CHEYENNE SOUTH CROW
DIVERSION PROJECT

PHASE II
FINAL REPORT

Submitted to:

Wyoming Water Development Commission
Herschler Building, 4th Floor
122 West 25th Street
Cheyenne, WY 82002

Submitted by:

ECI
The Water Resource Division of FR Harris, Inc.
5660 Greenwood Plaza Boulevard
Suite 500
Englewood, CO 80111

August 2000
August 8, 2000

Mr. Bruce Brinkman  
Wyoming Water Development Commission  
Herschler Building 4th Floor West Wing  
122 West 25th Street  
Cheyenne, WY 82002

Dear Bruce:

Enclosed are 35 copies of our approved Final Report for the above-referenced project. The required reproducible unbound copy and the digital version of the report and other project files are also being submitted separately at this time. These services were performed per our contract dated June 6, 1999.

This report summarizes the services performed for Phases I and II of this study, including an inventory of the existing system, problem identification, development of preliminary alternative plans, conceptual plans, geotechnical evaluations, identification of permitting and easements required, cost estimates and an economic analysis and project financing. We finalize this report after receipt of review comments from Wyoming Water Development Commission and the City of Cheyenne Board of Public Utilities.

This completes our work on this project. We appreciated your support and the input and help we received from the good staff at the City's Board of Public Utilities. If you need anything, please let us know and we will try to provide it.

We look forward to continuing our relationship. Please call if you have questions.

Sincerely,

R. Joseph Bergquist  
Project Manager

RJB/td

Enclosures
CHEYENNE SOUTH CROW DIVERSION PROJECT

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Submitted to:

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August 2000
PROFESSIONAL CERTIFICATION

I, Richard Joseph Bergquist, P.E., hereby certify that the professional services required for the Cheyenne South Crow Diversion Project, Level I and II, were performed by me or under my direction and that I am a professional Engineer licensed in Wyoming as required by the provisions of W.S. 33-29-105 through W.S. 33-29-113. IN WITNESS WHEREOF I have hereunder set my hand and affixed my seal.

By: Richard Joseph Bergquist, P.E. #6330
Projects Manager, ECI

I, Neil R. Sherrod, P.G., hereby certify that I am a Professional Geologist licensed as required by the provisions of W.S. 33-41-101 through W.S. 33-41-121, and that all geological work performed in relation to the Cheyenne South Crow Diversion Project, Level I and II, was performed by me or under my direction. IN WITNESS WHEREOF, I have hereunder set my hand and affixed my seal.

By: Neil R. Sherrod, P.G. #202
Senior Engineer Geologist, Terracon
EXECUTIVE SUMMARY

INTRODUCTION

In 1999 the Wyoming Legislature authorized the Wyoming Water Development Commission to secure engineering services to conduct an investigation of the existing South Crow diversion dam and pipeline rehabilitation needs for the City of Cheyenne’s Board of Public Utilities. The investigation was divided into two phases; a reconnaissance (Level I) study and, a feasibility (Level II) study.

The Commission selected ECI, in association with AVI and Terracon Consultants, to provide the necessary engineering services for the study and evaluation. The main tasks of the two-phased study were the following:

- Determine the feasibility of either reconstructing the diversion structure or constructing a new dam upstream of the existing diversion structure.
- Establish the benefits of both alternatives: (1) rehabilitation of the diversion structure and (2) a new upstream water storage dam.
- Evaluate the condition of the existing 2.5-mile-long, 16-inch-diameter pipeline.
- Determine the remedial measures required to repair the existing pipeline, concrete diversion structure, and valve house.

PROJECT DESCRIPTION

The South Crow Diversion Dam is located approximately 22 miles west of Cheyenne, Wyoming, in Laramie County in the NW1/4 section 1, T13N, R70W. It was constructed in 1911 by the City to impound and divert the flows of South Crow Creek for municipal use. The structure also serves as diversion for local irrigation users.

Based on available information, the South Crow diversion structure is approximately 26 feet high and 207 feet long. The impoundment is approximately 12-acre feet (acft) of water. Depending upon the time of year, a maximum of approximately 3 mgd can be diverted to help meet Cheyenne’s water supply needs. A 16-inch gate valve at the diversion controls the amount of water diverted through the pipeline to the City’s treatment facilities. Only manual control of the valve can be performed at the site.

INVENTORY

The study documented the following:

- The diversion structure and reservoir are located on City owned property, however normal access to the structure is across privately-owned land.
• The 90-year-old dam has structural problems and has deteriorated due to freeze-thaw action over the years. Past repairs have not been durable. Leaks through the structure in several locations along the horizontal construction joints have caused severe deterioration on the downstream side.

• There is limited erosion and no aggregation of the streambed downstream of the structure.

• The sluice has not been operated recently and is assumed to be blocked by sediment that have reduced the reservoir storage from 12 to 9 acft.

• The actual structure is shorter than depicted in the 1910 design drawings. It also was built without a dissipation basin and the valve house is closer to the spillway than the drawings indicate.

• The City has a diversion water right of 7 cfs with a 1910 date for the South Crow diversion structure. The City has an expired storage permit referred to as Lake Helen for a South Crow Creek Reservoir.

• The City has a sixty-foot easement for the South Crow Pipeline which grants the City the right to enter upon the land for the purpose of constructing, operating, repairing, maintaining, and inspecting the pipeline.

• BOPU staff has established the value of raw water to be approximately $1.32 per 1000 gallons or $200/acft. Treated water costs $2.02 per 1,000 gallons in 1999. They serve 65,000 residents in the City and surrounding area.

• The diversion structure and thus the future rehabilitation project is located in potential Prebles meadow jumping mouse protection areas.

Field work conducted during the summer and fall established the following:

• The pipeline appears to be in good condition and has limited corrosion damage.

• Modern day devices, such as air-release valves, remote control, and motorized valve operators are not installed on the pipeline.

• Operation of the pipeline in the last 10 years has been limited to flushing and occasionally supplying water to the City of Cheyenne water treatment plants. The lack of more frequent operation has been due to water quality issues, especially the amount of turbidity associated with South Crow discharges.

**EVALUATION**

**Pipeline Condition Assessment**

Considering the existing pipeline was constructed over 90 years ago, it appears to be in good condition. Based on the sections examined, the exterior of the cast iron pipeline has minimal pitting. Soil analysis
and corrosivity tests indicated that even though the pipeline runs through “hot” areas (conducive to corrosion) it was best to not disturb the situation by adding cathodic protection at this time.

Hydraulic capacity of the South Crow pipeline is dependant on how it is operated in conjunction with the rest of the system. The estimated capacity of the South Crow pipeline with the Hecla pipeline in operation varies from 7.3 to 7.5 mgd (11.3 to 11.6 cfs). The South Crow line, with the Hecla line closed, can discharge 7.4 mgd (11.5 cfs). According to the water rights the City holds for the diversion the maximum discharge is 4.5 mgd (7 cfs).

The evaluation concluded that the pipeline is in good condition and, except for the noted deficiencies such as air valves, could be placed back into operation. Provided BOPU maintenance practices are continued, it is estimated that a number of years of serviceable life remains.

Diversion Structure Condition Assessment

According to the State of Wyoming dam safety criteria the South Crow Diversion is classified as a low hazard small size structure with aging problems. Portions of the diversion structure, due to deterioration from freeze-thaw action and ice loading, is not in good condition. Sections of the upstream face of the structure, below the depth of ice on the reservoir are in good condition. The most severe deterioration is on the downstream face along the right side of the non-overflow section. Exposed concrete on the left side of the non-overflow section appears to be in reasonably good condition compared to the right side.

The gravity method of analysis was used to check the overturning and sliding stability of this dam. The results indicated the existing South Crow Diversion Dam requires modification to be stable and meet the required guidelines. The conclusion was that unless the diversion structure is rehabilitated, it could possibly fail in the near future. Failure will be dependant on the rate of continued concrete deterioration.

Easements and Permits

Construction access easements will be required for the diversion structure rehabilitation. The last 2.5 miles of access road to the site from the Crystal Lake Road will require upgrading. Prior to the start of construction the BOPU should negotiate and obtain a permanent maintenance easement for access and finalize the appropriate permits. A summary of the most significant permits are included in the main report. Any new dam alternative would also require a current day priority right to store additional water and a filing for adjudication for new storage.

DIVERSION STRUCTURE REHABILITATION ALTERNATIVES

Evaluation of existing and collected data and discussions with project sponsors identified the following potential rehabilitation alternatives for the South Crow diversion structure:

- Replacement of existing structure.
- Seal upstream surface of diversion structure with sealants, geotextiles, or grout.
Overlay upstream face of existing structure with concrete.
Overlay downstream face of existing structure with concrete, RCC, or soil-cement.

The downstream overlay alternatives have an advantage, compared to an upstream overlay or surface treatments, because concrete work can be performed on the downstream side of the diversion structure in the dry without draining the reservoir. These alternatives were costed and compared. Environmental and other constraints were considered in selecting the alternative to carry into Phase II.

**Phase I Conclusions and Recommendation**

From the results of the Phase I work it was concluded:

- The downstream concrete overlay alternative would provide the best protection of the existing structure, create a more stable structure satisfying State Engineer criteria, and have the lowest cost.
- The pipeline requires addition of air release and blowoff construction valves.
- The valve house requires an overhaul of the structure, valves and addition of new controls.
- The New Dam upstream alternative is uneconomical and would require trans basin imported water to function as an effective storage facility.

Phase I ended with the recommendation that the downstream concrete overlay with a 3-foot-high crest raise rehabilitation is the cost-effective alternative to carry forward into Phase II.

**Phase II Conceptual Design**

Components of the selected alternative based on the conceptual design are described in the following paragraphs:

**Pipeline**

Three air release valves should be located along the South Crow pipeline route: one downstream of the valve house, two at the high points in the pipeline. Two blowoff valves should be installed per BOPU instructions.

**Diversion Structure's Concrete Gravity Section**

A downstream concrete overlay should be used to improve and stabilize the existing concrete gravity section. Stability analysis performed on this section, concluded the new mass of concrete should extend from the edge of the existing structure approximately 3.5 feet at the top to the downstream side. The shape and a new slope of 0.725H:1.0V should be used to define the new gravity section following
concrete gravity dam design criteria as required by Wyoming’s SEO Dam Safety Engineer. Above the junction of the sloping and vertical wall at elevation 7124, the concrete section becomes a rectangular section extending to elevation 7135.

**Spillway**
The spillway section should be similar to the non-spillway section with a new crest elevation set at 7132.42. Per free board requirements the top of dam should be set at elevation 7135. The new spillway should have a crest length of 50 feet. The rancher’s irrigation diversion intake should be incorporated into the new spillway section. Grouted riprap will be considered during the final design if rock is not sufficient at the base of the spillway.

**Sluice Outlet**
Rehabilitation for the existing sluice outlet should include replacement of the existing gate with a new silt proof gate housed in a new tower that provides access to the gate. The arrangement keeps the gate free from being block by sediment. This work should be conducted while the reservoir is lowered to repair the upstream concrete face.

**Valve House and Valves**
The valve house should be rebuilt above elevation 7132 and equipped with new motorized valves and a SCADA system. Grating should be added along with new trash screens and access ladders and safety railing.

**SCADA**
A system to provide monitoring of the water quality and control water releases from South Crow Diversion should be added. The preferred method of communications is via radio interface with the existing system. An alternative system would be to utilize a satellite or telephone line if phone lines are made available from Crystal Road to the site. When built the operation and management SCADA system should include receivers at the South Crow Dam site and the South Crow pipeline junction with the 20-inch Hecla south pipeline supported by a store and forward repeater situated at the Cooper King Mine site. For purposes of this study it was assumed a repeater could be located at this site if needed.

**Access Road**
A new construction road will be required for the 2.5 miles from the Crystal Lake Road to the site and should be built and eventually converted into an operation and maintenance road.

**Transmission Line**
Power for the South Crow facility should be provided by a new single phase 14.4 Kilovolt power line (approximately 2.5 miles) following the proposed access road from the Hecla pipe line junction to the South Crow Diversion.
**Project Cost**

The opinions of probable construction cost for these conceptualized components of the project provided herein are made on the basis of ECI's experience and qualifications and represent our best judgement as an experienced and qualified professional engineers generally familiar with the construction industry. The estimated probable cost for the total project including engineering, site access easement, and permit assistance is $1,060,000.

**ABILITY TO PAY**

A repayment plan would be based on a 50 percent grant and 50 percent loan available from WWDC. The term of the loan would be 20 years with an interest ratio of 7.25 percent. These figures generate an annual cost to the City of Cheyenne's BOPU of $61,000 per year, which works out to be an increase of $2.66 per equivalent 3/4 inch tap per year.

**PHASE II FINAL CONCLUSIONS AND RECOMMENDATIONS**

Based on the results of the work completed for Phase II the following conclusions and recommendations are provided:

1. Final design and rehabilitation efforts should proceed within the next two years to prolong the life and stabilize the structure and preserve the City’s diversion water rights.

2. The new dam alternative will only be economical if out-of-basin water can be imported and stored. The watershed annual runoff is already fully allocated and no new water for storage is available.

3. The recommended rehabilitation alternative for the diversion structure is a downstream concrete overlay.

4. The recommended rehabilitation alternative for the pipeline is addition of air release and blowoff valves in identified locations.

5. The recommended rehabilitation for the valve house includes removal of part of the old valve house structure, addition of new valves and motorized operators, and a new SCADA system.

6. An easement for construction access and future operation will be required. It is recommended that the City begin negotiations with landowners as soon as possible to secure this access.
Cheyenne South Crow Diversion Project
Phase II Final Report

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ABBREVIATIONS

acft = acre feet
AVI = AVI Professional Corporation, Inc.
BOPU = Board of Public Utilities
cfs = cubic feet per second
City = City of Cheyenne
cuft = cubic feet
ECI = ECI, the Water Resource Division of Frederic R. Harris, Inc.
FERC = Federal Energy Regulatory Commission
GPM = gallons per minute
HGL = hydraulic grade line
MCE = Maximum Credible Earthquake
mgd = million gallons per day
N = North
NOAA = National Oceanic and Atmospheric Association
NRCS = Natural Resource Conservation Service
NW = North West
pH = numerical measurement of acidity or alkalinity
PMF = Probable Maximum Flood
PMP = Probable Maximum Precipitation
psi = pounds per square inch
R = Range
RCC = Roller Compacted Concrete
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<td>United States Geological Survey</td>
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<td>Watershed Modeling Software</td>
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<td>Warren Air Force Base</td>
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INTRODUCTION

GENERAL

In their 1999 session, the Wyoming Legislature authorized the Wyoming Water Development Commission (WWDC) to secure engineering services to conduct an investigation of the existing South Crow diversion dam and pipeline rehabilitation needs. The investigation was divided into two phases; the first phase was a reconnaissance (Level I) study and the second phase was a feasibility (Level II) study. The funding allocation was in response to a study funding application submitted to the WWDC by the City of Cheyenne’s Board of Public Utilities (BOPU).

The WWDC selected ECI, in association with AVI and Terracon Consultants, to provide the necessary engineering services for the study and evaluation. The main tasks of the two-phased study were the following:

- Determine the feasibility of either reconstructing the diversion structure or constructing a new dam upstream of the existing diversion structure.
- Establish the benefits of both alternatives: (1) rehabilitation of the diversion structure and (2) a new upstream water storage dam.
- Evaluate the condition of the existing 2.5-mile-long, 16-inch-diameter pipeline from the South Crow diversion to the Hecla pipeline and the 26-foot-high concrete diversion structure.
- Determine the remedial measures required to repair the existing pipeline, concrete diversion structure, and valve house.

The pipeline and diversion structure are owned by the City of Cheyenne and operated by the City’s BOPU. If constructed, the upstream storage project also would be owned and operated by the City.

ECI entered into a contract with the WWDC to perform the work on June 6, 1999. A Scoping Meeting was held on June 10, 1999 and the official Notice to Proceed was received on June 22, 1999. The contract period is from June 1, 1999 to June 30, 2000. Work was scheduled to be performed in two phases. Phase I was focused on performing an inventory of the existing system and a basic evaluation of repair options. Phase II included a feasibility study of the selected Phase I repair option and a cost estimate of the upstream storage project. This report presents the results of the study.

PROJECT DESCRIPTION

The South Crow Diversion Dam is located approximately 22 miles west of Cheyenne, Wyoming, in Laramie County in the NW1/4 section 1, T13N, R70W, as shown in Figure 1. It was constructed in 1911 by the City to impound and divert the flows of South Crow Creek for municipal use. The structure also serves as diversion for local irrigation users.
During the past several years, the City has not relied upon the waters of the system and the diversion dam has developed several structural problems. Numerous leaks through the dam and severe concrete spalling at various locations have been observed. Although the structure is not being fully utilized, the City realizes it has potential to be a valuable component of its water supply system. As the City grows, the use of both potable and raw water will increase and the need for additional water storage on the east side of the divide will increase. The use of raw water for irrigation is presently projected for the Northwest Parks area (Airport, Country Club and WAFB golf courses and for Lions Park). Therefore, in 1998 the City decided to seek State funding support for this study to identify the need for rehabilitation of the system.

Available information established the dimensions of the South Crow diversion structure at approximately 26 feet high and 207 feet long. The impoundment is approximately 12-acft of water. Depending upon the time of year, a maximum of approximately 3 mgd can be diverted to help meet Cheyenne’s water supply needs. With a drainage basin of about 13 square miles, an estimated 500 acft of water can be diverted to the City of Cheyenne per year.

A 16-inch gate valve at the diversion controls the amount of water diverted through the pipeline to the City’s treatment facilities. Only manual control of the valve can be performed at the site. Therefore, it was decided that the South Crow diversion rehabilitation project should include Supervisory Control and Data Acquisition (SCADA) and metering to make it compatible with the rest of the City’s water supply control system.

**PROJECT OBJECTIVE**

The purpose of the project was to examine the feasibility of rehabilitating the existing dam structure and pipeline or to reconstruct the dam upstream of the existing structure and rehabilitate the pipeline. The project was completed in two phases as described below to develop a preliminary design and cost estimate for the preferred alternative. The City as the project sponsor, and WWDC will use the information from this study to develop the request for a Level III construction project.

Phase I (Level I Study), the diversion structure evaluation, included four tasks. The first task was a scoping meeting. The meeting was used to start and give focus to the project. Task 2 consisted of an inventory and evaluation of the existing system. Task 3 dealt with the development and analysis of rehabilitation plans. In Task 4 a finding report was developed and presented to document the findings.

Level I tasks were focused on:

- determining the structural integrity of the South Crow Diversion Dam,
- evaluating the foundations conditions of the existing site,
- providing recommendations for the repair and upgrade of the diversion structure dam,
- evaluating the outlet works,
- providing recommendations for the repair and upgrade of the outlet works, including the addition of SCADA equipment,
• evaluating approximately 2.5 miles of cast iron raw water delivery pipeline (14" - 16" diameter) from the diversion structure to the 20" diameter south (Hecla) water line,
• providing recommendations to repair and upgrade the delivery pipeline,
• evaluating water rights,
• providing preliminary cost estimates for repairs and other required work, and
• selecting the option to carry into Phase II.

The objectives of Phase II (Level II Study) were the conceptual design and cost estimate of the selected alternative. Phase II was comprised of eight tasks: conceptual design, permitting assistance, geotechnical evaluation, surveying, cost estimates, economic analysis and project financing, a report, and project result presentations.

SCOPING MEETING

A Scoping Meeting was held on June 10, 1999 in the City's BOPU office in Cheyenne to clarify the objectives of the study. Attendees at the meeting were the following:

WWDC: Bruce Brinkman - Project Manager
BOPU: Jeff Pecenka - Project Manager, and
         Herman Noe
         Chuck Blackburn
         John Trefren
ECI : Joe Bergquist - Project Manager
AVI : Bruce Perryman

In the meeting, Mr. Brinkman, explained that the Project is a combined Level I and II study with:
• an emphasis on the existing structures and their rehabilitation, and
• a minimal design effort required to develop costs for an upstream dam.

During the meeting it was agreed that:

1. Geologic mapping will be the basis for dam foundation determination and will be used for dam construction cost estimating. The topographic mapping would start at the existing structure and extend upstream through the canyon to cover the upstream dam site.

2. The diversion structure evaluation should be accomplished in Phase I. All data collected on the condition of the structure should be included in the Phase I Report. Depending on structure repair recommendations and the results of data collection review, additional investigations could follow
in Phase II. During Phase I, the concrete diversion structure should be surveyed and examined by a concrete specialist. The need for hand core drilling to extract concrete samples should be based on the concrete specialist’s assessment.

3. Hydrology data from the USGS and Water Rights Records from the State should be used to evaluate availability and combined with a watershed model to size the storage and diversion requirements. Project impacts on the water rights should be established with the support of a spreadsheet analysis (direct flow and storage). Extensive water transfer options, water rights analyses and modeling should not be performed to size a larger storage potential.

4. Land ownership records and field surveys should be used to establish the locations of the diversion structure and neighboring land ownership. For reporting and cost estimating purposes, ownership impacts should be separated into construction, access, or inundation categories.

5. The pipeline evaluation should include soil corrosivity measurements, ultrasonic thickness measurements, and visual inspection in six open pits excavated to expose the pipe at high and low points along the pipeline alignment.

6. The Phase I Study results should be a rehabilitation plan and estimated costs for the diversion structure and pipeline. The report should address the need to dredge the reservoir and SCADA to the system.

7. The Phase I report presented before the Phase II work can start should emphasize data collection documentation. The report should be between five to eight pages in length with data included in an appendix. The first phase report should document selection of an alternative to carry into Phase II conceptual design for rehabilitation of the diversion structure.

8. The final Phase II report should be submitted as a draft by May 1 and finalized by June 1, 2000. The report should be finalized after receipt of comments from key individuals from review of the draft report. At a minimum, the report should address the following:

- sediment problems
- water availability
- potential storage rights
- structure rehabilitation options and costs
- pipeline condition
- new controls and SCADA system
- economics
- permits/requirements
BACKGROUND

During the Phase I study the condition of the existing diversion structure and pipeline, and geology of the area was inventoried. The inventory started with a site visit on March 10, 1999. During the site visit, BOPU officials provided details on the existing access, structures and pipeline. They explained the system’s operation problems and gave their understanding of the rehabilitation effort required to place the system in good working order. The inventory process continued with a series of trips to the site and to other offices in Cheyenne including the Wyoming State Engineer’s Office (SEO) and Laramie County Assessors’ office. Field visits included surveying work, geologic mapping, and special investigations of the pipeline and structures.

Throughout the study BOPU employees, especially Mr. Herman Noe, Mr. Jeff Pencenka and Mr. John Trefren, provided valuable insight to the condition and operation of the diversion structure and pipeline, for which we are very grateful.

The results of the inventory (the work products from the first phase of study) are documented in Appendix A. A brief summary of the key findings are presented below.

DIVERSION STRUCTURE

During the site visit, the BOPU officials stated that the diversion structure should be rebuilt or rehabilitated to prevent further deterioration of the facility. They also indicated that the operation should be upgraded and provided with remote control capability even if the structure is not rebuilt or rehabilitated. They stressed the purpose of the repair and upgrading is to have the South Crow Diversion facility become a functional part of the water supply system.

The visit to the SEO of dam safety produced the following documents:

- Drawing dated 1910 showing alignment of auxiliary water supply mains for the City of Cheyenne.
- Drawing dated 1910 showing plan, elevation and sections of diversion dam. This drawing indicates the dam is 40 feet high at the maximum section and 226 feet long with a downstream slope of about 0.66H:1V.
- Drawing dated 1910 showing plan and sections of the 16-inch pipe intake.
- Listing of water rights on South, Middle and North Crow Creek and Crow Creek.
- Drawings (2 sheets) dated 1930 showing proposed Lake Helen upstream of South Crow Diversion Dam.
Surveyors and specialist project team members visited the site throughout the summer to determine and document the condition of the structure. Efforts used to inventory and assess the condition of the diversion and valve house structures included:

- assessment of the structure’s location with respect to fence lines and section lines established during field surveys.
- field surveys of the diversion structure, reservoirs foot print, and sediment deposition.
- assessment of the condition of the existing concrete structures with respect to the type of materials used to build the facility and potential repair methods, costs, and remaining design life.
- determination of downstream erosion or aggregation.
- identification of the geologic formations and foundation conditions.
- photographic and video documentation of the structure.

The findings documented in the appendixes, especially in Appendix A, concluded that:

- The diversion structure and reservoir are located on City owned property (see Land Ownership section in Appendix A).
- Access to the structure is across privately-owned land. The City’s easement for the pipeline is not used as a normal access route.
- The 90-year-old dam has structural problems and has deteriorated due to freeze-thaw action over the years. Past repairs have not been durable.
- Leaks through the structure in several locations along the horizontal construction joints have caused severe deterioration on the downstream side.
- There is limited erosion and no aggregation of the streambed downstream of the structure.
- The sluice has not been operated recently and is assumed to be blocked by sediment.
- Sediment deposition has reduced the reservoir storage from 12 to 9 acft.
- The actual structure is shorter than depicted in the 1910 design drawings. It also was built without a dissipation basin and the valve house is closer to the spillway than drawings indicated as shown in the as-designed and as-surveyed figures in Appendix A.
PIPELINE

The inventory and investigation of the condition of the pipeline was conducted during the summer and fall. The project team made field visits and collected existing data to assess the condition of the 90-year-old cast iron raw water supply pipe. Since the pipeline is buried, the inventory and investigative work included:

- Exposing the pipe at six locations for ultrasonic thickness determination, visual inspection, and soil sampling.
- Soil corrosivity - soil resistivity testing at 500-foot intervals or less, along the pipeline route from the diversion structure to its junction with the Hecla pipeline.
- Soil samples testing.
- Flow capacity tests.
- A review of historic operation and maintenance efforts.

The focus was to determine the alignment, size, and capacity of the pipeline. Data were collected to evaluate the pipeline's remaining design life and establish the need for replacement. The results of the inventory process are documented in Appendix A and the results of the pipeline corrosion assessment are documented in Appendix B. The evaluation of the pipeline’s capacity follows in the next section.

It was concluded from the inventory process that:

- The pipeline appears to be in good condition and has limited corrosion damage.
- Modern day devices, such as air-release valves, remote control, and motorized valve operators are not installed on the pipeline.
- Operation of the pipeline in the last 10 years has been limited to flushing and occasionally supplying water to the City of Cheyenne water treatment plants. The lack of more frequent operation has been due to water quality issues, especially the amount of turbidity associated with South Crow discharges.

Flow capacity tests were inconsistent and thus were only considered indicative.

OTHER ASPECTS

The inventory established the following:

- The City has a diversion water right of 7 cfs with a 1910 date for the South Crow diversion structure.
- The City has an expired permit referred to as Lake Helen for a South Crow Creek Reservoir.
• Beyond the land in Section 2, T13N, R70W, owned by the City on which the reservoir is located, the City owns a small piece of land in Section 1, T13N, R70W in 1912 where the diversion structure is located.

• The City has a sixty-foot easement for the South Crow Pipeline which grants the City the right to enter upon the land for the purpose of constructing, operating, repairing, maintaining, and inspecting the pipeline.

• The diversion structure is located in the widened part of a narrow valley in rock from the Precambrian metasediments. The bottom of the creek is heavily vegetated with grasses, brush and trees.

• The west abutment of the structure is keyed into a rock outcrop consisting of granitic rock interbedded with highly weathered schist. The east abutment appears to be in a small alluvial fan or a gently eroded soil slope consisting of slope wash, several feet in thickness.

• The pipeline is located almost entirely in the White River Formation. This formation consists of alternating claystone, sandstones and conglomerates.

• BOPU staff has established the value of raw water to be approximately $1.32 per 1000 gallons or $200/acft. Treated water costs $2.02 per 1,000 gallons in 1999.

• BOPU serves approximately 65,000 residents in the City and surrounding area.

• The diversion structure and thus the future rehabilitation project is located in potential Prebles meadow jumping mouse protection areas, see Figure 2.
Figure 2

Laramie County, Wyoming
EVALUATION

INTRODUCTION

Each project feature inventoried was evaluated to determine:

- What needs to be replace or repaired.
- If it can be repaired, how should it be rehabilitated.
- Is it worth rehabilitating.

The evaluations included an assessment of pipeline flow capacity and control requirements, structural stability, and remaining design life. Documentation and calculations for all of the evaluations are included in Appendixes A through F. Evaluation of the SCADA system requirements is presented in Appendix F.

PIPELINE

Condition Assessment

Considering the existing pipeline was constructed over 90 years ago, it appears to be in good condition. Based on the sections examined, the exterior of the cast iron pipeline has minimal pitting. The apparent excellent condition is attributed to the external lacquer coating and original cast iron wall thickness. The single small pocket of corrosion found on the tee (Test Pit No. 5) to a blow-off will be removed when the blow-off is replaced.

Soil analysis and corrosivity tests indicated the pipeline runs through “hot” areas (conducive to corrosion) as well as non-corrosive conditions. The conclusion was to not disturb the situation by adding cathodic protection. If leaks develop in the future, minimal corrosion protection would be installation of galvanic anodes for hot spot protection.

Ultrasonic testing of wall thickness at five locations indicated virtually no internal corrosion damage at these locations. Therefore, it was concluded that the remainder of the pipeline is in similar condition. In the future, if corrosion leaks start to occur on a frequent basis, the BOPU has the option to clean and line the pipeline with a polyethylene liner or replace sections of the pipe.

Hydraulic Analysis

Hydraulic capacity of the South Crow pipeline is dependant on how it is operated in conjunction with the rest of the system. From the diversion structure, the pipeline runs 2.5 miles to a junction with a 20-inch cast iron raw water pipeline from Hecla Intake, as shown in Figure 3. The 20-inch south line then joins a network of pipelines delivering water to the City’s water treatment plants as shown in Figure 4. The South Crow line capacity is affected by operation of the rest of the system.
Factors included in the analysis were whether the Hecla line operates at full or partial capacity and if it is submerged at the inlet. Operation of the south line in conjunction with the north line was another factor to consider. If the hydraulic grade line (HGL) between the two pipelines is equalized, the capacity of the South Crow pipeline could be different than if equalization does not occur. Therefore, to establish the capacity of the South Crow line, the hydraulic analysis was performed to consider the following cases:

South Crow line in operation alone with a constant head at the South Crow Diversion structure and no downstream backwater pressure at the junction (Appendix A).

South Crow water flowing into the 20-inch south line and on to the WTP under a constant head at the South Crow Diversion structure and at the Hecla intake (Appendix C).

South Crow line operating with a constant discharge and constant elevations at the both intakes (Appendix C).

The 20-inch line from Hecla operating with a constant head at the Hecla intake to carry water to the WTP (Appendix C).

The South Crow line operating with a constant head at the diversion structure and Hecla closed (Appendix C).

Prior to conducting the hydraulic analysis, the pipes inlet and exit elevations; pipe lengths and sizes; pipe roughness and location of major flow obstructions had to be determined. The survey and inventory process provided the elevations, lengths and pipeline sizes. Friction factors, which are an indication of pipe roughness, were first assumed and then refined through an iteration process. Location of obstructions, such as air relief valves, were assumed.

According to BOPU personnel, an insufficient number of air relief valves on the line limits smooth flow characteristic by resulting in entrained air and foam under full flow conditions. Therefore, the hydraulic analysis was performed assuming air relief valves will be installed in the future.

Friction losses were estimated with the aid of pipe roughness handbooks and the Darcy-Weisbach equation. The Darcy-Weisbach friction factors were compared to the Hazen-Williams friction coefficient “C” and found comparable to past estimates between 90 to 100. Bend and other losses were estimated as a function of velocity for a given discharge.

Based on these data and estimated friction factors, the calculated capacity of the South Crow and Hecla pipeline varies from 7.3 to 7.5 mgd (11.3 to 11.6 cfs) depending on the case analyzed. The South Crow line, operated with the Hecla line closed, can discharge 7.4 mgd (11.5 cfs) to the WTP (Appendix C). This compares favorably with other estimates and flow tests on the 20-inch south line.

Earlier studies reported the capacity of the two 20-inch lines (south and north lines) which deliver water to Round Top Water Treatment Plant (WPT) at about 12 mgd (18.6 cfs) (Black and Veatch, 1994). States West Water Resources Corporation, in a 1996 report, states that based on discussions with BOPU personnel, the 20-inch line capacity is 9-10 mgd (14-15.5 cfs). In the same report, flow test results for two
conditions indicate the flow to be 6.6 mgd (10.2 cfs) with the WYE valve open (HGL equalized between the south and north lines) and 6.28 mgd (9.7 cfs) with the WYE valve closed. The closed WYE condition is similar to Case No. 5, which is South Crow line in operation with Hecla intake closed.

However, the calculated capacity of 11.5 cfs is more than permitted by the decreed water right to be discharged from South Crow Creek at this diversion point if more senior water rights holders downstream have requested the water. According to the water rights, maximum discharge is 4.5 mgd (7 cfs). Until the pipeline can be rehabilitated with the addition of air relief valves and tested, the maximum discharge should be limited to less than 7 cfs. Once the line has been improved, BOPU can attempt another flow tests to confirm the capacity and ability to withstand the pressure. Until the repairs are made and tested, it is only possible to state that:

- During a flow test the South Crow pipeline discharge was measured to be 3 cfs.
- Under limited flow conditions in South Crow Creek, the maximum flow the South Crow pipeline can carry is 7 cfs.
- During a flow test, the 20-inch South pipeline with the WYE closed discharged at 9.7 cfs.

Mathematically, the maximum flow capacity of the South Crow pipeline is 11.5 cfs.

**Remaining Design Life**

The pipeline appears to be in good condition and, except for the noted deficiencies such as air valves, can be placed back into operation. Provided BOPU maintenance practices are continued, it is estimated that at a number of years of serviceable life remain.

**DIVERSION STRUCTURE**

Evaluation of the diversion structure started with collection of data from the field and office inventory effort. This information is summarized under pertinent data below. The assessment consisted of structure size verification, flood routing study and hazard classification and stability analysis.

This same assessment was performed for the proposed new dam upstream of the diversion structure at the Lake Helen dam site and document in Appendix G.

### TABLE 1: Pertinent Data on the Existing Diversion Structure

<table>
<thead>
<tr>
<th>Drainage Area</th>
<th>12.05 sq.mi.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Discharges and Capacities</td>
<td></td>
</tr>
<tr>
<td>Spillway capacity</td>
<td>345 cfs</td>
</tr>
<tr>
<td>Outlet (sluiceway)</td>
<td>Unknown</td>
</tr>
<tr>
<td>City’s Raw Water Pipeline</td>
<td>7.0 cfs (per water right decree)</td>
</tr>
</tbody>
</table>
Reservoir

Normal Water Surface Elevation 7,130.75 ft
PMF Water Surface Elevation 7,140.00 ft

Elevations

Top of dam 7,132.00 ft
Spillway crest 7,130.75 ft
Invert outlet structure 7,125.00 ft
Sluice invert 7,112.8 ft (per 1990 survey)

Storage

Normal Water Surface El. 7,130.75 12.0 acft (per design)
Normal Water Surface El. 7,130.75 8.88 acft (per 1990 survey)

Dam

Type-Concrete gravity
Crest Length 128 ft
Height (maximum) 26 ft
Crest width 4.0 ft
Side slopes:  Downstream 0.65H:1.0 V
Upstream  Vertical

Size and Hazard Classification

According to the State of Wyoming dam safety criteria obtained in discussions with the Dam Safety Officer, the South Crow Diversion is classified as a low hazard small size structure.

The basis for the low hazard classification is the lack of residential dwellings or other buildings or state highways located immediately downstream. In the event of failure of the diversion structure, excessive damage would not occur to downstream property nor would there be a potential for loss of life.

The diversion structure is classified as a small size dam because its height is less than 40 feet and storage less than 50 acft.

Condition Assessment

The diversion structure was constructed over 90 years ago. Due to deterioration from freeze-thaw action and ice loading the structure is not in good condition. Most all concrete surfaces inspected exhibit some form of cracking or spalling. Exposed steel portions were rusted. The sluice gate is inoperable.
The valve house also shows some concrete deterioration. There are no bulkheads, gratings, or trash screen. The valve has a manual operator housed in a small room with no electricity.

The drop inlet for the adjacent ranch's irrigation diversion is broken. It is also difficult to reach. Access to the spillway is over the top of the valve house where the rancher's inlet is located. Access to the sluice gate is also over the valve house.

Concrete is deteriorated from the level of normal pool down to several feet below water line. The upstream face of the structure, below the depth of ice on the reservoir in good condition. The most severe deterioration is on the downstream face along the right side of the non-overflow section, looking downstream. Construction joints on the sloping surface below the crest are severely eroded to a depth of more than 6 inches and possibly as much as 12 inches in some locations. Substantial water flow was observed from holes along these construction joints at two locations.

Erosion of deteriorated concrete in the spillway section appears to be reasonably uniform across the sloping downstream face. One severely eroded horizontal construction joint about 5 feet below the crest appears to extend from one abutment to the other and makes the top of the dam appear as a rectangular beam. The concrete below this construction joint appears to be in much better condition except for some slightly deeper section near the toe of the spillway.

Exposed concrete on the left side of the non-overflow section appears to be in reasonably good condition compared to the right side. A few vertical cracks were observed on the right side of the non-overflow section but do not appear to be contributing to the deterioration.

There was no evidence of sulfate attack or cement-aggregate reactivity.

The downstream slope is steeper than modern design criteria and permitted by the Wyoming SEO rules and regulations. The combination of structural deterioration and steepness of the downstream slope has created a potentially unsafe structure because the factor of safety is low.

Structure Stability

The gravity method of stability analysis (Appendix D) was used to compute the factors of safety for sliding and overturning for the diversion structure. The gravity method is based on the assumptions that a straight gravity dam is comprised of a number of unit width vertical cantilever elements, each of which carries its own load to the foundation without transfer of load from or to adjacent vertical elements and that vertical stresses vary linearly. The factors of safety and stresses are computed at the base elevation of the dam or at selected elevations above the base. This method is used solely for straight gravity dams in which the transverse contraction joints are neither keyed nor grouted.

The gravity method of analysis provides an approximate means for determination of the overturning and sliding stability of a gravity concrete dam. It is applicable to the general case of a gravity section with a vertical upstream face and a constant downstream slope and to situations where there is a variable slope on either or both faces. Use of the gravity method requires several simplifying assumptions concerning the application of loads on the dam and the structural behavior of the dam. These assumptions include: (1) All
loads are transmitted to the foundation through cantilever action of the dam without support from adjacent monoliths; and (2) Normal stresses are distributed uniformly on horizontal planes.

Cracked Base Analysis:
The dam must be safe either with or without uplift; therefore, the concrete stresses and foundation reactions were computed with and without uplift to determine the maximum conditions. Base pressures were calculated excluding uplift. These pressures were compared to the uplift pressures at each point across the base to identify any portion of the base not in compression (i.e., where the foundation reaction is less than the uplift pressure a "cracked " section was assumed and, thereby, subjected to full headwater pressure). This condition requires that the uplift diagram be modified and a cracked base analysis be performed. This method of handling uplift was developed by the USBR and is considered the most appropriate for determining the initiation of interface cracking for existing dams [FERC, 1987].

Material Properties:
The concrete material properties assumed in the model were based on published data for similar 1910 concrete gravity structures. Concrete was assumed to have a minimum compressive strength of 2,450 lb/in² and have an average unit weight of 150 lb/ft³. The instantaneous modulus of elasticity for the concrete was estimated from Section 8.5.1 of ACI 318-95 based on the compressive strength, and which yields a value of 3,474,472 lb/in². The sustained modulus of elasticity was assumed to be 70% of the instantaneous value [USBR, 1977].

The effective cohesion for the dam foundation was estimated to be 15 psi (Appendix E). The effective internal friction angle used was assumed to be 45 degrees. A summary of the assumed material properties in original structure is presented in Table 2 below.

<table>
<thead>
<tr>
<th>Table 2: Summary of Material Properties</th>
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<tbody>
<tr>
<td>Material</td>
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<tr>
<td>---------------------------</td>
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<tr>
<td>Concrete Strengths</td>
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<tr>
<td>Conc. Modulus of Elasticity</td>
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<tr>
<td>Concrete</td>
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<tr>
<td>Foundation</td>
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<td></td>
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<tr>
<td></td>
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<tr>
<td>Water</td>
</tr>
</tbody>
</table>

*Based on tensile and shear strength equal to 10% of the compressive strength.
Loads:
The stability of the dam was analyzed for static and dynamic loads. The static loads included gravity, normal and flood reservoir elevations, sediment, ice, uplift, and normal and assumed tailwater elevations. The dynamic loads used pseudo-static analysis methods to evaluate the effects of the Maximum Credible Earthquake (MCE.)

Gravity:
The gravity load used a unit weight of 150 lb/cuft for existing concrete.

Silt:
The submerged sediment density was assumed to be 22.5 lb/cuft horizontally and 57.5 lb/cuft vertically.

Reservoir:
The hydrostatic loads corresponding to the normal and PMF flood reservoir levels were simulated using 62.4 lb/cuft. The maximum normal water surface for the reservoir was assumed to be at elevation 7,135 ft. The maximum reservoir water surface due to the PMF condition was assumed to be elevation 7,138 ft. The tailwater was estimated to be 4.15 feet of depth based on the 100-year hydraulic event (per flood routing analysis in the next section).

Uplift:
Uplift was applied to the section for computing the sliding stability factor of safety along the dam/foundation contact. For a cracked base condition the uplift load was assumed to be equal to the full reservoir head within the cracked portion of the base, then vary linearly from the full reservoir pressure at the crack tip to the tailwater pressure at the downstream end (Jansen, 1988). The effect of drains was not considered in the analysis.

Ice:
Ice loading was developed using USBR design criteria [USBR 1965]. Ice load was assumed to be 5,000 lbs. per linear foot of contact between the ice and dam for an ice depth of one foot applied at normal water surface.

MCE (Maximum Credible Earthquake):
Earthquake loading was applied using the pseudo static method of analysis from the USBR Design of Small Dams as well as the Uniform Building Code (UBC 94). The region is categorized as Zone 1 for earthquakes. A value of 0.052 was used for the horizontal ground acceleration. Hydrodynamic loading was also applied for the seismic analysis.

Each load was applied independently to the model to study the effect on the structure. This approach provides a method for checking the combined load stresses and for determining the loads that contribute the majority of stress and deflection to each load combination. The load combinations were obtained by
summing the stresses and deflections from each independent load analysis. The loads on the maximum cross section of the dam are shown in Appendix D.

Load Combinations:

The load combinations used to evaluate the behavior of the dam are summarized as follows:

- **Usual**: Normal water surface, dead load, silt, ice, uplift, tailwater.
- **Unusual**: Maximum water surface, dead load, silt, tailwater, uplift.
- **Extreme**: Usual loading, MCE.

Stability Criteria:

Basic requirements for stability of a gravity dam for all conditions of loading are [USBR, 1977]:

- Allowable unit stresses in the dam or foundation are not excessive. The allowable stresses are determined by dividing the ultimate strengths of the materials by the appropriate safety factors.
- Safe against failure by shear or sliding along a horizontal plane within the structure, at its contact with the foundation or within the foundation.
- Safe against overturning along any horizontal plane within the structure, at its contact with the foundation or within the foundation. Stability against overturning is usually determined by calculation of the location of the resultant of all forces acting on the structure and by determination of internal, interface and foundation stresses. Foundation stresses usually are determined without uplift included to check for the cracked base condition.
- Capable of surviving the MCE without failure of a type that would result in loss of life or significant damage to downstream property. Inelastic behavior with associated damage is permissible under the MCE which is specific to the site.

Factors of Safety:

The factors of safety were established in accordance with Wyoming’s SEO dam safety officer’s statements to use FERC guidelines. Accordingly, the factors of safety used for comparison with computed allowable stresses and shear friction factors of safety within the structure and at the concrete to foundation interface are listed below in Table 3:

<table>
<thead>
<tr>
<th>Table 3: Minimum Factors of Safety</th>
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<tbody>
<tr>
<td><strong>Loading Condition</strong></td>
</tr>
<tr>
<td>Usual</td>
</tr>
<tr>
<td>Unusual</td>
</tr>
<tr>
<td>Extreme</td>
</tr>
</tbody>
</table>
Allowable Stresses:

The following allowable stresses were established based on USBR criteria [USBR, 1977]:

**Concrete Compressive Stress:**
- Usual Load Combination 1,000 lb/in²
- Unusual Load Combination 1,500 lb/in²
- Extreme Load Combination 2,450 lb/in²

**Concrete Tensile Stress:**
- Usual Load Combination 100 lb/in²
- Unusual Load Combination 150 lb/in²
- Extreme Load Combination 245 lb/in² *

*Based on a factor of safety greater than 1.0 and tensile strength equal to 10% of the compressive strength. Tensile strength of the concrete-foundation interface was assumed to be zero.

**Results:**

The existing South Crow Diversion Dam requires modification to be stable and meet the required guidelines. The calculated and allowable factors of safety are summarized below in Table 4. Alternatives to improve the stability are discussed in a subsequent section of the report and in Appendix A.

<table>
<thead>
<tr>
<th>Table 4: Existing Diversion Structure Stability - Factors of Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Sliding Stability</strong></td>
</tr>
<tr>
<td><strong>Load Case</strong></td>
</tr>
<tr>
<td>Usual</td>
</tr>
<tr>
<td>Unusual</td>
</tr>
<tr>
<td>Extreme</td>
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</tbody>
</table>

**Remaining Design Life**

Unless the diversion structure is rehabilitated, it could possibly fail in the near future. Failure will be dependant on the rate of continued concrete deterioration.
HYDROLOGY AND HYDRAULIC ANALYSIS

INTRODUCTION

A hydrology and hydraulic analysis was completed to support the evaluation of the existing diversion structure as well as the development and evaluation of alternatives including the New “Lake Helen” Dam. The analysis was based on USGS streamflow gage data, topographic maps, NRCS soil maps, NOAA Atlas 2 Precipitation - Frequency Atlas of Western United States Volume II - Wyoming, and NOAA Hydrometeorological Report No. 55A. A summary of the analysis and results are presented below. Documentation of the analysis is presented in Appendix H.

FLOOD HYDROLOGY

Basin Description
South Crow Reservoir is located on South Crow Creek a tributary to Crow Creek in Laramie County, between Laramie and Cheyenne, Wyoming. The drainage area of the reservoir is approximately 12.2 square miles, which consists of approximately 40 percent steep rock slopes and 60 percent shallow soils. The diversion structure is situated approximately at 41°07' north latitude and 105°31' west longitude. The stream distance between the farthest reach of the watershed and the dam is approximately 3 miles. The watershed varies from El. 7132 feet at the diversion site to about El. 8100 feet at the highest point along the longest stream giving an average slope of 322 feet per mile. The contributing watershed is shown in Figure 1.

Runoff Characteristics
Flood peaks in the basin can occur as a result of runoff from either snowmelt or intense rainfall. Rainfall runoff has been the predominate factor in producing flood discharges. This can be observed in the 36 years of USGS records (Gage #0675500, 1933 to 1969) of the South Crow Creek flows downstream of the diversion structure. The largest recorded peak of 110 cfs occurred in July 1945. The second largest peak of 109 cfs occurred in June 1965.

Flood Frequency Analysis
The USGS stream gage records on South Crow Creek are too short for accurate flood frequency analysis. The available stream flow records on Crow Creek are not much longer. Lack of stream flow data required estimation of flood peaks at the diversion structure and the new dam using rainfall runoff modeling techniques.

Because the diversion structure is classified as a low hazard dam, the required flood routing can be performed for a 100-year flood peak. The new dam will be rated as a high hazard due to size. Therefore, the flood routing for the upstream site should be performed for the Probable Maximum Flood (PMF).

For comparison, the 100-year peak flood runoff was generated using two different methods. The first method used the HEC-1 model of the BOSS Watershed Modeling Software (WMS) to determine the 100-year peak flow, see Appendix H. The second method used the guidelines for analysis of runoff from
small drainage basins in Wyoming to determine the 100-year peak flow [Graig, 1978]. The WMS HEC-1 model required the 100-year rainfall intensity which was determined using the procedures outlined in NOAA Atlas 2, Volume II for Wyoming. The analysis indicated 3.6 inches of rainfall per hour for a 24-hour storm and 100-year return period. This storm would generate a 100-year discharge peak of 3,202 cfs at the diversion structure. Details of the analysis are in Appendix H. For comparison, the small drainage basin method indicated the 100-year discharge peak to be under 1,000 cfs. Comparison of rehabilitation alternatives was performed for the larger peak.

Determination of the PMF at this site was based on determination of the probable maximum precipitation (PMP) as described below.

**Probable Maximum Precipitation**

The 6-hour local and general storms were computed based on the methods outlined in the HMR No. 55A published by the U.S. Department of Commerce and the U.S. Department of Army [NOAA, 1988]. Both the local and general storm were used to determine the largest runoff.

For the local storm the average depth method was used to compute the 6-hour storm. This method was used because of the size of the watershed and the values are more conservative than those for the areal distribution method. An average 1-hour, 1-square mile PMP for the South Crow Creek watershed was estimated to be 10.85 inches. The average 1-hour, 1-square mile depth was 9.60 inches after reduction by an elevation factor of 0.885 percent. An areal reduction factor was used as it was applicable to this size of watershed, to reduce the 6/1-hour ratios. The National Weather Service (NWS) precipitation sequencing was used to sequenced the hourly PMP increments. The temporal sequence of the 15-min values for the largest 1-hour value was sequence as outlined in HMR-55A.

The general storm PMP on the 12.2 square mile watershed was determined using the indexes for the 1, 6, 24, and 72 hour storms. An area reduction factor was applied and the depth duration curve plotted.

The PMP calculations for both storms are presented in Appendix H.

**Runoff Potential**

The runoff potential for the watershed was computed using the Soil Conservation Service (SCS) Curve Number Method [SCS, 1972]. The curve number was estimated based on an evaluation of hydrologic soil groups and antecedent moisture conditions provided by Mr. Staples the soil conservationist for Natural Resource Conservation Service (NRCS) in Cheyenne, Wyoming.

Using data provided, it was determined that the watershed consists of rocky bare ground, lightly wooded, natural desert landscape, good pasture and fair grass lands. The watershed was estimated to consist of 40 percent rock and 60 percent shallow to very shallow, stony soils. Based on the above information, the principal hydrologic soil group for the watershed was selected as "D".

The curve number depends on the antecedent moisture condition (AMC) of the watershed. There are three classes of AMC's: dry, average, and wet (AMC I, II, and III). Conditions for annual flood estimates are usually AMC-II. AMC-III is usually used for PMF estimates depending on the type and duration of
the storm. Usually a period of light precipitation occurs prior to a PMP storm event. The light precipitation will cause the watershed to become saturated prior to the onset of the PMP storm event. The AMC-III value for the watershed was estimated to be 86.2. The detailed development of the curve numbers is presented in Appendix H.

Probable Maximum Flood
The WMS HEC-1 component of the watershed model was used to develop the rainfall-runoff relationship for the South Crow Creek watershed. The SCS dimensionless unit hydrograph was used to describe the morphologic relationships for the watershed. The SCS curve number was used to estimate the amount of runoff as previously described. It was assumed that the outlet works would be closed because current practice is to assume no credit for water releases through any low level outlet works when gated. It was also assumed that the New Dam spillway would be unobstructed and allowed to pass the design discharge during the analysis. At the diversion structure is was assumed the diversion structure would be overtopped and the flood would pass unobstructed.

The peak discharge computed for the PMF for the local storm was 14,660 cfs and for the general storm was 29,290 cfs. The larger general storm PMF was routed through the new reservoir and the dam spillway was sized to pass the peak discharge. The same storm was routed over the existing and proposed new diversion structures and they were both overtopped by 5.0 feet. The flood routing results are presented in Appendix H.
EASEMENTS, ENVIRONMENTAL ISSUES, AND PERMITS

EASEMENTS

Construction access easements will be required for construction of the diversion structure rehabilitation. The last 2.5 miles of access road to the site from the Crystal Lake Road, which is off Happy Jack Road, (Figure 1) will require upgrading. Access to the south side of the reservoir and creek will need to be developed. For the most part, the only landowner to be impacted would be Mr. Ferguson. The remaining impacted areas are owned by the State of Wyoming or the City of Cheyenne (Figure A-1 in Appendix G).

Along with the construction access easement, the BOPU should negotiate and obtain a permanent maintenance easement for the same route. The unimproved road is presently being used for access to the site.

The BOPU will be required to obtain borrow site and excavation access and new pipeline route easements before the new “Lake Helen” dam proceeds to construction. Depending on the extent of the reservoir footprint, the BOPU may need to acquire additional land for the inundation. The actual area to be inundated will depend on the amount of water diverted from the Stage I pipeline. Options are available to trade BOPU land for lands that will be inundated.

ENVIRONMENTAL ISSUES

A study of potential environmental impacts will be required prior to any rehabilitation construction. Due to the small size of the existing site, coupled with the small enlargement potential, the environmental assessment would not be too involved. The assessment should be based on presence or absence of endangered species and critical habitats.

A more complete environmental study would be required for the new dam, including assessment of historic and archaeological resources. This project would require a complete Environmental Assessment (EA), or perhaps even an Environmental Impact Statement (EIS) be performed.

Construction plans for both the diversion structure rehabilitation and the new dam should be developed to protect endangered and threatened species of wildlife and plants and their habitats, in accordance with the Endangered Species Act, if any impacts are found. Construction of a new dam should also take place in a manner which will ensure preservation of all objects of historical architectural or archaeological significance found in the area.

PERMITS

Finalization of the project will also require the appropriate permits. During the Level III design process and prior to any construction, several permits and clearances would need to be obtained. A summary of the most significant permits required follows:
U.S. Army Corps of Engineers (COE) would need to be informed regarding any disturbance of wetlands. This would likely apply to any stream crossings and would definitely apply to any work on the new South Crow dam. It is expected that a COE 404 Permit would be required for the new dam and a Nationwide Permit for the rehabilitation. Conceptual drawings should be submitted to Mr. Johnson at the COE regional office in Cheyenne with a request for a review of the planned construction and determination of permits that would be required.

Plans and specifications detailing construction of the rehabilitation or the new dam repairs should be submitted to the Wyoming State Engineer's Office (SEO) for review and comment. All changes in surface and water rights storage volume should be adjudicated before the State Board of Control (see Appendix I). These approvals should be obtained during the Level III design work. By today's legal standards for diversion structures, a reservoir permit would be required. To formally recognize this reservoir, the SEO will require two petitions and a map to enter the project into the water rights records. The petitions include the following:

- A petition to the State Engineer should request the acceptance of an amended map and that an endorsement be added to Permit No. 10492 to reflect the storage capacity behind the diversion dam.
- A petition to the Board of Control requesting an amended certificate be issued to recognize the storage behind the diversion dam.

If the selected rehabilitation option increases the capacity of the reservoir above the 8.87 acft, then a new reservoir application should be submitted to permit the additional storage capacity.

Where applicable, permission should be negotiated for right-of-access for all construction activities associated with South Crow. Where the access roads are under the jurisdiction of the City of Cheyenne or State of Wyoming, permitting by letter should be sufficient.

Prior to construction, the Wyoming Department of Environmental Quality/Water Quality Division (DEQ/WQD) will require a Permit to Construct for all transmission and storage facilities. The DEQ will also require a Storm Water discharge Permit (NPDES) and a plan showing best construction practices.

The Wyoming Game and Fish Department should be contacted to inform them of the project and to determine if they should be contacted for clearance prior to any construction.

Laramie County should be contacted during the Level III planning to obtain approval to construction activity within the county road easement.

Formal approval by the State Historic Preservation Office (SHPO) also will be required for the new dam. As soon as practical, the proposed construction plans should be submitted to the SHPO to allow them to review the proposed limits of construction within the known areas of historic resources.

The new dam would also require a current day priority right to store additional water and a filing for adjudication for new storage (see Appendix I).
GEOTECHNICAL EVALUATION

BACKGROUND

No geotechnical reports or drill logs for the existing structure were found. Geologic investigations for this study were completed with the aid of topographic and geologic maps prepared by the State of Wyoming [Love, 1985]. No drilling or laboratory testing of materials was completed for the diversion structure. Lab tests were completed on soil samples taken along the pipeline to determine “hot” soils (areas conducive to corrosion). Results of these tests are documented in Appendix B. Geologic field observations for the South Crow Diversion Structure site and the New Dam site were made in July 1999. Results of these observations are presented in Appendix A and Appendix E.

PROJECT AREA GEOLOGY

The geology in the project area is shown in Figure A-3 contained in Appendix A. The area is underlain by Precambrian metasediments consisting of schist and granite gneiss. The Precambrian rock is in contact with the Tertiary White River formation just north of the axis of the diversion dam. The White River Formation consists of claystones and arkosic conglomerates. Alluvium and slope wash consisting of sand and gravel, colluvium, alluvial fan and eolian deposits of Pleistocene or Recent Age, overlay the bedrock in large portions of the project area.

Existing Diversion Structure Site Geology

The diversion structure is located in the widened part of a narrow valley along South Crow Creek. The valley has moderate to steep side slopes to the west and gently sloping soil-covered slopes on the east side of the diversion dam. Further to the west and south the canyon walls steepen. Outcrops were noted on both sides of the valley upstream from the diversion structure. The rock in this area consists of Precambrian metasediments. The bottom of the creek is heavily vegetated with grasses, brush and trees.

The west abutment of the structure is keyed into a rock outcrop consisting of granitic rock inter bedded with highly weathered schist. The east abutment is in a small alluvial fan on a gently eroded soil slope consisting of slope wash, several feet in thickness. Rock outcrops of the same Precambrian metasediments encountered on the west abutment were noted further up the slope of the east abutment.

The bedrock in the vicinity of the existing diversion dam is highly jointed; three major joint sets occur. One major joint set has a strike of approximately N23 W to N27 W and dips of 76 to 86 degrees to the east and west, respectively. The second major joint set measured varied from N32 E to N36 E with dips ranging from 55 degrees to the east to 86 degrees to the west. The third joint set measured was N65 to 70 east to near vertical.

Pipeline Geology

The pipeline is located almost entirely in the White River Formation. This formation consists of alternating claystone, sandstones and conglomerates.
Several test pits were dug along the pipeline. These pits indicate that the top of the pipe is 6 to 8 feet below the surface and surrounded by soils made up of sandstones and conglomerates.

**FOUNDATION CONDITIONS**

The project is located within the Denver basin which consists of nearly horizontal strata of late Tertiary rocks. The bedrock underlies portions of the site at relatively shallow depth and is overlain by alluvial sands and gravel. Other portions of the site are exposed rock.

As noted above, the west abutment of the diversion structure is keyed into a rock outcrop consisting of granitic rock interbedded with highly weathered schist. The east abutment is in a small alluvial fan on a gently eroded soil slope consisting of slope wash. The soil cover is estimated to be 5 to 10 feet thick. The existing structure extends downward to bedrock. Enlargement of the existing diversion dam will encounter minimal geologic hazards. Extension of the east abutment will require removal of over burden. An enlarged west abutment will be keyed into the existing rock outcropping.

Foundation grouting may be necessary to minimize seepage through the bedrock jointing and highly weathered schist. Present seepage through the foundation is very minimal and not a concern.

**FUTURE EXPLORATION**

If the decision is made to proceed with a New Dam, geotechnical investigations will be required to establish soil and rock characteristics and parameters for final design.

During Level III design of the rehabilitation for the diversion structure, a minimal amount of geotechnical investigations will be required to confirm the level and nature of the foundation rock.

These investigations will consist of hand auger and backhoe excavation work.
REHABILITATION ALTERNATIVES

INTRODUCTION

Rehabilitation alternatives identified in the evaluation of the existing facilities consisted of those required to stabilize and protect the structure from future deterioration plus increase the storage capacity of the reservoir. Rehabilitation measures recommended for the valve house and pipeline did not require an evaluation of alternatives.

DIVERSION STRUCTURE REHABILITATION ALTERNATIVES

Evaluation of existing and collected data and discussions with project sponsors identified the following potential rehabilitation alternatives for the South Crow diversion structure:

- Replacement of existing structure.
- Seal upstream surface of diversion structure with:
  - Special sealants applied to surface
  - Geotextile materials fixed to surface
  - Concrete grouts placed as paste on the surface
- Overlay upstream face of existing structure with concrete.
- Overlay downstream face of existing structure with:
  - Concrete
  - RCC
  - Soil-cement

The above set of alternatives were selected because they covered the range of rehabilitation methods available in price and in approach. The selection provides removal, repairing the inside, and covering the structure alternatives. The costs are expected to increase from the lowest, being the encapsulation alternative, to the highest, being the replacement alternative.

Alternatives not considered were cement or epoxy grouting of the diversion structure body. These types of repair alternatives were eliminated from further consideration due to material costs, difficulty of access for construction, and insufficient structure stability of the existing structure, which would not be improved with this approach.

The downstream overlay alternatives have an advantage, compared to an upstream overlay or surface treatments, because concrete work can be performed on the downstream side of the diversion structure in the dry without draining the reservoir.

Each alternative is described in more detail in Appendix A.
ALTERNATIVE ANALYSIS

The analyses required to compare and evaluate each of the alternatives included:

- Determining the amount of storage volume lost due to sediment to establish the height the crest of the dam should be raised to reclaim lost storage.

- Determining the Probable Maximum Flood (PMF) and 100 year flood volumes for the flood routing analyses.

- Flood routing to determine the overtopping depth for the stability analysis.

- Stability analysis of the existing structure, improved structure, and raised structures to determine the amount of concrete required for the downstream and upstream overlay alternatives.

- Developing cost estimates.

For documentation of the storage lost to sediment and PMF and 100-year flood routings, see Appendix H. The analysis of water rights, pipeline characteristics, and control requirements are also documented in the Appendixes.

Lost storage due to sedimentation was determined by comparing a field survey to topographic data. The amount of storage lost was estimated to be 3 acft. Thus a raise of 3 feet would be required for the crest and spillway of the structure to restore storage lost due to sediment.

Details and results of the stability analyses are contained in Appendix D. The results of the stability analysis of the downstream and upstream concrete overlays show both alternatives satisfy the stability criteria and should be carried forward in the comparison process. See Table 5 below. The stability analysis established the upstream overlay thickness at about four feet and the downstream overlay thickness at about 3.5 feet.

<table>
<thead>
<tr>
<th>Option One - Downstream Concrete Overlay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sliding Stability</td>
</tr>
<tr>
<td>Load Case</td>
</tr>
<tr>
<td>Usual</td>
</tr>
<tr>
<td>Unusual</td>
</tr>
<tr>
<td>Extreme</td>
</tr>
</tbody>
</table>
Table 5: Results of Overlay Alternative Stability Analysis

<table>
<thead>
<tr>
<th>Option Two - Upstream Concrete Overlay</th>
<th>Sliding Stability</th>
<th>Flotation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Usual</td>
<td>2.03</td>
<td>2.00</td>
</tr>
<tr>
<td>Unusual</td>
<td>1.86</td>
<td>1.25</td>
</tr>
<tr>
<td>Extreme</td>
<td>1.89</td>
<td>1.00</td>
</tr>
</tbody>
</table>

**COST COMPARISON**

The replacement alternative was not considered in the cost comparison as it was considered too expensive. The upstream concrete overlay alternative requires more concrete than a downstream concrete overlay thereby eliminating this option from further consideration.

The selected upstream sealant alternative was estimated to cost about $104,000. This cost does not include a 2-foot thick downstream overlay to improve stability and additional concrete to raise the dam crest. Since the cost of a downstream overlay alternative was less than the upstream sealant alternative, this alternative was not considered further. In a similar phase, the upstream concrete overlay was not considered further.

The initial alternative evaluation process eliminated all alternatives from further consideration except for the three downstream overlay alternatives. Selection of the most economical downstream alternative was based on the following costs:

- Concrete overlay costing approximately $80,000.
- RCC overlay costing approximately $132,000 or more depending on the cost of mobilization.
- Soil-cement overlay costing approximately $83,000 or more depending on the soil type and availability of material.

**NEW DAM ALTERNATIVE**

A New Dam alternative exists that is similar to the replacement alternative mentioned above. A potential site for the New Dam alternative is the proposed Lake Helen site. The City did have a storage right which they abandon as shown in the water right exhibits in Appendix I. The geology of the site would provide a good dam foundation and a narrow canyon for dam construction, which are factors that would help reduce the cost of a new dam.

The hydrologic analysis performed with the available 36 years of flow records and the existing water rights on only South Crow Creek indicates that all of the available water is allocated. Therefore, there is no benefit for a new dam unless it can be established by storing imported new water.
The analysis of dam type and cost contained in Appendix G indicate a new 30-foot-high dam would cost approximately $1,000,000. The cost curve was established as a function of dam height and a RCC dam type.

The cost of a new dam was developed: (1) to facilitate a comparison to the selected rehabilitation alternatives, and (2) help the BOPU decide on the value of new storage. The cost comparison of alternatives follows the development of conceptual designs.
PHASE I CONCLUSIONS AND RECOMMENDATION

CONCLUSIONS

From the results of the Phase I work it was concluded:

• The downstream concrete overlay alternative would provide the best protection of the existing structure, create a more stable structure satisfying State Engineer criteria, and have the lowest cost.

• The pipeline requires addition of air release and blowoff construction valves.

• The valve house requires a major overhaul of the structure, valves and addition of new controls.

• The New Dam upstream is uneconomical and would require imported water via a basin transfer to function as an effective storage facility.

REHABILITATION ALTERNATIVE RECOMMENDATION

The downstream concrete overlay with a 3-foot-high crest raise is the most cost-effective alternative to carry forward into Phase II.
CONCEPTUAL DESIGN

INTRODUCTION

Conceptual design efforts were focused on the recommended and approved Phase I alternative for the South Crow diversion rehabilitation. Approval to proceed with Phase II came during the presentation and discussion of Phase I rehabilitation alternatives in the meeting with WWDC and BOPU personnel on the January 26, 2000. The consensus reached in that meeting was to accept the Phase I recommendation and proceed with the conceptual design of the downstream concrete overlay alternative. Conceptual design work started in February and was completed in May of this year.

Components of the approved alternative determined with the Phase I studies include the following items:

- Rehabilitation of the pipeline with the addition of air release valves and blowoff valves.
- Rehabilitation of the diversion structure with a downstream concrete overlay and crest raise of 3 feet.
- Rehabilitation of the valve house and the addition of new valves, operators, screens, and controls.
- Addition of a SCADA system to control the opening and closing of valves, measuring of reservoir water levels and quality, and other monitoring.

This section of the report presents the development of each component’s design and future function.

PIPELINE

The assessment concluded the pipeline required air release and blowoff valves to facilitate the pipeline’s operation. Locating the air release valves was accomplished with the use of the surveyed pipeline and hydraulic analysis. Typically, an air release valve is located at the high points in the pipeline and near the start of the pipeline.

For the South Crow pipeline alignment it was determined that three air release valves should be located along the route. The first was proposed to be inserted into the pipeline just downstream of the valve house, the next two were located at the high points in the pipeline.

Two blowoff valves were also considered. One was to replace the excavated blowoff valve tee where the corrosion was found. The other was located between the high points where the new air release valves were located.

Locations of all five new valves are shown on Figure 5.
NOTE
SURVEY DATA AND DESIGN
INFORMATION PROVIDED AT 1:4

WATER DEVELOPMENT COMMISSION
CHEYENNE SOUTH CROW DIVERSION PROJECT
SOUTH CROW PIPELINE
PROFILE WITH
PROPOSED IMPROVEMENTS

FEASIBILITY STUDY 

FIGURE 5
The blowoff installation envisioned for costing purposes consists of a tangential outlet (tee) from the pipeline with an isolation valve, a throttling valve, and discharge piping. The discharge piping can be slit to allow the blowoff piping to drain and not freeze during winter. The BOPU has used this method successfully on the blowoff piping recently installed with the isolation valve near the 20-inch Hecla pipeline. Installation is accomplished by cutting the existing pipe then installing a new steel or ductile iron tee between two new adapting couplers.

The air release valve can be combined with a vacuum valve and be installed as an air and vacuum release valve. A combination valve releases the entrapped air but also allow air in to protect the pipeline from low pressures due to water column separation. Cost estimates were based on an air release valve being installed and not the combination valve and no redundancy in the system. The air release valve installation consists of a tee, an isolation valve, piping, and an air release valve inside a vault.

If the pipeline develops leaks in the future, sections of the pipeline should be replaced.

**DIVERSION STRUCTURE**

Alternative analysis identified the downstream concrete overlay with a 3-foot-high crest raise as the most cost-effective alternative. The 3-foot crest raise was set by freeboard requirements and the determination of the new spillway crest elevation. Stability analysis was used to establish the configuration of the typical section. As documented in Appendix A this analysis was started prior to the commencement of the conceptual design.

Sufficient detail was develop in Phase II to complete the conceptual design and cost estimate presented herein this section and the next. Details were developed on the following items:

- Drainage filter placed between the new and existing concrete.
- Drainage collection system installed to capture leakage through the existing diversion structure.
- Dowels used to bond the new to the existing concrete.
- Upstream existing structure's concrete face repairs to prevent further disintegration.
- Improvements to the existing rancher's irrigation diversion from the diversion structure.
- Sluice gate rehabilitation to improve sediment flushing.
- Installation of riprap for overtopping protection.
- Valve house repair to upgrade the system's operation.

The plan and general layout of the downstream overlay rehabilitation project envisioned is shown on Figure 6. Sections of the overlay established with the stability analysis results and the new structure’s general layout are shown on Figure 7.
PLAN
PROPOSED CONCRETE OVERLAY OVER EXISTING DAM
SCALE A

NORTH ELEVATION
PROPOSED CONCRETE OVERLAY OVER EXISTING DAM
SCALE A

NOTES:
1) EXISTING DAM AND HOUSE DIMENSIONS ESTABLISHED BY
FIELD SURVEY CONDUCTED IN JUNE, 1999 BY A/E.

FEASIBILITY STUDY

WATER DEVELOPMENT COMMISSION
CHEYENNE SOUTH CROW DIVERSION PROJECT

DIVERSION STRUCTURE
PROPOSED REHABILITATION
PLAN AND PROFILE

FIGURE 6
FEASIBILITY STUDY

WATER DEVELOPMENT COMMISSION

GREAT SOUTHERN DIVERSION PROJECT

DIVERSION STRUCTURE
PROPOSED REHABILITATION SECTIONS

FIGURE 7

NOTES

1) EXISTING DAM AND HOUSE DIMENSIONS ESTABLISHED BY FIELD SURVEY CONDUCTED IN JUNE, BY AAI.

2) CLEAN SURFACE AND REMOVE DETERIORATED CONCRETE THEN SEAL CRACKS WITH DR 2000 JOINT SEALANT OR EQUIVALENT AND SEAL SURFACE WITH CONCRETE SEALER OR 100% BY EQUIVALENT.

3) EXISTING DOWNSTREAM SURFACE SANDBLASTED AND PREPPED FOR NEW CONCRETE OVERLAY.
Concrete Gravity Section
The approach envisioned was to use a downstream concrete overlay, founded on bedrock, to improve and stabilize the existing concrete gravity section, see Figure 6. Stability analysis performed on this section, concluded the new mass of concrete should extend from the edge of the existing structure approximately 3.5 feet at the top to the downstream side. The shape and a new slope of $0.725H:1.0V$ used to define the new gravity section following concrete gravity dam design criteria as required by Wyoming’s SEO Dam Safety Engineer. This basic configuration and slope was maintained for all structure heights investigated. A minimum thickness of an overlay required to protect the new and the old from freeze-thaw effects is 2 feet. Stability calculations determined the thickness of the overlay to be thicker than this minimum requirement and thus govern the final shape envisioned for the section.

Above the junction of the sloping and vertical wall at elevation 7124, the concrete section becomes a rectangular section extending to elevation 7135. Where bedrock is encountered at or above elevation 7124 the concrete section does not have a sloping portion, see Figure 6.

Spillway
The spillway section will be similar to the non-spillway section. The relationship between potential storage volume and elevation determined by a survey of the site was used to establish the new dam and spillway crest elevations. It was determined in order to storage the 12 acft assumed to be the original volume captured by the diversion structure the new crest elevation had to be set at 7132.42. Free board requirements placed the top of dam at elevation 7135.

The shape of the new spillway envisioned remained the same which was a crest length of 50 feet and a depth of approximately 3 feet. Flood routing studies indicated the new structure will be overtopped in the same manner as the existing by approximately 2.7 feet during a 100-year flood. Having a overall crest width of about 7.5 feet would allow the new shape to be converted into an ogee weir and be recessed into the section. The ogee concept and recess would be more costly.

The plan is to incorporate the rancher’s irrigation diversion intake into the new spillway section. The proposed trench across the spillway crest would be covered with a grate, see Figure 6. The pipe inlet can be gated.

Sluice Outlet
Rehabilitation for the existing sluice outlet will consist of removing the existing gate while the reservoir is lowered to repair the upstream concrete face. A new silt proof gate would be installed on the backside of a new outlet tower built on the upstream face. The new tower provides access to the gate. The arrangement keeps the gate free from being block by sediment.

Design Consideration
It is recommended that all new concrete will be bonded to the existing structure. This requires the use of dowels with the downstream overlay and clean surfaces and good grouting techniques used on the upstream face. It is important to remove as much of the deteriorated concrete as possible from both surfaces.
A filter and a new drainage system of pipes are planned to capture the water leaking through the existing structure and drain it down and away from the new overlay.

Protection of the new structure and the downstream dissipation area is envisioned to be provided by a layer of riprap. The riprap would be grouted where velocities exceeded 6 feet per second and placed along the toe of the structure to bring the surface back to the existing ground elevation. The combination of riprap and bedrock in the creek bed is expected to continue to dissipate discharges over the spillway. Confirmation of bedrock in the creek will be performed during construction. If rock is fractured or missing a concrete slab will be added to the end of the spillway.

**Valve House**

The rehabilitation of the valve house was performed to add new motorized valves and a SCADA system. The result of the valve size analysis required the most of valve house to be remodeled. The first step in the rehabilitation process envisioned was the removal of the existing structure above elevation 7132 and most of the house backside. The concept required the pipe to be cut so a new piece of pipe with new valves could be installed. The floor was raised to elevation 7135. Motors and electronic equipment was located above the PMF elevation of 7140. Grating was added along with new trash screens and access ladders and safety railing. The door selected was one that is bullet proof. The inside walls were insulated covered with a water proof sheet. A heater was provided to provide for operations, if required, during the colder months. Details of the improvements envisioned are on Figure 8.

**Valves**

The existing intake was modified to install a control valve and a guard valve. The control valve selected was a 16-inch diameter plug valve similar to the eccentric plug valve manufactured by Dezurik. A plug valve was proposed because it is suitable for regulating silty waters due to its rugged construction, and has a linear discharge. The guard valve proposed was a butterfly valve, which is the most economical and eminently suitable for emergency closure. Both the valves were provided with electric motor operators which can be remotely controlled. The operators with electric motors will have extended stems so that they can be located on the raised floor level, above the 100-year flood. Hydraulic cylinder operators were a possible option, and can be studied during detailed design stage.

It is proposed for the SCADA and control that the butterfly valve open-close positions be indicated remotely by green and red indicating lights respectively.

It is noted that the BOPU installed on the South Crow pipeline a 16-inch butterfly valve and a 6-inch blowoff valve just upstream of the pipeline’s junction with the Hecla pipeline. The butterfly valve intends to serve as an isolation valve.

**SCADA**

Identification of the SCADA system to provide monitoring of the water quality and control water releases from South Crow Diversion was performed to complete the conceptual rehabilitation design. The preferred method of communications is via radio interface with the existing system. Alternative system is to utilize telephone if phone lines are available nearby. A separate path study was completed of South Crow to
Table Mountain Repeater, which showed an obstruction at 3 miles out. Therefore, information from KNS Communications and BOPU were used to determine paths and sites with the aid of the RFCAD software and a map data base. KNS work covers the addition of a SCADA system to the rest of the City’s water supply system and the South Crow diversion can be added to that plan.

When built the SCADA system envisioned for the project will include the following three sites:

1. A receiver at the South Crow Dam site consisting of status and control of two valves, water quality and RTU with radio. The site is isolated without power or telephone.

2. A receiver at the South Crow pipeline junction with the 20-inch south pipeline coming from the Hecla diversion site is an inline pipeline valve consisting of status and control of an isolation valve and RTU with radio. The Hecla junction valve site is along the road and near an AC power line.

3. A store and forward repeater unit at the suggested Copper King Mine site.

This system’s coverage envisioned provides for monitoring and control of both valve sites and will be integrated with the Cheyenne BPU’s SCADA System. The RTUs and radios were selected to be compatible or equal to the existing BPU equipment. Controls were integrated with the existing system.

The proposed communications selected was one that will utilize 450 or 900 MHZ radio with the probability of a repeater near Copper King Mine. The repeater could be solar powered, or with the proximity of the proposed power line for South Crow, AC powered. Other options included a phone line or using the newest addition to this field, a satellite connection. Land ownership at the Copper King Mine site will be investigated by BOPU prior to final design work.

It was envisioned that the diversion site will be equipped with the RTU, radio, power distribution and other equipment. The controls were located above the flood plane, see Figure 9.

The telemetry and control (SCADA) system recommended utilized Motorola MOSCAD RTUs and Motorola Darcom 900 MHZ Radios. The system will link with and be compatible with the BOPU SCADA System. A direct radio link to one of BOPU’s repeater or other site was preferred, but the terrain studies suggest a repeater, either solar/battery or AC/battery will be required. A repeater at Copper King mine was recommended for reliable operation. This could also serve the Hecla site.

Main components of the proposed system included:

- Wood pole powerline from pipeline junction with Hecla pipeline to South Crow Diversion Structure.

- Two transformers a 14,400:120/240 15 KVA at the diversion and 14,400:120/240 10 KVA at the Hecla junction, each with lightning protection and distribution panels with a possible third for the repeater.

- Control, position and status circuits for valve operators, water level at diversion, water quality at the diversion and Hecla pipeline.
• Motorola MOSCAD RTU's for the South Crow Diversion and Hecla pipeline junction sites.

• Motorola Darcom telemetry radios and repeater. Radio system to be in compliance with BOPU radio plan and FC requirements. Radio systems at the dam and Hecla junction should be battery operated with AC chargers. Solar power is suggested for the repeater.

ACCESSORY ITEMS

To complete the rehabilitation project a number of accessory items were needed. One was access, another was power, both are highlighted below.

Access Road

A new construction road will be required for the 2.5 miles from the Crystal Lake Road to the site. It is envisioned that this road be converted into an operation and maintenance road, and so be constructed in accordance with Laramie County road standards with a gravel surface.

Transmission Line

It is envisioned that power for the South Crow facility would be provided by a new single phase 14.4 Kilovolt power line (approximately 2.5 miles) following the proposed access road from the Hecla pipe line junction to the South Crow Diversion. Transformers are to be provided at the diversion and the junction with the Hecla pipeline. A possible third transformer might be required for the repeater near Copper King mine. Power requirements include two valve operators, heating and lighting, and radio, control and telemetry. Power requirements for the Hecla junction include one-valve operator, and radio, control and telemetry. All power transformers are to have disconnects and lightning protection.
ESTIMATED CONSTRUCTION COSTS

GENERAL

A detailed cost estimate for construction of the proposed project is included in Table 6. The estimate is presented in two parts; the first for Engineering and Administration and the second for construction of the Project Components - including costs for inspection, testing and engineering support during construction. A contingency is included to cover unidentified and unlisted items since this is a preliminary design.

BASIS OF COSTS

Engineering and Administration Cost

- The estimate of costs for preparation of final designs, drawings and specifications were developed based on our experience with similar projects. The estimate was based on a percentage of the estimated construction cost. Estimated costs for assistance during bidding through award of the construction contract are also included in this section.

- The estimate of costs for Permitting and Mitigation is also based on a percentage of estimated construction cost. The estimates assume no unusual problems with these items.

- Legal fees also were estimated as a percentage of construction cost in anticipation that the City's standard contract terms will be incorporated into the bid documents and contract.

- A significant allowance has been made for the acquisition of access and rights of way. It was anticipated that easements will be obtained from the property owner(s) for access during construction and for access to perform maintenance and operation functions during the life of the project.

Direct Construction Cost

Direct costs for the project components were estimated using unit prices where appropriate. Quantities of work items were calculated from engineering drawings prepared for the Phase II Report. The unit prices were established using the following guidelines:

- Historical prices for similar scopes of work.

- Personal construction experience of our staff.

- Quotations from manufacturers, suppliers and vendors.

- Means Heavy Construction Cost Data Book.
Escalation was applied at 5 percent of the unit prices to establish a baseline cost for the year 2001. The unit prices are for materials in place and include contractor's overhead and profit. No allowance has been made for changed conditions or force majeure events which could impact the construction effort.

An integral part of the cost of construction is the effort required by the engineer for resident engineering, inspection, testing and home office support (submittal reviews, requests for information from the contractor, design clarifications, etc.). Based on past experience with similar projects, a factor of 10 percent was assumed to cover such costs. No allowance was made for owner's costs to administer the construction contract or for the cost of financing the project.

**Contingency**

A construction contingency is typically applied to construction cost estimates to account for unforeseen conditions during construction, miscellaneous items not included in the estimate and potential adjustments to the assumed unit prices. For a preliminary design construction cost estimate, it is normally recommended that a 30 percent contingency factor be added to the direct construction cost.

**Project Total Cost**

The sum of the Direct Construction Cost, including Engineering during construction, contingency, plus the Engineering and Administration Cost provides our opinion of the total project cost.

The opinions of probable construction cost provided herein are made on the basis of ECI's experience and qualifications and represent our best judgement as an experienced and qualified professional engineer generally familiar with the construction industry. However, since we have no control over the cost of labor, materials, equipment or services furnished by others, or over the contractors' methods of determining prices, or over competitive bidding or market conditions, ECI cannot and does not guarantee that proposals, bids or actual construction cost will not vary from the opinions of probable construction cost prepared by us.
## Table 6: Cheyenne South Crow Diversion Project

### Engineers Opinion of Probable Cost

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<tr>
<th>Item</th>
<th>Description</th>
<th>Unit</th>
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<th>Unit Price</th>
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April 2000
ECONOMIC ANALYSIS

INTRODUCTION

An economic analysis was completed to assess the ability of BOPU to pay for the project. Presently the BOPU charges a fee for the water supplied to residents of the City of Cheyenne. The consumer price for treated water in 1999 was $2.02 per 1,000 gallons. During this last year BOPU staff has established the value of raw water to be approximately $1.32 per 1,000 gallons. BOPU supplies treated water to approximately 65,000 residents with 22,916 equivalent 3/4-inch diameter taps.

ABILITY TO PAY

The cost estimate established the total project cost at $1,049,000.00. A repayment plan would be based on a 50 percent grant and 50 percent loan. The term of the loan would be 20 years with an interest ratio of 7.25 percent. These figures generate an annual cost to the City of Cheyenne's BOPU of $60.475, per Table 6 below.

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<th>Table 7: Annual Cost</th>
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<td>Capital Cost of Project</td>
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<td>50% of Capital Cost as a Loan</td>
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<td>20 Year Loan 7.25% Annual Interest Total of Payments</td>
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<tr>
<td>Total of Payments</td>
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<td>Annual Payment</td>
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<td>Annual O&amp;M Cost Increase</td>
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<td>Annual Cost to BOPU</td>
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</table>

Payments of the annual cost would increase the cost of treated water to the users by about $2.66 per tap/per year assuming the cost is distributed over all equivalent users.
CONCLUSIONS AND RECOMMENDATIONS

Based on the work completed in the analysis of the rehabilitation required at the Cheyenne South Crow Diversion, the following conclusions and recommendations are provided:

1. Final design and rehabilitation efforts should proceed within the next two years to prolong the life and stabilize the structure and preserve the City’s diversion water rights.

2. The new dam alternative will only be economical if out-of-basin water can be imported and stored. The watershed annual runoff is already fully allocated and no new water for storage is available.

3. The recommended rehabilitation alternative for the diversion structure is a downstream concrete overlay.

4. The recommended rehabilitation alternative for the pipeline is addition of air release and blowoff valves in identified locations.

5. The recommended rehabilitation for the valve house includes removal of part of the old valve house structure, addition of new valves and motorized operators, and a new SCADA system.

6. An easement for construction access and future operation will be required. It is recommended that the City begin negotiations with Mr. Ferguson as soon as possible to secure this access.
LIST OF REFERENCES

American Concrete Institute (ACI), 1989, Building Code Requirements for Reinforced Concrete, ACI 318-89, Michigan.


Letter to ECI from Mr. Staples at Soil Conservation Service (SCS), 1999. U.S. Department of Agriculture, Cheyenne, Wyoming.


U.S. Bureau of Reclamation (USBR), 1981, Control of Cracking in Mass Concrete Structures, Engineering Monograph No. 34, Revised Reprint, Denver, Colorado.
APPENDIX A

Phase I
Documentation
INTRODUCTION

The first phase of the Cheyenne South Crow Diversion Project was completed in December and the Phase I draft report submitted. The approval to proceed with Phase II and the conceptual design of the recommended concrete downstream overlay alternative was given on January 26 in a meeting with Mr. Bruce Brinkman and BOPU officials.

Work on Phase II conceptual design started in February and was completed in April. Directions were given in the meeting on the 26th that specified the inclusion of the Phase I Report as an appendix to the Phase II Report. Following those directions the Phase I Report with the exception of the first Introduction section is included herein as this Appendix A.

The Introduction section from the Phase I Report is used to introduce the Phase II Report and is not repeated here in Appendix A.
Inventory and Evaluation

INTRODUCTION

The primary task of Phase I was to inventory and evaluate the condition of the diversion structure and pipeline. During the inventory, a number of observations were made, including the realization that the 1910 design drawings show a larger diversion structure than was actually built.

Access for future construction and maintenance of the project was evaluated and found to be lacking. Water rights were inventoried. Water availability and land ownership were determined.

The pipeline was exposed and flow tested. The diversion structure was surveyed and the concrete condition was examined. The foundations for the existing diversion structure and potential upstream dam and the surrounding geological features were mapped.

The results of these examinations and evaluations is presented herein. The information collected was used in the development of the rehabilitation project.

SOUTH CROW DIVERSION WATER RIGHTS

The water rights on South Crow Creek and Crow Creek, from its confluence with South Crow to the City of Cheyenne, are listed in Table A.1 and shown in Figure A-1. These data were provided by the State Engineer’s Office and indicate existing rights as of March 18, 1999. The largest and the first water right is for 12,480 cfs on the main stem of Crow Creek that is held by the City of Cheyenne. The City of Cheyenne holds six other water rights for a total of 11.5 cfs with three of the six being secondary or supplemental rights. Mr. A. Gilchrist is the next largest water right holder listed in the table with a right to divert a total of 62.06 cfs to five ditches to irrigate 890 acres.

The City’s experimental station has one of the existing storage rights for 103.5 acft. The other storage right is held by Mr. S. Shockley for 0.599 acft. Because the City has used the diversion structure to store water, it is assumed the City could obtain a storage right for the diversion structure. An answer to this question has been requested in a letter to the State Engineer.

The 12,480-cfs right held by the City far exceeds the normal water availability in the Crow Creek watershed. If the City would make a call on the system for this amount, all other water users would most likely challenge the call. It has been the City’s policy to divert the water they can while acknowledging the water rights of others. Overall, on just South Crow Creek, the annual average flow can satisfy the water right demand approximately 3 out of 5 years. The dryness of the Crow Creek watershed has promoted the City to develop a more reliable water supply from the western slopes of the Continental Divide. This water is delivered via the Stage I and Stage II pipelines to Crystal Lake and Granite Reservoir for storage until needed.

The City also has an old expired permit at the location of the proposed upstream storage site referred to as Lake Helen. This permit, No. 1557R, South Crow Creek Reservoir, was filed by the City in
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1909. It expired in 1912. The City will be required to request cancellation of this old filing per SEO instructions. The State will also require installation of measuring devices near the outlets. This will permit a more accurate accounting of the South Crow Creek waters to senior downstream water users.

LOCAL LAND OWNERSHIP

A search of the County Real Estate Office records revealed the following about the diversion structure and pipeline:

- Book 177, Page 187; The City of Cheyenne purchased a small piece of land in Section 1 (T13N, R70w) where the diversion structure is located. The land was purchased in 1912 from a party named Roberts.

- Book 157, Page 361; Book 159, Page 456; Book 166, Pages 121 and 270: The City of Cheyenne obtained a sixty-foot wide easement for the South Crow Pipeline. The easement grants the City the right to enter into and upon the land for the purpose of constructing, operating, repairing, maintaining and inspecting said pipeline.

Further review of the County Records revealed the surrounding land ownership to be as follows:

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<th>ID #</th>
<th>NAME/ADDRESS</th>
<th>LEGAL DESCRIPTION</th>
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<tr>
<td>013</td>
<td>Ferguson Ranch Inc. 650 Co. Rd. 210 Cheyenne, WY 82007</td>
<td>Sec 30, T 14 N. R 69 W</td>
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<td>Ferguson Ranch Inc. 650 Co. Rd. 210 Cheyenne, WY 82007</td>
<td>Sec 25, T 14 N. R 70 W</td>
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<td>016</td>
<td>State of Wyoming State Lands Herschler Building Cheyenne, WY 82002</td>
<td>Sec 36, T 14 N, R 70 W</td>
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<td>001</td>
<td>John Eldon Sutherland General Delivery Granite Canyon, WY 82059</td>
<td>Sec 1, T 13 N, R 70 W</td>
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<td>002</td>
<td>City of Cheyenne 2101 O'Neil Avenue Cheyenne, WY 82001</td>
<td>N 1/2 Sec 2, T 13 N, R 70 W and NW1/4 Sec 4, T 13 N, R 70 W</td>
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<td>003</td>
<td>Willadsen Brothers partnership Granite Canyon, WY 82059</td>
<td>SI/2 Sec 2, Sec 3, T 13 N, R 70 W and NW 1/2 Sec 34, T 14 N, R 70 W (Shown on Figure A-2 with Davis)</td>
</tr>
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<td>094 &amp; 095</td>
<td>Carl F. Peterson 250 Eagle Drive Dillon, MT 59725</td>
<td>Portion of Sec 35, T 14 N, R 70 W (Shown on Figure A-2 as the Red Baldy Ranch)</td>
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<td>087</td>
<td>Marc Randal Strahn P.O. Box 3020 Cheyenne, WY 82003-3020</td>
<td>Portion of Sec 35, T 14 N, R 70 W (Shown on Figure A-2 as the Red Baldy Ranch)</td>
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<td>010</td>
<td>Robert J. Rockwell 630 S. 192 Circle Omaha, NE 68154</td>
<td>SW 1/4 Sec 34, T 14 N, R 70 W (Shown on Figure A-2 as Davis)</td>
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<td>040</td>
<td>Anna E. Johnson 5940 Bighorn Crossing Ft. Collins, CO 80526</td>
<td>E 1/2 Sec 34, T 14 N, R 70 W (Shown on Figure A-2 as Davis)</td>
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</tbody>
</table>
Current access to reservoir is via a 2-track access road on private lands, which is not covered by an easement or agreement. The locations of these land holdings and access road are shown on Figure A-2.

The records show that the City owns the land where the diversion structure and reservoir are located. A temporary construction access easement would need to be obtained by the City from the Ferguson Ranch and State of Wyoming before Level III funding or compensation could be secured.

**PROJECT AREA GEOLOGY**

The geology in the project area is shown in Figure A-3. The majority of the area is underlain by Precambrian metasediments consisting of schist and granite gneiss. The Precambrian rock is in contact with the Tertiary White River formation just north of the axis of the diversion dam. The White River Formation consists of claystones and arkosic conglomerates. Alluvium and slope wash consisting of sand and gravel, colluvium, alluvial fan and eolian deposits of Pleistocene or Recent Age, overlay the bedrock in large portions of the project area.

The specific geological conditions for each of the project components are described in the following paragraphs:

**Existing Diversion Site Geology**

The diversion structure is located in the widened part of a narrow valley along South Crow Creek. The valley has moderate to steep sides to the west and gently sloping soil-covered slopes on the east side of the diversion dam. Further to the west and south the canyon walls steepen. Outcrops were noted on both sides of the valley upstream from the diversion structure. The rock in this area consists of Precambrian metasediments. The bottom of the creek is heavily vegetated with grasses, brush and trees.

The west abutment of the structure is keyed into a rock outcrop consisting of granitic rock inter bedded with highly weathered schist. The east abutment appears to be in a small alluvial fan or on a gently eroded soil slope consisting of slope wash, several feet in thickness. Rock outcrops of the same Precambrian metasediments encountered on the west abutment were noted further up the slope of the east abutment.

The bedrock in the vicinity of the existing diversion dam is highly jointed; three major joint sets occur. One major joint set has a strike of approximately N23 W to N27 W and dips of 76 to 86 to the east and west. The second major joint set measured varied from N32 E to N36 E with dips ranging from 55 to the east to 86 to the west. The third joint set measured was N65 to 70 east to near vertical.

**Pipeline Geology**

As indicated by the geology shown in Figure A-3, the pipeline is located almost entirely in the White River Formation. This formation consists of alternating claystone, sandstones and conglomerates.
PLAN
SOUTH CROW DIVERSION STRUCTURE AND PIPELINE
(NOT TO SCALE)

FEASIBILITY STUDY

WATER DEVELOPMENT COMMISSION
CHEYENNE, SOUTH CROW DIVERSION PROJECT
SOUTH CROW PIPELINE
LAND OWNERSHIP MAP

Figure A-2
LEGEND

Qal  RECENT ALLUVIUM. Unconsolidated sand and gravel deposits.

Twrc TERTIARY WHITE RIVER FORMATION, CHANDRON MEMBER. Claystones, tuffaceous sandstones and arkosic conglomerates.

Xsv PRECAMBRIAN METASEDIMENTS. Shists and granite gneisses.

Ys PRECAMBRIAN SHERMAN GRANITE.

MAP REFERENCES

TOPOGRAPHY: USGS 1:24,000 Scale Quadrangle for Hecla, Wyoming.

GEOLOGY: Geologic map of Wyoming by J.D. Love and A.C. Christiansen, 1985

FEASIBILITY STUDY

SOUTH CROW DIVERSION
PROJECT AREA GEOLOGY

Figure A-3
Several test pits were dug along the pipeline at the locations indicated in Figure A-4. These pits indicate that the pipe is no deeper than 6 to 8 feet and surrounded by sandstones and conglomerates.

**FOUNDATION CONDITIONS**

During the first site visit, the decision was made to use field observations and a geologic survey to determine foundation conditions for both dam sites at this phase of investigation. Field observations were initialized and completed in July. Results of the geologic survey are contained in Appendix D.

The project is located within the Denver basin which consists of nearly horizontal strata of late Tertiary rocks. The bedrock underlies portions of the site at relatively shallow depth and is overlain by alluvial sands and gravel. Other portions of the site are of exposed rock.

As noted above, the west abutment of the diversion structure is keyed into a rock outcrop consisting of granitic rock. The east abutment is in a small alluvial fan on a gently eroded soil slope consisting of slope wash. The soil cover is estimated to be 5 to 10 feet thick. Enlargement of the existing diversion dam will encounter minimal geologic hazards. The existing structure extends to bedrock. An extension of the east abutment will require removal of over burden. An enlarged west abutment will be keyed in to the existing rock outcropping.

Foundation grouting may be necessary to minimize seepage due to the bedrock jointing and highly weathered schist. Present seepage through the foundation, which is very minimal, is not a concern.

**PIPELINE**

The inventory and evaluation of the existing pipeline between the diversion structure the Hecla pipeline consisted of the following:

- Visual inspection of the exposed pipeline (six of the eight locations where the pipeline was exposed were inspection pits excavated by BOPU staff. The pits averaged 6 to 10 feet deep and 6 to 20 feet long)
- Soil Corrosivity soil resistivity data collection and analysis over the entire length of pipeline (soundings were taken approximately every 500 feet)
- Soil samples were collected for corrosivity – resistivity analysis
- Geologic mapping of pipeline
- Ultrasonic thickness determination
- Flow capacity tests
FEASIBILITY STUDY

NOTE
SHEET SIZE AND DRAWING INFORMATION PRINTED ON ALL SHEETS

1"=100' AT FULL SIZE

WATER DEVELOPMENT COMMISSION
CHEYENNE SOUTH CROW DIVERSION PROJECT
SOUTH CROW PIPELINE PROFILE

Figure A-5
PLAINT - PIPELINE CENTERLINE ALIGNMENT

PLAN - PIPELINE CENTERLINE ALIGNMENT SCALE A

EXISTING 14" DIAMETER PIPE

EXISTING 16" DIAMETER PIPE

EXISTING GROUND

PIPELINE PROFILE
SCALE A HVR : SCALE B VER.

SCALE A
200' AT FULL SIZE

SCALE B
1"=200' AT FULL SIZE

NOTE
DRAFT DATED AND DRAWING INFORMATION REVISED BY DE

FEASIBILITY STUDY

WATER DEVELOPMENT COMMISSION
CHEYENNE SOUTH CROW DIVERSION PROJECT
SOUTH CROW PIPELINE
PLAN AND PROFILE
STA 46+00 TO 92+00

Figure A-7
Visual inspections, soil corrosivity – resistivity data collection and ultrasonic thickness determination were conducted by a corrosion specialist. Flow tests were performed by BOPU staff. Geologic mapping was included with site and structure mapping. The pipeline alignment survey was performed when the diversion structure and valley cross sections were surveyed.

Corrosion investigation details and data analysis results are contained in Appendix B.

Based on the data collected and observation made, it was determined that the pipeline:

- Consists of a nominal 16-inch diameter cast iron pipe from the reservoir to the highest topographic point on a ridge at STA 56+92 (Figure A-7) and a nominal 14-inch cast iron pipe from the high point to the end of the alignment.
- Is constructed of cast iron pipe with lead sealed joints that are not electrically bonded.
- Has a very thin (< 10 mils) external coating.
- Has an exterior surface with little or no corrosion damage in the eight sites exposed.
- Has no corrosion activity at the spigot end of any of the pipe joints.
- Has minimal internal corrosion damage.

**PIPELINE HYDRAULICS**

The high point at STA 56 + 92 controls the capacity of the pipeline. If the hydraulic grade line (HGL) drops below the pipeline elevation at this point, flow is reduced. Because the pipeline is in good condition, any effort to relocate the pipeline to avoid the high point is not recommended. The use of air-release valves would likely improve pipeline operation and flow capacity.

Flow measurement tests on the South Crow pipeline were conducted in October 1999 with the support of BOPU staff. The test consisted of measuring the pressure and discharge at the pipeline’s junction with the Hecla pipeline. From test data, effective friction factors were back-calculated, allowing an estimate to be made of the maximum capacity of the pipeline. Inside pipe diameters for the various lengths of pipe; location and types of bends and tees; and elevations and slopes were required for this analysis. This necessary information is shown on Figures A-6, A-7 and A-8. Flow test results did not cover the full range of flows for the pipeline, but they did provide data that allowed estimation of internal roughness and insight into the hydraulic characteristics. Although the the flow tests could not be readily stabilized, the maximum measured discharge was 1800 gpm (3 cfs) for a pressure of 80 psi.

Assuming the South Crow pipeline discharges to atmosphere at the connection to the Hecla pipeline, and based on the internal friction factors derived from the flow tests, hydraulic calculations estimate the pipeline capacity to be approximately 7 cfs. This matches with the diversion structure’s water rights of 7cfs per permit No. P10192D. However, during normal operations the HGL in the South Crow pipeline will also be constrained and defined by the water pressure in the Hecla pipeline. The pressure and flows at the Hecla/South Crow pipeline junction is limited by the elevation of the diversion structure in Middle Crow Creek which is the beginning of the Hecla pipeline.
The flow tests and hydraulic calculations established the following characteristics for the pipeline:

- Discharge capacity of 7 cfs (5,400 gpm)
- Estimated friction factor C, is of 103 at the design capacity
- Good condition
- Suitable for continuous service without any other consideration for corrosion control
- Addition of air-release valves would improve function.

The Phase II evaluation of the pipeline will include hydraulic analysis of the South Crow and Hecla Pipeline system working together and in isolation. It will determinate the maximum capacity of the South Crow pipeline without impacts to operation of the Hecla pipeline.

**DIVERSION STRUCTURE**

South Crow Creek diversion dam is located in the NW 1/4 of Section 1, T13N, R70W, as shown in Figure A-9. The City of Cheyenne completed construction of the diversion structure in 1911 to impound and divert the flows of South Crow Creek for municipal use.

Existing drawings show the South Crow diversion dam as a concrete structure 40 feet high, with a crest length of about 207 feet and impounding about 12-acft of water. Field surveys during Phase I established actual dimensions. Comparison of the 1910 drawings with the actual dimensions revealed a smaller structure was built than designed. The survey established the crest length at 157 feet and the maximum height at 22 feet, as shown on Figure A-10. The smaller structure was the result of moving the diversion dam upstream from the original location to place the west abutment on an existing rock outcrop.

Differences between the 1910 design and the “as-built” structure can also be seen in the spillway section. The original design included a spillway dissipation basin, while the as-built survey showed no dissipation basin and a higher elevation for the sluice pipe.

The existing diversion dam is a concrete gravity dam constructed with a vertical upstream face and a downstream slope of 0.7H to 1.0V. The crest width of the non-overflow section is approximately 3 feet with the downstream face vertical for approximately 3 feet. The exact height of the structure is difficult to determine due to severe erosion of original concrete along the downstream face.

The central portion of the dam is an uncontrolled spillway section, which is approximately 18 inches lower in elevation than the non-overflow sections on each side. A sluice gate and 24-inch diameter discharge pipe are located on the left side of the spillway. The wheel for the sluice gate is missing from the gate shaft and is likely in storage. The inlet valve and beginning of the water
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GENERAL PLAN
DIVERSION DAM & RESERVOIR

Figure A-9
NOTES

1) EXISTING DAM DIMENSIONS ESTABLISHED BY FIELD SURVEY CONDUCTED BY AVI IN JUNE, 1999.

2) 1910 DESIGN DRAWINGS FROM STATE OF WYOMING ORIGINALLY PREPARED BY CITY OF CHEYENNE ENGINEER'S OFFICE CHEYENNE, WYOMING DATED 12/5/1910, ARE SHOWN AS HIDDEN LINES.
supply pipeline to Cheyenne are contained in a concrete building, which is part of the left abutment of the dam. The concrete gate house extends about 10 feet above the crest of the dam. A 6-foot section of concrete wall extends from the building to the left abutment contact with the rock outcrop and serves as an entrance.

The gate valve is used to control the amount of water diverted through the raw water delivery system to the City of Cheyenne treatment facilities. The structural dimensions of the gate house shown on Figure A-11 reveal that the structure was built close to the original design. The biggest difference is in the location of the pipeline center line. Since this facility was built before 1910, many of today's desirable operation features and controls are lacking.

**Condition of the Diversion Structure**

The diversion structure concrete has been affected by freeze-thaw action. Along the upstream face looking downstream, the concrete is deteriorating from the level of normal pool to several feet below water line. Localized repairs were made along the upstream face at some time in the past. Tapping of the repair work indicates that these repairs are not well bonded. The upstream face below the depth of freezing is in good condition based on past experience at similar structures.

The most severe deterioration is on the downstream face along the right side of the non-overflow section, looking downstream. Construction joints on the sloping surface below the crest are severely eroded to a depth of more than 6 inches and possibly as much as 12 inches in some locations. Substantial water flow was observed from holes along these construction joints at two locations. Nearly all of the downstream face is wet on the right side of the non-overflow section. Vegetation is growing over much of this area.

Erosion of deteriorated concrete in the spillway section appears to be reasonably uniform across the sloping downstream face. One severely eroded horizontal construction joint about 5 feet below the crest appears to extend from one abutment to the other and makes the top of the dam appear as a rectangular beam. The concrete below this construction joint is in much better condition except for some slightly deeper erosion near the toe of the spillway.

Exposed concrete on the left side of the non-overflow section appears to be in reasonably good condition compared to the right side. What appears to be original form marks can be seen on the surface of the sloping downstream face and a major portion of this surface is dry.

A few vertical cracks were observed on the right side of the non-overflow section but do not appear to be contributing to the deterioration. No pronounced vertical erosion channels were observed on the downstream face.

There was no evidence sulfate attack or cement-aggregate reactivity.

The combination of structural deterioration and steepness of the downstream slope has created a potentially unsafe structure. Stability calculations show that the existing structure does not satisfy current dam safety design criteria.
Evaluation of the existing diversion structure resulted in the following conclusions:

- Leakage and continued concrete deterioration should be eliminated.
- Monitoring of discharges should be improved.
- The dam should be buttressed by adding mass to the downstream face of anchored to the foundation to satisfy safety criteria.

**EROSION AND SEDIMENT**

Sediment accumulation in the reservoir and downstream degradation has been limited. The creek section downstream of the diversion structure exhibits limited signs of erosion. The structure has functioned for over 89 years without an energy stilling basin and has withstood spring overtopping floods. Sediment accumulation was surveyed and is estimated to be less than 2 to 3 acft. Over 15 feet of sediment is lodged against the right abutment, and appears to be fairly stable.

**PROJECTED VALUE OF EXISTING FACILITY**

The City’s BOPU staff has established the value of raw water to be approximately $0.63 per 1000 gallons or $200/acft. Treated water is $2.25 per 1000 gallons in 1999. Other consultants have established the value of raw water in Wyoming to be approximately $0.82 per 1000 gallons. In Colorado, the value of raw water averages $1.50 per 1000 gallons. The projected value of the existing facility is the value of water delivered. Based on the estimated 500 acft per year at $200 per acft, the yearly benefit is $100,000.
REHABILITATION PLAN

INTRODUCTION

The Phase I inventory identified the rehabilitation required to return the diversion structure, pipeline, controls and other features of the South Crow Diversion Project to an acceptable working order or an improved condition. Based on the analysis and discussions with project sponsors, those repairs identified as essential to the rehabilitation project were listed. The listing of needs is summarized below and separated into (1) common features, or (2) alternatives to be analyzed.

Common features include all items required for future rehabilitation. Examples of common features are new valves for pipeline control and pipeline air release valves. These features are listed as essential components of any the rehabilitation alternatives. They were not considered in the comparison or costed at this stage in the study.

Alternatives include rehabilitation needs that require a comparison of costs and other factors to decide which alternative should be the recommended rehabilitation project. Items in this category include the following:

- Diversion structure repair alternatives
- Diversion structure height alternatives

This chapter documents the analysis and comparison used to select the rehabilitation project.

REHABILITATION REQUIRED

The results of the inventory showed that the following rehabilitation and modernization is required:

Pipeline:
- Air release valves.
- Blow-off tees and valves.

Operation Controls:
- A guard valve.
- A control valve.
- A SCADA system.
- Intake screens.
**Diversion Structure:**

- Repairs to arrest leakage and stop concrete deterioration.
- Structural mass to improve stability.
- Improve valve house.

**Reservoir:**

- Increase storage capacity at the existing site to account for storage loss or sediment removal and to regain storage capacity loss

Only the items listed under the Diversion Structure were considered in the alternative evaluation.

**DIVERSION STRUCTURE REHABILITATION ALTERNATIVES**

Evaluation of existing and collected data and discussions with project sponsors established the following potential rehabilitation alternative for the South Crow diversion structure:

- Replacement of existing structure
- Seal upstream surface of diversion structure with:
  - special sealants
  - geotextile materials
  - concrete grouts
- Overlay upstream face of existing structure with concrete
- Overlay downstream face of existing structure with:
  - concrete
  - RCC
  - soil-cement

Alternatives not considered included grouting the concrete structure or epoxy injections. These types of repair alternatives were eliminated from further consideration due to material costs, access problems, and structure stability factors. Injection of grout, epoxy, or other penetrating substances to seal the existing structure would require a drilling rig to be brought to the site and moved onto the structure. Both the site, and diversion dam layout, are not readily accessible for this type of repair work and developing access for this alternative would be more expensive than for the other alternatives.

Downstream overlay alternatives have an advantage compared to upstream treatments or overlay, because concrete work can be performed on the downstream side of the diversion structure in the dry. If a downstream overlay is required, the question becomes: “why spend the money to seal the
upstream face with a geomembrane material when a downstream overlay is easier and less expensive to construct”.

Replacement and overlay alternatives provide the opportunity to raise the crest of the diversion structure to create additional storage.

Each alternative is described in some more detail in the following paragraphs.

Replacement Alternative

Replacement requires removal of the existing structure and construction of a new diversion or large dam structure at the same location. The decision to pursue this alternative depends on the condition of the existing structure, cost of the new structure, and the ability to store imported water via the Stage I pipeline (Figure A-2) or capture additional water from the South Crow.

If this alternative is selected, work would start with the construction of a diversion system and the draining of the reservoir. Upstream and downstream cofferdams would be needed. Once the area is protected and the reservoir drained, demolition of the existing structure would start and construction of a new structure would proceed.

The benefits of considering this alternative are that it facilitates the installation of new equipment and monitoring systems instead of retrofitting. It replaces a 90-year old structure with a new one. It also provides the opportunity to raise the crest elevation and store more water.

The biggest disadvantage to the selection of this alternative is its cost.

Sealants

There are a number of sealing systems from various manufactures that have excellent containment properties. Most are expensive to apply. A review of collected literature and discussions with suppliers (including Thortex, Belzona, Superior Environment Products, and Carpi) identified the following applications for further consideration:

- Geotextile-geomembrane material coupled with a crack sealant
- Upstream grout cover coupled with a crack sealant
- Upstream concrete slurry wall requiring the removal of the sediment

Because the reservoir can be drained, the upstream concrete slurry wall alternative was converted to a cast-in-place upstream concrete “vertical wall” overlay instead of a displacement slurry wall. The benefits of overlays are discussed in the next section. Examples of the geotextile and upstream overlay alternatives are shown on Figure A-12.

Rehabilitation of the structure with a sealant alternative would start with the construction of a diversion system and the draining of the reservoir. Once drained, the structure face would be sandblasted and cleaned. Sandblasting prepares the surface for the application of a smooth fabric or other type of sealant. Each sealant application is different, some require an injection of a crack sealant, others use a
EXISTING STRUCTURE WITH GEOTEXTILE SEAL OPTION

CONCRETE UPSTREAM WALL OPTION

LENGTH VARIES

LENGTH VARIES

GEOTEXTILE FABRIC FASTENED TO UPSTREAM SURFACE

REINFORCEMENT NOT SHOWN

SCALE

1"=10' AT FULL SIZE

WWDC

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4.0' 4.0'

0.695

0.695

0.695

0.695

Figure A-12
penetrating primer, and some count of the combination of these applications. One that is price competitive is Peraseal which is an easily applied roll out membrane with a bentonite layer which bonds to the concrete structure. A disadvantage is that such a solid seal prevents water in the structure to drain back into the reservoir from behind the membrane.

The benefits of the sealant alternatives are that they seal the upstream face. Once sealed, leakage through the structure is stopped, and concrete sloughing on the downstream side stops. The alternative also fosters less use of concrete if the stability analysis indicates the structure is stable.

The disadvantages of upstream sealant alternatives include application difficulties, material costs, and the loss of increased storage potential through a raise in the structure height.

**Overlay Alternatives**

These alternatives involve increasing the structure mass on the downstream side with either an overlay of concrete as shown on Figure A-13 or RCC or soil-cement, as shown on Figure A-14. If RCC is constructed with dirt working equipment, it will require a minimum working width of 10 feet. The increase of downstream structure mass increases the stability of the structure. An alternative to a downstream overlay is an upstream overlay (Figure A-12).

Construction of a downstream overlay would proceed without lowering the reservoir. The existing sluice would function as the diversion during construction. Sandblasting and surface preparation would proceed the construction of the overlay. Installation of fabric, pipe, or cut drains on the downstream surface would follow the surface cleaning. These drains will control and direct the leakage through the existing structure. The overlay would be doweled to the old structure.

The upstream alternative construction would follow a similar pattern with the exception of having a need to drain the reservoir.

The amount and depth of the overlay depends on the type of overlay. A minimum concrete overlay is a layer of concrete 2 feet perpendicular to the slope of the existing structure. This minimum requirement of a two-foot thickness of concrete overlay is due to weather’s freeze and thaw action. Stability calculations can trigger an increase in the overlay thickness to satisfy stability requirements. The RCC and soil-cement alternatives require a minimum layer thickness of 10 feet.

The benefits of downstream overlays are that it causes the minimum amount of disturbance to the site, is easier to construct, and provides a means for the addition of safety features such as a dissipation basin. Increasing the structure’s height to capture more storage is also a possibility with an overlay alternative.

Continued leakage through the existing structure, which necessitates the use of drainage fabric, is one disadvantage of downstream overlay alternatives.
SOIL/Cement Overlay NTS

RCC Overlay NTS

LENGTH: VARIES

MIN 10.0'

MAX 15.0'

SCALE A

1"=10' AT FULL SIZE

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REHABILITATION
RCC & SOIL CEMENT OVERLAYS

Figure A-14
ALTERNATIVE ANALYSIS

The focus in Phase I was on the comparison of the diversion rehabilitation alternatives. The analyses required to accomplish this comparison included:

- Determining the amount of storage volume lost to sediment. This information established the amount of raise in the crest of the dam required to reclaim the storage volume lost.

- Determining Probable Maximum Flood (PMF) and 100 year flood volumes to be used in flood routing analyses.

- Flood routing to determine the overtopping elevation used in the stability analysis. This was determined to be approximately 3 feet for a 100-year flood.

- Stability analysis of the existing structure, improved structures, and raised structures. These were used to determine the amount of concrete required for the downstream and upstream overlay alternatives.

- Developing cost estimates for the alternative comparison and the eventual elimination of the more expensive alternatives.

For documentation of the lost storage, PMF and 100 year flood routing, see Appendix H in the Phase II Report. The analysis of water rights, pipeline characteristics, and control requirements are also documented in the Phase II Report's Appendixes.

Lost storage was determined by comparing a field survey to topographic data. The amount lost was estimated to be 3 acft. The increases in height of diversion spillway crest elevation to regain this volume was established with the same field survey, see Figure A-9. The raise required was 3 feet. This moved the dam crest from elevation 7132 to 7135, see Figure A-13.

Stability Analysis

Details and results of the stability analyses are contained in Appendix D.

From the stability analysis the downstream overlay requirement, shown in Figure A-13, was determined to be approximately 3.5 feet at the top and 4.5 feet at the bottom. The minimum thickness recommended by concrete experts for an overlay was 2 feet. This 2-foot thickness is required to protect the new and the old from freeze-thaw effects. The stability calculations have determined the overlay for stability reasons to be thicker than the minimum requirement and thus govern the amount of concrete required. The final configuration selected requires approximately 500 cubic yards of concrete to construct the downstream overlay and raise the crest of diversion structure raised to elevation 7135.

Construction requirements more than stability requirements governed the shape of the RCC or soil-cement overlay. The top width for both alternatives was set at 10 feet, see Figure A-14. Both the RCC and soil-cement overlay required concrete to raise the crest of the structure to elevation 7135.
volume of material required for the RCC overlay with a crest of 7135 includes 2590 cubic yards of RCC and approximately 15 cubic yards of concrete. The volume of material for the soil-cement overlay with a crest of 7135 includes 2200 cubic yards of an assumed soil-cement mix of 1 to 3 and 15 cubic yards of concrete.

The stability analysis was used to establish the upstream overlay thickness at four foot thick.

The volume of concrete required with the 4-foot thick upstream overlay and bring the crest elevation to 7135.00, is approximately 790 cubic yards of concrete.

Cost Comparisons

The replacement alternative was not considered in the cost comparison. The volume of concrete or RCC to build a new structure would be more than any of the overlay alternatives and thus by a volume comparison alone the replacement alternative was considered too costly.

Calculations determined that the upstream concrete overlay alternative requires more concrete than a downstream concrete overlay. This fact alone eliminated the upstream overlay option from further consideration. In addition to the volume difference, the upstream overlay would require construction efforts be carried out in a drained reservoir. It would also require the removal of the sediment before the wall could be constructed. Both of these factors increase the cost and support the decision to eliminate this alternative from further consideration.

The selected upstream seal alternative consists of (1) injecting moisture cured sealant slurry into the cracks, (2) applying a concrete sealer to the upstream face, (3) laying down a geotextile fabric, and then (4) coating the fabric. The total cost for this alternative with labor was estimated at $104,000. This cost increases to cover the cost of a minimum 2-foot thick downstream freeze and thaw protection overlay and the cost of concrete to raise the dam crest. Because the total cost of a downstream overlay alternative was less than a sealant alternative, the seal alternative was not carried forward for further consideration.

This process eliminates all alternatives from further consideration except for the three overlay alternatives. The selection of which downstream overlay alternative should be selected for further consideration depended on cost:

- Concrete overlay costing approximately $80,000
- RCC overlay costing approximately $132,000 or more depending on mobilization cost estimates
- Soil-cement overlay costing approximately $83,000 and more depending on the soil type and availability.

Conclusion

Based on rehabilitation requirements, stability analysis and cost comparisons the downstream concrete overlay provides the best protection, a safe structure, and has the lowest comparable cost. The RCC overlay alternative provides the same degree of protection at a higher cost.
RECOMMENDATION

REHABILITATION ALTERNATIVE RECOMMENDATION

The results of the alternative comparison analysis ends with the conclusion that the downstream concrete overlay with a crest raise is the alternative to carry forward into Phase II.

In Phase II the conceptual design for this alternative will be completed and the complete project will be costed. The cost estimate will include the essential common items not considered at this point in the study. The economic analysis will be completed to establish the feasibility of the complete (diversion and pipeline) rehabilitation project.

NEW DAM RECOMMENDATION

In the second phase of the project to establish the benefit of storing imported water in the South Crow Basin an upstream dam site will be selected and studied. The availability of water to fill the proposed reservoir will be established. A cost curve as a function of dam height will be established.

The cost of a new dam will be compared to the costs required to rehabilitate the diversion structure.
APPENDIX B

Pipeline Condition Assessment
This is a report of the corrosion testing conducted on the existing South Crow Reservoir Pipeline.

Introduction

As part the project work task #2 a preliminary evaluation of the existing South Crow Reservoir dam and pipeline was completed in July. The work included a complete corrosion assessment of the 90 year old, 14 inch and 16 inch cast iron pipeline. The assessment consisted of the following tasks:

1. Obtain soil resistivity measurements at approximately 500 foot intervals along the pipeline alignment to evaluate general soil corrosivity.
2. Inspect the pipe exterior at six pot hole sized excavations sites to observe and classify any corrosion activity or damage.
3. Obtain pipe wall thickness measurements at the six excavation sites using an ultrasonic thickness instrument.
4. Review the results from the laboratory analyses conducted on the grab bag soil samples obtained at each excavation site.
5. Document all field and laboratory data, conclusions and recommendations.

Conclusions

1. The pipeline consists of nominal 16 inch diameter cast iron from the reservoir to the highest topographical point on a ridge downstream and nominal 14 inch cast iron from the high point to the downstream end of the line.
2. The pipeline is constructed of cast iron pipe with lead sealed joints. The pipe joints are not electrically bonded other than might inadvertently occur through the leaded joints.
3. The pipe appeared to have a very thin (< 10 mils) external coating that could be easily scraped, filed, or ground off but showed no evidence of flaking or peeling.

4. A total of six excavations of the pipeline were completed at various locations (wet and dry) along the alignment by Cheyenne Public Utilities personnel. The exterior surface of the pipeline showed little or no corrosion damage in any of the excavations. There was evidence of very minor corrosion (light surface rust) on the end of the bell of several pipe joints. This appeared to be the result of galvanic corrosion between the lead in the pipe joint seal and the bell of the pipe. No corrosion activity was observed on the spigot end of any of the pipe joints. No pitting corrosion was observed in any excavation.

5. Significant corrosion in the form of graphitization (loss of iron atoms from the metal matrix leaving behind only the carbon atoms) was observed at an existing blow off valve in Excavation #5. The damage occurred on a short piece (approx. 1 foot long) of 6 inch cast iron pipe between the tee in the main line and the blow off valve. The graphitization was easily scraped and dug at with a knife to a depth exceeding 200 mils at which point no further graphite removal was attempted.

6. Twenty three soil resistivity measurements were obtained along the pipeline alignment using the Wenner 4-Pin Method (ASTM G57). Soil resistivities ranged from a low value of 720 ohm-cm to a high value of 29,108 ohm-cm with ten readings below 2,000 ohm-cm (corrosive to very corrosive to ferrous metals), ten readings between 2,000 ohm-cm and 5,000 ohm-cm (moderately corrosive), and three readings above 5,000 ohm-cm (mildly corrosive to slightly corrosive).

7. Pipe wall thickness measurements were obtained in each excavation using a digital ultrasonic thickness meter. Between 30 and 60 measurements were obtained at each excavation site. Wall thickness ranged from 0.572 inches to 0.997 inches which indicates minimal internal corrosion damage at these locations.

8. Grab bag soil samples obtained from each excavation were laboratory tested for pH, chlorides, moisture content, Redox potential, and soil resistivity (at both in-situ and saturated moisture levels). These soil chemistry data indicated the soils are generally classified as moderately to mildly corrosive to ferrous metals.

Recommendations

Based upon the field testing, lab results, and visual pipe inspections, complete the following recommendations 1, 2, and 3 prior to placing the pipeline in service:

1. Replace the blow off valve and piping located at Excavation #5 that showed signs of corrosion by graphitization. The new piping should be wrapped with polyethylene encasement.

2. Locate and excavate all other blow-off valves and replace if the piping shows evidence of corrosion or graphitization. Any new piping should be wrapped with polyethylene encasement.

3. Retain any pipe coupons obtained from the installation of new air/vac valves for examination of internal corrosion and presence of any lining material.

4. Cathodic protection is not recommended due to the following:
   a) There is no evidence that external corrosion presently threatens or will threaten the integrity of the pipeline.
   b) Cathodic protection is not cost effective due to a lack of joint bonding for reliable electrical continuity and a lack of a dielectric external coating.
Discussion

The City of Cheyenne Board of Public Utilities personnel completed six excavations on the South Crow Reservoir raw water pipeline on June 22-23, 1999. The excavations were located in areas of different topographical features, i.e., one on a high ridge, one in a creek bottom, one in a drainage draw, etc. The excavations were completed to allow verification of pipe diameter, visual examination of the condition of the pipeline, pipe wall thickness measurements, and to obtain soil samples for lab analysis. While the excavation work was being completed, we conducted soil resistivity testing at approximately 500 foot intervals along the pipeline alignment to enable classification of general soil corrosivity.

The South Crow Raw Water Pipeline consists of nominal 14 inch and 16 inch diameter cast iron pipe installed in 1910. It runs from the South Crow Reservoir eastward approximately 12,000 feet to a tie-in with another raw water line from the Crystal Lake area.

Soil resistivity measurements using the Wenner 4-Pin Method (ASTM G57) were obtained along the pipeline alignment to assess general soil corrosivity. In an attempt to obtain measurements to a depth of the pipe invert despite varying surface topography, three readings with a pin spacing of 6 feet, one reading with a pin spacing of 10 feet, and 19 readings with a pin spacing of 8 feet were obtained. Testing started at a point approximately 250 feet downstream from the South Crow Reservoir Dam and continued at approximately 500 foot intervals to the end of the line. Soil resistivities ranged from a low value of 720 ohm-cm to a high value of 29,108 ohm-cm with ten readings below 2,000 ohm-cm (corrosive to very corrosive to ferrous metals), ten readings between 2,000 ohm-cm and 5,000 ohm-cm (moderately corrosive), and three readings above 5,000 ohm-cm (mildly corrosive to slightly corrosive). Based on the 4-Pin soil resistivity data, the soil environment around the pipe is generally classified as corrosive to moderately corrosive to ferrous metals. Refer to the test data in Appendix A.

Excavation No. 1 was located approximately 1,500 to 2,000 feet from the downstream end of the pipeline. The depth to the top of the pipe was approximately seven feet and the pipe diameter was a nominal 14 inches. The soils were a dry, light brown sand with some gravels and silts. No rust or other corrosion activity was observed anywhere on the approximately 4 feet of exposed pipe. The pipe appeared to have a thin (< 10 mils thick) black external coating that was uniform and intact but easily scraped and filed off.

Excavation No. 2 was located approximately 2,000 feet upstream from Excavation No. 1. The depth to the top of the pipe was approximately five feet and the pipe diameter was a nominal 14 inches. A bell and spigot type joint with a lead type seal was visible. The soils consisted of a dry, medium brown sand with some gravels and silts. No rust or other corrosion activity was observed anywhere on the barrel portions of the approximately 4 feet of exposed pipe. Very minor surface rust was observed on portions of the pipe bell which is speculated to be the result of a galvanic reaction with the lead in the joint seal. The pipe appeared to have a thin (< 10 mils thick) black external coating that was uniform and intact but easily scraped and filed off.

Excavation No. 3 was located approximately 2,000 feet upstream from Excavation No. 2. This hole was dug in the bottom of a wet drainage ditch. The depth to the top of the pipe was approximately five feet and the pipe diameter was a nominal 14 inches. The soils consisted of a wet, dark brown and gray sand with some gravels,
sils and clays. Free ground water was evident in the bottom of the excavation. No rust or other corrosion activity was observed anywhere on the approximately 4 feet of exposed pipe. The pipe appeared to have a thin (<10 mils thick) black external coating that was uniform and intact but easily scraped and filed off.

Excavation No. 4 was located approximately 1,500 feet upstream from Excavation No. 3. This hole was dug at the top of a ridge which appeared to be the highest elevation point downstream of the reservoir. The depth to the top of the pipe was approximately 9 feet and the pipe diameter was a nominal 16 inches. The soils consisted of a slightly moist, medium brown sand with some gravels and silts. Minor surface rust was observed in a few small areas on the pipe as well as on the pipe bell. No other corrosion activity was observed anywhere on the approximately 4 feet of exposed pipe. The pipe appeared to have a thin (<10 mils thick) black external coating that was uniform and intact but easily scraped and filed off.

Excavation No. 5 was located approximately 1,500 feet upstream from Excavation No. 4. This hole was dug at a topographical low point but not in a ditch or wet area. A blow off valve was uncovered during excavation. The depth to the top of the pipe was approximately 5 feet and the pipe diameter was a nominal 16 inches. The soils consisted of a slightly moist, medium to light brown sand with some gravels and clays. No rust or other corrosion activity was observed anywhere on the barrel portions of the approximately 5 feet of exposed pipe. Very minor surface rust was observed on portions of the pipe bell which is speculated to be the result of a galvanic reaction with the lead in the joint seal. The pipe appeared to have a thin (<10 mils thick) black external coating that was uniform and intact but easily scraped and filed off. The blow off consisted of an approximately one foot long piece of 6 inch cast iron pipe running from a tee in the mainline to a 6 inch buried valve. This short piece of 6 inch pipe had incurred serious corrosion damage in the form of graphitization (loss of iron atoms from the cast iron metal matrix leaving behind only the carbon atoms). A pocket knife was used to dig a hole in excess of 200 mils deep into the graphitized 6 inch pipe. It is recommended that this 6 inch pipe and blow off valve be replaced prior to returning the line to service.

Excavation No. 6 was located approximately 1,200 feet upstream from Excavation No. 5. This hole was dug at a higher topographical point but in an area where the 4-Pin soil resistivity data suggested was corrosive. The depth to the top of the pipe was approximately 7 feet and the pipe diameter was a nominal 16 inches. The soils consisted of a slightly moist, light brown silty sand. No rust or other corrosion activity was observed anywhere on the approximately 4 feet of exposed pipe. The pipe had a thin (<10 mils thick) black external coating that was uniform and intact but easily scraped and filed off.

Contact with the Ductile Iron Pipe Research Association (DIPRA) regarding the thin black coating on the pipe exterior revealed that some cast iron waterlines in the early 1900’s were dipped in a hot tar like substance to provide both an internal lining and an external coating. It is speculated that the South Crow raw water line may have been coated in this manner. It is recommended that when the City installs new air/vac valves, the pipe coupons from the new taps be retained for examination for corrosion and the presence of a lining.

Wall thickness measurements were obtained in each excavation using an ultrasonic thickness meter on July 1, 1999. A grid with a cell size of approximately 3 inches longitudinally by 6 inches circumferentially was marked on the surface of the pipe for a length of two feet and measurements were obtained at each gridline intersection for a total of 30 to 60 measurements in each excavation. The pipe surface was made smooth at each test point.
by grinding off the coating and any surface irregularities with a power grinding wheel. Wall thickness ranged from 0.572 inches to 0.997 inches which suggests that virtually no internal corrosion damage has occurred at these locations. It should be noted that no wall thickness measurements were obtained in Excavation No. 3 as this hole was backfilled immediately after visual inspection due to high groundwater. Refer to Appendix A for the wall thickness test data.

Soil samples were obtained from each excavation site adjacent to the pipe. These grab bag samples were collected for the purpose of examining the chemistry of the soil environment around the pipe. The samples were laboratory tested for pH, chlorides, moisture content, Redox potential, and soil resistivity (at both in-situ and saturated moisture levels) by Terracon with Stewart Environmental performing the chloride tests. These soil chemistry data indicated the soils at the six excavation sites are generally classified as moderately to mildly corrosive to ferrous metals. Refer to Appendix A for the test data.

Summary

An initial corrosion evaluation was performed on the City of Cheyenne’s South Crow Raw Water Pipeline. This nominal 14 inch and 16 inch diameter cast iron pipeline line was installed in 1910. The pipe was visually inspected in six excavations completed by City personnel and ultrasonic wall thickness testing was also completed. Soil samples were obtained from each excavation site to assess soil chemistry and soil resistivity measurements were obtained at approximately 500 foot intervals along the pipeline alignment to assess general soil corrosivity.

Despite soil resistivity and soil chemistry data indicating moderately to mildly corrosive soil conditions, there was only very minor corrosion activity observed anywhere on the pipeline. It appears that a thin coating material, possibly some type of tar, was applied to the pipe exterior which may be responsible for the excellent external condition of the pipe. In addition, the pipe wall thickness data indicates little or nor internal corrosion has likely occurred. This pipeline is in outstanding condition and appears to be suitable for being returned to service without any other considerations for corrosion control.

Attachment: Sub-Appendix B1
Sub-Appendix B2 - Photos
SUB-APPENDIX B1

Table 1 – Soil Resistivity Data
Table 2 – Soil Sample Analyses
Table 3- Excavation No.1 Wall Thickness Data
Table 4- Excavation No.2 Wall Thickness Data
Table 5- Excavation No.4 Wall Thickness Data
Table 6- Excavation No.5 Wall Thickness Data
Table 7- Excavation No.6 Wall Thickness Data
<table>
<thead>
<tr>
<th>Test Point</th>
<th>4-Pin Spacing (ft.)</th>
<th>Meter Reading</th>
<th>Meter Multiplier</th>
<th>Calculated Resistivity (Ohm-cm)</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8</td>
<td>1.9</td>
<td>10</td>
<td>29,108</td>
<td>250' downstream of reservoir</td>
</tr>
<tr>
<td>2</td>
<td>6</td>
<td>1.2</td>
<td>10</td>
<td>13,788</td>
<td>at 2nd blue painted marker post</td>
</tr>
<tr>
<td>3</td>
<td>6</td>
<td>3.2</td>
<td>1</td>
<td>3,677</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>6</td>
<td>3.3</td>
<td>1</td>
<td>3,792</td>
<td></td>
</tr>
<tr>
<td>4A</td>
<td>10</td>
<td>1.6</td>
<td>1</td>
<td>3,064</td>
<td>same location as Test Point 3</td>
</tr>
<tr>
<td>5</td>
<td>8</td>
<td>1.2</td>
<td>1</td>
<td>1,838</td>
<td>SW end of hay field near marker</td>
</tr>
<tr>
<td>6</td>
<td>8</td>
<td>6.7</td>
<td>0.1</td>
<td>1,026</td>
<td>low spot in hay field near blowoff</td>
</tr>
<tr>
<td>7</td>
<td>8</td>
<td>7.7</td>
<td>0.1</td>
<td>1,180</td>
<td>Adjacent to Excavation #6</td>
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<tr>
<td>8</td>
<td>8</td>
<td>4.7</td>
<td>0.1</td>
<td>720</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>8</td>
<td>7.6</td>
<td>0.1</td>
<td>1,164</td>
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<tr>
<td>11</td>
<td>8</td>
<td>1.3</td>
<td>1</td>
<td>1,992</td>
<td>Adjacent to Excavation #4</td>
</tr>
<tr>
<td>12</td>
<td>8</td>
<td>2.3</td>
<td>1</td>
<td>3,524</td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>8</td>
<td>1.1</td>
<td>1</td>
<td>1,685</td>
<td></td>
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<tr>
<td>14</td>
<td>8</td>
<td>8.9</td>
<td>0.1</td>
<td>1,363</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>8</td>
<td>2.4</td>
<td>1</td>
<td>3,677</td>
<td>In drainage draw with culvert</td>
</tr>
<tr>
<td>16</td>
<td>8</td>
<td>1.7</td>
<td>1</td>
<td>2,604</td>
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<tr>
<td>17</td>
<td>8</td>
<td>2.5</td>
<td>1</td>
<td>3,830</td>
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<td>1</td>
<td>4,749</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>8</td>
<td>3</td>
<td>1</td>
<td>4,596</td>
<td></td>
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<tr>
<td>21</td>
<td>8</td>
<td>1.5</td>
<td>1</td>
<td>2,298</td>
<td></td>
</tr>
<tr>
<td>22</td>
<td>8</td>
<td>3.8</td>
<td>1</td>
<td>5,822</td>
<td>Approx. 500' from DS end of line</td>
</tr>
</tbody>
</table>

**NOTES:**
1. All data obtained using a Nilsson Laboratories Model 400 Soil Resistivity Meter
2. Measurements obtained on approx. 500' intervals along pipe alignment
3. Wenner 4-Pin Method (ASTM G57) used to obtain data
4. Standard Soil Resistivity Classifications:
   - 0 to 1,000 Ohm-cm = Very Corrosive
   - 1,000 to 2,000 Ohm-cm = Corrosive
   - 2,000 to 5,000 Ohm-cm = Moderately Corrosive
   - 5,000 to 10,000 Ohm-cm = Mildly Corrosive
   - Above 10,000 Ohm-cm = Slightly Corrosive
TABLE 2
CITY OF CHEYENNE
SOUTH CROW DIVERSION PROJECT
EXISTING RAW WATER PIPELINE
SOIL SAMPLE ANALYSES
JUNE 1999

<table>
<thead>
<tr>
<th>Excavation Number</th>
<th>pH</th>
<th>Chloride (ppm)</th>
<th>In-Situ Moisture Content (%)</th>
<th>Redox Potential (mv)</th>
<th>Resistivity At In-Situ Moisture (ohm-cm)</th>
<th>Resistivity At Saturated Moisture (ohm-cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7.99</td>
<td>ND</td>
<td>15.0</td>
<td>25.5</td>
<td>3,200</td>
<td>3,200</td>
</tr>
<tr>
<td>2</td>
<td>7.57</td>
<td>ND</td>
<td>7.9</td>
<td>21.9</td>
<td>8,800</td>
<td>9,200</td>
</tr>
<tr>
<td>3</td>
<td>8.99</td>
<td>ND</td>
<td>18.7</td>
<td>351.7</td>
<td>3,200</td>
<td>3,200</td>
</tr>
<tr>
<td>4</td>
<td>7.93</td>
<td>ND</td>
<td>14.0</td>
<td>53.7</td>
<td>3,200</td>
<td>2,800</td>
</tr>
<tr>
<td>5</td>
<td>7.91</td>
<td>ND</td>
<td>24.1</td>
<td>85.6</td>
<td>2,000</td>
<td>2,000</td>
</tr>
<tr>
<td>6</td>
<td>8.42</td>
<td>ND</td>
<td>20.3</td>
<td>15.4</td>
<td>2,000</td>
<td>2,800</td>
</tr>
</tbody>
</table>

NOTES:  
1. ND = Not Detected  
2. Lab Analyses completed by Terracon and Stewart Environmental
# TABLE 3

CITY OF CHEYENNE
SOUTH CROW DIVERSION PROJECT

EXISTING RAW WATER PIPELINE
PIPE WALL THICKNESS DATA
JULY 1999

EXCAVATION NO. 1

<table>
<thead>
<tr>
<th>Location Longitudinally On Pipe</th>
<th>Location Circumferentially on Pipe</th>
<th>Top Centerline</th>
<th>Location Circumferentially on Pipe</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>24&quot;</td>
<td>18&quot;</td>
<td>12&quot;</td>
</tr>
<tr>
<td>0</td>
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<td></td>
</tr>
<tr>
<td>3&quot;</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6&quot;</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>9&quot;</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>12&quot;</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15&quot;</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>18&quot;</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>21&quot;</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>24&quot;</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTES:**
1. All thickness measurement data in mils (thousandths of an inch).
2. Directions around and along the pipe are referenced to looking downstream in each excavation.
3. Data obtained using an ultrasonic thickness meter.
4. Pipe surface cleaned to smooth finish at all test points using a power grinding wheel.
### TABLE 4

**CITY OF CHEYENNE**  
**SOUTH CROW DIVERSION PROJECT**  
**EXISTING RAW WATER PIPELINE**  
**PIPE WALL THICKNESS DATA**  
**JULY 1999**

#### EXCAVATION NO. 2

<table>
<thead>
<tr>
<th>Location Longitudinally On Pipe</th>
<th>Location Circumferentially on Pipe Counterclockwise</th>
<th>Top Centerline</th>
<th>Location Circumferentially on Pipe Clockwise</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>24&quot;</td>
<td>18&quot;</td>
<td>12&quot;</td>
</tr>
<tr>
<td>0</td>
<td>649</td>
<td>695</td>
<td>666</td>
</tr>
<tr>
<td>3&quot;</td>
<td>698</td>
<td>677</td>
<td>654</td>
</tr>
<tr>
<td>6&quot;</td>
<td>672</td>
<td>756</td>
<td>725</td>
</tr>
<tr>
<td>9&quot;</td>
<td>699</td>
<td>695</td>
<td>653</td>
</tr>
<tr>
<td>12&quot;</td>
<td>741</td>
<td>675</td>
<td>647</td>
</tr>
<tr>
<td>15&quot;</td>
<td>702</td>
<td>660</td>
<td>647</td>
</tr>
<tr>
<td>18&quot;</td>
<td>727</td>
<td>631</td>
<td></td>
</tr>
<tr>
<td>21&quot;</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>24&quot;</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTES:**  
1. All thickness measurement data in mils (thousandths of an inch).  
2. Directions around and along the pipe are referenced to looking downstream in each excavation.  
3. Data obtained using an ultrasonic thickness meter.  
4. Pipe surface cleaned to smooth finish at all test points using a power grinding wheel.
## TABLE 5

**CITY OF CHEYENNE**  
**SOUTH CROW DIVERSION PROJECT**  
**EXISTING RAW WATER PIPELINE**  
**PIPE WALL THICKNESS DATA**  
**JULY 1999**

**EXCAVATION NO. 4**

<table>
<thead>
<tr>
<th>Location Longitudinally On Pipe</th>
<th>Location Circumferentially on Pipe Counterclockwise</th>
<th>Location Circumferentially on Pipe Clockwise</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>24&quot;</td>
<td>18&quot;</td>
</tr>
<tr>
<td>0</td>
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<tr>
<td>3&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>9&quot;</td>
<td>725</td>
<td>648</td>
</tr>
<tr>
<td>12&quot;</td>
<td>754</td>
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<td>769</td>
<td>690</td>
</tr>
<tr>
<td>18&quot;</td>
<td>879</td>
<td>727</td>
</tr>
<tr>
<td>21&quot;</td>
<td>873</td>
<td>775</td>
</tr>
<tr>
<td>24&quot;</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**NOTES:**  
1. All thickness measurement data in mils (thousandths of an inch).  
2. Directions around and along the pipe are referenced to looking downstream in each excavation.  
3. Data obtained using an ultrasonic thickness meter.  
4. Pipe surface cleaned to smooth finish at all test points using a power grinding wheel.
# EXCAVATION NO. 5

<table>
<thead>
<tr>
<th>Location Longitudinally On Pipe</th>
<th>Location Circumferentially on Pipe</th>
<th>Top Centerline</th>
<th>Location Circumferentially on Pipe</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>24&quot;</td>
<td>18&quot;</td>
<td>12&quot;</td>
</tr>
<tr>
<td>0</td>
<td>862</td>
<td>737</td>
<td>674</td>
</tr>
<tr>
<td>3&quot;</td>
<td>795</td>
<td>764</td>
<td>669</td>
</tr>
<tr>
<td>6&quot;</td>
<td>807</td>
<td>827</td>
<td>727</td>
</tr>
<tr>
<td>9&quot;</td>
<td>825</td>
<td>792</td>
<td>715</td>
</tr>
<tr>
<td>12&quot;</td>
<td>896</td>
<td>781</td>
<td>762</td>
</tr>
<tr>
<td>15&quot;</td>
<td>835</td>
<td>807</td>
<td>796</td>
</tr>
<tr>
<td>18&quot;</td>
<td>870</td>
<td>817</td>
<td>733</td>
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<td>21&quot;</td>
<td>864</td>
<td>812</td>
<td>765</td>
</tr>
<tr>
<td>24&quot;</td>
<td>762</td>
<td>734</td>
<td>894</td>
</tr>
</tbody>
</table>

## NOTES:

1. All thickness measurement data in mils (thousandths of an inch).
2. Directions around and along the pipe are referenced to looking downstream in each excavation.
3. Data obtained using an ultrasonic thickness meter.
4. Pipe surface cleaned to smooth finish at all test points using a power grinding wheel.
TABLE 7
CITY OF CHEYENNE
SOUTH CROW DIVERSION PROJECT
EXISTING RAW WATER PIPELINE
PIPE WALL THICKNESS DATA
JULY 1999

EXCAVATION NO. 6

<table>
<thead>
<tr>
<th>Location Longitudinally On Pipe</th>
<th>Location Circumferentially on Pipe Counterclockwise</th>
<th>Location Circumferentially on Pipe Clockwise</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>24&quot; 18&quot; 12&quot; 6&quot;</td>
<td>6&quot; 12&quot; 18&quot; 24&quot;</td>
</tr>
<tr>
<td>0</td>
<td>794 806 790 726</td>
<td>691 904 841</td>
</tr>
<tr>
<td>3&quot;</td>
<td>873 791 735 817</td>
<td>715 746 890</td>
</tr>
<tr>
<td>6&quot;</td>
<td>768 729 747 747</td>
<td>663 711 704 801</td>
</tr>
<tr>
<td>9&quot;</td>
<td>706 610 727 620</td>
<td>684 706 820</td>
</tr>
<tr>
<td>12&quot;</td>
<td>706 610 727 620</td>
<td>684 706 820</td>
</tr>
<tr>
<td>15&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>18&quot;</td>
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<tr>
<td>24&quot;</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

NOTES:
1. All thickness measurement data in mils (thousandths of an inch).
2. Directions around and along the pipe are referenced to looking downstream in each excavation.
3. Data obtained using an ultrasonic thickness meter.
4. Pipe surface cleaned to smooth finish at all test points using a power grinding wheel.
SUB-APPENDIX B2

Photos
South Crow pipeline blowoff at junction with 20-inch diameter Hecla pipeline

At Test Pit No. 4 looking back toward junction with Hecla pipeline.

On South Crow pipeline easement looking uphill towards STA 56 & 62 and Test Pit No. 4 (Refer to Figure A-4)

Excavation of Test Pit No. 1 by BOPU staff
Pipeline exposed in Test Pit No. 6 (Note: pipeline is in good condition)

Exposed pipe in Test Pit No. 2

Examination of pipeline in Test Pit No. 4

Exposed pipe in Test Pit No. 1
Tee and blowoff with gate valve in Test Pit No. 5 (Note: corrosion on small pipe to valve)

Exposure of pipe in Test Pit No. 4

Examination of corrosion in Test Pit No. 5

Markings in 1910 pipe still readable
APPENDIX C

Pipeline Hydraulic Analysis
PROJECT: South Crow Diversion Pipeline
DATE: 3/10/00
SUBJECT: Capacity Analysis South Line Existing Conditions
South Crow & HECLA joining at wye and continuing to Round Top Water Treatment Plant (WTP)

Given:
South Crow Alignment Drawings (Plan & Profile)
16" Nominal Dia from Sta 00+00 to Sta 34+74
14" Nominal Dia from Sta 34+74 to 132+04.13
Map Showing Water Supply System 1985
Demand Management fax

LEGEND:
\(\bigcirc\) Blowoff Valve
\(\square\) Reservoir
\(\blacktriangle\) Wye Junction
\(\blacktriangledown\) WSEL Symbol
ID 1 Inlet at South Crow Reservoir
ID 3 Inlet at HECLA Reservoir
ID 2 Pipe Class Change
ID 4 Junction of HECLA & South Crow
ID 5 Terminus of South Line
Round Top WTP

<p>| South Crow | HECLA | South Line |</p>
<table>
<thead>
<tr>
<th>ID</th>
<th>Station</th>
<th>CL El</th>
<th>ID</th>
<th>Station</th>
<th>CL El</th>
<th>ID</th>
<th>Station</th>
<th>CL El</th>
</tr>
</thead>
<tbody>
<tr>
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<td>00+00.00</td>
<td>7123.80</td>
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<td>34+74.00</td>
<td>7095.23</td>
<td>3</td>
<td>00+00.00</td>
<td>6799.17</td>
<td>4</td>
<td>00+00.00</td>
<td>6722.54</td>
</tr>
<tr>
<td>4</td>
<td>132+04.13</td>
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<td></td>
<td></td>
<td></td>
<td>5</td>
<td>897+60.00</td>
<td>6366</td>
</tr>
</tbody>
</table>

ID 3 6805 = WSEL
ID 1 7130.8 = WSEL
ID 2 Line 1 (South Crow)
Line 2 (HECLA)
ID 4 (NTS)
ID 5 WSEL = 6366

Map Showing Water Supply System 1985
Demand Management fax
Assumptions:

Fittings: \( h = K_v \frac{v^2}{2g} \)

Butterfly Valves & Gate Valves (assumed Open)

Sudden Contraction: \( h = K_v \frac{v^2}{2g} \)

Contraction from 16" to 14" dia pipe

Sudden Expansion: \( h = K_v \frac{v^2}{2g} \)

Expansion from 14" to 20"

Bends: \( h = K_v \frac{v^2}{2g} \)

Minor bends < 10 degrees

Friction: \( h = (fL/d) \frac{v^2}{2g} \)

See "Friction" worksheet for Darcy W. f value, e is for asphalt-dipped cast iron pipe

Entrance: \( h = K_v \frac{v^2}{2g} \)

Entrance loss for "Reentrant" type

<table>
<thead>
<tr>
<th>Pipe ID</th>
<th>Station</th>
<th>Diameter</th>
<th>Area</th>
<th>Length</th>
<th># Bends</th>
<th>K_b</th>
<th>Valves</th>
<th>K_v</th>
<th>Darcy f</th>
<th>K_e</th>
<th>D_2/D_1</th>
<th>K_e</th>
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</thead>
<tbody>
<tr>
<td>1-2</td>
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<td>34+74.00</td>
<td>1.33</td>
<td>1.40</td>
<td>3474.0</td>
<td>8</td>
<td>0.03</td>
<td>36&quot; GV</td>
<td>0.16</td>
<td>Attached</td>
<td>0.8</td>
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</tr>
<tr>
<td>2-4</td>
<td>34+74.00</td>
<td>132+04.13</td>
<td>1.17</td>
<td>1.07</td>
<td>9730.1</td>
<td>12</td>
<td>0.03</td>
<td>316&quot; BF</td>
<td>0.3</td>
<td>Attached</td>
<td>N/A</td>
<td>0.875</td>
</tr>
<tr>
<td>3-4</td>
<td>00+00.00</td>
<td>52+80.00</td>
<td>1.67</td>
<td>2.18</td>
<td>5280.0</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>Attached</td>
<td>0.8</td>
<td>1</td>
<td>1</td>
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<tr>
<td>4-5</td>
<td>00+00.00</td>
<td>897+60.00</td>
<td>1.67</td>
<td>2.18</td>
<td>89760.0</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>Attached</td>
<td>N/A</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

1 assumes minor losses in the 20" lines are 4% of the friction losses

2 assumes 6" gate valve fully open - USACE/EPRI Guide 1989

3 assumes 16" butterfly valve fully open - USACE/EPRI Guide 1989; t/d=0.15 & average of "Best Design" and "typical commercial valves"
### Solution:

**Energy Eqn from 1 to 4 (South Crow Line):**

\[ Z_1 = H_4 + \lambda_{\text{friction}} + h_{\text{minor}} \]

\[ Z_2 = H_4 + \lambda_{\text{friction}} + h_{\text{minor}} \]

\[ Z_3 = H_4 + \lambda_{\text{friction}} + \lambda_{\text{minor}}Q_1^3 \]

Solving for \( Q_1 \):

| \( H_4 \) ASSUMED | \( \lambda_{\text{minor}} \) base | Darcy \( f \) z | \( \lambda_{\text{friction}} \) | \( \lambda_{\text{minor}} \) base | Darcy \( f \) z | \( \lambda_{\text{friction}} \) | \( Q_1 \) | Darcy \( f \) base | \( \lambda_{\text{friction}} \) base | \( \lambda_{\text{minor}} \) base | \( Q_2 \) |
|-------------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|
| CASE III          | 6804.96        | 0.0096         | 0.01850        | 0.3839         | 0.0101         | 0.01895        | 2.1471         | 11.30           | 0.0177         | 3.1043         | 0.1274         | 11.65           |
| CASE I            | 7003.90        | 0.0096         | 0.01885        | 0.3911         | 0.0101         | 0.019229       | 2.1791         | 7.00            | 0.0174         | 3.0215         | 0.1241         | -25.59          |
| CASE II           | 6800.66        | 0.0096         | 0.01885        | 0.3911         | 0.0101         | 0.019229       | 2.1791         | 7.00            | 0.0176         | 3.1056         | 0.1275         | 11.50           |
| CASE IV           | 0.0096         | 0.01849        | 0.3837         | 0.0101         | 0.018939       | 2.1462         | 0.017679       | 3.1062         | 0.1275         | 11.50           |

**Energy Eqn from 4 to 5 (South Line):**

\[ H_4 = Z_5 + \lambda_{\text{friction}} + h_{\text{minor}} \]

\[ H_4 = Z_5 + \lambda_{\text{friction}} + h_{\text{minor}} \]

\[ H_4 = Z_5 + \lambda_{\text{friction}}Q_1^2 + \lambda_{\text{minor}}Q_1^3 \]

Solving for \( Q_2 \):

<table>
<thead>
<tr>
<th>( H_4 ) ASSUMED</th>
<th>( \lambda_{\text{minor}} ) base</th>
<th>Darcy ( f ) base</th>
<th>( \lambda_{\text{friction}} ) base</th>
<th>( \lambda_{\text{minor}} ) base</th>
<th>Darcy ( f ) base</th>
<th>( \lambda_{\text{friction}} ) base</th>
<th>( Q_2 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>CASE I</td>
<td>0.028302</td>
<td>0.017115</td>
<td>0.017679</td>
<td>3.1056</td>
<td>0.1275</td>
<td>11.50</td>
<td></td>
</tr>
<tr>
<td>CASE II</td>
<td>9064.28</td>
<td>0.017692</td>
<td>3.1085</td>
<td>0.1276</td>
<td>11.32</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CASE III</td>
<td>0.017692</td>
<td>0.0182853</td>
<td>0.0099</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Energy Eqn from 4 to 3 (HECLA):**

\[ Z_5 = H_4 + \lambda_{\text{friction}} + h_{\text{minor}} \]

\[ Z_3 = H_4 + \lambda_{\text{friction}} + h_{\text{minor}} \]

\[ Z_3 = H_4 + \lambda_{\text{friction}}Q_1^2 + \lambda_{\text{minor}}Q_1^3 \]

Solving for \( Q_3 \):

### CASE I

Flow from line 1 and elevations from reservoirs ID1 & ID3 are known, determine flows in line 2 & 3 and the reservoir elevation at ID5.

### CASE III

All three reservoir water surface elevations are known, determine flows from each line. (\( H_4 \) is adjusted until conservation of flow discharge is satisfied.)

### CASE IV

Flow from line 1 and elevations from reservoirs ID3 & ID5 are know, determine flows in line 2 & 3 and the required elevation at which pressure flow begins to satisfy elevations at ID3 & ID5.

### CASE II

Maximum capacity from HECLA to Round Top WTP with S. Crow closed.

### CASE IV

Maximum capacity from S Crow to Round Top WTP with HECLA closed.
### CASE III Calculation of Darcy Weisbach Friction Factor

<table>
<thead>
<tr>
<th>Pipe ID</th>
<th>Capacity (cfs)</th>
<th>Vel (fps)</th>
<th>Absolute Roughness</th>
<th>Kinematic Viscosity</th>
<th>Reynolds Number</th>
<th>Darcy</th>
<th>Moody (1st iter.)</th>
<th>Moody (2nd iter.)</th>
<th>Moody (3rd iter.)</th>
<th>Moody (4th iter.)</th>
<th>Moody (5th iter.)</th>
<th>Moody (6th iter.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-2</td>
<td>16.00</td>
<td>11.30</td>
<td>8.09</td>
<td>0.00090</td>
<td>0.000001664</td>
<td>648479.9</td>
<td>6.750E-04</td>
<td>0.018903</td>
<td>0.018903</td>
<td>0.018903</td>
<td>0.018903</td>
<td>0.018903</td>
</tr>
<tr>
<td>2-4</td>
<td>14.00</td>
<td>11.30</td>
<td>10.57</td>
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<td>0.000001664</td>
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<td>0.019043</td>
<td>0.019043</td>
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<td>0.019043</td>
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</tr>
<tr>
<td>4-5</td>
<td>20.00</td>
<td>11.65</td>
<td>5.34</td>
<td>0.00885</td>
<td>0.000001664</td>
<td>534852.4</td>
<td>5.000E-04</td>
<td>0.017775</td>
<td>0.017769</td>
<td>0.017769</td>
<td>0.017769</td>
<td>0.017769</td>
</tr>
<tr>
<td>3-4</td>
<td>20.00</td>
<td>0.35</td>
<td>0.16</td>
<td>0.00885</td>
<td>0.000001664</td>
<td>16068.9</td>
<td>5.000E-04</td>
<td>0.028187</td>
<td>0.028187</td>
<td>0.028187</td>
<td>0.028187</td>
<td>0.028187</td>
</tr>
</tbody>
</table>

**Darcy Weisbach f value is found using Swamee and Jain formula (attached).**

Darcy Weisbach f value from Swamee and Jain formula is iterated using Colebrook and White Eqn for a better approximation of the f value.
APPENDIX D

Diversion Structure Stability
Analysis
South Crow Diversion Space
Structure Stability Analysis

0.1 Project Description

(See Main Report)

0.2 Objective

The objective was to perform a structural stability analysis of the existing concrete gravity dam to determine if it was safe or in need of rehabilitation for safety reasons. The modeling assumptions for the South Crow Diversion dam include material properties for the concrete dam and foundation rock, evaluation criteria, and static and dynamic loads. The individual loads were combined to simulate the usual, unusual and extreme loading conditions.

0.3 Material Properties

The concrete material properties used in these analyses are based on data published by the U.S Bureau of Reclamation, Federal Energy Regulatory Commission and American Concrete Institute.

0.4 Foundation Material Properties

The effective cohesion for the dam foundation was assumed to be 15 psi. (See geotechnical investigation results) The effective internal friction angle was assumed to be 45 degrees. (See geotechnical investigation results)

0.5 Evaluation Criteria

The safety of South Crow Diversion Dam was evaluated for sliding-stability, overturning, and floatation. The bearing pressures were assumed adequate since the structure has not settled in 90 years due to good rock foundation.

Sliding Stability was computed using the following equation as given by the Design of Gravity Dams USBR 1976 A water resources publication.

\[ F.S. = \frac{C \times A + (W-U) \times \tan \phi}{V} \]

\[ C = \text{cohesion} \]
\[ A = \text{area in compression} \]
\[ W = \text{weight} \]
\[ U = \text{uplift} \]
\[ V = \text{shear} \]
\[ \phi = \text{angle of friction} \]

Overturning Stability F. S. was computed by computing moments at the toe.

\[ F.S. = \frac{\text{Sum of resisting M}}{\text{Sum of overturning M}} \]

Flotation F.S. is a ratio of weight to uplift
South Crow Diversion Space
Structure Stability Analysis

F.S. = Summation of Weight
      Summation of Uplift

0.6 Factors of Safety

The minimum allowable sliding factors of safety were developed based on FERC Criteria as recommended by the Wyoming State Engineer's Office regarding low-hazard potential dams.

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Usual</td>
<td>2.0</td>
</tr>
<tr>
<td>Unusual</td>
<td>1.25</td>
</tr>
<tr>
<td>Extreme</td>
<td>1.0</td>
</tr>
</tbody>
</table>

0.7 Loads

The stability of the dam (original and proposed rehabilitation alternatives) was analyzed for static and dynamic loads. The static loads included gravity, normal and flood reservoir elevations, sediment, and ice loading, uplift, and normal and assumed tailwater elevations. The dynamic loads used pseudo-static analysis methods to evaluate the effects of the MCE.

The gravity load used a unit weight of 150 lb/ft³ for existing concrete. The submerged sediment density was assumed to be 22.5 lb/ft³ horizontally and 57.5 lb/ft³ vertically. The hydrostatic loads corresponding to the normal and PMF flood reservoir levels were simulated using 62.4 lb/ft³. The maximum normal water surface for the reservoir was taken at an elevation of 7135 ft per hydrology analysis and flood routings. The maximum reservoir water surface due to the PMF condition was taken at an elevation of 7138 ft. The tailwater was calculated to be 4.15 feet of depth for the 100 year hydraulic event. (Per flood routing analysis)

Uplift was applied to the section for computing the sliding stability factor of safety along the dam/foundation contact. For a cracked base condition the uplift load was assumed to be equal to the full reservoir head within the cracked portion of the base, then vary linearly from the full reservoir pressure at the crack tip to the tailwater pressure at the downstream end.

The reference Advanced Dam Engineering Jansen 1988 incorporated a direct solution to solve for crack length. The effect of drains was not considered.

An ice condition was developed using USBR design criteria from the Design of Small Dams USBR 1965. This was taken as 5000 lbs. per linear foot of contact between the ice and dam for an ice depth of 1 foot and applied at the dam crest.

The earthquake loading used the psuedo static method of analysis form the USBR Design of Small Dams as well as the Uniform Building Code (UBC 94). The region is categorized as Zone 1 for earthquakes. This results in a value of 0.052 for the horizontal ground acceleration. Hydrodynamic loading was considered in the design.

0.8 Load Combinations

The load combinations used to evaluate the behavior of the dam are summarized below

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Usual</td>
<td>Normal water surface, dead load, silt, ice, uplift, tailwater</td>
</tr>
<tr>
<td>Unusual</td>
<td>Maximum water surface, dead load, silt, tailwater, uplift</td>
</tr>
<tr>
<td>Extreme</td>
<td>Usual loading, MCE</td>
</tr>
</tbody>
</table>
South Crow Diversion Structure Stability Analysis

1.9 Stability Results

The South Crow Diversion Dam requires some modifications as noted below under the column labeled “orig” (original dam configuration without accounting for deterioration) to be assured stable and meet the required guidelines. Alternative one is to reface with an overlay the downstream side of the dam. Alternative two is to reface with an overlay the upstream face of the dam. Both rehabilitation alternatives included a 3-foot raise of the crest. The factors of safety are summarized below for the original condition, the downstream overlay alternative and the upstream overlay alternative. Please see attached cross sections and calculations for the aforementioned alternatives (options one and two).

### Downstream Overlay Alternative as Option One

<table>
<thead>
<tr>
<th>Sliding stability</th>
<th>Flotation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Orig</td>
</tr>
<tr>
<td>Usual</td>
<td>1.62</td>
</tr>
<tr>
<td>Unusual</td>
<td>1.53</td>
</tr>
<tr>
<td>Extreme</td>
<td>1.36</td>
</tr>
</tbody>
</table>

### Upstream Overlay Alternative as Option Two

<table>
<thead>
<tr>
<th>Sliding stability</th>
<th>Flotation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Alter 2</td>
</tr>
<tr>
<td>Usual</td>
<td>2.03</td>
</tr>
<tr>
<td>Unusual</td>
<td>1.86</td>
</tr>
<tr>
<td>Extreme</td>
<td>1.89</td>
</tr>
</tbody>
</table>
LOAD CONDITION

ALTERNATIVE ONE
DOWNSTREAM OVERLAY

EL. 7135.00 (RAISED CREST)
EL. 7132.00 (EXISTING CREST)

EXISTING DIS FACE (≈ H:V)

NEW DIS FACE

H = 45°

7.5' 18.7' 4.5'
24.2' 30.7'

EL VARIES

MAX. ASSUMES

H = 45°
EVALUATE STABILITY OF DAM WITH NEW U/S SECTION

EVALUATE NEW CONCRETE SECTION FOR SLIDING, OVERTURNING, FLOATATION & STRESS
EASE FOR UNUSUAL EXTREME LOADING CONDITIONS

ALTERNATIVE TWO
UPSTREAM OVERLAY
### Summary

1. To satisfy all stability requirements, add 6" overlap of new concrete as shown on Figure 1 on Sheet 11.

2. Per P23, Dam crest to be raised 3'-0" to El. 7135.00

<table>
<thead>
<tr>
<th>Load Condition</th>
<th>Sliding Factor of Safety</th>
<th>Flotation Factor of Safety</th>
<th>Resultant Location/Base in Compression</th>
</tr>
</thead>
<tbody>
<tr>
<td>Actual</td>
<td>Required</td>
<td>Actual</td>
<td>Required</td>
</tr>
<tr>
<td>Usual</td>
<td>2.02</td>
<td>2.00</td>
<td>2.53</td>
</tr>
<tr>
<td>Unusual</td>
<td>1.80</td>
<td>1.25</td>
<td>2.19</td>
</tr>
<tr>
<td>Extreme</td>
<td>1.90</td>
<td>1.00</td>
<td>2.40</td>
</tr>
</tbody>
</table>
Figure 1
USUAL LOAD CONDITION

NWS EL 7136.00

ICE THICKNESS 1.0'

F

EL. 7136.00 (RAISED CREST)

EL. 7132.00 (EXISTING CREST)

3.5'

EL. 7124.00

EXISTING DIS FACE (0.010H:1V)

NEW DIS FACE

Wcb

0.725

F

F

F

Wcb

EL. 7092.00

30.9'

26.2'

18.7'

7.6'

4.5'

0(TDS)

UPRIFT
Project: South Crow | Job No: 3714 | Sheet #: 3
Feature: Diversion Dam | By: DJB | Date: 01/07/00
Detail: Design Loads - Maximum Reservoir Water Surface Elevation | Child By: 
File: South Crow (Usual Loading)

1. Thrust Block Cross-section:
   (a) Overall Dimensions, Elev., Width, & Crest Station:
   - Top Elevation = 7135.000 feet
   - Section Elevation = 7092.000 feet
   - Dam's Base Elevation = 7092.000 feet
   - Top Thickness = 7.500 feet
   - Bottom Thickness, b = 30.700 feet
   - Upstream Slope (Horiz./Vert.) = 0.000
   - Upstream Slope Top Elevation = 7092.000 feet
   - Downstream Slope (Horiz./Vert.) = 0.725
   - Downstream Slope Top Elevation = 7124.000 feet
   - Width of section = 1.000 feet
   - Length Conversion Factor = 12.000 inches

2. Material Properties:
   - Concrete Unit Weight = 150.000 pcf
   - Water Unit Weight = 62.400 pcf
   - Ice Force = 5000.000 lbs./lin.ft.
   - Silt (horizontal) Unit Weight = 22.500 pcf
   - Silt (vertical) Unit Weight = 57.500 pcf
   - Foundation Cohesion = 15.000 psi
   - Internal Friction Angle = 45.000 degrees
   - Allowable Tensile Strength of Concrete = 396.377 psi
   - Allowable Compressive Strength of Concrete = 2082.500 psi
   - Safety Factor = 1.000

3. Reservoir and Tailwater Elevations:
   - Reservoir Water Surface Elevation = 7135.000 feet
   - Reservoir Silt Surface Elevation = 7102.000 feet
   - Tailwater Elevation = 7092.000 feet
   - Thickness of Ice = 1.000 feet

4. Earthquake Parameters:
   - Z Seismic Zone Factor = 0.000 | UBC 1994
   - I Seismic Importance Factor = 1.000 | UBC 1994
   - C Numerical Coefficient = 2.750 | UBC 1994
   - Rw Numerical Coefficient = 4.000 | UBC 1994
   - Horizontal Ground Acceleration = 0.000 g
   - Vertical Ground Acceleration = 0.000 g
     - 1/2 of horizontal ground acceleration

5. External Forces on Thrust Block:
   i) Thrust Block Gravity Load:
      (a) Gravity Forces:
      - Dead Load = -104055 lbs.
      - Measured from Dam Toe Centroid, x = 20.81 feet
      - Measured from Dam Base Centroid, y = 15.70 feet
      - Resisting Moment @ Toe = -2164890 lb.-ft.
      - Resisting Moment @ Center = -567646 lb.-ft.
ii) Reservoir Hydrostatic Pressure due to Water, Silt, and Ice:

(a) Reservoir & Silt Gravity Forces:
Dead Load = 0 lbs.
Centroid, x = 0.00 feet

(b) Reservoir, Silt & Ice Hydrostatic Forces:
Thrust Load = 63814 lbs.
Centroid, y = 16.41 feet
Hydrostatic Water Load = 57689 lbs.
Hydrostatic Silt Load = 1125 lbs.
Ice Load = 5000 lbs.

Combined, Hydrostatic, Silt & Ice Moments:
Overturning Moment @ Toe = 1046872 lb.-ft.
Overturning Moment @ Center = 1046872 lb.-ft.

(d) Vertical Reservoir & Silt Pseudo-static Forces:
Vertical Inertia Load = 0 lbs.
Overturning Moment @ Toe = 0 lb.-ft.
Overturning Moment @ Center = 0 lb.-ft.

(e) Horizontal Reservoir, & Silt Pseudo-static Forces:
Horizontal Force = 0 lbs.
Overturning Moment @ Toe = 0 lb.-ft.
Overturning Moment @ Center = 0 lb.-ft.

iii) Tailwater Hydrostatic Pressure:
(a) Tailwater Gravity Forces:
Dead Load = 0 lbs.
Centroid, x = 0.00 feet

(b) Tailwater Hydrostatic Forces:
Thrust Load = 0 lbs.
Centroid, y = 0.00 feet

(e) Tailwater Moments:
Overturning Moment @ Toe = 0.00E+00 lb.-ft.
Overturning Moment @ Center = 0.00E+00 lb.-ft.
(d) Vertical Tailwater Pseudo-static Forces:

Vertical Inertia Load = 0 lbs.
Overturning Moment @ Toe = 0 lb.-ft.
Overturning Moment @ Center = 0 lb.-ft.

(c) Horizontal Tailwater Pseudo-static Forces:

Horizontal Inertia Load = 0 lbs.
Centroid, y = 0.00 feet
Overturning Moment @ Toe = 0 lb.-ft.
Overturning Moment @ Center = 0 lb.-ft.

iv) Uplift Forces:

Pressure @ Upstream Heel, u1 = 18.63 psi
Pressure @ Downstream Toe, u2 = 0.00 psi
Total Hydrostatic Force = 41187 lbs.

Moment about Toe = 842963 lb.-ft.

6. Stress Analysis:

Sum Thrust Forces, V = 63814 lbs.
Sum Vertical Force, P = -104055 lbs.
Area, A = 4421 in.²
Sum Moment (center), M = 5750712 lb.-in.
Sum Moment (toe), M = -13416219 lb.-in.
Base Centroid (from toe), c = 184.200 inches
Base Moment of Inertia, I = 49998718 in.⁴

Upstream Stress = (P/A) + (Mc/I) = -2.35 psi Compression
Downstream Stress = (P/A) - (Mc/I) = -44.72 psi Compression

7. Cracked Base Analysis:

\[ \text{total base width} = B \quad 368.400 \text{ inches} \]
\[ \text{sum of moments at the toe excluding uplift} = Mo \quad 13416219 \text{ lb.-in.} \]
\[ \text{sum of vertical forces excluding uplift} = V \quad 104055 \text{ lbs.} \]

Find base width in compression = b 31.2576 feet
Find crack length = c -0.56 feet Not a Cracked section
J. Stability Analysis:

1. Sliding-Stability:

\[ \text{F.S.} = \frac{C A + (W - U) \tan\text{(internal friction)}}{V} \]

\[ \frac{66312}{62868} + 63814 \]

F.S. = 2.02  \text{Good}

2. Overturning Moment:

\[ \text{Sum Resisting Moment(toe)} \]

\[ \text{F.S.} = \frac{\text{Sum Overturning Moment(toe)}}{2164890} \]

\[ \frac{1889835}{1889835} \]

F.S. = 1.15  \text{Good}

3. Floatation:

\[ \text{Summation Weight} \]

\[ \text{F.S.} = \frac{\text{Summation Uplift}}{104055} \]

\[ \frac{41187}{41187} \]

F.S. = 2.53  \text{Good}
UNUSUAL LOAD CONDITION

PMF
EL. 7038.00

ESTIMATED (CONC) VOLUME OF WATER ON CREST DURING OVERTOPPING
EL. 7135.00 (RAISED CREST)

EL. 7132.00 (EXISTING CREST)

SILT
EL. 7102.00

F_{SILT}

F_{WATER 1}

F_{WATER 2}

EXISTING C/FACE (HORIZONTAL)

NEW C/FACE

W_{C/S}

W_{C/S}

\( F_{WATER} \)

\( W_{SILT} \)

\( W_{FILL} \)

7.5'

18.7'

4.5'

26.2'

30.9'

Q_{(TOL)}

EL. 7042.00

EL. 7098.15

F_{FILLWAT}

F_{SILTWAT}

\( 3744-01 \)

\( M.A. \)

\( 7 \)
[1]. Thrust Block Cross-section:
   (a) Overall Dimensions, Elev., Width, & Crest Station:
      - Top Elevation = 7135.000 feet
      - Section Elevation = 7092.000 feet
      - Dam's Base Elevation = 7092.000 feet
      - Top Thickness = 7.500 feet
      - Bottom Thickness, b = 30.700 feet
      - Upstream Slope (Horz./Vert.) = 0.000
      - Upstream Slope Top Elevation = 7092.000 feet
      - Downstream Slope (Horz./Vert.) = 0.725
      - Downstream Slope Top Elevation = 7124.000 feet
      - Width of section = 1.000 feet
      - Length Conversion Factor = 12.000 inches

[2]. Material Properties:
   - Concrete Unit Weight = 150.000 pcf
   - Water Unit Weight = 62.400 pcf
   - Ice Force = 5000.000 lbs./lin.ft.
   - Silt (horizontal) Unit Weight = 22.500 pcf
   - Silt (vertical) Unit Weight = 57.500 pcf
   - Foundation Cohesion = 15.000 psi
   - Internal Friction Angle = 45.000 degrees
   - Allowable Tensile Strength of Concrete = 396.377 psi
   - fct=6.7*(f'c)*.5, worst case=0
   - Allowable Compressive Strength of Concrete = 2082.500 psi
   - Fo = .85*(.7)*f'c
   - Safety Factor = 1.000

[3]. Reservoir and Tailwater Elevations:
   - Reservoir Water Surface Elevation = 7138.000 feet
   - Reservoir Silt Surface Elevation = 7102.000 feet
   - Tailwater Elevation = 7096.150 feet
   - Thickness of Ice = 0.000 feet

[4]. Earthquake Parameters:
   - Z Seismic Zone Factor = 0.000
   - I Seismic Importance Factor = 1.000
   - C Numerical Coefficient = 2.750
   - Rw Numerical Coefficient = 4.000
   - Horizontal Ground Acceleration = 0.000 g
   - Vertical Ground Acceleration = 0.000 g
   - 1/2 of horizontal ground acceleration

[5]. External Forces on Thrust Block:
   i) Thrust Block Gravity Load:
      (a) Gravity Forces:
      - Dead Load = -104757 lbs.
      - Measured from Dam Toe Centroid, x = 20.85 feet
      - Measured from Dam Base Centroid, y = 15.70 feet
      - Resisting Moment @ Toe = -2184687 lb.-ft.
      - Resisting Moment @ Center = -576667 lb.-ft.
      - Dead Load Area 1 = 0 lbs.
      - Dead Load Area 2 = -48375 lbs.
      - Dead Load Area 3 = -55680 lbs.
      - Dead Load Area 4 = -702 lbs.
ii) Reservoir Hydrostatic Pressure due to Water, Silt, and Ice:

(a) Reservoir & Silt Gravity Forces:

- Dead Load = 0 lbs.
- Centroid, x = 0.00 feet

Water Dead Load = 0 lbs.
Silt Dead Load = 0 lbs.
Water, centroid, x = 0.000 feet
Silt, centroid, x = 0.000 feet

(b) Reservoir, Silt, & Ice Hydrostatic Forces:

- Thrust Load = 66863 lbs.
- Centroid, y = 15.01 feet

Hydrostatic Water Load 1 = 8050 lbs.
Hydrostatic Water Load 2 = 57689 lbs.
Hydrostatic Silt Load = 1125 lbs.
Ice Load = 0 lbs.

Combined Hydrostatic, Silt, & Ice Moments:

- Overturning Moment @ Toe = 1003689 lb.-ft.
- Overturning Moment @ Center = 1003689 lb.-ft.

(d) Vertical Reservoir & Silt Pseudo-static Forces:

- Vertical Inertia Load = 0 lbs.
- Overturning Moment @ Toe = 0 lb.-ft.
- Overturning Moment @ Center = 0 lb.-ft.

(e) Horizontal Reservoir & Silt Pseudo-static Forces:

- Horizontal Force = 0 lbs.
- Overturning Moment @ Toe = 0 lb.-ft.
- Overturning Moment @ Center = 0 lb.-ft.

iii) Tailwater Hydrostatic Pressure:

(a) Tailwater Gravity Forces:

- Dead Load = -390 lbs.
- Centroid, x = 1.00 feet

(b) Tailwater Hydrostatic Forces:

- Thrust Load = -537 lbs.
- Centroid, y = 1.38 feet

(c) Tailwater Moments:

- Overturning Moment @ Toe = -1.13E+03 lb.-ft.
- Overturning Moment @ Center = 4.85E+03 lb.-ft.
South Crow Diversion Dam

Design Loads - Maximum Reservoir Water Surface Elevation

**South Crow (Unusual Loading)**

(4) **Vertical Tailwater Pseudo-static Forces:**

- Vertical Inertia Load = 0 lbs.
- Overturning Moment @ Toe = 0 lb.-ft.
- Overturning Moment @ Center = 0 lb.-ft.

(5) **Horizontal Tailwater Pseudo-static Forces:**

- Horizontal facia Load = 0 lbs.
- Centroid, y = 0.00 feet
- Overturning Moment @ Toe = 0 lb.-ft.
- Overturning Moment @ Center = 0 lb.-ft.

iv) **Uplift Forces:**

<table>
<thead>
<tr>
<th>Pressure @ Upstream Heel, u1 - u2</th>
<th>For a Non-Cracked Section</th>
<th>For a Cracked Section</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>18.14 psi</td>
<td>0.00 psi</td>
</tr>
<tr>
<td>Pressure @ Downstream Toe, u2</td>
<td>1.80 psi</td>
<td>1.80 psi</td>
</tr>
<tr>
<td>Total Hydrostatic Force</td>
<td>40086 lbs.</td>
<td>48036</td>
</tr>
<tr>
<td>Moment about Toe</td>
<td>942452 lb.-ft.</td>
<td>0 lbs.</td>
</tr>
</tbody>
</table>

[6]. **Stress Analysis:**

- Sum Thrust Forces, V = 66326 lbs.
- Sum Vertical Force, P = -105147 lbs.
- Area, A = 4421 in.²
- Sum Moment (center), M = 5182421 lb.-in.
- Sum Moment (toe), M = -14185578 lb.-in.
- Base Centroid (from toe), c = 184.200 inches
- Base Moment of Inertia, I = 49998718 in.⁴

Upstream Stress = \( \frac{P}{A} + \frac{M_{c}}{I} \) = -4.69 psi Compression

Downstream Stress = \( \frac{P}{A} - \frac{M_{c}}{I} \) = -42.88 psi Compression

[7]. **Cracked Base Analysis:**

- total base width = B 368.400 inches
- sum of moments at the toe excluding uplift = Mo 14185578 lb.-in.
- sum of vertical forces excluding uplift = V 105147 lbs.
- Find base width in compression = b 32.8921 feet
- Find crack length = c -2.19 feet Not a Cracked section
1. Sliding-Stability:

\[ \text{F.S.} = \frac{C A + (W - U) \tan(\text{internal friction})}{V} \]

\[ \begin{align*}
66312 & + 57111 \\
66326 & \\
\text{F.S.} & = \frac{66326}{66326} \\
\text{F.S.} & = 1.86 \quad \text{Good}
\end{align*} \]

2. Overturning Moment:

\[ \text{F.S.} = \frac{\text{Sum Resisting Moment(toe)}}{\text{Sum Overturning Moment(toe)}} \]

\[ \begin{align*}
2184687 & \\
1945007 & \\
\text{F.S.} & = \frac{2184687}{1945007} \\
\text{F.S.} & = 1.12 \quad \text{Good}
\end{align*} \]

2. Floatation:

\[ \text{F.S.} = \frac{\text{Summation Weight}}{\text{Summation Uplift}} \]

\[ \begin{align*}
105147 & \\
48036 & \\
\text{F.S.} & = \frac{105147}{48036} \\
\text{F.S.} & = 2.19 \quad \text{Good}
\end{align*} \]
[1]. Thrust Block Cross-section:
   (a) Overall Dimensions, Elev., Width, & Crest Station:
      Top Elevation = 7135.000 feet
      Section Elevation = 7092.000 feet
      Dam's Base Elevation = 7092.000 feet
      Top Thickness = 7.500 feet
      Bottom Thickness, b = 30.700 feet
      Upstream Slope (Horz./Vert.) = 0.725
      Upstream Slope Top Elevation = 7092.000 feet
      Downstream Slope Top Elevation = 7124.000 feet
      Width of section = 1.000 feet
      Length Conversion Factor = 12.000 inches

[2]. Material Properties:
   Concrete Unit Weight = 150.000 pcf
   Water Unit Weight = 62.400 pcf
   Ice Force = 5000.000 lbs./lin.ft.
   Silt (horizontal) Unit Weight = 22.500 pcf
   Silt (vertical) Unit Weight = 57.500 pcf
   Foundation Cohesion = 15.000 psi
   Internal Friction Angle = 45.000 degrees
   Allowable Tensile Strength of Concrete = 396.377 psi
   Allowable Compressive Strength of Concrete = 2082.500 psi
   Safety Factor = 1.000

[3]. Reservoir and Tailwater Elevations:
   Reservoir Water Surface Elevation = 7135.000 feet
   Reservoir Silt Surface Elevation = 7102.000 feet
   Tailwater Elevation = 7092.000 feet
   Thickness of Ice = 0.000 feet

[4]. Earthquake Parameters:
   Z Siesmic Zone Factor = 0.075 UBC 1994
   I Siesmic Importance Factor = 1.000 UBC 1994
   C Numerical Coefficient = 2.750 UBC 1994
   Rw Numerical Coefficient = 4.000 UBC 1994
   Horizontal Ground Acceleration = 0.052 g
   Vertical Ground Acceleration = 0.026 g
   1/2 of horizontal ground acceleration

[5]. External Forces on Thrust Block:
   i) Thrust Block Gravity Load:
      (a) Gravity Forces:
         Dead Load = -104055 lbs.
         Measured from Dam Toe Centroid, x = 20.81 feet
         Measured from Dam Base Centroid, y = 15.70 feet
         Resisting Moment @ Toe = -2164890 lb.-ft.
         Resisting Moment @ Center = -567646 lb.-ft.
Project: South Crow Diversion Dam
Feature: Design Loads - Maximum Reservoir Water Surface Elevation
Detail: South Crow (Extreme loading)

(b) Vertical Gravity Inertia Forces:
- Vertical Inertia Load = 2683 lbs.
- Overturning Moment @ Toe = 55814 lb.-ft.
- Overturning Moment @ Center = 14635 lb.-ft.

(c) Horizontal Gravity Inertia Forces:
- Horizontal Inertia Load = 5365 lbs.
- Overturning Moment @ Toe = 84252 lb.-ft.
- Overturning Moment @ Center = 84252 lb.-ft.

ii) Reservoir Hydrostatic Pressure due to Water, Silt, and Ice:
(a) Reservoir & Silt Gravity Forces:
- Dead Load = 0 lbs.
- Centroid, x = 0.00 feet
- Water Dead Load = 0 lbs.
- Silt Dead Load = 0 lbs.
- Water, centroid, x = 0.000 feet
- Silt, centroid, x = 0.000 feet

(b) Reservoir, Silt, & Ice Hydrostatic Forces:
- Thrust Load = 58814 lbs.
- Centroid, y = 14.19 feet
- Hydrostatic Water Load = 57689 lbs.
- Hydrostatic Silt Load = 1125 lbs.
- Ice Load = 0 lbs.

Combined Hydrostatic, Silt, & Ice Moments:
- Overturning Moment @ Toe = 834372 lb.-ft.
- Overturning Moment @ Center = 834372 lb.-ft.

(d) Vertical Reservoir & Silt Pseudo-static Forces:
- Vertical Inertia Load = 0 lbs.
- Overturning Moment @ Toe = 0 lb.-ft.
- Overturning Moment @ Center = 0 lb.-ft.

(e) Horizontal Reservoir, & Silt Pseudo-static Forces:
- Horizontal Force = 3246 lbs.
- Overturning Moment @ Toe = 57366 lb.-ft.
- Overturning Moment @ Center = 57366 lb.-ft.

iii) Tailwater Hydrostatic Pressure:
(a) Tailwater Gravity Forces:
- Dead Load = 0 lbs.
- Centroid, x = 0.00 feet

(b) Tailwater Hydrostatic Forces:
- Thrust Load = 0 lbs.
- Centroid, y = 0.00 feet

(c) Tailwater Moments:
- Overturning Moment @ Toe = 0.00E+00 lb.-ft.
- Overturning Moment @ Center = 0.00E+00 lb.-ft.
(d) Vertical Tailwater Pseudo-static Forces:
- Vertical Inertia Load = 0 lbs.
- Overturning Moment @ Toe = 0 lb.-ft.
- Overturning Moment @ Center = 0 lb.-ft.

(e) Horizontal Tailwater Pseudo-static Forces:
- Horizontal Inertia Load = 0 lbs.
- Centroid, $y = 0.00$ feet
- Overturning Moment @ Toe = 0 lb.-ft.
- Overturning Moment @ Center = 0 lb.-ft.

iv) Uplift Forces:

- Pressure @ Upstream Heel, $u_1 = 18.63$ psi  
  For a Non-Cracked Section
- Pressure @ Downstream Toe, $u_2 = 0.00$ psi  
  For a Cracked Section
- Total Hydrostatic Force = 41187 lbs.  
  Moment about Toe = 842963 lb.-ft.

[6]. Stress Analysis:
- Sum Thrust Forces $V = 67425$ lbs.
- Sum Vertical Force, $P = -106738$ lbs.
- Area, $A = 4421$ in.$^2$
- Sum Moment (center), $M = 5075749$ lb.-in.
- Sum Moment (toe), $M = -13597035$ lb.-in.
- Base Centroid (from toe), $c = 184.200$ inches
- Base Moment of Inertia, $I = 49998718$ in.$^4$

  Upstream Stress = $(P/A) + (M/c/I) = -5.44$ psi  
  Compression

  Downstream Stress = $(P/A) - (M/c/I) = -42.84$ psi  
  Compression

[7]. Cracked Base Analysis:

- total base width = $B = 368.400$ inches
- sum of moments at the toe excluding uplift = $M_0 = 13597035$ lb.-in.
- sum of vertical forces excluding uplift = $V = 106738$ lbs.

  Find base width in compression = $b = 30.8706$ feet

  Find crack length = $c = -0.17$ feet  
  Not a Cracked section
2. Floating:

\[ \text{P.S.} = \frac{\text{Summation Weight}}{\text{Summation Uplift}} \]

\[ \text{P.S.} = \frac{101372}{41187} \]

\[ \text{P.S.} = 2.46 \quad \text{Good} \]
SUMMARY

- To satisfy all stability requirements, add 4'-0" thick concrete section to US face of dam w/ dowels into existing concrete.
- Per RJB, dam crest to be raised 3'-0" to EL 7135.00 - see Max cross section on page 1.

<table>
<thead>
<tr>
<th>LOAD CONDITION</th>
<th>SLIDING FACTOR OF SAFETY</th>
<th>FLATATION FACTOR OF SAFETY</th>
<th>RESULTANT LOCATION/ BASE IN COMPRESSION</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ACTUAL</td>
<td>REQUIRED</td>
<td>ACTUAL</td>
</tr>
<tr>
<td>Usual</td>
<td>2.03</td>
<td>2.00</td>
<td>2.59</td>
</tr>
<tr>
<td>Unusual</td>
<td>1.84</td>
<td>1.25</td>
<td>2.24</td>
</tr>
<tr>
<td>Extreme</td>
<td>1.89</td>
<td>1.00</td>
<td>2.59</td>
</tr>
</tbody>
</table>
1. Evaluate stability of dam with new U/S section

2. Evaluate max. cross section for sliding, overturning, flotation & stress

3. Base for usual, unusual, & extreme loading conditions
*Usual Load Condition:

\[ W_{c1} = (4.0')(43.0')(1.0')(150 \text{ lb/ft}^3) = 25,800 \text{ lb} \]
\[ W_{c2} = \left(\frac{1}{2}\right)(22.2')(32.0')(150 \text{ lb/ft}^3) = 53,280 \text{ lb} \]
\[ W_{c3} = (4.0')(43.0')(1.0')(150 \text{ lb/ft}^3) = 25,800 \text{ lb} \]
\[ F_w = \left(\frac{1}{2}\right)(62.4\text{ lb/ft}^3)(43.0')^2(1.0') = 57,690 \text{ lb} \]
* Usual Load Condition Cont.

\[ F_s = \frac{(9/2)(22.5 \text{ lb} / \text{ft}^3)(10.0')^2 (1.0')}{(30.2)(62.4 \text{ lb} / \text{ft}^3)(43.0')(1.0')} = 1.125 \text{ lb} \]

\[ U_t = (9/2)(30.2)(62.4 \text{ lb} / \text{ft}^3)(1.0') = 40,516 \text{ lb} \]

\[ F_s = \frac{(15,000 \text{ lb/ft})(1.0')}{(30.2)} = 5,000 \text{ lb} \]

\[ \text{CHECK SLIDING STABILITY:} \]

\[ F_s = \frac{cA + (W_t - U_t) \tan \phi}{E_f} \]

\[ W_t = 104,880 \text{ lb} \]

\[ U_t = 40,516 \text{ lb} \]

\[ \phi = 45^\circ \]

\[ c = 15 \text{ lb/ft} \]

\[ A = (30.2' \times 1.0')(12.4'/2) = 4,344 \text{ in}^2 \]

\[ E_f = 57,690 + 1,125 + 5,000 = 63,815 \text{ lb} \]

\[ \therefore F_s = \frac{151 \text{ lb/ft} \times 4,344 \text{ in}^2}{63,815} \times \tan 45^\circ = 2.03 > 2.00 \text{; OK IN SLIDING} \]

\[ \text{CHECK OVERTURNING STABILITY:} \]

\[ F_s = \frac{6 \left((EM_r)_a \right)}{\left((EM_o)_o \right)} \]

\[ (EM_r)_a = (25,800)(24.2) + (53,280)(2.3)(22.2) + (25,800)(28.2) \]

\[ = 2,140,464 \text{ ft-lb} \]
# Stability Analysis

- **Usual Load Condition Cont.:**

\[
\begin{align*}
\left( EM_s \right)_b &= (57,690)(43.0)(3) + (1,125)(10)(3) + (5,000)(42.5) \\
&\quad + (40,560)(30.2) = 1,858,862 \text{ lb} \\
\therefore \text{F.S.} &= \frac{2,140,444}{1,858,862} = 1.15
\end{align*}
\]

- **Check Floatation Stability:**

\[
\text{F.S.} = \frac{W_t}{U_t} = \frac{104,980}{40,511} = 2.59 > 2.00 \therefore \text{OK}
\]

- **Bearing Capacity / Stress Analysis:**

\[
\sigma = \frac{P}{A} + \frac{M_c}{I} = \frac{P}{A} + \frac{P_e c}{I}
\]

\[
P = W_t = 104,980 \text{ lb}
\]

\[
A = 30.2 \text{ ft}^2
\]

\[
I = (4/12)(1.0')^3(30.2')^3 = 2,295 \text{ ft}^4
\]

\[
c = \frac{30.2'}{2} = 15.1 \text{ ft}
\]

**Determine Location of Resultant:**

\[
\Sigma M_0: \quad R(x) + (57,690)(43.0/3) + (1,125)(10)/3 + (5,000)(42.5)
\]

\[
= 2,140,444 \quad \Rightarrow \quad x = \frac{1,097,324}{104,980} = 10.5'
\]

**Acceptable Range for x:**

\[
\frac{2}{3}(30.2) \leq x \leq \frac{1}{3}(30.2)
\]

\[
\rightarrow 20.1' \leq x \leq 10.1' \therefore \text{OK}
\]
* Usual load condition cont.

\[ e = \frac{30.2'}{-10.5'} = 4.6' \text{ towards toe} \]

\[ \sigma_{\text{TOE}} = \frac{104,880}{30.2} + \frac{(104,880)(4.6')(15.1')}{2,295} = 10,647 \text{ lb/ft}^2 = 46.2 \text{ lb/in}^2 \]

\[ \sigma_{\text{HEEL}} = \frac{104,880}{30.2} - \frac{(104,880)(4.6')(15.1')}{2,295} = 299 \text{ lb/ft}^2 = 2.1 \text{ lb/in}^2 \]

\[ \Rightarrow \text{Entire base is in compression} \]

46.2 psi
UNUSUAL LOAD CONDITION:

WEIGHTS & FORCES

\[ W_{c1} = 75,000 \text{ lb} \]
\[ W_{c2} = 53,280 \text{ lb} \]
\[ W_{c3} = 25,800 \text{ lb} \]

\[ W_d = \left( \frac{1}{2} \right)(8.0)(62.4)(4/12)(3.0)(1.0) = 749 \text{ lb} \]

\[ W_d = 104,880 \text{ lb} \]

\[ W_d = 105,629 \text{ lb} \]
UNUSUAL LOAD CONDITION CONT.

\[ F_{w1} = 57,090 \text{ lb} \]
\[ F_{w2} = (U_2 + \frac{1}{2})(3.0')(3.0')(1.0') = 8,050 \text{ lb} \]
\[ F_5 = 1,125 \text{ lb} \]
\[ U_1 = (U_2 + \frac{1}{2})(3.0')(3.0')(1.0') = 7,821 \text{ lb} \]
\[ U_2 = (\frac{1}{2})(3.0')(3.0')(4.15')(4.15')(1.0') = 47,254 \text{ lb} \]
\[ F_{w3} = (\frac{1}{2})(U_2 + \frac{1}{2})(4.15')^2(1.0') = 537 \text{ lb} \]

→ CHECK SLIDING STABILITY :

\[ F.S. = \frac{(15.1')(4.349')^2 + (105,629 - 47,254)}{\tan 45°} \]
\[ = \frac{57,090 + 8,050 + 1,125 - 537}{46,326} \]

\[ F.S. = 1.36 > 1.25 \therefore \text{OK} \]

→ CHECK OVERTURNING STABILITY :

\[ (\Sigma M_2) = 2,140,464 + (749)(30.2' - 3/3') + (537)(4.15/3) \]
\[ = 2,140,464 + 3,764 - 1,135 \]
\[ (\Sigma M_0) = (57,090)(43)(3) + (8,050)(43)(3) + (1,125)(30)(3) \]
\[ + (3,821)(30)(2) + (39,433)(2)(30)(2) = 1,915,730 \text{ lb\cdot in} \]

\[ \therefore F.S. = \frac{2,140,829}{1,915,730} = 1.13 \]
UNUSUAL LOAD CONDITION CONT.:

- CHECK FLOTAION STABILITY

\[ F.S. = \frac{105,629}{47,254} = 2.24 > 1.25 \therefore \text{OK} \]

- BEARING CAPACITY / STRESS ANALYSIS:

\[ P = 105,629 \text{ lb} \]
\[ A = 30.2 \text{ ft}^2 \]
\[ I = 2,295 \text{ ft}^4 \]
\[ c = 15.1 \text{ ft} \]

DETERMINE LOCATION OF RESULTANT:

\[ EM_0 = R(x) + (57,690)(43.0/3) + (8,060)(43.0/2) + (1,125)(10/3) \]
\[ = 2,164,829 \Rightarrow x = \frac{1,158,114}{105,629} = 10.96' \]

Acceptable Range: \(20.1' \leq x \leq 10.96'\) \therefore OK

\[ e = \frac{30.2' - 10.96'}{2} = 4.14' \text{ towards toe} \]

\[ \sigma_{\text{toe}} = \frac{105,629 + (105,629)(4.14)(15.1)}{30.2} = 6,395 \text{ lb/ft}^2 = 44.3 \text{ psi} \]

\[ \sigma_{\text{heel}} = \frac{2295}{2.295} = 6201 \text{ lb/ft}^2 = 43 \text{ psi} \]

\( \therefore \) ENTIRE BASE IS IN COMPRESSION
**EXTREME LOAD CONDITION:**

\[ F_3 = 1,125 \text{ lb (conservative - calculated as at-rest - now active - minus force is to not apply)} \]

\[ U_1 = 40,516 \text{ lb} \]

\[
\begin{align*}
W_{e1} &= 25,800 \text{ lb} \\
W_{e2} &= 53,280 \text{ lb} \quad 104,880 \text{ lb} \\
W_{e3} &= 25,800 \text{ lb} \\
F_w &= 57,690 \text{ lb}
\end{align*}
\]
0 EXTREME LOAD CONDITON CONT,

\[
W_{CA} = (0.052)(25,800 \text{ lb}) = 1,342 \text{ lb}
\]

\[
W_{C2} = (0.052)(53,280 \text{ lb}) = 2,771 \text{ lb}
\]

\[
W_{C3} = (0.052)(25,800 \text{ lb}) = 1,342 \text{ lb}
\]

\[F_{SE} = 1,125 \text{ lb} \quad \text{(VERY CONSERVATIVE - ACTUAL VALUE MUCH LESS FOR SEISMIC LOADING -}
\text{ SINCE FORCE IS SO SMALL, NO NEED TO PERFORM DETAILED CALC FOR THIS SMALL FORCE)}
\]

\[F_{EW} = \frac{(0.724)(1.0)}{\sqrt{(0.724)(0.735)(0.052)(0.416)(1.0)(43.0)(43.0)(1.0)}}
\]

\[= 3,201 \text{ lb} \quad \text{(SEE ATTACHED PHOTOCOPIES FOR REFERENCE TO ABOVE EQUATION)}
\]

\(\Rightarrow\) CHECK SLIDING STABILITY:

\[W_{t} = 104,880 \text{ lb}
\]

\[U_{t} = 40,514 \text{ lb}
\]

\[EF = 57,090 + 1,125 + 1,125 + (2)(1,342) + 2,771 + 3,201 = 68,594 \text{ lb}
\]

\[\Rightarrow F.S. = \frac{(15 \text{ lb/m}^2)(4,349 \text{ in}^2) + (104,880 - 40,514) \tan 45^\circ}{68,594}
\]

\[= 1.89 > 1.0 \Rightarrow \text{OK}
\]

\(\Rightarrow\) CHECK OVERTURNING STABILITY:

\[EM_{R} = 2,140,464 \text{ ft-lb} \quad \text{(PAGE 3)}
\]

\[EM_{O} = (57,090)(43.0/4) + (1,125)(10/3) + (40,514)(2/3)(30.2)
\]

\[+ (1,125)(2/3)(10) + (0.299)(102.4)(43.0^2)(1) + (2)(1,342)(43.0/2)
\]

\[+ (2,771)(32.0/3) = 1,797,848 \text{ ft-lb}
\]

\[\Rightarrow F.S. = \frac{2,140,464}{1,797,848} = 1.19
\]
<table>
<thead>
<tr>
<th>PROJECT</th>
<th>SOUTH CROW</th>
</tr>
</thead>
<tbody>
<tr>
<td>JOB NO.</td>
<td>37H-01</td>
</tr>
<tr>
<td>SHEET</td>
<td>11 OF</td>
</tr>
<tr>
<td>FEATURE</td>
<td>DIVERSION DAM</td>
</tr>
<tr>
<td>CHECKED BY</td>
<td>MLM</td>
</tr>
<tr>
<td>CHKD. BY</td>
<td>DATE 13/3/99</td>
</tr>
<tr>
<td>DETAILS</td>
<td>STABILITY ANALYSIS</td>
</tr>
</tbody>
</table>

- **EXTREME LOAD CONDITION CONT:**
  
  - CHECK FLOTAION STABILITY:
    
    - SAME AS USUAL LOAD CONDITION
  
  - BEARING CAPACITY STRESS ANALYSIS:
    
    - OVERTURNING MOMENT SLIGHTLY LESS THAN THAT CALCULATED FOR USUAL LOAD CONDITION (ECLI FORCE VS. SEISMIC FORCES) - RESISTING MOMENT SAME: BASED ON USUAL LOAD CONDITION CASES, ENTIRE BASE IS IN COMPRESSION FOR EXTREME LOAD CONDITION.
APPENDIX E

Geologic Survey Report and Foundation Condition Assessment
Dear Mr. Bergquist:

At your request a senior engineering geologist of Terracon performed a field observation of the above referenced site with Mr. Jim Swaisgood of ECI on July 13, 1999.

The project area lies within the Great Plains Physiographic Province characterized by a large belt of highlands with slope gradually eastward through the Rocky Mountains to the Central Lowlands. Structurally, the area lies within the Denver basin. The Denver basin consists of a series of nearly horizontal strata of late Tertiary rocks overlying unconformable rocks of Paleozoic and Mesozoic Age. These rocks have been downfolded into a large trough which was formed when the uplift of the Front Range occurred. Locally, the area is located in the "Gangplank" which is a narrow remnant of flat-lying late Tertiary rocks preserved along the east front of the Laramie Mountains. The Laramie Mountains are a direct continuation of the Colorado Front Range. These Tertiary rocks extend eastward to the Nebraska border. The narrow strip to Tertiary rocks provides a bridge between the surface of the high plains to the east and the uplands to the west. Bedrock underlying the area consists of Precambrian metasediments and Tertiary claystones and conglomerates. It is anticipated that bedrock underlies portions of the site at relatively shallow depth and is overlain by alluvial sands and gravel, and slope wash consisting of colluvium, and eolian deposits.

The majority of the site is underlain by PreCambrian metasediments consisting of schist and granite gneiss. The PreCambrian rock is in contact with the Tertiary White River Formation just east of the axis in the dam. The White River Formation consists of claystones and arkosic conglomerates. Alluvium and slope wash consisting of sand and gravel, colluvium, alluvial fan and eolian deposits of Pleistocene or Recent Age, overlay the bedrock in the project area.
The existing dam is a concrete structure approximately 40 feet high and 226 feet long. A concrete valve house is located along the east abutment. The dam has an overtopping spillway along with a pipe collection structure which carries some water from the spillway off the top of the dam into a 24-30 inch CMP pipe. The pipe empties into a small ditch along the west abutment north of the dam. The concrete in the dam is deteriorated and seepage was noted through the concrete in the dam. Vegetation was noted growing in the dam on the east abutment. Seepage was also noted at the toe of the dam, along the east abutment. The overflow outlet pipe at the top of the dam is leaking badly in several locations. The reservoir at the time of the site observation was full and water was overtopping the dam through the spillway. Water was noted pooling at the base of the dam.

The west abutment of the dam is keyed into a rock outcrop consisting of granitic rock interbedded with highly weathered schist. The bedrock is highly jointed. The east abutment appears to be in a small alluvial fan or on a gently eroded soil slope consisting of slope wash. The soil cover appears to be several feet in thickness. Rock outcrops of the same PreCambrian metasediments encountered on the west abutment were noted further up the slope of the east abutment. The dam is located in the widened part of a narrow valley along South Crow Creek. The valley has moderate to steep sides to the west and gently sloping soil covered slopes on the east side of the dam. Further to the west and south the canyon walls steepen. Outcrops were noted on both sides of the valley upstream from the dam. The rock in this area consists of PreCambrian metasediments. The bottom of the creek is heavily vegetated with grasses, brush and trees.

The attitude of several joint sets were measured at the time of field observation with a Brunton compass. One major joint set has a strike of approximately N23° to N27° and dips of 76° to 86° to the east and west. The second major joint set measured varied from N32° E to N36° E with dips ranging from 55° to the east to 86° to the west. The third joint set measured was N65° to 70° E with dips of between 70° east to near vertical.

It would appear soil cover up to 5 to 10 feet could be anticipated along the east abutment and bedrock at relatively shallow depths may be anticipated along the west abutment. The bedrock at the west abutment is highly weathered and jointed in places.

It appears repair of the existing dam or construction of a new dam either in the existing location or in a new location, upstream from the existing dam site will encounter minimal geologic hazards. It is anticipated the dam would extend to the bedrock at the abutments and in the bottom of the stream. It would appear some soil cover would be encountered along the east abutment and little soil cover would be
encountered along the west abutment of the existing dam site. Little soil cover is anticipated upstream from the existing dam in the area of a potential new dam site.

The depth to bedrock in the bottom of the stream is not known but it is anticipated it may be 10 to 30 feet below the bottom of the dam. Due to the jointing of the bedrock and the highly weathered schist encountered on the west abutment, grouting of the abutments and/or bottom of the dam foundation may be required to minimize seepage. Similar grouting requirements may be necessary for a new dam constructed upstream of the existing structure where bedrock is anticipated to be encountered near the surface along the abutments of the new dam. The depth to bedrock in the bottom of the stream, upstream from the existing dam, is anticipated to be at similar depths of 10 to 30 feet. If an earthened dam is considered to replace the existing structure, the claystone of the White River Formation and the overburden material adjacent to the dam would probably be suitable for use in construction of an earthen dam.

It is recommended that a geotechnical investigation be performed at the site prior to final design and repair of the existing dam or replacement of the structure.

If you have any questions regarding our observation and recommendations, please do not hesitate to contact us.

Sincerely,

TERRACON

Neil R. Sherrod, P.G.
Senior Engineering Geologist

Reviewed by:

Douglas Jobe, P.E.
Office Manager

Copies to: Addressee (3)
Interoffice Correspondence

To: Joe Bergquist, Project Manager  Date: March 20, 2000
From: Dave Ehler  File No: 3714 - reports
Subject: Cheyenne S. Crow Diversion Project - Pipeline

Requirement

Identification of the SCADA system to provide monitoring of the water quality and control water releases from South Crow Diversion was performed to complete the conceptual rehabilitation design. The preferred method of communications is via radio interface with the existing system. Alternative system is to utilize telephone if phone lines are available nearby.

Path Study

A path study was completed between various sites as listed below to see how the signal from the water treatment plant at Sherard could reach the South Crow Diversion site and be returned. This work was completed as part of Phase II scope of work. It was performed to identify what would be required for the future SCADA system. The investigation used the following NAD 27 State system coordinates. Conversions were made using U.S. Army Corps of Engineers, Corpscon for Windows Ver 5.11.03 and State Plane NAD 27 Wyoming I, U.S. Survey Feet.

<table>
<thead>
<tr>
<th>SITE NAME</th>
<th>ELEV.</th>
<th>COORDINATES NAD27</th>
<th>COORDINATES USGS</th>
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<tr>
<td></td>
<td></td>
<td>Easting</td>
<td>Northing</td>
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<tr>
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<td>494113</td>
<td>174269</td>
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<tr>
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<td>6755</td>
<td>498378</td>
<td>178508</td>
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<td>493011</td>
<td>168089</td>
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<tr>
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<td>6543</td>
<td>521840</td>
<td>181619</td>
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<tr>
<td>Round Top</td>
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<td>575242</td>
<td>189924</td>
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<tr>
<td>Sherard</td>
<td>6412</td>
<td>562001</td>
<td>171798</td>
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<tr>
<td>Buffalo Ridge Tank</td>
<td>6310</td>
<td>602510</td>
<td>186070</td>
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The study was performed with the RFCAD and a 1:100,000-map database. Paths were evaluated using 5-Watt transmitters with 10-foot high antenna for control sites and 20-foot high antenna for repeaters. Antennas for the study were omnidirectional.

All possible paths from South Crow were evaluated and selected paths were evaluated for completion of the South Crow and Hecla data circuits. Path maps and profile sheets were provided for all paths investigated. In the end the paths were separated into good, marginal, and obstructed and shown on Figure F1, F2, and F3 respectively and listed in the summary table below.

Site path summary table

<table>
<thead>
<tr>
<th>Site 1</th>
<th>Site 2</th>
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<th>Marginal</th>
<th>Obstructed</th>
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The conclusion to the path studies is that one needs to use the Copper King Repeater site. Copper King provides a number of paths back to the system including Crystal Dam, Round Top and Buffalo Ridge Tank.

Before selecting a marginal path, it should be field verified. These paths have a reasonable signal loss for reliable communications but appear to be obstructed. Slight relocation or additional antenna
height may provide a solid communications path. The obstruction may be on the edge of the pattern. The use of a gain Yaggi antenna would also improve signal strength and reliability.

**General Description**

Based on the path study and the identified control needs at the site the SCADA system for the project should include the following three sites:

1. The South Crow Dam site consisting of status and control of two valves, water quality and RTU with radio. The site is isolated without power or telephone.

2. The South Crow pipeline junction with the 20-inch south pipeline coming from the Hecla diversion site is an inline pipeline valve consisting of status and control of an isolation valve and RTU with radio. The Hecla junction valve site is along the road and near an AC power line.

3. Copper King Mine is the suggested store and forward repeater site.

This system’s coverage will provide for monitoring and control of both valve sites and will be integrated with the Cheyenne BPU’s SCADA System. The RTUs and radios shall be compatible or equal to the existing BPU equipment. Control will be integrated with the existing system.

The proposed communications will utilize 450 or 900 MHz radio with the probability of a repeater near Copper King Mine. The repeater could be solar powered, or with the proximity of the proposed power line for South Crow, AC powered.

The diversion site will need a small control building for the RTU, radio, power distribution and other equipment. The control building will be located above the flood plane. Other sites will utilize pole mounted NEMA 4 weatherproof enclosures. If security is of concern, small control buildings can be included.

Power for South Crow facility will be provided by a new single phase 14.4 Kilovolt power line (approximately 2.5 miles) following the proposed access road from the Hecla pipe line junction to the South Crow Diversion. Transformers are to be provided at the diversion and the junction with the Hecla pipeline. A possible third transformer might be required for the repeater near Copper King mine. Power requirements include two valve operators, heating and lighting, and radio, control and telemetry. Power requirements for the Hecla junction include one-valve operator, and radio, control and telemetry. All power transformers to have disconnects and lightning protection.

The telemetry and control (SCADA) system recommended utilized Motorola MOSCAD RTUs and Motorola Darcom 900 MHz Radios. The system will link with and be compatible with the BOPU SCADA System. A direct radio link to one of BOPU’s repeater or other site was preferred, but the terrain studies suggest a repeater, either solar/battery or AC/battery will be required. Preliminary studies indicate a very marginal path between South Crow Dam and Crystal Lake Reservoir Dam. (See Appendix E.) Therefore, a repeater at Copper King mine is recommended for reliable operation. This could also serve the Hecla site. A profile study was not done for this site, but the site is visible from the proposed repeater site. All paths should be verified prior to installation. Telephone should
be considered as a viable alternative if final studies can not provide verification of radio paths and repeater site is not available.

System Components

Main components of the proposed system include:

1. Wood pole powerline from pipeline junction with Hecla pipeline to South Crow Diversion Structure.

2. Two transformers a 14,400:120/240 15 KVA at the diversion and 14,400: 120/240 10 KVA at the Hecla junction, each with lightning protection and distribution panels with a possible third for the repeater.

3. Control, position and status circuits for valve operators, water level at diversion, water quality at the diversion and Hecla pipeline.

4. Motorola MOSCAD RTU’s for the South Crow Diversion and Hecla pipeline junction sites.

5. Motorola Darcom telemetry radios and repeater. Radio system to be in compliance with BOPU radio plan and FC requirements. Radio systems at the dam and Hecla junction should be battery operated with AC chargers. Solar power is suggested for the repeater.

Detailed Equipment And Material List (Electrical And SCADA)

1. AC Power And Power Line

   Power line approximately 15,000 feet long.

   At South Crow Diversion Structure
   Transformer, pole-mounted 14,400: 120/240-volt single-phase 15 KVA
   Lightning arrestors
   Indoor 120/240-volt single phase distribution panel

   Near the Hecla pipeline junction
   Tap and disconnect switch
   Transformer, pole-mounted 14,400: 120/240 single-phase 10 KVA
   Lightning arrestors
   Outdoor pole mounted 120/240-volt single-phase distribution panel

   Copper King Mine (optional site for repeater location)
   Transformer 14,400: 120/240-volt single-phase 10 KVA or less
   Lightning arrestors
   Outdoor pole mounted 120/240-volt single-phase distribution panel
2. Telemetry System

At South Crow Diversion
Motorola MOSCAD RTU system
Valve positioning shall utilize a valve setpoint system; direct control via the communications link is not acceptable. The system shall have the capability of making water level/quality automatic operations.
The system shall include the following:

- Water Quality – Hydrolab water quality (temp, conductance, pH, dissolved oxygen) sdi-12 interface
- Water level – analog, possibly bubbler.
- Guard Valve position – analog (optional)
- Control Valve position - analog
- Valve limits - Control Valve high and low limits, Guard Valve high and low limits
- Valve controls - Control Valve open and close, Guard Valve open and close
- Structure security alarm
- Loss of power status

Hecla pipeline junction
Valve positioning shall utilize a valve setpoint system; direct control via the communications link is not acceptable.
The system shall include the following:

- Nema 4 pole mounted enclosure
- Motorola MOSCAD RTU system
- Control Valve position - analog
- Valve limits - Control Valve high and low limits
- Valve controls - Control Valve open and close
- Structure security alarm
- Loss of power status

Copper King Mine repeater
Store and forward repeater.
The system shall include the following:

- Nema 4 pole mounted enclosure
- Structure security alarm
- Loss of power status

3. Radio System

Radio system to be in compliance with BOPU radio plan and FC requirements. Radios are integral components of the MOSCAD RTUs and will include the following:

- Motorola Darcom telemetry radio at the diversion with a Yaggi antenna
- Motorola Darcom telemetry radio at the Hecla pipeline junction with a Yaggi antenna
- Repeater at Copper King Mine with an Omni Antenna
- Coax, cables, lightning arrestors, and connectors as required at all sites
4. Miscellaneous Equipment

It is estimated that the following items will be required:

- Valve operators 1 to 2 HP, 240 Volt AC single Phase – at the diversion site
- Valve operators 2 to 5 HP, 240 Volt AC single Phase – at the Hecla pipeline junction
- Valve Position sensors – Part of valve operators
- Valve limit switches – Part of valve operators
- Water level sensor - Sutron Bubbler is suggested – in valve house
- Water Quality sensors - Hydrolab is suggested – in valve house
- Batteries and chargers – for radios and RTUs
- Loss of power sensor
- Intrusion switch
- Control bldg. heater and ventilation
- Equipment enclosure heater

Enclosures

The South Crow Diversion valve operators should be installed in the existing expanded structure above the 100-year flood level using long stems on the valves. The power, RTU and radio equipment should be installed in a new room constructed on the roof of the structure. Antenna should be installed on a pole above the roof. Solar panels, if this option is selected, should be installed low on the roof to minimize wind loading.

Hecla pipeline junction equipment shall be installed in a pole mounted Nema 4 enclosure.

The radio repeater shall be installed in a pole mounted Nema 4 enclosure.

Solar Alternative

A solar option is presented for South Crow Diversion. Solar operation option is viable for installations where occasional operational and corrections are required and constant level operation is not required. The operational concept is one in which the releases are not modulated, but rather open/close in nature.

This is a viable solution due to the availability of more efficient solar power systems and low voltage DC valve operators. Two independent solar/battery systems are required. This allows continued telemetry even with the depletion of the batteries for the valve operator.

This alternative requires the substitution of DC operators for the diversion guard valve and control valve. A 100-watt 12-volt solar panel battery system is required for the RTU, radio and telemetry equipment. A 200 watt 12 or 24 volt system is required for the valve operators. The repeater is solar operated using a 100-Watt system.
The Hecla Pipeline Junction would remain as an AC operated site due to the proximity of the 24.9/14.4-KV three-phase power line.

**Rough Cost Estimate**

See attached spreadsheets

**Comments and Answers**

A series of comments were generated by the final draft report on the proposed SCADA system. Each one was responded to and discussed with BOPU staff. The question and final acceptable answer has been incorporated into this report as Appendix F-1. The impact to this appendix was the additional path study work. It also required that the cost estimate for the entire project to increase approximately $10,000 to accommodate heaters and additional monitoring equipment.
Cost of Hardware for AC option

Power line for diversion 2.5 mi @ $20,000  $50,000
Regulator  $230
Battery Gel Cell  $126  $356
Regulator  $230
Battery  $250  $480
Solar panels for repeater
100 watt panels  $658
Mounting  $155
Regulator  $230
Battery Gel Cell  $126  $1,169
AC for Hecla  $1,000
DC power supply  $150
Battery Gel Cell  $126  $1,276
Valve operators
AC Operator (Auma) 3@  $2,200 each  $6,600
Motorola MOSCAD RTUs  3@  $2,000  $6,000
Programming  lot  $6,000
Radio systems 450 or 900 MHz to match available paths
Radio, SCADA 2@  $1,000  $2,000
Radio, repeater digital  $2,000  $2,000
Antenna, mast & coax 3@  $500  $1,500  $77,381
Labor, installation etc  $21,381  $98,762
Clear Signal Path Figure F-1
Marginal Signal Path Figure F-2
Obstructed Signal Path Figure F-3
APPENDIX - F1
CHEYENNE SOUTH CROW DIVERSION PROJECT
PHASE II FINAL REPORT
ANSWERS TO COMMENTS ON DRAFT VERSION

REVIEWER: CARY CHAPMAN, BOPU dated May 15, 2000
Questions indicated by numbers.

Comments to the review prepared by the ECI team. June 9 through June 20 follow each question as well as the results for extra work on this subject required to finish the final report.

SCADA

1. Were radio paths from South Crow to Roundtop, the new Sherard, and the Buffalo Ridge tank evaluated? These sites will all have radio by the time this project is constructed. They would appear to be better options since the signals do not have to go over the canyon walls that exist at the South Crow site.

Originally only a one radio path was investigated with the assistance of an outside source that had the mapping data to work with RFCAD. Table Mountain was obscured based on the coordinates used in that study. We did not have coordinates for other sites (Sherard and Buffalo ridge) nor the ability to complete a more comprehensive study at that time. Therefore, our report used information provided by the BOPU’s SCADA contractor.

Our preference is to do a more extensive study and we have made the arrangements to purchase the necessary mapping data to do the study. Coordinates of the possible alternative sites have been acquired and the study is underway. We are doing that comprehensive study on paths to existing site (repeaters and other stations) coordinates as provided by BOPU. The study was not, and will not be, field verified. Our work will be valuable on marginal paths and we will be able to make recommendation for the most direct selected paths.

We are using RFCAD, a radio path and propagation program based on radio theory and USGS map base.

Result of new work prior to finalizing the final report:
As shown in Figure 1, 2 and 3 all possible sites were evaluated. The result is the original conclusion is the same – a repeater is needed at Cooper King mine – for a good signal path.
2. Has the option of using VHF radio been evaluated? It could be added to the Table Mountain site and might allow getting the signal into the South Crow Canyon.

VHF radio is a little more forgiving and would improve a marginal UHF path. VHF due to the crowded spectrum may be harder to get frequency allocations and would typically have more noise and possible interference. This can be a consideration if existing licenses are available. We have asked about existing licenses and have not received a reply, if one is not received before the report is finalized we will state that VHF is not an option.

Result of new work prior to finalizing the final report:
As of the time this report was finalized, no new information on existing licenses has been received.

3. Were actual on site frequency checks done to determine the viability of the different radio frequencies or were the path surveys purely computer-based studies?

No on site checks were made. The original path studies were based on results done by others. We are doing additional studies and will be able to provide verification on any marginal or long paths.

Result of new work prior to finalizing the final report:
As noted above additional path studies were performed. Field verification will be left for the design phase.

4. Although radio is the preferred communication medium, the cost of adding a repeater site at the Copper King Mine just to serve this site may be cost prohibitive. Does the BOPU have access rights to this location or own the land? There are other options that should be explored as part of this study.

This site was suggested as a possibility by others. We did not have information on property rights or possible leased sites. BOPU offered to check this site's ownership status and report back to us. We will incorporate the results of those efforts into the final report, or make the statement that this information is required and BOPU will acquired it before a final decision is made on the use of a repeater at this site, if our study indicated this site should be used.

Result of new work prior to finalizing the final report:
No new information on ownership has been submitted for use in the final report. This will be settled during the design phase.
A. If grid power is to be brought into the South Crow site, what would it cost to bring in a data phone line and what is the monthly cost of operating the phone line?

We agree a dedicated telephone line is an option. We will investigate installation costs. We would also like to state that radio systems are less expensive to operate after the initial installation.

Result of new work prior to finalizing the final report:
ATT and US West were contacted. The quote received for this work was $11,500 per mile to string phone line on the proposed “existing” power transmission lines going into the site. Thus the repeater and radio option was maintained in the cost estimate as that option is more economical.

B. What would it cost to run fiber optic cable in our pipeline easement from the South Crow site to the Crystal reservoir site? A fiber to copper converter is available and would allow the Motorola RTU’s to communicate over the fiber. With this option there would be no monthly operating charge.

We realise this is also an option. We have recommended fiber optics in other applications. The costs have come down considerably. From our cost investigation we have determined that this system would run about $10 per liner foot including installation. We feel the radio system will be the system of choice for the City since fiber optics would require the use of a repeater at the junction of S. Crow pipeline with the Hecla pipelines, or installation of fiber lines all the way to the WTPs near Cheyenne.

C. Similar to using fiber in B. above, would copper be an option? If copper is used could tone generators be used to transmit the data from site to site?

Copper can be used although long lines cut into the available bandwidth. We do not recommend tone generator approach. Typically slow and much better solutions are available using copper wire.

5. Flow measurement at the South Crow site needs to be included.

Flow meters can be added. Most flow meters are not freeze tolerant so some heating or removing from them from the water is seasonally required. Heating can be made available with the AC power line option. The cost of such a development would be another $700 for the addition of heat and $1,500 for the addition of a flow meter to our total cost estimate.
6. Will electric heat be needed at the South Crow site to prevent freeze problems? If so this will preclude the solar option. If differential pressure flow meters are used, the heat will be required.

If AC is brought to the site heating would be provided. This option helps in the installation and maintaining water instruments water quality and flow during the cold season. This also lessens the problems with moisture in the equipment and control cabinets. As noted the additional cost is about $700, which we will add to our total cost estimate when we complete the final report.

7. Three additional digital points are recommended for the South Crow site. Water on the floor alarm, local-remote switch indication, and if heat is added, then a low temperature point will also be required.

The use of programmable equipment easily provides for more digital inputs. The water on floor, Local-remote, and low temperature/heat on can easily be provided. The cost of doing so is negligible, but we will mention these additions in our write-up.

8. If the solar option is used, then it is recommended that a propane generator be used for backup. An inverter to convert from DC to AC would allow using standard AC operators on the valves.

The propane generator with solar allows for more versatility in operation. It can be used for heating and AC gate Operators. The unit is started for gate operation and when heating is required. Reliability on starting is a concern. We have used something similar for canal check structures. Our preference is to use the AC grid power and install a heating unit.

The inverter to convert DC to AC is not recommended for two reasons:
- Inverters are inefficient, and
- The DC motor operators are designed for DC operation from batteries and have low power consumption. AC operators are designed for AC.
power and typically overpowered designed for fast operation and are low on power efficiency.

For any of these options involving a mix of power systems or DC only system the RTU and radio equipment would be on a separate battery system.

9. Costs for the RTU's is lower than what we have seen in the past. For a basic 6 slot RTU in an enclosure, with the radio, the costs have been around $5,500.

Rtu costs were based on information received from Motorola on equipment similar to existing BOPU equipment.
APPENDIX G

New “Lake Helen” Dam Analysis and Cost
APPENDIX G

SELECTION OF DAM TYPE AND TOTAL PROJECT COST ESTIMATE
CHEYENNE SOUTH CROW PROJECT
LAKE HELEN DAM SITE

INTRODUCTION
The work entailed an evaluation of the dam type appropriate for the New Upstream (Lake Helen) dam site given known and assumed site conditions, and the development of an appraisal level cost curve for the recommended dam type based on various dam heights. Site conditions were based on the Cheyenne South Crow Diversion Project, Phase I, Draft Report (ECI Project No. 3714-02, December 1999), and a topographic survey of the proposed dam axis by AVI in July 1999.

SUMMARY
From the analysis performed, the most appropriate and least expensive dam for the Lake Helen site is a Roller Compacted Concrete (RCC) dam. This dam has cost, construction, and operational advantages over the other dam types considered. A tentative dam section and topographic map of the project site is shown in Figure G-1.

Total project cost for an RCC dam at the Lake Helen site was estimated as described herein. Costs for a 30-, 50-, and 70-ft high dam are presented below. A curve relating cost to dam height is shown in Figure G-2. Dam height refers to the vertical distance between the proposed dam crest and the existing streambed. Prices reflect January 2000 dollars. We expect that these costs are within about 20% of actual project costs.

<table>
<thead>
<tr>
<th>Dam Height (Ft)</th>
<th>Total Project Cost (January 2000 Dollars)</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>920,000</td>
</tr>
<tr>
<td>50</td>
<td>1,620,000</td>
</tr>
<tr>
<td>70</td>
<td>2,490,000</td>
</tr>
</tbody>
</table>
SELECTION OF DAM TYPE

The dam type most suitable or economical for a site is usually difficult to select, and the process can require development of preliminary designs and cost estimating. Selection requires the consideration of each dam type with regard to site conditions, dam purpose, safety, economy, and other factors. Once acceptable safety criteria are met, construction cost is usually the overriding factor in the selection process.

The Lake Helen site is typical of sites where RCC, concrete arch, and concrete gravity sections have been built, but were also considered earthfill and rockfill sections for this work. Concrete gravity dams have been priced out of the market by RCC construction techniques over the last several years, and were therefore eliminated from further consideration.

RCC and concrete arch dam sections are typically the dams of choice where steep valley walls, narrow streambeds, thin overburden, and strong foundations are found; these are the conditions and anticipated conditions at the Lake Helen site. Earthfill dams are generally not built at such sites because of the lack of adequate or sufficient materials, requirements for river diversion, and construction sequencing. Rockfill dams, whether concrete-faced or not, can be built at such sites if sufficient materials are readily available, but also require significant river diversion. Steep canyon walls can lead to settlement problems for dams with cores.

RCC and concrete arch dams have certain economic and technical advantages over earthfill and rockfill dams. These include:

Less costs for water diversion during construction. Earthfill and rockfill structures are highly susceptible to major damage if overtopped during construction. In contrast, RCC and concrete arch sections will receive only minor damage in this event. Larger, more expensive diversion structures are needed for earthfill and rockfill dams.

Lower costs for spillways. Earthfill and rockfill dams generally have spillways located in abutments, separate from the dam embankments to reduce possible erosion problems. The RCC and concrete arch
sections themselves serve as a spillway, allowing flood waters to flow directly over the dams. In RCC structures, the spillway chute is stepped, and this stepped face dissipates much of the energy of discharges over the spillway. This results in a smaller energy dissipation structure at the toe of the dam than compared to other dam or spillway sections. Spillways incorporated directly into the dam body are generally much less expensive than spillways located in an abutment.

**Less risk of damage if spillway capacity is exceeded.** If the spillway capacity of an earth or rockfill dam is exceeded, the dam will be overtopped with a high risk of major damage or complete failure caused by the erosive action of the flowing water. RCC or concrete arch dams resist this erosion; damage that might result would probably not be catastrophic.

**Less susceptibility to internal erosion (piping).** Gradual piping of embankment materials has caused many problems in earthfill dams, and some of these events have resulted in total embankment failure. RCC and concrete arch sections are not susceptible to internal erosion.

**Less total volume of material required for construction.** The volume of an RCC dam is typically one-tenth to ¼ of the volume of earthfill or rockfill dams at a given site. There is therefore less disruption of the environment from quarrying operations for construction materials and excavation for a spillway chute. The time required for construction of the smaller volume RCC dam is much less than that for the larger earthfill or rockfill dams.

In consideration of whether the RCC or concrete arch section is more appropriate for the Lake Helen site, additional factors should be taken into account. These include the foundation strength, reservoir operation, and seasonal temperature fluctuations that cannot be properly evaluated now. All other factors being equal, our experience indicates that an RCC section will be less expensive to construct than a concrete arch section. We recommend that the RCC dam section be the dam type selected for this study.

**TOTAL PROJECT COST ESTIMATE**
Cost estimates were prepared for dam heights ranging between 30 and 90 feet. The approximate current maximum dam height being considered due to site constraints is about 70 feet. The results shown graphically in Figure 2 correspond to January 2000 dollars. These costs are within about 20% of actual project costs.

**Approach to the Cost Estimate:**
The Tarbox and Hansen (1988) approach was used for this appraisal-level total project cost estimate. Tarbox and Hansen (1988) developed cost versus dam volume curves using statistical data from constructed RCC dams. Existing site information was used in the volume calculations, as was guidance in Tarbox and Hansen (1988). The curves have been updated to include data from RCC dam projects constructed after 1988 and have been brought to January 2000 dollars using Bureau of Reclamation Construction Cost Trends.

The method includes consideration of basic design, estimated cost of the RCC dam structure itself, and upstream seepage reduction system. Tarbox and Hansen (1988) provide information needed to estimate the total project cost, that includes work items such as mobilization, river diversion, foundation excavation, grouting and drainage, spillway and stilling basin, inlet structures, and outlet conduits.

This approach was checked by performing a second cost estimate using unit construction costs and comparing the results. The unit construction costs were compiled by comparing various unit costs from recent cost estimate studies. Both methods gave similar total project costs.

**Design Assumptions:**
The assumed RCC dam cross section is shown in Figure 2. It has a vertical upstream face, a 16-ft wide crest, and a 0.7H:1V slope to the downstream toe. The spillway is incorporated into the dam as an overflow structure. The low-level conduit used in the river diversion scheme during construction would be integrated into the final dam.
Based on known and assumed foundation conditions, an average depth-to-bedrock of 20 feet in the valley bottom, and an average of five feet on valley walls, was assumed. A five-foot over-excavation of rock in the valley bottom, bringing the total foundation excavation depth to 25 feet there was also assumed. A 15-foot over-excavation of rock perpendicular to the valley walls was assumed. This brought the average total abutment excavation to 34 feet horizontally on both abutments.

**Construction Materials:**

Based on the geologic mapping documented in Appendix D, the majority of dam construction materials will be sourced from within the limits of the proposed Lake Helen reservoir. These materials will primarily be aggregates for the RCC that will be obtained from alluvial materials or from quarries established in the valley walls.
REFERENCES


SCHEMATIC DAM SECTION ALONG SPILLWAY
(NOT TO SCALE)
Figure G2

Estimated cost of RCC Dam at Lake Helen Site

Note: (using costs calculated using Tarbox & Hansen procedure with USBR Construction Cost Trends)
APPENDIX H

Hydrology and Flood Routing Calculations

1. Precipitation Duration Frequency Curves
2. General and Local Storm PMP’s
3. Existing Diversion Rehabilitation Hydrology and Flood Routing
4. New Dam Location Hydrology and Flood Routing
Precipitation Duration
Frequency Curves
Rainfall Data Analysis: South Crow Reservoir

Region: 3
Location: 
Elevation: 7130 ft

<table>
<thead>
<tr>
<th>Tr</th>
<th>5-min</th>
<th>10-min</th>
<th>15-min</th>
<th>30-min</th>
<th>1-hr *</th>
<th>2-hr</th>
<th>3-hr</th>
<th>6-hr **</th>
<th>12-hr *</th>
<th>24-hr **</th>
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<tr>
<td>2</td>
<td>0.23</td>
<td>0.36</td>
<td>0.46</td>
<td>0.63</td>
<td>0.80</td>
<td>0.94</td>
<td>1.04</td>
<td>1.20</td>
<td>1.48</td>
<td>1.75</td>
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<tr>
<td>5</td>
<td>0.25</td>
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<td>0.49</td>
<td>0.67</td>
<td>0.85</td>
<td>1.11</td>
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<td>1.90</td>
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<td>0.74</td>
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<td>1.57</td>
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<td>2.30</td>
<td>2.60</td>
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<td>25</td>
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<td>0.67</td>
<td>0.93</td>
<td>1.18</td>
<td>1.60</td>
<td>1.91</td>
<td>2.40</td>
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<td>2.90</td>
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<tr>
<td>50</td>
<td>0.46</td>
<td>0.72</td>
<td>0.91</td>
<td>1.26</td>
<td>1.60</td>
<td>1.96</td>
<td>2.23</td>
<td>2.65</td>
<td>3.03</td>
<td>3.40</td>
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<tr>
<td>100</td>
<td>0.70</td>
<td>1.09</td>
<td>1.38</td>
<td>1.92</td>
<td>2.42</td>
<td>2.62</td>
<td>2.77</td>
<td>3.00</td>
<td>3.30</td>
<td>3.60</td>
</tr>
</tbody>
</table>

Note: spreadsheet is valid for region 3 in Wyoming only.

Region 3

P_{1hr} = 1.897 + 0.439[(X_{6-hr})(X_{6-hr}/X_{24-hr}) - 0.008Z

P_{2hr} = 0.218 + 0.709[(X_{6-hr})(X_{6-hr}/X_{24-hr})]

where Z = point elev in hundreds of feet

Adjustment Factors to Obtain n-min Estimates from 1-hr values

<table>
<thead>
<tr>
<th>Duration (min)</th>
<th>5</th>
<th>10</th>
<th>15</th>
<th>30</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ratio to 1-hr</td>
<td>0.29</td>
<td>0.45</td>
<td>0.57</td>
<td>0.79</td>
</tr>
</tbody>
</table>

* Obtain these values from NOAA vol.II precipitation depth-duration diagrams
** Obtain these values from NOAA Atlas 2, Vol.II - Wyoming
General and Local Storm PMP’s
PURPOSE: Generate the General and Local Probable Maximum Precipitation for the South Crow Watershed

GIVEN: Watershed location - approx 12 miles west of Cheyenne, Wyoming
Watershed location - Long = 105°11'31.2" & Lat = 41°07'41.2"
Watershed location - Region 3, Subregion "B"
Watershed Elevation approx = 7200
Watershed area - approx 12.2 square miles
NOAA Atlas 2, Volume II
HMR 550 Plates Ib-VIb
HMR 55a

SOLUTION: General-Storm PMP Computation

1. Drainage map outline
   12.2 square miles & approximately 3 miles in length extending west

2. Determination of 1-, 6-, 24-, and 72-hr index PMP estimates

<table>
<thead>
<tr>
<th>Duration (hr)</th>
<th>PMP (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-</td>
<td>12.75</td>
</tr>
<tr>
<td>6-</td>
<td>22.00</td>
</tr>
<tr>
<td>24-</td>
<td>28.25</td>
</tr>
<tr>
<td>72-</td>
<td>33.00</td>
</tr>
</tbody>
</table>

3. Selection of appropriate subregion and subdivision
   Orthographic/Subdivision "C"

4. Determine areal reduction factors

<table>
<thead>
<tr>
<th>Duration (hr)</th>
<th>&quot;C&quot; Ortho</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-</td>
<td>98.25</td>
</tr>
<tr>
<td>6-</td>
<td>98.5</td>
</tr>
<tr>
<td>24-</td>
<td>98.75</td>
</tr>
<tr>
<td>72-</td>
<td>98.85</td>
</tr>
</tbody>
</table>

5. Computation of average 1-, 6-, 24-, and 72-hr PMP estimates for drainage

<table>
<thead>
<tr>
<th>Duration (hr)</th>
<th>Areal Adj PMP (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12.53</td>
</tr>
<tr>
<td>6</td>
<td>21.67</td>
</tr>
<tr>
<td>24</td>
<td>27.90</td>
</tr>
<tr>
<td>72</td>
<td>32.62</td>
</tr>
</tbody>
</table>
6. Depth-duration curve for drainage

![Depth-duration curve graph]

7. PMP Estimates for intermediate durations

<table>
<thead>
<tr>
<th>Duration (hr)</th>
<th>PMP (in)</th>
<th>Best fit Log</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>16.07</td>
<td>$y = 4.698714 \ln(x) + 12.816921$</td>
</tr>
<tr>
<td>3</td>
<td>17.98</td>
<td>$R^2 = 0.998324$</td>
</tr>
<tr>
<td>4</td>
<td>19.33</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>20.38</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>21.24</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>24.49</td>
<td></td>
</tr>
<tr>
<td>18</td>
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<td>24</td>
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<td>42</td>
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<td>48</td>
<td>31.01</td>
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<td>54</td>
<td>31.56</td>
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<td>60</td>
<td>32.06</td>
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<tr>
<td>66</td>
<td>32.50</td>
<td></td>
</tr>
<tr>
<td>72</td>
<td>32.91</td>
<td></td>
</tr>
</tbody>
</table>
SOLUTION: Local-Storm PMP Computation

1. Index 1-hr 1-mi^2 PMP estimate at 5,000-ft elevation

   \[ \text{PMP (in.)} = 10.85 \]

2. Adjustment for mean elevation of drainage

   From Figure 14.3, at 7200 feet the persisting dew point temp does not affect the elevation adjustment. Therefore, if the dew point temp is between 70 and 80 degrees, the elevation adjustment is

   \[ \text{El Adj} = 0.885 \]

3. Index 1-hr 1-mi^2 PMP estimate at mean elevation of drainage

   \[ \text{El Adj PMP} = 9.60225 \]

4. Depth Duration Curve for 1 mi^2

<table>
<thead>
<tr>
<th>Duration (hr)</th>
<th>% of 1-hr</th>
<th>PMP (hr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>0.68</td>
<td>6.53</td>
</tr>
<tr>
<td>0.50</td>
<td>0.86</td>
<td>8.26</td>
</tr>
<tr>
<td>0.75</td>
<td>0.94</td>
<td>9.03</td>
</tr>
<tr>
<td>1.00</td>
<td>1.00</td>
<td>9.60</td>
</tr>
<tr>
<td>2.00</td>
<td>1.16</td>
<td>11.14</td>
</tr>
<tr>
<td>3.00</td>
<td>1.23</td>
<td>11.81</td>
</tr>
<tr>
<td>4.00</td>
<td>1.28</td>
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<td>12.67</td>
</tr>
<tr>
<td>6.00</td>
<td>1.35</td>
<td>12.96</td>
</tr>
</tbody>
</table>

   Table 12.4

5. Areal reduction factors

<table>
<thead>
<tr>
<th>Duration (hr)</th>
<th>Areal Adj</th>
</tr>
</thead>
<tbody>
<tr>
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<td>0.50</td>
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<tr>
<td>0.75</td>
<td>0.790</td>
</tr>
<tr>
<td>1.00</td>
<td>0.810</td>
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<tr>
<td>2.00</td>
<td>0.830</td>
</tr>
<tr>
<td>3.00</td>
<td>0.845</td>
</tr>
<tr>
<td>4.00</td>
<td>0.855</td>
</tr>
<tr>
<td>5.00</td>
<td>0.870</td>
</tr>
<tr>
<td>6.00</td>
<td>0.875</td>
</tr>
</tbody>
</table>

   fig 12-12
6. PMP estimates for the basin

<table>
<thead>
<tr>
<th>Duration (hr)</th>
<th>PMP (in)</th>
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</thead>
<tbody>
<tr>
<td>0.25</td>
<td>4.64</td>
</tr>
<tr>
<td>0.50</td>
<td>6.32</td>
</tr>
<tr>
<td>0.75</td>
<td>7.13</td>
</tr>
<tr>
<td>1.00</td>
<td>7.78</td>
</tr>
<tr>
<td>2.00</td>
<td>9.25</td>
</tr>
<tr>
<td>3.00</td>
<td>9.98</td>
</tr>
<tr>
<td>4.00</td>
<td>10.51</td>
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<tr>
<td>5.00</td>
<td>11.03</td>
</tr>
<tr>
<td>6.00</td>
<td>11.34</td>
</tr>
</tbody>
</table>

7. Incremental PMP Estimates for the basin

<table>
<thead>
<tr>
<th>Duration (hr)</th>
<th>PMP (in)</th>
<th>Equation: ( y = 2.082470\ln(x) + 7.688204 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.25</td>
<td>4.64</td>
<td>( R^2 = 0.998476 )</td>
</tr>
<tr>
<td>0.50</td>
<td>6.32</td>
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</tr>
<tr>
<td>0.75</td>
<td>7.13</td>
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</tr>
<tr>
<td>1.00</td>
<td>7.78</td>
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</tr>
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<td>2.00</td>
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<td>72</td>
<td>16.59</td>
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</table>
Figure 14.3.—Adjustment for elevation for local-storm PMP based on procedures developed in the report and maximum persisting 12-hr 1000-mb dew point (F).
Figure 12.10.—Depth-duration curve for 6-/1-hr ratio of 1.35.

Table 12.4.—Percent of 1-hr local-storm PMP for selected durations for 6-/1-hr ratio of 1.35 (HMR No. 49)

<table>
<thead>
<tr>
<th>Duration (hr)</th>
<th>Percent of 1 hr</th>
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</thead>
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<td>1/4</td>
<td>.68</td>
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<td>.86</td>
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<tr>
<td>3/4</td>
<td>.94</td>
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<td>5</td>
<td>1.32</td>
</tr>
<tr>
<td>6</td>
<td>1.35</td>
</tr>
</tbody>
</table>

Figure 12.12.—Depth–area relations adopted for local-storm PMP in the CD-103
Existing Diversion
Rehabilitation Hydrology and Flood Routing
PURPOSE: Route the 100-yr storm runoff through the South Crow watershed basin with and without the proposed storage dam to determine the overtopping effects for the existing and proposed diversion structure.

GIVEN: Watershed Modeling Software file created to delineate the watershed basin(s) based on the chosen outlet locations (existing diversion structure and proposed storage dam).

- **WMS Filename:** P:\DOMESTIC\3714-01\Data\WMS\00210\2basins.sup
- **Precipitation depth-duration curves generated using NOAA Atlas 2, Vol. II-Wyoming**
- **PDF Filename:** P:\DOMESTIC\3714-01\CALCUL\PDF & PMP.xls
- **Reservoir volume capacity analysis generated from topo data for the existing diversion structure and the proposed storage dam.**
- **Prop Dam A-C Filename:** P:\DOMESTIC\3714-01\CALCUL\Diversion Structure\area capacity pro_stordam.xls
- **Exist Dam A-C Filename:** P:\DOMESTIC\3714-01\CALCUL\Diversion Structure\area capacity ex_divdam.xls

SOLUTION: Using HEC-1, the storm runoff will be generated and routed through the upper part of the watershed basin/proposed reservoir and combined with the runoff generated from the lower part of the watershed basin. The combined hydrographs will be routed through the diversion reservoir where the characteristics of the existing and proposed diversion dam calculations of overtopping will be made.

**Diversion Dam Physical Properties:**

<table>
<thead>
<tr>
<th></th>
<th>Existing</th>
<th>Proposed</th>
<th>Iter #1</th>
<th>Iter #2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dam Crest El</td>
<td>7132</td>
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<tr>
<td>Dam Crest Length</td>
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<tr>
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<td>Low Level Outlet CA</td>
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<td>Discharge Coefficient Dam</td>
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<td>2.70</td>
<td>(assumes broad crest overlay; 4' breadth &amp; 2.5' overtop)</td>
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**Storage Dam Physical Properties:**

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Loss Method:

SCS
Initial Abstraction = 0.1 (assumed a little lower than recommended as it is assumed antecedent storm will reduce abstractions.)
Curve Number = 86
% Impervious = 20 (assumed a little higher than recommended as it is assumed antecedent storm will have already saturated the ground.)

See SCS parameter Calcs

RESULTS: Overtopping of Diversion Dam with Proposed Storage Dam in Place

HEC-1 Filename: 2res100yr_60prop.hc1 input file
2res100yr_60prop.out output file

Diversion Dam Results:

<table>
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<tr>
<th>Iter #1</th>
<th>Iter #2</th>
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</thead>
<tbody>
<tr>
<td>Max Stage</td>
<td>7137.64 msl</td>
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<tr>
<td>Peak Discharge</td>
<td>3284 cfs</td>
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<tr>
<td>Peak Storage</td>
<td>21.4 acre-feet</td>
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<tr>
<td>Time to Peak</td>
<td>3.83 hours</td>
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<tr>
<td>Height of Overtop</td>
<td>2.64 ft</td>
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</table>

RESULTS: Overtopping of Diversion Dam without Proposed Storage Dam in Place

HEC-1 Filename: 1res100yr.hc1 input file
1res100yr.out output file

Diversion Dam Results:

<table>
<thead>
<tr>
<th>Iter #1</th>
<th>Iter #2</th>
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</thead>
<tbody>
<tr>
<td>Max Stage</td>
<td>7137.58 msl</td>
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<tr>
<td>Peak Discharge</td>
<td>3202 cfs</td>
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<tr>
<td>Peak Storage</td>
<td>21.2 acre-feet</td>
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<tr>
<td>Time to Peak</td>
<td>3.67 hours</td>
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<td>Height of Overtop</td>
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CONCLUSIONS:

Dam overtopping remains less than the allowable overtopping height of 3 feet for scenarios of sluice gate opened or closed on the low level outlet. The physical appurtenances of the proposed storage dam upstream have not been finalized, but small changes should not affect the hydrologic conditions of the lower diversion structure too much.
PURPOSE: Generate reservoir area-capacity curve for the South Crow Diversion Dam using survey data by a.v.i. In addition, determine the new spillway elevation which will store 12 acre-feet of water at crest elevation.

GIVEN: a.v.i. survey calculation attached

SOLUTION:

<table>
<thead>
<tr>
<th>Elevation (ft)</th>
<th>Capacity (Acre-ft)</th>
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<tbody>
<tr>
<td>7130.56</td>
<td>8.877</td>
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<td>7131.56</td>
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<td>7134.56</td>
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<tr>
<td>7135.56</td>
<td>17.784</td>
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</table>

Find elevation @ 12 acre-ft capacity

\[ X \quad Y \]
\[ (\text{Acre-ft}) \quad (\text{ft}) \]
\[ 12 \quad 7132.4162 \quad \text{Required Spillway Crest Elevation to store 12 acre-feet of water} \]
\[ 18.4 \quad 7136 \quad \text{Estimated Reservoir capacity for Flood Routing Studies} \]
\[ 20.2 \quad 7137 \quad \text{Estimated Reservoir capacity for Flood Routing Studies} \]
\[ 22.0 \quad 7138 \quad \text{Estimated Reservoir capacity for Flood Routing Studies} \]
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<tr>
<th>Reservoir</th>
<th>Storage Volumes</th>
<th>% Inc.</th>
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<td>7130.56</td>
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<td>10.93 Acre</td>
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<td>15.788 Acre</td>
</tr>
<tr>
<td>7135.56</td>
<td></td>
<td>17.784 Acre</td>
</tr>
</tbody>
</table>
PURPOSE: Generate reservoir area-capacity curve for the proposed storage dam as located.

GIVEN: Topo created from Eagle Point Software using Digital Elevation Map Data

SOLUTION:

<table>
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</table>

Years when all downstream water rights could be met.
Rainfall Data Analysis: South Crow Reservoir

Region: 3
Location: Elevation: 7130 ft

<table>
<thead>
<tr>
<th>Tr</th>
<th>5-min</th>
<th>10-min</th>
<th>15-min</th>
<th>30-min</th>
<th>1-hr *</th>
<th>2-hr</th>
<th>3-hr</th>
<th>6-hr **</th>
<th>12-hr *</th>
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</thead>
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</tbody>
</table>

Note: spreadsheet is valid for region 3 in Wyoming only.

** Adjustment Factors to Obtain n-min Estimates from 1-hr values **

<table>
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<th>Duration (min)</th>
<th>5</th>
<th>10</th>
<th>15</th>
<th>30</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ratio to 1-hr</td>
<td>0.29</td>
<td>0.45</td>
<td>0.57</td>
<td>0.79</td>
</tr>
</tbody>
</table>

* Obtain these values from NOAA vol II precipitation depth-duration diagrams
** Obtain these values from NOAA Atlas 2, Vol. II - Wyoming

\[ Y_{100} = 1.897 + 0.439[(X_{6-hr})(X_{6-hr}/X_{24-hr})] - 0.008Z \]

\[ Y_2 = 0.218 + 0.709[(X_{6-hr})(X_{6-hr}/X_{24-hr})] \]

where Z = point elev in hundreds of feet

\[ Y = 0.342(6-hr) + 0.658(1-hr) \]

\[ Y = 0.597(6-hr) + 0.403(1-hr) \]
<table>
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<th>0.20</th>
<th>0.30</th>
<th>0.45</th>
<th>0.60</th>
<th>0.75</th>
<th>0.90</th>
<th>1.20</th>
<th>1.50</th>
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A = 11.75 mi^2
BS = 0.0722 ft/ft
Shape = 9.61 mi^2/mi^2
CTOSTR = 1259.22 ft
CSD = 24723.55 ft

A = 0.30 mi^2
BS = 0.2714 ft/ft
Shape = 1.39 mi^2/mi^2
CTOSTR = 550.89 ft
CSD = 3417.39 ft
THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.

THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION

NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE, SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL, LOSS RATE:GREEN AND AMPT INFILTRATION
KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

HEC-1 INPUT PAGE 1

<table>
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<td>2</td>
<td>ID</td>
<td>Return Period 100 yr; Filename: Ires100yr.out</td>
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**DIAGRAM**

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 X X X X X X X
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 XXXXXX XXXX X XXXXX X
 X X X X X X X
 X X X XXXXXX XXXXX XXX
```

**100-yr**

```
IT 10 1JAN94 0 432
IO 0
KK Lower So Crow Watershed Basin
KO 0 0 0 0 0
BA 12.06
PB 0
IN 15 1JAN94 0
* 100-yr
```
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*Lower res elev*

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### Schematic Diagram of Stream Network

**Input Line (V) Routing**
- (-----) Diversion or Pump Flow

**No. Connector**
- (<---) Return of diverted or pumped flow

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<tr>
<td>23</td>
<td>1R</td>
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(***) Runoff also computed at this location

---

#### HEC-1 Analysis

- **Analysis using WMS 10/20/99**
- **Precipitation Derived from NOAA 2 Vol 11-Wyoming**
- **Return Period 100 yr**

**Output Control Variables**
- **IPRTN**: 0 (Print Control)
- **IPLOT**: 0 (Plot Control)
- **QSCAL**: 0 (Hydrograph Plot Scale)

**Hydrograph Time Data**
- **NMIN**: 10 (Minutes in Computation Interval)
- **IDATE**: 1JAN94 (Starting Date)
- **ITIME**: 0000 (Starting Time)
- **NQ**: 432 (Number of Hydrograph Ordinates)
- **NDDATE**: 3JAN94 (Ending Date)
- **NDTIME**: 2350 (Ending Time)
- **ICENT**: 19 (Century Mark)

**Computation Interval**: 0.17 Hours
**Total Time Base**: 71.83 Hours

**English Units**
- **Drainage Area**: Square Miles
- **Precipitation Depth**: Inches
- **Length, Elevation**: Feet
FLOW  CUBIC FEET PER SECOND
STORAGE VOLUME  ACRE-FEET
SURFACE AREA  ACRES
TEMPERATURE  DEGREES FAHRENHEIT

**************

6 KK  Lower  So Crow Watershed Basin

**************

7 KO  OUTPUT CONTROL VARIABLES

IPRINT  0  PRINT CONTROL
IPLT    0  PLOT CONTROL
QSCAL   0.  HYDROGRAPH PLOT SCALE
IPNCH   0  PUNCH COMPUTED HYDROGRAPH
IOUT    22  SAVE HYDROGRAPH ON THIS UNIT
ISAV1   1  FIRST ORDINATE PUNCHED OR SAVED
ISAV2   432  LAST ORDINATE PUNCHED OR SAVED
TIMINT  .167  TIME INTERVAL IN HOURS

10 IN  TIME DATA FOR INPUT TIME SERIES

JXMIN  15  TIME INTERVAL IN MINUTES
JXDATE  1JAN94  STARTING DATE
JXTIME  0  STARTING TIME

8 BA  SUBBASIN CHARACTERISTICS
TAREA  12.06  SUBBASIN AREA

9 PB  PRECIPITATION DATA

STORM  3.60  BASIN TOTAL PRECIPITATION

11 PI  INCREMENTAL PRECIPITATION PATTERN

.92  .64  .36  .17  .17  .17  .03  .03  .03  .03
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.00  .00  .00  .00  .00  .00  .00  .00  .00  .00  .00
.00  .00  .00  .00  .00  .00  .00  .00  .00  .00  .00
21 LS  
SCS LOSS RATE
STRTL .10 INITIAL ABSTRACTION
CRVNBR 86.00 CURVE NUMBER
RTIMP 20.00 PERCENT IMPERVIOUS AREA

22 UD  
SCS DIMENSIONLESS UNITGRAPH
TLAG 3.06 LAG

***
UNIT HYDROGRAPH
94 END-OF-PERIOD ORDINATES

DA MON HRMN ORD RAIN LOSS EXCESS COMP Q
1 JAN 0000 1 .00 .00 .00 .00 .00 .00 .00 .00 1.
1 JAN 0010 2 .92 .52 .40 12. * 2 JAN 1200 217 .00 .00 .00 1.
1 JAN 0020 3 .64 .18 .46 39. * 2 JAN 1210 218 .00 .00 .00 1.
1 JAN 0030 4 .36 .07 .29 91. * 2 JAN 1220 221 .00 .00 .00 1.
1 JAN 0040 5 .17 .03 .14 166. * 2 JAN 1230 222 .00 .00 .00 1.
1 JAN 0050 6 .17 .03 .14 263. * 2 JAN 1240 223 .00 .00 .00 1.
1 JAN 0100 7 .17 .02 .14 383. * 2 JAN 1250 224 .00 .00 .00 1.
1 JAN 0110 8 .03 .00 .03 526. * 2 JAN 1300 225 .00 .00 .00 1.
1 JAN 0120 9 .03 .00 .03 697. * 2 JAN 1310 226 .00 .00 .00 1.
1 JAN 0130 10 .03 .00 .03 897. * 2 JAN 1320 227 .00 .00 .00 1.
1 JAN 0140 11 .03 .00 .03 1126. * 2 JAN 1330 228 .00 .00 .00 1.
1 JAN 0150 12 .03 .00 .03 1384. * 2 JAN 1340 229 .00 .00 .00 1.
1 JAN 0200 13 .03 .00 .03 1656. * 2 JAN 1350 230 .00 .00 .00 1.
1 JAN 0210 14 .03 .00 .02 1929. * 2 JAN 1400 231 .00 .00 .00 1.
1 JAN 0220 15 .03 .00 .02 2188.

29. 63. 132. 205. 293. 391. 508. 643. 800. 973.
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1819. 1760. 1698. 1629. 1557. 1478. 1389. 1291. 1180. 1062.
960. 862. 790. 722. 663. 605. 556. 510. 474. 439.
403. 370. 341. 311. 282. 259. 239. 220. 200. 185.
170. 156. 141. 130. 120. 109. 99. 92. 85. 77.
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30. 27. 25. 23. 21. 19. 17. 16. 15. 14.
14. 13. 12. 10. 9. 8. 7. 6. 5. 4.
3. 2. 1. 0.

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HYDROGRAPH AT STATION   Lower
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<th>72-HR</th>
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**PEAK OUTFLOW IS 3203. AT TIME 3.67 HOURS**
### Runoff Summary

**Flow in Cubic Feet Per Second**

**Time in Hours, Area in Square Miles**

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<th>Operation</th>
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<th>Peak Flow</th>
<th>Time of Peak</th>
<th>Average Flow for Maximum Period</th>
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<td>6-Hour</td>
<td>24-Hour</td>
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<td>HYDROGRAPH AT</td>
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**Summary of Dam Overtopping/Breach Analysis for Station 1R**

(Peaks shown are for internal time step used during breach formation)

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<th>Top of Dam</th>
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<th>Ratio of PMF</th>
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<th>Maximum Outflow</th>
<th>Maximum Over Top</th>
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**Three Normal End of HEC-1**
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**CUMULATIVE AREA =** 12.06 SQ MI
THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

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DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL  LOSS RATE:GREEN AND AMPT INFILTRATION
KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

HEC-1 INPUT

LINE ID........1........2........3........4........5........6........7........8........9........10
1   ID      HEC-1 Analysis using WMS 02/11/2000
2   ID      General Storm PMP - NOAA Atlas 2, Vol II
3   ID      Engineer: David J. Priske; Filename: 2res_gen_7360prop.out

*DIAGRAM

*** LIST ***

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6   KK      Upper So Crow Watershed
7   KO      0  0  0  0  22
8   BA      11.751
9   BF      0  0  0
10  PB      0
11  IN      15 1JAN94  0

2 reservoirs

general storm
* General Storm PMP

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| PC | 26.85 | 26.9063 | 26.9625 | 27.0198 | 27.075 | 27.1312 | 27.1875 | 27.2437 | 27.3 | 27.3562 |
| PC | 27.412 | 27.4688 | 27.525 | 27.5813 | 27.6375 | 27.6938 | 27.75 | 27.7937 | 27.8375 | 27.8813 |
| PC | 29.508 | 29.5437 | 29.5792 | 29.6146 | 29.6496 | 29.6840 | 29.7108 | 29.7412 | 29.7717 | 29.8021 |
| PC | 30.695 | 30.7213 | 30.7475 | 30.7738 | 30.8 | 30.8263 | 30.8525 | 30.8788 | 30.905 | 30.9313 |
| PC | 32.06 | 32.0793 | 32.0967 | 32.115 | 32.1333 | 32.1517 | 32.17 | 32.1883 | 32.2067 | 32.225 |
| PC | 32.243 | 32.2617 | 32.278 | 32.2993 | 32.3167 | 32.335 | 32.3533 | 32.3717 | 32.39 | 32.4083 |
| PC | 32.426 | 32.445 | 32.4633 | 32.4817 | 32.5 | 32.5171 | 32.5342 | 32.5512 | 32.5683 | 32.5854 |
| PC | 32.602 | 32.6196 | 32.6367 | 32.6538 | 32.6708 | 32.6879 | 32.705 | 32.7221 | 32.7392 | 32.7562 |
| PC | 32.773 | 32.7904 | 32.8075 | 32.8246 | 32.8417 | 32.8588 | 32.8758 | 32.8929 | 32.91 | 32.93 |

LS 0.1 86 20

UD 2.7587

43 44 45 46 47 48
HEC-1 INPUT

PAGE 2

LINE
ID.............1........2........3........4........5........6........7........8........9........10

49 SE 7340 7344 7348 7352 7356 7360
50 SS 7348 200 3.6 1.5
51 SL 7308 4.91 0.7 0.5
52 ST 7360 204.34 2.6 1.5

53 KK Lower So Crow Watershed Basin
54 KO 0 0 0 0 22
55 BA 0.3036
56 PB 0
57 IN 15 1JAN94 0

* General Storm PMP

61 PC 22.052 22.1879 22.3233 22.4587 22.5942 22.7296 22.865 23.0004 23.1358 23.2712
66 PC 26.85 26.9063 26.956 27.0188 27.075 27.1312 27.1875 27.2437 27.3 27.3562
67 PC 27.412 27.4688 27.525 27.5813 27.6375 27.6938 27.75 27.7937 27.8375 27.8813
72 PC 29.508 29.5437 29.5792 29.6146 29.65 29.6804 29.7108 29.7412 29.7717 29.8021
73 PC 29.832 29.8629 29.8933 29.9237 29.9542 29.9846 30.015 30.0454 30.0758 30.1062
76 PC 30.695 30.7213 30.7475 30.7738 30.8 30.8263 30.8525 30.8788 30.905 30.9313
77 PC 30.957 30.9838 31.01 31.0329 31.0558 31.0787 31.1017 31.1246 31.1475 31.1704
82 PC 32.06 32.0783 32.0967 32.115 32.1333 32.1517 32.17 32.1863 32.2067 32.225
83 PC 32.243 32.2617 32.28 32.3 32.3267 32.3533 32.3717 32.39 32.4083
84 PC 32.426 32.4452 32.4633 32.4817 32.5 32.5171 32.5342 32.5512 32.5683 32.5854
85 PC 32.602 32.6196 32.6367 32.6538 32.6708 32.6879 32.705 32.7221 32.7392 32.7562
86 PC 32.773 32.7904 32.8075 32.8246 32.8417 32.8588 32.8758 32.8929 32.91
87 LS 0.1 86 20
88 UD 0.4546

89 KK 1C CNAME 1R
90 KO 0 0 0 0 22
91 HC 2
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SCHEMATIC DIAGRAM OF STREAM NETWORK

INPUT
LINE (V) ROUTING (--->) DIVERSION OR PUMP FLOW

NO. (.) CONNECTOR (<---) RETURN OF DIVERTED OR PUMPED FLOW

6 Upper
V
V
43 3R
.
53 .  Lower
.
.
89 1C............
V
V
92 1R

(***) RUNOFF ALSO COMPUTED AT THIS LOCATION

*******************************************************************************

* FLOOD HYDROGRAPH PACKAGE (HEC-1) *
* MAY 1991 *
* VERSION 4.0.1E *
* RUN DATE TIME *

*******************************************************************************

HEC-1 Analysis using WMS 02/11/2000
General Storm PMP - NOAA Atlas 2, Vol II
Engineer: David J. Friske

5 IO
OUTPUT CONTROL VARIABLES
IPRINT 0 PRINT CONTROL
IPLOT 0 PLOT CONTROL
QSCAL 0 HYDROGRAPH PLOT SCALE

IT
HYDROGRAPH TIME DATA
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ITIME 0000 STARTING TIME
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NDDATE 3JAN94 ENDING DATE
NDTIME 2350 ENDING TIME
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**TOTAL RAINFALL = 32.90, TOTAL LOSS = 1.32, TOTAL EXCESS = 31.58**

**PEAK FLOW TIME MAXIMUM AVERAGE FLOW**

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<th>Type</th>
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<th>24-HR</th>
<th>72-HR</th>
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**CUMULATIVE AREA = 11.75 SQ MI**
PEAK OUTFLOW IS 29486. AT TIME 4.00 HOURS

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<td>+ 7149.62</td>
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<td>7146.39 7140.14 7136.89 7136.89</td>
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CUMULATIVE AREA = 12.05 SQ MI
RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

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SUMMARY OF DAM OVERTOPPING/BREACH ANALYSIS FOR STATION 3R
(Peaks shown are for internal time step used during breach formation)

PLAN 1 .......... INITIAL VALUE SPILLWAY CREST TOP OF DAM
ELEVATION   7348.00       7348.00       7360.00
STORAGE    1046.         1046.         1848.      
OUTFLOW    174.          174.          30129.     

RATIO     MAXIMUM MAXIMUM MAXIMUM MAXIMUM DURATION TIME OF TIME OF
OF RESERVOIR W.S.ELEV DEPTH STORAGE OUTFLOW OVER TOP MAX OUTFLOW FAILURE
PMF   1.00  7359.75 0.00 1830.0 29212.0 0.00 4.00 .00

SUMMARY OF DAM OVERTOPPING/BREACH ANALYSIS FOR STATION 1R
(Peaks shown are for internal time step used during breach formation)

PLAN 1 .......... INITIAL VALUE SPILLWAY CREST TOP OF DAM
ELEVATION   7132.45       7132.40       7135.00
STORAGE    12.           12.           17.       
OUTFLOW    136.          136.          709.      

RATIO     MAXIMUM MAXIMUM MAXIMUM MAXIMUM DURATION TIME OF TIME OF
OF RESERVOIR DEPTH STORAGE OUTFLOW OVER TOP MAX OUTFLOW FAILURE
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*** NORMAL END OF HEC-1 ***
New Dam Location Hydrology and Flood Routing
PURPOSE: Define the approach taken to size the spillway for the appropriate percentage of the probable maximum flood.

SOLUTION:

1. Incorporating "Watershed Modeling Software" from BOSS International to delineate the watershed basin for the existing dam location which will include a basin outlet for the location of the proposed dam.

WMS is merely a pre-processor for the development of the HEC-1 input file. Digital elevation maps (DEM) are input into the model to which the locations (in metric Northing and Easting) of the two dam sites are placed. The model delineates the watershed for the outlet points and computes the basin areas, slopes, centroid stream distance, centroid stream slope, perimeter, shape factors and others.

Upon successful delineation of the contributing watershed (12.06 m³), the HEC-1 input for the watershed parameters are created within a file to which precipitation information, reservoir information for the proposed dam location, and channel routing information is included.

The proposed dam site will require information regarding the volume-capacity, the spillway crest length, elevation and discharge coefficient, low-level outlet centerline elevation & cross-sectional area, and dam crest length, elevation, and discharge coefficient. These physical features are determined by routing the local storm probable maximum Precipitation as required by the regulating authority.
This map complies with national map accuracy standards for sale by U.S. Geological Survey, Denver, Colorado 80225, or request a folder describing topographic maps and symbols is available on...
**FLOOD HYDROGRAPH PACKAGE (HEC-1)**

MAY 1991

**VERSION 4.0.1E**

**RUN DATE TIME**

THESE PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HECIGS, HECIDB, AND HECIRW.


NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE, SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY, DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL, LOSS RATE:GREEN AND SMFT INFILTRATION.

KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM.

**HEC-1 INPUT**

**LINE**

| ID | MEC-1 Analysis using WMS 02/11/2000 | Engine: David J. Priske | ID File: LIB020.ncl |

**LIST**

| IT | 10 | L2AN94 | 0 | 432 |

| KK | Upper So Crow Watershed |
| KO | 0 | 0 | 0 | 22 |

| BA | 11.751 |
| BF | 0 | 0 | 0 |

| FB | 0 |

| IN | 15 | L2AN94 |

* Local Storm PMF

| PC | 0 | 4.64 | 6.32 | 7.13 | 7.78 | 8.1475 | 8.515 | 8.8825 | 9.25 | 9.4325 |
| PC | 11.03 | 11.1075 | 11.185 | 11.2625 | 11.34 | 11.4033 | 11.4667 | 11.53 | 11.5933 | 11.6567 |
| PC | 15.086 | 15.1525 | 15.2185 | 15.2845 | 15.3505 | 15.4165 | 15.4825 | 15.5485 | 15.6145 | 15.6805 |
| PC | 15.22 | 15.2865 | 15.3525 | 15.4185 | 15.4845 | 15.5505 | 15.6165 | 15.6825 | 15.7485 | 15.8145 |
| PC | 15.363 | 15.4295 | 15.4955 | 15.5615 | 15.6275 | 15.6935 | 15.7595 | 15.8255 | 15.8915 | 15.9575 |

**PAGE 1**
### Schematic Diagram of Stream Network

**Input Line**

- **(V) Routing**: Version of pump flow
- **(.) Connector**: Return of diverted or pumped flow

#### Definitions

- **V**: Upper
- **JR**: Lower

#### Connections

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#### Notes

- "(**) Runoff also computed at this location"

---

### Flood Hydrograph Package (HEC-1)

- **Analysis Using**: WMS 02/11/2000, Local Storm PMF - NOAA Atlas 2, Vol II
- **Engineer**: David J. Friske

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#### Computation Interval

- **Duration**: 0.17 hours
- **Total Time Base**: 71.63 hours

### Units

- **English Units**
  - **Drainage Area**: Square miles
  - **Precipitation Depth**: Inches
  - **Length, Elevation**: Feet
  - **Flow**: Cubic feet per second
  - **Storage Volume**: Acre-feet
  - **Surface Area**: Acres
  - **Temperature**: Degrees Fahrenheit

---
**Upper So Crow Watershed**

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### OUTPUT CONTROL VARIABLES

- **IPRINT**: 0 (PRINT CONTROL)
- **IPLOT**: 0 (PLOT CONTROL)
- **QSCAL**: 0.6 (HYDROGRAPH PLOT SCALE)
- **IPMCH**: 0 (PUNCH COMPUTED HYDROGRAPH)
- **ISAV1**: 1 (FIRST ORDINATE PUNCHED OR SAVED)
- **ISAV2**: 432 (LAST ORDINATE PUNCHED OR SAVED)
- **TIMINT**: .167 (TIME INTERVAL IN HOURS)

### TIME DATA FOR INPUT TIME SERIES

- **JXMIN**: 15 (TIME INTERVAL IN MINUTES)
- **JXDATE**: 1JAN94 (STARTING DATE)
- **JXTIME**: 0 (STARTING TIME)

### SUBBASIN CHARACTERISTICS

- **TAREA**: 11.75 (SUBBASIN AREA)

### BASE FLOW CHARACTERISTICS

- **STRTQ**: .00 (INITIAL FLOW)
- **QRCSN**: .00 (BEGIN BASE FLOW RECESSION)
- **RTIOR**: 1.00000 (RECESSION CONSTANT)

### PRECIPITATION DATA

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APPENDIX I

Water Rights Exhibits
Dear Mr. Bergquist:

This letter is provided in response to your letter dated November 18, 1999, regarding the amount of water stored behind the South Crow Division Dam for the City of Cheyenne Board of Public Utilities. Your questions and requests for comments and clarification are as follows:

1) South Crow Pipeline, Permit No. 10192, was permitted in 1910 as a municipal diversion for the City of Cheyenne. At the time of permitting, no reservoir permit was required. The water right record for the pipeline is clear that there was a diversion dam necessary to effectuate the diversion of the water. This office has no official record of the amount of storage occurring behind the dam. In 1987, during a State Engineer's Office safety-of-dams inspection of the facility, the storage was visually estimated to be 12 acre-feet. By today's legal standards for diversion dams, a reservoir permit would be required for a facility of this type. It is not the intent of this office to take any water from this needed facility but we also cannot authorize any actions or changes to the diversion dam that would injure any other water appropriator. This office is willing to recognize and memorialize the existing storage capacity behind the diversion dam. From your Item #2, it appears that this storage amounts to a total of 8.87 acre-feet. It also appears from the plans of the diversion facility that this storage capacity is split into two components: 1) water stored below the invert of the South Crow Pipeline and 2) water stored above the pipeline to provide the head necessary to achieve the seven (7) c.f.s. diversion under Permit No. 10192. This storage is considered an in-place use to create the head necessary for the diversion and could not (on its own) be diverted for municipal use.
To formally recognize this reservoir, this office will require two petitions and a map to enter this information into the water right records. 1) A petition to the State Engineer should request the acceptance of an amended map and that an endorsement be added to Permit No. 10492 to reflect the storage capacity behind the diversion dam as described above. The amended map must meet the criteria for filing a reservoir map including a location map, reservoir contours, area-capacity table, etc. 2) A petition to the Board of Control requesting an amended certificate be issued to recognize the storage behind the diversion dam.

2) The information submitted should reflect the current survey details. The 12 acre-feet of capacity was only an estimate by this office, so the current surveyed information should be used in the amended map.

3) If the selected rehabilitation option increases the capacity of the reservoir above the 8.87 acre-feet, then a new reservoir application should be submitted to permit the additional storage capacity. The additional capacity may be used to temporarily store water from Stage I or II, since a current-day priority date on South Crow Creek may not be in priority to provide the water necessary to fill the enlargement of the reservoir. There may be conveyance losses charged to the Stage I and II water as it flows down the creek and into the reservoir.

4) A new reservoir, similar in size and location to the proposed Lake Helen is possible. It is possible to consider it part of the conveyance system and capable of restoring water previously stored in Rob Roy Reservoir. Again, delivery of water to and at this facility would be subject to conveyance, evaporation, and seepage losses. All new filings in this office are routinely reported to the other parties to the Cooperative Agreement. The procedures regarding new depletions have not yet been finalized under an agreed to Recovery Program so it is hard to advise you regarding the City's obligations for mitigation of the depletions from these facilities. This would be triggered by any federal permitting that may be necessary for this project. Also, the $215 per acre-foot that you refer to was determined in the interim programmatic EIS regarding the endangered species in central Nebraska and applies only to depletions less than 25 acre-feet. Larger depletions were to be mitigated by removing or abandoning another water right with similar depletions.
Our records indicate an old, expired permit at the location of the proposed Lake Helen. Permit No. 1557R, South Crow Creek Reservoir, was filed by the City of Cheyenne in 1909 and expired in 1912. The City of Cheyenne will be required to request cancellation of this old filing.

In any case, there will be a requirement for installation of measuring devices and outlets so that an accounting of South Crow Creek water could be determined for administrative purposes as it may be necessary to regulate Crow Creek water to downstream senior appropriators.

Please feel free to contact John Barnes or Phil Velez in the Surface Water Division as questions arise or if more information is necessary. I am also available to meet with you to further discuss these proposed projects.

With best regards,

GORDON W. FASSETT
State Engineer

cc: John Barnes, Administrator - Surface Water
    Allan Cunningham, Administrator - Board of Control
    Randy Tullis, Superintendent - Water Division No. 1
    Doug Elgin, Hydrographer- District 1
    Bruce Brinkman, WWDC - Project Manager
    Phil Velez, Water Rights Supervisor
November 18, 1999

Mr. Gordon W. Fassett
Wyoming State Engineer
Wyoming State Engineer’s Office
Herschler Building, 4th Floor East
Cheyenne, Wyoming 82002

Dear Mr. Fassett,

We are working on the Level I and II study of the South Crow Diversion Project for the Wyoming Water Development Commission and City of Cheyenne Board of Public Utilities. The work requires an evaluation of the existing diversion structure, the feasibility of its rehabilitation, and the potential to store more water in the watershed. Additional water storage depends upon the storage rights that could be secured for the site.

Therefore, on behalf of the Cheyenne Board of Public Utilities, we are requesting clarification and/or opinions from the State Engineers Office on the following;

1. Since the adjudicated water right does reference a 12.0 AF storage amount, can a 1912 reservoir permit be obtained? The structure and impoundment has existed and has been maintained since that time.

2. If the response to Question 1 is no, then can a 1912 reservoir storage permit be obtained for the 1999 surveyed amount of 8.87 AF? The difference between the two storage volumes appears to be sedimentation loss over the last 87 years.

3. One dam rehabilitation option would involve encapsulating the existing structure in 3 feet of new concrete. If this were the selected approach, the spillway would likewise be raised 3 feet; and the new storage amount would be approximately 13.90 AF, which is 1.9 AF greater than the original 1912 amount. Assuming the answer to Question 1 is yes, can the additional amount of storage be made up with Stage I water discharged five (5) miles upstream from the dam structure via the Stage I pipeline?

4. If response to Question 3 is yes, can a new reservoir, similar in size and location to the proposed Lake Helen reservoir upstream from the existing diversion, be filled with Stage I water and be considered part of the system conveyance? Would any of the storage...
amounts considered above be subject to the Platte River mitigation fees of $215.00/AF per one time filling fee?

We would like to meet with you when your schedule permits, to discuss these water-right questions. Let us know a time that would be convenient for you.

Sincerely,

R. J. Bergquist, P.E.
Project Manager
Phone No. (303) 773-3788

cc: Bruce Brinkman - WWDC, Project Manager
Jeff Pecenka - BOPU, Project Manager
Bruce Perryman - AVI
Phillip Velez - SEO, Water Rights Supervisor

RJB:ljj