RAY LAKE ENLARGEMENT, LEVEL II
Final Report

SUBMITTED BY
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IN ASSOCIATION WITH
States West Water Resources Corporation

July 2005
Final Report

RAY LAKE ENLARGEMENT
LEVEL II STUDY

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1. INTRODUCTION

The Little Wind River basin experiences chronic irrigation water shortages. Enlargement of Ray Lake on the Wind River Reservation near Lander, Wyoming, has been under consideration for several years to help alleviate these shortages. The existing 28 feet high embankment dam impounds a reservoir with a capacity of 6980 acre-feet at its normal high water line elevation of 5526 feet. The off-stream reservoir is supplied by canals and diversions from the South Fork and the North Fork of the Little Wind River. The general location of the project is shown on Figure 1.1.

Ray Lake Dam was built between 1924 and 1925. The dam has a history of dam safety issues including excessive foundation and embankment seepage, embankment cracking, cracking and leakage of the primary outlet structure, poor condition of the upstream slope protection, and inadequate emergency spillway capacity. These deficiencies were documented between 1983 and 1992 by U.S. Bureau of Reclamation (USBR) inspections and studies associated with their Safety Evaluation of Existing Dams (SEED) program. Due to the structural deficiencies identified by the SEED reports, the reservoir has been operating under a restricted pool elevation of 5525 feet (one foot below the normal high water line) since about 1991. The estimated storage capacity under the restricted pool is approximately 6040 acre-feet, or about 940 acre-feet less than could be stored if the restriction were lifted.

In 1993, the Tribal Water Engineer for the Eastern Shoshone and Northern Arapahoe Tribes hired GEI Consultants Inc. (GEI) to develop conceptual designs for dam safety modifications to the existing dam (GEI, 1993a). At the time, GEI also conducted a feasibility study for enlarging the dam and reservoir (GEI, 1993b).

Safety of Dams (SOD) and Enlargement Project alternatives and designs were carried forward in tandem studies by Western Water Consultants (now doing business as WWC Engineering) between 1994 and 1996 (WWC 1994, 1995a, 1995b, 1995c, and 1996). WWC advanced the designs, and in 1996 near-final (95%) construction plans and specifications for both the SOD and Enlargement Projects were completed and submitted to USBR for review.

Funding was provided to the tribes for the SOD modifications by the Bureau of Indian Affairs (BIA). However, the SOD work was delayed, evidently in the hope that the Enlargement Project could be approved and advanced, with the SOD funds applied to the enlargement construction costs. The Enlargement Project was not constructed at that time due to lack of funding and unresolved project technical issues, primarily concerns expressed by USBR about the inadequacy of the geotechnical information to support the enlargement design. The SOD improvements also have not been completed as of the date of this report (May 2005).

1.1 Project Purpose and Authorization

In 2003, the Wyoming Legislature authorized the Joint Business Council of the Eastern Shoshone and Northern Arapahoe Indian Tribes to sponsor water development projects under the state’s water development program in the same manner as political subdivisions and special districts of the state. The Joint Business Council (Sponsor) applied for funding of this Level II
study to review the existing (95%) design for the Enlargement Project, and to identify outstanding issues that will have to be coordinated between various stakeholders in order to advance the project.

This study has been carried out under the direction and funding of the Wyoming Water Development Commission (WWDC) by Gannett Fleming, Inc. (Gannett Fleming), in association with WWC Engineering (WWC), and States West Water Resources Corporation (States West). The scope of services, as documented in this report, included the following activities:

- compilation and review of previous reports and studies relevant to the project;
- engineering review of previous enlargement designs and construction cost estimates;
- evaluation of supplemental geotechnical investigation requirements and costs;
- analyses of hydrology, water availability, and reservoir size;
- review of the Little Wind River irrigation system, and assessment of how the Ray Lake Enlargement project would operate within the system;
- identification of potential permitting requirements; and
- facilitation of meetings and discussions between potential stakeholders.

### 1.2 Project Scoping and Study Prioritization

The initial scope of work for the project was limited to review and study of the Ray Lake Enlargement Project, and did not explicitly involve review or evaluation of the Safety of Dams (SOD) rehabilitation project. A project scoping meeting was held at the office of the Tribal Water Engineer in Fort Washakie on July 14, 2004. The meeting was attended by 21 people, including 4 representatives from the consulting team (Debora Miller/Gannett Fleming – Project Manager, Victor Anderson/States West, Murray Schroeder/WWC Engineering, and Patrick Plumley/Plumley & Associates), and Phil Ogle (WWDC Project Manager) and John Jackson from the WWDC staff. The meeting attendance sheet and presentation summary are provided in Appendix A. At the time of the scoping meeting no changes to the proposed scope of work were requested by the project Sponsors (the tribes).

Consulting team members Debora Miller and Patrick Plumley met in Denver on Friday, August 30, with Russ Carter of the USBR, and with Steve Pollock of the BIA in Billings attending via teleconference. The initial intent of this meeting was to review USBR’s previous concerns about the geotechnical investigations and analyses for the enlargement design that had been at issue following their review of the 1996 design. As a result of this meeting, however, it became apparent to the consulting team that the principal concern from BIA’s perspective was that the SOD work had not yet been completed by the tribes. The team learned at this time that the BIA was only authorized to fund projects pertaining to dam safety repairs, and operations and maintenance issues. Following this meeting, discussions among the project team, WWDC, and the Tribal Water Engineer concluded that the scope of work should be modified to include a technical review of the information available on the SOD design. The primary purpose of this review was to determine if the SOD work could be accomplished in a manner that could later be incorporated in the enlargement design, in order to avoid demolition and waste of the SOD improvements, especially to the outlet works. The SOD review and evaluation as it relates to the
enlargement design was added to the scope of services. The findings are presented in Section 3.5.1 of this report.
2. PREVIOUS STUDIES FOR ENLARGEMENT AND SAFETY OF DAMS PROJECTS

Ray Lake project records are extensive. As part of this study, Gannett Fleming reviewed the available reports and files, most of which are archived at the offices of WWC Engineering in Laramie, Wyoming. Other relevant reports and studies were made available to the team by WWDC and by the Tribal Water Engineer’s office. A complete inventory of the documents is provided in Appendix B, along with an annotated bibliography of the key studies and reports. The annotated bibliography provided in Appendix B was documented in a similar format as the “Wind River Irrigation Project Study Compilation – Final Report”, which was prepared for the tribes in August 2000 by Graham, Dietz and Associates (GDA, 2000). The GDA report is essentially a “document data base” that can be used for organizing and searching for various studies on the Wind River Indian Reservation Irrigation Project. By organizing the Ray Lake Annotated Bibliography in a similar format, this information could be easily incorporated into the data base, if desired.

Key study findings for the Ray Lake project are briefly summarized in this section under the main headings of Engineering Studies and Planning Studies. Refer to the Annotated Bibliography in Appendix B for more detailed summaries of the various studies. The project history can be summarized briefly as follows:

- 1924-1925: Ray Lake Dam Constructed
- 1993: GEI Consultants Preliminary Design Studies for Safety of Dams (SOD) and Feasibility Studies for Enlargement Options (11,450 and 20,000 acre-feet)
- 1994: WWC Alternatives Evaluation for Enlargement Projects (11,450, 16,000, and 41,650 acre-feet) and 50% Conceptual Design of SOD Project (6,980 acre-feet)
- 1995: Ray Lake Systems Analysis by NRCE which optimized enlargement size at 35,000 acre-feet
- 1995-1996: 35% through 95% Final Designs for 35,000 acre-feet Enlargement Project and 6,980 acre-feet SOD Project
- 1995-1996: USBR Reviews of Geotechnical Program and 35% Final Designs for SOD and Enlargement
- 2003: USBR Review of 95% Plans and Draft Specifications for SOD Modifications

2.1 Engineering and Design Studies

Ray Lake Dam was constructed in 1924-1925. There is essentially no documentation of the original design and construction, or records of maintenance or operation of the dam in the period between its construction and the first Safety Evaluation of Existing Dams (SEED) inspection by U.S. Bureau of Reclamation (USBR) on August 16, 1983.
Between 1983 and 1992 USBR conducted several SEED examinations of Ray Lake Dam, and performed a number of additional analyses and studies that are documented in technical memoranda. Key dam safety concerns identified by USBR during this time period are summarized as follows:

- **Embankment and foundation seepage**
  - Uncontrolled seepage is observed in the embankment near the outlet works (Sta 3+50 to 5+00), and through the foundation (Sta 5+50 to 10+50), with the most active seepage exit along the foundation near Sta 10+50.
  - Seepage appears when the reservoir elevation reaches approximately 5526 feet.
  - Two piezometers were installed adjacent to the outlet works in November 1984. The piezometers were not monitored after September 13, 1985.
  - Seepage exiting from cracks in the outlet conduit has the potential to cause internal erosion of embankment materials.
  - **Seepage in the vicinity of the outlet works constitutes a dam safety deficiency.**

- **Cracking and concrete deterioration of the outlet works conduit and stilling basin**
  - Divers inspected upstream conduit in September 1984 and found a 5/16-inch wide crack 16 feet upstream from the gate chamber. Water could seep through this crack into the adjacent embankment zone under full reservoir head. Crack was temporarily repaired in December 1984 using a fast-setting waterstop compound that sets and seals in the presence of water.
  - Additional inspections also found cracks in the conduit downstream from the gate chamber.
  - Gate seal was partially eroded and leaking.
  - Conduit cracks were attributed to a combination of settlement of the dam foundation and lateral spreading due to settlement of the overlying embankment fill.
  - Soils and water samples were tested and found to have high sulfate contents (3.81% for soil samples and 6499 mg/l for water samples). Type II cement was probably not used at time conduit was built, and so concrete is vulnerable to sulfate attack. Concrete deterioration was attributed to severe sulfate attack, freeze-thaw, and frost action.

- **Transverse and longitudinal cracks in the embankment near the crest of the dam**
  - Minor cracking was observed during initial SEED inspection in August 1983, but major cracking was documented in subsequent examination in August 1984. Additional cracks were discovered during inspection of outlet works in November 1984.
  - Development of cracks accelerated between 1983 and 1984 for unknown reason.
  - Cracks are more common near the outlet works. Maximum crack depth estimated to be about 6 feet below the crest of the dam.
  - No clear explanation as to cause of cracking. Cracks present a hazard at higher reservoir elevations.
  - **Cracks in the embankment constitute a dam safety deficiency.**

- **Inadequate capacity and erosion protection for the emergency spillway for the inflow design flood (PMF).**
  - Spillway is a 50-feet wide, unlined notch cut into the crest of the dam north of the outlet works (approximately Sta 9+25 to 9+75).
Probable Maximum Flood (PMF) was updated in 1990 (previous PMF study was 1983). Two PMF events were computed as follows:

<table>
<thead>
<tr>
<th>Type of Flood</th>
<th>Peak Flow (cfs)</th>
<th>Volume (acre feet)</th>
<th>Duration (hours)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Local thunderstorm PMF</td>
<td>12,200</td>
<td>2,800</td>
<td>15</td>
</tr>
<tr>
<td>General Storm PMF</td>
<td>14,000</td>
<td>6,800</td>
<td>72</td>
</tr>
</tbody>
</table>

- Flood routing used general storm PMF, and found that dam would be overtopped for approximately 4 hours at a maximum depth of 7 inches. Existing spillway can pass only 40% of PMF with 0 freeboard.
- Unlined spillway notch would likely erode rapidly if the reservoir water surface elevation ever exceeded the elevation of the notch (5527).
- Conclusion was that dam could safely store only 17% of the PMF below spillway crest (between normal pool elev. 5526 and spillway crest at 5527).
- **Overtopping of the dam due to inadequate spillway capacity is a dam safety deficiency.**
  - Failure and poor condition of the upstream riprap slope protection.
    - No bedding under the riprap layer.
    - Embankment upstream slope severely eroded by wave action for its entire length where riprap has been displaced.
    - During dive inspection in September 1984, divers found 2 feet of cobbles and boulders, and other debris layering the invert of the outlet conduit upstream from the gate shaft. It was assumed that the riprap had been washed into the outlet by wave action, and that therefore, the riprap was undersized.
    - Rocks and other debris were removed from the conduit in December 1984 during repairs to the cracks in the outlet conduit.

### 2.1.2 GEI Consultants (1993)

GEI Consultants Inc. (GEI) developed conceptual designs for dam safety modifications to the existing dam (GEI, 1993a), and conducted a feasibility study to enlarge the dam and reservoir in combination with the dam safety modifications (GEI, 1993b). The dam safety modification project was intended to address the deficiencies previously identified by USBR as summarized in the previous section. The scope of work included preliminary geotechnical and hydraulic analyses, Class III cultural resource evaluations, and reconnaissance-level assessments of threatened and endangered species.

The enlargement project that was being considered at the time of the 1993 studies was an 8 ft raise of the normal pool elevation, from elevation 5526 to 5534 ft, which would increase storage from 6,980 acre-feet to 11,450 acre-feet. This option would avoid impacting residential structures and Highway 287. Alternative designs for the 11,450 acre-feet enlargement project incorporated structural modifications to address the dam safety issues.

Later in the GEI (1993b) study, an enlargement to 20,000 acre-feet was evaluated at a reconnaissance level. GEI (1993b) reported that the 20,000 acre-feet enlarged reservoir would
require complete rebuilding of the main dam, plus several saddle dikes on the reservoir rim, relocation of a portion of Highway 287, relocation of one residence, and a new emergency spillway on the southwest rim of the reservoir.

In 1993, following review of GEI’s conceptual design summary report for SOD modifications, a value engineering team comprised of USBR geotechnical, structural and hydraulics design engineers, presented six proposals for alternatives to the GEI conceptual design to reduce costs or improve function of the SOD construction. The USBR value engineering (VE) proposals and GEI’s responses are provided in the Annotated Bibliography (Appendix B). A key VE proposal that was ultimately adopted in the subsequent designs of the SOD modifications by WWC was to use a 42-inch diameter steel lining of the upstream portion of the outlet conduit, in combination with replacement of the intake and downstream stilling basin, instead of complete removal and replacement of the outlet conduit.

2.1.3 WWC Engineering (1994-1997)
Between 1994 and 1996, Western Water Consultants (now doing business as WWC Engineering of Laramie, Wyoming) conducted alternatives evaluations, and advanced designs for both the SOD and enlargement projects in tandem. A number of enlargement sizes were considered at preliminary design levels, but the designers ultimately settled on a size of 35,000 acre-feet, based on recommendations in the Ray Lake Systems Analysis (NRCE, 1995). These efforts culminated in 1996 with plans and specifications for near final (95%) designs for both the SOD improvements, and a 35,000 acre-feet reservoir enlargement.

During this time frame, USBR was performing technical reviews of the analysis and designs for both projects, and were providing comments on behalf of the BIA. By the end of 1996, there remained unresolved technical issues for both the SOD and Enlargement projects, although the SOD issues were more easily manageable. The project records indicate that USBR’s design reviews were based on the 35% design reports by WWC, although the design work was being advanced to 95% plans for both projects. Gannett Fleming could not locate a 95% design report for either project, and believe that none exists.

The latest documentation in the files is a review done in late 2003 by USBR that focuses on the 95% plans and draft specifications for the SOD project. USBR evidently did not provide comments on the 95% construction documents for the 35,000 acre-feet enlargement project.

Gannett Fleming’s evaluation of the final (35,000 acre-feet) enlargement design is presented in Section 3 of this report.

2.1.4 Summary of Engineering and Design Studies
Table 2.1 summarizes the key SOD design studies, listed in chronological order. SOD designs addressed deficiencies previously identified by USBR including: embankment cracking, seepage problems and structural damage along the outlet conduit, inadequate erosion protection on the upstream slope, and hydraulic inadequacy of the spillway to pass the PMF.
Table 2.1  Ray Lake Safety of Dams (SOD) Project Design History

<table>
<thead>
<tr>
<th>Reference (date)</th>
<th>Subject</th>
</tr>
</thead>
<tbody>
<tr>
<td>GEI (1993)</td>
<td>Conceptual Design SOD</td>
</tr>
<tr>
<td>USBR (1994)</td>
<td>USBR Value Engineering of GEI SOD Conceptual Design</td>
</tr>
<tr>
<td>WWC (1994)</td>
<td>50% Conceptual Design</td>
</tr>
<tr>
<td>WWC (1995a)</td>
<td>Final Conceptual Design</td>
</tr>
<tr>
<td>WWC (1995c)</td>
<td>35% Final Design</td>
</tr>
<tr>
<td>USBR (1995)</td>
<td>Review of 35% Final Design</td>
</tr>
<tr>
<td>WWC (1996)</td>
<td>95% Final Design Plans</td>
</tr>
<tr>
<td>USBR (2003)</td>
<td>Review of 95% Plans and Draft Specifications for SOD</td>
</tr>
</tbody>
</table>

Table 2.2 summarizes the key enlargement design studies, listed in chronological order. Enlargement options ranged from 11,450 acre-feet to 41,650 acre-feet.
<table>
<thead>
<tr>
<th>Reference</th>
<th>Brief Title</th>
<th>Reservoir Size(s) (acre-feet)</th>
<th>Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td>GEI (1993b)</td>
<td>Feasibility Design Summary Report for Enlargement of Ray Lake Dam</td>
<td>11,450 and 20,000</td>
<td>11,450 acre-feet option avoided impacts to residences and Highway 287; 20,000 acre-feet option considered at reconnaissance level – would require complete rebuilding of main dam, several saddle dikes on reservoir rim, relocation of portion of Hwy 287, relocation of one residence, and new spillway on southwest rim of reservoir.</td>
</tr>
<tr>
<td>WWC (1994)</td>
<td>50% Conceptual Design</td>
<td></td>
<td>Option 1 - Safety of dams modifications to outlet (per GEI, 1993a).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6980</td>
<td>Option 2 – Low impact raise + dam safety modifications to outlet (per GEI, 1993b).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>11,450</td>
<td>Option 3 – Dam safety modifications to outlet + centerline raise of main dam + extension dike on north abutment + relocate spillway from main dam to south abutment. Would impact residences on south abutment.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>16,000</td>
<td>Option 4 – Similar to Option 3 with larger quantities + involving special considerations for Hwy 287 relocation.</td>
</tr>
<tr>
<td>WWC (1995a)</td>
<td>Final Conceptual Design</td>
<td>20,000, 31,000 and 41,650</td>
<td>All alternatives assume relocation of Hwy 287 to north portion of dam, and relocation/demolition of existing residence.</td>
</tr>
<tr>
<td>WWC (1995b)</td>
<td>Conceptual Design Outlet Canals</td>
<td>36,000</td>
<td>Canal capacities based on findings of Ray Lake System Analysis Report (NRCE, 1995) recommendations of 60 cfs (south outlet) and 270 cfs (north outlet).</td>
</tr>
<tr>
<td>WWC (1996)</td>
<td>Draft Plans and Specifications for the Ray Lake Enlargement</td>
<td>35,000</td>
<td>Indicated as “95% Design Specs” – with accompanying plan sheets (51 sheets). Plans show embankment plans and profiles; typical embankment sections; spillway plan, profile, and cross-section; riprap, toe drain, and finger drain details; typical highway section; causeway section and details, north and south outlet works plan layouts, structural details; lateral 37C realignment plan and profile; north and south outlet control details.</td>
</tr>
</tbody>
</table>
2.2 Planning Studies

2.2.1 NRCE Studies of the Wind River Irrigation System (1994)
Natural Resources Consulting Engineers (NRCE) of Fort Collins, Colorado, conducted several studies in association with the Tribal Water Engineer in 1994 to analyze the Wind River Irrigation System and develop a master plan to address system deficiencies. The comprehensive master plan is presented in the report entitled “Wind River Irrigation Project Assessment and Plan” (NRCE, 1994a). Incorporated in the master plan was a system reconfiguration plan, which was divided into several proposed projects based on geographic location. Projects of particular relevance to evaluation of the Ray Lake Enlargement project include the Upper and Lower Ray Canal Reconfiguration Reports (NRCE, 1994b and 1994c).

2.2.2 NRCE Ray Lake System Analysis (1995)
This Technical Memorandum by NRCE (1995) considered the areas currently served by Ray Lake, which include the lower portion of the Coolidge Canal and the entire area serviced by the Sub-Agency Canal. The study analyzed hydrologic and other pertinent data to estimate the potential for additional utilization of flows from the North and South Forks of the Little Wind River to meet current and future demands. The study also developed preliminary layouts for supply and distribution canals, and enlargement options for Ray Lake to service an enlarged service area.

2.2.3 Draft Environmental Assessment (1996)
The Tribal Water Engineer and NRCE developed the draft Environmental Assessment (EA) for Reservoir Enlargement and Dam Safety Modifications to Ray Lake (NRCE, 1996). The EA was prepared for the Bureau of Indian Affairs (BIA), Wind River Agency, and was based on the 95% Final Design (WWC, 1996) of a 35,000 acre-feet reservoir as the Proposed Action. Other alternatives considered included No Action, Dam Safety Modifications Only, Dam Safety Modifications plus Water Conservation, and Other Reservoir Sites.

2.2.4 Upper Wind River Storage Project – Level 1 Study (2001)
This study by Short Elliott Henderson (SEH, 2001) assessed the need for water storage in the Upper Wind River basin, identified alternative reservoir sites, and performed a screening analysis of the sites. Following the Phase I alternative storage site screening, Phase II detailed analyses were conducted to evaluate short-listed sites in more detail, including preparation of conceptual-level designs and cost estimates. Enlargement of Ray Lake up to 41,650 acre-feet passed the Phase I screening and was recommended as a project to be considered for future study. It was not considered necessary to conduct Phase II conceptual design because of the substantial design work already completed at the site.

2.2.5 Wind/Bighorn River Basin Plan Final Report (2003)
The basin plan was completed in October 2003, and identified the Ray Lake Enlargement project (41,650 acre-feet) as a short-listed candidate for further consideration (BRS, 2003).
3. REVIEW EXISTING ENLARGEMENT DESIGN

The Ray Lake System Analysis (NRCE, 1995) included an evaluation of hydrology and water supply, and system demands. The study concluded that available water supply from the North Fork and South Fork of the Little Wind River was on the order of 22,800 acre-feet per year. Active storage requirements in Ray Lake were optimized at approximately 26,000 acre-feet. This size allows for an average annual drawdown of approximately 16,500 acre-feet and an average annual carry-over of approximately 10,400 acre-feet.

The 95% final design (WWC, 1996) sized the reservoir at 35,000 acre-feet, with approximately 10,000 acre-feet of inactive storage/recreational pool. This size evidently was based on achieving close to the desired active storage of 26,000 acre-feet, while still having enough driving head near the bottom of the active pool level to be able to deliver water via the proposed new south canal. Thus the current design has approximately 25,000 acre-feet of active storage, and is based on anticipated average annual need of 16,500 acre-feet.

The final design that is shown on the 1996 “95% Plans and Specifications” is for a project with the following features:

- 35,000 acre-feet reservoir with normal high water line elevation of 5565 feet;
- removal of the old dam;
- a new, approximately 70-feet high embankment dam with a crest elevation of 5571 ft;
- 3 distinct embankment components: North Embankment, South Embankment, and Lateral 37C Dike;
- new emergency spillway cut in native materials on the south side of the reservoir;
- two new outlet works with appurtenant structures on the south and north sides of the reservoir;
- work associated with Highway 287 realignment and rebuilding along a portion of the crest of the new dam (North Embankment), and a causeway across a low spot on the north side of the reservoir;
- realignment, rehabilitation, and enlargement of Lateral 37C between Ray Canal and Ray Lake on the north side of the reservoir; and
- construction of a new canal pipeline to serve Lateral 67C-19 on the south side of the reservoir.

As part of this study, Gannett Fleming reviewed this design and conducted analyses to evaluate the following:

- evaluation of water supply needs and how an enlarged Ray Lake would work as an integral component of the Little Wind River Irrigation System;
- verification of the optimum reservoir size for the enlargement at 35,000 acre-feet, based on an independent evaluation of water availability and reservoir operations modeling;
• assessment of the available geotechnical data and analyses to support the 1996 design, which was questioned extensively by USBR in their technical reviews.

The results of these assessments and analyses are presented in the following sections.

3.1 Little Wind River Irrigation System and System Demands Evaluation

3.1.1 Little Wind River Irrigation System

The Wind River Irrigation Project was begun in 1905 as part of an agreement between the Indian Tribes and the federal government to exchange a portion of the reservation for an irrigation system to serve the Indian lands. The Ray Canal was one of the first parts of the system to be built; however, it was nearly 50 years later before the entire system was considered complete. There are still areas where improvements can be made, and new environmental concerns have also been raised, including damage to fish habitat and movement, erosion, drainage, and sediment controls, and a desire for better efficiency overall (NRCE, 1994).

In 1994, the Tribes completed a comprehensive study of the Wind River Irrigation Project (WRIP) titled the “Wind River Irrigation Project Assessment and Plan” (NRCE, 1994a). The evaluation was an in-depth analysis of the WRIP and presented a comprehensive master plan to remedy system deficiencies. The plan contained five separate interrelated components: a management plan, a system reconfiguration plan, a plan for on-farm improvements, an equipment plan, and a financial plan. As reported in the assessment study, the WRIP project is characterized by high structure and canal density, low irrigation efficiency, and onerous operation and maintenance problems.

The system reconfiguration plan component of the 1994 study was an eight volume report that detailed the assessment work and improvement plans for the Lower Ray Canal System, the Coolidge Canal System, and Sub-Agency Canal Systems in the Little Wind Unit; the Dry Creek Canal System, Meadow and Willow Creek Canal System, and Wind River ‘A’ Canal System in the Upper Wind Unit; and the Johnstown Unit and Lefthand Unit. For all of these eight study areas, the plan examined two different irrigation scenarios. The first one was based on the water rights decreed in the Big Horn Adjudication and existing on-farm efficiencies (1.5 cfs/70 acres). The second scenario was based on a more efficient on-farm system which includes improvements to transmission, distribution and measurement (0.9 cfs/70 acres). The 1994 NRCE work was focused on identifying reconfigurations to the system that would improve irrigation efficiency and operation. A complete analysis of an enlargement of Ray Lake water storage should reflect whether or not these system improvements have been or will be made.

In 1995, NRCE issued a report titled “Ray Lake System Analysis” (NRCE, 1995). This report made a recommendation to expand Ray Lake storage to at least 26,000 acre-feet, based on a reservoir sizing analysis that relied on limited hydrologic data. The reservoir sizing analysis showed that a 26,000 (active) acre-feet storage reservoir would serve an average of 16,500 acre-feet of demand 80% of the time (with shortage the remaining 20%). The report also recommended sizes for the Ray Lake inlet canal as well as the North and South delivery canals.
3.1.2 Irrigation System Demands Evaluation

The 1995 enlargement design was based on a previous estimate of the average annual un-met demand for supplemental irrigation water of 16,500 acre-feet (NRCE, 1995). This estimate was based on irrigation demands for the current service areas of both Ray Lake and Washakie Reservoir, and considered an expanded Ray Lake contribution to the combined service area to allow Washakie Reservoir to more efficiently serve the remaining smaller portion of the combined service area. It did not consider potential future irrigation water needs that are in addition to the current service area irrigated lands.

The Upper Wind River Storage Project –Level I Study was produced in 2001 by Short Elliot Hendrickson, Inc. (SEH, 2001), for the Wyoming Water Development Commission. The focus of the report was a basin-wide examination of water storage. The report presented estimates of irrigation system demands for many of the areas of interest to this current study. Table 3.1 summarizes the estimated current and future un-met demands that were identified by the SEH (2001) study that could potentially be served by an enlarged Ray Lake. These potential un-met demands are described in the following paragraph.

Using an incomplete 1998 version of the Wyoming Integrated River System Operation Study (WIRSOS) model, SEH estimated that there are un-met demands in the Little Wind River basin of about 31,500 acre-feet/year (100 percent level) and 18,500 acre-feet/year (80 percent level). The 80-percent demand level is the annual flow required to meet all calls except in 10 of the 50 driest years modeled, and is a common sizing criterion for irrigation water storage and supply. The incomplete WIRSOS model did not include Futures and Type 7 and 8 Awards. The South Fork of the Little Wind River has 11,179 acre-feet/year of Types 7 and 8 Awards that are not currently irrigated (SEH, 2001). Although there are no Futures Awards in the South Fork of the Little Wind River basin, the Futures Awards in other areas within the basin are potentially serviceable from Ray Lake. The Riverton East Futures Area has a combined 17,536 acre-feet/year of water need (719 acre-feet to be derived from the Little Wind River and 16,817 acre-feet from the Big Wind River). This water could be delivered from Ray Lake via the North Outlet Canal to the Little Wind River. The Futures Area needs in the Big Horn Flats project are estimated to be 7,212 acre-feet/year (2,616 acre-feet/year from the Little Wind River and 4,596 acre-feet from the Big Wind). This project also could be potentially served by storage in Ray Lake. The SEH study concluded that “the need is clear and compelling for significant additional upper basin storage to address existing and future demands.” Based simply on the needs identified in this paragraph, a conservative estimate of the total annual irrigation needs serviceable by Ray Lake and Washakie Reservoir via the Little Wind River is on the order of 33,000 acre-feet. An additional 21,913 acre-feet per year of need on the Big Wind below the confluence could also be feasibly served from Ray Lake, as summarized on Table 3.1.

---

1 The Eastern Shoshone and Northern Arapahoe Tribes were awarded Federal reserved water rights, of which approximately 209,000 acre-feet/yr are referred to as Futures Awards. Type 7 and 8 Awards are federally reserved Tribal water rights tied either to lands that have been irrigated in the past but are not currently irrigated (Type 7), or lands that have been identified as potentially irrigable but have never been irrigated (Type 8) (SEH, 2001).


### Table 3.1 Estimated Additional Irrigation Water Needs Potentially Served by an Enlarged Ray Lake (derived from SEH, 2001)

<table>
<thead>
<tr>
<th>Demand</th>
<th>80% Un-met Demand Ave. Annual (acre feet)</th>
<th>Future Demand Ave. Annual (acre feet)</th>
<th>Source/Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Existing un-met irrigation demands – lower Little Wind River basin</td>
<td>18,500</td>
<td></td>
<td>1998 WIRSOS (SEH, 2001, Table 2-5) (NRCE, 1995, estimated 16,500 acre-feet)</td>
</tr>
<tr>
<td>Type 7 and 8 Awards</td>
<td></td>
<td>11,179</td>
<td>South Fork Little Wind River (SEH, 2001, Table 2-5)</td>
</tr>
<tr>
<td>Futures Awards (Little Wind River Source)</td>
<td>719</td>
<td></td>
<td>Little Wind River Portion of Riverton East Project (SEH, 2001, Table 2-3)</td>
</tr>
<tr>
<td></td>
<td>2,616</td>
<td></td>
<td>Little Wind River Portion of Big Horn Flats Project (SEH, 2001, Table 2-3)</td>
</tr>
<tr>
<td>Sub-Total Little Wind River Source of Diversions</td>
<td>18,500</td>
<td>14,514</td>
<td></td>
</tr>
</tbody>
</table>

**Total Estimated Irrigation Water Needs - Little Wind River Basin = 33,041 acre-feet**

<table>
<thead>
<tr>
<th>Demand</th>
<th>80% Un-met Demand Ave. Annual (acre feet)</th>
<th>Future Demand Ave. Annual (acre feet)</th>
<th>Source/Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Existing un-met irrigation demands – Big Wind River basin below Little Wind River</td>
<td>500</td>
<td></td>
<td>1998 WIRSOS (SEH, 2001, Table 2-5)</td>
</tr>
<tr>
<td>Futures Awards (Big Wind River Source)</td>
<td></td>
<td>16,817</td>
<td>Big Wind River Portion of Riverton East Project (SEH, 2001, Table 2-3)</td>
</tr>
<tr>
<td></td>
<td>4,596</td>
<td></td>
<td>Big Wind River Portion of Big Horn Flats Project (SEH, 2001, Table 2-3)</td>
</tr>
<tr>
<td>Sub-Total Big Wind River Source of Diversions</td>
<td>500</td>
<td>21,413</td>
<td></td>
</tr>
</tbody>
</table>

**Total Estimated Irrigation Water Needs - Big Wind River below Little Wind River = 21,913 acre-feet**

### 3.1.3 Irrigated Areas

The current and possible future service areas for Washakie Reservoir and Ray Lake (common areas) are shown on Plate 1 (attached in pocket). Note that not all of the irrigated areas in the basin are shown on the map, only those areas in the lower portion of the drainage that could be served by both Washakie Reservoir and Ray Lake. Because Washakie Reservoir is situated on the higher western topography, it is positioned to deliver water to the entire Little Wind Unit. Enlargement of Washakie reservoir is not feasible due to geotechnical site constraints (GEI, 1993a). However, irrigation demand on Washakie could be reduced by increasing water storage in Ray Lake to serve the common areas (NRCE, 1995). By increasing the service area of Ray...
Lake, and decreasing the area that must be served by Washakie, late season strains on Washakie Reservoir could be reduced.

3.1.3.1 Existing Irrigation Service Area
Currently, the 6,980 acre-feet of storage in Ray Lake is used for supplemental irrigation on lands served by the lower portion of the Coolidge Canal (via Mill Creek) and lands served by the Sub-Agency Canal (via Mill Creek and the South Fork Little Wind River). Plate 1 identifies these two areas. The acreages of irrigated land currently assessed by the BIA for these areas are 4,021 acres (Sub-Agency) and 2,698 acres (Mill Creek), for a total of 6,719 acres (Billy Vassau, BIA Irrigation Office, 2005, personal communication).

3.1.3.2 Potential Additions to Existing Irrigation Service Area with Ray Lake Enlargement
With an enlarged Ray Lake, irrigation service would be continued to the existing Ray Lake service area (see Plate 1), and possibly expanded in the current service area to include Type 7 and 8 Awards acreages that are not currently being irrigated. In addition, service could be extended to two areas not previously served by Ray Lake.

One of the potential new service areas is additional acreage that could be served via the Coolidge Canal. Presently this area is irrigated by direct stream flow diversions out of the South Fork of the Little Wind, supplemented by storage supplies from Washakie Reservoir via the South Fork. The total assessed acreage on the Coolidge Canal is 6,062 acres, and the gross acreage in this area is 11,184 acres.

The second area that could be potentially served by an enlarged Ray Lake includes lands served by Lateral 65C-19 that are presently irrigated via Canal 65C, a diversion from the lower Ray Canal. The gross acreage in this area is 1,820 acres, and the current assessed acreage for this land is estimated to be about 910 acres (estimate only).

New irrigation within the two areas currently served via Coolidge Canal and Lateral 65C-19, could be as follows:

- as supplemental supply on existing irrigated grounds within the identified service areas, and/or
- irrigation on new grounds within these areas, including Type 7 and 8 Awards acreages.

Under the proposed enlargement project, the Coolidge Canal lands would receive water via a new North Canal Outlet for the enlarged Ray Lake Reservoir, and the lands on Lateral 65C-19 would be served by a new South Outlet Canal. Plate 1 shows these two irrigated areas, the existing canals, and the proposed new outlet canals.

In addition to the potentially irrigated areas described above that could be served by new outlets from Ray Lake to the Coolidge Canal and to Lateral 65C-19, there are two downstream Futures Awards projects that also could potentially be served from an enlarged Ray Lake. These new areas are within the Riverton East and Big Horn Flats Projects. SEH (2001, Table 3-2)
summarize potentially irrigable Futures Awards acreages within these two projects that could be served from the Little Wind River and Big Wind River supply sources (and therefore could be supplemented by storage from Ray Lake). Total acreages from the Little Wind River source include 968.6 acres in the Big Horn Flats Project and 156.4 acres in the Riverton East Project; and total acreages from the Big Wind River source include 1,701.4 acres in the Big Horn Flats Project and 3,657.6 acres from the Little Wind River source. Water from the Big Wind and Little Wind Rivers for the Riverton East project would be diverted to canal take-out, or to river pump stations for delivery to these areas (Nelson Engineering, 2001). It is assumed that similar systems would be required to deliver water to Big Horn Flats. For both projects, an enlarged Ray Lake could work in tandem with Washakie reservoir to supplement river flows that could be accessed via the pump and diversion systems that are installed.

3.1.4 Other Potential Beneficial Uses

Enlargement of Ray Lake has the potential to provide multiple benefits and increased flexibility for system operation, in addition to addressing existing late season irrigation deficiencies in the current service area. The Law and Order Code of the Eastern Shoshone and Northern Arapaho Tribes lists 15 beneficial uses of water. Potential project benefits with an enlarged Ray Lake reservoir could include the following subset of the potential benefits listed by the tribes:

- Improving in-stream flows on the Wind River below its confluence with the Little Wind River (including in-stream flow enhancement for fisheries, wildlife, pollution control, aesthetics and cultural purposes)
- Recreational benefits
- Potential downstream municipal uses
- Agricultural uses
- Hydropower
- Industrial uses
- Marketing and transfer
- Cultural and religious uses

The Sponsor has expressed the desire to consider these potential beneficial uses in the evaluation of an enlarged Ray Lake (Gary Collins, personal communication, 2005). Economic analyses were not included in the scope of work for the present study. As such, no specific demands or quantitative estimates for these supplemental benefits have been produced. Gannett Fleming recommends that future economic evaluations attempt to quantify the benefits associated with the items listed above, and possibly other potential benefits in addition to irrigation benefits.

3.2 Water Availability and Reservoir Size Optimization

The current enlargement design (WWC, 1996) is sized based on recommendations that were developed by the 1995 Ray Lake Systems Analysis study (NRCE, 1995). The 1995 hydrologic availability analysis estimated the average divertable water for storage in an enlarged Ray Lake as 22,900 acre-feet. This amount of available water supply was derived from both the North and South Fork of the Little Wind River, and the estimate was based on a customized operations
model that simulated available stream flows and existing diversions on those streams that must be satisfied before any water can be diverted for storage. The reservoir was sized to adequately meet an estimated irrigation demand of 16,500 acre-feet annually. Active storage requirements of 26,000 acre-feet were recommended, which would meet the identified irrigation demand 13 out of 16 years, with an average carry-over storage of 10,400 acre-feet. The configuration of the reservoir and delivery canal system also requires an approximately 10,000 acre-feet recreation pool to provide sufficient head to deliver water out of the new outlets. The total storage capacity of the 1995 design is 35,000 acre-feet, which provides 25,000 acre-feet of active storage, approximately 1000 acre-feet less than the size recommended by the systems analysis study. It is unclear why the design size (35,000 acre feet) was slightly smaller than the recommended size (36,000 acre feet) in the 1995 studies.

Preliminary modeling and analyses were carried out as part of this Level II study to provide an independent estimate for the optimum size of an enlarged Ray Lake. The analyses included the following activities:

- Identification of potential water supplies
- Development of a spreadsheet model
- Identification of demand
- Analysis of alternative reservoir and canal sizes
- Recommendations of optimum reservoir sizes

### 3.2.1 Potential Water Supplies

The potential water supplies for an enlarged Ray Lake include four possible sources: early runoff reserved water rights (1868 priority), canal efficiency improvements, present-day priority water rights, and transfer of decreed reserve water rights under Type 7 and Type 8 awards.

Under present reservoir operations, water is generally stored in Ray Lake in March and early April before the heavy irrigation season diversion and deliveries begin. These early-season water diversions have been accounted for as a portion of the 1868 priority reserved water rights decreed to the Tribes which have an annual volume limitation. Recent past reservoir operations have not usually resulted in the volume limitation being reached. Historically, up to 7,000 acre-ft of water was annually diverted and stored in Ray Lake. More recently, the safety of dams restriction has limited storage. Under this first alternative water supply, the proposed operation of an enlarged Ray Lake could continue to divert and store this early runoff water which avoids conflicts with the direct flow diversions (many of which also have 1868 priority) and continue to be accounted for as a portion of the reserved water rights.

The second alternative source of water supply could be derived through the conservation of water within the existing irrigation systems in the Little Wind River drainage by reducing canal seepage and water losses. The efficiency of the gravity flood irrigation systems has been reported to be approximately 27% which is quite low (i.e. 73% of the amount diverted is lost to seepage and related system losses). Previous studies have identified operating measures and system improvements to reduce losses. If the system efficiency could be increased by 10% to 20%, additional water could be available during the historic diversion periods. A portion of
these water savings could be stored in an enlarged Ray Lake.

The third potential source of water supply investigated was to assume a present day priority date for the diversion and storage in the enlarged Ray Lake. Under this alternative any new diversions and storage would not likely be accounted for as a portion of the reserved water rights. A spreadsheet model was put together to evaluate this alternative water supply. The model and the results are discussed in a separate section of this report. Because of the junior status, this water supply would not be available every year and would require the inclusion of carry-over storage space in the enlarged facility. The model study identifies the potential reservoir yields of a present day water right.

The fourth potential source of storage water could be derived through the transfer of the decreed reserved water rights associated with the Type 7 and Type 8 awards, or a small portion of the future project lands that are located within the Little Wind River drainage. These potentially irrigated lands are identified in a separate section of this report. These lands, while not currently in production, have a decreed 1868 priority water right and an annual volume limitation. Early season water could be potentially stored in an enlarged Ray Lake through diversion of these rights. This water would likely be counted against decreed annual volume limitation similar to the first alternative.

### 3.2.2 Present Priority Storage Water Right Yield

An analysis of the potential yield from a present day priority storage water right was performed. A spreadsheet model was developed specifically to evaluate the water supply yields from an enlarged Ray Lake under this alternative and others. As indicated in the model discussion, the water right was assumed to be subject to regulation from the senior priority storage water right (1945) for Boysen Reservoir. This priority limitation would not allow diversions to an enlarged Ray Lake in dry years. The modeling of the Ray Lake enlargement identified the necessity of providing carry-over storage to firm up the water supply yields. The model analyzed different sizes of enlargement, North Fork Diversion Canal capacities, and Ray Canal capacities. A discussion of the model and results follows.

### 3.2.3 Ray Lake Yield Analysis Spreadsheet Model

Reviewing the technical memo prepared in 1995 by NRCE (NRCE, 1995), the preliminary yield analysis did not present a model that allowed for variation of reservoir size and yield to determine optimal properties. The previous model’s apparent thrust was the determination of water needs in the basin, and it did not provide significant water supply information. For this reason, States West prepared a simplified spreadsheet model that uses streamflow gaging data from South and North Fork of the Little Wind River, and the NRCE water usage to model the operations and yields of Ray Lake. The current model permits variation of reservoir size, yield, minimum pool, Lateral 37C Canal capacity (the limiting stretch of Ray Canal), and North Fork Diversion Canal capacity in order to determine the firm yield percentage of Ray Lake. The model also includes the storage of water in Washakie Reservoir with the size and yield also provided as variables. The flow input includes the initial flow information (1976-1992) presented in the NRCE report and augments it with data from 1992 to 2003. This allows the inclusion of the severe drought experienced from 2000-2003. Figure 3.1 is the schematic
flowchart for the model.

After initial review of the model, it was determined that the ability for Boysen Reservoir to “call-out” Ray Lake due to the relative priority dates needed to be addressed. The model accomplishes this task by only allowing water to be stored in Ray Lake after Boysen Reservoir is beyond its permitted storage capacity of 664,908 acre-feet. The end-of-month storage data for Boysen Reservoir was taken from the Bureau of Reclamation website and converted to a single column of data that was directly incorporated into the spreadsheet model. The limiting storage in Boysen Reservoir is a variable in the model to allow differing values to be used. This is a conservative approach, since the priority advantage over a present day storage water right is only for an annual “one-fill” pursuant to Wyoming water law.

3.2.3.1 Assumptions
The creation of the spreadsheet model required several appropriate, yet simplifying assumptions to be made in order to limit the scope of this project element and allow for efficient operation of the model. The assumptions are as follows:

1. There are no water calls from outside the basin, except for Boysen Reservoir. After satisfying all diversions on the Little Wind River as stated in the NRCE report (approximately 28,867 acres and municipal rights), all additional water is available for storage. The model does take into account storage in the Bureau of Reclamation’s Boysen Reservoir. As for other water rights, regulation of the Little Wind River Basin has not occurred even through the recent drought event.

2. Return flows from the private ditches diverting from the North and South Forks of the Little Wind River are available for downstream use. At the stated 27% irrigation efficiency, the remaining 73% is returned with 60% of the return flow occurring the same month it is diverted, 20% occurring in the next month and the remaining 20% during subsequent months. The model assumes all 73% is recaptured to the stream system.

3. If water is available for storage in Ray Lake, it can be diverted and stored. The exception is November through February when the canal is not able to convey any available water to the off-stream reservoir. The model does take into account the “one fill rule” for Ray Lake as provided by Wyoming State Statutes.

4. Although the total municipal demands are approximately 2 cfs, the model subtracts 12 cfs as the diversions. This is due to the fact that the actual municipal pump points require 12 cfs in the river to properly work according to the 1995 NRCE report.

5. The reservoir permits are assumed to be junior to all diversions. All diversions must be satisfied prior to storing water in Washakie Reservoir or Ray Lake.

6. The streamflow record for the North Fork Little Wind River was linearly regressed by NRCE to fill in missing records from 1976 to 1987.
7. Leap year was not considered. February always has 28 days.

8. The water supply from the minor tributaries such as Trout Creek and Mill Creek are not included in the analysis.

9. Releases from Washakie Reservoir have no influence on water available to store in Ray Lake.

10. Full capacity of Lateral 37C is available to deliver water to Ray Lake. This value was included as a variable to allow consideration of alternative canal sizing.

11. The water supply yields from both Washakie Reservoir and Ray Lake are not determined by irrigation need, but only by a set amount to be released which is a variable in the model. Since the model does not contain individual permits, it can not determine when junior rights require supplemental water; therefore, reservoir yields are determined by the maximum amount of water than can be supplied without consideration of actual irrigation need.

12. All calculations are done in acre-feet of water. Initial cfs streamflow rates are converted to acre-ft per month flows.

13. Initial reservoir storage is 50% of capacity.

14. Return flows from downstream diversions located along the mainstem of the Little Wind River are not considered in this model. Since this reduces the amount of available flow to store, the model is conservative in predicted yields.

3.2.3.2 Variables

The model was created to allow flexibility in design of an expansion of Ray Lake. To accomplish this, several properties in the model are set as variables that can easily be altered to see their effects. The variables are as follows:

1. **Ray Lake Capacity** – The active total acre-ft capacity of the reservoir.

2. **Ray Lake Yield** – The amount of water that will annually be released.

3. **Ray Lake Minimum Pool** – Is the minimum storage allowed in Ray Lake. This variable was set to 200 acre-ft resulting in the reservoir capacity equaling the active storage only.

4. **Lateral 37C Capacity** – The deliverable capacity of the supply canal to Ray Lake.

5. **North Fork Diversion Canal** – The deliverable capacity of the supply canal conveying water from the North Fork to South Fork Little Wind River for re-diversion to Ray Lake.

6. **Washakie Reservoir Capacity** – The total acre-ft capacity of the reservoir.
7. **Washakie Reservoir Yield** – The amount of water that will annually be released.

8. **Minimum Boysen Reservoir Storage** – Until this volume in Boysen Reservoir is fulfilled, water is not made available to the junior priority Ray Lake. This assumes the water right to Ray Lake is junior to Boysen Reservoir’s permit. Because this model is relying upon historic Boysen Reservoir storage and operating information, it will present a conservative (lower estimated reservoir yield) results.
Ray Lake Yield Analysis Flowchart

States West Water Resources

USGS Gage 2855
South Fork Little Wind River 1976-2003

Subtract Private Ditches
And Add Return Flows

Yes

No

Can Flow
in North Fork
Satisfy
Downstream
Diversions?

Can Flow
in North and
South Fork
Satisfy
Downstream
Diversions?

No

Yes

All South Fork Flow
Available to Washakie
Reservoir for Filling

Yes

No

Washakie Reservoir
Diverts Available Flow
Until Full

Yes

No

Remainder of Flow
in South Fork Available to
Washakie Res. for Filling

Remainder of Flow
in South Fork Available to
Ray Lake for Filling

No Flow Available to Ray
Lake for Filling

Yes

No

Can Flow in North Fork Satisfy Downstream Divisions?

Can Flow in North and South Fork Satisfy Downstream Divisions?

Yes

No

All South Fork Flow Available for Ray Lake Storage

No Flow Available to Ray
Lake for Filling

Remainder of South Fork Flow Available for Ray Lake Storage

Ray Lake Diverts Available Flow Until Full

Reservoir Yield

Is Ray Lake Filled?

No

Yes

Is Boysen Reservoir Filled?

No

Yes

Is Available Flow Greater Than Canal L37 Capacity?

Canal Capacity Available to Divert to Ray Lake

No

Yes

Is Available Flow Greater Than Canal Capacity?

Canal Capacity Available to Divert to Ray Lake

No

Yes

Canal Capacity Available to Divert to South Fork

All Flow Goes to South Fork Diversion for Ray Lake Diversion

No

Yes

Can Flow in North Fork Satisfy Downstream Diversions?

No

Yes

Can Flow in North Fork Available to Divert to South Fork

No Flow Available to Ray Lake for Filling

Remainder of Flow in
North Fork Available to
Divert to South Fork

Remainder of Flow in
South Fork Available to
Washakie Reservoir
Diverts Available Flow
Until Full

Unable to Deliver Water to Ray Lake for Filling

Determines Percentage of Years that Yield is Satisfied

Ray Lake Diverts Available Flow Until Full

Reservoir Yield

Figure 3.1 Spreadsheet Model Schematic Flowchart
3.2.3.3 Outputs

Based on the assumptions and variables provided, the model calculates the monthly storage in Washakie Reservoir and Ray Lake. The spreadsheet reports the number of years that storage falls below minimum pool in Ray Lake. This number is converted to a percentage of the 27 years of record that the specified yield is achieved. The model also provides the statistics of minimum, maximum and average Ray Lake storage over the period of record. With manual iteration of the model, Ray Lake yields can be determined for various reservoir sizes. The model has been run for 50%, 70%, 80%, 90%, and 100% yields and reservoir sizes of 7,000, 10,000, 15,000, 20,000, 25,000, 30,000, 35,000, 40,000 and 45,000 acre-ft. An 80% yield provides the full requested release for 8 out of every 10 years. The spreadsheets contain the tabular data and graphs for these yield conditions. Figure 3.2 represents the yield analysis with current canal capacities.

Figure 3.2. Yield Analysis for Enlargement of Ray Lake with Existing Canal Capacities

3.2.3.4 Canal Capacity Scenarios

The model was originally run assuming there were no alterations in the existing canal infrastructure. Once this model was completed, various scenarios were run to assess the results on potential Ray Lake enlargements by increasing the Ray Canal and the North Fork Diversion Canal capacities. As figure 3.3 shows, increasing the North Fork Diversion Canal capacity from its current stated 50 cfs to a higher value does not significantly increase the average water supply stored in Ray Lake. Average storage was used in this instance since no change in yields was noted with the variation of the North Fork Diversion Canal.

As for the Ray Canal capacity scenarios, the main assumption made was that the canal would provide the applied capacity from the diversion on the South Fork of the Little Wind River to Ray Lake. The 1995 NRCE report lists the current canal capacity from the river to head of
Lateral 37C as 340 cfs and Lateral 37C’s capacity as 125 cfs. With a large majority of stored water occurring during the months of May and June, it needs to be determined if the existing canal capacities need to simultaneously service the direct flow irrigation deliveries and convey water to the reservoir. This determination was beyond the scope of this modeling effort, but should be addressed in the future modeling effort discussed later in this section. As figure 3.4 illustrates, minimal gain is obtained by increasing the canal delivery capacity to over 200 cfs, but a 60% increase can be obtained by increasing from the current limiting section’s capacity of 125 cfs to 200 cfs. Appendix C contains the yield analyses for the various scenarios considered. This includes Ray Canal capacities of 125, 300, 400, and 500 cfs.

Figure 3.3. North Fork Diversion Canal Capacity versus Average Ray Lake Storage

Figure 3.4. Ray Canal (Lateral 37C) Capacity versus Average Ray Lake Storage
3.2.4 Future Modeling Needs
The creation of this simplified spreadsheet model was completed to illustrate the availability of water to an enlarged Ray Lake. Even with a junior priority water right, the results show that enough water exists during runoff periods to yield an adequate water supply to meet the previously-determined irrigation needs. The next step in the modeling process for the proposed enlargement will be the construction of a deterministic model that details all water usage from the Little Wind River. Some of the current programs in use for this purpose include OPSTUDY, HYDROSS, WIRSOS, and STATEMOD. Each program has its advantages and disadvantages. When deciding on the final program to be used, care must be taken to ensure the needs of the user will be met and regulatory agencies including the US Army Corps of Engineers, the US Bureau of Reclamation, the US Fish and Wildlife Service, and the Wyoming State Engineer’s Office will be satisfied with the results. Since this model must illustrate individual impacts, a stochastic (statistically averaged) model such as the Wyoming Water Development Commission Basin Plan models cannot be used. A majority of the information in the basin plan models can be salvaged and programmed into the new model, but the lack of streamflow data will require simulations to be created that illustrate continuous periods of varying flow.

This model will need to be of such a scale as to allow analysis of individual water rights to assess potential impacts and benefits. This model will also allow for a more accurate analysis of reservoir yield since releases from Ray Lake can then be tied to actual water needs in the basin. Since one of the primary inhibiting factors of the amount of yield derived from an enlarged Ray Lake Reservoir is the storage in Boysen Reservoir, the model may need to address the entire basin associated with the inflows and demands upon Boysen Reservoir. If an earlier or equal priority date to Boysen Reservoir can be obtained, modeling will likely only need to address water rights usage and return flows in the Little Wind River Basin.

3.3 Canal Sizing Design Considerations
In 1995-96 WWC Engineering prepared conceptual designs for alternative plans for inlet and outlet canals to an enlarged Ray Lake Reservoir. This section summarizes the previous work and identifies budget level costs for final design and construction.

3.3.1 Existing Conceptual Designs

3.3.1.1 Inlet Canal
In 1995 and 1996, WWC Engineering prepared inlet canal conceptual designs and cost estimates (WWC, 1996b) for four alternatives. The present inlet canal system is a combination of the upper reach of the Ray Canal and Lateral 37C, as shown on Plate 1. The inlet canal designs were based on sizing recommendations in the Ray Lake System Analysis (NRCE, 1995). Cost estimates for the four alternatives ranged from $2.1 million to $7.6 million.

The recommended design included an enlargement of Lateral 37C to 340 cfs capacity from the Ray Canal to the reservoir (see Plate 1), including the replacement of existing drop structures. According to information at the time, the design did not require improvements to the Ray Canal from the Little Wind River diversion point to the Lateral 37C turnout, which had a reported
capacity of 340 to 420 cfs. The conceptual layout of the inlet canal and the appurtenant structures is presented WWC (1996a). The cost estimate for the recommended project was $2.1 million. The cost estimates include construction costs, allowances for final design, construction administration and contingencies in conformance with standard WWDC cost requirements.

3.3.1.2 Outlet Canals.
In 1995, WWC Engineering prepared outlet canal conceptual designs and cost estimates for two canals that would deliver water to areas not previously served by Ray Lake (WWC, 1995b). The outlet canal designs were based on sizing recommendations in Ray Lake System Analysis report (NRCE, 1995).

South Outlet Canal. The purpose of this canal is to expand the service capacity in Lateral 65C. The South Outlet Canal would convey flow from an enlarged Ray Lake Reservoir to Lateral 65C (See Plate 1). The outlet canal design was based on peak summer demands in July of 60 cfs with the reservoir at 20,000 acre-feet (10,000 acre-feet active storage pool), and 20 cfs based on peak September demands after all active storage is depleted and the reservoir storage is at the minimum recreational pool level (10,000 acre-feet). The conceptual design included a short pressurized pipeline outlet feeding a trapezoidal channel designed to carry 60 cfs. The proposed outlet will be regulated such that water may be delivered to either the South Canal through the pressurized pipe, Mill Creek by an inverted siphon, or both. The conceptual design included two dirt road culvert crossings, wasteway, and a Highway 287 culvert crossing. The conceptual layout of the canal and the appurtenant structure locations are presented WWC (1995b). The South Outlet Canal cost estimate was $900,000 (1995 dollars). The cost estimates include construction costs, allowances for final design, construction administration and contingencies in conformance with standard WWDC cost requirements.

North Outlet Canal. This canal allows expansion of the service area to the north. The North Outlet Canal would convey flow from an enlarged Ray Lake Reservoir to the Coolidge Canal (See Plate 1). The design was based on an approximate peak summer demand in July of 270 cfs when the reservoir has 10,000 acre-feet of active storage and a September delivery requirement of 120 cfs when all active storage has been depleted. The conceptual design included two separate canal segments intercepting at an approximate elevation of 5520. The first segment of approximately 4,000 feet is characterized as a deep cut through the ridge which separates the Mill and Trout creek drainages. The second segment is 21,000 feet long and would run along the side hill parallel to Trout creek to the confluence with the Coolidge Canal. The conceptual design of the second segment included a trapezoidal channel, three dirt road culverts crossing, and a drop structure. The conceptual layout of the canal and the appurtenant structures are presented WWC (1995b). This cost estimate for this project was $2.5 million (1995 dollars). The cost estimate includes construction cost, allowances for final design, construction administration and contingencies in conformance with standard WWDC cost requirements.

3.3.2 Inlet Canal - Final Design Considerations
The reservoir sizing work presented in section 3.2 makes recommendation for an inlet canal size
of 200 cfs for storage water. This is the amount that would be needed in addition to direct flow irrigation water delivery (below the reservoir and downstream in the Ray canal) that might be occurring concurrently. Typically, peak stream runoff and diversion of flow to Ray Lake would occur in May and early June in any given year. The reservoir outlet canals were sized to deliver peak irrigation flows in July, which would typically be well past the peak of diversions to Ray Lake. At this time, the timing of firm irrigation requirements and timing of river diversion to Ray Lake are too conceptual to make a recommendation on how large the inlet system should actually be. Based on our current understanding, the conceptual design capacity of 370 cfs appears to be adequate to meet the needs of reservoir filling and simultaneous irrigation.

3.4 Preliminary Reservoir and Canal Sizing Summary

Needs analysis by previous studies (NRCE, 1995 and SEH, 2001), summarized in Section 3.1 of this report, indicate that there are identified needs on the order of 16,500 to 30,000 acre-feet per year. This is based solely on existing un-met and future potential irrigation demands. Additional beneficial uses for an enlarged storage facility also are identified (Section 3.1.4).

Previous system analysis studies (NRCE, 1995), and the current model study presented in Section 3.2 of this report, indicate that a Ray Lake enlargement size of at least 35,000 acre-feet (25,000 acre-feet active storage and 10,000 acre-feet recreation pool for proper system operation) is viable based on water supply and conservative (present-day) water rights assumptions.

To adequately supply the enlarged Ray Lake Reservoir, the North Fork Diversion Canal should be upgraded to carry an additional 50 cfs of water for storage, and the Ray Canal and Lateral 37C upgraded to carry 200 cfs of water for storage. These capacities would be in addition to canal capacity required for simultaneous delivery of direct flow irrigation water.

This combination of reservoir size and supply canal size could deliver an average supplemental water demand of 16,500 acre-feet in 9 out of 10 years. Increasing the active storage capacity above 25,000 acre-feet would improve the ability of the project to meet the supplemental water demands. More detailed system modeling is recommended to provide more accurate analysis of reservoir yield by tying releases from Ray Lake directly to actual water needs in the basin. Since one of the primary inhibiting factors of the amount of yield derived from an enlarged Ray Lake Reservoir is the storage in Boysen Reservoir, the model may need to address the entire basin associated with the inflows and demands upon Boysen Reservoir.

3.5 Geotechnical Considerations

3.5.1 Dam Safety and Enlargement Design Issues
As part of this study, Gannett Fleming reviewed the basis of design for both the safety of dams (SOD) repairs and the enlargement design. A primary motivation for reviewing the SOD design was to evaluate the feasibility of modifying the repair concept for the existing outlet works such that the repaired structure could be later incorporated into the south outlet concept for enlargement design. Gannett Fleming concluded that this was not feasible, primarily due to
weak foundation conditions. This section provides some background and the basis for the SOD design of the outlet repairs, and the rationale for why this design cannot be reasonably modified to be incorporated into an enlarged project.

3.5.1.1 Existing Outlet Condition and Repair Design
The existing outlet works is in very poor condition. The 4x4 feet square reinforced concrete box conduit has a major crack upstream from the centrally-positioned gate housing. USBR temporarily repaired the crack with a sealant in 1984. There are numerous other cracks in the conduit downstream from the gate. The concrete is badly deteriorated due to sulfate attack and possibly reactive aggregate problems. The gate leaks and has a bad seal. There is no trash rack and large debris had accumulated in the pipe which was cleaned out during the 1984 crack repair. Concrete in the downstream energy dissipater structure is so badly deteriorated that the reinforcing steel and embankment are exposed behind the walls. There is strong evidence that before the upstream crack was sealed, significant seepage had occurred on the outside of the pipe, emerging from behind the downstream head wall and left wing wall of the energy dissipater structure.

GEI (1993) recommended complete removal and replacement of the outlet works in their feasibility design of SOD repairs. USBR reviewed that design in a Value Engineering Study, and offered an alternative to slip-line the upstream portion of conduit, plus replace the gate and downstream energy dissipater structure, and replace the intake with a new intake that includes a trash rack. The current SOD design by WWC (1995) calls for a full-length, 48-inch steel slip liner through conduit, plus replacement of the existing central gate, installation of an upstream gate and trash rack, and complete removal and replacement of the downstream energy dissipator structure. This design avoids removal of the embankment, and shortens the construction time and minimizes loss of irrigation supply. This design has been reviewed and essentially approved by USBR with some minor modifications. BIA provided funds to the tribes for constructing the outlet repairs and other dam safety corrective measures in 1996. This work has not been completed as of the date of this report (May 2005).

3.5.1.2 Peer Review Comments on the SOD Design for the Outlet Works
Gannett Fleming reviewed the SOD design as it is currently presented on the 1995 drawings. **We strongly caution that the design and installation of the downstream filter zone around the slip-lined pipe is a critical element of the repair that deserves special attention to ensure proper gradation design of the filter to be compatible with the embankment soil, and care during construction to ensure that it is properly placed around the outside of the pipe without segregation.** Our main concern is that, with the slip-liner in place, pore pressures are likely to increase in the downstream zone of the dam, as illustrated in Figure 3.5. Under current conditions, when the gate is closed, seepage through the embankment is probably draining into the downstream section of the pipe through numerous thin cracks. Readings on the two piezometers that were installed by USBR in 1984 seemed to indicate that this was, in fact, occurring. Once the lining is installed, however, seepage pressure will build up around the outside of the pipe. An additional concern is that the embankment has probably already been compromised and weakened in the vicinity of the pipe due to the long-term internal seepage that has occurred in the past. We are aware of at least one dam (Tin Cup Dam in Montana) that
failed due to downstream slope instability following slip-lining. The downstream filter zone around the pipe is a critical design element that deserves special attention.

Figure 3.5 Slip-liner effect on pore pressure in an embankment dam
(courtesy of Hal van Aller, Maryland Dam Safety Division)

3.5.1.3 SOD Outlet Design Incompatibility with Enlarged Dam Outlet Design
If the dam is enlarged to the extent that is now being considered (approximately 35,000 acre-feet; 70 feet high dam), a new outlet conduit will be constructed at the same location as the existing structure. Consideration was given to a downstream raise of the dam that could involve an extension of the existing (rehabilitated) conduit. As a possible alternative, the Gannett Fleming team also considered the option of revising the SOD design to remove and replace the outlet section of the existing dam in order to install a new conduit that could then be tied into the extended outlet conduit at a later date when the dam was raised. The obvious objective of this evaluation was to look for opportunities to apply a portion of the rehabilitation costs towards a future enlarged facility.

Review of the geotechnical information, plus other considerations, has led our team to the conclusion that the SOD design cannot be reasonably adapted to be incorporated into a later enlarged project. This is based on the following considerations:

- The foundation in the vicinity of the existing outlet is weak. The characteristics of the cracking patterns in the existing box conduit suggest that cracking was caused by settlement of the soft foundation under the embankment loading. The foundation for the conduit under the raised dam should be stripped down to remove the soft residual soils and weathered bedrock on which the current pipe is now founded in order to found the new pipe on stiffer, unweathered bedrock, about 6 to 10 feet deeper. If the SOD pipe design were to be made compatible with the raised dam design, a large cut in the
existing embankment would be needed to remove the old conduit and extend the stripping depth.

- Dealing with the soft, saturated sediments upstream from the existing dam, and dewatering of the excavation for the extended stripping depth cut, could prove to be costly and would likely require temporary construction cutoff and retaining systems such as sheet piling.

- Gannett Fleming is recommending two significant modifications to the existing enlargement design, as described in Section 4. One of these recommendations is to shift a portion of the south embankment alignment downstream such that the new dam footprint is entirely positioned downstream from the existing dam. The existing dam could thus serve as a cofferdam to allow uninterrupted irrigation water delivery during the construction period. With this shift in the alignment, the existing rehabilitated pipe would extend beyond the upstream toe of the new dam, and would, therefore, not be logically incorporated as part of the new outlet structure.

- There remain significant issues that must be resolved before the enlargement project can be advanced, including uncertainties about project funding, supplemental geotechnical investigation requirements (see next section), and environmental/permitting issues. It may be several years before these issues are resolved and the enlargement project is finally realized. In the meantime, the existing dam safety issues are paramount. The rehabilitation work should be completed as soon as possible to correct the dam safety deficiencies because the seepage and concrete deterioration problems will continue to worsen with time.

3.5.2 Bureau of Reclamation Concerns Regarding 1997 Enlargement Design

There remain significant geotechnical uncertainties regarding the foundation and embankment design, based on documented concerns expressed by the USBR reviews of the 35,000 acre-feet project. Because this is a serious issue, and represents a potential roadblock to moving the project forward either as it is currently designed, or as modified enlarged project, Gannett Fleming recommends that a supplemental geotechnical investigation program be implemented as part of a follow-on Phase II, Level II study to address this key deficiency. The key outstanding geotechnical issues and our proposed program are described in the following section.

**Geotechnical Issues.** A significant concern that must be addressed before the enlargement project can move forward is the adequacy of the geotechnical information that was the basis for final design by WWC (WWC, 1996a). The USBR served in a technical review capacity on behalf of the BIA throughout the final design reporting process. USBR’s geotechnical engineers raised concerns regarding the adequacy of the geotechnical investigations that had been performed in several phases by Hadley & Hollingsworth (H&H) under subcontract to WWC (Hadley & Hollingsworth, 1994, 1995a, 1995b, 1995c, 1996). Specifically, the major issues of concern as we understand them were as follows (USBR, 1996a):

- **Insufficient geologic mapping and information:** The geotechnical reports and design memoranda submitted by H&H did not present fundamental information such as:
• no site-specific field geologic mapping, or description of jointing, or measured dip and strike of the bedrock units;
• no drill hole logging, or test pit logging of lithologic variations which could indicate potential pervious or weak seams in the bedrock.

- **Insufficient field permeability data:** H&H used laboratory test specimens to evaluate foundation permeability and assumed isotropic values in their analyses. USBR was concerned that without knowledge of the presence or absence of pervious seams in the foundation, and without in-place permeability measurements in boreholes, there was insufficient information to determine the adequacy of the seepage collection and cutoff systems.

- **No exploration data along relocated highway alignment.**
- **No exploration data to support large cut slope stability for the North Outlet Canal.** USBR was particularly concerned that potential adverse dipping beds (based on review of published information) in the area could present “significant stability problems” in the deep cut slope on the north side of the reservoir.
- **No adequate documentation of absence of dipping, weak bentonite beds:** USBR pointed out that their literature reviews indicated the Cody Shale in the project area could contain bentonitic layers that could cause unstable foundation conditions. They were concerned that the sampling procedures used by H&H and lack of detailed geologic mapping of exposures could have obscured or missed evidence of these features.

In addition to their concerns regarding the adequacy of the geologic and geotechnical field and laboratory investigation program, USBR also expressed specific concerns regarding the geotechnical design of the embankment and other features. Their concerns included the following:

- Use of materials vulnerable to cracking near the dam crest
- Inadequate crest width
- Uncertainty about suitable fill materials from south abutment excavations
- Uncertainty about adequacy of camber design
- Incorrect assumptions of anisotropy for seepage analyses and drain/cutoff design
- Potential for adverse seepage in north abutment ridge – no seepage cutoff or control measures in design
- Need for deep filtered drain at downstream toe
- Chimney drain width too narrow
- Internal filter does not meet filtering criteria
- Toe drain slope too flat
- Questions regarding slope stability analyses and shear strength testing and assumptions
- Need for deformation analyses under earthquake loading

The issues raised by USBR were based primarily on their review of the 35% design report submittal and were documented in a series of letters and responses from H&H. The latest letter on file is dated December 20, 1996, and is a review by USBR of a proposed supplemental exploration program developed by H&H to partially address USBR’s concerns (USBR, 1996b).
USBRe formulated an example Geological/Geotechnical Exploration and Testing Program, and prioritized activities in Phases of work. In particular, they emphasized as critical priorities the need for geologic mapping using topographic coverage at 1 in = 50 ft, 2 ft contour interval base maps. Detailed logging of drillhole core samples in general accordance with USBR’s field manuals was recommended. USBR’s review of H&H’s proposal for the supplemental investigation noted that the amount of proposed laboratory testing was excessive for the first phase of work, at the expense of the field work which they considered higher priority.

3.5.3 Supplemental Geotechnical Investigation Program Recommendations

Based on our site reconnaissance and detailed review of the available geotechnical data, including the extensive USBR technical comments, Gannett Fleming concludes that the additional geologic and geotechnical information is required for design of the enlargement project. A supplemental geotechnical investigation program is necessary to identify any potential fatal flaws or significant geotechnical constrains that could impact the feasibility or construction costs. The subsections provided below provide (1) an overview of the known geologic conditions, (2) summary of key geotechnical concerns, and (3) recommended supplemental surface and subsurface geologic/geotechnical investigations for project design.

3.5.3.1 Geologic Conditions

The project site is located in the Wind River Basin. The Wind River Basin is a large structural and sedimentary basin that formed during major deformational events associated with uplift of the Rocky Mountains. The basin is surrounded by folded and faulted strata that forms the flanks of the adjacent mountain ranges. Regional geologic maps and structural contour maps of the region indicate that the site is underlain by Cretaceous Bedrock that dips towards the northeast (away from the Wind River Mountains). The bedrock consists of a sequence of interbedded shale, siltstone, and sandstone (Cody Shale and Frontier Formations) that regionally contain thin interbeds of weak bentonite clay that can be subject to instability. The sandstone and siltstone beds are permeable and should be evaluated as potential seepage pathways. This site is also situated in a seismically active region capable of producing moderate earthquakes that need to be considered in the geotechnical design.

Our geologic reconnaissance of the existing dam and reservoir area indicates that the bedrock underlying the site is exposed locally along the margin of the reservoir. The bedrock consists of a thinly interbedded sequence of shale, siltstone and sandstone that is weak to friable with a low hardness. The bedrock structure, or bedding planes within the sequence, dip towards the east to northeast (or downstream) at 10 to 16 degrees. The downstream dipping sequence and potential for weak bentonite beds indicates that there could be issues associated with potential sliding plane failure at the site. The sequence also contains occasional high angle fractures that have visible openings where the bedrock is deeply weathered. The upper several feet of bedrock is generally deeply weathered with soil like properties and relic bedrock structure. The weathered bedrock is covered by up to a couple of feet of residual soil that contains dessication cracks and other features suggesting the soils likely have some expansion potential.

In summary, review of existing geotechnical information previously relevant to the site indicates that significant geotechnical constrains for development need to be further evaluated prior to
design, particularly the potential for the dam foundation rocks to contain the following:

- Weak, adverse dipping clay (bentonite) beds;
- Seepage pathways (sand beds, or open fractures);
- Materials susceptible to internal erosion or dissolution.

The following outlines our recommended supplemental investigations to support the project advancement.

### 3.5.3.2 Engineering Geologic Mapping

The purpose of the surficial engineering geologic mapping (and subsequent subsurface investigation) is to identify any potential geologic hazards and define the site engineering geologic conditions for the embankment areas, emergency spillway, outlet and inlet canals, and reservoir area. Mapping will also assist in better defining potential borrow source areas for use in dam construction.

The scope of work for this subtask should consist of performing detailed geologic mapping of the dam and reservoir area. Except for isolated locations around the existing reservoir margin, the bedrock in this area is covered by surficial soils. For this reason, a trackhoe should be used to excavate shallow test trenches (up to 15 feet deep) to examine the soil and bedrock conditions (including bedrock structure) concealed beneath the surface soils. Additional information on the test pit logging methodology is provided in the following paragraphs. The test pits combined with surface mapping provide the most efficient method for mapping the geologic conditions across the project area. Key features to be mapped include geologic contacts, bedrock structure (i.e. bedding plane and fracture orientations, and fold, faults and shears), and surficial geologic units (such as colluvial and alluvial sediments). It is anticipated that mapping of the key components of the dam, spillway will be performed at a scale of 1 inch = 100 feet, or larger. Engineering geologic mapping of the reservoir areas, supply canals, and inlet outlet canal should be carried out at a scale of 1 inch = 400 feet, or larger. Reservoir area mapping should focus on identifying (1) areas susceptible to future slope instability, and (2) borrow materials.

### 3.5.3.3 Subsurface Geological Conditions

The subsurface conditions in the north and south embankments, emergency spillway, north outlet/inlet canal excavation, and borrow areas will be explored by a combination of drilling and in-situ testing, shallow backhoe test pits, and laboratory testing of select samples. The subsurface exploration program is summarized by project component in Table 3.2. Exploration methodologies are described below.
Table 3.2 Recommended Level of Effort for Supplemental Subsurface Investigation Program  
(refer to Plate 2 for locations of design features)

<table>
<thead>
<tr>
<th>Location/Feature</th>
<th>Approx.Max. Height or Depth</th>
<th>Length (ft)</th>
<th>Width (ft)</th>
<th>Drilling</th>
<th>Test Pits</th>
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<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>No. Borings</td>
<td>Depth Range</td>
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<tr>
<td>North Dam</td>
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<td>9500</td>
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<td>10</td>
<td>20-110</td>
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<tr>
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<td>10-20</td>
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<tr>
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<td>50’ cut</td>
<td>2000</td>
<td>1500</td>
<td>3</td>
<td>25-50</td>
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<tr>
<td>Spillway</td>
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<td>5000</td>
<td>125</td>
<td>3</td>
<td>20-40</td>
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<td></td>
<td>3000</td>
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<td>2</td>
<td>20</td>
</tr>
<tr>
<td>Borrow Area</td>
<td></td>
<td>3000</td>
<td>1000</td>
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</tr>
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<td>#1</td>
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<td>Total</td>
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<td>31</td>
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Wyoming Water Development Commission
**Exploration Drilling.** Detailed geologic logs for the previously drilled borings were not provided in the geotechnical reports prepared by Hadley & Hollingsworth. In addition, the borings were intermittently sampled, and critical in situ permeability tests were not performed. Therefore, supplemental drilling should be performed in the dam foundation area to characterize the subsurface conditions, particularly the presence, location and orientation of bentonite clay beds, beds containing soluble minerals or erodable materials, and permeable zones (i.e. sand beds or lenses, or open fracture zones). Field permeability testing (constant head packer test, or gravity falling head tests) should also be conducted in all bedrock core holes drilled along the dam alignment at approximate 10-foot vertical intervals as the hole is advanced, using a single packer system. The results of these water tests should be used to characterize the permeability of the bedrock in the dam foundation and to identify and map high permeability zones. Additional drilling may also be required to evaluate the physical characteristics of the bedrock to be encountered in the south shore excavation, and excavations for the emergency spillway, north outlet/inlet canal, lateral 37C relocation, and for dam borrow material.

The drilling would likely be performed using a combination of hollow stem auger and rotary wash drilling techniques. Where unconsolidated sediments such as residual soil, alluvium, or colluvial material cover the bedrock, the borehole would be initiated by drilling with 8-inch (OD) hollow stem augers, with continuous drive samples or Shelby tube samples collected as appropriate for the material types encountered. Once bedrock is reached, drilling and sampling would continue to final depth with continuous sampling using coring techniques. Our experience has shown that continuous coring using the HQ triple-tube system provides the highest level of recovery possible even if the bedrock is weak, highly fractured, or poorly consolidated.

The drilling conditions and recovered core should be carefully logged by a qualified engineering geologist as drilling proceeds (and prior to disturbance of the core by placement in core boxes). The lithology, bedding characteristics and orientation, texture, hardness, strength, cementation, degree of fracturing and weathering, and Rock Quality Designation (RQD) should be recorded on the core logs. Any unusual conditions encountered during drilling, such as water loss zones, voids, or soft zones encountered during drilling will also be recorded on the drill logs. Core samples should be preserved in core boxes that indicate the core hole and depth interval. Soil samples collected during drilling of unconsolidated material should be properly labeled and preserved as possible samples for laboratory analysis.

**Test Pits.** A trackhoe is recommended to investigate the shallow subsurface conditions (generally less than 15 feet) at selected locations in the dam foundation and spillway area, and along the supply canal and outlet canal; and at the diversion structure. The backhoe excavations will generally consist of shallow test pits or trenches up to 15 feet deep. The excavations will allow for removal of surface soils for examination of the character and competency of shallow bedrock materials. Test pits will also allow for determination of bedrock structure (strike and dip) in areas of interest where the bedrock is covered by surficial soils. Test pits would also be used to evaluate the general depth of alluvial, colluvial (slope deposits), and shallow landslide deposits, as necessary. The material exposed in the test pits or trenches will be carefully logged.
by an engineering geologist who should also collect representative samples for soils classification and laboratory testing.

**Borrow Material Investigation**
Supplemental investigations will be performed to further characterize and quantify the availability of suitable materials for dam construction. Based on our preliminary understanding of the geology of the area, it is likely that suitable clayey soil materials are available on site to construct an embankment dam that would be highly resistant to earthquake damage. Potential sources of materials for impervious zones will be mapped and sampled as necessary to supplement the existing geotechnical data. Primary sources of borrow material include planned excavation areas, and previously identified borrow source areas within the reservoir inundation area. Alluvial materials, and possibly other sources of granular materials such as gravel terrace deposits will be investigated as resources for filter, drain, and shell materials in the embankment, as well as possible concrete aggregate sources. Representative samples will be collected from test pits excavated in potential borrow source areas. Bulk samples will be collected, and the test pits logged to document the characteristics and variability of the materials, depth to water table, and other relevant information. Information provided in published geologic maps and reports for the area suggest that suitable riprap materials are unlikely to be present in the near vicinity of the dam and reservoir sites. A geologic reconnaissance will be performed to identify sources of hard bedrock materials for riprap located within reasonable transport distance.

**Laboratory Testing.**
Table 3.3 summarizes the number and types of tests anticipated to evaluate the foundation materials. Soil and rock core samples recovered from the foundation of the main dam should be tested for engineering index properties, strength, and compressibility. Soil and rock materials from cut slope areas should be tested for index properties and strength, and bulk soil samples recovered from the borrow site exploratory trenches will be tested for engineering index properties, compaction characteristics, remolded shear strength, compressibility, and dispersivity. Testing of soils relevant to potential concrete aggregate problems will also be done to address USBR concerns.

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**Table 3.3  Laboratory Tests for Dam Foundation, Cut Slope and Borrow Area Materials**
<table>
<thead>
<tr>
<th>ASTM Designation</th>
<th>Test Description</th>
<th>Number of Tests Anticipated</th>
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</thead>
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<td>Soil Moisture</td>
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</tr>
<tr>
<td>D422</td>
<td>Grain Size Analysis (GSA) to −200 sieve</td>
<td>20</td>
</tr>
<tr>
<td>D422/D854</td>
<td>GSA &amp; Specific Gravity w/Hydrometer</td>
<td>20</td>
</tr>
<tr>
<td>D4318</td>
<td>Atterbergs</td>
<td>35</td>
</tr>
<tr>
<td>D4767</td>
<td>Consolidated, undrained triaxial shear</td>
<td>7</td>
</tr>
<tr>
<td>D4644</td>
<td>Slake durability</td>
<td>3</td>
</tr>
<tr>
<td>D5607</td>
<td>Direct shear</td>
<td>15</td>
</tr>
<tr>
<td>D3148</td>
<td>Unconfined compression</td>
<td>30</td>
</tr>
<tr>
<td>D4221</td>
<td>Double hydrometer (dispersivity test)</td>
<td>6</td>
</tr>
<tr>
<td>D698</td>
<td>Standard Proctor compaction</td>
<td>6</td>
</tr>
<tr>
<td>D2435</td>
<td>Consolidation test</td>
<td>6</td>
</tr>
<tr>
<td>HACH method</td>
<td>Corrosion tests for sulfate and resistivity</td>
<td>3</td>
</tr>
</tbody>
</table>

### 3.5.3.4 Seismic Evaluation

The Upper Wind River Basin is characterized by a low level of historic seismicity. The largest recorded earthquake to affect the region was the Magnitude 5.0 Lander earthquake that occurred in 1984 (USBR, 1988). However, even this moderate earthquake is believed to have resulted in cracking at the Worthen Meadows Dam located in the Wind River Mountains southeast of the site. There are no known active faults in the project area. The closest known active faults are the Cedar Ridge/Dry Creek fault and the Stagner Creek Fault located in northern portion of the Wind River Basin (USBR, 1988). The supplemental geotechnical investigation should include a site-specific review of potentially active faults, and estimation of potentially damaging ground motions parameters for dam design. The scope of work should include compiling and reviewing available information on active and potentially active faults, and seismicity for the region. This information will be used in a seismotectonic evaluation to determine the design earthquake and peak ground acceleration (PGA) for use in the stability analysis. All known active and potentially active faults in the region should be reviewed with respect to proximity to the site, and estimated Maximum Credible Earthquake (MCE). Historic earthquake epicenters that have occurred in the region should be compiled and reviewed based on information derived from the USGS National earthquake Information Center.

The PGA for the dam site should be determined based on available seismotectonic information developed for the site and surrounding area. Two different types of earthquakes are typically considered in the analysis of the PGA for dam design: (1) the estimated Maximum Credible Earthquake (MCE) generated along known active faults in the region, and (2) the background MCE that could occur anywhere in the region as a random earthquake, not associated with a recognized active fault. The random event will be determined based on the seismic record for the region and probabilistic estimates developed by the USGS. All relevant sources of earthquakes (active and potentially active faults) that could impact the site should be considered
in the evaluation. For each MCE, the PGA should be determined based on applicable attenuation relationships. These are empirical relationships developed from recorded earthquake events that relate magnitude and distance to PGA. As a check, the estimated PGA determined for the site should be compared to probabilistic estimates of PGA for the region developed by the USGS. The results of this analysis will provide a site-specific estimate of the PGA for use in evaluation of embankment stability under seismic loading.

3.5.3.5 Supplemental Geotechnical Investigation Report

The results of the surface and subsurface investigations, borrow materials investigation, and seismic evaluation should be presented as a supplemental engineering geology and geotechnical report. This report should describe the geologic and geotechnical conditions and potential geologic hazards identified during review of existing data, and geologic mapping and subsurface investigation as outlined above. Interpretive cross-sections should be prepared to correlate and show the variation of materials from the right to left abutment and from upstream to downstream at the dam site. Additional cross-sections should be constructed to illustrate the subsurface conditions at each of the planned cuts. The location of any potentially problematic zones detected during drilling such as adverse dipping clay beds, seepage zones, or unstable zone should be shown on the cross-sections.

The report should include (1) a detailed engineering geologic map for the dam site, and reconnaissance level engineering geologic map for the reservoir and canals; (2) interpretive geologic cross-sections of the dam foundation areas, and excavations; (3) description of the geologic conditions in the dam foundation areas, outlet works, spillway areas, canals, and reservoir areas, (4) description of availability of construction materials (embankment fill, filter/drain and aggregate, and riprap materials) and (5) a discussion of geotechnical constraints, geologic hazards, and geotechnical considerations for design. The report should also include conclusions and recommendations for design measures to address geologic hazard or geotechnical constraints that may be identified during the course of these supplemental investigations.

3.5.4 Cost Estimate for Supplemental Geotechnical Investigation

The estimated cost for the supplemental geotechnical program outlined above is presented on Table 3.4.

Table 3.4 Estimated Costs for Supplemental Geotechnical Investigation Program

<table>
<thead>
<tr>
<th>Investigation Component</th>
<th>Estimated Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Labor (geologic mapping, subsurface investigations, analyses, reporting)</td>
<td>$99,700</td>
</tr>
<tr>
<td>Drilling and Test Pitting</td>
<td>$85,900</td>
</tr>
<tr>
<td>Laboratory Testing</td>
<td>$27,000</td>
</tr>
<tr>
<td>Total</td>
<td>$212,600</td>
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4. RECOMMENDED CHANGES FOR FINAL DESIGN

4.1 Separate Dam Enlargement and Highway Relocation Projects

The 1996 Ray Lake Dam Enlargement concept (WWC, 1996a) incorporated Wyoming State Highway 287 on top of the dam embankment. The justification for this design was believed at the time to be the least-cost configuration for constructing the dam and the highway (P. Rechard, 2005, personal communication). The current (2005) review of the 1996 concept has explored the costs and identified design concepts that would simplify construction and contracting, and improve operation, maintenance and safety of the facility, at similar project costs. The following points summarize the rationale for considering this design modification.

- The 1995 design has dam outlet works operating facilities for the North Outlet in close proximity to the traveled way of the highway. The current study team believes that there should be significantly more operation and maintenance room provided at this outlet structure. Secondly, from a safety perspective, it would be desirable if the approach to this operating area were not located adjacent to the highway. The Wyoming Department of Transportation (WYDOT) discourages the construction of such approaches off state highways.
- The 1995 design envisioned a multi-construction season project, which would require the installation of a permanent highway detour around the embankment construction. The 1995 design estimated the cost of this detour at $1 million.
- The 1995 design dam embankment section includes an 88 foot road section on top of the North Embankment segment of the combined dam/highway section. This is substantially larger than the width needed for an embankment which does not include a state highway. The potential cost savings of constructing a narrower dam embankment was also considered in the following comparative analysis.

4.1.1 Option 1 – 1995 Design with Combined Road/Dam Embankment

The 1995 plans prepared by WWC Engineering merged the north embankment of the reservoir and State Highway 287, forming a single embankment to serve a both the dam and highway embankment. The total length of roadway reconstruction is approximately 1.53 miles.

Advantages:
- One location for all environmental clearances
- Combined right-of-way for dam and roadway
- Lower total construction cost

Disadvantages:
- Detour, for at least 2 construction seasons, construction & removal. This will be a very significant detour and will be required to meet all the same criteria of the final roadway. It is not clear in the existing plans where this detour was to be located.
• Higher construction cost/ mile
• Anticipate greater wetland conflicts than Option 2
• Complications for highway and dam maintenance, i.e., access to causeway manhole, outlet works, embankment riprap etc.
• Recommend guardrail for this option due to safety concern along reservoir, this was not accounted for in original design

4.1.2 Option 2 – Separate Dam and Highway Projects
This option re-aligns the highway northeast of the existing centerline. There are two roadway alignments currently under consideration, as shown on Figure 4.1. (Note, figures for this section are attached at the end of Section 4). Alternate 1 has less total acreage impacts, but impacts more wetlands. The total length of roadway for Alternative 1 is approximately 2.3 miles, and approximately 3.6 miles for Alternative 2.

Advantages:
• No safety concern due to reservoir
• Separation of design/ construction schedules for each project
• Separation of maintenance for highway and dam
• No construction detour required, use of the existing roadway could be maintained during the construction at the new location
• Lower cost per mile
• Reduced outlet pipe length requirements for the North Outlet Works pipe (from 604.5 feet to 555 feet), and the Lateral 37C culvert pipe (from 625 feet to 585 feet)
• Reduced dam embankment volumes for the north portion of the dam

Disadvantages:
• Higher total construction-related costs (relative to cost savings associated with combined road and dam construction such as construction oversight, mobilization, etc.)
• Higher total miles of reconstruction
• More right-of-way acquisition required
• More survey data required
• Environmental clearances for separate corridors

4.1.3 Option 1 & 2: Preliminary Engineering Recommendations
Review of the roadway reconstruction design developed for the 1996 Ray Lake Dam Enlargement Plans indicates that they were not prepared to WYDOT standards and did not go through WYDOT’S required review process. Regardless of the alignment location selected in final design, reconstruction plans for State Highway 287 will be required by WYDOT to go through the same design and review phases as they require for their construction projects. This will include the following preliminary engineering work and submittals:
• Preliminary Engineering
• Survey Data – Aerial or Field data to WYDOT criteria
• Environmental Clearances & Inventories– Wetlands, Archeological, T & E
• Geotechnical Investigation
• Design Plan Submittals
• Preliminary Plans
• Grading Plans
• R/W & Engineering Inspection Plans
• Title search & Official R/W Plans
• R/W & Utility Plans
• Final Plans

All geotechnical investigations, material testing, surfacing recommendations, geology recommendations, environmental inventories, designs etc. will be required to go through a review process by the applicable WYDOT departments.

4.1.4 Construction Cost Comparisons for Highway Realignment Options

Option 1: The cost of many of the roadway items (such as embankment construction) in the 1995 plans are incorporated with the dam construction estimate. The 2006 estimate provided on Table 4.1 assumes that these items are accounted for in the dam construction estimate and are not accounted for in the 2006 cost. Construction unit costs developed in the 1995 study were escalated for this report using the WYDOT data base (BIDTABS) program for unit prices. The total length of roadway reconstruction for this option is approximately 1.53 miles.

Option 2: The estimates provided for option 2 alternatives should be considered feasibility-level only. Detailed quantities and unit costs were not developed for these alternatives at this level of study.

Construction Engineering Costs: No cost estimate had been previously evaluated for construction engineering. Option 1 would be able to combine some of the construction engineering for the dam with that of the roadway, and Option 2 would require additional construction seasons, so it is anticipated that Option 2 would have higher construction engineering costs.
Table 4.1  Comparison of Costs for Highway Realignment Options

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Preliminary Engineering (15%)</td>
<td>-</td>
<td>$330,000</td>
<td>$390,000</td>
<td>$600,000</td>
</tr>
<tr>
<td>Construction</td>
<td>$1,775,000</td>
<td>$2,200,000</td>
<td>$2,600,000</td>
<td>$4,000,000</td>
</tr>
<tr>
<td>Construction Engineering (10%)</td>
<td>-</td>
<td>$220,000</td>
<td>$260,000</td>
<td>$400,000</td>
</tr>
<tr>
<td>TOTAL ESTIMATE</td>
<td>$1,775,000*</td>
<td>$2,750,000</td>
<td>$3,250,000</td>
<td>$5,000,000</td>
</tr>
</tbody>
</table>

*Costs for engineering and construction not included

4.2  Shift Enlarged Embankment Alignment Downstream

Another design modification that is recommended by the Gannett Fleming team is to shift the 1995 dam alignment downstream where the raised dam is constructed adjacent to the existing dam. Figure 4.2 shows the embankment section at the location of the south outlet works. Plate 2 (provided in the map pocket), shows the footprint for the recommended revised dam alignment, including this realigned southern segment, and the revised northern segment alignment with the highway section relocated off the dam (as discussed in Section 4.1). The advantages and disadvantages of shifting the raised dam downstream from the existing dam are as follows:

Advantages include:
- The existing dam would serve as a cofferdam during the construction period, retaining water supply storage, as well as the soft reservoir sediments and stormwater runoff from the upstream drainage area which will facilitate construction.
- Current reservoir water supplies can continue to be delivered without interruption over the anticipated 2 year construction period via a temporary bypass constructed through the new south outlet works.

Disincentives for this option are that a somewhat larger new disturbance area will be created in the dam footprint of the south embankment reach (adjacent to the existing dam), and more extensive foundation dewatering may be required during construction. These disadvantages are considered minor.

4.3  Embankment Design Modifications

Based on a review of the 1995 design and USBR comments regarding the embankment design, Gannett Fleming recommends the following modifications to the embankment:
• Widen the dam crest to 24 feet wherever the embankment height exceeds 50 feet, which is approximately between dam stations -5+00 to 15+00. This wider crest will meet minimum criteria per USBR guidelines. Figure 4.2 (attached at the end of this Section 4) shows the modified dam section with the widened crest (see Plate 2 for stationing on modified embankment alignment).

• Narrow the dam crest to 20 feet on the north dam segment (north of Station 15+00), with the highway section relocated as described in Section 4.1. Figures 4.3 and 4.4 illustrate the reduced section at the location of the north outlet works and near Station 10+54, respectively, on the new alignment.

• Install a full blanket drain under the downstream zone between stations -5+00 to 20+00 (approximately), in place of the finger drains on 50-feet centers as currently shown on the design plans.

• Based on the findings of the supplemental geotechnical investigation (outlined in Section 3.5.3), additional foundation excavation, or positive deep cutoff may be required between approximately Stations -5+00 to 20+00, and in the vicinity of the North Outlet works. Recommendations for final foundation treatment cannot be made until the geotechnical investigations and analyses have been completed.

4.4 Outlet Conduit Modifications

The geotechnical information and cracking of the existing conduit indicates that the foundation and embankment fill materials are compressible. This indicates that the outlet conduits may experience significant deformation under the embankment loads. Considering this, Gannett Fleming recommends incorporating into the conduit design a joint system that can accommodate a reasonable degree of conduit length change if foundation deformation is experienced, or if horizontal stresses will be induced due to embankment consolidation after fill placement. Reinforced concrete cylinder pipe (embedded steel cylinder) with articulated joints should be considered in lieu of the steel pipe as shown on the drawings.

Advantages include:

• Provides a means of relieving stress concentrations (will require some geotechnical input for anticipated range horizontal embankment spreading due to embankment material consolidation and induced foundation deformation).

• Pipe is less expensive than steel.

• Carrier pipe will be cement lined and less susceptible to tubercular formations that would reduce hydraulic capacity over the long term.

• Pipe can be provided with pre-stressing reinforcement if required to accommodate internal pressure.

Typically the pipe is installed in concrete encasement similar to what is shown in the 1996 drawings, except expansion joints are provided at each pipe joint (20 to 24 ft. pipe lengths).

4.5 Sluice Gate Operating System Design
The operating arrangement for the sluice gates (referenced as slide gates throughout the 1996 specifications and drawings) could be prone to binding if differential settlement of support pedestals occurs. Also, there is not much cover to protect the stem and oil-filled casing from damage due to ice loading. It is suggested that to minimize the potential for problems that consideration be given to the following:

- Some provisions should be made to prevent the actuator thrust bearing wall (founded on dam crest) from impinging on the operating stem/stem casing pipe if vertical displacement of the wall occurs due to settlement. This could be achieved by providing adequate annular spacing around the stem casing pipe as it passes through the wall. In addition, the design would also require modifying the orientation of the actuator thrust-bearing collar so it interfaces the bearing wall in the vertical plane. The actuator mounting plate/collar (attaching the operator to the wall) could be provided with slotted openings that would permit relative motion between the wall and the operator without inducing substantial vertical forces against to the operating stem.

- The centralizers planned to be provided within the oil filled operating stem casing pipe should be provided as roller or ball bearing assemblies to facilitate gate operation considering that a sizable component of the weight of the stem will bear on these units.

- To address ice loading, an additional thickness of riprap could be installed over and along the alignment of the operating stem (approximately 5 feet top width, 2 feet thick, 1:1 taper to embankment face).

- Sluice gate specification needs to identify seating head type sluice (Heavy Duty) with corresponding design head. Also specification should identify maximum allowable leakage. (typically a function of gate perimeter).

- Gate valves might best be specified as “resilient” disk type. Larger valves are now available as resilient, as opposed to double disk metal seats.

### 4.6 Spillway

The HEC-1 computations that were included in the “Spillway Design” computations were reviewed by Gannett Fleming. The following comments are provided about the analysis and spillway design:

- The initial conditions for the storage routings used starting water surface elevation one foot below spillway crest elevation (El. 5665 feet versus 5566 feet), thus overstating the storage available at initiation of the storm runoff event.

- Assumptions regarding the model input parameters for the rating curve appear to be incorrect. The input for routing was for trapezoidal and ogee control sections, and a tail-water rating curve is required input data for these configurations. However, the control section is not an ogee, and no tail-water rating curve was supplied in the input array. Inspection of output indicates control was set as critical depth for the input trapezoidal geometry. The spillway profile shows the longitudinal length of the outlet channel is approximately 2,800 feet at a 0% slope. It is suggested that top of dam elevation be verified using an outflow rating generated from backwater computations,
and rerouting the inflow hydrograph. The review also noted that the low-level outlet capacity was included in the routing, and although this will likely have a small impact, this is usually not included as part of the outlet ratings for spillway sizing.

- The spillway plan topography indicates what appears to be a drainage swale or ditch traversing a course leading to and along the toe of the south embankment. It appears from the mapping that a portion of the spillway out-flow might be conveyed along this drainage course, potentially inducing erosion along the toe of the embankment. It is recommended that a berm (embankment side of discharge channel) be provided to insure all spillway outflows are directed away from the embankment.
NOTES:
1. Erosion zones, chamfer and blanket drain not shown for clarity.
2. Refer to Plate 3 for location of section.
5.1 Updated Cost Estimates for Dam and Highway Construction

Engineering cost estimates prepared for the 1995 design were escalated to 2006 dollars by adjusting the unit prices. For dam elements, escalated unit costs were derived from the Engineering News Record (ENR) construction cost index ratio (CCIR) between December 1995 to April 1995 (CCIR = 1.344), plus 4% inflation to estimate April 2006 unit costs (inflation factor – 1.04). Highway costs were escalated as described in Section 4.1.4, based the WYDOT data base (BIDTABS) program for unit prices.

5.2 Cost Estimates for Canal Construction

As part of this current effort, WWC Engineering reviewed the cost estimates for new canal construction that were prepared in 1995 and 1996. Our review was limited to a cursory examination of the quantities and unit prices used in the original estimates. This effort included a review of the working papers which WWC still maintains. The review did not identify any unreasonable deficiencies in the conceptual design quantities, but changes will undoubtedly occur as the final design is advanced. As a check on the reasonableness of the estimates, WWC recomputed cost estimates using 2004 unit price data. Conceptual design estimates include engineering, surveys, construction administration, easements, residential relocation, utility relocation and final plans and specifications. The 2004 cost estimates were then escalated to year 2006 costs using ENR indexes. The cost estimates for canal construction are summarized on Table 5.1.

<table>
<thead>
<tr>
<th></th>
<th>1995 Estimate</th>
<th>2004 Unit Prices</th>
<th>2006 Estimated Cost*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inlet Canal</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Recommended Design</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(AH 1)</td>
<td>$2,700,000</td>
<td>$3,400,000</td>
<td>$3,900,000</td>
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<tr>
<td>South Outlet Canal</td>
<td>$900,000</td>
<td>$1,000,000</td>
<td>$1,300,000</td>
</tr>
<tr>
<td>North Outlet Canal</td>
<td>$2,500,000</td>
<td>$2,700,000</td>
<td>$3,700,000</td>
</tr>
<tr>
<td><strong>Subtotal: Cost of</strong></td>
<td><strong>$6,100,000</strong></td>
<td><strong>$7,100,000</strong></td>
<td><strong>$8,900,000</strong></td>
</tr>
<tr>
<td><strong>Canal Construction</strong></td>
<td><strong>$6,100,000</strong></td>
<td><strong>$7,100,000</strong></td>
<td><strong>$8,900,000</strong></td>
</tr>
</tbody>
</table>

*based on ENR construction cost index ratio and 4% inflation

Table 5.2 summarizes the base construction costs for all elements of the project, including the 1995 cost estimates, the 2006 escalated cost estimates for the 1995 design, and 2006 escalated cost estimates for the modified design (separate highway and dam projects) for two different highway re-alignment options. The Ray Lake Enlargement project can only be achieved if all major project components are considered, including costs for dam, outlet and spillway, and Lateral 37C relocation construction; Highway 287 relocation; new canal construction; final design engineering; and permitting and mitigation costs. Table 5.3 summarizes the estimated total project costs for the combined project components.
Table 5.2  Base Construction Cost Estimates

<table>
<thead>
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<th></th>
<th></th>
<th></th>
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<tbody>
<tr>
<td>Site Work</td>
<td>$2,558,000</td>
<td>$3,575,000</td>
<td>$3,555,000</td>
<td>$3,555,000</td>
<td>$3,555,000</td>
<td>Modified design has shorter causeway pipe</td>
</tr>
<tr>
<td>Embankment</td>
<td>$10,022,000</td>
<td>$14,004,000</td>
<td>$12,624,000</td>
<td>$12,624,000</td>
<td></td>
<td>Modified design has less Zone 1 and no highway special subgrade due to removal of road; more Zones 2 and 3A due to wider crest width and full blanket drain in max. dam section</td>
</tr>
<tr>
<td>North Outlet Works</td>
<td>$1,043,000</td>
<td>$1,477,000</td>
<td>$1,302,000</td>
<td>$1,302,000</td>
<td></td>
<td>Modified design has shorter, lower cost pipe; upstream slide gate and downstream valve costs were adjusted for current (2005) model prices (new unit prices) and inflation for both escalated 1995 cost and modified costs</td>
</tr>
<tr>
<td>South Outlet Works</td>
<td>$711,000</td>
<td>$899,000</td>
<td>$826,000</td>
<td>$826,000</td>
<td></td>
<td>Modified design has longer length, but lower cost pipe; upstream slide gate and downstream valve costs were adjusted for current (2005) model prices (new unit prices) and inflation for both escalated 1995 cost and modified costs</td>
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<tr>
<td>Subtotal dam</td>
<td>$14,334,000</td>
<td>$19,955,000</td>
<td>$18,307,000</td>
<td>$18,307,000</td>
<td></td>
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<tr>
<td>Subtotal Highway Reconstruction</td>
<td>$1,775,000</td>
<td>$2,200,000</td>
<td>$2,600,000</td>
<td>$4,000,000</td>
<td>See Table 4.1</td>
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<tr>
<td>Subtotal Canals</td>
<td>$6,100,000</td>
<td>$8,900,000</td>
<td>$8,900,000</td>
<td>$8,900,000</td>
<td>See Table 5.1</td>
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<tr>
<td>Subtotal Construction</td>
<td>$22,209,000</td>
<td>$31,055,000</td>
<td>$29,807,000</td>
<td>$31,207,000</td>
<td></td>
<td></td>
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<td>Mobilization (10%)</td>
<td>$2,221,000</td>
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<td>$2,981,000</td>
<td>$3,121,000</td>
<td></td>
<td></td>
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<tr>
<td>Base Construction Cost Subtotal</td>
<td>$24,430,000</td>
<td>$34,161,000</td>
<td>$32,788,000</td>
<td>$34,328,000</td>
<td></td>
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</tr>
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Table 5.3  Total Project Costs (2006 Dollars)

<table>
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<tr>
<th></th>
<th></th>
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<th></th>
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</thead>
<tbody>
<tr>
<td>Base Costs for Construction</td>
<td>$34,161,000</td>
<td>$32,788,000</td>
<td>$34,328,000</td>
<td>Table 5.2</td>
</tr>
<tr>
<td>Engineering (10% Construction)</td>
<td>$3,416,000</td>
<td>$3,279,000</td>
<td>$3,433,000</td>
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<tr>
<td>Construction + Engineering</td>
<td>$37,577,000</td>
<td>$36,067,000</td>
<td>$37,761,000</td>
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</tr>
<tr>
<td>Contingency (15% Construction + Engineering)</td>
<td>$5,637,000</td>
<td>$5,410,000</td>
<td>$5,664,000</td>
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<tr>
<td><strong>Subtotal: Cost of Construction and Engineering w/contingency</strong></td>
<td><strong>$43,214,000</strong></td>
<td><strong>$41,477,000</strong></td>
<td><strong>$43,425,000</strong></td>
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<tr>
<td>Final Design &amp; Specifications (15% of Construction Costs)</td>
<td>$5,124,000</td>
<td>$4,918,000</td>
<td>$5,149,000</td>
<td></td>
</tr>
<tr>
<td>Wetland Mitigation</td>
<td>$3,309,000</td>
<td>$4,036,000</td>
<td>$3,459,000</td>
<td>31.72 acres – 1995 design \ 41.42 acres – Alt. 1 \ 33.72 acres – Alt. 2 \ 12.4 acres - Canals Estimated at replacement rate of 1.5 acres/acre disturbed X $50,000/acre constructed wetland</td>
</tr>
<tr>
<td>Land Acquisition</td>
<td>$464,000</td>
<td>$552,000</td>
<td>$602,000</td>
<td>232 acres – 1995 design \ 276 acres – Alt. 1 \ 301 acres – Alt. 2 Estimated as $2000/acre for open agricultural land</td>
</tr>
<tr>
<td>Residential Property Purchase</td>
<td>$900,000</td>
<td>$900,000</td>
<td>$900,000</td>
<td>6 @ $150,000 each altern.</td>
</tr>
<tr>
<td>Loss of Reservoir Storage During Construction</td>
<td>$280,000</td>
<td>$0</td>
<td>$0</td>
<td>$20/acre-feet x 6980 acre-feet x 2 seasons</td>
</tr>
<tr>
<td><strong>TOTAL PROJECT</strong></td>
<td><strong>$53,291,000</strong></td>
<td><strong>$51,883,000</strong></td>
<td><strong>$53,535,000</strong></td>
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</table>
6. NEPA AND PERMITTING REQUIREMENTS

6.1 Anticipated NEPA Process

Compliance with the National Environmental Policy Act (NEPA) is required whenever federal funds are involved in a project. A meeting was held on March 7, 2005 between various stakeholders (WWDC, BIA, Tribal Water Engineer’s Office, and Tribal Water Board members) to discuss the anticipated NEPA process and requirements. Members of the discussion group agreed that an Environmental Impact Study (EIS) would likely be required due to potential impacts to existing cultural sites and wetlands. Individual property owners and tribal properties both could be impacted.

The BIA would be the lead agency, with the Tribes as cooperating agencies. The Tribal Water Engineer’s office has been reviewing available information relevant to NEPA, and anticipates that the U.S. Army Corps of Engineers (COE) also would be involved due to wetlands issues that need to be addressed.

6.2 Key Issues

6.2.1 Cultural Resources

Although the project area was surveyed about 8 to 10 years ago, the previous study looked at a smaller reservoir (10,000 to 15,000 acre-feet), not the 35,000 acre-feet reservoir that is currently being considered. However, the previous work was fairly comprehensive, and could likely be built upon with supplemental work for the larger project. Typically, representatives from each tribe would be involved, and would make recommendations on how to handle the sites.

6.2.2 Wetland Impacts and Mitigation

The 1995 Ray Lake Enlargement design work estimated a wetland impact of about 11 acres that would result from the proposed dam enlargement project (WWC, Dec 29, 1995). The estimate was based on an actual field delineation effort. However, that estimate was based only on the areas immediately impacted by the embankments; borrow areas, and high waterline inundation. The estimate did not consider impacts from a new highway re-alignment, new canal construction or widening of existing canals.

As part of the current study, the Project Team examined the previous wetlands inventory and prepared a revised estimate of the potential wetlands impact. This revised assessment considered the impact of new outlet canals, dam embankments, reservoir inundation, highway construction and enlargement or modification of existing canals. Figure 6.1 presents a map that depicts the potential impacted wetland areas and quantities. (Note, figures for this section are attached at the end of Section 6). These wetland areas are those that are currently identified on the National Wetlands Inventory Mapping prepared in the mid 1990’s and do not likely represent actual areas at this time.

The cost to mitigate wetlands should be considered in program funding development. Because
specific design details are not known at this time, the following guidance is offered:

- Assume that the wetland replacement requirement is 1.5 acre replacement for each acre of disturbance.
- For unit price to design, permit, and construct wetland mitigation, assume $50,000 /acre in 2006. This estimate is approximate and based in part on recent bid data from a WYDOT project and from conceptual design estimates presented in the 1995 wetlands report (WWC, 1995). This price includes post-construction monitoring that is required.

### 6.2.3 Land Ownership and Acquisition

The enlargement of Ray Lake Reservoir and associated activities would require the acquisition of considerable right-of-way, easement, or purchase of lands. As part of this current study, an estimate of these needs was prepared, and the results of that effort are presented on Figure 6.2. This estimate includes an allowance for increasing the right-of-way along existing canals, providing right of way for new canals, providing for new reservoir inundation area, the embankments of the dam, and alternative state highway re-alignments.

Time to complete appraisals is often lengthy when BIA attempts to acquire land. Third party appraisers may be used to expedite this process. There may be opportunities for land transfers within tribal lands.

The project team discussed the procedures and potential difficulties associated with right-of-way acquisition on the Wind River Indian Reservation with the Tribal Transportation Planner. Lands on the Reservation are classified as allotted lands, trust lands (held in trust by the BIA) and fee lands. Despite the differences in classification, the value of the lands would be appraised and right-of-way could be obtained through all areas, just as right-of-way is obtained for projects off the Reservation. At the time of this report preparation, we had not yet received data from the Tribes for specific ownership information in the reservoir area. Despite this, we do not feel that the specific ownership information is critical to the results of this current study.

No title work or appraisal work was performed for this study. The Project Cost estimate presented elsewhere in this report assumes the value of open agricultural lands at $2,000/acre. Homes/building that could be displaced in the immediate vicinity of the dam would be valued considerable higher. Presently there are an estimated 6 homes/buildings along the north side of the roadway within the impact area of the proposed embankment.
### Impact Area Description

<table>
<thead>
<tr>
<th>Impact Area Description</th>
<th>Gross Area (ac)</th>
<th>Wetland Area (ac)</th>
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<tr>
<td>Roadway Alignment</td>
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<tr>
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<tr>
<td>Extension</td>
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<td>5.1</td>
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<td>Lower Ray Canal</td>
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Construction Impact Areas

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<tr>
<td>Road alt. 2 w/150 ft. ROW</td>
<td>63</td>
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<tr>
<td>Widened Lateral 37C to Lake</td>
<td>6</td>
</tr>
<tr>
<td>North Canal ROW (100 ft.)</td>
<td>56</td>
</tr>
<tr>
<td>South Canal ROW (100 ft.)</td>
<td>35</td>
</tr>
<tr>
<td>Proposed Spillway</td>
<td>56</td>
</tr>
<tr>
<td>Embankment Impact Area</td>
<td>85*</td>
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<tr>
<td>Total land to be acquired (Alt 1)</td>
<td>276</td>
</tr>
<tr>
<td>Total land to be acquired (Alt 2)</td>
<td>301</td>
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</table>

* Impact area for embankment estimated at 20% more than footprint.
7. ISSUES COORDINATION

A crucial aspect of the study was to define issues that must be coordinated among the state, tribes and federal agencies that are involved. Two Issues Coordination meetings were held during the course of the project as indicated below. Complete meeting notes are provided in Appendix A.

- Issues Coordination Meeting 1 was held on November 3, 2004 in Ethete, Wyoming. The primary purpose of this meeting was to update the Sponsors and WWDC on the progress of the study following initial review of the project records, and to present results and conclusions from the preliminary hydrologic system modeling. At this meeting the project team also presented our conclusion that the structural measures required for the SOD project could not be made economically compatible with the Enlargement Project, primarily due to geotechnical foundation concerns. At this time the study team made the recommendation that each project should be advanced separately, and that the SOD improvements be implemented as soon as possible.

- Issues Coordination Meeting 2 was conducted by teleconference on March 7, 2005. The primary purpose of this meeting was to discuss anticipated NEPA permitting issues for the Enlargement Project.

Issues identified and addressed by this study (section where each issue is presented and evaluated is indicated in parenthesis) include the following:

- Irrigation and other water needs in the project area (Section 3.1)
- Water availability and optimum reservoir size (Section 3.2)
- Canal system upgrade requirements (Section 3.3)
- Consideration of SOD and enlargement project timelines (Section 3.5.1)
- Adequacy of geotechnical information to support final design of enlargement project (Section 3.5.2 and 3.5.3)
- Enlargement design review and recommended modifications (Section 4)
- Anticipated NEPA and permitting requirements (Section 6)
- Costs (Section 5)

Technical issues listed above have been identified and preliminarily addressed by this study. Recommendations for future studies that will be needed to advance the project on these issues are provided in Section 7. In addition to the previously identified issues, there are two additional outstanding issues that will require further coordination and consideration if the project is to be advanced. These issues are described in the following sections.

7.1 State of Wyoming Permitting of an Enlarged Reservoir

The existing Ray Lake has no storage permit from the State of Wyoming. The normal State of Wyoming procedures for the permitting of an enlarged Ray Lake would be to issue a present day
priority date. One of the purposes of the optimum reservoir sizing presented in Section 3.2 was to determine if the reservoir priority date was critical to the enlargement project feasibility.

Based upon the results of this study, the reservoir priority date would not affect the ability to expand the storage and canal delivery capacities for diverting the available early season stream flows, in addition to those presently stored in Ray Lake. These diversions for storage in the enlarged reservoir could continue to be accounted for as a part of the existing 1868 priority diversion volume as decreed.

Water saved as a result of conservation activities through reduction of seepage losses could also be stored in an enlarged Ray Lake. This water would also be accounted for in the existing diversion right.

Additional irrigated acreage as allowed for in the decree and identified as Type 7, Type 8, or future project lands could be served water from the storage provided in an enlarged Ray Lake. Portions of these decreed water rights could also be stored in an enlarged Ray Lake.

Even greater amounts of water stored under a present day storage water rights would be available in larger water years. Consequently, expanded canal conveyance capacities and carry-over storage capacity would be needed to efficiently utilize the available water under these high streamflow conditions.

In summary, a present day priority storage water right for Ray Lake would not affect the feasibility of the enlargement project. However, water rights issues will undoubtedly remain a key issue in future discussions among project stakeholders.

7.2 State of Wyoming Permitting of Rehabilitated Reservoir

The potential rehabilitation of Ray Lake at its present capacity is a likely scenario regardless of whether the reservoir is enlarged in the future. Based on discussions with representatives of the SEO during the course of this study, the rehabilitation project may not be granted a construction permit from the SEO without a reservoir storage permit. The SEO procedures, as previously discussed, would be to issue a present day priority date for the reservoir. As discussed in the previous section, this priority date would not affect the ability to store early water as is presently done.
8. CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

Key findings from this study are summarized as follows:

- The Ray Lake Enlargement Project has already passed several rounds of screening in previous water planning studies, and was recommended for advancement through both the Upper Wind River Storage Project, Level I Study (SEH, 2001) and the Wind/Bighorn Basin Plan (BRS, 2003). This study confirms that the project is justified and viable in terms of both identified needs and water availability.

- Needs analysis by previous studies (NRCE, 1995 and SEH, 2001), summarized in Section 3.1 of this report, indicate that there are identified needs of at least 16,500 acre-feet per year, and may be as much 33,000 acre-feet per year, based solely on existing unmet and future potential irrigation demands in the Little Wind River basin that could be serviced by an enlarged Ray Lake. Additional irrigation needs have also been identified downstream from the confluence of the Little Wind and Big Wind Rivers that potentially could be supplemented from Ray Lake storage. Other beneficial uses for an enlarged storage facility also are identified (Section 3.1.4).

- Previous system analysis studies (NRCE, 1995), and the current model study presented in Section 3.2 of this report, indicate that a Ray Lake enlargement size of at least 35,000 acre-feet (25,000 acre-feet active storage and 10,000 acre-feet recreation pool for proper system operation) is viable based on water supply and conservative (present-day) water rights assumptions.

- To adequately supply the enlarged Ray Lake Reservoir, the North Fork Diversion Canal would need to be upgraded to carry an additional 50 cfs of water for storage, and the Ray Canal and Lateral 37C upgraded to carry 200 cfs of water for storage. These capacities would be in addition to canal capacity required for simultaneous delivery of direct flow irrigation water.

- This combination of reservoir size and supply canal size could deliver an average supplemental water demand of 16,500 acre-feet in 9 out of 10 years. Increasing the active storage capacity above 25,000 acre-feet would improve the ability of the project to meet the supplemental water demands.

- The existing dam has significant dam safety issues, and will require rehabilitation work, whether or not the enlargement project moves forward. The safety of dams (SOD) design is substantially complete and approved by USBR subject to minor design change recommendations (USBR, 2003), and it has been funded by the BIA.
  - This study concluded that it was neither economically or technically feasible to incorporate the SOD work components into a future enlargement design, and the team recommends constructing the SOD improvements as soon as possible.

- Although significant engineering design has already been completed for a 35,000 acre-feet reservoir, one important issue that must be resolved is the inadequate geotechnical and geologic information at the site to support the final design of the embankment dam.
In Section 3.2 of this report, Gannett Fleming has outlined a proposed detailed plan for supplemental geotechnical work to address this issue in order to prevent this from being a barrier to consideration of an enlargement project.

- The design review under this study identified several recommended changes to the 1995 design to facilitate construction, and to simplify long-term operation and maintenance of the project, without substantially impacting the construction cost estimates. The primary recommended changes include:
  - **Separate the highway realignment from the dam.** The 1995 design shows the Highway 287 embankment realigned and joined with the north portion of the raised embankment dam. The current study team recommends separating these projects for a variety of reasons, including the administrative advantage of having separate construction packages and schedules for the different types of construction, elimination of a costly temporary detour, separation of highway and dam maintenance and operations, and other considerations, as described in Section 4.1. Two preliminary feasible alternative alignments for the highway are identified.
  - **Shift the enlarged embankment alignment downstream along the segment adjacent to the existing dam.** The 1995 design shows the upstream toe of the raised dam coincident with the upstream toe of the existing dam. Shifting the raised dam downstream will allow the reservoir to remain in operation during the anticipated two-year construction period, and will allow the existing dam to provide a protective cofferdam retaining unstable reservoir sediments and storm water runoff during construction.
  - **Modify the embankment sections** to incorporate a wider crest width across higher sections of the dam, and to reduce the crest width where the embankment no longer has to carry the highway over its crest. Additional recommendations are made regarding blanket filter configuration, and possible foundation treatment requirements, but these are preliminary and require additional geotechnical site investigations before final recommendations can be made.
  - **Use a different type of pipe for the outlet works.** The 1995 design shows a steel pipe encased in concrete. Considering the compressible nature of the foundation and embankment materials, Gannett Fleming recommends using specially fabricated reinforced concrete cylinder pipe (embedded steel cylinder) with articulated joints. This type of pipe would provide better performance at a lower cost.
  - **Recommendations for final design analysis.** Additional recommendations regarding final design analysis and details for the sluice gate operating system and spillway design are also provided in Section 4.5 and 4.6.

- Cost estimates for the 1995 design, and the recommended modified design were escalated to 2006 prices using ENR index methods and bid tabs information. Base construction cost estimates for the highway, dam, and new canals range from $32.8 million to $34.3 million, depending on the highway alternative selected. When estimated costs for final design, engineering, wetland mitigation, land acquisition, contingencies, and other items are factored in, the total construction costs range from
$51.9 million (for the modified design Alternative 1 highway alignment) to $53.5 million (for the modified design Alternative 2 highway alignment).

8.2 Recommendations

The following additional analyses are recommended for a follow-on Phase II, Level II study if the project is advanced by the Sponsors:

1. **Additional System Modeling.** Additional detailed system modeling is recommended to provide more accurate analysis of reservoir yield by tying releases from Ray Lake directly to actual water needs in the basin. Existing water shortages and future water supply needs are demonstrated in the project service area. Existing annual un-met irrigation demand is estimated to be at least 16,500 acre-feet (NRCE, 1995 estimate), up to 18,500 acre-feet (SEH, 2001 estimate). Future potential irrigation demand in the Little Wind River basin is estimated to be on the order of about 14,500 acre-feet (Type 7, Type 8 and Futures Awards). Since one of the primary inhibiting factors of the amount of yield derived from an enlarged Ray Lake Reservoir is the storage in Boysen Reservoir, the model may need to address the entire basin associated with the inflows and demands upon Boysen Reservoir.

2. **Supplemental Geotechnical Investigations and Analysis.** Additional geotechnical information is required to support final design of the enlargement project. A detailed program is outlined in Section 3.5, including estimated costs for labor, drilling and test pits, and laboratory testing (Table 3.4). In addition to the costs associated with the field work and geotechnical report shown on Table 3.4, additional budget should be provided for pre-final engineering analysis and design of modifications (as needed) to the embankment section and foundation treatment requirements based on the findings of the supplemental geotechnical investigation. It should be noted that the investigation outlined in Section 3.5 is comprehensive, and is anticipated to complete the requirements for final design, such that Level III could proceed directly to finalization of detailed plans and specifications for the project.

3. **Supplemental Analysis and Design Verification for Emergency Spillway.** The design review identified some potential problems with the spillway routing model regarding input assumptions. The spillway hydraulics should be reviewed, and appropriate methods and routing assumptions applied to develop the rating curve. Also, additional analysis are recommended, once the geotechnical information is available, to evaluate the spillway erodibility and potential for scour at the toe of the embankment adjacent to the spillway, using the SITES model developed by the Natural Resources Conservation Service (NRCS).

4. **Pre-Final Design of Ray Lake Inlet and Outlet Canals, and Improvements to Existing Ray Canal System.** In 1995 and 1996, WWC prepared conceptual designs for alternatives to provide a new inlet to an enlarged Ray Lake, and for new outlet canals to deliver water to areas northeast and southeast of Ray Lake (WWC 1995b, 1996b). Following additional system analysis to confirm the optimum size of the enlarged reservoir (see recommendation
1), these designs could be advanced to a preliminary level in a follow-on Level II, Phase II study. Also, to adequately supply the enlarged Ray Lake Reservoir, the North Fork Diversion Canal should be upgraded to carry at least an additional 50 cfs of water for storage, and the Ray Canal and Lateral 37C upgraded to carry 200 cfs of water for storage. These capacities would be in addition to canal capacity required for simultaneous delivery of direct flow irrigation water. Designs and cost estimates for these canal enlargements should also be advanced in further studies.

5. **Evaluation of Lateral 37C Realignment at North End of Dam.** The current (1995) embankment design plans show Lateral 37C passing through the dam above the North Outlet Works pipe near dam Station 85+00 (See Plate 2). To avoid this penetration, it is recommended that the next phase of design evaluate alternative alignments of the lateral around the north end of the dam.

6. **Preliminary Selection and Design of Alternative Highway Relocation.** A key recommendation that originated from this study is to separate the highway and dam construction projects, as discussed in Section 4.1. Two very preliminary possible alignments have been identified and feasibility-level cost estimates presented for these options. If this recommendation is to be advanced, supplemental studies will be required to evaluate optimum alignment options and provide a more detailed design basis for cost estimating.

7. **Additional Cultural Resources and Wetland Impact Studies.** This study developed a preliminary wetland inventory based on National Wetlands Inventory Mapping prepared in the mid 1990’s and do not likely represent actual areas at this time. Also, cultural resource surveys conducted in the mid-1990’s were developed for smaller reservoir alternatives, and did not include areas potentially impacted by a larger reservoir inundation area, as well as new canal and road construction disturbance areas. Field mapping for wetlands and cultural resources surveys specific to the final project features should be completed to support the permitting and identify mitigation requirements.

8. **Economic Analysis and Project Financing.** This Level II study did not include detailed economic analysis and identification of project financing opportunities. These tasks also should be included in a Level II, Phase II study.
9. REFERENCES


prepared by Thomas A. Brown, Team Leader, Geotechnical Services, USBR, Denver, CO, to Area Director, Bureau of Indian Affairs, Billings Area Office, Nov 20.


November 10, 2004

Mr. Phil Ogle
Wyoming Water Development Commission
6920 Yellowtail Road
Cheyenne, WY 82002

Re: Ray Lake Enlargement Project, WWDC Project No. 05SC0292446
    Report on Issues Coordination Meeting held November 3, 2004 in Ethete, WY

Attendees: See attached attendance sheet

Agenda: See attached agenda

Dear Mr. Ogle:

This letter provides a summary of the first Issues Coordination meeting for the Ray Lake Dam Enlargement project. The meeting was held on Wednesday evening, November 3, 2004, at the Wyoming Indian High School Technical Center in Ethete, Wyoming. The following sections summarize the meeting, following the agenda outline.

INTRODUCTION
The meeting was attended by fourteen people, as shown on the attached attendance sheet, representing the following:

- Wyoming Water Development Commission (WWDC) and their consultant team for the Level II study (Gannett Fleming and States West Water Resources);
- Bureau of Indian Affairs (BIA), representatives from both Wind River Agency and Rocky Mountain Regional office;
- Tribal Water Engineer;
- Tribal Water Board;
- Shoshone Business Council, and;
- Wind River Irrigation interests.

Phil Ogle (WWDC) introduced the project team, and briefly described the WWDC program. The stated purposes of the meeting were to report on the status of the Ray Lake Enlargement, Level II study, and to provide a forum for presenting and discussing issues relevant to the project.
PRESENTATION
Project team members Deb Miller (Gannett Fleming, Project Manager) and Victor Anderson (States West Water Resources, Meeting Facilitator) made a brief presentation to the group using PowerPoint. A disk with the PowerPoint presentation, and the handouts provided to the attendees are attached with this letter. The presentation included a brief review of the project background and scope of work of the current study, and two key study findings to date. The presentation content is summarized as follows:

Project Background: Detailed review of the project archives and previous studies revealed that the existing dam has significant dam safety issues, as identified through the U.S. Bureau of Reclamation’s (USBR) Safety Evaluation of Existing Dams (SEED) studies that were conducted between 1983 and 1992. The safety of dams (SOD) deficiencies include the following:

- Uncontrolled seepage in the embankment near the outlet works, and through the foundation;
- Cracking and concrete deterioration of the outlet works conduit;
- Transverse and longitudinal cracks near the crest of the dam;
- Inadequate spillway capacity and erosion protection for the inflow design flood; and
- Failure of the upstream riprap slope protection.

Between 1993 and 1996 designs were developed by GEI Consultants (1993a and 1993b) and Western Water Consultants (WWC) (1994; 1995a; 1995b; and 1996) on two separate tracks, as follows:

Safety of Dams Modifications
- Capacity remains at 6980 AF
- Addresses USBR dam safety deficiencies

Enlargement Designs
- Various enlargement sizes considered (up to > 40,000 AF)
- Larger options also involved major new canal structures

Enlargement options were intended to address the dam safety issues (by replacing the existing deficient dam and outlet structure with new facilities), and to provide additional water in storage to address existing irrigation supply shortages. Several basin and irrigation system planning studies also were conducted between 1994 and 2003 (NRCE, 1994; NRCE, 1995; NRCE, 1996; SEH, 2001; and BRS, 2003). These studies confirmed the chronic irrigation supply shortages in the basin and, through screening processes, recommended enlargement of Ray Lake as a viable project to help address these needs. It was also recognized that additional potential project benefits with an enlarged reservoir could include:

- Ability to deliver water to additional irrigation users south of Mill Creek and north towards Trout Creek through additional reservoir outlets;
- System operations would be considered to look at opportunities for Ray Lake storage to be used in tandem with Washakie Reservoir;
- Improving in-stream flows on the Wind River;
- Additional irrigation benefits in the Riverton East project;
Recational benefits;
Potential downstream municipal uses; and
Futures awards uses.

The status of the previous studies for both the SOD modifications and the Enlargement projects was also presented. The presentation is summarized as follows:

- 35% design reports for both the SOD Modifications Project, and for an Enlargement Project for a 36,000 acre-feet reservoir, were completed and provided to USBR for review in 1995.
- 95% Plans and Draft Specifications for both projects were completed by WWC in 1996.
- USBR reviewed the SOD designs in 1995-1996 (35% design report) and 2003 (95% plans and draft specifications). Their review comments indicate the SOD design is acceptable, with relatively minor recommendations/clarifications to details of the design, and a requirement for a final design report.
- USBR reviewed the Enlargement Project design in 1995-1996 (35% design report and supplemental geotechnical studies). The basis for the 35% design was questioned by USBR, primarily due to inadequacies in the geotechnical site investigation and embankment design. The questions and issues that were raised by USBR are perceived by the project team as a potential “road block” to the Enlargement Project moving forward. The scope of work for the Level II study addresses these issues.

**Scope of Work of this Level II Study:** The team attempted to make it clear that this study is about the Enlargement Project, not the SOD Modifications Project. The presentation briefly outlined the major study tasks which include the following:

1. Background Information Review
2. Review Existing Designs for Ray Lake Enlargement
3. Evaluate Increasing Enlargement of Ray Lake
4. Review Little Wind River Irrigation System
5. Issues Coordination
6. Reports
7. Scoping and Project Meetings

**Study Findings to Date:** Two key study findings were then presented, as follows:

1. **The Safety of Dams (SOD) Project cannot be economically modified to accommodate a later Enlargement Project.** This finding is based on the team’s review of the geotechnical information and the poor condition of the existing dam and outlet works. An enlargement project requires complete rebuilding of the dam and outlet structures, with additional excavation to a suitable foundation for the higher structure. The team recommends that the SOD project proceed as it is currently designed, and as a separate effort, independent of any studies that are done to consider a future enlargement of Ray Lake.
2. **Storage water is available for the 36,000 acre-feet enlarged reservoir size.** This is based on preliminary, independent hydrologic analysis conducted by the project team as part of this study. These analyses assumed a 2004 water storage right, existing gage data, and consideration of regulation by Boysen Reservoir. The 36,000 acre-feet size is validated, and is sized to provide a 10,000 acre-feet recreation pool (“dead pool”) needed to bring the reservoir elevation up high enough to deliver water out through long canal systems north and south of the reservoir. An additional approximately 10,000 acre-feet of carry-over storage is also needed to ensure adequate supplies during drought periods, for a firm yield of approximately 16,000 acre-feet annually. These findings are consistent with previous system analyses studies (NRCE, 1995).

**BIA COMMENTS**

Mr. *Steve Pollock* with Bureau of Indian Affairs (BIA) in Billings, Montana, provided the following comments on behalf of the BIA Rocky Mountain Region:

1. As the owner, BIA has ultimate responsibility to ensure that the dam is safe.
2. Early warning systems have been developed and implemented for both Ray Lake and Washakie dams.
3. Of 116 dams under their jurisdiction, Ray Lake is #16 on the BIA priority rating system, nationwide.
4. USBR has felt that a restriction on the reservoir level to a maximum elevation 5525 feet is required and has been in effect since 1991. Given the enduring controversy and inaction in making necessary repairs, there has been a considerable loss of water supply.
5. The Ray Lake SOD project was a congressional line budget item. Money ($2.6 million) was made available through a contract with the Joint Business Council (JBC) to address the dam safety deficiencies.
6. This money has been retracted by the BIA because although minor work has been done, the major dam safety modifications have not. Every year the dam has deteriorated more and more. BIA terminated their contract with the JBC for Ray Lake SOD funds. The JBC has appealed this contract termination, and BIA is currently responding to the appeal.
7. BIA's current position is that they cannot wait any longer for the Safety of Dams (SOD) work to be completed by the JBC. Failure to correct these deficiencies is “unconscionable”. Their current goal is to have the 95% SOD design finalized and construction underway on the dam repairs as soon as possible. However, they currently do not have access to the funding to complete the plans and specifications on the SOD modifications until this litigation is settled.
8. Original plan was to have construction underway by August 2005. However, incomplete work at Washakie Dike No. 1 seepage concerns is presently considered a higher priority. This has been the focus and attention between BIA and USBR. Once the work at Washakie is completed, BIA will turn full attention to Ray Lake SOD improvements.
9. Mr. Pollock stated that he felt “vindicated” that our team had reached a similar conclusion as USBR regarding the possibility of modifying the SOD design, especially in the area of the outlet works, in order to accommodate a future enlargement. The conclusion from BIA’s perspective is that the SOD work should be completed as it is currently designed. USBR is satisfied with the technical aspects of the SOD design and supporting field investigations. From BIA’s perspective, the SOD project should go forward as a separate project, not tied to any further consideration of an enlargement project.
DISCUSSIONS AND QUESTIONS
A question was raised by Phil Ogle as follows: “What is the BIA’s position on the Enlargement Project?”

- Ray Nation of the BIA Wind River Agency responded that the BIA would like to have a better understanding of the acres that would be served, impacts including the enlargement of the Ray Canal, and opportunities to work in tandem with Washakie Reservoir.

- The team replied that these issues would be addressed as part of this Level II study. A map was shown which outlined the acres that are currently irrigated, and potential new acreages that would be served with the enlargement project.

- Gary Collins (Tribal Water Engineer) reiterated the other potential project benefits, including working in tandem with Washakie to better serve other irrigated areas, and project purposes besides irrigation such as those listed previously (Riverton East, futures awards, etc.).

A question was asked by Steve Pollock about the funding mechanisms under the WWDC program.

- Phil Ogle explained that the Level II study is fully funded by the WWDC. Level III, final design and construction is a cost sharing program, typically 50% loan and 50% grant. Sponsors may choose to take the loan portion or find alternative sources for that half of the funding. Additionally a project sponsor could advance the project to final design and construction using other sources of funding and not state funding. The state legislature approves these studies and projects on a case-by-case basis.

Other topics that were briefly discussed include:

- Ray Lake Enlargement technical issues: One meeting participant pointed out that there used to be an old resort on the south side of the reservoir, and that there are possibly buried pipes and gas tanks to be aware of. In addition, there has been an increase in sediment load to the reservoir in recent years due to brush fires in the basin.

- Changes in Tribal Water Board: Sandra C’Bearing introduced herself as the new chairman and on behalf of the Arapahoe Tribe requested a meeting with the BIA representatives the following day to discuss several issues. She stated that she is interested in getting a better understanding of the project(s).
CONCLUSIONS
In general, the meeting was well attended by a representative group of potential project stakeholders. The meeting served to get interested parties up to date on the status of both the SOD and Enlargement Projects for Ray Lake. Important study findings to date were presented, as follows:

1. The SOD project cannot be economically modified to accommodate a later enlargement project. The team recommends that these projects continue on separate, independent design tracks. As it will likely be several years before the enlargement project is realized, the team recommends the SOD project be implemented as soon as possible to ensure dam safety and minimize risk of dam failure in the interim.
2. The team’s independent preliminary hydrologic modeling analyses indicate that sufficient water supply is available for an enlargement project at least 36,000 acre-feet in size. This is the recommended size of the enlargement that has been considered and designed in concept to date. The preliminary hydrologic modeling was completed using conservative assumptions regarding present-day (2004) storage water rights.

The BIA made their position clear – they intend to move forward with SOD modifications to the existing dam. The timing for completion of the SOD final design and construction is uncertain at this point due to ongoing litigation between the BIA and the JBC over the contract administration, and due to priority dam safety issues at Washakie Dam. The BIA expressed their continued interest in the enlargement project, and the additional information that will be provided through this Level II study.

Uncertainties remain as to the position(s) of the Tribes and the JBC with regard to the enlargement project. Tribal, BIA, and local irrigators’ support will be necessary to move the project forward.

Sincerely,
GANNETT FLEMING, INC.

Debora J. Miller, P.E., Ph.D.
djmiller@gfnet.com
REFERENCES


**RAY LAKE ISSUES COORDINATION MEETING**  
6-8 pm, November 3, 2004  
Wyoming Indian High School Technical Center, Ethete, Wyoming

<table>
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<tr>
<th>Name</th>
<th>Address</th>
<th>Phone No.</th>
<th>Representing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phil Ogle</td>
<td>Cheyenne, WY</td>
<td>(307)777-7626</td>
<td>Wyoming Water Development Commission</td>
</tr>
<tr>
<td>Alden Lock</td>
<td>Lander, WY</td>
<td>(307)332-4996</td>
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<tr>
<td>Ray Nation</td>
<td>Fort Washakie, WY</td>
<td>(307)332-3718</td>
<td>BIA/Wind River Agency</td>
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<tr>
<td>Douglas Davis</td>
<td>316 N. 26&lt;sup&gt;th&lt;/sup&gt;, Billings, MT 59101</td>
<td>(406)247-7998</td>
<td>BIA/Rocky Mountain Region</td>
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<td>Stephen Pollock</td>
<td>316 N. 26&lt;sup&gt;th&lt;/sup&gt;, Billings, MT 59101</td>
<td>(406)247-7998</td>
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<tr>
<td>John Jackson</td>
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<td>John Stall</td>
<td>Crowheart, WY</td>
<td>(307)486-2241</td>
<td>Water Board</td>
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<td>Victor Anderson</td>
<td>Cheyenne, WY</td>
<td>(307)634-7848</td>
<td>States West Water Resources Corp.</td>
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<td>Debora Miller</td>
<td>Windsor, CO</td>
<td>(970)686-5716</td>
<td>Gannett Fleming</td>
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<tr>
<td>Sandra C’Bearing</td>
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<td>(307)332-5318</td>
<td>Wind River Water Regulation and Control Board</td>
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<td>Loren Antelope</td>
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<td>(307)332-5659</td>
<td>Wind River Irrigation</td>
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<td>Frank Capotte</td>
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<td>(307)332-6464</td>
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<td>Gary Collins</td>
<td>Arapaho</td>
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<td>Tribal Water Engineer</td>
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<td>Kassil Weeks</td>
<td>Fort Washakie, WY</td>
<td>(307)332-3532</td>
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RAY LAKE ENLARGEMENT PROJECT
LEVEL II STUDY
WYOMING WATER DEVELOPMENT COMission

Issues Coordination Meeting

6-8 pm, November 3, 2004
Wyoming Indian High School Technical Center
Ethete, Wyoming

AGENDA

INTRODUCTIONS (Wyoming Water Development Commission)

PRESENTATION (Project Study Team)
- Project Background
- Scope of Work of Current Study
- Study Findings to Date, Part 1: Geotechnical Issues Review (Dam Safety and Enlargement Projects)
- Study Findings to Date, Part 2: Hydrologic Modeling/Yield Analysis Results

BIA COMMENTS

STAKEHOLDER DISCUSSIONS AND QUESTIONS (Lead by Project Team Facilitator)

CONCLUDING REMARKS AND SUMMARY (Project Team)
Memorandum

To: Doug Davis, U.S. Department of the Interior, Bureau of Indian Affairs, Rocky Mountain Regional Office, 316 N. 26th Street, Billings, MT 59101-1397

Gary Collins, Shoshone and Arapaho Tribes, Office of the Tribal Water Engineer, Box 217, 21 South Fork Road, Fort Washakie, WY 82514

Ray Nation, U.S. Department of the Interior, Bureau of Indian Affairs, Wind River Agency, P.O. Box 158, Fort Washakie, WY 82514

From: Phil Ogle, Project Manager, Wyoming Water Development Commission

Date: March 14, 2005

Subject: Issues Coordination Meeting by Teleconference, March 7, 2005

Deb Miller and I have prepared meeting notes from our March 7, 2005 conference call for the meeting file. If you have additions or corrections, send them to me at the above address or by e-mail pogle@state.wy.us. We will include any added information in the meeting file.

Gary feel free to distribute the notes to the Tribal Water Board and especially to those members that participated in the conference call.
RAY LAKE ENLARGEMENT PROJECT, LEVEL II STUDY
WYOMING WATER DEVELOPMENT COMMISSION

Issues Coordination Meeting by Teleconference
2:00 – 2:30 pm, March 7, 2005

PARTICIPANTS
Phil Ogle – Wyoming Water Development Commission (WWDC), Cheyenne
Debora Miller – Gannett Fleming, consultant to WWDC
Ray Nation – BIA/Wind River Agency, Fort Washakie
Gary Collins – Tribal Water Engineer’s Office
Merle Glick – Tribal Water Engineer’s Office
Ronnie Givens – Tribal Water Board Member
Bill O’Neal – Tribal Water Board Member
Pete Calhoun – Tribal Water Board Member
Doug Davis – BIA/Rocky Mountain Region, Billings
Ernie Sun Rhodes – Tribal Water Board Member

MEETING NOTES

Purposes of the call (Phil Ogle)
(1) Determine direction of the Ray Lake Enlargement Project if it is to go forward.
(2) Open up discussions to ideas for moving the project forward, and clarifying who
would lead specific efforts, especially NEPA permitting, and funding.

1. How will NEPA be addressed should the project move forward? Current
Environmental Assessment (EA) is probably not sufficient.
NEPA is required whenever federal funds are involved. Members of the discussion
group agreed that an Environmental Impact Study (EIS) would likely be required due
to potential impacts to existing cultural sites and wetlands and the scale of the project.
Individual property owners and tribal properties could both be impacted.

2. Who would be the lead agency for NEPA?
BIA would be the lead agency, with Tribes as cooperating agencies.

The Tribal Water Engineer’s office has been reviewing available information relevant
to NEPA, and anticipates that the U.S. Army Corps of Engineers (COE) also would
be involved due to wetlands issues that need to be addressed.

3. What are the main environmental and other issues?
(1) Cultural resources: Although the project area was surveyed about 8 to 10 years
ago, the previous study looked at a smaller reservoir (10,000 to 15,000 acre-feet),
not the 35,000 acre-feet reservoir that is currently being considered. However,
the previous work was fairly comprehensive, such that supplemental cultural
resources studies for an enlargement project would not be starting from square
Typically, representatives from each tribe would be involved, who would make recommendations on how to handle the sites. New regulations may be coming out soon that could impact the way cultural resource issues are handled.

(2) Wetlands: Wetland delineation and mapping in potentially impacted areas would be required.

(3) Land acquisition: Time to complete appraisals is often lengthy when BIA tries to acquire land. Third party appraisers may be used to expedite this process. There may be opportunities for land transfers within tribal lands.

Key issues from the BIA’s perspective are related to: (1) previous technical questions and concerns by the Bureau of Reclamation about the dam foundation and geotechnical issues, and (2) water availability (supply) and needs. These questions will be addressed to a degree by the current ongoing Level II WWDC study. Project benefits may include recreational and other considerations in addition to irrigation benefits.

4. What are avenues for potential project funding?
BIA does not have a mechanism available for funding enlargement projects. Funding is only available for rehabilitation or improvement projects.

Special funding requests for an enlargement project must be initiated by the tribes to Congress.

The State of Wyoming may be able to provide up to 50% of project costs through the water development program.

Tribes have been working to acquiring funds ($10 million over 4 years) for irrigation projects. The Wyoming congressional delegation is supportive and encouraging.

Other Topics
Ray Canal enlargement needs to be considered in conjunction with enlargement of Ray Lake because the canal is typically running full during the prime spring runoff (June). This means that additional diversions to fill Ray Lake would have to be accommodated on top of the normal diversions. This is being considered in the current study and recommendations for additional studies to optimize the canal sizing analysis will be provided in the report.

BIA is moving forward with the Safety of Dams rehabilitation project for Ray Lake.

Next steps to move the enlargement project forward are:

1. Complete the current study and report to document the preliminary water supply modeling analyses, issues coordination discussions, and recommendations for future geotechnical and other studies to support final design of an enlarged dam and reservoir; and

2. Conduct recommended supplemental geotechnical, hydrologic, and environmental permitting activities under Level II, Phase 2 studies.
APPENDIX B
Annotated Bibliography of Ray Lake Project Design Records
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<td>1/30/96</td>
<td>Conceptual Design Report, Ray Lake Inlet Canal, Ray Lake Enlargement</td>
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<td>80</td>
<td>8/25/95</td>
<td>Design of Enlargement Ray Lake Dam, Design Memo No. 2, Geotechnical</td>
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<td>WY State Highway Dept.</td>
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<td>83</td>
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<td>Ray Lake Dam Rehabilitation and Enlargement, 50% Conceptual Design of Enlargement Report</td>
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<td>Drawing Set - Ray Lake Dam Enlargement, 35% Design Plans</td>
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<td></td>
<td>Ray Lake Dam Project Notebook (Work Plans, Budget, Photos, Correspondence, Survey Data)</td>
<td>WWC</td>
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<td>88</td>
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<td>SOD Final Design Project Notebook</td>
<td>WWC</td>
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<td></td>
<td>USGS Ethete-Ray Lake 7.5 minute Quad maps</td>
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<td>Folder: Misc. correspondence re: follow-up cost and budget estimates for supplemental geotech &amp; aerial photo investigations</td>
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<td>Folder: H &amp; H responses to USBR comments</td>
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<td>Folder: Color infrared aerial photo and map of disturbance areas</td>
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Introduction

This annotated bibliography was compiled from available project archives at the offices of WWC Engineering in Laramie Wyoming, and from various other sources including the Wyoming Water Development Commission, and the Shoshone and Arapahoe tribal records. Available tribal records on the project are listed in a compilation report prepared in August 2000 by Graham, Dietz and Associates Consulting Engineers, entitled “Wind River Irrigation Project Study Compilation Final Report – GDA - 2000.” That compilation included a number of references that are specifically relevant to Ray Lake, and, as such, are also included in this bibliography.

This annotated bibliography is intended to document and summarize the previous studies and related work associated with Ray Lake Dam and Reservoir dam safety and enlargement modifications. Key documents and studies are summarized in chronological order to summarize the history and progression of the Ray Lake studies and designs.

1924 - 1983
Ray Lake Dam was constructed in 1924-1925. There is essentially no documentation of the original design and construction, or records of maintenance or operation of the dam in the period between its construction and the first Safety Evaluation of Existing Dams (SEED) inspection by U.S. Bureau of Reclamation (USBR) on August 16, 1983.


Summary
Between 1983 and 1992 the USBR conducted several SEED examinations of Ray Lake Dam, and performed a number of additional analyses and studies that are documented in technical memoranda. Table 1 lists the USBR studies and key findings in chronological order.
Table 1. Chronology of USBR Dam Safety Activities at Ray Lake Dam

<table>
<thead>
<tr>
<th>Date</th>
<th>Action</th>
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<tr>
<td>8/16/83</td>
<td>Performed on-site SEED Examination No. 1</td>
</tr>
<tr>
<td>9/20/83</td>
<td>Approved Probable Maximum Flood</td>
</tr>
<tr>
<td>3/29/84</td>
<td>Signed TM No. SAR-1632-42 (Seismotectonic Issues)</td>
</tr>
<tr>
<td>3/30/84</td>
<td>Issued Initial SEED Report for examination No. 1</td>
</tr>
<tr>
<td>8/27-29/84</td>
<td>Field examination for geotechnical issues</td>
</tr>
<tr>
<td>9/29/84</td>
<td>Examination of outlet conduit by divers</td>
</tr>
<tr>
<td>11/2/84</td>
<td>Drill 2 test borings adjacent to outlet and install porous-tube piezometers</td>
</tr>
<tr>
<td>11/29-30/84</td>
<td>Dewatering and field examination of outlet works</td>
</tr>
<tr>
<td>12/7/84</td>
<td>Removal of debris from outlet conduit and repair of crack in conduit</td>
</tr>
<tr>
<td>4/26/85</td>
<td>Complete laboratory testing of embankment and foundation soils</td>
</tr>
<tr>
<td>5/17/85</td>
<td>Signed TM No. RL-230-1 (Geotechnical Issues)</td>
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<td>Signed TM No. RL-230-1A (Geotechnical – Conduit Crack Evaluation)</td>
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<td>Signed TM No. SOD-RLD-222-1 (Hydrologic/Hydraulic Issues)</td>
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<td>6/13/85</td>
<td>Signed TM No. SOD-RLD-222-2 (Structural Issues)</td>
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<td>6/4/86</td>
<td>Performed on-site SEED Examination No. 2</td>
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<tr>
<td>10/1/86</td>
<td>Issued Report for Examination No. 2</td>
</tr>
<tr>
<td>6/22/88</td>
<td>Performed on-site Examination No. 3 (Special Examination)</td>
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<tr>
<td>8/12/88</td>
<td>Issued Report for Examination No. 3 (Special Examination)</td>
</tr>
<tr>
<td>9/6/88</td>
<td>Prepared Downstream Hazard Classification (Leedshill-Herkenhoff, Inc.)</td>
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<tr>
<td>7/20/89</td>
<td>Performed on-site SEED Examination No. 4</td>
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<tr>
<td>4/26/90</td>
<td>Issued SEED Report for Examination No. 4</td>
</tr>
<tr>
<td>7/12/90</td>
<td>Performed on-site Examination No. 5 (Special Examination)</td>
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<td>9/18/90</td>
<td>Issued Report for Examination No. 5 (Special Examination)</td>
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<tr>
<td>10/3/90</td>
<td>Approved Revised Probable Maximum Flood</td>
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<td>Signed TM No. 3610-91-11 (Revised Seismotectonic)</td>
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<td>Signed TM No. BIA-RLD-3110-1 (Hydrologic/Hydraulic)</td>
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<td>11/7/91</td>
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<td>3/31/92</td>
<td>Issued SEED Report</td>
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<tr>
<td>1/14/93</td>
<td>Issued Intermediate SEED Examination Report</td>
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A synopsis of the key findings contained in the USBR reports and technical memos are provided as follows:
1. Report documents SEED team assignment conducted between August through September, 1983.
2. On-site examination of Ray Lake Dam was performed August 16, 1983. Reservoir water surface elevation 5524.
3. Ray Lake dam considered to represent a **significant hazard** according to USBR guidelines.
4. Recommended development of Standing Operating Procedures (SOP) to cover operation for authorized personnel and attendance under adverse conditions.
5. Recommended development of outlet works discharge curve.
6. Preliminary hydrologic assessment indicated dam is unsafe and would be overtopped or breached by flow over the unlined spillway during the Inflow Design Flood (IDF), which is equivalent to the Probable Maximum Flood (PMF).
7. PMF analyses indicated peak discharge = 16,900 cfs and 72-hour volume = 6,735 acre-feet.
8. Regional geologic data review indicated bedrock is composed of marine shale with lenticular sandstone beds of the Cody Shale Formation of Upper Cretaceous age. Formation is dipping a few degrees in an easterly direction. No records were found to show that the dam foundation was ever investigated or grouted, or that a geology report was prepared. Foundation preparation was concluded to have been satisfactory based on performance to date.
9. Flat reservoir floor covered with weathered Cody Shale is impervious and effectively seals the reservoir.
10. No known active faults. Project in area of low seismicity.
11. **No seepage** known to have occurred through embankment, abutments or foundation.
12. Normal freeboard (8 feet) is adequate.
13. Riprap generally satisfactory, but some minor areas of beaching were observed.
14. Adequate erosion protection on crest and downstream face provided by grass and weeds. Recommended removal of one cottonwood tree, and several cottonwood and willow tree shoots on upstream face.
15. No misalignment, slumps, bulges, sinkholes, or other significant distortions.
16. Spillway is an unlined notch on the crest of the dam. Noted as an unsatisfactory configuration.
17. **No instrumentation.** Recommended installing instrumentation to meet minimum guidelines approved by the Bureau of Indian Affairs (BIA).
18. Ray Canal intake control structure in disrepair. Failure of this structure would result in uncontrolled releases into Ray Lake. Not considered dam safety concern due to dam’s high visibility.
19. Slide gate on outlet works leaks considerably. Mechanical features judged to be in fair condition.
20. Recommendations:
   a. 83-SOD-A – Develop outlet works discharge curve
   b. 83-SOD-B – Route PMF to verify hydraulic adequacy of spillway
   c. 83-SOD-C – Evaluate capability of spillway to safely pass large or sustained discharges if PMF routing indicates flows would occur
   d. 83-SOD-D – Evaluate need for seismotectonic study

**1984 Initial SEED Examination Report** (March 30, 1984 memorandum from Chris Veesaert to Chief, Inspections Branch, U.S. Bureau of Reclamation, Denver, CO)
1. Report describes geotechnical engineering Safety of Dams (SOD) concerns resulting from a field examination conducted between August 27 and 29, 1984, by USBR personnel, and follow-on analyses, inspections and activities.

2. The following SOD concerns and responses are summarized in the documents:
   a. 84-SOD-AB – Determine erosion characteristics of the unlined notch spillway in the crest of the embankment. Response: Spillway notch is an emergency feature that may erode rapidly if the water surface in the reservoir exceeds the bottom of the notch. The rate that erosion will occur can only be estimated by present engineering techniques.

   b. 84-SOD-AC – Determine the cause of transverse and longitudinal cracks on the crest of the embankment and develop a remedial treatment method that will prevent cracking in the future. Response: Minor cracking was observed during initial (August 16, 1983) SEED examination. Major cracking was discovered during the August 27-29, 1984 inspections. Additional cracks were found during the November 29-30, 1984 examination of the outlet works. Apparently, the development of cracks accelerated, for some unknown reason, during the year following the initial SEED examination. Maximum crack depth was estimated to be about 6 feet below the crest of the dam, which coincides with the normal reservoir water surface level. Cracks are more common in the vicinity of the outlet works.

   c. 84-SOD-AD – Establish a seepage monitoring and instrumentation system to determine effects of the seepage adjacent to the control works and along the embankment-foundation contact plane. Response: Active seepage areas were identified during the August 27-29, 1984 site inspection. Area with most active flow was behind the outlet headwall and on both sides of the conduit, between station 3+50 to 5+00. The foundation-embankment contact seepage zone extends from about station 5+50 to station 10+50, with the most active seepage exit along the foundation at station 10+50. This seepage was not noted during the August 1983 SEED examination, but could be identified on aerial photographs dated September 1, 1982, where they did not appear to be as extensive. Piezometers were installed adjacent to the outlet conduit in November 1984.

   d. 84-SOD-AE – Determine the extent that brush and trees should be removed from the embankment to facilitate seepage monitoring and to improve dam safety. Response: Brush and weed cover interfere with observations of embankment behavior from about station 0+00 to 13+00.

   e. 84-SOD-AF – Evaluate the requirements for upstream slope protection. Response: Embankment has been severely eroded by wave action for its entire length with scarps as high as 5 feet. Replacement of riprap is necessary. In addition, there is no bedding under the existing riprap. On September 29-30, 1984, divers found 2 feet of cobbles and boulders, pieces of concrete, and a 55-gallon drum layering the invert of the outlet conduit upstream from the gate shaft. It was assumed that the cobbles and boulders were
washed into the outlet from the adjacent riprap by wave action. The conclusion was that
the existing riprap is undersized with respect to the wind-generated forces acting on it.

3. Significant findings from follow-on analyses to address the SOD issues are summarized as follows:
a. 84-SOD-AC – No clear explanation for the cause of embankment cracking, or for
increased cracking in the interval from 1983 to 1984. Cracks present an added hazard at
higher reservoir levels.
b. 84-SOD-AD – BIA personnel noted that seepage adjacent to the outlet works had become
more noticeable in the previous 2 years (1983-84), especially on the right side of the
conduit. Divers inspected the upstream conduit on September 29, 1984 and determined
(1) that there was no trash rack on the inlet, (2) the outlet conduit upstream from the gate
was filled to a depth of 2 feet with cobbles and boulders (mostly 8-12-inches in size),
pieces of concrete and a 55-gallon drum, and (3) a 5/16-inch wide crack was located 16
feet upstream from the gate chamber. The crack permitted ingress of water under full
reservoir head into the adjacent embankment. Additional inspections found cracks in
the conduit downstream from the cast iron gate. Leakage past the gate was estimated at
about 10 cfs under a head of 23 feet, and was suspected as being due to partial erosion of
the gate sill. Seepage was also noted on both sides of the downstream head walls.
{Note: Two, 6-inch diameter borings were drilled adjacent to the outlet conduit in
November 1984. Representative soil samples were collected and tested. A porous-tube
piezometer was installed in each boring to allow monitoring of pore pressures. The
crack located 16 feet upstream from the gate chamber was temporarily repaired in
December 1984, and the rocks and other debris were removed. Laboratory testing for
engineering and index properties of the embankment materials was completed in April
1985.}
c. 84-SOD-AF – Deficiency in larger sizes (12 to 24-inch) of riprap may be the reason for
the riprap failure, in addition to lack of bedding.

4. Conclusions:
a. Transverse and longitudinal cracks have been observed on crest of the embankment.
Transverse cracks up to 1.5-inches wide and up to 6 feet deep, extend across the crest,
and are more numerous near the outlet works.
b. Crest of dam is uneven (undulates), and differential elevations may be as much as 12
inches.
c. Seepage is flowing through cracks in the outlet headwall, from the embankment on both
sides of the outlet conduit near the headwall, and from a zone near the embankment-
foundation contact that extends from the left wingwall about 720 feet left to the bend in
the embankment. Water is seeping through transverse and diagonal cracks and
construction joints in the 4-feet wide by 4-feet high concrete conduit downstream from
the gate.
d. Divers found a 5/16-inch wide crack in the outlet conduit, 16 feet upstream from the gate
well. The crack permitted full reservoir head to act on the embankment.
e. Divers found 2 feet of cobbles and boulders, concrete, and a 50-gallon drum in the
conduit upstream from the gate.
f. Concrete has spalled at the headwall, exposing reinforcement steel bars and the
embankment; seepage exits from these places.
g. Considerable brush and a few trees are growing on the dam. Vegetation may contribute to seepage problem by providing root holes through which water can percolate. Removal of trees and brush will facilitate monitoring seepage and cracking conditions.

1985 Memorandum, Subject: Physical Properties Test Results (Index Tests) - Ray lake Dam (April 25, 1985, memo to Chief, Embankment Dams Branch from Chief, Geotechnical Branch; U.S. Bureau of Reclamation, Denver, CO)

1. Memo transmits results from geotechnical laboratory tests performed on specimens from push tube samples obtained from 2 borings drilled in November 1984, adjacent to the outlet works. Purpose of testing was to provide data on physical properties of the soil for use in evaluation of repairs to the outlet works and probable future modifications to the embankment.
2. 24 push tube samples were obtained from boring B-1 and B-2. This report provides geologic boring logs and results from the following lab tests: gradation analyses, Atterberg limits, specific gravity, moisture and dry unit weight. Results for samples tested for petrographic analyses, and shear strength are provided in separate memos.
3. Index test results are summarized on a table and in figures provided in the report. All samples except one classified as CL according to the Unified Soil Classification System (USCS). One sample classified as CL-CH.

1985 Memorandum, Subject: Petrographic Examination and Soluble Sulfate Test of Soil Samples - Ray lake Dam (March 15, 1985, memo to Head, Soil Mechanics Section from Head, Chemistry, Petrography and Chemical Engineering Section; U.S. Bureau of Reclamation, Denver, CO)

Memo transmits results of petrographic and sulfate tests on two specimens. Purpose of examinations was to provide mineralogical composition with emphasis on clay mineral content, and water-soluble sulfate content.

1985 Memorandum, Subject: Physical Properties and CU (Consolidated-Undrained) Triaxial Shear Test Results - Ray lake Dam (May 9, 1985, memo to Chief, Embankment Dams Branch from Chief, Geotechnical Branch; U.S. Bureau of Reclamation, Denver, CO)

1. Memo transmits results from testing of select, undisturbed soil samples for shear strength using the CU triaxial testing procedure. Memo provides details of test procedures and analysis of results.
2. Results from these tests indicated the following shear strength parameters

**CU Triaxial Shear Strength Summary**
<table>
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<tr>
<th>Sample</th>
<th>USCS</th>
<th>$G_s$</th>
<th>$\phi'$</th>
<th>$c'$ (psi)</th>
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<tr>
<td>62H-45</td>
<td>CL</td>
<td>2.70</td>
<td>23.2$^\circ$</td>
<td>6.4</td>
</tr>
<tr>
<td>62H-47</td>
<td>CL</td>
<td>2.67</td>
<td>21.5$^\circ$</td>
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<td>62H-50</td>
<td>CH-CL</td>
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<td></td>
</tr>
<tr>
<td>62H-51</td>
<td>CL</td>
<td>2.74</td>
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**1985 Memorandum, Subject: Geologic Input to Division of Dam and Waterway Design**  
**Technical Memorandum for SEED Analysis Report - Ray lake Dam**  
(April 10, 1985, memo to Dam Safety Coordinator from Chief, Division of Geology; U.S. Bureau of Reclamation, Denver, CO)

1. Memo provides brief summary of regional geology based on literature review.  
2. According to regional geologic maps, Cody Shale comprises bedrock at the site. Literature describes the formation as gray, soft marine shale with lenticular, gravelly sandstone beds. It may contain some bentonitic shales. Formation is nearly horizontal. No outcrops occur near the dam and reservoir.  
3. Review of bedrock samples from 2 borings drilled in November 1984 that penetrated the embankment and about 5 to 7 feet of the bedrock, indicated ½ to 1-inch lenses of soft to firm silt and clay with many thin (less than ¼ inch) lenses of lighter-colored, very fine sand or coarse silt. Pockets of similar, very fine sand were noted in the core and photographs.  
4. Bedrock materials sampled are not shale (as noted on the geologic maps), but rather interbedded claystone and siltstone as noted on the drill logs. **Apparently, the foundation for the outlet structures was bedrock. It is unlikely that the embankment was founded on bedrock, but rather residual soil developed on the Cody Formation.**

**1985 Technical Memorandum RL-230-1A, Subject: Analysis of Geotechnical SOD Issue: Open Crack 16 Feet Upstream from the Gate Chamber**  
(memo to BIA, from Division of Dam and Waterway Design, U.S. Bureau of Reclamation, Denver, CO; 1st draft was prepared 1/8/85, text revised 5/1/85, 7/22/86, and minor editorial changes made 7/26/91)

1. Memo summarizes evaluation of cause of open crack in the outlet conduit, 16 feet upstream from the gate chamber.  
2. Dive team confirmed existence of crack on September 19, 1984. Crack was open about $5/16$ -inch at the bottom of the conduit and about $1/4$ - inch at a point 12 inches above the bottom of the conduit. Crack was permitting reservoir head to act on the adjacent embankment fill about 60 feet downstream from the upstream toe. Seepage at the time was exiting the downstream slope of the embankment on both sides of the outlet conduit in the wingwall/headwall area. Seepage has been increasing since 1982, and the embankment in this vicinity was soft.  
3. Consensus based on discussions among BIA officials and USBR engineers assigned cause of cracking to either settlement of the structure into the foundation material, or lateral spreading of the embankment on the foundation-embankment contact plane, or a combination of both.
4. On December 7, 1984 the crack was sealed with a patented, fast setting compound “Waterstop” that will seal and set in the presence of water.

5. Construction joints downstream from the gate chamber were also seeping water. One of the joints is near the headwall in the area where concrete has been most severely damaged by sulphate reaction.

6. Stress analyses indicated that the openings in the outlet conduit are at locations where there is an appreciable differential of effective stress. Alternate cycles of consolidation and expansion of the Cody Formation on which the conduit is founded may have caused the failure.

7. Known high sulfate content of the soils and seepage may have reduced the strength of the 1920 concrete since type II cement was probably not used. The extent of sulfate attack will be determined from concrete cores that were sampled from the conduit.

8. Recommendations were to conduct tests on the embankment and foundation soils for consolidation and expansion characteristics, and tests to determine the extent of sulfate attack on the concrete cores.

1985 Technical Memorandum SOD-RLD-222-2, Subject: SEED Analysis of Structural Issues (June 13, 1985, memo to BIA from Division of Dam and Waterway Design; U.S. Bureau of Reclamation, Denver, CO)

1. Report describes recommendations related to possibility of structural failure of the outlet works due to progressive deterioration of the concrete. The analysis is based on site inspections in August, September and November, 1984, and review of available data at the time.

2. Deterioration is most likely to be caused by a combination of very severe sulfate attack, freeze-thaw, and frost action.

3. Failure of the concrete stilling basin and/or conduit could lead to downstream slope failure in the embankment dam. The crack in the upstream conduit is a major concern because seepage through this crack could cause internal erosion in the dam. Seepage is most likely contributing to the deterioration of the concrete downstream.

4. The following structural SOD recommendations were made as a result of analyses performed:
   a. 84-SOD-A G - Evaluate structural damage and potential for structural failure of the outlet works. Response: Transverse cracking and damaged concrete were observed in the outlet works conduit and stilling basin. Initial damage is probably due to settlement and spreading of the embankment. Further damage is probably related to seepage in these areas and caused by a combination of sulfate attack, alkali-aggregate reaction, freeze-thaw and frost action. Lab tests confirmed presence of sulfate in the soil and seepage, and sulfate attack in the concrete.
   b. 84-SOD-A H - Evaluate condition of the outlet works slide gate. Response: There has been concern over the condition of the outlet works slide gate and gate frame since onset of excessive leakage. Inspections have revealed damage to the brass gate seal. Leakage through the seal may cause excessive buildup of ice in the downstream conduit and stilling basin.

5. Analysis: Historical performance of outlet works was satisfactory until about 1982, based on discussions with the water master for Ray Lake Dam. At that time, excessive leakage
beneath the slide gate became evident. Comparison of photographs between 1st SEED examination and in August 1983 and examinations in August 1984 showed an increase in concrete deterioration. Leakage through gate also increased during that time frame. Concrete repair work at some unknown date was evident on downstream end of stilling basin walls. Deterioration of downstream conduit had progressed so that reinforcement bars and embankment material were exposed.

6. Repair of the upstream crack using “Water Stop” compound was viewed as a temporary repair by USBR, because the durability of this material is unknown.

7. Sulfate concentrations in the soil and water samples were found to be 3.81% and 6499 mg/L, respectively. This level of sulfate concentration will result in severe sulfate attack. Petrographic examination of three concrete core samples taken from the outlet works conduit revealed that alkali-aggregate reactions and sulfate attack has occurred in one of the cores.

8. It is impossible to prevent further sulfate attack without either replacing the existing concrete with sulfate-resistant concrete, or preventing contact between the concrete and the seepage or soil containing high concentrations of sulfate.

9. Replacement of the conduit may not be necessary if the seepage can be stopped and repairs are made.

10. The slide gate seal for the bottom of the gate is not intact. The loose brass seal may cause the gate to jam during an emergency. Pins on the gate stem were badly corroded.

11. Conclusions: Many of the structural problems are related to seepage through the embankment. Seepage problems should be eliminated. Concrete in the outlet works stilling basin and conduit is in poor condition. Without repair or replacement, the damage will probably progress to the point of failure. At the time the dam was built, sulfate-resistant concrete materials were not available.

1986 2nd SEED Examination Report (October 1, 1986 memorandum from R. C. Rocklin to Chief, Inspections Branch, U.S. Bureau of Reclamation, Denver, CO)

1. SEED team assignment period June 1986.
3. Hazard classification unchanged (significant hazard).
4. Lack of operating instruction for Ray Lake Dam is unsatisfactory.
5. PMF approved on September 20, 1983 should be routed through reservoir and spillway evaluated.
6. Evidence of seepage observed on ground surface downstream from the dam on the left side of the outlet works. This was not observed during first SEED examination, when reservoir water surface was 3.4 feet lower.

7. Seepage observed on downstream face of dam adjacent to left outlet works stilling basin wingwall. Water seeping over top of wingwall and entering stilling basin. Seepage was observed in this area in 1984, and no evidence that the seepage rate had changed. Seepage attributed to cracks in the upstream outlet works conduit; however, cracks were repaired during rehabilitation of the outlet works in 1985. Seepage may be flowing along outside of the conduit. No evidence of piping.
8. Upstream face has suffered a significant amount of erosion in places due to wave action. Willows growing on upstream face of the dam. Recommended placing riprap slope protection.

9. Small crack observed in downstream face in fill material placed during installation of piezometers in 1984. No cracks observed during this 1986 examination. Probable cause attributed to desiccation shrinkage and cracking.

10. Outlet works has suffered a significant amount of deterioration. This includes cracks in the upstream conduit and spalls and cracks in the downstream conduit and stilling basin. Repairs were made to the upstream conduit in 1985, but repairs are still required to the concrete in the downstream conduit and stilling basin. Outlet works considered structurally sound at the time.

11. Two piezometers were installed near the outlet works. After 10 months of readings taken between November 1984 and September 1985, performance was considered satisfactory.

12. Mechanical equipment considered to be in satisfactory condition, although experienced problems due to trash accumulation. Recommended addition of a trash rack.

13. New (1986) SOD Recommendations included:
   a. 1986-SOD-A – Determine cause of seepage on downstream side of the dam and evaluate its effect on safety of the dam.
   b. 1986-SOD-B – Determine cause of cracking in the dam embankment and evaluate its effect on safety of the dam.

1988 Special SEED Examination Report (August 12, 1988 memorandum from R. C. Rocklin to Chief, Inspections Branch, U.S. Bureau of Reclamation, Denver, CO)

1. Special SEED team assignment for purpose of examining seepage below the dam and cracking in the dam embankment.
2. Special on-site examination June 22, 1988. Reservoir water surface elevation 5527 (approximate)*.
3. No seepage observed on downstream left side of outlet works wing wall, in area where seepage had been reported in the previous (October 1986) SEED report. Reservoir elevation about 5 feet lower* than elevation at time of 1986 SEED examination. *(NOTE: This contradicts reported elevation of 5527. Reservoir elevation during October, 1986 inspection was 5527.4. Suspect that what they meant to refer to was reservoir levels at time of inspections in 1984, when significant seepage was observed.)*
4. Damp area observed downstream from dam between left side of outlet works and curve in the embankment several hundred feet to left of the outlet works. Attributed to diffuse seepage through dam and foundation. Unchanged since 1986 SEED inspection.
5. Longitudinal cracks observed on crest of the dam. Cracks of limited length and depth attributed to desiccation shrinkage of embankment materials.
6. Recommended subsequent examinations be done in spring when reservoir water surface is at its highest elevation to determine if seepage flows previously observed are still occurring at higher reservoir levels.
1. Purpose of report was to present downstream hazard classification and means and rationale used in its determination.
2. Ray Lake Dam classified as **significant hazard**. Failure of dam has the potential for loss of life in the flood plain below the dam.
4. Methodology: downstream hazard classification based on inundation study performed using National Weather Service Simplified Dam Break Model (SM PDBK). Topographic and physical characteristics of the dam for input into SM PDBK were obtained from the Ray Lake 7.5-minute quadrangle map, and from photographs and data obtained by USBR on-site examinations.
5. Dam was assumed to fail by overtopping caused by hypothetical storm. Dam assumed to breach in right abutment at location of large drainage channel below the outlet works.
6. Floodwave was routed downstream to a point about 1.2 miles downstream from the dam.
7. Peak breach outflow estimated as 32,640 cfs at the dam. This results in highway 287 overtopping by up to 7 feet in some areas. Routing assuming failure of the highway embankment was used for the hazard classification.
8. First residence in the floodplain would be subject to peak flow of 28,300 cfs, and flooded to depth of about 3.9 feet above the foundation.
9. Based on these results and USBR guidelines for hazard classification the downstream hazard classification is **significant**.


1. On-site examination July 20, 1989. **Reservoir water surface elevation 5524.5.**
2. Hazard classification unchanged (significant hazard).
3. Outlet works discharge curve and reservoir evacuation potential still have not been determined.
4. No SOP or EPP. Recommendations for this O&M item have been made previously.
5. PMF was developed in 1983. Hydraulic adequacy of spillway not yet determined (1983-SOD-B).
6. Evidence of past seepage, but no seepage observed during this examination. Damp soils and alkali deposits on ground surface downstream from the toe of the dam, both to the left and right of the outlet works indicate seepage through foundation.
7. A few, short desiccation cracks, both longitudinal and transverse, were observed in the dam crest to the left of the outlet works. Cause of cracking has never been determined and remains a dam safety concern.
8. Downstream portion of outlet works was examined, and found minor surface spalling and erosion of concrete near the slide gate, but no leaks through cracks or joints in the roof and walls of the conduit. Opaque water was present from leakage of slide gate. Unknown if color of water was indication of reservoir water quality or indication of potential dam safety concerns related to seepage.

9. Two, porous-tube piezometers located 5 feet to the left of the outlet works. No monitoring of these piezometers since September 13, 1985.

10. New SOD recommendation: 1989-SOD-A – Implement program for sampling reservoir water and gate leakage to determine whether opacity of water is related to reservoir conditions or to other causes which might threaten the safety of the dam.


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**1990 Special SEED Examination Report** (September 18, 1990 memorandum from Leon E. Faris to Head, Dam Safety Inspection Section, U.S. Bureau of Reclamation, Denver, CO)

1. Special on-site examination July 12, 1990, for purpose of examining seepage below the dam and cracking in the dam embankment

2. **Reservoir water surface elevation 5524.5.**

3. Recent monitoring indicates **seepage appears when reservoir elevation reaches approximately 5526 feet.** No seepage observed during this inspection with reservoir about 1.5 feet below this elevation. Area to left of outlet works was damp. Area along downstream toe between left side of outlet works and curve in embankment was dry.

4. Lack of adequate slope protection on upstream face of dam has resulted in erosion in a number of areas. Erosion has encroached several feet into the crest of the dam at a location approximately 60 feet to the right of the outlet works. Damage also observed in several other locations.

5. No significant changes observed in cracks in the embankment.

6. Vehicle traffic has damaged downstream face of the dam.

7. Crest survey is needed. Crest elevation from recent survey of piezometers indicates crest elevation at 5532 feet, not 5534 feet as documented in previous SEED reports.

8. The following previous SOD recommendations are incomplete: (1989-SOD-A, 1986-SOD-A, 1986-SOD-B)

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**1990 Technical Memorandum Subject: Revised Probable Maximum Flood (PMF) for Ray Lake Dam, Wyoming,** (October 3, 1990, memo to Chief, Geotechnical Engineering and Geology Division, from Manager, Planning Services Staff, U.S. Bureau of Reclamation, Denver, CO)

1. Report details computations and results of an updated PMF study. The update was necessary due to changes in meteorologic criteria published in 1988, and it was expected that the revised values would be somewhat smaller. The previous PMF study was completed in 1983.
2. Two PMF events were computed by these analyses as follows:

<table>
<thead>
<tr>
<th>Type of Flood</th>
<th>Peak Flow (cfs)</th>
<th>Volume (acre-feet)</th>
<th>Duration (hours)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Local Thunderstorm PMF</td>
<td>12,200</td>
<td>2,800</td>
<td>15</td>
</tr>
<tr>
<td>General Storm PMF</td>
<td>14,000</td>
<td>6,800</td>
<td>72</td>
</tr>
</tbody>
</table>

3. Peak discharge was reduced from previous PMF study (1983) from 16,900 cfs to 14,000 cfs, but there was no significant change in volume.


1. Technical memo serves as addendum to SEED Analysis of Hydrologic/Hydraulic Engineering Issues technical memorandum for Ray Lake Dam, which addresses recently (October, 1990) approved PMF study. This addendum describes flood routings to determine adequacy of Ray Lake Dam to safely pass the PMF and smaller floods.
2. Failure Potential Assessment: High failure potential is assessed due to inability of the dam and appurtenant structures to safely accommodate the PMF. Erosion of the dam embankment near the spillway is very probable since spillway is unlined notch cut in the embankment.
3. These analyses were done in response to 90-SOD-AA - perform flood routing studies to determine adequacy of dam to safely pass the current PMF. Results of the PMF study (October 3, 1990) and this memo documenting flood routing indicate that the dam cannot accommodate either a general storm or local thunderstorm PMF event.
4. Methodology: Routings began at normal water surface elevation 5526.0 (1 foot below the spillway crest).
5. Conclusions: General storm PMF is the governing flood event; this PMF will overtop the dam embankment for approximately 4 hours at a maximum depth of 7 inches. Dam and appurtenances could pass 84% and 40% of the PMF with 0 feet and 3 feet of freeboard, respectively. These results are based on assumption that the embankment and spillway would not fail. However, the unlined spillway is likely to erode if used and could subsequently cause dam failure. Therefore the dam can safely store 17% of the PMF below spillway crest (i.e., between elevations 5526.0 and 5527.0).

2. The conclusions of the Geomatrix report concerning seismic sources and maximum credible earthquakes (MCEs) differ from the 1984 SEED technical memorandum and supersede those values.

3. The following specific conclusions should be used for analysis of Ray Lake Dam:
   a. Stagner Fault, a 38-km long normal fault located along the south flank of the Owl Creek Mountains, is the only potentially active fault of significance to Ray Lake Dam. The estimated MCE for the Stagner Creek fault is an \( M_s 6\frac{3}{4} \) event at a focal depth of 10-20 km and an estimated epicentral distance from Ray Lake Dam of 60 km. The fault has an estimated recurrence interval of 8000 to 20,000 years.
   b. The following random earthquakes and corresponding probabilistic epicentral distances should be used for analysis of Ray Lake Dam:

<table>
<thead>
<tr>
<th>Magnitude</th>
<th>10^{-5}</th>
<th>2 \times 10^{-5}</th>
<th>10^{-4}</th>
<th>10^{-3}</th>
<th>Focal Depth (km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.5</td>
<td>24</td>
<td>34</td>
<td>75</td>
<td>239</td>
<td>10-15</td>
</tr>
<tr>
<td>6.0</td>
<td>13</td>
<td>18</td>
<td>40</td>
<td>126</td>
<td>10-15</td>
</tr>
<tr>
<td>5.5</td>
<td>7</td>
<td>9</td>
<td>21</td>
<td>66</td>
<td>5-15</td>
</tr>
<tr>
<td>5.0</td>
<td>3</td>
<td>5</td>
<td>11</td>
<td>35</td>
<td>5-15</td>
</tr>
</tbody>
</table>

   c. Random events with magnitudes smaller than the 6.5 magnitude random MCE could occur at smaller epicentral radii, and consequently, could produce higher peak accelerations than the random MCE. A random earthquake smaller than the MCE could be the controlling event, and should be considered in analysis of the dam.
   d. No new data to suggest landsliding, liquefaction, reservoir-induced seismicity, or surface fault displacement are significant hazards to Ray Lake Dam.
1991 Special SEED Examination Report (September 4, 1991 memorandum from R.C. Rocklin to Head, Dam Safety Inspection Section, U.S. Bureau of Reclamation, Denver, CO)

1. Report documents a special on-site examination of Ray Lake Dam performed August 7, 1991. Reservoir water elevation 5523.9 feet. Restricted operating level 5525.0 feet.
2. Purpose of examination was to examine seepage conditions and embankment for signs of instability and cracking to see if conditions have changed since previous examinations.
3. Conclusions:
   a. No evidence of active seepage during the examination. Currently (1991) reservoir is being restricted to elevation 5525 because of seepage concerns. Areas downstream from the dam are damp and salt encrusted which indicates a high ground water table at the toe of the dam caused by seepage from the reservoir. **Seepage does not appear to be a dam safety deficiency if the reservoir elevation remains at 5525 feet or lower. However, if the reservoir elevation rises above 5526, which could occur during flood flows, then seepage conditions could deteriorate to the point that the dam could be endangered.**
   b. Desiccation cracking near the dam crest was observed near the outlet works during this examination. Cracking appeared to be random orientation and shallow in depth. A longitudinal crack was observed on upstream crest near the bend in embankment. This crack was noted in previous (1989 SEED) examination. Crack is attributed to settlement of dumped fill that had been placed to prevent erosion.
   c. Erosion of upstream face is occurring in vicinity of the outlet works, and is caused by wave action during windy conditions. Erosion has reduced the cross section at this location and is of concern.
4. No new SOD recommendations as a result of this examination.

1991 Deficiency Verification Analysis (DVA) (August 30, 1991 memorandum to Billings Area Office from Division of Geotechnical Engineering & Geology, U.S. Bureau of Reclamation, Denver, CO)

1. Purpose: The purpose of this “decision memorandum” is to document which of the potential dam safety deficiencies that have been identified for Ray Lake Dam present a threat to the integrity of the dam and its appurtenances. Potential safety of dams (SOD) deficiencies were identified through previous SEED examinations in 1983, 1986, 1988, 1989 and 1990. This memorandum also provides closure for issues which were considered potential SOD deficiencies but have been determined not to be by further investigations and analyses.
2. Background: Each potential problem has been investigated to determine if SOD deficiencies exist, and analysis documented in various Technical Memorandums (SOD-RLD-222-1, SOD-RLD-222-2, RL-230-1, RL-230-1A, SAR-1632-42, BIA-RLD-3110-1, and 3610-01-11), and in a seismotectonic study by Geomatrix Consultants in 1988.
3. Decision: The following were considered to be dam safety deficiencies:
   a. Transverse cracking of the embankment crest.
   b. Inadequate erosion protection on the upstream face of the dam.
   c. Seepage instability along and in the vicinity of the outlet works conduit.
   d. Structural damage and the potential for structural failure of the outlet works.
   e. Hydraulic inadequacy of the spillway to pass the PMF.

2. Downstream hazard classification (significant) is unchanged.

3. Emergency preparedness considerations (communication system, access to the site, security, operating instructions, and reservoir evacuation potential) are considered satisfactory.

4. Updated PMF study (1990) indicates dam will be overtopped during PMF. The Deficiency Verification Analysis (DVA) concluded that overtopping of the dam due to inadequate spillway capacity is a dam safety deficiency.

5. No outstanding dam safety concerns regarding geologic and seismic considerations.

6. The DVA concluded that seepage exiting from cracks in the outlet conduit has the potential to cause internal erosion of embankment materials if sufficient head is present. Seepage instability of the dam embankment in the vicinity of the outlet works constitutes a dam safety deficiency.

7. No significant changes in condition of the embankment, which is considered to be fair. Upstream face requires periodic addition of riprap protection to retard wind and wave action erosion. Desiccation crack observed in previous examinations have swelled shut due to wet conditions. The DVA concluded that cracks in the embankment constitute a dam safety deficiency.

8. Additional dam safety deficiencies include: inadequate capacity and potential erosion of the spillway during large flood flows, and potential structural failure of outlet works due to concrete deterioration.

9. Plots of two porous tube piezometers indicate that readings vary in direct proportions to the reservoir surface elevation and show no unusual behavior.

10. No new SOD recommendations as a result of this examination.
1993 DESIGN STUDIES FOR SAFETY OF DAM (SOD) AND ENLARGEMENT PROJECTS (11,450 and 20,000 ACRE-FEET) BY GEI CONSULTANTS

Summary

GEI Consultants Inc. (GEI) developed conceptual designs for dam safety modifications to the existing dam (GEI, 1993a), and conducted a feasibility study to enlarge the dam and reservoir in combination with the dam safety modifications (GEI, 1993b). The dam safety modification project was intended to address deficiencies previously identified by the U.S. Bureau of Reclamation (USBR) in their SEED examinations conducted between 1983 and 1992 that included embankment cracking, seepage problems and structural damage along the outlet conduit, inadequate erosion protection on the upstream slope, and hydraulic inadequacy of the spillway to pass the probable maximum flood (PMF). The scope of work included preliminary geotechnical and hydraulic analyses, Class III cultural resource evaluations, and reconnaissance-level assessments of threatened and endangered species.

The enlargement project that was being considered at the time of the 1993 studies was an 8 ft raise of the normal pool elevation, from elevation 5526 to 5534 ft, which would increase storage from 6,980 ac-ft to 11,450 ac-ft. This option would avoid impacting residential structures and Highway 287. Alternative designs for the 11,450 ac-ft enlargement project incorporated structural modifications to address the dam safety issues.

Later in the GEI (1993b) study, an enlargement to 20,000 ac-ft was evaluated at a reconnaissance level. GEI (1993b) reported that the 20,000 ac-ft enlargement would require complete rebuilding of the main dam, plus several saddle dikes on the reservoir rim, relocation of a portion of Highway 287, relocation of one residence, and a new emergency spillway on the southwest rim of the reservoir.


1. The report presented in 3 volumes describes conceptual designs for dam safety modifications to address dam safety deficiencies identified by previous studies by USBR that were summarized in the Deficiency Verification Analysis (DVA) report dated August 30, 1991, as well as an additional stability deficiency identified by GEI in their investigations.
   a. Volume I provides the text, tables and figures summarizing the project background, conceptual design alternatives, hydrologic/hydraulic evaluations, geotechnical evaluations, structural evaluations, permitting requirements, construction cost estimates, and final design recommendations.
   b. Volume II contains three technical memoranda that present detailed analyses and results for 1) Downstream Hazard Assessment, 2) Hydraulic/hydrologic Evaluations, and 3) Geotechnical Evaluations.
   c. Volume III presents detailed construction cost estimates including quantity estimates, unit price development, and summary tables.
2. Three alternatives were developed and evaluated in this study, as follows:
   a. **Alternative No. 1** - Replace the outlet works with a new outlet works structure and modify the dam and auxiliary spillway to safely pass the PM F.
   b. **Alternative No. 2** - Replace the outlet works with a new outlet works/service spillway structure and raise the dam to temporarily store the PM F. In this alternative, the flood would be passed through the service spillway.
   c. **Alternative No. 3** - Breach the main embankment dam.

3. Design alternatives were intended to address SOD deficiencies identified by USBR, plus additional SOD deficiencies identified by GEI. These deficiencies included the following:
   a. D1: Random cracking of the dam crest.
   b. D2: Inadequate erosion protection on the upstream face of the dam.
   c. D3: Seepage instability along and in the vicinity of the outlet works conduit.
   d. D4: Structural damage and potential for structural failure of the outlet works.
   e. D5: Hydraulic inadequacy of the spillway to pass the PM F.
   f. D6: Embankment erosion during spillway discharges.
   g. D7: Adverse seepage through the dam foundation.

4. Alternative No. 1 was recommended as the preferred alternative because it provided the most cost-effective design, and had the least environmental impacts. Alternative No. 1 involved the following construction elements:
   a. Replace the existing outlet works with a new structure (corrects for deficiencies D3 and D4). The new structure would be designed and constructed to minimize the impacts of seepage along the conduit. In addition, one operation and maintenance (O&M) concern regarding debris accumulation in the outlet would be corrected by installation of a trash rack.
   b. Enlarge the emergency spillway to have a crest elevation at 5527 feet, and a crest width of 700 feet (corrects deficiencies D5 and D6).
   c. Replace the upstream slope protection (corrects deficiency D2). This would involve removal of existing riprap on the upstream slope, regrading the slope, and replacing new riprap, or placing soil-cement or roller-compacted concrete (RCC) facing instead of riprap. As part of upstream slope regrading, up to three feet of the main embankment dam crest would be removed and replaced with materials resistant to shrinkage cracking (corrects deficiency D1). This will also level the crest elevation which has been cited as an O&M concern. Dam crest would be raised to elevation 5532.5 for riprap revetment option, or to 5533.5 for soil-cement/RCC options to provide adequate freeboard for wind and wave run-up from maximum reservoir pool.
   d. Place a stability berm along the downstream toe of the main dam (corrects deficiency D6). This berm is required to provide adequate factors of safety against slope failures based on analyses done as part of this study.
   e. Construct a toe drain along the downstream toe of the main embankment (corrects deficiency D7). Would be removed for construction of a toe drain and addition of a stability berm on the downstream slope.

5. Hydrologic/Hydraulic Evaluations conducted as part of this study included:
   a. Downstream hazard assessment: concluded that the Inflow Design Flood (IDF) for Ray Lake Dam should be the PM F
b. IDF: USBR identified 2 PMF events, a general storm and local thunderstorm. The general storm PMF was used for conceptual design (peak inflow = 14,000 cfs, 72-hour inflow volume = 6,800 acre-feet)

c. Freeboard analysis: Results using USBR criteria for wind and wave runup analysis indicated 3.9 and 5.9 feet of normal freeboard (above the emergency crest elevation) for riprap and soil-cement upstream slope protection, respectively.

d. Flood routings: Auxiliary (emergency) spillway crest elevation was set at 5527 so that reservoir could safely store floods up to a 100-year event without overtopping the unlined spillway. The PMF routing indicated that the required length of the auxiliary spillway is 700 feet to maintain the reservoir below elevation 5530 feet and provide the required minimum freeboard of 2.3 feet.

e. Outlet works: Design criterion was that new structure should provide the same capacity as the existing structure. Replacement structure was therefore designed as a 4-feet-square, reinforced concrete conduit. The stand-alone outlet works would consist of a low-level intake with a sloping trashrack, a 4-feet-square conduit, a pre-fabricated slide gate housed in a central control tower, and a USBR-impact basin. The tower was planned to be located immediately upstream from the embankment crest. It was noted that relocation of the tower to accommodate future enlargement could be incorporated into final designs, if necessary. An energy dissipater was selected using USBR recommended guidelines.

6. Stability analyses had not been documented in the previous SEED reporting by USBR, and so were completed as part of this study. Based on these analyses, the upstream slope was found to be stable with acceptable factors of safety for both steady seepage and rapid drawdown conditions. However, the downstream slope was found to have factors of safety lower than acceptable criteria. This was the basis for GEI’s concerns that cracking on the dam crest may be an indication of slope problems, and not simply attributable to desiccation shrinkage. Also, GEI’s conceptual designs incorporated a toe drain and stability berm to address this deficiency.

7. Permitting and NEPA Compliance: In anticipation of NEPA compliance issues, a reconnaissance-level cultural resource evaluation and biological assessment were conducted.

8. Cultural resource studies: Several cultural sites are located around the fringe of Ray Lake. All sites are located above the normal water surface elevation of Ray Lake, so no impacts to these sites from the dam safety project are expected.

9. Biological assessments: Not completed at the time of this report.

10. Construction costs estimates: The estimated total project costs for SOD modifications range from $1.5 to $3.2 million (1993 dollars), not including the estimated costs to mitigate the loss of water supply during construction. GEI recommended project funding be based on Alternative 1B (soil cement or RCC upstream facing) cost estimate at $1.73 million.

11. Recommendations for final design analyses and evaluations included the following:

a. hydraulic evaluations including reservoir routing, wave runup, and freeboard analyses, and optimizing the design of the auxiliary spillway,

b. geotechnical evaluations including 1) seepage stability and need for a chimney drain connected to the proposed toe drain system, 2) filter compatibility of the chimney/toe drain system, 3) further evaluation of upstream slope protection alternatives, and 4) pavement design for the replacement of Ray Lake Road,
c. geotechnical investigations to identify appropriate borrow materials for construction and the extent of crest and dam modifications necessary to address crest cracking and wave damage concerns,
d. structural and hydraulic analyses for design of the outlet works structure,
e. evaluation of alternatives for emergency dewatering of the outlet works structure including an emergency slide gate and bulkheads,
f. resolution of right-of-way considerations with respect to U.S. Highway 287, Ray Lake Road, and adjacent property owners; this task will require the assistance of the Tribes, BIA, and possibly the TWE, and
g. if required, consideration of reservoir enlargement as part of the final design package.


A value engineering team comprised of USBR geotechnical, structural and hydraulics design engineers, conducted assessments and presented a report that listed 6 proposals for alternatives to the GEI conceptual design to reduce costs or improve function of the SOD construction. The USBR VE proposals and GEI comments regarding each one are summarized as follows:

1. **VE Proposal No. 1 - Use 42-inch diameter steel lining of upstream portion of outlet, in combination with replacement of the intake and downstream stilling basin as designed by GEI instead of complete removal and replacement of the outlet conduit.** GEI Response: A) Seepage concerns along the outlet structure other than from the upstream conduit are not addressed (previous inspection reports indicate cracks in the conduit are not the only source of seepage, and will not eliminate safety concerns created by past erosion of embankment materials along the conduit), B) No repairs are proposed for the tower, and seepage under and around the gate is not corrected, C) Outlet works capacity is substantially reduced, D) Durability and life of steel liner may be less than concrete, E) Potential for removal of embankment materials through cracks in the downstream portion of the conduit during high discharges is not addressed. GEI recommended that this proposal not be adopted because: 1) additional deterioration of the existing concrete is likely and this deterioration will adversely affect the long term performance of the gate and outlet works structure. Continued deterioration of the concrete may also result in the development of preferential seepage paths that could lead to a seepage failure of the dam, 2) This proposal does not address the seepage concerns and possible voids and piping along the outside of the outlet works conduit, 3) This alternative reduces the capacity of the outlet works, and 4) This proposal (in GEI opinion) represents an interim repair and not a long-term solution to the deterioration problem.

2. **VE Proposal No. 2 - Revise Riprap Design.** The conceptual design was based upon a design wind speed of 100 mph and the suggest VE modification is based upon a design wind speed of 55 mph. GEI response: Recommend adopting this proposal with following modifications: 1) Hourly wind data from regional weather stations should be used to develop a design wind speed, and 2) prevailing wind direction should be evaluated to determine if the design wind speed for riprap required on the upstream slope can be reduced below value selected.
3. VE Proposal No. 3 – Retain the PMF. This proposal is a modification to GEI concept design Alternative No. 2. The VE proposal raises the dam by over-steepening both the upstream and downstream slopes to 1H:1V using geogrids in the earthfill; crest would be capped with 1-foot thick layer of soil cement and “Jersey” barriers used at the top of the upstream slope to account for wave action. GEI response: GEI recommended eliminating this proposal from future consideration because 1) Cost over GEI’s preferred Alternative No. 1 is increased by approximately $150,000, and 2) Embankment stability has not been considered and would likely increase costs even further.

4. VE Proposal No. 4 – Optimize Soil Cement Mixtures for Conceptual Design Alternatives. This VE proposal recommends that both local borrow sources and pit run material sources from aggregate suppliers be considered to reduce the unit cost for soil cement. Also recommends that silty sand materials with up to 30% fines be considered for use in soil cement. GEI response: GEI recommends adopting this proposal as follows: 1) GEI’s regional borrow studies indicated suitable local soils are not available and that additional evaluations be limited to potential pit run sources at commercial suppliers.

5. VE Proposal No. 5A – Avoid Crest Removal and Replacement. Removal and replacement of up to 3 feet of material and replacement with low plasticity materials was proposed to repair wave damaged areas and reduce potential for desiccation shrinkage cracking that was identified as a dam safety concern. The VE proposal would eliminate all removal and replacement of desiccated and cracked crest materials. GEI response: Recommended that this proposal not be adopted. The GEI concept design recommended determining the amount of required crest removal and replacement by digging test pits to investigate extent of cracking.

6. VE Proposal No. 5B – Stability Berm. The VE proposal recommends the stability berm not be compacted, the width of the berm at the outlet works be minimized, and the berm size beyond the outlet works be maximized. GEI response: GEI recommends this proposal be adopted.

7. VE Proposal No. 6 – Use Precast Concrete Elements for New Outlet Conduit and Tower. This proposal is to replace the 4 x 4 feet cast-in-place concrete conduit with a 4 x 4 feet precast concrete conduit and to replace the cast-in-place 11 x 16 feet tower with a 4 x 4 precast elements. Water tight connections are proposed between the precast sections. GEI response: Recommended not adopting this proposal for the following reasons: 1) potential cost savings will be reduced by costs incurred for additional analyses required to design details for the tower, gate openings, gate connections, etc, and for evaluations needed to assess the reliability of, and methods to construct watertight joints, and 2) Long-term reliability of the water tight joints is a concern.


1. The report documents feasibility-level and reconnaissance-level studies for enlargement of Ray Lake.
a. Feasibility-level design (including preliminary layouts and cost estimates) was for an enlargement option that was limited to a maximum dam crest elevations that would not impact any residences adjacent to the reservoir at elevation 5540 (a raise of about 8 feet over the existing dam crest at elevation 5532). This option involved a downstream raise of the existing embankment, along with SOD modifications (replacement of the outlet works and upstream slope protection, and installation of a downstream chimney and toe drain).

b. GEI also did a reconnaissance-level study for an enlargement concept to increase the reservoir storage to 20,000 acre-feet. This concept involved complete replacement of the embankment with a new dam.

2. Feasibility-level design of 11,450 acre-feet reservoir (8 feet raise of existing dam):
   a. Raise existing dam to increase the normal water surface elevation from 5526 to 5534, increasing storage capacity by about 4,470 acre-feet.
   b. Downstream raise of existing embankment. Downstream slope would be constructed at 2.75H:1V to provide stability without a toe berm.
   c. Incorporate chimney/toe drain in downstream slope design to provide control of seepage.
   d. Flatten upstream slope to 3H:1V. Thus the upstream toe would be shifted slightly. The upstream slope would include slope protection (riprap or soil cement/RCC).
   e. Raise the berm that forms the left side of the dam by upstream construction to avoid disturbance of Ray Lake Road.
   f. Auxiliary spillway (unlined notch cut into crest of dam) sized at 725 feet wide with a crest elevation of 5535 feet. Spillway located on left side of the embankment at the same location as for SOD modifications.
   g. Downstream slope protection comprising cellular concrete mat (CCM) on the auxiliary spillway section to protect the downstream slope during flow events in the spillway.
   h. Outlet structure would be similar to one proposed for SOD modifications (replacement of the existing structure), except the height of the control tower and length of the conduit would be modified to accommodate the raise.
   i. Cost estimate for feasibility-level design (raise of existing dam): If SOD and enlargement modifications are constructed concurrently, the total cost was estimated (1993 dollars) to be approximately $3.59 million. Costs were also computed for options to construct the SOD modifications and enlargement modifications separately.

3. Reconnaissance-level design of 20,000 acre-feet reservoir (replace existing dam with new dam):
   a. Enlargement of this size would require complete rebuilding of a substantially larger main dam, several saddle dikes on the reservoir rim, relocation of a portion of U.S. Highway 287, relocation of one residence and its occupants, and a complete relocation of the emergency spillway to the southwest rim of the reservoir.
   b. New dam crest elevation at 5552 feet.
   c. Rough cost estimates were made by using same unit costs as developed for the feasibility level cost estimates (for raise of existing dam), and appropriate adjustments to percentage items (e.g. permitting and mitigation, construction management, and contingencies).
   d. Potential foundation stability concerns cannot be adequately resolved without additional field explorations. Therefore embankment quantities were estimated for a broad range of potential slope inclinations ranging from 2.5H:1V to 5H:1V downstream slopes to develop a reasonable range of unit costs per acre-foot of storage. Costs were estimated in
the range between $850 and $1,050 per acre-foot for the 13,000 additional acre-feet of storage.


The Emergency Action Plan (EAP) was prepared by GEI for the Tribes, and provides general guidelines and information for the use of existing emergency resources during an emergency at the dam. The EAP did not include a detailed Local Emergency Operations Plan (LEOP) for warning and evacuating the population at risk in the dam-break floodplain. A LEOP was included in the Washakie Dam EAP, and is referenced by the Ray Lake EAP.
1994-1996 CONCEPTUAL AND FINAL DESIGNS FOR ENLARGEMENT OF RAY LAKE AND SAFETY OF DAMS MODIFICATIONS TO RAY LAKE BY WESTERN WATER CONSULTANTS (WWC)

Between 1994 and 1996, Western Water Consultants (now WWC Engineering of Laramie, Wyoming) prepared designs in parallel for dam and reservoir enlargement options and safety of dams (SOD) modifications for Ray Lake. These efforts culminated in 1996 with plans for near final (95%) designs for both the SOD improvements, and a 35,000 acre-feet reservoir enlargement. The documents summary provided in this section follows the chronology of these designs.

Summary: This report presents 50% conceptual designs and construction cost estimates for four options as follows:

Option 1 - SOD modifications of the existing dam and retain present capacity of 6,980 acre-feet
Option 2 - 11,450 acre-feet enlargement
Option 3 - 16,000 acre-feet enlargement
Option 4 - 20,000 acre-feet enlargement without relocating Highway 287
   4a - construct highway on a portion of the dam crest
   4b - construct the dam parallel with the highway outside of the right-of-way

The concept designs for these four options are summarized as follows:

Option 1 - SOD Modifications: This concept was developed starting from the SOD feasibility design presented by GEI (1993a). GEI (1993a) proposed modifications to address specific dam safety concerns identified through previous studies and examinations by USBR. The main components of the GEI design included 1) replacement of upstream slope protection, 2) installation of a downstream toe drain and stability berm, 3) removal and replacement of the existing outlet works, 4) removal and replacement of up to 3 feet of fill at the crest of the dam, and 5) enlargement of the emergency spillway including erosion control lining and lowering of Ray Lake Road adjacent to the spillway. The WWC design concept features are summarized as follows:

1. Upstream Slope Protection: WWC conducted additional analyses and cost estimates, and concluded that riprap would be the most economical choice for replacing the defective upstream slope protection on the 3H:1V slope (GEI had suggested 2 options be considered: riprap and soil cement or RCC).
2. Downstream Toe Drain: WWC essentially retained the concept of a downstream toe drain to control foundation and embankment seepage, as presented by GEI (1993a).
3. Outlet Works: WWC evaluated a number of alternatives, and recommended the use of an upstream gate configuration in conjunction with slip lining the 4 x 4 feet outlet conduit, and complete removal and replacement of the downstream energy dissipation structure. WWC
cited the following reasons for recommending this option over GEI’s recommendation to completely remove and replace the outlet structure: 1) cost savings, 2) concerns about embankment stability resulting from removal and replacement of existing fill material through the maximum dam section, 3) improved dam safety by having the entire outlet conduit dewatered under normal non-operational conditions, and 4) greatly reduced construction time which may avoid loss of an irrigation season. Potential drawbacks of this design compared to the replacement concept were listed as: 1) possibility of damage to exposed gate stems by ice and wave action, 2) possibility of seepage along the exterior of the existing outlet conduit, 3) a reduction in the capacity of the existing outlet works, and 4) maintenance of the upstream gate requires dewatering of the reservoir or underwater work by divers.

4. Embankment Fill, Crest Replacement and Downstream Stability Berm: WWC retained recommendations by GEI to 1) excavate and replace up to 3 feet of fill at the crest of the dam with low-plasticity materials that are not as susceptible to shrinkage cracking, and 2) to place a weight berm at the toe of the dam over the new toe drain to improve stability factors of safety for the downstream slope.

5. Spillway enlargement: WWC performed analyses on the enlarged spillway section recommended by GEI and determined that flow velocities would be erosive in an unlined cut. WWC investigated several options for lining the spillway, and selected a cast-in-place slope paving concept.

6. Cost estimate: WWC estimated the costs for SOD modification described above at approximately $800,000 (1994 dollars).

Option 2 - Enlargement to 11,450 acre-feet: This enlargement size was evaluated previously by GEI (1993a) as the maximum that could be accomplished without requiring relocation of nearby residential structures. The conceptual design presented by WWC involves the following elements:

1. Centerline raise of the main embankment to crest elevation 5540, with upstream slope at 3H:1V and downstream slope at 2.5H:1V. The flattened downstream slope eliminates the need for the stability toe berm that is recommended for the SOD modifications.

2. Chimney and toe drain placed on the downstream slope of the existing dam under the new downstream fill zone to provide control of seepage.

3. Riprap and bedding as upstream slope protection.

4. Similar outlet works rehab as proposed for the SOD modification (slip lined conduit with upstream gate and replaced downstream energy dissipation structure) except that a pipe extension would be required both upstream and downstream.

5. Cost estimate: WWC estimated the costs for 11,450 acre-feet enlargement described above at approximately $1,500,000 (1994 dollars).

Option 3 - Enlargement to 16,000 acre-feet: This enlargement size would require relocation of residents from the south abutment, and relocation of the emergency spillway from the main dam section to the south abutment. The conceptual design presented by WWC is similar to Option 2 presented above, except for an increase in earthwork, riprap and other construction quantities, additional extension of the outlet pipe, and a longer spillway crest. The dam crest would be raised by centerline construction to elevation 5547 feet, and the normal high water line to 5541 feet. The spillway crest elevation was set at 5542 feet to provide storage of the 100-year flood.
The spillway crest length was estimated as 1850 feet. Analyses showed the spillway flow velocities would not exceed 8 feet per second, and it was concluded that a lining would not be required. The cost estimate for this option was approximately $3,000,000 (1994 dollars).

**Option 4 - Enlargement to 20,000 acre feet:** This enlargement size would require relocation of residents from the south abutment, relocation of the emergency spillway from the main dam section to the south abutment, and special considerations associated with Highway 287, which are presented as sub-options in the report. The conceptual design presented by WWC is similar to Option 3 presented above, except for an increase in earthwork, riprap and other construction quantities, additional extension of the outlet pipe, modification associated with highway relocation. The dam crest would be raised by centerline construction to elevation 5553 feet, and the normal high water line to 5547 feet. The spillway crest elevation was set at 5548 feet to provide storage of the 100-year flood. Topographic constraints limit the spillway crest length to 1050 feet. Analyses showed the spillway flow velocities would not exceed 8 feet per second, and it was concluded that a lining would not be required.

This option differs from the other options due to impacts to Highway 287 along the north end of the extension dike. Two options were developed to deal with the highway impacts as follows
- Option 4a - raise the highway along its current alignment making the highway embankment serve as the dam for a portion of its section
- Option 4b - construct an extension dike parallel with the highway and outside of the highway right-of-way

The cost estimate for Option 4a was approximately $5,000,000, and Option 4b was approximately $5,200,000 (1994 dollars).

**Summary:** This report presented conceptual designs for reservoirs sized at 20,000, 31,000 and 41,650 acre-feet. All alternatives assumed relocation of Highway 287 to the north portion of dam, and relocation/demolition of an existing residence.


**Summary:** This report presents conceptual designs for the north and south outlet works and canals. The sizing for these structures was based on the results of the system analysis study (NRCE, 1995) which indicated that the south canal should have a capacity of 60 cfs and the
north canal a capacity on the order of 270 cfs. These capacities are based on peak summer demands in July. The system analyses also indicated that the optimum active storage capacity for Ray Lake should be approximately 26,000 acre-feet. The reservoir was sized at 36,000 acre-feet to provide sufficient driving head on the north outlet works and to provide a recreational pool. The systems analysis estimated that of the 26,000 acre-feet active storage, there would be approximately 10,000 acre-feet of active carry over storage in an average year.

1. South canal design summary
   a. Design Criteria: 1) deliver July peak demands of 60 cfs with reservoir at 10,000 acre-feet active storage (average end-of-season reservoir level), and 2) deliver September peak demands of 20 cfs when all active storage has been depleted and the reservoir storage is at the minimum recreational pool level.
   b. South Outlet Canal:
      i. Convey flows from Ray Lake to Lateral 65C.
      ii. Canal length approximately 18,000 feet.
      iii. Accounting for head losses, need a reservoir water surface elevation of 5532 feet to deliver water to Lateral 65C. This results in dead pool volume of 10,000 acre-feet.
   c. South Outlet Works:
      i. Single, low-level outlet to serve south outlet canal via a short, pressurized pipeline and Mill Creek
      ii. Outlet will have valve system capable of delivering water to either the south canal through the pressurized pipe, to Mill Creek, or to both.
      iii. Hydraulic analysis indicated a 42-inch outlet conduit through the embankment, connected to a 36-inch south canal pipeline would meet the hydraulic requirements.
   d. Appurtenant structures:
      i. 200-feet long inverted siphon at Mill Creek: 48-inch diameter RCP
      ii. 3 culverts: 2 dirt road crossing culverts and crossing culvert at Highway 287
      iii. 2 wasteways
   e. Cost Estimate: approximately $900,000
   f. Final Design: surveys will be required to accurately locate canal route and check hydraulic computations. Also advise checking efficiency of smaller, lined canal section.

2. North canal design summary
   a. Design Criteria: 1) deliver July peak demands of 270 cfs with reservoir at 10,000 acre-feet active storage (average end-of-season reservoir level), and 2) deliver September peak demands of 120 cfs when all active storage has been depleted and the reservoir storage is at the minimum recreational pool level.
   b. North Outlet Canal:
      i. Convey flows from Ray Lake to Coolidge Canal.
      ii. Canal length approximately 25,000 feet in two segments: 1) approximately 4000 feet characterized by deep cut through ridge that separates Mill and Trout Creek drainages, and 2) approximately 21,000 feet running sidehill parallel to Trout Creek.
      iii. Higher reservoir pool means lower costs associated with cut requirements. For minimum pool at elevation 5532 (10,000 acre-feet recreation pool storage), cut volume is approximately 600,000 cubic yards.
   c. North Outlet Works:
i. 5-feet diameter conduit that will pass under Lateral 37C within the dam embankment.
ii. Capable of conveying 120 cfs when reservoir is at the recreation pool level (5532 feet).

d. Appurtenant structures:
i. Pipe drop structure to control decent into Coolidge Canal.
ii. 3 dirt road crossing culverts: each comprising two 60-inch CMPs

e. Cost Estimate: approximately $2,500,000

f. Final Design: surveys will be required to accurately locate canal route and check hydraulic computations. Also advise checking efficiency of smaller, lined canal section, and possibly relocating the dirt road to the east bank of the canal to simplify route and minimize crossings.


Summary: This report presents transmits the 35% final design plans, essentially as proposed in the report “Ray Lake Dam Rehabilitation and Enlargement Final Conceptual Design Report” (WWC, January, 1995), with the following changes since the January report:

1. Reservoir Capacity: The January 1995 Final Conceptual report presented enlargement designs for 31,000 and 41,650 acre-feet reservoirs. Based on the “Ray Lake System Analysis Report” (NRCE, 1995), and subsequent action by the Joint Tribal Business Council, the reservoir size was optimized for final design at 35,000 acre-feet. The 35% final design plans submitted with this report show an embankment dam with a crest elevation at 5571 feet, and normal high water line at elevation 5565 feet. This size will require a small (2 feet) dike along lateral 37C south of the highway.

2. North and South Outlet Canals: In the conceptual design phase, the size and required flows for the north outlet for Ray Lake had not yet been determined. Since then, NRCE (1995) completed a study that optimized the discharge capacities for these canals, and WWC completed conceptual designs for the outlet canals (WWC, 1995a). The 35% plans reflect the conceptual designs presented in the previous report.

3. Alignment of Reconstructed Highway 287: Previous conceptual designs for enlargement alternatives assumed that Highway 287 would remain within its existing right-of-way. Based on data received from NRCE and comments from the Tribes, the BIA, and Wyoming Department of Transportation, the option of relocating the highway alignment to the north is considered acceptable. This revised alignment has numerous benefits including savings in fill requirements, temporary construction detours, reduced riprap quantities, and increase reservoir storage volume. The revised alignment would require relocation of Lateral 37C.

4. Chimney Drain: Conceptual designs included an exposed external drain placed on the downstream slope. Potential problems were recognized with this design, including protecting the filter from erosion and contamination. The revised design uses an internal chimney filter/drain.
5. **North Embankment Causeway**: The proposed dam alignment and existing highway cross a swampy depression area located north of Ray Lake. To alleviate potential drainage problems in this area, the design incorporates the area as part of the raised reservoir inundation area, with a causeway constructed across the area. A 36-inch culvert will allow water from the reservoir to drain into and out of the area. Some slope protection is anticipated for the highway embankment section, and some diking for Lateral 37C is also anticipated.

6. **South Embankment Location**: The south dam abutment was shifted to avoid placement of fill on the steep eroded shoreline.

7. **South Outlet Works**: The conceptual design called for repair of the existing outlet works using a slip-liner. Concern about possible settlement and condition of the existing outlet prompted a change to require complete removal and replacement of the old structure.

Total estimated costs for the 35,000 acre-feet option were $20,300,000 (1995 dollars).

Also included with this transmittal are two geotechnical documents, “Geotechnical Field and Laboratory Investigations” and “Design Memorandum No. 2” (Hadley & Hollingsworth, 1995b and 1995c). Summaries of those reports are provided in subsequent sections of this bibliography.
1994-1996 GEOTECHNICAL FIELD INVESTIGATION REPORTS AND DESIGN MEMORANDA BY HADLEY AND HOLLINGSWORTH (H&H), AND REVIEW COMMENTS BY USBR

Between 1994 and 1996, Hadley and Hollingsworth, Ltd. (H&H), a geotechnical consulting firm based in Lakewood Colorado, provided subcontract services to WWC in support of dam and reservoir enlargement designs and safety of dams (SOD) modifications for Ray Lake. The U.S. Bureau of Reclamation (USBR) provided comments on these investigations at various times. Summaries of the geotechnical reports and USBR comments are provided in this section, in chronological order. The following table summarizes the geotechnical site studies and reports by H&H, and USBR Comments documents.

Summary of Geotechnical Reports by H & H and USBR Reviews between 1994-1996

<table>
<thead>
<tr>
<th>Report Date</th>
<th>Title</th>
<th>Brief Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>8/1/94</td>
<td>Phase I Field and Laboratory Investigation for Design of Safety of Dam Modifications and Enlargement of Ray Lake Dam</td>
<td>Reports on field and lab investigations conducted in June 1994; 17 test pits downstream from dam and local gravel pit sampling</td>
</tr>
<tr>
<td>10/31/94</td>
<td>Phase II Field and Laboratory Investigations for Design of Safety of Dam Modifications and Enlargement of Ray Lake Dam</td>
<td>Reports on field and lab investigations completed in Sept-Oct 1994 during reservoir drawdown; 10 test pits upstream from dam, 8 borings in dam and foundation</td>
</tr>
<tr>
<td>1/9/95</td>
<td>Conceptual Design of Enlargement, Ray Lake Dam, Design Memorandum No. 1, Geotechnical Considerations</td>
<td>Recommendations for centerline raises, embankment zoning, foundation preparation, and slope stability analyses for 3 raise options with dam heights of 50, 63, and 75 feet</td>
</tr>
<tr>
<td>6/6/95</td>
<td>Letter from USBR to BIA regarding Phase II site investigation</td>
<td>USBR recommended specific additional final design geotechnical information that needed to be obtained for enlargement scenarios</td>
</tr>
<tr>
<td>8/23/95</td>
<td>Report of Study, Geotechnical Field and Laboratory Investigations for Design of Safety of Dam Modifications and Enlargement of Ray Lake Dam</td>
<td>Summary data report that includes Phase I, Phase II and additional Phase II investigations that included additional borings</td>
</tr>
<tr>
<td>8/25/95</td>
<td>Design of Enlargement, Ray Lake Dam, Design Memorandum No. 2, Geotechnical Considerations</td>
<td>Revised geotechnical analyses and recommendations for a 68 feet high dam (35,000 acre-feet reservoir).</td>
</tr>
<tr>
<td>11/20/95</td>
<td>Review of 35% Final Design Report, Ray Lake Enlargement (dated 9/1/95), and Ray Lake SOD 35% Final Design memorandum (dated 7/14/95)</td>
<td>USBR provided comments on both SOD and enlargement design reports: SOD study and design were determined to be essentially adequate; but geotechnical program for enlargement design was determined to be seriously inadequate</td>
</tr>
<tr>
<td>1/12/96</td>
<td>Design Memorandum No. 3 – Ray Lake Dam Enlargement</td>
<td>Memo provides supplemental analysis that was requested by USBR for slope stability and seepage for the 68 feet high dam</td>
</tr>
</tbody>
</table>
This report documents results of the first phase of geotechnical investigations conducted by Hadley & Hollingsworth (H&H) in support of the design efforts by WWC between 1994 and 1996. The work conducted under Phase I had the stated purpose of identifying any geotechnical fatal flaws or conditions that might significantly impact construction costs. The major tasks that were reported to have been completed under the Phase I effort are listed as follows:

1. Geologic reconnaissance and mapping of the reservoir site
2. Review of potential borrow areas to confirm suitability and availability of impervious borrow and granular materials for dam construction
3. Field investigation program consisted of
   a. Excavating 17 test pits by backhoe
   b. Sampling 3 existing gravel pits north of Ray Lake
   c. Laboratory testing of sampled materials to determine moisture-density (compaction) relationships, index properties (grain size analysis, specific gravity, and Atterberg limits), dispersive potential, and swell characteristics.
4. The report includes a map (Figure 1) showing the locations of excavated test pits, a “Geologic Sketch Map” (Figure 2) that shows only the approximate boundary between the Cody Shale and the Frontier Formation overlain on the site plan, stick logs of the test pits (Figure 3) with an accompanying legend (Figure 4), lab test result graphs (Figures 5 through 12), and table summarizing the lab test results (Table 1).
5. The conclusions from the Phase I field investigation were as follows:
   a. No fatal flaws or conditions that would significantly impact construction costs were found.
   b. Adequate impervious soil borrow areas were found below the proposed normal pool level in the reservoir area. Adequate sources of granular soils were located from established local gravel pits.
   c. Foundation conditions for the proposed raised dam sections and new dike sections were interpreted to consist of soft to medium stiff clays, soft weathered shale and soft shale. These materials were considered to be suitable for founding the embankment at relatively flat slopes. The slopes of the conceptual sections at 3H:1V upstream and 2.5H:1V downstream were judged to likely be stable with proper foundation preparation.
   d. The foundation of the existing dam appeared to not have been stripped of organic material prior to fill placement. However, this report considered the foundation suitable to support both the SOD modifications and the dam enlargement at relatively flat slopes.
   e. The foundation for a new auxiliary spillway in the south abutment was described as soft sandy clay with an interlayer of gravel overlying soft shale. The report states that stripped to the shale, the foundation materials should be suitable to support a crest structure.


This report documents results of the second phase of geotechnical investigations conducted by Hadley & Hollingsworth (H&H) in support of the design efforts by WWC between 1994 and 1996. The field investigation consisted of excavating ten test pits and drilling 8 exploratory borings. The reservoir was drawn down to the lowest level possible at the time of the exploration, which allowed access to areas upstream from the dam. Silt buildup up to 10 feet deep was visually evident on the upstream toe. Erosion of the upstream face of the dam also was observed. This report combines the Phase I investigation description and findings with the additional Phase II work. The major (new) tasks that were reported to have been completed under the Phase II effort are listed as follows:

1. Excavation of 10 test pits upstream from the existing dam.
2. Drilling of 8 exploratory borings: two borings (B-1 and B-2) were drilled within the upstream portion of the footprint of the raised dam, three borings (B-3, B-4, and B-5) were drilled through the existing embankment and into bedrock, two borings (B-6 and B-7) were drilled through the existing dike and into the foundation, and one boring (B-8) was drilled along the proposed U.S. 287 embankment re-alignment.
3. Laboratory testing of samples collected from test pits and borings, including testing for index properties (Atterberg limits, natural moisture content and dry density, and percentage of material finer than #200 sieve size); standard Proctor compaction; consolidation testing of undisturbed samples of the existing embankment, foundation and bedrock; permeability testing of bedrock core samples; unconfined compressive strength of embankment and
foundation samples; direct shear of bedrock samples; dispersion potential of claystone bedrock; and concentration of water soluble sulfate in the weathered bedrock. Laboratory test results are provided on graphs (Figures 5 through 30) and a summary table (Table I) in the report. These figures and summary table combine the results of Phase I and Phase II investigations.

4. Recommendations of material properties for design:
   a. Foundation materials: the report provides a summary (Table II) of the recommended values for design parameters for sandy clay, weathered claystone, and claystone foundation materials including: wet and saturated unit weights, shear strength (“friction angle” and cohesion), and permeability for three materials. Shear strengths reported on the summary table were based on an average of direct shear tests of the foundation materials. Permeability values were based on results of falling head tests on core samples in the laboratory.
   b. Embankment materials: Table III in the report summarizes the recommended values to be used in design for existing embankment and new embankment materials including: wet and saturated unit weights, shear strength (“friction angle” and cohesion), and permeability. Shear strengths reported on the summary table were based on direct shear tests conducted as part of this study, and on triaxial shear tests conducted by USBR in 1984. Permeability values were based on results of falling head tests on remolded samples in the laboratory.

At the time of this report (January 1995), three enlargement options were being considered, as shown on the following table:

<table>
<thead>
<tr>
<th>Alternative</th>
<th>Size (acre-feet)</th>
<th>Crest Elev. (ft)</th>
<th>Normal Pool Elev. (ft)</th>
<th>Structural Dam Height (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Existing Dam</td>
<td>6890</td>
<td>5532</td>
<td>5526</td>
<td>29</td>
</tr>
<tr>
<td>I</td>
<td>20,000</td>
<td>5553</td>
<td>5547</td>
<td>50</td>
</tr>
<tr>
<td>II</td>
<td>30,000</td>
<td>5566</td>
<td>5560</td>
<td>63</td>
</tr>
<tr>
<td>III</td>
<td>40,000</td>
<td>5578</td>
<td>5572</td>
<td>75</td>
</tr>
</tbody>
</table>

This report presents recommendations for locations of the centerlines of the raised dam sections, sources of embankment materials, preparation of the embankment foundation, configuration and zoning of the raised dams, and stability analyses to support the recommended configurations. The analyses are based on the Phase I and Phase II geotechnical site investigations. Key recommendations are summarized as follows:

1. Geotechnical constraints: H&H reported that there were no geotechnical constraints to raising Ray Lake Dam to at least 75 feet based on available information.

2. Location of Raised Dam Centerline: All options considered leaving the existing dam in place, and constructing a centerline raise over the existing dam. H&H state that they preferred this type of raise because “...this method will put new foundation preparation and dam embankment both upstream and downstream of the existing embankment. Thus, the existing embankment, the condition of which the design team has little control over, will be in the bottom, center of the raised dam embankment which is the least critical location with respect to stability and seepage.”

3. Foundation Conditions: The reservoir bed has up to 10 feet of soft, wet silt and clay in the upstream toe of the existing dam. The report notes that it will be difficult to dewater and excavate beneath the entire upstream section of the raised dam embankment to a suitable foundation. They noted that the distance that the upstream toe of the raised dam will extend into the reservoir area will depend on the dam height, the upstream slope, and the alignment of the centerline. They recommended placing the upstream toe of the raised dam embankment closer than 200 feet of the potential level of the lowered reservoir based on available topography. On the downstream side, H&H stated that there were no geotechnical constraints to construction of the dam.

4. Sources of embankment materials: The Phase I and Phase II geotechnical field investigations located significant quantities of impervious borrow materials for construction of a homogeneous raised embankment.

5. Foundation Preparation: H&H provided recommendations for removal of all brush, vegetation, and topsoil beneath the footprint of the raised dam. They also recommended removal of “minor areas” of wet, soft clay downstream from the existing dam that are present as a result of seepage from the reservoir. Removal of soft, wet silt and clay deposits up to 10 feet thick down to medium stiff clay or weather claystone, whichever is encountered first, was recommended for the upstream toe of the main dam. H&H reported that “...There doesn’t seem to be any purpose for excavating a keyway or cutoff trench on the foundation. There is no particularly different foundation stratum to key into. Therefore, a keyway trench would not serve to decrease seepage or increase stability. We do not recommend a keyway or cutoff trench for the raised dam.”

6. Embankment Configuration: A homogeneous embankment constructed of mainly sandy clay and weathered claystone materials from local borrow sources was recommended. Upstream slopes of 3H:1V and downstream slopes of 2.5H:1V were recommended for all raise configurations. Two configurations were considered for downstream seepage control as follows:
   a. Toe drain and drainage blanket placed on the downstream slope to mid-height of the dam.
   b. Horizontal drainage blanket extending from the downstream toe to a point 1.5 times the raised dam height from the centerline, and an inclined chimney drain on a 1H:1V slope to mid height of the raised dam.

7. Drain material requirements were recommended to meet Wyoming Department of Transportation Standard Specifications for Road and Bridge Construction, Gravel for Drains, Grading C, Section 703.17. No specifications or design calculations were shown for filter requirements.

8. Stability Analyses: Stability analyses were performed using strength parameters recommended in the Phase II geotechnical report. Factors of safety are reported for deep,
circular failure surfaces for steady seepage, earthquake, rapid drawdown, and end of construction loading cases. Earthquake loading analysis was a pseudo-static analysis using a “maximum ground acceleration” equal to 0.1.

1995 Letter from USBR to BIA regarding Phase II site investigation: letter dated June 6, 1985, from Thomas A. Brown, Team Leader, Geotechnical Services, USBR to Area Director, BIA, Billings; Subject: Review of Hadley & Hollingsworth (1994b) Phase II Field and Laboratory Investigations for Design of Safety of Dam Modifications and Enlargement of Ray Lake Dam, Fort Washakie, Wyoming,

This letter report documents comments from the USBR geotechnical review team on the Phase II geotechnical site investigation conducted by Hadley & Hollingsworth (H&H). USBR recommended that additional final design geotechnical information data needed to be obtained for enlargement scenarios. It is unclear whether or not USBR believed that adequate information was presented in the report to support the SOD modifications. USBR recommended that additional information for the enlargement concept was needed to address the following concerns:

1. Potential weak zones in the foundation.
2. Foundation seepage.
3. Foundation data to the left and right of the main embankment.
4. Inadequate information in the area of the outlet works foundation.
5. Inadequate information in the area of the proposed emergency spillway.

The letter report provides specific comments regarding the type and quality of geologic and geotechnical information that they believed was lacking and necessary. Geologic data that was lacking included: continuous sampling of bore holes, detailed geologic logs of investigations, and preparation of geologic cross sections along with overall geologic plan map showing strike and dip information of all surficial deposits, variations of lithology within those deposits, fractures, joints, faults, shears or other structural information as can be identified, and location of geologic sections. Geotechnical issues included: sampling and testing methods for “undisturbed” specimens, use of direct shear and unconfined test methods to determine shear strength instead of the preferred triaxial test procedures, inappropriate analysis and assumptions used in estimation of permeability of the dam foundation materials, lack of adequate consolidation and strength testing for structural elements, and no testing or investigations on filter materials. In addition, the review noted that the final version of the report (dated January 9, 1995) had different statements in the “Limitations” section than the October 31, 1994 version that had been previously submitted for review, and that the final (January 9, 1995) version did not mention the need to remove the soft foundation soils under the raised dam footprint both upstream and downstream from the dam.
This report is bound with the WWC report entitled “35% Final Design Report, Ray Lake Enlargement” (WWC, 1995) and documents the combined results of Phase I and II geotechnical investigations conducted by Hadley & Hollingsworth (H&H) in 1994, the 1984 geotechnical investigations by USBR, and “additional Phase II” investigations by H&H conducted in 1995. The additional Phase II investigations included drilling 11 additional exploratory borings (B-9 through B-19), drilled between April 3-5, 1995. These borings were evidently intended to fill in gaps and acquire data based on the evolving design which was considering locating the emergency spillway on the south side of the reservoir, and a new irrigation outlet structure on the north side of the reservoir. The major (new) tasks that were reported to have been completed under the additional Phase II effort are listed as follows:

1. Field investigations: Five borings (B-9 through B-13) were drilled in a proposed borrow area on the northwest side of the reservoir. One boring (B-14) was drilled next to Highway 287 near the location where the north outlet canal will cross the highway. One boring (B-15) was drilled at the downstream toe of the dam on the north side of the existing outlet. Two borings (B-16 and B-17) were drilled in the right (south) abutment ridge on the raised dam alignment, and two borings (B-18 and B-19) were drilled near the proposed alignment of the north outlet canal.

2. Laboratory testing of samples collected from these borings included testing for index properties (Atterberg limits, natural moisture content and dry density, and gradation analyses); standard Proctor compaction; swell-consolidation testing of undisturbed samples of bedrock; permeability testing of bedrock core samples; unconfined compressive strength of foundation samples; and direct shear of bedrock samples; Complete (Phase I, Phase II, and additional Phase II) laboratory test results are provided on graphs (Figures 8 through 68) and a summary table (Table I) in the report.

3. In addition to providing the additional Phase II site investigation information, this report summarizes information that was not previously reported on borrow source materials, foundation conditions, and recommended values for design.

4. Borrow area – the intent of these investigations was to evaluate potential borrow source materials from the raised reservoir area, from excavations required for the North Outlet Canal, and from the spillway area. The subsurface conditions typically indicated approximately 6-inches of topsoil and 0 to 10.5 feet of medium stiff to stiff sandy clay, and 2 to at least 11.5 feet of soft to medium weathered claystone overlying medium hard to very hard claystone bedrock to the depth investigated (up to 26 feet).

5. Foundation conditions along alignment of raised dam section and dike, as follows:
   a. Upstream from existing dam and dike: up to 10 feet of loose, wet sandy silt over up to 10 feet of soft, wet, plastic sandy clay overlying weathered claystone and claystone. The weathered claystone is red-brown to grey, medium hard to hard with fine sand stringers throughout, and moist.
b. Downstream from existing dam and dike: 1.5 to 3 feet of embankment fill overlying sandy clay in the dam toe area. Downstream from the toe boring B-15 indicated 6-inches of topsoil over 4.5 feet of sandy clay, and 3.5 feet of weathered claystone overlying claystone. Embankment fill in the toe area is described as moist, medium stiff, plastic clay. The contact between fill and foundation contained minor amount of organic material indicated the foundation had not been stripped. Weathered claystone was moist, soft to firm, with numerous fine sand lenses. Claystone bedrock was moist, medium hard to hard with numerous fine sand lenses throughout. Downstream from the dike the foundation is described as 0 to 3-inches of topsoil and 1 to 5.5 feet of sandy clay overlying weathered claystone. The weathered claystone is moist and soft to firm.

c. Centerline of Existing Dam and Dike: Foundation conditions along the existing dam are described as 0 to 9 feet of sandy clay, and 5 to 7 feet of weathered claystone overlying claystone bedrock. The sandy clay is moist to wet, light brown, soft to medium stiff, and plastic. The contact between the fill and foundation contained organics. Weathered claystone and claystone were described as moist, red-brown, and firm to hard with numerous fine sand lenses throughout.

d. Foundation conditions along raised section: 0 to 3 inches of topsoil, 6 inches to 5 feet of sandy clay, and 0 to 4.5 feet of weathered claystone overlying claystone. Sandy clay was described as moist to wet, light brown, soft to medium stiff, and plastic. Both the weathered claystone and claystone were described as firm, and moist to wet.

e. Foundation conditions for U.S. Highway 287 embankment: Boring B-8 indicated 9 feet of sandy clay over 5 feet of weathered claystone overlying claystone bedrock.

f. Foundation conditions for right abutment of raised dam (proposed location of emergency spillway): 6 inches of topsoil over 2 to 5.5 feet of sandy clay, over 2.5 to 7.5 feet of weathered claystone over claystone bedrock. A layer of gravel, 8-inches thick was encountered in TP-17 at a depth of 2 feet, 8 inches. Weathered claystone is described as moist to wet, and soft to firm. The claystone bedrock is described as medium hard to hard.

g. Foundation conditions for the North Outlet Canal Crossing at Highway 287: 6-12 inches topsoil over 2 feet of sandy clay, and 4 feet of weathered claystone overlying claystone bedrock.

h. Subsurface conditions for North Outlet Canal (at location of large cut): Borings B-18 and B-19 indicated 6-12 inches of topsoil, 3 to 12.5 feet of sandy clay, and 9.5 to 13 feet of weathered claystone overlying claystone bedrock. The clays are sandy to very sandy with scattered gravel, medium stiff to stiff, moist and light brown. The weathered claystone is described as firm, sandy, moist and brown. The bedrock is described as medium hard to very hard, sandy and moist.

6. Recommendations of material properties for design:

a. Embankment materials: Table II in the report summarizes recommendations for design parameters for three types of fill materials: (1) existing embankment, (2) new embankment from borrow area “E”, and (3) new sandy silt embankment from the reservoir bed. Effective stress strength parameters for existing and new borrow source “E” materials is based on previous triaxial tests by USBR (1984), and total stress strength parameters are based on direct shear testing done as part of these investigations. Undrained strength (Su) values also are listed for these materials based on unconfined compressive strength tests. Vertical permeability values are based on falling head lab test
results. Effective and total stress strength values for the sandy silt materials derived from the upstream foundation excavations were estimated based on direct shear test results.

b. Foundation materials: Table II also summarizes the recommended values for design parameters for 1) sandy clay, 2) weathered claystone, and 3) claystone foundation materials including: wet and saturated unit weights, shear strength ("friction angle" and cohesion), and permeability for three materials. Effective stress strength values for the sandy clay foundation materials was estimated based on triaxial testing of embankment soils by USBR (1984). Shear strengths reported on the summary table for weathered claystone and claystone were also estimated based on tests done by USBR (1984). Permeability values were based on results of falling head tests on core samples in the laboratory.

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<th>Alternative</th>
<th>Size (acre-feet)</th>
<th>Crest Elev. (ft)</th>
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At the time of this report (August 1995), one enlargement option was being considered in addition to the SOD modification, as shown on the following table:

This design memorandum discusses the geotechnical issues related to the 35,500 acre-feet enlargement alternative, and presents recommendations for location of the raised dam centerline, sources of embankment materials, preparation of the foundation, configuration and zoning of the raised dam, and stability analyses for the raised dam section. This report also recommends foundation design parameters for the new outlet works and the North Outlet Canal crossing. Key recommendations are summarized as follows:

1. Location of Raised Dam Centerline: This report differs from previous design recommendations by H&H (1995a) in that the dam centerline is shifted downstream to avoid placing new fill on the soft, wet reservoir sediments at the upstream toe. The design that is promoted in this memorandum maintains the raised embankment upstream toe at the same location as the existing dam upstream toe. Further, although not a specific recommendation or requirement by H&H, the report does state that they “... have no objection... to completely remove the existing embankment.”

2. Foundation Preparation under main dam: H&H provided recommendations for removal of all brush, vegetation, and topsoil beneath the footprint of the raised dam. They
recommended dewatering of all embankment foundation areas, and removal of “minor areas” of wet, soft clay downstream from the existing dam that are present as a result of seepage from the reservoir. H&H recommended leaving the existing dam in place under the upstream shell of the new raised dam. They stated that foundation seepage cutoff would be provided by the new foundation preparation in the downstream area, so that any deficiencies in the foundation of the existing dam would be cut off. According to the report... “We see no compelling reason to completely remove the existing embankment but agree that it would be a conservative approach...” H&H recommends foundation preparation in the area where the embankment is removed consist of removal of all organic material and soft materials down to the medium stiff clay or weathered bedrock, whichever is encountered first.

3. North abutment foundation preparation: This area is new embankment that serves to impound the reservoir and possibly also as Highway 287 embankment. Foundation preparation recommended by H&H consisted of excavation of organic materials and soft materials down to medium stiff clay or weathered bedrock, whichever is encountered first.

4. South abutment foundation preparation: At the time of this report, consideration was being given to re-aligning the south end of the dam to avoid placement of fill on steep eroded banks of the reservoir rim in this area. Foundation preparation recommendations were to remove all organic, soft and granular soils down to the medium stiff clay or weathered bedrock, whichever is encountered first.

5. Seepage control: USBR had expressed concerns that sand lenses in the claystone bedrock are a continued source of seepage under the existing dam and dike. H&H stated that they did not see a purpose for excavating a keyway or cutoff trench in the foundation, or attempting to grout these materials. They stated that their filter and drain features in the main dam section were adequate to control seepage and maintain stability of the raised dam.

6. Sources of embankment materials: The Phase I and Phase II geotechnical field investigations located significant quantities of impervious borrow materials for construction of a homogeneous raised embankment. H&H recommended using silt and low plasticity clay materials to be stripped from the upstream reservoir bed near the existing dam near the crest to minimize risk of shrinkage cracking.

7. Embankment Configuration: A homogeneous embankment constructed of mainly sandy clay and weathered claystone materials from local borrow sources was recommended. Upstream slope of 3H:1V and downstream slope of 2.5H:1V were recommended for the raise configuration.

8. Seepage control: Two configurations were considered for downstream seepage control as follows:
   a. Toe drain and drainage blanket placed on the downstream slope to mid-height of the dam.
   b. Horizontal drainage blanket extending from the downstream toe to a point 1.5 times the raised dam height from the centerline, and an inclined chimney drain on a 1H:1V slope to mid height of the raised dam.

9. Drain material requirements were recommended to meet Wyoming Department of Transportation Standard Specifications for Road and Bridge Construction, Gravel for Drains, Grading C, Section 703.17. No specifications or design calculations were shown for filter requirements.

10. Stability Analyses: Stability analyses were performed using strength parameters recommended in the compilation geotechnical report (H&H, 1995b). Factors of safety are reported for deep, circular failure surfaces for steady seepage, earthquake, rapid drawdown,
and end of construction loading cases. Earthquake loading analysis was a pseudo-static analysis using “maximum ground accelerations” equal to 0.1g and 0.35g. The 0.35g ground acceleration was evaluated based on recommendation a review by USBR. Pseudo-static factors of safety were found to be less than 1 for the 0.35g loading assumption. H&H therefore performed a simplified deformation analysis using a program developed by USBR, and computed deformations under the 0.35g condition to be on the order of 1.3 feet. This was considered by H&H to be acceptable since the freeboard for the dam is approximately 6 feet.

11. Settlement Analyses: Settlement was estimated for the maximum section, under the assumption that the foundation is stripped down to weathered claystone. Total settlement of 1.4 feet was predicted, including an estimated 0.8 feet of embankment consolidation and 0.6 feet of foundation consolidation. An overbuild of 1.5 feet at the maximum section was recommended to accommodate this settlement.

12. South Outlet Works: Recommendations for foundation preparation for this structure included cleaning of all disturbed and soft materials down to the weathered claystone. The recommendations further state that “… it seems prudent to plan on 0.8 feet of settlement under the centerline and 0.2 feet of settlement at the inlet and outlet structures.”

13. North Outlet Works: It was recommended that the structure be excavated well into claystone and designed for settlement of 0.2 feet at the highway embankment, 0.1 feet at the intake structure, and no settlement at the energy dissipater structure. Also, it is noted that the Lateral 37C conduit will pass through the embankment above the outlet works conduit. It was recommended that the entire conduit should be founded on the same depth of fill to equalize settlement. Backfill of the outlet conduit and the foundation fill for Lateral 37C conduit should be sandy clay or weathered claystone placed close to optimum moisture content and compacted to at least 100% standard Proctor maximum density.

November 20, 1995 Letter from USBR to BIA regarding 35% Final Designs of Enlargement and SOD:

This letter report documents comments from the USBR geotechnical review team on both the enlargement project design and the SOD project design.

USBR comments pertaining to the 35% Final SOD design are summarized as follows:

1. The final design alternative for the SOD modification was determined to be in general accordance with Reclamation and industry practice.
2. Drain system issues:
   a. 12-inch thickness of filter material under outlet works conduit extension and under the stilling basin should be specified as a minimum, or increased to ensure continuity of intact filter blanket under the conduit and basin.
b. Toe drain should be installed deeper using “agricultural drain” installation procedures. Toe drain filter should be protected from erosion and contamination by covering with Zone 4 material.

c. Chimney drain is too thin.

3. Earthwork issues:
   a. Inadequate information provided in reports to review/evaluate suitability of materials used in embankment, foundation preparation, and compaction requirements.
   b. Stripping depth of 12 inches should be minimum. Actual stripping may need to go deeper to remove soft or wet foundation materials, not just organic materials and topsoil.
   c. Details for tie-in between base of new riprap and existing ground are unclear. In some places, soft, saturated, silty reservoir sediment is present which will be unsuitable to found the new riprap facing.
   d. Need to provide slope stability analysis for the 2.5H:1V downstream slope without a toe berm. This is a change from the conceptual design of 2.75H:1V downstream slope with a berm.

4. Outlet Works: Concrete quality is poor based on USBR examinations, due to alkali-aggregate reaction and sulfate attack. These conditions need to be addressed in evaluating the durability of the existing structures and design of new structures.

5. Intake Structure: Bellmouth entrance would provide more efficiency, but would complicate construction. Trashracks need to be included. Steel liner thickness within the intake should be designed to accommodate external loads when the bulkhead is in place and the pipe is drained. Include a filling valve and air vent in the blind flange for the bulkhead at the upstream end.

6. Conduit issues:
   a. Need to review capacity of the existing conduit for its ability to withstand unwatering loading that would be possible with upstream bulkhead in place. Conduit may not have been designed for this loading.
   b. Alignment (horizontal and vertical) of the existing pipe needs to be checked to ensure that there is sufficient room to allow “threading” of the 42-inch liner into place with only 2 inches of clearance at the quarter sections.
   c. Good potential for settlement of the foundation under the 20-feet long outlet pipe extension. USBR recommends adding a control joint within this section to allow for more flexibility to accommodate settlement.
   d. Need more details on chimney and blanket drain wrapped around downstream portion of conduit.
   e. Recommend hoop bar reinforcement around conduit, not just exterior only as shown.

7. Guard gate structure: USBR cautions that existing structure must be able to support the new guard gate, with consideration of long-term reliability before implementing this design. May be some difficulty providing adequate support for the gate stems when the stem guides are located very far from the support face. Air vent is required for closing the guard gate under flow conditions or draining the downstream conduit.

8. Vault Structure and Energy Dissipator: No details provided for review, but concept appears reasonable.
9. Spillway Modification:
   a. Spillway crest was lowered from 5527 (50% conceptual design) to 5529 (this 35% final design). This will increase the frequency of flood discharges from the spillway. BIA needs to verify that the revised spillway discharge elevation is appropriate for the site.
   b. New spillway design shows much shorter crest length which will result in higher flow velocities. Erosion protection is concrete cantilever walls on each side of the spillway, and gabions on the inlet area and flow surface. USBR has not used gabions for spillway structures. USBR offers several concerns in the review regarding the gabion design.

10. Standard drawing details need to be updated to current USBR standards and codes

11. Cost estimate: USBR is concerned that cost estimates for “demolition of existing outlet structure”, and “dewatering” may be low. Need more information on how these costs were developed. Allowance for mobilization and preparatory work (10%) may be too high. USBR typically uses 5% for this item.

USBR comments pertaining to the 35% Final Design of the Enlargement project are as follows:

1. General lack of complete geologic mapping, drill hole logging, and site characterization, as well as identification of foundation and borrow material engineering properties, as detailed in USBR’s June 6, 1995 review of the “Report of Study Phase II Field and Laboratory Investigations for Design of Safety Modifications and Enlargement of Ray Lake Dam” by Hadley and Hollingsworth (1994b). These deficiencies lead to numerous design and construction uncertainties. (Refer to summary presented previously for specific geologic and geotechnical concerns).

2. Geology:
   a. Specific concerns relate to potential for excessive foundation seepage. Current design does not adequately address potential abutment and foundation seepage by cutoff, and/or interception, collection, filter and discharge.
   b. Need geologic plan map and cross sections, and a foundation excavation plan.
   c. No exploration along alignment of the relocated highway.
   d. Bedding dip not adequately documented. Actual bedding dip needs to be confirmed and considered in embankment and cut slope designs.
   e. Need to conduct additional exploration to evaluate presence and effect of bentonitic clay beds in the foundation.
   f. Additional borrow studies need to be done to evaluate presence of CaCO$_3$ and gypsum, which are potentially soluble minerals that would be undesirable in the core section of the dam.

3. Geotechnical Foundation issues:
   a. USBR recommends complete removal of existing embankment in order to found the new, higher dam on suitable foundation materials and to ensure continuity of construction of the new dam.
   b. Inadequate information to evaluate foundation treatment requirements. Need excavation plan and basis for suitable foundation.
c. Design should include identification of suitable materials for construction by type, density, and moisture content.
d. Data collected to date does not reliably reflect seepage properties of the foundation.

4. Geotechnical Material Properties and Embankment Construction issues:
a. Use of low-plasticity silts derived from reservoir sediments on dam crest to alleviate cracking is questioned. USBR recommend suitable materials would be a blend of low PI, semi-pervious materials with significant sand content, such as clayey sand or silty-clayey sand, for use above the normal high water line to minimize risk of desiccation cracking.
b. Use of lean and fat clays placed at 2-3% above optimum moisture is questioned. USBR recommends fill moisture content be limited to no more than 2% dry to 1% wet of optimum. Wet soils must be dried prior to placement and will be very difficult to work.
c. No compaction data were provided for borrow source materials derived from required cut on the south abutment. Information on particle breakdown needs to be developed to ensure that use of mixed materials does not lead to a fill of hard, dry fragments in a wet soil matrix.
d. No specific information was provided on material requirements, including composition, placement, moisture and compaction for various zones.
e. A abrupt change is crest width from 20 to 15 feet at a sharp, almost 90 degree turn is inadvisable from a traffic safety standpoint. Further, USBR guidelines for crest width for a 68-feet high dam are 24-feet, minimum. USBR recommends oversteepening top 3 or 4 feet to achieve the recommended top width.
f. No materials distribution chart was provided.
g. No instrumentation plans for embankment were provided.

5. Geotechnical Settlement Analysis issues:
a. Sample disturbance of consolidation test specimens is suspected.
b. Foundation settlement under maximum embankment sections should be evaluated at sections including the outlet works and other locations to design required crest camber.
c. USBR questioned applicability of the consolidation test data used as the basis for the settlement computations, as the sample density for the test specimen used was exceedingly low compared to the reported typical maximum Proctor density. USBR recommends testing recompacted embankment soils.

6. Geotechnical Seepage Analysis and Filter Drain System Design issues:
a. No anisotropy used in the analysis, which is unrealistic for compacted materials.
b. Need to evaluate left abutment seepage for possible end-run or flow beneath the toe drain system and damage to portions of Highway 287. May need additional seepage control measures in this location.
c. Toe drain seepage interception trench should be extended deeper into the foundation.
d. Embankment filter/drain system does not meet USBR current design practice or current practice.
e. Chimney drain need to extend to elevation 5565 to provide protection from embankment cracking near dam crest. Width of filter/drain zone should be increased.

f. Wyoming DOT specification referenced in 35% Design Report is unsuitable for filter material. ASTM C-33 concrete sand may meet filter requirements, if additional limits are placed on minus #200 sieve size materials.

g. Toe drain slope is too shallow in some reaches.

7. Geotechnical Slope Stability Analyses issues:
   a. Basis for stability analysis results is inadequately documented.
   b. Need correct seepage analysis using anisotropic permeabilities to more accurately evaluate pore pressures for stability analyses.
   c. All stability analyses should incorporate foundation layering, as well as embankment zoning.
   d. No indication that analyses other than circular failure surfaces were evaluated. Bedding controlled failure surfaces may be more critical and should be analysed.
   e. Strength parameter assumptions are incorrect.
   f. Reclamation outlines appropriate assumptions and analyses that should be conducted for each loading case (I - Steady State Seepage, II - Earthquake Loading, III – Rapid Drawdown, and IV – End-of-Construction).

8. Hydraulic Structures design issues:
   a. Concrete quality: design of new concrete structures should account for sulfate attack and alkali-aggregate reactions that are evident in existing concrete structures.
   b. Outlet works (both north and south structures which are similar):
      i. Proposed gate scheme for unwatering the outlet works should be re-evaluated. USBR has experienced operational difficulties with similar gate systems.
      ii. Conduit design is satisfactory. Drainage details were not provided; chimney filter and drainage blanket should encase the conduit to control seepage that may be occurring along the outside of the structure.
      iii. Slight downstream slope along the invert of the conduit may facilitate unwatering.
   c. Vault and energy dissipater/South Outlet Works: Size and layout appear generally satisfactory. USBR provides specific suggestions for possible improvements.
   d. South Canal Pipeline: Insufficient information available to review.
   e. Inlet Channel/North Outlet Works: 2H:1V side slopes of the inlet channel may be unstable during drawdown during normal irrigation demands when reservoir is low.
   f. Vault and Stilling Basin/North Outlet Works: No details provided. Details of Lateral 37C crossing were limited. Need to provide adequate filter protection in this portion of the embankment.

9. Spillway (1000-feet wide excavated channel through reservoir rim right of end of the embankment): Consider grade control sill, and erosion protection on the left side of the spillway channel. Prevent spillway releases from flowing along toe of the dam.
10. Standard Drawings: Need to incorporate recent revisions in USBR standard drawings and recent code changes.
11. Specifications: Major work items appear to be covered.
12. Cost Estimates: Cost for dewatering appears low and should be reviewed. Costs for mobilization (10%) are high based on USBR experience. USBR typically allows 5% for mobilization and preparatory work.


1. At the time of this report, this was considered to be supporting geotechnical analyses for the 100% final design of the 68-feet high dam enlargement (crest elevation 5571, reservoir volume 35,000 acre-feet). This report documents stability and seepage analyses for the maximum embankment section, with the chimney drain moved into the dam section. Both circular and sliding block failure surfaces were analyzed.

2. Estimated factors of safety were calculated for all but the pseudo-static condition with 0.35g seismic coefficient assumption. Yield acceleration was computed at 0.25g, and a revised deformation analysis was performed. The estimated deformation ranged from 0.2 to 0.3 feet, which was considered acceptable.

3. Seepage analyses were performed by preparing a flow net for a transformed section. It was assumed that the horizontal permeability was nine times the vertical permeability based on laboratory tests. The flow through the embankment was estimated at $1.33 \times 10^{-7}$ ft$^2$/sec per foot of embankment for the main dam.

USBR comments pertaining to the 35% Final SOD design are summarized as follows:

1. USBR noted that responses to the review comments for the 35% Final SOD design were “generally reasonable and responsive”. USBR in this letter provides comments to WWC’s responses to indicate where areas of uncertainty or concern continue to exist.
2. The final design alternative for the SOD modification was determined to be in general accordance with Reclamation and industry practice.

USBR comments pertaining to the 35% Final Design of the Enlargement project are as follows:

1. USBR noted that responses to review comments on geotechnical components of the 35% final design of enlargement project were “unsubstantiated and did not adequately address Reclamation’s technical review comments.”

This Technical Memorandum considered the areas currently served by Ray Lake, which include the lower portion of the Coolidge Canal and the entire area serviced by the Sub-Agency Canal. The study analyzed hydrologic and other pertinent data to estimate the potential for additional utilization of flows from the North and South Forks of the Little Wind River to meet current and future demands. The study also developed preliminary layouts for supply and distribution canals, and enlargement options for Ray Lake to service an enlarged service area. The study included the following tasks:

2. Identify and obtain hydrological data, crop surveys, and other pertinent information including USGS records, previous reports and on-going studies.
3. Analyze data to determine future needs for stored water in the Little Wind River unit in addition to, and beyond, the existing storage system and service area.
4. Analyze data and project facilities to determine the extent to which high flows in the North and South Forks of the Little Wind River can be used to meet the current and potential demand.
5. Develop preliminary layouts for the supply canal, distribution canals, and enlargement scenarios for Ray Lake, including modifications of the Ray Lake service area.

Key findings resulting from this study are as follows:
- Enlargement of Ray Lake is a cost-effective way to better meet irrigation demands on the Little Wind Unit of the Wind River Irrigation Project.
- Analysis concluded that an average 22,900 acre-feet annually is divertable from the North Fork and South Fork of the Little Wind River for storage in Ray Lake.
- An average of 16,500 acre-feet annually of Ray Lake storage is required by crops in the Little Wind Unit.
- Active storage requirement for the existing irrigation season during an 80% exceedence year, based on current requirements, is 26,000 acre-feet, with approximately 10,400 acre-feet of annual carry-over storage.
- The inlet canal was sized to be large enough to allow conveyance of storage water and irrigation water during the irrigation season.
  - The inlet canal requirement was determined to be 290 cfs.
  - A new inlet location was proposed that would divert directly from Ray Canal and enter the lake through a draw to the northwest of Ray Lake. This would require enlarging about ¼ mile of Ray Canal between the headgate for Lateral 37C to the new inlet canal location.
- Two outlet structures and new canals will allow an increase in the Ray Lake service area, and alleviate some of the demand on Washakie Reservoir.
  - The south outlet canal was sized at a capacity of 60 cfs
  - The north outlet canal was sized at approximately 274 cfs
- The reservoir was sized at 36,000 acre-feet. This provides 26,000 acre-feet of active storage. The remaining 10,000 acre-feet is recreational pool. The large recreational pool is necessary to keep the reservoir elevation at the bottom of active storage up at about 5532 feet elevation. This head is necessary to deliver water out of the south outlet to Lateral 65C and through the north outlet to the Coolidge Canal. The south outlet must have enough driving head to get water to Lateral 65C and overcome head losses associated with 18,000 feet of canal, 3 road culvert crossings and an inverted siphon under Mill Creek. On the north end, the main issue for water delivery to the Coolidge Canal is that a large cut is required through the ridge that separates Mill Creek and Trout Creek drainages. The higher the reservoir elevation, the less cut is required. The report recommended optimizing the elevation of the north outlet canal during final design.


The Draft Environmental Assessment (EA) was based on the 95% Final Design (WWC, 1996) of a 35,000 ac-ft reservoir as the Proposed Action. Other alternatives considered included No Action, Dam Safety Modifications Only, Dam Safety Modifications plus Water Conservation, and Other Reservoir Sites.
2001 - 2003 BASIN PLANNING STUDIES


This study assessed the need for water storage in the Upper Wind River basin, identified alternative reservoir sites, and performed a screening analysis of the sites. Following the Phase I alternative storage site screening, Phase II detailed analyses were conducted to evaluate short-listed sites in more detail, including preparation of conceptual-level designs and cost estimates. Enlargement of Ray Lake up to 41,650 ac-ft passed the Phase I screening and was recommended as a project to be considered for future study. It was not considered necessary to conduct Phase II conceptual design because of the substantial design work already completed at the site.


The basin plan was completed in October 2003, and identified the Ray Lake Enlargement project (41,650 ac-ft) as a short-listed candidate for further consideration.
APPENDIX C
Yield Analysis with Canal Sizing Scenarios
### 100-400 Reservoir Size

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### 50-300 Reservoir Size

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### 50-125 Reservoir Size

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Ray Lake Yield Analysis with Boysen Reservoir Calls and No Winter Delivery

50% Yield

Reservoir Size (Acre-Ft)

Firm Yield (Acre-Ft)

- North Fork = 50, Lateral 37C = 300
- North Fork = 150, Lateral 37C = 500
- North Fork = 100, Lateral 37C = 400
- North Fork = 50, Lateral 37C = 125
Ray Lake Yield Analysis with Boysen Reservoir Calls and No Winter Delivery

70% Yield

- North Fork = 50, Lateral 37C = 300
- North Fork = 150, Lateral 37C = 500
- North Fork = 100, Lateral 37C = 400
- North Fork = 50, Lateral 37C = 125

Reservoir Size (Acre-Ft) vs. Firm Yield (Acre-Ft) graph.
Ray Lake Yield Analysis with Boysen Reservoir Calls and No Winter Delivery

80% Yield

Firm Yield (Acre-Ft)

North Fork = 50, Lateral 37C = 300
North Fork = 150, Lateral 37C = 500
North Fork = 100, Lateral 37C = 400
North Fork = 50, Lateral 37C = 125
Ray Lake Yield Analysis with Boysen Reservoir Calls and No Winter Delivery

90% Yield

Firm Yield (Acre-Ft) vs. Reservoir Size (Acre-Ft) Graph

- North Fork = 50, Lateral 37C = 300
- North Fork = 150, Lateral 37C = 500
- North Fork = 100, Lateral 37C = 400
- North Fork = 50, Lateral 37C = 125
Ray Lake Yield Analysis with Boysen Reservoir Calls and No Winter Delivery

100% Yield

Firm Yield (Acre-Ft)

North Fork = 50, Lateral 37C = 300
North Fork = 150, Lateral 37C = 500
North Fork = 100, Lateral 37C = 400
North Fork = 50, Lateral 37C = 125