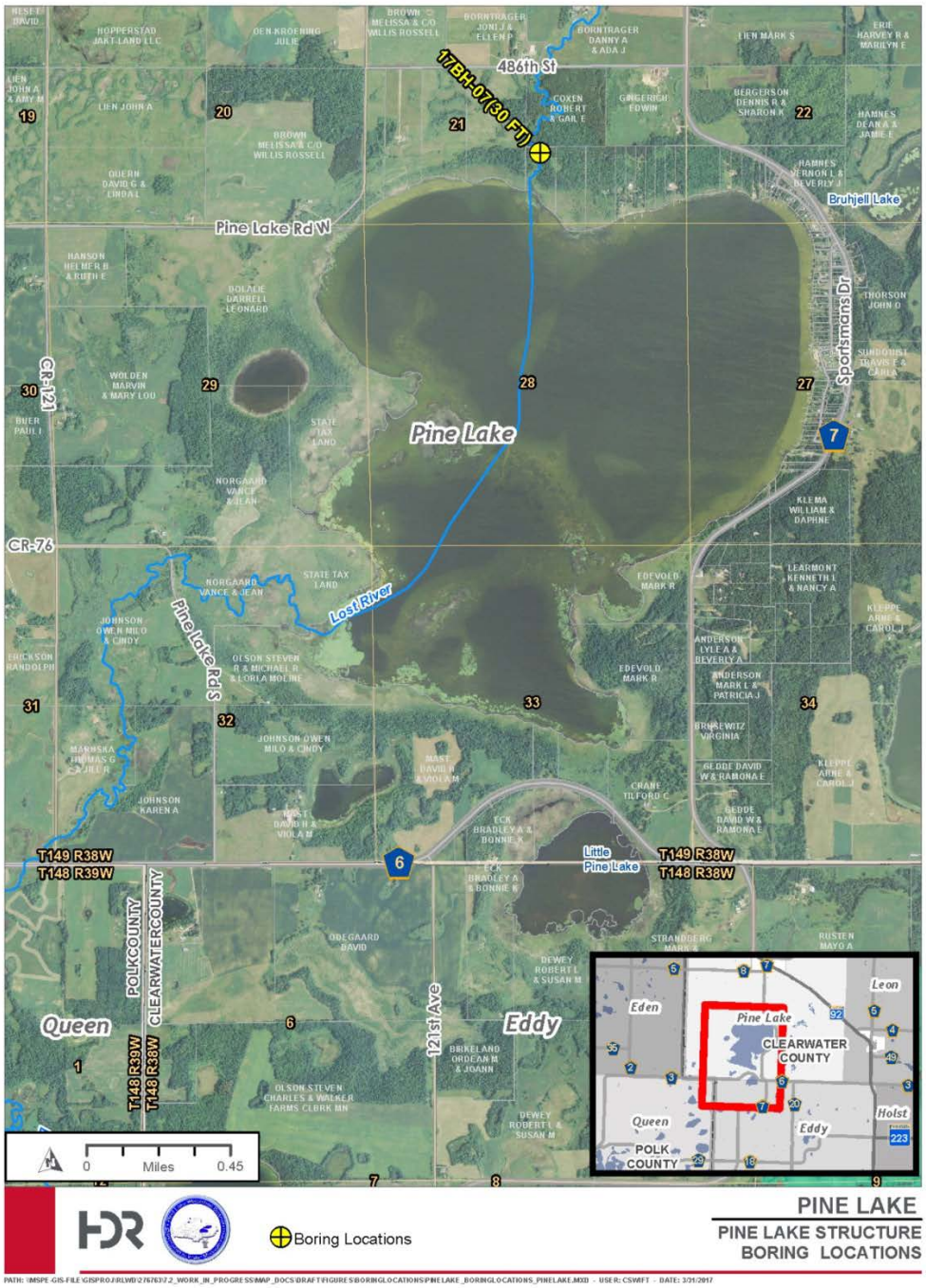

Figure 8. Boring Location Plan for Pine Lake Outlet Structure

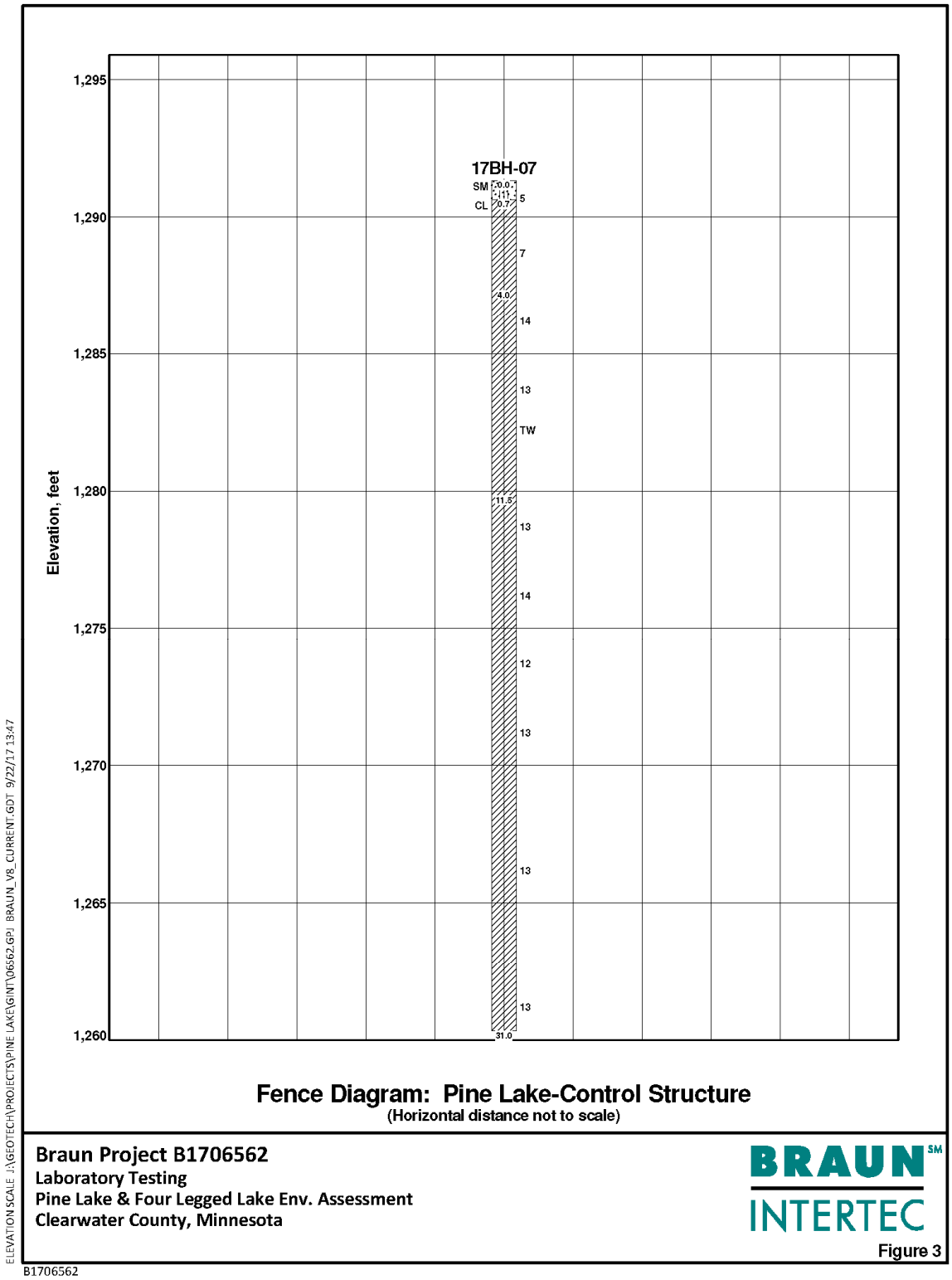

Figure 9. Boring Location Plan for Pine Lake Outlet Structure

5.2 Interpretation of Data

5.2.1 Retention Basins

A steady state seepage analysis was performed in Geostudio 2016 Seep/W Version 8.15.5.11.777 to evaluate the seepage condition, particularly the hydraulic gradient, for Site D and E embankments. Per guidance in USACE EM 1110-2-1901 Seepage Analysis and Control for Dams (1993), a critical hydraulic gradient of 0.5 was adopted for analysis. The generalized cross-section for the Site D and E embankments consist of a 5H:1V slope on the upstream side and 4H:1V slope on the downstream side. The geotechnical evaluation assumed a principal spillway crest elevation of approximately 1357 ft and 1390 ft for Sites D and E, respectively, and auxiliary spillway crests elevations of El. 1359 and 1391 ft, respectively. For analysis purposes, the embankments are assumed to be constructed of clay above the foundation of the dam (El. 1330 ft at Site D and E. 1365 ft at Site E).

Generalized stratigraphy and soil parameters were adopted below the bottom of the dam based on Borings 17BH-01 through 17BH-03 at Site D and Borings 17BH-04 through 17BH-06 at Site E. The soil conditions observed at each location are described in detail in Sections 5.1.1 and 5.1.2. Given the variability in stratigraphy between the borings at each location, a sensitivity analysis was performed as part of the steady-state seepage analysis to evaluate the effect of different stratigraphy on the seepage. For seepage analysis at Site D, the upstream water elevation was modeled at El. 1,357 ft based on the principal spillway crest and it was assumed the downstream water elevation was consistent with the existing ground surface at El. 1,330 ft. For seepage analysis at Site E, the upstream water elevation was modeled at El. 1,390 ft based on the principal spillway crest and it was assumed the downstream water elevation was consistent with the existing ground surface at El. 1,365 ft.

The steady state seepage analyses at both locations, generally indicated the hydraulic gradient at the downstream toe was elevated based on the proposed cross-section and without improvements (i.e. sand filter, toe berm, clay, cutoff, etc). Initially, a sand filter and toe berm were added to the Site D and E embankments to reduce the hydraulic gradient and address stability concerns, which will be discussed later. However, even with the sand filter and toe berm, the hydraulic gradient was still elevated. Furthermore, sensitivity analysis, which was performed to account for potential variability in the stratigraphy, demonstrated the hydraulic gradients may be even higher, particularly where the sand seams were closer to the existing ground surface or thicker than expected. Therefore, a clay cutoff/inspection pit and 3 ft thick grout curtain were also added to lengthen the seepage path and help remediate the potentially high hydraulic gradient.

Table 17 summarizes the hydraulic gradient for the Site D and Site E embankments assuming stratigraphy based on both of the borings drilled at this location. With the sand filter, toe berm, and clay cutoff/grout curtain incorporated into the design cross section, the resulting modeled hydraulic gradient is less than 0.4, which is less than the maximum acceptable hydraulic gradient of 0.5. The seepage results were also evaluated for sensitivity to dam height. This evaluation concluded that the grout curtain could be eliminated for Site D if the dam height were lowered by approximately 5 feet (the sand filter and rock toe would need to remain). The conditions at Site E do not allow elimination of the grout curtain unless the dam height is lowered by more than 13 feet. This amount of lowering would be so great as to make the dam ineffectual at meeting the purpose and need for the project so it is assumed at this stage of analysis that a grout curtain, sand filter, and rock toe will be required for Site E regardless of any minor refinements to dam height.

Table 17. Summary of Seepage Analysis Site D and E

Site	Stratigraphy Basis for Seepage Model	Improvements to Dam	Critical Hydraulic Gradient, i_{cr}	Estimated Hydraulic Gradient, i	Hydraulic Gradient Requirements Satisfied? (Yes/No)
Retention Basin D	Boring 17BH-03	Sand Filter/Toe Berm/Cutoff*	0.5	0.40	Yes
	Boring 17BH-01	Sand Filter/Toe Berm/Cutoff*		0.36	Yes
Retention Basin E	Boring 17BH-05	Sand Filter/Toe Berm/Cutoff	0.5	0.20	Yes
	Boring 17BH-05 and 17BH-06	Sand Filter/Toe Berm/Cutoff		0.42	Yes

* Sensitivity analysis indicates the grout curtain can be eliminated for Site D if the embankment height is lowered by 5 feet.

Stability analysis was performed in Geostudio Slope/w software 2016 (version 8.15.511777). The geometry, soil parameters, stratigraphy, and groundwater table conditions discussed above for the seepage analysis were evaluated for stability analysis. The end of construction (short-term) condition, after construction (long-term) condition, steady state seepage condition, and drawdown condition were evaluated. The minimum required factors of safety for new dams were adopted for each of the loading conditions based on recommendations presented in USACE EM 1110-2-1902 Slope Stability (2003), Table 3-1. Table 18 summarizes the factors of safety resulting from the stability analyses.

Table 18. Summary of Slope Stability Analysis for Site D and E

Site	Loading	Slope	Groundwater Table	Factor of Safety	
				Minimum required	Calculated
Retention Basin D	End of Construction-Undrained Loading	4H:1V Downstream	1,330	1.3	2.64
	After Construction-Long-term	4H:1V Downstream	1,330	1.5	2.21
	Steady State Seepage-Drained (Long-term)	4H:1V Downstream	1,357 to 1,330	1.5	2.03
	Rapid Drawdown	5H:1V Upstream	Before: 1,357 to 1,330 After: 1,328	1.2	2.28
Retention Basin E	End of Construction-Undrained Loading	4H:1V Downstream	1,365	1.3	2.17
	After Construction-Long-term	4H:1V Downstream	1,365	1.5	2.23
	Steady State Seepage-Drained (Long-term)	4H:1V Downstream	1,390 to 1,365	1.5	1.88
	End of Construction-Undrained Loading	5H:1V Upstream	Before: 1,390 to 1,365 After: 1,365	1.2	2.18

Stability requirements are satisfied with the current design at both Sites D and E. The assumed groundwater table for each of the conditions is summarized in Table 18. For the steady state condition, the groundwater table shown in Table 18 was first used to estimate the pore water pressures in the embankment based on a seepage analysis. The pore water pressures from the

seepage analysis were used in the stability analysis. For the drawdown analysis, the initial groundwater table was assumed to be in a steady state seepage condition. At Site D, the groundwater dropped from 1357 ft at the upstream side to 1330 ft at the downstream side. At Site E, the groundwater dropped from 1390 ft at the upstream side to 1365 ft at the downstream side. The groundwater table after drawdown was assumed to be at the elevation of the bottom of the gated structure.

Based on the proposed geometry and stratigraphy, a settlement analysis was performed to estimate the potential settlement and required overbuild of the embankment. Based on the analysis, it is estimated that the embankments at Site D and E will settle approximately 10 inches and 16 inches, respectively. It is likely that settlement will be substantially complete (>90%) within 6 months of completion of the embankment.

Organic soils were observed in Boring 17BH-05 at Site E, which is consistent with the geology map indicating organic soils underlie a portion (~25%) of the embankment based on its current alignment. Additionally, although not observed in the borings drilled for Site D, organic soils are anticipated under a significant portion (~75%) of the Site D alignment based on a review of geology. Organic soils were observed within Boring 17BH-05 to a depth of approximately 10 to 15 feet below the proposed ground surface at Site E. A similar thickness of organic soils should be expected at Site D and E where organic soils underlie the embankment. Where the organic soils are observed, the soils should be undercut and replaced with cohesive soils (lean clay with sand) and the grout curtain should be extended below the undercut to the bottom elevations shown in the typical details. The bottom elevations shown in the details were selected to key the cutoff into the underlying clay and silt soils, which are assumed to be less permeable than the clays. Depending on the soils encountered, the cutoff may need to be extended deeper within the soil profile.

The borings at Sites D and E were also studied to evaluate the potential use of borrow material as fill. Consistent with observations below the foundation level of the embankment, the stratigraphy was variable within the elevations of the borings where borrow material may be taken. Generally, the soils at Site D were predominantly cohesive materials (lean clay with sand) while the soils at Site E contained more sand (poorly graded sand or silty sands). The materials at both locations are generally appropriate for use as fill materials. Although not clear in the borings, a significant portion of the alignments at both sites may have organics within the stratigraphy as noted above. These materials would not be appropriate for reuse as fill and would have to be transported offsite.

5.2.2 Pine Lake Outlet Structure

The proposed Pine Lake outlet structure is an inverted T shape and is assumed to have a 10” stem, 18” thick footing, and 6 ft wide base, which is embedded 6 ft below the ground surface. The top of the structure extends 3 ft above the ground surface, which is at El. 1281 ft, to El. 1284 ft. The water level at the upstream side of the outlet structure is assumed to be El. 1284 ft and the water level at the downstream side is assumed to be El. 1281 ft.

A steady state seepage analysis was performed in Geostudio 2016 Seep/W Version 8.15.5.11.777 to evaluate the hydraulic gradient as well the uplift and hydrostatic water pressures on the outlet structure. External stability and settlement analyses were performed to evaluate the foundation design (using the uplift and hydrostatic water pressures from the seepage analysis as input).

The stratigraphy and soil parameters were adopted based on Borings 17BH-07, which indicated the foundation would be underlain by stiff lean clay with sand. The soil conditions observed at the outlet location are described in detail in Section 5.1.3.

Table 19 summarizes the results of the seepage analysis at the Outlet Structure. Per guidance in USACE EM 1110-2-1901 Seepage Analysis and Control for Dams (1993), a critical hydraulic gradient of 0.5 was adopted for analysis. For the proposed design, the estimated hydraulic gradient is less than the critical hydraulic gradient and design requirements appear to be satisfied.

Table 19. Summary of Seepage Analysis at Outlet Structure

Wall Ht (ft)	Hydraulic Gradient, i			Hydrostatic Pressures from SEEP/W Analyses (psf)		
	Critical i_{cr}	Estimated from SEEP/W	Requirements Satisfied? (Yes/No)	Uplift	Hydrostatic Pressure on Upstream Side	Hydrostatic Pressure on Downstream Side
8	0.5	0.16	Yes	400	El. 1284 to 1281 ft: 0 to 187 El. 1281 to 1277.5 ft: 187 to 375 El. 1277.5 to 1276 ft: 375 to 434	El. 1281 to 1277.5 ft: 0 to 245 El. 1277.5 to 1276 ft: 245 to 375

External stability, which includes sliding, overturning, and bearing capacity, was also evaluated for the outlet structure foundation (based on the hydrostatic pressures summarized in Table 19 as well as any additional loads on the foundation). The loading on the outlet structure was predominately from the unbalanced hydrostatic pressures as the ground surface is at the same elevation (El. 1281 ft) on both sides of the wall near the sides of the structure.

Minimum required factors of safety for sliding, overturning, bearing capacity of 1.5, 2.0, and 3.0, respectively, were adopted per guidance in Chapter 10: Earth Retaining Structures of the Soils and Foundations Reference Manual (Santani and Nowatski, 2006) and Engineering Manual 1110-2-2502 (USACE, 1989). For the proposed outlet structure, the calculated factors of safety are greater than the minimum required factors of safety for sliding, overturning, and bearing capacity, and design requirements are satisfied.

Based on the proposed geometry and stratigraphy, a settlement analysis was performed to estimate the potential settlement of the outlet structure. It is estimated a maximum settlement of 2 inches will occur at the outlet structure.

Given the existing and proposed geometry at the site, it is anticipated excess material will be removed from the site. The soils are generally consistent with the material observed at the potential borrow locations and could be reused for fill at outside locations.

6 Sensitivity Analysis for Dam Height

The Site D and Site E configurations evaluated previously in this memorandum are based on maximizing the use of available storage at each site (i.e. the embankments are at the highest practical level for the location). To identify the optimal balance between project costs and benefits, a sensitivity analysis must be performed to identify how hydraulic performance, cost, and benefits, change based on embankment heights of basins D and E. Lowering the overall embankment height would reduce the amount of storage available for flood attenuation and would reduce the potential benefits downstream. This lowering of the embankment would also reduce the construction cost of the project. This section provides a discussion on how varying the height of embankments at Sites D and E affects downstream hydraulic performance. Sections 9 and 10 provide discussion on how alternative embankment heights affect costs, benefits, and the resulting impact to benefit-cost ratios.

6.1 Pine Lake Structures

The primary purpose of the project is to provide flood protection to structures around Pine Lake. LiDAR data was used to estimate the elevations at which structures would become inundated by flooding. Figure 10 shows the relationship between lake levels for various storm events and structures that are located around the lake. As shown in this figure, modeling indicates that a 100-year 10-day design event would result in a lake water surface elevation of approximately 1286.7 feet which would flood approximately 79 structures. Under a 100-year 24 hour storm event, a lake elevation of approximately 1285.6 feet would flood approximately 39 structures. Reduced numbers of structures are impacted for the 50 year and smaller (more frequent) events.

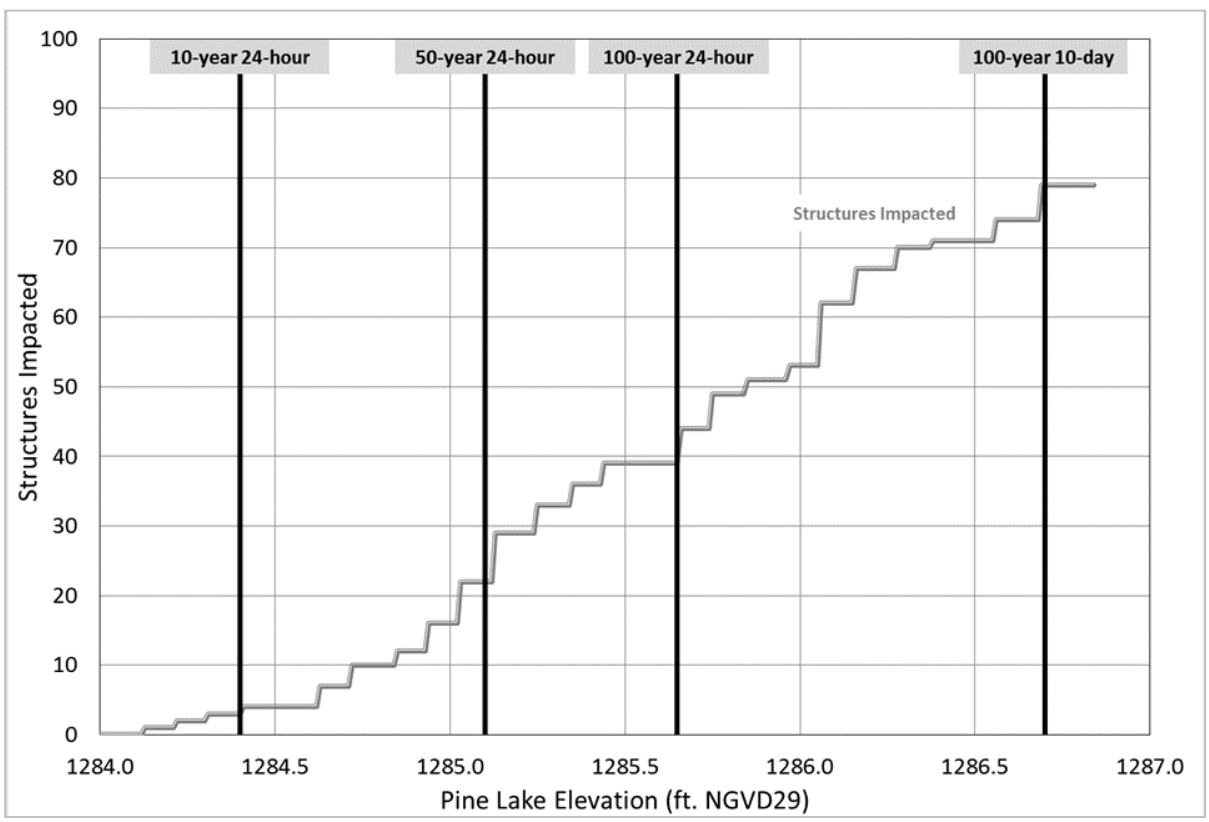

Figure 10 Pine Lake Structure Impacts by Event

6.2 Modifications to Site D Retention Basin

The Site D Retention Basin was evaluated for three alternative configurations:

- Original embankment height with top elevation of 1363 feet (as documented in Section 4)
- 11 feet lower embankment height (top elevation of 1352 feet)
- 6 feet lower embankment height (top elevation of 1357 feet)

Modeling of the 11 foot lower embankment showed that Pine Lake water surface elevations would be reduced from existing conditions by 0.1 feet or less for the 100-year 10-day event. Under this scenario, no structures would appreciably benefit from the upstream presence of the Site D retention basin. For this reason, the 11 foot lowered Site D alternative was dismissed from further consideration.

The full height embankment configurations documented in Section 4 assumed that the floodplain/impoundment area upstream of the embankments are empty, and that the low flow gate is closed immediately prior to the design storm event. Timing of low flow gate closure was evaluated to determine the impact of refining the performance of retention basin D. Delayed closure allows a structure's storage to be engaged during the most beneficial time. Models were run with increasing closure times to identify the time at which gate closure would be most effective. Optimal closure times for Sites D under the 100-year 10-day event are at 8 days for the full height embankment and at 8.75 days for the 6 foot lower embankment. Evaluation also showed that timing of gate closure did not change Pine Lake water elevations for the 100-year 24-hour event.

The performance of the original height configuration, 6 foot lower configuration, and existing conditions are plotted on Figure 11. Shading or 'bands' are provided to indicate the range of performance that could be anticipated from changing the timing of gate closure. The 100-year 10-day events are shown on the right side of the figure, and the 100-year 24-hour events are shown on the left side of the figure. As shown in Figure 11, under the full height scenario, between 62 and 70 structures could become inundated by a 100-year 10-day event (depending on the timing of gate closure). Between 71 and 74 structures could be inundated by a 100-year 10-day event for the 6 foot lowered (reduced height) embankment. As shown on the left side of Figure 11, the width of the performance bands is very small indicating that gate closure is not critical for the 100-year 24-hour event. A 6 foot reduced height embankment would result in approximately 36 structures being flooded. Per the assumed structure elevations as determined by LiDAR, the full height embankment would limit flooding to approximately 16 structures.

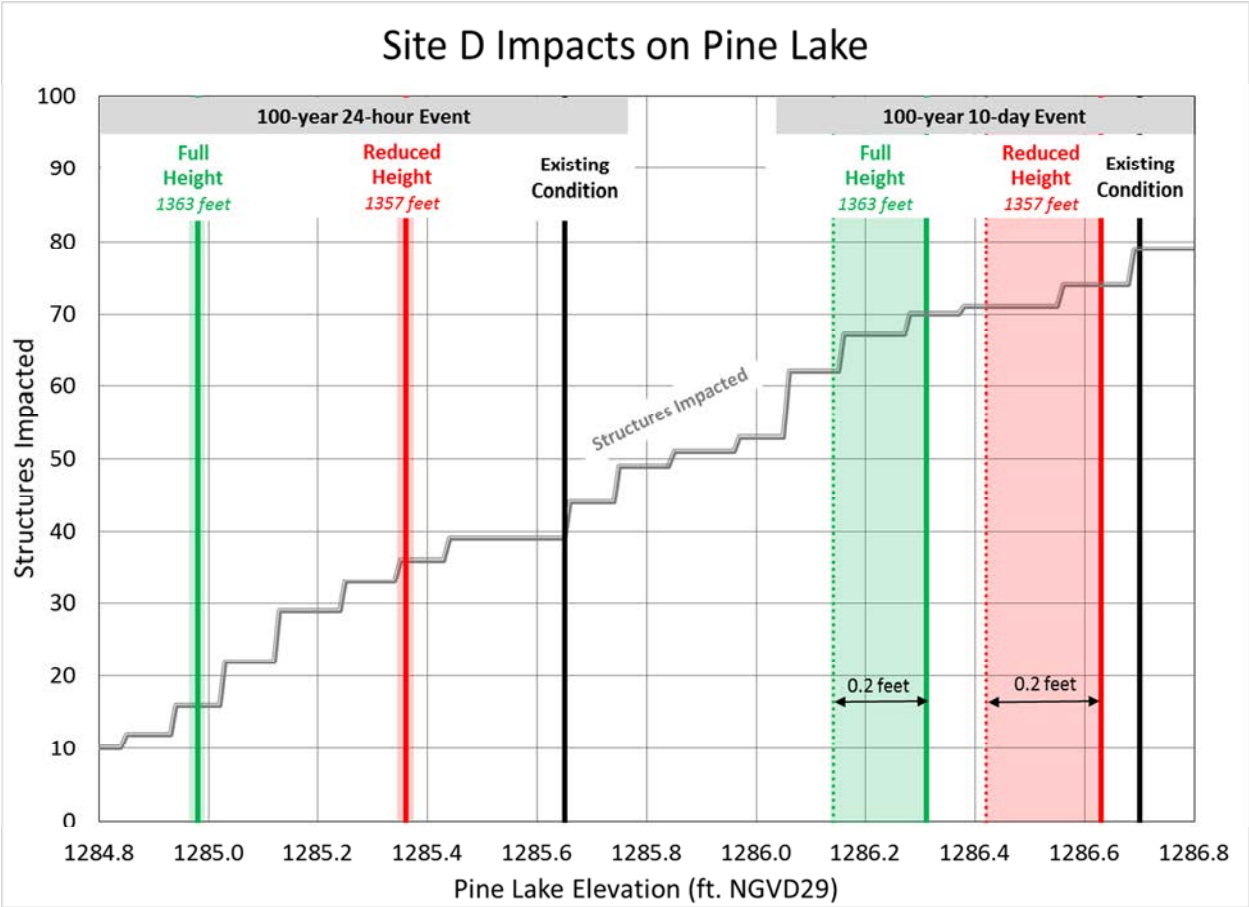

Figure 11 Site D Impacts on Pine Lake

6.3 Modifications to Site E Retention Basin

The Site E Retention Basin was evaluated for three alternative configurations:

- Original embankment height with top elevation of 1393.2 feet (as documented in Section 4)
- 10 feet lower embankment height (top elevation of 1383.2 feet)
- 5 feet lower embankment height (top elevation of 1388.2 feet)

Modeling of the 10 foot lower embankment showed that Pine Lake water surface elevations would be reduced from existing conditions by 0.1-0.2 feet for the 100-year 10-day event. Under this scenario, few if any structures would appreciably benefit from the upstream presence of the Site E retention basin. For this reason, the 10 foot lowered Site E alternative was dismissed from further consideration.

As for Site D, the full height embankment configurations for Site E documented in Section 4 assumed that the floodplain/impoundment area upstream of the embankments are empty, and that the low flow gate is closed immediately prior to the design storm event. Timing of low flow gate closure was evaluated to determine the impact of refining the performance of retention basin E. Delayed closure allows a structure's storage to be engaged during the most beneficial time. Models were run using increasing closure times to identify the time at which gate closure would be most effective. Optimal closure times for Sites E under the 100-year 10-day event are at 2 days for the full height embankment and at 3.75 days for the 5 foot lower embankment. Consistent with what was

determined for Site D, modeling showed that timing of gate closure at Site E did not change Pine Lake water elevations for the 100-year 24-hour event.

The performance of the original height configuration, 5 foot lower configuration, and existing conditions are plotted on Figure 12. Shading or 'bands' are provided to indicate the range of performance that could be anticipated from changing the timing of gate closure. The 100-year 10-day events are shown on the right side of the figure, and the 100-year 24-hour events are shown on the left side of the figure. As shown in Figure 12, under the full height scenario, approximately 62 structures could become inundated by a 100-year 10-day event. Approximately 70 structures could be inundated by a 100-year 10-day event for the 5 foot lowered (reduced height) embankment. As shown on the left side of Figure 12, the width of the performance bands is very small indicating that gate closure is not critical for the 100-year 24-hour event. Per the assumed structure elevations as determined by LiDAR, the full height and 5 foot reduced height embankments would limit flooding to approximately 29 structures.

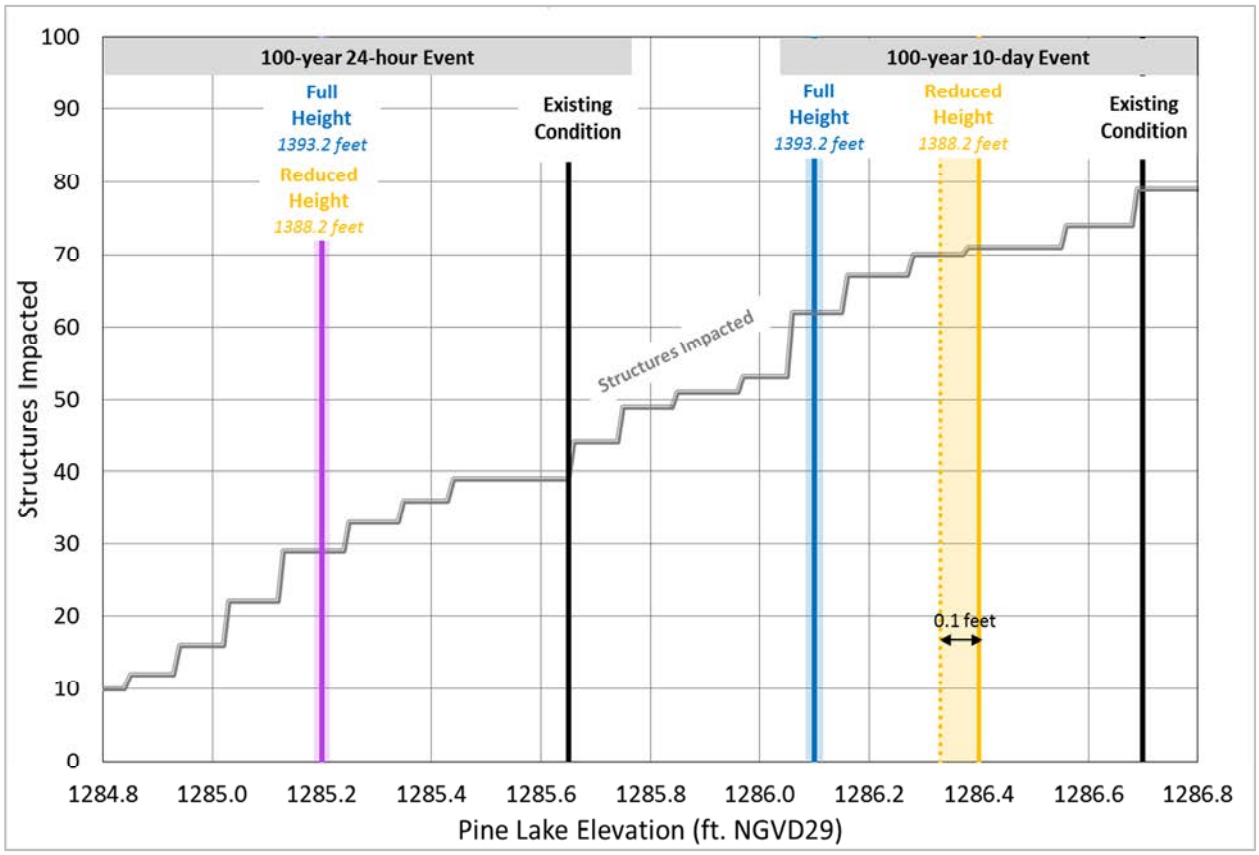

Figure 12 Site E Impacts on Pine Lake

7 Breach Analysis and Hazard Classification

Dam breach failures were evaluated for each of the two retention basins under consideration. The evaluated dam breach failures were completed assuming a “sunny day failure” with a water level at the elevation of the earthen auxiliary spillway. The maximum breach outflow rate was determined using guidance contained in TR-60 (NRCS 2005). The calculated peak discharge estimate used for this analysis was approximately 34,400 cubic feet per second. Once this flow rate was established, methods outlined in NRCS TR-66 (NRCS 1985) were used to develop an outflow hydrograph for use in the HEC-RAS hydraulic model. The HEC-RAS hydraulic model produced inundation extent boundaries, time of arrival estimates, and estimates of maximum depth, velocity, and discharge. The models extended from the downstream toe of each retention basin to the confluence of the Lost River with Silver Creek at Anderson Lake (approximately 3 miles downstream of Pine Lake). The maximum inundation areas are shown in Figures 21 - 26 in Appendix C.

For Basin D, the flood wave from the breached retention basin would arrive at Pine Lake in approximately 6.75 hours. Flow depths and water surface elevations were tabulated at several primary road crossings. Maximum depths varied between the road crossings. Results indicate the overtopping depths range from roughly 1 foot to 6 feet at the crossings examined. A dam breach would result in potential property damage and risk to human life by affecting one home and four farm buildings and by overtopping nine road crossings (Table 20 and Table 22). The home identified in the flood zone would be inundated by an estimated 0.5 feet of water; the farm buildings would be inundated by between an estimated 1 and 3 feet of water. No schools, hospitals, or municipal buildings were identified near the Basin D breach flood zone.

For Basin E, the flood wave from the breached retention basin would arrive at Pine Lake in approximately 7.25 hours. Results indicate the roadway overtopping depths would range from roughly 0.5 foot to 11 feet at the crossings examined. A dam breach would result in potential property damage and risk to human life by affecting one home and two farm buildings and by overtopping ten road crossings (Table 21 and Table 23). The home identified in the flood zone would be inundated by an estimated 0.4 feet of water; farm buildings would be inundated by between an estimated 0 and 2 feet of water. No schools, hospitals, or municipal buildings were identified near the Basin E breach flood zone.

Table 20. Retention Basin D Affected Structures Breach Summary – Estimated Number of Buildings in the Breach Flood Zone

Building Type	Inundation Depth 0 to 3 feet	Inundation Depth 3 to 6 feet	Inundation Depth 6 to 9 feet
Home	1	0	0
Farm building/other	4	0	0
Commercial	0	0	0
Total	**5**	**0**	**0**

Table 21. Retention Basin E Affected Structures Breach Summary – Estimated Number of Buildings in the Breach Flood Zone

Building Type	Inundation Depth 0 to 3 feet	Inundation Depth 3 to 6 feet	Inundation Depth 6 to 9 feet
Home	1	0	0
Farm building/other	2	0	0
Commercial	0	0	0
Total	**3**	**0**	**0**

Table 22. Retention Basin D Breach Impacts to Roadways

Structure Description	Structure Location (miles)[a]	Maximum Depth (feet)[b]	Maximum Water Surface Elevation (feet)[c]	Top of Road at Crossing Structure (feet)[d]	Maximum Overtopping Depth near Crossing (feet)[e]	Travel Time to Structure (minutes)[f]	Time to Peak Discharge at Structure (minutes)[g]
CR 18	0.8	11.0	1,333.7	1,327.4	6.3	15	45
CR 7	1.5	10.6	1,332.0	1,326.3	5.7	30	60
440th Street	2.8	12.5	1,330.0	1,327.0	3.0	75	120
420th Avenue SE	5.7	2.4	1,301.7	1,298.4	3.3	240	345
460th Street	6.3	9.6	1,301.3	1,299.1	2.2	225	435
109th Avenue	6.4	7.8	1,298.6	1,295.4	3.2	240	450
109th Avenue	7.4	4.0	1,292.7	1,290.6	2.1	435	495
South Pine Lake Road	7.7	6.0	1,292.3	1,291.0	1.3	405	585
139th Avenue	12.2	4.5	1,212.9	1,212.6	0.3	4,560	5,415

[a] Distance downstream of dam face.
[b] Depth at the upstream side of the crossing within the channel
[c] Elevations in NGVD29 feet.
[d] Top of road elevation estimated from LiDAR in NGVD29 feet.
[e] Maximum overtopping is the overtopping depth at the lowest roadway elevation near the structure. Lowest roadway elevation may be located away from structure.
[f] Travel time is the time from dam breach formation to the arrival of the flood wave at a given location.
[g] Time to peak discharge is the time measured after the dam breach until the peak flow rate has developed at a given location.

Table 23. Retention Basin E Breach Impacts to Roadways

Structure Description	Structure Location (miles)[a]	Maximum Depth (feet)[b]	Maximum Water Surface Elevation (feet)[c]	Top of Road at Crossing Structure (feet)[d]	Maximum Overtopping Depth near Crossing (feet)[e]	Travel Time to Structure (minutes)[f]	Time to Peak Discharge at Structure (minutes)[g]
141st Avenue	0.4	17.0	1,374.1	1,362.7	11.4	15	15
Nesset Creek Drive	1.7	9.6	1,351.1	1,350. 8	0.3	30	45
CR 18	3.4	9.9	1,332.6	1,327.4	5.2	60	90
CR 7	4.1	9.3	1,330.7	1,326.3	4.4	75	105
440th Street	5.4	11.6	1,329.2	1,327.0	2.2	120	165
420th Avenue SE	8.3	1.6	1,301.0	1,298.4	2.6	345	435
460th Street	8.9	9.2	1,300.9	1,299.1	1.8	255	510
109th Avenue	9	6.7	1,297.5	1,295.4	2.1	285	510
109th Avenue	10	3.1	1,291.8	1,290.6	1.2	435	645
South Pine Lake RD	10.3	5.3	1,291.6	1,291.0	0.6	465	675

[a] Distance downstream of dam face.
[b] Depth at the upstream side of the crossing within the channel
[c] Elevations in NGVD29 feet.
[d] Top of road elevation estimated from LiDAR in NGVD29 feet.
[e] Maximum overtopping is the overtopping depth at the lowest roadway elevation near the structure. Lowest roadway elevation may be located away from structure.
[f] Travel time is the time from dam breach formation to the arrival of the flood wave at a given location.
[g] Time to peak discharge is the time measured after the dam breach until the peak flow rate has developed at a given location.

8 Sedimentation

A sedimentation evaluation was performed to estimate the amount of sediment that would accumulate in either of the alternative retention basins (Basins D or E). No records of dredging within the existing Pine Lake basin or within the wetlands of Site F were found. It was assumed that the project life would be 100 years for the features evaluated by this Watershed Plan/Environmental Assessment. The amount of sediment that is assumed to accumulate over the 100-year project life in the retention basins was removed from the storage volume considered to be available for attenuation of stormwater flows.

The evaluation used USDA's Revised Universal Soil Loss Equation (RUSLE2) to estimate potential losses of soil from rainfall events. The RUSLE2 approach considers watershed soil characteristics, slopes, and land uses. A simplified sediment transport evaluation was used to route sediment to each proposed impoundment location. Comparison of the calculated sediment accumulation rate was made using previous regional studies.

The average sediment deposition rate for Site D was estimated to be 1.64 acre-feet per year. The deposition rate for Site E was estimated to be 0.69 acre-feet per year. It was assumed that there would be no future changes in land uses because only small land use changes have occurred over the last 50 years. Operations and maintenance, including ditch dredging and sediment placement within the impoundment, were assumed to completely capture sediment within the impoundments. The majority of the Highly Erodible Land (HEL) was within ½ to 1 mile of Lost River. The sediment pool for a 100-year design life was estimated to be 164 acre-feet for Site D and 69 acre-feet for Site E.

8.1 Watershed Characteristics

Watershed characteristics, including topography, soils, and land uses, affect soil erosion rates. Higher hillside slopes and stream gradients result in increased flow velocities that can accelerate the removal and transport of sediment. Additionally, locations closer to the impoundments will have a greater chance to deliver sediment than locations farther away. The less cohesive a soil is, such as silt, the higher the rate of erosion. Additionally, land use and soil management practices, specifically related to vegetative cover, will affect erosion rates. The Site D and Site E watersheds were evaluated separately for these characteristics. The Site D watershed is the larger of the two watersheds and contains the smaller Site E watershed.

8.1.1 Topography

Watershed elevations were obtained from LiDAR from the International Water Institute (2010). Elevations were evaluated using a 1-meter resolution grid, while slopes were computed using a 30-meter resolution grid. Elevations within the Site E watershed have a relief of 255.5 feet. Elevations within the Site D watershed have a relief of 285.7 feet. Stream slopes range from 1% to 1.5%. Average hillside slopes range from 4.5% to 5%. The greater hillside slopes, from 20% to 30%, occur in a corridor within ½ mile of the Lost River and other drainages.

The drainage areas were estimated from terrain data developed by USACE (USACE 2013). The Site D drainage area is approximately 11,853 acres, and the drainage area of Site E is approximately 6,119 acres. The Lone Lake area (approximately 460 acres) in the upper watershed was assumed to be non-contributing based on review of the terrain data.

8.1.2 Watershed Soils

Generalized watershed soils information was taken from the Clearwater County soil survey (USDA 1997). Figure 13, Table 24, and Table 25 provide the types and distribution of soils in the watersheds for impoundments D and E. The soil types are generally the same for both sites. The

watershed is mostly loams, with some organic soils (“mucks”). While mucks are organic in nature, some of these soils can have a loamy component. Loams containing a higher percentages of silt tend to have higher erosion potential.

Predominant loams are the Naytahwaush, Mankonce, and Auganaush series. These loams are glacial in origin, with generally low permeability (USDA 2004). Naytahwaush soils occur throughout the watershed, from high slopes to nearly flat areas. Mahkonce soils are limited to flat areas while Auganaush soils tend to occur in swales. Loamy sands, where present, tend to form near drainages.

The second-most predominant soil types are mucks. These soils tend to occur in depressions and are formed from organic and vegetative matter. The Cathro series, for example, contains heterogeneous units of loamy to organic materials with a wide range of infiltration rates.

8.1.3 Land Use

Land uses were characterized using the National Land Cover Dataset (NLCD) (Homer et al, 2015). The NLCD is a remotely sensed interpretation of land use that is generally updated every 5 years. The most recent NLCD information is from 2011. The NLCD information is shown in Figure 14 and tabulated in Table 26. Within the watershed above Site E, forests cover over 60% of the watershed. In the Site D watershed, forests cover 54% of the total area. Human-modified land uses, primarily pastures, account for 22% (Site E) to 28% (Site D) of the areas. Wetlands and developed areas (primarily roads and ditches) are less than 5% of these areas.

The Cover Management Factor (C) and Practice Management Factor (P) are components of the RUSLE regression equation that are based on land uses. The C factor considers the type of vegetative cover in place. The cover might be natural conditions, such as forests or wetlands, or cultivated uses, such as homesteads, crops, or pastures. The P factor is how land is managed over a year. Practice management might reflect the natural condition or various tillage approaches.

The Cover Management Factors (C) were adopted from the International Water Institute’s Prioritize, Target, and Measure Application (PTMApp) rural watershed planning process (International Water Institute 2016). These values are shown in Table 27. Natural land uses, such as forests, have lower C values, indicating less potential for erosion. The Practice Management Factor (P) assumes a value of 1 for modified land uses (crops and pastures) with loose soils with more uniform drainage slopes. Natural land uses (forests and wetlands) are assumed to be rough, irregular, and more compacted soils; the P value for this is equal to 0.9.

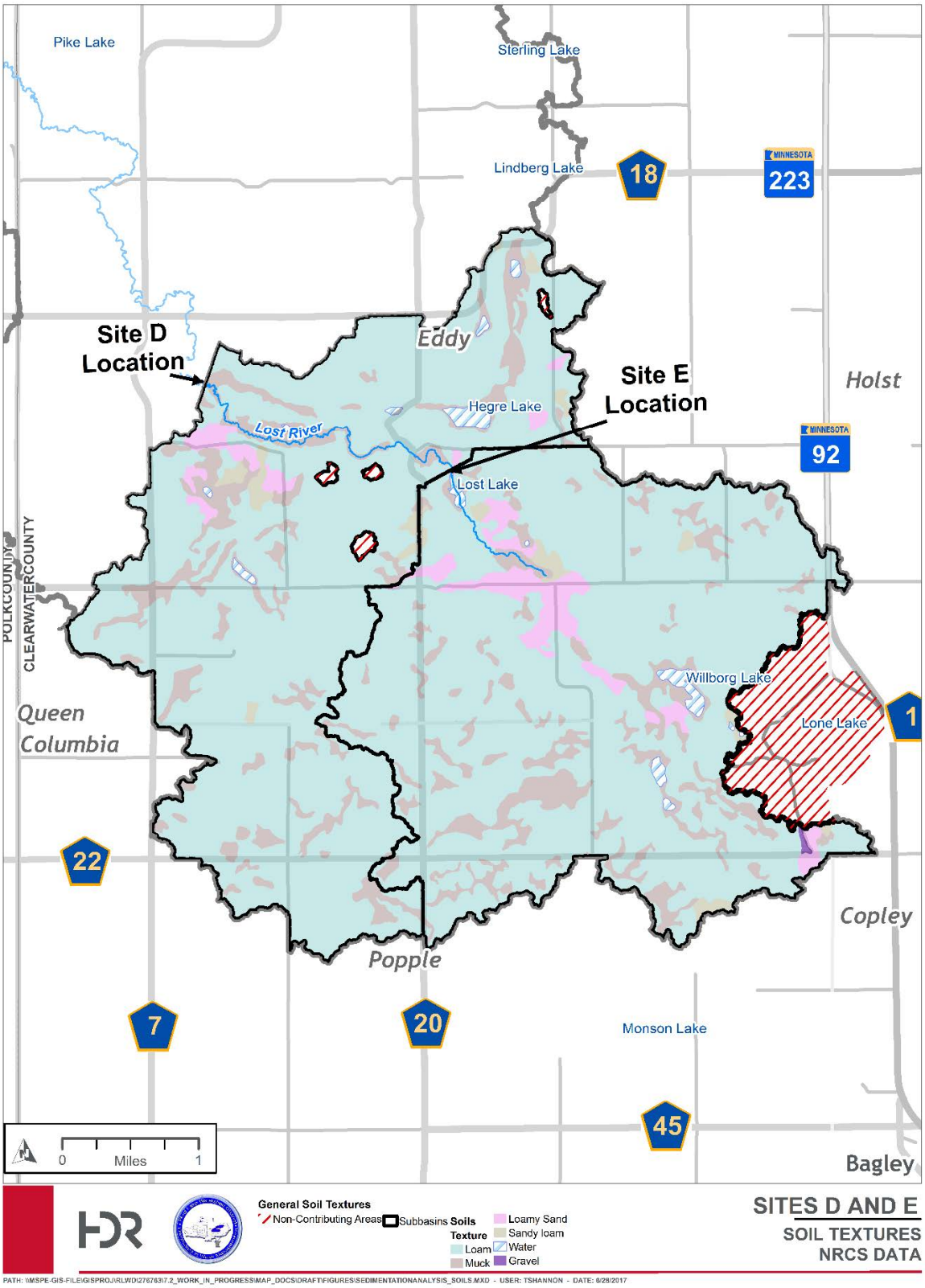

Figure 13. Sites D and E Area Soils

Table 24. Site D Soils Properties

Soil Texture	Area		Median Erosion Factor (k_f)	Median Composition			Average SCS 24-hour Curve Number
	Acres	Percentage		Sand	Silt	Clay	
Gravel	5.7	<1%	0.24	—	—	—	35
Loam	9,582.4	81%	0.35	37%	42%	21%	75
Loamy sand	411.4	3%	0.13	83%	11%	6%	64
Muck	1,566.9	13%	0.32	—	—	—	79
Sandy loam	178.1	2%	0.28	5%	65%	27%	64
Water	108.9	1%	—	—	—	—	93
Total/Average	**11,853.4**	**100%**	**0.34**	**34%**	**48%**	**18%**	**75**

Table 25. Site E Soils Properties

Soil Texture	Area		Median Erosion Factor (k_f)	Median Composition			Average SCS 24-hour Curve Number
	Acres	Percentage		Sand	Silt	Clay	
Gravel	5.7	<1%	0.24	—	—	—	33
Loam	4,880.7	80%	0.35	37%	42%	21%	75
Loamy sand	290.8	5%	0.10	84%	9%	6%	62
Muck	795.8	13%	0.32	—	—	—	80
Sandy loam	91.0	1%	0.28	65%	27%	9%	62
Water	54.7	1%	—	—	—	—	93
Total/Average	**6,118.7**	**100%**	**0.33**	**35%**	**46%**	**18%**	**75**

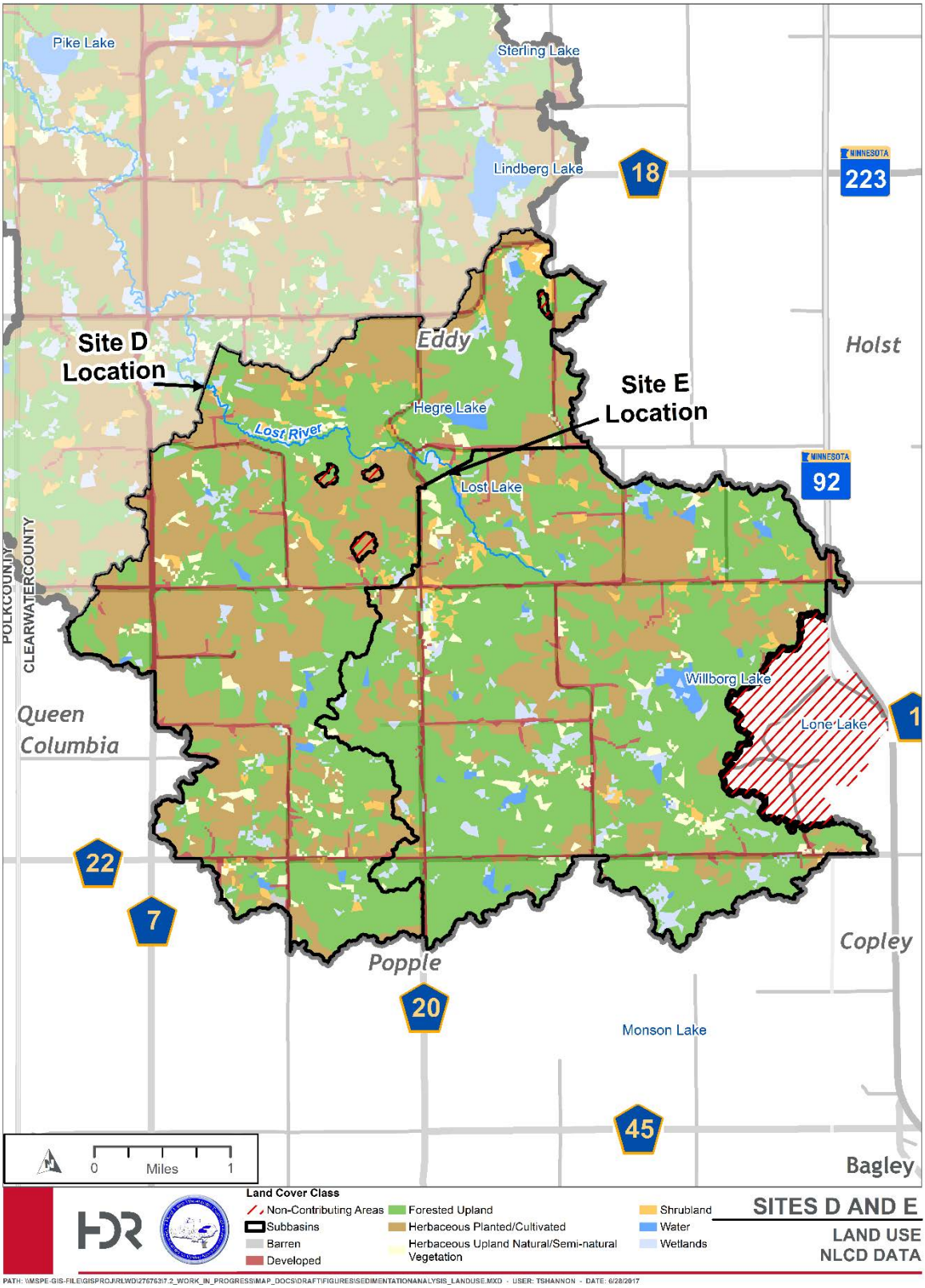

Figure 14. Sites D and E Land Uses

Table 26. Summary of Land Uses

Land Use	Percentage of Area	
	Site D	Site E
Open water	1%	2%
Developed, open space	4%	4%
Developed, low intensity	<1%	<1%
Developed, medium intensity	<1%	<1%
Barren land	<1%	<1%
Deciduous forest	47%	54%
Evergreen forest	7%	7%
Mixed forest	<1%	<1%
Shrub/scrub	2%	2%
Grassland/herbaceous	3%	4%
Pasture/hay	28%	22%
Cultivated crops	3%	1%
Woody wetlands	1%	2%
Emergent herbaceous wetland	2%	3%

Table 27. RUSLE Cover Management Factors

Cover Management Factor (C)	Land Use Types
0	Open water
0.001	Grassland/herbaceous Woody wetlands Emergent herbaceous wetlands
0.002	Deciduous forest Evergreen forest Mixed forest
0.003	Developed, low intensity Developed, medium intensity
0.1	Developed, open space Barren land Shrub/scrub Pasture/hay
0.2	Cultivated crops

Source: International Water Institute, 2016.

8.1.4 Rainfall Average Annual Erosivity

The rainfall average annual erosivity value (R) is the sum of the energy from a maximum 30-minute rainfall intensity for all storms in one year. Previous NRCS mapping of R incorporated earlier droughts and did not include recent trends in increasing rainfall intensities. The R value was obtained for Clearwater County, Minnesota, from the USDA electronic RUSLE2 databases (USDA 2013), which include rainfall data from 1960 to 1999.

8.2 Sedimentation Estimates

The estimate of sediment erosion within the watershed and deposition in a proposed impoundment followed the following steps:

1. Establish future condition scenarios: the extent that existing conditions may change in the future can affect the rate of erosion and sediment transport. Both future conditions with and without the impoundment projects were evaluated.
2. Determine local hydraulic erosion rates: the potential for erosion from rainfall prior to transport further downstream was determined.
3. Determine reservoir sedimentation: the liberated sediment was routed downstream to the impoundment. As sediment travels downstream, a portion can settle prior to reaching the impoundment site. Assumptions were made as to where sediment that reaches the impoundment would settle, and the extent of sediment that would be lost through the outlet structure.

8.2.1 Sedimentation Scenarios

The available land use, soils, and terrain data are existing conditions. Land uses were based on 2011 conditions, the soil surveys were from 1997, and terrain data was collected in 2010. Of these datasets, future land use changes related to pastures, cultivated crops, or homesteads are most likely to directly affect sedimentation rates.

The watershed area in the FWoP or future with project scenarios were assumed to be the same as existing conditions. This assumption was based on two historic trends related to land uses:

- Consistent land use over time: land uses in the watersheds over the last 50 years have shown little change.
- Low population growth: Populations in the townships that include the watersheds have declined.

The projects themselves would be operated as dry-pool impoundments, filling only for a short duration during select flood events. Land uses in the impoundments were, therefore, assumed to be unaltered by the projects.

Consistent Land Use over Time

The NLCD was compared between 2001 and 2011 for the watershed. The watershed above Site D had a slight change (about 1%) from 2001 to 2011 in land uses. For the watershed above Site E, about 2% of the forested area in 2001 was converted into wetlands and open water.

Over a longer timeframe, forested land uses were assessed from aerial photographs from 1960/1961(University of Minnesota 1960) and compared with 2015 (USDA 2015). The Site D watershed had a net loss of 0.1 square mile (less than 1% of the total area), converted from forested area to pasture, cropland, or homestead uses. The Site E watershed had a net gain of about 1% of the total area over this timeframe. Between 3% to 4% of the Sites D and E watersheds converted from forests into wetlands or open water during this timeframe. A review of aerials over a longer timeframe (from 1960 to 2015) also showed few changes in land use. This suggests that future conditions will have similar land use and that no significant changes to erosion potential attributable to land use should exist.

Low Population Pressures

Homestead densities and other land uses associated with increased potential for erosion can be driven by population. The watersheds upstream from Sites E and F consist of land from the Eddy and Popple Townships. These townships have low population densities. Eddy Township’s population

was 322 in the 2000 Census and was estimated at 345 in 2015, or an average increase of 0.4% per year (Minnesota State Demographic Center. 2016). Popple Township had 564 people in the 2000 Census and an estimated 529 people in 2015, or a change of –0.4% per year.

8.2.2 Potential for Hydraulic Erosion

Local hydraulic erosion is estimated soil loss from an area attributable to rainfall and surface runoff, but prior to sediment transport to remote locations in the watershed. Estimates of local hydraulic erosion were calculated using the RUSLE2 regression equation (USDA 2013). Environmental Systems Research Institute ArcGIS version 10.4.1 was used to determine the variables used in the RUSLE2 equation. The ArcGIS process used cells of 30-meter resolution that combined land uses, hillside slopes, and management practices. Figure 15 plots the computed local hydraulic erosion. Areas with higher proportions of natural forests and wetlands and areas with flatter slopes tend to have lower rates of local hydraulic erosion. Higher rates of erosion (up to 21 tons/acre/year) occur in cultivated areas, pastures, and within a corridor of higher hillside slopes about a mile from the Lost River. Figure 16 shows the location of HEL. The HEL areas are mostly within ½ to 1 mile of Lost River. Table 28 shows calculated averages for the local hydraulic erosion rates by land use. Average local erosion rates are 3.5 tons/acre/year for cultivated crops, 2 tons/acre/year for pastures, and less than 1 ton/acre/year for forested land.

A Sediment Delivery Ratio (SDR) regression equation (Maidment 1993) was used to estimate the amount of sediment that, once eroded by the local hydraulic erosion process, would be delivered to the dam locations. The average SDR was 20% to 27% for Site D and 21% to 25% for Site E. A detailed estimate of sediment transport zones is shown in Table 29 and Table 30. Average sediment delivered to Site D is 0.225 tons/acre/year for a total average of 2,667 tons per year. Average sediment delivered to Site E is 0.183 tons/acre/year for a total average of 1,120 tons per year.

During times when the low flow opening of the impoundment outlet structure is closed, no sediment would pass through the low flow conduit, and sediment would have an opportunity to settle upstream of the dam embankment. During times when the low flow opening of the impoundment outlet structure is open, sediment could pass through the low flow conduit. The volumes calculated in this analysis represent an upper bound on the anticipated amount of sediment captured.

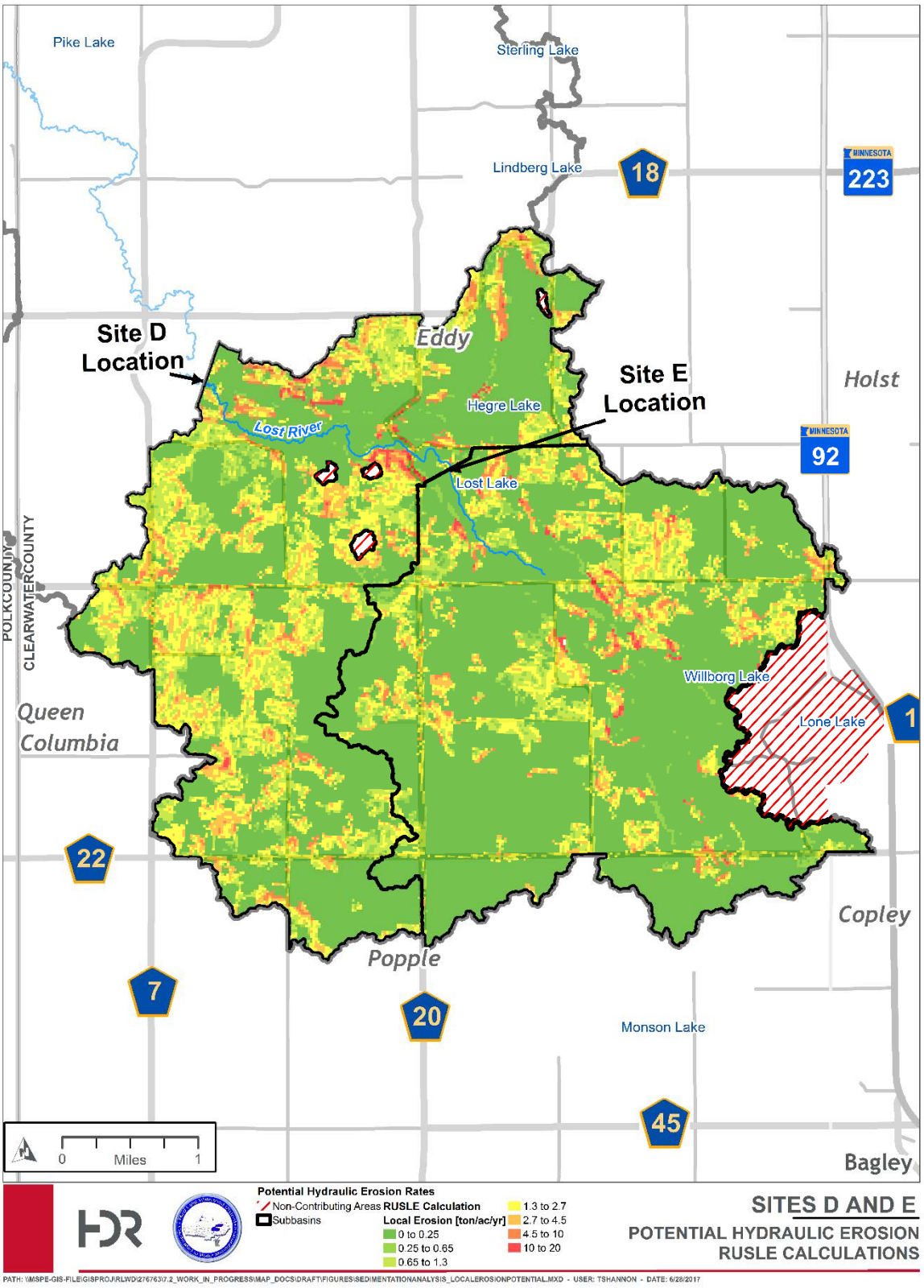

Figure 15. Local Hydraulic Erosion Potential

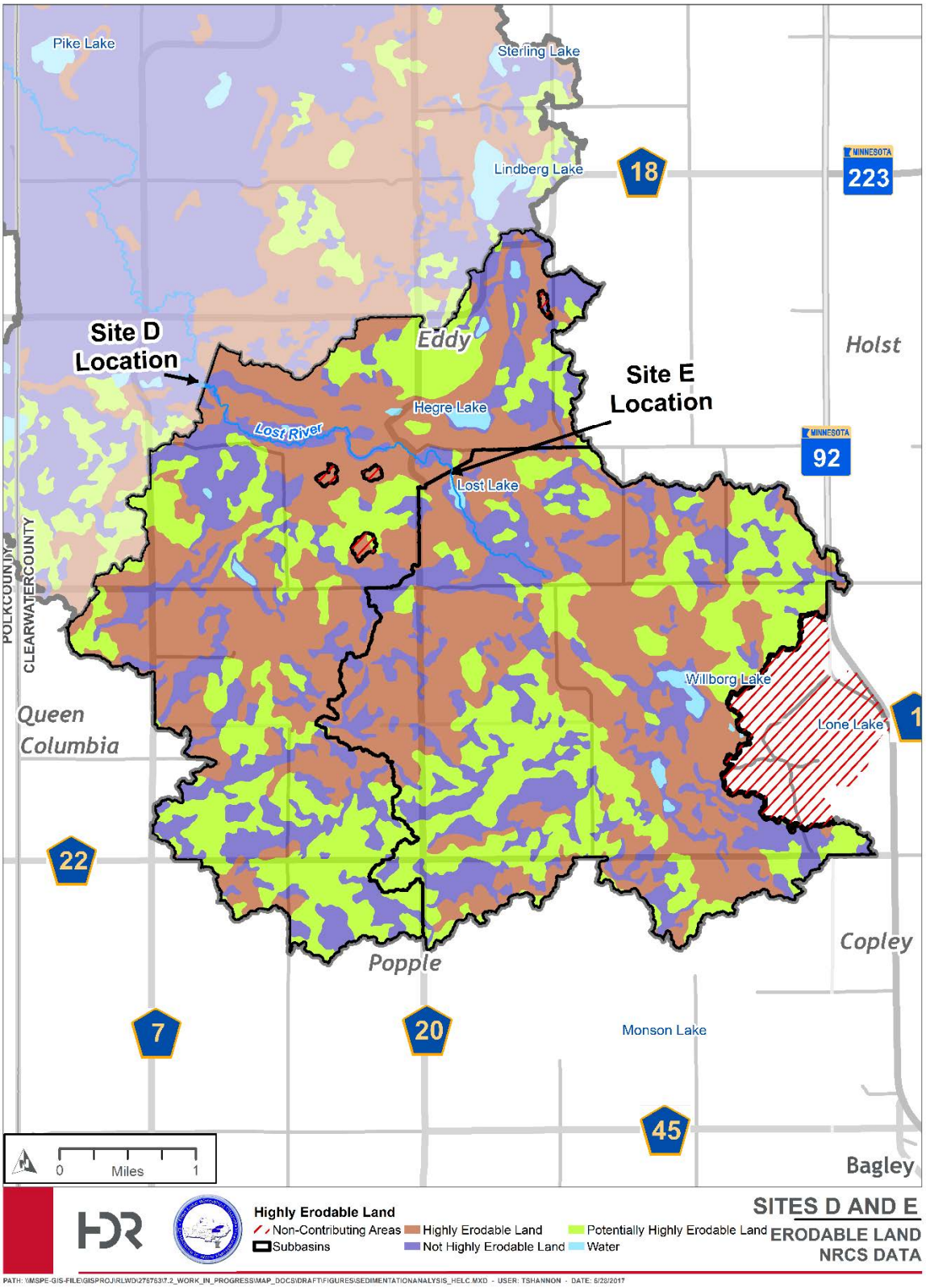

Figure 16. Highly Erodible Land

Table 28. Summary of Local Hydraulic Erosion

Land Use	Local Hydraulic Erosion (tons/acre/year)	
	Site D	Site E
Open water	<1	<1
Developed, open space	<1	<1
Developed, low intensity	<1	<1
Developed, medium intensity	<1	<1
Barren land	2.78	2.28
Deciduous forest	<1	<1
Evergreen forest	<1	<1
Mixed forest	<1	<1
Shrub/scrub	2.09	1.99
Grassland/herbaceous	<1	<1
Pasture/hay	2.05	2.22
Cultivated crops	3.48	3.53
Woody wetlands	<1	<1
Emergent herbaceous wetlands	<1	<1

Table 29. Sediment Delivery Estimates to Site D

Sediment Delivery Ratio to Dam	Area of Capture Zone (acres)	Sediment Delivery Rate (tons/acre/year)	Total Sediment Delivered to Dam (tons/year)
70% to 100%	57.36	0.84	48.2
40% to 70%	336.65	0.53	178.4
30% to 40%	1,559.86	0.42	655.1
25% to 30%	2,358.07	0.24	565.9
20% to 25%	5,418.02	0.19	1,029.4
0% to 20%	2,123.44	0.09	191.1
Watershed Average	**11,853.40**	**0.225**	**2,667.0**

Table 30. Sediment Delivery Estimates to Site E

Sediment Delivery Ratio to Dam	Area of Capture Zone (acres)	Sediment Delivery Rate (tons/acre/year)	Total Sediment Delivered to Dam (tons/year)
70% to 100%	4.89	0.61	2.9
40% to 70%	378.59	0.47	177.9
30% to 40%	941.58	0.24	225.9
25% to 30%	1,592.11	0.25	398.0
20% to 25%	3,171.98	0.10	317.2
0% to 20%	29.55	0.05	1.5
Watershed Average	**6,118.7**	**0.183**	**1,119.7**

8.2.3 Reservoir Sedimentation

Estimated sedimentation pool volumes for Site D and Site E are provided in Table 31. The loam soils of the area are primarily silt and sand with some clay content. Based on dry sediment densities (USDA 1983), the average sediment density is approximately 75 pounds per cubic foot. The projected sediment pool volume using a 100-year design life is 164 acre-feet for Site D and 69 acre-feet for Site E.

Two comparable sources of information were examined for sedimentation data. The U.S. Geological Survey (USGS) Reservoir Sedimentation (RESSED) Database (USGS 2009) contains historic volume surveys of storage projects. The change in volumes over time is assumed to be attributable to sedimentation. Sedimentation data from RESSED for Minnesota projects in the Red River Basin is in Table 32.

The closest projects include a farm stock pond near Winger, Minnesota, and the Frazee, Minnesota, Town Lake. These two projects have smaller storage volumes than the proposed Sites D or E. Drainage areas to these comparable projects range from less than Site E to much more than Site D. Two other projects from RESSED are Lake Bronson and Orwell Dam. Lake Bronson is comparable in storage capacity to proposed Site D or Site E, although with a much larger drainage area. Orwell Dam is much larger in storage volume and drainage area than the proposed Site D or Site E.

The calculated sediment accumulation by weight in Site D or Site E is roughly two to ten times more than from RESSED survey, which provides sediment accumulation by weight. The latest survey data were no later than 1971. The RESSED rates are related to some extent with less intense rainfall in this earlier period of record. Additionally, differences in land uses, soils, and topology could account for these differences. Neglecting sediment transport through the proposed impoundment outlet would contribute to more conservative estimates for Site D or Site E.

Another source of comparison is water quality sampling performed by USGS on at the Lost River at Oklee, Minnesota (USDGS 2016) (USGS Gage Number 05078230). This sampling site is 25 river miles downstream from Pine Lake. Drainage area to this gauge is 254 square miles. Pine Lake regulates 45 square miles of this drainage area.

The average annual flow volume at this site from 1961 to 2016 was 53,225 acre-feet. Eight water quality samples that included suspended sediment concentrations were available from this site. Two samples were taken in 1979, while the remainder was from 2000 to 2001. Suspended sediment concentrations ranged from 36 milligrams per liter (mg/l) to 108 mg/l. The average concentration was 70 mg/l. Sediment rate passing this gauge is 0.03 tons per acre per year. This rate is also less than the calculated rate at Site D or Site E. The RUSLE2 calculations are conservative with respect to, but also within an order of magnitude of, the observed data.

Table 31. Sedimentation Pool Volumes

Item	Site D	Site E
Average sediment delivery (tons/acre/year)	0.225	0.183
Sediment delivery area (acres)	11,853.4	6,118.7
Sediment delivery (tons/year)	2,667.0	1,119.7
Sediment composition (%)		
Sand	34%	35%
Silt	48%	46%
Clay	18%	18%
Sediment density (dry) (pounds/cubic foot)	74.8	74.9
Sediment volume (acre-feet/year)	1.64	0.69
Volume (100-years) (acre-feet)	164	69
Volume (100-years) (inches)	0.0012	0.0009

Table 32. Reservoir Sedimentation (RESSED) Database for Minnesota Projects in the Red River Basin

Reservoir	Distance from Pine Lake (miles)	Total Storage (acre-feet)	Drainage Area (square miles)	Sediment Survey Dates	Sediment Deposits	
					acre-feet/year	tons/ acre/year
Lake Bronson (east of Lake Bronson, MN)	97	3,792	438.5	1940 to 1950	16.6	0.04
Frazee Town Lake (Frazee, MN)	60	155	210.0	1926 to 1952	0.689	0.02
John McWilliams (farm pond southeast of Winger, MN)	24	40	1.5	1958 to 1971	0.034	0.04
Orwell Dam (southwest of Fergus Falls, MN)	100	30,200	1,820.0	1954 to 1970	7.5	0.09

8.2.4 Sedimentation to Pine Lake

The RUSLE calculations were applied to estimate the sediment retention in Pine Lake, with results shown in Table 33. Potential sediment capture by Little Pine Lake and the Wildlife Management Area were not included. Under FWoP conditions, estimated sediment to Pine Lake is 1.75 acre-feet/year. Implementing the Site D impoundment would potentially reduce the sediment delivery to Pine Lake to 0.96 acre-feet/year. Over 100 years, this would reduce the potential sediment volume delivered to Pine Lake by 79 acre-feet. The Site E impoundment would potentially reduce sediment delivery to Pine Lake to 1.43 acre-feet/year. Over 100 years, this would reduce the potential sediment volume delivered to Pine Lake by 32 acre-feet.

Table 33. Sedimentation Scenarios for Pine Lake

Item	FWoP	With Site D	With Site E
Average sediment delivery (tons/acre/year)	0.115	0.111	0.121
Sediment delivery area (acres)	27,512.6	15,659.2	21,393.9
Sediment delivery (tons/year)	3,170.7	1,731.3	2,594.7
Trap efficiency (%)	90%	90%	90%
Sediment volume (acre-feet/year)	1.75	0.96	1.43
Volume (100-years) (acre-feet)	175	96	143

8.3 Sedimentation Summary and Conclusions

The estimated maximum average sediment deposition rates for Site D are 1.64 acre-feet per year and for Site E are 0.69 acre-feet per year. It was assumed that there are no future changes in land uses because, historically, only small land use changes have occurred in the last 50 years. No sediment transport below the impoundment was assumed based on operation of the proposed impoundments only during flood events and assumed maintenance involving ditch dredging and sediment placement within the impoundment. Cultivated and pasture lands within ½ to 1 mile of Lost River were a primary driver of this hydraulic erosion and sediment transport. The sediment pool for a 100-year design life is estimated at 164 acre-feet for Site D and 69 acre-feet for Site E.

9 Opinion of Probable Construction Costs

This section is pending and will be completed for a future submittal.

10 Economics Evaluation

This section is pending and will be completed for a future submittal.

11 Environmental Evaluation

The environmental evaluation (EE) is an NRCS planning process as described in the NRCS National Planning Procedures Handbook. The EE identifies and analyzes the economic, environmental, and social concerns. This planning process is then summarized on the CPA-52 Environmental Evaluation for Conservation Planning form. This EE planning process started with the identification of problems and opportunities and continues through the application and evaluation of the project.

11.1 NE-CPA-52

The CPA-52 has been developed according to guidance found in the NRCS National Environmental Compliance Handbook and policy from the General Manual, Section J. Special Environmental Concerns of the form address the primary laws, executive orders, and policy that are of planning concern. For each of these concerns there is an Evaluation Procedure Guide Sheet that has been developed to assist the planner in determining the status of their project in relation to that particular concern.

For planning purposes of this Watershed Plan Supplement and EA, the CPA-52 has been used for scoping and documentation of concerns and then has been updated as the planning process has proceeded. When a resource concern was found to be not relevant and sufficient rationale is provided, then the concern was eliminated from further consideration. Each of the scoping concerns that are noted in the EA as “Relevant to the Proposed Action” was carried forward to the alternatives analysis. Within Chapter 4 of the EA, scoping concerns were further reviewed to see if they were pertinent to the individual alternatives. Those pertinent concerns were then evaluated for that alternative in Chapter 5, Environmental Consequences.

The CPA-52 form and associated Evaluation Procedure Guide Sheets are provided on the following pages.

12 References

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