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Final Report

Owl Creek Irrigation District Storage

Level II Study

Submitted to:

Wyoming Water Development Commission
Cheyenne, Wyoming

Submitted by:

Short Elliott Hendrickson Inc.

In association with:

Anderson Consulting Engineers, Inc.
Plumley & Associates, Inc.

July 30, 2008

Owl Creek Irrigation District Storage
Level II Study

SEH No. A-WWDC00502.00

July 30, 2008

Professional Engineer (Civil)
Douglas M. Yadon
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Date
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I hereby certify that this report was prepared by me or under my direct supervision, and that I am a duly Licensed Professional Engineer under the laws of the State of Wyoming.

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PROFESSIONAL GEOLOGIST
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I hereby certify that the site geologic conditions described in Sections 5.3.4 and 5.4.2 of this report (except for seismotectonic conditions) were prepared by me or under my direct supervision, and that I am a duly Licensed Professional Geologist under the laws of the State of Wyoming.

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Table of Contents

List of Tables

List of Figures

Final Report

Owl Creek Irrigation District Storage
Level II Study

Prepared for Wyoming Water Development Commission

1.0 Introduction

1.1 Purpose and Scope

The primary purposes of the Owl Creek Irrigation District Storage, Level II Study are to:

- Assess the source of apparent losses in or around Anchor Reservoir, evaluate concepts to mitigate those losses, and examine the feasibility of expanding existing storage in Anchor Reservoir by raising internal dikes.
- Evaluate alternative storage sites identified in the Owl Creek Master Plan Level I and provide conceptual designs and cost estimates for selected preferred alternative(s) that serve the estimated supplemental irrigation needs of the District.
- Perform a flow conveyance analysis to address the potential to route South Fork Owl Creek flows to North Fork Owl Creek in conjunction with alternate storage described above.

The location of the study area is shown on Figure 1.1-1. The scope of this study is fully responsive to the Scope of Services in Exhibit "A" of the Consultant Contract for Services.

1.2 Authorization and Responsibility

This project was authorized by Consultant Contract for Services No. 05SC0292679 effective July 1, 2005, as modified by Amendment No. 1 effective November 14, 2006 and Amendment No. 2 effective November 8, 2007, between the Wyoming Water Development Commission (WWDC) and Short Elliott Hendrickson Inc. (SEH). The official contractual representative for the WWDC was Lawrence M. Besson, Director of the Wyoming Water Development Office (WWDO). Ron Vore served as the WWDC Project Manager and primary point of contact for SEH on both technical and administrative matters.

SEH's Project Manager for this study was Douglas M. Yadon, Wyoming PE No. 4650, and all engineering work on the project was performed under his responsible charge. Except as described herein, the work for this project was performed by Mr. Yadon and other selected SEH staff including Alan C. Jewell, PE, William R.

Kelly, PE, Aaron S. Ritter, EIT, Christopher Wichmann, Tom Stanfill and Amber Leyba. Anderson Consulting Engineers, Inc. (ACE) of Fort Collins, Colorado performed the hydrology and water rights task and assisted in the needs assessment. The work by ACE was led by Jay Schug. Mr. Patrick Plumley, Wyoming PG No. 3387 supervised the geologic aspects of the project, including alternative storage site reconnaissance and exploration. Consultation on site geology during a dam site reconnaissance was provided by W. Roger Hail, PG of Fort Collins, Colorado. Finally, Joe Campbell, President of the Owl Creek Irrigation District (District) and Anthony Martinez, Anchor Reservoir Dam Tender provided invaluable assistance and information throughout the project. Personal artifacts and information on Anchor Reservoir from W. Roger Hail and Mr. Willard Wilson also proved invaluable.

2.0 Information Collection and Literature Review

2.1 General Information

Relevant and applicable background information on all of the key topics and tasks was collected for this Level II study. This included, but was not necessarily limited to, seeking information from and/or contact with the following sources:

- U.S. Bureau of Land Management (BLM)
- U.S. Geological Survey (USGS)
- U.S. Bureau of Reclamation (USBR)
- Wyoming Water Development Commission (WWDC)
- Wyoming Department of Environmental Quality (WDEQ)
- Wyoming Game and Fish Department (WGFD)
- Wyoming State Engineer’s Office (WSEO)
- Wyoming State Geological Survey (WSGS)
- Wyoming Board of Land Commissioners/State Lands and Investments Board (WBLC/SLIB)
- Wyoming Geographic Information Science Center (WyGISC)
- Bureau of Indian Affairs (BIA)
- Office of the Tribal Water Engineer (TWE), Wind River Reservation
- Arapaho Business Council
- Hot Springs County Assessors Office

Information was acquired from these agencies and entities by one or more of the following methods: download from their internet site, acquisition of published materials from the agency/entity or libraries, telephone/email contacts, and personal visits. Information gathered through these efforts formed the basis for the subsequent tasks, and significant sources of information are listed in the references in Section 8.0 and are also incorporated in the project GIS described below.

2.2 Project GIS (Task 7)

The results of a portion of the data collection efforts were incorporated into a Geographic Information System (GIS). A GIS can be described as a three-

dimensional mapping tool that can be used to evaluate and compare spatial data pertaining to a wide range of topics. Numerous maps can be "stacked" to overlay information; each map (or "theme") incorporates data (or "attributes") pertaining to the theme.

The primary objective of this GIS was to compile the directly relevant available information pertaining to this area and combine it with new GIS data generated within this project. A large amount of spatial data and imagery is available free of charge for download via the Internet from various federal, state, and local agency websites. Additional GIS data was also gathered through personal contact with relevant agencies. Sources included:

- Wyoming Geographic Information Science Center (WyGISC)
- Natural Resources Conservation Service (NRCS) Spatial Data Gateway
- United States Geological Survey (USGS)
- Wyoming Water Development Commission (WWDC)
- Hot Springs County
- Wyoming Game & Fish Department (WGFD)
- Bureau of Land Management (BLM)

These sources were accessed early in the study and a preliminary GIS was compiled to which relevant additional information collected or generated during the investigation was added. Information collected from these sources and incorporated into the project GIS includes, but is not limited to, the following:

- Roads;
- Hydrography;
- Public Lands Survey (PLSS);
- Soils and geology;
- Crucial big game habitat;
- Sage grouse lek buffer zones;
- NWI wetlands; and
- Property ownership.

In addition to these already available GIS coverages, data collected during the completion of field and office efforts were incorporated into the project GIS as appropriate. These data sets include the following:

- Potential reservoir locations;
- Potential new diversions and canal alignments;
- Sinkhole locations;
- Geologic mapping and information; and
- Subsurface exploratory information.

The Project Deliverables CD contains the GIS coverages as shapefiles and indicates their source.

The project GIS was not intended to be a complete and comprehensive collation of all available spatial data. Only those available themes deemed pertinent to this Level II Study were incorporated. Also, GIS data from the Level I Study were not included, except as it pertains to this project. Should the project sponsors acquire additional GIS information in the future, it can easily be incorporated at that time at their discretion.

3.0 Anchor Dam and Reservoir Operations Analysis (Task 3)

The purpose of this task was to evaluate and resolve discrepancies between existing stream gage inflows above and below Anchor Reservoir and to provide recommendations to mitigate losses due to seepage. The respective upstream (SFOY) and downstream (SFOC) gages in relation to Anchor Reservoir are shown in Figure 3.0-1. Figure 3.0-2 illustrates the discrepancies between the gages on an annual basis for the available data. The Level I Study examined these gage records and estimated the following apparent losses between the gages (after accounting for evaporation):

- 1960-1969: 9402 ac-ft/yr
- 1970-1984: 7809 ac-ft/yr
- 1991-2002: 6101 ac-ft/yr

Given demonstrated water shortages in the basin, it is critical to understand the apparent large ongoing losses in the vicinity of Anchor Reservoir since they account for approximately 30 percent (or more) of the South Fork Owl Creek yield above Anchor Reservoir. Specifically, this task analyses the above gage discrepancies to accomplish the following goals:

- Estimate the portion of the total loss allocable to storage loss in Anchor Reservoir;
- Estimate the portion allocable to seepage/conveyance loss in the canyon below Anchor Dam; and
- Make recommendations for economically feasible alternatives (if any) for lessening seepage in either of the above cases.

Obtaining an accurate picture of the extent and nature of any losses in the vicinity of Anchor Reservoir is necessary to evaluate the economic feasibility of seepage mitigation measures (Section 4.0), and/or optimization of Anchor Reservoir operations to maximize usable storage individually or in tandem with constructed additional storage downstream of Anchor Reservoir (Section 5.0).

The following analyses and field investigations were completed to accomplish the above goals:

- **Review of Historic Seepage and Mitigation.** The history of Anchor Reservoir was reviewed to understand the nature, location and quantity of previous sinkholes and leaks as well as to understand the technologies/remedial measures applied to address those losses and their effectiveness (Section 3.1).

- **Canyon Loss Evaluation.** Losses were evaluated in the canyon below Anchor Reservoir using a WWDC gage installed specifically for this Level II Study directly below Anchor Dam (Figure 3.0-1). Flows from this gage were calibrated and directly compared to the downstream (SFOC) gage to analyze losses specific to the canyon (Section 3.2.1).
- **Water Balance Model.** A computerized daily water balance model was developed to not only quantify the reservoir losses, but also to assess their potential location and characteristics to the degree practicable. By comparing storage, inflow, outflow, evaporation, runoff from ungaged basins and other factors on a daily basis for the period of record, a more accurate picture of the nature of losses in Anchor Reservoir has been developed. Specifically, the relationship of losses at Anchor relative to elevation and time has been developed. Losses have been modeled as a series of sinkholes or leaks obeying the orifice equation with a specific start date, elevation and exponential time decay; and as seepage through reservoir sediments (Section 3.2.2).
- **Reservoir Drogue Evaluation.** To supplement the above water balance model, a field investigation was conducted using drogues during high reservoir pool in an attempt to pinpoint any large and potentially repairable losses occurring in Anchor Reservoir and to confirm to the extent possible conclusions of the Water Balance Model (Section 3.2.3).
- **Reservoir Reconnaissance.** Reconnaissance of the reservoir bottom during empty reservoir conditions was conducted to confirm identified seepage areas and also to identify other seepage areas (Section 3.2.4).

Conclusions of the above analyses are presented in Section 3.2.5, and potential seepage mitigation alternatives are discussed in Section 3.2.6. Section 3.3 (Subtask 3B – Operations Analysis) discusses the implications of the conclusions of the above analyses on the operation of Anchor Reservoir both with and without alternative storage downstream. Finally, Section 4.0 utilizes the results of the above analyses to evaluate the feasibility of raising the elevation of the internal Anchor Reservoir Isolation Dikes.

3.1 Pertinent Anchor Reservoir History

Anchor Reservoir was originally conceived as early as 1934 (U.S. Bureau of Reclamation, 1950). Initial studies for the project were conducted in 1940 (U.S. Bureau of Reclamation, 1971) and various studies were conducted in the years following. Anchor Dam was finally constructed between 1957 and 1960 with an intended design capacity of 17,400 acre-feet (U.S. Bureau of Reclamation, 1971). It is a thin concrete arch dam having a structural height of 208 feet and 69,350 cubic yards of concrete were used in its construction. The crest of the service spillway is at elevation 6451.

The piezometric surface at the reservoir site is in excess of 1,000 feet below the bottom of the reservoir. This condition precludes seepage mounding to the reservoir elevation and thus the hydraulic gradient of seepage remains high. Sinkhole #1 was identified in the reservoir area as early as 1952, and numerous sinkholes have developed since first filling. Anchor Reservoir has remained well below its original intended capacity ever since, although heavy rains in 1991 reportedly filled the reservoir to within three feet of the service spillway (Kurtz, 1991). This would correspond to an elevation of approximately 6448.5; however the Bureau states on

their website (U.S. Bureau of Reclamation, 2003) that "*the maximum reservoir elevation to date is believed to have been at Elevation 6418.5 on July 5, 1967*."

3.1.1 Dam Foundation Construction

Solution cavities and other problematic phenomena were noted from the beginning of construction. Two to three foot wide solution cavities were noted along bedding dip in a six foot thick bed of dolomite in the Tensleep Sandstone at the toe of the left dam abutment. Discovery of these features prompted additional investigations including an extensive 35-hole drilling program that uncovered additional solution cavities. In April 1958, Owl Creek was intentionally diverted into a solution cavity near the dam foundation. A total of 254 acre-feet was routed into this cavern over a two week period, with no trace of emergence of the water in the canyon below and no sign of any impact to water levels in any exploratory holes. A team of consultants recommended concreting of known caverns and grouting of solution channels and concluded that the relatively tight shales of the Chugwater/Dinwoody formations underlying the rest of the reservoir would impede significant seepage. Construction proceeded with additional excavation and plugging of cavities.

3.1.2 Sinkhole #3

In 1958, a 300-foot diameter circular crack in the reservoir area was discovered 700 feet upstream of the right abutment. In 1961, during first filling of the reservoir, this area collapsed, forming Sinkhole #3 (Figure 3.1-1). Sinkhole #3 is believed to be the largest sinkhole formed in the reservoir. An estimated 90,000 cubic yards of material and 880 acre-feet of water over a 26-hour period were consumed by the sinkhole. In one particular half-hour period, the estimated inflow to this sinkhole was estimated to be 2,210 cubic feet per second (cfs). Maximum vertical displacement was measured as 70 feet. Later that year, an inverted, graded filter was constructed in an attempt to plug the leak. About 45,000 cubic yards of compacted rock, cobbles, gravel, sand, and compacted earth was placed in the sinkhole. Additional attempts to plug the hole were conducted in 1962, when 70 bales of hay, 26,000 cubic yards of soil via dozer, and 5,175 cubic yards of material via blasting were placed in the hole. Later that year, a dike was constructed around the sinkhole to allow storage to elevation 6375. By 1964, a total of 65,000 yards of material had been placed or sluiced into Sinkhole #3. In 1975, isolation dikes having a crest elevation of 6415 and a volume of 55,000 cubic yards were constructed to isolate the Waste Rock Area (described below) from the reservoir. In 1978, one of these dikes was overtopped and breached, and later reconstructed with a spillway capacity of 75 cfs and a crest elevation of 6412.8 (U.S. Bureau of Reclamation, 2003).

3.1.3 Sinkhole #11

Sinkhole #11 (Figure 3.1-1) developed in 1963 along the strike of a known fault and was 300 feet long, 35 feet wide, and 11 feet deep. Various attempts to plug this sinkhole ensued, including blanketing of the sinkhole in 1965. In 1967, an additional compacted earth blanket and isolation dike were installed in an attempt to minimize losses.

3.1.4 Waste Rock Area and Construction of Isolation Dikes

Leaks were confirmed in the waste rock area (Figure 3.1-1) in 1963 and large reservoir losses were noted. Various efforts were made to plug numerous leaks in this area, and sometime before or during 1967 a dike was constructed to elevation 6387 to isolate the numerous leaks in the waste rock area. Numerous additional

sinkholes formed in 1974, and in 1975, and as noted previously, isolation dikes having a crest elevation of 6415 and a volume of 55,000 cubic yards were constructed to isolate the Waste Rock Area from the reservoir.

3.1.5 Other Sinkholes and Leaks

The above represent only a small portion of the various sinkholes and leaks identified by the Bureau over the years. In excess of 50 sinkholes have reportedly been cataloged (Jarvis, 2002). A 1991 news article (Kurtz, 1991) quoted Bureau spokesman Ted Christenson as follows:

> *"The sinkholes generally appear 200 yards above the dam, and some years they do not appear at all. Other years, as many as 25 sinkholes will drain the reservoir, so there's no telling if the water will be available."*

3.1.6 Repairs with Sluiced Material

Several attempts to repair leaks via sluicing material were documented. Several internal dikes were constructed in the interior of the reservoir to capture sediment to be used for repairs. One breakthrough that consumed water at a rate of an estimated 200 cfs was plugged via sluicing the following year with 50 cubic yards of material. The area was later ponded intentionally and accepted no water. The later performance of this plug is undocumented.

3.1.7 Reservoir Operations Restrictions

On March 30, 1992, Contract 1-07-60-W0701 was signed between the Bureau and the Owl Creek Irrigation District and amended in 1993 (Nelson Engineering, 2004) that placed operation restrictions on the reservoir and turned maintenance operations over to the District. To prevent damage to the waste rock isolation dikes and to minimize sinkholes, the reservoir is operated to not exceed elevation 6415, and the Bureau must be notified anytime the reservoir elevation exceeds 6400 feet (U.S. Bureau of Reclamation, 2003). A notch in the dikes begins spilling at elevation 6412.8 feet, where storage capacity is reported as 7,572 acre-feet (U.S. Bureau of Reclamation, 2003). Under the "Sinkhole Maintenance" clause, the contract states:

> *"The District may make repairs to the sinkholes and dikes. Repairs that the District chooses not to accomplish shall be made solely at the discretion of Reclamation. The cost of any discretionary repairs, determined necessary by Reclamation, shall be paid by the United States.*"

The Bureau's website (U.S. Bureau of Reclamation, 2003) describes the current state of the reservoir as follows:

> *"No attempt is made to store water in the reservoir from season to season under present operating conditions. The dam is currently operated to provide flood control and extend the irrigation season with what water can be stored. Sinkholes continue to develop in the reservoir each summer as the weight of the water collapses the floor and sinkholes develop. Some of the sinkholes are new, whereas others are old sinkholes that reappear from previous years"*.

3.1.8 Level I Study Loss Mitigation Alternatives

The Owl Creek Master Plan Level I (Nelson Engineering, 2004) summarizes various technologies that were utilized in the 1960s and 1970s to determine the nature of the

seepage and also to attempt to repair the reservoir. The Level I Study also examined alternatives for lining Anchor Reservoir and concluded that full restoration of storage (i.e., to original capacity) in Anchor Reservoir would be too costly compared to constructing alternate storage downstream. The Level I Study identified various potential incremental strategies for restoring storage in Anchor Reservoir that became the basis for Task 3 and Subtask 4A of this study.

3.2 Subtask 3A – Evaluation of Losses and Mitigation Alternatives

3.2.1 Evaluation of Losses in Canyon Below Anchor Reservoir

3.2.1.1 Gage Calibration

As part of this project, WWDC installed a continuously recording stream gage immediately below Anchor Dam to record runoff season flows for comparison to the SEO-operated gage below Anchor Reservoir (SFOC) shown on Figure 3.0-1. Readings were physically downloaded from the WWDC gage at regular intervals using WWDC-supplied equipment during the 2005 runoff season. Two methods of processing and calibrating the data downloaded from the WWDC gage were evaluated:

- **Calibration via In-stream Cross Section and Velocity Measurements.** Streamflow was measured using a calibrated Marsh-McBirney in-stream flowmeter at several rates of flow from Anchor Dam to develop a calibration curve to convert stage (pressure) readings from the WWDC gage to flowrates. In-stream readings were collected at various times during the runoff season and also in conjunction with the second method described below.
- **Calibration via Anchor Dam Hollow-Jet Valve Outlet Rating Curves.** In cooperation with Anthony Martinez, the hollow-jet valves were opened progressively in a series of increasing flowrates. The USBR-developed outlet rating curves provided by Mr. Martinez were utilized to estimate a flowrate and develop a second independent rating curve for the WWDC gage.

Both methods of gage calibration resulted in apparent gains (rather than losses) in the stream in the canyon below Anchor Dam after analysis and correction of the data as discussed later in Section 3.2.1.2. Calibration via the in-stream flow method resulted in higher apparent gains than the outlet calibration method. The calibration via the Anchor Dam outlet rating curves was believed to provide a more reliable gage rating curve for the following reasons:

- Due to high velocities and the irregular, rocky bed conditions in the stream, flows greater than 50 cfs could not be measured in-stream safely by conventional methods. Measurements at both low and high flows exhibited considerable variability.
- Alluvium at the WWDC gage location (and throughout the canyon below Anchor Dam) is very coarse (up to boulder sizes) and judged capable of conveying significant unmeasurable underflow.
- Rating curves of fixed hydraulic structures (i.e., the Anchor Dam outlet) would be expected to be more repeatable than those based on in-stream flow measurement calibrations.

The gage stage-discharge calibration curve was evaluated using several known fitting methods, including power, exponential and a USGS method utilizing a best fit of the following formula (USGS, 1977):

$$Q = C\,(G - e)^{b}$$

Where:

Q = discharge,

G = stage, and

C, e and b are coefficients to match the curve to the data.

The USGS and exponential curves provided the best data fit, and the exponential curve was selected as the preferred rating curve due to it's slightly better data fit.

3.2.1.2 Gage Comparison

Once the rating curve for the WWDC gage was developed, flow data at the WWDC gage could be compared to data at the existing SEO gage (SFOC) below the canyon. Based on high frequency data from the SEO gage, a travel time of approximately 60 minutes between the Anchor Dam Outlet and the SEO gage was computed and used to adjust the data as necessary. Because the SEO gage includes a small intervening drainage basin (Figure 3.0-1), a water-balance was utilized in an attempt to at least approximately account for runoff from this basin. Precipitation records from the nearest available weather station with daily records for the period (Thermopolis 2 located in Thermopolis, and available at http://www4.ncdc.noaa.gov/cgi-win/wwcgi.dll?wwDI~StnSrch~StnID~20022886) was utilized to model runoff from the basin.

The gage regression results are shown in Figure 3.2-1 after adjusting for basin runoff. The regression indicates that the stream has apparent net gains of approximately 6 percent between the two gages, and that there are no indications of any net measurable losses in the canyon below Anchor Dam. The sources of these relatively small apparent net gains may include:

- Residual, unavoidable errors due to underflow at the WWDC gage;
- Losses from the reservoir to the channel (abutment seepage) not measured by the WWDC gage;
- Groundwater (ambient discharge from channel slopes between the gages); and/or
- Unavoidable limitations on measurement and analysis accuracy (biased towards gains).

A plot of the data residuals (apparent gain or loss in the stream for individual data points) versus reservoir head is shown in Figure 3.2-2. This plot indicates that there appears to be little relationship between apparent gains and losses in the stream and head in the reservoir. Because the apparent gains in the stream are within the probable measurement and analysis accuracy of the canyon water loss evaluation, it is assumed for purposes of other analyses in this report, particularly the Reservoir

Water Balance Model described in the next section, that gains or losses in the canyon below Anchor Dam are zero.

3.2.2 Reservoir Water Balance Model

3.2.2.1 Methodology and Assumptions

A computerized water balance model was developed that computes a historical daily reservoir routing and water balance of the reservoir and computes a best-fit estimate of the losses due to sinkholes/orifices as the independent variable in the water balance equation. Specifically, the water balance model is composed of the following components:

- ***Anchor Reservoir storage volumes*** (from USBR Hydromet data for Anchor Dam)

Inflows:

- ***South Fork Owl Creek inflow*** (via the USGS/SEO stream gauge above Anchor Reservoir (SFOY in Figure 3.0-1))
- ***Local reservoir area inflow*** (precipitation/snowmelt from the ungaged local basin draining to Anchor Reservoir, including Middle Fork Owl Creek)

Outflows:

- ***Anchor Dam discharge*** (from the USGS/SEO stream gage below Anchor Reservoir (SFOC in Figure 3.0-1))
- ***Evapotranspiration*** (based on local pan evaporation data or similar climate station records for end-of-month surface area and any significant losses due to local area phreatophytic evapotranspiration)
- ***Sinkhole/Leak Zone losses*** (solved for by model). Examination of the data indicated that apparent losses are strongly related to the square root of head in the reservoir. A sinkhole model was developed whereby sinkholes are modeled hydraulically as single orifices whose loss rates are according to the orifice equation:

  $$Q = C3.14D^2 (H - E)^{0.5}$$

  Where:

  Q = discharge (cfs),

  H = Reservoir head (feet),

  E = Elevation of the sinkhole (feet),

  D = Effective orifice diameter of the sinkhole in feet, and

  C = Constant related to the characteristics of the opening.

  Sinkholes are modeled as having a specific start date and specific elevation. Examination of the data also indicated that apparent sinkhole losses tend to decrease over time, potentially due to repairs and/or dynamic changes in the sinkhole itself. To model this behavior, an exponential decay equation was adopted:

$$Q(t) = Q\, e^{(-(1 / (1.44269 * h)) * (t - tstart)/365)}$$

Where:

Q is computed per the orifice equation above,

Q(t) is the discharge from the sinkhole at time t,

h = Sinkhole half-life (years), and

tstart = “start” date/time of the sinkhole (days)

- ***Reservoir bottom seepage*** (solved for by model). In addition to the sinkhole losses described above, a reservoir bottom seepage model was also incorporated. This model assumes that a constant thickness permeable blanket overlies the reservoir bottom, making seepage losses proportional to the effective reservoir head over the blanket as well as the area inundated. This model effectively results in losses that are generally much more linear in relation to reservoir head since they are governed by a Darcian flow regime. This seepage model attempts to approximately account for seepage through sediments present in the reservoir bottom area before construction and accumulated over time, but does not allow for any change in thickness over time.
- ***Base loss*** (solved for by model). In addition to the above losses, a small, constant base loss was modeled when an examination of the data indicated that a persistent loss appears to be occurring somewhere in the system that is unrelated to any other model component. Potential sources of the base loss may include model and/or measurement error, stream losses between the two gages above and below Anchor either flowing into or out of the reservoir, and/or losses deep within the reservoir bottom. For purposes of modeling, the base loss was modeled as independent of any other model component (including reservoir head) except for Anchor inflow. The base loss was computed as the Anchor inflow or the constant base loss, whichever is smaller. A potential physical explanation for such a loss is that during the relatively long periods where inflows to Anchor are occurring, but no water is being stored, water is infiltrating into the reservoir sediments and finding seepage pathways out the bottom of the reservoir. Thus, such losses are unrelated to reservoir head as modeled.

3.2.2.2 Model Limitations

It is important to note that the model as outlined above cannot distinguish between single and multiple sinkholes, but it can identify their approximate aggregate size. Sinkholes/leak zones identified by the model are best conceptualized as an elevation zone of leakage that could potentially be distributed areally over a large portion of the reservoir bottom. The model cannot distinguish the x-y location or even extent of an individual leak zone, but it can locate the larger leakage zones by an approximate elevation. Also, the “start date” computed by the model is not necessarily the actual start date of the sinkhole/leak zone but rather a best-fit of the model equations to the data and may represent one or more events. Because of the above, it is not necessarily possible or recommended to correlate modeled sinkholes/leak zones to records of historic sinkholes and other recorded phenomena in the reservoir except in general terms.

3.2.2.3 Model Results

Sinkholes and Leak Zones were incorporated into the model on a manual trial and error basis by examination of the daily records of the residual storage losses and gains after adjusting for all previously accepted water balance inputs and outputs. A total of 16 major Sinkholes/Leak Zones were modeled. Anchor Reservoir is believed to have experienced many more loss events than this based on historic records, and examination of the data indicate that a large number of relatively high loss but apparent short duration (several day or month) events have occurred and no attempt was made to model such losses in order to prevent over-fitting of the model to the data. The 16 major losses and other losses modeled generally have multi-year impacts on storage and appear to account for the bulk of observed losses. Parameters for the modeled losses are given in Table 3.2-1.

Figure 3.2-3 illustrates the model fit (predicted losses versus actual losses). Unmodeled excursions are evident on the plot, including a probable localized precipitation/runoff event that illustrates the limitations associated with attempts to account for ungaged runoff, as well as a probable unmodeled short duration Sinkhole/Leak Zone. Figure 3.2-4 illustrates the evolution of the total modeled losses (from Sinkholes/Leak Zones, Reservoir Seepage and Base Losses) over the recorded history of the reservoir.

General observations and conclusions regarding the above model results are as follows:

- ***The model fit is good***. A regression analysis of the predicted and actual loss rates is given in Figure 3.2-3. The relatively high R^2 value of 0.83 indicates a good fit of the model to the 1961-2007 data. This would indicate that the relatively simple loss model (Sinkholes/Leak Zones, Reservoir Seepage and Base Losses) appears to predict the physical losses in Anchor Reservoir reasonably well. The unique signature associated with the bulk of the losses (Sinkhole/Leak Zones with specific elevation and start date) tends to match well with historic descriptions of the source of losses. Additionally, it is unlikely that any large errors associated with either of the stream gauges or the canyon below Anchor would exhibit such a unique loss signature. It is concluded that the majority of the apparent historic losses in Anchor Reservoir are likely associated with physical phenomena in the reservoir and are generally not the result of stream gauge error or downstream channel losses.
- ***Overall reservoir losses have reduced over time.*** Figure 3.2-4 indicates a significant reduction in loss rates over time. Reduction in loss rates is likely due to both physical repairs (including the installation of internal dikes and repairs to larger sinkholes) as well as natural processes such as sedimentation of the reservoir bottom. It is also possible that some potential loss zones have naturally partially sealed over the years due to collapse and/or sedimentation. The model may also be indicating that the repairs that have been initially successful have tended to remain effective over the years. Decays in loss rates below approximately elevation 6350 to 6355 may in large part be due to the substantial sediment accumulation in the reservoir below these approximate levels. To some extent, loss rate decays above 6350 to 6355 may be due to some accumulation of sediment blanket, although there does not appear to be substantial thicknesses accumulated in those areas. Relatively low sediment accumulation does not necessarily preclude large loss rate reductions if sediments can form a stable plug

that switches loss flows from an orifice regime to a Darcian regime. Such a regime change could theoretically decrease flows substantially with a relatively small amount of sediment accumulation.

- ***Unexpected losses can occur at any time.*** The model indicates that new modeled Sinkholes/Leak Zones may appear or reappear/reopen at any time (with the most recent modeled Sinkhole/Leak Zones occurring as recently as 1995 and accounting for approximately 14 percent of all apparent losses), typically in response to high pool elevations in the reservoir. While these sudden increases in loss rate tend to be offset by the overall reduction in losses described above, this unpredictable behavior of the reservoir must be factored into the analyses of any repairs of the reservoir as part of ongoing operations.

- ***Losses are likely higher than indicated by a direct comparison of the gages or by the Level I Study.*** The substantial ungaged intervening area between the gage above Anchor Reservoir and Anchor Dam (including Middle Fork Owl Creek) results in an apparent significant inflow to the reservoir that is not offset by relatively lower evaporation losses or other factors in the Water Balance Model. This inflow is unaccounted for in a direct comparison of the gages above and below Anchor Reservoir. While the Level I Study accounted for evaporation losses, it did not account for intervening basin runoff gains, which appear to be significant. After accounting for these gains, the model predicts that overall losses have been approximately 12,500 ac-ft/yr (Table 3.2-1) as opposed to the following losses reported in the Level I report (Nelson, 2004):

  - 1960-1969: 9402 ac-ft/yr
  - 1970-1984: 7809 ac-ft/yr
  - 1991-2002: 6101 ac-ft/yr

  It is important to note, however, that the ungaged inflows are predicted by applying a precipitation runoff estimate to a basin area, and thus are likely only approximations of actual inflows. This also means that the magnitude of the above apparent excess losses (relative to a direct comparison of the gages) may also be approximate.

- ***Approximately 1/4 of historic losses are modeled as attributable to Base Losses***. About 1/4 of the reservoir losses come from what are defined as base losses which are not related to reservoir head and are modeled as constant (but not to exceed reservoir inflows). Because these losses appear to be persistent and constant, and Sinkhole/Leak Zone losses appear to heal over time, these apparent losses account for a significantly larger relative portion of the total losses under current reservoir conditions. Of all the modeled losses, these apparent losses are the least understood because there are several explanations of what could potentially cause them. These losses could be explained by any or all of the following causes:

  - *Cause #1: Stream losses in ungaged basins*. Ungaged basin runoff occurring between the area gaged above Anchor Reservoir (SFOY) and Anchor Dam (including the Middle Fork Owl Creek basin) could conceivably be seeping into leak zones outside the typically inundated area of the reservoir. Such leak zones could presumably be located anywhere within the ungaged basin where they could cause a persistent, relatively constant loss of runoff.

Although such losses could conceivably be widely distributed through the basin, it would seem more likely that they would occur along drainage paths (e.g., swales, stream channels). No records or observations of such concentrated seepage losses on the Middle or South Forks above Anchor or elsewhere in the intervening drainage basins are known. Even if such concentrated losses are present, it would likely be very challenging to identify and quantify them. Furthermore, even if found, it may be technically difficult, problematic to permit, and costly to mitigate such losses.

- *Cause #2: Unavoidable measurement or model errors biased towards a constant loss.* Measurement bias errors associated with stream gages or errors with the model itself could potentially result in apparent constant losses. A more refined and sophisticated model could potentially provide more insight on this potential cause, but such a result is not certain by any means given the inherent complexity of the conditions being modeled and the unavoidable errors associated with even the most carefully collected stream gage data.

- *Cause #3: Losses relatively deep within the reservoir attributable to periods of low or no reservoir storage.* Inflows to Anchor Reservoir during periods when there is little or no storage could potentially be finding their way to low level leakage zones where at least a portion of the flows are intercepted and lost. Such losses, if they exist, could prove difficult to verify, locate and potentially difficult to mitigate, because they may underlie significant soft sediment accumulation. To the degree such losses primarily occur outside the irrigation season, they are less of a concern.

Given the history of Anchor Reservoir, it is judged likely that at least some or most of the Base Losses are attributable to Cause #3, since low-level leakage zones appear to exist, and reservoir sediments are not completely impermeable. Inflow to the reservoir is thus likely to be saturating a portion of the reservoir bottom, enabling losses to deep-seated, potentially buried leaks. Given the conditions noted above, attempting to determine the actual cause(s) of the apparent Base Losses indicated by the model would be very expensive and the results may very well be inconclusive. Thus, further study of the Base Loss issue does not appear warranted.

- The bulk of historic Sinkhole/Leak Zone losses are limited to four elevation zones within the reservoir. Up to 69 percent of the historic losses can be modeled using only five elevation zones as follows:

  - Elevation 6306 (Sinkhole/Leak Zone 7) accounts for approximately 12 percent of historic losses and appear to be located deep within the reservoir.

  - Elevation 6340 to 6345 (Sinkhole/Leak Zones 4, 9 and 13) accounts for approximately 8 percent of historic losses.

  - Elevation 6347 to 6353 (Sinkhole/Leak Zones 2, and 10) accounts for approximately 6 percent of historic losses.

  - Elevation 6370 to 6377 (Sinkhole/Leak Zones 3, 5, 11, and 16) accounts for approximately 20 percent of historic losses.

- Elevation 6387 to 6397 (Sinkhole/Leak Zones 6, 8, 12, and 14) accounts for approximately 23 percent of historic losses.

Figure 3.2-5 illustrates the locations of these elevation zones relative to a plan view of Anchor Reservoir and previously mapped sinkholes and leak zones. Since all of the above indicated losses utilize the loss decay model, leakage through many of these individual Sinkholes/Leak Zones has already decayed to relatively low levels. However, given the apparent recurrent nature of Sinkholes/Leak Zones within these zones, new losses could potentially reappear at any time.

3.2.2.4 Projected Losses

As indicated in Figure 3.2-4, the model estimates only a small overall decay in loss rate between 2000 and 2007. Loss rates have experienced a relatively slow decay since as far back as 1980, which is believed to be approximately when major repair efforts in Anchor Reservoir ceased. This is in contrast to relatively large decays in loss rates in previous periods, especially below elevation 6395. Sedimentation rates in the reservoir may have increased beginning around approximately 1990 when the low-level sediment outlet was closed. The potential sealing effect of sediments may be slowing as the dead pool storage volume between the closed lower and still open upper outlet is filled. It is likely that the current outlet invert represents an upper limit above which the rate and extent of further sediment accumulation may be less than previously due to flushing of sediment through the outlet. Some sediment would still accumulate in the delta in the upper pool area as it has historically, but the rate of areal expansion of the delta would likely be relatively slow as compared to the previous filling of dead pool volume.

Due to the above factors, the probability that new breakthrough Sinkholes/Leak Zones will periodically occur, and the existence of apparently constant and persistent Base Losses, there is no reason to believe that loss rates in the coming decades will naturally decrease substantially below current levels. Thus, 2007 loss rates are judged likely to approximate future loss rates, given what is known about historic losses and reservoir history. Loss rates could also increase significantly if new sinkholes or leak zones develop.

3.2.3 Reservoir Drogue Evaluation

3.2.3.1 Methodology

Drogues are relatively inexpensive devices utilized by the United States Navy, the National Oceanic and Atmospheric Administration and numerous other entities in the study of both large and very small currents in ocean and lake environments. Drogues are designed to float underwater with the current while tethered to a low-drag surface float that prevents them from sinking and allows their movements to be tracked from the surface. The drogue device utilized in Anchor Reservoir was the “Holey Sock Drogue” illustrated in Figure 3.2-6. This device has been perfected over several decades and was adapted for use in Anchor Reservoir.

The use of drogues in Anchor Reservoir was predicated on the theory that small, but detectable currents associated with concentrated zones of leakage would attract the drogues. If such seepage-induced currents attracted the drogues, then they would be expected to be eventually drawn to the location of the seep. Currents as small as 1-2 feet per minute or smaller are reportedly detectable by these devices.

3.2.3.2 Results

A set of ten drogues was constructed to be deployed as an array throughout the reservoir to search for concentrated leaks (Figures 3.2-7 and 3.2-8). The deployment was conducted during the week of June 23, 2005. Pool level conditions were ideal, with a relatively high approximate pool elevation of 6400 achieved during the deployment. Drogues were deployed at various depths corresponding to seepage zones identified by the Water Balance Model throughout the reservoir and allowed to search.

Because the experiment required several days, it was deemed that closing the outlets to eliminate outlet-induced currents was not feasible. High winds prevailing generally from the southwest during the deployment hampered efforts substantially by creating surface drag on the drogues. The outlet-induced currents and wind drag prevented a reliable estimate of any seepage induced currents from being calculated and also limited the ability of the drogues to conduct an unbiased search for leaks in the reservoir, because drogues were generally drawn to the northwest by both the wind drag and outlet current. Efforts were also impeded in some cases by deeper drogues becoming entangled by relatively high topography at the search depth within the reservoir.

Figure 3.2-5 illustrates the location of four potential leaks identified by the drogue search along the northwest rim of the reservoir. Three of these potential leaks at an elevation of approximately 6350 to 6355 (identified as #4, #5 and #6 on Figure 3.2-5) were later confirmed as having apparent surface expressions during field reconnaissance as described in Section 3.2.4. One leak (#7 "Rockfall Area" on Figure 3.2-5) could not be confirmed by field reconnaissance because it appears to be located within a large, steep talus slope (Figure 3.2-9). Due to the steepness of the slope relative to the height of the drogue, it was not possible to pinpoint an elevation of this apparent leak with any degree of precision. The elevation(s) of this leak is estimated to lie within the elevation range 6365 to 6395.

An effort was made to measure currents in the vicinity of the leaks using a current meter specially constructed for the project (Figure 3.2-6), however no currents were detected at any of the identified apparent leaks above the estimated precision of the device (approximately 0.1 foot per second). It is estimated that the measurable induced currents at the surface of the reservoir bottom associated with these leaks could be from 0.1 feet per second to several orders of magnitude lower. This is particularly true at the Rockfall Area, because a relatively large flow could be distributed through a large area on the talus slope.

3.2.4 Site Reconnaissance

3.2.4.1 Methodology

The reservoir bottom was examined on August 17, 2005 when the reservoir was effectively empty. This reconnaissance allowed inspection of the reservoir for any surficial expressions of leakage and to verify the results of the Drogue Study. Due to the accumulation of very soft sediments/ground, a swath of reservoir bottom approximately 300 to 600 feet wide and extending approximately 2000 feet directly west of the dam could not be examined. A total of six surficial expressions were located during the reconnaissance and their locations are shown on Figure 3.2-5.

3.2.4.2 Results

Photos of the surficial expressions of the seepage features are shown in Figures 3.2-10 through 3.2-16. Features 1, 2, and 3 coincidently lie along an internal canal that is believed to have been used by the Bureau in repair operations in the 1960s. It is not apparent why these features are coincident with the canal. Features 4, 5, and 6 are clustered near the northwest side of the reservoir on the gently sloping banks and coincide with leak zones identified by the Drogue Study discussed in Section 3.2.3. All of the Features 1 through 6 were located with a portable GPS unit and lie at an elevation of approximately 6370, which indicates these features experienced approximately 35 feet of reservoir head at the peak 2005 pool elevation.

3.2.4.3 Estimated Significance of Located Features

The soils surrounding each of the six features appeared to be fine-grained and relatively erodible. Each of the six features appeared to be relatively intact and undisturbed. They also did not appear to be any part of a larger, more subtle feature such as a wider, circular depression detectable by visual examination. Assuming these apparent leaks are controlled by orifice flow, this would indicate that the surficial expression of these leaks is larger than the associated buried orifice because the surrounding soils would erode significantly if orifices were the same scale as the surface features and losses were sustained over a long period. Assuming a maximum tolerable average flow velocity (without further erosion) of 4 feet/sec into these features, then the largest feature (#6) could sustain a maximum flow of approximately 1 cfs. Assuming this peak flow occurred at peak 2005 pool level, this largest feature would be associated with a buried orifice the equivalent of 2-3 inches in diameter and would only account for approximately 3 percent of the total projected 2007 loss rate of the Water Balance Model at the 6400 pool elevation per Figure 3.2-4. Extending this calculation to all features, it is estimated that all six features together would account for no more than 8-9 percent of the total projected 2007 loss rate at a pool of 6400.

3.2.5 Gage Accuracy

Both the gages above and below Anchor Reservoir (SFOY/SFOC of Figure 3.0-1) were examined as part of this project to evaluate District concerns regarding the accuracy of the gages. Specifically, the reliability of the upper gage was questioned due to the apparent settlement the gage has suffered over the years as well as the large amount of sediment that accumulates in the approach to the gage.

Any stream gage is expected to have some amount of day-to-day error due to changing streambed conditions, gage settlement (if applicable) and various other factors. These errors will result in reported readings that are both higher and lower than the actual flow in the stream creating positive and negative errors. According to the SEO, gages are calibrated monthly via field flow measurements according to standard USGS stream gauging procedures. Such frequent calibrations would be expected to correct the conditions described by the District over the long period of record such that positive and negative errors effectively cancel out.

As long as there is no systematic bias in the gage readings either high or low, then the conclusions of this study are not impaired by gage errors, because the expected day-to-day gage errors will not significantly impact the Water Balance Model long-term average results. There are reasons to conclude that there is no systematic bias in the gage readings:

- The frequent calibration of the gages by the SEO is according to well-researched standard procedures used throughout Wyoming and the United States. It is presumed that calibrations were performed in a similar manner when the gages were under the jurisdiction of and operation by the USGS prior to 1995.
- The fit of the Water Balance Model strongly suggests that the bulk of the losses are explained by the orifice model described in Section 3.2.2 whereby large losses begin at a certain date at a specific elevation, orifice size and specific exponential time decay. The unique nature of the losses does not match with the characteristics that would be exhibited by apparent losses caused by a systematic bias in the gages. Systematic biases in the gages, if present at all, appear to be small and hidden in the "noise" of the model and/or potentially the relatively smaller "Base Losses" of the Water Balance Model.

3.2.6 Conclusions and Recommendations

The following conclusions and recommendations have been developed based on the results of the various investigations described above:

3.2.6.1 <u>Losses in Canyon Below Anchor Dam</u>

There appear to be no significant losses in the canyon below Anchor Dam above the SEO Gage Station (SFOC). Analyses have indicated apparent gains of approximately 6 percent of flow in the canyon between Anchor Dam and the SEO gage immediately below Anchor Dam ("SFOC" of Figure 3.0-1). These apparent gains have several potential causes as described in Section 3.2.1, including measurement and analysis error. For purposes of the other analyses described herein, it has been assumed that there are no losses or gains in this portion of South Fork Owl Creek. Based on this conclusion, the following action is recommended:

- *Continue recording Anchor Dam outlet settings.* It is recommended that recording of the settings of the Anchor Dam Outlet Gates continue and include values of percent open, date and time of setting change, and notations of any condition that may be altering the flow out the gates not related to pool level such as clogging or debris in the trashrack that appears to be reducing flow below what would otherwise be expected. The above values should ideally be tabulated in a logbook at each setting change to form a continuous record. These records would help any future analysis of losses in Anchor Reservoir as well as provide a basis to verify conclusions regarding losses in the canyon below Anchor over a longer period of record.

3.2.6.2 <u>Losses in Anchor Reservoir</u>

There is strong evidence of substantial historic losses in Anchor Reservoir. An investigation of reservoir history (Section 3.1), the development and analysis of an Anchor Reservoir Water Balance Model (Section 3.2.2), field reconnaissance via drogue deployment (Section 3.2.3) and later dryland reservoir bottom reconnaissance (Section 3.2.4) all provide evidence that sinkholes and leakage zones within Anchor Reservoir can explain most or all of the apparent losses between the SFOY and SFOC stream gages shown on Figure 3.0-1. In fact, losses may significantly exceed those indicated by a direct gage comparison due to runoff gains from a large, ungaged basin (which includes Middle Fork Owl Creek) that drains to Anchor Reservoir below the upper gage. Average losses over the history of the reservoir are estimated to be as much as 12,500 ac-ft/yr (including any losses in the ungaged basin

above Anchor Reservoir), but loss rates appear to have decreased substantially over the history of the reservoir.

Anchor Reservoir is believed to still be experiencing significant losses. Although loss rates have apparently decreased over the years, projected 2007 loss rates are still substantial enough to render long-term storage (multi-month) in Anchor Reservoir ineffective (Table 3.2-1). Some potential sources of a portion of the losses were identified in the field reconnaissance investigations and via the Water Balance Model (Section 3.2.2), and potential mitigation alternatives are discussed in Section 3.2.7. There is also strong evidence that losses in Anchor Reservoir are subject to sudden large increases due to the opening of new seepage pathways and/or sinkholes typically in response to higher pool levels.

3.2.7 Mitigation Alternatives for Identified Losses

This section presents concepts for potential mitigation alternatives for identified seepage losses in Anchor Reservoir. As described later in Section 3.3, Anchor Reservoir will remain an integral part of water operations for the District whether alternate storage is constructed downstream or not. The findings of this study described above indicate that detailed and reliable cost benefit analyses cannot be performed based on the results of this study for several reasons including the following:

- The continued apparent recurrence of sinkholes/leak zones cannot be reliably predicted for the reservoir as a whole based on the results of this study. Any small-scale mitigation of known leaks could be rendered ineffective at a later date by the appearance of a new sinkhole or leak zone in another area. While the existing history may give some indication of the recurrence interval of new sinkholes/leaks, there is no way to know if this recurrence rate would be predictive of future recurrence rates. Thus, risk cannot be adequately quantified based on current information. An extensive and costly subsurface investigation of the reservoir bottom could potentially provide some insights as to expected recurrence rates, but such an investigation cannot be justified by the results of this study. Without a reliable recurrence rate, reliable cost-benefit analyses cannot be performed
- Leaks must be evaluated on a case-by-case basis and leakage rates for the specific leak and reservoir as a whole must be known. To estimate the benefit of repairing a given sinkhole/leak, the effect of the repair in terms of net storage gained must be known. To correctly evaluate this, a full reservoir routing simulation would need to be conducted utilizing current loss rates with and without repairs to estimate the net storage gained. A lifespan would be assigned to the repair to account for the recurrence interval (assuming a reliable recurrence can be calculated or negotiated) of new leaks as described above. None of the currently identified leaks are believed to be large enough to justify such an intensive analysis. In fact, the cost of repairs described in Section 3.2.7.1 for some small scale features such as those identified by this study (Features 1 through 6 of Section 3.2.4) are likely to cost less than the economic analyses required to justify them. The appearance of larger sinkholes or leaks in the future could potentially warrant such an analysis, however.

The District has expressed the desire to have some guidance provided as to potential small-scale repairs that could be constructed in a low-cost manner to address some of

the potential leaks identified above. While economic analyses of these repairs cannot be easily computed or justified for the reasons listed above, applying very low-cost repairs to known small scale leakage features is considered to be low-risk if suitable funds can be located. The following section describes some low-cost repair concepts and recommendations.

3.2.7.1 Repairs to Small Surface Features on Level Ground

Surface features similar to those identified in Section 3.2.4 (Features 1 through 6) have the potential to be repaired fairly easily as they are identified. Reconnaissance of the reservoir bottom following periods of relative high pool levels could likely identify most of the potential leak zones with visible surface expressions in the portion of the reservoir where travel by foot is possible. Relatively thorough reconnaissance of the accessible reservoir bottom can be conducted in a day or less. Once identified, two potential repair concepts to reduce seepage through identified features could be applied, depending on the materials available. These concepts would be suitable for repairs to small-scale features such as Features 1 through 6 identified in section 3.2.4. They would not be suitable for repairs to larger-scale features. The concepts are described below and illustrated in Figure 3.2-17:

Concept A. This repair concept utilizes a very well graded soil ideally composed of a uniform blend of materials sized over a large gradation range to replace a portion of the soils surrounding a potential leak. Where materials cannot be found that meet the coarser end, an optional coarse layer can be laid under a finer-grained layer. This concept would be ideally applied where such material can be found nearby in the reservoir area to minimize haul distances. The area around the identified feature would be excavated and the well graded borrow would be placed in a mound over the excavated area. The objective of this repair is to provide a “bridge” over the feature that would ideally at a minimum reduce large orifice flows to smaller Darcian-type flows. It is estimated that such a repair could be constructed by a contractor for $1,600 or less, assuming reasonable equipment and labor rates.

Concept B. This repair concept is similar to Concept A, except it utilizes a high-strength geotextile to provide a more reliable “bridge” over the identified feature, and also allows finer-grained borrow soils to be utilized. This concept would likely provide a slightly more reliable repair and may be more accommodating of locally available borrow at the expense of a somewhat higher cost. It is estimated that such a repair could be constructed by a contractor for $2,500 or less, assuming reasonable equipment, material and labor rates. Wherever feasible, Concept B is recommended in place of Concept A.

3.2.7.2 Rockfall Area Potential Leak

The area identified as Feature 7 as part of reservoir drogue evaluations described in Section 3.2.3 is a potential source of concentrated leakage. Unfortunately, site and weather conditions did not permit an estimate of losses associated with this apparent leakage area. A conceptual mitigation alternative for this leak, depending on its characteristics, is to remove the existing loose rock and debris from the slope, if feasible, and grout the resulting bedrock around the leak area if it can be identified. Access to the area is problematic due to the soft sediments in the reservoir bottom adjacent to the slope. Without substantial additional information, the feasibility of repairing this area cannot be assessed. The following is a base framework for a plan to further investigate the feasibility and potential benefit of repairing this area:

- **Conduct a supplemental drogue experiment.** A small scale version of the drogue experiment described in Section 3.2.3 could provide the necessary information regarding an estimated leakage rate from this area by observing the movements of drogues towards the leak. To provide the highest value from the effort, the supplemental drogue evaluation should ideally be conducted at a high pool elevation (6395 or higher) during a period when the Anchor Dam outlets can be closed (for a period of 1-2 days), and preferably conducted when winds are at a minimum (consideration should be given to night operations). The drogue(s) should be redesigned so that they are shorter and wider so that a more accurate estimate of the elevation of the leak can be obtained. The drogues should also be redesigned so that surface drag is greatly reduced or eliminated.
- **Detailed dryland reconnaissance, including geologic mapping and geotechnical investigation.** After the location of the leakage area is refined, a detailed dryland reconnaissance should be conducted to map the local geology in the vicinity of the leak and estimate the depth of talus on the slope near the leak as well as provide information on the characteristics of the underlying bedrock. If feasible, a subsurface investigation should be conducted to try to identify the leak. Such information would provide a basis for recommending the nature and extents of remediation alternatives.
- **Collect geotechnical information and survey on the reservoir sediments.** Conducting a geotechnical exploration program to collect information on the reservoir bottom sediments would provide information as to the feasibility of constructing any potential remedy identified above.

It is necessary to point out that the cost of the above exploratory program and the risk that it fails to identify a significant source of leakage amenable to repair may outweigh the benefit of any ultimate water savings. As discussed previously, it is not feasible to evaluate the costs versus the benefits to arrive at defensible conclusion as to proceeding with this program.

3.2.7.3 Base Losses

The Water Balance Model described in Section 3.2.2 estimated that approximately 25 percent of historic losses are "Base Losses" that appear to be unrelated to reservoir head. Three potential causes of these losses were identified as follows:

- *Cause #1: Stream losses in ungaged basins*
- *Cause #2: Unavoidable measurement or model errors biased towards a constant loss*
- *Cause #3: Losses relatively deep within the reservoir attributable to periods of low or no reservoir storage*

To the extent losses are attributable to Cause #1 or Cause #3, there is some potential that the source(s) of these losses could be repaired; however, the model cannot provide any more information as to the probable cause of these losses without significant additional effort.

Cause #1 could potentially be addressed by repairing any concentrated seeps in the ungaged drainage basin above Anchor Reservoir. Cause #3 could be addressed by providing a lined conveyance canal that would route flows through the reservoir area during periods where the reservoir contains no or minimal storage. This lined

conveyance channel could conceivably serve to reduce infiltration when flows are being passed through the reservoir during the irrigation season. If alternate storage is constructed downstream, such a conveyance could potentially provide benefits in the non-irrigation season as well. If the channel were constructed relatively high in the reservoir, it is conceivable that sources of seepage that may exist in the western end of the reservoir could be partially avoided even when some storage is in the reservoir (i.e. in the eastern portion of the reservoir). If feasible, this concept could extend potential loss mitigation to periods when the reservoir is partially full as well. Construction of such a canal could, however, be highly problematic in the last 600 feet prior to the dam outlet due to the presence of soft sediments and very steep, rocky slopes. An alternative would be to end the canal prematurely. It is not possible to analyze such a conveyance without significant further information on the nature of the sediments as well as the cause of the apparent losses. The following plan is a base framework for investigating the potential for mitigation of Base Losses.

- ***Revise the Water Balance Model.*** The Water Balance Model could theoretically be revised to estimate a more accurate picture of where Base Losses are originating as follows. A more accurate runoff estimate for the ungaged basin above Anchor Reservoir would be made by detailed analysis of the hydrology of the basin. Such estimates would be compared to runoff of nearby gaged basins with what are judged to be similar key hydrologic characteristics and adjustments made to the runoff estimates if/as appropriate. Additionally, the "signature" of unmodeled runoff from the ungaged basin is believed to appear as unaccounted gains in storage in the current model. Unaccounted storage gains could be separated from the model and used to further refine the ungaged basin runoff estimate. Finally, the refined runoff estimate and a new source of loss representing potential stream losses in the ungaged basin would be incorporated into the model. The model would then be used to estimate the amount of losses attributable to Cause #1 and potentially provide more insight as to the quantity attributable to Cause #3. It is important to understand, as noted previously, that unavoidable uncertainties and errors in the approach described above could mask or otherwise make unreliable the conclusions of the refined model. Unfortunately, it is not possible to accurately assess the risk of this outcome on the front end of the effort.
- **Gage the stream in the interior of Anchor Reservoir.** Providing a temporary stream gage or gages for the collected inflow at an interior point(s) within Anchor Reservoir could help provide additional insight and ideally confirmation of the results of the analyses described above and could help estimate losses associated with Cause #3 as well as the potential benefit of an interior canal. However, establishing a sufficiently accurate and stable temporary gage in the accumulated soft sediments comprising the channel banks and bed in this reach may be problematic.
- **Collect geotechnical information and survey on the reservoir sediments.** Conducting a geotechnical exploration program to collect information on the reservoir bottom sediments would provide information as to the feasibility of constructing an interior canal as well as provide information on the permeability and thickness of reservoir sediments which could help refine the revised Water Balance Model. Such a program would require significant and costly effort to access areas for test pit, drill hole or in situ testing (such as cone penetrometer and water loss/permeability). In the end, even if an adequate model of the

geotechnical and hydrogeologic characteristics of the reservoir sediments was gained, it is important to understand that this program would not provide information on underlying existing or potential future sources of concentrated leakage. It would only be intended to support alignment and design of a lined channel to carry low flows through the reservoir area.

Based on the above discussion, and as noted previously, it appears that the high cost and risk of incomplete or potentially misleading conclusions from a thorough study of Base Losses and design of mitigation or repair measures outweigh the potential water savings. As a result, such a study is not recommended.

3.3 Subtask 3B – Operations Analysis

3.3.1.1 Without Alternate Storage Downstream

The initial storage required to yield a given supplemental irrigation water release after various durations of storage is given in Table 3.2-2 for projected 2007 loss rates. These loss rates reflect the best estimate of the Water Balance Model described in Section 3.2.2 and are supported by various other investigations described in Sections 3.1 and 3.2. The tabulated results indicate that loss rates, even after what appear to be substantial reductions in loss rates over the history of Anchor Reservoir, are still very high and storage in Anchor Reservoir is relatively unreliable. It is also evident from the table and Water Balance Model that there is no threshold storage below which losses decrease significantly. The model indicates that significant losses occur even when flows are being “routed through” Anchor Reservoir (with no significant or intentional storage held).

The basic conclusion from this table and the Water Balance Model is that the duration and quantity of storage should both be minimized as much as possible if the objective is to minimize losses, because losses increase with both duration and amount of storage. However, storage, even with the resultant losses, is beneficial in meeting at least some of the irrigation shortages below Anchor Reservoir. It is believed that current operating procedures effectively achieve this goal to the degree allowable under the reservoir operation restrictions described in Section 3.1.7. Without construction of alternate storage downstream, there appear to be no improvements to be made to existing operations.

3.3.1.2 With Alternate Storage Downstream

The surface water availability and shortages analyses described later in Section 5.1 indicate that there are shortages in the Upper Area on South Fork Owl Creek above the confluence with North Fork Owl Creek. Anchor Reservoir is ideally situated to address these localized shortages and could be operated in tandem with alternate new storage downstream. The recommended alternate storage sites described in Section 5.3.7 located on North Fork Owl Creek are ideally situated to serve identified shortages in the Middle Area below the confluence.

This means that Anchor Reservoir could be operated in a reduced storage level (relative to current operations) to serve only needs on the South Fork Owl Creek above the confluence. If Anchor Reservoir were operated in a reduced storage capacity, losses in the reservoir would be reduced per Table 3.2-2 and more water would be available for storage at an alternate site. The analyses of Section 5.2 and 5.3 indicate that depending on the storage capacity of the alternate site selected, transfer of some storage (without harming South Fork users served by Anchor) to the alternate site may be beneficial. Additional study to verify or refine this model is

recommended as described in Section 5.1.3. A more refined model may indicate the storage in Anchor could be further optimized to provide maximum benefit to water users. Such refined reservoir operations analyses would be appropriate during final design of alternate storage downstream.

It is not recommended, however, that operations be permanently altered (diminished) in Anchor any more than necessary for the following reasons. Given the unreliable nature of the storage (unpredictability of sudden loss rate increases), existing storage capacity should be operated as described in the previous section to provide maximum achievable benefit and reliability to users on the South Fork Owl Creek above the confluence.

Further study of alternate storage or final design of that storage should include an analysis that examines optimized operations that may provide benefits to the District such as during extended drought conditions.

4.0 Water Storage Site Evaluation and Analysis (Subtask 4A) – Anchor Reservoir

4.1 Feasibility of Raising the Elevation of Internal Isolation Dikes

The feasibility of providing additional storage in Anchor Reservoir by raising the internal isolation dikes was examined as part of evaluating alternative storage in the basin (Task 4). Providing additional storage in Anchor Reservoir is problematic for several reasons, including the following:

- ***No Carryover Storage.*** Existing losses in Anchor Reservoir predicted by the Water Balance Model described in Section 3.2.2 are high enough that carryover storage from year to year cannot be attained. The losses that can be expected for various durations of storage are illustrated in Table 3.2-1. Carryover storage is essential to serve irrigation shortages in dry years. As discussed in Section 5.2, the alternate downstream storage reservoirs are sized to provide carryover storage to yield supplemental supply for up to two "dry" years back to back. Thus, providing additional storage in Anchor Reservoir at current loss rates provides significantly less benefit than alternate storage downstream.
- ***Potential to increase loss rates.*** There is significant evidence that increasing storage in Anchor Reservoir can lead to sudden increases in loss rates by the development of new Sinkholes/Leak Zones. Such loss rate increases are unpredictable, which imparts more uncertainty into either short-term or long-term storage yield estimates. Irrigators relying on the additional storage in Anchor Reservoir in any given year cannot necessarily count on existing storage still being available weeks to months later when they need it. This problem is exacerbated significantly at higher pool levels. While evidence indicates that loss rates have decreased in Anchor Reservoir over time, a sudden, large, permanent increase in loss rate cannot be ruled out, which compounds the risk associated with increasing storage in Anchor Reservoir.
- ***Water lost from Anchor Reservoir does not reemerge within the District.*** Available evidence from previous studies discussed in Section 3.0 indicates that losses from Anchor Reservoir do not reemerge downstream within the boundaries of the District. Thus, the additional water lost by increasing storage in Anchor Reservoir is water that is removed from basin supply and cannot be stored as carryover.

Notwithstanding the above considerations, both 10-foot and 20-foot potential raises to the existing internal isolation dikes were evaluated. For purposes of this analysis, it was assumed that the dikes could be raised using an upstream raise technique and locally available borrow. It was further assumed that no special filtering zones are required to prevent internal piping within the dikes and that the existing dikes and their foundations are safe from piping and dissolution at the higher reservoir heads. A geologic and geotechnical subsurface sampling and testing program would be needed to confirm these potentially optimistic assumptions. The estimated costs to construct the raises and their additional storage provided (assuming no losses) are as follows:

- 10-foot raise:
  - 2,700 ac-ft additional storage capacity (no losses)
  - Estimated total cost = $2,300,000
- 20-foot raise:
  - 6,000 ac-ft additional storage capacity (no losses)
  - Estimated total cost = $5,500,000

The capacities listed above must be adjusted to account for losses in any analyses to compute the "effective" additional storage provided. A proper simulation would involve a full reservoir routing analysis which was beyond the scope of this study. An approximation of the effective storage can be computed via Table 3.2-2. Assuming enough runoff is present in a given year to fill the reservoir to the maximum capacity of the isolation dikes, the following table provides the estimated effective capacities based on the duration of storage by comparing the storage gained relative to the existing condition after losses:

Storage Duration	10-foot Raise	20-foot Raise
1 month	1800 ac-ft	3600 ac-ft
2 months	1000 ac-ft	2200 ac-ft
3 months	600 ac-ft	1200 ac-ft

4.2 Economic Cost Benefit Analyses

Due to the unquantifiable risk associated with potential sudden increases in storage losses and other unknown risks associated with the behavior that Anchor Reservoir may exhibit if storage is raised, the risks associated with increasing storage in Anchor Reservoir cannot be estimated reliably. Techniques that may provide a potentially reliable estimate of risk are discussed in Section 3.2.7, but the expense associated with these analyses cannot be justified by the results of this study. In addition, costs associated with the dike raises as discussed above are potentially optimistic due to geotechnical and geologic unknowns. Because of all of these factors developing a reliable estimate of cost to benefit is not feasible.

An evaluation of the Anchor Reservoir inflows as well as modeled surface water availability and shortages downstream, however, indicates that there is little apparent benefit to be obtained from increasing storage in Anchor Reservoir, with the losses experienced during storage in the reservoir. Figure 4.2-1 illustrates the maximum historic pool elevation attained in Anchor Reservoir versus the total annual inflow for a given year (for years with full annual records). This figure indicates that based on past operating conditions, Anchor Reservoir does not appear to reach levels that would fully utilize the capacity provided by the 10-foot raise, except in wet normal or wet years. The 20-foot raise appears to not be fully utilized except in what are considered to be wet years. Due to limited supply (i.e., natural runoff) Anchor Reservoir does not appear to be capable of utilizing this storage to provide benefit during dry years, even at the significantly reduced loss rates post-1980. The Wind/Bighorn River Basin model discussed later in Section 5.1 indicated the following shortages in the combined Upper and Middle Areas served by Anchor Reservoir:

- Normal year: 5,400 ac-ft
- Dry Year: 13,600 ac-ft

Thus, providing additional storage in Anchor Reservoir would not satisfy the substantial dry-year shortages. At best, it is estimated that the additional storage in Anchor Reservoir would help satisfy only a small fraction of the normal year shortage, because these shortages are more apt to occur in drier normal years than wetter ones. Given that the additional storage can only be filled in wetter normal years, it's conceivable that none of the normal year shortages would be benefited by the additional storage. The basin model described in Section 5.1 is not detailed enough to quantify the actual benefit provide by additional storage (if any). The more sophisticated and detailed basin-specific modeling recommended in Section 5.1.3 is required to estimate the benefit. It is highly likely that the 20-foot raise would show no benefit at all.

The following table shows the cost of raises on a cost per ac-ft basis based on the total cost and effective storage discussed in Section 4.1 for various assumed durations:

Storage Duration	10-foot Raise	20-foot Raise
1 month	$1,300 / ac-ft	$1,500 / ac-ft
2 months	$2,300 / ac-ft	$2,500 / ac-ft
3 months	$3,800 / ac-ft	$4,600 / ac-ft

Based on the cost per ac-ft of storage for alternate sites downstream as discussed later in Section 5.0, additional storage in Anchor is only competitive if the storage can be utilized to full benefit in less than two months (and assuming that loss rates do not increase substantially due to sudden development of one or more new Sinkholes/Leak Zones). Because serving some of the shortages discussed above may require even longer durations than this, the effective storage (and thus benefits) would likely be relatively low and the unit costs relatively high resulting in an unfavorable benefit to cost ratio.

Coupled with proper accounting for risks in the analyses, it is concluded that providing additional storage in Anchor Reservoir would fare very poorly in an economic comparison relative to alternate storage downstream (Section 5.7). This is primarily because alternate, reliable storage would provide benefits in both dry and normal years (via carryover storage) as well as late season benefits whereas additional Anchor Reservoir storage would only provide benefit in what could be a small fraction of normal years and no additional benefit at all in dry years. Also, additional storage in Anchor Reservoir provides little, if any, savings on a cost per ac-ft basis relative to storage downstream, and may cost significantly more, especially in the likely event that the shortages occur later in the irrigation season.

5.0 Water Storage Site Evaluation and Analysis (Subtask 4B) – Alternate Storage Sites

This section describes the evaluation of surface water availability and shortages (Section 5.1); the selection of a target reservoir storage capacity (Section 5.2) based on water availability and shortages; the evaluation of six reservoir sites from the Level I Study relative to their ability to provide the necessary capacity (Section 5.3); and the selection of preferred alternatives (Section 5.3.7) that represent the best of the six alternatives. Permitting considerations (Section 5.5); conceptual design (Section 5.4); estimated costs (Section 5.6) and economic analyses (Section 5.7) of the preferred sites are also addressed. Key characteristics of the study area, including major drainage basins, the six Level I sites, the locations of irrigated lands, and classes of land ownership are shown on Figure 5.0-1.

In regard to land ownership, it should be noted that the available land/property ownership datasets exhibit some ambiguity in the vicinity of the alternative storage sites discussed later in this Section 5.0. There were two ownership datasets acquired for this project. One was obtained from the Wyoming Geographic Information Science Center (WyGISC) website and was completed as a part of the Wyoming Gap Analysis in 1994. The second was obtained directly from the Hot Springs County Assessors office and was updated in 2005. Neither dataset designated ownership in the area surrounding Alternative Storage Site 1 as shown on Figure 5.3-1. The 7.5-minute USGS topographic map labels this area as a part of the Wind River Indian Reservation, but no other source could be found that verified this designation. In addition, the WyGISC dataset showed all land south of South Fork Owl Creek to be included in the Reservation. The Hot Springs dataset, however, designated some of this land immediately south of the creek to be privately owned as noted on Figure 5.3-1. Furthermore, the Hot Springs dataset did not include a designation of Reservation lands, but rather identified what are believed to be Reservation lands as BLM land. The resolution of these uncertainties and ambiguities is beyond the scope of this study.

5.1 Surface Water Availability and Shortages

The primary quantifiable need for storage in the Owl Creek watershed is to provide supplemental irrigation supply to address to the degree possible existing shortages known to be experienced during normal and dry years by the Owl Creek Irrigation District in the Upper and Middle Areas. Once shortages are quantified, a determination must be made of how much, if any, physically and legally available flows there are within the basin that can be stored at a given reservoir/storage location for beneficial use in addressing the identified irrigation shortages.

5.1.1 Overview

The evaluation of flows available for potential storage projects and of irrigation shortages within the Owl Creek watershed was based upon results of the Wyoming Water Development Commission (WWDC) basin planning model developed for the Wind/Bighorn River watershed (BRS, Inc., 2003). This section summarizes key aspects of that effort. Much of the discussion of the model, assumptions inherent to it, and its limitations was extracted from previous reports and modified with information pertinent to this study. It is included herein to provide the background necessary to interpret model results.

Because the basin planning model was originally developed to address overall conditions in the entire Wind/Bighorn basin, it is only able to provide an approximate estimate of the shortages and available, storable flows in individual subbasins such as Owl Creek. That said, the basin plan model currently represents the best available tool to assess availability and shortages within the scope of this study. There are reasons to believe as discussed below, however, that portions of the available model may be over- or under-predicting certain available flows and shortages utilized for sizing storage in this study. Recommendations for future studies to correct any deficiencies of the current Owl Creek model results are provided in Section 5.1.3, should a storage project in the Owl Creek basin advance to further consideration.

5.1.2 Wind/Bighorn River Basin Model

The Wind/Bighorn River Basin Model is a water accounting spreadsheet that incorporates multiple diversions, gaging stations, and other water resources data within the Bighorn River Basin. One of the primary purposes of the model is to provide a planning tool for Bighorn River water users and the State of Wyoming for use in determining those river reaches in which flows may be available to Wyoming water users for future development.

For the purposes of this study, the spreadsheet model developed during completion of the Wind/Bighorn River Basin Planning Study was utilized without modification. The Wind/Bighorn River model consists of ten individual spreadsheet models, each representing a specific subbasin of the watershed. The Owl Creek watershed is tributary to the Bighorn River within the limits of the Upper Bighorn River subbasin model.

The individual spreadsheet models are linked to enable data generated in one model to be “passed along” to subsequent models. Furthermore, models were generated to reflect each of three hydrologic conditions: dry, normal, and wet year water supply. The spreadsheets each represent one calendar year of streamflow data, on a monthly time step. Each spreadsheet relies on a calibration model that reflects available historical data from the study period to estimate the hydrologic conditions. Streamflow, consumptive use, diversions, and irrigation return flows are the basic input data to the model. For all of these data, average values drawn from the dry, normal, or wet subset of the study period were computed for use in the spreadsheets. The model does not explicitly account for water rights, reservoir operations, compact allocations, or the management of the basin water supply based on these legal constraints. It is assumed that the historic discharge data reflect effects of any limitations that may have been placed upon water users by water rights or compact restrictions as well as reservoir operations. In the case of Anchor Reservoir, the

model utilizes the gage data above and below to approximate reservoir operations, including the substantial losses discussed in Section 3.0.

To mathematically represent the Bighorn River system subbasins, each basin was first divided into reaches based primarily upon the location of USGS gaging stations. Each reach was then sub-divided by identifying a series of individual nodes representing locations where diversions occur, basin imports are added, tributaries converge, or other significant water resource features are located. At each node, a water budget computation is completed to determine the amount of water that flows out of the node. Total flow into the node and diversions or other losses from the node are calculated. The difference between total inflow and diversions/losses is the amount of flow available to the next node downstream. Mass balance, or water budget calculations, are repeated for all nodes in a reach, with the outflow of the last node being the inflow to the beginning node in the next reach. Figure 5.1-1 displays a graphical representation of the water balance approach. For each reach, ungaged stream gains (e.g., ungaged tributaries, groundwater inflow, and return flows from unspecified diversions) and losses (e.g., seepage, evaporation, and unspecified diversions) are computed as the difference between average historical gage flows (or outflows) and model-predicted outflow from the reach. Stream gains are input at the top of a reach to be available for diversion throughout the reach and losses are subtracted at the bottom of each reach.

5.1.2.1 Model Limitations

There are several limitations to the model, which must be considered when reviewing the model and results generated by its use. These limitations and their implications with respect to a determination of water availability are discussed as follows:

- The spreadsheet model does not explicitly account for diversions from the river in accordance with Wyoming water law and is not operated on these legal principals. Simply stated, this means that the model cannot forego a diversion to an upstream junior water appropriator to satisfy a downstream senior water right. The basin planning model was originally developed under the assumption that if this situation occurred historically, the diversion data would reflect this occurrence and the junior appropriator would incur a shortage.
- The model does not incorporate reservoir operational rules for release or storage of water. For each simulation condition (normal-, dry- and wet-year conditions), reservoir releases do not deviate from historic releases. For example, releases from Boysen Reservoir remain consistent with historic patterns despite changes to reservoir inflow and storage. The implication of this limitation is that Boysen Reservoir behaves as a “buffer” between the upper and lower portions of the basin.
- The spreadsheet model does not contain logic to evaluate impacts upon the state's obligations under the Yellowstone River Compact (Compact). The Yellowstone River Compact between Montana, North Dakota and Wyoming was signed in 1950. The compact outlines allocations for several rivers in northern Wyoming, including the Bighorn River. On the Bighorn River, water is to be allocated 80 percent to Wyoming and 20 percent to Montana. Pre-1950 water rights are guaranteed. The Compact does not affect Native American rights to Yellowstone River water. Consequently, all estimates of available flows presented in this report do not include consideration of the Yellowstone River Compact.

- Comparison of historic data with full supply diversion estimates indicates that irrigators typically operate under supply-limited conditions. The model simulates diversion data related to a multitude of uses (irrigation, municipal, industrial, etc.). Given the magnitude of the irrigation diversions, however, special attention is devoted to the water requirements associated with irrigated lands. To fully understand this potential limitation, it is important to know that the spreadsheet model can be run in three different modes:
  - *Calibration (Historical):* This mode simulates the historical diversions where data are available. This mode is typically used for model calibration because historic diversion data are utilized.
  - *Full Supply for Existing Irrigated Lands:* This mode reflects full supply diversions, based on computed diversion requirements for existing irrigated lands (lands presently irrigated and mapped during the planning process).
  - *Full Supply for Existing Irrigated Lands and Futures Projects:* This mode simulates full supply, based on computed diversion requirements, for existing irrigated lands and Tribal futures projects. Within the Owl Creek portions of the model, there were no Tribal Futures projects; consequently, there were no local impacts of their potential implementation. The "Futures" version of the model was used, however, because Tribal Futures will impact availability of the Bighorn River downstream of the confluence with Owl Creek, which could ultimately affect availability of flows within the Owl Creek watershed.

5.1.2.2 Available Flows Analysis

To estimate how much of the physical supply is available for storage at any given model node, "available water" is defined in the model as that portion of the physically available streamflow that could be stored without causing a shortage to existing water users in any downstream river reach. In other words, the water available at any node was determined as the minimum of the physically available flow at that point or the minimum available flow at any node downstream in the system (including the Bighorn River).

Results of the model availability analyses indicate that there is flow available for storage without incurring a shortage in downstream reaches as summarized in Table 5.1-1 and on Figure 5.1-2 for modeled stream reaches within the study area. The total annual available flow for alternate storage sites located on North Fork Owl Creek (including a transbasin diversion of available, storable flows from South Fork Owl Creek) is estimated in the model as about 21,000 ac-ft for a normal (6 out of 10 years) condition and 4,400 ac-ft for a dry (2 out of 10 years) condition. The model results show that the large majority of available flows occur in April, May and June as would be expected in this hydrologic setting.

Evaluation of available gage data that were not utilized in the basin plan model and the experience of long-time residents and irrigators suggest that these model predictions of available flow may be high. However, the results are believed to be within order-of-magnitude of the actual values. Given the model estimates to date, we conclude that there is sufficient physically/legally available flow for storage during normal and wet years to mitigate at least a significant portion (if not all) of the existing irrigation shortages and thus justify more detailed follow-on study.

5.1.3 Recommendations for Future Hydrologic and Reservoir Operations Studies

It is recommended that consideration be given to development of a StateMod (or equivalent) hydrologic model for the Owl Creek watershed should this project advance so that native inflows and shortages can be evaluated in more detail, and appropriate exercise of water rights and reservoir operations can be included in the more detailed evaluations. It is recommended that natural ungaged headwaters inflows estimated in the current model using regression methods (based on the work of Lowham, 1988) be reevaluated and consideration be given to making these estimates by correlation with gaged streams with similar climatic, topographic and hydrologic properties. In particular, further evaluation of drainages in the central portion of the watershed (such as Red Creek and Mud Creek) should be performed as part of the StateMod modeling effort. It is recommended that the records of all gages in the Owl Creek and generally comparable basins in the region be thoroughly analyzed and all relevant data incorporated in the more detailed modeling to at least incrementally improve the calibration of the overall model.

It is also important to note that the results of the availability analyses for the Tribal Futures Condition Scenario from the existing Wind/Bighorn Basin Plan model used for this Level II study were identical to those of the Existing Condition Scenario. The reason for this reflects the method in which Boysen Reservoir releases are computed within the model. As previously discussed, the model does not adjust reservoir releases; releases are based on historically calibrated data regardless of the inflows to the reservoir. The modeling simulation does indicate that the reservoir incurs a reduction in storage with the Tribal Futures Condition compared to the Existing Condition Scenario. This issue would best be further addressed by utilizing a StateMod (or comparable) model approach should a storage project advance to further study.

5.2 Reservoir Capacity and Yield

Reservoir capacity was established at the alternative sites based on either physical site limitations or estimated needs and water available for storage. Estimated need is identified as “irrigation shortage” for either “normal years” (six years out of a ten year period) or “dry years” (two years out of a ten year period) and was determined from the Wind/Bighorn River Basin Model for the Owl Creek basin (model) as discussed in Section 5.1.2.

Irrigation reservoirs are ideally sized large enough so that they can provide carryover storage in most years, providing a reserve capacity to address needs (“shortage”) in the inevitable dry year periods. The reservoir “yield” is assumed to be the water that can be made available from storage for beneficial use (supplemental irrigation) during a given year. The “available flow”, as used in the model, is the annual flow that can be stored in the reservoir for subsequent use by irrigators without adversely impacting existing use below the site.

During a “normal year” the water stored from “available flow” should satisfy the “normal year shortage” and provide some additional water for replenishing reservoir capacity depleted during “dry years”. Under this scenario the normal year “yield” would be equal to the “normal year shortage” or need. It is assumed that at the end of the irrigation season of such a typical year that the reservoir is drawn down by an amount equal to the normal year shortage.

At the start of a "dry year" the reservoir would already be drawn down based on usage at the end of the prior year. During the spring runoff, it would then be partially replenished by the "available dry-year flow". The runoff water thus stored plus a portion of the storage at the beginning of the year would be used in an attempt to satisfy the "dry-year shortage". The analysis herein assumes that the initial carryover storage is split between two following dry years.

Accordingly, the maximum or "target" reservoir size was set to accommodate two consecutive "dry years" preceded by a "normal year" drawdown. In addition, an allowance was made for evaporation and seepage losses together with a minimum pool and low-flow releases.

Storage Capacity is then calculated as:

> Storage Capacity = [2 x (Dry Year Yield – Dry Year Available Flow)] + Normal Year Yield + (Losses + Minimum Pool + Low-Flow Releases)

Yield is then calculated as:

> Dry Year Yield = 0.5 x [Storage Capacity – Normal Yield - (Losses + Minimum Pool + Low-Flow Releases)] + Dry Year Available Flow

A storage capacity of 25,000 acre-feet was selected for Sites 2, 5, 17 and 18 to achieve the calculated storage capacity providing 10,200 acre feet dry-year yield for two consecutive years. This provides an allowance of 6670 acre-feet for a minimum pool, losses and low-flow releases over a two year period. Sites 1 and 8 are constrained by site topography and are incapable of providing the maximum storage capacity required to satisfy the full dry-year shortage as identified above. However, these two sites illustrate the potential benefits of a smaller reservoir and its impact on yield during normal and dry periods. Its impact on cost is also explored in Sections 5.6 and 5.7. A storage capacity of 9000 acre-feet was calculated for Site 1 and a capacity of 9500 acre-feet was calculated for Site 8. In the calculations for yield at Sites 1 and 8 losses, minimum pool, and low-flow releases were ignored to provide a best case scenario relative to addressing supplemental irrigation needs (most applicable to Site 1).

Table 5.2-1 lists, for each site, estimated annual "available" water and "irrigation shortage" for both dry and normal years. Assumed yield from each reservoir is also listed. For "normal year" irrigation, for all sites the yield was established as the shortage below the site since the model suggests that "normal year" shortage would be satisfied with the "available" flow that could be stored. As indicated above, for "dry year" supplemental irrigation, yield was based on the assumption that the reservoir was drawn down by the "shortage" (yield) from a "normal year" followed by a two-year back-to-back "dry-year" period. Yield was then calculated as the average storage supplied during that two-year period (which includes the "dry-year" flow "available" for storage).

The maximum storage capacity and yield analysis discussed above is based on the Wind/Bighorn basin model described in Section 5.1, which also described the model limitations and provided recommendations for further hydrologic and reservoir operations analyses to address these limitations. Because of these limitations, and for purposes of this study, Sites 1 and 8 were not automatically screened out on the basis that they are below the capacity calculated as necessary to fully satisfy dry-year

shortage. This is because it is our judgment, based on a review of the basin model, available gage data, and input from local irrigators and long-time residents, that there is a reasonable potential that a smaller reservoir may achieve or be adequately close to the capacity computed by the more sophisticated basin modeling recommended in Section 5.1.3.

5.3 Evaluation and Screening

5.3.1 Alternate Storage Sites from Level I Study

The Owl Creek Master Plan Level I Study identified six sites for advancement to Level II as alternative storage sites. These six sites are illustrated on Figure 5.3-1. Sites 2, 5, 8 and 18 are located onstream on North Fork Owl Creek above the confluence with South Fork Owl Creek. Site 1 is the only offstream site, located just north of the confluence. Each of the above sites could receive additional supply from South Fork Owl Creek by one of the diversion alternatives discussed in more detail in Section 6.0. Figure 5.3-1 shows the lower South Fork Owl Creek Diversion, which is the recommended diversion alternative from this study. Site 17 is located immediately above the confluence (spanning both creeks) and would require no diversions to receive supply from both South and North Fork Owl Creek.

5.3.2 Methodology

A process was developed to screen the selected alternatives from the Owl Creek Master Plan Level I into a manageable few for further analysis and ultimately to the most effective and feasible alternative(s) which best serves the needs of the project and upon which an application for follow-on study and eventual funding can be based. The process involves a multidisciplined approach that utilized three screening and analysis steps (Three Phase Approach) to achieve the goals of Task 4BI as described in Section 5.3.2.1. A key collection, evaluation, analysis and presentation tool that was utilized throughout the process is described in the following section.

5.3.2.1 <u>Storage Site Matrix</u>

The storage site matrix shown on Table 5.3-1 serves three purposes as follows:

- **Information compilation and summary.** A wide array of relevant information about the six alternative storage sites was compiled as part of this study, from information on environmental issues identified as part of the work documented in Section 5.3.5, geologic and geotechnical investigations documented in Section 5.3.4, from conceptual-level modeling of the sites as described later in this Section 5.4, and from cost analyses described in Section 5.6.
- **Comparative ranking.** A comparative ranking of selected parameters/conditions for each of the alternative sites is shown by the color-coding on Table 5.3-1. It is not necessarily intended that the comparative ranking be used to further screen these sites without additional interpretation and consideration as discussed in Section 5.3.7. Rather, the coding shows the relative degree to which each site may be impacted by various factors or how well the sites are suited to their individual role and is an important tool in support of decision-making during the screening process.
- **Design analyses.** Table 5.3-1 is derived from an Excel spreadsheet that links the compiled site information to a number of equations and algorithms that support the conceptual design and costing described in the remainder of this section and in Sections 5.4 and 5.6.

5.3.2.2 Three Phase Screening Approach

A three-step approach was utilized in screening the sites. This approach consists of three sequential phases as follows:

- **Phase I – Preliminary Investigations and Analysis.** This phase provides for the preliminary analysis and screening of all sites identified in the Owl Creek Master Plan Level I Reconnaissance Study. At this phase of analysis, basic information from multiple disciplines (geologic, hydrologic, environmental, etc.) is collected, analyzed and compiled in Table 5.3-1. Examples of the type of information collected or analyzed during this phase included available geologic mapping, refined infrastructure mitigation requirements and costs, refined quantity and capacity calculations, preliminary impacted landowner contact and coordination (including meetings with tribal authorities), and preliminary permitting requirements analyses, and geologic reconnaissance. The emphasis during this phase was to collect the readily available information that will lead to further differentiation of the preferred sites identified in the Level I report, to conduct the groundwork analyses that enable the following phases, and to subsequently utilize this information to further compare sites prior to collecting more detailed information in Phase II. The result of this phase was to select sites for further subsurface investigation as discussed in Section 5.3.4.
- **Phase II – Detailed Investigation and Analysis.** This phase provides for more detailed investigation of the selected sites from Phase I. The investigations and analyses at this level are necessary to further refine alternatives. Examples of the types of investigations and analyses gathered at this level include: targeted subsurface geological information; design configuration and optimization of selected sites; extents of borrow areas; optimal storage sizing of alternatives; and further refined cost estimates. The first objective of this phase was to collect the more detailed information, but only for the screened sites selected from Phase I. The second objective was to utilize this information for final screening of alternatives and as a basis for the final analyses of Phase III. The results of this Phase II evaluation are discussed in Section 5.3.7.
- **Phase III – Final Investigation and Analysis.** This phase includes the final investigation and analysis of the preferred alternative(s). The investigations and analyses at this level represent the last and most detailed necessary to fulfill the scope requirements and to prepare the conceptual design drawings and cost estimates to support decisions regarding the future direction of the project. Examples of the types of analyses conducted at this level are preparing conceptual design plans and sections along with final cost estimates, refinement of final permitting requirements, and economic analyses. The information generated during this phase is described in Sections 5.4 through 5.7.

5.3.3 Site Location and Size Relative to Need and Available Supply

All six of the alternate sites carried forward from the Level I study are located within an approximately seven mile reach extending along North Fork Owl Creek from about six miles above the confluence with South Fork Owl Creek to about a mile below the confluence (see Figure 5.3-1). Four of the sites (2, 5, 8 and 18) are located on North Fork Owl Creek. Site 17 is located immediately above the confluence of the North and South Forks to capture flows from both streams. Site 1 is an offstream site located just adjacent to the confluence to the north. As described further below, all of these sites can serve all of the irrigated lands in the Middle Area of the District

below the confluence of the North and South Forks of Owl Creek (Figure 5.3-2). It is assumed that the irrigated lands on the South Fork in the Upper Area would continue to be served by releases from Anchor Dam as discussed previously in Section 3.3.

Supply to all of the six Level I alternate sites is available from North Fork Owl Creek, and to the extent legally and physically available, from South Fork Owl Creek by transbasin diversion described in Section 6.0 and shown on Figure 5.3-1. Because Sites 1 and 8 are limited in capacity, the surface water analyses described in Section 5.1 indicate that these sites can be supplied via North Fork Owl Creek flows only, and a transbasin diversion is likely to not show any benefit by the recommended advanced modeling described in Section 5.1.3. Site 17 already captures South Fork Owl Creek and therefore does not require transbasin diversion either. The remaining Sites 2, 5 and 18 would utilize the diversion to collect excess available flows when they are legally and physically available. The diversion would be operated to divert flows only when they are not required downstream on South Fork Owl Creek. Because Anchor Reservoir is proposed to be operated in a diminished capacity (Section 3.3) if alternate storage is built, some greater flow than at present is expected to be available for South Fork irrigators and/or storage in an alternate North Fork reservoir because there will be less lost at Anchor.

Site 1 (Hamilton Dome). Site 1 is an offstream site located just north of the confluence of the North and South Forks of Owl Creek. Two relatively small dams would be required to provide storage up to a practical maximum of about 9,000 ac-ft (due to topographic limitations), which is less than the maximum target capacity discussed in Section 5.2. Reservoir supply would be diverted from North Fork Owl Creek via a canal alignment of approximately 1.9 miles across relatively gentle terrain. A 1.6 mile supply canal would route releases from storage to a point near the confluence as shown on Figure 5.3-1. The Level I Study considered the further possibility of diverting treated water extracted from the Hamilton Dome oil field to serve as a supplemental supply; however this water is currently being used for beneficial purposes in the Cottonwood Creek basin and is not expected to be available.

Site 2 (Onstream #3). This site is situated on North Fork Owl Creek immediately above (west) of the confluence with the South Fork. The site is estimated as capable of storing the maximum target capacity (Section 5.2) of approximately 25,000 ac-ft. The site requires a main dam across North Fork Owl Creek, and depending on the storage capacity selected, a smaller saddle dam to the north. Supply would be from North Fork Owl Creek, and as necessary and legally and physically available, via the diversion of flows from South Fork Owl Creek as described above. Gravity release from Site 2 could serve irrigated lands in the Middle Area as described previously.

Site 5 (Onstream #4). This site is located about a mile and a half upstream of Site 2. It is able to store the maximum target reservoir capacity of 25,000 acre-feet but is somewhat less efficient in terms of dam volume to reservoir capacity than Site 2. As for Site 2, this site would require a main dam and a small saddle dam to achieve the target storage capacity. As a stand-alone site, supply and demand characteristics for Site 5 are essentially the same as for Site 2.

Site 8 (Onstream #5). Site 8 is located on North Fork Owl Creek about six miles above the confluence with the South Fork approximately two miles northeast of the historic town of Embar. The site is judged capable of storing up to about 9,500 acre-

feet before wing dikes become necessary to prevent “spilling” into South Fork Owl Creek.

Because 9,500 ac-ft is near the maximum capacity that could be efficiently supplied by North Fork Flows alone, the extra cost associated with the wing dikes as well as incremental costs and seepage loss risks associated with the nearby karst geology were judged as excessive relative to the potential storage capacity gained by considering a larger reservoir at this site. Additional flows from South Fork Owl Creek could be diverted to increase capacity, but the invert elevation of the diversion supply canal to Site 8 also constrains capacity. For all of the above reasons, the practical capacity was limited to 9,500 ac-ft which provides reasonably good storage efficiency for the single dam required. Gravity releases can serve all but the irrigated lands on the South Fork as described previously. It is also possible to divert water from storage to South Fork Owl Creek users via a tunnel below the outlet through the ridge between the two streams. However, due to the constrained capacity of Site 8, and the potential for utilizing Anchor Reservoir as a source of supply to meet at least some of the supplemental South Fork Owl needs, there appears to be little or no benefit to this approach, so it was not carried forward as an option.

Site 8 could be considered together with Site 1 for a greater combined capacity of 18,500 ac-ft (closer to the targeted capacity of 25,000 ac-ft). This would require that water from South Fork Owl Creek be diverted into this reservoir across the low divide between the two streams so supplement North Fork natural flows which alone are not sufficient to supply both Sites 1 and 8. Because Site 8 is generally less favorable from several perspectives (per Table 5.3-1), it was judged that building a single larger dam (e.g., Site 2, 5, 17 or 18) was preferred compared to this particular combination of smaller dams, and this option was not carried forward.

Site 17 (Confluence). This site is located immediately upstream of the confluence of North and South Forks of Owl Creek. Site 17 would require a single long dam to capture flows from both streams. The maximum practical storage capacity at this site with reasonable dam to reservoir volume efficiency is well in excess of the maximum target capacity of 25,000 acre-feet (per Section 5.2). Site 17 is the only site of the six sites from Level I capable of capturing South and North Fork Owl Creek, Red Creek, and Dry Cottonwood Creek flows without diversion and could provide the greatest degree of downstream flood protection. However, as noted below there are a number of other important considerations for this site including loss of irrigated lands, significant wetlands impacts, and road and power line relocations, among others.

Site 18 (Onstream #6). This is an onstream site on North Fork Owl Creek located between Sites 5 and 8. This site is capable of providing the maximum target storage capacity of 25,000 ac-ft on its own, but it is the least efficient site in terms of storage efficiency. Site 18 could serve users in the Middle Area by gravity. Site 18 could conceivably utilize a tunnel (similar to Site 8, but with a much longer tunnel) to serve a portion of the irrigators on South Fork Owl Creek, however it is believed that Anchor Reservoir would still need to be retained to supply users not served by this tunnel. Because of the significant cost of the tunnel, combined with the relatively small irrigated area potentially served, it was judged that utilizing Anchor Reservoir (per Section 3.3) to service these users was the better option to carry forward in analyses.

5.3.4 Site Geologic and Geotechnical Conditions

Overview. A detailed geologic reconnaissance was conducted of five of the six alternate sites brought forward for further consideration from the Level I study. (Site 8 was not included in the reconnaissance as legal access could not be acquired.) This effort involved review of available aerial photography, ground observation, reconnaissance-level geologic mapping, and photographing of key and typical conditions and exposures. Table 5.3-2 provides a summary of key geologic information from the site reconnaissance for the five sites for which access was acquired. The results of the geologic mapping are shown on Figure 5.3-3.

A brief description of the geologic conditions expected at each of the six Level I alternate sites is presented below. These descriptions are based on the literature review and initial reconnaissance conducted during Level I and the more detailed geologic reconnaissance performed during Level II. Note that the geologic reconnaissance confirmed the conclusion from the Level I study that all of these alternate dam sites are best suited for earth dams, and that the relatively low strength sedimentary rocks present at all of the sites are not adequate to support concrete or RCC gravity dams without extraordinary and costly foundation improvement measures.

Seismotectonic Conditions. Seismotectonic conditions in the site area (i.e., the potential for strong ground shaking due to earthquakes) are equally applicable to all of the sites under consideration. In terms of screening of the six alternate sites, the primary consideration related to earthquake shaking is the presence of potentially susceptible saturated surficial deposits in the dam foundation(s) at a given site. The shallow alluvial and colluvial deposits anticipated at Site 1 do not appear to present a significant hazard due to loss of strength under earthquake shaking as they could (and likely would for other reasons including static stability and seepage control) be economically removed. Conditions at Sites 2, 5, 8 and 18 are judged likely sufficiently similar such that any required design and construction measures to mitigate potential earthquake effects, although variable in detail, would be of the same order of magnitude at each site. Due to the anticipated greater depths and extent of alluvial deposits at Site 17, it is anticipated that significantly greater foundation treatment (by over-excavation or other means) would likely be required as compared to the other sites.

A more detailed discussion of the basis for and levels of strong ground shaking that would need to be accommodated in the design of a dam at any of the preferred sites resulting from the screening in this section is provided later in Section 5.4.2.

Site 1. This site is primarily underlain by Cody Shale bedrock composed of gray to black marine shale likely with some bentonite lenses. The upper part of this formation locally contains thin calcareous sandstone beds. Reservoir seepage losses to the Cody Shale are expected to be minimal. The bedding of the Cody Shale at Site 1 dips generally to the north-northwest and thus into the left abutments to generally upstream at both dam sites. Although some jointing should be anticipated, the intense fracturing associated with the crest of the Owl Creek Anticline as at Site 17 as discussed below is not expected at Site 1. It appears that the locally steep folding associated with the anticline does not extend into this area even though it is present relatively close to the south. The weathered Cody Shale and overlying clayey colluvium/slopewash present in abundance at the site are suitable for use as core material for a zoned earth fill dam.

Terrace gravels of adequate quality appear to be available in sufficient quantity for use in filters and drains. The terrace gravels also appear to be available in sufficient quantities for use as shell material. Rock for use as riprap upstream slope protection is not present in the near proximity of Site 1. The more competent cliff-forming sandstones of the upper Mesaverde formation present on the higher northern slopes above Sites 5 and 18 (more than two miles from Site 1) may provide a source of moderate quality riprap. Note, however, that a quarry in these materials would likely be highly visible, difficult to reclaim, and relatively expensive to open and operate. As an alternative, the terrace deposits at Site 1 are available in adequate quantity and are judged capable of being processed to provide aggregate of adequate quality for use in soil cement or RCC slope protection. The on-site alluvial and colluvial deposits are likely relatively fine-grained given the nature and small size of the drainage area, and thus may not be suitable as an efficient source of aggregate. However, the alluvial deposits in the North Fork valley are relatively close by and may be of suitable quality for use as aggregate with appropriate processing (screening and washing). Even though likely relatively impervious, the alluvial and colluvial deposits should be removed beneath the dam footprint due to their assumed compressibility and low shear strength

Site 2. Cody Shale (refer to Site 1 above for description) underlies the main dam site at Site 2. The saddle dam alignment is also underlain by Cody Shale except that the left abutment may overlie the lower Mesaverde formation mapped in the area depending on reservoir size. The reservoir area is also mostly underlain by Cody Shale; a portion of the pool along the north rim may locally overlie Mesaverde formation again depending on the reservoir size (see discussion of Site 18 in Section 5.4.2 below for a more detailed description of the Mesaverde formation in the study area). Given these conditions, reservoir seepage at Site 2 is anticipated to be low to negligible. The crest of the Owl Creek Anticline located to the south of Site 2 projects toward the right abutment area of the main dam site, but no evidence of the anticline is present in outcrop exposures and bedrock-controlled topography visible in this area. Rather, the available exposures show a consistent bedding dip to the west-northwest, generally upstream and slightly into the left abutments at the main and saddle dam sites. Like Site 1, weathered shale and overlying fine-grained colluvium/slopewash are an abundant source of low permeability core material for a zoned earthfill dam.

The main dam site in the North Fork floodplain is underlain by alluvial deposits composed of sand, gravel and overbank silts estimated to be on the order of 15-20 or more feet thick. These deposits are likely sufficiently pervious and of low enough shear strength that will need to be removed at least under the central portion of the dam. The ridge south of the reservoir including the area of the right abutment of the main dam site and the hill between the main and saddle dam sites are capped with terrace gravels. These terrace deposits are similar to those at Site 1 and are adequate in quantity and judged suitable in quality for use as shell, filter and drain material. These materials and the alluvial deposits described above are also judged potentially suitable for use as aggregate for soil cement or RCC upslope erosion protection as an alternative to the potential use of Mesaverde sandstone as described for Site 1 above.

Site 5. Site 5 is underlain by the lower of two units of the Mesaverde formation present in the study area. This lower unit in the study area is typically comprised of a thinly interbedded sequence of relatively weak mudstone, sandstone and siltstone and

somewhat stronger thinly bedded sandstone. Reconnaissance mapping suggests that this unit in the vicinity of Site 5 may be transitional to the underlying Cody Shale which is present just downstream (to the east). These bedrock units are judged to be of moderate bulk permeability, especially near surface where bedding planes and joints are more open. As a result, reservoir seepage losses would be expected to be greater than at Sites 1 or 2, but not likely enough to pose a significant economic penalty for lost storage water. Bedding in this area dips gently (on the order of 5-7 degrees) to the northwest and thus generally upstream at the main dam site. These bedrock units should be suitable for shell material but may require special attention in design and construction to avoid zones of stronger, larger pieces/clasts of sandstone intermixed with zones of friable, sand to silt size material in the fill. Low permeability weathered bedrock and surficial deposits for use as core material are judged relatively scarce directly at Site 5, but could be borrowed within reasonable distance from sources just downstream.

The main dam site in the floodplain section is underlain by alluvial deposits composed of sand, gravel and overbank silts estimated to be on the order of 15-20 or more feet thick. These deposits are judged sufficiently permeable, compressible and low strength to require removal at least under the central portion of the main dam and the entire footprint of the saddle dam. A small patch of terrace gravels is present at the right abutment of the main dam; extensive terrace gravels cap the ridge to the south of the dam and reservoir area at some distance from the main dam site. These deposits appear suitable for use as filter and drain material, may be suitable as aggregate for soil cement or RCC upstream slope protection (as an alternative to the relatively nearby upper unit Mesaverde sandstone discussed under Site 1 above), and may serve as shell material, all for a zoned earthfill dam. The lower slope in the right abutment area of the main dam site is mantled by a relatively thick accumulation of colluvium and two moderate size landslides. The landslides appear to be up to several tens of feet deep and may be founded in weathered bedrock. All of the landslide debris and colluvium in the right abutment of the main dam site would have to be removed as part of foundation preparation. However, some if not all of this material may be able to be reused as embankment fill.

Site 8. Foundation conditions at this site are less known than at the other sites due to the fact that legal access could not be obtained for a site reconnaissance as noted above. The available small-scale published geologic mapping indicates that the site is underlain by Cody Shale. However, reconnaissance geologic mapping during this Level II study demonstrates that the regional mapping is in error along the North Fork from at least Site 18 to Site 5, and that Mesaverde formation underlies the area along the North Fork in this reach and not Cody Shale. Similar topographic expression and the character of the ground as observed in aerial photographs suggest that bedrock conditions at Site 8 are likely generally similar to Sites 5 and 18. As a result, reservoir seepage is expected to be as described for Site 5 above and Site 18 below. Bedding attitudes mapped to the north and southeast of Site 8 suggest moderate dips (probably in the range of 10-40 degrees) to the north-northeast, and thus with an unfavorable component in the downstream direction. Assuming that the site is underlain by lower Mesaverde formation, these sedimentary units should provide a source of adequate quality shell material (subject to the caveats for this unit noted above for Site 5). Core materials, however, are problematic at Site 8 (as described for the presumed similar Site 18 in more detail in Section 5.4.2). For this reason, either core material would have to be hauled in (presumably from up to five

miles away in the vicinity of Site 2) or a homogeneous embankment design would have to be considered.

A large landslide is mapped in the right abutment just upstream of the proposed dam centerline at Site 8. The depth of this slide is not known, but may be deep enough to involve bedrock given its areal extent. The depth of alluvium in the valley section at the dam site is estimated to be on the order of 10 to 20 feet, but could be more. These deposits would have to be excavated as described previously for Sites 2 and 5 above. It is likely that terrace sands and gravels are present capping the right abutment ridge, but quantities are probably limited in the immediate vicinity of the dam due to the narrowness of the ridge. More abundant terrace materials are present further downstream on the right (south) side ridge where it widens appreciably. These materials are assumed to be a suitable source of borrow for filters and drains, shell material, and possibly aggregate for soil cement or RCC for use as upstream slope protection. Consideration could be given to use of upper unit Mesaverde sandstone as described above for Site 5, subject to the same issues noted in the discussion of Site 1 above.

Site 17. Bedrock conditions at the left (northwest) abutment and northern portions of the reservoir area at Site 17 are generally similar to those described above for Site 2, with bedding dips on the order of 10-15 degrees to the northwest (generally upstream). The southeast abutment of Site 17 would be in rocks of the older Cretaceous-age Frontier formation (and possibly the Mowry and/or Thermopolis Shales) dipping downstream at about 12 degrees. Significant bedrock fracturing in these units is associated with the Owl Creek Anticline, a relatively steep-flanked structural fold that extends from the dam axis in the left abutment along the prominent ridge to the southeast. It is judged possible that through-going, high angle joints/fractures in the crest of this anticline may provide hydraulic connection from a reservoir at this site to the underlying Chugwater formation which is well known for karst features elsewhere in the basin and in other areas of Wyoming. This condition, and the generally higher bulk permeability anticipated for the Frontier formation are expected to result in higher to possibly excessive seepage losses without significant, costly foundation treatment (presumably shallow blanket and deeper vertical and angled curtain grouting).

Weathered Cody Shale and overlying fine-grained residual soils and colluvium are available in and beyond the left abutment for core material, and Frontier formation sandstones/shales are available in the right abutment area for use as shell material and possibly riprap (depending on the quantity and quality of more durable sandstone in the section in this area). Relatively thick masses (up to 20-30 feet) of slope debris (included toppled sandstone blocks) are present on the left abutment and will have to be removed. Alluvium overlies bedrock in the combined North and South Fork valleys beneath nearly the full length of the embankment footprint. It is estimated that the depth of alluvium may well exceed 25-30 feet at this convergence of two major tributary drainages. It is likely that the alluvium at Site 17 is an interlayered sequence of relatively coarse sand and gravel and finer-grained overbank silts and possibly clays. These pervious to locally soft and compressible materials would have to be removed at least in a core trench beneath the dam. The extent of removal under the shells would depend on the specific nature of the materials present and could be significant and costly at this site. Terrace sands and gravels are locally available for filters and drains, shell material, and potentially for soil cement or RCC aggregate as

described for Site 2 previously. If the local sandstones in the Frontier formation are not suitable for riprap, consideration would be given to use of Mesaverde sandstone or soil cement or RCC as upstream slope protection as described previously for the other sites.

Site 18. A more detailed investigation of Site 18, including test pit exploration, was conducted as described in Section 5.4.2. The geologic and geotechnical conditions encountered are discussed in some detail in that section and not repeated here.

Key Observations and Conclusions. Key observations from the reconnaissance effort relevant to comparing and screening the six Level I alternate sites on the basis of geologic/geotechnical conditions are summarized as follows:

- *Valley Stress Relief Fracturing*. Valley stress relief fracturing was observed in the units that comprise the abutments of several potential dam sites, and is assumed to be present at all of the dam sites whether conditions permitted observation or not. Stress relief fractures are known to occur on the walls and floors of valleys as a response to the erosion of material from the valley and resulting stress relief in the walls and floor of the valley. The fracturing typically occurs in walls parallel to the valley and if not properly addressed during dam design and construction, represents potential zones of weakness and/or preferential seepage/piping in the dam abutments. Fracturing in the valley floor typically occurs horizontally or along bedding planes and can be an issue for seepage, foundation pore pressures, and piping (internal erosion) depending on specific site conditions. High Savery Dam, for example, exhibited these stress relief features and required a substantial cutoff trench in one abutment and other special measures in the other abutment, as well as special foundation measures in the valley section to mitigate the known and potential problems posed by these features.
- *Borrow Materials*. In general, borrow materials for zoned earth dams appear to be more abundant and of more favorable quality at the lower three sites (Sites 1, 2 and 17) than at the upper three sites (Sites 5, 18 and 8). Based on the reconnaissance-level study, it appears that the availability of materials at the upper sites may dictate design and construction with less favorable materials and/or hauling of more favorable materials from the lower sites, with both options likely resulting in higher costs.
- *Dam and Reservoir Site Foundation Conditions*. In general, Sites 1 and 2 are believed to mainly overlie a bedrock unit (Cody Shale) expected to have the most favorable seepage characteristics of any of the bedrock units in the study area. Also, bedding orientation appears to be generally favorable at these sites in terms of potential resistance to sliding in the event local weaker beds/layers are present. Sites 5, 18 and 8 overlie formations which are expected to be generally more permeable. In addition, the Site 8 reservoir area is in close proximity to (and may overlie at relatively shallow depths) geologic units that are known to be karstic as described in the Level I report and there is a large landslide in the right abutment area. There are colluvium and landslide deposits at Site 5 that would have to be removed in the right abutment area. Site 17 exhibits unfavorable fracturing of brittle sedimentary rocks in the right abutment and is believed to be in relatively close structural proximity to Chugwater/Dinwoody formations red bed units that often are characterized by soluble gypsiferous minerals and formation of karst (solution voids and openings resulting in very high bulk

permeability). Site 17 also lies in a wide alluvial valley judged to be underlain by variable, relatively deep pervious, weak and compressible alluvial fill. These conditions could result in greater foundation preparation requirements than required for the other sites.

On the basis of geologic/geotechnical considerations, Sites 1 and 2 are strongly favored over the other four sites. Sites 5 and 18 are generally similar in terms of the anticipated unfavorable geologic/geotechnical issues of higher foundation/abutment seepage, greater depths/extents of likely required removal of surficial and weak bedrock materials, and less desirable locally available embankment fill. Site 17 is also judged less desirable than Sites 1 and 2 due to anticipated greater depths and variability of surficial deposits in the foundation that would need to be removed or otherwise addressed, and the observed significant bedrock fracturing in at least the right abutment. Site 8 is considered as the least favorable site due to the potential that it overlies karstic bedrock, there is a large landslide present in the right abutment, and the site is believed to otherwise share the less favorable materials characteristics of Sites 5 and 18.

5.3.5 Environmental Considerations

Introduction. Environmental analyses were conducted during the Level I study (Nelson Engineering, 2004) for five of the six sites recommended for advancement to Level II. Site 5 was not included in the Level I study, but is addressed herein. The Level I analyses addressed the following known or potential environmental issues:

- Threatened, Endangered, Candidate, and Sensitive (TEC&S) Species
- Wildlife and Fish
- Riparian Areas, Wetlands, and Aquatic Areas

The conclusions of the Level I analyses for five of the alternate sites (updated as appropriate based on additional information and evaluations during Level II) and of similar analyses conducted during Level II for Site 5 are summarized here and on Table 5.3-1 as part of the basis for final screening. Maps of WGFD-defined crucial big game habitat (see Figure 5.3-4), BLM-identified sage grouse leks with 2-mile buffer zones for nesting/brooding areas (see Figure 5.3-5), and NWI wetlands (see Figure 5.3-6) are included to illustrate the locations and scale of these key environmental conditions relative to the six alternate dam and reservoir sites. In the discussion below topics common to all of the six sites are addressed first, followed by discussion of each site and the other environmental considerations relevant to each site. Environmental permitting and mitigation considerations for the short-listed sites resulting from this screening process are addressed later in Section 5.5.

Threatened, Endangered, Candidate, and Sensitive Species. All six of the Level I alternate sites are judged generally similar in terms of the potential for impacts to TEC&S species. In the absence of detailed, site specific studies beyond the scope of the Level I and II studies, it is not possible to fully assess the potential for impacts to the TEC&S species identified and discussed in the Level I report. However, a preliminary assessment arrives at the following conclusions:

- *Bald Eagles* – Significant impacts judged unlikely; absence of known nests or winter roost sites presumably due to lack of an adequate fishery to sustain bald eagle population within or near the alternate reservoir sites.

- *Black-footed Ferrets* – Significant impacts judged unlikely; absence of known occurrences of black-footed ferrets within the project area; potential for occurrence requires further evaluation at all sites except Sites 8 and 18 where prairie dog complexes (required food source) have been observed.
- *Grizzly Bears* – Significant impacts judged unlikely; absence of essential habitat for grizzly bears (preferred food and den sites).
- *Gray Wolves* – Significant impacts judged unlikely; absence of know occurrences of gray wolves in the project area; some potential for indirect impacts to a pack whose range is south of the project area if reservoir development significantly changes mule deer distribution.
- *Canada Lynx* – Significant impacts judged unlikely; absence of known occurrences of Canada lynx resulting from lack of preferred food (snowshoe hares).
- *Ute Ladies' Tresses* – Significant impacts judged unlikely; riparian areas and wetlands in project area not expected to support this species.
- *Yellow-billed Cuckoo* – Significant impacts judged unlikely; this species not anticipated east of the Continental Divide.

Stream Classifications. The reaches of the North and South Forks of Owl Creek directly and indirectly affected by any of the six alternate sites are designated Class IV by the Wyoming and Game Fish Department (WGFD). Class IV streams are described as "*low production trout waters – fisheries frequently of local importance, but generally incapable of sustaining substantial fishing pressure*". The fisheries in these streams within the project area are comprised mainly of mountain suckers and long nose dace; trout are not known to occur in these waters. When contacted during the Level I study, WGFD reportedly requested that native, non-salmonid species be afforded protection regarding movement and migration by providing adequate instream flow from any of the alternate onstream reservoirs considered.

North Fork Owl Creek is classified 2C by the Wyoming Department of Environmental Quality (WDEQ). Class 2C waters are "*are those known to support or have the potential to support only nongame fish populations or spawning and nursery areas at least seasonally including their perennial tributaries and adjacent wetlands. Class 2C waters include all permanent and seasonal nongame fisheries and are considered "warm water". Uses designated on Class 2C waters include nongame fisheries, fish consumption, aquatic life other than fish, recreation, wildlife, industry, agriculture, and scenic value…*". South Fork Owl Creek and Owl Creek below the confluence are classified cold water 2AB. These waters "*are those known to support game fish populations or spawning and nursery areas at least seasonally and all their perennial tributaries and adjacent wetlands and where a game fishery and drinking water use is otherwise attainable. Class 2AB waters include all permanent and seasonal game fisheries and can be either "cold water" or "warm water" depending upon the predominance of cold water or warm water species present. All Class 2AB waters are designated as cold water game fisheries unless identified as a warm water game fishery by a "ww" notation in the "Wyoming Surface Water Classification List". Unless it is shown otherwise, these waters are presumed to have sufficient water quality and quantity to support drinking water supplies and are protected for that use. Class 2AB waters are also protected for nongame fisheries, fish consumption, aquatic life other than fish, recreation, wildlife,*

industry, agriculture and scenic value uses…". Any reservoir project will be required to maintain the protected uses of the stream reaches potentially impacted by operation of the reservoir.

Site 1. Known and potential environmental affects at Site 1 involve loss of antelope crucial winter range, non-crucial mule deer winter range, sage grouse nesting/brooding habitat (the site lies well within a sage grouse lek 2-mile buffer zone), migratory bird habitat, and possibly (but judged unlikely) black-footed ferret habitat. The amount and significance of these potential habitat losses would need to be addressed if Site 1 advances to the next level of study. Due to the absence of other than ephemeral runoff at this offstream site, no significant impacts to wetlands, riparian areas or aquatic resources are anticipated.

Site 2. Known and potential environmental affects at Site 2 include loss of mule deer and antelope crucial and non-crucial winter yearlong range, migratory bird habitat, possibly some sage grouse nesting/brooding habitat (the site lies at the edge but outside of a 2-mile sage grouse lek buffer zone), and possibly (but judged unlikely) some black-footed ferret habitat. Wetlands along the North Fork Owl Creek channel and overbank would be inundated; indirect impacts could also result downstream from diversion of flows from North Fork Owl Creek (and likely South Fork Owl Creek) to storage. Direct impacts to riparian habitat would occur due to dam construction and reservoir inundation; additional indirect impacts could occur as for wetlands. Aquatic resources will be directly impacted along the North Fork channel inundated by the reservoir; indirect impacts downstream could also occur without adequate reservoir releases.

Site 5. Known and potential environmental affects at Site 5 include loss of mule deer and some antelope crucial and non-crucial winter yearlong range, migratory bird habitat, and possibly (but judged unlikely) some black-footed ferret habitat. Wetlands along the North Fork Owl Creek channel and overbank would be inundated; indirect impacts downstream could also result from diversion of flows from North Fork Owl Creek (and likely South Fork Owl Creek) to storage. Direct impacts to riparian habitat would occur due to dam construction and reservoir inundation; additional indirect impacts could occur as for wetlands. Aquatic resources will be directly impacted along the North Fork channel inundated by the reservoir; indirect impacts downstream could also occur without adequate reservoir releases.

Site 8. Known and potential environmental affects at Site 8 include loss of mule deer and crucial and non-crucial winter yearlong range, non-crucial antelope winter yearlong range, migratory bird habitat, and some sage grouse nesting/brooding habitat (the site lies at the edge of but mostly within a 2-mile sage grouse lek buffer zone). Wetlands along the North Fork Owl Creek channel and overbank would be inundated; indirect impacts downstream could also result from diversion of flows from North Fork Owl Creek to storage. Direct impacts to riparian habitat would also occur due to dam construction and reservoir inundation; additional indirect impacts could occur as for wetlands. Aquatic resources will be directly impacted along the North Fork channel inundated by the reservoir; indirect impacts downstream could also occur without adequate reservoir releases.

Site 17. Known and potential environmental affects at Site 17 involve loss of mule deer crucial and non-crucial winter yearlong range, non-crucial (and a very small area of crucial) antelope winter yearlong range, migratory bird habitat, possibly some sage

grouse nesting/brooding habitat (the site lies just outside a 2-mile sage grouse lek buffer zone), and possibly (but judged unlikely) some black-footed ferret habitat. Wetlands along the North Fork and South Fork Owl Creek channels and overbanks would be inundated; indirect impacts downstream could also result from diversion of flows from both creeks to storage. Direct impacts to riparian habitat would occur due to dam construction and reservoir inundation; additional indirect impacts could occur as for wetlands. Aquatic resources will be directly impacted along the North and South Fork channels inundated by the reservoir; indirect impacts downstream could also occur without adequate reservoir releases.

Site 18. Known and potential environmental affects at Site 18 involve loss of mule deer crucial and non-crucial winter yearlong range, non-crucial antelope winter yearlong range, migratory bird habitat, and possibly some sage grouse nesting/brooding habitat (the site lies at the edge of but mostly within a 2-mile sage grouse lek buffer zone). Wetlands along the North Fork Owl Creek channel and overbank would be inundated; indirect downstream impacts could also result from diversion of flows from North Fork Owl Creek (and likely South Fork Owl Creek) to storage. Direct impacts to riparian habitat would occur due to dam construction and reservoir inundation; additional indirect impacts could occur as for wetlands. Aquatic resources will be directly impacted along the North Fork channel inundated by the reservoir; indirect impacts downstream would need to be mitigated by maintaining adequate reservoir releases.

Conclusions. Based on the analyses performed during the previous Level I study and this Level II study, a comparison and relative ranking of the six alternate dam and reservoir sites on the basis of environmental conditions was performed. The results of this effort are summarized in Table 5.3-1. As shown, the offstream Site 1 is judged to result in significantly less overall environmental consequences and Site 17 somewhat more than the other four sites (including especially much greater direct wetlands impacts). Of the remaining sites, Site 8 has the least apparent cumulative impact potential, followed in order by Site 18, Site 5 and Site 2. Note that these comparisons and rankings are at best only semi-quantitative; more detailed analyses beyond the scope of this study and final determination of the reservoir size and impacted areas would be necessary to refine these rankings. However, even if refined studies were conducted, the differences among Sites 2, 5, 8 and 18 would likely not be especially significant in terms of permitting and mitigation except in the case of wetlands at Site 2. As noted on Table 5.3-1, there are an estimated 33 acres of wetlands at Site 2 as compared to only about 15 acres at Site 18 to as little as 4 acres at Site 5.

5.3.6 Other Considerations

Additional considerations that are not addressed in the previous sections include: infrastructure, land ownership, cultural resources, irrigated land impacts, irrigation impacts, site access, construction and cost considerations not discussed above, and general reservoir operations considerations. To the degree any of these considerations further differentiate the various sites, they are discussed below. Other of the above considerations that do not differentiate the sites are included in costing analyses, but are generally not discussed here**.**

County Road Impacts. Site 17 generally blocks access to both the North Fork and South Fork Owl Creek valleys and would require the relocation of as much as 1.5 miles (or significantly more) of the paved county road that lies within its inundation

area. Relocation of the county road would be not only costly, but would require either relocation through the Wind River Indian Reservation to the south, or a significant traverse through public and private lands to the north with at least three bridge or culvert stream crossings required. A third potential relocation option is across the dam, whereby the road would scale the downstream slope of the dam or one of the abutments and cross the reservoir on the dam crest to the ridge between South and North Forks Owl Creek. All three of these relocation options appear to involve relatively steep grades, significant road curvatures, and/or significantly lengthened distances of travel, all of which would increase travel times for local residents and increase road maintenance requirements.

Sites 2, 5 and 18, due to their need for transbasin diversion from South Fork to North Fork Owl Creek (as discussed in Section 5.3.3), will require a culvert crossing of the county road for the diversion canal. This is a relatively minor consideration and is factored into costs of the diversion.

Access. Each of the sites will require, at a minimum, a maintained gravel road to access the dam control facilities (outlet control, instrumentation houses, etc.). Ideally, other facilities, such as spillways and dam crests are accessible by maintained gravel road to the degree feasible. In addition, access considerations include improving and/or providing utilities (e.g., if needed for remote outlet operation) to the facility. Site access for the six sites is a relatively minor consideration, having impacts primarily to costs and potentially some impacts to irrigated lands. Sites 1, 2 and 17 are considered as having the most favorable site access due to their close proximity to the existing county road. Sites 5 and 18 could be either accessed via the North Fork valley or over the ridge via the county road from the South Fork valley. Either access would require significantly more right of way acquisition and maintenance than Sites 1, 2, and 17. Site 8 would be accessed from the county road to the south via an over the ridge route and is considered to have moderate access considerations. Site 1, while having relatively close access to the dams via the county road, would also require an access route along the supply canal to the diversion headgate on North Fork Owl Creek.

Other Infrastructure. Other infrastructure concerns for the six sites are relatively minor. Site 17 is known to impact approximately 1.8 miles of the electrical power distribution line for the South Fork valley, but it is not expected that the cost of relocation of this relatively low voltage power line will significantly impact the site's overall ranking. A similar conclusion applies to relocation of local service telephone lines in the Site 17 inundation area.

Storage Losses. Potential losses due to seepage from the reservoir are discussed in Section 5.3.4. Evaporation losses are generally a minor concern, but do have non-negligible impacts to costs, requiring incrementally larger reservoirs to provide the same target storage capacity. Site 17, with its larger surface area, has the highest estimated loss rates. The rest of the reservoirs have lower estimated loss rates and by virtue of their size, Sites 1 and 8 have the lowest losses.

Operations. Operational considerations at the six sites are generally a minor factor in terms of screening; however, there are a few differences worth noting among the various sites. Site 1 is generally the least favorable from a reservoir operations standpoint, having both an inlet and outlet canal subject to maintenance, and is the only site that requires two outlets to fully drain the reservoir. Sites 2, 5 and 18, by

virtue of their reliance on the transbasin diversion (due only to their larger capacity) have slightly less favorable operating considerations than Sites 8 and 17.

Unknowns. The analyses discussed in this report, while appropriate for a Level II study, must necessarily be based on certain assumptions and contain estimates appropriately based on engineering judgment. While the design and cost analyses contained herein, including the 15 percent contingency factor incorporated into WWDC cost analyses are believed to include appropriate conservatism, unknowns at this stage of analyses can potentially and unavoidably alter the costs presented in Table 5.3-1 and/or alter expected performance. All other factors equal, those sites with unknowns most likely to impact costs or performance significantly are less favorable. While many of the potential unknowns have been identified in previous sections, the “bottom line” impacts of the various factors are integrated below.

The impact of geologic considerations on embankment design was summarized in detail in Section 5.3.4. The configuration of the dam embankments, however, can have “multiplier effects” on the geologic considerations depending on their configuration. All geologic factors equal, the sites with the shortest length of dam, smallest dam footprint, shallowest expected alluvium depth, and to some extent lowest dam height, smallest inundation area and smallest quantity of embankment material would generally be the most favorable, because unknowns generally have the least potential to impact revisions to costs in more detailed follow-on investigations.

Site 17 is considered the least preferred site in this regard due to its relatively large embankment volume, deep alluvium, and large inundation area, and unknowns associated with the right abutment geology. Site 8, with it’s potential association with the karst geologic units, and because it is the only site for which permission to access the site for reconnaissance was not granted, has the largest potential geologic unknowns and is the next least preferable site despite it’s otherwise favorable dam configuration. Site 18, due to its large embankment volume and highest dam height of the six sites, would generally be the next least preferred; however, given that Site 18 is the only site with subsurface investigation its unknowns are considered moderate. Also, Site 18 becomes significantly less of a concern and compares better with other sites at lower capacities. Sites 2 and 5, by virtue of having multiple dams with long lengths, are considered to have moderate unknowns and compare approximately equal to Site 18. Site 1 is the most preferred in terms of unknowns, due to its small dam sizes, shallow foundation cutoffs and favorable geology. Site 1 is benefited in this regard by its small size relative to the other sites, but even after adjusting for this advantage, Site 1 is still preferred.

Irrigated Lands. Site 17 has substantial impact to existing irrigated lands, inundating 447 acres (with potential impacts to additional acreage) and impacting three residences/ranches. Loss of irrigated land directly impacts the economics of the operations losing the irrigated acres and indirectly impacts the economy of the region overall as a result of the broader benefits of irrigated agriculture. Sites 2, 5 and 18, while not inundating irrigated lands, will result in minor loss of irrigated lands beneath the South to North Fork Owl Creek supply canal and possibly some operational impacts to adjacent irrigated lands. Site 8, while avoiding the need for the transbasin diversion (due to capacity constraints), would impact irrigated lands because the proposed emergency spillway for the site would be routed from the tail

(upstream) end of the reservoir over to the South Fork Owl Creek along an alignment through irrigated lands.

Cultural Resources. Cultural resources in this context include prehistoric and historic cultural resources, paleontological resources, and natural history resources. The only known documented historic cultural feature identified during this study was a portion of the Rawlins to Montana (locally Thermopolis to Meeteetse) stage road that passes through Site 18 over the low divide between the South Fork and North Fork of Owl Creek (U. S. Bureau of Land Management, 1996). Although a class I cultural resources survey was not included in the scope of the Level I study or this Level II study, it is judged very likely that other cultural resources will be found in the project area at potential project sites or along potential project alignments. This is due to the fossils found in various of the bedrock geologic units, prehistoric and historic use of the area by various Native American tribes (including the Northern Arapaho and Eastern Shoshone Tribes), and historic ranching (such as the Embar Ranch operations on South Fork Owl Creek near Site 18).

5.3.7 Recommended Alternatives

Based on the information and evaluations described in the previous sections, sites were selected for further analyses, including conceptual design and costing, permitting, and economic analyses. The site matrix tool (Table 5.3-1) described in Section 5.3.2 was populated fully and used in the selection process. Selection was based on finding those site options that best meet the needs of the District in a rational evaluation.

The analyses have identified two recommended alternatives that are best thought of as two "options" rather than two sites. The alternative selection process is generally most valuable if it does not carry forward a lot of alternatives that will not meet the goals of the screening process. Also, each alternative is tied to a specific plan of action for further study, should this project advance to further consideration. Finally, it is our opinion that each alternative should have a "backup" plan in case further studies uncover unexpected conditions. The two recommended alternatives are discussed below.

Alternative	Selected Site	Target Storage Capacity Range (ac-ft)	Backup Plan
1	1	9,000 or less	Site 2 or 18
2	2	9,001 – 25,000	Site 18

Alternative 1 (Site 1). Site 1 stands out as the most favorable dam and reservoir site overall, edging out Site 2, if the required storage capacity is less than 9,000 ac-ft. Site 2 is a close second at this smaller storage capacity. When other sites were analyzed at capacities similar to the 9,000 ac-ft maximum capacity of Site 1, however, the advantages of Site 1 become more significant across a wide range of multi-disciplinary factors. Specifically, the offstream nature of this site provides a significant benefit due to lower cost flood routing, due primarily to the lack of need for a primary spillway, and would result in significantly less environmental impacts than the other sites. Site 1 clearly does not match the target maximum storage

capacity of 25,000 ac-ft discussed in Section 5.2. However, as discussed in that section, it was recommended that consideration be given to retaining the option of a smaller capacity site pending follow-on hydrologic and reservoir operations studies recommended in Section 5.1.3. Thus, Site 1 is retained as the preferred site if follow-on studies indicate that a 9,000 ac-ft or less storage capacity will fulfill the needs of the District.

Alternative 2 (Site 2). Site 2 stands out as the most favorable site at capacities exceeding 9,000 ac-ft. Site 2, closely followed by Site 18, is the most favorable site for a narrow capacity range of 9,001 acre-feet to under 15,000 acre-feet. Site 2 is generally more efficient than Site 18 in this size range, has more favorable borrow materials potential, and is believed to have more favorable dam foundation and reservoir seepage conditions. Site 2 becomes the single favored site for capacity targets above 15,000 acre-feet, primarily since it retains it's relatively high efficiency across a broad capacity range, has favorable spillway opportunities, and retains its generally favorable known and anticipated geologic characteristics. In addition, Site 2 generally ranks near the top in most of the other multi-disciplinary categories shown in Table 5.3-1. It is recommended that if, based on the results of this report, the District and/or WWDC choose to pursue further studies, that discussions with tribal authorities regarding permission to conduct subsurface investigations be held as early as possible. Subsurface investigation as early as feasible in the process is recommended to confirm the results of this study for Site 2 which were necessarily based solely on surface reconnaissance.

Backup Sites. Site 18 is selected as a backup site for either of the alternatives above. Site 18 is generally the third-most preferred site overall, especially at capacities less than 15,000 ac-ft. Site 18 is further analyzed in the sections that follow, including development of conceptual designs and cost estimates, to provide flexibility to the District should the preferred Sites 1 or 2 prove unfavorable under further consideration. Finally, Site 2 is also considered a preferred backup site to Site 1 under Alternative 1.

5.4 Conceptual Designs of Selected Sites

5.4.1 Overview

A conceptual design was prepared for each of the dam and reservoir sites identified in Section 5.3.7 (Sites 1, 2 and 18) that form the basis of the selected alternatives. These conceptual designs are based on information developed throughout the project under various work tasks and significant prior experience in the planning, design and/or construction oversight of numerous dam and reservoir projects in Wyoming over the past 25 years.

Key factors influencing conceptual design (and estimated cost) include reservoir capacity and dam size, anticipated geological conditions, flood hydrology and the associated spillway sizing, and permitting and environmental considerations and mitigation. Reservoir capacity (which then dictated dam size) was established as described previously in Section 5.2. The next two of these factors are discussed in the following subsections. Permitting and environmental considerations are discussed in Section 5.5 below.

5.4.2 Anticipated Geologic Conditions

Introduction. Subsurface exploration was initially anticipated to support conceptual design and cost estimating of the preferred site(s) resulting from Level II screening of the six alternate sites carried forward from the Level I study. A substantial effort was made over a significant portion of the study schedule to acquire legal access to the preferred Sites 1, 2 and 18. Ultimately, access was only granted for subsurface exploration on state land at Site 18. Design and costing of Sites 1 and 2 are based on the conditions observed and inferred from the reconnaissance level studies and geologic mapping summarized above in Section 5.3.4 and the discussion regarding those conditions presented below. The site exploration conducted at Site 18 and the resulting geologic and geotechnical conditions relevant to conceptual design and costing are presented below. Finally, the basis for and anticipated levels of future strong ground shaking at the preferred alternative and backup sites are also discussed below.

Sites 1 and 2. Although subsurface investigations have not yet been performed at Site 1 or Site 2, the geologic reconnaissance information suggests that both of these sites are potentially more favorable for dam construction as compared to Site 18. Specifically, Sites 1 and 2 are located in an area underlain completely or predominantly by Cody Shale. The weathered Cody Shale, and overlying fine-grained residual soils and colluvium are a potential borrow source for low permeability soils for construction of a low permeability core for a zoned earthfill dam. In addition, the Cody Shale appears to occur at relatively shallow depths on both abutments at both dams at both sites. In other words, thick fine-grained granular soils mantling bedrock are not anticipated at either site. Although the depth of alluvium in the valley section at Site 2 is unknown, it is probably similar in depth to the alluvium at Site 18. The footprint area of the main dam at Site 2 is smaller than the dam footprint at Site 18. Thus, the total amount of alluvial excavation should be significantly less at Site 2 than at Site 18 for comparable storage capacity. Also, as discussed in more detail below, there are relatively thick eolian (wind placed) surficial deposits at Site 18 that will need to be excavated; these deposits are not present at Site 1 or Site 2. Therefore, it is judged likely that the amount of total foundation excavation required for site 2 will be considerably less than at Site 18. Stripping requirements under the shells at the offstream Site 1 dam sites are anticipated to be significantly less than at Site 2, but similar core trench excavation depths are conservatively assumed in the conceptual design for Site 1 described below. In addition, the results of the field mapping and literature review indicate that there is no evidence of an anticlinal structure transecting either the Site 1 or Site 2 dam sites; and the potential for the sites to be impacted by karstic type features (as occur at Anchor Dam) is very low. The conditions at Sites 1 and 2 also suggest more favorable (less costly) conditions for spillway and outlet construction than at Site 18.

Site 18. Access was granted to conduct subsurface investigation on the State land (Section 36) at Site 18. However, access was not granted to access the adjacent privately owned land (Section 1) located immediately south of Section 36. Because of this access constraint subsurface exploration of conditions in the plateau area (for potential borrow source materials) immediately south of the dam site was not possible. In addition, this access constraint also limited the locations that could be explored in the right (or south) abutment slope.

Test pit exploration was conducted at Site 18 between December 10 and 11, 2006. Descriptions of these test pits are presented in Table 5.4-1. Ten test pits (OCTP-1 through OCTP-10) were excavated using a trackhoe to expose, examine, and sample the shallow subsurface materials (<16 feet) within the site vicinity. The locations of the test pits are shown on Figure 5.4-1 (note that the conceptual alignment of the Site 18 dam was shifted slightly to the west subsequent to the test pit exploration program; the results of the exploration are judged still generally applicable to the revised site layout) and an interpretive geologic section through these exploratory features is shown on Figure 5.4-2. Test pits OCTP- 1 through OCTP-5 were located in the left abutment area; OCTP-6 and -7 were located in the valley bottom area; and OCTP-8 and -9 were located mid-slope in the right abutment area. Additional test pits in the right abutment were not excavated due to topographic and access constraints discussed above. The test pits ranged from approximately 5.5 to 15.5 feet deep, and up to 20 feet long. After logging and sampling, the test pits were backfilled with the spoil material, and covered with the top soil stockpiled during excavation. The surface was then smoothed by tracking over the area with the trackhoe.

The general site geologic and geotechnical conditions are inferred based on the results of the geologic reconnaissance described previously and the limited shallow subsurface exploration described herein. As summarized in Table 5.4-2, the soil and bedrock materials can be subdivided into four primary units that include: the Mesaverde formation, eolian deposits, and colluvial and alluvial soil deposits.

Bedrock associated with the lower sequence of the Mesaverde formation is presumed to underlie the entire site at variable depth. In the upper portion of the left abutment, and lower portion of the right abutment, the bedrock consists of a thinly interbedded sequence of mudstone, sandstone and siltstone that is low hardness, weak to friable, deeply weathered, and closely fractured. In the lower portion of the left abutment (in the slope immediately adjacent to the valley section), the bedrock consists of thinly bedded sandstone that is weak to moderately strong. These deposits are anticipated to exhibit moderate permeability due both to the primary permeability of the sandstone units and the secondary permeability of open bedding planes and joints and fractures common in these brittle rocks. As a result, reservoir seepage losses are expected to be higher than at Sites 1 or 2, although not necessarily excessive in terms of loss of economic value of the stored water.

The bedrock in the left abutment dips towards the northwest at approximately 6-7 degrees. Bedrock in the mid and upper portions of the right abutment is covered beneath surficial deposits and was not encountered at the locations and depths of the investigations performed to date.

The left abutment slope is locally covered by up to an estimated 20 feet of relatively homogeneous, massive, very fine-grained sand and silt that is interpreted to have been deposited by eolian (wind) processes. These deposits are locally interfingered with sandy gravels, and gravelly sands that are interpreted to be colluvium derived from weathering and erosion of the Mesaverde formation rocks. Thick colluvial soils also mantle the bedrock on the right abutment.

Alluvial deposits blanket the floor of the valley bottom. The alluvium appears to consist of silt and sandy silt with lenses of cobbles and gravel. The thickness of the alluvium is at least >10 feet. Based on the geomorphology of the valley it seems

likely that the thickness of the alluvium in the valley is on the order of 20 feet or more. The actual thickness of the alluvium and other surficial materials (that were thicker than the depth of the backhoe exploration) cannot be further defined without exploratory drilling.

The results of the test pit exploration indicate that there is up to an estimated 20 feet of fine-grained sands and silts covering a large portion of the left abutment. There is also an estimated >10 feet of granular colluvial sediments mantling bedrock in the lower middle slope of the right abutment. The silts and fine-grained sands in these various surficial deposits are highly susceptible to internal erosion and piping. For dam design consideration, it is reasonable to assume that all of these surficial materials and deeply weathered bedrock would need to be stripped (or over-excavated) from the dam foundation area prior to dam construction. Considering the length of the dam and estimated thickness and distribution of these materials, a relatively large amount of stripping/excavation would be required for dam construction.

The investigation found no suitable low permeability material available at the site for constructing a zoned earthfill type dam (with a low permeability core). The predominant materials available at the site and within the dam foundation area are sands, silts, and gravels. The design considerations and cost of constructing a dam using these materials is further evaluated in Sections 5.4.4 and 5.6.

If a service and/or emergency spillway are envisioned around the left (or north) abutment of the dam, special consideration of foundation conditions is warranted. The gentle slopes in this area are locally blanketed by fine-grained sands and silts that are likely up to 10-20 feet thick. The bedrock underlying these materials is also weak and friable. Because the surficial materials are highly erodible, it should be anticipated that these surficial materials (and possibly also the friable deeply weathered bedrock) may need to be removed or lined as part of spillway construction to minimize erosion during flood events. If removed, soils could potentially be used as borrow.

The results of this preliminary investigation suggest that Site 18 would be a challenging and potentially expensive site for dam construction primarily due to lack of locally available core zone borrow materials. As discussed previously for other sites founded on Mesaverde formation, it will be necessary to either haul core zone material from a borrow site in the vicinity of Site 2 (a distance of about 3 miles) or to design and construct a homogeneous dam at this site. The later option, although judged technically possible, carries with it the potential for more costly than normal processing, placing, moisture control and compacting of much of the main dam fill material and likely thicker than normal chimney and blanket filter/drain zones to ensure safe performance of the dam.

Seismotectonic Conditions. The Owl Creek basin is an area of minor historic seismicity. A total of only 11 earthquakes have been recorded in northern Fremont and Hot Springs Counties with magnitude greater than 3.0. The largest of these recorded earthquakes was a magnitude 4.7 event in 1967 with its epicenter located approximately 32 miles southwest of Thermopolis. No damage was reported anywhere in Fremont or Hot Springs County as a result of this earthquake. The only historic earthquake known to have occurred in or near the basin was an event in 1950

of Modified Mercalli intensity V that reportedly caused houses to shake and dishes to rattle at Hamilton Dome. (Case, et al., 2002a, 2002b)

There are no mapped active or potentially active faults in the Owl Creek basin. The closest mapped potentially active fault is the Stagner Creek fault located about 40 miles to the southeast of the project area immediately downstream of Boysen Dam. There is reportedly evidence of recurrent latest Quaternary activity (i.e., within the past approximately 15,000 years) based on detailed mapping and morphometric studies of this fault zone. The apparent slip rate of this fault is relatively low (<0.2 mm/yr). An estimate of the recurrence interval for earthquakes on the Stagner Creek fault is 8,000-22,000 years. The maximum credible earthquake for this fault has been estimated as M6.75. This size event has been estimated to be capable of producing a peak horizontal ground acceleration of approximately 0.06g at Hamilton Dome. The next closest potentially active structure is a portion of the Cedar Ridge/Dry Fork fault zone located over 80 miles to the west of the Stagner Creek fault. If it is conservatively assumed that this entire fault zone is active, a M7.1 earthquake could occur. Such an event would result in ground shaking of about 0.029g at Hamilton Dome. Lower accelerations than any of these estimates would be predicted in the Owl Creek basin. (Case, et al., 2002a, 2002b) These levels of ground shaking would not result in any significant damage to well designed and constructed reservoir storage dams.

Because fairly large earthquakes can and do occur where no known source structure (e.g., an active fault or fold) is known to be present, it is prudent to design critical structures such as high hazard dams assuming that a "floating or random" earthquake could occur near the facility. A previous study related to Boysen Dam recommended that a M6.25 event be assumed at a distance of 15 km to address this potential in the seismotectonic province that includes the Owl Creek basin. Such an event would result in peak horizontal accelerations on the order of 0.15g. (Case, et al., 2002a, 2002b) This level of shaking could result in some effects such as triggering metastable landslides or weakening loose, saturated silty to sandy soils. Again, well designed facilities on adequate foundations should not experience significant damage under this level of ground shaking.

The U.S. Geological Survey has developed an interactive program that predicts probabilistic levels of ground shaking in a given area based on the presence of the surrounding known or suspected active earthquake generating structures and the potential for a floating or random earthquake event. This program can be accessed at: http://earthquake.usgs.gov/research/hazmaps/products_data/2002/wus2002.php. For the purposes of this study, a map of the predicted peak horizontal ground accelerations within the Owl Creek basin with an annual probability of 2 percent of not being exceeded within any 50-year period was generated (see Figure 5.4.3). Maximum predicted accelerations in the area of the preferred alternative and backup sites are in the range of 0.16-0.18g. These levels of ground shaking require that any loose, saturated soils (especially sandy to silty gradations) be removed from the dam foundation to depths below which they cannot affect the stability of the foundation soils or overlying embankment. A properly designed and constructed earthfill dam on an adequate foundation will readily withstand these levels of ground shaking without significant deformation or risk of slope instability.

5.4.3 Flood Hydrology and Spillway Sizing

Dam Safety Classification and Inflow Design Flood Requirements. Requirements for dam safety, including inflow design flood (IDF) size, for any jurisdictional dam and reservoir project are promulgated and administered by the Safety of Dams Program, Surface Water Division of the Wyoming State Engineer's Office. The size of the IDF required for any new storage reservoir is determined by the hazard classification of the dam. There are four classifications (I through IV) based on the potential for loss of life and/or significant property damage in the event of dam failure. For the purposes of hazard classification, an assumption is made that the dam under review fails in a clear weather breach. The likely consequences of that failure are then evaluated to arrive at the dam's classification. The definitions for each of the four classes are as follows:

- A "Class I" dam is a dam for which loss of human life is expected in the event of the failure of the dam.
- A "Class II" dam is a dam for which significant damage is expected to occur, but no loss of human life is expected in the event of failure of the dam. Significant damage is defined as damage to structures where people generally live, work, or recreate, or public or private facilities exclusive of non-primary roads and picnic areas. Damage means rendering the structures uninhabitable or inoperable.
- A "Class III" dam is a dam for which loss of human life is not expected, and damage to structures and public facilities as defined for a "Class II" dam is not expected in the event of failure of the dam.
- A "Class IV" dam is a dam for which no loss of human life is expected, and for which damage will occur only to the dam owner's property in the event of failure of the dam.

Conceptual-Level Hazard Classification. Hazard classification requires determination of the potential for inundation of existing structures, recreational areas or primary roads. Completing such an evaluation requires dam break analysis and routing of flood waters and is beyond the scope of this Level II study. Accordingly, judgment has been used to "select" hazard classifications and provide an initial basis for sizing the IDF and thereby the conceptual spillway type and size. In general, sparsely populated areas with structures well out of the flood plain and/or significantly downstream from the reservoir (enabling dissipation of a flood wave), and/or small reservoirs which would provide minimal impacts on failure offer a reduced threat to property and human life.

Based on this concept and a review of topographic mapping downstream from each site, the three preferred alternative and backup dam sites were preliminarily assumed to be either Class I or Class II. This assumption recognizes that there is a paved road below the site(s) and numerous dwellings along Owl Creek.

Inflow Design Flood Determination. The required IDF for both Class I and II dams is the Probable Maximum Flood (PMF), unless an incremental damage/loss of life analysis (IDA) demonstrates that a lesser IDF is applicable. The IDF for a Class III dam is the 100-year flood, and no spillway is required for a Class IV dam.

At this level of study it is not practicable to perform a detailed and fully defensible IDA. Accordingly, the IDF was conservatively assumed to be the PMF for assumed Class I and II dams. Because the consequences of the potentially significant

conservatism inherent in this assumption can be substantial in terms of the cost of the spillway capacity required, it would be appropriate to evaluate the IDF further in a subsequent study if the preferred alternative is otherwise found to be favorable.

Probable Maximum Flood Estimation. At this level of study it is not warranted to perform the extensive and detailed analyses necessary to most accurately determine the characteristics of a PMF at each site. Instead, estimates of PMF peak flows were made from correlations of peak flow versus drainage area based on past studies of other sites in Wyoming. Initially, data on a total of 27 dam sites for which previous estimates of PMF peak flow had been made by others were compiled and evaluated. The source of the data for these previous PMF study results was the U.S. Bureau of Reclamation and other projects completed for WWDC. Correlations of peak flow versus drainage area were investigated using this complete data set and various subsets of the data to determine if a reasonably reliable correlation could be derived.

Reasonable correlations (i.e., with relatively high R^2 values) were found once obvious outliers were excluded from the data set, leaving 23 data points with areas ranging from 1.3 sq miles to nearly 20,000 sq miles. Because drainage areas for the majority of sites on this project were relatively small (i.e., less than 200 square miles) the analysis was limited to sites with similar areas, leaving 14 data points for which the following regression equation was determined:

$Q_{PMF} = 5755(A^{0..58})$, where Q is peak PMF discharge in cfs and A is drainage area in mi^2

Inflow Design Floods. The results of the IDF evaluations described above are summarized in Table 5.3-1. As shown on the table, PMF IDF peak flow ranges from 10,000 cfs at the offstream Site 1 to 85,000 cfs at Site 2. As noted previously, it will be necessary to evaluate IDF flows at a significantly greater level of detail (likely including an IDA study) should any of the storage projects advance to the next level of study.

5.4.4 Conceptual Dam and Appurtenances Design

The bases for establishing conceptual-level dam and reservoir size and spillway capacity for each of the alternatives have been described previously in this subsection. These parameters and others relevant to the conceptual design and cost estimating of the alternative projects are among the information summarized on Table 5.3-1. It is important to note, however, that these parameters potentially will have to be appropriately modified should further studies regarding needs, reservoir operations, and site-specific hydrologic and geologic/geotechnical conditions be undertaken.

In particular, storage capacity should be tied to the desired reservoir yield and operations, and spillway capacity should reflect appropriate project-, site- and/or region-specific analyses to account for factors such as IDF determination, reservoir routing and attenuation (i.e., flood storage), incremental downstream damage/loss of life potential, and practicality of downstream warning/evacuation. In addition, final slope configurations, size and extents of filter and drain layers, depths of core cutoffs, and other factors affecting the embankment design will require refinement based on extensive subsurface investigation and laboratory analyses of collected samples. Finally, note that the storage capacities reported for each site are assumed to include normal storage, carry-over storage, and a modest minimum pool to accommodate

sedimentation. More detailed evaluation of the need for and required capacity for each of these storage components should be carried out if any of these alternatives are advanced to the next level of study.

Earth Dam/Abutment Spillway Concept. The approach to the conceptual design of the dams, spillways and outlet works for Sites 1, 2 and 18 involved first establishing a “typical design” representative of the concept which was assumed to be a homogeneous or zoned earth dam with an abutment spillway. The typical design was then applied to each alternative site as appropriate and modified as necessary. Site 1, for example, due to it’s relatively small drainage basin, did not require an abutment service spillway, and Sites 1 and 2, due to their borrow characteristics are especially favorable for zoned earth dam design. The conceptual design for Site 18 also adopts a zoned configuration pending more detailed characterization of the available on-site borrow materials. It is possible that a homogeneous embankment design could be used, but this would require that the available borrow produces a fill of sufficiently low permeability without requiring excessive processing, sorting and/or placement measures.

For this approach to be appropriate site topography needs to be generally favorable to construction of an abutment or reservoir rim emergency spillway. This means that a “saddle” (a low swale in the reservoir rim relative to the flood pool elevation), a gradually sloping abutment, or a relatively low abutment as compared to the flood pool elevation must be present at the sites. Otherwise, the excavation necessary to make room for the spillway could result in a required volume of excavation which greatly exceeds the volume of fill that could be productively used in construction of an earth embankment. Site topography was reviewed to confirm that this is generally the case at the alternative sites. However, as any given site is being considered for various potential storage capacities, such conditions should be confirmed for the capacity ultimately selected. Otherwise, an alternative spillway concept would be required.

In the typical design, an emergency spillway would be constructed by excavation into the abutment of the dam, with the excavated material used (to the extent feasible) in construction of the earth dam in the valley section. The emergency spillway is assumed to incorporate a concrete cutoff at its crest, and to be cut in natural earth or rock in the entrance and tailrace channels. The service spillway is assumed to be a concrete chute-type spillway located in one of the abutments with an ogee crest and stilling basin. Service spillway capacity was assumed at 10 percent of the estimated PMF inflow and emergency spillway capacity was assumed at 90 percent of the estimated PMF inflow. No attempt to route the inflow flood was made at this level of study.

A low-level outlet works is assumed to be a cut-and-cover low-level pipe outlet with gate control. The outlet works is assumed to be located on one side or the other of the valley section, founded on rock or otherwise competent material. For the alternative at Site 1, two separate outlet works are assumed.

The site conditions, dam and hydraulic structures layouts, and maximum dam sections at Sites 1, 2 and 18 are illustrated on a set of figures for each site (Site 1 – Figures 5.4-4 through 5.4-7, Site 2 – Figures 5.4-8 through 5.4-11, and Site 18 – Figures 5.4-12 through 5.4-16). The first two or three figures in each set are photographs showing the general site topographic, vegetative, and geologic

conditions. The next figure in each set shows the reservoir area, dam footprint, spillway locations, and outlet works alignment in plan view on a base map derived from a USGS 7.5-minute topographic quadrangle map with 20-foot contour interval. The design maximum section used in estimating earthwork quantities for each site is shown in the last figure in each set.

The maximum dam sections utilize an assumed embankment configuration having a 3:1 upstream slope, 2.5:1 downstream slope, 30-foot crest width, and a variable depth of foundation stripping under the dam shells and cutoff excavation under the core. Freeboard, which allowed for the PMF discharge head, was added to the normal storage pool volume to set the dam crest elevation. The dam designs incorporate an impervious core zone/core trench founded on competent foundation for Sites 1, 2 and 18, internal filters and drains to control seepage and prevent internal erosion/piping, and upstream slope protection (either riprap or RCC/soil cement depending on material availability and cost). If needed, a grout curtain (or possibly a relief well system) would be installed to control seepage and pore pressures in the deeper foundation.

5.5 Permitting

The following discussion presents the results of a regulatory process analysis for a dam and reservoir project in the Owl Creek basin. The purpose of this analysis is to characterize the known and likely environmental processes, permits and related requirements and conditions associated with the alternative projects, including identification of environmental documentation, permits, agency clearances and approvals, and agency coordination steps that would be required for implementation of the proposed actions and alternatives.

It is likely that any of the preferred projects described in this plan will be subject to the National Environmental Policy Act (NEPA) and other federal environmental regulations administered by federal agencies such as the EPA, Bureau of Land Management (BLM), Army Corps of Engineers (COE), and/or the U.S. Fish and Wildlife Service (FWS). The Wyoming agencies which may have environmental, land use, and other regulatory approval requirements include, but are not necessarily limited to the Department of Environmental Quality (WDEQ), State Engineer's Office (WSEO), State Historic Preservation Officer (SHPO), Board of Land Commissioners through the State Lands and Investments Board (SLIB), and Game and Fish Department (WGFD).

NEPA compliance and documentation is presented in Section 5.5.1, permitting and approvals are discussed in Section 5.5.2. Known and potential environmental issues were previously discussed in Section 5.3.5. Cultural resources were addressed in Section 5.3.6. Mitigation is discussed in Section 5.5.3. These discussions are based upon various assumptions about the proposed actions and alternatives. These assumptions may change if/as project planning progresses from this Level II study. Ultimately, the applicability of the individual federal and Wyoming permits, clearances and approvals to the project(s) will depend upon the alternative(s) selected and their implications.

5.5.1 NEPA Compliance And Documentation

NEPA applies to any of the proposed actions for which the project site is located on federal land, federal funds may be used, and/or when formal federal agency actions are necessary for the project to move forward. One of the primary intentions of the

NEPA process is to avoid, minimize and mitigate adverse environmental consequences of federal actions. NEPA requires analysis and documentation of potential adverse and beneficial effects of a proposed action and alternatives and an open public involvement process.

For this project, it is likely that BLM would be the lead federal agency for implementation of the NEPA process for projects on lands under their administration. The COE would presumably be the lead federal agency otherwise where wetlands may be impacted. It is also possible that these agencies may work out a shared lead under a Memorandum of Understanding (MOU) if there are significant issues best led by both agencies for a given project. For projects on or impacting Reservation lands it is assumed that the Bureau of Indian Affairs (BIA) would assist the Tribes in their participation in the NEPA process, and the Bureau of Reclamation would likely serve as a technical advisor for both the NEPA and subsequent design and construction phases.

The following discussion characterizes the basic steps of the NEPA process applicable to a dam and reservoir storage project.

Prepare a Purpose and Need Statement for the Project. It is important to develop an accurate and defensible Purpose and Need statement for the project as one of the first steps in the NEPA process. The Purpose and Need statement provides an overall or basic purpose for the proposed action and presents details supporting various needs for the project. The Purpose and Need statement should provide enough information to develop and support a "reasonable range" of alternatives. More specifically, the Purpose and Need statement guides the alternative development and screening process. With the COE as the lead agency, the Purpose and Need would include a reference to finding the "least damaging practicable alternative." This reference relates to the Clean Water Act Section 404 requirements that are under the jurisdiction of the COE and is an important part of the NEPA process for a reservoir storage project. Additional details about the Section 404 process are provided in Section 5.5.2. The project sponsor, WWDC, other project participants, and the public should all be part of the process of defining the Purpose and Need statement.

Develop Project Alternatives and NEPA Documentation Determination. The NEPA process requires analysis of the No Action alternative and a reasonable range of alternatives that fully address the project's purpose and need. The reasonable range of alternatives may include one or more "build" alternatives, depending on the nature and extent of anticipated project impacts and level of NEPA documentation to be provided.

For new, expanded or reconstructed reservoir storage projects, key issues associated with alternative development will or may include:

- Loss of wetland and riparian habitat from direct inundation by a new, expanded or reconstructed reservoir;
- Potential impacts on threatened and endangered species;
- Potential impacts on fish and other aquatic species; and
- Potential impacts on other wildlife (e.g., migratory birds, sage grouse; big game).

Given these issues and risk management considerations, the project team anticipates that an EIS will likely be the appropriate NEPA documentation for reservoir storage projects. An EIS involves analysis of more than one build alternative and typically takes up to several years to complete. An Environmental Assessment (EA) may or may not involve analysis of more than one build alternative and can typically be completed in less than 18 months. The outcome of an EA is either a Finding of No Significant Impact (FONSI) or a recommendation to prepare an EIS. If an EA is prepared, there is a possibility that the outcome might be that an EIS is needed. This could occur as a result of "significant impact findings" or as a result of substantial public controversy over the project's effects. If this occurs at the end of the EA process, the EIS process would need to start from the beginning, wasting a considerable amount of time and money. At this time, it appears it would be prudent to assume that an EIS process would be applicable, while leaving the option open for an EA/FONSI, rather than to proceed with an EA and take the risk that an EIS will ultimately be needed. This decision should be reviewed during a subsequent study (should the project advance) when more detailed information is available on a preferred proposed action and its appropriate alternatives.

Conduct a Proactive Public Involvement Program. The NEPA process begins with public and agency outreach and related input focused on alternatives and potential impacts. Education about the project's purpose and need, project details and issues is provided and input is solicited in various ways. It is very important that the public have a clear understanding of the benefits and potential adverse impacts of the proposed action and alternatives. Public involvement is continuous throughout the project and can influence alternative development, alternative screening, issues addressed, mitigation measures, the level of NEPA documentation to be prepared (EA or EIS), and the selection of the preferred alternative.

Collect and Analyze Environmental Baseline Data. It is important to carefully identify environmental constraints and considerations early and incorporate them into alternative development efforts as a means of avoiding and minimizing potential impacts. Early field investigations and agency consultation and coordination efforts help to focus this effort and streamline subsequent analysis methods, schedule needs, and budget requirements. Creating "self-mitigating" alternatives is highly advantageous and fully consistent with the intent of NEPA.

Many NEPA analyses relate to compliance with various laws and regulations. Integrating the NEPA, National Historic Preservation Act, Endangered Species Act and other compliance processes will reduce overall permitting timeframes and costs, and streamline agency decision-making. These issues are discussed in Section 5.5.2.

Prepare the Draft and Final Environmental Impact Statement. The Draft EIS would be prepared in two versions. A Preliminary Draft EIS would be prepared for internal review. The Draft EIS would respond to comments on the Preliminary Draft EIS. The Draft EIS would be circulated for public review and would be the subject of a public hearing. The Final EIS would also be prepared in two versions. A Preliminary Final EIS would be prepared for internal review. The Final EIS would respond to comments on the Preliminary Final EIS. The Final EIS would be circulated for public review and would be the subject of a public hearing. A Record of Decision would be prepared to complete the NEPA process.

5.5.2 Permitting/Clearances/Approvals

In addition to the U.S. Army Corps of Engineers (COE) Section 404 Permit, there are numerous other permits and/or approvals required for new dam and reservoir construction. Presented below are the primary additional permits and/or approvals that would be required for any of the alternative projects under consideration.

Section 404 Permit. Like all water development projects, any dam and reservoir storage project in the Owl Creek basin (within the larger Bighorn River basin) will face environmental permitting issues. Typically the most significant environmental permit to be secured is a Section 404 Dredge and Fill permit from the COE, Omaha District. Even when impacts are anticipated to be modest, the process of obtaining a Section 404 permit for new storage projects may take several years from initiation of the NEPA process.

The primary guidance in embarking on the permitting process for a new dam and reservoir storage project is the development of a defensible Purpose and Need for the project. The NEPA process dictates that the least environmentally damaging practicable alternative that addresses the purpose and need be pursued. This is the alternative most likely to be successfully permitted.

Endangered Species Act (Section 7 Consultation). The lead agency would prepare a biological assessment to determine project effects on threatened and endangered plant and animal species listed or proposed for listing (candidate species) under the Endangered Species Act (16 U.S.C. § 1531 et seq.). U.S. Fish and Wildlife Service (FWS) would then issue an opinion on whether federal actions are likely to jeopardize the continued existence of a threatened or endangered species, or destroy or adversely modify critical habitat. FWS must approve the preparation of a biological assessment to comply with the Endangered Species Act in order to render its decision. If FWS determines that the preferred alternative would jeopardize the continued existence of a species, it may offer a reasonable and prudent alternative that would preclude jeopardy.

Fish and Wildlife Coordination Act. The Fish and Wildlife Coordination Act requires federal agencies involved in actions that will result in the control or structural modification of any natural stream or body of water for any purpose to take action to protect the fish and wildlife resources which may be affected by the action. It requires federal agencies or applicants to first consult with state and federal wildlife agencies to prevent, mitigate and compensate for project-caused losses of wildlife resources, as well as to enhance those resources.

Laws and Regulations Addressing Cultural Resources. Because federal approvals are likely involved with any of the identified alternatives, a consideration of effects on cultural resources must be undertaken (Section 106 consultation), as required under the following laws and regulations: the National Historic Preservation Act (NHPA) of 1966 (16 U.S.C. § 470 et seq.); the National Environmental Policy Act (NEPA) of 1969 (42 U.S.C., § 4321); the Archaeological Resources Protection Act (ARPA) of 1979 (16 U.S.C. § 470aa et seq.); the National Park Services (NPS) procedures concerning the National Register of Historic Places (NR) (36 CFR Part 60); the Advisory Council on Historic Preservation's Procedures for the Protection of Cultural Properties (36 CFR Part 800); the Treatment of Archaeological Properties of 1980: Determination of Eligibility for Inclusion in the NR (36 CFR 63); the Secretary of Interior's Standards and Guidelines for Archaeological Historical Preservation of

1983; Reservoir Salvage Act of 1960; and the l974 Amendment to the Reservoir Salvage Act of 1960. The State of Wyoming Historic Preservation Office (SHPO) coordinates with federal agencies in determining the significance of cultural resources potentially affected by ground disturbing activities.

In addition, consultation with relevant Native American groups concerning traditional cultural properties is required under the American Indian Religious Freedom Act of 1978 (AIRFA, P.L. 95-341.42 U.S.C. § 1996) and Section 4 of ARPA of 1979. Guidelines for evaluation of traditional cultural properties are contained in Bulletin 38 issued by the National Park Service. This would be an especially critical aspect of the permitting program should Site 2 be selected for further study as this site is located on Tribal (i.e., Arapaho Ranch) lands.

Wyoming Board of Land Commissioners. The Wyoming Board of Land Commissioners through the State Lands and Investments Board (SLIB) is responsible for regulating all activities on state lands, including granting of rights-of-way. Any facility, utility, road, railroad, ditch or reservoir to be constructed on state or school lands must have a right-of-way, as required in the "Rules and Regulations Governing the Issuance of Rights Of Way" (W.S. 36-20 and W.S. 36-202).

Wyoming State Engineer's Office Surface Water Storage Permit. The State Engineer's Office administers the water rights system of appropriation within the state. The Applicant must obtain the necessary water rights permits from the State of Wyoming for the diversion and storage of the State's surface water.

Wyoming State Engineer's Office Permit to Construct/Dam Safety Review. The Wyoming Dam Safety Law (W.S. 41-3) requires that any persons, public company, government entity or private company who proposes to construct a dam which is greater than 20 feet high or which will impound more than 50 acre-feet of water, or a diversion system which will carry more than 50 cubic feet of water per second, must obtain approval for construction of the dam or ditch from the Wyoming State Engineer's Office. The approval by the State Engineer's Office of a dam's construction is contingent upon the Office's review and approval of all dam plans and specifications, which must be prepared by a registered professional engineer licensed in Wyoming. Design, construction, and operation of jurisdictional dams must also comply with dam safety regulations promulgated pursuant to the Dam Safety Act. At present, these regulations are in final draft form and formal issuance is anticipated in the not distant future.

Wyoming Department of Environmental Quality – National Pollution Discharge Elimination System (NPDES) permit and Section 401 Certification. The federal Clean Water Act is administered in Wyoming by the Department of Environmental Quality (WDEQ), Water Quality Division (WQD) consistent with the Wyoming Environmental Quality Act. The Section 401 Certification is the State's approval to ensure that the activities authorized under Section 404 meet state water quality standards and do not degrade water quality. Any discharge of pollutants into the broadly defined "waters of the state" requires application to and permit issuance by WQD in accord with WQD's Rules and Regulations. This body of regulations sets forth classification of surface and groundwater uses and establishes water quality standards (Wyoming Water Quality Standards). The WQD administers the NPDES permit system including storm water permits and construction-related, short-term discharge permits.

Implementation of any of the action alternatives would require application for and compliance with the provisions of the statewide general NPDES Construction Storm Water Discharge Permit (WYR10-000). Construction activities associated with dam construction or enlargement often result in the requirement to temporarily discharge pumped water. These discharges are provided for in a general permit. Upon acceptance of the application by DEQ, the temporary discharge must be in compliance with the terms of the general permit and any stipulations applied as a result of the application's review.

EPA has oversight responsibility for federal Clean Water Act programs delegated to and administered by the State Water Quality Division. EPA also may intervene to resolve interstate disputes where discharges of pollutants in an upstream state may affect water quality in a downstream state.

Mining Permit. A Wyoming mining permit is not required for development of an aggregate and/or borrow material source solely for use in construction of one of the various reservoir alternatives and whose product is not for commercial sale. Commercial sources of aggregate, rock, or other mined materials are responsible for obtaining and maintaining all required permits and clearances for their operations.

Special Use Permits/Rights-of-Way/Easements. Special use permits, rights-of-way (ROW) or easements will be required wherever access across the lands of others (private, state or federal) is needed for construction and/or operation of the project facilities. These may be temporary (e.g., access to a temporary borrow area or quarry site to be closed and reclaimed; construction of a new haul road; etc.) or permanent (e.g., construction of a relocated county road alignment). Usually privately owned lands that will be rendered permanently unavailable (such as the dam and reservoir footprint of a storage project) would be purchased unless the owner desired (and the sponsoring entity agreed to) a permanent easement. Permanent use of BLM lands would most likely be administered under a grant with an appropriate term issued under their ROW process. An easement or ROW from Hot Springs County may also be required where the South Fork to North Fork diversion crosses the Owl Creek road. The specific requirements for rights-of-way, special use permits and easements vary widely and should be determined as part of the early stages of planning for a specific proposed project. This will help to avoid the potential for significant project delay, higher costs, or required changes in location/alignment or design during project development and implementation.

Other. In addition to the above, there may be other permits and clearances required for a given dam and reservoir project. These might include permits typically required to be provided by the construction contractor (e.g., air quality permit; trash/slash burning permit; etc.).

5.5.3 Mitigation

Based on the Level I and Level II environmental evaluations described previously and prior experience, mitigation could be required at any of the identified preferred alternative or backup dam and reservoir sites to address impacts to wetlands, riparian vegetation, stream channel habitat, cultural resources, fish and game resources, and possibly threatened or endangered species.

Detailed mitigation plans would need to be prepared and approved to replace any lost wetlands identified and quantified by formal wetlands delineation, and riparian

vegetation communities. However, given the relatively small acreages of wetlands at the alternative dam and reservoir sites (ranging from 0 to 33 acres), it is anticipated that mitigation of this resource will be possible by constructing additional wetlands nearby, ideally in the mainstem stream valley and/or in one or more close-by lower tributary reaches. Clearly mitigation of 33 acres at Site 2 will be more challenging than at Site 1 where no or negligible mitigation is envisioned or at Site 18 where only approximately 15 acres would require mitigation.

Mitigation of potential migratory bird and big game (mule deer and antelope) impacts would generally involve control of certain construction activities during sensitive time periods, and avoidance of direct disturbance of the subject species. Mitigation of potential sage grouse lek impacts will need to be given special consideration in cooperation with WGFD and the U.S. Fish and Wildlife Service (FWS). If any T&E species were encountered at a given site special studies would be required to determine if appropriate mitigation could be implemented. In general, any such impacts would be avoided to the greatest extent possible by relocation of site facilities. It may be necessary to conduct special studies of habitat conditions at some or any prairie dog colony at or in the near vicinity of a preferred site to assess the potential for black-footed ferrets.

Additional cultural and historic resource fieldwork would need to be completed to identify and document any such resources that would be inundated or otherwise impacted as a result of constructing any one (or more) of the alternative dam and reservoir projects. This would include, in turn, a class I (literature search) survey, a class II (reconnaissance inventory) survey, and if needed, a class III (intensive inventory) survey. Ultimately, a mitigation plan for cultural resources would be developed which would culminate in a Memorandum of Agreement (MOA) between the Wyoming SHPO and the lead federal agency with concurrence by the project sponsor(s), and affected Native American tribes. The agreement would require approval from the Advisory Council on Historic Preservation.

5.6 Cost Estimates of Selected Sites

A similar approach was used to prepare conceptual level estimates for each of the surface water storage alternative sites recommended in Section 5.3.7. This approach first involved estimation of construction costs for each of the major project components (i.e., dam, spillway and outlet works, canals), miscellaneous items (e.g., mobilization, clearing, fencing, etc.), and known interferences from existing infrastructure such as road relocation. Indirect costs (e.g., construction engineering and contingency) were then added as a percentage of the construction cost estimate.

The construction cost estimating approach included preparation of a conceptual layout of a potential dam and spillway at the subject alternative site on USGS 7.5-minute topographic mapping. See Section 5.4.4 for additional information on the layouts used in the conceptual designs on which cost estimates are based. Volumes of earthwork and storage capacity were then calculated at various dam heights (using multiple sections) to provide an approximate curve of dam height versus storage capacity and excavation/fill requirements. The overall dam volume was apportioned based on dam geometry (i.e., average dam height and length) to estimate various materials requirements such as riprap, core, shell, drain, etc.

The estimated construction cost for the service spillway was based on a formula used to estimate the approximate concrete volume considering dam height, head on the

spillway, slope of abutment, and spillway width. The derived formula was then checked against prior dam construction projects. The emergency spillway cost was based on construction of a concrete crest (cutoff wall) and open cut rock/earth spillway in one of the abutments. If estimated earthwork cut volumes for the spillway exceeded that required for construction of the dam embankment, an additional cost was added to cover hauling this material to a local waste fill assumed to be within the reservoir area or immediate vicinity of the dam and reservoir.

The outlet works estimate for new reservoirs was based on a formula which related outlet works size to storage volume (and associated dam safety drawdown requirements) and height of dam (which is related to conduit diameter and length). The generalized formula was then checked for reasonableness against costs from prior projects. Note that costs for Site 1 assume two separate outlet works due to the requirement for two dams and the presence of an intervening ridge.

Costs were also included on a case by case basis to provide an allowance for other items such as instrumentation, revegetation, site access, supply canal and/or wetlands mitigation, as appropriate.

Costs for the non-project component items (i.e., preparation of final design and specifications, construction engineering, contingency, legal fees, acquisition of access and rights of way, and environmental permitting and mitigation) were included as a percent of construction cost based on average percentages of these costs on prior project estimated and actual costs, or based on WWDC contract-designated percentages where appropriate.

The conceptual-level estimated project costs for each of the Level II preferred alternatives and backup are presented in Tables 5.6-1 through 5.6-3 for Site 1, Site 2 and Site 18, respectively. These estimates are in the contract-required WWDC format and are based on 2008 dollars.

Table 5.6-4 presents a summary of the total project cost for each alternative as well as the estimated cost per acre-foot of storage. Cost curves of total project cost versus reservoir capacity for each of the sites are included on Figure 5.6-1. These curves were derived from the model used to estimate costs at the capacity included in Tables 5.6-1 through 5.6-3 and are provided to illustrate how cost may vary with reservoir size should alternative sizes at any of the sites be considered in the future.

5.7 Economic Analyses

This section analyses order-of-magnitude economics and considers potential funding scenarios for the selected surface water storage project alternative sites identified in Section 5.3.7 above. The economic analyses are based on the direct and indirect benefits of using stored water for supplemental irrigation of existing irrigated lands. The funding scenarios assume WWDC grant/loan funding of an eligible local project sponsor in accordance with the contract scope of work. The economic and financing analyses address the following key elements:

- Benefits associated with the alternative projects;
- The ability-to-pay of local irrigators based on anticipated revenue from increased water yield;
- The minimum cost of water to irrigators under current WWDC guidelines; and

- The sponsor's ability-to-pay under different grant/loan scenarios.

5.7.1 Benefits Analysis

The economic benefits of supplemental irrigation water are measured by the marginal increase in farm income that would be generated by a given amount of additional water. This section develops an estimate of the marginal increase in farm income attributable to an additional acre-foot of water for a typical irrigated operation in the Owl Creek watershed. To develop this estimate, several variables must be known or estimated, including:

- The efficiency with which the additional water would be utilized by the crop or crops under consideration;
- The yield response of the crop or crops to the application of the additional water;
- The market value of the additional yield that would be generated; and
- The marginal production costs that would be incurred in harvesting the additional yield.

Estimates of this type are typically developed through crop enterprise budget analyses. These analyses can consist of developing site-specific estimates of all relevant variables through exhaustive studies of local agronomic conditions and farm production practices. They can also be generated by adapting estimates developed for other areas with similar characteristics. For this study, the latter approach was used.

Estimates of irrigation efficiency were adapted from the Wind/Bighorn Basin Plan, (BRS, Inc., 2003). That plan estimates the overall weighted annual average irrigation efficiency at 28 percent for the Owl Creek watershed. Irrigation efficiency was assumed to vary during the irrigation season, however, from a low of 17 percent in October to a high of 41 percent in August. An efficiency estimate of 35 percent was used in developing the benefit estimates described in this report.

Estimates of the crop yield response to additional irrigation water were adapted from studies undertaken for the North Platte River Basin in eastern Wyoming near Torrington (Pochop, et. al, 1992). Those studies indicate that an assumed crop mix of 50 percent alfalfa and 50 percent grass hay would have a yield response averaging 1.48 tons per acre-foot of consumptively used irrigation water. Studies by the University of Wyoming indicate that the average growing season in the Owl Creek area is approximately ten percent shorter than in the Torrington area (Pochop, et. al, 1992). Assuming that changes in the yield response to irrigation water are proportional to changes in the growing season results in an estimated yield response of 1.42 tons per acre-foot of consumptively used irrigation water.[1]

The market value of the additional yield that would be generated from an acre-foot of additional irrigation water is dependent upon crop prices. For the past four years (2003 – 2006) the market price of alfalfa hay in Wyoming has averaged $80.12 per ton. The corresponding average price for grass hay has been $75.50.[2] Because hay production in the Owl Creek watershed is estimated as about three times as much

[1] The growing season for alfalfa in the Owl Creek vicinity Wyoming was estimated as the average of the Riverton and Thermopolis growing seasons, or 211 days. The average growing season for the Torrington area is 220 days – WRS-19, University of Wyoming.

[2] http://www.nass.usda.gov/Statistics_by_State/Wyoming/index.asp

alfalfa as grass, an average price of $79.00 per ton was used to derive the benefit estimates described in this report.

The final piece of data needed to arrive at an estimate of the marginal value of supplemental irrigation water is an estimate of the marginal increase in production costs that would be incurred to harvest the incremental yield. In the case of hay production, these marginal costs consist primarily of increased costs for irrigation, baling, loading, and stacking activities. A study of these costs for North Platte River irrigators resulted in an estimated cost increase of $26.12 per ton in 1995 dollars (Watts and Brookshire, 2000). Updating these costs to 2006 dollars (consistent with other cost data and estimates herein) results in a marginal production cost estimate of $30.24 per ton[3].

The information described above can be used to estimate the economic benefits of an acre-foot of supplemental irrigation water as follows. One acre-foot of water at an irrigation diversion point applied with 35 percent efficiency will result in an estimated increase in hay production of 0.35 x 1.42 = 0.50 tons. At an average market price of $79.00 per ton, the estimated value of the increased production is $39.50. Subtracting a marginal production cost increase of 0.50 x $30.24 = $15.12 results in a net benefit estimate of $24.38 for each acre-foot of supplemental irrigation water available for diversion when and where it is needed.

5.7.1.1 Indirect Irrigation Benefits

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

Some economists argue that it is inappropriate to include indirect benefits in project evaluations because building an irrigation project in one part of Wyoming may result in economic losses elsewhere in the state. For example, in a report prepared for the U.S. Environmental Protection Agency, Huszar (1990) argued that indirect benefits should not be considered in an economic evaluation of the Sandstone Project. His reasoning was that funding for the project came from taxes on Wyoming citizens, and that without the project, taxes could be lowered with an equally large stimulating effect on the Wyoming economy. Although the logic of this argument is sound, the facts are incorrect. WWDC construction funds are derived from minerals severance taxes, not taxes on Wyoming citizens. If minerals severance taxes were lowered, the likely result would be a benefit to out-of-state users of Wyoming's coal or out-of-state shareholders of energy companies doing business in Wyoming, not Wyoming residents. For this reason, it is appropriate to consider indirect benefits in evaluating projects funded largely by minerals severance taxes.

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[3] Costs were updated using the index of production costs paid by Wyoming farmers and ranchers available from the Wyoming Office of the National Agricultural Statistics Service at the following website: http://www.nass.usda.gov/Statistics_by_State/Wyoming/index.asp

The Bureau of Economic Analysis of the U.S. Department of Commerce (USDOC) produces periodic estimates of indirect income multipliers for Wyoming's agricultural sector. Their latest published estimate of this multiplier is 3.36 (USDOC, 1992); meaning that for each dollar of additional farm income, total income in Wyoming increases by $3.36. The $3.36 is comprised of $1.00 of farm income and $2.36 of indirect income, which can be an indirect benefit of new irrigation projects.

5.7.2 Ability to Pay Analysis

An irrigator's ability to pay for irrigation water is bounded by the magnitude of direct irrigation benefits that would be generated by the additional water. For example, the analysis used in Section 5.7.1 is based on the estimate that supplemental irrigation water will generate a direct benefit of $24.38 per acre-foot. The benefit estimate reflects the market value of additional crop production minus the costs associated with producing the additional crop. Table 5.7-1 presents an analysis of maximum potential benefits of the various project alternatives based on estimated normal and dry-year shortages, available yield, and project benefits for each of the alternatives. The column labeled "Maximum Potential Annual Benefits from Supplemental Irrigation Only" is based on the estimated direct benefit of $24.38 per acre-foot of water utilized. The next to last column in the table is the "Present Value" of those benefits based on an interest rate of 4.0 percent and a loan period of 50 years. Numbers in that column could be compared to the capital cost of an alternative which would represent a break even project cost if no grant money was provided and if a loan of 4.0 percent for a 50-year period was received. The final column represents the total estimated value of the increased benefit of the project to the state economy based on a ratio of 3.36 total benefit to increased direct farm income.

The above figures represent an upper bound on ability to pay because they are estimates of the total additional farm income that could be generated from additional storage. If irrigators were required to forfeit all of the additional income to repay project expenditures, however, they would have no incentive to participate in the project. Thus, ability-to-pay studies assume that ability to pay is some reasonable fraction of direct project benefits. For purposes of this analysis, ability to pay is assumed to be 50 percent of the additional income that could be generated from new storage, or $12.19 per acre foot.

5.7.3 Financing Under WWDC Guidelines

The current WWDC project financing default standard is 67 percent grant with the remaining 33 percent of project costs to be repaid by the project sponsor. Repayment can be financed over 50 years at the State Loan Board interest rate, which is currently four (4.0) percent. The implications of applying these funding criteria to the project surface water storage alternatives can be seen in the results in Table 5.7-2A. For comparison, additional funding scenarios of 75 percent Grant/25 percent Loan and 90 percent Grant/10 percent Loan are presented in Table 5.7-2B and Table 5.7-2C, respectively. Each of these three tables show the estimated Level III cost of each alternative (column 2), the project sponsor's share of those costs based on the assumed grant (column 3), and the sponsor's annual repayment amount assuming a 50-year loan at 4.0 percent interest (column 4). The estimated Level III cost is the estimated total project cost less the estimated Level II/ Phase III costs.

The remaining two columns of the table show estimates of the maximum amount irrigators could repay each year based upon an ability-to-pay rate of $12.19 per acre-

foot of storage. These amounts are substantially less than would be required to fund any of the alternatives under current WWDC guidelines for sponsored projects.

The last column of the tables presents ability to pay as a percentage of a sponsor's share of total project costs, for the assumed funding percentage, interest rate and loan period. These results indicate a limited ability for project sponsors to repay estimated project costs without substantial additional state assistance in the form of a much higher than average grant or some other source(s) of significant funding.

Table 5.7-3 presents the recurring annual cost over the life of a 50-year loan (i.e., the amortized non-grant share of total project cost) per acre-foot of available yield. The annual cost in Table 5.7-3 is the sponsor's share of the total project cost based on the three assumed funding scenarios of 67 percent, 75 percent and 90 percent grant.

The "annual yield" used in the calculations in Table 5.7-3 is the "dry-year yield" developed as described previously in Section 5.2. The cost to the sponsor per acre-foot of yield on this basis ranges from a low of $11 per acre-foot to a high of $106 per acre-foot. The range of annual cost/acre-foot is due in large part to the size/cost of the reservoir relative to the need served and the range of grant percentages considered.

6.0 Flow Conveyance Analysis

Because all but Site 17 of the alternative sites described above lie on North Fork Owl Creek, and the majority of the storable basin yield lies on South Fork Owl Creek, a diversion from the South Fork to the North Fork Owl Creek is required, depending on the storage capacity needed. Specifically, Sites 2 and 18 of the selected alternatives, if constructed to the maximum target storage capacity discussed in Section 5.2, would require this diversion. The Level I report also identified the potential for diverting South Fork flows to the North Fork either above or below Anchor Reservoir (Figure 6.0-1). A discussion of the analyses of these diversions follows.

6.1 South Fork to North Fork Above Anchor Reservoir

This diversion would divert South Fork water above Anchor Reservoir to the North Fork as an alternative to the diversion below Anchor Reservoir near the historic Embar site as shown on Figure 6.0-1. Due primarily to the long alignment required to construct this diversion (because of the elevation of the ridge dividing the two basins and the distance separating them), three potential cost-reduction options were examined: canal, canal with tunnel, and canal with deep cut. A significant portion of the diversion would be through karst terrain and is presumed to require seepage mitigation measures (i.e., liner) to prevent excessive loss of conveyed water, adding to costs. Additionally, depending on the option chosen, the diversion traverses relatively rugged terrain, which also increases the cost of building the diversion.

This diversion has the potential to avoid a portion of the extensive losses at Anchor Reservoir (discussed in Section 3.0) by bypassing some or all of the South Fork flows around Anchor for storage at alternative sites below. The results of this study, however, recommend that Anchor Reservoir continue to serve a role in tandem with alternate storage downstream to provide users on the South Fork Owl Creek with at least some storage (as discussed in Section 3.3). In order to prevent the most loss in Anchor under this scenario, the diversion would be operated to "peel off" early runoff available flows and route them to alternate storage before losses could occur in Anchor. Later runoff flows would be stored in Anchor to serve South Fork Owl

Creek users per Section 3.3. The savings in water (relative to diverting below Anchor) would be limited to the incremental portion of the "base losses."[4] Because the nature of these base losses are unknown, the actual incremental savings could be much less, conceivably (and even likely) zero.[5] For these reasons that are very complex and well beyond the scope of this study to resolve, it is assumed that this diversion option provides no incremental savings in water relative to the below Anchor diversion option. Assuming, however, that the savings is the maximum conceivable amount, this diversion cannot be justified economically as discussed later in Section 6.3.

The three alternatives considered for a South Fork to North Fork diversion above Anchor Reservoir are summarized as follows:

- **Canal Alternative 1.** An overland diversion route was identified under this scenario to convey water from South Fork Owl Creek to North Fork Owl Creek. Due to the presence of a high ridge between the two creeks, water must be diverted approximately 1.5 miles up the South Fork Owl canyon above Anchor Reservoir. The route is relatively long at 5.53 miles and requires an aqueduct (or siphon) across Middle Fork Owl Creek. The estimated cost to construct the alternative is $4,800,000.
- **Tunnel Alternative 2a.** A second alternative was evaluated to examine the feasibility of shortening the route by tunneling through the high ridge. The route can be shortened to 3.57 miles, but the estimated cost of this option is significantly higher than Alternative 1.
- **Excavation Alternative 2b.** A third alternative, similar to the tunnel alternative but with an excavated cut instead of a tunnel, was evaluated and cost was estimated to be significantly higher than Alternative 1.

6.2 South Fork to North Fork Below Anchor Reservoir

This diversion would divert South Fork water to the North Fork near Embar as an additional source of supply for Sites 2 and 18. This diversion is short and over gentle terrain. The cost for this diversion is included in the costs for Sites 2 and 18 (see Section 5.6).

6.3 Recommendations

The diversion above Anchor Reservoir is not cost effective relative to the diversion below Anchor at Embar in a direct comparison of costs. While the diversion above Anchor Reservoir could conceivably save an indeterminate amount of water (estimated to be a maximum 1,500 ac-ft/yr, but likely much less, even zero), it is still

[4] See section 3.2.2 for a discussion of base losses. This is because flows equivalent to the diverted flow would be routed through the reservoir (under the below Anchor diversion option) and thus not subject to additional incremental loss due to increased reservoir head. In all likelihood, some additional loss due to the routing may occur, depending on the efficiency of proposed reservoir operations and flood attenuation, but this is presumed to be small.

[5] Base losses, as discussed in Section 3.2.2, could conceivably be caused by losses in portions of the ungaged basin above Anchor and may not impact flows into Anchor from South Fork Owl Creek. In this case, there would be no incremental savings. Alternatively, because the diversion would not capture flows from the ungaged basin, these flows would still be subject to loss in Anchor. Additionally, if base losses are due to infiltration of water into the sediments at Anchor (and loss out the bottom), which is expected to be the most likely case, then some "sacrificial' flow must be lost in Anchor to keep the sediments saturated so that flow can be provided to users on South Fork Owl Creek during the "peel off" season. Depending on the amount of sacrificial flow required, incremental savings could again be zero. Finally, if base losses are do to model error, savings would again be zero.

considered to be uneconomic relative to the diversion below Anchor. This is because according to the basin plan model, flows are available to make up this savings by increasing storage incrementally (and storing available flows at the alternate reservoir) at a much lower cost per acre-ft than utilizing the upper diversion. Thus, under any conceivable scenario described in this report, utilizing the diversion below Anchor is the recommended alternative.

An unlikely, but conceivable option would be to completely "route around" Anchor, providing no storage whatsoever in Anchor and examining a third diversion, from North Fork Owl Creek back to South Fork Owl Creek. This would still require alternate storage to serve users downstream of the confluence. Users on South Fork Owl Creek would not benefit from storage in Anchor, but could conceivably save more water than outlined above. Whether the timing of this savings would be late enough in the season to outlast storage available in Anchor under the proposed operating scenario is unknown and perhaps unlikely. Given the complexity of the problem, and the substantial analyses that would be required to resolve the remaining unknowns regarding losses at Anchor, the complex modeling and optimization required (including reservoir routing and the basin modeling recommended in Section 5.1.3) it is not likely that this option is worth pursuing for a benefit that may be minimal at best and unsupportable.

7.0 Conclusions and Recommendations

Summary conclusions and recommendations are presented below for the analyses of Anchor Reservoir, alternate storage site evaluation in both Anchor Reservoir and for six sites below Anchor Reservoir, and an analysis of flow conveyance diversions from the South to North Forks Owl Creek.

7.1 Conclusions

7.1.1 Anchor Dam and Reservoir Operations Analysis

The following major conclusions were established regarding the current operations at Anchor Reservoir:

- There appear to be no significant losses occurring in the canyon below Anchor Dam based on a comparison of readings for a gage installed as part of this project immediately below Anchor Dam and an existing gage at the mouth of the canyon. Results indicate that the stream in this reach is a slightly gaining stream (6 percent of flow). For purposes of evaluating Anchor Reservoir, this stream section was assumed to have no net gain or loss.

- A reservoir water balance model was created to evaluate losses at Anchor Reservoir. Model fit statistics indicate the model is a good predictor of losses at Anchor based on the following data inputs: gage data above and below Anchor, and storage volume in Anchor. Model results indicated the following:

  - Actual losses in Anchor Reservoir are more than indicated by a direct comparison of gage records above and below Anchor (after accounting for evaporation from Anchor Reservoir) due to significant inflows from ungaged basins above Anchor. Estimated average actual historic losses in Anchor Reservoir (and the ungaged basin above) are 12,500 ac-ft/year.

  - Anchor loss rates appear to have diminished significantly over its history, but the rate of decay has slowed since 1980. This "healing" of Anchor Reservoir

may be due to man-made repairs, sedimentation in the reservoir, and/or natural plugging of leaks.

- New breakthroughs (significant increases in loss rates) have occurred periodically in Anchor Reservoir according to the model. The most recent significant breakthrough occurred as recently as 1995, and new breakthroughs appear to be unpredictable, making Anchor Reservoir storage unreliable.
- Twenty-five (25) percent of the modeled losses are due to "Base Losses." Base losses are predicted by the model as 4.8 cfs or the inflow to Anchor Reservoir, whichever is less. These losses are believed to be due to one of three causes: losses in ungaged basins above Anchor, model/measurement errors, and/or seepage into reservoir sediments in Anchor during periods of no or minimal storage in Anchor Reservoir.
- Approximately 70 percent of these losses are due to Sinkhole/Leak Zone losses, which are modeled using the orifice equation (related to head in the reservoir). A series of 16 Sinkholes/Leak Zones were modeled, each with a specific start date, orifice elevation, orifice size and characteristic, and "half-life" to model apparent decay. 69 percent of these losses are attributable to five elevation zones.

▪ A drogue evaluation was conducted in Anchor Reservoir during a relatively high reservoir pool elevation. (Drogues are devices designed to float with the current at a specific depth to locate leaks.) The devices found three leaks that were confirmed by later visual reconnaissance of the floor of the reservoir when it was essentially empty. One of the devices found an additional apparent leak that appears to be hidden within a talus slope and could not be confirmed visually.

▪ Another three apparent leaks were found during the empty-reservoir reconnaissance that were not located by the drogues (probably due to high winds and Anchor Dam outlet currents affecting the drogue performance during the study period).

▪ The six leaks that could be seen visually appear to account for only a small portion of current losses in Anchor Reservoir.

▪ An evaluation of the gage data above and below Anchor indicated that potential inaccuracies in the gage readings are unlikely to be a significant cause of the apparent losses.

▪ Potential mitigation alternatives were developed to address localized openings in the reservoir floor found during field reconnaissance and leakage characterized in the model as Base Losses. However, reliable economic analysis of the effectiveness of these measures was not possible, primarily due to the unpredictability of new breakthroughs in Anchor Reservoir that could occur and result in new losses exceeding any gains made by the repairs.

7.1.2 Water Storage Site Evaluation and Analysis

7.1.2.1 <u>Anchor Reservoir</u>

The feasibility of raising the internal dikes within Anchor Reservoir was examined. The following conclusions were reached:

- Raising the internal pool level in Anchor is problematic because there is no carryover storage possible at Anchor due to the high loss rates, and raising pool levels could increase the development of new breakthroughs that significantly increase loss rates.
- Dike raises of 10-feet and 20-feet were examined. Due to the existing high loss rates, the effective capacity gained is significantly less than the theoretical capacity gained at 1 to 3 month holding periods. The effective capacity gained diminishes rapidly with storage duration.
- Based on post-1980 records of actual pool levels and an analysis of South Fork Owl Creek inflows to Anchor Reservoir, it appears that inflows are only sufficient to utilize this capacity in wet-normal and/or wet years, when extra storage is less valuable. Since this extra storage cannot be carried over to the following year, its benefits are very limited.
- The economics of the raises cannot be estimated reliably due to the above factors and the unpredictability of breakthroughs at Anchor, but it is known that a per acre-foot cost comparison of additional storage in Anchor would compare very poorly relative to providing storage in the alternate storage sites downstream.

7.1.2.2 Alternate Storage Sites

Six alternate storage sites identified by the Level I Study were evaluated. The results of those evaluations are summarized as follows:

- **Reservoir Capacity.** Based on the irrigation shortage and available flow results of the Wind River/Bighorn Basin Plan Model, an estimated maximum reservoir capacity of 25,000 acre-feet was selected to provide firm yields sufficient to meet all shortages in the Middle Area of the District for two back-to-back dry years preceded by a normal year.
- **Model Reliability.** Review of the basin model indicates that it may be over-predicting shortages, and that a smaller reservoir capacity may be adequate. As described later, a framework for constructing a more sophisticated model to resolve these uncertainties is recommended.
- **Site Screening.** The six alternate storage sites were screened to two alternatives, with a "backup plan" for each. Specifically, Site 1, with a maximum physically practicable capacity of 9,000 ac-ft, is the preferred alternative if the recommended refined modeling described later indicates this capacity will adequately serve the District's needs. If the modeling indicates a higher capacity than 9,000 acre-feet is required, then the larger capacity (up to 25,000 acre-feet or more) Site 2 is the preferred alternative. Site 18 serves as backup to either alternative, and Site 2 serves as backup to Site 1.
- **Conceptual Design.** Zoned earthfill dams with earthen cut emergency spillways and concrete service spillways in the abutments (except Site 1, which does not require a service spillway) are the selected design configurations for Sites 1, 2 and 18.
- **Environmental/Permitting.** The primary environmental considerations in the Owl Creek watershed in relation to permitting of dam and reservoir sites include: wetlands, sage grouse leks, and crucial big game habitat. This is not to say that other considerations are not important or may not significantly influence the ability to permit a project. It is considered most likely that permitting of a dam

and reservoir site would require that a full EIS process be followed under the lead of either the BLM or the COE, or possibly under their joint lead.

- **Cost Estimates.** The estimated costs for development of alternate storage vary substantially depending mainly on the size and overall efficiency of the dam and reservoir site. The total project cost of the three sites that form the selected alternatives range from about $14,900,000 for the 9,000 acre-feet capacity Site 1 – Hamilton Dome, $43,300,000 for the 25,000 acre-feet capacity Site 2 to $77,900,000 for the 25,000 acre-feet capacity Site 18. The estimated costs per acre-foot of storage on a capital cost basis for the six projects studied in more detail range from about $1,700 to $3,200.

- **Economic Analyses.** Under the most favorable assumption of a 90 percent grant and the WWDC's standard loan terms for the remainder of the cost, the estimated sponsor's ability to pay (as a percentage of the sponsor's annual repayment obligation) ranges from 16 percent for Site 18 to 66 percent for Site 1. The ability to pay percentages assuming 67 percent grant for the same projects are estimated as 5 percent and 20 percent.

7.2 Recommendations

The recommendations discussed in more detail in the preceding sections of this report are summarized below:

7.2.1 Anchor Reservoir

- **Operations.** Present operations should continue as-is, as long as alternate storage is not available downstream. The continuing high loss rates in Anchor Reservoir do not allow carryover storage to be held year-to-year. If one of the selected alternate storage sites (Site 1, 2, or 18) is built downstream, an exchange should be considered if/as appropriate such that the alternate site serves all users in the Middle Area of the District to the maximum extent possible and short-term storage in Anchor Reservoir is utilized to serve shortages on South Fork Owl Creek in the Upper Area of the District that can not be served by the alternate storage site. It is recommended that Anchor reservoir operations otherwise remain unchanged, except as described above.

- **Large Scale Mitigation Alternatives for Identified Losses.** Not enough information is available about the Base Losses and the apparent "Rockfall Area" losses to economically justify any mitigation alternatives identified in this study. Economic justification of these losses would require loss rates and their exact sources to be known, sophisticated reservoir modeling, and some reliable estimate of future losses in Anchor Reservoir, plus additional design analyses, none of which are available within the scope of this study. Acquiring this information would require a substantial effort at high cost that still may not generate the information required. Thus, further consideration of these mitigation options cannot be justified and are not recommended.

- **Small Scale Mitigation Alternatives for Identified Losses.** Two conceptual low-cost mitigation alternatives were identified for visually observable leak areas in Anchor Reservoir. Although these measures would very likely marginally decrease losses if implemented, it is estimated that the decreases would be so small as to not be measurable. Thus, the value of even these low-cost mitigation measures would likely remain unknown. As a result, implementation of these measures is not recommended.

7.2.2 Alternate Storage

If the District requests and WWDC agrees to further pursue further study of alternate storage downstream of Anchor Reservoir, then the following recommendations are offered:

- Based on the results of existing modeling of available flows and irrigation shortages and careful evaluation of those results during this study, more detailed study of the potential for development of surface water storage in the Owl Creek watershed is justified and recommended. The study should include, but not be limited to, more detailed hydrologic, water rights and reservoir operations modeling using the StateMod or an equivalently robust model to support refinement of the existing irrigation needs, available flows in the watershed, and opportunities to most efficiently utilize Anchor Reservoir and recommended alternate storage.
- If, based on the results of the modeling described above, a storage capacity of 9,000 ac-ft or less can be shown to satisfy shortages in the Middle Area of the District, then it is recommended that Site 1 become the preferred site and subsurface geotechnical and geologic studies be conducted if permission to access the site can be obtained.
- If, based on the model, a storage capacity of more than 9,000 ac-ft is required, then it is recommended that Site 2 become the preferred site and subsurface geotechnical and geologic studies be conducted if permission to access the site can be obtained.
- If the subsurface studies indicate that Site 1 or Site 2 is infeasible, then Site 2/18 and Site 18 should be considered as backup alternatives, respectively.

7.2.3 Transbasin Diversion

If Sites 2 or 18 are constructed as alternate storage downstream as described above, the South Fork to North Fork Owl Creek Diversion below Anchor Reservoir is the recommended transbasin diversion alternative.

8.0 References

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List of Tables

Table 3.2-1
Anchor Reservoir Loss Model

	Sinkhole/Leak Zone #																Reservoir Seepage**	Base Losses***	Total
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16			
Portion of Historic Loss	1.6%	2.6%	14.4%	2.9%	2.3%	14.0%	11.8%	6.9%	2.7%	1.2%	3.5%	0.4%	2.2%	1.9%	0.9%	0.6%	5.3%	24.8%	100.0%
Diameter (feet)	0.70	3.03	4.06	1.58	3.53	3.38	4.07	3.70	1.07	1.73	2.04	25.84	0.77	3.00	10.00	2.00			
HalfLife (years)	3.84	1.08	3.37	1.19	0.45	13.48	0.59	3.58	2.15	0.47	2.35	0.15	1.87	0.10	0.13	1.09			
Elevation	6336	6353	6370	6340	6377	6397	6306	6387	6340	6347	6376	6391	6344	6394	6411	6376			
Start Date	5/11/1960	8/6/1961	1/2/1963	6/16/1965	5/15/1973	6/24/1974	9/30/1995	11/8/1966	5/18/1976	2/1/1970	10/27/1988	1/5/1994	4/30/1991	6/4/1995	3/19/1991	7/23/1983			
Net Loss (ac-ft)*	202	328	1,801	367	285	1,762	1,483	871	335	145	439	45	280	242	108	72	669	3,112	12,546

* Due to data gaps, net losses are not necessarily reflective of an average annual rate.

** Modeled as an assumed 10-foot thick (average) blanket of sediments with an effective permeability of 1 x 10^{-5} cm/s.

*** Modeled as Anchor Reservoir inflow or 4.8 cfs, whichever is lower.

Table 3.2-2

Required Storage in Anchor Reservoir (ac-ft) to Achieve Desired Storage Yield*

Desired Storage Yield (ac-ft)	Duration of Storage (months)											
	1	2	3	4	5	6	7	8	9	10	11	12
0	297	629	990	1,385	1,821	2,300	2,829	3,396	3,998	4,992	6,892	9,681
500	850	1,231	1,651	2,113	2,623	3,180	3,760	4,520	6,073	8,469	12,611	-
1,000	1,397	1,833	2,313	2,844	3,411	4,017	5,032	6,955	9,782	-	-	-
1,500	1,947	2,439	2,982	3,553	4,197	5,423	7,548	10,787	-	-	-	-
2,000	2,498	3,046	3,619	4,292	5,622	7,835	11,321	-	-	-	-	-
2,500	3,048	3,621	4,295	5,628	7,844	11,338	-	-	-	-	-	-
3,000	3,572	4,223	5,478	7,629	10,933	-	-	-	-	-	-	-
3,500	4,127	5,271	7,324	10,391	-	-	-	-	-	-	-	-
4,000	4,996	6,898	9,691	-	-	-	-	-	-	-	-	-
4,500	6,035	8,416	12,498	-	-	-	-	-	-	-	-	-
5,000	6,904	9,701	-	-	-	-	-	-	-	-	-	-
5,500	7,661	10,992	-	-	-	-	-	-	-	-	-	-
6,000	8,367	12,394	-	-	-	-	-	-	-	-	-	-
6,500	9,084	13,985	-	-	-	-	-	-	-	-	-	-
7,000	9,854	-	-	-	-	-	-	-	-	-	-	-
7,500	10,700	-	-	-	-	-	-	-	-	-	-	-
8,000	11,644	-	-	-	-	-	-	-	-	-	-	-
8,500	12,677	-	-	-	-	-	-	-	-	-	-	-
9,000	13,790	-	-	-	-	-	-	-	-	-	-	-
9,500	14,988	-	-	-	-	-	-	-	-	-	-	-
10,000	-	-	-	-	-	-	-	-	-	-	-	-
10,500	-	-	-	-	-	-	-	-	-	-	-	-
11,000	-	-	-	-	-	-	-	-	-	-	-	-
11,500	-	-	-	-	-	-	-	-	-	-	-	-
12,000	-	-	-	-	-	-	-	-	-	-	-	-
12,500	-	-	-	-	-	-	-	-	-	-	-	-
13,000	-	-	-	-	-	-	-	-	-	-	-	-
13,500	-	-	-	-	-	-	-	-	-	-	-	-
14,000	-	-	-	-	-	-	-	-	-	-	-	-
14,500	-	-	-	-	-	-	-	-	-	-	-	-
15,000	-	-	-	-	-	-	-	-	-	-	-	-

* Based on 2007 Model loss rates; assumes no change in storage due to inflow/outflow

Highlighted cells assume that internal dikes are raised (per Task 4A)

Table 5.1-1
Shortages and Available Flows in Study Area

Reach	Shortages (ac-ft)		Available Flow (ac-ft)	
	Dry Year	Normal Year	Dry Year	Normal Year
A - South Fork Owl Creek	3,462	1,560	1,468	11,752
B - North Fork Owl Creek	0	0	2,961	9,563
C - Owl Creek below confluence (upper reach)	10,152	3,838	n/a	n/a
D - Owl Creek below confluence (lower reach)	19	0	n/a	n/a
Totals	**13,633**	**5,398**	**4,429**	**21,315**

Table 5.2-1
Reservoir Capacity and Yield

Alternative	Reservoir Capacity (ac-ft)	In Stream Total Available Dry Year (ac-ft)	"Dry Year" Irrigation Shortage Below Alternative Site (acre-ft)	Assumed Annual Reservoir Yield - Dry Year (ac-ft)	In Stream Total Available Normal Year (ac-ft)	"Normal Year" Irrigation Shortage Below Alternative Site (acre-ft)	Assumed Annual Reservoir Yield - Normal Year (ac-ft)
Site 1	9,000	2,960	10,200	5,500	9,560	3,850	3,850
Site 2	25,000	2,960	10,200	10,200	19,750	3,850	3,850
Site 5	25,000	2,960	10,200	10,200	19,750	3,850	3,850
Site 8	9,500	2,960	10,200	5,800	9,560	3,850	3,850
Site 17	25,000	2,960	10,200	10,200	19,750	3,850	3,850
Site 18	25,000	2,960	10,200	10,200	19,750	3,850	3,850

Table 5.3-1
Storage Site Matrix

	1	2	5	8	17	18
Site Name	Hamilton Dome	Onstream #3	Onstream #4	Onstream #5	Confluence	Onstream #6
Locational Information						
USGS 7.5-minute Topographic Quadrangle	Arapahoe Ranch	Arapahoe Ranch	Embar	Embar	Arapahoe Ranch	Embar
Tributary (Onstream) or Supply (Offstream)	North Fork Owl	North Fork Owl	North Fork Owl	North Fork Owl	North and South Fork Owl	North Fork Owl
Onstream / Offstream	Offstream	Onstream	Onstream	Onstream	Onstream	Onstream
Location Relative to Demand	Above Middle District	Above Middle District	Above Middle District	Above Middle District	Above Middle District	Above Middle District
Basin Characteristics & Hydrology						
Drainage Areas (square miles)						
Direct Runoff to Reservoir	2.7	104.4	100.9	92.7	305.2	98.5
Diversion to Reservoir	100.9	0.0	0.0	0.0	0.0	0.0
Estimated PMF Flood Characteristics						
Estimated Peak Discharge (thousand cfs)	10	85	84	80	159	82
Estimated Runoff Volume (thousand acre-feet)	1.2	54.3	52.4	48.0	165.0	51.1
Reservoir Characteristics & Operation						
Demand						
Normal Year Shortages (ac-ft)	3,850	3,850	3,850	3,850	3,850	3,850
Dry Year Shortages (ac-ft)	10,200	10,200	10,200	10,200	10,200	10,200
Supply						
Normal Year Available Flow (ac-ft)	9,560	19,750	19,750	9,560	19,750	19,750
Dry Year Available Flow (ac-ft)	2,960	2,960	2,960	2,960	2,960	2,960
Normal High Water						
Target Capacity (acre-feet)	9,000	25,000	25,000	9,500	25,000	25,000
Surface Area (acres)	363	650	597	316	902	490
Water Surface Elevation	5,408	5,465	5,536	5,779	5,416	5,733
Average Water Depth (feet)	25	38	42	30	28	51
Site Geology						
Geology						
Valley Stress Relief Features	Assumed	Present	Assumed	Assumed	Present	Present
Adverse Structure Impacts	Not expected	Not expected	Not expected	Possible	Present	Not expected
Relative Proximity to Karst Units	Far	Far	Far	Close	Moderate	Far
Expected Relative Foundation Cutoff Depths	Shallow	Moderate	Moderate	Moderate	Deep	Moderate
Bedding Dip at Damsite	Favorable	Favorable	Favorable	Unfavorable	Unfavorable	Favorable
Anticipated Reservoir Seepage	Low	Low	Moderate	Potentially High	Moderate	Potentially High
Landslides/Slope Instability	Not apparent	Not apparent	Present	Present	Not apparent	Not apparent
Borrow						
Relative apparent availability	Favorable	Favorable	Adequate	Marginal	Adequate	Marginal
Relative apparent quality	Favorable	Favorable	Adequate	Marginal	Adequate	Marginal
Site Environmental Conditions						
Big Game Habitat - Crucial						
Mule Deer	0	542	649	301	489	528
Antelope	381	268	128	0	36	0
Sage Grouse Leks	x	x		x		x
T&E Species (BFF-Black-footed ferret)	BFF(?)	BFF(?)	BFF(?)		BFF(?)	
Riparian Habitat		x	x	x	x	x
NWI Wetlands (acres - direct impact)	0	33	4	12	108	15
Aquatic Habitat (feet of channel - direct impact)	0	22,500	22,100	10,100	23,100	19,900
WDEQ Stream Classification		2C	2C	2C	2AB & 2C	2C
WGFD Stream Classification		IV	IV	IV	IV	IV
Infrastructure and Ownership						
Infrastructure/Utilities Conflicts						
Residences/Ranches	0	0	0	0	3	0
Highway/Paved Road (miles)	0.0	0.0	0.0	0.0	4.0	0.0
Improved Unpaved Road (miles)	0.0	0.0	0.0	0.0	0.0	0.0
Pipelines (miles)	0.0	0.0	0.0	0.0	0.0	0.0
Powerlines (miles)	0.0	0.0	0.0	0.0	1.8	0.0
Irrigated Lands (acres)	0	0	0	0	447	0
Land Ownership (acres owned)						
Private	0	743	648	202	508	257
State	2	0	0	0	0	230
Federal	129	4	6	128	0	68
Tribal	251	0	0	0	478	0
Dam Characteristics & Hydraulic Structures						
Dam						
Proposed Type	Zoned Earthfill	Zoned Earthfill	Zoned Earthfill	Zoned Earthfill	Zoned Earthfill	Zoned Earthfill
Number of Dams	2	2	2	1	1	1
Freeboard/Head over Spillway (feet)	5	10	10	10	15	10
Crest Elevation (feet)	5,413	5,475	5,546	5,789	5,431	5,743
Total Crest Length (feet)	2,189	5,036	6,146	1,118	4,124	3,322
Crest Width (feet)	30	30	20	30	30	30
Maximum Dam Height (feet)	83	115	131	99	91	168
Foundation Excavation (thousand cy)	161	647	699	400	1,130	1,135
Total Embankment Volume (thousand cy)	787	2,752	2,823	1,706	4,095	5,544
Storage Efficiency (ac-ft/1000cy)	11	9	9	6	6	5
Outlet Works						
Number of Outlets	2	1	1	1	1	1
Proposed Type	Cut/cover conduit	Cut/cover conduit	Cut/cover conduit	Cut/cover conduit	Cut/cover conduit	Cut/cover conduit
Outlet Elevation (feet)	5,330	5,360	5,415	5,690	5,340	5,575
Service Spillway						

Table 5.3-1
Storage Site Matrix

	1	2	5	8	17	18
Site Name	Hamilton Dome	Onstream #3	Onstream #4	Onstream #5	Confluence	Onstream #6
Proposed Type	Integrated Outlet Works	Unregulated Concrete	Unregulated Concrete	Unregulated Concrete	Unregulated Concrete	Unregulated Concrete
Design Capacity (cfs)	1,000	8,500	8,300	7,900	15,800	8,200
Approximate Width (feet)	24	71	69	66	72	68
Emergency Spillway						
Proposed Type	Cut Channel	Cut Channel	Cut Channel	Cut Channel	Cut Channel	Cut Channel
Design Capacity (cfs)	8,200	76,300	74,800	71,200	142,600	73,800
Approximate Width (feet)	190	630	620	590	650	610
Approximate Length (feet)	2,000	1,800	2,500	1,800	2,000	2,500
Height Efficiency (feet/thousand ac-ft)	9.2	4.6	5.2	10.4	3.7	6.7
Supply and Delivery Facilities						
South Fork to North Fork or North Fork to Reservoir Supply Diversion						
Length (miles)	1.9	1.2	1.2	0.0	0.0	1.2
Terrain	Favorable	Favorable	Favorable	None	None	Favorable
Storage Delivery Channel						
Length (miles)	1.6	0.0	0.0	0.0	0.0	0.0
Terrain	Favorable	None	None	None	None	None
Costing						
Total Project Cost	$14,934,220	$43,278,971	$44,550,133	$25,854,775	$68,001,085	$77,891,701
Total Project Cost per cubic yard of fill	$ 18.98	$ 15.73	$ 15.78	$ 15.15	$ 16.61	$ 14.05
Total Project Cost per ac-ft of storage	$ 1,659	$ 1,731	$ 1,782	$ 2,722	$ 2,720	$ 3,116

(green)	Excellent or more than adequate
(yellow)	Favorable or adequate
(light orange)	Potential fatal flaw or unfavorable value
(orange)	Probable fatal flaw or very unfavorable value

Table 5.3-2
Summary of Engineering Geologic Observations

Dam Site	Dam Description	Geologic Conditions	Geotechnical Concerns	Clay Core Borrow Source	Filter / Drain Borrow Source	Other Comments/ Notes
1	Off channel storage site Two dams (east and west)	1. Cody Shale 2. Qc: 4-6' of Qc on slopes 3. Qal :5-10 ft. Qal in valley 4. Terrace Deposits (avg. 10' thick)	Suitability of Qal for dam needs to be evaluated	Weathered Cody Shale (abundant)	Terrace Deposits	
2	Primary Dam on North Fork with saddle dam on swale to north	1. Cody Shale (exposed by steep stream cut on right abutment); dipping NW (upstream) 10-15 degrees; appears tight except for open stress relief fractures 2. Alluvium: est. 15-30 ft. thick in valley bottom (Note: channel incised 10-15 feet below surface of flood plain) 3. Terrace Deposits: 0-5 feet on Rt. abut.; 10-15' on Lt. abut.	Valley Stress Relief Features in South Abutment with openings up to ~2 inches or more filled near surface Clay beds in sequence?	Weathered Cody Shale available in both abutments	Terrace Deposits available in both abutments	
5		1. Bedrock: Interbedded sequence of sandstone and siltstone; beds are ½ inch to 3 inches; rock is weak, closely fractured; sandstone is cemented, but siltstone is generally uncemented 2. Bedrock Dip: NW 5-7 degrees (upstream) 3. Older Flood Plain Deposits: Bedrock covered by older flood plain deposits (predominately fine sand and silt) that extend up to 30 ft above active stream; and mantle bedrock across and along the margins of the valley 4. Landslide Deposits: oversteepended older flood plain deposits are slumping and creeping on slope flanks of Rt. abut. 5. Terrace Deposits. 10-15 ft. thick on Lt. abut.; 0-5 feet thick on Rt. abut.	Thick older floodplain deposits would need to be removed in dam foundation area;	No obvious source of clay core material	Terrace Deposits	
17	Large dam that would span across both North and South Forks	**Right Abutment**: 1. Frontier Formation: Alternating cross-bedded sandstone (to 30 feet) and shale beds 2. Located on the northeast limb of anticline **Left Abutment:** 1. Cody Shale 2. Structure dips to NW 10-15 degrees 3. Covered by 5-10 ft. of Qc and older Qal floodplain deposits 4. Channel incised 15-20 feet below surface of flood plain deposit	1. Open fracture sets in sandstone beds (with openings to 2 inches or greater) 2. Vertical Stress relief fractures parallel to valley wall 3. Masses of slope debris up to 20-30 feet thick locally associated with toppling of sandstone 4. Sequence dips downstream at 12 degrees 5. Depth to Gypsum Springs and Chugwater Group unknown 6. Thickness of Alluvium that will likely have to be removed in valley	Weathered Cody Shale available in north abutment area	Terrace Deposits available in north abutment area. Suitability of Qal in valley unknown.	Structural discontinuity between the north and south limb of Owl Creek Anticline
18		1. Bedrock: Exposed in 30 ft. exposure in creek bottom; thinly interbedded sandstone and siltstone as at 5. 2. Bedrock Structure: Gentle anticline through the dam site; with dips < 5 degrees. 3. Colluvium: up to 5 feet thick	Valley stress relief fractures; some filled with gypsum to ¼ inch.	No obvious source of clay core material	Limited (up to few feet on Lt. abut?)	

Table 5.4-1
Site 18 Test Pit Descriptions

Test Pit	Depth (feet)	Sample	Field Description	Interpretation	Notes
OCTP-1	0.0-1.2		SILTY CLAY (CL) with gravel. Stiff, moist	Residual soil	North end of Left Abutment; subtle knoll
	1.5-5.5	Bag	INTERBEDDED MUDSTONE, SANDSTONE, SILTSTONE. Thinly interbedded fine-grained sandstone, mudstone, and siltstone. Mudstone is dark olive gray, varved, with wavy interbeds; with poorly developed shaly partings; low hardness, and weak to friable. Sand and silt interbeds are mostly .1"-1", wavy to lenticular; with occasional thicker, moderately hard sandstone beds to 4" thick, Feox staining. Approx. 50-60% mudstone. Approximate strike and dip S65W7NW; Widely spaced fracture sets :(1) N87E Vertical; open near surface (up to 1/8"); (2) N87E68SE, Irregular, 1/4" oscillations, Fe+CaCO3 coating; both sets appear tight below about 4-4.5 feet.	Mesaverde Formation	
OCTP-2	0.0-1.0		SILTY SAND (SM) medium dense, moist with minor clay.		Left abutment; east side of small ephemeral drainage
	1.0-6.5	Bucket	SILTY SAND (SM/ML), very fine-grained sand, medium loose, uncemented	Eolian (loess) ?	
	6.5-9.0	Bag	SANDY GRAVEL (GP), medium dense, uncemented, poorly sorted, clasts (upper Mesaverde Fm. Sandstones) are angular to subangular with occasional cobble to boulders up to 8" in diameter.	Colluvium	
	9.0-15.5	Bucket	SAND (SP) with SANDY GRAVEL (GP) interbeds and occasional cobbles and boulders	Interbedded eolian ? and colluvium	
OCTP-3	0.0-8.0	Bag	SAND (SP) fine-grained to very fine-grained, medium brown, slight increase in grain size with depth, medium loose, small patches (<1") of CaCO3 and strong rxn. with HCL, very weak cementation	Eolian (loess) ?	Left Abutment; north of access road
	8.0-10.5	Bag	SILT/SILTY SAND (ML/SM), predominately silt,, sand is very fine-grained, medium dense	Eolian (loess) ?	
	10.5-12.0	Bag	SAND (SP), fine-grained, orange brown, medium dense	Eolian (loess) ?	
OCTP-4	0.0-1.1		SILTY SAND (SM) loose, moist with occasional rock clasts	Colluvium	Left Abutment; downhill from TP-3, north of access road; excavated to located bedrock below surficial materials
	1.1-8.0		SANDSTONE, light grey, low hardness, weak to friable, deeply weathered, FeO, and CaC03 along bedding; thinly bedded (1-5" thick), closely fractured (no obvious consistent fracture sets; fractures appear to be controlled by weathering and creep) sandstone is immature with subangular - subround mafic grains. Dips into hill (north) at 6-7 degrees (absolute strike not measurable due to weathering)	Mesaverde Formation	
OCTP-5	0.0-3.2		GRAVEL W/SAND (GP), medium dense, clast supported, gravel is angular and poorly sorted, mostly clast supported	Colluvium	Left Abutment, south of access road
	3.2-8.0	Bag	SILTY SAND (SM), very fine-grained sand, yellow brown, medium dense, uncemented, massive (unstratified), with faint relic bedrock texture grades to weathered sandstone at depth	Residual soil	
	8.0-12.0	Bag	SANDSTONE	Mesaverde Formation	
OCTP-6	0.0-3.0		SANDY GRAVEL (SP), yellow brown, medium dense, angular clast 1-3", derived from Mesaverde Fm.,predominately clast supported, irregular basal contact; upper o.8' minor clay, roots	Colluvium	Left Abutment, south of access road, on bench just above valley section
	3.0-4.0		SANDSTONE, hard, moderately weak, closely fractured cemented	Mesaverde Formation	
	4.0-9.0		SILTSTONE/MUDSTONE interbedded, deeply weathered, dense soil like properties, v. friable bedrock with relic bedding texture. N68E10NW	Mesaverde Formation	
OCTP-7	0.0-9.0		SILT/SANDY SILT (ML/SM); medium dense, moist, homogeneous	Alluvium - overbank deposits	Valley Bottom; older floodplain surface located 10-15 feet above recent floodplain
OCTP-8	0.0-5.5		SILT/SANDY SILT (ML/SM); medium dense, moist, homogeneous	Alluvium - overbank deposits	Valley Bottom; older floodplain surface located 10-15 feet above recent floodplain
	5.5-8.0		COBBLES/GRAVEL (GP), loose, well rounded cobbles to 6", clast supported, coarse sandy matrix	Alluvium - channel deposits	
OCTP-9	0.0-1.8		SILTY CLAY (CL), brown, stiff, moist, with roots and occasional gravel Stiff, moist		Right Abutment, downstream from dam alignment (only accessible location), on mid slope bench
	1.8-9.0	Bag	SILT W/ GRAVEL (ML), yellow brown, stiff, moist, variable size and volume of gravel to 3" (mostly 1.2-1" in diameter), includes distinct rounded purple gravel clasts; gravel increases with depth,	Colluvium	
OCTP-10	0.0-1.5		SILTY CLAY (CL), brown, stiff, moist, with roots and occasional gravel Stiff, moist		Right Abutment, downstream from dam alignment (only accessible location); located
	1.5-10.0		SILT W/ GRAVEL (ML), yellow brown, stiff, moist, variable size and volume of gravel to 3" (mostly 1.2-1" in	Colluvium	

Table 5.4-2
Summary of Geologic Units at Site 18

Geologic Unit	Description	Est. Thickness[1] (feet)	Occurrence		
			Left Abutment	Valley Bottom	Right Abutment
Alluvium	Consists of medium dense silt and sandy silt (overbank deposits) with localized lenses of loose cobbles and gravel (channel deposits). Thickness in valley section unknown.	>15		x	
Colluvium	Includes unconsolidated gravelly sands, sandy gravel, and silt with gravel deposits that mantle slopes. Derived from weathering and downslope movement of predominately sandstone material from the upper Mesaverde Formation sequence. Locally interbedded with clean fine-grained, homogeneous sands and silts (Eolian Deposits) on left abutment.	0-9 (left)	x		x
	Thickness of colluvium overlying bedrock on right abutment unknown.	>10 (right)			
Eolian (wind blown) Deposits[2]	Unconsolidated beds of homogeneous silt, very fine-grained sand, and fine-grained sand and silty sand beds that overlie bedrock on the left abutment. Locally interfingers with colluvium beds.	0-20	x		
Mesaverde Formation	Lower unnamed subunit of the Mesaverde Fm. that consists of two materials: (1) thinly interbedded mudstone, sandstone and siltstone that is low hardness, and weak to friable; and (2) thinly bedded sandstone, low to moderately hard, weak to moderately strong.	>200	x	x	x

[1] Estimated thickness is based on geologic mapping, geomorphic evidence, and limited shallow test pit exploration. Actual thickness of units that extend below the depth of the backhoe test pits can be verified using deeper subsurface exploration (i.e., exploration drilling).

[2] An eolian origin is inferred based on the texture and physical characteristics of the deposit

Table 5.6-1
Site 1 Conceptual Cost Estimate

Cost Item	Cost Estimate	
Preparation of Final Designs and Specifications	$1,296,000	
Permitting	$156,000	
Mitigation	$3,000	
Legal Fees	$52,000	
Acquisition of Access and Rights of Way	$311,000	
Non-Construction Cost Total		***$1,817,000***
Project Components		
Mobilization	$587,000	
Dam	$6,260,000	
Spillway(s)	$940,000	
Outlet Works	$833,000	
Other	$1,749,000	
Construction Cost Subtotal #1		***$10,369,000***
Engineering Costs = CCS#1 x 10%	$1,037,000	
Subtotal #2		***$11,406,000***
Contigency = Subtotal #2 x 15%	$1,710,919	
Construction Cost Total		***$13,117,000***
Project Cost Total		***$14,934,000***
Less Level II/Phase III Costs [1]	$1,452,000	
Project Cost Used in Ability to Pay Analysis		***$13,482,000***

[1] Preparation of Final Designs and Specifications; and Permitting

Table 5.6-2
Site 2 Conceptual Cost Estimate

Cost Item	Cost Estimate	
Preparation of Final Designs and Specifications	$3,734,000	
Permitting	$448,000	
Mitigation	$260,000	
Legal Fees	$149,000	
Acquisition of Access and Rights of Way	$896,000	
Non-Construction Cost Total		***$5,488,000***
Project Components		
Mobilization	$1,706,000	
Dam	$21,146,000	
Spillway(s)	$3,579,000	
Outlet Works	$1,782,000	
Other	$1,661,000	
Construction Cost Subtotal #1		***$29,874,000***
Engineering Costs = CCS#1 x 10%	$2,987,000	
Subtotal #2		***$32,862,000***
Contigency = Subtotal #2 x 15%	$4,929,257	
Construction Cost Total		***$37,791,000***
Project Cost Total		***$43,279,000***
Less Level II/Phase III Costs [1]	$4,182,000	
Project Cost Used in Ability to Pay Analysis		***$39,097,000***

[1] Preparation of Final Designs and Specifications; and Permitting

Table 5.6-3
Site 18 Conceptual Cost Estimate

Cost Item	Cost Estimate	
Preparation of Final Designs and Specifications	$6,751,000	
Permitting	$810,000	
Mitigation	$121,000	
Legal Fees	$270,000	
Acquisition of Access and Rights of Way	$1,620,000	
Non-Construction Cost Total		***$9,572,000***
Project Components		
Mobilization	$3,064,000	
Dam	$42,270,000	
Spillway(s)	$4,687,000	
Outlet Works	$2,359,000	
Other	$1,627,000	
Construction Cost Subtotal #1		***$54,007,000***
Engineering Costs = CCS#1 x 10%	$5,401,000	
Subtotal #2		***$59,408,000***
Contigency = Subtotal #2 x 15%	$8,911,223	
Construction Cost Total		***$68,319,000***
Project Cost Total		***$77,892,000***
Less Level II/Phase III Costs [1]	$7,561,000	
Project Cost Used in Ability to Pay Analysis		***$70,331,000***

[1] Preparation of Final Designs and Specifications; and Permitting

Table 5.6-4
Alternative Projects Cost Summary

Site	Storage Capacity (ac-ft)	Total Project Cost	Cost/Acre-ft Storage
1	9,000	$14,900,000	$1,656
2	25,000	$43,300,000	$1,732
5	25,000	$44,600,000	$1,784
8	9,500	$25,900,000	$2,726
17	25,000	$68,000,000	$2,720
18	25,000	$79,000,000	$3,160

Table 5.7-1

Summary of Maximum Potential Benefit of Project Alternatives

Alternative	Reservoir Capacity (ac-ft)	In Stream Total Available Dry Year (ac-ft)	"Dry Year" Irrigation Shortage Below Alternative Site (acre-ft)	Assumed Annual Reservoir Yield – Dry Year (ac-ft)	In Stream Total Available Normal Year (ac-ft)	"Normal Year" Irrigation Shortage Below Alternative Site (acre-ft)	Assumed Annual Reservoir Yield – Normal Year (ac-ft)	Maximum Potential Annual Benefits From Supplemental Irrigation Only ($ per year)	Maximum Potential Present Value of All Direct Irrigation Benefits ($)	Maximum Potential Value of Direct and Indirect Irrigation Benefits ($)
Site 1	9,000	2,960	10,200	5,500	19,750	3,850	3,850	83,140	1,786,000	6,001,000
Site 2	25,000	2,960	10,200	10,200	19,750	3,850	3,850	106,050	2,278,000	7,654,000
Site 18	25,000	2,960	10,200	10,200	19,750	3,850	3,850	106,050	2,278,000	7,654,000

Note: Analysis assumes no new area placed under irrigation; interest rate of 4.0% assumed, for fifty years

Table 5.7-2A
Summary of Ability to Pay for Project Alternatives - 67 % Grant / 33% Loan

Alternative	Level III Project Cost ($ Millions)	Sponsor's Share of Project Costs ($ Millions)	Sponsor's Annual Payment ($)	Sponsor's Maximum Ability to Pay ($)	Sponsor's Percentage Ability to Pay (%)
Site 1	13.5	4.45	207,105	41,570	20.1
Site 2	39.1	12.90	600,591	53,025	8.8
Site 18	70.3	23.21	1,080,394	53,025	4.9

Table 5.7-2B
Summary of Ability to Pay for Project Alternatives - 75 % Grant / 25% Loan

Alternative	Level III Project Cost ($ Millions)	Sponsor's Share of Project Costs ($ Millions)	Sponsor's Annual Payment ($)	Sponsor's Maximum Ability to Pay ($)	Sponsor's Percentage Ability to Pay (%)
Site 1	13.5	3.37	156,897	41,570	26.5
Site 2	39.1	9.77	454,993	53,025	11.7
Site 18	70.3	17.58	818,481	53,025	6.5

Table 5.7-2C
Summary of Ability to Pay for Project Alternatives - 90 % Grant / 10% Loan

Alternative	Level III Project Cost ($ Millions)	Sponsor's Share of Project Costs ($ Millions)	Sponsor's Annual Payment ($)	Sponsor's Maximum Ability to Pay ($)	Sponsor's Percentage Ability to Pay (%)
Site 1	13.5	1.35	62,759	41,570	66.2
Site 2	39.1	3.91	181,997	53,025	29.1
Site 18	70.3	7.03	327,392	53,025	16.2

Table 5.7.3

Annual Yield Cost by Alternative Storage Site

Alternative	Assumed Yield[1] (ac-ft)	Annual Cost Per Acre-Foot Yield		
		With 67 Percent Grant ($/ac-ft)	With 75 Percent Grant ($/ac-ft)	With 90 Percent Grant ($/ac-ft)
Site 1	5,500	$38	$29	$11
Site 2	10,200	$59	$45	$18
Site 18	10,200	$106	$80	$32

[1] The "assumed yield" is the estimated dry-year yield as presented in Table 5.7.2-1.

List of Figures

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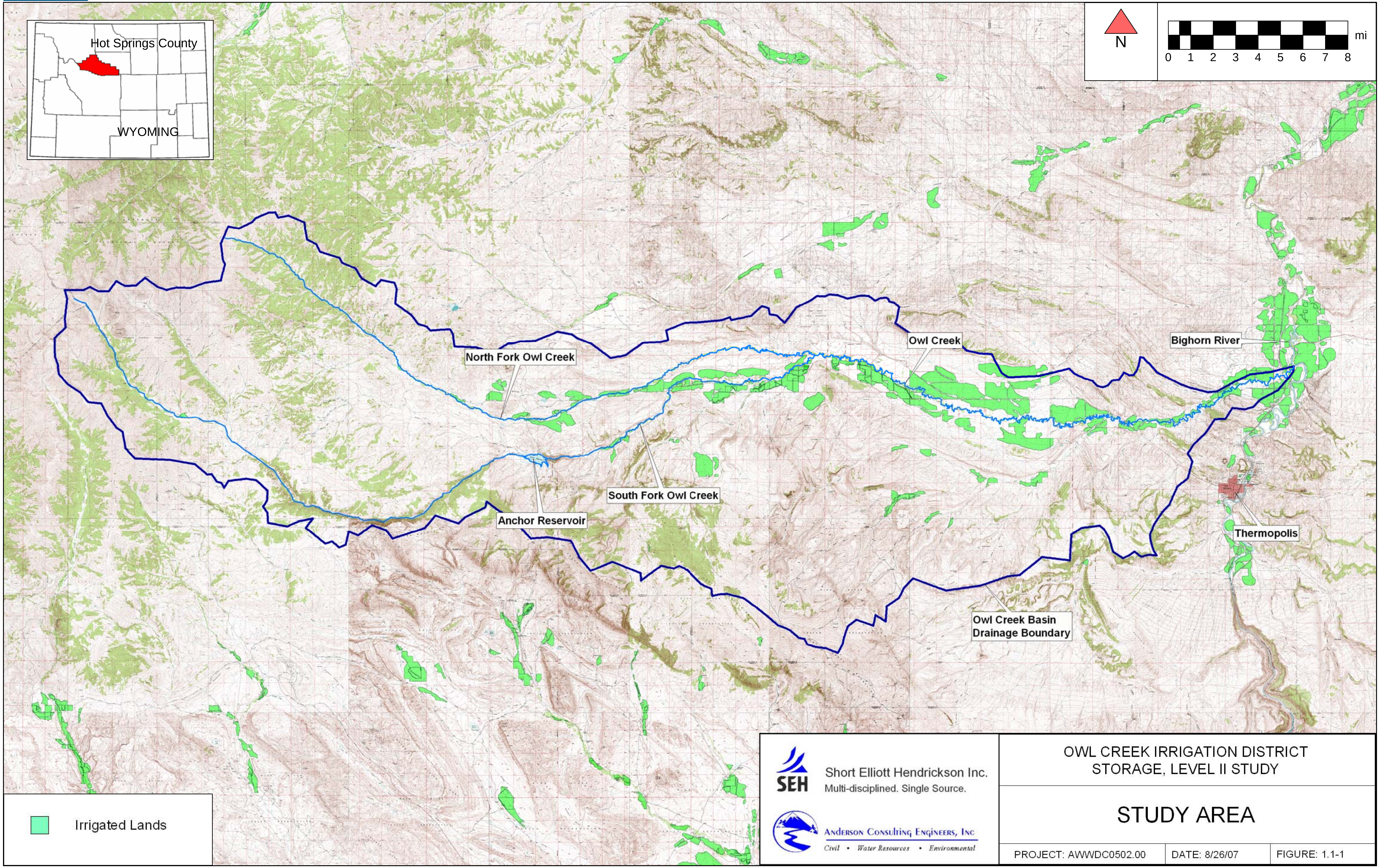

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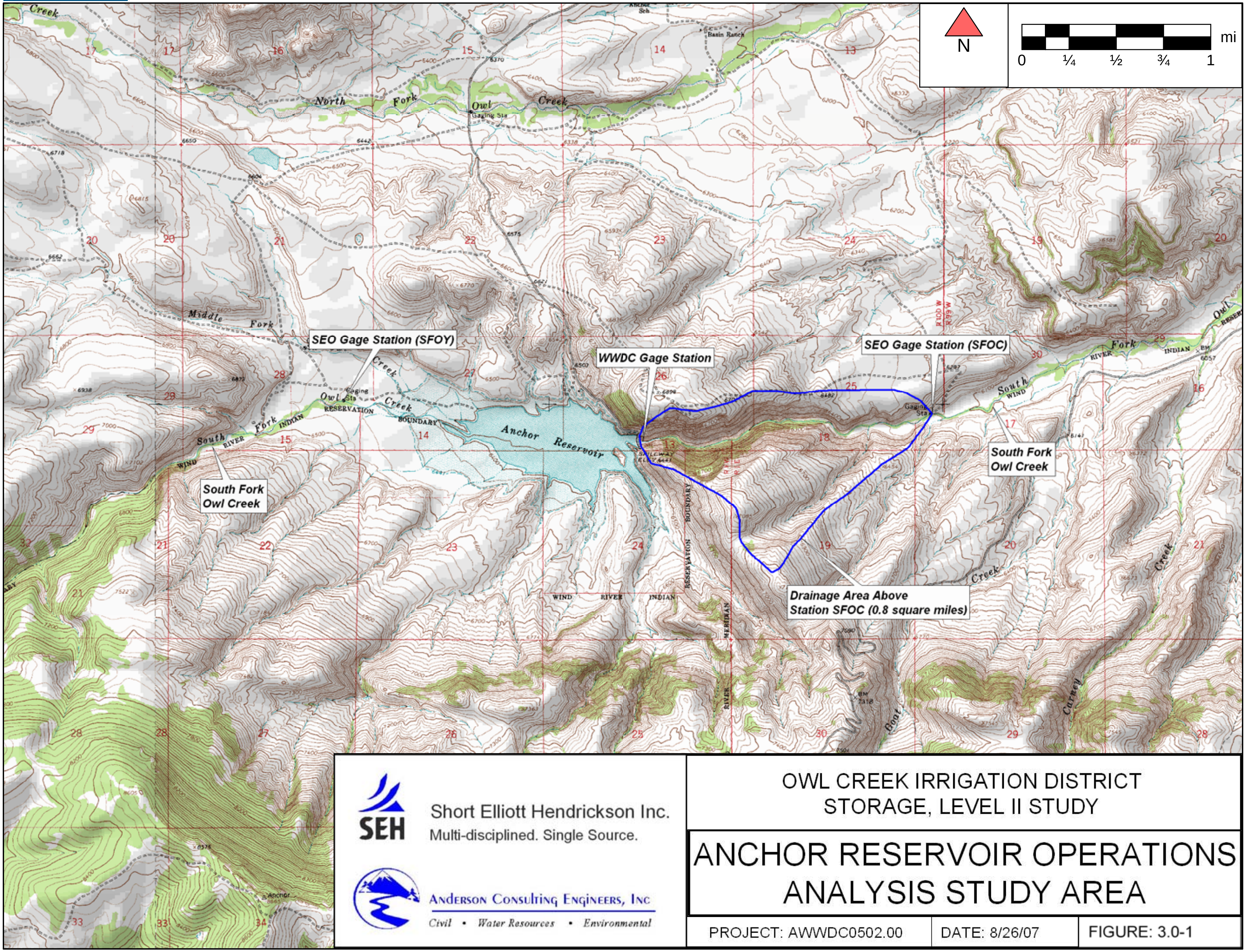

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ANCHOR RESERVOIR OPERATIONS ANALYSIS STUDY AREA

PROJECT: AWWDC0502.00	DATE: 8/26/07	FIGURE: 3.0-1

Figure 3.0-2
Annual Flows Above and Below Anchor Reservoir

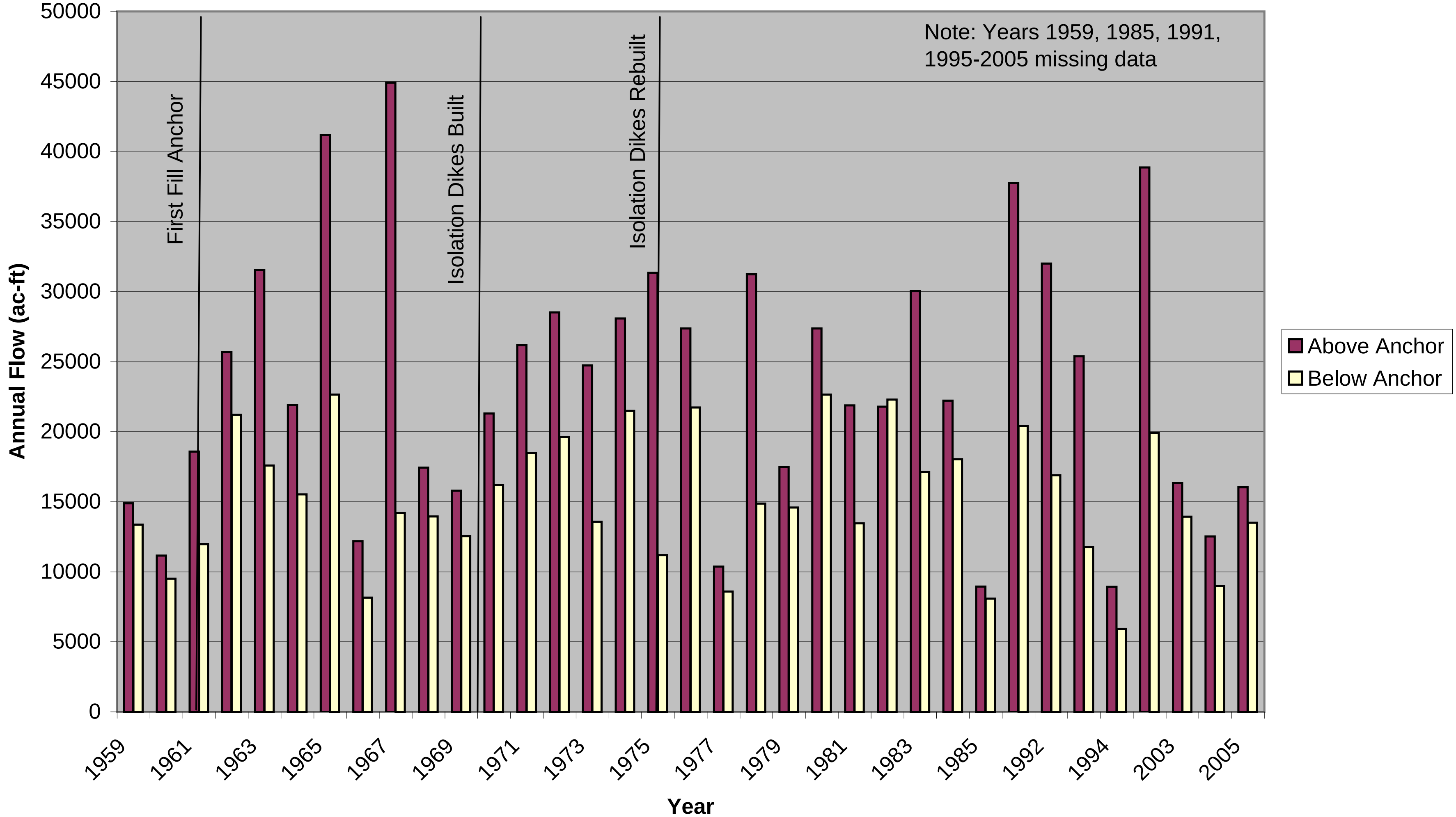


Figure 3.1-1

Aerial View of Anchor Dam, Looking Southeast, August 4, 1967.

Photo courtesy Willard Wilson.

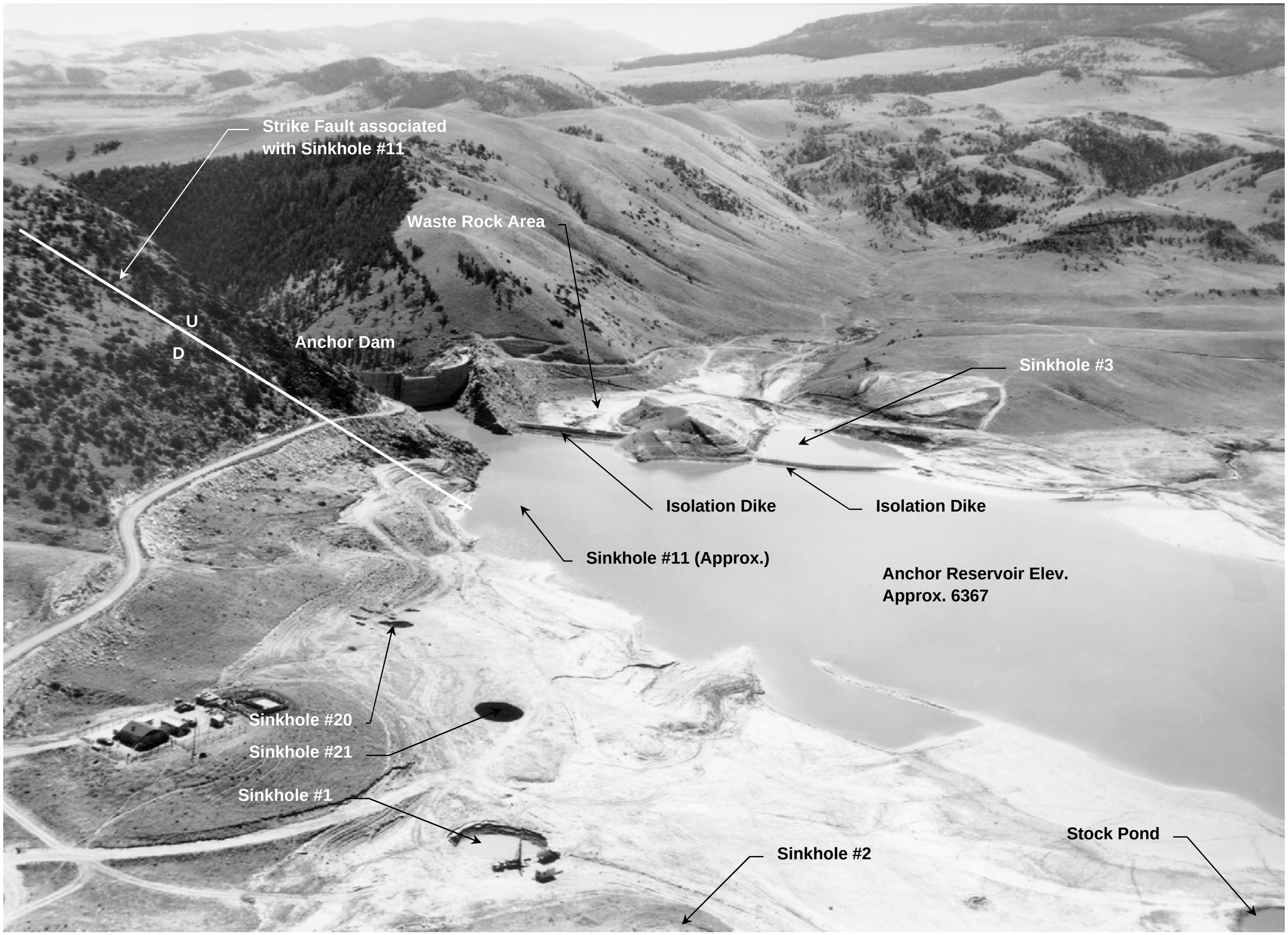


Figure 3.2-1

Gage Regression

WWDC Gage below Anchor Dam and SEO Gage (SFOC)

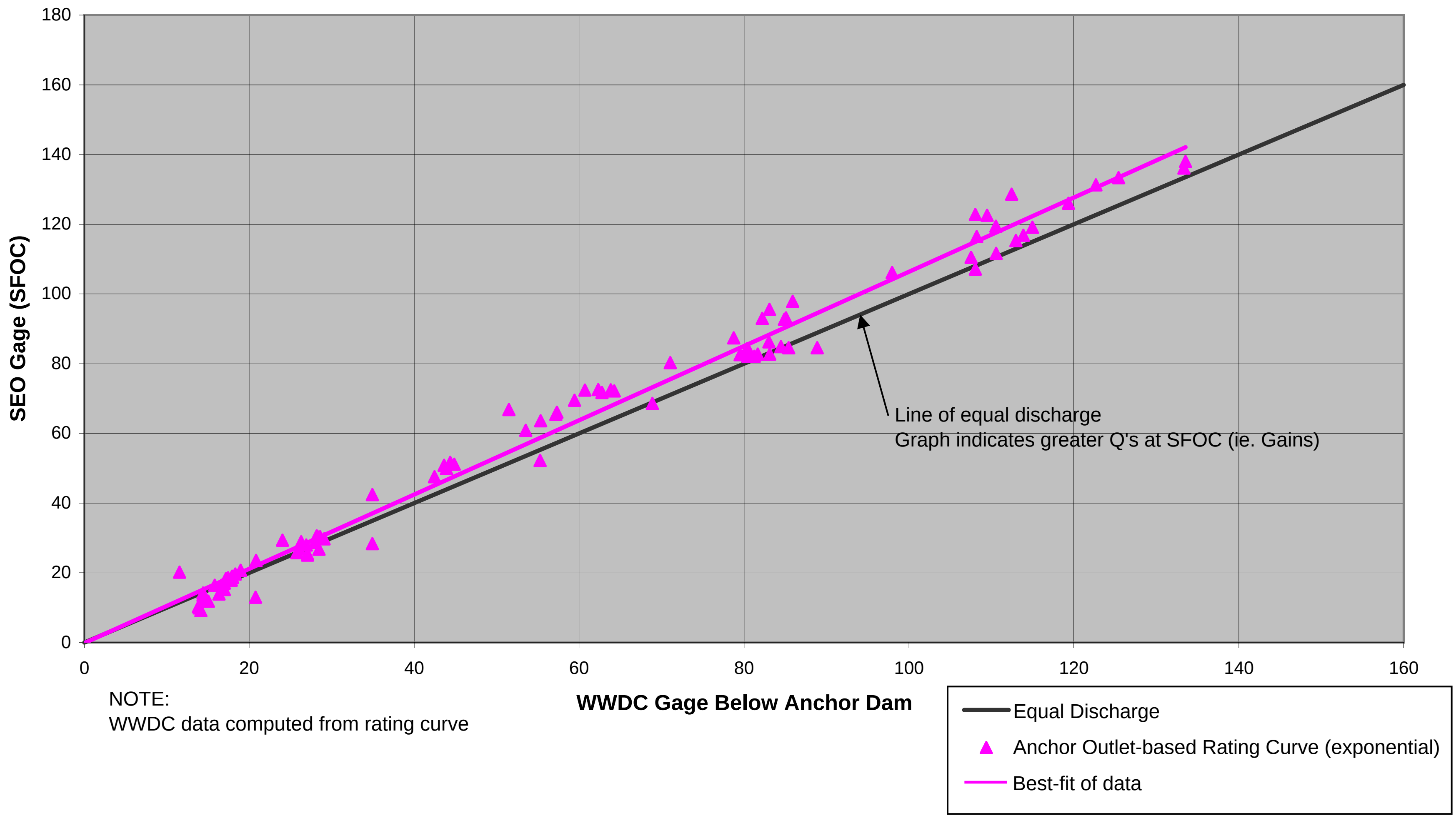


Figure 3.2-2
Residual Gain/Loss in Canyon vs. Reservoir Head

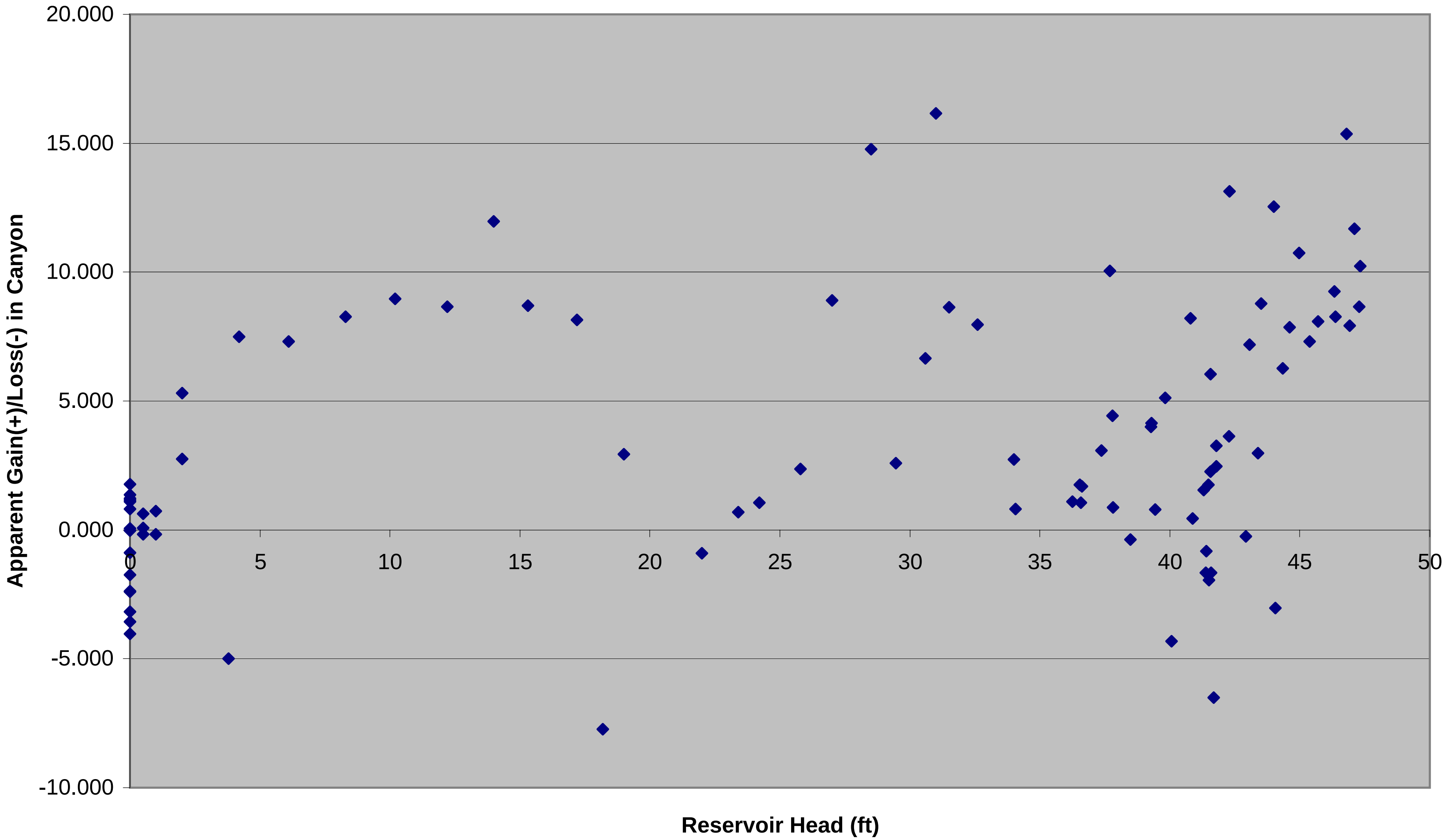


Figure 3.2-3
Anchor Reservoir Water Balance Model
Model Fit: Actual versus Predicted

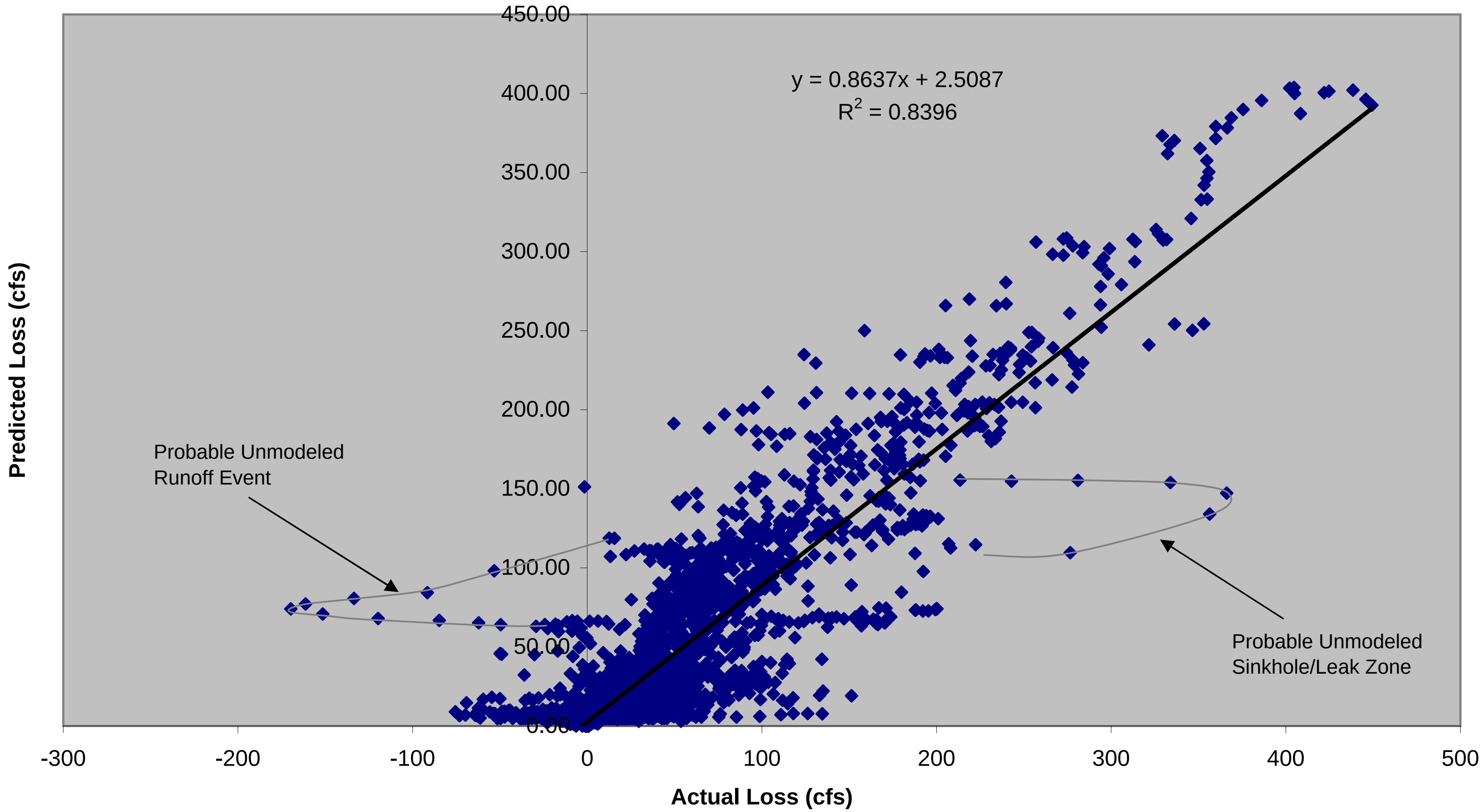
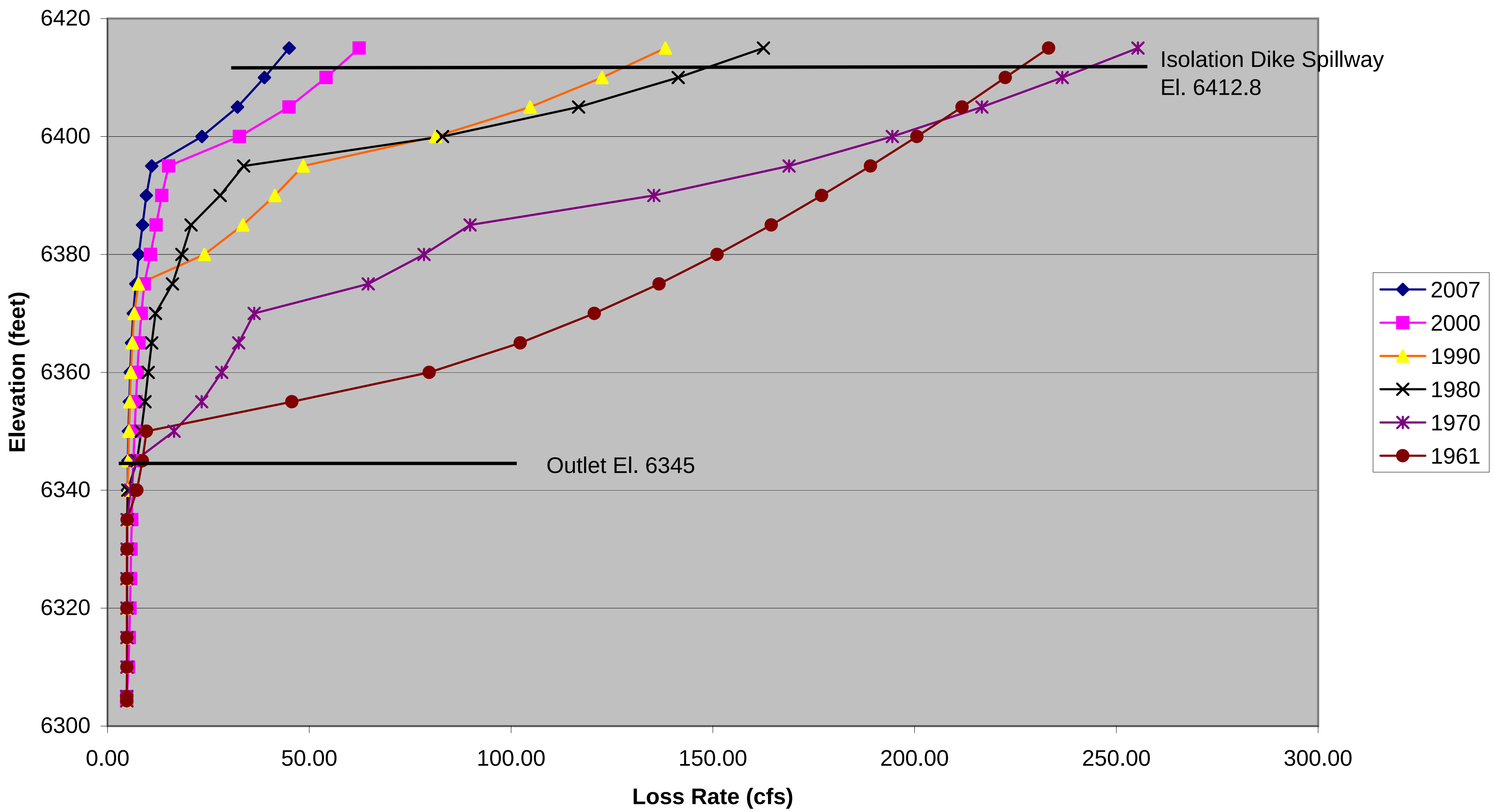


Figure 3.2-4

Anchor Reservoir Loss Rates over Time

(Sinkholes/Leak Zones + Reservoir Seepage + Base Losses)



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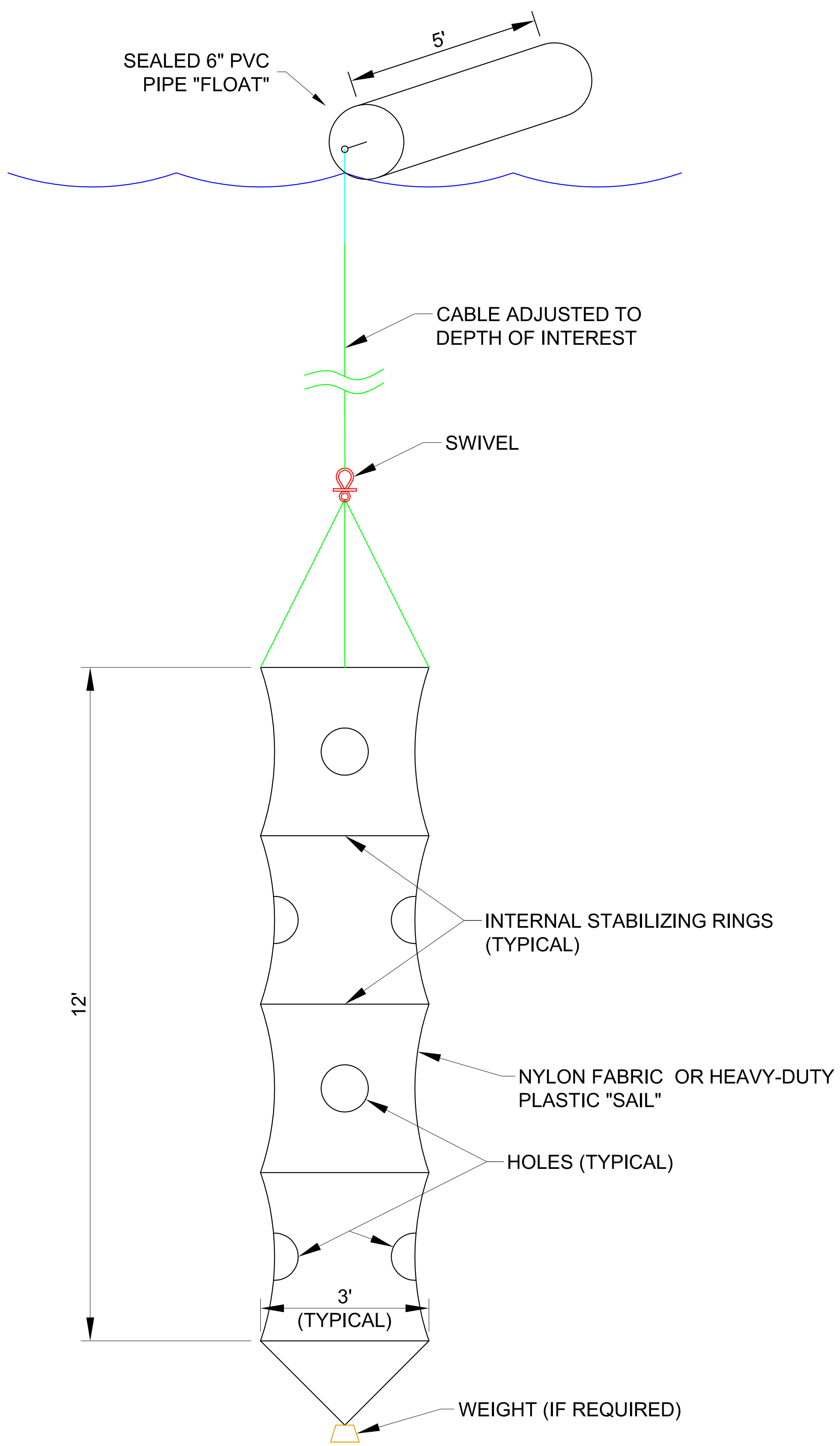
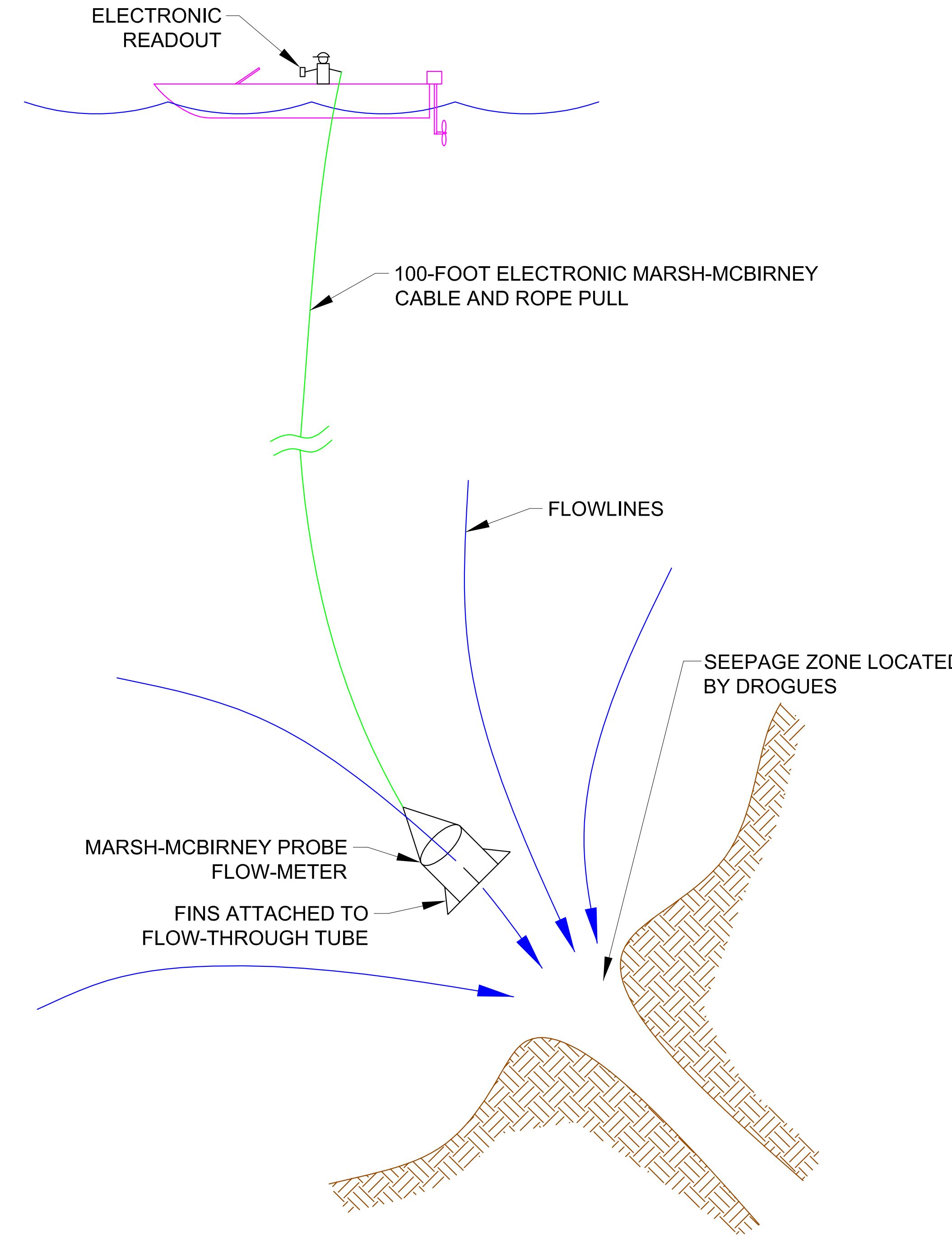
RESULTS OF FIELD RECONNAISSANCE OF ANCHOR RESERVOIR

PROJECT: AWWDC0502.00	DATE: 8/26/07	FIGURE: 3.2-5

005DWWDC-OC.DWG



"HOLEY SOCK" TRACKING DROGUE
PROTOTYPE DESIGN



DEEP-WATER FLOW METER

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"HOLEY SOCK" DROGUE AND
DEEP-WATER FLOW METER

PROJECT: AWWDC0502.00	DATE: 8/26/07	FIGURE: 3.2-6

Figure 3.2-6_Holey Sock Drogue.dwg

Figure 3.2-7: Drogues Prior to Deployment

Figure 3.2-8: Deployed Drogue

Figure 3.2-9: Drogue near "Rockfall Area" Looking NW from Anchor Dam Right Abutment

Figure 3.2-10: Site Reconnaissance – Feature 1

Figure 3.2-11: Site Reconnaissance – Feature 2

Figure 3.2-12: Site Reconnaissance – Feature 3

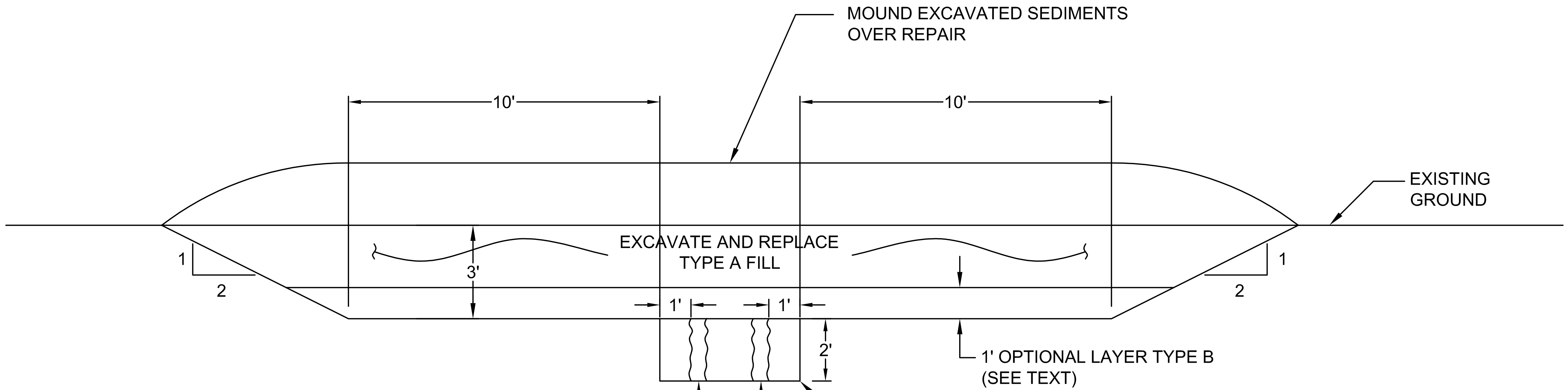
Figure 3.2-13: Site Reconnaissance – Feature 4

Figure 3.2-14: Site Reconnaissance – Feature 5

Figure 3.2-15: Site Reconnaissance – Feature 6

Figure 3.2-16: Site Reconnaissance – Feature 6

CONCEPT A

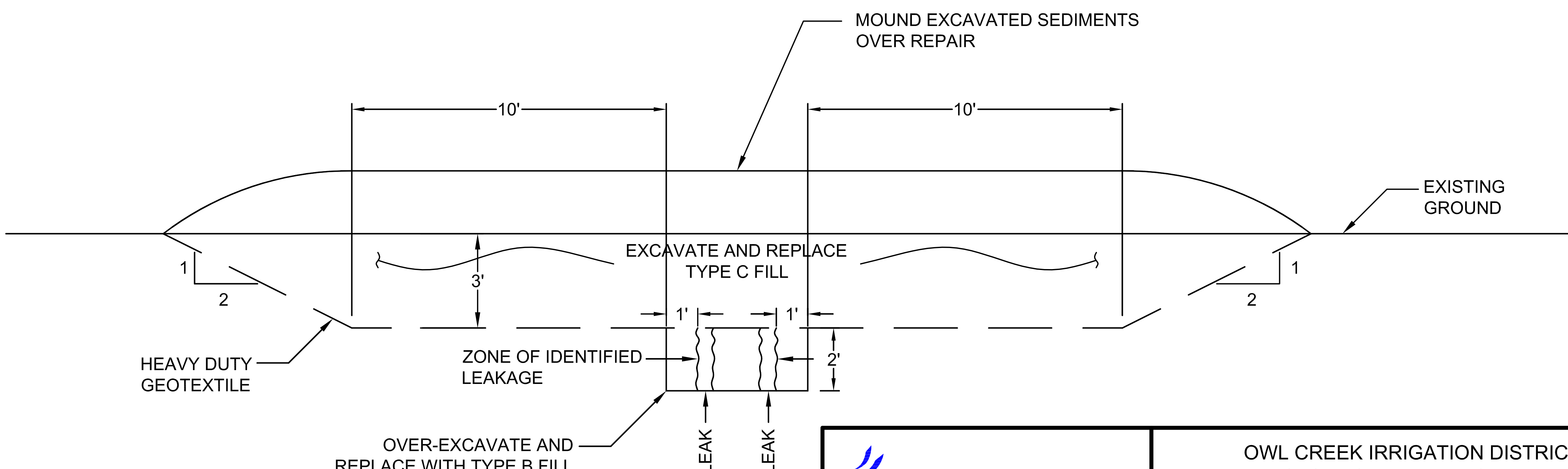


TYPE A FILL: WELL-GRADED SOIL FOUND ON STEEPER SLOPES CONTAINING APPROXIMATELY EQUAL AMOUNTS OF SAND SIZE AND SMALLER PARTICLES AND LARGER THAN SAND-SIZED PARTICLES (UP TO 6" SIZE)

TYPE B COBBLES: LOCALLY AVAILABLE 1" TO 6" COBBLES WITH LESS THAN 20% BY WEIGHT SUB 1-INCH PARTICLES

TYPE C: FINE-GRAINED RESERVOIR BOTTOM SEDIMENTS

CONCEPT B



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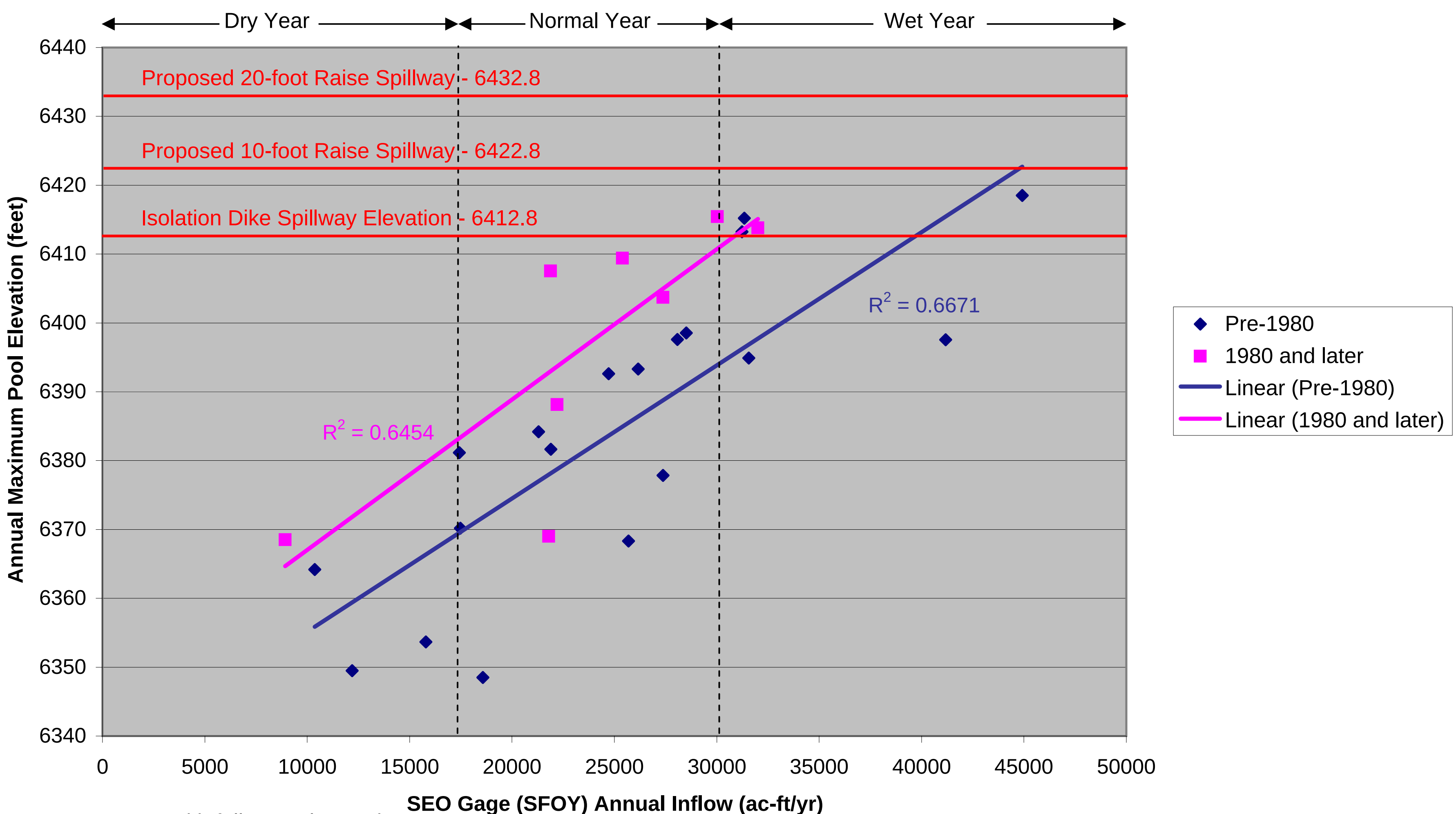
SMALL SCALE ANCHOR RESERVOIR
REPAIR CONCEPTS

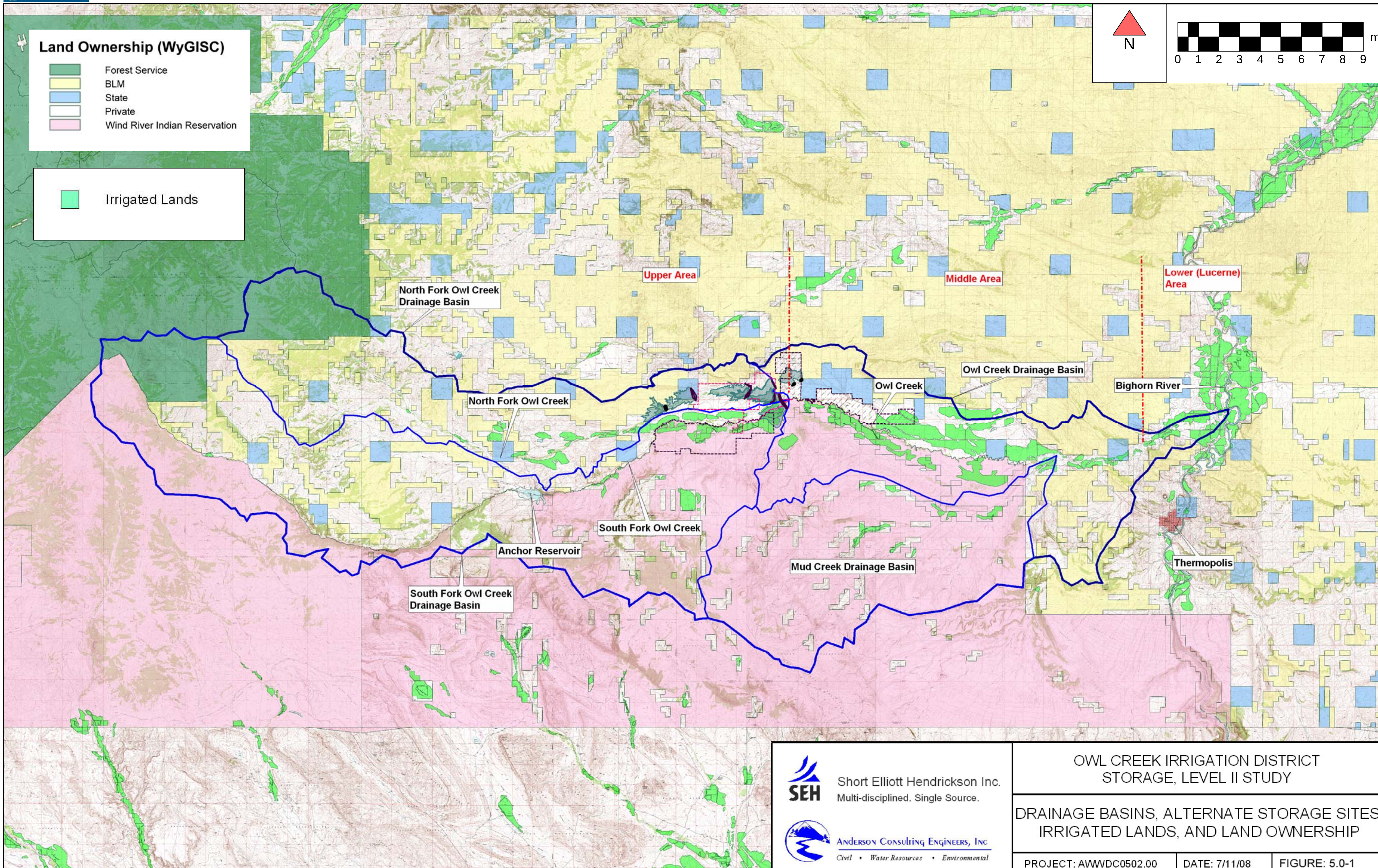
PROJECT: AWWDC0502.00	DATE: 8/26/07	FIGURE: 3.2-17

Figure 4.2-1

Anchor Reservoir Maximum Pool Elevations for Dry, Normal and Wet Years*



* For years with full annual record


Land Ownership (WyGISC)
Forest Service
BLM
State
Private
Wind River Indian Reservation
Irrigated Lands
N
0 1 2 3 4 5 6 7 8 9
mi
Upper Area
Middle Area
Lower (Lucerne) Area
North Fork Owl Creek Drainage Basin
North Fork Owl Creek
Owl Creek Drainage Basin
Owl Creek
Bighorn River
South Fork Owl Creek
Anchor Reservoir
South Fork Owl Creek Drainage Basin
Mud Creek Drainage Basin
Thermopolis
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DRAINAGE BASINS, ALTERNATE STORAGE SITES, IRRIGATED LANDS, AND LAND OWNERSHIP
PROJECT: AWWDC0502.00
DATE: 7/11/08
FIGURE: 5.0-1

Figure 5.1-1
Diagram of Model Water Budget Computations

Outflow to
Downstream Node =

Inflow from upstream
+
Irrigation Returns
-
Diversions
+
Tributary Inflow
+/-
Imports/Exports
+/-
Reach Gains/Losses

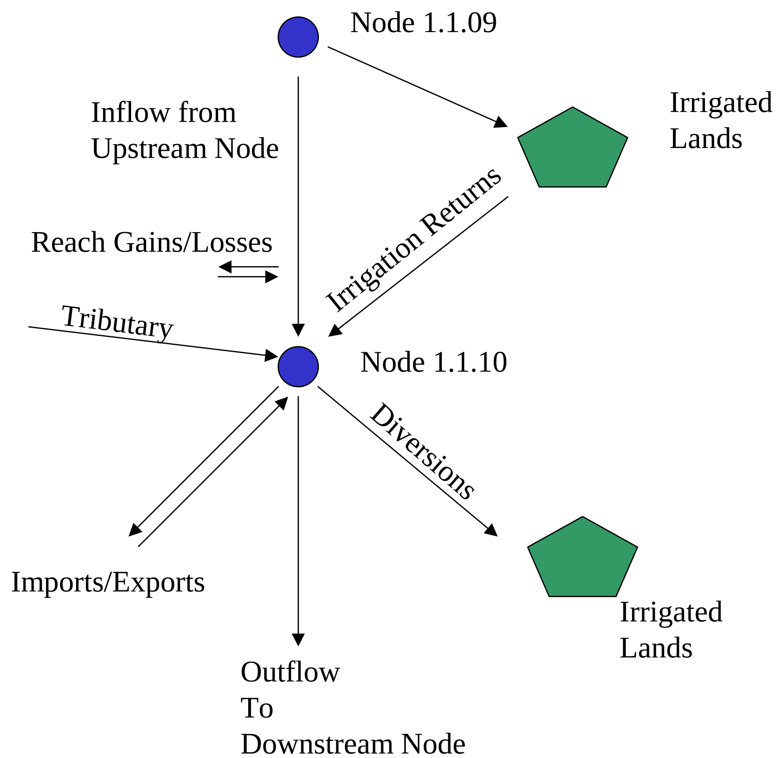
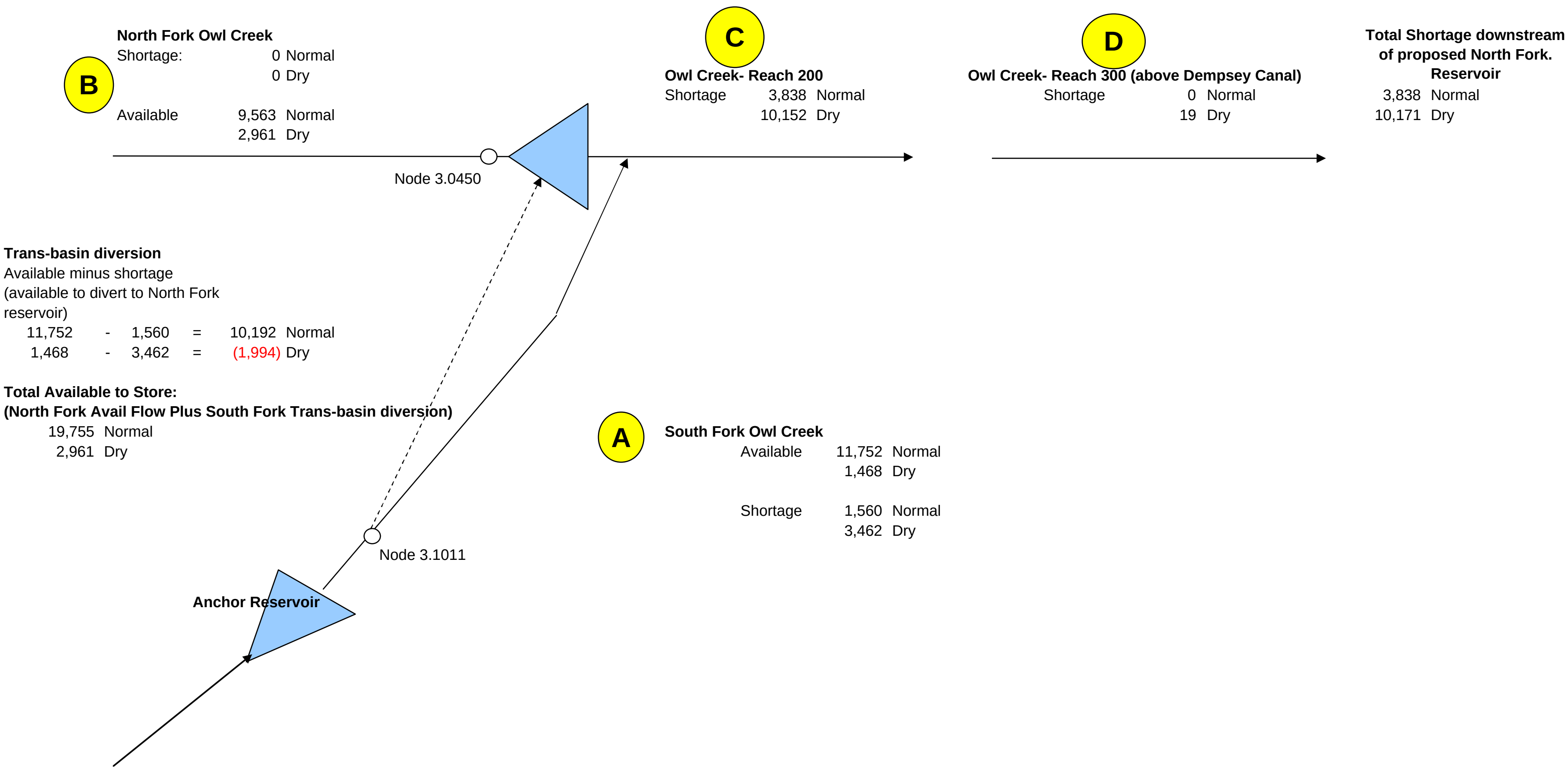


Figure 5.1-2
Results of Basin Plan Model Within the Study Area



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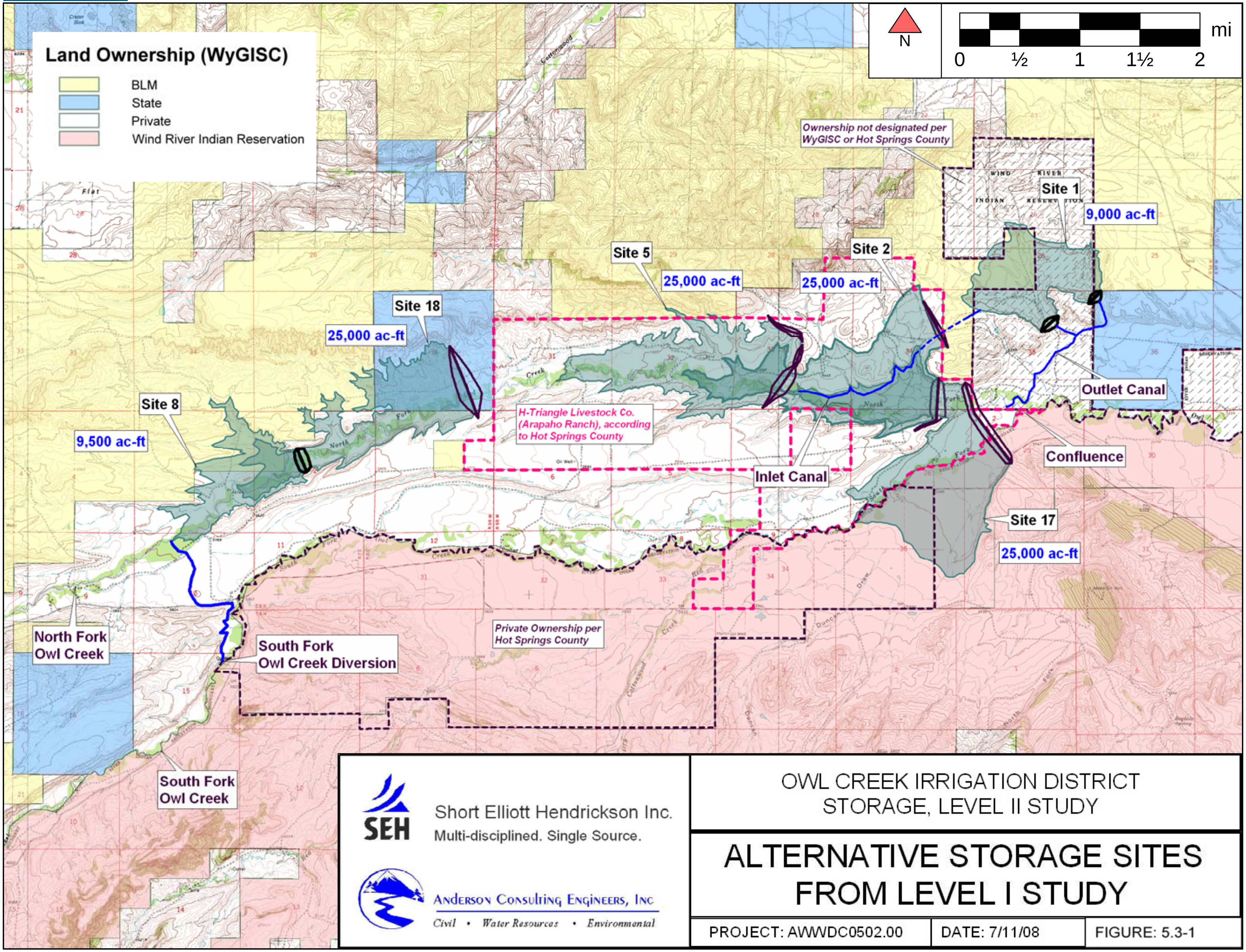

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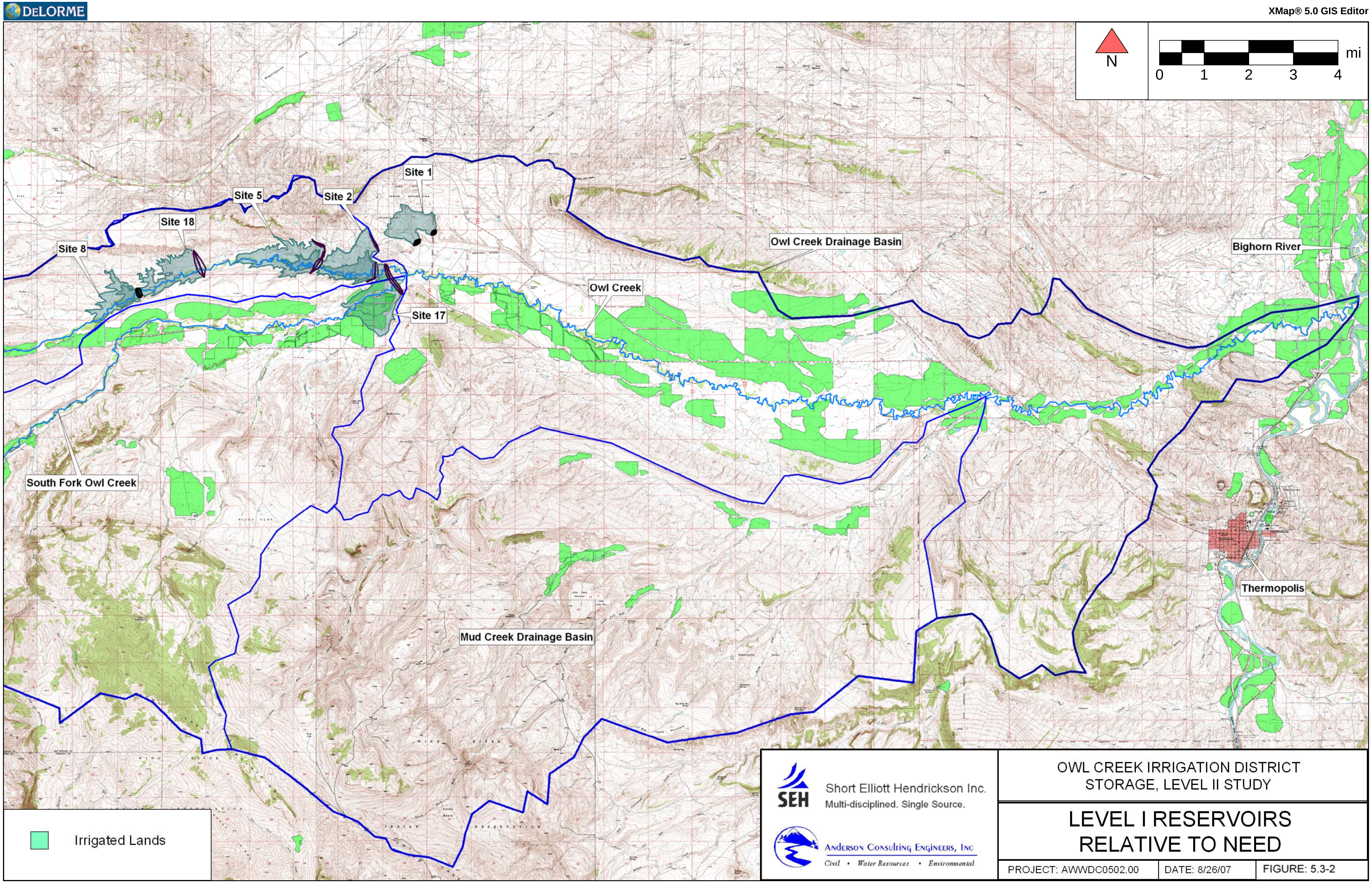

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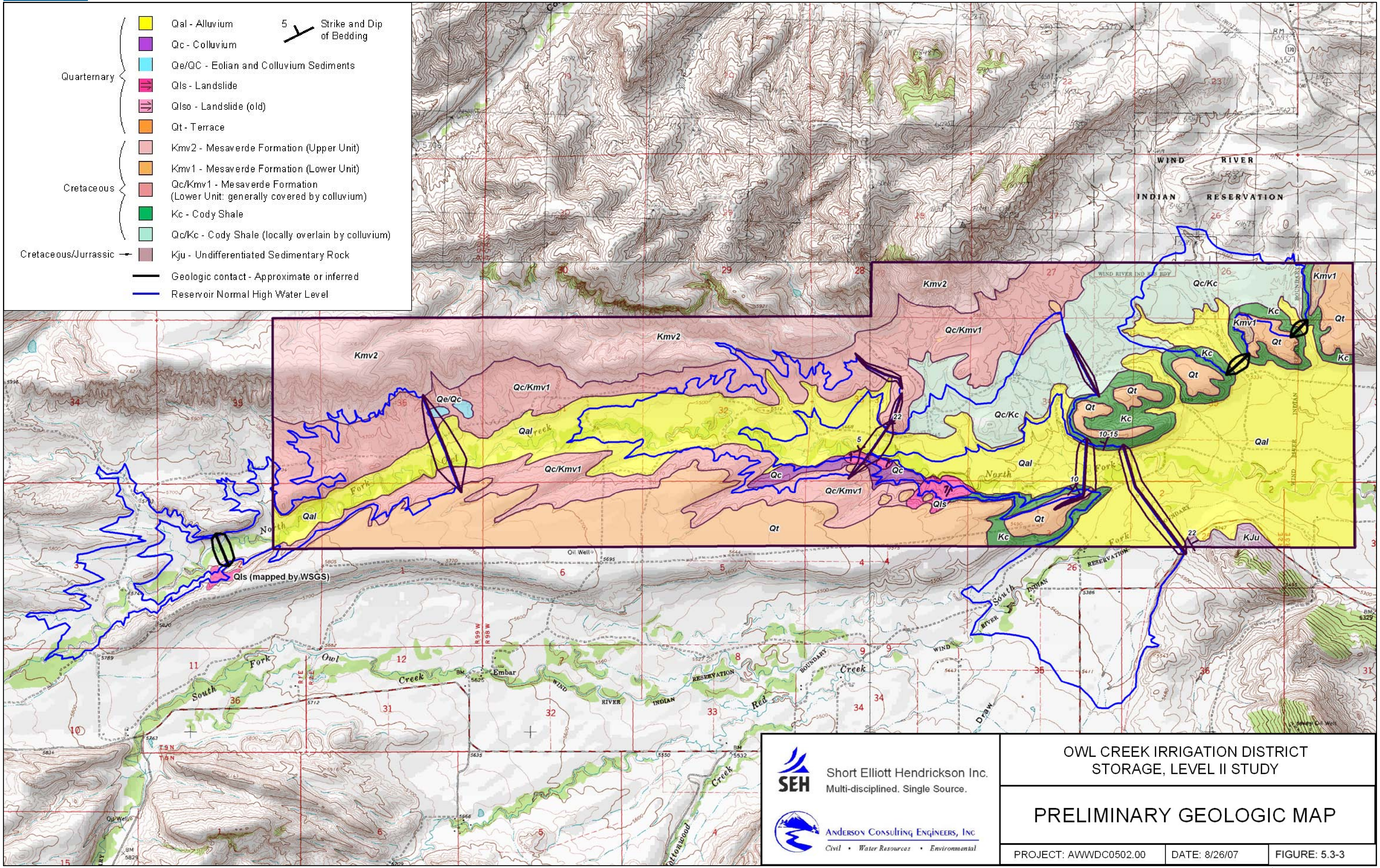
LEVEL I RESERVOIRS RELATIVE TO NEED

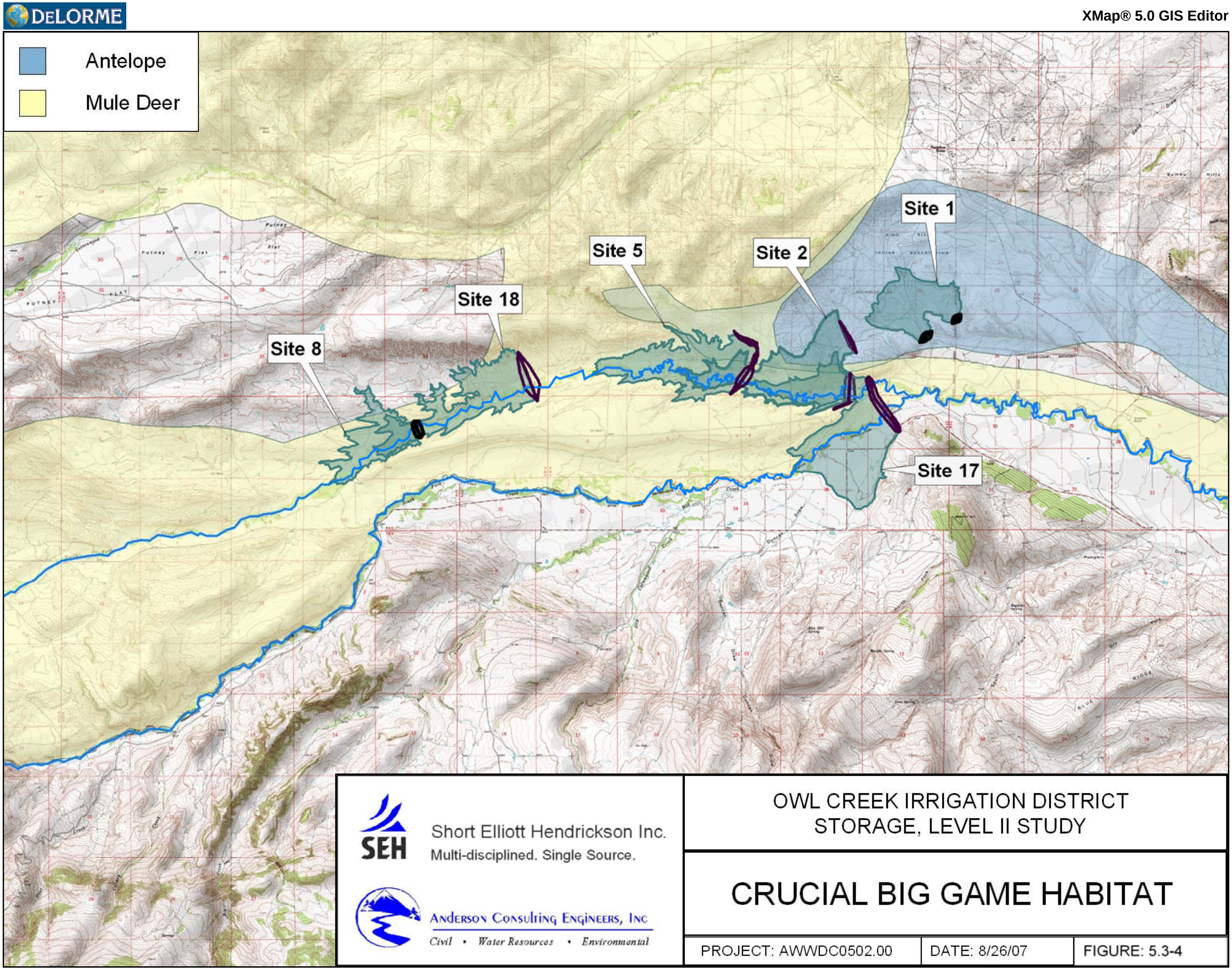
PROJECT: AWWDC0502.00 | DATE: 8/26/07 | FIGURE: 5.3-2



TN
MN (11.6°E)

Scale 1 : 34,375
1" = 2,864.6 ft
Data Zoom 12-6

XMap® 5.0 GIS Editor



TN
MN (11.7°E)

Scale 1 : 100,000

1" = 1.58 mi

Data Zoom 11-0

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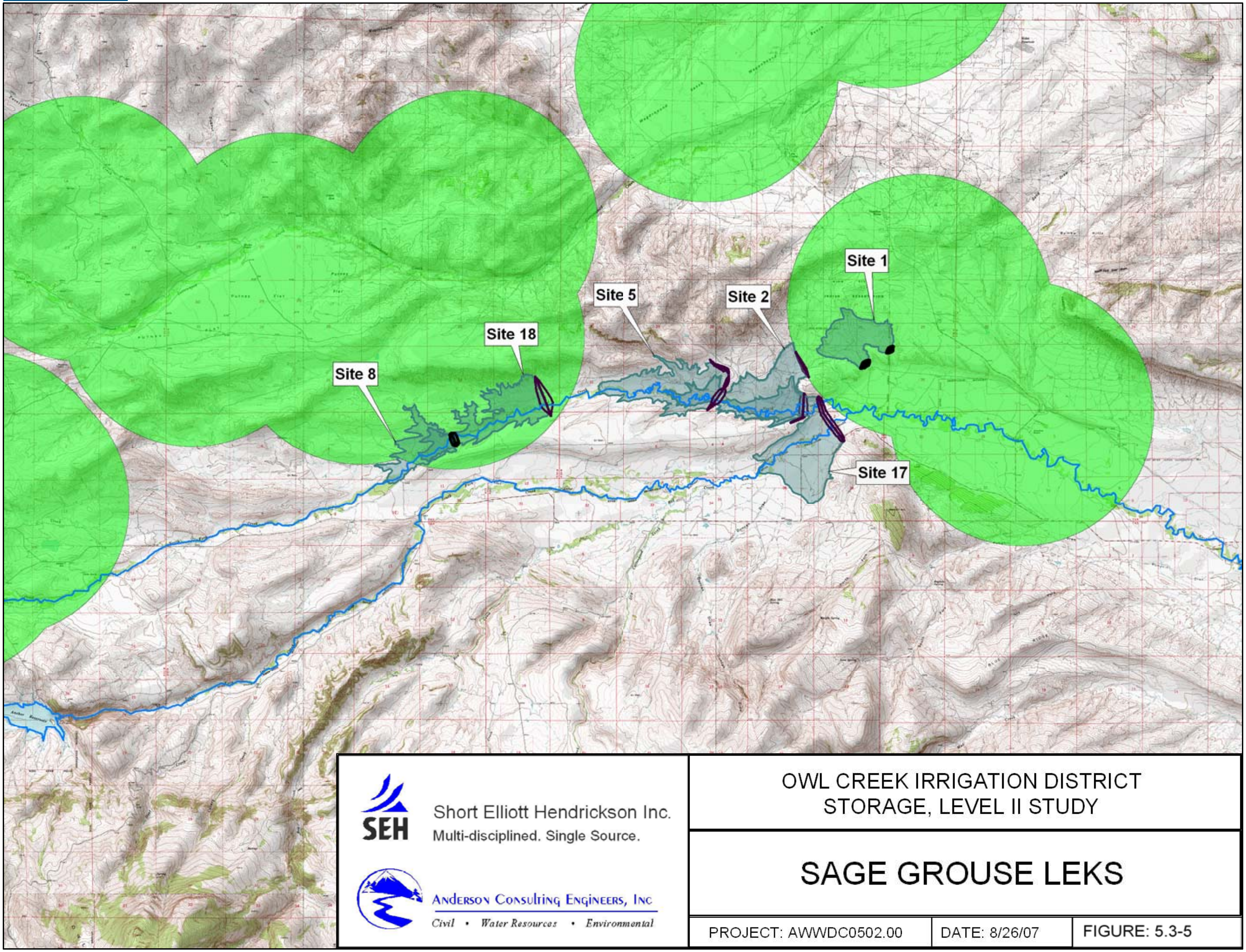

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SAGE GROUSE LEKS

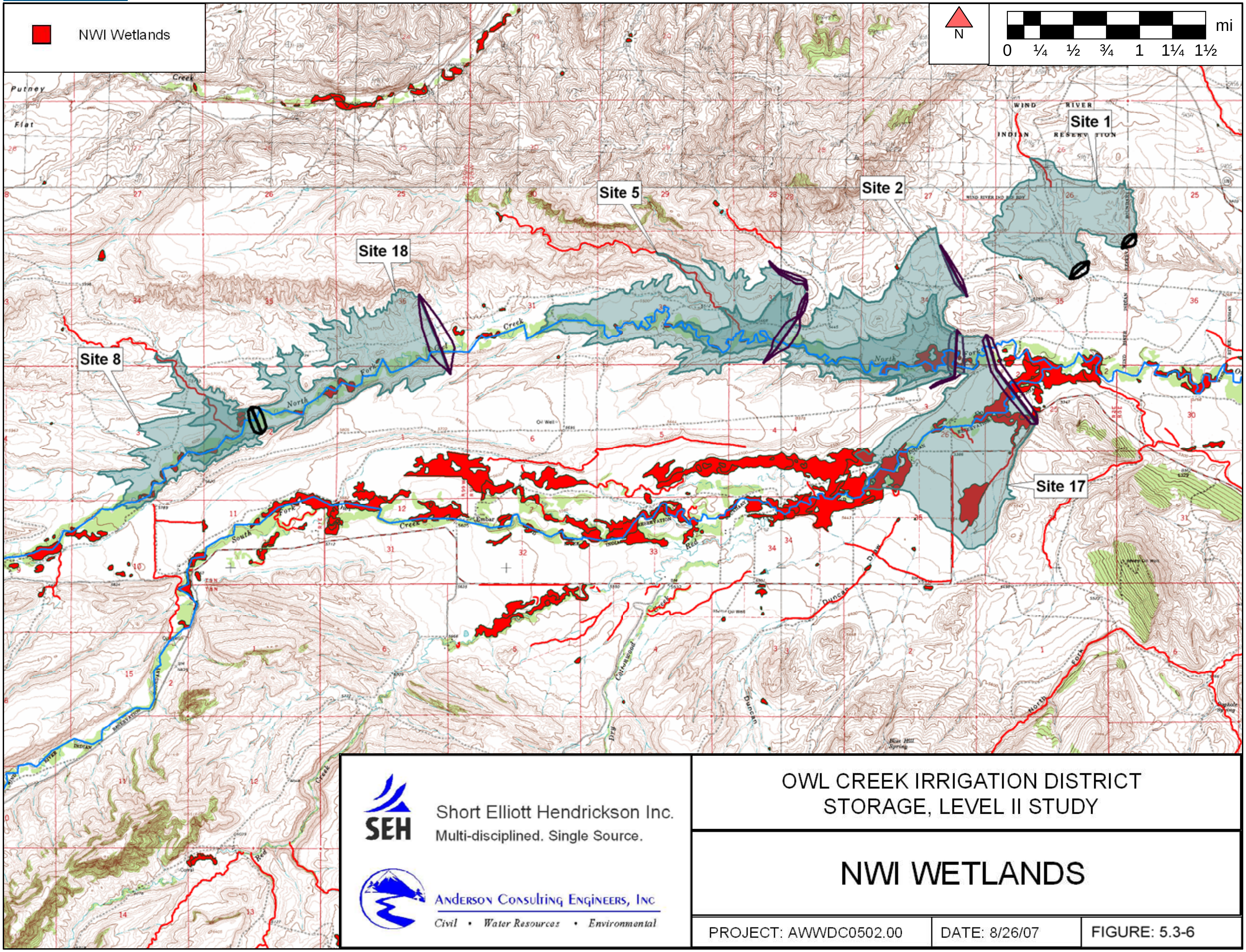
PROJECT: AWWDC0502.00	DATE: 8/26/07	FIGURE: 5.3-5

TN
MN (11.7°E)

Scale 1 : 125,000

1" = 1.97 mi   Data Zoom 11-0

XMap® 5.0 GIS Editor



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NWI WETLANDS

PROJECT: AWWDC0502.00	DATE: 8/26/07	FIGURE: 5.3-6

Figure 5.4-1 and 2_Site 18 Geo

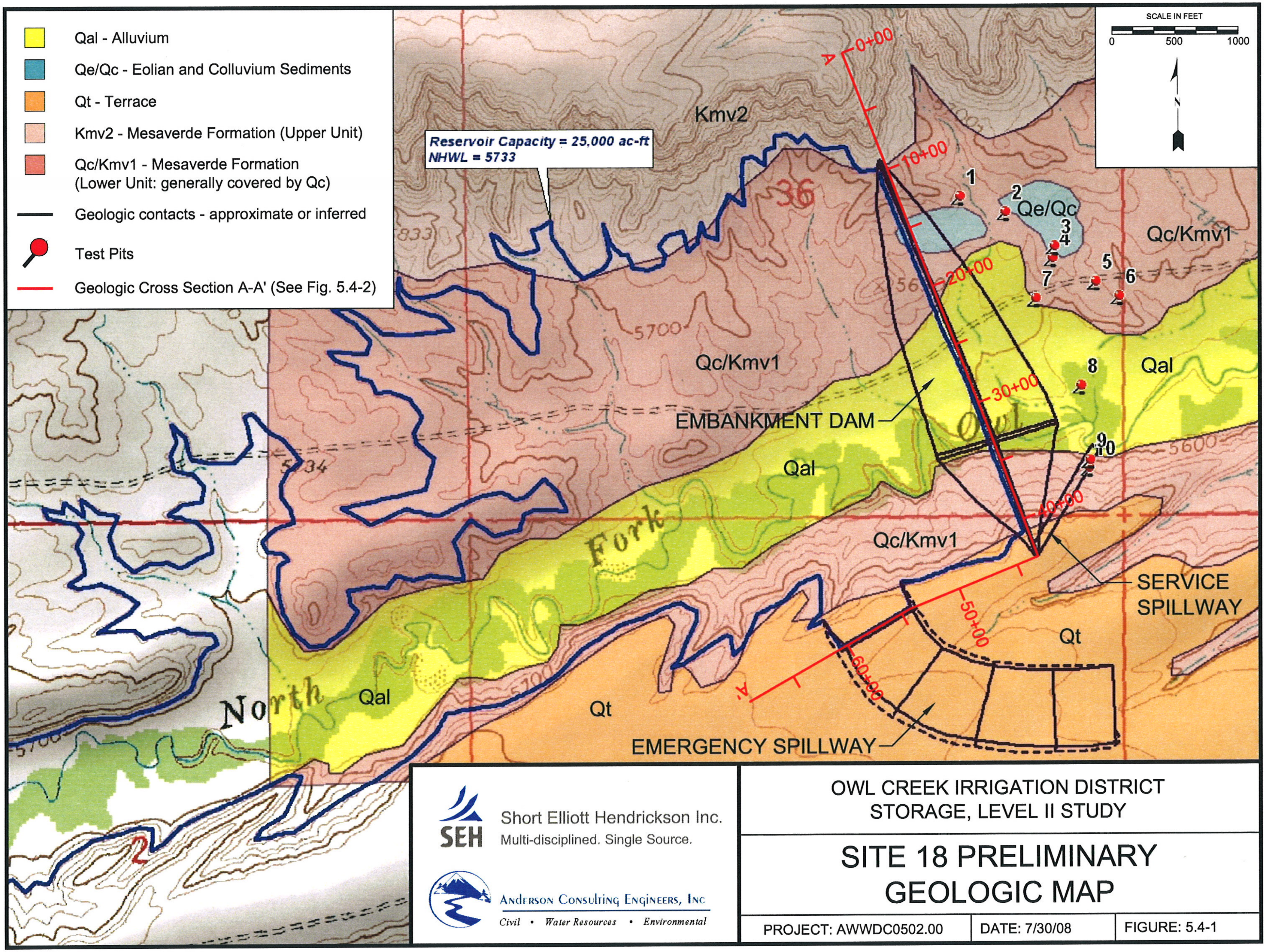
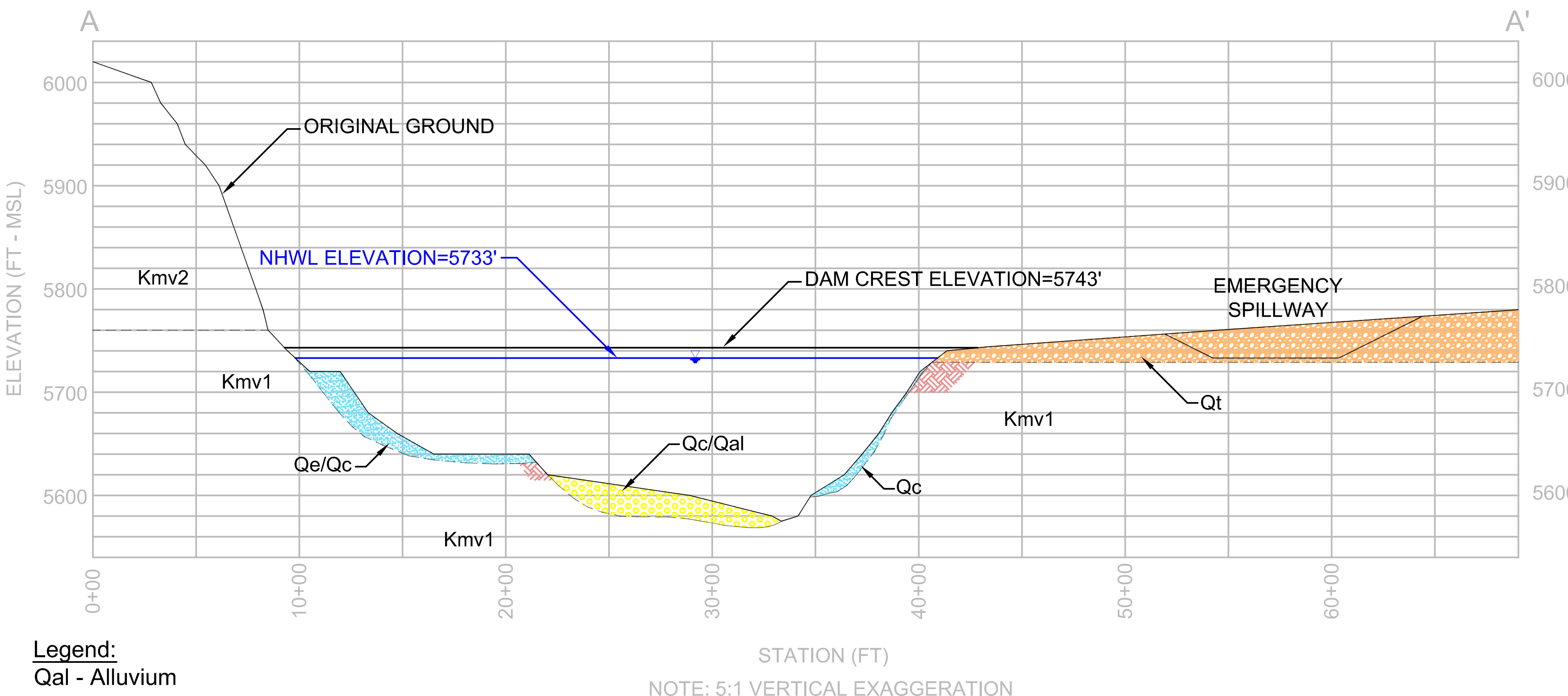


Figure 5.4-1 and 2_Site 18 Geo



Legend:

Qal - Alluvium

Qc - Colluvium

Qe/Qc - Eolian and Colluvium Sediments

Qt - Terrace

Kmv2 - Mesaverde Formation (Upper Unit)

Kmv1 - Mesaverde Formation (Lower Unit)

— — Geologic Contact - Approximate or inferred

Note: This is a preliminary interpretive geologic cross-section based on projection of limited shallow subsurface infromation. Actual conditions need to be confirmed by subsurface exploration (including drilling).

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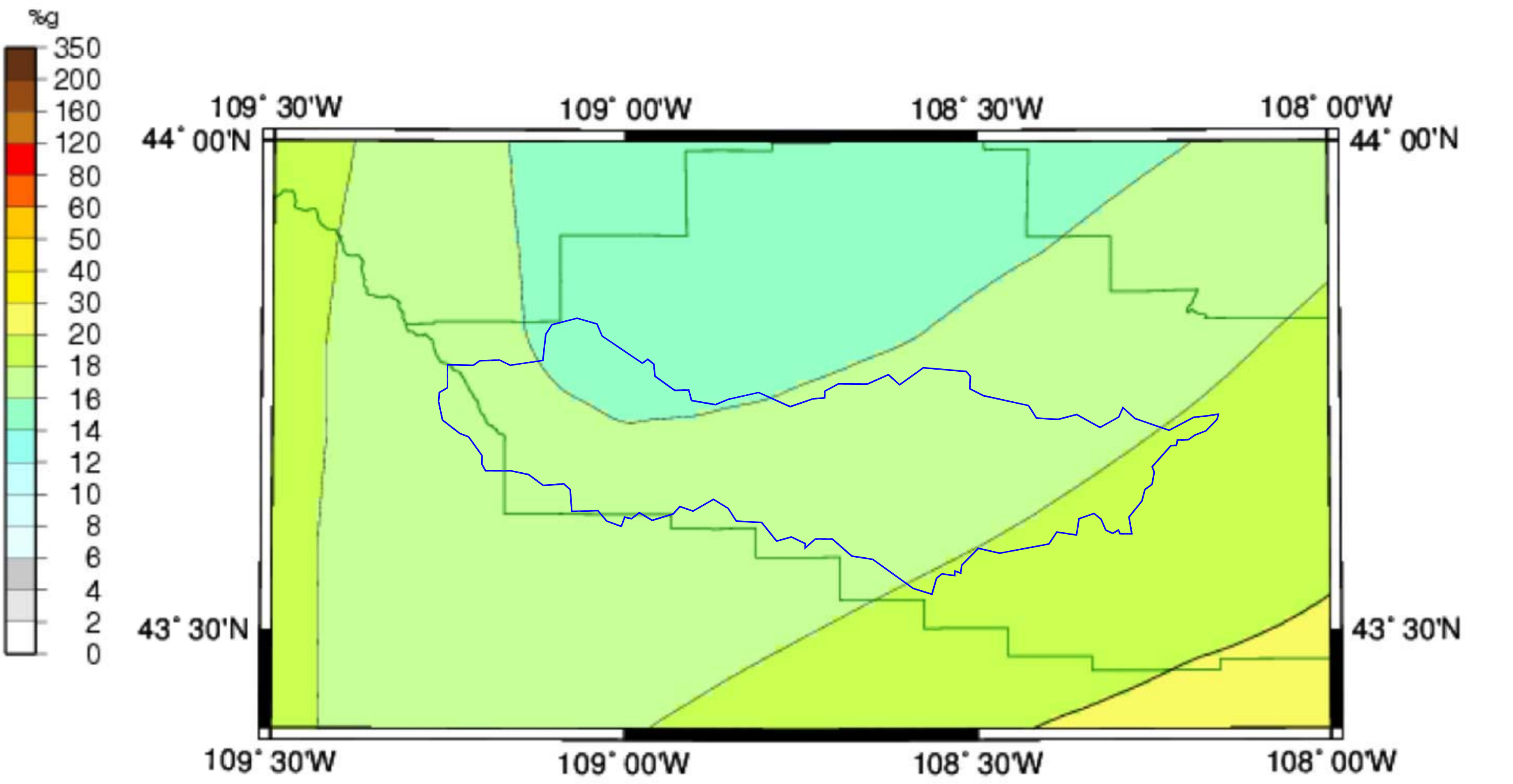
SITE 18 INTERPRETIVE GEOLOGIC CROSS-SECTION A-A'

PROJECT: AWWDC0502.00	DATE: 7/30/08	FIGURE: 5.4-2



Peak Acceleration (%g) with 2% Probability of Exceedance in 50 Years
site: NEHRP B-C boundary
U.S. Geological Survey
National Seismic Hazard Mapping Project

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	PROJECT: AWWDC0502.00 — DATE: 8/26/07 — FIGURE: 5.4-3

Figure 5.4-4: Site 1 – Looking South Towards Northwest Dam Area

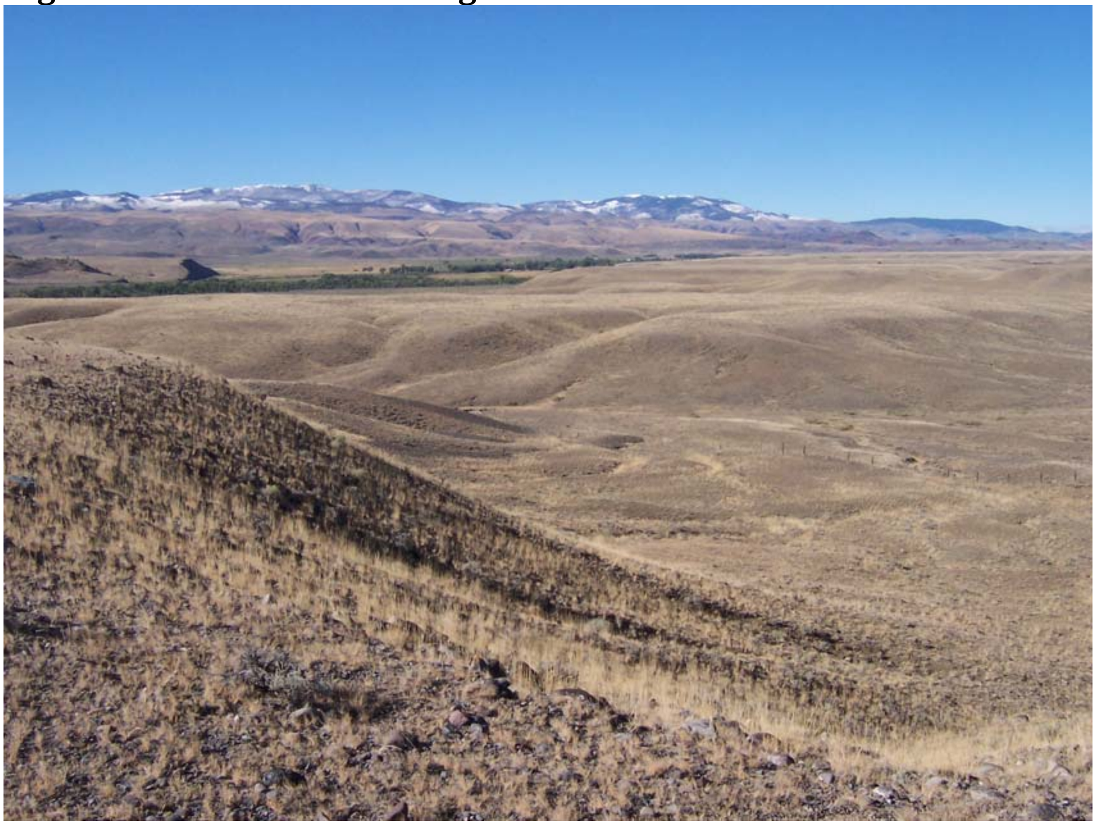
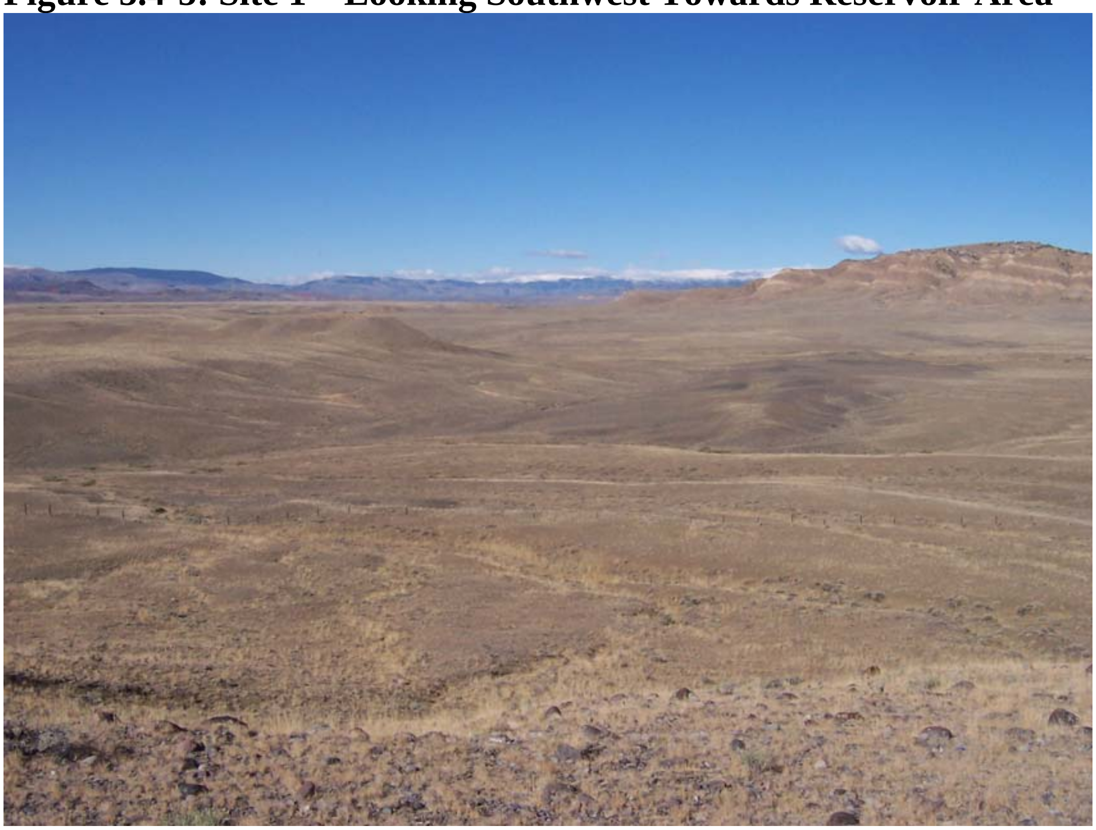
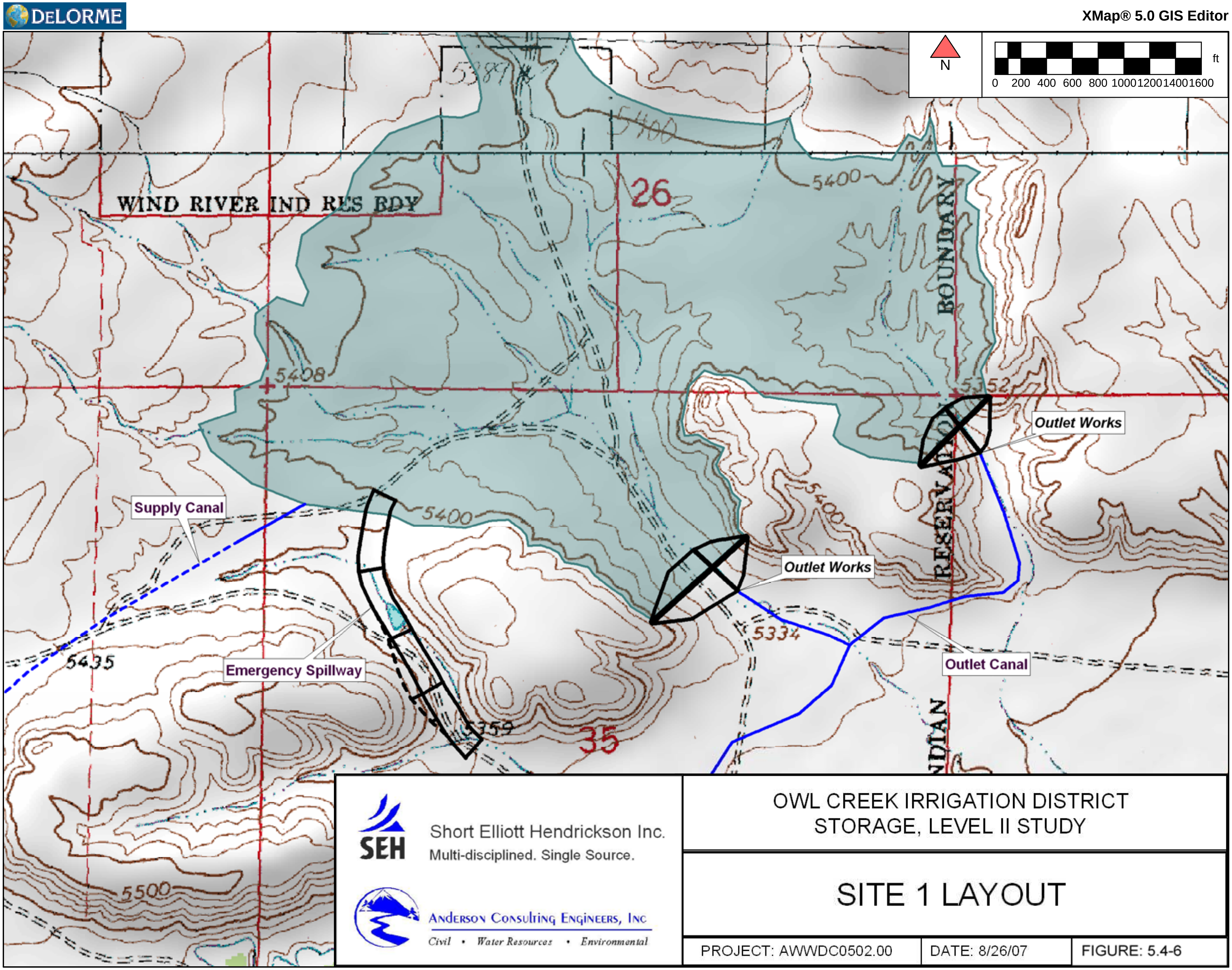


Figure 5.4-5: Site 1 – Looking Southwest Towards Reservoir Area





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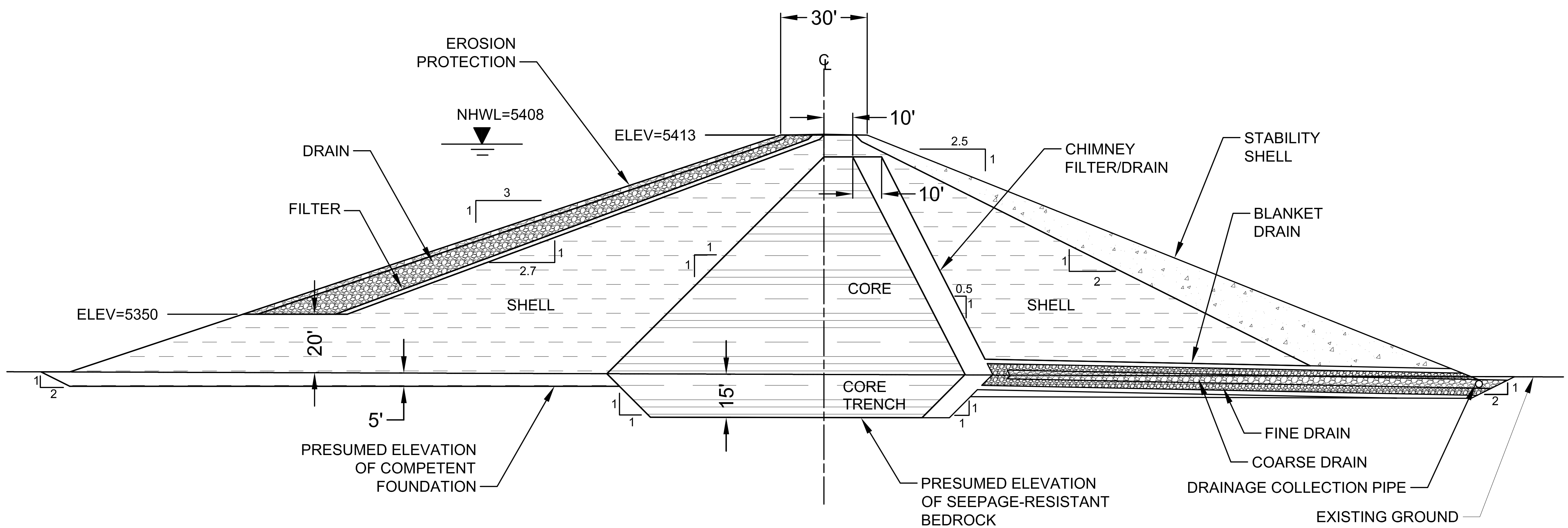
SITE 1 LAYOUT

PROJECT: AWWDC0502.00	DATE: 8/26/07	FIGURE: 5.4-6

SCALE IN FEET
0 20 40



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SITE 1 MAXIMUM SECTION (SOUTHWEST DAM)

PROJECT: AWWDC0502.00	DATE: 7/30/08	FIGURE: 5.4-7

Figure 5.4-8: Site 2 – Looking Northeast Towards Main (South) Dam Area

Figure 5.4-9: Site 2 – Looking Northwest Towards Reservoir Area

DeLORME

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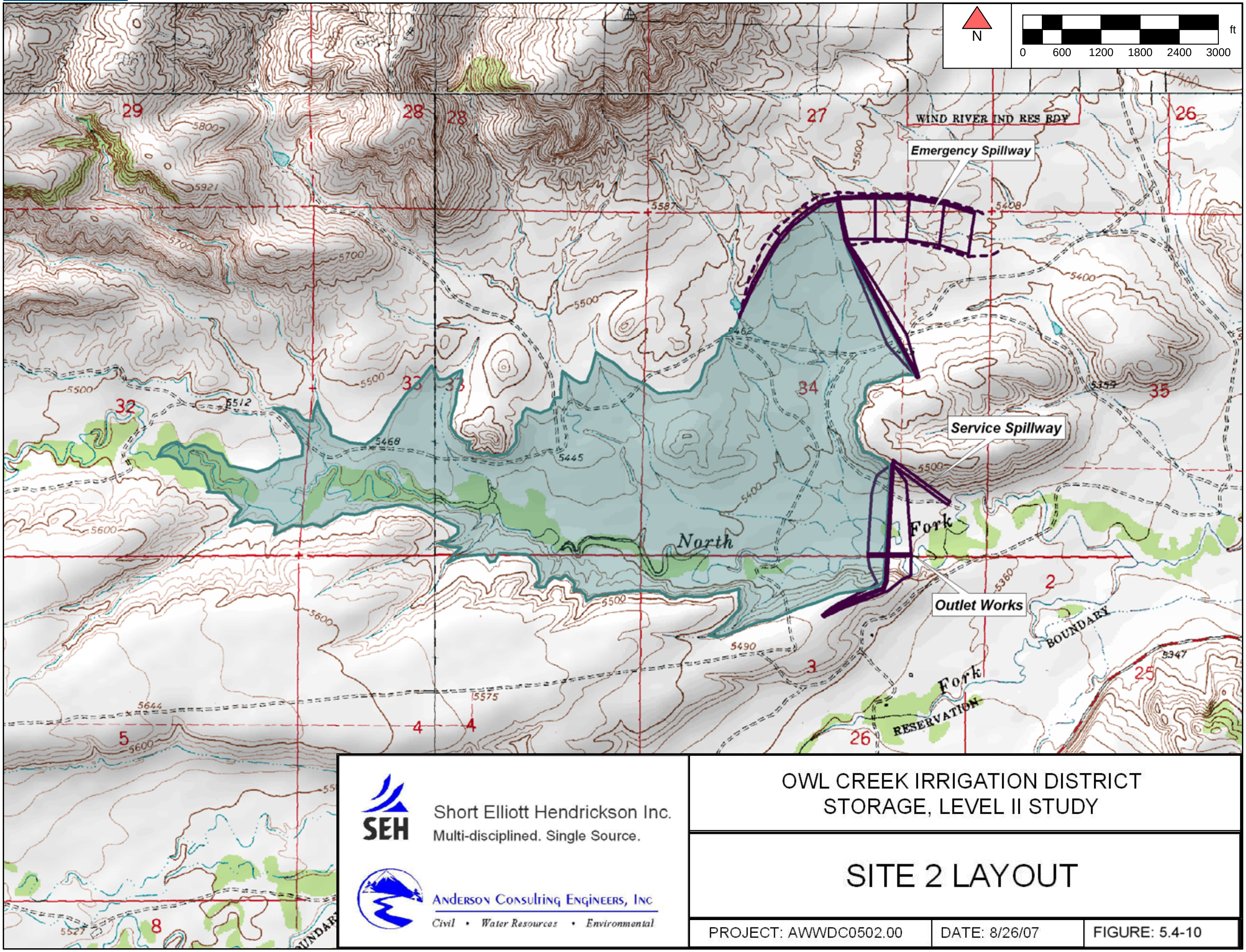

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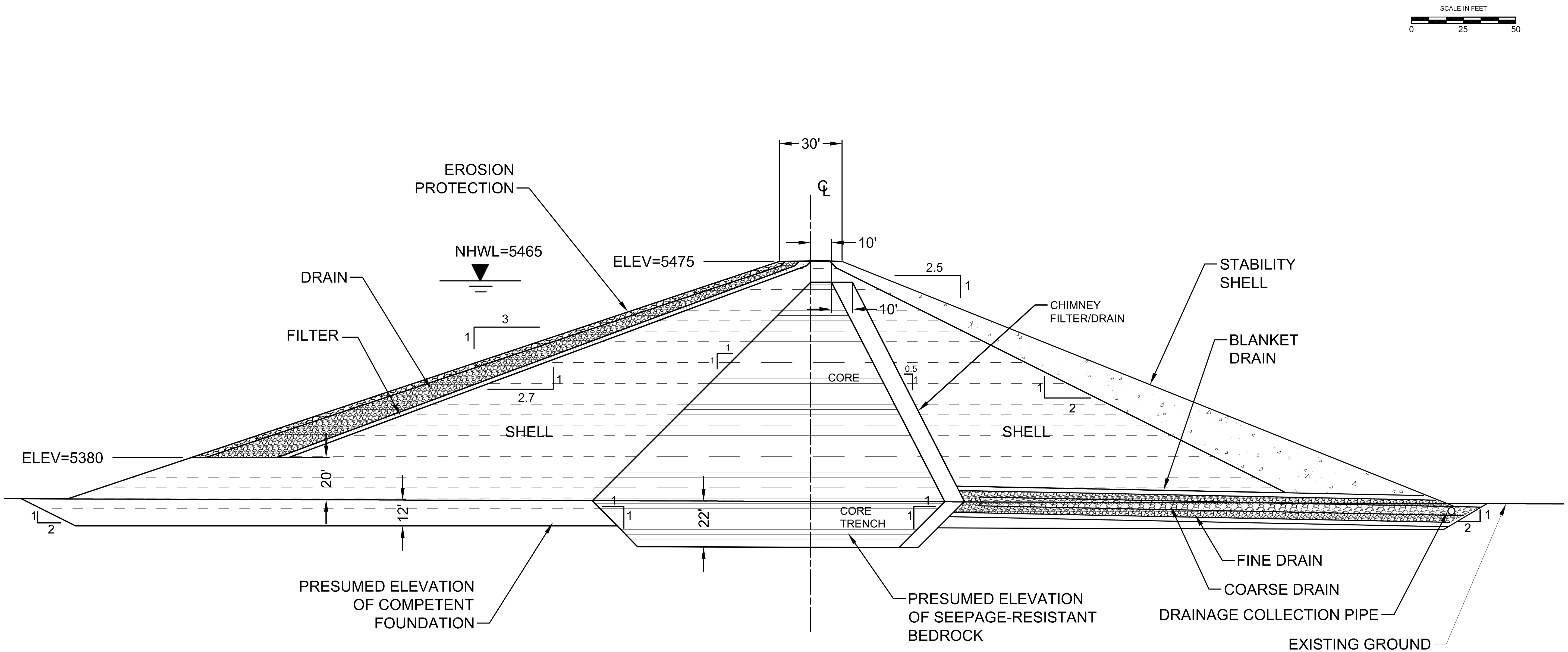
SITE 2 LAYOUT

PROJECT: AWWDC0502.00	DATE: 8/26/07	FIGURE: 5.4-10



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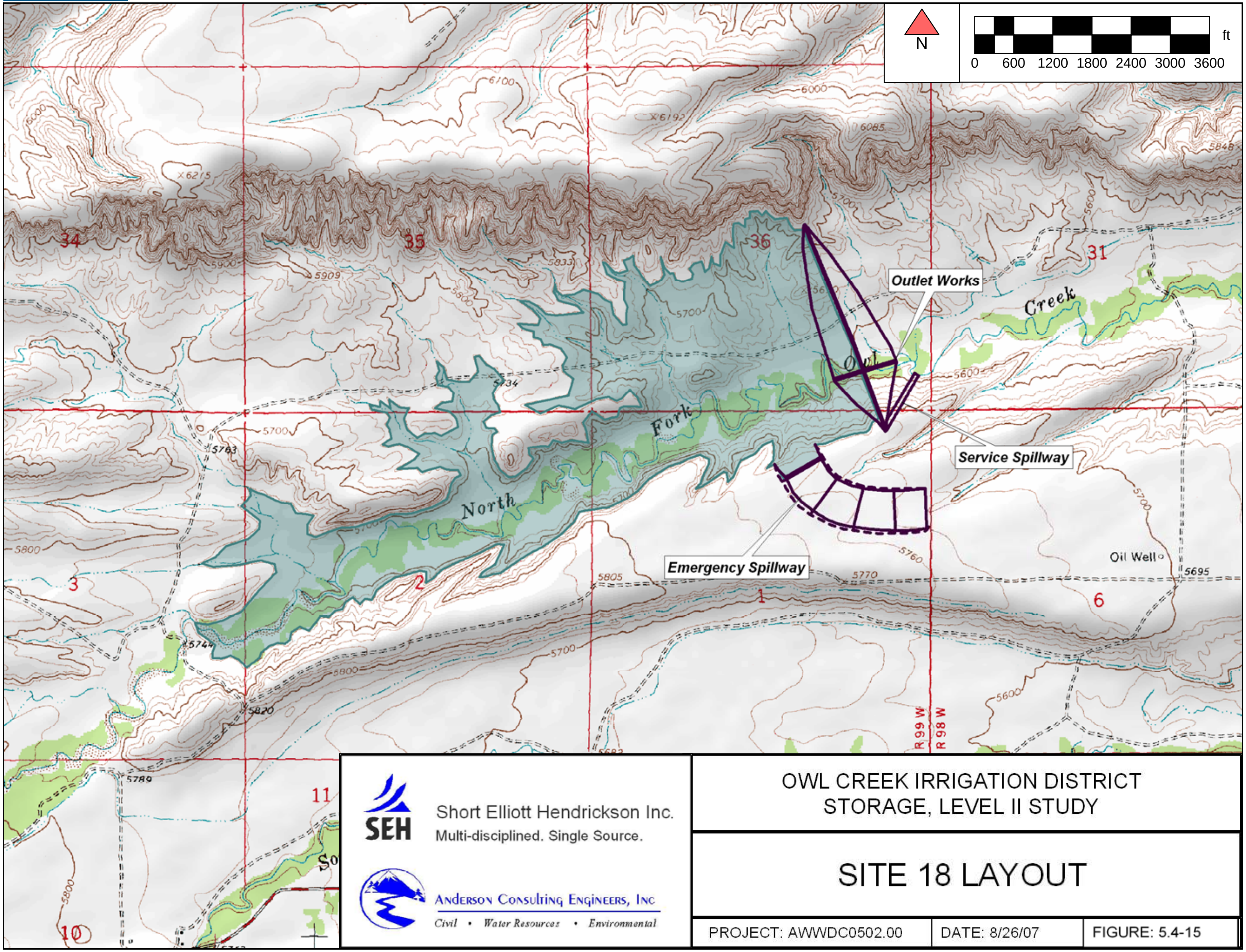
SITE 2 MAXIMUM SECTION
MAIN (SOUTH) DAM

PROJECT: AWWDC0502.00	DATE: 7/30/08	FIGURE: 5.4-11

Figure 5.4-12: Site 18 – Looking Northeast Towards Left Abutment

Figure 5.4-13: Site 18 – View Along Channel at Dam Alignment

Figure 5.4-14: Site 18 – Interbedded Mesaverde Formation near Site 18 Dam Alignment



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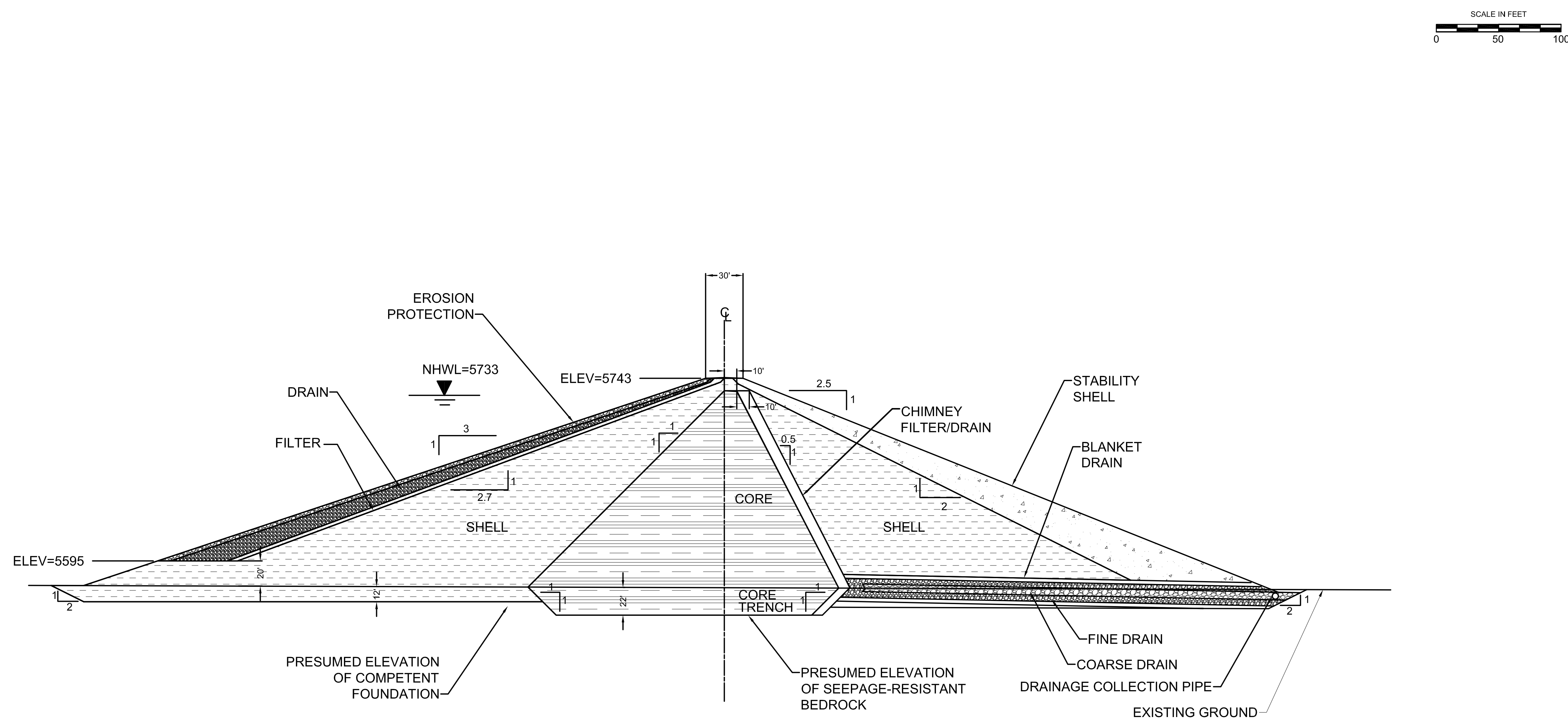
SITE 18 LAYOUT

PROJECT: AWWDC0502.00	DATE: 8/26/07	FIGURE: 5.4-15



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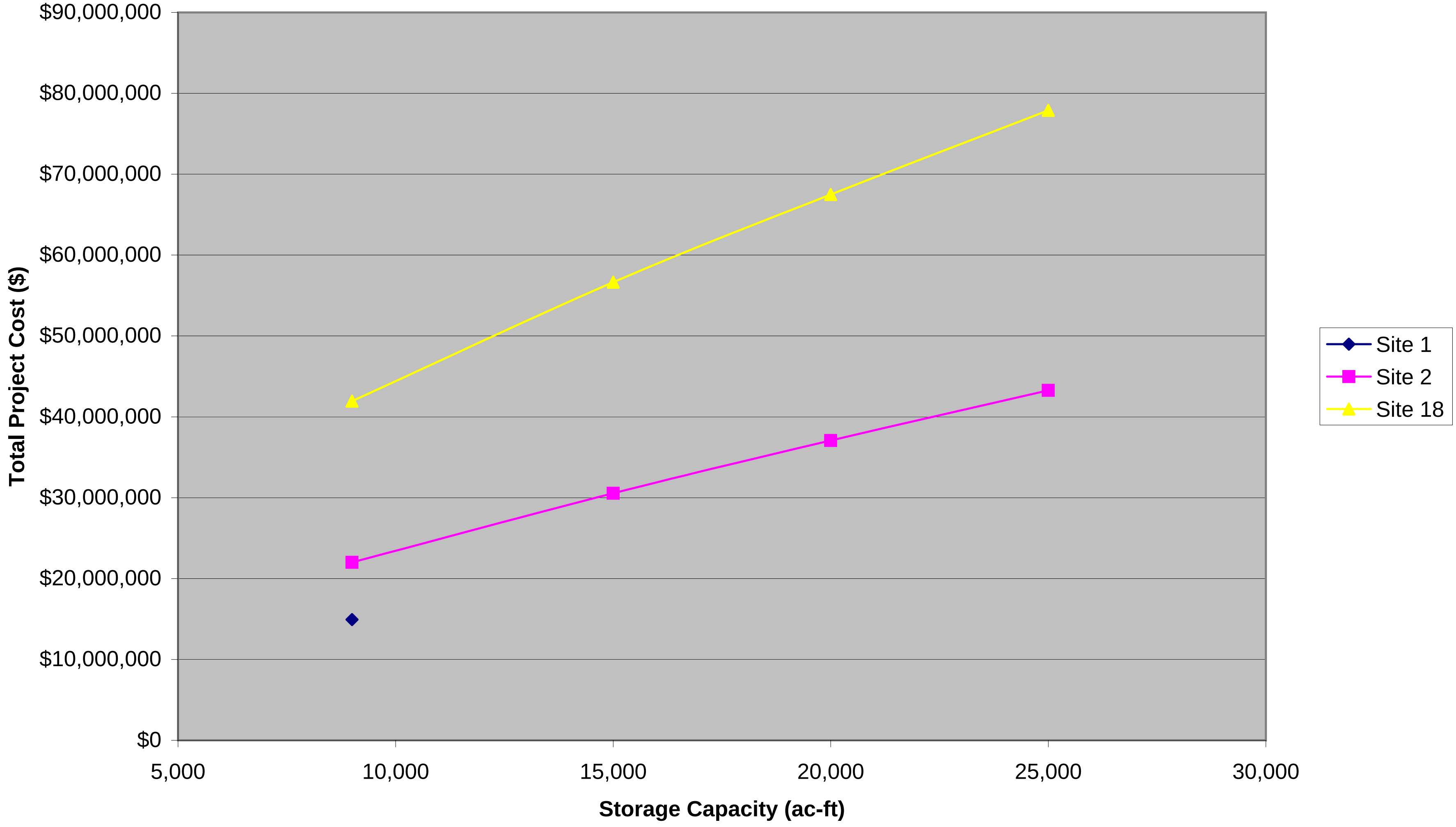
STORAGE, LEVEL II STUDY

SITE 18 MAXIMUM SECTION

PROJECT: AWWDC0502.00	DATE: 7/30/08	FIGURE: 5.4-16

Figure 5.6-1
Selected Alternative Storage Site Cost Curves



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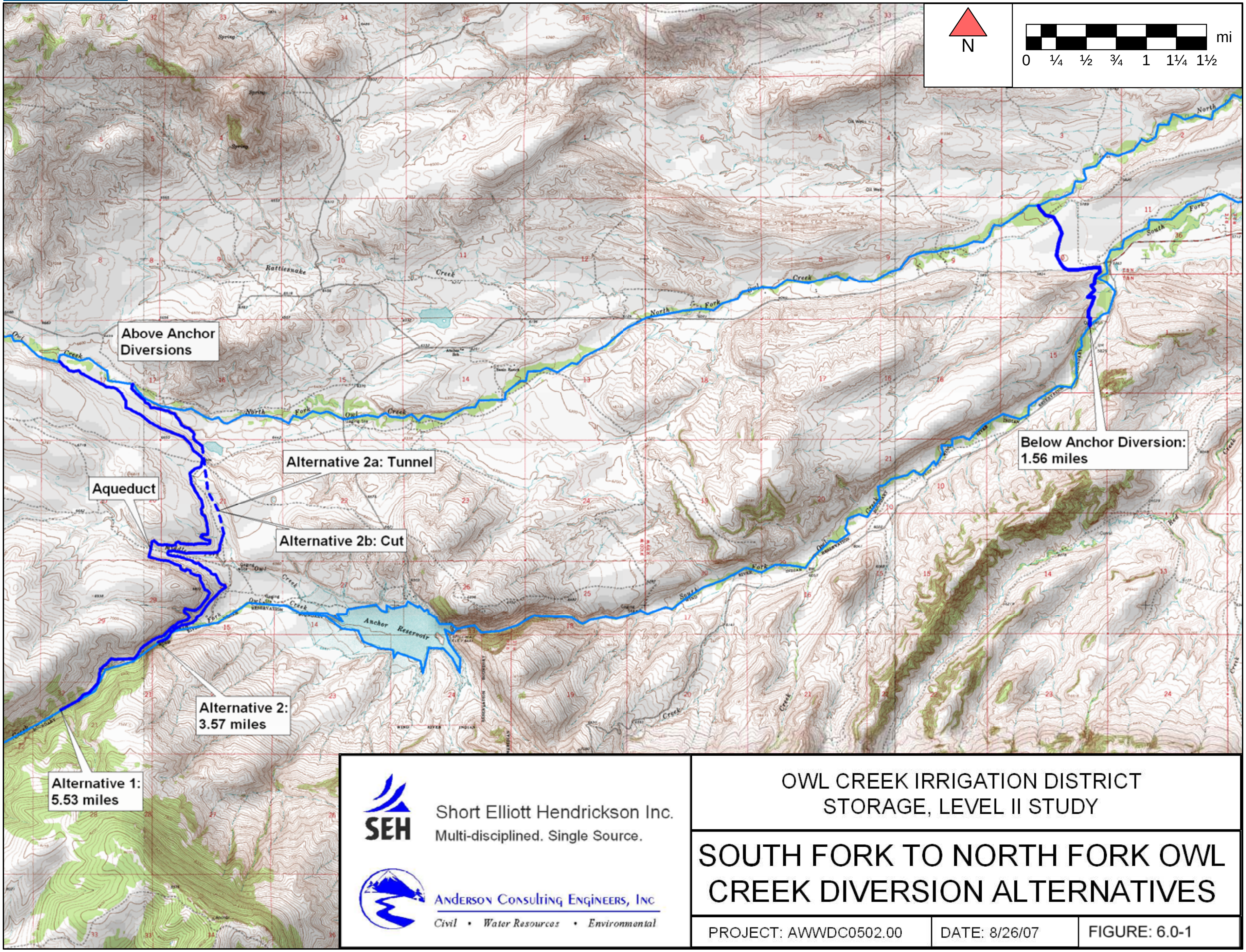

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SOUTH FORK TO NORTH FORK OWL CREEK DIVERSION ALTERNATIVES

PROJECT: AWWDC0502.00	DATE: 8/26/07	FIGURE: 6.0-1