RAWLINS RAW WATER STORAGE LEVEL II PHASE III REPORT

Prepared for:
Wyoming Water Development Commission

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RAWLINS RAW WATER STORAGE LEVEL II,
PHASE III REPORT

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**Level II, Phase III Report**

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1.0 EXECUTIVE SUMMARY

1.1 Objective and Background

The Rawlins Raw Water Storage Project, Level II is a Wyoming Water Development Commission study to evaluate the feasibility of constructing a raw water storage reservoir. Phase II of the study has already evaluated a reservoir adjacent to the existing Peaking Reservoir designated the Peaking II reservoir. This Phase II continuation study examines the Five-mile reservoir site, the replacement of the Atlantic Rim Pipeline, decommissioning of the Atlantic Rim Reservoir, and provides a conceptual solution to address the supply to the BLM Rim Lakes. Figure 1.1 shows the locations referenced in this report. For the comparison purposes of this report, the previous Phase II work will be referred to as “The Peaking II Study”, while the present work will be referred to as “The Five-Mile Study”.

1.2 Water Supply Evaluation

In 2006, WWC performed a water supply evaluation (The Phase I Study – Appendix to the Rawlins Raw Water Storage Level II (Peaking II Study), Phase I and II Report, WWC, 2006), which examined several water supply and water demand scenarios. The study identified the need either for additional reliable storage or preparation for assertive pumping to address projected shortfalls in peak water use during drought events. The water supply evaluation utilized a 50 year planning horizon. The relationship between the available supplies and the estimated demands is shown on Figure 1.2. Several weaknesses in the Atlantic Rim reservoir were also identified, including on-going leakage and the inability to store water from the Platte River. Therefore, the study team made a recommendation to replace the Atlantic Rim reservoir.

1.3 Reservoir Evaluation

The Peaking II Study included a conceptual design for a 500 acre-foot reservoir at the Peaking II site, which is adjacent to the existing Peaking reservoir. The present study modifies and extends the initial study findings to bring the 500
acre-foot concept design up to 644 acre-feet. This was done in order to completely replace the Atlantic Rim Reservoir and enable a direct transfer of the storage rights associated with that facility. A cost estimate for the Peaking II site (reservoir construction only) is $11.1 Million.

The present study now also provides a 644 acre-foot concept plan for the Five-mile site, consisting of an earth dam on the east side in the naturally occurring drainage area just north of the Five-mile ridge. The locations and configurations for the two reservoirs are shown on Figure 1.1. Designs for the Five-mile site include provision for through drainage from the small drainage area above the site, as well as identification of a need for wetlands permitting. The cost estimate for the Five-Mile site is $10.5 Million.

The two primary sites were compared using similar design criteria including dam construction, inlets and outlet works, clay lining, monitoring, wave run-up and riprap protection, security fencing, access, and stability criteria. Intangibles have been identified, including proximity and staffing costs, security, and land ownership issues, among other factors.

1.4 Decommissioning of Atlantic Rim Reservoir

The Atlantic Rim Reservoir is currently located on BLM property. During this investigation no third parties expressed an interest in taking over the facility, although several mining interests did consider. The BLM requires that the site be restored to its former state, but some of the materials such as riprap and clay materials have been incorporated into the planning for a new reservoir site. The estimated cost for decommissioning Atlantic Rim is $420,000.

1.5 Water Supply of Rim Lakes

The current leakage from the Atlantic Rim Reservoir provides a baseline flow which supplies the two Rim Lakes. The BLM has built recreational facilities at the Rim Lakes, which include park amenities and fishing. They would like to maintain at least one of the Rim Lakes as a recreational fishing lake. An option to extend a supply pipeline to lakes, with a meter and control valve would cost an estimated $490,000. The recreational nature of this improvement allows for some
additional options for financing. Also, while the BLM was unable to commit to a payment obligation at this time for purchasing raw water, there may be a possibility of collecting for this water in the future.

1.6 Replacement of Atlantic Rim Pipeline

The Peaking II Study presented a conceptual design for a pipeline that would replace the existing pipeline between Atlantic Rim Reservoir and the water treatment plant. Some evidence of the need for the pipeline replacement are the pipe failures reported by the staff and collaborated by the failed pipeline sections strewn alongside the pipeline at several locations. During this current study, there has not been enough additional investigation to definitively recommend pipeline replacement. We believe that there are opportunities to use the existing line in some way; however, for planning purposes we assume total replacement. This pipeline is a critical link in the City water supply under all possible scenarios, so a line (or lines) with adequate capacity are needed. As a preliminary planning estimate, the cost of replacing or installing a parallel pipeline from Atlantic Rim to the treatment plant is estimated at $4.0 Million.

1.7 Project Financing

The costs for developing the Five-mile site are currently estimated to be $600,000 less than the Peaking II site, but the Peaking II site has some operating and security advantages and is located on property which would be relatively easy to obtain. In contrast, no resolution with Anadarko has been reached for the obtainment of their property, and the City might have to use a condemnation procedure, resulting in probable delays and political complications. Table 1.1 summarizes the estimated costs for all the improvements previously discussed.

Assuming WWDC participation with a 67% grant and 33% loan, the annualized cost of constructing the Peaking II reservoir and related projects is about $306,000. This annual cost translates to an approximate user fee increase of $6.74 per month/water tap. An alternative to the reservoir construction has always been to pump from the Platte River; however, the average cost of doing
this is calculated at $6.70 per month/water tap but could vary greatly, depending on energy costs and severity of drought.

1.8 **Recommendations**

The team recommends that the City apply for a WWDC-funded Level III final design of Peaking II to include the following tasks:

- Provide an engineered design and a bid package for a reservoir at the Peaking II site, including optimizing the configuration of the reservoir and its operating pool elevation to best serve the system, and performing additional engineering analyses and value engineering that could potentially identify cost savings for reservoir construction.

- Complete an engineered design and bid package for decommissioning and reclaiming the Atlantic Rim Reservoir. Important considerations will include phasing this work so as to avoid wetlands permitting issues.

- Provide an engineered design and bid package for provision of an alternative water supply for the Bureau of Land Management’s Rim Lakes which were previously supplied partly from leakage of the Atlantic Rim Reservoir.

- Further evaluate the need for upgrades or replacements to the Atlantic Rim pipeline. At this time, complete replacement is a conservative recommendation.
### Table 1-1 Financing Summary

<table>
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<tr>
<th>Project</th>
<th>Estimate</th>
<th>67% WWDC grant</th>
<th>Loan pmt, Annualized¹</th>
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1. Assumes WWDC financing of 33% of project cost at 4% for 30 years, includes operating costs for pumping option.
2. Based on 3,605 Rawlins system taps + 283 Sinclair system taps billed at 60% per sharing agreement.
3. 2006 estimate + 6% (to 2008)
NOTES:
CENTER BAR WITHIN THE WATER RIGHTS BOXES REPRESENT THE ANNUALIZED AVAILABILITY.
MILLER HILL AND SAGE CREEK, ON AVERAGE CAN MEET THE PRESENT AVERAGE DEMAND ON AN ANNUAL BASIS BUT REQUIRE STORAGE OR PUMPING TO MEET THE PEAK DEMAND.

FIGURE 1-2
PROJECTED DEMANDS AND SUPPLIES
RAWLINS RAW WATER STORAGE LEVEL II
PHASE III REPORT
CARBON COUNTY, WYOMING
2.0 INTRODUCTION

2.1 Background

The City of Rawlins has municipal sources of water from three diversified sources, including snow melt collection from the Sage Creek Springs, the Miller Hill wells, and water from the North Platte River. Previous study of the system (Phase I Level I supply study, WWC, 2006 – Appendix to the Peaking II Study) has confirmed the reliability of the supplies, while identifying some areas of potential concern, including water storage and the reliability of the transmission pipeline.

In the past, the City has diligently evaluated their water supply system capability and taken action when infrastructure improvements were shown to be needed. In 1983, the City completed a comprehensive water supply master plan (Montgomery, 1983) that examined the relationship between increasing water demands and the need for system improvements. This work resulted in the expansion of the City’s resources through a new well field and the replacement/enlargement of the critical water transmission pipeline to the Sage Creek Basin. In 1997, the City again examined their supply capability and strengthened it with the reconstruction of their North Platte River water supply pipeline (WWC, 1997). A complete list of references appears at the end of this report.

The 2006 Phase I, Level I supply study (Appendix to the Peaking II Study) considered recent drought conditions, and took into account projected changes to City demographics and population. Currently, there is a substantial development of energy resources in the area, and an increase in temporary workforce is noted. There is at present also a planned update (supported by the WWDC) to the City Water Master Plan (Montgomery, 1983) which will include a study of possible water reuse systems as well as potential for water conservation for the City.
2.2 **Objectives of the Peaking II Study**

The Peaking II Study evaluated the feasibility of increasing raw water reservoir storage to maintain water system reliability under future changes in water supply and/or water demand. The objectives of the evaluation included the following:

1. To estimate the hydrologic availability of water resources and the physical and legal constraints on those resources.
2. To examine technical issues regarding the siting and design of raw water reservoir storage at various locations, based on geotechnical, environmental, financial and other factors.
3. To develop a conceptual design for a reservoir project, suitable for Level III funding.

2.3 **Project Phasing**

The initial Peaking II Study included two phases. The Phase I study examined the legal and physical framework for building a reservoir and looked at numerous alternatives. The Phase I report was not separately issued but was included as an appendix to the Peaking II Study. The Phase II work included in the Peaking II Study was directed at preparing a detailed conceptual reservoir design for the Peaking II site, located adjacent to the Peaking Reservoir.

The current Level II continuation which we here refer to as the Five-Mile Study is a detailed study of the Five-Mile site identified as the most likely among the various options previously examined in the Phase I study. The preliminary design of a Five-Mile Reservoir was carried out to a comparable level of the previous Peaking II study so that a comparison of the two sites could be achieved.
3.0 SUMMARY OF THE PHASE I RESERVOIR EVALUATIONS

3.1 Introduction

This chapter presents a brief summary of the Phase I work which was presented in Appendix B of the Peaking II Study. During the Phase I Study, several water supply alternatives were considered. Among the alternatives were modifications to existing reservoirs, using more aggressive pumping from the Platte River, and several locations for a possible new reservoir.

The Phase I reservoir evaluation work demonstrated that additional water supply storage was reasonably feasible with regard to the following issues:

- Water supply and demand-The water supply evaluation showed that there are conditions of demand and supply that result in the need for water storage and/or operational changes (assertive pumping). The City has expressed a desire to pursue alternatives that reduce or eliminate the need to frequently resort to the pumping alternatives, preferring to expend funds on capital improvements rather than less predictable pumping costs.

- Geotechnical- The geotechnical evaluation work has shown that there are several sites that could be used for reservoir storage. The Peaking II Reservoir and the Five-mile Ridge Reservoir sites are both viable options, though both show a need for lining and other methods for minimization of seepage or leakage.

- Financial-The financial evaluation showed that the City has the ability to finance a water reservoir construction project. Although household incomes in Rawlins are below state average, so are the water rates. Furthermore, the financial evaluation work also showed that the financial impact to water users would be approximately the same if assertive pumping were used (relatively high operating cost) versus the construction of additional reservoir storage (low operating cost but high capital cost).

Due to the problems at Atlantic Rim Reservoir, it appears that lining and continued monitoring of this facility would be required regardless of other options taken for water supply. More discussion of this issue appeared in Chapter 5 of the
Peaking II Study and is discussed in Chapter 6 of this report. The present study will address additional issues related to taking the Atlantic Rim Reservoir out of service.

3.2 Reservoir Site Options

Five 500 ac-ft. reservoir sites were evaluated (see figure 3-1) considering geological/geotechnical conditions. Conceptual cost estimates were developed for the options deemed practical. The options considered were:

Option 1: Enlarging Peaking Reservoir No. 1 by raising the existing embankments. This was ultimately found not to be practical to achieve the recommended 500 additional acre-feet of capacity.

Option 2: Peaking II Reservoir consisted of constructing a new 500 ac-ft. reservoir adjacent to Peaking No. 1 located on property partially owned by the Bureau of Land Management.

Option 3: Enlarging Atlantic Rim was considered, but ultimately not pursued due to the structural problems and leakage issues identified by the Geotechnical experts. In addition, toward the end of the study period, it was suggested that taking Atlantic Rim out of service would eventually become necessary, and adjustments were made to the supply model to reflect this change.

Option 4: Five-mile Ridge Reservoir consists of constructing a new reservoir and dam in the Five-mile drainage area southwest of Peaking Reservoir on property owned by Anadarko Land Resources Company (formerly UPRR). This site was identified as likely having the lowest development cost due to taking advantage of the natural terrain to minimize earthwork.

Option 5: A below grade basin near the Peaking Reservoir would minimize embankment heights, but the depths to bed rock caused us to drop this from consideration.
3.3 **Phase I Findings**

The Phase I work recommended that a Phase II scope of work, including field investigations, proceed at the Five-mile Ridge Reservoir site for the following reasons:

- Least expensive conceptual reservoir option.
- Hydraulic benefits with similar or better storage elevation than the existing Peaking Reservoir.
- Uses existing pipeline corridor.
- The North Platte pumping system can probably be configured to pump to this site without changing pumps, although additional pipe would be needed. A slight increase in operating pressures at the Thayer pump station needs to be further studied to see if any changes would be required to this facility. A reconfiguration of the booster pumps at the treatment plant could also assist the passage of water from Thayer to the new reservoir, although a detailed study of these issues should be part of the scope for the final design considerations.
4.0 PHASE II FIVE-MILE RESERVOIR DESIGN

4.1 Introduction

As presented at the end of the last section, the Five-mile Ridge Reservoir site was a preferred option for field investigations under Phase II. Therefore, WWC Engineering approached the surface owners of the Five-mile Ridge Reservoir site for ingress and egress to perform field investigations. Unfortunately, permission for ingress and egress was not obtained in time to allow for the field work and reporting to be completed in advance of the WWDC’s August 15th, 2006 funding application deadline. The Peaking II alternative was studied instead. The Peaking II site has many of the same advantages listed previously for the Five-Mile Site, with the exception that there is no natural depression to reduce earth moving quantities.

Access to the Five-Mile site was obtained from Anadarko Land on August 9, 2007, allowing more detailed study of this option, including survey work and geotechnical explorations. WWC Engineering performed surface survey work during the week of August 13, 2007 and the geotechnical field investigation was done by RJH Consultants on September 18, 2007. The reservoir layout was refined based on the field data.

The geotechnical consultants have determined that foundation conditions at the Five-Mile site are similar to Peaking Reservoir, and a liner would be necessary for either Five-Mile or Peaking II. The present proposed clay liner system is different from the system previously proposed for Peaking II, and has been applied to both reservoirs.

The Five-Mile reservoir would use the same pipeline route as the existing system, but would have only a portion available for gravity transfer to the existing Peaking Reservoir. WWC also recommends that a delivery pipeline (16" PVC) be installed from the terminus of the supply line from the Platte River to allow for storage and blending of the Platte River water. This additional piping is one of the differences in cost between the previously studied (and now updated) Peaking II site and the Five-Mile site. An analysis of the hydraulics for the new site indicate that the proposed storage at Five-
Mile site has a somewhat higher energy potential than the existing Peaking Reservoir, although the very lowest levels of the reservoir lack sufficient elevation head for the water to arrive at Peaking by gravity feed. The useable storage range for the reservoir would be limited to water stored above the elevation of the highest point on the pipeline between the site and the treatment plant. There would be a potential gain of approximately 4 feet of useable storage if the replacement line between Five-Mile and Peaking were increased from the proposed 24" to a 30" diameter line, due to a reduction in head loss from a larger line.

The costs, including the new pipe routing, ultimately are similar to the costs for the Peaking II site, however, so both options will be carefully considered. Both site options also account for monitoring instrumentation, construction of outlet works, and providing an access road. We have also considered as a part of our design the need for security fencing as recommended by Homeland Security.

4.2 Conceptual Design

4.2.1 Reservoir Size

Based on the previous studies and the possibility of transferring the storage right from the Atlantic Rim Reservoir to this new reservoir, the Five-Mile site was designed to hold approximately 644 acre-feet. The design in a natural draw results in a deeper pool than what is possible with the Peaking II design. At a high water level of 7165 feet, the reservoir surface area would be approximately 33 acres, with a maximum depth of 65 feet over the old stream channel near the dam.

The 100 to 200 acre-foot volume should be added to the 500 acre-feet recommended in the Water Supply Evaluation (Chapter 3), which recommended adding a reservoir of 500 acre-feet to assure the present level of reliability through drought and growth scenarios. The resulting size is conveniently similar in capacity to the present permitted storage for Atlantic Rim Reservoir at 644 acre-feet. It appears that preserving the present storage right would align with the projected needs of the City.
4.2.2 Reservoir Design

The preliminary design layout for the 644 acre-foot reservoir at the Five-Mile site is shown in Figure 4-1. The design consists of a dam just west of the present pipeline and pipeline access track in the draw to the north of the ridge designated as Five-Mile Ridge. The geotechnical study indicated that on average, between 5 and 10 feet of the surface material could be removed in the reservoir area and used for the dam, to create a near balance of cuts and fills. The proposed high water elevation was chosen just below and to the East of the Right-of-way for State Highway 71. This elevation (7165 feet) was the highest possible without impacting the highway, which would result in complications (highway relocation) that would be better avoided, or could require the installation of a more expensive upstream embankment, negating the cost effectiveness of the site.

On the East side of the proposed reservoir, an embankment dam up to 70 feet in height will be constructed across the drainage. The crest of the embankment is at 7170 feet with a reservoir high water level (HWL) of 7165 feet. The upstream slopes will be a 3H:1V with a 20 foot wide crest and a 2H:1V downstream slope. The upstream or wet side of the embankment would be protected with a riprap layer over a granular bedding material that could be screened from the on-site materials.

In addition to the riprap protection, the current dam concept calls for the clay liner to extend under the dam and transition to a cut-off wall forming the center or core of the embankment. Downstream of the cut-off wall, a two-stage sand filter wall would relieve pore pressure within the dam and provide a method for monitoring any leakage. The sand filter would discharge to a system of pipes to capture and transmit leakage water to a control point at the deepest section of the dam. The 1st stage of the sand filter would likely be an imported material, while the second stage material could be screened from the on-site granular materials.

The bottom of the reservoir will be excavated approximately 8 feet deep to gain additional reservoir capacity and to provide earthen material for construction of the embankment. Once the reservoir and embankment foundations have been
excavated to final grade, the rock surfaces should be cleaned and inspected, and large fractures and joints (larger than about ½-inch wide) should be slush grouted. The reservoir bottom will then be lined with a 3 foot thick clay liner consisting primarily of material from the Atlantic Rim borrow source. Figure 4-2 displays two typical cross-sections through the proposed embankment and cut slopes.

Primary dam construction material will consist of the naturally occurring weathered sandstone and siltstone. Material over 6 inches in size will be removed prior to embankment construction, and can be used for riprap at the surface. Embankment material should be compacted to at least 95% of the maximum dry density at moisture contents ±2% of optimum, as determined by ASTM D-1557.

The present design uses on-site riprap for surface protection. An alternative using an exposed geomembrane system was also considered, but the cost of this system has since risen to the point where it is probably not a viable option. The synthetic membranes are subject to fluctuations in the cost of petroleum.

4.2.3 Reservoir Outlet and Inlet Piping

WWC Engineering met with the water treatment plant operators to discuss facilities for filling and draining the proposed reservoir. A conceptual design for those facilities is included on Figure 4-3. Note that the operators preferred the proposed reservoir be operable in parallel with the existing Peaking (I) reservoir, with interlinked hydraulics based on equal design highwater levels. The different design highwater levels can be controlled by means of the outlet and inlet controls. Each of the reservoirs can be operated independently of the other, including supply of the treatment plant from either reservoir if one of the reservoirs needs to be taken out of service for maintenance.

The outlet for the new reservoir would be similar in style to the existing Peaking (I) Reservoir, with a free standing tower intake in the reservoir. The intake would have ports at several elevations to allow for selective water withdrawal. The outlet conduit would be buried below the bottom of the reservoir, backfilled with concrete and routed to connect to the existing inlet/outlet pipeline at a new valve vault at the east side of the proposed embankment. The Atlantic
Rim Pipeline would run through this vault on its way to the Peaking Reservoir. Unfortunately, given the elevations of the proposed reservoir, not all of the desired capacity of the reservoir would flow by gravity to the Peaking Reservoir. An analysis of the elevation and piping losses shows that approximately 20% of the proposed Five-Mile reservoir capacity would fall below the peak usability elevation which would deliver water to the treatment plant intake at the 20 psi working pressure requested by the operator. The desired elevation to supply the requested residual pressures comprises the upper 25 feet of the reservoir capacity. The lower elevations can supply the plant at lesser pressures and volumes drawing down to elevation 7130 (leaving 7% dead capacity). This scenario, however, would require an increase of the delivery pipeline from the 24" proposed diameter, to 30", to reduce the hydraulic losses when supplying full plant capacity. We note that the existing Peaking reservoir can only supply this working pressure for approximately the upper 10 feet of reservoir capacity. A similar analysis for the Peaking II reservoir appears in chapter 5.

Water from the Sage Creek Springs would come through the Atlantic Rim Pipeline, through the described vault, then could be diverted into the proposed reservoir through the outlet/inlet conduits. The new inlet/outlet conduit would be a minimum of 24 inch diameter, though a 30" diameter inlet/outlet pipe diameter would be advantageous. Final design calculations would have to demonstrate that the outlet could drain the reservoir sufficiently fast so as to meet dam safety evacuation criteria.

WWC also discussed with water treatment plant operators the option for providing a fine screening arrangement at the intake tower to reduce the inflow of small materials into the plant. However, plant operations and finished water quality would not appear to improve with these intake screens, and so that concept was not evaluated further.

An additional inlet piping system would originate at a tee off the North Platte Pipeline as shown in Figure 5-3. The pipe would be 16-inch diameter, one size smaller than the North Platte Pipeline. At this connection, gate valves would
be installed so that the City could control which of the two reservoirs would receive North Platte River water.

More complex motor operated valves could be installed to direct water to one reservoir or the other, or both, but they are not directly included in the conceptual design cost estimate presented later in this chapter. The inlet end of the inlet piping in the reservoir would be located as far as practically possible from the outlet to reduce the probability of short circuiting. Isolation of one reservoir for repairs while using the other is provided for in the design, though the ability to transfer water from one reservoir to the other is limited since they are hydraulically at similar levels though the upper 16 feet of Five-Mile is hydraulically higher than the Peaking Reservoir high water level.

4.2.4 Overflow Protection Spillway

For the Five-Mile option, there is a drainage area that must be accommodated with an overflow spillway in the design. The drainage area is 195 acres, including the reservoir itself, resulting in a requirement to allow for 3,441 cfs per the results of the Trihydro 98 calculation based on the maximum predicted rainfall event (19") indicated by Becky Matheson at the State Engineer's office. This rainfall event was based on the HMR55a method, but was not re-verified for this study. Allowing for pond routing and the use of up to three feet of the required 5 feet of freeboard on the dam, this flow could easily be passed with a riprap protected spillway on the order of 35 to 50 feet wide, which has been included to give an estimate of the cost of this feature. The conceptual spillway includes a weir and heavy riprap, as well as a riprap stilling basin at the downstream end. The stilling basin can also serve as a discharge basin when using the outlet works to drain the bottom of the reservoir since approximately 7% of the reservoir cannot drain to the treatment plant by gravity.

4.3 Water Rights Considerations

The City of Rawlins has an adjudicated water right for Atlantic Rim Reservoir, Permit 8016 Res, for 644.5 acre-foot of capacity with a priority date of July 20, 1978. The City also has reservoir supply water rights to fill this reservoir
from diversions from Sage Creek and from the City’s Sage Creek basin network of springs. This reservoir has been used to temporarily re-regulate the supply of water from these sources and groundwater from the Miller Hill wellfield in combination with Rawlins Peaking Reservoir, to meet the municipal demands for water.

In order to maintain and protect the City’s full portfolio of existing water right assets, and for the technical considerations elaborated elsewhere in this report, WWC recommends that the City pursue the transfer of the location of the entire storage water right for the Atlantic Rim Reservoir to the proposed new reservoir site. This will require a petition to the State Board of Control, seeking approval for the change in location of the storage reservoir water right. The State Engineer’s office has indicated that this requested change will be more beneficial, and procedurally simpler than seeking a new reservoir water right for a new water storage facility within the North Platte River Basin.

The benefits of this approach are; first, the City will maintain the historic use and existing priority date for this reservoir facility. Second, it appears from the water supply reliability analysis that at least 500 AF of storage would be required. Third, the City will be continuing the use the reservoir for the same historic beneficial purposes, and to provide both year-to-year carry-over storage and seasonal re-regulation benefits of the existing set of municipal water supplies. As a consequence of the continued increasing nature of enforcement, administration and regulation of water rights in the North Platte River system, WWC recommends that the City continue to fully exercise and protect these existing appropriations of water, rather than to seek new permits for such a facility.

As a part of the recommended water right petitioning process for Atlantic Rim Reservoir, the City should also seek to clarify and describe the re-regulation operations of the City’s North Platte River water supplies since one of the benefits of the preferred alternative reservoir site is that it will allow for the management of the entire portfolio of municipal water supplies, including the North Platte River, along with the Miller Hill wellfield and the Sage Creek and Sage Creek Springs systems.
4.4 Cost Estimate

The total project cost estimate for the Five-Mile site along with associated appurtenances is shown in Table 4-1. For comparison, the updated cost of the Peaking II reservoir is also shown. The detailed estimates show a projected cost of the Five-Mile site reservoir of $10,500,000. The unit price data was obtained from bid tabulations from recent projects and the Means cost estimating publication.

4.5 Clearances, Easements and Permits

As part of the previous work (the Peaking II Study), a Class III Cultural Resource Inventory was completed for the Peaking II reservoir site. The report on this work, included as Appendix C to that study, found no cultural materials and recommended that clearance for construction be given. The Five-Mile site is near enough and similar enough to the original Peaking II site, that it is assumed for the purposes of this work that no additional impacts will be found for this site. If this site is developed in the future, a formal study would have to be performed.

Also as part of the previous study (Peaking II study), a Biological Resources Clearance was completed for the Peaking II site. The report presents the results of surveys for threatened, endangered, proposed and candidate species, BLM sensitive species and wetlands and other waters of the U.S. The report also provides information on big game winter range. The report on this work, included as Appendix D in that report, recommends further consultation with BLM on surveys for some species, although in general the report does not identify any significant biological issues that would impact project feasibility. The same species would likely impact both the Five-Mile and the Peaking II sites.

The Peaking II site is located on a ridge line away from drainages and thus is unlikely to have any wetlands. The Five-Mile site is located in a natural draw which does apparently have some ephemeral flows and would have to be more carefully evaluated for wetlands impacts. In either case, it is not projected that wetlands will be a significant factor in permitting these sites. Preliminary discussions with the U.S. Army Corps of Engineers (USACOE) have indicated
the filling in the Five-Mile draw would require a jurisdictional determination and possibly a permit.

Anadarko Land Resources Company (formerly UPRR) owns the property at the Five-Mile site. In the past, Anadarko has agreed to give properties to the public entities in exchange for good will and tax benefits. At present, it is unknown whether such a deal could be worked out. If the property were purchased at fair market value (approximately $2000/acre) we would have to assign a cost for 80 acres of about $160,000. WWC has assigned such a cost to the estimate as it is still uncertain what position Anadarko will take.
### TABLE 4-1 - RESERVOIR CONCEPTUAL DESIGN PROJECT COST ESTIMATE

<table>
<thead>
<tr>
<th>Item</th>
<th>Unit</th>
<th>Quantity</th>
<th>Quantity</th>
<th>Cost/Unit</th>
<th>Total Cost</th>
<th>Total Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mobilization and Bonds (% of Items)</td>
<td>10%</td>
<td>$674,759</td>
<td>$714,937</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 Reservoir (3:1 upstream, 2:1 downstream)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a. Cut and Fill</td>
<td>CY</td>
<td>280,000</td>
<td>472,051</td>
<td>$5</td>
<td>$1,400,000</td>
<td>$2,360,255</td>
</tr>
<tr>
<td>b. Cleaning and Slush Grouting of Large Joints/Fractures</td>
<td>AC</td>
<td>34.0</td>
<td>30.9</td>
<td>$8,091</td>
<td>$275,094</td>
<td>$250,012</td>
</tr>
<tr>
<td>c. Gravel (on site, sifted) over clay liner (9&quot;)</td>
<td>CY</td>
<td>169,000</td>
<td>37,380</td>
<td>$5</td>
<td>$845,000</td>
<td>$186,900</td>
</tr>
<tr>
<td>d. Clay Liner Installed (3 ft. thick)</td>
<td>CY</td>
<td>195,000</td>
<td>149,523</td>
<td>$7</td>
<td>$1,365,000</td>
<td>$1,046,661</td>
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<tr>
<td>e. 3' thick Sand/Drainage Layer 1 Placement</td>
<td>CY</td>
<td>16,116</td>
<td>35,170</td>
<td>$21</td>
<td>$338,433</td>
<td>$738,573</td>
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<tr>
<td>f. 3' thick Sand/Drainage Layer 1 Placement</td>
<td>CY</td>
<td>15,446</td>
<td>33,633</td>
<td>$21</td>
<td>$324,372</td>
<td>$706,288</td>
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<tr>
<td>g. 6&quot; Rip-Rap bedding (on-site, sifted)</td>
<td>CY</td>
<td>8,000</td>
<td>19,000</td>
<td>$7</td>
<td>$56,000</td>
<td>$133,000</td>
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<tr>
<td>h. Riprap Placement (from Atlantic Rim)</td>
<td>CY</td>
<td>0</td>
<td>0</td>
<td>$50</td>
<td>$0</td>
<td>$0</td>
</tr>
<tr>
<td>i. Riprap Placement (from quarry site)</td>
<td>CY</td>
<td>3,000</td>
<td>0</td>
<td>$7</td>
<td>$21,000</td>
<td>$0</td>
</tr>
<tr>
<td>j. 50' Spillway riprap 1 ft. thick</td>
<td>CY</td>
<td>40</td>
<td>0</td>
<td>$1,000</td>
<td>$40,000</td>
<td>$0</td>
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<tr>
<td>k. Spillway control structure concrete</td>
<td>CY</td>
<td>6,750</td>
<td>4,610</td>
<td>$50</td>
<td>$337,500</td>
<td>$230,500</td>
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<tr>
<td>l. Security Fencing (incl. gates, etc.)</td>
<td>LF</td>
<td>200</td>
<td>130</td>
<td>$1,000</td>
<td>$200,000</td>
<td>$130,000</td>
</tr>
<tr>
<td>3 Intake Structure</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a. Concrete(Slab and Tower)</td>
<td>CY</td>
<td>200</td>
<td>130</td>
<td>$1,000</td>
<td>$200,000</td>
<td>$130,000</td>
</tr>
<tr>
<td>b. Piping (Inside structure up to 90° bend under foundation)</td>
<td>LF</td>
<td>10</td>
<td>10</td>
<td>$164</td>
<td>$1,640</td>
<td>$1,640</td>
</tr>
<tr>
<td>c. 42&quot; x 42&quot; Heavy Duty Sluice Gate</td>
<td>EA</td>
<td>3</td>
<td>3</td>
<td>$16,000</td>
<td>$48,000</td>
<td>$48,000</td>
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<tr>
<td>d. Trash racks</td>
<td>EA</td>
<td>3</td>
<td>3</td>
<td>$6,700</td>
<td>$20,100</td>
<td>$20,100</td>
</tr>
<tr>
<td>e. Walkway from Structure to Embankment</td>
<td>EA</td>
<td>1</td>
<td>1</td>
<td>$100,000</td>
<td>$120,000</td>
<td>$100,000</td>
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<tr>
<td>f. Guardrail</td>
<td>LF</td>
<td>48</td>
<td>40</td>
<td>$75</td>
<td>$3,600</td>
<td>$3,000</td>
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<tr>
<td>4 Utility Relocation (Rocky Mt. Power-Line)</td>
<td>LS</td>
<td>1</td>
<td>0</td>
<td>$100,000</td>
<td>$100,000</td>
<td>$0</td>
</tr>
<tr>
<td>5 Piping (Intake Structure to Existing Peaking Res. Outlet)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a. 24&quot; Steel Pipe, 300#</td>
<td>LF</td>
<td>650</td>
<td>1,400</td>
<td>$164</td>
<td>$106,600</td>
<td>$229,600</td>
</tr>
<tr>
<td>b. Increase proposed 24&quot; pipe to 30&quot;</td>
<td>LF</td>
<td>3,631</td>
<td>0</td>
<td>$25</td>
<td>$90,775</td>
<td>$0</td>
</tr>
<tr>
<td>c. Appurtenances</td>
<td>LS</td>
<td>1.2</td>
<td>1</td>
<td>$25,000</td>
<td>$30,000</td>
<td>$25,000</td>
</tr>
<tr>
<td>d. Concrete Backfill</td>
<td>CY</td>
<td>208</td>
<td>130</td>
<td>$350</td>
<td>$72,800</td>
<td>$45,500</td>
</tr>
<tr>
<td>6 Piping (Platte River Pipeline to Reservoir)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a. 16 PVC C905</td>
<td>LF</td>
<td>3316</td>
<td>1800</td>
<td>$85</td>
<td>$281,860</td>
<td>$153,000</td>
</tr>
<tr>
<td>b. Appurtenances</td>
<td>LS</td>
<td>1</td>
<td>1</td>
<td>$25,000</td>
<td>$25,000</td>
<td>$25,000</td>
</tr>
<tr>
<td>c. Concrete (Inlet Structure Pad)</td>
<td>CY</td>
<td>4</td>
<td>4</td>
<td>$350</td>
<td>$1,400</td>
<td>$1,400</td>
</tr>
<tr>
<td>9 Unlisted Items (10% of Items 2-7)</td>
<td>LS</td>
<td>1</td>
<td>1</td>
<td>10%</td>
<td>$613,417</td>
<td>$649,943</td>
</tr>
<tr>
<td>A Construction Cost Subtotal</td>
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<td></td>
<td></td>
<td></td>
<td>$7,422,350</td>
<td>$7,864,309</td>
</tr>
<tr>
<td>B Engineering: Construction Supervision Costs (10% of A)</td>
<td>10%</td>
<td>$742,235</td>
<td>$786,431</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C Subtotal (A+B)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$8,164,585</td>
<td>$8,650,740</td>
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<tr>
<td>D Contingency (15% of C)</td>
<td>15%</td>
<td>$1,224,688</td>
<td>$1,297,611</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>E CONSTRUCTION COST TOTAL (C+D)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$9,389,273</td>
<td>$9,948,351</td>
</tr>
<tr>
<td>F Prepare Final Design and Specs (10% of E)</td>
<td>10%</td>
<td>$938,927</td>
<td>$994,835</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>G Permitting and Mitigation</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$10,000</td>
<td>$10,000</td>
</tr>
<tr>
<td>H Legal Fees</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$10,000</td>
<td>$10,000</td>
</tr>
<tr>
<td>I Acquisition of Access and ROW</td>
<td>AC</td>
<td>71.5</td>
<td>26.8</td>
<td>$2,000</td>
<td>$143,000</td>
<td>$53,600</td>
</tr>
<tr>
<td>PROJECT TOTAL COST</td>
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<td></td>
<td></td>
<td></td>
<td>$10,491,200</td>
<td>$11,016,786</td>
</tr>
<tr>
<td>ROUNDED TOTAL COST</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$10,500,000</td>
<td>$11,100,000</td>
</tr>
</tbody>
</table>

9/19/2008
5.0 PEAKING II RESERVOIR DESIGN UPDATE

5.1 Introduction

As presented at the end of Chapter 3, the Five-Mile Ridge Reservoir site was the preferred option identified in the Phase I study (Appendix to the Peaking II Study) recommended for field investigations under Phase II. WWC Engineering approached the surface owners of the Five-Mile Ridge Reservoir site for ingress and egress to perform field investigations. Unfortunately, permission for ingress and egress was not obtained in time to allow for the field work and this reporting to be completed as fast as the City of Rawlins needed the information. The City was very interested in having the draft reporting completed well in advance of the WWDC’s August 15th, 2006 funding application deadline.

This Chapter presents a revised design for a reservoir at the Peaking II site adjusted for the current capacity requirements. The capacity was increased from 500 acre-feet to the 644 acre-feet as previously discussed, and the construction design has been updated based on the findings of the current geotechnical team. Further analysis of the functionality of the reservoir and the hydraulics is also presented here.

5.2 Conceptual Design

5.2.1 Reservoir Size

The previous conceptual design for the Peaking II reservoir was for a 500 acre-foot capacity, based on the recommendations of the Phase I study (appendix in the Peaking II Study). To achieve the secondary objective of full transfer of the 644 acre-foot storage right from decommissioning the Atlantic Rim Reservoir, the conceptual design had to be updated to allow for 644 acre-feet of storage.

5.2.2 Reservoir Design

A 644 acre-foot reservoir was designed for the Peaking II site, shown in Figure 5-1. The design consists of a quasi-rectangular reservoir adjacent to the existing Peaking Reservoir and the City’s water treatment plant. The high point
for the reservoir is on the southeast side at 7160 feet and gently slopes down to 7115 feet on the northwest side. Along the northeastern side of the reservoir, the site also slopes downward into a small drainage that empties into the gulch below.

The crest of the embankment would be at elevation 7165 feet with a high water level (HWL) of 7160 feet. The upstream (inside or wet) design slopes are 3H:1V with a 20 foot wide crest and a 2H:1V downstream (outside or dry) slope. The reservoir would be impounded by a cutslope on the southeast side into a shallow ridge. On the remaining sides, an embankment dam up to 50 feet in height will be constructed. On the southwest side, the Peaking II site will abut the existing Peaking Reservoir. The bottom floor of the reservoir will be excavated approximately 8 feet deep to gain additional reservoir capacity and provide earthen material for construction of the embankment. Once the reservoir and embankment foundations have been excavated to final grade, the rock surfaces should be cleaned and inspected, and large fractures and joints (larger than about ½-inch wide) should be slush grouted.

The reservoir bottom is lined with a 3 foot thick clay liner consisting of material from the Atlantic Rim borrow source. Figures 5-2 displays typical cross-sections through the proposed embankment and cut slopes. A riprap erosion protection lining was also utilized for the cost estimate. The estimate also accounts for monitoring instrumentation, modification of outlet works, installation of new supply pipe, and providing an access road.

A seismic study was performed on this site, in addition to examination of the geotechnical information gathered from the original construction of Peaking Reservoir. The current design recommends an increase in clay liner thickness but eliminates the synthetic liner incorporated into the original design.

Primary construction material will consist of on-site weathered sandstone and siltstone. Material over 6 inches in size will be removed prior to embankment construction and can be used for riprap. Embankment material should be compacted to at least 95% of the maximum dry density at moisture contents ±2% of optimum, as determined by ASTM D-1557.
5.2.3 Reservoir Outlet and Inlet Piping

WWC Engineering met with the water treatment plant operators to discuss facilities for filling and draining the proposed reservoir. A conceptual design for those facilities is included on Figure 5-5. Note that the operators preferred the new reservoir be operable in parallel with the Peaking I reservoir, with equal design high water levels. The current design shows different design high water levels which will be more complex to operate. The Peaking II reservoir would store the top 14 feet of water at a higher equivalent elevation than the high water level of Peaking reservoir, though the existing Sage Creek pipeline has plenty of residual hydraulic energy to fill this.

The outlet for the new reservoir would be similar in style to the existing Peaking Reservoir, with a free standing tower intake in the reservoir. The intake would have ports at several elevations to allow for selective water withdrawal. The outlet conduit would be buried below the bottom of the reservoir, backfilled with concrete and routed to connect to the existing inlet/outlet pipeline in the valve vault at the west side of the existing Peaking Reservoir. The Atlantic Rim Pipeline connects to this same existing vault just west of the Peaking Reservoir.

Water from the Sage Creek Springs would come through the Atlantic Rim Pipeline, through the described vault, then finally into each reservoir through the outlet/inlet conduits. To fill the proposed Peaking II reservoir to the upper levels, the valve for Peaking I would need to be closed or at least restricted. The new inlet/outlet conduit would be a minimum of 24 inch diameter. Final design calculations would have to demonstrate that the outlet could drain the reservoir sufficiently fast so as to meet dam safety evacuation criteria.

An additional inlet pipe would originate at a tee off the North Platte Pipeline as shown in Figure 5-1 to allow for supply into the reservoir from the North Platte pumping system. The pipe could be 16-inch diameter. At this connection, gate valves would be installed so that the City could control which of the two reservoirs would receive North Platte River water. The additional reservoir height at Peaking II, and the slight increase in hydraulic friction due to the 16" supply line would add about 8 psi of pressure at the Thayer booster
station. A detailed check of that facility against the operating pressure increase needs to be performed as part of any final design.

More complex motor operated valves could be installed to direct water to one reservoir or the other, or both, but they are not directly included in the conceptual design cost estimate presented later in this Chapter. The inlet end of the inlet piping in the reservoir would be located as far as practically possible from the outlet to reduce the probability of short circuiting. Isolation of one reservoir for repairs is provided in the design while independently operating the other reservoir to supply the system.

5.3 **Cost Estimate**

The total project cost estimate for the Peaking II Reservoir and its associated inlet/outlet piping is $11,100,000. Table 4-1 presents the detailed cost estimate, alongside the estimate for the Five-Mile site. The unit price data was obtained from bid tabulations from recent projects and the Means cost estimating publication.

5.4 **Clearances, Easements and Permits**

As previously mentioned, a Class III Cultural Resource Inventory was completed for the Peaking II reservoir site. The report on this work, Appendix G in the Peaking II Report, found no cultural materials and recommended the site be approved for development.

Also, a Biological Resources Clearance was completed for the Peaking II site. The report presents the results of surveys for threatened, endangered, proposed and candidate species, BLM sensitive species and wetlands and other waters of the U.S. The report also provides information on big game winter range. The report on this work, Appendix F in the Peaking II Report, recommends further consultation with BLM on surveys for some species, although in general the report does not identify any significant biological issues that would impact project feasibility.

Currently the BLM owns some of the land at the site for Peaking Reservoir II. WWC contacted the Rawlins BLM real estate department to
determine options for the City to acquire the land. Per conversation with Chuck Valentine, the following options exist for the City:

1. **Right-of-Way Grant**—The BLM could grant a right-of-way for reservoir construction/operation for a 30 year term, which is renewable. There would be little to no cost for this option. The water utility would also need access and transmission right-of-ways. This process for the BLM is neither automatic nor guaranteed, but can be done in a matter of months. Mr. Valentine indicated that other projects for WWDC (such as High Savery Dam) have been done using this mechanism. For a right-of-way grant, the BLM would require compliance with the NEPA process for environmental impacts, including involvement of the Wyoming SHPO, USACOE, U.S. Fish & Wildlife Service, Wyoming Game and Fish, FEMA, etc.

2. **Purchase Property**—Rawlins could purchase the property outright, but this would be a much longer process than the Option 1 and would require publication, notices, and it would have to be a competitive sale involving the risks of going to public auction.

3. **Long-Term Lease**—A long-term lease does not seem to be a preferred option by the agency (BLM) at this time.

If this location is chosen for the additional reservoir, WWC recommends that the City pursue Option #1 and obtain a right-of-way grant from the BLM for the construction and operation of the Peaking II site.
LEGEND

- TEST PIT
- CONTROL POINT
- WATER VALVE (PROPOSED)
- WATER VALVE (EXISTING)
- POWER POLE (EXISTING)
- WATER LINE (PROPOSED)
- WATER LINE (EXISTING)
- FENCE (EXISTING)

FIGURE 5–3
PEAKING II CONTROLS FOR RESERVOIR PIPING

RAWLINS RAW WATER STORAGE LEVEL II PHASE II REPORT CARBON COUNTY, WYOMING


6.0 ATLANTIC RIM RESERVOIR

6.1 Introduction

The Atlantic Rim Reservoir has a history of severe seepage problems that include very troubling evidence of internal erosion and piping in the foundation, as indicated by sand boils near the downstream toe of the dam (details in Appendix C of the Peaking II Report). The City also provided seepage analysis from weir measurements taken from April 1991 to February 1992 indicating a mean leakage rate from the Atlantic Rim Reservoir of 216,000 gallons per day (0.216 MGD). Because of the leakage, the City has been forced to operate the reservoir at levels well below its rated capacity.

Based on previous studies at the site, the seepage problems are believed to be caused by either: 1) the presence of soluble minerals in the foundation soils, or 2) the presence of highly weathered and fractured bedrock in the deeper foundation and exposed on the eastern margin of the reservoir. These adverse foundation conditions were first recognized over 20 years ago and the current project team believes it is likely that the shallow (soil) foundation, and possibly the dam embankment itself, may have been further compromised over time under long-term seepage forces. Ongoing dissolution of soluble minerals, and possibly additional internal erosion of fines through piping features or into fractured bedrock, could have opened or widened seepage pathways and softened or weakened the dam and its foundation over time.

Currently, Atlantic Rim Reservoir active capacity is limited to approximately the middle third of the total reservoir volume since the reservoir cannot be filled due to leakage problems, and the bottom third seems to be inaccessible, according to the operator, because of elevation constraints.

Although the conceptual estimated costs as generated in the Phase I (Appendix to Peaking II Report) evaluation for enlarging Atlantic Rim Reservoir appeared lower than the other options, those estimates assume the foundation of the existing facility is adequate to support continued use and enlargement of the reservoir. However, this is a very optimistic assumption, and it is likely that costs could be significantly higher for rehabilitating Atlantic Rim to ensure future dam safety.
The team estimated that an appropriately detailed geotechnical evaluation to evaluate the condition of the embankment and its foundation could cost between $150,000 and $250,000. Even following a rigorous geotechnical evaluation, there would be remaining uncertainties and risk associated with continued use of the facility, especially if it were raised or operated under higher hydraulic heads. Given the high costs for supplemental geotechnical investigations, with no guarantee of a positive outcome, and the likelihood that there would be residual uncertainties and risks at its conclusion, the project team was directed by the Sponsors and the WWDC to concentrate our resources and efforts on other solutions.

The Sponsor, WWDC and consulting team made a decision in May 2006 to investigate the Peaking II site alternative, as it was also one of the preferred locations for a new reservoir. In addition to this decision, the consulting team recommended that the City consider removing the existing Atlantic Rim Reservoir from service as time allowed, and to replace that reservoir with a new one of equal size near the water treatment plant.

6.2 **Wetlands Protection Considerations**

Because of the water leakage at Atlantic Rim, there is growth of vegetation, which even from aerial photographs appears to indicate the formation of wetlands and stream channels or ditches between the Atlantic Rim Reservoir and the Rim Lakes to the South. The decommissioning of Atlantic Rim Reservoir would likely cause these wetlands to dry up and revert to their natural uplands regime.

The U.S. Army Corps of Engineers specifically regulates the filling or placement of fills in wetlands or waters of the United States. In an off-the-record conversation with a staff member, mechanisms for closing the reservoir were discussed which would ease the regulatory burden for this operation. Specifically, there is no provision under the law which prevents the cessation of artificially provided water sources in a situation such as that existing downstream of the Atlantic Rim Reservoir. Therefore, it is recommended that the reservoir be vacated first and breached without placing any fills in the wetlands. Once the
vegetation and soils have stabilized over time, without the provision of water from the transmission pipelines, a determination will have to be made about whether there are any viable wetlands remaining, which would be subject to regulation.

The reuse of the riprap and clay materials from the reservoir itself could be done as a removal operation, but any stockpiling of such materials within the old reservoir basin would be subject to Army Corps of Engineers regulation unless it can be demonstrated that the reservoir is not part of the "waters of the United States". It should be possible to demonstrate this once the reservoir is breached and no longer in service.

6.3 Rim Lakes Water Supply

Leakage from the Atlantic Rim Reservoir was previously measured at 216,000 gallons per day. At an assigned value of $2 per 1000 gallons for raw water, this represents a loss to the City of $432/day or nearly $160,000 per year. Assuming the losses for overland flows to be about 50%, a pipeline supply of about 100,000 gallons of raw water per day should provide an equivalent supply to the Rim Lakes. The exact demand for these lakes has not been studied in detail but the revenue from metering and charging for this water at $2/1000 gallons could be around $70,000 per year, depending on an exact determination of the BLM demands. Regardless of a payment agreement with the BLM, this would assume a substantial savings (at least 50% less wasted) in water from controlled piping versus seepage flows.

A conceptual design for a 6” pipeline was prepared running from the Sage Creek pipeline to the lakes along the section line, a distance of 5350 feet. Also included was a vault with a 4” meter and a control valve. Since this pipeline may be classified a recreational use, special funding consideration may apply. Alternatives for funding the construction of this line were beyond the scope of the present study, but an arrangement which includes support from the BLM, either in grant funding, or in a purchase agreement for raw water could be very beneficial for the City as well as the BLM.
6.4 **Recommendation for Decommissioning Atlantic Rim Reservoir**

6.4.1 **Procedures and Time Line**

The procedure for decommissioning the Atlantic Rim Reservoir is generally to remove as much stored water as is feasible by draining it to the Peaking reservoir. This would logically be done at the end of the dry season when the reserve would not likely be needed until the following peak demand. The remaining water, if any, could be drained overland toward the Rim Lakes. If needed for fully draining the facility, the dam could be breached as long as no fill is placed in the wetlands downstream of the leaky dam.

Once the reservoir has dried out sufficiently, materials slated for reuse such as clay liner material, riprap, and fencing could be removed for use at the new reservoir site. At this point, planned fill operations will have to be cleared with the Army Corps of Engineers. The process of drying sufficiently to get free clearance to fill from the Corps of Engineers may take several years. The bulk of the dam material could then be smoothed into the depression leaving an amphitheater shaped area to be graded and reclaimed as indicated by the BLM. Once the BLM has accepted the reclamation as complete, the lease for this location can be voided.

6.4.2 **Materials**

Materials to be reclaimed from the Atlantic Rim Reservoir site include chain link fencing, gates, gravel, riprap, clay liner material, and possibly some piping. In order to reclaim the land, some topsoil will have to be brought in (from the proposed new reservoir site), as well as range seeding materials. Normal right-of-way fencing will have to be reinstalled along the section where the chain link fence is removed adjacent to the road.
6.4.3 Water Rights Transfer

Since the goal will ultimately be to transfer the water storage right from Atlantic Rim Reservoir to a new reservoir of the same size, the only change to the storage right proposed is the “point of storage”. If the State Engineer is properly informed there should be no loss of priority for this right. The condition for transfer of the water storage right will require that the Atlantic Rim Reservoir be removed from service, thus the closure of Atlantic Rim cannot be unlinked from the main reservoir project, though the timing of some phases may be adjustable.

6.5 Cost Estimates

The cost estimate for the decommissioning of Atlantic Rim Reservoir assumes that the BLM will not require a large amount of imported material to precisely match the pre-construction contours of the site. In addition, the area adjacent to the Atlantic Rim site is also the identified source of additional clay materials which are needed for the construction of a new reservoir. Certain aspects of the reclamation such as seeding and final grading may be advantageously combined with reclamation efforts related to the removal of the clay materials.

A cost estimate for the reclamation efforts for the Atlantic Rim site are $910,000, including allowances for detailed engineering, permitting, construction monitoring, and contingencies. The detailed cost estimate appears as table 6-1.
### TABLE 6-1 COST ESTIMATE FOR DECOMMISSIONING ATLANTIC RIM RESERVOIR

<table>
<thead>
<tr>
<th>Item</th>
<th>Unit</th>
<th>Quantity</th>
<th>Cost/Unit</th>
<th>Total Cost</th>
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<tr>
<td>1 Mobilization and Bonds (% of Items)</td>
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<tr>
<td>2 Reclamation of Atlantic Rim Reservoir</td>
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<td>H Legal Fees</td>
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<td>$10,000</td>
</tr>
<tr>
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<td>AC</td>
<td>5</td>
<td>$2,000</td>
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### TABLE 6-2 COST ESTIMATE FOR RIM LAKES SUPPLY PIPELINE

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<tr>
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<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
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<td>4&quot; meter, vault, and tap.</td>
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</tr>
<tr>
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<tr>
<td>D Contingency (15% of C)</td>
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<tr>
<td><strong>ROUNDED TOTAL COST</strong></td>
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</table>
7.0 ATLANTIC RIM PIPELINE

7.1 Introduction

In 1997, the Peaking II Study presented a conceptual design for the replacement of the existing Atlantic Rim Pipeline (ARP), between the Atlantic Rim Reservoir and the water treatment plant. At that time, a rationale for replacement was that the ARP was not reliable enough during periods when the Peaking reservoir was off line and the Sage Creek Basin water delivery to the treatment plant was directly through the ARP. The plant operators had experienced at least a couple of pipeline failures during this operational mode, although the exact cause of the failure was not identified.

If the Atlantic Rim Reservoir is abandoned and the pipeline extended to the water treatment plant, then the Sage Creek Pipeline system may not have the benefit of an intermediate location at which the “hydraulic grade line” is fixed by the reservoir water level (and standpipe at 33+80). Instead, the hydraulic grade line for a new closed pipeline system could increase operating pressures in the pipeline near the plant, especially when flows from the spring collection systems are throttled at the WTP. As noted in Section 7.4, the benefits and feasibility of operating a pressure pipeline still have not been completely addressed by this study. Figure 7-1 presents pipeline hydraulic gradelines for the high flow and low flow conditions in the Sage Creek pipeline. The present analysis used an EPANET model and AWWA C403-00 guidelines to check the theoretical pressures, including surge pressures, against the capability of the T-40 class A-C pipe. WWC has determined from the system model that surge pressure fluctuations may exceed the allowable pressure rating of the pipeline under some conditions. If a new pipeline were installed under the planned conditions, given the AWWA C403-00 criteria, WWC has determined that the appropriate pipeline would be greater than T-40 class, depending on the exact location within the profile.

The remainder of this Chapter describes a conceptual design for a new pipeline between Atlantic Rim Reservoir and the water treatment plant. The existing line would have some value as a backup line, and may continue to
function as long as precautions are taken to prevent large fluctuations in operating pressures and velocities. It is also unknown if installation problems such as point loadings or environmental exposure have compromised the pipeline. A physical examination of the line for weaknesses was not included in the scope of this report. Additional study of the line, especially including a field study, is advisable before making a final determination as to the pipeline’s appropriateness for use under the new conditions.

7.2 Pipeline Design Capacity

Based on discussions with current operators, the Sage Creek Springs area peak discharge has not exceeded 3,600 gpm and the potential contribution from the Miller Hill Well Field is about 400 gpm. Based on this information, a reasonable pipeline design capacity is 4,000 gpm.

From a water rights viewpoint, the design flow capacity would be slightly more. Current water rights of record for the Sage Creek basin (6.92 cfs = 3,100 gpm) and the Miller Hill Wells (1,350) total 4,450.

Finally, we note that one as-constructed drawing set for the Sage Creek Transmission Pipeline (Montgomery, 1988) indicates that the design flow could be as high as 6,340 gpm (1,340 from Wells and 5,000 from Springs), for a pipeline system with a booster station at the Miller Hill Wellfield junction.

Based on the above information, for the purposes of a conceptual design, we used the largest of the three values for pipeline design (6,340 gpm). This capacity also exceeds the full-flow capacity of the treatment plant of 8 MGD (5,555 gpm), which would allow full treatment plant capacity plus some amount of reservoir storage.

7.3 Conceptual Design

7.3.1 Alignment

As shown in Figure 7-2, the Atlantic Rim Pipeline (ARP) begins at the existing Atlantic Rim Reservoir and extends to the valve vault just south of the Water Treatment Plant (WTP). The new ARP will connect to the existing 24” steel transmission line from the wellfield and Sage Creek Basin springs at Atlantic
Rim Reservoir and follow the alignment of the existing 18” asbestos concrete pipe to the WTP. The ARP will end approximately at Station 120+00 with a connection to the existing valve vault which serves as an inlet/outlet from Peaking Reservoir No.1. A sleeve control valve is recommended for controlling line discharge and is included in the conceptual design.

The vertical alignment will follow the alignment of the existing 18” pipeline and remain below the hydraulic grade line. We note that the new hydraulic profile would allow the pipeline to be installed at standard depth through the crossing of highway

7.3.2 Material

The existing Sage Creek Basin Pipeline is a coated and cathodically protected steel pipeline. Steel pipe is recommended for the proposed new pipeline. Although steel pipeline requires cathodic protection, the City staff is trained on the maintenance of corrosion control for pipelines. Alternative materials such as PVC and HDPE are at present very expensive for the pressure rating that would be required in this application. A new look at pricing of alternative materials should be part of the final design scope, to allow for fluctuations in material prices.

7.3.3 Existing Utilities and Road Crossing

No research was performed into existing utilities that are along the proposed alignment, though none were apparent to visual observation. Also, no underground utilities were flagged within the Five-Mile study area when One-Call of Wyoming was advised of the geotechnical investigation.

It is assumed that an open cut would be allowed on the highway crossing. Such a cut was used on the original construction and the highway is not in such a condition that a well-constructed surface patch would be noticeable to the motorists on this highway.
7.4 **Hydraulic Design**

WWC performed a hydraulic evaluation of the Sage Creek pipeline as it is proposed to be modified by this project. An EPANET pressure pipe model was assembled for the existing system based on partial as-constructed information (Montgomery, 1988). An important difference; however, is that the standpipe at 33+80 was not included in the model. The pipeline diameters and hydraulic grade line elevations are shown on Figure 7-2. The results of the evaluation show that a 24-inch pipeline between Atlantic Rim and Peaking will be adequate to convey the design flows presented in a previous section of this Chapter. This finding is regardless of whether the line is run as it currently is with the 33+80 standpipe or in a full pressurized mode. This calculation should be confirmed during final design.

We note that it is possible that system hydraulics are controlled by the intake capacity of the spring box collection system, when Rawlins Reservoir is not hydraulically connected to the Sage Creek Pipeline. The use of an open standpipe at 33+80 confirms that the City operates the system by taking all of the water they can collect, as opposed to controlling discharge in a valve controlled pressure pipe system. The feasibility of converting the Sage Creek Pipeline to a pressure system was not addressed in this study. Possible benefits include providing more precise flow control to the plant and providing energy to power plant functions (eliminating booster station operation).

7.5 **Cost Estimate**

The total project cost estimate for the 24-inch diameter Atlantic Rim Pipeline is $4,000,000. Table 7-1 presents the detailed cost estimate. The unit price data was obtained from bid tabulations from recent projects and the Means cost estimating publication.

7.6 **Permits and Easements**

The pipeline project will fall under the US Army Corps of Engineer’s jurisdiction. In accordance with Section 404 of the Clean Water Act, a Nationwide Permit will be required for a utility crossing of waters of the United
States. A Nationwide Permit No.12 for Utility Line Activities must be acquired for the Atlantic Rim Pipeline.

The easement for the existing pipeline has not been researched. That document should be consulted to ensure that adequate right of way exists and that pipe replacement work is allowed under the easement.

Cultural resources inventories and wildlife clearances should be performed as were done for the Peaking II reservoir.
### TABLE 7-1 - ATLANTIC RIM PIPELINE CONCEPTUAL DESIGN PROJECT COST ESTIMATE

<table>
<thead>
<tr>
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<tr>
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<td>b.</td>
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<td>c.</td>
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<td>d.</td>
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<td>e.</td>
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<td>LS</td>
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### Summary

- **Construction Cost Subtotal**: $2,848,502
- **Engineering-Construction Supervision Costs (% of A)**: 10% $284,850
- **Subtotal (A+B)**: $3,133,353
- **Contingency (% of C)**: 15% $470,003
- **CONSTRUCTION COST TOTAL (C+D)**: $3,603,356
- **Prepare Final Design and Specs (% of E)**: 10% $360,336
- **Permitting and Mitigation**: $10,000
- **Legal Fees**: $10,000
- **Acquisition of Access and ROW**: $10,000

### Total Cost

- **PROJECT TOTAL COST**: $3,993,691
- **ROUNDED TOTAL COST**: $4,000,000
8.0 PROJECT FINANCING

This Chapter presents an analysis of project financing for the Peaking II site and other projects. From this information, monthly fee increases to the end user were estimated.

The Wyoming Water Development Commission (WWDC) is a State program available to assist the community with water system improvement projects such as a raw water storage reservoir. WWC assumed that all funding will come from the WWDC or the City.

At the present time, eligible rehabilitation projects receive a 67% grant and 33% loan arrangement. The loan portion of the financing, if needed, does not have to originate from the WWDC. But if it does, the present loan rate is 4% for a 30 year (negotiable) term. Another good source of funding is the 2% county and capital facilities tax, which may allow a reduction in the cost to end users. A detailed study of this tax was not included in the analysis, but City Council did express an interest and support for using some of these funds in the future as needed for this project.

Table 8-1 presents the project cost estimates, project financing and estimated rate increase required to retire project debt and operational costs. Based on 2007 water system records the number of taps in Rawlins is 3,888 (including the Town of Sinclair which is billed at 60% of the Rawlins water rates). Currently the average water bill in Rawlins is $24.38, and the average water bill for the state is $35.45 (WWDC, 2007 – taking medium to larger WY towns only), a difference of $11.07. The average billing rate in Rawlins is 31% lower than that of the state, while the average mean income is 12% lower (Department of Administration and Information, Wyoming 10 Year Outlook, July 2007 and 2005 Census data by county, adjusted by growth trend to 2007). Assuming that an affordable rate increase for the citizens of Rawlins should be proportional to average incomes, they can afford to pay 88% of the average state water bill ($31.20), the citizens could afford the Peaking II reservoir project. This project has an estimated $6.74/month/tap water rate increase for the City of Rawlins and an increase of $4.05 for the Town of Sinclair.
Table 8 - 1  Project Financing

<table>
<thead>
<tr>
<th>GRANTS AND LOANS</th>
<th>Peaking Reservoir No. 2</th>
<th>Five-Mile Reservoir</th>
<th>Atlantic Rim Pipeline</th>
<th>Assertive North Platte Pumping</th>
</tr>
</thead>
<tbody>
<tr>
<td>A PROJECT COST ESTIMATE</td>
<td>3' Clay Liner</td>
<td>3' Clay Liner</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1 Reservoir Project</td>
<td>$11,100,000.00</td>
<td>$10,500,000.00</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 Atlantic Rim Pipeline</td>
<td>$4,000,000.00</td>
<td>$4,000,000.00</td>
<td>$4,000,000.00</td>
<td>$4,000,000.00</td>
</tr>
<tr>
<td>3 Decommission Atlantic Rim</td>
<td>$420,000.00</td>
<td>$420,000.00</td>
<td></td>
<td>$420,000.00</td>
</tr>
<tr>
<td>4 Supply to Rim Lakes</td>
<td>$490,000.00</td>
<td>$490,000.00</td>
<td></td>
<td>$490,000.00</td>
</tr>
<tr>
<td>B GRANTS</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1 Total of WWDC Grant Eligible</td>
<td>$16,010,000.00</td>
<td>$15,410,000.00</td>
<td>$4,000,000.00</td>
<td>$4,910,000.00</td>
</tr>
<tr>
<td>2 WWDC Total Grant (67% of Cost Est.)</td>
<td>$10,726,700.00</td>
<td>$10,324,700.00</td>
<td>$2,680,000.00</td>
<td>$3,289,700.00</td>
</tr>
<tr>
<td>C LOANS</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Proposed WWDC</td>
<td>4.00%</td>
<td>$5,283,300.00</td>
<td>$5,085,300.00</td>
<td>$1,320,000.00</td>
</tr>
<tr>
<td>Loan Amount (B1-B2)</td>
<td>4.00%</td>
<td>$305,600.00</td>
<td>$294,100.00</td>
<td>$76,400.00</td>
</tr>
</tbody>
</table>

USER COSTS

ANNUAL OPERATIONS AND MAINTENANCE

| (adjusted for inflation and distance from plant to reservoir) |                      |                    |                      |                                |
| Proposed Reservoir WWDC Loan Annual Repayment | $305,600.00 | $294,100.00 | $76,400.00 | $93,800.00 |

ANNUAL OPERATIONS AND MAINTENANCE

B Based on NRCS O&M data

| (adjusted for inflation and distance from plant to reservoir) |                      |                    |                      |                                |
| $37,996.00 | $38,716.00 | $203,520.00 |

C TOTAL ANNUALIZED COSTS (see note 1)

|                      | $305,600.00 | $294,100.00 | $76,400.00 | $297,320.00 |

WATER RATES

<table>
<thead>
<tr>
<th>(From April 07 to Mar 08 cycle)</th>
<th>Average</th>
<th>Taps</th>
</tr>
</thead>
<tbody>
<tr>
<td>Current Rawlins Monthly Water Bill =</td>
<td>$ -</td>
<td>3605</td>
</tr>
<tr>
<td>Current Sinclair Monthly Water Bill =</td>
<td>$ -</td>
<td>283</td>
</tr>
<tr>
<td>Calculated Rawlins Rate increase</td>
<td>$6.75</td>
<td>$6.50</td>
</tr>
<tr>
<td>Calculated Sinclair Rate increase</td>
<td>$4.05</td>
<td>$3.90</td>
</tr>
</tbody>
</table>

note 1: Operations and Maintenance should not increase over the cost of operating and maintaining Atlantic Rim, so this amount is not added to the total annualized cost.

note 2: The number of taps for Rawlins proper is 3605. The number of taps for Sinclair is 283 which are billed at 60% of Rawlins rate.
REFERENCES


Wyoming Department of Transportation’s (WYDOT) Endangered, Threatened, Candidate & Proposed Species Resource Manual (1999)


LEVEL II GEOTECHNICAL DATA REPORT
FOR FIVEMILE SITE
RAWLINS RAW WATER STORAGE STUDY
CARBON COUNTY, WYOMING

Submitted to
WWC Engineering, Inc.
611 Skyline Road
Laramie, Wyoming 82070

Submitted by
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January 2008
RJH Project 07112
WWC Project 2007158

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Project Manager

Edwin R. Friend, P.E., P.G.
Engineering Geologist
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Appendix E Geologic Reconnaissance Data
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SECTION 1 – INTRODUCTION

1.1 Purpose

The purpose of the Rawlins Raw Water Storage Level II Study is to evaluate the feasibility of constructing a new raw water storage reservoir for the City of Rawlins (City) at a site designated as “Fivemile Site” (the Site). The purpose of this report is to present geologic and geotechnical data collected for the Project.

1.2 Objectives

The objectives of the geotechnical investigation were to collect geologic and geotechnical information required to identify if construction of a dam and reservoir is feasible at the site and to support the development of a feasible dam and reservoir concept for the Level II Study.

1.3 Scope of Work

RJH Consultants, Inc. (RJH) and our subcontractors performed the following services for the project:

- Reviewed available geologic literature, including maps and reports, containing information pertinent to the Site.
- Performed a geologic reconnaissance.
- Excavated seven test pits at the Site.
- Observed the Atlantic Rim Dam and Reservoir and collected a sample of the reservoir bottom to evaluate the potential for reuse as a liner for the proposed Fivemile Reservoir.
- Performed laboratory tests on representative samples of soil collected during subsurface exploration.
- Based on the data obtained, developed an interpretation of the subsurface materials and conditions at the Site.
- Prepared this report to present the data collected.
1.4 Authorization and Project Personnel

The work described in this report was performed in accordance with the Agreement for Professional Services between WWC and RJH dated August 3, 2007. RJH personnel responsible for the execution of this work included:

- Project Manager: Robert J. Huzjak, P.E.
- Project Engineer: A. Tom MacDougall, P.E. (1)
- Staff Geological Engineer: Adam B. Prochaska, Ph.D., E.I.
- Staff Engineer: Emily P. Tyler, E.I.

(1) Registered as a Professional Engineer in Arizona and Colorado.
(2) Registered as a Professional Engineer in Colorado.

RJH retained the services of Michael D. Hattel, P.G., R.E.A. with Apex. Mr. Hattel, a geologist registered in the State of Wyoming, assisted in performing geologic reconnaissance and oversaw subsurface logging.
SECTION 2 - PROJECT DESCRIPTION

2.1 Background

The City and the Wyoming Water Development Commission (WWDC) are studying the feasibility of constructing a new raw water storage reservoir for the City. The purpose of the reservoir is maintain the reliability of the City’s municipal water supply system while accommodating increased water demand and or decreased water supply (e.g., due to drought). Currently, the City’s largest reservoir, the Atlantic Rim Reservoir, has storage restrictions because of excessive seepage and is not providing the intended service to the water supply system.

A previous Level II Study was performed to develop and evaluate alternatives for maintaining the reliability of the City’s water supply. The study involved a reconnaissance level (Phase I) evaluation of five alternative locations for raw water reservoir storage. Of the five alternatives, a conceptual level (Phase II) evaluation was performed at the site designated Peaking No. 2 Reservoir (which is adjacent to the existing Peaking Reservoir). The Phase I evaluation also recommended that a Phase II study be performed for the Fivemile Site.

WWC informed RJH that the Atlantic Rim Reservoir may be decommissioned due to safety concerns and the embankment fill and clay liner may be available for reuse as liner or fill materials for the proposed Fivemile project.

2.2 Site Location

The Fivemile Site is located immediately north of Fivemile Ridge on an unnamed, ephemeral tributary to Hay Gulch, which is tributary to Sugar Creek. The Site is approximately 3.5 miles south-southwest of Rawlins, Wyoming, immediately east of Highway 71, in Section 1, Township 20N, Range 88 W of the 6th Principle Meridian. The Atlantic Rim Reservoir is located approximately 1.5 miles south of the Fivemile site, immediately west of Highway 71. The locations of the Fivemile site and the Atlantic Rim Reservoir are shown on Figure 2.1.

2.3 Description of Fivemile Dam and Reservoir Concept

The Fivemile dam and reservoir concept includes an earthen embankment dam approximately 70 feet high with a crest elevation of 7,170 feet. The dam will impound approximately 644 acre-feet with a surface area of approximately 32 acres (at maximum normal pool). The dam is anticipated to be an earthen structure. The reservoir will likely
need to be lined with clay material to reduce seepage losses. Preliminary embankment slopes are 3 Horizontal (H) to 1 Vertical (V). The spillway is anticipated to be an excavated channel through the abutment. A low-level outlet works capable of draining the reservoir contents is also anticipated.
SECTION 3 GEOLOGIC AND GEOTECHNICAL DATA COLLECTION

3.1 General

RJH collected geologic and geotechnical data by reviewing existing information, performing geologic site reconnaissance, excavating test pits at the Fivemile Site, visiting the Atlantic Rim Reservoir, and performing laboratory testing.

3.2 Review of Existing Information

RJH reviewed the following existing documents relating to the geologic and geotechnical conditions at the Site:

- Preliminary Map of Known Surficial Structural Features for the Rawlins 1° x 2° Quadrangle. WGS, King, J.K., Greer, P.L., and Ver Ploeg, A.J. Open File Report 87-10.
- Atlantic Rim Reservoir Design Drawings. CTL/Thompson, September 1981, Sheets 1 through 3 of 3.
- Preliminary Natural Resource Conservation Service (NRCS) Soil Map-Carbon County, Wyoming. This data is unpublished and was provided to RJH with the understanding that all information is preliminary. Wyoming Office of the NRCS.

Report contains the results and recommendations of a preliminary geotechnical reconnaissance performed to support the evaluation of five reservoir alternatives. The report contains summaries of geologic data collected for the general region, a description of seismicity of the area, summaries of subsurface conditions based on available information, preliminary designs and cost estimates, and recommendations for additional geotechnical study at the Atlantic Rim Site. This report also contains a chronology of significant events for the Atlantic Rim Reservoir.


This technical memorandum documents Gannett Fleming’s geotechnical investigations, preliminary design, and preliminary engineer’s cost opinions for construction of a 645 acre-foot reservoir at the Peaking No. 2 site. In addition, the report summarizes a 1978 geotechnical investigation for Peaking Reservoir No. 2 by CTL Thompson. Gannett Fleming and CTL Thompson explored the Atlantic Rim Site as a potential borrow area for the Peaking Reservoir No. 2 project.

3.3 Geologic Reconnaissance

On September 18, 2007, Apex personnel visited the site to observe conditions and perform a geologic reconnaissance. The geologic reconnaissance included mapping surficial deposits, noting surface features, measuring the strike and dip of fractures at bedrock outcrops, classifying rock types of surface exposures, and making general geologic observations. Locations of outcrops where fracture orientations were measured are shown on Figure 3.1. Following the site visit, field data was compared to published data. Site geology is discussed in Section 4.

3.4 Test Pits

3.4.1 General

Seven test pits were excavated in the proposed dam and reservoir area on September 18, 2007. Test pits were excavated by A&D Oilfield Dozers, Inc. of Rawlins, Wyoming, under subcontract to RJH. A summary of the test pits is presented in Table 3.1 and the test pit locations are shown on Figure 3.1. An RJH geotechnical engineer and a registered Geologist
from Apex were onsite during excavation to visually classify the soil and rock, collect
samples, prepare field logs for each test pit, and record relevant excavation information. The
purposes of the test pit explorations were to observe subsurface soil and rock conditions;
perform preliminary evaluation of the foundation strength and permeability; obtain samples
for laboratory testing; and evaluate material types, quantities, and availability of potential
borrow material.

### TABLE 3.1
**SUMMARY OF TEST PITS**

<table>
<thead>
<tr>
<th>Borehole ID</th>
<th>Coordinates of Location(1)</th>
<th>Ground Surface Elevation(2)</th>
<th>Ground Water Depth(3)</th>
<th>Total Depth (ft)</th>
<th>Approximate Depth to Bedrock (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Latitude</td>
<td>Longitude</td>
<td>(ft)</td>
<td>(ft)</td>
<td></td>
</tr>
<tr>
<td>TP-101</td>
<td>41.7363</td>
<td>-107.2597</td>
<td>7088</td>
<td>6.0</td>
<td>9.0</td>
</tr>
<tr>
<td>TP-102</td>
<td>41.7360</td>
<td>-107.2605</td>
<td>7094</td>
<td>N/E(4)</td>
<td>7.0</td>
</tr>
<tr>
<td>TP-105</td>
<td>41.7359</td>
<td>-107.2636</td>
<td>7155</td>
<td>N/E(4)</td>
<td>8.0</td>
</tr>
<tr>
<td>TP-106</td>
<td>41.7352</td>
<td>-107.2632</td>
<td>7137</td>
<td>N/E(4)</td>
<td>6.5</td>
</tr>
<tr>
<td>TP-107</td>
<td>41.7349</td>
<td>-107.2617</td>
<td>7138</td>
<td>N/E(4)</td>
<td>9.0</td>
</tr>
<tr>
<td>TP-108</td>
<td>41.7348</td>
<td>-107.2599</td>
<td>7126</td>
<td>N/E(4)</td>
<td>7.0</td>
</tr>
<tr>
<td>TP-112</td>
<td>41.7369</td>
<td>-107.2613</td>
<td>7136</td>
<td>N/E(4)</td>
<td>9.0</td>
</tr>
</tbody>
</table>

Notes:
1. Coordinates obtained using hand-held GPS system.
2. Elevations approximated using the topographic map prepared by WWC Engineering.
3. Measured at time of excavation.
4. N/E = not encountered.

### 3.4.2 Excavation Methods

The test pits were excavated with a Link Belt 210LX track-mounted excavator. The test pit
depths ranged from 6.5 to 9.0 feet below the ground surface and were generally about 3 feet
wide and 20 feet long. Each of the test pits were backfilled with the excavated material upon
completion.

### 3.4.3 Logging and Sampling Procedures

Soils were classified according to ASTM D 2488 (visual-manual method). Rock was
classified according to the U.S. Bureau of Reclamation *Engineering Geology Field Manual*
(USBR, 2001). Stratigraphic changes were measured from approximately the middle of the
trench (lengthwise). Classifications of soil and rock were based on observing the excavated
spoils and trench sides. Representative samples were collected from the excavated spoils
and placed in plastic bags. Photographs were taken of the pit following excavation and prior
to backfilling. Additional explanation regarding the soil and rock descriptors used on the logs is provided in Appendix A, test pit logs are provided in Appendix B, and photographs of test pits are provided in Appendix C.

### 3.5 Atlantic Rim Site Reconnaissance

RJH visited the Atlantic Rim Dam and Reservoir site to preliminarily evaluate the site as a potential borrow source. RJH observed surface soils, observed discharge from the seepage collection system, and sampled the reservoir bottom at the northeast corner. The reservoir was partially full and the eastern side of the reservoir bottom was exposed. Slope protection consisted of sandstone riprap. The reservoir bottom that was exposed (presumably the liner, although possibly reservoir sediment) consisted of sandy silt. Areas of accumulated evaporates are present on the ground surface to the north of the dam and vegetation was sparse in those areas. Discharge from the seepage collection system was observed flowing at two discharge points.

Data at the Atlantic Rim Reservoir was collected previously by CTL Thompson of Denver, Colorado (1981) and by Gannett Fleming, of Phoenix, Arizona (2006). Data collected from these studies are provided in Appendix F.1 and Appendix F.2

### 3.6 Laboratory Testing

Laboratory testing was performed on two samples to evaluate index properties of the on-site alluvium and the Atlantic Rim Reservoir bottom. RJH engaged HP Geotech of Parker, Colorado to perform the testing. Atterberg limits were tested in accordance with ASTM D 4318 and grain-size analysis was performed in accordance with ASTM D 422. The results are summarized in Table 3.2 and included in Appendix D.

<table>
<thead>
<tr>
<th>Boring or Test Pit ID</th>
<th>Sample Depth Interval (feet)</th>
<th>General Description</th>
<th>Atterberg Limits</th>
<th>Gradation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Liquid Limit, LL (%)</td>
<td>Plasticity Index, PI (%)</td>
</tr>
<tr>
<td><strong>Atlantic Rim Reservoir Bottom</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>AR-1</td>
<td>0.0 - 0.5</td>
<td>Sandy Silt</td>
<td>24</td>
<td>1</td>
</tr>
<tr>
<td><strong>Fivemile Site Alluvium</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TP-101</td>
<td>0.0 - 2.0</td>
<td>Silty Sand</td>
<td>20</td>
<td>4</td>
</tr>
</tbody>
</table>

**TABLE 3.2**

**SUMMARY OF LABORATORY TEST RESULTS**
Previous studies included laboratory testing of the potential borrow area at the Atlantic Rim Reservoir. For convenience, laboratory test results for these previous studies are included in Appendix F.3.
LEVEL II GEOTECHNICAL DATA REPORT
FIVEMILE SITE
RAWLINS RAW WATER STORAGE STUDY
PROJECT NO. 07112
January 2008  Figure 3.1
SECTION 4 - SITE GEOLOGY AND SUBSURFACE PROFILE

4.1 Regional Geology

The Site is located approximately 3.5 miles south-southwest of Rawlins, Wyoming in the Wyoming Basin Province of Rocky Mountain Physiographic Region. The Wyoming Basin Province is characterized by broad structural and topographic basins that are partially filled with Tertiary (67 to 2 million-year-old) deposits and separated by low anticlines (Hunt, 1967).

Bedrock structure in the vicinity of the Site is controlled by several faults and anticlines. A possibly active thrust/reverse fault is located approximately 2.5 miles north of the Site. This fault is oriented approximately east/west with the north block being upthrown. Normal faults with various orientations are present in the vicinity of the Site, and most indicate east/west extension. The nearest normal fault to the Site is located approximately 2.5 miles to the north. The anticlines generally trend north/south or northeast/southwest. The hinge line of the nearest anticline to the Site trends north/south along the alignment of Highway 71 and plunges to the north. A map of the regional structural geology is shown on Figure 4.1.

The geologic history of the region includes deposition of sediments during the regression of the Cretaceous Seaway. Regional folding occurred throughout the Late Cretaceous and Tertiary Periods (100 to 2 million years ago), and was likely associated with the east/west crustal shortening of the Laramide Orogeny.

The surficial deposits at the Site lie unconformably on the Mesaverde Group. Continued erosion of this formation, combined with reworking and deposition of surficial soils, has formed the present-day landscape at the Site.

4.2 Fivemile Site Geology and Subsurface Profile

4.2.1 Geologic Setting

The Site is situated on the east limb of an anticline and the proposed dam crosses an incised valley with moderate side slopes (generally about 7 percent). Various bedrock outcrops are present and vegetation consists of small shrubs and bushes. The proposed right and left abutments are comprised of a thin veneer (generally 2.5 feet or less) of colluvial, residual, or eolian deposits overlying intensely weathered sandstone. The valley bottom contains alluvial
deposits consisting of silty sand and sandy silt. Figure 4.2 shows the surficial geology at the Site.

Bedrock at the Site consists of Late Cretaceous (100 to 67-million-year old) Haystack Mountains formations, which is in the Mesaverde Group. The Haystack Mountains formation consists primarily of sandstone with minor amounts of shale. Surficial soils at the Site consist of residual, eolian, and alluvial deposits (Hallberg and Case, 1998). The NRCS classifies the surficial soils at the Site as ranging from sandy loam to clay loam, well drained, with moderate to moderately slow permeability.

4.2.2 Bedrock Structure

RJH subcontracted Apex to measure the orientations of the strike and dip of fractures in exposed bedrock outcrops at the Site. A total of 11 measurements were recorded at 11 locations (Figure 3.1). Two fracture sets exist at the Site, with the following orientations:

- N15E, 90
- N8E, 77W

Additional details of fracture orientations are presented in Appendix E.

4.2.3 Generalized Subsurface Profile

The generalized subsurface profile at the Fivemile Reservoir Site consists of alluvium (in the valley bottom) and eolian deposits and/or residual soils (on the valley sides) overlying sandstone bedrock.

In the valley bottom and channels at the site, the subsurface consists of an alluvial stratum of silty sand or clayey sand up to approximately 8 feet thick. The silty sand contains between 35 and 45 percent fines and between 5 and 10 percent gravel. The fines within the silty sand are low to non-plastic. The clayey sand is similar to the silty sand except the fines are slightly more plastic.

On the valley sides above the ephemeral channels, the surficial soil consists of eolian deposits or residual soil. The soil consists of silty sand and extends to approximately 1.5 feet below the ground surface. The soil is generally loose and dry to slightly moist.

Sandstone bedrock underlies the surficial eolian, residual, and alluvial deposits. This sandstone is part of the Haystack Mountains formation, which is part of the Mesaverde
Group. Outcrops are present at the site and the sandstone extended beyond the depths explored (up to 9 feet). Within the zone observed, the sandstone is fine to medium-grained, very intensely to moderately weathered (generally decreased weathering with depth), and very intensely to moderately fractured. A generalized subsurface profile along the dam axis is shown on Figure 4.3.

4.3 Atlantic Rim Reservoir Geology and Subsurface Conditions

Subsurface conditions at the Atlantic Rim Reservoir consist of surficial soils overlying Steele shale. The surficial soils include fill (the dam embankment); sandy silt (within the reservoir bottom); lean clay with sand and sandy silty clay (to the west of the existing reservoir); and clayey to silty sand and sandy clay (to the north of the reservoir). Fines contents of the surface soils vary between about 25 and 85 percent. Plasticity indices range from 6 to 18. The fines generally consist of more than 50 percent silt and dispersivity of one sample tested was 25 percent.

Steele shale is present below the surficial soils at the site and consists primarily of claystone. Intermittent zones of “highly weathered bedrock,” which is presumed to be a very intensely weathered and very intensely fractured claystone, are shown on CTL Thompson’s logs. Neither laboratory test results nor in-situ test results on the Steele shale were encountered in our review of the existing data at the site.

4.4 Geologic Hazards

4.4.1 Faulting and Ground Motion

A reverse fault that forms the Rawlins Uplift, which is located approximately 2.5 miles to the north, is the closest fault to the site and is considered possibly active. According to the USGS, the peak ground motion at the site with a 2 percent probability of exceedance in 50 years (i.e., 2,475-year return period) is 0.19g.

4.4.2 Soluble Evaporates

If borrow material at the Atlantic Rim Reservoir site may contain evaporates within the soil. Anhydrite, which is one type of evaporate, will alter to gypsum in the presence of water, which produces a volume increase and swell pressures on surrounding structures (West, 1995). Dissolution of evaporates within the fill may also be a concern. In a future stage of investigation, the amounts and specific types of evaporates that exist within the soil at the potential borrow area should be identified.
NOTES:

1. BASEMAP CREATED FROM US GEOLOGIC SURVEY BY TOPO®, © 2006 NATIONAL GEOGRAPHIC.


ANTICLINE – arrows perpendicular to axis show symmetry, i.e., short arrow indicates flank with steeper dip. Axis is dashed where covered or approximately located. Arrow on axis indicates direction of plunge.

NORMAL FAULTS – ball on downthrown block (dashed where covered or approximately located). Queried where existence is questionable.

THRUST/REVERSE FAULTS – teeth on upthrown block (dashed where covered or approximately located). Queried where existence is questionable.

POSSIBLE ACTIVE FAULT – as identified by Case (1998)
Figure 4.3

PROPOSED DAM CREST
EL 7170

EQUAN DEPOSITS
ANG/OR RESIDUUM

PROPOSED FILL

INTENSELY WEATHERED
SANDSTONE BEDROCK
(HAYSTACK MOUNTAINS
FORMATION)

INTENSELY WEATHERED
SANDSTONE BEDROCK
(HAYSTACK MOUNTAINS
FORMATION)

ALLUVIUM

SECTION A.1
PROPOSED DAM CENTERLINE 3.1

EXISTING GROUND

PROPOSED GRADE

ASSUMED CONTACT
BETWEEN SUBSURFACE UNITS

HORIZONTAL SCALE IN FEET

VERTICAL SCALE IN FEET

0 100 200 400 600

0 20 40 80 120

December 2007
SECTION 5 - REFERENCES


APPENDIX A

EXPLANATION OF SOIL AND ROCK DESCRIPTORS
FINE GRAINED SOILS

FOR SOILS WITH ≥ 50% FINES

GROUP SYMBOL | GROUP NAME
--- | ---
CL (P<1) | LEAN CLAY
< 30 % + No. 200 | LEAN CLAY WITH SAND
15-29 % + No. 200 | LEAN CLAY WITH GRAVEL
≥ 30 % + No. 200 | SANDY LEAN CLAY

CL-ML (4d <7) | SANDY LEAN CLAY WITH GRAVEL
< 30 % + No. 200 | SANDY LEAN CLAY WITH GRAVEL
15-29 % + No. 200 | SANDY LEAN CLAY WITH SAND
≥ 30 % + No. 200 | GRAVELY LEAN CLAY

ML (P<4) | SILT
< 30 % + No. 200 | SILT WITH SAND
15-29 % + No. 200 | SILT WITH GRAVEL
≥ 30 % + No. 200 | SANDY SILT

CH | FAT CLAY
< 30 % + No. 200 | FAT CLAY WITH SAND
15-29 % + No. 200 | FAT CLAY WITH GRAVEL
≥ 30 % + No. 200 | SANDY FAT CLAY

MH | ELASTIC SILT
< 30 % + No. 200 | ELASTIC SILT WITH SAND
15-29 % + No. 200 | ELASTIC SILT WITH GRAVEL
≥ 30 % + No. 200 | SANDY ELASTIC SILT

ORGANIC SOIL | ORGANIC SOIL
< 30 % + No. 200 | ORGANIC SOIL WITH SAND
15-29 % + No. 200 | ORGANIC SOIL WITH GRAVEL
≥ 30 % + No. 200 | SANDY ORGANIC SOIL

PLASTICITY CHART

GUIDE FOR STIFFNESS OF FINE-GRAINED SOILS:

STIFFNESS | CRITERIA (MANUAL TESTS)
--- | ---
VERY SOFT | - EXTRUDED BETWEEN FINGERS WHEN SQUEEZED
SOFT | - MOLDED BY LIGHT FINGER PRESSURE
MEDIUM | - MOLDED BY STRONG FINGER PRESSURE
STIFF | - READILY INDENTED BY HUMB
VERY STIFF | - PENETRATED WITH GREAT EFFORT
HARD | - INDENTED WITH DIFFICULTY BY THUMBNAIl
COARSE GRAINED SOILS

FOR SOILS WITH < 50% FINES

GROUP SYMBOL | GROUP NAME
--- | ---
GW | WIDELY GRADED SAND
GP | NARROWLY GRADED SAND
GW-GM | WIDELY GRADED SAND WITH SILT
GW-GC | WIDELY GRADED SAND WITH CLAY (OR SILTY CLAY)
GP-GM | NARROWLY GRADED SAND WITH SILT
GP-GC | NARROWLY GRADED SAND WITH CLAY (OR SILTY CLAY)
GM | SILTY SAND
GM | SILTY SAND WITH SAND
GC | CLAYEY SAND
GC | CLAYEY SAND WITH SAND
GW | WIDELY GRADED SAND WITH GRAVEL
GP | NARROWLY GRADED SAND WITH GRAVEL
GW-GM | WIDELY GRADED SAND WITH SILT AND GRAVEL
GW-GC | WIDELY GRADED SAND WITH CLAY AND GRAVEL (OR SILTY CLAY)
GP-GM | NARROWLY GRADED SAND WITH SILT AND GRAVEL
GP-GC | NARROWLY GRADED SAND WITH CLAY AND GRAVEL (OR SILTY CLAY)
SM | SILTY SAND WITH GRAVEL
SC | CLAYEY SAND
SC | CLAYEY SAND WITH GRAVEL
SW | WIDELY GRADED SAND WITH SILT
SP | NARROWLY GRADED SAND WITH SILT
SW-SM | WIDELY GRADED SAND WITH SILT AND GRAVEL
SW-SC | WIDELY GRADED SAND WITH CLAY AND GRAVEL (OR SILTY CLAY)
SP-SM | NARROWLY GRADED SAND WITH SILT AND GRAVEL
SP-SC | NARROWLY GRADED SAND WITH CLAY AND GRAVEL (OR SILTY CLAY)
SM | SILTY SAND WITH GRAVEL
SC | CLAYEY SAND
SC | CLAYEY SAND WITH GRAVEL
SW | WIDELY GRADED SAND WITH SILT
SP | NARROWLY GRADED SAND WITH SILT
SW-SM | WIDELY GRADED SAND WITH SILT AND GRAVEL
SW-SC | WIDELY GRADED SAND WITH CLAY AND GRAVEL (OR SILTY CLAY)
SP-SM | NARROWLY GRADED SAND WITH SILT AND GRAVEL
SP-SC | NARROWLY GRADED SAND WITH CLAY AND GRAVEL (OR SILTY CLAY)
SM | SILTY SAND WITH GRAVEL
SC | CLAYEY SAND
SC | CLAYEY SAND WITH GRAVEL
SW | WIDELY GRADED SAND WITH SILT
SP | NARROWLY GRADED SAND WITH SILT
SW-SM | WIDELY GRADED SAND WITH SILT AND GRAVEL
SW-SC | WIDELY GRADED SAND WITH CLAY AND GRAVEL (OR SILTY CLAY)
SP-SM | NARROWLY GRADED SAND WITH SILT AND GRAVEL
SP-SC | NARROWLY GRADED SAND WITH CLAY AND GRAVEL (OR SILTY CLAY)
SM | SILTY SAND WITH GRAVEL
SC | CLAYEY SAND
SC | CLAYEY SAND WITH GRAVEL

RELATIVE DENSITY OF SANDS ACCORDING TO SPT BLOW COUNTS:

<table>
<thead>
<tr>
<th>NUMBER OF BLOWS, N</th>
<th>RELATIVE DENSITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 4</td>
<td>VERY LOOSE</td>
</tr>
<tr>
<td>4 - 10</td>
<td>LOOSE</td>
</tr>
<tr>
<td>10 - 30</td>
<td>MEDIUM</td>
</tr>
<tr>
<td>30 - 60</td>
<td>DENSE</td>
</tr>
<tr>
<td>OVER 50</td>
<td>VERY DENSE</td>
</tr>
</tbody>
</table>

ANGULARITY

- ROUNDED
- SUBROUNDED
- SUBANGULAR
- ANGULAR
TABLE 1
BEDDING, FOLIATION, OR FLOW TEXTURE DESCRIPTIONS

<table>
<thead>
<tr>
<th>Descriptor</th>
<th>Thickness/Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Massive</td>
<td>Greater than 10 ft. (3m)</td>
</tr>
<tr>
<td>Very Thickly (Bedded, Foliated, or Banded)</td>
<td>3 to 10 ft. (1 to 3m)</td>
</tr>
<tr>
<td>Thickly</td>
<td>1 to 3 ft. (300mm to 1m)</td>
</tr>
<tr>
<td>Moderately</td>
<td>0.3 to 1 ft. (100 to 300mm)</td>
</tr>
<tr>
<td>Thinly</td>
<td>0.1 to 0.3 ft. (30 to 100mm)</td>
</tr>
<tr>
<td>Very Thinly</td>
<td>0.03 [3/8-in.] to 0.1 ft. (10 to 30mm)</td>
</tr>
<tr>
<td>Laminated (Intensely Foliated or Banded)</td>
<td>Less than 0.03 ft. [3/8-in] (10mm)</td>
</tr>
</tbody>
</table>

Note:
The dip of the bedding noted on the logs is measured from horizontal for vertical boreholes and normal to the axis on angled boreholes.

TABLE 2
ROCK HARDNESS / STRENGTH DESCRIPTORS

<table>
<thead>
<tr>
<th>Alphanumeric Descriptor</th>
<th>Descriptor</th>
<th>Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>H1</td>
<td>Extremely Hard</td>
<td>Core, fragment, or exposure cannot be scratched with knife or sharp pick; can only be chipped with repeated heavy hammer blow.</td>
</tr>
<tr>
<td>H2</td>
<td>Very Hard</td>
<td>Cannot be scratched with knife or sharp pick. Core or fragment breaks with repeated heavy hammer blow.</td>
</tr>
<tr>
<td>H3</td>
<td>Hard</td>
<td>Can be scratched with knife or sharp pick with difficulty (heavy pressure). Heavy hammer blow required to break specimen.</td>
</tr>
<tr>
<td>H4</td>
<td>Moderately Hard</td>
<td>Can be scratched with knife or sharp pick with light or moderate pressure. Core or fragment breaks with moderate hammer blow.</td>
</tr>
<tr>
<td>H5</td>
<td>Moderately Soft</td>
<td>Can be grooved 1/16 inch (2 mm) deep by knife or sharp pick with moderate or heavy pressure. Core or fragment breaks with light hammer blow or heavy manual pressure.</td>
</tr>
<tr>
<td>H6</td>
<td>Soft</td>
<td>Can be grooved or gouged easily by knife or sharp pick with light pressure. Can be scratched with fingernail. Breaks with light to moderate manual pressure.</td>
</tr>
<tr>
<td>H7</td>
<td>Very Soft</td>
<td>Can be readily indented, grooved, or gouged with fingernail, or carved with a knife. Breaks with light manual pressure.</td>
</tr>
</tbody>
</table>

Note:
Source: U.S. Bureau of Reclamation, Engineering Geology Field Manual
### TABLE 3
**FRACTURE CONTINUITY DESCRIPTORS**

<table>
<thead>
<tr>
<th>Descriptor</th>
<th>Lengths</th>
</tr>
</thead>
<tbody>
<tr>
<td>Discontinuous</td>
<td>Less than 3 ft (&gt; 1 m)</td>
</tr>
<tr>
<td>Slightly Continuous</td>
<td>3 to 10 ft (1 to 3 m)</td>
</tr>
<tr>
<td>Moderately Continuous</td>
<td>10 to 30 ft (3 to 10 m)</td>
</tr>
<tr>
<td>Highly Continuous</td>
<td>30 to 100 ft (10 to 30 m)</td>
</tr>
<tr>
<td>Very Continuous</td>
<td>Greater than 100 ft (&gt; 30 m)</td>
</tr>
</tbody>
</table>

*Note:* Measure inclination of fractures and joints from horizontal.

### TABLE 4
**FRACTURE SPACING DESCRIPTORS**

<table>
<thead>
<tr>
<th>Joint or Fracture Spacing Descriptor</th>
<th>True Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Extremely Widely Spaced</td>
<td>Greater than 10 ft (&gt; 3 m)</td>
</tr>
<tr>
<td>Very Widely Spaced</td>
<td>3 to 10 ft (1 to 3 m)</td>
</tr>
<tr>
<td>Widely Spaced</td>
<td>1 to 3 ft (300 mm to 1 m)</td>
</tr>
<tr>
<td>Moderately Spaced</td>
<td>0.3 to 1 ft (100 to 300 mm)</td>
</tr>
<tr>
<td>Closely Spaced</td>
<td>0.1 to 0.3 ft (30 to 100 mm)</td>
</tr>
<tr>
<td>Very Closely Spaced</td>
<td>Less than 0.1 ft (&lt; 30 mm)</td>
</tr>
</tbody>
</table>

### TABLE 5
**FRACTURE OPENNESS DESCRIPTORS**

<table>
<thead>
<tr>
<th>Descriptor</th>
<th>Openness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tight</td>
<td>No visible separation</td>
</tr>
<tr>
<td>Slightly Open</td>
<td>Less than 0.003 ft [1/32 in] (&lt; 1 mm)</td>
</tr>
<tr>
<td>Moderately Open</td>
<td>0.003 to 0.01 ft [1/32 to 1/8 in] (1 to 3 mm)</td>
</tr>
<tr>
<td>Open</td>
<td>0.01 to 0.03 ft [1/8 to 3/8 in] (3 to 10 mm)</td>
</tr>
<tr>
<td>Moderately Wide</td>
<td>0.03 to 0.1 ft [3/8 to 1.2 in] (10 to 30 mm)</td>
</tr>
<tr>
<td>Wide</td>
<td>Greater than 0.1 ft [1.2 in] (&gt; 30 mm) record actual openness</td>
</tr>
</tbody>
</table>
### TABLE 6
FRACTURE ROUGHNESS DESCRIPTORS

<table>
<thead>
<tr>
<th>Roughness Descriptor</th>
<th>Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stepped</td>
<td>Near-normal steps and ridges occur on the fracture surface.</td>
</tr>
<tr>
<td>Rough</td>
<td>Large angular asperities can be seen.</td>
</tr>
<tr>
<td>Moderately Rough</td>
<td>Asperities are clearly visible and fracture surface feels abrasive.</td>
</tr>
<tr>
<td>Slightly Rough</td>
<td>Small asperities on the fracture surface are visible and can be felt.</td>
</tr>
<tr>
<td>Smooth</td>
<td>No asperities, smooth to the touch.</td>
</tr>
<tr>
<td>Polished</td>
<td>Extremely smooth and shiny.</td>
</tr>
<tr>
<td>Slickensided</td>
<td>A polished fault surface, often with a lineation parallel to the displacement direction.</td>
</tr>
</tbody>
</table>

### TABLE 7
FRACTURE FILLING THICKNESS DESCRIPTORS

<table>
<thead>
<tr>
<th>Fracture Filling Descriptor</th>
<th>Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clean</td>
<td>No film or coating</td>
</tr>
<tr>
<td>Very Thin</td>
<td>Less than 0.003 ft [1/32 in] (&lt; 1 mm)</td>
</tr>
<tr>
<td>Moderately Thin</td>
<td>0.003 to 0.01 ft [1/32 to 1/8 in] (1 to 3 mm)</td>
</tr>
<tr>
<td>Thin</td>
<td>0.01 to 0.03 ft [1/8 to 3/8 in] (3 to 10 mm)</td>
</tr>
<tr>
<td>Moderately Thick</td>
<td>0.03 [3/8 in] to 0.1 ft (10 to 30 mm)</td>
</tr>
<tr>
<td>Thick</td>
<td>Greater than 0.1 ft (&gt; 30 mm)</td>
</tr>
<tr>
<td>Weathering Descriptor</td>
<td>Chemical Weathering – Discoloration and/or Oxidation</td>
</tr>
<tr>
<td>-----------------------</td>
<td>-----------------------------------------------------</td>
</tr>
<tr>
<td>fresh</td>
<td>No discoloration, not oxidized</td>
</tr>
<tr>
<td>lightly weathered to fresh (^1)</td>
<td>Discoloration or oxidation is limited to surface or short distance from, fractures: some feldspar crystals are dull</td>
</tr>
<tr>
<td>lightly weathered</td>
<td>Discoloration or oxidation extends from fractures, usually throughout: Fe-Mg minerals are &quot;rusty&quot;, feldspar crystals are &quot;cloudy&quot;</td>
</tr>
<tr>
<td>moderately to slightly weathered (^1)</td>
<td>Discoloration or oxidation is limited to surface or short distance from, fractures: some feldspar crystals are dull</td>
</tr>
<tr>
<td>moderately weathered</td>
<td>Discoloration or oxidation extends from fractures, usually throughout: Fe-Mg minerals are &quot;rusty&quot;, feldspar crystals are &quot;cloudy&quot;</td>
</tr>
<tr>
<td>densely to moderately weathered (^1)</td>
<td>Discoloration or oxidation extends from fractures, usually throughout: Fe-Mg minerals are &quot;rusty&quot;, feldspar crystals are &quot;cloudy&quot;</td>
</tr>
<tr>
<td>densely weathered</td>
<td>Discoloration or oxidation extends from fractures, usually throughout: Fe-Mg minerals are &quot;rusty&quot;, feldspar crystals are &quot;cloudy&quot;</td>
</tr>
<tr>
<td>very intensely weathered</td>
<td>Discoloration or oxidation extends from fractures, usually throughout: Fe-Mg minerals are &quot;rusty&quot;, feldspar crystals are &quot;cloudy&quot;</td>
</tr>
<tr>
<td>decomposed</td>
<td>Discolored or oxidized throughout, but resistant minerals such as quartz may be unaltered; all feldspars and Fe-Mg minerals are completely altered to clay</td>
</tr>
</tbody>
</table>

Notes:
This chart and its horizontal categories are most readily applied rocks with feldspars and mafic minerals. Weathering in various sedimentary rocks, particularly limestones and poorly indurated sediments, will not always fit the categories established. This chart and weathering categories may have to be modified for particular site conditions or alteration such as hydrothermal effects; however, the basic framework and similar descriptors are to be used.

1. Combination descriptors are permissible where equal distribution of both weathering characteristics are present over significant intervals or where characteristics present are "in between" the diagnostic feature. However, dual descriptors should not be used where significant, identifiable zones can be delineated. When given as a range, only two adjacent terms may be combined (i.e., decomposed to slightly weathered or moderately weathered to fresh are not acceptable).

2. Does not include directional weathering along shears or faults and their associated features. For example, a shear zone that carried weathering to great depths into a fresh rock mass would not require the rock mass to be classified as weathered.

# TABLE 9
## FRACTURE DENSITY DESCRIPTORS

<table>
<thead>
<tr>
<th>Descriptor</th>
<th>Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unfractured</td>
<td>No observed fractures.</td>
</tr>
<tr>
<td>Very Slightly Fractured</td>
<td>Core recovered mostly in lengths greater than 3 feet (1 m).</td>
</tr>
<tr>
<td>Slight to Very Slightly Fractured</td>
<td>Core recovered mostly in lengths from 1 to 3 feet (300 to 1,000 mm) with few scattered lengths less than 1 foot (300 mm) or greater than 3 feet (1,000 mm).</td>
</tr>
<tr>
<td>Slightly Fractured</td>
<td>Core recovered mostly in lengths from 0.33 to 1.0 foot (100 to 300 mm) lengths with most lengths about 0.67 foot (200 mm).</td>
</tr>
<tr>
<td>Moderately to Slightly Fractured</td>
<td>Lengths average from 0.1 to 0.33 foot (30 to 100 mm) with scattered fragmented intervals. Core recovered mostly in lengths less than 0.33 foot (100 mm).</td>
</tr>
<tr>
<td>Moderately Fractured</td>
<td>Core recovered mostly in lengths greater than 3 feet (1 m).</td>
</tr>
<tr>
<td>Intensely to Moderately Fractured</td>
<td>Core recovered mostly in lengths from 0.33 to 1.0 foot (100 to 300 mm) lengths with most lengths about 0.67 foot (200 mm).</td>
</tr>
<tr>
<td>Intensely Fractured</td>
<td>Core recovered mostly as chips and fragments with a few scattered short core lengths.</td>
</tr>
<tr>
<td>Very Intensely to Intensely Fractured</td>
<td>Core recovered mostly as chips and fragments with a few scattered short core lengths.</td>
</tr>
<tr>
<td>Very Intensely Fractured</td>
<td>Core recovered mostly as chips and fragments with a few scattered short core lengths.</td>
</tr>
</tbody>
</table>

Note:
1. Combinations of fracture densities are permissible (e.g., very intensely to intensely fractured or moderately to slightly fractured) where equal distribution of both fracture density characteristics are present over a significant core interval or exposure, or where characteristics are "in between" the descriptor definitions.
APPENDIX B

TEST PIT LOGS
TEST PIT LOG

PROJECT NAME: RAWLINS RAW WATER STORAGE
PROJECT NUMBER: 07112
LOCATION: Lat. 41.7363 Long. -107.2597
EQUIPMENT USED: LINK-BELT 210LX

LOGGED BY: ATM
CONTRACTOR'S NAME/COMPANY: A & D DOZERS

PIT DIMENSIONS: LENGTH: 20.0
WIDTH: 3.5
DEPTH: 9.0
WATER LEVEL ELEV: 6.2
BOTTOM OF OVERBURDEN (FT): 8.0

CHECKED BY: EPT
DATE: 9/18/07

SAMPLE

<table>
<thead>
<tr>
<th>ELEV. (FT)</th>
<th>DEPTH (FT)</th>
<th>TYPE AND NO.</th>
<th>SAMPLE DEPTH INTERVAL (FT)</th>
<th>REMARKS</th>
<th>Lithological Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>7088.0</td>
<td>0</td>
<td>D-1</td>
<td>0.0 - 1.0</td>
<td></td>
<td></td>
<td>Ground Surface</td>
</tr>
<tr>
<td>7085.0</td>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7080.0</td>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7079.0</td>
<td>8.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7079.0</td>
<td>9.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7079.0</td>
<td>10</td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>7079.0</td>
<td>12</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>7079.0</td>
<td>14</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

OBSERVATIONS:

GROUNDWATER seepage at west end of pit
Bedrock at 8.0 ft.
Bottom of pit at 9.0 ft.

0.0 - 3.0 ft. Silty Sand
Mostly sand, fine to coarse grained, subrounded to angular; 35-45% fines, low to nonplastic, 5 - 10% gravel, fine to coarse grained, angular, max size = 2 in, slightly moist, tan. (SM)
[ALLUVIUM]

3.0 - 8.0 ft. Clayey Sand
Mostly sand, fine to coarse grained, subrounded to angular; 15 - 20% fines, low plasticity; 10 - 15% gravel, fine to coarse grained, subrounded to angular, max size = 2 in, moist, brown. (SC)
[ALLUVIUM]

Note: Sandstone cobbles in pit at 3.0 - 8.0 ft.

8.0 - 9.0 ft. Sandstone
Mostly sand, fine to medium grained; intensely to very intensely weathered; thinly bedded; moderately to intensely fractured; H5 to H7, gray (Seams of clayey sandstone)
[HAYSTACK MOUNTAINS FORMATION]
**TEST PIT LOG**

**PROJECT NAME:** RAWLINS RAW WATER STORAGE

**PROJECT NUMBER:** 07112

**LOCATION:** Lat. 41.7360, Long. -107.2605

**EQUIPMENT USED:** LINK-BELT 210LX

**LOGGED BY:** ATM

**CONTRACTOR'S NAME/COMPANY:** A & D DOZERS

**PIT DIMENSIONS:**
- **LENGTH:** 20.0
- **WIDTH:** 3.5
- **DEPTH:** 7.0

**WATER LEVEL ELEV:** Not Encountered

**BOTTOM OF OVERBURDEN (FT):** 6.0

**CHECKED BY:** EPT

**DATE:** 9/18/07

---

<table>
<thead>
<tr>
<th>ELEV. (FT)</th>
<th>DEPTH (FT)</th>
<th>Type and No.</th>
<th>SAMPLE DEPTH INTERVAL (FT)</th>
<th>REMARKS</th>
<th>Lithological Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>7084.0</td>
<td></td>
<td></td>
<td></td>
<td>Ground Surface</td>
<td></td>
<td>0.0 - 2.0 ft. Silty Sand</td>
</tr>
<tr>
<td></td>
<td>0</td>
<td></td>
<td></td>
<td>Mostly sand, fine to coarse grained, subrounded to angular, 35-45% fines, low to nonplastic, 5-10% gravel, fine to coarse grained, angular, max size - 2 in., slightly moist, tan. (SM) [ALLUV/UM]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7082.0</td>
<td>2</td>
<td></td>
<td></td>
<td>2.0 - 6.0 ft. Cobbles and Boulders with Silty Sand</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Mostly cobbles, sandstone with claystone seams or pockets, angular; 20% boulders, sandstone, angular, roots; 15-20% silty sand infill in fractures and voids; slightly moist, tan. [INFILL ZONE]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7088.0</td>
<td>6</td>
<td></td>
<td>Bedrock at 6.0 ft.</td>
<td></td>
<td>6.0 - 7.0 ft. Sandstone</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Mostly sand, fine grained; intensely weathered, intensely fractured, H6, slight reaction with Hydrochloric acid (HCl), iron staining, slightly moist, brownish gray. [HAYSTACK MOUNTAINS FORMATION]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7087.0</td>
<td>8</td>
<td>Bottom of pit at 7.0 ft.</td>
<td></td>
<td></td>
<td>End of Log</td>
<td></td>
</tr>
<tr>
<td></td>
<td>10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>12</td>
<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td>14</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**OBSERVATIONS:**

Infill zone consists of colluvial cobbles of Hay Stack Mountains sandstone or claystone and silty sand infill in wide fractures.
**TEST PIT LOG**

**PROJECT NAME:** RAWLINS RAW WATER STORAGE  
**PROJECT NUMBER:** 07112  
**LOCATION:** Lat. 41.7359 Long. -107.2636  
**EQUIPMENT USED:** LINK-BELT 210LX  
**LOGGED BY:** ATM  
**PIT DIMENSIONS:** LENGTH: 20.0  
**WATER LEVEL ELEV:** Not Encountered  
**CHECKED BY:** EPT

<table>
<thead>
<tr>
<th>ELEV. (FT)</th>
<th>DEPTH (FT)</th>
<th>TYPE AND NO.</th>
<th>SAMPLE DEPTH INTERVAL (FT)</th>
<th>REMARKS</th>
<th>SAMPLE</th>
<th>Lithological Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>7155.0</td>
<td>0.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Ground Surface</td>
</tr>
<tr>
<td>7154.0</td>
<td>2.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>8.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>10.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>12.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>14.0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**SAMPLE**

- **Remarks:**
  - **0.0 - 1.0 ft. Silty Sand**
  - Mostly sand, fine to coarse grained, subrounded to angular, 35-45% fines, low to nonplastic, 5-10% gravel, fine to coarse grained, angular, max size = 2in., slightly moist, tan (SM) [EOLIAN/RESIDUUM]
  - **1.0 - 8.0 ft. Sandstone**
  - Mostly sand, fine to medium grained; very intensely weathered, very intensely fractured, H5- H7, light reddish brown. [HAYSTACK MOUNTAINS FORMATION]
  - **Note:** Decreasing weathering and fracturing with depth.

**OBSERVATIONS:**

- | 8.0 | Bottom of pit at 8.0 ft. | |

**END OF LOG**
## TEST PIT LOG

**PROJECT NAME:** RAWLINS RAW WATER STORAGE  
**PROJECT NUMBER:** 07112  
**LOCATION:** Lat. 41.7352 Long. -107.2632  
**EQUIPMENT USED:** LINK-BELT 210 LX

**LOGGED BY:** ATM  
**CONTRACTOR'S NAME/COMPANY:** A & D DOZERS  
**PIT DIMENSIONS:** LENGTH: 20.0  
**WATER LEVEL ELEV:** Not Encountered  
**BOTTOM OF OVERBURDEN (FT):** 1.0  
**LOGGED BY:** EPT  
**DATE:** 9/18/07

<table>
<thead>
<tr>
<th>ELEV. (FT)</th>
<th>DEPTH (FT)</th>
<th>TYPE AND NO.</th>
<th>SAMPLE DEPTH INTERVAL (ft)</th>
<th>REMARKS</th>
<th>Lithological Symbol</th>
<th>Description</th>
</tr>
</thead>
</table>
| 7137.0     | 0          |              |                             |         | Ground Surface      | 0.0 - 1.0 ft. Silty Sand  
Mostly sand, fine to coarse grained, subrounded to angular, 35-45% fines,  
low to nonplastic, 5-10% gravel, fine to coarse grained, angular, max size=  
2 in., slightly moist, tan. (SM)  
[HAYSTACK MOUNTAINS FORMATION] |
| 7136.0     | 1          |              | Bedrock at 1.0 ft.          |         |                     | 1.0 - 6.5 ft. Sandstone  
Mostly sand, fine to medium grained; intensely weathered, intensely  
fractured, H6 - H5, slightly moist, light gray.  
[HAYSTACK MOUNTAINS FORMATION] |
| 7132.0     | 4          |              |                             |         |                     | 2.5 - 3.0 ft. Seams of Brown Sandstone |
| 7130.5     | 6          |              | Hard                        |         |                     | 5.0 - 6.5 ft. Sandstone  
Becomes moderately weathered, moderately fractured, H4 to H6, brown.  
[HAYSTACK MOUNTAINS FORMATION] |
|            | 8          |              |                             |         |                     |                                        |
|            | 10         |              |                             |         |                     |                                        |

**OBSERVATIONS:**
TEST PIT LOG

PROJECT NAME: RAWLINS RAW WATER STORAGE
PROJECT NUMBER: 07112
LOCATION: Lat. 41.7349 Long. -107.2617
EQUIPMENT USED: LINE-BELT 210LX

LOGGED BY: ATM
CONTRACTOR'S NAME/COMPANY: A & D DOZERS

PIT DIMENSIONS: LENGTH: 20.0 WIDTH: 3.5 DEPTH: 9.0
WATER LEVEL ELEV: Not Encountered
BOTTOM OF OVERBURDEN (FT): 0.33

CHECKED BY: EPT
DATE: 9/18/07

SAMPLE

<table>
<thead>
<tr>
<th>ELEV. (FT)</th>
<th>DEPTH (FT)</th>
<th>TYPE AND NO.</th>
<th>SAMPLE DEPTH INTERVAL (FT)</th>
<th>REMARKS</th>
<th>Lithological Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>7138.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Ground Surface</td>
</tr>
<tr>
<td>7138.2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Bedrock at 0.33 ft.</td>
</tr>
<tr>
<td></td>
<td>2-3</td>
<td>B-1</td>
<td>1.0-2.0</td>
<td></td>
<td></td>
<td>Mostly sand, fine to coarse grained, subrounded to angular, 35-45% fines, low to nonplastic, 5-10% gravel, fine to coarse grained, angular, max size = 2 in., slightly moist, tan. (SM) [EOLIAN/RESIDUUM] End of Log</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7120.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Bottom of pit</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

OBSERVATIONS:

Note: Iron staining from 2.0 - 3.0 ft. Becoming less weathered with depth.
TEST PIT LOG

PROJECT NAME: RAWLINS RAW WATER STORAGE
PROJECT NUMBER: 0712
LOCATION: Lat. 41.7348 Long. -107.2599
EQUIPMENT USED: LINK-BELT 210/LX

LOGGED BY: MDH (APEX)/ATM
CONTRACTOR'S NAME/COMPANY: A & D DOZERS

PIT DIMENSIONS:
LENGTH: 18.0
WIDTH: 3.5
DEPTH: 7.0
WATER LEVEL ELEV: Not Encountered
BOTTOM OF OVERBURDEN (FT): 1.5

SAMPLE

<table>
<thead>
<tr>
<th>ELEV. (FT)</th>
<th>DEPTH (FT)</th>
<th>TYPE AND NO.</th>
<th>SAMPLE DEPTH INTERVAL (FT)</th>
<th>REMARKS</th>
<th>Lithological Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>7126.0</td>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Ground Surface</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.0 - 1.5 ft. Silty Sand</td>
<td>Mostly sand, fine to coarse grained, subangular to angular, 35-45% fines, low to nonplastic, 5-10% gravel, fine to coarse grained, angular, max size = 2 in., slightly moist, tan. (SM) [EOLIAN/RESIDUUM]</td>
</tr>
<tr>
<td>7124.5</td>
<td>2</td>
<td></td>
<td>Bedrock at 1.5 ft.</td>
<td></td>
<td></td>
<td>1.5 - 7.0 ft. Sandstone</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Mostly sand, fine grained; intensely weathered, intensely fractured, H6 to H7, 75 - 80 degrees at N 10° E, blocky (3 in x 6 in), roots, iron staining, gray. [HAYSTACK MOUNTAINS FORMATION]</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Decl. = 15 degrees</td>
</tr>
<tr>
<td>7119.0</td>
<td>7</td>
<td></td>
<td>Bottom of pit at 7.0 ft.</td>
<td></td>
<td></td>
<td>End of Log</td>
</tr>
</tbody>
</table>

OBSERVATIONS:

End of Log
# TEST PIT LOG

**PROJECT NAME:** RAWLINS RAW WATER STORAGE  
**PROJECT NUMBER:** 07112  
**LOCATION:** Lat. 41.7369 Long. -107.2613  
**EQUIPMENT USED:** LINK-BELT 210LX

**LOGGED BY:** ATM  
**CONTRACTOR'S NAME/COMPANY:** A & D DOZERS

**PIT DIMENSIONS:**  
- LENGTH: 20.0  
- WIDTH: 4.0  
- DEPTH: 9.0

**WATER LEVEL ELEV:** Not Encountered  
**BOTTOM OF OVERBURDEN (FT):** 1.0

**CHECKED BY:** EPT  
**DATE:** 9/18/07

<table>
<thead>
<tr>
<th>ELEV. (FT)</th>
<th>DEPTH (FT)</th>
<th>TYPE AND NO.</th>
<th>SAMPLE DEPTH INTERVAL (ft)</th>
<th>REMARKS</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>7138.0</td>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td>0.0 - 1.0 ft, Silty Sand</td>
</tr>
<tr>
<td>7135.0</td>
<td></td>
<td></td>
<td>Bedrock at 1.0 ft.</td>
<td></td>
<td>Mostly sand, fine to coarse grained, subrounded to angular, 35-45% fines, low to nonplastic, 5-10% gravel, fine to coarse grained, angular, max size = 2 in., slightly moist, tan. (SM)</td>
</tr>
<tr>
<td>7134.4</td>
<td>2</td>
<td>B-1</td>
<td>1.0 - 2.5 ft, Almost residual soil</td>
<td></td>
<td>1.0 - 2.5 ft, Sandstone</td>
</tr>
<tr>
<td>7127.0</td>
<td>4</td>
<td>B-2</td>
<td>5.0-9.0 ft.</td>
<td></td>
<td>Mostly sand, fine to medium grained, very intensely fractured, very intensely weathered, iron staining, minor root penetration, blocky, H7, slightly moist, light gray. [HAYSTACK MOUNTAINS FORMATION]</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td></td>
<td>Bottom of pit at 0.0 ft.</td>
<td></td>
<td>2.5 - 9.0 ft.</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td></td>
<td></td>
<td></td>
<td>Intensely weathered, intensely fractured, average clast = 6 in, H6, grayish brown.</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td></td>
<td></td>
<td></td>
<td>End of Log</td>
</tr>
</tbody>
</table>

**OBSERVATIONS:**
TP-101: Alluvium with a few intermittent cobbles.

TP-102: Alluvium and an infill zone.
TP 105: Thin eolian/residuum layer over sandstone bedrock.

TP 106: Thin eolian/residuum layer over sandstone bedrock.
TP-107: Thin eolian/residuum layer over sandstone bedrock.

TP-108: Thin eolian/residuum layer over sandstone bedrock.
TP-112: Eolian and residuum overlying a light grey sandstone layer, overlying a grayish-brown sandstone.
APENDIX D

LABORATORY TEST RESULTS
GRANULAR:

100% GRAVEL
43% SAND
57% SILT AND CLAY
24% LIQUID LIMIT

SAMPLE OF: Silt (ML), very sandy

BORING: AR1 @ 0-0.5

PLASTICITY INDEX: 1%

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. 4</td>
<td>100</td>
</tr>
<tr>
<td>No. 8</td>
<td>100</td>
</tr>
<tr>
<td>No. 16</td>
<td>100</td>
</tr>
<tr>
<td>No. 30</td>
<td>100</td>
</tr>
<tr>
<td>No. 50</td>
<td>99</td>
</tr>
<tr>
<td>No. 100</td>
<td>92</td>
</tr>
<tr>
<td>No. 200</td>
<td>57</td>
</tr>
</tbody>
</table>
HYDROMETER ANALYSIS
SIEVE ANALYSIS

TIME READINGS
U.S. STANDARD SIEVES
CLEAR SQUARE OPENINGS

CLAY < 2 D (Silt-Size)
FINE GRAVEL
COBBLES
MEDIUM GRAVEL
FINE Silt
COARSE GRAVEL
COARSE Silt
COARSE CLAY

GRAVEL: 9%
SAND: 44%
SILT AND CLAY: 47%
LIQUID LIMIT: 20%
PLASTICITY INDEX: 4%
SAMPLE OF: SAND (SM) very silty
BORING: TP101 @ 0-2'

Sieve Size | Percent Passing
-----------|-----------------'
No. 3/4"   | 100
No. 3/8"   | 97
No. 4      | 91
No. 8      | 86
No. 16     | 83
No. 30     | 79
No. 50     | 75
No. 100    | 63
No 200     | 47
APPENDIX E

GEOLOGIC RECONNAISSANCE DATA
<table>
<thead>
<tr>
<th>Location</th>
<th>Strike</th>
<th>Dip</th>
</tr>
</thead>
<tbody>
<tr>
<td>F-1</td>
<td>N15E</td>
<td>89</td>
</tr>
<tr>
<td>F-2</td>
<td>N15E</td>
<td>90</td>
</tr>
<tr>
<td>F-3</td>
<td>N14E</td>
<td>89</td>
</tr>
<tr>
<td>F-4</td>
<td>N10E</td>
<td>80E</td>
</tr>
<tr>
<td>F-5</td>
<td>N15E</td>
<td>90</td>
</tr>
<tr>
<td>F-6</td>
<td>N05E</td>
<td>78W</td>
</tr>
<tr>
<td>F-7</td>
<td>N10E</td>
<td>80W</td>
</tr>
<tr>
<td>F-8</td>
<td>N18E</td>
<td>85W</td>
</tr>
<tr>
<td>F-9</td>
<td>N07E</td>
<td>75W</td>
</tr>
<tr>
<td>F-10</td>
<td>N15E</td>
<td>90</td>
</tr>
<tr>
<td>F-11</td>
<td>N10E</td>
<td>90</td>
</tr>
</tbody>
</table>

Michael Hattel
Tuesday, December 4, 2007
Conterminous 48 States
2002 Data
Hazard Curve for PGA
Latitude = 41.735602
Longitude = -107.260644
Data are based on a 0.05 deg grid spacing
Frequency of Exceedance values less than 1E-4 should be used with caution.

<table>
<thead>
<tr>
<th>Ground Motion (g)</th>
<th>Frequency of Exceedance (per year)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.005</td>
<td>3.3687E-02</td>
</tr>
<tr>
<td>0.007</td>
<td>2.5846E-02</td>
</tr>
<tr>
<td>0.010</td>
<td>1.910E-02</td>
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<tr>
<td>0.014</td>
<td>1.3616E-02</td>
</tr>
<tr>
<td>0.019</td>
<td>9.3235E-03</td>
</tr>
<tr>
<td>0.027</td>
<td>6.1765E-03</td>
</tr>
<tr>
<td>0.038</td>
<td>3.9911E-03</td>
</tr>
<tr>
<td>0.053</td>
<td>2.5171E-03</td>
</tr>
<tr>
<td>0.074</td>
<td>1.5666E-03</td>
</tr>
<tr>
<td>0.103</td>
<td>9.7199E-04</td>
</tr>
<tr>
<td>0.145</td>
<td>5.9241E-04</td>
</tr>
<tr>
<td>0.203</td>
<td>3.6154E-04</td>
</tr>
<tr>
<td>0.284</td>
<td>2.1787E-04</td>
</tr>
<tr>
<td>0.397</td>
<td>1.2822E-04</td>
</tr>
<tr>
<td>0.556</td>
<td>7.240E-05</td>
</tr>
<tr>
<td>0.778</td>
<td>3.8738E-05</td>
</tr>
<tr>
<td>1.090</td>
<td>1.9049E-05</td>
</tr>
<tr>
<td>1.520</td>
<td>8.2697E-06</td>
</tr>
<tr>
<td>2.130</td>
<td>2.6334E-06</td>
</tr>
</tbody>
</table>

Ground Motion | Freq. of Exceed. | Return Pd. | P.E. | Exp. Time
(g)           | (per year)       | (years)    | %    | (years)     
0.1882        | 4.0404E-04       | 2475.0     | 2.00 | 50.0        

By: ATM 12/28/07
Checked by: ABP 1-17-08
Appendix F

Data from Previous Studies at Atlantic Rim Reservoir

F.1 CTL Thompson 1981 Boring Logs on Repair Plans
F.2 Gannett Fleming 2006 Test Pit Logs
F.3 Gannett Fleming 2006 Laboratory Test Results
APPENDIX F.1

CTL THOMPSON 1981 BORING LOGS ON REPAIR PLANS
NOTES:
1. ORIGINAL DESIGN SURFACE, TOP AT END EMBANKMENT AND AT YET
   UNTAPE MORTAR MIDDLE CREST FROM REVERSE TO REF.
2. LOCATION OF OBSERVATION WELLS ESTABLISHED BY TANGENT FROB WELLS
   MATURAL PLUMB SO THE SUR.
3. ELEVATION OF COMMUNE SURFACE WELLS ELEAGED BY A LEVEL.

PROFILE AT C OF EMBANKMENT
SCALE: HORIZ. 1"=200' VERT. 1"=20'

LEGEND:
- HIGHLY WEATHERED BEDROCK
- SANDY CLAY TO SILTY SAND
- CLAYEY TO SILTY SHALE
- SANDY, "SILT SAND"

ATLANTIC RIM RESERVOIR
RAWLINS, WYOMING
PROFILE LONG E OF DAM AND CROSS
SECTION A-A' AT STATION 31+00

CROSS SECTION A-A' AT STATION 31+00
SCALE: HORIZ. 1"=200' VERT. 1"=20'

CTL/THOMPSON, INC.
CONSULTING GEOTECHNICAL
AND MATERIALS ENGINEERS
211 WEST 7TH AVENUE, DENVER, COLORADO 80204

JOB NO. 12345
DRAWN BY:...
CHECKED BY:...
INCH. 2 OF 3
NOTES:
1. SURVEYING AND LANDSCAPE DESIGN SERVICES RENDERED TO RESERVOIR DESIGNERS, INC. IN CONNECTION WITH THIS PROJECT.
2. SURVEYING WELLS WERE LOCATED BY CARTESIAN COORDINATE METHOD, HAVING A GOOD DEGREE OF ACCURACY.
3. SURVEYING WELLS WERE ERECTED AT POINTS WHERE HANGING WALLS WERE TO BE ERECTED.
4. SURVEYING WELLS WERE LOCATED AT POINTS WHERE HANGING WALLS WERE TO BE ERECTED.
5. SURVEYING WELLS WERE LOCATED AT POINTS WHERE HANGING WALLS WERE TO BE ERECTED.
6. SURVEYING WELLS WERE LOCATED AT POINTS WHERE HANGING WALLS WERE TO BE ERECTED.
7. SURVEYING WELLS WERE LOCATED AT POINTS WHERE HANGING WALLS WERE TO BE ERECTED.

LEGEND:
- OBSERVATION WELLS
- SURVEYING WELLS
- LOCATION OF OBSERVATION WELLS
- SEEPAGE AREAS

ATLANTIC RIM RESERVOIR
RAWLINS, WYOMING

LOCATION OF OBSERVATION WELLS
AND SEEPAGE AREAS

CTL/THOMPSON, INC.
CONSULTING GEOTECHNICAL
AND MATERIALS ENGINEERS
7800 GRAF AVENUE, DALLAS, TEXAS 75220

SIGNED:
BY:

1981
APPENDIX F.2

GANNETT FLEMING 2006 TEST PIT LOGS
**TEST PIT LOG**

**Date Started:** 05-24-06  **Date Finished:** 05-24-06

**Total Depth of Pit:** 10.0 Ft.

**Inspector:** Jessica Humble, EIT, GIT  **Project:** City of Rawlins - Atlantic Rim Clay Liner Borrow

**Photographic Log:** Yes  **Excavation Contractor:** A & D Oil Field Dozers, Inc.

**Groundwater Observations**

<table>
<thead>
<tr>
<th>Depth (Ft.)</th>
<th>Sample No.</th>
<th>Legend</th>
<th>Description of Materials</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1.0 - 2.9</td>
<td>B-3</td>
<td></td>
<td>Lean clay with sand, residual, brown, moist, medium plasticity, medium dry strength, % dispersion = 25</td>
<td>6&quot; - 2&quot; Considerable concentration of evaporites</td>
</tr>
<tr>
<td>2.5</td>
<td></td>
<td></td>
<td>Thin lenses of clay containing evaporites throughout Moisture content = 10.2%</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td>Bottom of Test Pit = 10.0 Feet</td>
<td></td>
</tr>
</tbody>
</table>

**Surface Elevation:** 7188.0 Ft.
Date Started: 05-24-06
Date Finished: 05-24-06
Total Depth of Pit: 10.0 Ft.

Inspector: Jessica Humble, EIT, GIT
Photographic Log: Yes
Excavation Contractor: A & D Oilfield Dozers, Inc.

---

Total Depth of Pit: 10.0 Ft.

Groundwater Observations
Not Encountered

<table>
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<th>Depth (Ft)</th>
<th>Sample No.</th>
<th>Legend</th>
<th>Description of Materials</th>
<th>Remarks</th>
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<tr>
<td>0</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.5</td>
<td></td>
<td></td>
<td>Sandy silty clay, residual, brown, moist, medium plasticity, medium dry strength</td>
<td></td>
</tr>
<tr>
<td>4.0 - 5.0</td>
<td>B-4A</td>
<td></td>
<td>Thin lenses of clay containing evaporites throughout</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td>Water content = 10.9%</td>
<td></td>
</tr>
<tr>
<td>6.0 - 8.0</td>
<td>B-4B</td>
<td></td>
<td>Maximum Dry Density = 117.5 psf, OMC = 12%</td>
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<td>10</td>
<td></td>
<td></td>
<td>Bottom of Test Pit = 10.0 Feet</td>
<td></td>
</tr>
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Remarks:

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Surface Elevation: 7208.0 Ft.

E Coordinate: Not Encountered

---

Remarks:

---

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APPENDIX F.3

GANNETT FLEMING 2006 LABORATORY TEST RESULTS
Atlantic Rim, TP-3 @ 1-2'

ASTM D422 Sieve Analysis plus Hydrometer

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<th>Sieve Size</th>
<th>Percent Passing</th>
</tr>
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<tbody>
<tr>
<td>TP-3 @ 1-2'</td>
<td>06/02/06</td>
<td>no. 10</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>no. 20</td>
<td>98</td>
</tr>
<tr>
<td></td>
<td></td>
<td>no. 40</td>
<td>94</td>
</tr>
<tr>
<td></td>
<td></td>
<td>no. 100</td>
<td>87</td>
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<tr>
<td></td>
<td></td>
<td>no. 200</td>
<td>80.2</td>
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<tr>
<td></td>
<td></td>
<td>0.041 mm</td>
<td>33.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.030 mm</td>
<td>30.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.019 mm</td>
<td>30.0</td>
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<td></td>
<td></td>
<td>0.011 mm</td>
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<td></td>
<td>0.008 mm</td>
<td>26.3</td>
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<tr>
<td></td>
<td></td>
<td>0.006 mm</td>
<td>25.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.003 mm</td>
<td>17.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.002 mm</td>
<td>8.2</td>
</tr>
</tbody>
</table>

ASTM D4221 Double Hydrometer

<table>
<thead>
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<th>Description</th>
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<th>Sieve Size</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP-3 @ 1-2'</td>
<td>06/02/06</td>
<td>0.051 mm</td>
<td>19.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.036 mm</td>
<td>19.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.023 mm</td>
<td>11.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.013 mm</td>
<td>7.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.010 mm</td>
<td>7.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.007 mm</td>
<td>7.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.003 mm</td>
<td>3.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.002 mm</td>
<td>1.9</td>
</tr>
</tbody>
</table>

ASTM D4221 Percent Dispersion

\[
\%\text{Dispersion} = \frac{\%\text{passing 5 - \mu m in ASTM D4221} \times 100}{\%\text{passing 5 - \mu m in ASTM D422}}
\]

\[
\%\text{Dispersion} = \frac{5.84 \times 100}{23.30} = 25.06\%
\]

As delivered moisture content: 10.2%

ASTM D4318 Plasticity Index

Liquid Limit: 34
Plastic Limit: 16
Plasticity Index: 18
CLIENT: WWC Engineering
PROJECT: Atlantic Rim & Peaking 2 Test Pit Samples
PROJECT LOCATION: Rawlins, Wyoming
SAMPLE LOCATION: TP-3 @ 1-2'

DATE: June 21, 2006
PROJ. NO: 24061081

--- Table ---

<table>
<thead>
<tr>
<th>SAMPLE LOCATION</th>
<th>SOIL CLASSIFICATION</th>
<th>MC%</th>
<th>LL</th>
<th>PI</th>
<th>Cu</th>
<th>Cp</th>
<th>CL</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP-3 @ 1-2'</td>
<td>Sandy Lean Clay (CL)</td>
<td>10.2</td>
<td>34</td>
<td>18</td>
<td>3.41</td>
<td>33.27</td>
<td></td>
</tr>
</tbody>
</table>

--- Table ---

<table>
<thead>
<tr>
<th>SAMPLE LOCATION</th>
<th>D100</th>
<th>D60</th>
<th>D30</th>
<th>D10</th>
<th>% GRAVEL</th>
<th>% SAND</th>
<th>% SILT</th>
<th>% CLAY</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP-3 @ 1-2'</td>
<td>2.000</td>
<td>0.060</td>
<td>0.019</td>
<td>0.002</td>
<td>19.8</td>
<td>56.9</td>
<td>23.3</td>
<td></td>
</tr>
</tbody>
</table>

REMARKS: NR: DENOTES NOT REPORTED DATA   NV: DENOTES NO VALUE
Atlantic Rim, TP-4 @ 6-8’

ASTM D422 Sieve Analysis plus Hydrometer

<table>
<thead>
<tr>
<th>Description</th>
<th>Date Delivered</th>
<th>Sieve Size</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP-4 @ 6-8’</td>
<td>06/02/06</td>
<td>3/8”</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>no. 4</td>
<td>99</td>
</tr>
<tr>
<td></td>
<td></td>
<td>no. 10</td>
<td>93</td>
</tr>
<tr>
<td></td>
<td></td>
<td>no. 20</td>
<td>86</td>
</tr>
<tr>
<td></td>
<td></td>
<td>no. 40</td>
<td>80</td>
</tr>
<tr>
<td></td>
<td></td>
<td>no. 100</td>
<td>69</td>
</tr>
<tr>
<td></td>
<td></td>
<td>no. 200</td>
<td>57.4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.044 mm</td>
<td>52.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.032 mm</td>
<td>49.6</td>
</tr>
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<td></td>
<td>0.020 mm</td>
<td>46.5</td>
</tr>
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<td></td>
<td></td>
<td>0.012 mm</td>
<td>41.8</td>
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<td></td>
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<td>0.009 mm</td>
<td>38.7</td>
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<td></td>
<td>0.006 mm</td>
<td>35.6</td>
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<tr>
<td></td>
<td></td>
<td>0.003 mm</td>
<td>29.4</td>
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<tr>
<td></td>
<td></td>
<td>0.002 mm</td>
<td>13.9</td>
</tr>
</tbody>
</table>

ASTM D4318 Plasticity Index

Liquid Limit: 24
Plastic Limit: 18
Plasticity Index: 6
**CLIENT:** WWC Engineering  
**PROJECT:** Atlantic Rim & Peaking 2 Test Pit Samples  
**PROJECT LOCATION:** Rawlins, Wyoming  
**SAMPLE LOCATION:** TP-4 @ 6-8'  
**DATE:** June 21, 2006  
**PROJ. NO:** 24061061

---

### Grain Size Distribution

**GRAIN SIZE IN MILLIMETERS (mm)**

<table>
<thead>
<tr>
<th>Percent finer by weight (%)</th>
</tr>
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<tbody>
<tr>
<td>100.000</td>
</tr>
</tbody>
</table>

---

### Soil Classification Table

<table>
<thead>
<tr>
<th>SAMPLE LOCATION</th>
<th>SOIL CLASSIFICATION</th>
<th>MCX</th>
<th>LL</th>
<th>PI</th>
<th>CG</th>
<th>Cu</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP-4 @ 6-8'</td>
<td>Sandy Silty Clay (CL-ML)</td>
<td>10.9</td>
<td>24</td>
<td>6</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

### Particle Size Analysis Table

<table>
<thead>
<tr>
<th>SAMPLE LOCATION</th>
<th>D100</th>
<th>D60</th>
<th>D30</th>
<th>D10</th>
<th>% GRAVEL</th>
<th>% SAND</th>
<th>% SILT</th>
<th>% CLAY</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP-4 @ 6-8'</td>
<td>9.500</td>
<td>0.092</td>
<td>0.003</td>
<td>1.0</td>
<td>1.0</td>
<td>41.6</td>
<td>24.2</td>
<td>33.2</td>
</tr>
</tbody>
</table>

---

**REMARKS:**  
NR: DENOTES NOT REPORTED DATA  
NV: DENOTES NO VALUE
Material Information
Contractor: WWC Engineering
Source of Material: Atlantic Rim
Proposed Use: CL-ML

Sample Information
Sampled By: Leather L. Rogers
Sample Location: TP - 4 @ 6-8'
Sample Description: Sandy Silty Clay

Laboratory Test Data
Test Procedure: ASTM D698-91
Method A
Wet Preparation
Mech. Rammer
Maximum Dry Unit Weight, pcf: 117.5
Optimum Water Content, %: 12.0

Moisture Density Relations
Zero Air Voids Curve for assumed specific gravity 2.63

Specifications
Result

Liquid Limit: 24
Plastic Limit: 18
Plasticity Index: 6
% Passing #200: 57.4
% Passing #40: 80.0

Services-Obtain a sample of treated subgrade at the project site and return it to the laboratory. Laboratory test data is performed by Terracon in substantial accordance with ASTM D 4318, Liquid Limit, Plastic Limit, and Plasticity Index of Soils; ASTM D 1140, Amount of Soils Finer than the No. 200 Sieve; and ASTM D 422, Particle Size Analysis of Soils.
Atlantic Rim, TP-4 @ 4-5'
As delivered moisture content: 10.9%

Peaking #2, TP-5 @ 8-10'
As delivered moisture content: 3.7%

Peaking #2, TP-6 @ 2-3'

**ASTM D422 Sieve Analysis**

<table>
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<th>Sieve Size</th>
<th>Percent Passing</th>
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</thead>
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<tr>
<td>TP-6 @ 2-3'</td>
<td>06/02/06</td>
<td>2&quot;</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1 1/2&quot;</td>
<td>95</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1&quot;</td>
<td>87</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3/4&quot;</td>
<td>82</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1/2&quot;</td>
<td>75</td>
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<td></td>
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<td>3/8&quot;</td>
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<td></td>
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<td>no. 4</td>
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<td></td>
<td></td>
<td>no. 10</td>
<td>57</td>
</tr>
<tr>
<td></td>
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<td>no. 20</td>
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<tr>
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<td>no. 200</td>
<td>13.2</td>
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</table>
LEVEL II GEOTECHNICAL DESIGN MEMORANDUM
FOR THE FIVEMILE SITE
RAWLINS RAW WATER STORAGE PROJECT
CARBON COUNTY, WYOMING

Submitted to
WWC Engineering, Inc.
611 Skyline Road
Laramie, Wyoming 82070

Submitted by
RJH Consultants, Inc.
9800 Mt. Pyramid Court, Suite 330
Englewood, Colorado 80112
303-225-4611
www.rjh-consultants.com

August 2008
Project 07112

Robert J. Huzjak, P.E.
Project Manager
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<td>Slope Stability</td>
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<td>Embankment Section Concept</td>
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SECTION 1 - INTRODUCTION

1.1 Purpose

The purpose of the Rawlins Raw Water Storage Level II Study is to evaluate the feasibility of constructing a new raw water storage reservoir for the City of Rawlins (City) at a site designated as "Fivemile Site" (the Site). The purpose of this report is to present a summary of the Level II geotechnical analysis and design performed for the Project.

1.2 Objectives

The objectives of the Level II geotechnical analysis and design are to: 1) Assist WWC Engineering (WWC) develop a conceptual design of an embankment that creates a 644 acre-foot reservoir at the Site, and 2) evaluate technical issues and economics of the dam and reservoir concept.

1.3 Scope of Work

RJH Consultants, Inc. (RJH) performed the following services for this project:

- Performed geologic and geotechnical data collection at the Site and potential borrow area. This data is presented in a Geotechnical Data Report prepared by RJH, dated January 2008.
- Evaluated the geologic and geotechnical conditions at the Site for suitability of a dam and reservoir.
- Performed slope stability and wave run-up analyses to assist in developing a dam and reservoir concept.
- Developed a typical dam section.
- Estimated quantities for embankment fill, imported clay fill, and sand filter material.
- Prepared this report and geotechnical portions of the Project Notebook.

1.4 Authorization and Project Personnel

The work described in this report was performed in accordance with the Agreement for Professional Services between WWC and RJH dated August 3, 2007. RJH personnel responsible for the execution of this work included:
Level II Geotechnical Design Memorandum - Fivemile Site - Rawlins Raw Water Storage Project
August 2008

Project Manager
Robert J. Huzjak, P.E.

Project Engineer
A. Tom MacDougall, P.E.\(^{(1)}\)

Staff Engineer
Emily P. Tyler, E.I.

Technical Review
Edwin R. Friend, P.E.\(^{(1)}\), P.G.

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SECTION 2 - PROJECT DESCRIPTION

2.1 General

The City and the Wyoming Water Development Commission (WWDC) are studying the feasibility of constructing a new raw water storage reservoir for the City. The purpose of the reservoir is to maintain the reliability of the City’s municipal water supply system while accommodating increased water demand and/or decreased water supply (e.g., due to drought). Currently, the City’s largest reservoir, the Atlantic Rim Reservoir, has storage restrictions because of excessive seepage and is not providing the intended service to the water supply system.

A previous Level II Study was performed to develop and evaluate alternatives for maintaining the reliability of the City’s water supply. The study involved a reconnaissance level (Phase I) evaluation of five alternative locations for raw water reservoir storage. Of the five alternatives, a conceptual level (Phase II) evaluation was performed at the site designated Peaking No. 2 Reservoir (which is adjacent to the existing Peaking Reservoir). The Phase I evaluation also recommended that a Phase II study be performed for the Fivemile Site.

WWC informed RJH that the Atlantic Rim Reservoir may be decommissioned due to safety concerns and the embankment fill and clay liner may be available for reuse as liner or fill materials for the proposed Fivemile project.

2.2 Site Location

The Site is located immediately north of Fivemile Ridge on an unnamed, ephemeral tributary to Hay Gulch, which is tributary to Sugar Creek. The Site location is described and is included on Figure 2.1 provided in our previously issued Geotechnical Data Report (January 2008).

2.3 Conceptual Design Requirements

In general, the dam will be designed according to the Wyoming State Rules and Regulations (Rules (1992)). According to the Rules, RJH understands the dam will be a Class III dam. To meet City requirements, the reservoir will need to store 644 acre-feet at the maximum normal pool elevation and seepage losses into the foundation need to be as low as practical because all water needs to be pumped into the reservoir.
SECTION 3 - GEOLOGIC/GEOTECHNICAL CONDITIONS

3.1 General

Geologic and geotechnical information used in pre-design was obtained from published maps and reports and Level II Geotechnical Data Report for Fivemile Site (RJH, 2008). Additional geotechnical and geologic data will need to be collected for later design phases of this project. Consequently, the information presented herein may be superseded and/or modified at later stages of the design process.

3.2 Geology

The Site is located approximately 3.5 miles south-southwest of Rawlins, Wyoming in the Wyoming Basin Province of Rocky Mountain Physiographic Region. The Wyoming Basin Province is characterized by broad structural and topographic basins that are partially filled with Tertiary (67 to 2 million-year-old) deposits and separated by low anticlines (Hunt, 1967).

The Site is situated on the east limb of an anticline and the proposed dam crosses an incised valley with moderate side slopes (generally about 7 percent). Various bedrock outcrops are present and vegetation consists of small shrubs and bushes. The proposed right and left abutments are comprised of a thin veneer of colluvial, residual, or eolian deposits (Hallberg and Case, 1998) overlying intensely weathered sandstone. The valley bottom contains narrow incised channels in-filled with alluvial deposits.

Bedrock at the Site consists of Late Cretaceous (100 to 67-million-year old) Haystack Mountains formations, which is in the Mesaverde Group. The Haystack Mountains formation consists primarily of sandstone with minor amounts of shale.

3.3 Subsurface Conditions

The generalized subsurface profile at the Site consists of alluvium (in the valley bottom) and eolian deposits and/or residual soils (on the valley sides) overlying sandstone bedrock.

In the valley bottom and channels at the Site, the subsurface consists of an alluvial stratum of silty sand or clayey sand up to approximately 8 feet thick. The silty sand contains between 35 and 45 percent fines and between 5 and 10 percent gravel. The fines within the silty sand are low to non plastic. The clayey sand is similar to the silty sand except the fines are slightly more plastic.
On the valley sides above the ephemeral channels, the surficial soil consists of eolian deposits or residual soil. The soil consists of silty sand and extends to approximately 1.5 feet below the ground surface. The soil is generally loose and dry to slightly moist.

Sandstone bedrock underlies the surficial eolian, residual, and alluvial deposits. This sandstone is part of the Haystack Mountains formation, which is part of the Mesaverde Group. Outcrops are present at the Site and the sandstone extends beyond the depths explored (up to 9 feet). Within the zone observed, the sandstone is fine to medium grained, very intensely to moderately weathered (generally decreased weathering with depth), and very intensely to moderately fractured. A generalized subsurface profile along the dam axis is shown on Figure 3.1.

3.4 Groundwater

The depth to groundwater at the Site is below the depths explored in our geotechnical exploration. Additional groundwater and foundation permeability data will need to be collected in subsequent stages of design.

3.5 Seismicity

A reverse fault that forms the Rawlins Uplift, which is located approximately 2.5 miles to the north, is the closest fault to the site and is considered possibly active. According to the USGS, the peak ground motion at the site with a 2 percent probability of exceedance in 50 years (i.e., 2,475-year return period) is 0.19g. According to the Rules, there are no seismic design criteria for Class III dams.
SECTION

PROPOSED DAM CENTERLINE

LEGEND

- - - - EXISTING GROUND
- - - - PROPOSED GRADE
- - ? - ASSUMED CONTACT BETWEEN SUBSURFACE UNITS

0 100 200 400 600
HORIZONTAL SCALE IN FEET

0 20 40 80 120
VERTICAL SCALE IN FEET

NOTE:

1. SECTION IS LOOKING DOWNSTREAM.
SECTION 4 - DESIGN CONSIDERATIONS

4.1 Slope Stability

RJH evaluated stability of various slope configurations to develop preliminary embankment geometry for the Dam. Our slope stability evaluation consisted of three general steps:

1. Develop preliminary material properties and build a computer model.
2. Apply the anticipated “worst case” load conditions.
3. Iteratively calculate factors of safety for a given slope configuration and loading condition until appropriately stable slopes are determined.

4.1.1 Preliminary Material Properties

RJH developed preliminary material properties based on laboratory test results presented in Level II Geotechnical Data Report for Fivemile Site (RJH, 2008), published empirical data and correlations, and experience with similar materials. Most of the Level II design strengths were based on correlations and experience and not laboratory data, and may change when additional geotechnical data is obtained. Based on experience, RJH selected conservative material properties to reduce the possibility that changes in material properties will require design modification that increase project costs.

Preliminary material properties (for unit weight, strength, and hydraulic conductivity) are summarized in Table 4.1.

<table>
<thead>
<tr>
<th>TABLE 4.1</th>
<th>PRELIMINARY MATERIAL PROPERTIES</th>
</tr>
</thead>
<tbody>
<tr>
<td>Property</td>
<td>Alluvium</td>
</tr>
<tr>
<td>General</td>
<td>Silty or Clayey Sand</td>
</tr>
<tr>
<td>Unit Weight Parameters</td>
<td>Dry Density, $\gamma_d$ (psf)</td>
</tr>
<tr>
<td></td>
<td>Moist Density, $\gamma$ (psf)</td>
</tr>
<tr>
<td></td>
<td>Sat. Density, $\gamma_s$ (psf)</td>
</tr>
</tbody>
</table>
4.1.2 Load and Strength Conditions

Based on our experience and the geotechnical site conditions, RJH considered the critical load cases to be:

- Rapid drawdown for the upstream slope.
- Steady state seepage for the downstream slope.

The dam was modeled to be about 70 feet high at the maximum section. The normal pool was set 5 feet below the crest and the phreatic surface for both loading conditions was estimated using procedures outlined in the USACE EM 1110-2-1902 – Slope Stability (USACE, 2003). Figure 4.1 shows the sections analyzed with the corresponding material properties and phreatic surfaces.

4.1.3 Computer-Aided Calculations

Two-dimensional stability was evaluated using the computer program Slope/W©2004. The Morgenstern-Price method was used to calculate driving forces, resisting forces, and factors of safety. Slope/W performed the iterative task of locating the critical failure surfaces and calculating minimum factors of safety. If safety factors were unacceptable, slope geometries were modified and computations were performed again. The results of the stability analyses for the selected configurations are presented in Table 4.2.
TABLE 4.2
SLOPE STABILITY RESULTS

<table>
<thead>
<tr>
<th>Slope</th>
<th>Load</th>
<th>Slope</th>
<th>Minimum Computed FOS</th>
<th>Required FOS$^{(1)}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Downstream</td>
<td>Steady State</td>
<td>2H:1V</td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td>Upstream</td>
<td>Rapid Drawdown</td>
<td>3H:1V</td>
<td>1.3</td>
<td>1.3</td>
</tr>
</tbody>
</table>

Note:
1. The State does not have specific requirements for Factors of Safety (FOS). RJH used the minimum FOS shown based on the USACE published requirements and our experience.

Calculations and computer model output of the slope stability analysis are provided in the Project Notebook.

4.2 Wave Run-Up

RJH calculated wave run-up to evaluate the appropriate freeboard and to size riprap slope protection on the upstream slope. We used procedures from the USACE Manuals 1110-2-1414 (1989) and 1110-2-1100 (2002); USBR Assistant Commissioner – Engineering and Research Technical Memorandum No. 2 (1992); and extreme wind speed data from the USBR online data base HYDRO-MET for the analysis. The design wave was generated based on an adjusted 8-minute wind speed of approximately 78 miles per hour. The resulting wave run-up was 2.15 feet, and a wind setup height was calculated to be 0.059 foot. With an additional 1.5 feet of residual freeboard, the minimum required freeboard was calculated to be 3.71 feet. Calculations of the wave run-up are provided in the project notebook.

Based on the results of the wave height calculations, nominal riprap size was selected. RJH selected 18 inches of riprap with a $D_{50}$ of 9 inches to protect the upstream slope from wave erosion. A 9-inch layer of riprap bedding underlies the riprap. This layer may not be needed based on the gradation of the embankment and should be evaluated at a later stage of design. We utilized the procedure for sizing riprap outlined in USBR’s Standard 13, Chapter 7. The riprap bedding was designed generally according to the USBR’s Standard 13, Chapter 7 and USACE EM 1110-2-2300 (USACE, 2004). The size of the bedding will prevent migration of bedding particles through the riprap. The gradation of the riprap bedding has not been defined during this Level II Study. Calculations of wave height and required riprap size are provided in the Project Notebook.

4.3 Seepage

The reservoir site geology, the dam foundation conditions, and the material available for the embankment fill consists primarily of fractured sandstone. The fractured sandstone (both in-
situ and re-used in the embankment) has a high hydraulic conductivity and seepage will need to be controlled. RJH qualitatively evaluated three concepts to control and reduce seepage:

- A clay core integral with a foundation cutoff.
- A synthetic seepage barrier and reservoir liner.
- A clay core and clay reservoir liner.

Due to the highly permeable nature of the bedrock, significant losses into the fractures of the underlying bedrock formation are likely. As a result, a foundation grout curtain would need to extend relatively deep (30 to 50 feet below the ground surface) and the curtain would need to extend laterally beyond the length of the dam to control losses through the abutments. The volume of grout is expected to be very high and result in very high project costs. This concept would only provide partial control of the potential for significant seepage losses into the bedrock foundation.

RJH considered providing a synthetic “core” and liner as the seepage barrier for this project. Constructing a synthetic liner at this site would require site grading to eliminate bedrock protrusions (that could potentially puncture the liner) and sand bedding under the liner. The liner would extend into the dam and be the seepage barrier. RJH concluded that this system would require significant import of material, significant earthwork, and the system would have moderately high risk of failure due to the pressure differential across the thin synthetic liner.

A clay core and liner were considered to be the most efficient design. Clay is not available on site but can be obtained at a moderate cost from the Atlantic Rim Site. A clay core will provide a safer seepage barrier than a synthetic liner because it is about 1,200 times thicker. A clay reservoir liner can be compacted over more irregular surfaces than a synthetic liner and will provide a positive seepage barrier against losses into the bedrock formation. Clay core particles will be prevented from migrating into the adjacent embankment fill with a sand filter system. During future stages of design, the gradation of the filter will need to be identified. For this design, due to the large particle size anticipated for the embankment fill, RJH assumed a two-stage filter would be needed. The unit cost of ASTM C33 fine aggregate can be used to develop material costs for the first filter zone (closest to the clay core) and ASTM C33 course aggregate can be used to estimate the costs of the second filter zone material.
A quantitative seepage analysis was not performed at this stage in design for three reasons:

- A properly designed filter will be utilized to create an internally stable embankment.
- The entire reservoir will be lined to mitigate seepage losses.
- Calculation of seepage quantities is used to size the drainpipes and drainpipe sizes were not needed for a Level II design.

The core location and geometry for this conceptual design were selected based on the following considerations:

- Reducing the volume of clay will be beneficial to the economic feasibility of the project because clayey soils will need to be imported from an off-site source.
- The reservoir will likely fluctuate significantly and the clay core will need to be protected from desiccation.
- An upstream sloping core would create a relatively unstable upstream slope, especially during rapid drawdown conditions.
- The minimum thickness of the core should be 8 feet for constructability.
- The minimum thickness of the core at the base of the dam should be at least one-fourth of the maximum hydraulic head.

The reservoir liner concept was developed considering:

- The geology at the Site consists of a thin veneer of sandy soil underlain by highly fractured sandstone bedrock to depths of at least 12 feet. Without a liner, RJH anticipates significant seepage losses into the fractured sandstone. If future geotechnical exploration identifies a relatively impermeable zone below the fractured bedrock, the clay liner may be eliminated.
- Significant seepage losses through the reservoir are not acceptable.
- To reduce project costs, the volume of clay should be minimal because clayey soils will need to be imported from an off-site source.
- The reservoir will likely fluctuate significantly and the clay liner will need to be protected from desiccation.
- To reduce the potential for the migration of fines into open fractures in the underlying bedrock, slush grouting should be performed.
4.4 Settlement and Camber

The foundation materials consist of clayey or silty sand overlying sandstone bedrock. These materials generally do not exhibit post-construction consolidation when subjected to loading. Although a formal analysis during final design should be performed, for this Level II Study, the camber of the dam can be estimated to be 2 percent of the height of the dam, or about 1.5 feet of post-construction settlement.

4.5 Embankment Section Concept

Based on result of analyses and experience, RJH developed the concept for the embankment section as shown on Figure 4.2. The upstream slope is 3H:1V, the downstream slope is 2H:1V. The dam is approximately 70 feet high at the maximum section and is a zoned earth dam. The zones include a clay core, embankment fill, a fine sand filter and a course sand/gravel filter and drain. The upstream slope has 18 inches of riprap slope protection overlying 9 inches of riprap bedding.

4.6 Geotechnical Impacts on Hydraulic Structures

4.6.1 Outlet Works

Geotechnical design for the outlet works should incorporate the following to mitigate the potential for seepage and settlement-related issues.

- The dam core should be compacted tightly against the outlet conduit. This is commonly accomplished by encasing the conduit in concrete that forms a trapezoid and compacting core soil against the concrete encasement. Vertical or negatively-sloped interfaces between the core and the outlet conduit should be avoided.
- The sand filter should be in contact with the outlet conduit.
- The pipe should be consistently supported through the embankment to avoid differential settlement of the outlet works. This can be accomplished by locating the outlet conduit north or south of the main alluvial channel where it can be founded on bedrock.

4.6.2 Spillway

The primary geotechnical issues associated with the design of the spillway include:
- Erosion of the spillway channel.
- Seepage beneath the spillway control structure.

If velocities of the spillway flows are less than 8 feet per second (fps), it is appropriate to have an unlined channel. If velocities are more than 8 fps, riprap, soil-cement, or other channel lining should be considered.

The reservoir liner should be connected into the spillway control structure and the liner protected with a downstream filter zone.
MAXIMUM NORMAL WATER SURFACE ELEVATION 7165 FT

CLAY CORE/LINER (NOTE 1)
$\gamma$ SAT = 135 pcf
$\gamma$ = 131 pcf

EMBANKMENT FILL
$\gamma$ SAT = 134 pcf
COHESION = 0
$\phi$ = 33 DEGREES
$\gamma$ = 127 pcf

RAPID DRAWDOWN PHREATIC SURFACE

EXISTING GROUND

ASSUMED TOP OF BEDROCK

EMBANKMENT FILL
$\gamma$ SAT = 134 pcf
COHESION = 0
$\phi$ = 33 DEGREES
$\gamma$ = 127 pcf

STEADY-STATE PHREATIC SURFACE

COMPUTED STEADY-STATE FAILURE SURFACE (F.O.S. = 1.5)

WEATHERED SANDSTONE BEDROCK
$\gamma$ SAT = 138 pcf
COHESION = 0
$\phi$ = 35 DEGREES
$\gamma$ = 132 pcf

EMBANKMENT FILL
$\gamma$ SAT = 134 pcf
COHESION = 0
$\phi$ = 33 DEGREES
$\gamma$ = 127 pcf

NOTE:

1. FOR STEADY-STATE LOADING CONDITION, $\phi$ = 28 DEGREES AND COHESION = 0 psf. FOR RAPID DRAWDOWN LOADING CONDITION, $\phi$ = 0 DEGREES AND COHESION = 1000 psf.
1. EMBANKMENT FILL TO CONSIST OF ON4.2SITE SANDSTONE GRavel AND COBBLES, SILTY SAND, AND CLAYEY SAND.
2. CLAY CORE AND LINER TO CONSIST OF CLAY, SANDY CLAY, AND CLAYEY SAND IMPORTED FROM THE ATLANTIC RIM RESERVOIR SITE. 2 FEET OF CLAY LINER IS TO COVER THE RESERVOIR AREA AND BE COVERED WITH 12 INCHES OF ON4.2SITE GRAVEL.
3. TWO4.2STAGE FILTER AND DRAIN TO CONSIST OF AN IMPORTED FINE CONCRETE AGGREGATE ZONE AND AN IMPORTED COARSE AGGREGATE ZONE.
4. SEEPAGE COLLECTION PIPE TO CONSIST OF AWWA C900 PIPE.
5. RIPRAP NOMINAL D50 IS 9 INCHES.
6. USE WDOT STANDARD RIPRAP BEDDING OR SIMILAR.
SECTION 5 - CONSTRUCTION CONSIDERATIONS

5.1 General

Primary site and constructability issues for the dam currently being considered include existing physical features and constraints, stream diversion, dewatering, excavation and foundation preparation, and sources of construction materials.

5.2 Existing Physical Features

Existing facilities that are potentially impacted by the proposed dam and reservoir include an overhead electric power line, two unpaved roadways, and a water pipeline.

The overhead power line traverses the proposed reservoir generally in a north-south direction. The power line would need to be relocated along a new alignment or encased and buried. A poorly-defined unpaved “two-track” roadway generally follows the alignment of the overhead electric line and would be inundated by the proposed reservoir. It is likely this roadway can be eliminated if the overhead power lines are realigned. A well-defined unpaved roadway is located downstream of the proposed toe of the dam. The emergency spillway intersects this road and dam access may be limited during emergency spillway flows. Additionally, this roadway would likely be damaged from spillway flows. A water pipeline is located adjacent to the well-defined unpaved roadway and may be overstressed if buried under the proposed dam, or may conflict with outlet works piping.

5.3 Stream Diversion

The outlet works would be located near the existing stream channel. Excavation, foundation treatment, and fill placement for the embankment will likely be required within the stream channel. Stream diversion will be required to protect the construction work in the valley. As the proposed dam is located on an ephemeral channel, stream diversion, coupled with timing the construction to occur during typically dry months, will likely be a cost effective method of addressing this issue.

Diversion concepts will likely include construction of an earthen cofferdam upstream of the dam and placement of diversion conduits to pass flow through the construction area. The outlet works could be incorporated into the stream diversion concept to take advantage of the completed outlet works conduit to pass stream flows during construction.
5.4  Dewatering

Dewatering will likely not be needed for this project due to the limited depth of excavation and the lack of shallow groundwater.

5.5  Excavation

Excavation into bedrock will be required to construct the dam foundation, outlet works, and spillway. Most of the shallow bedrock consists of sandstone that is moderately to intensely weathered and intensely to very intensely fractured. We expect that the bedrock excavation will require large earthwork equipment including dozers, track-mounted excavators, and rippers. We do not anticipate that blasting will be required for excavations.

5.6  Foundation Treatment

Foundation treatment for the dam will require clearing and grubbing of the vegetation within the limits of dam footprint and reservoir and preparing the final subgrade.

Most of the vegetation within the limits of the embankment and appurtenant facilities consists of grasses and small sage brush that are not anticipated to have deep root systems. We have assumed that within the stream channels, the upper 4 to 8 feet of the soil horizon consists of clayey sand and outside of the channels the upper 3 inches of the soil horizon consists of silty sand material. We anticipate that this material will be removed and stockpiled for re-use as topsoil.

Slush grouting will be needed below the clay liner in the reservoir and below the dam to mitigate the potential for migration of fines into open bedrock fractures.

Based on available data, the degree of fracturing of the bedrock below the clay liner is relatively high. We have assumed the volume of grout needed to accomplish the slush grouting will be approximately 1,500 cubic yards (cy).
5.7 Construction Materials

5.7.1 General

The primary materials required for dam construction are several types of earthfill for the embankment, concrete for the spillway and outlet works, and riprap. Primary sources of these materials are described in the following sections.

5.7.2 Embankment Fill

The primary source of embankment fill is the soils and weathered bedrock within the reservoir basin below the elevation of normal pool. The volume required to build the embankment is approximately 260,000 cy, which is available within the reservoir basin. Benefits of using material from within the reservoir basin to build the dam are that removal of this material would increase the overall storage capacity of the reservoir and the haul distance is short.

5.7.3 Clay Core and Liner

The primary source of clay core and liner material is the Atlantic Rim Reservoir area. Based on review of existing data, either the existing embankment material or nearby areas contain sufficient clay to construct the core and liner. The approximate volume of clay required to construct the core and liner is 141,000 cy. The haul distance from the Atlantic Rim Reservoir to the Site is approximately 1.5 miles.

5.7.4 Filter/Drain Sand

Based on our review of local geology, it is unlikely that material suitable for use in the filter drain can be developed economically on site. Therefore, it should be assumed that this material would be obtained from a commercial quarry and transported to the Site in trucks.

5.7.5 Riprap

Riprap may be obtained from two sources: The Atlantic Rim Reservoir or by screening the on-site bedrock excavation material. Riprap derived from the Atlantic Rim Reservoir would require hauling and the size is not confirmed. It may be possible to screen the material excavated from the reservoir area to develop riprap that is suitable for use as upstream slope protection. Economically obtaining riprap from on-site deposits may be difficult.
potential to develop on-site sources of riprap should be further evaluated in a later phase of project development. For purposes of developing a cost opinion for the project, it should be assumed that riprap would be obtained from a commercial quarry and transported to the site in trucks. If an on-site source can be identified and developed, the net result would be a reduction in overall project costs. Riprap from on-site sources would likely not meet the durability requirements commonly used for similar projects. Durability should also be further studied in subsequent phases of the project.

5.7.6 Concrete

Conventional concrete for outlet works and spillway construction would be obtained from existing commercial concrete plants in Rawlins and trucked to the site.
SECTION 6 - REFERENCES


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