Final Report

LeClair/Riverton Valley Irrigation Storage Project, Level II Study

Professional Services No. 0451

Prepared for:

Wyoming Water Development Commission
Cheyenne, Wyoming

Prepared by:
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In Association with:
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Watts and Associates, Inc. – Laramie, Wyoming

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1. INTRODUCTION

1.1 General

This report presents the findings of the LeClair / Riverton Valley Irrigation Storage Level II Feasibility Study. This study develops conceptual designs, cost estimates, and operational plans for the addition of storage reservoirs to the LeClair and Riverton Valley Irrigation Districts. The study was conducted for the LeClair and Riverton Valley Irrigation District under direction and funding of the Wyoming Water Development Commission (WWDC) by States West Water Resources Corporation in association with Apex Surveying, Inc., Gannett-Fleming, Inc., Western EcoSystems Technology, Inc., and Watts and Associates, Inc.

1.2 Description and Scope

This study investigated the economic and technical feasibility of adding new storage reservoirs to the LeClair and Riverton Valley ID for the purpose of capturing and re-regulating flows in the two district’s canals. This study looked at the potential for adding storage to alleviate water shortages during the irrigation season. The scope put an emphasis on collaboration between the two districts to create a synergy where both districts would see benefit. The project required the review of previous studies and the development of conceptual reservoir designs for the sites that indicated reasonable potential for construction. The conceptual designs were to include detailed geotechnical investigations resulting in embankment configuration and design parameters. Components of the geotechnical investigation included surface reconnaissance, subsurface exploration, and materials testing along the dam centerline, abutments, and emergency spillway. An evaluation of potential barrow areas was also conducted. Environmental impacts of the sites were determined and permitting issues were evaluated. Spillway requirements and the associated costs were determined in conjunction with the Dam Hazard Classification of each site. Affected landowners were contacted and informed of project related activities and estimates of land acquisition costs were developed. An in-depth analysis of the ability-to-pay and a benefit cost analysis of the potential storage facilities was conducted.

1.3 Purpose

The purpose of this study was to develop conceptual designs with cost estimates, operational plans, and recommendations for the LeClair and Riverton Valley ID to assist with the determination of the feasibility of constructing re-regulation reservoir facilities. The study was to evaluate the need for additional storage, identify all feasible storage locations, select the preferred alternative sites, conduct thorough investigations of the preferred alternatives, and present detailed alternatives to the two irrigation districts.
2. OVERVIEW

2.1 General

LeClair ID serves approximately 15,000 acres in the Riverton valley area. The diversion is from the Big Wind River located in Section 32, Township 2 North, Range 3 East, W.R.M., Fremont County, Wyoming. The LeClair canal is approximately 31 miles long mainly consisting of unlined earth canal.

Riverton Valley ID serves approximately 8,500 acres in the Riverton valley area. The diversion is from the Big Wind River located in Section 27, Township 1 North, Range 3 East, W.R.M., Fremont County, Wyoming. The Riverton Valley canal is approximately 18 miles long mainly consisting of unlined earth canal. The Riverton Valley diversion is approximately 11 miles down river of the LeClair diversion. The Riverton Valley canal is approximately 160 feet lower than the LeClair canal.

Three irrigation districts comprised of Midvale ID, LeClair ID, and Riverton Valley ID share the same 1906 water right and have entered a tripartite agreement to allocate the water. The agreement allocates one cfs of direct flow per 70 acres of irrigated land to LeClair and Riverton Valley ID when their water right is in priority. Midvale ID is allowed one cfs of direct flow per 70 acres of irrigated land when available in the river and relies on storage in Bull Lake for additional water. The diversion flow rate varies with irrigation demand and available flow in the river, however, under normal conditions (one cfs per 70 acres) LeClair ID typically diverts 220 cfs and Riverton Valley typically diverts 120 cfs for a total of 340 cfs. During flood conditions (two cfs per 70 acres) LeClair ID is allowed to divert 440 cfs and Riverton Valley is allowed to divert 240 cfs.

2.2 Problem Identification

There are typically four times per year when the two irrigation districts experience irrigation water supply shortages. These shortages usually occur in early May when the irrigation districts first begin operation for the season when pre run-off flows in the Wind River are low, early July after the first cutting of hay, early August after the second cutting of hay, and lastly the two irrigation districts historically have late season shortages. When the farmers cut hay they shut down irrigation for approximately two weeks to allow their fields to dry out enough to gain access and cut hay. During these periods system demand decreases and excess water is wasted. The demand peaks and the irrigation districts experience shortages after each hay cutting when irrigation resumes. It would be beneficial to the irrigation districts to release water from storage during these four times of shortage. Several different reservoir configurations are presented in this study to solve these water shortage problems.
3. SITE RECONNAISSANCE

The design goals used to establish the potential storage reservoir sites are as follows: 1) store optimal volume of direct flow, 2) maximum benefit to both irrigation districts, 3) benefit land owner, 4) low environmental impact, 5) cost effective, 6) technically sound, 7) safe. The first step was to quantify the storage volume needed. The LeClair and Riverton Valley Irrigation Districts have historically been short of water several times per year. The first shortage occurs early May when the irrigation districts first start up for the season and pre run-off flows in the river are low. The second and third shortages occur in early July and early August after the first and second cutting of hay. The irrigation districts are again historically short late in the season. It would be beneficial to the irrigation districts to release supplement supply from storage during these times of shortage. The irrigation districts have indicated a need for 30 cubic feet per second (cfs) for 30 days. This equals a storage volume of 1,800 acre-feet (ac-ft). Land status maps that specifically identified land ownership were developed. All tribal land was removed from consideration for development because land purchase is not possible and easements or land trades are not feasible. All potential storage reservoir sites were identified within this land constraint and storage volume criteria.

3.1 Site Identification

Three types of sites were investigated in this study. The first type of site could be located in between the two canals and operate with an exchange between the two irrigation districts. LeClair irrigation district could divert a portion of Riverton Valley irrigation district’s water through their diversion and divert this water along with a match to storage. When supplemental supply is needed LeClair irrigation district could continue taking a portion of Riverton Valley irrigation district’s water. Riverton Valley irrigation district could release their supplemental supply from storage. This type of site would be the most beneficial located near the upper portion of Riverton Valley irrigation district’s system and could benefit both irrigation districts. A second type of reservoir configuration investigated in this study could be located on each irrigation district’s canal and operated independently of the other irrigation district to supply supplemental water to the respective canal. This storage option could be supplied by and deliver water to the same canal. It would re-regulate flows in the canal to better meet irrigation district needs. The third type of reservoir configuration investigated in this study could be located to capture new sources of water available to the two irrigation districts.

Twelve areas were identified as possible reservoir sites. Most areas had several different storage configuration options. The following section describes each potential reservoir site. All potential reservoir sites studied can be seen on Figures 3.1 through 3.6.
FIGURE 3.4
POTENTIAL RESERVOIR SITES 5, 5A, & 5B

LECLAIR/RVID IRIGATION STORAGE PROJECT

STATES WEST WATER RESOURCES CORPORATION

WARNING
IF YOU USE THIS MAP, YOU MUST RESTORE IT TO MAP SCALE.

LECLAIR IRRIGATION DISTRICT

EXISTING RESERVOIR

WETLAND (PEMC)

ARMED FORCES

BOUNDARY

WITHDRAWAL AREA

KINNEAR DRAIN
ELEV. = 534.13'

INIAN RES

RESERVOIR SUPPLY PIPE

SITE NO. 5B

WETLAND (PEMC)

SITE NO. 5A

APPLIED FOR BY:

APEX SURVEYING, INC.
ENGINEERING AND LAND SURVEYING

427 West Adams Avenue, Box 1774
Klamath, Oregon 97601
(541)271-1001

STORAGE

RESERVOIR AREA

DEPARTMENT

STAGE (FT)

ORIG.

DATA:

APPROVED:

21 MARCH 2005

LECLAIR/RVID IRIGATION STORAGE PROJECT

POTENTIAL RESERVOIR SITES

SITE NO.

STORAGE

RESERVOIR AREA

DEPARTMENT

STAGE (FT)

ORIG.

DATA:

APPROVED:

21 MARCH 2005

LECLAIR/RVID IRIGATION STORAGE PROJECT

POTENTIAL RESERVOIR SITES

SITE NO.

STORAGE

RESERVOIR AREA

DEPARTMENT

STAGE (FT)

ORIG.

DATA:

APPROVED:

21 MARCH 2005
Figures 3.7 through 3.20 attached at the end of this section show detailed drawings of each potential reservoir site. Key statistics can be seen in Table 3.1.

Sites 1, 2, 3, & 4 all share the same concept, but vary slightly in storage volume and wetland impacts. Each site is located on small drainages directly above the Riverton Valley canal and could be supplied by diverting water from the LeClair canal and conveying the water down each site’s respective drainage. Water could be released from these sites and delivered directly to the Riverton Valley canal through their respective outlet works. The erosive nature of the soils at Sites 1 through 4 present some geotechnical challenges. The soil permeability will likely require some type of waterstop. These sites could be lined with an impermeable membrane to prevent seepage, however, this type of construction could be expensive and the benefits may not outweigh the costs.

3.1.1 Site Number 1
Site 1 is the site of an existing dam that was breached. This site can be seen on Figure 3.7. The Riverton Valley canal is elevated across the drainage. Only the top portion of the reservoir could be utilized in the canal. This physical constraint would create approximately 5 ac-ft of unusable storage in Site 1. This site could store approximately 60 ac-ft. The embankment would contain approximately 11,000 cubic yards (CY) of material above grade yielding a storage efficiency of 183 CY of embankment per ac-ft of storage (CY/ac-ft). Total embankment volume is dependent upon subsurface condition and influenced by stripping depth. U.S. Fish and Wildlife Service National Wetland Inventory (NWI) maps were overlain on each site to quantify wetland impacts. There is an area of wetland directly downstream of the dam. Site 1 could inundate approximately 0.97 acres of wetlands.

3.1.2 Site Number 2
Site 2 is below Site 7 on the same drainage as shown on Figure 3.8. Site 7 could require a structure at Site 2 to facilitate water delivery to Riverton Valley’s canal. A reservoir at Site 2 was considered to serve this purpose. The drainage at Site 2 passes the Riverton Valley canal through an under-drain. Some unusable storage would be created at this potential reservoir site. Reservoir capacity is limited by the geometry of the drainage. This site could store approximately 87 ac-ft. The embankment would contain approximately 23,000 CY of material above grade yielding a storage efficiency of 264 CY/ac-ft. Site 2 contains approximately 1.9 acres of wetlands.

3.1.3 Site Number 3
Site 3 contains no wetlands according to NWI maps, however, storage volume is significantly limited due to several structures and the reservoir supply is routed across tribal land. This could potentially be a problem, however the Canal Act of 1890 could offer support on this issue. The Canal Act of 1890 was passed to specifically provide for a reservation of right of way for ditches or canals. This act provides a method for ditch companies to obtain a right of way for their ditches and canals and applies to public lands held by the Federal Government, including Indian reservations. Due to the limited storage this site was dismissed from consideration.

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1 Apex Surveying, Inc. (2003). Riverton Valley Diversion Modifications, Level II Study: Prepared for WWDC. Riverton, WY. Appendix F.
3.1.4 Site Number 4
Site 4 could store the largest volume of the four sites and is the favored site among the previous three sites. This site can be seen on Figure 3.9. The high-water line is limited by River View Road. Site 4 could impound approximately 91 acre-feet of water. The embankment would contain approximately 18,700 CY of material above grade yielding a storage efficiency of 206 CY/ac-ft. The NWI map shows 0.88 acres of wetland within the pool inundation area. This wetland area polygon corresponds with the existing reservoir at this site. The existing reservoir built circa 1980 has significantly silted in. Water could be supplied to this site by diversion from the LeClair canal and conveyance down the drainage to the reservoir. This water supply route crosses a 1/4 section of tribal land raising the same issues associated with Site 3. The drainage crosses over the Riverton Valley canal on a flume. Water released from this site could be conveyed down the drainage and diverted at the flume into the canal. Alternatively, water could be piped from the reservoir to the canal.

3.1.5 Site Numbers 5, 5A, & 5B
Site 5 is the site of an existing dam that stores about 3 acre-feet of water coming down the Kinnear drain as shown on Figure 3.10. The concept was to put a bigger facility at this location to take advantage of the supplemental flow in the Kinnear drain, however the high-water line of the existing facility abuts up to the tribal land boundary to the North. This eliminates any further development of this site. The existing embankment and outlet works is in need of rehabilitation. Sites 5A and 5B were identified on a small drainage adjoining the Kinnear drain further up from Site 5 as shown on Figure 3.11. The concept for these two sites would be to store the supplemental flow in the Kinnear drain and to store any return flows coming down the drainage the reservoirs are sited on. A pipeline to draw water from the Kinnear drain before the drain crosses onto tribal land would be required to supply the reservoir to an elevation of 5341’. Site 5A would be the largest facility that could be supplied with Kinnear drain water. Site 5B would be a five foot raise on Site 5A. The only supply for this additional five feet would be return flows coming down the drainage. These sites could store approximately 90 and 197 ac-ft of water respectively. The embankment would contain approximately 42,200 and 69,500 CY of material above grade respectively yielding a storage efficiency of 471 and 354 CY/ac-ft respectively. The sites would inundate 4 and 4.7 acres of wetland and 3.3 and 8.9 acres of farmed field respectively. The outlet works for these sites would have to release water across tribal land for approximately 25 feet to deliver water to the Kinnear drain. The Kinnear drain would then convey the flows across tribal land, through the existing reservoir at Site 5, over the LeClair canal on a flume, and discharge to the Wind River. These flows could be diverted at the flume for use in the LeClair canal or picked up from the Wind River at Riverton Valley’s diversion.

3.1.6 Site Numbers 6, 6A, & 6B
Sites 6, 6A, and 6B are located upstream of Site 4 on the same drainage as shown on Figures 3.12, 3.13, and 3.14. The embankments and storage volumes for Sites 6 and 6A would be constrained by the tribal land boundary to the South and housing development to the West. Site 6A would also be constrained by the tribal land boundary to the East. Both sites would inundate significant areas of wetlands. Sites 6 and 6A would cover 5.2 and 10.6 acres of wetland respectively. Site 6A would also inundate 6.3 acres of farmed field. Both of these sites could be
supplied by diverting water from the LeClair canal. Sites 6 and 6A could store approximately 104 and 458 ac-ft of water respectively. The embankment would contain approximately 25,200 and 120,000 CY of material above grade respectively yielding a storage efficiency of 243 and 262 CY/ac-ft respectively.

Site 6B would be located just above Sites 6 and 6A directly on the LeClair canal and would inundate approximately 43.5 acres of farmed field. Site 6B would be constrained by the tribal land boundary to the South and the wetlands to the West. The LeClair canal crosses the drainage in a flume. There is approximately five feet of drop in the canal associated with this structure. The top five feet of storage (approximately 325 ac-ft) could be delivered to the LeClair canal. Alternatively, the storage volume could be released and conveyed down the drainage and diverted into the Riverton Valley canal. This site could store approximately 1,090 ac-ft of water. The embankment would contain approximately 377,800 CY of material above grade yielding a storage efficiency of 346 CY/ac-ft. This site would inundate approximately 1.6 acres of wetlands.

3.1.7 Site Numbers 7, 7A, & 7B
Sites 7, 7A, and 7B are located upstream of Site 2 as shown on Figures 3.15 and 3.16. Site 7 is constrained by a housing development to the Southeast and would inundate 0.43 acres of wetlands and 1.3 acres of farmed field. This site could store approximately 129 ac-ft. The embankment would contain approximately 37,500 CY of material above grade yielding a storage efficiency of 290 CY/ac-ft. Site 7A is a much larger site constrained by the tribal land boundary to the West and North and a housing development to the East. Site 7A would be supplied by a head gate and pipeline from the LeClair canal. This site would be comprised of two separate embankments and a berm across a saddle between the two embankments. This facility could be expanded by passing water over the saddle and into Site 7B. Site 7A could potentially store approximately 1,440 ac-ft of water. The embankments would contain approximately 352,900 CY of material above grade yielding a storage efficiency of 244 CY/ac-ft. Site 7B could store approximately 146 ac-ft of water. The embankment would contain approximately 47,600 CY of material above grade yielding a storage efficiency of 326 CY/ac-ft. A pipeline from the West embankment parallel to River View Road over to the East drainage would be required to route flows from the West embankment to the East embankment. From there flows would continue down the drainage, through Site 2, and into the Riverton Valley canal. Since the drainage passes the Riverton Valley canal through an under-drain, a structure would be required at Site 2 to divert the supplemental supply from the drainage into the Riverton Valley canal. Alternatively, water released from these sites could be piped to the Riverton Valley canal. Sites 7A and 7B have minimal wetland impacts but site 7A would inundate 38 acres of farmed fields. These sites could pose the risk of significant damage due to a clear weather breach to several downstream structures. Dam Hazard Classification of these sites is discussed in section 4.2.7

3.1.8 Site Number 8
Site 8 is located above the LeClair canal at the site of a breached stock pond as shown on Figure 3.17. At this location the LeClair canal drops a sufficient amount of elevation to allow water stored in Site 8 to be released back into the LeClair canal. This site could be supplied with an existing headgate and lateral that turns out upstream of the canal drop section. The canal drop consists of two sections of concrete lined chutes. This site would have limited wetland impacts.
The reservoir pool would have minimal wetland impacts, however, there are some wetlands identified on the NWI maps below the reservoir. Increased flows in the drainage associated with reservoir releases and a diversion structure to deliver water to the LeClair canal may have limited wetland impacts. Alternatively, water released from this site could be piped to the LeClair canal and the wetland impacts could be avoided. The high-water line for Site 8 would be constrained by the tribal land boundary to the West. This site could store approximately 111 ac-ft of water. The embankment would contain approximately 16,800 CY of material above grade yielding a storage efficiency of 152 CY/ac-ft. This site would serve the LeClair canal and help to re-regulate and augment water shortages in the lower sections of the canal system. This site would likely require a high Dam Hazard Classification due to the risk of potential damage to several downstream structures. Dam Hazard Classification of this site is discussed in section 4.3.7

3.1.9 Site Number 9
Site 9 is located on the Riverton Valley canal at the outlet of the Highway 26 tunnel as shown on Figure 3.18. This site has significant wetland impacts and is limited in size due to the county road to the South, Highway 26 to the Northwest, and a housing development to the East. This site could store approximately 166 ac-ft of water. The embankment would contain approximately 46,400 CY of material above grade yielding a storage efficiency of 279 CY/ac-ft. This site would serve the Riverton Valley canal and help to augment water shortages in the lower sections of the canal system. This site would likely require a high Dam Hazard Classification due to the risk of potential damage to several downstream structures.

3.1.10 Site Number 10
Site 10 is located on Haymaker Draw and is constrained by tribal land boundaries to the East and North. This site can be seen on Figure 3.19. This site would contain approximately 7.4 acres of various types of wetlands and would pose the risk of significant damage due to a clear weather breach to several downstream structures. This site would likely require a Class I Dam Hazard Classification. This site could be supplied with the LeClair canal and release water down the drainage to the Riverton Valley canal. This site could store approximately 281 ac-ft of water. The embankment would contain approximately 32,700 CY of material above grade yielding a storage efficiency of 116 CY/ac-ft. This site is situated low in the system and would benefit irrigators near the end of the canal systems.

3.1.11 Site Number 11 & 11A
Site 11 is a small site located on drainage near the end of the Riverton Valley canal as shown on Figure 3.20. This potential site would be situated directly above the canal. The drainage flows water year round, however, the flow is influenced by agriculture. The drainage flows more water during the irrigation season by conveying return flows. The concept would be to store these flows in the drainage and release them into the Riverton Valley canal. With this sites limited storage its main function would be to divert stream flows into the canal while storing a surge for periods of high demand. This site could store approximately 12 ac-ft of water. The embankment would contain approximately 4,700 CY of material above grade yielding a storage efficiency of 385 CY/ac-ft. It would be possible to fill this reservoir multiple times. Alternatively, a diversion structure could be built at this site eliminating any significant storage while serving the main function of diverting stream flow into the canal. Both of these structures would serve the
Riverton Valley canal and supply a new source of supplemental water to the irrigators on the lower sections of the canal system. This site would have minimal wetland impacts.

Site 11A is located approximately 350 feet upstream of Site 11 as shown on Figure 3.20. This site could store approximately 9 ac-ft of water. The embankment would contain approximately 1,800 CY of material above grade yielding a storage efficiency of 199 CY/ac-ft. This site would be located at a narrower section of the drainage and would require less embankment material, however, the land under the dam footprint was recently sold and is in the process of being subdivided and developed. This site would require a pipeline to deliver supplemental supply to the Riverton Valley canal.

3.1.12 Site Number 12
Site 12 is a potential reservoir site identified near Pilot Butte Reservoir on the Midvale Irrigation District. The site would be on the U.S. Bureau of Reclamation Withdrawal Area. The site would potentially be supplied with Reclamation facilities. The concept for this site would be to divert LeClair Irrigation water through Diversion Dam and convey it through the Wyoming canal to an existing lateral to supply the reservoir. Water would be released from the reservoir and conveyed a quarter mile down the drainage over tribal land to the Wind River. This water would then be diverted at the LeClair and Riverton Valley diversion structures for use in their canals. A meeting with John Lawson, Area Manager, Wyoming Area Office, Bureau of Reclamation on Friday, September 23, 2005 identified several obstacles to this site. First, the land would need to be revocated from withdrawal status. This is the process of returning land back to the previous owner prior to its inclusion in the Reclamation withdraw. This could be difficult if the Tribes seek to have this land turned back over to them. It is unclear what entity would become the new land manager and if it could then be purchased. This first step could be avoided if the land is deeded. Secondly, the reservoir would need to be supplied using Reclamation facilities. It would first have to be proved that there was excess capacity in the Midvale canal system, then through the Warren Act a contract with the Federal government would need to be entered. The Warren Act allows excess capacity in Reclamation facilities to be used for irrigation purposes. This would evoke the Federal NEPA process. A second contract would need to be entered with Midvale ID for maintenance on their canal. Because of these obstacles this site is likely not feasible.
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<td>$2,563</td>
<td>$4,490</td>
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**Note:** The table provides a comprehensive overview of reservoir statistics, including proposed storage capacity, surface area, proposed type of reservoir, borrow material availability, crest elevation, total crest length, maximum dam height, total volume, storage efficiency, inflow design flood, return period, estimated peak inflow, estimated runoff volume, flood pool with no freeboard, flood pool with 1.5’ freeboard, flood freeboard, outlet works, principal spillway, reservoir supply, and total project cost along with the cost per cubic yard of fill and ac-ft of yield.
3.2 Site Alternatives Selection

The potential reservoir sites were screened using input from the project sponsors and a weighted comparison with a variety of inputs. Table 3.1, shown previously, gives pertinent data and allows for comparison of each potential site. The sites were evaluated using the following design criteria: 1) store 1,800 ac-ft of supplemental water, 2) storage shall be located and operated to serve maximum benefit to both irrigation districts, 3) benefit land owner, 4) impact less than one tenth of an acre of wetland, 5) benefits outweigh the costs, 6) technically sound, and 7) compliance with the State of Wyoming’s Dam Safety Laws. Each site was ranked using a weighted comparison as shown in Table 3.2. Sites 8, 7A, 11A, and 7B are the four top ranked potential sites and Sites 11 and 4 tie for fifth ranking. These site rankings compliment the irrigation district’s preferred sites list. The LeClair and Riverton Valley Irrigation Districts held a joint board meeting on November 29, 2005 to discuss reservoir site alternatives. Dick Inberg with Apex Surveying, Inc. was in attendance to answer any project related questions. Reservoir Sites 7A, 7B, 8, 11, 4, and 2 were determined to be the preferred alternative reservoir sites, and the irrigation districts were in agreement to pursue further investigation on these sites.

A single site capable of storing 1,800 ac-ft was difficult to identify. Sites 7A and B and Site 12 are the only two sites identified that potentially satisfy the storage volume criteria. This storage volume is easier accomplished with multiple smaller sites or multiple fills of one or more smaller sites. All but Sites 9 and 10 are ideally located offering storage high in the system to best serve the irrigation districts. These sites offer the most flexibility in operation plan to both irrigation districts. Sites 9 and 10 are located lower in the system and offer less flexibility to the irrigation districts. Sites 8, 11, and 11A, although located lower in the system, are suitably located to serve supplemental supply to their respective canal. They could help to augment shortages near the end of the canal systems. All the potential sites except for sites 7A, 7B, 8, and 11 exceeded the criteria for wetland impacts. This could potentially be a fatal flaw for the sites that inundated significant amounts of wetland due to extensive mitigation and permitting obligations.
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1 - Most Favorable
20 - Least Favorable
4. PREFERRED SITES CONCEPTUAL DESIGN

4.1 Introduction

Conceptual designs were developed for the preferred alternative sites and are presented below. These designs include recommendations on reservoir supply methods, geotechnical and geologic site conditions, embankment layout and sections, outlet works and appurtenances, emergency spillways, dam hazard classification, permitting requirements, land acquisition, and cost estimates. Conceptual designs were developed for Sites 7A, 8, 11 Storage option, and 11 Diversion option. Sites 7B, 4, and 2 were dismissed as preferred alternatives for reasons discussed in the following sections. Preliminary geologic/geotechnical reconnaissance was initiated to begin the design process and is presented below.

4.1.1 Regional Geology

The identified storage sites are situated in the central portion of the Wind River Basin. Published geologic maps for the region (Whitcomb and Lowry, 1968; Keefer 1965) indicate that two geologic units occur in the project area: alluvial deposits, and the Wind River Formation. Alluvial deposits mantel the floor of the Wind River Valley. These deposits include terrace and floodplain deposits, and consist of unconsolidated clay, silt, sand, and gravel. The thickness of the alluvium is variable but generally increases towards the Wind River. Terrace deposits composed of coarse sand and gravel occur in the project area at three different elevations in the valley. These terrace deposits have been dissected by stream erosion and are therefore, discontinuous in aerial extent. One of these terraces is situated 150-200 feet above river level and occurs between the LeClair and Riverton Valley canals.

The alluvial deposits are underlain by the Wind River Formation. The bedrock crops out, or is covered by thin colluvial soils along the margin of the valley. The Wind River Formation consists of an interbedded, flat lying sequence of shale and siltstone with lenticular beds of fine to coarse-grained sandstone. Overall, the formational materials tend to be finer grained towards the center of the basin (Whitcomb and Lowry, 1968). The shale and siltstones are characterized as weak, erodible sediments; whereas the interbedded sandstone tends to be cemented, and more resistant to erosion and forms outcrops and ledges.

4.1.2 Preliminary Site Seismicity Review

The Upper Wind River Basin is characterized by a low level of historic seismicity. The largest recorded earthquake to affect the region occurred on November 3, 1984, and was a magnitude 5.1, intensity VI earthquake that was detected approximately 10 miles northwest of Atlantic City. The earthquake was felt in Lander, Dubois, Atlantic City, and Casper. Residents in Lander and Atlantic City reported cracked walls, foundations, and windows (Casper Star-Tribune, November 4, 1984). This event was one of the largest earthquakes to occur in the southwestern quarter of the state (Case, et al., 2002). This earthquake also is believed to have resulted in cracking at the Worthen Meadows Dam.
Fremont County, Wyoming is situated in Uniform Building Code (UBC) Seismic Zones 1 and 2; Riverton is situated in Seismic Zone 1, with a designated 0.05g (g = gravitational acceleration) to less than 0.1g effective peak acceleration.

Case, et al. (2002) summarized the results of a seismological characterization for Fremont County, Wyoming. Table 4.1 summarizes the potential earthquake loading levels for Riverton derived by various methods. The loading levels are expressed as peak horizontal accelerations (PHA) for the maximum credible earthquake (MCE) assigned for each fault system. Each of these potential earthquake sources are summarized briefly in the following paragraphs.

Table 4.1. Summary of acceleration estimates for Riverton, Wyoming
(summarized from Case et al., 2002)

<table>
<thead>
<tr>
<th>Method</th>
<th>MCE</th>
<th>PHA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deterministic analysis of regional active faults having</td>
<td></td>
<td></td>
</tr>
<tr>
<td>surficial expression</td>
<td></td>
<td></td>
</tr>
<tr>
<td>South Granite Mountain fault system</td>
<td>6.75</td>
<td>0.0375g</td>
</tr>
<tr>
<td>Stagner Creek fault system</td>
<td>6.75</td>
<td>0.06g</td>
</tr>
<tr>
<td>Dry Fork fault system</td>
<td>6.7</td>
<td>0.023g</td>
</tr>
<tr>
<td>Cedar Ridge fault system</td>
<td>7.1</td>
<td>0.04g</td>
</tr>
<tr>
<td>Floating or random sources</td>
<td>6.25</td>
<td>0.15g</td>
</tr>
<tr>
<td>Probabilistic seismic hazard analysis</td>
<td>500 year (10% probability of exceedance in 50 years)</td>
<td>NA</td>
</tr>
<tr>
<td></td>
<td>1000 year (5% probability of exceedance in 50 years)</td>
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</tr>
<tr>
<td></td>
<td>2500 year (2% probability of exceedance in 50 years)</td>
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</tbody>
</table>

The Case et al. study included the deterministic evaluations of two known active fault systems (South Granite Mountain and Stagner Creek) and one suspected active fault system (Dry Fork) having surficial expression in Fremont County. There are no known active faults in the immediate project area. The closest known active structure is the Stagner Creek Fault, which is located near Boysen Reservoir, approximately 27 miles north of Riverton. One additional fault system was included in the study (Cedar Ridge), although the authors stated that there was no compelling reason to believe that this fault system was active. The deterministic analysis was performed by measuring the distance from the area of interest to the closest point on the fault system as a whole. The average magnitude was used for activation anywhere along the fault. The study reported significant earthquake magnitudes (MCEs), ranging from 6.75 to 7.1, but the relatively low potential site accelerations (0.023g to 0.06g) associated with these faults are due to their long distance from Riverton.

Areas having a uniform potential for earthquake occurrence where active faults have not been identified, or where earthquakes can occur on buried faults with no surficial expression, are termed tectonic provinces. “Floating earthquakes” are used to evaluate
the potential ground motions associated with earthquakes in tectonic provinces. These floating earthquakes are assumed to occur randomly within the province. The Case study reported the MCE in the Wyoming Foreland Structural Province would have a magnitude in the 6.0 to 6.5 range, with an average value of 6.25, and a corresponding PHA of 0.15g for Riverton.

The United States Geological Survey (USGS) publishes probabilistic ground motion maps for earthquakes having 500-year, 1,000-year and 2,500-year return frequencies (10%, 5%, and 2% probability of exceedance in 50 years, respectively). Based on these maps, the PHAs in Riverton are 0.07 to 0.08g (intensity V) for 10% in 50 years, 0.1 to 0.15g (intensity VI) for 5% in 50 years, and 0.2g (intensity VII) for 2% in 50 years.

### 4.1.3 Geologic/Geotechnical General

Preliminary geologic/geotechnical evaluations were performed at four preferred dam and reservoir storage sites. The four sites investigated were Sites 4, Site 7A (West and East embankments), Site 7B, and Site 8. In addition, geologic site reconnaissance was performed for two small diversion/re-regulating structures at Sites 2 and 11. The geotechnical work was performed by Gannett Fleming, Inc. of Windsor, Colorado, and Plumley & Associates of Fort Collins, Colorado.

Preliminary geotechnical site investigations for dam sites 4, 7A, 7B, and 8 included site geologic reconnaissance and surface mapping along with preliminary subsurface investigations with backhoe test pits and subsurface borings. Conceptual-level embankment designs have been prepared for feasible sites 7A West, 7A East, and Site 8, including preliminary recommendations for embankment zoning templates and foundation treatments. Conceptual design recommendations are presented in the following sections on a site-by-site basis in sufficient detail to allow preliminary evaluation of material quantities and construction cost estimates.
4.2 RESERVOIR SITE NUMBERS 7A & 7B

4.2.1 General
Sites 7A and 7B are located approximately midway between the LeClair and Riverton Valley canals, north of River View Road. These sites are located in the south half of Section 25, Township 1 North, Range 3 East as shown on Figure 4.1. Access to these sites is from River View Road to the south. The reservoir and appurtenances would all be located on deeded land owned by two different landowners. Currently there is approximately 38 acres of hayfield in production at this site that would be inundated. An old homestead site would also be inundated along with a corral and a small cabin. It is anticipated that the land needed for the dams and reservoirs would be purchased and owned by the two irrigation districts. Easements would be obtained for pipelines and other appurtenances. An alternative to purchasing the land inundated by the reservoir pool would be an easement to allow the current landowners control over access for non-irrigation purposes. All landowners were contacted for permission to survey and conduct study related investigations. No formal appraisals or land or easement purchase negotiations have taken place. Land with development potential is generally more valuable than farmland because growth of the town of Riverton and the surrounding area is limited to non-tribal deeded land. This restraint makes land in general in this area valuable. Land or easement purchase at the site was estimated at $10,000 per acre for use in this study.

4.2.2 Reservoir Supply
These reservoirs would be supplied with a diversion and pipeline from the LeClair canal as shown on Figures 4.2 and 4.3. The diversion point from the LeClair canal would be east of the adjacent tribal land boundary. The diversion structure would consist of a tilting weir check structure and a 36” head gate and pipeline. Water would be piped approximately 1,800 ft to Site 7A East. The pipeline outlet would consist of a riprap apron located near the east embankment. Water would first be stored in Site 7A East and would spill over into Sites 7A West and 7B. Water would have to fill to an elevation of 5115’ in Site 7A East before water would begin to flow over the saddle and into Site 7A West. Site 7B would be supplied through a 2’ inverted slide gate and pipe in the berm between Site 7A and 7B. Water could be supplied to Site 7B when the water level in Site 7A East reaches an elevation of 5118’. An alternative pipeline route shown on Figure 4.4 would supply water directly to Site 7A West if Site 7A East were not built.

4.2.3 Reservoir Sizing
Embankment efficiency could be achieved by dividing one large embankment creating one large reservoir into three smaller embankments creating Sites 7A and 7B. The irregularly-shaped reservoir area of Site 7A would be impounded by several, long embankment sections constructed between topographic highs along the south and west margins of the reservoir area. For purposes of describing the site characteristics and preliminary dam layouts, the site is segmented into two areas: 7A West and 7A East, separated by a north-south oriented bedrock ridge. Consequently two outlet works would be required for Site 7A. Water in Site 7A could be stored to a high water line of 5120’. The combined reservoirs would impound about 1090 acre-feet with no reservoir
stripping, and about 1290 acre-feet with approximately 12 feet (average) of additional excavation in the reservoir areas to provide additional, below-grade storage volume. This site would be constrained by the tribal land boundary to the West and North and a housing development to the East. Site 7B would be located between the two embankments of Site 7A separated from Site 7A East by a berm. Water in Site 7B could be stored to a high water line of 5106’.

4.2.4 Geotechnical/Geologic Investigations
Geotechnical/geologic site investigations and evaluation of site conditions are provided in the following section for these sites.

4.2.4.1 Site Conditions and Geologic/Geotechnical Concerns
Figure 4.5 (Site No. 7A and 7B Geologic Map) is a geologic map for the area that includes Sites 7A West, Site 7A East, and Site 7B. The site plan shows the locations of seventeen exploratory test pits (TP-1 through TP-17) and 2 borings (DH-1 and DH-4) that were used to investigate the subsurface conditions at these sites.

Sites 7A and 7B are situated on a gentle south-sloping hillside that is underlain by Tertiary Wind River formation (Tw) at varying depths below a mantle of alluvial (Qalo) and terrace (Qt) deposits. Bedrock is exposed on the side slopes of ridges and knobs that protrude above the valley-bottom alluvial sediments. Remnants of older gravelly and cobbley terrace deposits are found on the tops and lower benches of the exposed bedrock ridges. The dam embankments are laid out to take advantage of the bedrock highs to the degree practical to minimize embankment volumes between bedrock abutments.

Site 7B
Site 7B is a small reservoir site situated between the ridges that separate Site 7A West and 7A East. This site was explored by surface geologic reconnaissance and mapping, and by excavating five test pits (TP-1 through TP-5). A geologic profile along the dam centerline can be seen on Figure 4.6 (Site No. 7B Geologic Profile). Based on the field geologic reconnaissance, Site 7B was determined to be inefficient due to the presence of a large, resistant bedrock knob occupying the middle part of the proposed reservoir area. This bedrock knob takes up a large percentage of the potential reservoir storage volume, and would be difficult and costly to excavate in order to develop more storage. Site 7B was therefore not considered further.

Sites 7A West and 7A East
Figure 4.7 (Site No. 7A West Geologic Profile) shows a geologic profile along the centerline of the western dam embankments. This site was explored by excavating 8 test pits (TP-6 through TP-12, and TP-16), and one 69-feet deep boring (DH-1).

Figure 4.8 (Site No. 7A East Geologic Profile) shows a geologic profile along the centerline of the east dam embankment. This site was explored by excavating 4 test pits (TP-13 through TP-15, and TP-17), and one 30-feet deep boring (DH-4).
Three distinct geologic units were encountered. These units, and potential geotechnical uses or concerns associated with each, are described as follows:

**Older Alluvium (Qalo):** These deposits mantle the bedrock in the reservoir bottom, and range in thickness up to over 26 feet near the middle portions of both sites. Based on field observations and Standard Penetration Test (SPT) blow counts conducted in both test borings, these deposits generally are comprised of loose to very loose, silty fine sands and fine sandy silts. Representative gradation curves for these soils are presented in Appendix C (Geotechnical Lab Test Results). These non-plastic, loose, fine-grained soils are particularly vulnerable to surface erosion when exposed without vegetative cover, and to internal erosion and piping under seepage forces. In their natural, in-place, loose to very loose state, these soils are highly susceptible to potential liquefaction and loss of shear strength under moderate earthquake shaking. Compaction test results on these materials (presented in Appendix C Geotechnical Lab Test Results) indicate that these materials can be significantly improved and densified by applying compactive effort. Compacted samples also exhibit low permeability characteristics (approximately 3 \times 10^{-6} \text{ cm/s}). As such, when properly compacted, these materials are considered suitable for use as “impervious” embankment fill.

**Terrace Deposits (Qt):** Alluvial terrace deposits are present on the ridge tops and on lower-elevation bedrock benches above the valley bottom. Field logs indicate that these deposits have variable gradation characteristics, but are generally characterized as coarse-grained (sands and gravels) ranging up to mostly large cobble-sized particles in some areas. Cobbles ranging up to 1-foot in a clast-supported matrix with sand and gravel were encountered on the lower-elevation bedrock benches from Stations 2+00 to 3+00, and Stations 5+25 to 6+25 on the west dam alignment (see Figure 4.7 Site No. 7A West Geologic Profile). Laterally extensive terrace deposits up to over 15 feet thick are present on top of the left abutment ridge. With the dam crest set near the ridge top, a seepage cutoff may be required to prevent excessive seepage through these materials at higher reservoir pool elevations. Terrace gravels and cobbles provide a potential local borrow resource for erosion-protection surfacing on the slopes of embankments and cut slopes constructed from the finer-grained materials.

**Wind River Formation (Tw):** Generally flat-lying sequence of sandstone, mudstone/claystone, and siltstone. Based on core samples from DH-1, the bedrock appears to be weak, friable and weathered to depths up to about 10 to 15 feet below the bedrock surface. Fractures appear to be generally tight and infilled. Rock quality and degree of weathering improve with depth. This unit is judged to be suitable for the dam foundation with minimal treatment other than a shallow seepage cutoff through the upper weathered zone.
4.2.4.2 Recommendations

Site 7A East and 7A West are suitable for dam and reservoir construction, with the following specific considerations and recommendations for foundation treatment and embankment materials:

- **Excavate and replace alluvial materials in the dam foundations:** Loose, silty fine sand and fine sandy silt materials should be excavated down to bedrock, and re-compact ed in the dam foundations. In their existing, in-place condition, these materials are judged to be weak, subject to significant deformation and settlement under loading, and vulnerable to liquefaction and loss of shear strength under moderate earthquake shaking. However, laboratory test results indicate that these materials will compact well, and can be densified with standard compactive effort.

- **Dewatering Requirements:** Groundwater was encountered in the alluvial sediments, at a depth of 19 feet below ground surface in DH-1, and at 15 feet below ground surface in DH-4. Deep required excavations in the dam foundations for both Sites 7A West and 7A East will require dewatering of the alluvial sediments by either well points or by constructing drainage trenches and sumps. It is recommended that the contractor be required to submit a dewatering plan for approval as part of the construction contract.

- **Alluvial materials are suitable for dam embankment construction:** When properly compacted, the silty fine sand and sandy silt alluvial soils have low permeability (on the order of $k_{\text{vertical}} = 10^{-6}$ cm/s), and can be used to construct “impervious” zones for a zoned or homogeneous embankment dam. Shear strength was not tested directly, but based on laboratory compaction behavior, gradations, and the non-plastic characteristics of these materials, it is anticipated that shear strengths (effective friction angles) of the compacted fill will be on the order of $\phi' = 30^\circ$ to $35^\circ$. Preliminary embankment slopes should be based on these assumptions for shear strength.

- **Seepage cutoff required for terrace gravel deposits in the left abutment of 7A West Dam:** A positive seepage cutoff should be constructed through coarse-grained gravels, sand, and cobbles on the upper left abutment ridge. Limited-volumes of terrace gravels and cobbles on lower-elevation bedrock benches under the 7A West embankment should be removed and replaced with “impervious” embankment fill.

- **Provide a shallow seepage cutoff through the upper, weathered zone of bedrock:** The top 10 to 15 feet of the bedrock appears to be friable, fractured and weathered. A seepage cutoff should be provided through this zone under the dam. Extensive grouting is not considered necessary, as rock quality improves with depth, and fractures below the weathered zone appear to be tight and in-filled. The upper bedrock is considered to be a suitable foundation for the embankment without over-excavation or special treatment requirements.
4.2.5 Embankment and Foundation Design Recommendations

4.2.5.1 Design Considerations

Based on the geologic site investigations described in Section 4.2.4, and other sources of information, the key considerations for economical embankment design and foundation treatment requirements for Site 7A are summarized as follows:

1. Materials available for embankment construction are predominantly fine-grained granular soils (silty fine sand and sandy silt). More limited quantities of coarser materials (gravelly sands, gravels and cobbles) are available on higher elevation bedrock ledges and ridge tops above the valley section.

2. Loose, saturated fine sandy soils up to 26 feet thick above bedrock in the dam foundations should be excavated and re-compacted to reduce compressibility, increase shear strength, and reduce risk of instability or severe deformation under seismic shaking.

3. The upper 10 to 15 feet of the Wind River Formation bedrock is weathered, friable and fractured, and requires a shallow cutoff to control seepage. Bedrock quality in the vicinity of DH-1 (dam site 7A West) improves with depth, and fractures and bedding-plane joints are generally tight and in-filled. Based on the preliminary geotechnical findings, it is assumed that deep foundation grouting will not be necessary.

4. Good quality, durable riprap is not available locally. The closest sources for durable, angular, quarried hard rock are over 30 miles from the site, and unit costs for this material are estimated to be extremely high (over $70/cubic yard). However, a local aggregate supplier was consulted who could supply large, rounded alluvial oversize materials, ranging in size from 4 inches to 4 feet. Preliminary riprap sizing analyses indicate that the oversize alluvial cobbles and boulders would be suitable as a potential riprap source for sites with moderate erosion-protection requirements (small fetch distances and limited wave action).

5. Alluvial gravel and sand resources are locally abundant in the Riverton area. It is assumed that local aggregate processing facilities can supply good-quality granular filter and drain materials to required specifications for special zones at reasonable cost.

4.2.5.2 Preliminary Dam-Type Alternatives Considered

Three preliminary embankment alternatives were considered for the Site 7A dam section. These alternatives were as follows:

Exposed Upstream Geomembrane-Faced Sand/Silt Embankment: This design was considered because the exposed geomembrane facing would provide a reliable, robust water barrier, and would eliminate the need for potentially costly riprap upstream slope protection. A photo of this type of system that was used in a recent installation on a sand dam in harsh (freezing and windy) climatic conditions at a site in Iran is shown as Figure 4.9. Preliminary costs for this system were requested from an experienced manufacturer/installer (CARPI http://www.carpitech.com/ ). Although the upstream geomembrane option would be a reliable and technically effective alternative, the costs
for this alternative were found to be much higher than a conventional zoned embankment with riprap upstream slope protection. The estimate from CARPI for the geomembrane facing alone was $1.4 million (this would be in addition to the embankment fill costs). Therefore, this alternative was not pursued further.

Figure 4.9. Exposed Geomembrane System (CARPI) Installed on a Sand Dam Located at an Extremely Cold and Windy Site in Iran (this system was not selected for preliminary design due to higher estimated construction costs)

**Full-Height Central Concrete Core Wall:** This design alternative would use a concrete core wall extending from the embedment depth in the rock foundation to the full height of the reservoir, within the center of the embankment dam. The thin core wall would provide an internal “membrane” waterstop for a pervious sand embankment. The wall could be constructed by either forming and placing conventional reinforced concrete in panel segments that are brought up incrementally, with backfill placed on both sides of the wall; or by pre-placing the fill and then trenching back down through the fill and foundation rock and placing unreinforced, plastic concrete (concrete admixed with bentonite) in the trench. The trenching construction method could be done either in one, slurry-filled trenching operation, or in shorter vertical fill segments that could be trenched without slurry. This alternative was not selected when the preliminary laboratory test results indicated that the re-compacted silty sand fill materials would be sufficiently impervious such that a full-height membrane waterstop would not be necessary.

**Partial Concrete Core Wall with Broad Central Filter/Drain (Recommended Alternative):** The preferred alternative that was advanced for preliminary design uses a partial-height plastic concrete core wall through the foundation in the center of the dam, with a broad (8 feet wide) central vertical filter/drain extending from just below the top of the core wall to the maximum water surface elevation (5120 feet). The design also incorporates a high-capacity blanket drain under the downstream zone, extending from the central filter/drain to the downstream toe. Upstream slope protection would be riprap derived from locally-obtained (Riverton-area) alluvial oversize cobbles and boulders. The details and basis for this preliminary design template are described in the following section.
4.2.5.3 Dam Zoning Template and Foundation Treatment Recommendations - Main Dam Embankments: 7A East Dam (entire length), and 7A West Dam (Sta 0+00 through approximately Sta 11+00)

Conceptual dam templates were developed for the main embankments at Site 7A East and 7A West, as shown on Figures 4.10 through 4.12. The embankment design is intended to best utilize available on-site materials, and to accommodate anticipated foundation conditions to the best of our current understanding of the site conditions based on the preliminary geologic investigations. Key features of the design are as follows:

**Main dam embankments**: Main dam embankments would be constructed from compacted silty fine sand and sandy silt derived from stripping excavations in the reservoir pool areas and dam foundations. Main dam embankments have 3H:1V upstream slopes and 2.5H:1V downstream slopes; maximum heights above final grade of about 58 feet (west dam) and 35 feet (east dam); and 20 feet crest widths at elevation 5125 feet. The east dam is approximately 1400 feet long, and the west dam, main section is approximately 1100 feet long. (Note: The transition/closure section for the west reservoir is based on a different design template that is described in the next section of this report.)

The 7A West main dam section between Stations 3+60 and 4+80, is the highest embankment section on the project (58 feet). A cobble toe berm is included in this section, as shown on Figure 4.10 (7A West Dam Section at Sta 4+00). The cobble material would be derived from required stripping of the local terrace deposits on the lower benches at that site.

**Foundation stripping**: Anticipate complete excavation to bedrock, up to 26 feet deep under the main dam embankments. The purpose of this stripping is to remove and re-compact loose silt and sand in the valley sections, and to remove porous gravel and cobble terrace deposits at higher elevations on the abutments. Foundation stripping under the main dam sections between about Stations 6+50 and 12+50 on the east dam, and between about Stations 3+60 and 4+80 on the west dam will require dewatering of the alluvial sediments.

**Plastic Concrete Foundation Core Wall**: A plastic concrete core wall would be constructed in the main dam foundations for both 7A West and 7A East. The wall also would be extended approximately 250 feet upstream from the left abutment of 7A West dam to cut off high-elevation gravel terrace deposits on the ridge top. The wall would be keyed 5 to 15 feet into bedrock, and would extend vertically up to the approximate elevation of the natural ground surface. Wall width would be (nominally) 2 feet. Plastic concrete is a high slump, low strength concrete mixture of cement, aggregate, water and bentonite. The mix is slow to set and is soft enough to excavate for several days to a few weeks after its initial placement. It is anticipated that the wall could be economically and simply built by trenching without slurry through limited thicknesses (estimated 8 to 10 feet, typical) of pre-placed fill, excavating down a few inches into the top of the
SITE NO. 7A - EAST DAM TYPICAL SECTION
MAXIMUM SECTION AT STA. 9+00
(TYPICAL FOR STA. 6+00 TO 10+50)

SITE NO. 7A - EAST DAM TYPICAL SECTION
SECTION AT STA 11+00
(TYPICAL FOR STA 10+50 TO 12+25)
previously placed wall section below to form a clean “joint”, and placing the plastic concrete mix into the excavated trenches from ready-mix pump trucks. The in-place wall will be essentially impervious, resistant to cracking, and “self-healing” along construction joints and cracks.

**Vertical Chimney Filter/Drain and Downstream Blanket Filter/Drain**: An 8-feet wide central filter/drain would be constructed in the main dam sections, extending from an elevation about 2 feet below, and enveloping, the top of the core wall up to elevation 5120 (normal high water line). Blanket drains would extend from the vertical chimney sections to the downstream toes. The filter/drain system serves two key functions, as follows:

1. The vertical filter/drain material gradation is specifically designed to serve as a “crack-stopper”, providing protection against piping and internal erosion of the fine sandy/silty embankment soils in case the embankment develops cracks due to differential settlement, hydrofracturing, or seismic loading. If a crack or multiple cracks were to develop for any reason, the filter gradation is designed to stop the erosion of fines (scour) caused by high pressure, high velocity flows within the crack. Erosion is stopped at the filter face, thus preventing cracks from widening and further eroding under long-term seepage forces. A properly-designed filter will plug cracks at the filter face, and will not propagate cracks through the filter zone.

2. The gradation of the filter/drain zone is designed to be more pervious than the surrounding fill. It is intended to serve as an internal drain with sufficiently high capacity to collect all seepage from the upstream embankment zones, and through the foundations (seepage through and under the core wall). The blanket drain is designed to safely convey anticipated seepage discharges within the drain zone to the downstream toe drain. This prevents a phreatic line from developing in the downstream fill zone and keeps seepage from emerging on the unprotected downstream face of the dam.

Gradation design for the Site 7 Chimney and Blanket Filter Drain Zones (Zone 2) is illustrated on Figure 4.13 (Site 7A Filter/Drain (Zone 2) Gradation Design). The design is based on procedures outlined in NRCS (1994). Filter design control points are shown as red circles. The critical design criteria are the filtering and permeability control points. A suitable filter material for this base soil can be obtained from standard fine aggregate gradations (ASTM C33-modified2 or WYDOT Fine Aggregate for Concrete). Although these gradations are more uniformly graded than is allowed by the filter design control points, these gradations meet the critical (filtering and permeability) criteria on the D15 sieve size, and are considered acceptable for this base soil.

The design intent for the filter/drain on the basis of hydraulic conductivity (k), is that k of the filter/drain zone should be several times higher than k of the base soil. This was a

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2 The standard gradation specifications for ASTM C33 fine aggregate must be modified to include an additional requirement on the maximum percentage passing the No. 200 sieve ≤ 3%. This modification to the standard gradation specification insures that the filter/drain has adequate hydraulic conductivity (permeability).
special concern for the Site 7A dams due to the granular characteristics of the Zone 1 fill. A representative compacted sample of the base soil (Site 7, TP-15 silty fine sand) was tested using a falling head permeability test. The test indicated that a relatively low hydraulic conductivity of \( k = 2.2 \times 10^{-6} \text{ cm/s} \) could be achieved following compaction of the material. The test sample was compacted to 95% relative density based on standard Proctor compaction (as tested dry density = 104.9 lb/ft\(^3\), based on maximum dry density = 110.4 lb/ft\(^3\)). Laboratory test results are provided in Appendix C.

To evaluate the permeability requirements for the filter/drain, the design filter gradation bands (ASTM C33-modified and WYDOT fine aggregate) were compared with published data correlating hydraulic conductivity \( (k, \text{ in cm/s}) \) and soil gradations on similarly graded sand and gravel filters. A comprehensive study was done in the 1970’s-1980’s by researchers at the former Soil Conservation Service Soil Mechanics Laboratory (Sherard et al., 1984; SCS, 1978). Two gradation curves for soils tested in those studies are provided on Figure 4.13 (labeled SCS #3 and SCS #4). These SCS soils have similar gradations to the fine side of the filter band for the fine aggregate material specifications, and would therefore represent a conservative estimate for hydraulic conductivity of those materials. As indicated on the figure, the average hydraulic conductivity of the SCS soils which are similarly graded as the specified filter/drain zone are on the order of about 9 \( \times 10^{-3} \) to 1 \( \times 10^{-1} \) cm/s, depending on density. These values are several orders of magnitude higher than the hydraulic conductivity of the silty sand base soil (Zone 1), which had a measured (vertical) laboratory permeability of \( 2.2 \times 10^{-6} \) cm/s.

Preliminary seepage analyses were completed to evaluate the effectiveness of the filter/drain to capture and convey the anticipated seepage through the embankment. Steady state seepage analysis at maximum reservoir pool elevation was modeled for the maximum section of the embankment (see Figure 4.10, 7A West, Section at Sta 4+00). For the analysis, it was assumed that the Zone 1 fill had a 10:1 horizontal to vertical anisotropy with respect to hydraulic conductivity. The preliminary analyses results are provided in Appendix B (Preliminary Dam Design Analyses – Site 7 Seepage Modeling). The seepage models indicate that a small amount of excess pore pressure would build up in the downstream zone for a 3-feet thick blanket drain having \( k = 2 \times 10^{-2} \) cm/s. This problem would be remedied by either increasing the blanket drain thickness, or by increasing the hydraulic capacity of the blanket drain by incorporating a 6-inch to 1-foot thick internal zone of clean drain rock (gravel), “sandwiched” between layers of the filter (Zone 2). The preliminary seepage results indicate that a 3-ft thick blanket drain having a higher hydraulic conductivity \( (k = 3 \times 10^{-1} \text{ cm/s}) \) that could be achieved with a central drain layer would have adequate capacity to convey the seepage without pore pressure buildup in the downstream zone. It is recommended for final design that additional sampling and testing be done to verify the compacted base soil (Zone 1) hydraulic conductivity. Additional seepage analyses should also be done to confirm these preliminary results and to optimize the blanket drain design.

**Upstream Riprap Slope Protection:** Preliminary riprap design was based on procedures outlined by the National Resources Conservation Service (formerly Soil Conservation Service) (SCS, 1983). Refer to Appendix B (Preliminary Design Analysis – Riprap
Design Calculations) for the details of the analysis. The design procedure generally involves the following steps:

1. Evaluate Design Wind Direction: This was done using climate data published by NOAA Weather Service for Riverton (Normals, Means, and Extremes, available at web site [http://www.crh.noaa.gov/riw/climate/clm/lcdcpr.php](http://www.crh.noaa.gov/riw/climate/clm/lcdcpr.php)). The speed and direction (azimuth in degrees clockwise from north) for the “fastest mile” wind for each month was tabulated in a spreadsheet calculator. The design wind direction is computed as:

\[
Design\ Wind\ Direction = \frac{\sum (Wind\ velocity)(Azimuth)}{\sum Wind\ velocity}
\]

The design wind direction for Riverton was determined to be from the west-southwest, at azimuth 259° (as illustrated below).

![Design Wind Direction Diagram](image)

2. Evaluate Effective Fetch \((F_e)\): When the design wind direction is generally towards the upstream face of the dam, the effective fetch is determined by assigning the central radial across the reservoir open water to the face of the dam in the same direction as the design wind direction. For Site 7A, however, the design wind direction is generally away from the upstream faces of the main embankments. For these conditions, the central radial is laid out from a point on the face of the dam to a point on the opposite shoreline in the direction that yields the longest distance over
open water. The effective fetch by this method for Site 7A West (longest fetch for both embankments) was $F_e = 0.17$ mile.

3. Estimate Design Wind Velocity ($U_d$): The design wind velocity was estimated by considering three methods, as summarized on Table 4.2.

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<th>Velocity (mph)</th>
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<td>1 – Ave. “fastest mile” from climate data</td>
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</tr>
<tr>
<td>2 – Generalized map data (Figure 4 in SCS, 1983)</td>
<td>80</td>
</tr>
<tr>
<td>3 – Maximum speed of record, adjusted for duration and effective fetch</td>
<td>59</td>
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</tbody>
</table>

A design wind velocity of $U_d = 70$ mph was chosen. No correction was made for over-water versus overland velocity due to the small fetch.

4. Determine Significant Wave Height ($H_s$): Significant wave height is determined from $U_d$ and $F_e$, using a nomograph provided in SCS (1983), as $H_s = 1.5$ feet.

5. Rock size: The required rock size is determined from $H_s$ and the embankment slope ($\alpha = 18.43^\circ$ for 3H:1V upstream slope) as follows:

$$W_{50} = \frac{19.5 G_s H_s^3}{(G_s - 1)^3 \cot \alpha}$$

where: $W_{50} =$ weight (lbs) of the median rock size. It was assumed that the specific gravity of the rock, $G_s = 2.65$. The median rock size was calculated as $W_{50} = 12.9$ lbs. Because the anticipated source for riprap in Riverton will be rounded to subrounded alluvial cobbles and boulders, this median size was increased by 1.5 times. The selected design median weight was therefore assumed to be $W_{50} = 19.4$ lbs. The average size of rock for this weight and specific gravity is about $D_{50} = 6$ to 7 inches.

6. Riprap and riprap bedding gradations and minimum thickness: Figure 4.14 (Site 7A Riprap (Zone 4) and Riprap Bedding (Zone 5) Gradations) shows the design gradation bands for Riprap (Zone 4) and Bedding (Zone 5). Using the recommended gradation for uniformly graded rock, the recommended riprap gradation specification is provided on Table 4.3.
Table 4.3. Riprap (Zone 4) Gradation Recommendations  
Site 7A

<table>
<thead>
<tr>
<th>% Passing</th>
<th>Size (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>9 - 12</td>
</tr>
<tr>
<td>50</td>
<td>6 - 9</td>
</tr>
<tr>
<td>15</td>
<td>4 – 6</td>
</tr>
</tbody>
</table>

Riprap bedding: The bedding gradation must be compatible with both the riprap gradation and the underlying Zone 2 filter layer. The gradation shown on Table 4.4 and on Figure 4.14 represents clean graded gravel that meets these criteria.

Table 4.4. Riprap Bedding (Zone 5) Gradation Recommendations  
Site 7A

<table>
<thead>
<tr>
<th>% Passing</th>
<th>Size (inches) or Sieve Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>3 – 6</td>
</tr>
<tr>
<td>50</td>
<td>¾ – 1 ½</td>
</tr>
<tr>
<td>15</td>
<td>No. 8 - ¾</td>
</tr>
<tr>
<td>3 (max.)</td>
<td>No. 200</td>
</tr>
</tbody>
</table>

Minimum thickness of the riprap layer (Zone 4) should be = maximum rock size = 12 inches. To provide additional protection due to the expected rounded characteristics of the rock, and the fine-grained characteristics of the base soil, it is recommended that the total thickness of the upstream slope protection (riprap and bedding) be 24-inches. Because of the fine-grained base soil (Zone 1), a layer of Zone 2 Filter/Drain material also will be needed to separate the riprap bedding from the Zone 1 fill.

7. Freeboard Requirement: Wind setup (S) and wave runup (R) are the principal design criteria used to determine freeboard requirements. These parameters are defined on Figure 4.15.

![Figure 4.15](image)

**Figure 4.15. Definition of parameters used to evaluate freeboard requirement**

The wave length (L) is determined from a nomograph (Figure 11 in SCS, 1983) from $F_e$ and $U_d$. For Site 7A West, the wave length $L = 20$ feet. The ratio $H_s/L = 1.5/20 = 0.075$. 

4-26
From the design manual (Figure 13 in SCS, 1983), the relative runup ratio $R/H_s = 1.23$; and $R = 1.85$ feet. This is for angular stone. The guidance documents recommend increasing this ratio by 1.2 for blocky-shaped rock or flagstone. Applying this correction factor due to the anticipated rounded characteristics of the riprap, the recommended design runup is $R = 2.2$ feet. Wind setup is taken as $S = 0.1 \times H_s = 0.1 \times (1.5 \text{ feet}) = 0.15$ feet. Therefore total required freeboard is approximately $2.2 + 0.15 = 2.35$ feet. As this value is less than the minimum freeboard for small dams recommended by the U.S. Bureau of Reclamation, it is recommended that **minimum freeboard = 4 feet** (USBR, 1987, 1992).
Figure 4.13. Site 7A Filter/Drain (Zone 2) Gradation Design
Figure 4.14. Site 7A and Site 8 Riprap (Zone 4) and Riprap Bedding (Zone 5) Gradations
4.2.5.4 Dam Zoning Recommendations - 7A West Dam Transition Section (Sta 11+00 to approximately Sta 14+00), and Closure Section (Sta 14+00 to right abutment Sta 23+00)

The design concept for the transition and closure sections, is to provide a safe transition from the main dam typical section to a broad, gently-sloped “spoil fill” berm along the northwestern margin of the reservoir, where hydraulic heads will be generally lower than on the main dam section on the south side of the reservoir.

Embankment Template and Foundation Preparation for Transition Section (Sta 11+00 to Sta 14+00): Figure 4.11 (7A West Dam Section at Sta 12+00) shows the design template for this section. Foundation preparation for the transition zone will include excavation and re-compaction of approximately 10 feet of loose, silty sand. There will be no cutoff to bedrock under this section, however the internal chimney and blanket filter/drain zones will be maintained due to the fairly significant hydraulic head (up to 40 feet) on portions of this transition section. The upstream slope will be flattened to 4H:1V. For economy, upstream slope protection will consist of 2 feet of “rock mulch” (Zone 6) derived from local excavations from select terrace gravel deposits. The rock mulch will be underlain by a geotextile filter fabric. It is anticipated that Zone 6 fill will be variably graded and will comprise varying amounts of sand, gravel, and cobbles, and may contain significant amounts of fines. Over time during the operational life of the dam, it is anticipated that there may be minor damage or erosion of this zone in places, especially on the upstream face of the embankments due to wave action. This minor erosion should be an anticipated maintenance item, but will not present a dam safety concern due to the very broad section of fill in these embankments.

Embankment Template and Foundation Preparation for Closure Section (Sta 14+00 to Sta 23+00): Figure 4.11 (7A West Dam Section at Sta 16+00) shows the design template for this section. This section is essentially a broad, flat-sloped spoil berm constructed of the material that will be stripped from the reservoir bottom to maximize storage capacity below grade. Foundation preparation will consist of nominal stripping to remove topsoil and vegetation only. There will be no cutoff to bedrock, or chimney filter/blanket drain for this section due to the extremely broad crest (100 feet wide) and modest hydraulic head (20 feet). Slope erosion protection for this section will be rock mulch underlain by geotextile on the upstream slope.

4.2.5.5 Recommendations for Supplemental Investigations and Final Design of Site 7A Dams and Reservoir

Final design of the Site 7A embankments and reservoirs will require additional geotechnical site investigations and analyses. Some of the key areas where additional work will be necessary are described as follows:

- **Supplemental geotechnical subsurface exploration and laboratory soils testing:**
  - Additional drilling and sampling should be done on the dam alignments to evaluate depth to bedrock and the condition of the bedrock along the alignment. Water pressure testing (packer testing) in boreholes within the bedrock is recommended to evaluate and optimize the core wall key-in depth requirements.
Core sampling and testing of the bedrock in the area of the spillway cut should be done to support analysis and evaluation of the unlined spillway, such as parameters needed to evaluate headcut erodibility index which is used in spillway erosion model analysis. Parameters to be evaluated typically include unconfined compressive strength, rock quality designation, and joint and fracture characteristics (NRCS, 1997).

Additional exploration of the terrace deposits on the ridge tops should be done to better define their lateral extent, depth and gradational characteristics. These deposits are the anticipated source for rock mulch (Zone 6) which is recommended for use as slope protection on all the downstream embankment slopes, and also on the upstream slopes for the 7A West Dam Transition and Closure sections (Figures 4.10 and 4.11). The investigation should confirm adequate quantities and gradational characteristics of these materials for these purposes.

Additional subsurface exploration is recommended in the reservoir stripping areas to evaluate depth to bedrock and to collect additional borrow samples of Zone 1 for laboratory testing. Consideration should be given to performing seismic refraction surveys across the reservoir areas to define the depth to bedrock in sufficient detail to optimize the below-grade stripping opportunities for increasing the reservoir volume. (The present conceptual design considers an average stripping depth of 12 feet. This is believed to be a conservative estimate, and additional storage volume may be feasible with deeper excavation of alluvial overburden deposits).

Additional laboratory testing of the silty sand, sandy silt (Zone 1) materials is needed. Tests should include additional hydraulic conductivity testing in triaxial devices that allow testing at various confining stresses. Consolidated, undrained triaxial shear strength testing also is necessary to support slope stability analyses. Consideration should also be given to conducting cyclic shear strength testing of the Zone 1 materials to understand potential for liquefaction or strength loss of these materials under dynamic (e.g., seismic) loading.

**Supplemental Geotechnical Analysis:**

- Slope stability analyses should be performed for both embankments (7A East and West) to document acceptable factors of safety for all loading conditions: (a) static, steady seepage at full and partial pool elevations (upstream and downstream slopes), (b) rapid drawdown from full and partial pools (upstream slope), (c) pseudo-static and seismic deformation analysis (upstream and downstream slopes). The embankment slopes should be adjusted (flattened) as necessary to achieve required minimum factors of safety.
- Additional seepage analyses should be performed to optimize the design of the blanket drain to ensure sufficient capacity to prevent phreatic line buildup in the downstream zone, and to size the toe drain outlet pipe.

### 4.2.6 Outlet Works

Conceptual designs were developed for the outlet works for these sites as shown on Figure 4.16 through 4.22. The Site 7A East outlet works would consist of an intake structure, a concrete gate tower incorporating two 36” sluice gates located within the embankment, 36” outlet pipeline, and an impact basin outlet structure. The concrete tower would contain two 3’ x 3’ sluice gates in
series. This would allow the head to be reduced across each gate, eliminating the need for a special head-breaking valve, which could be very expensive. The design of the outlet works was developed to allow a release of 30 cfs with minimal depth in the reservoir. Approximately one foot of head above the outlet pipe would be required to flow 30 cfs. The maximum capacity of these primary outlet works would be approximately 110 cfs. The concrete tower would act as a principal spillway through the use of an overflow weir on the center partition above the first gate. Water released from 7A East would be piped to the Riverton Valley canal. Site 7A West would have a similar outlet works design consisting of an intake structure, a concrete tower consisting of two 36” sluice gates located within the embankment, 36” outlet pipeline, and an impact basin outlet structure. The pipeline route would parallel the north side of River View Road to a point below site 7B then cross under River View Road and run south to the impact basin outlet into the Riverton Valley canal. The design of this outlet works was also developed to allow release of 30 cfs with minimal depth in the reservoir. Approximately one foot of head above the outlet pipe would be required to flow 30 cfs. The maximum capacity of these primary outlet works would be approximately 110 cfs. An impact basin outlet structure would be needed at the pipeline outlet to control outlet velocities. Site 7B would have a similar outlet works design consisting of an intake structure, a concrete tower consisting of two 36” sluice gates located within the embankment, and a 36” pipeline to connect this site with the pipeline connecting Site 7A West to the Riverton Valley canal. The outlet pipeline for site 7B would connect into the pipeline coming from Site 7A West at River View Road. The design of this outlet works was also developed to allow a release of 30 cfs with minimal depth in the reservoir. Approximately one foot of head above the outlet pipe would be required to flow 30 cfs. The maximum capacity of these primary outlet works would be approximately 105 cfs. Detailed calculations are presented in Appendix F.

4.2.7 Emergency Spillway

Conceptual designs for emergency spillways were developed. Site 7B does not require an emergency spillway because there is no drainage area for storm water runoff and the capacity of the outlet works exceeds that of the reservoir supply. The capacity of the primary outlet works is approximately 105 cfs and the capacity of the reservoir supply is approximately 40 cfs. Sites 7A and 7A East however require an emergency spillway. This site will likely require a Class I Dam Hazard Classification since loss of human life could be possible in the event of failure of the dam. This classification is based upon an evaluation of the consequences of a clear weather breach of the dam assuming the reservoir is at the high-water line. Safety of Dams Law states inflow design flood requirements for Class I dams are the Probable Maximum Flood (PMF). Spillway capacity must be designed according to the inflow design flood requirements. Generation of the PMF begins with the development of the Probable Maximum Precipitation (PMP) using Hydrometeorological Report No. 55A. The PMP was generated for both the general storm and the local storm. It was determined to use the local storm in spillway design calculations. The local storm generated higher peak flows and is characteristic of this region’s intense isolated storm events. The index 1 hr 1 mi² PMP estimate adjusted for mean drainage elevation was determined. Then the depth-duration curve for 1 mi² was generated using the 1 mi² factors for durations up to six hours. Next the areal reduction factors were applied. The result was the PMP depth-duration curve for the drainage basin above site 7A.
The Natural Resources Conservation Service classifies soils into four Hydrologic Soil Groups based on the soil’s potential for runoff. The four Hydrologic Soil Groups are A, B, C, and D. HSG A soils generally have the least runoff potential and HSG D soils have the greatest. Details for these classifications can be found in ‘Urban Hydrology for Small Watersheds’, Soil Conservation Service Technical Release 55 (June 1986). The drainage basin above site 7A consists of HSG B. The soils in the basin are deep and well drained with moderate infiltration rates when thoroughly wetted. Runoff is generally slow to moderate. The drainage basin is comprised of undeveloped rural rangeland. Land cover is good and generally consisting of grasses and forbs. The resulting pre-development Soil Conservation Service Curve Number based on land cover type, Hydrologic condition, and Hydrologic Soil Group is 61.

Hydrologic modeling of the drainage basin above site 7A was completed to determine the PMF. Stormwater runoff simulation was completed using U.S. Army Corps of Engineers developed HEC-HMS 2.2.2 hydrologic modeling system. The Soil Conservation Service (SCS) Unit Hydrograph method was used to generate the basin outflow hydrograph. The PMP depth-duration curve along with the drainage basin area, basin lag time, and drainage basin curve number were required input parameters. Basin lag time can be related to time of concentration for ungaged watersheds by:

\[ t_{\text{lag}} = 0.6 \ t_c \]  

(1)

Time of concentration is the time it takes for the most distant point in the watershed to contribute runoff at the design point. Runoff is assumed to travel as either sheet flow, shallow concentrated flow, and channel flow. Time of concentration is estimated as the sum of the travel times of these three types of flow. Flow velocities and basin geometry determine the time of concentration for the basin. The basin lag time was calculated to be 19 minutes for the west basin and 26 minutes for the east basin. The drainage area for the west and east basins are .11 and .41 mi² respectively as shown on Figure 4.23. Table 4.5 shows the peak flow and runoff volume for the local and general storm PMF for the drainage above site 7A.

<table>
<thead>
<tr>
<th></th>
<th>Local Storm PMF</th>
<th>Local Storm PMF + Canal Break</th>
<th>General Storm PMF</th>
<th>General Storm PMF + Canal Break</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Drainage Discharge (cfs)</td>
<td>1839</td>
<td>2279</td>
<td>1055</td>
<td>1495</td>
</tr>
<tr>
<td>Total Direct Runoff (acre ft)</td>
<td>203</td>
<td>418</td>
<td>573</td>
<td>3191</td>
</tr>
<tr>
<td>Time of Peak (min.)</td>
<td>41</td>
<td>41</td>
<td>77</td>
<td>77</td>
</tr>
<tr>
<td>Total Precipitation (inch)</td>
<td>12.7</td>
<td>12.7</td>
<td>27.1</td>
<td>27.1</td>
</tr>
<tr>
<td>Total Loss (inch)</td>
<td>5.38</td>
<td>5.38</td>
<td>6.4</td>
<td>6.4</td>
</tr>
<tr>
<td>Total Excess (inch)</td>
<td>7.32</td>
<td>7.32</td>
<td>20.7</td>
<td>20.7</td>
</tr>
</tbody>
</table>

The local storm PMF generates 203 ac-ft of water with a peak flow of 1,839 cfs. This flood volume could be absorbed in the freeboard on the dam however it is anticipated that the PMF may breach the LeClair canal causing the canal flow to enter the reservoir. For this reason the emergency spillway was sized for the PMF along with the maximum flow 440 cfs expected in the canal. This generates a peak inflow of 2,279 cfs with a total inflow of 418 ac-ft. This model assumed the canal broke immediately and ran 440 cfs into the reservoir for the 6 hour storm duration. It was assumed that the reservoir would be full when the storm occurred.
DRAINAGE BASIN STATISTICS
SITE 7A WEST
DRAINAGE AREA = 0.11 SQ. MI.
BASIN SLOPE = 496 FT/MI
BASIN LAG TIME = 20 MIN.

DRAINAGE BASIN STATISTICS
SITE 7A EAST
DRAINAGE AREA = 0.41 SQ. MI.
BASIN SLOPE = 500 FT/MI
BASIN LAG TIME = 26 MIN.

STATE NO. 7A WEST

STATE NO. 7A EAST

APEX SURVEYING, INC.
(307) 534-7841

LECLAIR/PWIS IRRIGATION STORAGE PROJECT
407 West Adams Avenue, P.O. Box 1147
Cheyenne, Wyoming 82001
(307) 634-7844

FIGURE 4.23
SITE NO. 7A DRAINAGE AREA
4-41
The emergency spillway would be constructed by excavating a channel with a bottom width of 50’ and 3:1 horizontal to vertical side slopes as shown on Figure 4.8. The water generated from the design storm was routed through the emergency spillway for reservoir 7A. The results of this analysis can be seen in Table 4.6.

<table>
<thead>
<tr>
<th>Local Storm PMF</th>
<th>Local Storm PMF + Canal Break</th>
<th>General Storm PMF</th>
<th>General Storm PMF + Canal Break</th>
</tr>
</thead>
<tbody>
<tr>
<td>Emergency Spillway Width (ft)</td>
<td>50</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>Peak Elevation (ft)</td>
<td>1.69</td>
<td>2.72</td>
<td>1.96</td>
</tr>
<tr>
<td>Peak Emer. Spillway Outflow (cfs)</td>
<td>285</td>
<td>583</td>
<td>357</td>
</tr>
<tr>
<td>Time of Peak Outflow (min.)</td>
<td>146</td>
<td>289</td>
<td>274</td>
</tr>
</tbody>
</table>

The peak water level rise in the reservoir would be 2.72 feet. This will maintain a minimum freeboard on the dam of 2 feet which meets the requirements falling under the Safety of Dams law of no residual freeboard required for dams designed for the PMF. The spillway would be re-vegetated with a seed mixture of native grasses. The emergency spillway would be located adjacent to the left abutment of the east embankment and discharge into the drainage below the toe of the dam.

### 4.2.8 SEO Permitting

Each reservoir, prior to construction, would require filing an application for a permit to appropriate surface water with the State Engineer (SEO). There are several different types of application forms to submit. Sites 7A and 7B would require Form S.W. 3 reservoir permits along with Form S.W. 2 enlargement permits on the reservoir supply (LeClair canal) and Form S.W. 2 permits for supplemental supply to the Riverton Valley canal. The Form S.W. 2 permit for supplemental supply application requires the reservoir capacity to be allocated to specific irrigated lands. This could be all land included in the Riverton Valley ID. In addition to these Wyoming SEO surface water storage permits, the Wyoming SEO would, prior to construction, need to review the plans and specifications for dam safety approval and to provide approval to construct the proposed facility.

Sites 7A and 7B would operate with a water exchange between the two irrigation districts. LeClair ID would divert an additional 15 cfs from the Wind River at their diversion leaving Riverton Valley ID 15 cfs short. Riverton Valley ID would release 30 cfs from storage and route it into their canal to replace the 15 cfs they exchanged with LeClair ID and to gain an additional 15 cfs of supplemental supply. A combined storage volume in Site 7A of 1,290 acre-ft would supply the additional 15 cfs to both irrigation districts for 22 days.

It is anticipated that the irrigation districts would get two fills allowing these reservoirs to yield twice their storage capacity. The irrigation districts could fill these reservoirs in the spring while they are filling their canal before the irrigation season begins. The irrigation districts are typically short of water in early May when pre-runoff flows in the river are low and irrigation demand is high. Water could be released from storage during this period and then stored again when the irrigators shut down to cut hay. The second fill could be used to supplement late season water to the irrigators. These reservoirs could be filled again as late season irrigation demand drops off and carried over the winter for supplemental supply for startup the following spring.
4.2.9 Additional Permits
In addition to these Wyoming SEO permits and approval, there are additional permits and approvals required for new dam construction. The Army Corp of Engineers regulates activities involving the waters of the United States. It is anticipated that an Individual Section 404 Permit will be required. This will require that an Environmental Assessment be prepared and submitted along with the Section 404 application. These include a Wyoming Department of Environmental Quality National Pollution Discharge Elimination System (NPDES) permit and Section 401 Certification. This permit controls the discharge of stormwater pollutants associated with construction activities. The Section 401 Certification is the State’s approval to ensure that the proposed activities meet state water quality standards and do not degrade water quality. U.S. Fish and Wildlife Service Endangered Species Act Compliance (Section 7) would be required. Coordination with the U.S. Department of Interior Advisory Council on Historic Preservation (Section 106), which protects cultural and historic resources, would be required. State of Wyoming Historic Preservation Office (SHPO) archaeological clearance which determines significance of cultural resources potentially affected by ground disturbing activities would be required.

4.2.10 Staged Construction
There is potential for these sites to be built in stages giving the irrigation districts the flexibility to delay construction as their financial situation and funding opportunities improve. The two embankments associated with Site 7A could be split into two phases. The first phase would include construction of either Site 7A East or West along with the reservoir supply diversion and pipeline, outlet works and Riverton Valley supply pipeline, and emergency spillway. A dike would need to be built to separate site 7A East from site 7A West. Phase two would include removal of the dike and construction of the other embankment along with outlet works and pipeline to connect with the Riverton Valley canal supply pipeline. Phase one construction of 7A East with the plan to complete phase two in the future is the most economical sequence due to the shorter reservoir supply pipeline, however, Site 7A East is a less economical site compared to Site 7A West. If phase two is never completed then Site 7A West is the most economical site to build.
4.2.11 Cost Estimates

Based on the conceptual designs presented above, conceptual cost estimates have been prepared. The cost estimates include costs for construction components, construction engineering, preparation of final design plans and specifications, permitting and mitigation, legal fees and acquisition of access and right-of-way. Construction costs were calculated using 2007 material and labor costs and are presented in the standard format used by the WWDC. The conceptual cost estimate for constructing Site No. 7A West is presented in Table 4.7.

Table 4.7 - Site Number 7A West

<table>
<thead>
<tr>
<th>No.</th>
<th>Item</th>
<th>Units</th>
<th>Estimated Quantity</th>
<th>Unit Cost</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Mobilization</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$195,000.00</td>
</tr>
<tr>
<td>2</td>
<td>Zone I Embankment</td>
<td>C.Y.</td>
<td>237,552</td>
<td>$3.80</td>
<td>$902,697.60</td>
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<tr>
<td>3</td>
<td>Zone IIA Embankment</td>
<td>C.Y.</td>
<td>89,415</td>
<td>$2.00</td>
<td>$178,830.00</td>
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<td>4</td>
<td>Zone II Embankment</td>
<td>C.Y.</td>
<td>21,752</td>
<td>$21.00</td>
<td>$456,792.00</td>
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<tr>
<td>5</td>
<td>Zone III Embankment</td>
<td>C.Y.</td>
<td>7,900</td>
<td>$4.50</td>
<td>$35,550.00</td>
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<td>6</td>
<td>Zone IV Embankment</td>
<td>C.Y.</td>
<td>9,648</td>
<td>$35.00</td>
<td>$337,680.00</td>
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<tr>
<td>7</td>
<td>Zone V Embankment</td>
<td>C.Y.</td>
<td>4,824</td>
<td>$21.00</td>
<td>$101,304.00</td>
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<td>8</td>
<td>Zone VI Embankment</td>
<td>C.Y.</td>
<td>26,521</td>
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<td>9</td>
<td>Plastic Concrete Core Wall</td>
<td>C.Y.</td>
<td>1,333</td>
<td>$175.00</td>
<td>$233,275.00</td>
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<td>10</td>
<td>Geotextile Filter Fabric</td>
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<td>11</td>
<td>Perforated Toe Drain Pipe</td>
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<td>1,400</td>
<td>$7.00</td>
<td>$9,800.00</td>
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<td>12</td>
<td>Pool Area Excavation and Spoil</td>
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<td>154,552</td>
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<td>$309,104.00</td>
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<td>13</td>
<td>Dewatering</td>
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<td>14</td>
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<td>Primary Outlet Works</td>
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<td>--</td>
<td>$565,000.00</td>
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<tr>
<td>16</td>
<td>Revegetation</td>
<td>Ac.</td>
<td>3</td>
<td>$1,000.00</td>
<td>$3,000.00</td>
</tr>
</tbody>
</table>

Construction Cost Sub-Total: $3,920,527.60

10% Engineering: $392,052.76

Sub-Total: $4,312,580.36

15% Contingency: $646,887.05

CONSTRUCTION COST TOTAL: $4,959,467.41

Preparation of Final Designs and Specifications: $310,000.00

Permitting and Mitigation: $25,000.00

Legal Fees: $5,000.00

Acquisition of Access and Rights of Way: $550,000.00

TOTAL PROJECT COST: $5,849,467.41

USE: $5,849,000.00

Assuming a 67% WWDC grant and 33% loan at 4% for 50 years, the annual repayment would be $89,900.
The conceptual cost estimate for constructing Site No. 7A East is presented in Table 4.8.

### Table 4.8 - Site Number 7A East

<table>
<thead>
<tr>
<th>No.</th>
<th>Item</th>
<th>Units</th>
<th>Estimated Quantity</th>
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<th>Cost</th>
</tr>
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<tbody>
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<td>--</td>
<td>--</td>
<td>$160,000.00</td>
</tr>
<tr>
<td>2</td>
<td>Zone I Embankment</td>
<td>C.Y.</td>
<td>209,073</td>
<td>$3.80</td>
<td>$794,477.40</td>
</tr>
<tr>
<td>3</td>
<td>Zone II Embankment</td>
<td>C.Y.</td>
<td>16,493</td>
<td>$21.00</td>
<td>$346,353.00</td>
</tr>
<tr>
<td>4</td>
<td>Zone IV Embankment</td>
<td>C.Y.</td>
<td>8,504</td>
<td>$35.00</td>
<td>$297,640.00</td>
</tr>
<tr>
<td>5</td>
<td>Zone V Embankment</td>
<td>C.Y.</td>
<td>4,252</td>
<td>$21.00</td>
<td>$89,292.00</td>
</tr>
<tr>
<td>6</td>
<td>Zone VI Embankment</td>
<td>C.Y.</td>
<td>5,251</td>
<td>$5.00</td>
<td>$26,255.00</td>
</tr>
<tr>
<td>7</td>
<td>Plastic Concrete Core Wall</td>
<td>C.Y.</td>
<td>1,630</td>
<td>$175.00</td>
<td>$285,250.00</td>
</tr>
<tr>
<td>8</td>
<td>Perforated Toe Drain Pipe</td>
<td>L.F.</td>
<td>1,500</td>
<td>$7.00</td>
<td>$10,500.00</td>
</tr>
<tr>
<td>9</td>
<td>Pool Area Excavation and Spoil</td>
<td>C.Y.</td>
<td>166,566</td>
<td>$2.00</td>
<td>$333,132.00</td>
</tr>
<tr>
<td>10</td>
<td>Dewatering</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$30,000.00</td>
</tr>
<tr>
<td>11</td>
<td>Reservoir Supply Pipeline</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$260,000.00</td>
</tr>
<tr>
<td>12</td>
<td>Primary Outlet Works</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$590,000.00</td>
</tr>
<tr>
<td>13</td>
<td>Emergency Spillway</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$4,000.00</td>
</tr>
<tr>
<td>14</td>
<td>Revegetation</td>
<td>Ac.</td>
<td>1</td>
<td>$1,000.00</td>
<td>$1,000.00</td>
</tr>
</tbody>
</table>

**Construction Cost Sub-Total:** $3,227,899.40  
10% Engineering: $322,789.94  
**Sub-Total:** $3,550,689.34  
15% Contingency: $532,603.40  
**CONSTRUCTION COST TOTAL:** $4,083,292.74  
Preparation of Final Designs and Specifications: $260,000.00  
Permitting and Mitigation: $25,000.00  
Legal Fees: $5,000.00  
Acquisition of Access and Rights of Way: $470,000.00  
**TOTAL PROJECT COST:** $4,843,292.74  
USE: $4,843,000.00

Assuming a 67% WWDC grant and 33% loan at 4% for 50 years, the annual repayment would be $74,500.
The conceptual cost estimate for constructing Site No. 7A West without over excavation within the pool area is presented in Table 4.9.

**Table 4.9 - Site Number 7A West w/Out Over Excavation**

<table>
<thead>
<tr>
<th>No.</th>
<th>Item</th>
<th>Units</th>
<th>Estimated Quantity</th>
<th>Unit Cost</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Mobilization</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$180,000.00</td>
</tr>
<tr>
<td>2</td>
<td>Zone I Embankment</td>
<td>C.Y.</td>
<td>237,552</td>
<td>$3.80</td>
<td>$902,697.60</td>
</tr>
<tr>
<td>3</td>
<td>Zone IA Embankment</td>
<td>C.Y.</td>
<td>89,415</td>
<td>$2.00</td>
<td>$178,830.00</td>
</tr>
<tr>
<td>4</td>
<td>Zone II Embankment</td>
<td>C.Y.</td>
<td>21,752</td>
<td>$21.00</td>
<td>$456,792.00</td>
</tr>
<tr>
<td>5</td>
<td>Zone III Embankment</td>
<td>C.Y.</td>
<td>7,900</td>
<td>$4.50</td>
<td>$35,550.00</td>
</tr>
<tr>
<td>6</td>
<td>Zone IV Embankment</td>
<td>C.Y.</td>
<td>9,648</td>
<td>$35.00</td>
<td>$337,680.00</td>
</tr>
<tr>
<td>7</td>
<td>Zone V Embankment</td>
<td>C.Y.</td>
<td>4,824</td>
<td>$21.00</td>
<td>$101,304.00</td>
</tr>
<tr>
<td>8</td>
<td>Zone VI Embankment</td>
<td>C.Y.</td>
<td>26,521</td>
<td>$5.00</td>
<td>$132,605.00</td>
</tr>
<tr>
<td>9</td>
<td>Plastic Concrete Core Wall</td>
<td>C.Y.</td>
<td>1,333</td>
<td>$175.00</td>
<td>$233,275.00</td>
</tr>
<tr>
<td>10</td>
<td>Geotextile Filter Fabric</td>
<td>S.Y.</td>
<td>17,080</td>
<td>$1.75</td>
<td>$29,890.00</td>
</tr>
<tr>
<td>11</td>
<td>Perforated Toe Drain Pipe</td>
<td>L.F.</td>
<td>1,400</td>
<td>$7.00</td>
<td>$9,800.00</td>
</tr>
<tr>
<td>12</td>
<td>Dewatering</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$30,000.00</td>
</tr>
<tr>
<td>13</td>
<td>Reservoir Supply Pipeline</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$400,000.00</td>
</tr>
<tr>
<td>14</td>
<td>Primary Outlet Works</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$565,000.00</td>
</tr>
<tr>
<td>15</td>
<td>Revegetation</td>
<td>Ac.</td>
<td>3</td>
<td>$1,000.00</td>
<td>$3,000.00</td>
</tr>
</tbody>
</table>

Construction Cost Sub-Total: $3,596,423.60
10% Engineering: $359,642.36
Sub-Total: $3,956,065.96
15% Contingency: $593,409.89
CONSTRUCTION COST TOTAL: $4,549,475.85
Preparation of Final Designs and Specifications: $285,000.00
Permitting and Mitigation: $25,000.00
Legal Fees: $5,000.00
Acquisition of Access and Rights of Way: $550,000.00
TOTAL PROJECT COST: $5,414,475.85
USE: $5,414,000.00

Assuming a 67% WWDC grant and 33% loan at 4% for 50 years, the annual repayment would be $83,300.
The conceptual cost estimate for constructing Site No. 7A East without over excavation within the pool area is presented in Table 4.10.

Table 4.10 - Site Number 7A East w/Out Over Excavation

<table>
<thead>
<tr>
<th>No.</th>
<th>Item</th>
<th>Units</th>
<th>Estimated Quantity</th>
<th>Unit Cost</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Mobilization</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$145,000.00</td>
</tr>
<tr>
<td>2</td>
<td>Zone I Embankment</td>
<td>C.Y.</td>
<td>209,073</td>
<td>$3.80</td>
<td>$794,477.40</td>
</tr>
<tr>
<td>3</td>
<td>Zone II Embankment</td>
<td>C.Y.</td>
<td>16,493</td>
<td>$21.00</td>
<td>$346,353.00</td>
</tr>
<tr>
<td>4</td>
<td>Zone IV Embankment</td>
<td>C.Y.</td>
<td>8,504</td>
<td>$35.00</td>
<td>$297,640.00</td>
</tr>
<tr>
<td>5</td>
<td>Zone V Embankment</td>
<td>C.Y.</td>
<td>4,252</td>
<td>$21.00</td>
<td>$89,292.00</td>
</tr>
<tr>
<td>6</td>
<td>Zone VI Embankment</td>
<td>C.Y.</td>
<td>5,251</td>
<td>$5.00</td>
<td>$26,255.00</td>
</tr>
<tr>
<td>7</td>
<td>Plastic Concrete Core Wall</td>
<td>C.Y.</td>
<td>1,630</td>
<td>$175.00</td>
<td>$285,250.00</td>
</tr>
<tr>
<td>8</td>
<td>Perforated Toe Drain Pipe</td>
<td>L.F.</td>
<td>1,500</td>
<td>$7.00</td>
<td>$10,500.00</td>
</tr>
<tr>
<td></td>
<td>Dewatering</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$30,000.00</td>
</tr>
<tr>
<td>9</td>
<td>Reservoir Supply Pipeline</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$260,000.00</td>
</tr>
<tr>
<td>10</td>
<td>Primary Outlet Works</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$590,000.00</td>
</tr>
<tr>
<td>11</td>
<td>Emergency Spillway</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$4,000.00</td>
</tr>
<tr>
<td>12</td>
<td>Revegetation</td>
<td>Ac.</td>
<td>1</td>
<td>$1,000.00</td>
<td>$1,000.00</td>
</tr>
</tbody>
</table>

Construction Cost Sub-Total: $2,879,767.40
10% Engineering: $287,976.74
Sub-Total: $3,167,744.14
15% Contingency: $475,161.62
CONSTRUCTION COST TOTAL: $3,642,905.76

Preparation of Final Designs and Specifications: $230,000.00
Permitting and Mitigation: $25,000.00
Legal Fees: $5,000.00
Acquisition of Access and Rights of Way: $470,000.00
TOTAL PROJECT COST: $4,372,905.76

USE: $4,373,000.00

Assuming a 67% WWDC grant and 33% loan at 4% for 50 years, the annual repayment would be $67,200.
The conceptual cost estimate for constructing Site No. 7A East and West combined is presented in Table 4.11.

<table>
<thead>
<tr>
<th>No.</th>
<th>Item</th>
<th>Units</th>
<th>Estimated Quantity</th>
<th>Unit Cost</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Mobilization</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$310,000.00</td>
</tr>
<tr>
<td>2</td>
<td>Zone I Embankment</td>
<td>C.Y.</td>
<td>415,625</td>
<td>$3.80</td>
<td>$1,579,375.00</td>
</tr>
<tr>
<td>3</td>
<td>Zone IA Embankment</td>
<td>C.Y.</td>
<td>89,415</td>
<td>$2.00</td>
<td>$178,830.00</td>
</tr>
<tr>
<td>4</td>
<td>Zone II Embankment</td>
<td>C.Y.</td>
<td>38,245</td>
<td>$21.00</td>
<td>$803,145.00</td>
</tr>
<tr>
<td>5</td>
<td>Zone III Embankment</td>
<td>C.Y.</td>
<td>7,900</td>
<td>$4.50</td>
<td>$35,550.00</td>
</tr>
<tr>
<td>6</td>
<td>Zone IV Embankment</td>
<td>C.Y.</td>
<td>18,152</td>
<td>$35.00</td>
<td>$635,320.00</td>
</tr>
<tr>
<td>7</td>
<td>Zone V Embankment</td>
<td>C.Y.</td>
<td>9,076</td>
<td>$21.00</td>
<td>$190,596.00</td>
</tr>
<tr>
<td>8</td>
<td>Zone VI Embankment</td>
<td>C.Y.</td>
<td>31,772</td>
<td>$5.00</td>
<td>$158,860.00</td>
</tr>
<tr>
<td>9</td>
<td>Plastic Concrete Core Wall</td>
<td>C.Y.</td>
<td>2,963</td>
<td>$175.00</td>
<td>$518,525.00</td>
</tr>
<tr>
<td>10</td>
<td>Geotextile Filter Fabric</td>
<td>S.Y.</td>
<td>17,080</td>
<td>$1.75</td>
<td>$29,890.00</td>
</tr>
<tr>
<td>11</td>
<td>Perforated Toe Drain Pipe</td>
<td>L.F.</td>
<td>2,900</td>
<td>$7.00</td>
<td>$20,300.00</td>
</tr>
<tr>
<td>12</td>
<td>Pool Area Excavation and Spoil</td>
<td>C.Y.</td>
<td>321,118</td>
<td>$2.00</td>
<td>$642,236.00</td>
</tr>
<tr>
<td>13</td>
<td>Dewatering</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$60,000.00</td>
</tr>
<tr>
<td>14</td>
<td>Reservoir Supply Pipeline</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$260,000.00</td>
</tr>
<tr>
<td>15</td>
<td>Primary Outlet Works</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$784,500.00</td>
</tr>
<tr>
<td>16</td>
<td>Emergency Spillway</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$4,000.00</td>
</tr>
<tr>
<td>17</td>
<td>Revegetation</td>
<td>Ac.</td>
<td>4</td>
<td>$1,000.00</td>
<td>$4,000.00</td>
</tr>
</tbody>
</table>

**Construction Cost Sub-Total:** $6,215,127.00  
10% Engineering: $621,512.70

**Sub-Total:** $6,836,639.70  
15% Contingency: $1,025,495.96

**CONSTRUCTION COST TOTAL:** $7,862,135.66

Preparation of Final Designs and Specifications: $495,000.00

Permitting and Mitigation: $25,000.00

Legal Fees: $5,000.00

Acquisition of Access and Rights of Way: $1,000,000.00

**TOTAL PROJECT COST:** $9,387,135.66

**USE:** $9,387,000.00

Assuming a 67% WWDC grant and 33% loan at 4% for 50 years, the annual repayment would be $144,400.
The conceptual cost estimate for constructing Site No. 7A East and West without over excavation within the pool area is presented in Table 4.12.

### Table 4.12 - Site Number 7A Combined w/Out Over Excavation

<table>
<thead>
<tr>
<th>No.</th>
<th>Item</th>
<th>Units</th>
<th>Estimated Quantity</th>
<th>Unit Cost</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Mobilization</td>
<td>L.S.</td>
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<td>--</td>
<td>$275,000.00</td>
</tr>
<tr>
<td>2</td>
<td>Zone I Embankment</td>
<td>C.Y.</td>
<td>415,625</td>
<td>$3.80</td>
<td>$1,579,375.00</td>
</tr>
<tr>
<td>3</td>
<td>Zone IA Embankment</td>
<td>C.Y.</td>
<td>89,415</td>
<td>$2.00</td>
<td>$178,830.00</td>
</tr>
<tr>
<td>4</td>
<td>Zone II Embankment</td>
<td>C.Y.</td>
<td>38,245</td>
<td>$21.00</td>
<td>$803,145.00</td>
</tr>
<tr>
<td>5</td>
<td>Zone III Embankment</td>
<td>C.Y.</td>
<td>7,900</td>
<td>$4.50</td>
<td>$35,550.00</td>
</tr>
<tr>
<td>6</td>
<td>Zone IV Embankment</td>
<td>C.Y.</td>
<td>18,152</td>
<td>$35.00</td>
<td>$635,320.00</td>
</tr>
<tr>
<td>7</td>
<td>Zone V Embankment</td>
<td>C.Y.</td>
<td>9,076</td>
<td>$21.00</td>
<td>$190,596.00</td>
</tr>
<tr>
<td>8</td>
<td>Zone VI Embankment</td>
<td>C.Y.</td>
<td>31,772</td>
<td>$5.00</td>
<td>$158,860.00</td>
</tr>
<tr>
<td>9</td>
<td>Plastic Concrete Core Wall</td>
<td>C.Y.</td>
<td>2,963</td>
<td>$175.00</td>
<td>$518,525.00</td>
</tr>
<tr>
<td>10</td>
<td>Geotextile Filter Fabric</td>
<td>S.Y.</td>
<td>17,080</td>
<td>$1.75</td>
<td>$29,890.00</td>
</tr>
<tr>
<td>11</td>
<td>Perforated Toe Drain Pipe</td>
<td>L.F.</td>
<td>2,900</td>
<td>$7.00</td>
<td>$20,300.00</td>
</tr>
<tr>
<td>12</td>
<td>Dewatering</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$60,000.00</td>
</tr>
<tr>
<td>13</td>
<td>Reservoir Supply Pipeline</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$260,000.00</td>
</tr>
<tr>
<td>14</td>
<td>Primary Outlet Works</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$784,500.00</td>
</tr>
<tr>
<td>15</td>
<td>Emergency Spillway</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$4,000.00</td>
</tr>
<tr>
<td>16</td>
<td>Revegetation</td>
<td>Ac.</td>
<td>4</td>
<td>$1,000.00</td>
<td>$4,000.00</td>
</tr>
</tbody>
</table>

**Construction Cost Sub-Total:** $5,537,891.00  
10% Engineering: $553,789.10  
**Sub-Total:** $6,091,680.10  
15% Contingency: $913,752.02  
**CONSTRUCTION COST TOTAL:** $7,005,432.12  
Preparation of Final Designs and Specifications: $440,000.00  
Permitting and Mitigation: $25,000.00  
Legal Fees: $5,000.00  
Acquisition of Access and Rights of Way: $1,000,000.00  
**TOTAL PROJECT COST:** $8,475,432.12  
USE: $8,475,000.00  

Assuming a 67% WWDC grant and 33% loan at 4% for 50 years, the annual repayment would be $130,300.
Foundation stripping, embankment barrow, and embankment material import costs were included in the respective zoned embankment unit costs. Excavation for the emergency spillway is included in the Zone I Embankment bid item. Tables 4.13 through 4.16 show detailed conceptual cost estimates for the 36” reservoir supply diversion and pipeline and outlet works lump sum items.

### Table 4.13 - Site 7A East 50' Concrete Gate Tower Outlet Works

<table>
<thead>
<tr>
<th>No.</th>
<th>Item</th>
<th>Units</th>
<th>Est. Quantity</th>
<th>Unit Cost</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Concrete</td>
<td>C.Y.</td>
<td>63</td>
<td>$500.00</td>
<td>$31,500.00</td>
</tr>
<tr>
<td>2</td>
<td>36” Sluice Gate</td>
<td>E.A.</td>
<td>2</td>
<td>$12,000.00</td>
<td>$24,000.00</td>
</tr>
<tr>
<td>3</td>
<td>36” Steel Pipe</td>
<td>L.F.</td>
<td>300</td>
<td>$200.00</td>
<td>$60,000.00</td>
</tr>
<tr>
<td>4</td>
<td>10'x12' Building</td>
<td>S.F.</td>
<td>120</td>
<td>$150.00</td>
<td>$18,000.00</td>
</tr>
<tr>
<td>5</td>
<td>Intake Box (4’x4’x6’) Concrete</td>
<td>C.Y.</td>
<td>5</td>
<td>$500.00</td>
<td>$7,500.00</td>
</tr>
<tr>
<td>6</td>
<td>36” Pipeline</td>
<td>L.F.</td>
<td>4,000</td>
<td>$110.00</td>
<td>$440,000.00</td>
</tr>
<tr>
<td>7</td>
<td>Impact Basin</td>
<td>E.A.</td>
<td>1</td>
<td>$10,000.00</td>
<td>$10,000.00</td>
</tr>
</tbody>
</table>

**Construction Cost Sub-Total:** $591,000.00

**USE:** $590,000.00

### Table 4.14 - Site 7A West 65' Concrete Gate Tower Outlet Works

<table>
<thead>
<tr>
<th>No.</th>
<th>Item</th>
<th>Units</th>
<th>Est. Quantity</th>
<th>Unit Cost</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Concrete</td>
<td>C.Y.</td>
<td>80</td>
<td>$500.00</td>
<td>$40,000.00</td>
</tr>
<tr>
<td>2</td>
<td>36” Sluice Gate</td>
<td>E.A.</td>
<td>2</td>
<td>$12,000.00</td>
<td>$24,000.00</td>
</tr>
<tr>
<td>3</td>
<td>36” Steel Pipe</td>
<td>L.F.</td>
<td>400</td>
<td>$200.00</td>
<td>$80,000.00</td>
</tr>
<tr>
<td>4</td>
<td>10'x12' Building</td>
<td>S.F.</td>
<td>120</td>
<td>$150.00</td>
<td>$18,000.00</td>
</tr>
<tr>
<td>5</td>
<td>Intake Box (4’x4’x6’) Concrete</td>
<td>C.Y.</td>
<td>5</td>
<td>$500.00</td>
<td>$7,500.00</td>
</tr>
<tr>
<td>6</td>
<td>36” Pipeline</td>
<td>L.F.</td>
<td>3,500</td>
<td>$110.00</td>
<td>$385,000.00</td>
</tr>
<tr>
<td>7</td>
<td>Impact Basin</td>
<td>E.A.</td>
<td>1</td>
<td>$10,000.00</td>
<td>$10,000.00</td>
</tr>
</tbody>
</table>

**Construction Cost Sub-Total:** $564,500.00

**USE:** $565,000.00

### Table 4.15 - Site 7A Supply Pipeline

<table>
<thead>
<tr>
<th>No.</th>
<th>Item</th>
<th>Units</th>
<th>Est. Quantity</th>
<th>Unit Cost</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Canal Diversion Structure</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$40,000.00</td>
</tr>
<tr>
<td>2</td>
<td>36” Pipeline</td>
<td>L.F.</td>
<td>1,900</td>
<td>$110.00</td>
<td>$209,000.00</td>
</tr>
<tr>
<td>3</td>
<td>Impact Basin</td>
<td>E.A.</td>
<td>1</td>
<td>$10,000.00</td>
<td>$10,000.00</td>
</tr>
</tbody>
</table>

**Construction Cost Sub-Total:** $259,000.00

**USE:** $260,000.00

### Table 4.16 - Site 7A West Supply Pipeline

<table>
<thead>
<tr>
<th>No.</th>
<th>Item</th>
<th>Units</th>
<th>Est. Quantity</th>
<th>Unit Cost</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Canal Diversion Structure</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$40,000.00</td>
</tr>
<tr>
<td>2</td>
<td>36” Pipeline</td>
<td>L.F.</td>
<td>3,175</td>
<td>$110.00</td>
<td>$349,250.00</td>
</tr>
<tr>
<td>3</td>
<td>Impact Basin</td>
<td>E.A.</td>
<td>1</td>
<td>$10,000.00</td>
<td>$10,000.00</td>
</tr>
</tbody>
</table>

**Construction Cost Sub-Total:** $399,250.00

**USE:** $400,000.00
The conceptual cost estimate for constructing Site No. 7B is presented in Table 4.17. Dam sections and detailed embankment quantity takeoffs were not completed for this site. The unit cost of embankment was estimated using a weighted average of the unit costs for Site 7A.

### Table 4.17 - Site Number 7B

<table>
<thead>
<tr>
<th>No.</th>
<th>Item</th>
<th>Units</th>
<th>Est. Quantity</th>
<th>Unit Cost</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Mobilization</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$34,000.00</td>
</tr>
<tr>
<td>2</td>
<td>Embankment</td>
<td>C.Y.</td>
<td>69,000</td>
<td>$7.00</td>
<td>$483,000.00</td>
</tr>
<tr>
<td>3</td>
<td>Reservoir Supply Works</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$25,000.00</td>
</tr>
<tr>
<td>4</td>
<td>Outlet Works</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$145,000.00</td>
</tr>
</tbody>
</table>

Construction Cost Sub-Total: $687,000.00

10% Engineering: $68,700.00

Sub-Total: $755,700.00

15% Contingency: $113,355.00

CONSTRUCTION COST TOTAL: $869,055.00

Preparation of Final Designs and Specifications: $55,000.00

Permitting and Mitigation: $5,000.00

Legal Fees: $5,000.00

Acquisition of Access and Rights of Way: $230,000.00

TOTAL PROJECT COST: $1,164,055.00

USE: $1,164,000.00

Assuming a 67% WWDC grant and 33% loan at 4% for 50 years, the annual repayment would be $17,900.

Tables 4.18 through 4.19 show detailed conceptual cost estimates for the reservoir supply and outlet works lump sum items.

### Table 4.18 - Site 7B 40' Concrete Gate Tower Outlet Works

<table>
<thead>
<tr>
<th>No.</th>
<th>Item</th>
<th>Units</th>
<th>Est. Quantity</th>
<th>Unit Cost</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Concrete</td>
<td>C.Y.</td>
<td>50</td>
<td>$500.00</td>
<td>$25,000.00</td>
</tr>
<tr>
<td>2</td>
<td>36&quot; Sluice Gate</td>
<td>E.A.</td>
<td>2</td>
<td>$12,000.00</td>
<td>$24,000.00</td>
</tr>
<tr>
<td>3</td>
<td>36&quot; Steel Pipe</td>
<td>L.F.</td>
<td>240</td>
<td>$200.00</td>
<td>$48,000.00</td>
</tr>
<tr>
<td>4</td>
<td>10'x12' Building</td>
<td>S.F.</td>
<td>120</td>
<td>$150.00</td>
<td>$18,000.00</td>
</tr>
<tr>
<td>5</td>
<td>Intake Box (4'x4'x6') Concrete</td>
<td>C.Y.</td>
<td>5</td>
<td>$500.00</td>
<td>$7,500.00</td>
</tr>
<tr>
<td>6</td>
<td>36&quot; Pipeline</td>
<td>L.F.</td>
<td>210</td>
<td>$110.00</td>
<td>$23,100.00</td>
</tr>
</tbody>
</table>

Construction Cost Sub-Total: $145,600.00

USE: $145,000.00

### Table 4.19 - Site 7B Supply Inclined Slide Gate

<table>
<thead>
<tr>
<th>No.</th>
<th>Item</th>
<th>Units</th>
<th>Est. Quantity</th>
<th>Unit Cost</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>24&quot; Inclined Slide Gate</td>
<td>E.A.</td>
<td>1</td>
<td>$10,000.00</td>
<td>$10,000.00</td>
</tr>
<tr>
<td>2</td>
<td>24&quot; Steel Pipe</td>
<td>L.F.</td>
<td>100</td>
<td>$150.00</td>
<td>$15,000.00</td>
</tr>
<tr>
<td>3</td>
<td>Riprap</td>
<td>C.Y.</td>
<td>6</td>
<td>$70.00</td>
<td>$420.00</td>
</tr>
</tbody>
</table>

Construction Cost Sub-Total: $25,420.00

USE: $25,000.00
4.3 RESERVOIR SITE NUMBER 8

4.3.1 General
Site 8 is located north and west of the LeClair Canal. This site is situated in a small, natural drainage feature that drains towards the east-southeast. Remnants of a small, breached embankment dam are evident on the same alignment as the proposed new dam alignment. The breach is located at the deepest part of the drainage near the toe of the left abutment. It is unknown whether the breach was intentional or occurred as a result of overtopping or failure of the dam. This site is located in the SE ¼ of the NE ¼ of Section 20, Township 1 North, Range 4 East as shown on Figure 4.24. Access to this site is from the LeClair canal bank road. The reservoir and appurtenances would be located on deeded land owned by E. Wayne Jr. and Wanda P. Major. The land is currently used as pasture. It is anticipated that the land needed for the reservoir would be purchased and owned by LeClair ID. Easements would be obtained for pipelines and other appurtenances. As an alternative to purchasing the land inundated by the reservoir pool, an easement could be obtained to allow the current landowner to control access for non-irrigation purposes. The landowner was contacted for permission to survey and conduct study related investigations. No formal appraisals or land or easement purchase negotiations have taken place. Land or easement purchase was estimated at $10,000 per acre for use in this study.

4.3.2 Reservoir Supply
An existing lateral used to supply irrigation water to several fields above the LeClair canal was replaced in the summer of 2005 with 24” PVC irrigation pipe. The design capacity for this piped lateral is 9.1 cfs. The reservoir would be supplied through this existing 24” PVC piped lateral. An existing 12” turnout to supply water to the reservoir site pasture would be upgraded to a 24” pipeline to supply the reservoir. Approximately 50 feet of 24” PVC pipeline would be required as shown on Figure 4.25. The supply capacity of the existing lateral could fill the reservoir in approximately six days.

4.3.3 Reservoir Sizing
Conceptual designs were developed for this reservoir site which is located above the LeClair canal. This site could store approximately 120 ac-ft of water to a high-water line of 5159’ with a 32-feet high, 680-feet long dam. This storage elevation, however, would inundate approximately 0.8 acres of tribal land to the west. A high-water line of 5154’ would limit the storage volume to 77 ac-ft, but would not inundate the tribal land boundary.

4.3.4 Geotechnical/Geologic Investigations
Geotechnical/geologic site investigations and evaluation of site conditions are provided in the following section for this site.

4.3.4.1 Site Conditions and Geologic/Geotechnical Concerns
The geologic site plan and profile for the proposed Site 8 reservoir and dam are provided as Figures 4.26 (Site No. 8 Geologic Map) and 4.27 (Site No. 8 Geologic Profile), respectively. Site 8 was investigated with seven test pits (TP-18 through TP-24) and one, 40-feet deep boring (DH-2). Wind River Formation sandstone, mudstone and siltstone comprises both abutments, and is encountered under alluvial sediments that range up to 15 feet thick over bedrock in the
channel bottom. The alluvial sediments comprise a fining-upward sequence ranging from clay at the ground surface grading to silt and finally to gravelly sand at depth above the bedrock. The geologic units encountered at Site 8 are described as follows:

**Alluvium (Qal)**: Surficial soils in the upper few feet of the valley bottom consist of lean clay from the ground surface down to about 2.5 to 3 feet depth. The surficial clay layer may be sediment that built up behind the old dam. The material classifies as CL, with a liquid limit = 48%, and plasticity index = 29%. Laboratory test results are provided in Appendix C (Lab Test Results). The clay layer was encountered in TP-20, TP-24, and DH-2, and, for purposes of quantity availability, is interpreted to be laterally persistent in the upper 2.5 feet in the valley bottom below about elevation 5145. This material is a potential resource for high quality, impervious core material for a zoned earth embankment. When compacted, the clay soil will have low permeability, and will be resistant to internal erosion or piping under seepage forces. The clay zone will also be resistant to loss of strength due to seismic shaking.

Underlying the clay layer in the valley bottom is approximately 3 to 4 feet of loose to medium dense silt and silty fine sand. The silt/silty sand layer is about 3 feet thick at DH-2, and was also encountered in valley bottom test pits TP-20 and TP-24.

**Older Alluvium (Qalo)**: Beneath the silt layer and extending to bedrock in the valley bottom is dense, gravelly sand (referred to as a weak, slightly cemented conglomerate) that is interpreted to be an older alluvial deposit. This material also exists as a thin blanket over bedrock on the valley slopes.

In the valley bottom this coarse-grained zone is about 7.5 feet thick (e.g., at DH-2). This dense, gravelly layer is judged to have high shear strength, and is not anticipated to be vulnerable to liquefaction under moderate seismic shaking. However, the conglomerate layer may be permeable, and would require a seepage cutoff to bedrock in the valley bottom.

**Wind River Formation (Tw)**: Generally flat-lying sequence of sandstone, mudstone, and siltstone. Based on core samples from DH-2, the bedrock appears to be friable and little weathered to about 5 feet below the bedrock surface. At depth the bedrock is tight and massive with few fractures. This unit is judged to be suitable for the dam foundation with minimal treatment other than possibly a shallow (3 to 5 feet deep maximum) seepage cutoff through the upper weathered zone.

### 4.3.4.2 Recommendations

Site 8 is suitable for dam and reservoir construction, with the following specific considerations and recommendations for foundation treatment and embankment materials:

- **Foundation stripping**: Clay and loose, silt or silty fine sand materials should be stripped from the dam foundation. In the valley bottom area, the stripping depth may be limited to the top of the older alluvium (conglomerate) layer, which should provide a strong foundation for an embankment dam. Shallow loose soil materials on the abutment slopes, and the thin layer

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3 Laboratory test results are provided in Appendix C.
of residual soils on the right abutment bedrock ledge should be completely stripped to bedrock under the dam.

- **Seepage cutoff:** If the conglomerate layer is left in place in the valley bottom, a cutoff trench should be excavated through this layer and keyed into bedrock to provide positive seepage cutoff in the foundation. On the abutments and along the bedrock ledge in the right abutment, the bedrock key trench should extend through the upper weathered zone (about 3 feet deep, typical).

**Borrow materials availability:** Materials for construction of a zoned embankment dam (excluding special filter/drain and riprap/bedding zones) may be obtained from the upstream reservoir area. Reservoir stripping for borrow should be limited to beyond about 100 feet upstream from the dam to maintain a seepage “blanket” adjacent to the dam. An estimated quantity of approximately 7400 CY of clay material is available from the remaining area in the reservoir bottom (below elevation 5145, and under the dam footprint, for use to construct a clay core and core trench. Sandy silt and gravelly sand materials available on site may be used to construct the outer shells of the embankment.

### 4.3.5 Embankment and Foundation Design Recommendations

#### 4.3.5.1 Design Considerations

Based on the geologic site investigations described in Section 4.3.4, and other sources of information, the key considerations for economical embankment design and foundation treatment requirements for Site 8 are summarized as follows:

1. Due to the relatively large drainage area for this small, off-channel impoundment (1.6 square miles), an emergency spillway will be necessary. It is anticipated that the emergency spillway will be an unlined channel cut into sandstone in the right abutment.
2. Materials for construction of a zoned embankment dam (excluding special filter/drain and riprap/bedding zones) may be obtained from the upstream reservoir area. Reservoir stripping for borrow should be limited to beyond about 100 feet upstream from the dam to maintain a seepage “blanket” adjacent to the dam. An estimated quantity of approximately 7400 CY of clay material is available from the remaining area in the reservoir bottom (below elevation 5145, and under the dam footprint, for use to construct a clay core and core trench. Sandy silt and gravelly sand materials available on site may be used to construct the outer shells of the embankment.
3. Clay and loose silty and sandy soils up to approximately 7 to 10 feet thick should be stripped from the dam foundation. In the valley bottom area, the stripping depth may be limited to the top of the older alluvium (dense sand and gravel “conglomerate”) layer, which should provide a strong foundation for an embankment dam. Shallow loose soil materials on the abutment slopes, and the thin layer of residual soils on the right abutment bedrock ledge should be completely stripped to bedrock under the dam.
4. If the conglomerate layer is left in place in the valley bottom, a cutoff trench should be excavated through this layer and keyed into bedrock to provide positive seepage cutoff in the foundation. On the abutments and along the broad bedrock ledge on the right side, the bedrock key trench should extend through the upper weathered zone (about 3 feet deep, typical).
5. The upper 3 to 5 feet of the Wind River Formation bedrock is weathered, friable and fractured, and requires a shallow cutoff to control seepage, as described in item 4 above. Bedrock quality in the vicinity of DH-2 improves with depth, and fractures and bedding-plane joints are generally tight and in-filled. Based on the preliminary geotechnical findings, it is assumed that deep foundation grouting will not be necessary.

6. Good quality, durable riprap is not available locally. The closest sources for durable, angular, quarried hard rock are over 30 miles from the site, and unit costs for this material are estimated to be extremely high (over $70/cubic yard). However, a local aggregate supplier was consulted who could supply large, rounded alluvial oversize materials, ranging in size from 4 inches to 4 feet. Preliminary riprap sizing analyses indicate that the oversize alluvial cobbles and boulders would be suitable as a potential riprap source for sites with moderate erosion-protection requirements (small fetch distances and limited wave action). As an alternative for this site, soil cement plating could be considered for upstream slope protection.

7. Alluvial gravel and sand resources are locally abundant in the Riverton area. It is assumed that local aggregate processing facilities can supply good-quality granular filter and drain materials to specifications for special zones at reasonable cost.

4.3.5.2 Dam Zoning Template and Foundation Cutoff Recommendations

The dam templates for the maximum section and right embankment for Site 8 are provided as Figure 4.28 (Dam Sections at Sta 2+00, and Sta 4+00). The embankment has a maximum height of 32 feet, with 15 feet crest width, 3H:1V upstream slope and 2.5H:1V downstream slope. Dam crest elevation is at elevation 5164 and normal pool (spillway crest) is at elevation 5159.

The recommended section is a zoned embankment with a central clay core (Zone 1) and upstream-offset clay cutoff trench, 10 feet wide at its base and keyed 3 feet (typical) into bedrock. The shells of the dam are comprised of silt and silty sand derived from local borrow excavation in the reservoir area. The shells of the maximum section (Sta 0+80 to Sta 2+50) of the dam can be founded on the dense sand and gravel (conglomerate) layer. The clay cutoff trench should completely penetrate this layer and be keyed into the underlying bedrock.

The clay core is protected by a vertical chimney filter/drain that ties into a horizontal blanket drain of the same gradation (Zone 2). Gradation design for Zone 2 was done using procedures outlined in NRCS (1994), and is illustrated on Figure 4.29 (Site 8 Filter/Drain (Zone 2) Gradation Design). As for Site 7A filter/drain zone, standard material specifications (ASTM C33-modified or WYDOT Fine Aggregate for Concrete) are suitable for this zone.

4.3.5.3 Preliminary Upstream Slope Protection Alternatives Considered

Two preliminary alternatives were considered for the Site 8 upstream slope protection. These alternatives are described as follows:

Riprap and Riprap Bedding: The design procedure for riprap slope protection was outlined in Section 4.2.5.3 for Site 7A, based on SCS (1983). A similar procedure was followed for evaluation of riprap requirements for Site 8. The effective fetch for Site 8 was only Fe = 0.11 mile. For this site the dam face is oriented such that the prevailing wind direction impinges on the upstream slope at an angle across the reservoir area. The design requirements for Site 8 were
very similar to Site 7A for riprap sizing and gradation. The gradations for riprap and bedding shown on Figure 4.14 (Site 7A and Site 8 Riprap (Zone 4) and Riprap Bedding (Zone 5) Gradations) are applicable to this site as well. Due to the anticipated fine grained characteristics of Zone 3, a zone of Filter/Drain material (Zone 2) will also be required under the riprap bedding at this site.

**Soil Cement Plating:** This potentially economical design alternative was considered for this site because of the relatively low height (25 feet) of this dam. The modest dam height would allow safe placement of a soil cement facing on a 3H:1V upstream slope, and the rapid drawdown stability concerns are not as significant as would be the case for a higher embankment, such as 7A West and East dams. (The riprap/bedding/filter zone on the upstream slopes of 7A embankments will facilitate rapid drainage during drawdown and reduces risk of drawdown-induced instability of the upstream slope.)

Soil cement is a mixture of soil, Portland cement and water that can be compacted to a high density using conventional earth-moving equipment. Soil cement is frequently used as slope protection where an economical source of rip-rap is not available.

Granular soils are recommended for soil-cement mixes since the soil cement may be subjected to repeated cycles of wetting/drying, freezing/thawing and wave action. Table 4.20 summarizes common performance standards for soil cement used in the U.S.

<table>
<thead>
<tr>
<th>Organization</th>
<th>Maximum Organic Content</th>
<th>Max. PI</th>
<th>Max. LL</th>
<th>2 inch Sieve % Passing</th>
<th>No. 4 Sieve % Passing</th>
<th>No. 8 Sieve % Passing</th>
<th>No. 200 Sieve % Passing</th>
<th>Wet-Dry Max % Loss</th>
<th>Freeze-Thaw Max % Loss</th>
<th>Min 7-Day UCC (psi)</th>
<th>Min 28-Day UCC (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>USACE</td>
<td>2%</td>
<td>121</td>
<td>**</td>
<td>100% Min</td>
<td>45% Min</td>
<td>**</td>
<td>6% - 35%</td>
<td>6%</td>
<td>8%</td>
<td>600</td>
<td>800</td>
</tr>
<tr>
<td>USBR</td>
<td>--</td>
<td>2</td>
<td>**</td>
<td>100% Min, 1.5%</td>
<td>65% Min</td>
<td>**</td>
<td>10% - 30%</td>
<td>6%</td>
<td>8%</td>
<td>600</td>
<td>875</td>
</tr>
<tr>
<td>PCA</td>
<td>65%</td>
<td></td>
<td>37%</td>
<td>65% Min</td>
<td>37% Min</td>
<td>25% Max</td>
<td>300 - 900</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Florida DOT</td>
<td>6%</td>
<td>19</td>
<td>25</td>
<td>100% Min</td>
<td>65% Min</td>
<td>37% Min</td>
<td>25% Max</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Georgia DOT</td>
<td>10%</td>
<td>25</td>
<td></td>
<td>65% Min</td>
<td>**</td>
<td>**</td>
<td>6% - 35%</td>
<td>6%</td>
<td>8%</td>
<td>200 - 800</td>
<td>250 - 1000</td>
</tr>
<tr>
<td>ACI Committee</td>
<td>2%</td>
<td></td>
<td></td>
<td>100% Min</td>
<td>65% Min</td>
<td>**</td>
<td>6% - 35%</td>
<td>6%</td>
<td>8%</td>
<td>200 - 800</td>
<td>250 - 1000</td>
</tr>
</tbody>
</table>

**Notes:**
1. USACE recommends "better quality granular materials" for levee protection.
2. USACE notes that clay balls can form at a PI as low as 8.
3. Bureau of Reclamation recommends "low plasticity or non-plastic fines".

For the types of soil materials generally available in the project area, optimum Portland cement contents are expected to range from about 5 to 9 percent by weight of soil, or 7-9 lbs per cubic foot of compacted soil cement. A portion of the Portland cement could be replaced by fly ash.

It is recommended that the soil cement facing be constructed in a thin slab (plating) configuration on a fairly flat (3H:1V) slope. The plating method consists of placing one or more layers of soil cement parallel to the slope, and is used where less severe exposure is expected (USACE, 2000). Although there are no definitive design criteria for lift thickness, experience has shown that 12 inches of protection is generally adequate in areas where plating is appropriate.
In the plating construction method, the slopes should be a maximum of 3H:1V in order to properly spread and compact the soil cement. The plating method would likely provide adequate protection for Site 8. It is recommended that a medium weight, non-woven geotextile (filter fabric) be placed on the slope under the soil cement to provide for drainage under the facing. A schematic representation of soil cement soil slope protection applied on a 3H:1V upstream slope is shown as Figure 4.30.

4.3.5.4 Recommendations for Supplemental Investigations and Final Design Analyses for Site 8 Dam and Spillway

Final design of the Site 8 embankment will require additional site investigations and analyses. Some of the key areas where additional work will be necessary are described as follows:

- **Supplemental geotechnical subsurface exploration and laboratory soils testing:**
  - Additional drilling and sampling should be done on the dam alignment to evaluate/confirm the condition of the overburden and bedrock along the alignment. Water pressure testing (packer testing) in boreholes within the bedrock is recommended to evaluate and optimize the clay core key-in depth requirements.
  - Core sampling and testing of the bedrock in the area of the spillway cut should be done to support analysis and evaluation of the unlined spillway, such as parameters needed to evaluate headcut erodibility index which is used in spillway erosion model analysis. Parameters to be evaluated typically include unconfined compressive strength, rock quality designation, and joint and fracture characteristics (NRCS, 1997).
  - Additional exploration and sampling of the clay deposits in the reservoir area should be completed to better define and confirm their lateral extent, thickness and engineering characteristics. These limited deposits are the anticipated source for the critical clay core zone (Zone 1). The investigation should confirm adequate quantities are available, and the engineering characteristics of these clay materials.
  - Sampling and laboratory testing of the clay (Zone 1), and the silty sand, sandy silt, and gravelly sand (Zone 3) materials is needed. Tests should include compaction tests and consolidated, undrained triaxial shear strength testing of both the Zone 1 and Zone 3 materials to support slope stability analyses.

- **Supplemental Geotechnical Analysis:**
  - Slope stability analyses should be performed to document acceptable factors of safety for all loading conditions: (a) static, steady seepage at full and partial pool elevations (upstream and downstream slopes), (b) rapid drawdown from full and partial pools (upstream slope), (c) pseudo-static and seismic deformation analysis (upstream and downstream slopes). The embankment slopes should be adjusted (flattened) as necessary to achieve required minimum factors of safety.
  - Seepage analyses should be performed to optimize the extent of upstream blanketing (clay and alluvium left in place upstream from dam), to design the blanket drain to ensure sufficient capacity to prevent phreatic line buildup in the downstream zone, and to size the toe drain outlet pipe.
Filter/Drain Design
Site 8

Site 8 Clay base soil
Permeability criteria
Filtering criteria

Filter Design Control Points

CLAY (plastic) TO SILT (non-plastic)
SAND
GRAVEL
COBBLES

Site 8 Clay Base Soil
ASTM C33 (modified)
WYDOT Fine Aggregate

Figure 4.29. Site 8 Filter/Drain (Zone 2) Gradation Design
Figure 4.30. Soil Cement Soil Plating Option for Upstream Slope Protection Site 8
4.3.6 Outlet Works

Conceptual designs were developed for the outlet works for this site as shown on Figures 4.25 and 4.31. The outlet works for this site would include an inverted slide gate on a 24” steel outlet pipe and an 18” pipeline to the LeClair canal. The design of the outlet works was developed to allow a supply of 15 cfs to be delivered to the LeClair canal with minimal depth in the reservoir. Approximately one foot of head above the outlet pipe would be required to flow 15 cfs. The 18” canal supply pipeline would be aligned along the south side of the draw to avoid the wetlands and riparian areas located below the reservoir in the bottom of the draw and to maintain a grade on the pipeline to the outfall. Water not delivered to the LeClair canal could be released below the toe of the embankment and conveyed down the draw to the existing underdrain structure which passes flows in the draw under the LeClair canal. The maximum capacity of the primary outlet works with discharge to the draw is approximately 56 cfs.

4.3.7 Emergency Spillway

Conceptual design for the emergency spillway was developed. This site will likely require a Class II Dam Hazard Classification since significant damage to downstream structures would be possible in the event of failure of the dam. This classification is based upon an evaluation of the consequences of a clear weather breach of the dam assuming the reservoir is at the high-water line. Safety of Dams Law states inflow design flood requirements for Class II dams are the Probable Maximum Flood (PMF). Spillway capacity must be designed according to the inflow design flood requirements. Generation of the PMF begins with the development of the Probable Maximum Precipitation (PMP) using Hydrometeorological Report No. 55A. The PMP was generated for both the general storm and the local storm. Hydrologic modeling of the drainage basin above site 8 was completed to determine the PMF. Stormwater runoff simulation was completed using U.S. Army Corps of Engineers developed HEC-HMS 2.2.2 hydrologic modeling system. The Soil Conservation Service (SCS) Unit Hydrograph method was used to generate the basin outflow hydrograph. The PMP depth-duration curve along with the drainage basin area, basin lag time, and drainage basin curve number were required input parameters. The basin lag time was calculated to be 75 minutes and the drainage area 1.58 mi² as shown on Figure 4.32. Table 4.21 shows the peak flow and runoff volume for the local and general storm PMF for the drainage above site 8. It was determined to use the local storm in spillway design calculations. The local storm generated higher peak flows and is characteristic of this region’s intense isolated storm events.

<table>
<thead>
<tr>
<th></th>
<th>Local Storm PMF</th>
<th>General Storm PMF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Drainage Discharge (cfs)</td>
<td>2578</td>
<td>2281</td>
</tr>
<tr>
<td>Total Direct Runoff (acre ft)</td>
<td>601</td>
<td>1714</td>
</tr>
<tr>
<td>Time of Peak (min.)</td>
<td>104</td>
<td>161</td>
</tr>
<tr>
<td>Total Precipitation (inch)</td>
<td>12.6</td>
<td>27.1</td>
</tr>
<tr>
<td>Total Loss (inch)</td>
<td>5.36</td>
<td>6.4</td>
</tr>
<tr>
<td>Total Excess (inch)</td>
<td>7.24</td>
<td>20.7</td>
</tr>
</tbody>
</table>

The local storm PMF generates 601 ac-ft of water with a peak flow of 2,578 cfs. The emergency spillway would be constructed by excavating a channel with a bottom width of 100’ and 3:1 horizontal to vertical side slopes. The water generated from the design storm was routed through
the emergency spillway for Reservoir 8. It was assumed that the reservoir would be full when
the storm occurred. The results of this analysis can be seen in Table 4.22.

<table>
<thead>
<tr>
<th>Table 4.22. Flood Routing Through Site 8 Emergency Spillway</th>
</tr>
</thead>
<tbody>
<tr>
<td>Emergency Spillway Width (ft)</td>
</tr>
<tr>
<td>Peak Elevation (ft)</td>
</tr>
<tr>
<td>Peak Emer. Spillway Outflow (cfs)</td>
</tr>
<tr>
<td>Time of Peak Outflow (min.)</td>
</tr>
</tbody>
</table>

The peak water level rise in the reservoir would be 4.53 feet. This meets the requirements falling
under the Safety of Dams law of no residual freeboard required for dams designed for the PMF.
The spillway would be re-vegetated with a seed mixture of native grasses. The emergency
spillway would be located adjacent to the right abutment of the embankment and discharge into
the drainage below the toe of the dam. The material excavated from the emergency spillway
could be used in zone 3 of the embankment.

4.3.8 SEO Permitting
Prior to construction, this reservoir would require filing an application for a permit to appropriate
surface water with the State Engineer. There are several required application forms to submit.
Site 8 would require filing for a S.W. 3 reservoir permit along with a S.W. 2 enlargement permit
on the reservoir supply (LeClair canal) and a S.W. 2 permit for supplemental supply to the
LeClair canal. The S.W. 2 permit for supplemental supply application requires the reservoir
capacity to be allocated to specific irrigated lands. This could be all land included in the LeClair
ID. In addition to these Wyoming SEO surface water storage permits, the Wyoming SEO would,
prior to construction, need to review the plans and specifications for dam safety approval and to
provide approval to construct the proposed facility.

LeClair ID would operate Site 8 independently from the Riverton Valley ID to store
supplemental supply. This site would serve the LeClair canal only. This storage reservoir would
not qualify as a re-regulation reservoir because it exceeds the one acre-ft size limit and the 24 hr
turnover requirement. It could fall under the one fill rule meaning it would be allowed one fill
per water year starting October 1st and ending September 30th. The one fill rule would only be
enforced when the Wind River is in regulation. The Wind River has only been in regulation one
time in the last twenty years. In practice this storage reservoir could be filled multiple times in a
season unless a call is placed on the river. If a call were placed on the river, then the storage
reservoir would only be allowed to fill once when its water right is in priority. The irrigation
district could fill Site 8 in the fall after the irrigation season or early spring before irrigation
begins to have a supplemental supply for startup when pre run-off flows in the river are low and
irrigation demand is high. Water could be again stored in this site when the farmers stop
irrigating to cut hay and then release water when the farmers resume irrigating and the demand is
high. This reservoir could be filled and emptied multiple times in a season. It is anticipated the
irrigation district could get two or three fills per season depending on the amount of runoff and
frequency of summer rainfall. This site could help reduce operational waste by storing water
when irrigation demand is less than irrigation supply. A storage volume of 120 ac-ft with two or
three reservoir fills would yield 240 to 360 ac-ft of water and would supply an additional 15 cfs
to the LeClair canal for approximately 8 to 12 days.

4.3.9 Additional Permits
In addition to the Wyoming SEO permits and approval, there are additional permits and
approvals required for new dam construction. The Army Corp of Engineers regulates activities
involving the waters of the United States. It is anticipated that an Individual Section 404 Permit
will be required. This will require that an Environmental Assessment be prepared and submitted
along with the Section 404 application. These include a Wyoming Department of Environmental
Quality National Pollution Discharge Elimination System (NPDES) permit and Section 401
Certification. This permit controls the discharge of stormwater pollutants associated with
construction activities. The Section 401 Certification is the State’s approval to ensure that the
proposed activities meet state water quality standards and do not degrade water quality. U.S.
Fish and Wildlife Service Endangered Species Act Compliance (Section 7) would be required.
Coordination with the U.S. Department of Interior Advisory Council on Historic Preservation
(Section 106), which protects cultural and historic resources, would be required. State of
Wyoming Historic Preservation Office (SHPO) archaeological clearance which determines
significance of cultural resources potentially affected by ground disturbing activities would be
required.
4.3.10 Cost Estimates

Based on the conceptual design presented above, conceptual cost estimates have been prepared. The cost estimates include costs for construction components, construction engineering, preparation of final design plans and specifications, permitting and mitigation, legal fees and acquisition of access and right-of-way. Construction costs were calculated using 2007 material and labor costs and are presented in the standard format used by the WWDC. The conceptual cost estimate for constructing Site No. 8 is presented in Table 4.23.

Table 4.23 - Site Number 8

<table>
<thead>
<tr>
<th>No.</th>
<th>Item</th>
<th>Units</th>
<th>Estimated Quantity</th>
<th>Unit Cost</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Mobilization</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$23,000.00</td>
</tr>
<tr>
<td>2</td>
<td>Zone I Embankment</td>
<td>C.Y.</td>
<td>6,524</td>
<td>$4.20</td>
<td>$27,400.80</td>
</tr>
<tr>
<td>3</td>
<td>Zone II Embankment</td>
<td>C.Y.</td>
<td>2,784</td>
<td>$21.00</td>
<td>$58,464.00</td>
</tr>
<tr>
<td>4</td>
<td>Zone III Embankment</td>
<td>C.Y.</td>
<td>34,590</td>
<td>$3.80</td>
<td>$131,442.00</td>
</tr>
<tr>
<td>5</td>
<td>Zone IV Embankment</td>
<td>C.Y.</td>
<td>2,327</td>
<td>$35.00</td>
<td>$81,445.00</td>
</tr>
<tr>
<td>6</td>
<td>Zone V Embankment</td>
<td>C.Y.</td>
<td>1,154</td>
<td>$21.00</td>
<td>$24,234.00</td>
</tr>
<tr>
<td>7</td>
<td>Key Trench</td>
<td>C.Y.</td>
<td>944</td>
<td>$6.93</td>
<td>$6,541.92</td>
</tr>
<tr>
<td>8</td>
<td>Perforated Toe Drain Pipe</td>
<td>L.F.</td>
<td>980</td>
<td>$4.00</td>
<td>$3,920.00</td>
</tr>
<tr>
<td>9</td>
<td>24&quot; Reservoir Supply Pipeline</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$10,000.00</td>
</tr>
<tr>
<td>10</td>
<td>Primary Outlet Works</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$75,000.00</td>
</tr>
<tr>
<td>11</td>
<td>Emergency Spillway</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$3,000.00</td>
</tr>
<tr>
<td>12</td>
<td>Revegetation</td>
<td>Ac.</td>
<td>10</td>
<td>$1,000.00</td>
<td>$10,000.00</td>
</tr>
</tbody>
</table>

**Construction Cost Sub-Total:** $454,447.72

10% Engineering: $45,444.77

**Sub-Total:** $499,892.49

15% Contingency: $74,983.87

**CONSTRUCTION COST TOTAL:** $574,876.37

Preparation of Final Designs and Specifications: $36,000.00

Permitting and Mitigation: $25,000.00

Legal Fees: $5,000.00

Acquisition of Access and Rights of Way: $160,000.00

**TOTAL PROJECT COST:** $800,876.37

**USE:** $801,000.00

Assuming a 67% WWDC grant and 33% loan at 4% for 50 years, the annual repayment would be $12,300.
Foundation stripping, embankment barrow, and embankment material import costs were included in the respective zoned embankment unit costs. Excavation for the emergency spillway is included in the Zone III Embankment bid item. Tables 4.24 and 4.25 show detailed conceptual cost estimates for the 24” reservoir supply pipeline and primary outlet works lump sum items.

Table 4.24 - Site 8 Reservoir Supply Pipeline

<table>
<thead>
<tr>
<th>No.</th>
<th>Item</th>
<th>Units</th>
<th>Estimated Quantity</th>
<th>Unit Cost</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>24&quot; Head Gate</td>
<td>E.A.</td>
<td>1</td>
<td>$5,000.00</td>
<td>$5,000.00</td>
</tr>
<tr>
<td>2</td>
<td>24&quot; Pipe</td>
<td>L.F.</td>
<td>50</td>
<td>$55.00</td>
<td>$2,750.00</td>
</tr>
<tr>
<td>3</td>
<td>Riprap</td>
<td>C.Y.</td>
<td>8</td>
<td>$70.00</td>
<td>$560.00</td>
</tr>
</tbody>
</table>

**Construction Cost Sub-Total:** $8,310.00

**USE:** $10,000.00

Table 4.25 - Site 8 Primary Outlet Works

<table>
<thead>
<tr>
<th>No.</th>
<th>Item</th>
<th>Units</th>
<th>Estimated Quantity</th>
<th>Unit Cost</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>24&quot; Inclined Slide Gate</td>
<td>E.A.</td>
<td>1</td>
<td>$10,000.00</td>
<td>$10,000.00</td>
</tr>
<tr>
<td>2</td>
<td>24&quot; Steel Pipe</td>
<td>L.F.</td>
<td>200</td>
<td>$150.00</td>
<td>$30,000.00</td>
</tr>
<tr>
<td>3</td>
<td>18&quot; HDPE or PVC Pipe</td>
<td>L.F.</td>
<td>680</td>
<td>$30.00</td>
<td>$20,400.00</td>
</tr>
<tr>
<td>4</td>
<td>Steel to HDPE Coupler</td>
<td>E.A.</td>
<td>1</td>
<td>$2,000.00</td>
<td>$2,000.00</td>
</tr>
<tr>
<td>5</td>
<td>24&quot; Valve</td>
<td>E.A.</td>
<td>2</td>
<td>$5,000.00</td>
<td>$10,000.00</td>
</tr>
<tr>
<td>6</td>
<td>Concrete</td>
<td>C.Y.</td>
<td>2</td>
<td>$500.00</td>
<td>$1,000.00</td>
</tr>
</tbody>
</table>

**Construction Cost Sub-Total:** $73,400.00

**USE:** $75,000.00
4.4 RESERVOIR SITE NUMBER 11

4.4.1 General
Site 11 is located in the E ½ of the NW ¼ of Section 28, Township 2 North, Range 5 East as shown on Figures 4.33 and 4.34. Access to this site is from the Riverton Valley canal bank road. The reservoir and appurtenances would be located on deeded land owned by Foster Ranch LLC and Eric D. and Cathleen G. Yochheim. The dam and all appurtenances would be located on the property owned by Foster Ranch LLC. The reservoir pool would begin to inundate the Yochheim property at an elevation of 4879’. The Yochheim property is in the process of being sold and subdivided. The developer was contacted and was favorable to the idea of having an irrigation storage reservoir inundate a portion of the development. Attached in Appendix D is a letter dated April 13, 2006 to Robert Olson with BC Development to pursue this storage option. Further coordination and written agreement with the developer would need to occur in order to pursue this storage option. It is anticipated that the land needed for the reservoir and appurtenances would be purchased and owned by Riverton Valley ID. As an alternative to purchasing the land inundated by the reservoir pool, an easement could be obtained to allow the current landowner to control access for non-irrigation purposes. The landowners were contacted for permission to survey and conduct study related investigations. No formal appraisals or land or easement purchase negotiations have taken place. Land or easement purchase was estimated at $5,000 per acre for use in this study.

4.4.2 Reservoir Supply
The supply for Site 11 would be the flow in the drainage. The drainage flows water year round, however, the flow is influenced by agriculture. The drainage flows more water during the irrigation season by conveying return flows from the LeClair ID. The concept would be to utilize these flows in the drainage. The drainage is estimated to flow approximately 3.5 – 6 cfs during the irrigation season. Two options were considered at this location. A small storage reservoir could store and release the flows to the Riverton Valley canal. The estimated supply flow rate could fill the reservoir in one to two days. The second option is to install a diversion structure to divert the flows in the drainage into the Riverton Valley canal. No testing has been completed on the quality of this water.

4.4.3 Reservoir Sizing
Conceptual designs were developed for both options at this site, which is located near the end of the Riverton Valley canal. The reservoir option could store water to a high-water line of 4890’. This site could store approximately 12 ac-ft of water. The diversion structure option would check the water level up to elevation 4882’ and create a small pool backing water approximately 380’ upstream. The embankment required to store water to an elevation of 4890’ would contain 5,840 CY of material yielding an embankment efficiency of 486 CY/ac-ft.

4.4.4 Geotechnical/Geologic Investigations
Geotechnical/geologic site investigations and evaluation of site conditions are provided in the following section for this site.

4.4.4.1 Site Conditions and Geologic/Geotechnical Concerns
Site 11 was explored by geologic reconnaissance. A geologic site map for the diversion dam, and an interpreted geologic profile on the proposed dam alignment, are provided as Figures 4.35 (Site No. 11 Geologic Map) and 4.36 (Site No. 11 Geologic Profile), respectively. The reconnaissance found an exposure of interbedded siltstone/sandstone (Wind River Formation) bedrock in the stream bottom about 100 feet downstream from the proposed diversion site. The bottom of the incised drainage contains alluvial deposits a few feet thick above bedrock. The banks are older alluvial sediments (Qalo) comprised of unconsolidated, very fine-grained silty sand deposits. Laboratory tests were conducted on a bulk sample taken from the left stream bank, including gradation, compaction and permeability tests. Test results are provided in Appendix C – Lab Test Results. The lab test data indicate that the silty fine sand materials comprising the abutments at Site 11 are relatively impervious \(k_\text{vertical} = 3 \times 10^{-6} \text{ cm/s}\). However, the non-plastic, fine-grained characteristics of these materials make them highly vulnerable to both surface erosion and internal seepage erosion or piping under even moderate seepage gradient through the abutments.

### 4.4.4.2 Recommendations

Site reconnaissance indicates that bedrock exists at fairly shallow depth below the channel bottom. It is recommended that the alluvial materials be stripped down to bedrock across the channel bottom in order to found the diversion structure on bedrock.

The channel bank materials are very susceptible to piping and internal erosion under even moderate hydraulic heads and gradients. It is recommended that a sheet pile seepage cutoff be constructed to reduce “end-around” seepage gradients adjacent to the structure. The sheet piling should be driven into bedrock and extended well into both abutments. In addition, a filter blanket should be placed on the downstream bank slopes below the structure to capture the end-around seepage exiting the slope and protect against initiation of piping erosion.

### 4.4.5 Embankment and Foundation Design Recommendations

Detailed storage option typical dam sections were not developed for this site due to the cost effectiveness of the diversion structure option. Cost estimates for the storage option embankment were developed using the average unit cost for embankment at Site 7A.

#### 4.4.5.1 Sheet Pile Structure Design Considerations

Based on the geologic site investigations described in Section 4.4.4, the key design considerations for the regulating structure at Site 11 are as follows:

1. Site reconnaissance indicates that sandstone bedrock exists at fairly shallow depth below the channel bottom. It is recommended that the alluvial materials be stripped down to bedrock across the channel bottom in order to found the diversion structure on bedrock.

2. The channel bank materials are very susceptible to piping and internal erosion under even moderate hydraulic heads and gradients. It is recommended that a sheet pile seepage cutoff be constructed to reduce “end-around” seepage gradients adjacent to the structure. The sheet piling should be driven into bedrock and extended into both abutments sufficiently to reduce seepage gradients to below critical values on the bank downstream from the cutoff wall.
3. A filter blanket should be placed on the downstream bank slopes below the structure to capture the end-around seepage exiting the slope and protect against initiation of piping erosion.

4.4.5.2 Seepage Analysis for “End-Around” Sheet Pile Cutoff

Preliminary seepage analyses were completed to evaluate the head loss and seepage exit gradients on the bank slope downgradient from the structure. Analyses are provided in Appendix B (Preliminary Dam Design Analyses – Site 11 Seepage Modeling).

The exit gradient and seepage around the sheet pile wall were calculated using three methods: (1) hand drawn flow net with unknown phreatic surface, (2) Corps of Engineers Method of Fragments, and (3) SEEP/W numerical model. These methods are typically used for calculating unconfined seepage flow through earth embankments, which was adapted to this problem by straightening the flow path around the ends of the sheet pile wall so that it could be represented in two dimensions. The flow path was assumed to be from the maximum (500-year) water surface elevation upstream from the pile to the tailwater downstream. The initially assumed cutoff length was 9 feet embedment horizontally from the point on the bank where the maximum water surface elevation (4889) meets the sheet pile wall. The results are summarized on Table 4.26.

Table 4.26. End-Around Seepage Analysis – Sheet Pile Cutoff Site 11

<table>
<thead>
<tr>
<th>Method</th>
<th>Q (gpd/ft)</th>
<th>i&lt;sub&gt;exit&lt;/sub&gt;</th>
<th>Exit point (ft above toe of slope)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flow net</td>
<td>3.8</td>
<td>0.28</td>
<td>NA1</td>
</tr>
<tr>
<td>Method of fragments</td>
<td>4.6</td>
<td>NA1</td>
<td>2.6</td>
</tr>
<tr>
<td>SEEP/W</td>
<td>2.9</td>
<td>0.31</td>
<td></td>
</tr>
</tbody>
</table>

Limited guidance was found regarding allowable exit gradients for unprotected seepage emerging on a natural slope. The U.S. Army Corps of Engineers, for underseepage in levee design, recommend a maximum exit gradient on the landward side = 0.5. Lane's weighted creep ratio is an empirical technique that was historically used for designing cutoffs or apron lengths to reduce uplift pressures under concrete dams. Lane recommends weighted creep ratios (WCR) for various soil material types. The WCR is the ratio of the weighted creep distance (= length of flow path around cutoff/apron etc) divided by the effective head loss. For silty fine sand to fine sand, Lane recommends WCR = 7 to 8.5. For the conditions at Site 11, this would be equivalent to exit gradients on the order of 0.12 to 0.14.

The exit gradients for a 9-feet pile embedment depth appear to be marginally safe to non-conservative, based on these two criteria. As such, additional protection of a weighted filter blanket on the downstream bank slopes is prudent and justified. A properly designed filter blanket will safely intercept seepage flows emerging from around the ends of the piles on the bank slopes downstream, and will protect against development of internal erosion and piping (undermining) at the abutments.
4.4.5.3 Sheet Pile Structure Filter Blanket Design

The lateral dimensions and typical cross section for the weighted filter blanket are shown on Figures 4.34 (Site No. 11 Diversion Structure Plan) and 4.37 (Site No. 11 Diversion Structure Details). The filter blanket gradation requirements are designed using similar procedures as described for the filter/drain zones for the embankment dams at Sites 7A and Site 8 (NRCS 1994). The design gradation band that is applicable for the base soil gradation is shown on Figure 4.38 (Site 11 Filter Blanket Gradation Design). The WYDOT Fine Aggregate (or ASTM C33 aggregate) standard gradations are too fine to meet these requirements. For comparison, another WYDOT standard (for Superpave and Marshall pavement mix designs with 1-inch aggregate) was found that is fairly close, and does meet the critical filtering criteria, as shown on Figure 4.38. However, it does not meet the permeability requirement. Local gravel suppliers should be able to produce a gradation that fits within the design filter band. A 9-inch minimum filter thickness (perpendicular to the bank slope) is recommended to accommodate the expected seepage discharges.

The filter blanket needs to be protected against erosion and scour during overtopping flows in the channel. It is recommended that a 2-feet thick, grouted riprap revetment be placed over the filter blanket. A 10 inch minimum grout penetration is assumed, based on the rock gradation available for the grouted rip rap (4” to 12” assumed). The filter blanket will extend from the structure downstream approximately 30 feet. The recommended vertical extent is from a toe-in to bedrock up to the high water line elevation (4889). Drain pipes should be provided through the grouted rip rap on 10 ft centers,

4.4.5.4 Recommendations for Supplemental Investigations and Final Design Analyses for Site 11

It is recommended that at least 2 borings be drilled to confirm depth to bedrock in both abutments to support final design of the Site 11 sheet pile cutoff structure.
Figure 4.37. Site 11 Filter Blanket Gradation Design
4.4.6 Outlet Works

4.4.6.1 Storage Reservoir Option
Conceptual designs are provided for the outlet works for this site as shown on Figures 4.39 and 4.40. The primary outlet works for this site would include a concrete inlet box, a 20 foot concrete gate tower, a 24” steel outlet pipe, an 18” steel pipeline to connect with the Riverton Valley canal, and an impact basin to slow outlet velocities released to the draw. The concrete gate tower outlet works design was chosen to maintain the normal high water level and allow excess flow to automatically pass through the outlet works and discharge to the drainage without activating the emergency spillway. Water not delivered to the Riverton Valley canal would be released through the impact basin and conveyed down the draw. The capacity of the 24” primary outlet works is approximately 38 cfs at normal reservoir head.

There are approximately 200 acres of irrigated lands below this site on the Riverton Valley canal. The design supply flow rate of 2 cfs per 70 acres of irrigated land computes to 5.7 cfs. The design of the canal supply pipeline was developed to allow this supply rate to be delivered to the Riverton Valley canal with minimal depth in the reservoir. Water delivered to the Riverton Valley canal via the 18” pipeline would tie-in to the existing 36” steel piped canal as shown on Figure 4.39.

4.4.6.2 Diversion Structure Option
Flow diverted with the diversion structure option would be piped with an 18” pipeline to the Riverton Valley canal as shown in Figures 4.34 and 4.37. The design of this pipeline was developed using the same supply flow rate as determined for the storage option. Flow in the drainage is checked up to a sufficient elevation with the use of the sheet pile weir to generate the required head to transmit the flows to the Riverton Valley canal.

4.4.7 Emergency Spillway
Conceptual design for the storage option emergency spillway was developed. This site will likely require a Class IV Dam Hazard Classification since no loss of human life would be expected and damage would occur only to the dam owner’s property in the event of failure of the dam. This classification is based upon an evaluation of the consequences of a clear weather breach of the dam assuming the reservoir is at the high-water line. Safety of Dams Law has no inflow design flood requirements for Class IV dams. The emergency spillway was sized by estimating the peak stream flow using the basin characteristics method for the plains region as described in the U.S. Geological Survey Water Resources Investigations Report 88-4045 (H.W. Lowham, 1988) for a 100 year recurrence interval. The regression equation used to determine the peak flow is:

\[ P_{100} = 130A^{0.58}S_B^{0.25}G_f \]  

Where:

- \( P_{100} \) = Annual Peak Flow (cfs) for 100 year recurrence interval.
- \( A \) = Contributing Drainage Area (square miles).
- \( S_B \) = Basin Slope (feet/mile).
- \( G_f \) = Geographic Factor, determined from Plate 1d. (1.0 was used)
SITE 11 DAM PROFILE

4870  4880  4890  4900  4910  4920  4930  4940  4950
0+00  1+00  2+00  3+00  4+00  5+00  6+00  7+00  8+00

18" CONCRETE TOWER

18" CONCRETE TOWER

60" EMERGENCY SPILLWAY

CONCRETE OUTLET TOWER

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CONC
The result of this investigation was that the drainage, shown on Figure 4.41, could produce a peak stream flow of approximately 825 cfs. The emergency spillway would be constructed by excavating a channel with a 60’ bottom width and 3:1 horizontal to vertical side slopes. This would allow the entire flow to pass the spillway with a maximum depth of 3 feet. This maintains no free board on the dam. The spillway will be re-vegetated with a seed mixture of native grasses. The emergency spillway would be located adjacent to the left abutment of the embankment and discharge into the drainage below the toe of the dam.

4.4.8 SEO Permitting
Prior to construction, this reservoir would require filing an application for a permit to appropriate surface water with the State Engineer. There are several required application forms to submit. Site 11 Storage Option would require filing for a S.W. 3 reservoir permit and a S.W. 2 permit for supplemental supply to the Riverton Valley canal. The S.W. 2 permit for supplemental supply application requires the reservoir capacity to be allocated to specific irrigated lands. This could be all land included in the Riverton Valley ID. The Diversion Option also would require filing for a S.W. 2 permit for supplemental supply to the Riverton Valley canal. In addition to these Wyoming SEO surface water storage permits, the Wyoming SEO would, prior to construction, need to review the plans and specifications for dam safety approval and to provide approval to construct the proposed facility.

Riverton Valley ID would operate Site 11 Storage option independently from LeClair ID to store supplemental supply. This site would serve the Riverton Valley canal only. This storage reservoir would not qualify as a re-regulation reservoir because it exceeds the one acre-ft size limit and the 24 hr turnover requirement. It could fall under the one fill rule meaning it would be allowed one fill per water year starting October 1st and ending September 30th. In practice, however, this storage reservoir could be filled multiple times in a season unless a call is placed on the river. If a call were placed on the river, then the storage reservoir would only be allowed to fill once when its water right is in priority. It would be necessary with this site’s limited capacity to fill multiple times in a season. A storage volume of 12 acre-ft would augment the 1 cfs per 70 acres of irrigated land allocation to the Riverton Valley canal for approximately 2 days.

4.4.9 Additional Permits
In addition to these Wyoming SEO permits and approval, there are additional permits and approvals required for new dam construction. The Army Corp of Engineers regulates activities involving the waters of the United States. It is anticipated that an Individual Section 404 Permit will be required. This will require that an Environmental Assessment be prepared and submitted along with the Section 404 application. These include a Wyoming Department of Environmental Quality National Pollution Discharge Elimination System (NPDES) permit and Section 401 Certification. This permit controls the discharge of stormwater pollutants associated with construction activities. The Section 401 Certification is the State’s approval to ensure that the proposed activities meet state water quality standards and do not degrade water quality. U.S. Fish and Wildlife Service Endangered Species Act Compliance (Section 7) would be required. Coordination with the U.S. Department of Interior Advisory Council on Historic Preservation (Section 106), which protects cultural and historic resources, would be required. State of Wyoming Historic Preservation Office (SHPO) archaeological
States West Water Resources Corporation

WARNING

If you do not observe the following.

DRAINAGE BASIN STATISTICS
SITE II
DRAINAGE AREA ~ 1.79 SQ. MI
BASIN SLOPE ~ 439 FT/MI
BASIN LAG TIME ~ 48 MIN.

Figure 4.41
SITE NO. 11 DRAINAGE AREA
4–86
clearance which determines significance of cultural resources potentially affected by ground disturbing activities would be required.

4.4.10 Cost Estimates
Based on the conceptual designs presented above, conceptual cost estimates have been prepared. The cost estimates include costs for construction components, construction engineering, preparation of final design plans and specifications, permitting and mitigation, legal fees and acquisition of access and right-of-way. Construction costs were calculated using 2007 material and labor costs and are presented in the standard format used by the WWDC. The conceptual cost estimate for constructing Site No. 11 Storage Option is presented in Table 4.27. Dam sections and detailed embankment quantity takeoffs were not completed for this site. The unit cost of embankment was estimated using a weighted average of the unit costs for Site 7A.

Table 4.27 - Site Number 11 Storage Option

<table>
<thead>
<tr>
<th>No.</th>
<th>Item</th>
<th>Units</th>
<th>Estimated Quantity</th>
<th>Unit Cost</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Mobilization</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$15,000.00</td>
</tr>
<tr>
<td>2</td>
<td>Embankment</td>
<td>C.Y.</td>
<td>5,837</td>
<td>$7.00</td>
<td>$40,859.00</td>
</tr>
<tr>
<td>3</td>
<td>PVC Liner</td>
<td>S.F.</td>
<td>100,000</td>
<td>$1.42</td>
<td>$142,000.00</td>
</tr>
<tr>
<td>4</td>
<td>Primary Outlet Works</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$94,000.00</td>
</tr>
<tr>
<td>5</td>
<td>Emergency Spillway</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$500.00</td>
</tr>
<tr>
<td>6</td>
<td>Revegetation</td>
<td>Ac.</td>
<td>4</td>
<td>$1,000.00</td>
<td>$4,000.00</td>
</tr>
</tbody>
</table>

Construction Cost Sub-Total: $296,359.00
10% Engineering: $29,635.90
Sub-Total: $325,994.90
15% Contingency: $48,899.24
CONSTRUCTION COST TOTAL: $374,894.14
Preparation of Final Designs and Specifications: $24,000.00
Permitting and Mitigation: $25,000.00
Legal Fees: $5,000.00
Acquisition of Access and Rights of Way: $20,000.00
TOTAL PROJECT COST: $448,894.14

USE: $449,000.00

Assuming a 67% WWDC grant and 33% loan at 4% for 50 years, the annual repayment would be $6,900.
Table 4.28 shows detailed conceptual cost estimates for the outlet works lump sum bid item.

Table 4.28 - Site 11 Concrete Gate Tower Outlet Works

<table>
<thead>
<tr>
<th>No.</th>
<th>Item</th>
<th>Units</th>
<th>Estimated Quantity</th>
<th>Unit Cost</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Concrete</td>
<td>C.Y.</td>
<td>31</td>
<td>$500.00</td>
<td>$15,500.00</td>
</tr>
<tr>
<td>2</td>
<td>24&quot; Sluice Gate</td>
<td>E.A.</td>
<td>1</td>
<td>$10,000.00</td>
<td>$10,000.00</td>
</tr>
<tr>
<td>3</td>
<td>24&quot; Steel Pipe</td>
<td>L.F.</td>
<td>130</td>
<td>$150.00</td>
<td>$19,500.00</td>
</tr>
<tr>
<td>4</td>
<td>18&quot; Head Gate</td>
<td>E.A.</td>
<td>1</td>
<td>$5,000.00</td>
<td>$5,000.00</td>
</tr>
<tr>
<td>5</td>
<td>18&quot; Steel Pipe</td>
<td>L.F.</td>
<td>80</td>
<td>$100.00</td>
<td>$8,000.00</td>
</tr>
<tr>
<td>6</td>
<td>10'x12' Building</td>
<td>S.F.</td>
<td>120</td>
<td>$150.00</td>
<td>$18,000.00</td>
</tr>
<tr>
<td>7</td>
<td>Intake Box (4'x4'x6') Concrete</td>
<td>C.Y.</td>
<td>5</td>
<td>$500.00</td>
<td>$7,500.00</td>
</tr>
<tr>
<td>8</td>
<td>Impact Basin</td>
<td>E.A.</td>
<td>1</td>
<td>$10,000.00</td>
<td>$10,000.00</td>
</tr>
</tbody>
</table>

Construction Cost Sub-Total: $93,500.00

USE: $94,000.00

The conceptual cost estimate for constructing Site No. 11 Diversion Option is presented in Table 4.29.

Table 4.29 - Site Number 11 Diversion Structure Option

<table>
<thead>
<tr>
<th>No.</th>
<th>Item</th>
<th>Units</th>
<th>Estimated Quantity</th>
<th>Unit Cost</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Mobilization</td>
<td>L.S.</td>
<td>--</td>
<td>--</td>
<td>$5,000.00</td>
</tr>
<tr>
<td>2</td>
<td>Sheet Pile</td>
<td>S.F.</td>
<td>1500</td>
<td>$25.00</td>
<td>$37,500.00</td>
</tr>
<tr>
<td>3</td>
<td>Grouted Riprap</td>
<td>C.Y.</td>
<td>172</td>
<td>$100.00</td>
<td>$17,200.00</td>
</tr>
<tr>
<td>4</td>
<td>Filter Blanket</td>
<td>C.Y.</td>
<td>47</td>
<td>$21.00</td>
<td>$987.00</td>
</tr>
<tr>
<td>5</td>
<td>Perforated Drain Pipe</td>
<td>L.F.</td>
<td>100</td>
<td>$10.00</td>
<td>$1,000.00</td>
</tr>
<tr>
<td>6</td>
<td>18&quot; Slide Gate</td>
<td>E.A.</td>
<td>1</td>
<td>$5,000.00</td>
<td>$5,000.00</td>
</tr>
<tr>
<td>7</td>
<td>18&quot; Diversion Pipe</td>
<td>L.F.</td>
<td>230</td>
<td>$30.00</td>
<td>$6,900.00</td>
</tr>
<tr>
<td>8</td>
<td>Revegetation</td>
<td>Ac.</td>
<td>1</td>
<td>$1,000.00</td>
<td>$1,000.00</td>
</tr>
</tbody>
</table>

Construction Cost Sub-Total: $74,587.00

10% Engineering: $7,458.70

Sub-Total: $82,045.70

15% Contingency: $12,306.86

CONSTRUCTION COST TOTAL: $94,352.56

Preparation of Final Designs and Specifications: $7,500.00

Permitting and Mitigation: $5,000.00

Legal Fees: $5,000.00

Acquisition of Access and Rights of Way: $10,000.00

TOTAL PROJECT COST: $121,852.56

USE: $122,000.00

Assuming a 67% WWDC grant and 33% loan at 4% for 50 years, the annual repayment would be $1,900.
4.5 RESERVOIR SITE NUMBER 4

4.5.1 General
Site 4 is situated in a relatively large, southward-draining, natural channel that is deeply incised through thick alluvial sediments. This site is the location of an existing small embankment dam. Site 4 is located in the E ½ of the SW ¼ of Section 26, Township 1 North, Range 3 East as shown previously on Figure 3.9. This site is located north of the Riverton Valley canal and south of River View Road near the Riverton Valley diversion. This site was considered as a potential small storage site that would require a 26-feet high, 400 feet long dam to impound approximately 90 acre feet. This site could store water to a high-water line of 5069’. Access to this site is a private road off of River View Road. The reservoir and appurtenances would all be located on deeded land owned by Bruce L. and Barbara A. Brimmer and Michael F. and Cathy L. Johnson. The dam and all appurtenances would be located on the Brimmer property. The reservoir pool would inundate approximately 0.2 acres of the Johnson property. It is anticipated that the land needed for the reservoir and appurtenances would be purchased and owned by the LeClair and Riverton Valley ID. As an alternative to purchasing the land inundated by the reservoir pool, an easement could be obtained to allow the current landowners control over access for non-irrigation purposes. The landowners were contacted for permission to survey and conduct study related investigations. No formal appraisals or land or easement purchase negotiations have taken place.

4.5.2 Design Flood
This site will likely carry a Class IV Dam Hazard Classification since no loss of human life is expected and damage will occur only to the dam owner’s property in the event of failure of the dam. This classification is based upon an evaluation of the consequences of a clear weather breach of the dam assuming the reservoir is at the high-water line. Safety of Dams Law has no inflow design flood requirements for Class IV dams. The emergency spillway could be sized by estimating the peak stream flow using the basin characteristics method for the plains region as described in the U.S. Geological Survey Water Resources Investigations Report 88-4045 (H.W. Lowham, 1988) for a 100 year recurrence interval. The regression equation used to determine the peak flow is:

\[
P_{100} = 130A^{0.58A^{-0.05}}S_B^{0.25}G_f
\]

Where:
- \( P_{100} \) = Annual Peak Flow (cfs) for 100 year recurrence interval.
- \( A \) = Contributing Drainage Area (square miles).
- \( S_B \) = Basin Slope (feet/mile).
- \( G_f \) = Geographic Factor, determined from Plate 1d. (1.0 was used)

The result of this investigation was that the drainage, as shown on Figure 4.42, could produce a 100 year peak stream flow of approximately 1,000 cfs. The emergency spillway could be constructed by excavating a channel with an 80’ bottom width and 3:1 horizontal to vertical side slopes. This would allow the entire flow to pass the spillway with a maximum depth of 3 feet.
Figure 4.42
SITE NO. 4 DRAINAGE AREA
4-90
4.5.3 Site Conditions and Geologic/Geotechnical Concerns

The proposed dam alignment would be located approximately 250 feet downstream from an existing small embankment dam. The reservoir behind the existing dam is completely silted in. According to local resident and project team member, Mr. Dick Inberg, the small dam was constructed in the mid-1980’s, and has been silted in for several years (personal communication April 11, 2006).

A geologic site map for the dam and reservoir area, and a geologic profile on the proposed dam alignment, are provided as Figures 4.43 (Site No. 4 Geologic Map) and 4.44 (Site No. 4 Geologic Profile), respectively. Geologic exploration of the site included site reconnaissance and mapping, and subsurface exploration of the left abutment by a 43-feet deep boring (DH-3)\(^4\). The field observations indicate that the abutments for the dam are comprised of unconsolidated, silts, sands and gravels. Siltstone bedrock was encountered in DH-3 at a depth of 42 feet, or about 5 to 10 feet below the channel bottom elevation. The granular soil materials above bedrock appear to grade coarser with depth in the abutment. Very loose to loose silt and silty fine sand was encountered from the ground surface in the upper abutment to about 24 feet depth. These finer-grained materials grade to medium dense sandy gravel and gravelly sand in the lower abutment. These pervious, coarse-grained soils were encountered from about 24 feet below ground surface to bedrock.

A well-established wetland vegetation community is present in the bottom of the drainage at the dam site, extending well upstream including and above the existing sediment-filled impoundment. There appears to be significant seepage from the existing, filled-in impoundment, which feeds the downstream wetland area.

The fine silty and sandy materials in the upper abutments at this site are also prevalent along the upstream channel that drains into the site. These materials are highly vulnerable to both surface erosion and internal seepage erosion or piping under seepage forces through the embankment and the abutments at this location. The coarse-grained gravelly sands and sandy gravels in the lower abutments are anticipated to be highly pervious.

Special design measures would be required to construct a safe dam at this site, including extensive seepage cutoffs into the abutments, and internal filters and drains to protect against seepage-erosion failure modes in the dam and both abutment areas. In addition, the upstream channel would probably require slope protection or lining to avoid excessive scour and accumulation of sediment behind the structure under the higher stream flows that would be expected under system operation. The relatively rapid sediment infilling of the smaller upstream reservoir is clear evidence that sedimentation is problematic for this site. Another serious constraint for this site is that the basin drainage area is large relative to the small size of the reservoir (over 2 square miles). Therefore, this structure would likely require an emergency spillway to protect the dam from overtopping during extreme flood events. The highly erodible, loose soil materials in the abutment areas are unsuitable foundations for a concrete-lined spillway.

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\(^4\) Subsurface exploration boring logs, test pit logs, and site photos are provided in Appendix C
4.5.4 Recommendations

Adverse geologic and other site conditions encountered at Site 4 are considered to represent fatal flaws for economic construction of a safe dam and reservoir at this site. These adverse conditions are summarized as follows:

- Absence of competent bedrock or dense impervious soil on which to found the dam in either abutment;
- Presence of loose, highly erodible silt and silty fine sand in the upper abutments;
- Presence of highly pervious sand and gravel layers in the lower abutments;
- Significant potential for rapid sedimentation (infilling) of the reservoir impoundment area; and
- Poor soil conditions in both abutments prohibit economic construction of an emergency spillway, which would be required due to the relatively large drainage basin area.

It is the recommendation of the project team that Site 4 be eliminated from further consideration for dam and reservoir construction.
4.6 RESERVOIR SITE NUMBER 2

4.6.1 General
Site 2 was initially considered as a potential site for a small re-regulating reservoir as shown previously on Figures 3.8 and 4.17. The water storage capacity at this site was anticipated to be less than 90 acre-feet. The primary function of this structure would be to facilitate water delivery from the Site 7A storage reservoir into the Riverton Valley canal.

4.6.2 Site Conditions and Geologic/Geotechnical Concerns
Site 2 is situated immediately north of the Riverton Valley Canal in a small, southward-draining, natural channel that is incised through alluvial sediments. Surface runoff in the drainage is conveyed under the canal via a 36” pipe culvert. Geologic reconnaissance of the site indicates the drainage is incised into relatively homogeneous, unconsolidated, very fine-grained sand and silty sand deposits. Well-established wetland vegetation covers the bottom of the drainage at the location of the proposed structure.

The fine silty and sandy materials at this site, which are also prevalent along the upstream drainage channel approaching the site, are highly vulnerable to erosion. These materials are problematic with regard to both surface erosion and internal erosion or piping caused by seepage forces through and under a dam at this location. Special design measures such as seepage cutoffs and internal filters and drains would be required to protect against seepage-erosion failure modes. In addition, the upstream channel would probably require slope protection or lining to avoid excessive scour and accumulation of sediment behind the structure under the higher stream flows that would be expected under system operation.

4.6.3 Recommendations
Based on the unfavorable foundation and abutment conditions regarding seepage and piping potential, high probability for erosion and scour in the channel, and the presence of high-quality wetlands at the site, it is recommended that a re-regulating structure not be constructed at this location. The recommended alternative is to convey water directly from Site 7A to the canal via an extended outlet pipeline.
5. ENVIRONMENTAL REPORT
WETLANDS, SENSITIVE SPECIES AND BIG GAME HABITAT ASSOCIATED WITH POTENTIAL RESERVOIR SITES – RIVERTON-LECLAIR IRRIGATION DISTRICT

Prepared for:

States West Water Resources Corporation
Cheyenne, Wyoming

Prepared by:

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2003 Central Avenue
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May 23, 2006
INTRODUCTION

The Wyoming Water Development Commission is considering construction of one or more reservoirs to supply additional storage for the Riverton-LeClair irrigation districts in Fremont County, Wyoming. Based on initial screening the number of potential reservoir sites has been reduced to 5, including sites 2, 4, 7, 8 and 11. This report looks at biological criteria including wetlands; threatened, endangered and sensitive wildlife and plants; migratory birds; and big game habitats.

METHODS

Maps showing locations of wetlands within the dam footprint, high water lines and other impact areas (i.e., supply canals) as based on U.S. Fish and Wildlife Service National Wetland Inventory (NWI) maps were provided by States West Water Resources, along with the size of each wetland potentially impacted. Each reservoir site was visited on May 8, 2006 and each of the NWI wetlands was examined to check accuracy of the NWI maps. Each site was also examined to determine presence of wetlands not shown on the NWI map.

Previously documented occurrences of species of concern within the project area (defined as all townships containing potential reservoir sites) were determined through searching the Wyoming Natural Diversity Database (WNDD) maintained by the University of Wyoming. The WNDD computer search included species of concern within their standard one township buffer around the township each site is in. To obtain information on big game habitats associated with each site, big game herd unit maps maintained at the Cheyenne office of the WGFD were examined. A letter was sent to the U.S. Fish & Wildlife Service (USFWS) requesting information on federally listed species potentially occurring in the project area.

RESULTS

Wetlands

Site 2. The proposed dam site for Site #2 is on a channel with extensive amounts of wetlands. The NWI wetland map appeared fairly accurate for this site and total wetland impacts would likely be around 3.5 acres.

Site 4. Site # 4 also has extensive amounts of wetlands. The NWI map appears to be inaccurate for this site. In addition to the 0.88 acres shown on the NWI map within the inundation zone, wetlands also extend further up the drainage as well as below the existing NWI wetland within the same drainage. It is likely that total wetland impacts for this site would approach 1.5 acres or more rather than just the 0.88 acres based on NWI mapping.

Site 7. No wetlands are on the NWI map for the inundation area of this reservoir. However, wetlands occur within two small drainages on the site. These wetlands occur along approximately 450 linear feet of channel and average approximately 10 feet in width, for a total of approximately 0.10 acres. In addition, irrigation runoff has formed wetlands in a low area within an irrigated field on the west half of the site. This area appears to total approximately 0.75 acres. It is not associated with a drainage and is therefore likely not jurisdictional.

Site 8. No wetlands are on the NWI map for the inundation area of this reservoir. However, examination of the site revealed a few small depressions containing wetlands. These all appeared to be irrigation induced and may not be jurisdictional. The total size of all of these wetlands is likely less than 0.25 acres. The canal from the dam to
the LeClair Canal at this site also traverses a large wetland that is fairly accurately mapped on the NWI map.

**Site 11.** The NWI map of this site does not show a wetland fringe along the drainage upstream from the dam site. The entire drainage upstream from the dam has a wetland fringe that averages 15 feet in width. Depending on the type of structure built here total wetland impacts will vary with the length of channel being inundated, and the total impact can be estimated by multiplying the length of channel inundated by 15 feet.

**Listed or other Sensitive Species**
According to the USFWS, federally listed species that may occur in the project area include bald eagle, black-footed ferret, and Ute’s ladies tresses. No records for any of these species are known for any of the reservoir sites. Because the entire project area is relatively developed, it is unlikely that bald eagles would nest or form winter roosts in the project area. No prairie dog towns were observed on any of the sites; therefore, there is no habitat for black-footed ferret. Ute’s ladies tresses occur in wetlands and all wetlands in the project areas could potentially provide habitat for this species. Therefore, once a final dam site(s) is determined, surveys may be required for this species. Surveys are typically conducted from late July through August.

Although not yet listed under the Endangered Species Act (ESA), the USFWS also expressed concern over two fish species that occur in the Wind River drainage. These two species (sauger and burbot) may be petitioned for listing under the ESA in the future. To reduce impacts to these species, as well as other sensitive fish species, the USFWS requests that diversion of water for the reservoir(s) should be limited to excess flow conditions. The USFWS letter is attached as Appendix A.

According to the WNDD, although several sensitive plant and animal species may potentially occur in the project area, there are no records of any sensitive species at any of the proposed reservoir sites. The nearest observation of a sensitive species in the project area is a yellow-billed cuckoo just south of the City of Riverton.

**Migratory Birds**
Two raptor nests were observed during the field reconnaissance within a half mile of potential reservoir construction activities. One of these nests is in a tree immediately adjacent to and on the west side of Site #7. This nest was not apparently active during the site visit. The other nest was an active red-tailed hawk nest in a tree approximately 0.25 miles east of the dam location for Site #8. Presence of active raptor nests within 0.5 miles of the project area may affect timing of construction so that potential impacts to nesting raptors are avoided.

**Big Game**
There is no crucial big game winter range or other important big game habitats on or near any of the reservoir sites. The project area is not considered to provide habitat for white-tailed deer or elk. It is considered yearlong range for both mule deer and pronghorn antelope.
6. ECONOMIC ANALYSES

6.1 Benefit Cost Analysis

6.1.1 Introduction

This section addresses the economic benefits and costs that would potentially accrue to local irrigators, the regional economy, and the State of Wyoming from enhancing water storage and delivery facilities for the LeClair and Riverton Valley Irrigation Districts. The two districts serve over 23,000 acres of irrigated land in central Fremont County and normally have adequate water supplies. Their predominant irrigated crop is alfalfa, mixed with smaller acreages of barley, corn, dry beans, grass hay and oats. Operators typically take two cuttings of alfalfa, one in late June or early July and the second in early August. They usually stop irrigating approximately two weeks prior to each cutting to allow their fields to dry out enough for harvesting. During this period, irrigation water supplies typically exceed demand and water is wasted. When irrigation resumes after each cutting, however, it can place enormous simultaneous demands upon the system and water shortages can result. Thus, the districts have a need to store water during periods of relative low demand and releasing it during periods of high demand.

Three potential project sites were identified that potentially could at least partially meet the needs of the two districts. They are:

- **Site 7A** – a reservoir site that would be supplied by a LeClair Canal diversion and a pipeline. Six different reservoir configurations were examined at this site, with differing levels of storage and associated costs. The two irrigation districts would share the costs and benefits of the reservoir and associated diversion. Assuming that a reservoir at this site could be filled twice during the irrigation season, between 636 and 2584 acre-feet of irrigation water could be released for use during times of need. Cost estimates for a reservoir at this site range from $4.37 to $9.39 million.

- **Site 8** – a 120 acre-foot reservoir site that would be supplied by an existing lateral from the LeClair Canal. We assumed that a small reservoir at this site could be filled two or three times per season – once early when irrigation demand is low, and once or twice in summer, depending upon the amount of runoff and the frequency of summer storms. The water could then be released back into the LeClair canal during periods of high demand. The estimated construction cost for the reservoir is $801,000.

- **Site 11** – this site would redirect runoff and irrigation water return flows from a small drainage into the Riverton Valley Canal by means of a small diversion facility. The site is located near the end of the canal, and the additional water it generates should be enough to serve the full irrigation needs of the approximately 150 to 200 acres of cropland located below it. The additional water produced at this site would free up water during times of need for diversion by irrigators located higher on the canal. The estimated cost of a diversion structure at this site is $122,000. A small reservoir at the
site would provide more flexibility regarding water releases and would increase project costs to $449,000.

Several other potential projects were evaluated during the course of the study and dropped from consideration for various reasons, including geotechnical considerations. The benefit-cost analysis focuses on the three sites, described above, that were deemed practical from engineering, geotechnical, and hydrologic perspectives.

6.1.2 Background

The direct economic benefits attributable to storing irrigation water for release when needed are measured by the sum of all farm income increases that would accrue to individual irrigators. To estimate these farm income increases, several variables must be known or estimated, including:

- The amount of additional water available and the efficiency with which it would be applied to crops;
- The crops that would benefit and their yield response to the application of additional water
- The market value of the additional yield that would be generated, and
- The marginal production costs that would be incurred in producing the additional yield.

When these variables are known or can be estimated, the additional farm income attributable to supplement water can be estimated as the value of additional crop production minus the marginal increase in production costs. Most of the data and information listed above is common to all three sites, and is discussed generically below. The amount of additional irrigation water to be made available is site specific and thus is discussed in the succeeding subsections dealing with the individual project sites.

Crops and Yield Response

Although several irrigated crops are grown in the area, alfalfa is predominant and would be the primary beneficiary of additional irrigation water. Smaller acreages of sugar beets and corn may also occasionally benefit from additional late season water. However, because alfalfa crops are expected to be the primary beneficiaries of additional water, the benefit estimates derived herein are based upon estimated alfalfa yield increases.

An estimate of the yield response of alfalfa to additional irrigation water was adapted from a study in eastern Wyoming.5 Because climatic conditions in the Riverton area are different than those in eastern Wyoming, the study results were adjusted to reflect differences in growing seasons between the two areas. For example, the eastern Wyoming study indicates that an

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additional acre-foot of water utilized by alfalfa in the form of ET will generate an additional 2.20 tons of alfalfa yield. The typical growing season in the Riverton area is 184 days, however, contrasted with 201 days in eastern Wyoming. Therefore, the 2.20-ton yield estimate was adjusted by the ratio 184/201 to reflect differences in climatic conditions. The resulting yield response estimate for the Riverton area is 2.01 tons of alfalfa per acre-foot of ET.

**Crop Prices**

Crop price estimates were developed from information published by the Wyoming Agricultural Statistics Service. Wyoming alfalfa prices were averaged over the last five years to avoid annual fluctuations attributable to drought and other factors. The resulting average alfalfa price is $92.20 per ton.

**Production Costs**

It also was necessary to estimate of the marginal increase in production costs that would be incurred to harvest the incremental yield. In the case of hay alfalfa production, these costs consist primarily of increased costs for irrigation, and harvesting activities. A marginal cost estimate for alfalfa production was taken from a production cost study for the Torrington area of eastern Wyoming, then updated to current dollars using production cost indices published by the Wyoming Agricultural Statistics Service. The resulting estimate is $21.44 per ton. There is a cost differential on the margin between putting up hay in big bales and small bales. The estimate of $21.44 assumes a 50/50 percent split between large and small bales among alfalfa hay producers in the Riverton area.

**Irrigation Efficiency**

Diversion records are available for the LeClair Canal for the period 1970 through 2005. These records indicate that the LeClair Irrigation District diverted an average of 5.45 acre-feet per acre annually. The consumptive irrigation requirement (CIR) for an average irrigated crop mix in Fremont County is 26.94 inches, or 2.25 acre-feet annually. If LeClair irrigators were meeting the full CIR requirements of an average crop mix, then average irrigation efficiency could be estimated as 2.25/5.45 = 41 percent. It is difficult to deliver the full CIR for a crop over an entire season, however, especially when water shortages occur, meaning that average irrigation efficiency is probably less than 41 percent for LeClair irrigators. Countering this fact

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8 Ibid.
9 Estimate provided by Dick Enberg, Apex Surveying.
10 Data provided by Dick Inberg, Apex Surveying.
12 An average cropping pattern was derived from irrigated crop statistics for Fremont County published by the Wyoming Agricultural Statistics Service.
is the probability that any additional water developed for release during times of high demand could be delivered with higher than average efficiency.

Efficiencies tend to be greater later in the irrigation season because earthen canal and lateral channels have been saturated and any system problems have likely been remedied. Taking all of these factors into account, we assumed an irrigation efficiency of 45 percent for additional water delivered to irrigators served by the LeClair Canal. This estimate applies to additional water diverted throughout the entire system. Some adjustments to this estimate are described in succeeding sections to account for the site-specific characteristics.

Average irrigation efficiencies for water diverted by Riverton Valley irrigators are estimated to be somewhat higher than for the LeClair irrigators because the Riverton Valley Canal predominantly uses piped laterals while the LeClair Canal does not. Overall irrigation efficiencies for Riverton Valley irrigators are estimated to be about 20 percent higher than for LeClair irrigators, or about 49 percent assuming full CIR. As with LeClair, however, efficiencies tend to be greater later in the irrigation season because earthen channels have been saturated and any system problems have likely been remedied. Taking all of these factors into account, we assumed an irrigation efficiency of 55 percent for additional water delivered to irrigators served by the Riverton Valley Canal.

7.1.3 Irrigation Benefits and Costs

Site 7A

Site 7A is located near the head of the Riverton Valley Canal. Water for this site would be diverted form the LeClair Canal and released into the Riverton Valley Canal in times of high demand. The two districts would share the benefits of this site through an exchange agreement. This site is located near the head of the Riverton Valley Canal and the overall efficiency with which water can be used from this site should be about the same as for typical canal diversions by both irrigation districts. Because the two districts would share the additional water from Site 7A, an average irrigation efficiency of \((45 + 55) / 2 = 50\) percent was used to estimated irrigation benefits for this site.

As mentioned above, a variety of reservoir configurations are available at this site, with different storage capacities and costs. The smallest configuration is 7A East, with a storage capacity of 318 acre-feet and an estimated construction cost of $4.37 million. If this reservoir were filled twice during the irrigation season, 636 acre-feet of water would be available for release during times of need. Of this amount, about \((636) \times (0.50) = 318\) acre-feet would be consumptively used by alfalfa, resulting in a total yield increase of \((318) \times (2.01) = 639.2\) tons annually. At $92.20 per ton, the increased yield would be valued at $58,900. Increased production costs for the alfalfa are estimated to be $21.44 per ton, so total cost increases are valued at \((639.2) \times (21.44) = 13,700\). Subtracting this amount from the gross income figure of $58,900 leaves a direct irrigation benefit estimate of $45,200.

That benefit estimate is shown in the second row of Table 6-1, along with corresponding estimates for the other project configurations at that site. For example, the third row of the table
shows that Site 7A east, with over excavation to increase storage capacity, would generate about $59,800 in annual direct benefits at a relatively small increase in costs. The last column of Table 6-1 presents benefit-cost ratios for the alternative project configurations. The results show that the project configuration with the highest ratio of benefits to cost is 7A West. With over excavation to increase storage capacity, its benefit cost ratio is 0.54. Without over excavation, the benefit cost ratio is slightly lower at 0.52. The other alternatives show in the table have benefit-cost ratios in the range of 0.27 to .50. The lowest ratios of benefits to costs are the two options associated with Site 7A East.

Table 6-1: Direct Irrigation Benefit and Cost Estimates for Site 7A.

<table>
<thead>
<tr>
<th>Configuration</th>
<th>Cost ($Millions)</th>
<th>One Fill (AF)</th>
<th>Releases (AF)</th>
<th>Crop ET (AF)</th>
<th>Annual Irrigation Benefit ($)</th>
<th>Benefit-Cost Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>7A East w/o Over Excavation</td>
<td>4.37</td>
<td>318</td>
<td>636</td>
<td>318</td>
<td>45,200</td>
<td>0.27</td>
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<td>4.84</td>
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<td>842</td>
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<td>59,800</td>
<td>0.32</td>
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<td>775</td>
<td>110,200</td>
<td>0.52</td>
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<td>5.85</td>
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<td>1,742</td>
<td>871</td>
<td>123,800</td>
<td>0.54</td>
</tr>
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<td>7A Combined w/o Over Excavation</td>
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<td>1093</td>
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</tr>
<tr>
<td>7A Combined w Over Excavation</td>
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<td>1292</td>
<td>2584</td>
<td>1292</td>
<td>183,600</td>
<td>0.50</td>
</tr>
</tbody>
</table>

Site 8

This 120 acre-foot storage site is located along the LeClair Canal about two thirds of the way down from its diversion point. It could be filled two or three times during the irrigation season, depending upon the amount of runoff and the frequency of summer storm events. Depending upon the year, between 240 and 360 acre-feet of water could be released for downstream use in periods of high demand. The reservoir would be financed by and used to benefit LeClair Irrigation District members. Because this site is located so far down the canal, overall irrigation efficiencies for release from this site should be higher than the average for LeClair irrigators. We assumed that overall irrigation efficiencies from this site would average 55 percent, or the same as for Riverton Valley irrigators. If the reservoir fills twice, approximately \((240 \times 0.55) = 132\) acre-feet of additional water would be consumptively used by crops. If the reservoir fills three times, this figure increases to \((360 \times 0.55) = 198\) acre-feet of additional consumptive use.

\(^{13}\) Benefit-cost ratios were computed by finding the present value of the annual benefit stream for each alternative using a 50-year time horizon and a 3 percent real discount rate.
Multiplying 2.01 tons of alfalfa by 132 acre-foot of ET and an average price of $92.20 gives an annual gross irrigation benefit of $24,500. Increased production costs are estimated to be $(2.01 \times 132 \times $21.44) = $5,700. Thus, net annual irrigation benefits for two fills at Site 8 are estimated to be roughly $18,800 annually. If the reservoir fills three times, annual irrigation benefits would increase to $28,200. For discussion purposes, we will assume a long-term average fill rate of 2.5 times per season, and an average annual irrigation benefit of $23,000. The present value of this stream of benefits over a 50-year time frame using a 3 percent discount rate is $591,800. Dividing this number by an estimated construction cost of $801,000 yields a benefit cost ratio of 0.74.

Site 11

Site 11 is located near the end of the Riverton Valley Canal, and consists of a small diversion works with or without an associated small storage reservoir. This site could supply the full irrigation water needs of the 150 to 200 acres of cropland located below it on the canal. The additional water produced by this site would free up approximately 655 acre-feet of water for diversion by irrigators located higher up on the canal.

Some of this water would be made available for diversion when needed in times of high demand. Other times it would be available when excess water is in the system, and thus be wasted unless new storage is available on the system to capture it. Thus, the potential benefit of Site 11 depend partially on whether it is a stand alone project or combined with other storage. For example, Riverton Valley ID could allow additional water to be diverted into the LeClair canal for storage at Site 7 if it is not needed downstream of Site 11. In this case, the potential benefits of the Site 11 project would be to make up to 655 acre-feet of water available (minus conveyance losses).

On the other hand, without additional storage, the benefit would be limited to times when the canal upstream of Site 11 is short of water. This situation generally occurs two to three times per season for a few days at a time. For estimation purposes, it was assumed that Site 11 could provide beneficial water for 10 irrigation days per season at flow rate averaging 5 cfs, which would be the equivalent of approximately 100 acre-feet of additional water annually. Assuming an irrigation efficiency of 55 percent, 55 acre-feet would be consumptively used by crops in the form of ET. At 2.01 tons of alfalfa per acre-foot of ET, the increase in yield would be 110.6 tons valued at $92.20 for a gross benefit of $10,200 annually. Increased production costs would be about $(110.6 \times $21.44) = $2,400, resulting in a net benefit of $10,200 – 2,400 = $7,800 annually.

The present value of this benefit discounted over a 50-year time frame with a 3 percent real discount rate is $200,700. If this benefit could be achieved with the diversion structure alone at a cost of $122,000, the direct benefit-cost ratio would be 1.65. If the small reservoir is incorporated into the project for a total cost of $449,000, the benefit-cost ratio drops to 0.45.

6.1.3 Indirect Irrigation Benefits

Indirect benefits, sometimes referred to as secondary benefits, stem from the economic multiplier effect of increases in income in a regional economy. For example, if irrigators’ incomes increase
because of a new irrigation project, some of that income will be spent locally, resulting in additional income increases in other sectors of the Wyoming economy. Thus, the total economic benefits associated with an irrigation project can be larger than direct income increases to irrigators alone.

The Bureau of Economic Analysis of the U.S. Department of Commerce (USDOC) produces periodic estimates of indirect income multipliers for Wyoming’s agricultural sector.\textsuperscript{14} Their latest published estimate of this multiplier is 3.36; meaning that for each dollar of additional farm income, total income in Wyoming increases by $3.36. That number was published in 1992 and may be somewhat out of date. Other information sources indicate that it is rare for an income multiplier in rural areas of the west to exceed 2.50 in magnitude. As a conservative estimate, a farm income multiplier of 2.50 was used to estimate indirect benefits for this report. Applying this multiplier to the estimates in Table 6-1 results in the following total benefit cost estimates for each project site and configuration.

The results in Table 6-2 indicate that when indirect benefits are taken into account, all of the projects except the Site 7A East configurations have estimated benefit-cost ratios in excess of one. Of the larger reservoir alternatives at Site 7A, the largest benefit-cost ratios are associated with Site 7A West, which shows a higher ratio of benefits to cost than the combined sites, and significantly higher ratio than Site 7A East.

\textbf{Table 6-2: Total Benefit and Cost Estimates for Project Alternatives}

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<tr>
<th>Configuration</th>
<th>Cost ($Millions)</th>
<th>One Fill (AF)</th>
<th>Releases (AF)</th>
<th>Crop ET (AF)</th>
<th>Annual Total Benefits ($)</th>
<th>Total Benefit-Cost Ratio</th>
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<tr>
<td>7A East w/o Over Excavation</td>
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<td>636</td>
<td>318</td>
<td>113,000</td>
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<td>7A East w Over Excavation</td>
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<td>7A West w Over Excavation</td>
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<td>Site 8</td>
<td>0.80</td>
<td>120</td>
<td>360</td>
<td>198</td>
<td>57,500</td>
<td>1.85</td>
</tr>
<tr>
<td>Site 11 Diversion</td>
<td>0.12</td>
<td>NA</td>
<td>100</td>
<td>55</td>
<td>19,500</td>
<td>4.11</td>
</tr>
<tr>
<td>Site 11 w Reservoir</td>
<td>0.45</td>
<td>12</td>
<td>100</td>
<td>55</td>
<td>19,500</td>
<td>1.12</td>
</tr>
</tbody>
</table>

6.1.5 Other Effects

Some of the sites described above have existing agricultural uses that would be disrupted if a reservoir and or diversion were constructed. Site 7A, for example, has about 8 acres of hay and 28 acres of pasture that could be inundated by some project configurations. Also, Site 8 would inundate about 3.4 acres of marginal pasture. The value of this lost agricultural production is taken into account in this analysis because it is included in land acquisition costs for each site. Because all the sites are located on private land, no public recreation activities are likely to be affected. Some disruption of wildlife grazing activities may occur, especially at Site 7A. On the other hand, this disruption may be offset by the creation of additional waterfowl habitat. These and other potential project effects would require further attention if project planning proceeds on one or more sites.

6.2 Ability to Pay

6.2.1 Introduction

This subsection considers potential funding scenarios for the project alternatives based upon the assumption of WWDC assistance and a local project sponsor. Other potential funding sources might be identified if detailed planning proceeds with one or more project alternative. The financing analysis considers three elements:

- The ability-to-pay of local irrigators;
- The minimum cost of water to irrigators under current WWDC guidelines, and
- The cost sharing arrangement that would be necessary to keep water payments within the ability-to-pay of local irrigators.

A sponsor’s ability to pay for irrigation water is bounded by the magnitude of direct irrigation benefits that would be generated by the additional water. For example, the estimates in Table 6-1 indicate that the direct benefits of additional storage to farmers from Site 7A range from approximately $45,200 to $183,600 annually, depending upon the project configuration. This benefit estimate reflects the market value of additional crop production minus the costs associated with producing the additional crop, but before making any additional payments for water.

This figure represents an upper bound on ability to pay because it estimates the additional farm income that could be generated from additional storage. If irrigators were required to forfeit all of the additional income to repay project expenditures, they would have little incentive to participate in the project. Thus, ability-to-pay studies assume that ability to pay is some reasonable fraction of direct project benefits.

Many of the benefit estimates in this section of the report are based upon professional judgments concerning the number of times reservoirs could be filled and water released during times of need. Without detailed hydrologic studies backing up these judgments, ability to pay estimates should necessarily be conservative to avoid over commitment on the part of local sponsors. For purposes of this analysis, two ability to pay scenarios were developed to bracket the ability of
local sponsors to pay for project components. The first scenario assumes that ability to pay is 30 percent of the estimated additional income that could be generated from the projects, while the second scenario assumes that ability to pay is 50 percent of the farm income increase. These two scenarios are discussed further below.

6.2.2 Financing Under WWDC Guidelines

Current WWDC guidelines provide for funding new storage and rehabilitation projects with a 67 percent project grant with the remaining 33 percent of project costs to be repaid by the project sponsor. Repayment can be financed over 50 years at four percent interest. The implications of applying these funding criteria to the project alternatives can be seen in the results in Table 6-3. That table shows the cost of each alternative (column 2), the project sponsor’s share of those costs at 33 percent (column 3), and the sponsor’s annual repayment amount assuming a 50-year loan at four percent interest (column 4).

<table>
<thead>
<tr>
<th>Project/Configuration</th>
<th>Project Cost ($Millions)</th>
<th>Sponsor’s Cost ($Millions)</th>
<th>Sponsor’s Annual Payment ($)</th>
<th>Sponsor’s Annual Ability to Pay ($)</th>
<th>Sponsor’s Percentage Ability to Pay (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7A East w/o Over Excavation</td>
<td>4.37</td>
<td>1.44</td>
<td>67,200</td>
<td>13,600</td>
<td>6.7</td>
</tr>
<tr>
<td>7A East w Over Excavation</td>
<td>4.84</td>
<td>1.60</td>
<td>74,500</td>
<td>17,900</td>
<td>7.9</td>
</tr>
<tr>
<td>7A West w/o Over Excavation</td>
<td>5.41</td>
<td>1.79</td>
<td>83,300</td>
<td>33,100</td>
<td>13.1</td>
</tr>
<tr>
<td>7A West w Over Excavation</td>
<td>5.85</td>
<td>1.93</td>
<td>89,900</td>
<td>37,100</td>
<td>13.6</td>
</tr>
<tr>
<td>7A Comb w/o Over Excavation</td>
<td>8.48</td>
<td>2.80</td>
<td>130,300</td>
<td>46,600</td>
<td>11.8</td>
</tr>
<tr>
<td>7A Comb w Over Excavation</td>
<td>9.39</td>
<td>3.10</td>
<td>144,400</td>
<td>55,100</td>
<td>12.6</td>
</tr>
<tr>
<td>Site 8</td>
<td>0.80</td>
<td>0.26</td>
<td>12,300</td>
<td>6,900</td>
<td>18.5</td>
</tr>
<tr>
<td>Site 11 Diversion</td>
<td>0.12</td>
<td>0.40</td>
<td>1,900</td>
<td>2,400</td>
<td>43.0</td>
</tr>
<tr>
<td>Site 11 w Reservoir</td>
<td>0.45</td>
<td>0.15</td>
<td>6,900</td>
<td>2,400</td>
<td>11.5</td>
</tr>
</tbody>
</table>

The remaining two columns of the table show the maximum amount irrigators could repay each year based upon an ability-to-pay rate of 30 percent of direct project benefits. Those amounts vary from a low of $2,400 per year for Site 11 to a high of $55,100 per year for Site 7A with over excavation. With the exception of the Site 11 diversion, these amounts are less than would be required to fund any of the projects under current WWDC guidelines.
The last column of Table 6-3 presents ability to pay as a percentage of total project costs, again assuming 50 year financing at four percent interest. A sponsor’s estimated ability to pay for the projects ranges from a high of 43.0 percent for the Site 11 diversion to a low of 6.7 percent for Site 7A East w/o over excavation. These results indicate a limited ability for project sponsors to repay estimated project costs without state assistance in the form of a higher than average grant or a state sponsored project.

Table 6-4 shows ability to pay estimates and financing scenarios based upon an assumed ability to pay of 50 percent of direct project benefits.

**Table 6-4: Sponsor’s Ability to Pay Summary (50 percent of direct benefits)**

<table>
<thead>
<tr>
<th>Project/Configuration</th>
<th>Project Cost ($Millions)</th>
<th>Sponsor’s Cost ($Millions)</th>
<th>Sponsor’s Annual Payment ($)</th>
<th>Sponsor’s Annual Ability to Pay ($)</th>
<th>Sponsor’s Percentage Ability to Pay (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7A East w/o Over Excavation</td>
<td>4.37</td>
<td>1.44</td>
<td>67,200</td>
<td>22,600</td>
<td>11.2</td>
</tr>
<tr>
<td>7A East w Over Excavation</td>
<td>4.84</td>
<td>1.60</td>
<td>74,500</td>
<td>29,900</td>
<td>13.2</td>
</tr>
<tr>
<td>7A West w/o Over Excavation</td>
<td>5.41</td>
<td>1.79</td>
<td>83,300</td>
<td>55,100</td>
<td>21.8</td>
</tr>
<tr>
<td>7A West w Over Excavation</td>
<td>5.85</td>
<td>1.92</td>
<td>89,900</td>
<td>61,900</td>
<td>22.7</td>
</tr>
<tr>
<td>7A Comb w/o Over Excavation</td>
<td>8.48</td>
<td>2.79</td>
<td>130,300</td>
<td>77,700</td>
<td>19.7</td>
</tr>
<tr>
<td>7A Comb w Over Excavation</td>
<td>9.39</td>
<td>3.09</td>
<td>144,400</td>
<td>91,800</td>
<td>21.0</td>
</tr>
<tr>
<td>Site 8</td>
<td>0.80</td>
<td>0.26</td>
<td>12,300</td>
<td>11,500</td>
<td>30.8</td>
</tr>
<tr>
<td>Site 11 Diversion</td>
<td>0.12</td>
<td>0.35</td>
<td>1,900</td>
<td>3,900</td>
<td>71.1</td>
</tr>
<tr>
<td>Site 11 w Reservoir</td>
<td>0.45</td>
<td>0.15</td>
<td>6,900</td>
<td>3,900</td>
<td>19.2</td>
</tr>
</tbody>
</table>

The results show that assuming ability to pay is 50 percent of estimated benefits increases the sponsor’s percentage ability to pay across the board for all alternatives. Unfortunately, however, only the Site 11 diversion meets the criteria of repaying 33 percent of project costs over a 50-year time horizon at 4 percent interest.
7. SUMMARY

This Level II Study conducted for the LeClair and Riverton Valley Irrigation Districts under the direction and funding of the Wyoming Water Development Commission develops conceptual designs, cost estimates, operational plans, economic analyses, and recommendations to assist with the determination of the feasibility of the addition of irrigation storage reservoirs to the LeClair and Riverton Valley irrigation systems.

This study evaluated the need for additional storage, identified twelve potential reservoir storage locations, selected the preferred alternative sites, conducted thorough investigations of the preferred alternatives, and present detailed alternatives to the two irrigation districts. Reservoir Sites 7A, 7B, 8, 11, 4, and 2 were determined to be the preferred alternative reservoir sites. Through thorough investigation of these potential reservoir sites, Sites 7B, 4 and 2 were dropped from the list of preferred alternatives for the reasons previously discussed. The storage volume in Site 7B would be limited by the presence of a bedrock knob resulting in an inefficient site. The bedrock knob was determined to be costly to remove and economical construction of this site would not be feasible. Sites 4 and 2 have several geotechnical concerns and it was determined that the economical construction of a safe dam at these sites would not be feasible.

Site 7A

This potential reservoir site would be located between the LeClair and Riverton Valley canal and could serve both canals through a water exchange between the two irrigation districts. This reservoir site could store water in the spring and when irrigation demand is less than irrigation supply as long as the Wind River is not in regulation. Water could be released during periods of high demand on the canal systems. It is anticipated that this site could be filled twice per season. Costs associated with this site would be split up between both irrigation districts based on acres served. Six storage volume scenarios and the resulting costs were analyzed for this potential reservoir site. The six storage configurations include 7A East, 7A West, and 7A East and West combined each with and without over excavation. Each storage configuration was evaluated with and without over excavation of the sandy silty material in the pool area to increase storage volume in the reservoir. Over excavation and spoil appears to be a cost effective method of increasing storage volume. These site configurations appear to be technically feasible, however, economically they are probably not feasible with the current funding package and economic environment.

Site 8

This potential reservoir site would be located above the LeClair canal near the mid point of the canal system. This site would serve the LeClair canal only. This site could help to regulate flows in the canal and reduce waste by storing water any time irrigation demand is less than irrigation supply assuming the Wind River is not in regulation. It is anticipated that this site could be filled two or three times per season depending on the amount of runoff, frequency of summer rainfall, and system operation. This site could store approximately 120 ac-ft of water per fill allowing this site to yield approximately 240 to 360 ac-ft per season. This site appears to be technical feasibility, however, economically it is probably not feasible with the current funding package and economic environment.
Site 11

Site 11 is located near the end of the Riverton Valley canal. This site would serve the Riverton Valley canal only. The source of water at this site is the flow in the drainage. Two options were developed at this site: 1) a small reservoir that could store approximately 12 ac-ft, and 2) a diversion structure to divert the flows in the drainage into the Riverton Valley canal. There appears to be sufficient flow in the drainage to supply the irrigated lands downstream from this site on the Riverton Valley canal. The storage option at this site is technically questionable and financially not feasible, however, economic and technical feasibility of the diversion structure appears to be favorable.

A summary of costs for these projects is presented in Table 7.1. The costs include construction costs, construction engineering, preparation of final design plans and specifications, permitting and mitigation, legal fees and acquisition of access and rights-of-way. This table also presents the annual repayment required by the two irrigation districts based on the WWDC funding of 67% grant and 33% loan at 4% interest for 50 years.

<table>
<thead>
<tr>
<th>Site</th>
<th>Project Cost</th>
<th>Storage (ac-ft)</th>
<th>Annual Repayment</th>
</tr>
</thead>
<tbody>
<tr>
<td>7A East w/out Over Excavation</td>
<td>$4,373,000</td>
<td>318</td>
<td>$67,200</td>
</tr>
<tr>
<td>7A East w/ Over Excavation</td>
<td>$4,843,000</td>
<td>421</td>
<td>$74,500</td>
</tr>
<tr>
<td>7A West w/out Over Excavation</td>
<td>$5,414,000</td>
<td>775</td>
<td>$83,300</td>
</tr>
<tr>
<td>7A West w/ Over Excavation</td>
<td>$5,849,000</td>
<td>871</td>
<td>$89,900</td>
</tr>
<tr>
<td>7A Combined w/out Over Excavation</td>
<td>$8,475,000</td>
<td>1093</td>
<td>$130,300</td>
</tr>
<tr>
<td>7A Combined w/ Over Excavation</td>
<td>$9,387,000</td>
<td>1292</td>
<td>$144,400</td>
</tr>
<tr>
<td>Site 8</td>
<td>$801,000</td>
<td>125</td>
<td>$12,300</td>
</tr>
<tr>
<td>Site 11 Diversion Option</td>
<td>$122,000</td>
<td>-</td>
<td>$1,900</td>
</tr>
<tr>
<td>Site 11 Storage Option</td>
<td>$449,000</td>
<td>12</td>
<td>$6,900</td>
</tr>
</tbody>
</table>

Table 7.1 – Summary of Construction Costs
APPENDIX A.

Letter from U.S. Fish and Wildlife Service regarding LeClair/Riverton Project
Victoria Poulton
Western Ecosystem Technology, Incorporated
2003 Central Avenue
Cheyenne, Wyoming 82001

Dear Ms. Poulton:

Thank you for your letter of March 16, 2006, received by our office on March 21, requesting a species list for the LeClair/Riverton Valley Irrigation Storage project near Riverton, in Fremont County, Wyoming. Several impoundments are proposed along the LeClair Irrigation Canal, which is supplied by the Wind River. The project will be located in sections 25, 26 and 36, T1N, R3E, section 20 T1N, R4E, and section 28 T2N, R5E.

The U.S. Fish and Wildlife Service (Service) is providing you with comments on (1) threatened, endangered and candidate species, (2) migratory birds, and (3) wetlands and riparian areas. The Service provides recommendations for protective measures for threatened and endangered species in accordance with the Endangered Species Act (Act) of 1973, as amended (16 U.S.C. 1531 et seq.). Protective measures for migratory birds are provided in accordance with the Migratory Bird Treaty Act (MBTA), 16 U.S.C. 703 and the Bald and Golden Eagle Protection Act (BGEPA), 16 U.S.C. 668. Wetlands are afforded protection under Executive Orders 11990 (wetland protection) and 11988 (floodplain management), as well as section 404 of the Clean Water Act. Other fish and wildlife resources are considered under the Fish and Wildlife Coordination Act and the Fish and Wildlife Act of 1956, as amended, 70 Stat. 1119, 16 U.S.C. 742a-742j.

In accordance with section 7 of the Act, my staff has determined that the following threatened or endangered species, or species proposed for listing under the Act, may be present in or near the project area. We would appreciate receiving information as to the current status of each of these species within the project area.

<table>
<thead>
<tr>
<th>SPECIES</th>
<th>STATUS</th>
<th>HABITAT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bald eagle</td>
<td>Threatened</td>
<td>Found throughout state</td>
</tr>
<tr>
<td>(Haliaeetus leucocephalus)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
**Black-footed ferret**
(*Mustela nigripes*)

**Endangered**

**Prairie dog towns**

**Ute ladies’-tresses**
(*Spiranthes diluvialis*)

**Threatened**

**Seasonally moist soils and wet meadows of drainages below 7000 feet elevation.**

**Bald eagle:** While habitat loss and human disturbance remains a threat to the bald eagle's full recovery, most experts agree that its recovery to date is encouraging. Adult eagles establish lifelong pair bonds and build large nests in the tops of large trees near rivers, lakes, marshes, or other wetland areas. During winter, bald eagles gather along open water to forage and night roost in large mature trees, usually in secluded locations that offer protection from harsh weather. Bald eagles often return to use the same nest and winter roost year after year. Because bald eagles are particularly sensitive to human disturbance at their nests and communal roosts, protective buffers should be implemented around these areas [Buehler et al. 1991, Greater Yellowstone Bald Eagle Working Group (GYBEWG) 1996, Montana Bald Eagle Working Group (MBEWG) 1994, Stalmaster and Newman 1978, U.S. Fish and Wildlife Service (USFWS) 1986].

In Wyoming, bald eagle nest buffer recommendations include avoiding project-related disturbance and habitat alteration within 1 mile of bald eagle. The nesting season occurs from February 1 to August 15 and bald eagle nest buffers should receive maximum protection during this time period. For some activities (construction, seismic exploration, blasting, and timber harvest), a home range buffer may include potential foraging habitat for 2.5 miles from the nest (GYBEWG 1996). We recommend that you contact the U.S. Fish and Wildlife Service to determine the potential impact of your activity to nesting bald eagles if your project will cause disturbance within one of these nest buffer areas.

A communal roost is defined as an area where six or more eagles spend the night within 100 meters (328 feet) of each other (GYBEWG 1996). For bald eagle communal winter roosts, we recommend that disturbance be restricted within 1 mile of known communal winter roosts during the period of November 1 to April 1. Additionally, we recommend avoiding disturbance and habitat alteration within 0.5 mile of active roost sites year round.

Disturbance sensitivity of roosting and nesting bald eagles may vary between individual eagles, topography, density of vegetation and intensity of activities. The buffers and timing stipulations, as described above, should be implemented unless site-specific information indicates otherwise (Stalmaster and Newman 1978, USFWS 1986). Modification of buffer sizes may be permitted where biologically supported and in coordination with the U.S. Fish and Wildlife Service.

**Black-footed ferret:** Black-footed ferrets may be affected if prairie dog towns are impacted. Please be aware that black-footed ferret surveys are no longer recommended in black-tailed prairie dog towns statewide or white-tailed prairie dog towns except those noted in our enclosed February 2, 2004, letter. However, we encourage the federal agency to protect all prairie dog towns for their value to the prairie ecosystem and the myriad of species that rely on them. We further encourage you to analyze potentially disturbed prairie dog towns for their value to future black-footed ferret reintroduction.
If white-tailed prairie dog towns or complexes greater than 200 acres will be disturbed, surveys for ferrets may be recommended in order to determine if the action will result in an adverse effect to the species. Surveys are recommended even if only a portion of the white-tailed prairie dog town or complex, as identified in our enclosed letter, will be disturbed. According to the Black-Footed Ferret Survey Guidelines (USFWS 1989), a prairie dog complex consists of two or more neighboring prairie dog towns less than 7 km (4.3 miles) from each other. If a field check indicates that prairie dog towns may be affected, you should contact this office for guidance on ferret surveys.

**Ute ladies'-tresses**

Ute ladies'-tresses (*Spiranthes diluvialis*) is a perennial, terrestrial orchid, 8 to 20 inches tall, with white or ivory flowers clustered into a spike arrangement at the top of the stem. *S. diluvialis* typically blooms from late July through August, however, depending on location and climatic conditions, it may bloom in early July or still be in flower as late as early October. *S. diluvialis* is endemic to moist soils near wetland meadows, springs, lakes, and perennial streams where it colonizes early successional point bars or sandy edges. The elevation range of known occurrences is 4,200 to 7,000 feet in alluvial substrates along riparian edges, gravel bars, old oxbows, and moist to wet meadows. Soils where *S. diluvialis* have been found typically range from fine silt/sand, to gravels and cobbles, as well as to highly organic and peaty soil types. *S. diluvialis* is not found in heavy or tight clay soils or in extremely saline or alkaline soils. *S. diluvialis* seems intolerant of shade and small scattered groups are found primarily in areas where vegetation is relatively open. Surveys should be conducted by knowledgeable botanists trained in conducting rare plant surveys. *S. diluvialis* is difficult to survey for primarily due to its unpredictability of emergence of flowering parts and subsequent rapid desiccation of specimens. The Service does not maintain a list of "qualified" surveyors but can refer those wishing to become familiar with the orchid to experts who can provide training or services.

**Migratory Birds**

Please recognize that consultation on listed species may not remove your obligation to protect the many species of migratory birds, including eagles and other raptors protected under the MBTA and BGEPA. The MBTA, enacted in 1918, prohibits the taking of any migratory birds, their parts, nests, or eggs except as permitted by regulations and does not require intent to be proven. Section 703 of the MBTA states, "Unless and except as permitted by regulations ... it shall be unlawful at any time, by any means or in any manner, to ... take, capture, kill, attempt to take, capture, or kill, or possess ... any migratory bird, any part, nest, or eggs of any such bird..." The BGEPA, prohibits knowingly taking, or taking with wanton disregard for the consequences of an activity, any bald or golden eagles or their body parts, nests, or eggs, which includes collection, molestation, disturbance, or killing.

In order to promote the conservation of migratory bird populations and their habitats, the Service recommends the Federal Highway Administration implement those strategies outlined within the Memorandum of Understanding directed by the President of the U.S. under the Executive Order 13186, where possible.

**Wetlands/Riparian Areas**

Wetlands may be impacted by the proposed project. Wetlands perform significant ecological functions which include: (1) providing habitat for numerous aquatic and terrestrial wildlife species, (2) aiding in the dispersal of floods, (3) improving water quality through retention and
assimilation of pollutants from storm water runoff, and (4) recharging the aquifer. Wetlands also possess aesthetic and recreational values. The Service recommends measures be taken to avoid and minimize wetland losses in accordance with Section 404 of the Clean Water Act, and Executive Order 11988 (floodplain management) as well as the goal of "no net loss of wetlands." If wetlands may be destroyed or degraded by the proposed action, those wetlands in the project area should be inventoried and fully described in terms of their functions and values. Acreage of wetlands, by type, should be disclosed and specific actions should be outlined to avoid, minimize, and compensate for all unavoidable wetland impacts.

Riparian or streamside areas are a valuable natural resource and impacts to these areas should be avoided whenever possible. Riparian areas are the single most productive wildlife habitat type in North America. They support a greater variety of wildlife than any other habitat. Riparian vegetation plays an important role in protecting streams, reducing erosion and sedimentation as well as improving water quality, maintaining the water table, controlling flooding, and providing shade and cover. In view of their importance and relative scarcity, impacts to riparian areas should be avoided. Any potential, unavoidable encroachment into these areas should be further avoided and minimized. Unavoidable impacts to streams should be assessed in terms of their functions and values, linear feet and vegetation type lost, potential effects on wildlife, and potential effects on bank stability and water quality. Measures to compensate for unavoidable losses of riparian areas should be developed and implemented as part of the project.

Plans for mitigating unavoidable impacts to wetland and riparian areas should include mitigation goals and objectives, methodologies, time frames for implementation, success criteria, and monitoring to determine if the mitigation is successful. The mitigation plan should also include a contingency plan to be implemented should the mitigation not be successful. In addition, wetland restoration, creation, enhancement, and/or preservation does not compensate for loss of stream habitat; streams and wetlands have different functions and provide different habitat values for fish and wildlife resources.

Best Management Practices (BMPs) should be implemented within the project area wherever possible. BMPs include, but are not limited to, the following: installation of sediment and erosion control devices (e.g., silt fences, hay bales, temporary sediment control basins, erosion control matting); adequate and continued maintenance of sediment and erosion control devices to insure their effectiveness; minimization of the construction disturbance area to further avoid streams, wetlands, and riparian areas; location of equipment staging, fueling, and maintenance areas outside of wetlands, streams, riparian areas, and floodplains; and re-seeding and re-planting of riparian vegetation native to Wyoming in order to stabilize shorelines and streambanks.

**Fisheries**

*Sauger* (*Sander canadense*) are native to the Wind-Big Horn, Tongue, Powder, and North Platte River Drainages in Wyoming. They are still present in the Wind-Big Horn River Drainage, but are now rare or extirpated in the other drainages in Wyoming. Additionally, they have been eliminated from much of their native range by habitat alteration, primarily changes associated with water development in the western portion of their range. Resource managers have a trust responsibility to Wind River Reservation Tribes to preserve native fish populations and their habitat. Furthermore, the Wyoming Game and Fish Department list sauger as a Wyoming "Species of Concern" and there is interest among environmentalists in petitioning the FWS to list.
sauger as threatened under the Endangered Species Act. Three recent, University of Wyoming graduate studies have defined the Wind River sauger as a low density, genetically pure population. Sauger have declined in abundance or become hybridized with walleye through much of their native range so it is imperative that habitat alterations in the Wind River drainage be done with caution to prevent further losses to the current population. It is therefore recommended that diversion of water (mainstem water and irrigation return flows) for the proposed ponds be limited to excess flow conditions.

Burbot
Burbot (Lota lota), frequently called ling, are native to river systems in Wyoming that flow north to the Yellowstone River in the upper Missouri River drainage, including the Bighorn-Wind, Tongue, and Powder River systems, and are rare in the Tongue and Powder River systems. Fisheries for burbot persist in the Bighorn-Wind River system, where they are prized as a food fish, particularly among the Shoshone people in the Wind River watershed. In some areas, including the Wind River watershed, burbot populations have declined and are threatened with localized extinction. For example, burbot have declined in the Kootenay Lake and Kootenay River system to the point that they were petitioned for listing as endangered under the Endangered Species Act. As with sauger, diversion of water for the proposed ponds should be limited to excess flow conditions.

Other Fish Species of Concern
Several other native fish species which have shown significant declines in the past years due to water regime alterations and losses through the irrigation systems include flathead chub (Platygobio gracilis) and various minnow (Cyprinidia sp.) species.

Thank you for your efforts to ensure the conservation of threatened, endangered and sensitive species in Wyoming. If you have any questions regarding this letter or your responsibilities under the Act, please contact Pat Deibert at letterhead address or by calling (307) 772-2374, ext. 26.

Sincerely,

[Signature]
Brian T. Kelly
Field Supervisor
Wyoming Field Office

Enclosures (2)

cc: (w/o enclosures)
WGFD, Statewide Habitat Protection Coordinator, Cheyenne, WY (V. Stelter)
WGFD, Non-Game Coordinator, Lander, WY (B. Oakleaf)
FWS, Management Assistance Office, Lander, WY (S. Roth)
References


APPENDIX B.

Geotechnical Design Analyses

1) Site 7A Filter/Drain Seepage Analyses
2) Riprap Design Sites 7A & 8
3) Site 11 Sheet Pile Seepage Analyses
Site 7 West Dam
3 foot thick blanket drain

Description:
Comments:
File Name: grid 1.sez
Last Saved Date: 5/26/2006
Last Saved Time: 10:39:14 AM
Analysis Type: Steady-State
Analysis View: 2-D

$K = 2 \times 10^{-2}$ cm/sec
$K_h = 3 \times 10^{-5}$ cm/sec
$K_v = 3 \times 10^{-6}$ cm/sec
$K = 1$ cm/sec
Site 7 West Dam
5 foot blanket drain

Description:

Comments:

File Name: grid 2.sez
Last Saved Date: 5/26/2006
Last Saved Time: 10:42:59 AM
Analysis Type: Steady-State
Analysis View: 2-D

Kh = 3E-05 cm/sec
Kv = 3E-06 cm/sec
K = 2E-02 cm/sec
K = 1 cm/sec
Site 7 West Dam
3 foot thick blanket drain

Description:
Comments:
File Name: grid 3.sez
Last Saved Date: 5/26/2006
Last Saved Time: 11:16:21 AM
Analysis Type: Steady-State
Analysis View: 2-D

$K = 3 \times 10^{-1} \text{ cm/sec}$

$K_{h} = 3 \times 10^{-5} \text{ cm/sec}$
$K_{v} = 3 \times 10^{-6} \text{ cm/sec}$

$K = 1 \text{ cm/sec}$
Riprap Design by NRCS Methodology.

NOAA Data, Normals, Means and Extremes for Latitude 42N Longitude 106 W (Riverton, WY)

**SITES 7A & 8**

**Fastest Mile Wind Data**

<table>
<thead>
<tr>
<th>Month</th>
<th>Direction (degrees)</th>
<th>Speed (mph)</th>
<th>V * Dir</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jan</td>
<td>200</td>
<td>58</td>
<td>11600</td>
</tr>
<tr>
<td>Feb</td>
<td>230</td>
<td>58</td>
<td>13440</td>
</tr>
<tr>
<td>Mar</td>
<td>250</td>
<td>81</td>
<td>20250</td>
</tr>
<tr>
<td>Apr</td>
<td>250</td>
<td>54</td>
<td>13500</td>
</tr>
<tr>
<td>May</td>
<td>320</td>
<td>58</td>
<td>18560</td>
</tr>
<tr>
<td>Jun</td>
<td>360</td>
<td>52</td>
<td>18720</td>
</tr>
<tr>
<td>Jul</td>
<td>250</td>
<td>52</td>
<td>13000</td>
</tr>
<tr>
<td>Aug</td>
<td>250</td>
<td>50</td>
<td>12500</td>
</tr>
<tr>
<td>Sept</td>
<td>320</td>
<td>53</td>
<td>16960</td>
</tr>
<tr>
<td>Oct</td>
<td>250</td>
<td>55</td>
<td>13750</td>
</tr>
<tr>
<td>Nov</td>
<td>250</td>
<td>49</td>
<td>12250</td>
</tr>
<tr>
<td>Dec</td>
<td>200</td>
<td>63</td>
<td>12600</td>
</tr>
</tbody>
</table>

**Overland Wind Velocity**

<table>
<thead>
<tr>
<th>V (mph)</th>
<th>ave. from climate data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Method 1</td>
<td>57</td>
</tr>
<tr>
<td>Method 2</td>
<td>80 (Fig. 4 (SCS 1983))</td>
</tr>
<tr>
<td>Method 3</td>
<td>59</td>
</tr>
</tbody>
</table>

Max V of record = 81

Use Design V = 70 mph

**Effective Fetch Site 7A**

*Method 2 - Prevailing Wind is away from dam face*

<table>
<thead>
<tr>
<th>Radial (a)</th>
<th>$\cos^2 a$</th>
<th>Distance (feet)</th>
<th>Distance (miles)</th>
<th>$D \times \cos^2 a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>42</td>
<td>0.552264</td>
<td>1350</td>
<td>0.256</td>
<td>0.1412039</td>
</tr>
<tr>
<td>36</td>
<td>0.654508</td>
<td>1340</td>
<td>0.254</td>
<td>0.1661063</td>
</tr>
<tr>
<td>30</td>
<td>0.75</td>
<td>1340</td>
<td>0.254</td>
<td>0.1903409</td>
</tr>
<tr>
<td>24</td>
<td>0.834565</td>
<td>1420</td>
<td>0.269</td>
<td>0.2244475</td>
</tr>
<tr>
<td>18</td>
<td>0.904508</td>
<td>1400</td>
<td>0.265</td>
<td>0.2398318</td>
</tr>
<tr>
<td>12</td>
<td>0.956773</td>
<td>1480</td>
<td>0.280</td>
<td>0.2681863</td>
</tr>
<tr>
<td>6</td>
<td>0.989074</td>
<td>1570</td>
<td>0.297</td>
<td>0.2940996</td>
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<tr>
<td>0</td>
<td>1</td>
<td>1800</td>
<td>0.341</td>
<td>0.3409091</td>
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<tr>
<td>6</td>
<td>0.989074</td>
<td>1140</td>
<td>0.216</td>
<td>0.21355</td>
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<tr>
<td>12</td>
<td>0.956773</td>
<td>640</td>
<td>0.121</td>
<td>0.1159725</td>
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<td>18</td>
<td>0.904508</td>
<td>610</td>
<td>0.116</td>
<td>0.1044981</td>
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<td>24</td>
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<td>510</td>
<td>0.097</td>
<td>0.0806114</td>
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<td>30</td>
<td>0.75</td>
<td>480</td>
<td>0.091</td>
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<td>0.654508</td>
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<td>42</td>
<td>0.552264</td>
<td>410</td>
<td>0.078</td>
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<td></td>
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<td></td>
<td>2.5453658</td>
</tr>
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</table>

**Effective Fetch Site 8**

*Method 1 - Prevailing Wind = Direction of Central Radial*

<table>
<thead>
<tr>
<th>Radial (a)</th>
<th>$\cos^2 a$</th>
<th>Distance (feet)</th>
<th>Distance (miles)</th>
<th>$D \times \cos^2 a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>42</td>
<td>0.552264</td>
<td>980</td>
<td>0.186</td>
<td>0.102504</td>
</tr>
<tr>
<td>36</td>
<td>0.654508</td>
<td>1000</td>
<td>0.189</td>
<td>0.12396</td>
</tr>
<tr>
<td>30</td>
<td>0.75</td>
<td>915</td>
<td>0.173</td>
<td>0.129972</td>
</tr>
<tr>
<td>24</td>
<td>0.834565</td>
<td>915</td>
<td>0.173</td>
<td>0.144626</td>
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<tr>
<td>18</td>
<td>0.904508</td>
<td>890</td>
<td>0.169</td>
<td>0.152465</td>
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<td>12</td>
<td>0.956773</td>
<td>850</td>
<td>0.161</td>
<td>0.154026</td>
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<tr>
<td>6</td>
<td>0.989074</td>
<td>775</td>
<td>0.147</td>
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<tr>
<td>0</td>
<td>1</td>
<td>680</td>
<td>0.129</td>
<td>0.128788</td>
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<td>6</td>
<td>0.989074</td>
<td>650</td>
<td>0.123</td>
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<tr>
<td>12</td>
<td>0.956773</td>
<td>620</td>
<td>0.117</td>
<td>0.112348</td>
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<tr>
<td>18</td>
<td>0.904508</td>
<td>580</td>
<td>0.110</td>
<td>0.099359</td>
</tr>
<tr>
<td>24</td>
<td>0.834565</td>
<td>550</td>
<td>0.104</td>
<td>0.086934</td>
</tr>
<tr>
<td>30</td>
<td>0.75</td>
<td>525</td>
<td>0.099</td>
<td>0.074574</td>
</tr>
<tr>
<td>36</td>
<td>0.654508</td>
<td>470</td>
<td>0.089</td>
<td>0.058261</td>
</tr>
<tr>
<td>42</td>
<td>0.552264</td>
<td>315</td>
<td>0.060</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1.667701</td>
</tr>
</tbody>
</table>

**Effective Fetch Site 7A** = 0.17 mile

**Effective Fetch Site 8** = 0.11 mile
Method 3 (SCS 1983)

$V = 59$ mph

Wind Velocity (Fig. 5)
Fetch Limited (Fig. 2)
Design Wave Calculations (SCS 1983)

Wave Height
Method 1 (fig. 2), for Design Fetch = 0.17 mi, Design Wind = 70 mph

\[ H_s = 1.5 \text{ ft} \]

Rock Size
(Fig. 8) Embankment slope 3H:1V, \( H_s = 1.5 \text{ ft} \), \( G_s = 2.65 \)

\[ W_{50} = 12.9 \text{ lb} \]
\[ W_{50} = 19.4 \text{ lb} \] corrected for rounded rock

\[ D_{50} = 0.61 \text{ ft sphere} \]
\[ D_{50} = 0.49 \text{ ft cube} \]

Use \( D_{50} = 6 \text{ inches} \)

Gradation Riprap

<table>
<thead>
<tr>
<th>% Passing</th>
<th>Min Size (inches)</th>
<th>Max Size (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>9</td>
<td>12</td>
</tr>
<tr>
<td>85</td>
<td>7.8</td>
<td>10.8</td>
</tr>
<tr>
<td>50</td>
<td>6</td>
<td>8.4</td>
</tr>
<tr>
<td>15</td>
<td>4.32</td>
<td>6.6</td>
</tr>
<tr>
<td>0</td>
<td>3.9</td>
<td>6</td>
</tr>
</tbody>
</table>
Assumptions (flow path around sheet pile (flow path shown on attached drawing)):
- Silty sand on backfill \( k = 1 \times 10^{-6} \text{ m}^2/\text{s} \)
- Steady state

\[
\frac{Q}{L} = \frac{KHf}{L} = \frac{(0.283 \text{ ft/d}) \times 8 \text{ ft} \times \frac{2}{4}}{L}
\]

\[
Q = 0.51 \text{ ft}^3/\text{ft}^2\text{day}
\]

\[
Q = 3.8 \text{ gal/day/ft}
\]

Reference: Lembke, Whitman, 1969
Ch. 18 Two-Dimensional Fluid Flow
Cedergren, 1989 Ch. 4 Flownet Construction
**Assumptions:**
- In pore fluid foundation (conservative)
- Steady state
- $\alpha = 12.0^\circ$
- $L = 1/8''$
- $k = 0.283 \text{ ft/day}$
- 3' horizontal embedment
- Free drain 4'8"; see attached drawing

### Region 1

$$Q = \frac{2a_h - 2a_r}{\cot \alpha} \ln \frac{h + h_1}{h - h_2}$$

### Region 2

$$Q = \frac{h_2 - h_1}{2}$$

$$Q = \left( \frac{7 \text{ ft}}{2(48 \text{ ft})} \right)^2$$

$$Q = \frac{2.167 \text{ ft}}{2}$$

### Region 3

$$Q = \frac{3a_1}{2L} \left( 1 + \frac{a_1 + a_2}{a_2} \right)$$

$$2.167 \text{ ft} = \frac{3a_1}{2.00} \left( 1 + \frac{a_1 + a_2}{a_2} \right)$$

$$6.50 \text{ ft} = 2a_1 + a_2 \left( \frac{a_1 + 9}{2L} \right)$$

$$a_1 = 2.6 \text{ ft}$$

### Reference:
- COE EM 1110-2-1901
- Sope analysis and control for dams
- Appendix E: Approximate method for analysis of seepage problems
LeClair/Riverton
Flow around sheet pile wall

Flux out of toe = 0.381 ft/day
Water exits 1 foot above toe
Maximum vertical gradient = 0.31 at toe
1/3 design high water level

grouted rip-rap

silty sand bank

filter
drain rock (6" min)

non-grouted rip-rap
be protection
bottom of stream channel

gravel penetration
(10" min)
depth to bedrock
Check required thickness of drain

\[ k_d = \frac{Q_d L_d}{A_d^2} \]

Cedergren, 1989

where \( Q_d = \text{flow in drain} \)

\( Q_d = \frac{1 \text{ ft}^3/\text{day}/\text{ft}}{} \) (from flow net analysis, with \( F_s = 2 \))

\( L_d = \text{length of drain} \)

\( L_d = 50 \text{ ft} \) (assumes filter/drain along flow line analyzed with flow net)

\( A_d = \text{area of drain} \)

Assuming the allowable maximum head in drain is less than the thickness of the drain

\[ k_d = \frac{(1 \text{ ft}^3/\text{day}/\text{ft}) \times 50 \text{ ft}}{A_d^2} \]

for \( h = 0.5 \text{ ft} \) \( k_d = 200 \text{ ft/day} \)

\[ = 7 \times 10^{-2} \text{ cm/sec} \]

Therefore, for a drain 0.5 ft thick, hydraulic conductivity of the drain material should be greater than 200 ft/day.
APPENDIX C.

Geotechnical Site Investigation

1) Boring Logs
2) Test Pit Logs
3) Site Photos
4) Laboratory Test Results
<table>
<thead>
<tr>
<th>Depth (Ft.)</th>
<th>Sample No.</th>
<th>Blows or RQD</th>
<th>Rec (%)</th>
<th>Description of Materials</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 5.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.0 - 6.5</td>
<td>S-1</td>
<td>2-3-3</td>
<td>100</td>
<td>Sand (sp), light yellow-brown, very fine-grained, very loose, moist</td>
<td></td>
</tr>
<tr>
<td>10.0 - 11.5</td>
<td>R-1</td>
<td>4-5-5</td>
<td>100</td>
<td>Sandy silt (sm), medium brown, fine-grained sand/silt with minor clay, loose, very moist</td>
<td></td>
</tr>
<tr>
<td>15.0 - 16.5</td>
<td>S-2</td>
<td>4-3-4</td>
<td>100</td>
<td>Sand (sp), mottled/yellow brown, fine to medium grained, medium dense</td>
<td></td>
</tr>
<tr>
<td>20.0 - 21.5</td>
<td>R-2</td>
<td>4-2-4</td>
<td>100</td>
<td></td>
<td></td>
</tr>
<tr>
<td>25.0 - 26.5</td>
<td>S-3</td>
<td>13-25-50/5&quot;</td>
<td>95</td>
<td>Sandstone interbedded with Mudstone, friable, uncemented</td>
<td></td>
</tr>
<tr>
<td>30.0 - 34.0</td>
<td>C-1</td>
<td>0</td>
<td>12</td>
<td></td>
<td>30' Switch to NX Coring Core material washing out, no recovery Drill Rate = 1 min/ft</td>
</tr>
<tr>
<td>34.0 - 37.0</td>
<td>C-2</td>
<td>0</td>
<td>33</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### BORING LOG

**PROJECT:** LeClair/Riverton, Site 7 West  
**COUNTY:** Fremont County  
**GEOLOGIST / ENGINEER:** Patrick Plumley, PG  
**DRILLING METHOD:** HSA, NX Coring  
**BORING NO.:** DH-1  
**DRILLER / COMPANY:** Maxim  
**TOTAL DEPTH OF HOLE:** 69.0 Ft.  
**DATE STARTED:** 05/15/06  
**DATE FINISHED:** 05/15/06  
**GROUNDWATER OBSERVATIONS:**  
**SURFACE ELEVATION:** 5071.0 Ft.  
**N COORDINATE:**  
**E COORDINATE:**

<table>
<thead>
<tr>
<th>Depth (Ft.)</th>
<th>Sample No.</th>
<th>Blows or RQD</th>
<th>Rec (%)</th>
<th>Legend</th>
<th>Description of Materials</th>
<th>Remarks</th>
</tr>
</thead>
</table>
| 37.0 - 39.0 | C-3        | 40           | 92      |        | Mudstone, gray to reddish brown, low hardness, friable, moderately weathered, flat lying laminations | Fractures healed with FeO3 stain  
Drill Rate = 5 min/ft  
Fractures/ft = 5  
Drill Rate = 5 min/ft  
Fractures/ft = 3 |
| 39.0 - 44.0 | C-4        | 52           | 70      |        | Sandstone, low to moderate hardness, weak, moderately weathered | Returns become black with carbon  
Drill Rate = 2 min/ft  
Fractures/ft = 5 |
| 44.0 - 49.0 | C-5        | 0            | 20      |        | Mudstone, blue gray, soft, friable, washes out | Drill Rate = 3 min/ft  
Fractures/ft = 5 |
| 49.0 - 54.0 | C-6        | 0            | 26      |        | Sandstone, light gray, moderately hard, weak, low to moderately fractured, weak silica cement, fine-grained, tight fractures | 53 Feet = Height of Dam  
'54-56' Drill Rate = 2 min/ft  
56-59' Drill Rate = 3 min/ft  
54-56' Fractures/ft = 3  
56-59' Fractures/ft = 0 |
| 59.0 - 64.0 | C-8        | 76           | 96      |        | Mudstone, blue gray, silty, friable, low hardness, thinly bedded, tight bedding | 59-60.5' Fractures/ft = 2  
60.5-62.5' Fractures/ft = 0  
62.5-64' Fractures/ft = 2 |
| 64.0 - 69.0 | C-9        | 76           | 100     |        | Silty sandstone, gray, moderately hard, weak, slightly weathered, fracture surfaces irregular | Drill Rate = 3 min/ft  
Fractures/ft = 0-3 |

Bottom of Boring = 69.0 Ft.
PROJECT: LeClair/Riverton, Site 8
COUNTY: Fremont County
BORING NO.: DH-2
BIRRILLER / COMPANY: Maxim
TOTAL DEPTH OF HOLE: 40.0 Ft.

SHEET 1 OF 2
GEOLOGIST / ENGINEER: Patrick Plumley, PG
DRILLING RIG: Truck Mount CME 55

DATE STARTED: 05/16/06
DATE FINISHED: 05/16/06

Remarks: Site 8, near TP-20
Hole backfilled with bentonite chips

Depth (Ft.) | Sample No. | Blows or RQD | Rec (%) | Description of Materials | Remarks
--- | --- | --- | --- | --- | ---
2 | | 2-5-12 | | Silty clay (cl), stiff, moist | Start with 4" hollow stem auger
5.0 - 6.5 | S-1 | | | Silt (ml), medium brown, medium dense, moist (alluvium) | El. 5136.0'
| | | | | | El. 5132.5'
| | | | Conglomerate, yellow-brown, fireable, weak, cemented, deeply weathered | | Switch to 8" hollow stem auger due to sloughing
| | | | Soil-like gravelly sand (sp), dense, moist with rounded gravel up to 1.5" (older alluvium) | | Wind River Formation
10.0 - 11.5 | R-1 | 29-23-12 | | Sandstone, light tan, very fine-grained, fireable, thinly bedded | Slough in hole. Discard sample.
| | | | | | El. 5125.0'
| | | | Mudstone, olive gray, fireable, slightly weathered, silty w/ faint laminations, locally iron oxide stained around occasional coarse sand lithic fragments | | El. 5121.5'
15.0 - 16.5 | S-2 | x-x-16 | | Siltstone and very fine-grained sandstone, light tan, low hardness, weak, tight | Drilling gets harder at 23 Feet
20.0 - 21.5 | R-2 | 9-15-34 | | Sandstone, medium gray, low hardness, weak, massive, medium grained, fractures appear to be mechanical | 26' Switch to NX Coring
| | | | | | Drill Rate = 2 min/ft
| | | | | Fractures/ft = 0-5
| 21.5 - 23.0 | S-3 | 18-16-21 | 0 | | El. 5116.0'
| 25.0 - 30.0 | C-1 | 70 | 88 | | El. 5115.5'
| 30.0 - 35.0 | C-2 | 50 | 88 | Mudstone/Claystone, mottled gray and yellow-brown, fireable, silty, local fractures are ox. stained | Fractures appear tight
| | | | | | Drill Rate = 3 min/ft
| | | | | Fractures/ft = 1
| 35.0 - 40.0 | C-3 | 62 | 98 | |
**BORING LOG**

**PROJECT:** LeClair/Riverton, Site 8  
**COUNTY:** Fremont County  
**GEOLOGIST / ENGINEER:** Patrick Plumley, PG  
**DRILLING METHOD:** HSA, NX Coring  
**DRILLING RIG:** Truck Mount CME 55  
**TOTAL DEPTH OF HOLE:** 40.0 Ft.

**BORING NO.** DH-2  
**DRILLER / COMPANY:** Maxim  
**TOTAL DEPTH OF HOLE:** 40.0 Ft.

**SHEET 2 OF 2**  
**GROUNDWATER OBSERVATIONS:** N COORDINATE:  
**DATE STARTED:** 05/16/06  
**X Groundwater not encountered**  
**DATE FINISHED:** 05/16/06  
**E COORDINATE:**  
**SURFACE ELEVATION:** 5139.0 Ft.

<table>
<thead>
<tr>
<th>Depth (Ft.)</th>
<th>Sample No.</th>
<th>Blows or RQD</th>
<th>Rec (%)</th>
<th>Legend</th>
<th>Description of Materials</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Sandstone, yellow-brown, weak, low hardness, massive with siltstone interbeds</td>
<td>El. 5100.5'</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Siltstone, gray</td>
<td></td>
</tr>
<tr>
<td></td>
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<td></td>
<td></td>
<td></td>
<td>Bottom of Boring = 40.0 Ft.</td>
<td></td>
</tr>
</tbody>
</table>
### Drilling Log

**Project:** LeClair/Riverton, Site 4  
**County:** Fremont County  
**Geologist/Engineer:** Patrick Plumley, PG  
**Drilling Method:** HSA, NX Coring  
**Drilling Rig:** Truck Mount CME 55  
**Total Depth of Hole:** 43.0 Ft.

#### Boring No. DH-3

**Driller/Company:** Maxim  
**Total Depth of Hole:** 43.0 Ft.

#### Groundwater Observations

- **Date Started:** 05/16/06  
- **Date Finished:** 05/16/06  
- **Surface Elevation:** 5063.0 Ft.

**Remarks:** Site 4, left abutment in cut area about 5' below natural ground surface. Hole backfilled with bentonite chips.

<table>
<thead>
<tr>
<th>Depth (Ft.)</th>
<th>Sample No.</th>
<th>Blows or RQD</th>
<th>Rec (%)</th>
<th>Legend</th>
<th>Description of Materials</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 5.0</td>
<td>S-1</td>
<td>3-2-3</td>
<td></td>
<td></td>
<td>Silt (ml)/Sandy silt, light tan, sand is very fine-grained, loose, dry</td>
<td>Drilling becomes hard at 7 Feet</td>
</tr>
<tr>
<td>5.0 - 10.0</td>
<td>R-1</td>
<td>7-7-9</td>
<td></td>
<td></td>
<td>Sand (sp), light brown, fine-grained, loose to very loose, ranges from silt to very fine sand, moist</td>
<td>El. 5049.5'</td>
</tr>
<tr>
<td>10.0 - 15.0</td>
<td>S-2</td>
<td>3-3-5</td>
<td></td>
<td></td>
<td>Sandy gravel and gravelly sand (gw/gp), dark brown, partially cemented, medium dense, saturated with gravel up to 2&quot;, dirty, subangular clasts</td>
<td>WATER TABLE AT 20 FEET</td>
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<tr>
<td>15.0 - 20.0</td>
<td>R-2</td>
<td>2-2-3</td>
<td></td>
<td></td>
<td></td>
<td>El. 5038.5'</td>
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<tr>
<td>20.0 - 25.0</td>
<td>S-3</td>
<td>15-21-27</td>
<td></td>
<td></td>
<td></td>
<td>Drilling becomes more difficult</td>
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<tr>
<td>25.0 - 30.0</td>
<td>R-3</td>
<td>50/0</td>
<td></td>
<td></td>
<td>No sample recovered</td>
<td>No sample recovered</td>
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<tr>
<td>30.0 - 35.0</td>
<td>S-4</td>
<td>50/1&quot;</td>
<td></td>
<td></td>
<td>No sample recovered</td>
<td></td>
</tr>
</tbody>
</table>

**Remarks:** Site 4, left abutment in cut area about 5' below natural ground surface. Hole backfilled with bentonite chips.
<table>
<thead>
<tr>
<th>Depth (Ft.)</th>
<th>Sample No.</th>
<th>Blows or RQD</th>
<th>Rec (%)</th>
<th>Legend</th>
<th>Description of Materials</th>
<th>Remarks</th>
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</thead>
<tbody>
<tr>
<td>40.0 - 41.5</td>
<td>S-5</td>
<td>12-27-40</td>
<td>100</td>
<td></td>
<td>Siltstone, light gray, low hardness, friable, silt to very fine sand, un cemented</td>
<td>Wind River Formation</td>
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<tr>
<td>42.6 - 43.0</td>
<td>S-6</td>
<td>50/5</td>
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<td></td>
<td>Bottom of Boring = 43.0 Ft.</td>
</tr>
<tr>
<td>Depth (Ft.)</td>
<td>Sample No.</td>
<td>Blows or RQD</td>
<td>Rec (%)</td>
<td>Legend</td>
<td>Description of Materials</td>
<td>Remarks</td>
</tr>
<tr>
<td>-------------</td>
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<td>---------</td>
<td>--------</td>
<td>---------------------------</td>
<td>---------</td>
</tr>
<tr>
<td>5.0 - 6.5</td>
<td>S-1</td>
<td>3-2-3</td>
<td>100</td>
<td></td>
<td>Sandy silt/Silty sand (ml/sm), interbedded, very loose, moist</td>
<td>Running sands so no intact sample except for 4&quot; at 16'</td>
</tr>
<tr>
<td>10.0 - 11.5</td>
<td>R-1</td>
<td>16-14-11</td>
<td>100</td>
<td></td>
<td>Sandy gravel (gp), medium dense, strong carbonate, subrounded gravel up to 2&quot;</td>
<td>WATER TABLE @ 15 FEET</td>
</tr>
<tr>
<td>15.0 - 16.5</td>
<td>S-2</td>
<td>1-2-3</td>
<td>33</td>
<td></td>
<td>Sand, very fine-grained, running sands Becoming medium-grained at bottom</td>
<td></td>
</tr>
<tr>
<td>20.0 - 21.5</td>
<td>R-2</td>
<td>5-8-14</td>
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<td></td>
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<td></td>
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<tr>
<td>21.5 - 23.0</td>
<td>R-3</td>
<td>15-21-27</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>25.0 - 26.5</td>
<td>S-3</td>
<td>5-9-17</td>
<td>100</td>
<td></td>
<td>Mudstone (silty), olive gray, friable, uncemented, tight, soil-like, stiff</td>
<td></td>
</tr>
<tr>
<td>29.0 - 29.7</td>
<td>S-4</td>
<td>37-50/2&quot;</td>
<td>0</td>
<td></td>
<td>Siltstone, gray, low hardness, weak, slightly fractured</td>
<td></td>
</tr>
</tbody>
</table>

Remarks: Site 7 East, along alignment in valley bottom, 10' west of irrigated pasture Hole backfilled with bentonite chips
1. Exploratory borings were drilled on @BDATE using a 4-inch diameter continuous flight power auger.

2. No free water was encountered at the time of drilling or when re-checked the following day.

3. Boring locations were taped from existing features and elevations extrapolated from the final design schematic plan.

4. These logs are subject to the limitations, conclusions, and recommendations in this report.

5. Results of tests conducted on samples recovered are reported on the logs.

Notes:

1. Exploratory borings were drilled on @BDATE using a 4-inch diameter continuous flight power auger.

2. No free water was encountered at the time of drilling or when re-checked the following day.

3. Boring locations were taped from existing features and elevations extrapolated from the final design schematic plan.

4. These logs are subject to the limitations, conclusions, and recommendations in this report.

5. Results of tests conducted on samples recovered are reported on the logs.
SITE 7  
LA CLAIR - RIVERTON

MIDDLE QM SITE

TP 1

[LT. ABOUT 1/3 UP SLOPE]
1. Sandy clay / clayey sand cr/sc
   med stiff, v. moist w/ occasional
   well rounded cobbles,
2. Silty clay stiff, moist, white/grey
   strong Calc [Residual Soil], moist
3. Siltstone, med. olive gray
   mod. hard, weak, closely
   fractured, mod. weathered
   NOTE: appears to be flat lying
   (grab sample)

TP 2

[LT. ABOUT THE 2/3 SLOPE AT 50 TO 60 DEGREES]
1. Sand / Silty Sand (SP) Orange brown
   loose to mod dense
2. Silt / mod. gray / v. stiff
   v. wk cement Calc [Older Alluvial Deposit]
3. Siltstone, moist
   olive gray / low-mod hard
   weak to friable / mod. weathered.

SPUR AIDOE

TP 3

[AT ABUTMENT 2/3 WAY UP ON]
1. Gravely sand (SP), w/ sit
   r minor clay, loose, moist
   [Residual Soil]
2. Sandy Gravel [Cobbles] 6"W
   20-40% cobble: 1" - 8"; well
   rounded w/ sand & gravel matrix
   medium gray; loose, caving
   [Terrace Deposit]
SITE 7
LaClare - Riverton
4/11/06

MIDDLE DAM SITE
TP 4

0
5
10

RT. ABUTMENT MID-SLOPE

1. Sandy silt (SM)
   Orange brown, loose / soft to firm, minor clay

2. Sandy silt w/ gravel (SM)
   Mottled gray / brown w/ str. CaCO3, grain to 2''

3. Siltstone, olive gray
   Weak, friable, duplex weathered, closely fractured
   [Fractures closed or filled with weather products]
   Massively banded - bedding planes not apparent.

TP - 5

525E

VALLEY BOTTOM

1. Silty sand (SM)
   H-brown, medium dense moist
   [Other Alluvium]

Cobble in lower 2'
**SITE 7**

**DIVERTON - LA CAIL**

**WEST DAM**

TP - 6

SBE

**TOP OF LT. ABUTMENT**

1. Silty sand / Mottled LT. Red brown & white (carbonate)
   W/ pebbles or coarse cobbles to 4ft

2. Sandy silt / Silty sand (melting)
   Middling moist W/ well rounded pebbles. Sand is v. fine-grained

3. Sand (5m) LT. Tan; v. fine grain.
   Middling moist

4. Sandy (SP) LT. Tan; medium
   Grained; middling moist

[OLDER ALLUVIAL TERRACE DEPOSIT]

Down 1/2 ft. Top, LT. ABUT

5. Sand + 3 w/ roots, organics

6. Silty sand (5m) / Sand (SP)
   LT. grey; Fine-coarse
   Grained Sandy moist

[Alluvial Terrace]

7. Mudstone
   Olive grey; weak; friable;
   Closely fractured high clay content; deeply weathered.
WEST DAM SITE

TP-8

LT. ABUT / MID SLOPE BENCH

Cobbles w/ sand (6W)
Hard, round cobbles to
1'; clast supported
Approx. 15%-25% gravel w/
Coarse sand
[Terrace Gravel/S]

TP-9

LT. ABUT / TOE AREA

1 Sand Silt (ML)
Mid dense / moist

2 Silts/Stone/ mudst.
Lt. gray green
Mod. hard / weak; closely
Fractured, appears to
dip slightly out of hill;
Locally sandy
[Wind River Fm.]

TP-10

VALLEY BOTTOM

Sand (SP)
Fine to medium-grained,
Loose to medium dense,
V moist, upper 1' roots (organics
No bedding
[Older Alluvium]
**SITE 7**

**WEST DAM SITE**

**TP - 11**

**NAOE**

1. Cobble w/sand loose/moist
Terrace cobbles - could

2. Sandstone/mudst
lt.-olive gray, low-mod hard/weak
deeply weathered; interbedded

**TP - 12**

**NAOE**

Cobble & sand loose/moist
[Terrace Deposit]

**TP - 13**

**DUE E**

East dam
right abut/Top
on terrace

1. Sandy gravel (SP/SW)
med. dense/moist
w/ cobble 15-20% to
6" [Terrace Deposit]

2. Mudstone
olive gray, rich
clay, deeply weathered
soil-like, intensely fractured, fractures
filled w/ rock & products

**RIGHT ABUT. TOE AREA**

4/11/06

**RIVERSTONE - LA CLAIR**
East Dam
Right Abutment

1. Sand (SP) MEO gray / mod. gravel
   Med. dense / moist
2. Sandstone / orange-tan
   Poorly consolidated, un lithified, low hard / friable
3. Mudstone / claystone / siltstone
   Low hardness, weak - friable
   Closely fractured, deeply weathered, interbedded

Valley Bottom

1. Sand (SP) v. fine-grained
   Med. brown, loose, v. moist
   [alluvium] MEO grit
2. Sandstone
   Med. gray, med. hard
   Weak to friable thinly bedded
   [Wind River Fm.]

West Dam

Valley Bottom
Upper central portion of reservoir basin

1. Sand (SP) fine-grained
   Loose, moist, clean

cobbles to 8"; prob. new dam of layout.
SITE 7 & 8
Riverton, La Claire
4/12/06

TP-17
Nor WP

SITE 7

1. Sandy silt (ML)
   stiff, dry

2. Silty mudstone
   deeply weathered, residual
   low hardness, friable
   closely bedded

3. Silty mudstone / siltstone
   dark olive - gray
   low to moderate hardness
   (Unia Riv. Fm.)

TP-18
S. 27 W.

SITE 8
Left Abutment
Mid - slope

1. Sandy silt w/ cobbles (ML)
   stiff, moist

2. Sandstone
   H. tany, low hardness/
   friable; wk cacao color
   medium to coarse-grained
   No visible fractures; appears
   low K overall
   (Wind River Fm.)

TP-19

SITE 8 Lt. Abutment
Top of pit 5' from top

1. Silty sand w/ gravel (SP)
   N. moist, loose
   (Side cast fill)

2. Cobble and sand gravel
   bed supported 60%
   gravel & cobbles; sand
   matrix [Terrace gravel]

3. Sandstone
   yellow brown / tan
   mid-grained; wk / friable
SITE B

RPWORTH LA CLAIB
4/12/06

TP-20

VALLEY BOTTOM
Between Power Pole & Branch

1. CLAY (CL) OR BROWN
V. STIFF, MOIST

2. SILT (ML) OR V. FINE SAND (SP)
MED. STIFF, V. MOIST
Becomes slightly coarser (to fine sand) near bottom
(Fining upward sequence)

TP-21

VALLEY BOTTOM
(WEST SIDE)

1. SILTY SAND (SM)
Mid orange brown, moist, med. loose, roots / organic upper 1/2 - 1'

2. Deeply weathered "SANDB" yell brown, v. weak, friable,
(soil like) uncremented w/ relic bedding, flat lying
becomes harder / more rock like w/ depth.

TP-22

1. SILTY SAND (SM)
W/ local clays / zones
STIFF / MOIST [FILL]

2. SANDST
Yell brown, deeply weathered,
weak, friable, low hardness
w/ relic horizon bedding / planes
TP - 23

CRST OLD EMB.EMEN T

SILTY SAND w/ gravel (SM)
15% rounded cobbles/gravel to 6"
matrix is medium brown, loose

[FILL]

TP - 24

VALLEY BOTTOM

1. CLAY (CL) mid brown
   v. stiff, moist

2. SILTY SAND (SM)/SF
   loose, moist
   v. fine-grained

[Alluvium]
Site Photos

Site 7A West. View Downstream towards dam site.
Site 7A West. View east towards left abutment.
Site 8 Panorama view towards the southwest from left abutment across to the right abutment.
Site 4, view southeast. DH-3 located upstream on the left abutment approximately 125 feet upstream from centerline of dam site. Note wetlands in valley bottom and culvert beneath older embankment.
Site 7B. TP-5 Terrace deposit with large cobbles.
Site 7A West. TP-7 Terrace Deposit
Site 7A West. View towards east along center line. TP-10 Valley Bottom. Note terrace surface on hill in background.
Site 7A West area, TP-15 upper reservoir area looking south towards dam.
Site 7A East. TP-17 Left abutment looking southwest towards right abutment on far side of valley.
Site 7A East. DH-4, valley bottom, view southwest.
Site 8, view northeast towards left abutment. Note older breached embankment located to the right (downstream) of the drill rig.
### Summary of Laboratory Test Data

<table>
<thead>
<tr>
<th>Site No.</th>
<th>Test Pit No. /Location</th>
<th>Depth (ft)</th>
<th>Description</th>
<th>% Gravel</th>
<th>% Sand</th>
<th>% Silt</th>
<th>% Clay</th>
<th>LL (%)</th>
<th>PL (%)</th>
<th>PI (%)</th>
<th>Natural Water Content (%)</th>
<th>Maximum Dry Density (pcf)</th>
<th>Optimum Moisture Content (%)</th>
<th>Permeability - Falling Head Test (cm/sec)</th>
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<tbody>
<tr>
<td>4</td>
<td>DH-3</td>
<td>5.5-6.5</td>
<td>Silty Sand</td>
<td>0</td>
<td>67.2</td>
<td>32.8</td>
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<td>4</td>
<td>DH-3</td>
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<td>7A East</td>
<td>TP-15</td>
<td>0-8</td>
<td>Silty Sand</td>
<td>1.2</td>
<td>61.8</td>
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<td>110.4</td>
<td>16.6</td>
<td>2.2x10^-6</td>
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<td>DH-4</td>
<td>Bulk</td>
<td>Sand with Silt and Gravel</td>
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<td>46.9</td>
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<td>7A West</td>
<td>DH-1</td>
<td>10.5-11.5 and 20-21.5</td>
<td>Sandy Silt</td>
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<td>48.9</td>
<td>29.1</td>
<td>22</td>
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<td>TP-20</td>
<td>Bulk</td>
<td>Clay</td>
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<td>47</td>
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<td>Bulk</td>
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<td>11</td>
<td>Left Abutment</td>
<td>Bulk</td>
<td>Silty Sand</td>
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<td>19.8</td>
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</table>
MOISTURE PERCENT
(ASTM D 2216)

459.6  Wet Wt. & Pan
401.2  Dry Wt. & Pan
201.9  Tare Pan
314.2  Loss
307.5  Dry Weight
19.0  % Moisture

G R A D A T I O N  D A T A
(ASTM D 422)

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Lbs. or Grs.</th>
<th>Retained %</th>
<th>Passing %</th>
<th>Specs.</th>
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<td>3/4&quot;</td>
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<td>3/8&quot;</td>
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<td>0</td>
<td>100</td>
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<tr>
<td>No. 4</td>
<td>0.2</td>
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<td>100</td>
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<td>No. 10</td>
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<td>No. 40</td>
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<td>No. 100</td>
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<td>No. 200</td>
<td>150.3</td>
<td>48.9</td>
<td>51.1</td>
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HYDROMETER ANALYSIS

SIEVE ANALYSIS

TIME READINGS

U.S. STANDARD SERIES

CLEAR SQUARE OPENINGS

CLAY (plastic) TO SILT (non-plastic)

SAND

GRAVEL

COBBLES

LeClair/Riverton, Site 7A West, DH-1
**Landmark Laboratories Ltd.**

**GRAIN SIZE DISTRIBUTION**

Project: Gannett Fleming  
Project No.: GANNEF-684J-02-702  
Leclair/Riverton Wyoming Dam

Description of Soil: Sandy Silt  
Sample No.: DH-1 @ 10.5-11.5'  
& 20.0-21.5'

Tested By: LAM  
Date of Testing: May 2, 2006

<table>
<thead>
<tr>
<th>Gravel</th>
<th>Sand</th>
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</table>

<table>
<thead>
<tr>
<th></th>
<th>Coarse to medium</th>
<th>Fine</th>
<th>Silt</th>
<th>Clay</th>
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<tr>
<td></td>
<td>U.S. standard sieve sizes</td>
<td></td>
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<td></td>
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</table>

- 100
- 80
- 60
- 40
- 20

**Percent finer**

<table>
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<th>Grain diameter, mm</th>
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<tr>
<td>0.002</td>
</tr>
<tr>
<td>0.001</td>
</tr>
<tr>
<td>0.01</td>
</tr>
<tr>
<td>0.075</td>
</tr>
<tr>
<td>0.035</td>
</tr>
<tr>
<td>0.15</td>
</tr>
<tr>
<td>0.42</td>
</tr>
<tr>
<td>0.84</td>
</tr>
<tr>
<td>1.75</td>
</tr>
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<td>3.15</td>
</tr>
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<td>80</td>
</tr>
<tr>
<td>90</td>
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<td>100</td>
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</table>

**Graph**

- Percent finer vs. grain diameter, mm
MOISTURE PERCENT
(ASTM D 2216)

<table>
<thead>
<tr>
<th></th>
<th>Wet Wt. &amp; Pan</th>
<th>Dry Wt. &amp; Pan</th>
<th>Tare Pan</th>
<th>Loss</th>
<th>Dry Weight</th>
<th>% Moisture</th>
</tr>
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<tbody>
<tr>
<td>402.9</td>
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MATERIAL  Silty Sand, Trace Gravel
TEST NO.  2

CLASSIFICATION

PIT NAME

AREA REP.  D. H. #2
TEST BY  LAM

SAMPLED BY  Client

GRADATION DATA
(ASTM D 422)

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LeClair/Riverton, Site 8, DH-2
MOISTURE PERCENT
(ASTM D 2216)

358.8  Wet Wt. & Pan  MATERIAL  Silty Sand  TEST NO. 3
345.4  Dry Wt. & Pan  CLASSIFICATION
96.1   Tare Pan
13.4   Loss
249.3  Dry Weight  PIT NAME
5.4    % Moisture

AREA REP.  D. H. 3  5.5-6.5  TEST BY  LAM
SAMPLED BY  Client

G R A D A T I O N  D A T A
(ASTM D 422)

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MOISTURE PERCENT
( ASTM D 2216)

391.9  Wet Wt. & Pan
371.2  Dry Wt. & Pan
93.8   Tare Pan
20.7   Loss
277.4  Dry Weight
7.5    % Moisture

MATERIAL  Sandy Silt
CLASSIFICATION
PIT NAME
A REA REP.    D. H. #3 @ 11-11.5' TEST BY LAM
SAMP LE D BY   Client

GRADATION DATA
( ASTM D 422)

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(ASTM D 2216)

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CLASSIFICATION
PIT NAME
AREA REP. | D. H. 4
TEST BY | LAM
SAMPLED BY | Client

% Moisture

G R A D A T I O N D A T A
(ASTM D 422)

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MOISTURE-DENSITY ANALYSIS

INBERG-MILLER ENGINEERS

CLIENT: Gannett Fleming
PROJECT: Irrigation Projects
JOB NO: 12555 RX
TEST DATE: 4-17-06
SOURCE: Test Pit
DESCRIPTION: Silty Sand

SAMPLE NO.: Site 7, TP-15, 0-8 ft
SAMPLED BY: Client
TESTED BY: DAL
TEST METHOD: D698-A

OPTIMUM WATER CONTENT (%): 16.6
MAXIMUM DRY DEN. (LBS/CU. FT): 110.4
PARTICLE SIZE ANALYSIS
INBERG-MILLER ENGINEERS

CLIENT: Gannett Fleming
PROJECT: Irrigation Projects
JOB NO.: 12555 RX
TEST DATE: 4-17-06
TESTED BY: DAL
TEST METHOD: C 136

SAMPLE NO.: Site 7, TP15
SAMPLED BY: Client
SOURCE: Test Pit
SAMPLE DESCRIPTION: Silty Sand

US STANDARD SIEVE SIZE OPENINGS

Grain Size in Millimeters

Percent Finer by Weight

Unified Soil Classification System (ASTM D2487)
# INBERG-MILLER ENGINEERS

## PARTICLE SIZE ANALYSIS

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<th>Garnet Fleming</th>
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FALLING HEAD PERMEABILITY TEST
REMOLDED SAMPLE OR UNDISTURBED SAMPLE

Project: Irrigation Projects
Client: Garnett Fleming
Source of Sample: Silty fine to medium SAND

Compaction Criteria: 95% Std Proctor at optimum moisture
Target Dry Density (pcf): 104.30
Moisture Content for Compaction: 12.6%

Determine dimension of remolded sample.
Tube Inside Diameter (cm): 5.35
Tube Length (cm): 15.24
Empty Tube Weight (w/ 2 caps) (g): 301.07
Tube Volume (cm³): 482.5
Required Weight of Moist Soil (g): 913.4

Sample Weights & Volume
Wt. Soil Tube & Caps (g): 1087.11
Unit Conversions
Tube, soil, sand & caps (g): 1887.0
Sand density (pcf): 99.5
Soil Volume (cm³): 418.5
Wet Density (g/cm³): 1.875
Dry Density (g/cm³): 1.620

Water Content
Pan No.: 92.23
Wt. Pan (g): 213.45
Moist Soil & Pan (g): 196.80
Moisture Content: 15.9%

LENGTH OF SOIL IN TUBE (cm) 13.22
PERMEAMETER HEAD CORR. (cm) 0
STANDPIPE NO. 1

CROSS SECTION AREA OF STANDPIPE (cm²) 0.709
SOIL TUBE AREA (cm²): 31.668
AVERAGE TEMPERATURE OF WATER AND SOIL (°C): 20
VISCOSITY RATIO U1/U20: 1.00

DATE START (YR, MO, DY) TIME START (HR, MN, SC) HEAD START (cm) DATE END (YR, MO, DY) TIME END (HR, MN, SC) HEAD END (cm) PERMEABILITY (cm/sec) ACCEPTED DATA

Note: Yellow shaded areas have formulas to carry down the end point data from previous period. These formulas should be overwritten whenever the tube is refilled.

INBERG-MILLER ENGINEERS
350 Parsley Blvd
Cheyenne, WY 82007
phone: 307-635-6827
fax: 307-635-2713
PROJECT NAME: Irrigation Projects
CLIENT: Gannett Fleming
CLIENT SAMPLE NO.: Site 8, TP-20
SOIL DESCRIPTION: Clay

DATE RECEIVED: 4/12/2006
TYPE OF SAMPLE: Bulk

Sieve Size | PARTICLE SIZE (mm) | FINER | PERCENT |
-----------|------------------|-------|---------|
|

Irbeig-Miller Engineers
124 East Main Street
Riverton, WY 82201
MOISTURE-DENSITY ANALYSIS

INBERG-MILLER ENGINEERS

CLIENT: Gannett Fleming
PROJECT: Irrigation Projects
JOB NO. 12555 RX
TEST DATE: 4-17-06
SOURCE: Test Pit
DESCRIPTION: Silty Sand

SAMPLE NO.: Site 11, Left Abutment
SAMPLED BY: Client
TESTED BY: DL/DAL
TEST METHOD: D698-A

OPTIMUM WATER CONTENT (%): 12.6
MAXIMUM DRY DEN. (LBS/CU. FT): 119.1
PARTICLE SIZE ANALYSIS
INBERG-MILLER ENGINEERS

CLIENT: Gannett Fleming
PROJECT: Irrigation Projects
JOB NO.: 12555 RX
TEST DATE: 4-17-06
TESTED BY: DAL

SAMPLE NO.: Site 11, Left Abutment
SAMPLED BY: Client
SOURCE: Test Pit
SAMPLE DESCRIPTION: Silty Sand

GRADATION DESCRIPTION:

US STANDARD SIEVE SIZE OPENINGS

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Unified Soil Classification System (ASTM D2487)
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FALLING HEAD PERMEABILITY TEST
REOMOLED SAMPLE OR UNDISTURBED SAMPLE

Project: Irrigation Projects
Client: Garnett Fleming
Source of Sample: Client
Description of Soil: Silty fine to medium SAND

Compaction Criteria: 95% Std Proctor at optimum moisture
Target Dry Density (pcf) 113.10
Moisture Content for Compaction 12.68

Determine dimension of remolded sample
Tube Inside Diameter (cm) 6.35
Tube Length (cm) 15.24
Empty Tube Weight (w/ 2 caps) (g) 299.67
Tube Volume (cm^-3) 482.5

Required Weight of Moist Soil (g) 984.8

Sample Weights & Volume
| Wt. Soil Tube & Caps (g) | 1157.58 |
| Tube, soil, sand & caps (g) | 1235.20 |
| Sand density (pcf) | 95.0 |
| Soil Volume (cm^-3) | 432.1 |
| Wet Density (g/cm^-3) | 1.985 |
| Dry Density (g/cm^-3) | 1.779 |

Length of Soil in Tube (cm) 13.65
Permeameter Head Corr. (cm) 0
Standpipe No. A

<table>
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<th>HEAD START (cm)</th>
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<th>TIME END (HR,MIN,SC)</th>
<th>HEAD END (cm)</th>
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<th>Accepted Data (cm/sec)</th>
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Note: Yellow shaded areas have formulas to carry down the end point data from previous period. These formulas should be overwritten whenever the tube is refilled.

File: permeability_reduction_cheyenne_cm_with_graph.xls

INBERG-MILLER ENGINEERS
350 Parsley Blvd.
Cheyenne, WY 82007
phone: 307-635-6827
dan: 307-635-2113
APPENDIX D.

Correspondence

1) Letter – BC Development, Developer at Site 11 property
2) Email – Loren Smith, Water Division III Superintendent
April 13, 2006

Robert Olson
BC Development
280 Ramshorn Drive
Riverton, WY 82501

RE: Potential Reservoir within the Proposed Fox Springs Subdivision

Dear Mr. Olson:

In reference to recent conversations between Frank DeeDee and Dick Inberg of Apex Surveying regarding a potential reservoir within your subdivision, I offer the following information.

Riverton Valley Irrigation District is studying a potential reservoir for storage of irrigation water to be used during high demand times on our system. The dam would be constructed on land belonging to Robert Foster and back water up the draw along the boundaries of proposed Lots 12, 13 and 14.

The conversations between Mr. Inberg and Mr. DeeDee outlined benefits for both BC Development and Riverton Valley Irrigation District.

At present we are conducting a Level II Study funded by the Wyoming Water Development Commission. If the study is favorable cost wise, our intent is to construct the reservoir.

This letter is to inform you of our long range plans and promote an understanding that would be beneficial for all parties concerned.

Sincerely,

Robert Rein, President

Cc: Ray Price, County Planner
From: Loren [smitty3@wyoming.com]
Sent: Friday, December 16, 2005 11:18 AM
To: vanderson@stateswest.com
Cc: dwade@stateswest.com
Subject: LeClair Riverton Valley Storage

Victor,

I thought last evening that maybe I should just write out the concept that I was trying to convey to folks at that original meeting, as I feel it is easily misunderstood but quite important.

At the point in time when LeClair (LID) has the need for an additional 30 cfs over their single appropriation (220 cfs), to supply demand, Riverton Valley (RVID) most likely has a similar increased demand over it’s first appropriation (120 cfs). If there is available to the districts a total of 340 cfs (basic 1/70 appropriation) the additional demand must then be satisfied solely from storage. Since the storage will most likely be held down gradient from LID, then an exchange contract will be necessary to provide LID with the additional water. Through exchange, 30 cfs of additional water would be diverted from the river at LID’s headgate on the river giving them a total diversion of 250 cfs. This then results in only 90 cfs of direct flow water being available for RVID to divert at their headgate. The original 30 cfs foregone by RVID would need to be made up from storage to return the supply to RVID to their 120 cfs single appropriation. In addition to this 30 cfs RVID would need to bring from storage whatever quantity of water that would be necessary for them to meet their demand over and above their single appropriation. This additional water could be up to another 30 cfs. If this scenario was to be played out over a 30 day period as was originally conceived, the total volume of storage used by RVID would be 3600 Acre-feet. (60 cfs for 30 days)

I hope this helps to explain this a bit better. Please contact me if you have any further questions.

Loren Smith
Superintendent, Water Division III

lsmith@seo.wyo.gov
APPENDIX E.

Sample Design Flood Calculations
Site 7A - Probable Maximum Precipitation Sample Calculation

index PMP = 9.4 in

PMP estimates for basin

<table>
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<th>Duration (hours)</th>
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<th>3</th>
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<td>8.8</td>
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Areal Reduction Factors

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<td>1</td>
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Incremental PMP estimates

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PMP intermediate estimates

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<th>1.5</th>
<th>1.75</th>
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<th>2.25</th>
<th>2.5</th>
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<tbody>
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<td>9.4</td>
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<td>10.1</td>
<td>10.4</td>
<td>10.9</td>
<td>10.9</td>
<td>11.1</td>
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Drainage Area = 0.4101 mi²
Curve Number = 61
100yr, 24hr Rainfall Amount = 3.0 in.
West Basin Lag Time = 0.325 hrs
East Basin Lag Time = 0.429 hrs

Cover type: Pasture, grassland, or range - continuous forage for grazing, Hydrologic condition: Good, Hydrologic soil group: B

Local Storm Depth-Duration Curve

PMP for Drainage Above Site 7A

y = 1.9589ln(x) + 9.3396

R² = 0.996
USGS Streamflows - Basin Characteristics Method
Annual Peak Flow for 100yr and 500yr Reoccurrence Interval

\[ P_{100} = 130A^{0.58}G^{0.25}S_b^{0.03} \]
\[ P_{500} = 245A^{0.57}G^{0.27}S_b^{0.03} \]

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<th>Site 7A West Drainage</th>
<th>Site 7A East Drainage</th>
<th>Site 7A</th>
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</thead>
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<tr>
<td>( A = 0.1092 ) ( \text{mi}^2 )</td>
<td>( A = 0.4101 ) ( \text{mi}^2 )</td>
<td>( A = 0.5193 ) ( \text{mi}^2 )</td>
</tr>
<tr>
<td>( S_b = 495.8 ) ( \text{ft/mi} )</td>
<td>( S_b = 500.2 ) ( \text{ft/mi} )</td>
<td>( S_b = 498 ) ( \text{ft/mi} )</td>
</tr>
<tr>
<td>( P_{100} = 146 ) ( \text{cfs} )</td>
<td>( P_{100} = 358 ) ( \text{cfs} )</td>
<td>( P_{100} = 415 ) ( \text{cfs} )</td>
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<tr>
<td>( P_{500} = 320 ) ( \text{cfs} )</td>
<td>( P_{500} = 771 ) ( \text{cfs} )</td>
<td>( P_{500} = 891 ) ( \text{cfs} )</td>
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</table>

Site 8 Drainage
\( A = 1.557 \) \( \text{mi}^2 \)
\( S_b = 470.1 \) \( \text{ft/mi} \)
\( P_{100} = 778 \) \( \text{cfs} \)
\( P_{500} = 1651 \) \( \text{cfs} \)

Site 11 Drainage
\( A = 1.788 \) \( \text{mi}^2 \)
\( S_b = 438.8 \) \( \text{ft/mi} \)
\( P_{100} = 825 \) \( \text{cfs} \)
\( P_{500} = 1747 \) \( \text{cfs} \)
\( PR = 9 \) \( \text{in} \)
\( Q_a = 0.048 \) \( \text{cfs} \)

Site 4 Drainage
\( A = 2.145 \) \( \text{mi}^2 \)
\( S_b = 638.77 \) \( \text{ft/mi} \)
\( P_{100} = 1001 \) \( \text{cfs} \)
\( P_{500} = 2130 \) \( \text{cfs} \)
\( PR = 9 \) \( \text{in} \)
\( Q_a = 0.056 \) \( \text{cfs} \)

Site 2 Drainage w/out Site 7A
\( A = 0.5642 \) \( \text{mi}^2 \)
\( S_b = 599.78 \) \( \text{ft/mi} \)
\( P_{100} = 457 \) \( \text{cfs} \)
\( P_{500} = 985 \) \( \text{cfs} \)
APPENDIX F.

Sample Hydraulic Calculations
Sample Hydraulic Calculations

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<th>value</th>
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<tr>
<td>ν=</td>
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<tr>
<td>g=</td>
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<tr>
<td>γ=</td>
<td>62.4</td>
<td>lb/ft³</td>
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</table>

\[
p_1 + \frac{V_1^2}{2g} + z_1 + h_p = p_2 + \frac{V_2^2}{2g} + z_2 + h_L
\]

\[
h_L = \frac{V^2}{2g} \left( f \frac{L}{D} + \sum K_L \right)
\]

\[
\operatorname{Re} = \frac{VD}{\nu}
\]

### Drop from LeClair Canal to 7A (PVC)

<table>
<thead>
<tr>
<th>quantity</th>
<th>value</th>
<th>units</th>
<th>quantity</th>
<th>value</th>
<th>units</th>
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<td>1800</td>
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<td>h_T or P=</td>
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<td>ft</td>
<td>L=</td>
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<td>ft</td>
<td>h_T or P=</td>
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<td>η=</td>
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<td>ft</td>
<td>hp shaft=</td>
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### Drop from LeClair Canal to 7A (Steel)

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<th>quantity</th>
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### 7A West to RV Canal (PVC)

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### 7A West to RV Canal (Steel)

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<th>value</th>
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<td>V=</td>
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<td>h_L=</td>
<td>101.868</td>
<td>ft</td>
<td>f=</td>
<td>0.0114</td>
<td>ft/s</td>
<td>h_L=</td>
<td>101.868</td>
<td>ft</td>
</tr>
<tr>
<td>L=</td>
<td>3850</td>
<td>ft</td>
<td>h_T or P=</td>
<td>0.000</td>
<td>ft</td>
<td>L=</td>
<td>3850</td>
<td>ft</td>
<td>h_T or P=</td>
<td>0.000</td>
<td>ft</td>
</tr>
<tr>
<td>K_L=</td>
<td>14</td>
<td></td>
<td>Δz (free discharge)=</td>
<td>105.426</td>
<td>ft</td>
<td>K_L=</td>
<td>14</td>
<td></td>
<td>Δz (free discharge)=</td>
<td>105.426</td>
<td>ft</td>
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<tr>
<td>hp + T or -P=</td>
<td>0</td>
<td></td>
<td>η=</td>
<td>0.7</td>
<td>ft</td>
<td>hp shaft=</td>
<td>0</td>
<td></td>
<td>Δz available=</td>
<td>105</td>
<td>ft</td>
</tr>
<tr>
<td>η=</td>
<td>0.7</td>
<td></td>
<td>h_L,TOTAL=</td>
<td></td>
<td>ft</td>
<td>η=</td>
<td>0.7</td>
<td></td>
<td>h_L,TOTAL=</td>
<td></td>
<td>ft</td>
</tr>
<tr>
<td>hp shaft=</td>
<td>0</td>
<td></td>
<td>Δz available=</td>
<td>105</td>
<td>ft</td>
<td>hp shaft=</td>
<td>0</td>
<td></td>
<td>Δz available=</td>
<td>105</td>
<td>ft</td>
</tr>
</tbody>
</table>
APPENDIX G.

Wetlands and Deepwater Habitats Classification
### WETLANDS AND DEEPWATER HABITATS CLASSIFICATION

**SYSTEM**

**SUBSYSTEM**

**CLASS**

**Subclass**

1. **M - MARINE**

   1 - **SUBTIDAL**
   - **RB – ROCK BOTTOM**
     - 1 Bedrock
     - 2 Rubble
   - **UB – UNCONSOLIDATED BOTTOM**
     - 1 Cobble-Gravel
     - 2 Sand
     - 3 Mud
     - 4 Organic
   - **AB – AQUATIC BED**
     - 1 Algal
     - 2 Rooted Vascular
     - 3 Floating Vascular
     - 4 Unknown Submergent
     - 5 Unknown Surface
   - **RF - REEF**
     - 1 Coral
     - 2 Worm
   - **OW - OPEN WATER/ Unknown Bottom**
     - 1 Coral
     - 3 Worm

2 - **INTERTIDAL**

   - **AB – AQUATIC BED**
     - 1 Algal
     - 2 Rooted Vascular
     - 3 Floating Vascular
     - 4 Unknown Submergent
     - 5 Unknown Surface
   - **RF – REEF**
     - 1 Coral
     - 2 Worm
   - **RS – ROCKY SHORE**
     - 1 Bedrock
     - 2 Rubble
   - **US – UNCONSOLIDATED SHORE**

**SYSTEM**

**SUBSYSTEM**

**CLASS**

**Subclass**

1. **E - ESTUARINE**

   1 - **SUBTIDAL**
   - **RB – ROCK BOTTOM**
     - 1 Bedrock
     - 2 Rubble
   - **UB – UNCONSOLIDATED BOTTOM**
     - 1 Cobble-Gravel
     - 2 Sand
     - 3 Mud
     - 4 Organic
   - **AB – AQUATIC BED**
     - 1 Algal
     - 2 Rooted Vascular
     - 3 Floating Vascular
     - 4 Unknown Submergent
     - 5 Unknown Surface
   - **RF – REEF**
     - 1 Coral
     - 2 Worm
   - **OW – OPEN WATER/ Unknown Bottom**
     - 1 Coral
     - 3 Worm

2 - **INTERTIDAL**

   - **AB – AQUATIC BED**
     - 1 Algal
     - 2 Rooted Vascular
     - 3 Floating Vascular
     - 4 Unknown Submergent
     - 5 Unknown Surface
   - **RF – REEF**
     - 1 Coral
     - 2 Worm
   - **SB – STREAMBED**
     - 1 Bedrock
     - 2 Sand
     - 3 Mud
     - 4 Organic
   - **US – UNCONSOLIDATED SHORE**

**SYSTEM**

**SUBSYSTEM**

**CLASS**

**Subclass**

1. **R - RIVERINE**

   1 – **TIDAL**
   - **RB – ROCK BOTTOM**
     - 1 Bedrock
     - 2 Rubble
   - **UB – UNCONSOLIDATED BOTTOM**
     - 1 Cobble-Gravel
     - 2 Sand
     - 3 Mud
     - 4 Organic
   - ***SB – STREAMBED**
     - 1 Bedrock
     - 2 Rubble
     - 3 Cobble Gravel
     - 4 Sand
     - 5 Mud
     - 6 Organic
     - 7 Vegetated
   - **AB – AQUATIC BED**
     - 1 Algal
     - 2 Rooted Vascular
     - 3 Floating Vascular
     - 4 Unknown Submergent
     - 5 Unknown Surface
   - **RS – ROCKY SHORE**
     - 1 Bedrock
     - 2 Rubble
   - **US – UNCONSOLIDATED SHORE**

2 – **LOWER PERENNIAL**

3 – **UPPER PERENNIAL**

4 – **INTERMITTENT**

5 – **UNKNOWN PERENNIAL**

*STREAMBED is limited to TIDAL and INTERMITTENT SUBSYSTEMS, and comprises the only CLASS in the INTERMITTENT SUBSYSTEM.

**EMERGENT is limited to TIDAL and LOWER PERENNIAL SUBSYSTEMS.**

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Classification of Wetlands and Deepwater Habitats of the United States  
Cowardin ET AL. 1979 as modified for National Wetland Inventory Mapping Convention
### WETLANDS AND DEEPWATER HABITATS CLASSIFICATION

#### SYSTEM

<table>
<thead>
<tr>
<th>Subsystem</th>
<th>L- LACUSTRINE</th>
<th>P - PALUSTRINE</th>
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<tbody>
<tr>
<td>Class</td>
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</tr>
<tr>
<td>subclass</td>
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<tr>
<td>1 Bedrock</td>
<td>1 Bedrock</td>
<td>1 Bedrock</td>
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<tr>
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<td>3 Sand</td>
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<td>1 Algal</td>
<td>1 Algal</td>
</tr>
<tr>
<td>2 Aquatic Moss</td>
<td>2 Aquatic Moss</td>
<td>2 Aquatic Moss</td>
</tr>
<tr>
<td>3 Rooted Vascular</td>
<td>3 Rooted Vascular</td>
<td>3 Rooted Vascular</td>
</tr>
<tr>
<td>4 Floating Vascular</td>
<td>4 Floating Vascular</td>
<td>4 Floating Vascular</td>
</tr>
<tr>
<td>5 Unknown Submerged</td>
<td>5 Unknown Submerged</td>
<td>5 Unknown Submerged</td>
</tr>
<tr>
<td>6 Unknown Surface</td>
<td>6 Unknown Surface</td>
<td>6 Unknown Surface</td>
</tr>
</tbody>
</table>

#### MODIFIERS

In order to more adequately describe the wetland and deepwater habitats one or more of the water regime, water chemistry, soil, or special modifiers may be applied at the class or lower level in the hierarchy. The farmed modifier may also be applied to the ecological system.

<table>
<thead>
<tr>
<th>Water Regime</th>
<th>Water Chemistry</th>
<th>Soil</th>
<th>Special Modifiers</th>
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</thead>
<tbody>
<tr>
<td>Non-Tidal</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>A Temporarily Flooded</td>
<td>H Permanently Flooded</td>
<td>K Artificially Flooded</td>
<td>*S Temporary-Tidal</td>
</tr>
<tr>
<td>B Saturated</td>
<td>J Intermittently Flooded</td>
<td>L Subtidal</td>
<td>*R Seasonal-Tidal</td>
</tr>
<tr>
<td>C Seasonally Flooded</td>
<td>K Artificially Flooded</td>
<td>M Irregularly Exposed</td>
<td>*T Semipermanent-Tidal</td>
</tr>
<tr>
<td>D Seasonally Flooded/ Well Drained</td>
<td>W Intermittently Flooded</td>
<td>N Regularly Flooded</td>
<td>U Unknown</td>
</tr>
<tr>
<td>E Seasonally Flooded/ Saturated</td>
<td>Y Saturated/ Semipermanent</td>
<td>P Irregularly Flooded</td>
<td>U Unknown</td>
</tr>
<tr>
<td>F Semipermanently Flooded</td>
<td>Z Intermittently Exposed</td>
<td>U Unknown</td>
<td>*These water regimes are only used in tidally influenced, freshwater systems.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Coastal Salinity</th>
<th>Inland Salinity</th>
<th>pH Modifiers for all Fresh Water</th>
<th>Soil</th>
<th>Special Modifiers</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Hyperaline</td>
<td>7 Hypersaline</td>
<td>a Acid</td>
<td>g Organic</td>
<td>b Beaver</td>
</tr>
<tr>
<td>2 Euthaline</td>
<td>8 Eusaline</td>
<td>9 Missaline</td>
<td>n Mineral</td>
<td>d Partially Drained/Ditched</td>
</tr>
<tr>
<td>3 Mixhaline (Brackish)</td>
<td>4 Polyhaline</td>
<td>0 Fresh</td>
<td>l Artificial Substrate</td>
<td>t Farmed</td>
</tr>
<tr>
<td>5 Mesohaline</td>
<td>6 Oligohaline</td>
<td>0 Fresh</td>
<td>x Soil</td>
<td>x Excavated</td>
</tr>
</tbody>
</table>

**NOTE:** Italicized terms were added for mapping by the National Wetlands Inventory program.
REFERENCES CITED


