PRELIMINARY ENGINEERING REPORT

FOR

PHASE II ENGINEERING EVALUATION OF AWSA DIVERSION AND STORAGE PROPOSALS

NOVEMBER 7, 2014

Prepared for:

State of New Mexico
Interstate Stream Commission
P.O. Box 25102
Santa Fe, New Mexico 87504-5102

Prepared by:

Bohannan Huston

Engineering
Spatial Data
Advanced Technologies

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ABBREVIATIONS AND ACRONYMS

ABCWUA	Albuquerque Bernalillo County Water Utility Authority
AF	acre-foot
AFY	acre-foot per year
AWSA	Arizona Water Settlements Act
BHI	Bohannan Huston, Inc.
bhp	brake horsepower
BOR	Bureau of Reclamation
CBC	Concrete Box Culvert
cfs	cubic feet per second
CUFA	Consumptive Use and Forbearance Agreement
CY	Cubic Yards
DEM	Digital Elevation Model
FWS	US Fish and Wildlife Service
GBIC	Gila Basin Irrigation Commission
gpm	gallons per minute
HGL	Hydraulic Grade Line
hp	horsepower
ICOLD	International Commission on Large Dams
ISC	New Mexico Interstate Stream Commission
JSAI	John Shomaker & Associates, Inc.
kWh	kilowatt-hour
LCY	Loose Cubic Yards
MEI	Mussetter Engineering, Inc.
NEPA	National Environmental Policy Act
NWS	National Weather Service
NMAC	New Mexico Administrative Code
NMDOT	New Mexico Department of Transportation
NMED	New Mexico Environment Department
NMGRT	New Mexico Gross Receipts Tax
NPDES	National Pollutant Discharge Elimination System
NRCS	Natural Resources Conservation Service
O&M	Operating and Maintenance
OSE	New Mexico Office of the State Engineer
OSHA	Occupational Safety and Health Administration
PER	Preliminary Engineering Report
PNM	Public Service Company of New Mexico
PV	Photovoltaic
RCP	Reinforced Concrete Pipe
SCS	Soil Conservation Service
SWRWS	Southwest Regional Water Supply
TBM	Tunnel Boring Machine
TDH	Total Dynamic Head
USFS	US Forest Service
USGS	US Geological Survey
WSEL	Water Surface Elevation

EXECUTIVE SUMMARY

This preliminary engineering report (PER) supplements the Bohannan Huston Inc. (BHI) *Preliminary Engineering Report for Gila River Diversion, Conveyance, and Storage Alternatives* (April 2014 PER) by identifying a new diversion location, considering in more detail both surface and subsurface diversion options and the conveyance to Spar (the first reservoir in the system), refining analysis of potential storage at Spar and Winn canyons, conducting a reach specific river geomorphologic review, completing geophysical and geotechnical site investigations and analysis for the reservoir sites, analyzing gravity and pumping diversion options, and evaluating potential sediment control measures.

The project area is located in the Gila River valley between Turkey Creek and Cherokee Canyon, near the southwestern New Mexico towns of Cliff and Gila in Grant County. BHI investigated a two-mile reach of the Gila River from Turkey Creek to just downstream of Brushy Canyon for potential AWSA diversion locations. In addition to Spar and Winn canyons, we evaluated the other storage sites selected in Phase I in more detail during this phase of work, including Pope, Sycamore, Maldonado and Dix canyons.

Storage of water near the top of the Cliff-Gila valley is critical to provide water for irrigation and to replenish the river. During irrigation season, the river commonly runs dry between the Fort West and Gila Farm irrigation diversion points. As part of this project, BHI revisited potential storage sites near the top of the Cliff-Gila farming valley, specifically Spar and Winn which, among other purposes, could be used to keep the river wet in this reach. In addition to storage of water, removal of sediment from the system is critical for system functionality. To this end, BHI has evaluated Spar Canyon as a sediment removal facility located upstream of the conveyance pipes and all other reservoirs in the system.

Tetra Tech, as a sub consultant to BHI, performed geomorphic and sediment-transport analysis of the reach of the Gila River in the vicinity of the proposed diversion structure along with an assessment of the sediment transport capacity of the proposed tunnel from the river diversion to Spar Reservoir. This analysis addresses two main concerns:

1. Diversion structure may result in excessive sedimentation that could limit the diversion's functionality.
2. Potential for tunnel blockage resulting from accumulation of sediments delivered by the diversion structure.

BHI, Tetra Tech, and the New Mexico Interstate Stream Commission (ISC) staff conducted a site visit in June 2014. We identified five potential surface diversion sites prior

to the site visit and three additional potential surface diversion sites during the site visit. Of these eight sites, Site 6 (upstream of Brock Canyon) was recommended by BHI and Tetra Tech and selected by ISC for further analysis for a surface diversion. The surface diversion would consist of a low profile concrete weir with tilted wedge-wire screen. The structure would extend across 115 feet of the river, which is less than 20 percent of the width of the 100-yr floodplain. The 100-yr river floodplain extends beyond the end of the proposed diversion structure approximately 700 feet. A low-flow bypass is included in the structure, which will allow for fish passage. Fish migration ability will not be impacted by the diversion structure. As part of the site investigation, the reach was also assessed for suitability of a sub-surface diversion, which would consist of a modified radial collector well or infiltration gallery.

BHI subcontractor Geo-Test Inc. performed preliminary geotechnical, geophysical and geologic investigations for this project. Based on these investigations, Geo-Test recommended installing impermeable liners at Spar, Winn, Pope and Sycamore reservoirs due to high permeability of underlying soils.

Due to Spar Canyon's location near the top of the Cliff-Gila farming valley, BHI evaluated the potential for Spar Canyon reservoir to replenish the Gila River during irrigation season around four constraints.

1. Provide up to 90 cfs of water supply for irrigation in the Cliff-Gila Valley.
2. Allow for sediment retention in the Spar Canyon reservoir.
3. Release water with suitable temperature from the upper 10 – 20 ft of the water column back to the river at the time of drought.
4. Provide a maximum outflow rate of 350 cfs to supply water to downstream storage sites.

These constraints are intended to ensure that Spar Canyon reservoir can aid flows for endangered aquatic species habitat and develop additional water rights afforded by the AWSA.

Alternatives analyzed previously for diverting, conveying, and storing water in the Cliff-Gila Valley, have been limited to gravity diversion and conveyance methods only. In this phase of work, pumping options were evaluated to benefit properties that cannot be served by gravity from storage and to maximize reservoir storage capacity for two scenarios.

1. Pumping from Winn Canyon to irrigate properties that could not be served by gravity releases from Winn if the reservoir was less than 20 percent full. The purpose of the Winn Canyon pump is to pump water from Winn Canyon to irrigated

properties in the far north end of the irrigated Cliff-Gila Valley. Water from Winn Canyon could also be discharged to the river, just downstream of the Upper Gila Diversion, to supply the reach that periodically dries up between the Upper Gila and Gila Farms diversions.

2. Pumping from the Gila River to Spar Canyon reservoir to maximize reservoir storage capacity. The purpose of the Gila River to Spar Canyon pump is to eliminate the gravity diversion and conveyance constraints which limit the storage capacity in Spar Canyon. In a pumping scenario, storage at Spar Canyon is dictated by site topography, rather than by the elevation of the gravity diversion.

BHI subcontracted with Consolidated Solar Technologies, LLC (CST) to estimate the configuration and costs of a solar photovoltaic array for diversion and conveyance purposes.

Based on the more in-depth reanalysis, three primary alternatives were developed. These alternatives and their approximate total project construction and non-construction costs including New Mexico Gross Receipts Tax (NMGRT) are summarized below.

ALTERNATIVE	DESCRIPTION	TOTAL STORAGE (AF)	TOTAL PROJECT COST
ALTERNATIVE 1A	Surface diversion at Site 6, tunnel from diversion to Spar Canyon, storage at Spar, Winn, Pope, and Sycamore Canyons, 10 cfs irrigation pump station at Winn Canyon	65,417	$711 M
ALTERNATIVE 1B	Surface diversion at Site 6, tunnel from diversion to Spar Canyon, storage at Spar, Winn, Pope, and Sycamore Canyons, 50 cfs irrigation pump station at Winn Canyon	65,417	$714 M
ALTERNATIVE 2A	Subsurface diversion for 150 cfs above Site 6 combined with surface diversion for 200 cfs at Site 6, tunnel from diversion to Spar Canyon, storage at Spar, Winn, Pope, and Sycamore Canyons, 10 cfs irrigation pump station at Winn Canyon	65,417	$741 M
ALTERNATIVE 2B	Subsurface diversion for 150 cfs above Site 6 combined with surface diversion for 200 cfs at Site 6, tunnel from diversion to Spar Canyon, storage at Spar, Winn, Pope, and Sycamore Canyons, 50 cfs irrigation pump station at Winn Canyon	65,417	$744 M
ALTERNATIVE 3	Diversion at elevation 4,640 (approximately 2,000 feet downstream of the confluence of Spar Canyon with the Gila River), pumping from diversion to Spar Canyon, storage at Spar, Winn and Pope Canyons	65,482	$703 M

The three alternatives developed as part of this project, summarized in the table above and presented in more detail in this report all meet the project objectives.

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I. INTRODUCTION

This preliminary engineering report (PER) supplements the BHI *Preliminary Engineering Report, Gila River Diversion for Conveyance, and Storage Alternatives* (April 2014 PER) by identifying a new diversion location, considering in more detail both surface and subsurface diversion options and the conveyance to Spar (the first reservoir in the system), refining analysis of potential storage at Spar and Winn canyons, conducting a reach specific river geomorphologic review, completing geophysical and geotechnical site investigations and analysis for the reservoir sites, analyzing gravity and pumping diversion options, and evaluating potential sediment control measures.

A. PROJECT OBJECTIVES

The objectives of work contracted by the New Mexico Interstate Stream Commission (ISC) with BHI for the project include the following:

- Recommend one new diversion location.
- Conduct site investigation of the river diversion reach with Tetra Tech and ISC staff.
- Assess selected diversion site with respect to sediment deposition/aggradation, suitability for a low profile concrete weir with tilted wedge-wire screen diversion and/or a modified radial collector well/infiltration gallery diversion methods.
- Summarize the river geomorphologic investigation of the potential diversion reaches (objectives of geomorphologic investigation are listed in Section III).
- Evaluate modified radial collector well or infiltration gallery configurations for viability of producing 150 cfs and 350 cfs.
- Assess viability of selected diversion site filling to optimal pool elevations in selected storage sites.
- Develop additional alternatives at Spar Canyon.
- Develop additional alternatives at Winn Canyon.
- Evaluate pumping options.
- Evaluate methods to minimize flood damage and sedimentation at the diversion structures and conveyance structures.
- Summarize geotechnical/geological/geophysical analysis (objectives of geotechnical/geological/geophysical analysis are listed in Section IV).

- Prepare elevation-area-capacity tables for all candidate reservoirs based on diversion site selection.
- Estimate configuration and costs of a solar array for diversion and conveyance purposes (objectives of solar array analysis are listed in Section VII).
- Prepare architectural renderings of diversion structure and impoundments in the selected storage sites.
- Prepare PER summarizing analysis and recommendations.
- Present findings to the ISC and stakeholders.

B. BACKGROUND

BHI prepared a *Preliminary Engineering Report for Gila River Diversion, Conveyance and Storage Alternatives* for the ISC in April 2014 (hereafter referred to as April 2014 PER) with alternatives for diversion, conveyance and off-stream storage of AWSA water along the Gila River between Turkey Creek and Cherokee Canyon. Locations for off-stream storage in the side canyons leading into the Gila River were evaluated. In addition to storage evaluation, that report also identified diversion locations to supply that storage (known as Category 2 diversions), as well as locations for diversions to improve and supply the existing irrigation system in the Cliff-Gila farming valley (known as Category 1 diversions). That report also evaluated different types of diversion structures, which are listed, along with their advantages and disadvantages, in Table I-1 below.

Table I-1 – Diversion Types, Advantages and Disadvantages

Diversion Type	Advantages	Disadvantages	Comments
Shallow Collector Wells with Horizontal Wells	Not as visible, Environmentally friendlier after construction, Construction of the wells has less impact on the river than constructing diversion weirs, Would not be impacted by ash or other river debris.	High maintenance at high flow rates,	Considered for both Category 1 and Category 2 diversions
Infiltration Galleries	Not as visible, Environmentally more friendly after construction, Operates efficiently even when the river has low flow, Construction of the wells has less long term impact on the river than constructing diversion weirs, Not be impacted by ash or other river debris.	High maintenance at high flow rates, limited capacity, more short term impact (disturbance) of the river from diversion construction, susceptible to plugging	Considered for both Category 1 and Category 2 diversions
Rock Cross Vane Weir	Provides grade control and reduced bank erosion, fish passage and enhanced fish habitat, Visually pleasing.	Less permanent than concrete structures, Leaky, Higher maintenance than concrete structures, Susceptibility to damage from large bedload materials.	Not considered for Category 2 diversions due to maintenance requirements.
Grouted Boulder Weir	Provides grade control and reduced bank erosion, fish passage and creates a riffle and a pool	Less permanent than concrete structures, Higher maintenance than concrete structures, Susceptibility to damage from large bedload materials.	Not considered for Category 2 diversions due to maintenance requirements.
Low Profile Concrete Weir with Tilted Wedge-Wire Screen	Self-maintaining, Hydraulic control	Possible concerns with fish passage, Susceptibility to damage from large bedload materials, Susceptibility to sediment deposition in the system	Not considered for Category 1 diversions due to cost
Concrete Weir with Control Gates	Generally self-maintaining, Hydraulic control, Reliability	Obtrusive, concerns with fish passage, Large afflux during floods which causes large submergence, Because the crest is at a high level, there is a great silting problem	Not considered for Category 1 diversions due to cost
Diversion Channel with Upstream Vanes	Least invasive to river	Space requirement, Susceptible to degradation/aggradation of river, Likely to be bypassed by river flow	Not considered for Category 2 diversions due to reliability

A major goal of the April 2014 PER effort was to achieve at least 65,000 AF of storage in side canyons to the Gila River, including 5,000 AF near the top of the Cliff-Gila valley. This stored water could then be made available to agricultural and municipal users through

a system of conveyances, piping, and pumping, as well as water releases during dry periods for the health of the river and its riparian habitat. A review of the side canyons was performed, and a multi-step selection process was used to focus the identification of preferred locations for water storage in side canyons along the Gila. Six canyons were recommended for storage as a result of that evaluation process. Table I-2 below lists the storage volumes that were calculated for the recommended canyons in the April 2014 PER.

Table I-2 – Storage Volumes for Recommended Canyons

Canyon Name	Diversion at el. 4770		Diversion at el. 4695		Diversion at el. 4640	
	WSEL (ft)	Volume (ac-ft)	WSEL (ft)	Volume (ac-ft)	WSEL (ft)	Volume (ac-ft)
Spar	4,755	2,512	N/A	N/A	N/A	N/A
Maldonado	4,750	3,424	N/A	N/A	N/A	N/A
Winn	4,685	7,589	4,680	6,454	N/A	N/A
Pope	4,640	12,857	4,640	12,857	4,600	4,534
Sycamore	4,600	41,399	4,600	41,399	4,595	38,706
Dix	4,600	3,593	4,600	3,593	4,595	3,166
Total Volume		71,374		64,303		46,406

The alternatives and resultant maximum system storage in the side canyons were dictated primarily by the elevation of the potential Category 2 diversions. Three Category 2 diversion sites were considered within the April 2014 scope. The most upstream potential Category 2 diversion site was located downstream of Turkey Creek, and the most downstream diversion site was located just upstream of Mogollon Creek. The three potential diversion locations, with approximate elevations ranging from 4,770 to 4,640 ft, enabled the development of a broad range of alternatives with respect to cost and ultimate system storage capacity to aid the ISC in selecting the optimal configuration for the proposed water system. For each diversion site, several types of diversion structures were evaluated. Table I-3 below summarizes the selection process for the Category 2 diversions. The April 2014 PER included conceptual site plans for each diversion location.

Table I-3 – Category 2 Diversion Selection

Location	Shallow Collector Wells with Horizontal Wells		Infiltration Galleries		Concrete Weir with Control Gates		Low Profile Concrete Weir With Tilted Wedge-Wire Screen		Inflatable Rubber Dams	
Diversion 1 at 4770	X	High maintenance at high flow rates	X	High maintenance at high flow rates	X	Obtrusive	O +	Requires the least maintenance	X	Complicated operational requirements
Diversion 2 at 4695	X	High maintenance at high flow rates	X	High maintenance at high flow rates	X	Obtrusive	O +	Requires the least maintenance	X	Complicated operational requirements
Diversion 3 at 4640	X	High maintenance at high flow rates	X	High maintenance at high flow rates	X	Obtrusive	O +	Requires the least maintenance	X	Complicated operational requirements

O – suitable
X – not suitable
+ – recommended

For each alternative, gravity conveyances between the diversion and storage sites were also evaluated. Several items were taken into consideration during the evaluation process, including type of conveyance (pipe versus open channel), constructability, demands on each conveyance segment and storage site and cost.

In addition to diversion, conveyance and storage of AWSA water within the Cliff-Gila valley, BHI evaluated the Southwest Regional Water Supply proposal, proposed by the City of Deming. Water diverted from the Gila River would be pumped from a reservoir in Pope Canyon through a pipeline over the continental divide to provide water to the Silver City area, mining district communities, and to the City of Deming. The total flow through the pipeline was assumed to be 10,000 AFY, or 6,190 gallons per minute (gpm), based on the proposal. Five booster stations were proposed along the pipeline alignment. BHI recommended a 36 inch pipeline from Pope Canyon to the high point along the proposed route and a 16 inch pipeline from the high point to communities east of the continental divide. From the high point of the pipeline alignment to Deming, there is considerable energy available in the pipeline due to the difference in elevation (approximately 2,000 feet). The April 2014 PER stated there may be potential to convert that energy to electricity, which would offset the electricity requirements of the booster stations east of the high point. As part of this Phase II PER, BHI has completed a hydropower analysis (discussed further in Section VIII below).

As a result of that analysis, three primary alternatives were developed. These alternatives and their approximate total project costs are presented in the April 2014 PER and are summarized in Table I-4 below.

Table I-4 – Comparison of April 2014 PER Alternatives

ALTERNATIVE	DESCRIPTION	TOTAL STORAGE (AF)	COST BEFORE NMGRT
ALTERNATIVE 1A	Diversion at elevation 4,770, open channel from diversion to Spar Canyon, storage at Spar, Maldonado, Winn, Pope, Sycamore and Dix Canyons	71,374	$559 M
ALTERNATIVE 1B	Diversion at elevation 4,770, tunnel from diversion to Spar Canyon, storage at Spar, Maldonado, Winn, Pope, Sycamore and Dix Canyons	71,374	$519 M
ALTERNATIVE 2A	Diversion at elevation 4,695, open channel from diversion transitioning to pipe to Winn Canyon, storage at Winn, Pope, Sycamore and Dix Canyons	64,303	$496 M
ALTERNATIVE 2B	Diversion at elevation 4,695, pipe from diversion through tunnel to Junction 5, pipe from Junction 5 to Winn Canyon, storage at Winn, Pope, Sycamore and Dix Canyons	64,303	$412 M
ALTERNATIVE 3	Diversion at elevation 4,640, pipe from diversion to Pope Canyon, storage at Pope, Sycamore and Dix Canyons	46,406	$354 M

II. PROJECT AREA DESCRIPTION

A. LOCATION

The project area is located in the Gila River valley between Turkey Creek and Cherokee Canyon, near the towns of Cliff and Gila in Grant County in southwestern New Mexico. The general project location is shown on Figure 1. The Planning Area is bounded on the north and south by the Gila National Forest. On the north end of the project area, the Gila Wilderness represents a federally protected area within which no construction can take place; therefore, no improvements were considered within the Wilderness area. Vehicle access to the river and side canyons is available from NM Highways 211, 153 (Turkey Creek Road), 180 and 293 (Box Canyon Road) and via McCauley Road and Bill Evans Road. The three existing irrigation diversions and the one existing non-irrigation diversion that provides water for Bill Evans Lake are shown on Figure 2. The existing irrigation diversions, from north to south, are Upper Gila, Fort West, and Gila Farms. The only existing permanent non-irrigation diversion in this reach of the River is owned and operated by Freeport-McMoRan Copper & Gold. Water from that diversion is pumped to Bill Evans Lake, shown on Figure 2, from where it is pumped to the Tyrone Mine, which is located approximately 13 miles southeast of Bill Evans Lake. A two mile reach of the river from Turkey Creek to a point downstream of Brushy Canyon was investigated for potential AWSA diversion locations. Four selected storage sites were evaluated in more detail during the Phase II study, including Spar, Winn, Pope, and Sycamore canyons. The diversion investigation reach and storages sites can be seen on Figure 2.

B. TOPOGRAPHY

Within the project planning area, the ground generally slopes from northeast to southwest, with the side canyons sloping toward the river. The elevation of the river descends from approximately 4,770 feet above mean sea level at the north end of the project planning area, to approximately 4,475 feet at the southern end of the planning area. The average slope of the river over this reach is approximately 0.3 percent. The walls of the river valley are generally steeper at the north end of the planning area. The river valley widens in the middle of the planning area, near Cliff and Gila, where the valley flattens out and there exists significant irrigated acreage, served by the existing irrigation diversions.

C. GEOLOGY

Geo-Test, Inc. completed a geologic investigation as part of the April 2014 PER. Geo-Test's report was included as an appendix to the April 2014 PER. That geologic study included a discussion of concerns that related to scoring each of nine candidate dam sites during the canyon selection process. The primary geologic consideration was the potential for the presence of faults within the proposed dam candidate sites. A preliminary review of seismic hazards for the area found that there are no Quaternary faults mapped in the area and that seismic design would be equivalent to background acceleration that should be the same for each of the dam sites. That report included annotated photographs. The second geologic concern raised in that geologic investigation was the very granular (non-cohesive) characteristics of the Gila Conglomerate soils. As a result, further geologic and geotechnical investigation was completed as part of this Phase II study to address this concern.

III. GEOMORPHOLOGIC INVESTIGATION

A. INTRODUCTION

BHI subcontracted with Tetra Tech's Surface Water Group to perform a geomorphic and sediment-transport analysis of the Gila River in the vicinity of the proposed diversion structure. This analysis is to assist BHI and ISC in addressing concerns regarding sedimentation that could limit functionality of the diversion and conveyances structures and the potential for tunnel blockage, as a result of accumulation of sediments delivered by the diversion structure. The following are Tetra Tech's objectives:

- Perform a field reconnaissance of the Gila River study area, including the approximately 2.3-mile reach from the confluence with Turkey Creek down to the "riffle" adjacent to the Brushy Canyon alluvial fan, located about 3,700 feet downstream of the proposed diversion structure (see Figure 3). From the field reconnaissance results, identify changes in hydraulic and geomorphic conditions since the MEI (2006) geomorphology study and to collect bed material samples to characterize the sediment load.
- Conduct topographic and bathymetric surveys of the approximate 2.3-mile project reach to provide input to the hydraulic and sediment-transport modeling.
- Summarize and update, if necessary, the findings from the MEI (2006) study.
- Using measured data from the Gila River near Gila, NM Gage (USGS Gage No. 09430500), evaluate the hydrologic conditions by performing mean daily flow duration and peak flood flow frequency analyses and developing representative annual hydrographs that include peak discharges equivalent to approximately the 2-, 5-, and 10-year return events.
- Develop a hydraulic model of the project reach.
- Convert the project-conditions hydraulic model into a mobile-boundary sediment-transport model to assess sediment loading in the vicinity of the proposed diversion structure.
- Use the model results and methods developed by Einstein (1950) along with measured suspended sediment gage data at the Gila River near Gila, NM to estimate the suspended sediment concentration profile by size fraction over a range of discharges. Develop a rating curve that defines the volume of material

that would be passed through the intake screen and into the tunnel over a range of discharges.

- Estimate hydraulic conditions in the tunnel and use the results to estimate the sediment-transport capacity of the tunnel for the range of sediment sizes that would pass the intake screen. Assess the potential for accumulation of sediment in the tunnel.
- Prepare a report that summarizes the methods, assumptions and results of the analysis.

B. PREVIOUS STUDIES

Two previous studies on geomorphology of the Gila River have been completed and were reviewed as part of this project. The first study is "Geomorphology of the Upper Gila River within the State of New Mexico," prepared by Mussetter Engineering, Inc. (MEI) in June 2006 for the ISC. The second study is the "Upper Gila River Fluvial Geomorphology Study," prepared by the U.S. Department of the Interior, Bureau of Reclamation (BOR) Fluvial Hydraulics & Geomorphology Team from the Technical Service Center in Denver, Colorado, between 2001 and 2004.

1. MUSSETTER ENGINEERING, INC. STUDY

The MEI study investigated the geomorphology, hydrology, hydraulics and sediment-transport characteristics of the Upper Gila River at five locations between the downstream boundary of the Gila Wilderness Area and the Arizona-New Mexico State Line. The study identified the geomorphic impacts of two Consumptive Use and Forbearance Agreement (CUFA) depletion scenarios provided by the ISC. The Turkey Creek site in the MEI study overlaps part of the reach of interest in this BHI report. At the Turkey Creek site, for the 2-year flow (1,930 cfs), the average channel velocity is about 5 feet per second (fps), hydraulic depth is about 4 feet, and the channel top width is about 98 feet. The Turkey Creek site has a representative median bed-material size of 49 mm, and D_{84} value of 101 mm, and critical discharges for bed mobilization and significant sediment transport of 2,500 and 4,000 cfs, respectively. Diversion of flows below 2,500 cfs at the Turkey Creek site will have no geomorphic impacts. The MEI study concluded that the maximum diversion rate of 350 cfs is unlikely to have a significant effect on sediment transport volumes during infrequent flows, when the bulk of the sediment is being transported.

2. BUREAU OF RECLAMATION STUDY

The Bureau of Reclamation's study was sponsored by the NMED, Surface Water Quality Bureau, with the goal of diagnosing the fluvial geomorphological attributes of the upper Gila River. The purpose of the study was to increase the awareness of these processes enabling improved local, state and federal management of the stream corridor. The study includes background information, field data, photographic analyses, and a variety of topographic, geomorphic, hydraulic and hydrologic analyses. The following were performed:

- Qualitative Assessment of Upper Box Geomorphology which concluded that the river has been stable for centuries.
- Flood Frequency, Flow Duration and Trends which included estimates of the peak flow rate for the 2 year through 100 year events ranging from 1,970 cfs to 38,600 cfs respectively.
- Stable Channel Analysis which concluded that the Upper Gila River Analysis Reach tends toward stability.
- Catalog of Historical Changes which concluded that channel changes are related to large floods such as water years 1941 (25,400 cfs), 1978 (32,400 cfs) and 1984 (35,200 cfs).
- Stream Corridor Assessment which synthesizes the above analyses and findings.

The study included a field data collection plan, which summarized the plans for field data collection to support the Upper Gila River Fluvial Geomorphology Study. The study reach includes the Gila River between the Arizona-New Mexico State border and roughly 1.6 miles upstream of Mogollon Creek, near Cliff, NM. BOR reviewed existing studies that contain information about the upper Gila River watershed that were thought to be useful in the Upper Gila River Fluvial Geomorphology Study. Over 80 documents were reviewed and include everything from hydrologic, geologic, and biologic studies to accounts of floods and precipitation events, studies of channel change and erosion, links between flood records and climate, land use planning documents, water quality studies and ground water studies. Field reconnaissance for this study resulted in qualitative observations of a clear record of stability of geomorphic surfaces that bound the Gila River in the Upper Box (roughly 37 miles of the river upstream of the Cliff-Gila valley). Observations indicated that the bed elevation of the Gila River in the Upper Box has been dynamically stable for the last several centuries.

C. SITE INVESTIGATION

Two site visits were conducted for the geomorphologic investigation portion of this project. The first was a field reconnaissance site visit and the second was a topographic/bathymetric survey and bed material sampling site visit.

1. FIELD RECONNAISSANCE

A site visit was conducted June 10, 2014 by ISC, Tetra Tech, and BHI staff. As part of the trip, a two-mile reach of the Gila River downstream of Turkey Creek was investigated, including five pre-selected potential diversion locations, shown on Figure 3. The river was walked and photographed. Photographs are included in Appendix A. The purpose of the site investigation was to evaluate each candidate diversion site for its suitability for diversion construction with respect to river geomorphologic conditions as well as identify any potential other sites. From this work, three additional potential diversion sites were identified and are shown on Figure 4.

2. TOPOGRAPHIC/BATHYMETRIC SURVEYING

During a second site visit conducted June 17 –19, 2014, Tetra Tech surveyed river sections (including repeat surveys of the cross sections that were included in the 2006 MEI report) from Turkey Creek down to well below the Brock Canyon area near the confluence with Brushy Canyon. The Cross Sections are numbered 1, 2, 3, etc. to correspond with the numbering from the 2006 MEI report. Tetra Tech also collected numerous sediment samples (bulk samples, pebble counts and boulder counts) including a bulk sample of the fine material that represents the fine suspended sediment load.

3. BED MATERIAL SEDIMENT SAMPLING

Tetra Tech's sediment sampling results indicate there is a very wide range of sediment sizes present along the project reach. This is consistent with the gradations of the 2006 sediment sampling. The subsurface samples collected in 2014 along the main stem are coarser than those collected in 2006, which suggests that materials were transported by high magnitude events in 2008 and 2013, whereas, the 2006 samples represent the finer fractions of the 2005 Brock Canyon debris flows. The sediment sampling is discussed in more detail in Appendix C.

D. MORPHOLOGIC CHANGES AND FINDINGS

Based on the available aerial imagery taken between 2006 and 2013, the most significant changes to channel planform have occurred at and downstream from the confluence with Brock Canyon, where erosion of the distal portions of the fan and subsequent deposition of the material along the left channel margin has resulted in shifting of the main channel by as much as 50 feet to the right. Along the remainder of the reach, localized bank erosion and bar formation or adjustment has occurred. Repeat surveys of the 2006 cross sections in 2014 indicate that the geometry of the main channel and overbanks has shifted to varying degrees from the 2008 and 2013 floods. Appendix C includes a technical memorandum from Tetra Tech summarizing updates to MEI's 2006 study.

The proposed surface diversion location, Site 6, is located in a reach that is stable laterally and vertically. The proposed diversion is located near River Station 51+00 of the Tetra Tech hydraulic model, which corresponds to Cross Section 6 (see Figure 3.10 of the Tetra Tech memo included as Appendix C). Based on comparison of Cross Section 6 from the 2014 survey and the 2006 survey (see Figure 4.9 of the Tetra Tech memo included as Appendix C) and other information including field observations, Tetra Tech concluded "this section experienced very little change during the period between the two surveys, although limited reshaping of the left bank is evident. This reshaping is attributed to 2008 and 2013 floods. Further, because this site is located at a bedrock-controlled hydraulic constriction, relatively high velocities are expected that would tend to limit deposition, at least in the immediate vicinity of the structure. Considering all of this information and assessment, Site 6 was recommended by BHI and Tetra Tech to the ISC as the preferred location for the diversion structure. The ISC then selected Site 6 for the diversion location.

E. MODELING AND RESULTS

Tetra Tech developed a hydraulic model of the project reach for a range of flow up to the 500-year peak flow to support the development of diversion alternatives. Tetra Tech created an existing conditions model, as well as a sediment-transport model.

1. HYDRAULIC MODELING

The existing conditions hydraulic model was created from cross-section surveys along approximately 2.3 miles of the Gila River from the confluence of Turkey Creek to 3,300 ft downstream of Site 6 (recommended new location for diversion). A one-dimensional HEC-

RAS model was developed for the purpose of hydraulic modeling. Where the cross sections needed to be extended beyond the limits of the survey to model higher-magnitude flood events, the USGS National Elevation Dataset (NED) was used to supplement the survey data. Manning's n-values ranged between 0.035 and 0.039 in the main channel, and between 0.035 and 0.08 in the overbanks, based on observations from the field reconnaissance, past experience with similar rivers, and published values for similar roughness conditions. The model was calibrated to the available information, which included the surveyed water surface elevations at the time of the surveys (Q = 28 cfs) and the measured high water marks from the 2013 flood (Q = 28,800 cfs). The model results match both datasets reasonably well. At 28 cfs, the maximum difference between the predicted and computed water-surface elevations is about 0.8 feet, and the average difference is about 0.3 feet. As such, the model should accurately predict the hydraulic conditions over the range of flows between 28 cfs and 28,800 cfs.

The calibrated model was modified to represent project conditions by incorporating the proposed diversion structure weir geometry into the model. The cross section output for the existing conditions and project conditions are included in Appendix D.

2. SEDIMENT TRANSPORT MODELING

An incipient-motion analysis was performed for the project site. The hydraulic model was converted to a sediment-transport model using HEC-6T. The existing and with-project conditions model geometries were incorporated directly into the sediment-transport model. The Wilcock and Crowe sediment-transport equation was selected for use in this study. Based on the computations using the overall reach-averaged hydraulic information, the bed begins to mobilize at about 1,600 cfs; it becomes fully mobilized at about 5,000 cfs (between the 2- and 5-year peak discharges). These results are somewhat different than the incipient motion calculations presented in MEI (2006), which indicated particle mobility occurs at about 2,500 cfs, and significant transport occurs at about 4,000 cfs, because that study focused on the much shorter reach from Sta 37+00 to Sta 57+00. Using the predicted reach-averaged hydraulic conditions through that portion of the study reach, the incipient motion calculations yield results that are very similar to the estimates presented in MEI (2006), with particle mobility occurring at about 2,900 cfs and significant transport at about 4,000 cfs. As expected, particle mobility occurs at lower discharges under the higher energy conditions at the geomorphic controls (i.e., bedrock outcrops, valley constrictions), with

mobilization beginning at about 1,100 cfs and measureable transport at discharges on the order of the 2-year event (2,200 cfs). More details are provided in Appendix D.

3. MODELING RESULTS

Results from the model for project conditions were compared to the results from the model for existing conditions to evaluate the effects of the proposed diversion structure on the hydraulic conditions at and upstream of the structure. The comparison of the water-surface elevations indicates that, compared to a weir that would span the entire cross section, the impact of the proposed diversion structure would be relatively minor, resulting in reduced water-surface elevations at and upstream from the structure over a distance of about 1,600 feet. Water-surface elevations in the vicinity of the weir are within 0.5 feet of the existing conditions water-surface elevations at discharges less than about 1,930 cfs (2-year peak flow) but are up to 1 foot lower at discharges greater than the 2-year event due to channel widening associated with the excavation along the left (east) bank and grading of the mid-channel bar that would be necessary to install the low profile weir. See Figure 4.11 in Appendix D for computed water surface elevations in the vicinity of the proposed diversion structure under existing and project conditions. The structure would lower main channel velocities by as much as 1 fps immediately upstream from the structure at the lower discharges (i.e., less than 1,930 cfs), due to backwater effects from the structure and increase velocities by as much as 1.8 fps at the higher discharges (i.e., greater than 1,930 cfs), because of the critical depth conditions that occur across the weir.

Under project conditions, the water-surface elevations are reduced primarily due to the in-channel and bank grading that would be required to install the structure. This does not necessarily indicate that pooling would not occur upstream from the structure. Considering the two-dimensional flow characteristics that would occur in the vicinity of the skewed structure, a 2-D model in the vicinity of the structure is recommended as part of the design phase.

The sediment-transport model results indicate that the overall project reach is roughly in equilibrium with the upstream sediment supply. In general, there is less than 2 feet of aggradation or degradation of the bed elevation at the majority of the cross sections, which is consistent with the magnitude of changes between the 2006 and 2014 field surveys. Results from the with-project conditions simulation indicate that, in general, the diversion structure would have relatively minor effects on the overall sediment-transport characteristics of the project reach. A comparison of the reach-wide volumes of aggradation

and degradation indicates that the diversion structure would result in very little change along the overall project reach, except during the 1978 and 1984 flood events, when the presence of the diversion structure would decrease the amount of degradation compared to existing conditions. By the end of the simulation, the total mass of degradation along the project reach increases slightly from about 6,400 tons to about 6,900 tons. The results also indicate that the diversion structure would have relatively little effect on the degree of aggradation and degradation in the vicinity of the structure, and the upstream extent of these effects is about 2,300 feet above the structure. It should be noted this 1-D sediment-transport model is probably not sufficient to evaluate the localized effects of the diversion structure on aggradation and degradation patterns in the immediate vicinity of the structure; more detailed 2-D modeling would be necessary to adequately assess these effects and should be completed as part of the design phase.

F. SUSPENDED SEDIMENT RATING CURVES

Suspended sediment load rating curves were prepared for bed material load, suspended bed-material load, and fine sediment load. Then, they were summed to represent the total sediment load rating curve that would enter the tunnel. The suspended sediment load rating curves are included in Appendix D.

The total tunnel load rating curve was integrated over the hydrographs for representative water years. The annual sediment load delivered to the tunnel in an average year is approximately 22 AF. Based on the ranked water-delivery volumes provided by ISC, water year 1998 (average year) represents a 56 percent ranking of the years between water year 1936 and water year 2012. Of the years between 1959 and 1967 (the period for which USGS suspended sediment data was recorded), water year 1964 (52 percent rank) and water year 1965 (47 percent rank) would probably result in similar sediment delivery to the tunnel. More details regarding wet and dry years are presented in Section V.E.1.

Results from Tetra Tech's analysis were compared to USGS measured data. This comparison indicates the estimated suspended sand loads match the measured sand loads reasonably well with the predicted curve falling within the scatter of measured data, so the bed-load capacities predicted by the model are also within reason. USGS suspended sediment data is discussed in more detail in Appendix D. Tetra Tech primarily focused on looking at bed load, while USGS was looking at suspended sediment load. The two can be related. Tetra Tech estimates of suspended sediment align well with USGS estimates.

G. DISCUSSION OF FINDINGS

The proposed surface diversion location, Site 6, is located in a reach that is stable laterally and vertically. A hydraulic model and incipient motion analysis were completed for the reach of the river including the diversion site. The comparison of the water-surface elevations indicates that, compared to a weir that would span the entire cross section, the impact of the proposed diversion structure would be relatively minor. The sediment-transport model results indicate that the overall project reach is roughly in equilibrium with the upstream sediment supply. The annual sediment load delivered to the tunnel in an average year is approximately 22 AF.

IV. GEOPHYSICAL AND GEOTECHNICAL ANALYSIS

A. INTRODUCTION

BHI subcontracted with Geo-Test Inc. to perform additional geotechnical/geologic investigations for this project. The following are Geo-Test's objectives:

- Perform soil sampling and rock coring at Spar, Winn, Pope, and Sycamore Canyons.
- Perform a boring and field permeability test at the diversion site.
- Determine the depth and extent of alluvium by geophysical methods.
- Characterize rock by geophysical methods.
- Determine if the ancient landslide identified at Spar Canyon is associated with a fault.
- Perform sediment sampling at two sediment control dams, one on each side of the Gila River.
- Prepare an engineering report presenting the results of field and laboratory investigations.

B. GEOPHYSICAL FIELD WORK

1. OBJECTIVE

The purpose of the geophysical work was to provide calculated two dimensional average shear wave velocity profiles at the approximate center of each dam embankment and potential dam abutment locations and the potential diversion site. This can assist in determining likely material types for each location and measuring the depth to bedrock. In addition, two arrays were placed to observe the nature of the lineament at Spar Canyon observed in the geologic analysis that was performed as part of the April 2014 PER to determine if the ancient landslide is associated with a fault.

2. METHODS AND LOCATIONS

Ambient noise/refraction micro tremor and refraction seismic data were recorded with one geophone array at each of ten locations, including canyon channels, proposed dam centerlines, and the north and south abutment at each dam site. There were 22 to 23 geophones in each array spaced about 10 feet apart. The approximate array locations and seismic profiles are shown in Appendix E.

3. RESULTS

The shear wave velocity profiles do not indicate that there is a fault at Spar Canyon. The depth to bedrock at the diversion site was measured to be 25 feet. At the dam sites, the depth to bedrock was measured to be between 24 to 83 feet deep, as listed in Table IV-1 below.

Table IV-1 – Depth to Bedrock Measurements

Location	Depth to Bedrock (ft)
Pope Canyon Channel	75
Pope Canyon Dam Centerline	75
Pope Canyon North Abutment	Inconclusive
Pope Canyon South Abutment	53
Winn Canyon Channel	31
Winn Canyon Dam Centerline	33
Winn Canyon North Abutment	83
Winn Canyon South Abutment	75
Spar Canyon Channel	75
Spar Canyon Dam Centerline	57
Spar Canyon North Abutment	59
Spar Canyon South Abutment	24
Sycamore Canyon Channel	59
Sycamore Canyon Dam Centerline	65
Sycamore Canyon North Abutment	59
Sycamore Canyon South Abutment	51

The results presented for the Pope Canyon North Abutment appear to show that the Gila Formation conglomerate is present at the surface and likely extends to the full depth explored in the geophysical analysis. Higher velocity materials, shown as undifferentiated rock in other profile descriptions at Pope Canyon were not present under the North Abutment array location to the depth explored. This is a preliminary presentation of the results of a preliminary level of field exploration. An understanding of why the conditions exist will need to be addressed when more information than a single geophone array and a single boring is available.

The information shown in Table IV-1 is an average of conditions extent under the entire length of the arrays. Examination of the cross sections of the channel and centerline arrays also provided in the report shows that the structure underlying the arrays is not homogeneous. The arrays cross in the approximate center of each array, where a channel appears present in the centerline array cross section. When considering the varying depth

to higher velocity materials along the centerline array, the location of the channel array and the effect of averaging to produce the results shown in Table IV-1, the results are not inconsistent.

C. GEOTECHNICAL FIELD WORK AND SOIL SAMPLING

1. OBJECTIVE

The purpose of the geotechnical field work was to perform soil sampling and rock coring at Spar, Winn, Pope and Sycamore canyons.

2. METHODS AND LOCATIONS

At each dam site, exploratory borings were drilled typically including one at each dam abutment and one at the center of the dam embankment with a few exceptions. Due to steep, rugged terrain, the north abutment at Winn Canyon and the south abutment at Spar Canyon were not accessible with a track-mounted drill rig. The north abutment at Spar Canyon was not accessible, due to being contingent upon the United States Forest Service's (USFS) permission. The soils encountered in the borings were continuously examined, visually classified and logged during the drilling operation. Also, field permeability tests were performed. Selected soil samples were tested in the laboratory to determine certain engineering properties of the soils. Sieve analyses and Atterberg limits tests were performed to aid in soil classification. The boring locations and logs are included in Appendix E.

3. RESULTS

The field permeability test results range from 23 ft/yr to greater than 10,502 ft/yr in the alluvium and from 23 ft/yr to 1720 ft/yr in the Gila Conglomerate formation. While the soils demonstrate relatively high permeability rates, the soils are otherwise suitable for dam construction provided a means controlling seepage is incorporated. As such, the recommendation is to line the dam reservoir pools and upstream face of the dam embankment with a black, 60 mil, HDPE liner, or equivalent. Geosynthetic liners have been used for more than 45 years to provide waterproofing on various types of large dams around the world. Geosynthetics Magazine contains numerous examples of geosynthetic liners that function properly. According to Geosynthetics Magazine (April 2012), as of 2006, of the 265 dams incorporating geomembranes cited in the 2010 International Commission on Large Dams Bulletin (ICOLD Bulletin No. 135) on geomembrane sealing systems for dams, 183 or 69 percent are earth or rock fill dams. One example of a large dam with a 60 mil HDPE liner

is the Newmont Mining Corporation's Boddington Gold Mine Tailings Dam, southeast of Perth, Western Australia. It was completed in June 2007 and included forest clearing, rock removal, excavation and reshaping of the liner subgrade, 12 inches of compacted clay subgrade, 185 acres of 60 mil HDPE liner, and an underdrainage system. The area of that liner is similar to the area for the liner recommended at Pope Reservoir. A similar application of liner technology exists at the Banner Mill Dam in Lordsburg, NM. This is a wet tailings dam and is approximately 35 feet high and will be raised to 70 feet high. It is 100 percent lined with a 60 mil HDPE liner to prevent process water from seeping into the environment. This application was permitted by the New Mexico Office of the State Engineer, Dam Safety Bureau. Pond liner is typically black, white, tan or gray. Most liner is manufactured black from the factory. There is an extra cost to tint the liner gray or tan. There is no real advantage to recommend any liner other than black. White liners are primarily used for landfills. Gray and tan are primarily for aesthetics in New Mexico to blend in with the landscape, or near landing strips to prevent a glare.

The liner could be backfilled for aesthetics and to mitigate the risk for damage from animals and humans. On the other hand, the liner could be left exposed. There are pros and cons to both options. If left exposed, the liner can be punctured. Driving of equipment on the liners is discouraged to prevent damage. Exposed liners could also be visually inspected. If backfilled, the soil cover could slough and erode due to fluctuating reservoir levels, and inspection of the liner would be more difficult. This could lead to higher O&M costs. To protect the liner from damage due to animals and humans, the site could be secured. A patch can be used to repair any punctures as long as there is no water on top of the area needing patching. Liners are commonly not covered with soil in similar water supply projects. ICOLD Bulletin No. 135 listed 47 cases of earth or rock fill dams with liners in an exposed condition. Given these facts, backfilling the liner with soil is not recommended for this project.

The remolded permeability test results on the sediment samples from Winn and Northrup sediment ponds indicate that all of the Winn sediment and the upper 5 feet of the Northrup sediment are suitable for dam core material or clay lining. BHI subsequently estimated the amount of sediment within the Winn and Northrup sediment ponds to be 183 and 13 acre feet, respectively. The proctor densities for the sediment at Winn range from 97 to 100 pcf. At Northrup, the proctor densities for the sediment are between 104 and 123 pcf. Overall shrinkage during construction is estimated to be on the order of 25 to 30 percent. This high quality but limited quantity material can be used as a 12-inch subgrade under the

recommended liner at each dam. Additional clay material from other sources would be required to provide this subgrade for all of the recommended dam sites. The combination of the clay subgrade layer and HDPE liner provide a higher factor of safety to meet dam safety standards. The final configuration of the dam embankment and seepage control measures will be included as part of the design of the project.

Winn Canyon's sediment pool (206 AF, the largest) is more than three times the size of the Bell Canyon sediment pool (62 AF, the second largest), and almost 60 times the size of the Rodriguez Canyon sediment pool (5.9 AF, the smallest). Winn Canyon's sediment pool is approximately 90 percent full, based on BHI's estimates. This estimate was based on photographs of the outlet tower and interpolation of the area capacity (ACAP) table from the record drawings. Some of the sediment control dams have been cleaned out recently. A more detailed assessment, including field measurements would be necessary to determine the total quantity of sediment available. While the material at the Winn and Northrup sediment pools was found to be suitable for use as core material for dams, additional testing of the sediments from the other sediment pools would be necessary to determine their suitability for core materials for dams. If ISC decides to pursue diversion of AWSA water, it would be beneficial to start stockpiling this material for phased construction. Geo-Test has recommended HDPE liners for all the dam sites; therefore, zoned embankments are not necessary. Moisture vapor transmission analysis will need to be included as part of the design of the liners and the subgrade for the liners.

D. PERMEABILITY TEST

At the time this report was written, the boring and field permeability test at the diversion site had not been performed, due to being contingent upon USFS permission. The proposed location for the diversion site and hence the location for the permeability test are within land held by the USFS. The permission process is ongoing.

E. DISCUSSION OF RESULTS

None of the dam sites were eliminated based on the results of Geo-Test's field and laboratory work. Geo-Test recommends installation of impermeable liners at Spar, Winn, Pope, and Sycamore reservoirs.

V. ENGINEERING ANALYSIS

A. DIVERSION EVALUATION

1. LOCATIONS FOR SURFACE AND SUBSURFACE DIVERSION

As discussed in Section III, five potential surface diversion sites were identified prior to the site visit, and three additional potential surface diversion sites were identified during the site visit, for a total of eight potential diversion sites. Of these eight sites, Site 6 was recommended by Tetra Tech and BHI and selected by ISC for further analysis. Site 6 and the potential surface diversion sites identified during the site visit can be seen on Figure 4. The selection of Site 6 was made for several reasons. First, the river appears to be laterally stable at this location. While there is potential for lateral shifting of the overbank in the vicinity of the recommended diversion site, with the 1996 aerial image clearly showing a secondary (higher) channel in the east overbank upstream from this location, none of the historical aerial photographs show significant planform adjustments at the actual location of the proposed diversion structure dating back to 1953 (see historical aerials included in Appendix B). The secondary flow channel that appears in the 1992 and 1987 photography re-enters the main channel more than 200 feet upstream from the proposed location of the structure. In addition, BHI recommends extending the structure laterally into the east overbank, which further limits the potential for flanking. Review of historical (1953 to 2011) aerial photography of the river shows the river thalweg (line of lowest elevation corresponding with the river's path) without exception aligned against the rock outcrop which forms the right (west) bank of the river at this location. This fact combined with the proposed placement of the river diversion along the right (west) bank eliminates concern that the river would meander leaving the diversion structure disconnected from the flow. Inundation mapping for 500 cfs and for 792 cfs is included in Appendix D.

In addition to the historical aerial photographs discussed above, historical mapping dating back to 1911 was reviewed; however, the resolution was not good enough to reach any conclusion on the stability of the river. The available mapping supports the recommendation on the surface diversion at Site 6.

Also, the river appears to be vertically stable at this location for two reasons. First, repeat survey data collected from this location in 2006 and 2014 indicates that the channel geometry at Site 6 was relatively unaffected by large floods in 2008 and 2013. Second, the results of Tetra Tech's sediment-transport model also indicate that very little change in mean bed elevation occurs at the proposed location of the diversion structure, so this location is

vertically stable. Relatively high velocities are expected at this site, which would limit sediment deposition in the vicinity of the proposed diversion structure. Finally, the site has good accessibility from Turkey Creek Road which will translate to lower construction and O&M costs. The site is closer to Spar Reservoir than the more upstream potential sites which are only slightly higher in elevation. The more upstream sites would have significantly longer tunnel conveyance requirements with only minimal benefit in terms of higher hydraulic grade line and greater resultant storage in Spar.

In addition to the surface diversion site, BHI has identified one location for a subsurface diversion. The recommended location for the subsurface diversion overlaps the upper half of the surface diversion field reconnaissance reach, as seen on Figure 4.

2. DIVERSION METHODS

BHI has assessed the selected diversion site with respect to two diversion methods: (1) low profile concrete weir with tilted wedge-wire screen (commonly known as Coanda Screens) and (2) modified radial collector wells or infiltration galleries. As part of this task, sedimentation issues at existing surface and subsurface diversion structures were researched, including the Santa Fe County Buckman Direct Diversion (BDD), the Albuquerque Bernalillo County Water Utility Authority (ABCWUA) Rio Grande Intake, and the ABCWUA Radial Collector Well. Information on these existing structures may be found in Appendix G.

a) *Low Profile Concrete Weir with Tilted Wedge-Wire Screen*

In the April 2014 PER, BHI recommended construction of a low profile concrete weir with a tilted wedge-wire screen and improved intake structure for the AWSA diversion. The recommendation was made on the basis of some major advantages, including superior hydraulic and sediment control, cost effectiveness with respect to the required large capacity, and its self-cleaning properties.

The configuration of the low profile concrete weir can be seen in Figure 5. The concrete weir is skewed approximately 30 degrees from perpendicular to achieve an effective screen width of 115 ft and optimize the river and diversion hydraulics. Installing the weir at a skewed angle, as opposed to perpendicular to the channel bank, encourages sediment movement around the structure which reduces the potential for channel aggradation upstream of the diversion structure. Additionally, the skewed orientation achieves the effective screen width necessary to divert the maximum allowable water while minimizing the encroachment of the diversion structure into the left (east) overbank. The

selected orientation and location of the low profile concrete weir provides hydraulic conditions conducive to bypassing flows in accordance with the AWSA while diverting up to 350 cfs from the flows higher than the required bypass flows. The geomorphic advantages to this orientation are discussed in Tetra Tech's geomorphologic report, which is included as Appendix D to this report. This orientation also provides an anchoring point to the bedrock outcrop for the west end of the structure. The east end and entire length of the structure would also be anchored with piles driven to bedrock below. On the east end of the structure, BHI and Tetra Tech recommend including armored bank protection upstream of the structure to guide the flow over the diversion and to protect the east end from localized scour/erosion. The extent of this bank protection should be determined along with other diversion design details as part of the design phase.

The Coanda Effect Screen Analysis Software program, public domain software from the BOR, was used to determine the preliminary screen size of the Gila diversion structure. Based on the Coanda program, a screen with a width of 115 ft and a screen length of 4 ft at a 45 degree decline is capable of diverting 350 cfs with a 1.5 factor of safety. The preliminary screen spacing was determined to be 0.5 mm. As such, debris and sediment greater than 0.5 mm in size will not be intercepted or conveyed by the diversion and conveyance tunnel. The final dimensions and configuration of the screen should be established as part of the design phase based on additional analysis and study including two-dimensional hydraulic modeling. A physical model of the diversion structure is also recommended as part of the design phase. The Coanda program calculations are included in Appendix G. Architectural renderings of the diversion structure are included in Appendix K.

b) *Modified Ranney Well or Infiltration Gallery Configurations*

Due to concerns raised related to the efficacy of the surface diversion, ISC asked BHI to revisit infiltration galleries as a diversion alternative. As part of this task, BHI reviewed "Desalination and Water Purification Research and Development Program Report No. 151, Research and Development for Horizontal/Angle Well Technology" (BOR, October 2008). That report describes radial collector wells and infiltration galleries, as well as angle wells and horizontal directional drilled wells.

(1) Modified Ranney Well Configurations

BHI also investigated modified Ranney well configurations. In the April 2014 PER, subsurface diversions were not recommended, due to their susceptibility to plugging in high

flow rates, which is when this diversion would be operating. Infiltration systems also generally do not produce the maximum capacity required for this project. The anticipated amount of infiltration pipe and overall footprint for an infiltration system was estimated to impact approximately 6.5 miles of river, which was believed to be excessive to achieve the required flows resulting in considerable adverse environmental impact during and after construction. The BOR report states that radial collector wells have a central caisson that must be located very close to a surface water source and may range from 8 to 20 feet in diameter. The caisson can be 120 to 260 feet in depth with six to eight (or more) horizontal laterals radiating outward from near the bottom of the vertical shaft. Currently, the length of laterals used for water supply is typically 127 feet to 240 feet. In permeable aquifers with a constant recharge source, collector wells are capable of very high production rates of 2 to 92 mgd (4 to 171 cfs) and have been used for both fresh water and seawater intakes. The presence of cobbles and boulders can prohibit, or severely limit, the distance to which the horizontal pipes can be advanced outward from the central caisson. If radial collector wells remain a viable option as the project moves forward, a geotechnical investigation will be required including investigative drilling that specifically develops information on depths to bedrock and in-situ hydraulic conductivity. That data will allow for a quick and definitive assessment of overall collector well feasibility.

(2) Infiltration Gallery Configurations

An infiltration gallery intake system consists of horizontal collector screens placed in permeable aquifer materials, usually adjacent to a water body or beneath its bed. Screens are placed in open trenches and backfilled with appropriate filter materials. Because they are constructed via open excavation, they are generally limited to depths of approximately 25 feet. Additionally, infiltration galleries can be very difficult to maintain due to incrustation caused by iron bacteria that thrive in an aerated environment and their tendency to clog if over-pumped or if left inactive. Another challenge related to infiltration galleries is river bed scour. More detailed analysis will be required to determine the frequency of plugging of the system. Unplugging of the system could include frequent cleaning of the perforated pipes. Rehabilitation of the system would include removal of fine sediment from the infiltration trench backfill material.

With the assistance of John Shomaker & Associates, Inc. (JSAI), BHI has assessed the capacity of infiltration gallery-type structures on the Gila River for diversion at a rate of 150 cfs and a rate of 350 cfs using only gravity flow to Spar Canyon. JSAI developed a multilayer groundwater flow model along the subject reach of the Gila River to simulate

maximum diversions from infiltration galleries. Given that there are no site specific hydraulic data for the alluvial sediments, JSAI performed the simulations using assumed hydraulic conductivity values intended to be representative of low, medium, and high hydraulic conductivity for alluvial sediments in this hydrogeologic setting. JSAI recommends using a medium hydraulic conductivity of 50 ft/day for this analysis. If site-specific permeability values become available, this estimate could be refined.

JSAI has evaluated the extent of infiltration galleries that are needed to produce the desired flow rates. The location of the infiltration galleries was based on topographic maps and aerial photographs where width, and presumably depth, of the alluvium is greatest. Based on JSAI's evaluation, a subsurface diversion to divert 350 cfs is not deemed feasible, due to the hydraulic conductivity that would be required (over 200 ft/day).

JSAI has provided a conceptual design for the configuration of the infiltration galleries. Perforated pipes are recommended to be at least 20-inch in diameter and have a minimum open area of 45 square inches per foot. High-strength low-alloy or a more corrosive-resistant pipe is recommended. The total length of the perforated pipe at each lateral drain should be at least 1,200 feet. The total length of pipe for the system is 19,200 feet. This configuration can produce 150 cfs. As such, a subsurface diversion alone would not produce the required diversion flow rate of 350 cfs, and a combined subsurface and surface water diversion would be required to achieve 350 cfs. The configuration can be seen in Figure 6. This configuration extends from Turkey Creek to approximately one mile downstream, impacting approximately 100 acres of the river valley. JSAI has summarized their work in a technical memorandum, which is included as Appendix F. This structure would cost approximately 31 million dollars (See Appendix J).

3. DISCUSSION

Below is a matrix listing advantages and disadvantages of each diversion method, such as sedimentation issues, amount of harvestable water, and geomorphologic and geophysical considerations.

Table V-1 – Diversion Method Comparison Matrix

Diversion Method	Advantages	Disadvantages
Low Profile Concrete Weir with Tilted Wedge-Wire Screen	Self-cleaning properties, Superior hydraulic control, Substantial harvesting capacity, Minimal structure footprint, Low maintenance, Cost-effective	Sediment removal from harvested water, Susceptible to algae growth in low flow conditions
Modified Radial Collector Well or Infiltration Gallery	Located in floodplain as opposed to in the main river channel, Ability to harvest water regardless of river stage	Large footprint, Limited harvesting capacity, Susceptible to plugging,

4. STORAGE VOLUMES IN THE SELECTED STORAGE SITES

The storage volumes in the selected storage sites were calculated from elevation storage curves based on the recommended storage configuration at each site (described in more detail in Sections V.B and V.C, below). Due to the recommendation from Geo-Test for impermeable liners at each storage site, BHI has revised the grading at each site to reflect the surface preparation required for each liner. The revised grading has increased the storage volume at Winn, Pope, and Sycamore canyons by a total of 1,930 AF. As these three canyons plus the storage at Spar provide more than 65,000 AF of total storage, BHI recommends removing Maldonado and Dix from consideration for storage. From the Soil Conservation Service (SCS) Map for Gross Annual Lake Evaporation for New Mexico (1972), the lake evaporation rate in the project area is approximately 60 inches per year. The evaporation rates for each dam site have been included in the elevation area capacity (ACAP) tables in Section V.C.

The surface diversion at Site 6 puts the upstream HGL at an elevation of 4736 ft, which is the elevation of the bottom of the tilted wedge-wire screen. From this point, a 108 inch diameter pipe will cross the river floodplain and then follow a tunnel at a slope of 0.05 percent through the ridge to Spar Canyon (See Figure 5). The design flow rate for the conveyance to Spar reservoir is 350 cfs for each alternative.

Downstream of Spar where flow is split to fill Winn and Pope followed by Sycamore, design flows were assigned based on a proportion of storage volume served by the conveyance to each reservoir. The design flow rates for each conveyance are given in tables in the following discussion of each alternative.

All pressure piping for all alternatives was sized using the Hazen and Williams empirical formula. Profiles for use in developing and evaluating the alternatives were drawn along the conveyance alignments using the New Mexico statewide mapping that was completed by BHI in 2005.

The subsurface diversion as shown in Figure 6 achieves the same upstream HGL for the system as the surface diversion. The HGL calculations are included in Appendix G. The total storage achieved with the subsurface diversion is equal to the total storage achieved with the surface diversion alone.

B. REVISITING UPSTREAM STORAGE SITES

Storage of water near the top of the Cliff-Gila valley is critical to provide water for irrigation and to replenish the river. During irrigation season, it is not uncommon for the river

to run dry between the Fort West and Gila Farm irrigation diversion sites. As part of this project, BHI was tasked with revisiting storage sites near the top of the Cliff-Gila farming valley, specifically Spar and Winn. In addition to storage of water, removal of sediment from the system is critical for system functionality. To this end, BHI has evaluated alternatives for Spar Canyon as a sediment removal facility, as this is the first storage facility in each alternative; the tunnel to Spar in gravity alternatives will have a constant slope down from the diversion to Spar. The tunnel will have no sags or low points that could potentially trap sediment.

1. SPAR CANYON

One of the objectives of this project is to develop additional gravity alternatives for Spar Canyon to maximize its function for both sediment removal and water storage. Spar is the furthest upstream candidate site that BHI has investigated and has the most potential to benefit the farmers in the Cliff-Gila valley, as well as to replenish the river. As such, additional alternatives have been developed for Spar Canyon.

a) *Methods and Possibility of Maximizing Storage*

BHI has evaluated options for maximizing storage at Spar by expanding into adjacent canyons, excavating the reservoir pool, and constructing multiple berms at an elevation that maximizes the availability of that water for irrigation and to replenish the river. These alternatives all include a tunnel from the recommended diversion site to minimize cost and head losses from the diversion to Spar Canyon. A summary of these alternatives is included in Appendix G.

Based on the recommended diversion location at Site 6, the WSEL at Spar reduced from 4755 to 4713. The reduction in WSEL is primarily a result of diverting water at a lower point along elevation and secondarily minimizing tailwater conditions at the tunnel outlet to Spar to improve sediment movement. Based on recommendations from Geo-Test, Spar will require an impermeable liner, which will require grading of the ground surface. Figure 7 shows the conceptual grading plan for the recommended Spar grading alternative.

Spar Canyon reservoir alternatives were also evaluated assuming the water supply to the reservoir was pumped from a diversion site. When considering a pump, the maximum water surface elevation is not limited by a gravity feed from the diversion site; rather, it is dictated by site topography. A total of four pumping options for Spar were evaluated. Table V-2 below shows the storage capacity and maximum water surface elevation for each of the options, compared to Alternative 1 from the April 2014 PER.

Table V-2 – Spar Storage Alternatives Based on Pumping from the Gila River

Alternative	Maximum WSEL (ft)	Approximate Maximum Storage (AF)
Proposed (April 2014 PER)	4,755	2,513
Pumping Option 1	4,820	9,500
Pumping Option 2	4,880	14,200
Pumping Option 3	4,900	10,000
Pumping Option 4	4,940	22,000

The options based on pumping presented above were analyzed in ArcGIS using the National Elevation Dataset from the USGS. The Proposed (April 2014 PER) option is the proposed grading plan for Spar Canyon provided in the April 2014 PER, which is 525 ft east of Turkey Creek Road. Pumping Option 1 is in the same location as the proposed option with the embankment raised to where storage is limited by topography at that location, rather than limiting storage by the maximum WSEL provided by the diversion. Pumping Option 2 relocates the embankment approximately 2,200 ft east of Turkey Creek Road, Pumping Option 3 relocates the dam embankment approximately 3,500 ft east of Turkey Creek Road, and Pumping Option 4 is in the same location as Pumping Option 3 but requires a secondary embankment along the south side to prevent water from spilling into Maldonado canyon. Figure 8 shows the reservoir footprints for the revised storage options presented above. Based on the analysis above, Spar Pumping Option 4 provides the most storage capacity and is the recommended storage site for the pumping option. The evaluation of pumping alternatives is discussed in more detail in Section V.D.2.

b) *Maximized Storage Capacity and Inundated Area*

The storage at Spar has been maximized under both gravity and pumping scenarios. Table V-3 below lists all the alternatives for maximizing the storage at Spar, including the inundated area.

Table V-3 – Spar Storage and Inundation Areas

Alternative Type	Alternative Name	Maximum WSEL	Storage (AF)	Inundated Area (acres)
Initial	Proposed (April 2014 PER)	4,755	2,513	65
Final	Recommended (Gravity)	4,713	1,642	38
Initial	Pumping Option 1	4,820	9,500	186
Initial	Pumping Option 2	4,880	14,200	261
Initial	Pumping Option 3	4,900	10,000	217
Initial	Pumping Option 4	4,940	22,000	338
Final	Recommended (Pumping)	4,960	46,037	283

Pumping Option 4 was the largest volume calculated in GIS. This is the option that is recommended for more detailed analysis, including the HGL analysis, as well as quantity and cost estimates. The recommended pumping option is the result of that more detailed analysis, and is the alternative that is included in the cost estimate for Alternative 3.

c) *Conveyance Alignments*

The gravity conveyance alignment from the gravity diversion to Spar Reservoir is recommended to be a tunnel through the ridge on the north side of Spar. The conveyance alignment from the pump diversion to Spar Reservoir is discussed in Section V.D.

d) *Conceptual Layout as Settling Basin*

To determine the required size and configuration for Spar to function as a settling basin in conjunction with a low profile concrete weir and tilted wedge-wire screen gravity diversion, three common settling basin design criteria were used. The first is based on the inflow rate and required surface area to settle a particular size particle. The second is the reservoir length to width ratio. The third is based on the amount of time the water remains in the reservoir and is a function of the inflow and outflow rates and the storage volume.

With the vast majority of the river sediments being filtered out by the tilted wedge-wire screen, only very fine particles passing through the screen are of concern with respect to the Spar settling basin. Wedge-wire screen spacing typically ranges between 0.3 and 0.5 mm for surface diversion structures. Therefore, depending on the final design for the wedge-wire screen, the proposed surface diversion structure is capable of removing sediments down to a particle size of 0.3 mm. For purposes of this study, a screen opening size of 0.5 mm has been selected, and as such particles smaller than 0.5 mm will pass through the diversion structure screen openings, into the intake structure trough, and then into the conveyance through which they will be discharged into the Spar Canyon reservoir.

The required size of a settling basin is a function of the settling velocity for the design particle to be removed through the process of sedimentation. The removal of solids from the water column by settling is described by Stokes' Law as stated below:

$$V = \frac{g\left({}^{\rho_1}/_{\rho} - 1\right)d^2}{18v}$$

Where:

V = settling velocity of the solid in ft/s

g = acceleration of gravity in ft^2/s

ρ_1 = mass density of the solid in slugs per cubic foot

ρ = mass density of the fluid in slugs per cubic foot

d = diameter of the solid (in feet, assuming spherical)

v = kinematic viscosity of the fluid, in ft^2/s

The density of water is nearly constant, though the kinematic viscosity varies with temperature. The colder the water, the slower the settling velocity and hence the larger settling basin required. The settling velocity of water at 0 degrees C (32 degrees F) is about 43 percent that for water at 40 degrees C (90 degrees F). For purposes of this study, the water temperature was conservatively assumed to be 0 degrees C. Settling velocities for various size particles, estimated using Stokes' Law and 0 degree C water are provided in Table V-4 below. From this and the following equation that relates settling velocity to settling basin surface area, it is possible to determine the minimum required surface area to settle various size particles. Equation 2 from the California State Water Resources Control Board's Sediment Basin Sizing guidance document relates the inflow rate, settling velocity of the design particle, and surface area of the basin.

$$A_s = \frac{1.2Q}{V_s}$$

Where:

A_s is the minimum surface area for trapping soil particles of a certain size in sf

V_s is the settling velocity of the design particle in fps

Q is the inflow rate in cfs

and 1.2 is a factor of safety to account for reduction in basin efficiency caused by turbulence and other non-ideal conditions.

From this equation, the minimum required surface area, based on the design inflow rate of 350 cfs, was calculated for various size particles and their associated settling velocities. See Table V-4 below.

Table V-4 – Minimum Surface Areas Required to Settle Particles

Target Particle Size (mm)	Particle Class Name	Inflow Rate, Q_{out} (cfs)	Settling Velocity, V_s (ft/s)	Surface Area, A_s (ft^2)
0.1	Very Fine Sand	350	0.0140	**30,000**
0.075	Very Fine Sand	350	0.0079	**54,000**
0.05	Coarse Silt	350	0.0035	**120,000**
0.015	Medium Silt	350	0.00032	**1,330,000**
0.01	Fine Silt	350	0.0001	**2,992,000**

The Buckman Direct Diversion sediment removal system (see Appendix G) has grit classifiers designed to remove particles 0.1 mm and greater prior to the flow being introduced into the water system for conveyance to the water treatment plant. As shown in Table V-4, this equates to a surface area of 30,000 sf as a minimum for Spar Reservoir to settle out a particle size of 0.1 mm, given the maximum inflow of 350 cfs. Based on the proposed configuration for Spar Reservoir, Table V-5 lists the surface areas that will be achieved, with 30,000 sf being realized at elevation 4644 or a depth of approximately 2 feet.

Table V-5 – Spar Reservoir Achieved Surface Areas

Elevation (feet)	Surface Area, A_s (ft^2)
4642 (Reservoir Bottom)	**0**
4644	**30,000**
4645	**114,571**
4650	**372,717**
4655	**650,292**
4660	**734,027**
4665	**819,181**
4670	**905,814**
4675	**993,967**
4680	**1,083,540**
4685	**1,174,490**
4690	**1,266,798**
4695	**1,360,451**
4700	**1,455,441**
4705	**1,551,770**
4710	**1,649,437**
4715	**1,748,438**
4720	**1,848,773**
4724 (Top of Dam)	**1,927,976**

To settle a design particle of 0.015 mm, medium silt, for the maximum inflow of 350 cfs, which represents the worst case in terms of requiring the largest surface area, a minimum surface area of 1,330,000 sf is required. This is achieved at approximately elevation 4693. At elevation 4713, corresponding with the proposed invert of the tunnel outlet, the achieved surface area is approximately 1,700,000, which would settle particles 0.015 mm and larger.

Second, for the settling basin to function properly, the basin configuration needs to be such that the length-to-width ratio is between 2:1 and 5:1, although 3:1 is used as the minimum length-to-width ratio to ensure settling of the design particle. The length-to-width ratio for Spar at a WSEL of 4713 is approximately 4:1.

The third criterion is retention time. At the elevation corresponding with the invert of the tunnel outlet, 4713, the storage volume is 1,642 AF, considering sediment storage (See "Maintenance Issues and Solutions" below). For a maximum inflow rate of 350 cfs if the reservoir is empty, the time to fill the reservoir to elevation 4713 is approximately 49 hours. Once full to this level as flow continues to enter the reservoir, flow will be released to the downstream system. Assuming the maximum inflow 350 cfs with a corresponding outflow of 350 cfs, it will take approximately 57 hours for water to travel across the reservoir. The flow through Spar reservoir is controlled by the configuration of the reservoir. The downstream outlet pipe has been sized to pass 350 cfs. Both of these measures of retention time satisfy the hydraulic retention time of at least 24 to 72 hours recommended by the California State Water Resources Control Board in the California Best Management Practice Construction Handbook.

e) *Maintenance Issues and Solutions*

Based on Tetra Tech's analysis of sediment load delivery to the tunnel, the annual sediment load delivered to Spar Canyon in an average year is expected to be approximately 22 AF. This sediment will be stored in Spar Canyon and removed approximately every ten years. As such, the volume of Spar Canyon has been reduced by 220 AF to account for sediment storage. Heavy equipment cannot drive on the HDPE liner that is recommended at Spar; therefore, an 18 inch thick soil cement grade control slab across the bottom of the reservoir is recommended as the impermeable liner along the bottom of the dam pool at Spar, in place of the HDPE liner. This will facilitate sediment removal and provide maintenance crews a fixed surface to ensure the reservoir is not over excavated during sediment removal. Below are two photos of an existing dam in the Albuquerque area with a soil cement grade control slab across the bottom of the reservoir.

Photo 1 – Sediment Removal Activities in a Dam with a Soil Cement Grade Control Slab

Photo 2 – Soil Cement Grade Control Slab after Cleaning

f) *Outlet Works*

Conceptual outlet works for Spar Canyon reservoir had to meet four requirements:

1. Provide up to 90 cfs of water supply for environmental purposes in the Cliff-Gila Valley
2. Allow for sediment retention in the Spar Canyon reservoir
3. Provide a maximum outflow rate to the downstream reservoirs of 350 cfs

The proposed outlet works consist of an inclined intake structure placed on the upstream embankment. Laying the intake structure along the embankment reduces costs by using the embankment as a support mechanism for the structure, as opposed to a free-standing vertical structure. Nichols and McClure Dam outlet structures in Santa Fe, New Mexico were recently renovated by BHI using this similar design concept. A photograph of one of these structures is included on Figure 9.

The proposed orifice openings for the Spar Canyon outlet works are spaced at 20 vertical feet increments and are sized to convey 90 cfs. Additionally, the final orifice opening near the emergency spillway of the dam is sized to convey the maximum flow rate of 350 cfs to prevent accidental overtopping of the dam. Configuring the outlet works in the manner described above allows Spar Canyon to be used as a sediment trap while still adhering to the other design constraints. Figure 9 shows the conceptual layout of the outlet works at Spar Canyon reservoir.

While Spar Canyon has been evaluated as the primary reservoir from which water will be supplied for environmental purposes, Winn Canyon could be used in this way as well. This provides an operational flexibility with water stored at both Spar and Winn Canyons to meet agricultural and environmental needs. This can depend on the time of year as well as the amount of water available at Spar and Winn Canyons. This will need to be evaluated more fully during future phases of work.

g) *Revised Total Cost Estimates*

The cost estimates for Spar Canyon reservoir have been revised to include the outlet works, impermeable liner, and soil cement grade control slab for both gravity and pumping alternatives. The revised cost estimates are included in Appendix J and summarized in Section VIII.

2. WINN CANYON

Additional alternatives have been developed for Winn Canyon to maximize the storage capacity, as it is near the top of the Cliff-Gila farming valley and can supply water to most of the farms by gravity (all farms if the reservoir is more than 20 percent full). If a pump is included at Winn, this canyon can supply water to the top of the Cliff-Gila farming valley at the location of the combined diversion for Upper Gila and Fort west, as recommended in the April 2014 PER in times when the reservoir is not as full. Pumping from Winn Canyon to the upstream valley is discussed in further detail in Section V.D.

a) *Methods and Possibility of Maximizing Storage*

BHI has evaluated expanding the volume of Winn Canyon into adjacent canyons, excavating the reservoir pool, constructing multiple berms, and extending the embankment as proposed in the April 2014 PER to the south to maximize the storage at an elevation that maximizes the availability of that water for irrigation and to replenish the river with at least 10 cfs. A summary of these alternatives is included in Appendix G.

Based on recommendations from Geo-Test, Winn will require an impermeable liner to prevent seepage losses. This liner will require grading to smooth out the surface upon which it will be placed. Figure 10 shows the revised grading plan for Winn Canyon.

b) *Maximized Storage Capacity and Inundated Area*

The storage at Winn has been maximized under both gravity and pumping scenarios. The maximized storage in both scenarios, based on the grading shown in Figure 10, is 10,713 AF at a WSEL of 4,685 ft. The storage pool for Winn Canyon would extend to the west, beyond the existing sediment control dam in Winn Canyon. The cost estimate for the storage dam in Winn Canyon includes a new stormwater detention and sediment removal facility upstream of the AWSA storage facility. Architectural renderings showing the proposed embankment location, as well as the inundation limits for Winn Canyon, are included in Appendix K.

c) *Conveyance Alignments*

The conveyance alignment from Spar to Winn is recommended to follow Highway 293 from Spar to J4 (approximately at the mouth of Guerrero Canyon, between Maldonado and Garcia Canyons), then to go west and cross under the Gila River approximately three miles upstream of the NM 211 bridge. The conveyance from Spar to Winn is recommended to be in the same location for both gravity and pumping alternatives. The conveyance

specifications from the Spar to Winn, both for gravity and pumping options are discussed in Section VI.

d) *Revised Total Cost Estimate*

The cost estimates for Winn Canyon reservoir have been revised to include the impermeable liner, a pump station, and a 20 inch pipeline from Winn Canyon to the Upper Gila Ditch and the Gila River (See Section V.D.1.b) below). The revised cost estimates are included in Appendix J and summarized in Section VIII.

C. ELEVATION AREA CAPACITY TABLES FOR THE SELECTED RESERVOIRS

Below are the elevation area capacity (ACAP) tables for the selected reservoirs for both gravity and pumping scenarios. Alternatives 1, 2 and 3 are described in detail in Section VI, below. The tables are based on the proposed grading at each site, which is included in Appendix G. Based on the results of the geotechnical lab work, impermeable liners will be required at Spar, Winn, Pope, and Sycamore reservoirs. The grading for the reservoir pools has been revised to reflect the earthwork that will be required for the liners.

Table V-6 – ACAP Table for Spar (Alternatives 1 and 2)

Spar Canyon						
Elevation	**Incremental Volume (cu ft)**	**Cumulative Volume (cu ft)**	**Acre-Feet**	**Storage Volume with Sediment (AF)**	**Surface Area (sq ft)**	**Annual Evaporation (AF)**
4,645	103,837	103,837	2	-	114,571	13
4,650	1,304,460	1,408,296	32	-	372,717	43
4,655	2,610,555	4,018,851	92	-	650,292	75
4,660	3,460,232	7,479,083	172	-	734,027	84
4,665	3,882,418	11,361,501	261	41	819,181	94
4,670	4,311,850	15,673,351	360	140	905,814	104
4,675	4,748,840	20,422,191	469	249	993,967	114
4,680	5,193,190	25,615,381	588	368	1,083,540	124
4,685	5,644,508	31,259,889	718	498	1,174,490	135
4,690	6,102,658	37,362,547	858	638	1,266,798	145
4,695	6,567,565	43,930,112	1,008	788	1,360,451	156
4,700	7,039,173	50,969,285	1,170	950	1,455,441	167
4,705	7,517,469	58,486,755	1,343	1,123	1,551,770	178
4,710	8,002,460	66,489,215	1,526	1,306	1,649,437	189
4,715	8,494,130	74,983,345	1,721	1,501	1,748,437	201
4,720	8,992,467	83,975,812	1,928	1,708	1,848,773	212
4,724	7,364,391	91,340,203	2,097	1,877	1,927,976	221

Table V-7 – ACAP Table for Spar (Alternative 3)

Spar Canyon					
Elevation	**Incremental Volume (cu ft)**	**Cumulative Volume (cu ft)**	**Acre-Feet**	**Surface Area (sq ft)**	**Annual Evaporation (AF)**
4,710	375,741	375,741	9	371,566	43
4,715	5,485,831	5,861,572	135	2,079,973	239
4,720	16,855,752	22,717,324	522	4,501,083	517
4,725	23,486,138	46,203,462	1,061	4,773,332	548
4,730	24,212,982	70,416,444	1,617	4,911,992	564
4,735	24,908,251	95,324,695	2,188	5,051,441	580
4,740	25,607,492	120,932,187	2,776	5,191,689	596
4,745	26,310,735	147,242,922	3,380	5,332,740	612
4,750	27,018,018	174,260,940	4,000	5,474,602	628
4,755	27,729,353	201,990,293	4,637	5,617,274	645
4,760	28,444,735	230,435,028	5,290	5,760,756	661
4,765	29,164,195	259,599,223	5,960	5,905,058	678
4,770	29,887,751	289,486,974	6,646	6,050,179	694
4,775	30,615,404	320,102,378	7,349	6,196,120	711
4,780	31,347,188	351,449,566	8,068	6,342,894	728
4,785	32,083,141	383,532,707	8,805	6,490,502	745
4,790	32,823,279	416,355,986	9,558	6,638,949	762
4,795	33,567,619	449,923,605	10,329	6,788,239	779
4,800	34,316,178	484,239,783	11,117	6,938,373	796
4,805	35,068,976	519,308,759	11,922	7,089,360	814
4,810	35,826,057	555,134,816	12,744	7,241,206	831
4,815	36,587,438	591,722,253	13,584	7,393,915	849
4,820	37,353,170	629,075,424	14,442	7,547,500	866
4,825	38,123,303	667,198,727	15,317	7,701,969	884
4,830	38,897,881	706,096,608	16,210	7,857,333	902
4,835	39,676,957	745,773,564	17,121	8,013,600	920
4,840	40,460,567	786,234,131	18,049	8,170,780	938
4,845	41,248,772	827,482,903	18,996	8,328,883	956
4,850	42,041,641	869,524,544	19,962	8,487,932	974
4,855	42,839,267	912,363,810	20,945	8,647,935	993
4,860	43,641,724	956,005,534	21,947	8,808,924	1,011
4,865	44,449,229	1,000,454,763	22,967	8,970,939	1,030
4,870	45,261,902	1,045,716,665	24,006	9,133,996	1,048
4,875	46,079,811	1,091,796,475	25,064	9,298,106	1,067
4,880	46,903,024	1,138,699,500	26,141	9,463,282	1,086
4,885	47,731,580	1,186,431,080	27,237	9,629,529	1,105

Spar Canyon					
Elevation	**Incremental Volume (cu ft)**	**Cumulative Volume (cu ft)**	**Acre-Feet**	**Surface Area (sq ft)**	**Annual Evaporation (AF)**
4,890	48,565,520	1,234,996,599	28,352	9,796,861	1,125
4,895	49,404,867	1,284,401,467	29,486	9,964,975	1,144
4,900	50,234,438	1,334,635,905	30,639	10,128,879	1,163
4,905	51,055,148	1,385,691,054	31,811	10,293,259	1,182
4,910	51,878,319	1,437,569,373	33,002	10,458,164	1,200
4,915	52,704,636	1,490,274,009	34,212	10,623,926	1,219
4,920	53,537,947	1,543,811,956	35,441	10,791,733	1,239
4,925	54,415,189	1,598,227,145	36,690	10,979,834	1,260
4,930	55,399,119	1,653,626,264	37,962	11,176,547	1,283
4,935	56,322,149	1,709,948,412	39,255	11,352,702	1,303
4,940	57,209,873	1,767,158,285	40,568	11,531,937	1,324
4,945	58,118,258	1,825,276,543	41,903	11,716,001	1,345
4,950	59,049,221	1,884,325,764	43,258	11,904,579	1,366
4,955	60,010,452	1,944,336,216	44,636	12,102,060	1,389
4,960	61,043,362	2,005,379,578	46,037	12,319,048	1,414

Table V-8 – ACAP Table for Winn

Winn Canyon					
Elevation	**Incremental Volume (cu ft)**	**Cumulative Volume (cu ft)**	**Acre-Feet**	**Surface Area (sq ft)**	**Annual Evaporation (AF)**
4,595	249,264	249,264	6	240,480	28
4,600	3,329,197	3,578,462	82	1,043,008	120
4,605	6,568,445	10,146,907	233	1,541,230	177
4,610	8,803,737	18,950,644	435	1,997,483	229
4,615	11,183,233	30,133,877	692	2,444,972	281
4,620	13,280,735	43,414,612	997	2,876,847	330
4,625	15,489,064	58,903,676	1,352	3,326,549	382
4,630	17,884,765	76,788,440	1,763	3,818,407	438
4,635	20,415,888	97,204,328	2,232	4,364,902	501
4,640	23,332,577	120,536,905	2,767	4,963,696	570
4,645	26,330,782	146,867,687	3,372	5,594,812	642
4,650	30,102,764	176,970,450	4,063	6,468,241	742
4,655	34,235,915	211,206,365	4,849	7,181,800	824
4,660	37,172,493	248,378,858	5,702	7,673,715	881
4,665	39,488,117	287,866,976	6,609	8,131,323	933
4,670	41,913,332	329,780,308	7,571	8,593,586	986
4,675	43,849,812	373,630,120	8,577	8,944,233	1,027
4,680	45,616,293	419,246,413	9,625	9,304,376	1,068
4,685	47,425,180	466,671,593	10,713	9,666,129	1,110

Table V-9 – ACAP Table for Pope

Pope Canyon					
Elevation	**Incremental Volume (cu ft)**	**Cumulative Volume (cu ft)**	**Acre-Feet**	**Surface Area (sq ft)**	**Annual Evaporation (AF)**
4,520	606,762	606,762	14	250,413	29
4,525	1,922,220	2,528,982	58	521,341	60
4,530	3,368,772	5,897,754	135	834,639	96
4,535	5,053,796	10,951,550	251	1,191,975	137
4,540	6,933,164	17,884,713	411	1,589,311	182
4,545	9,025,483	26,910,196	618	2,026,141	233
4,550	11,271,459	38,181,655	877	2,472,816	284
4,555	13,261,490	51,443,145	1,181	2,823,434	324
4,560	14,960,548	66,403,693	1,524	3,161,705	363
4,565	16,690,647	83,094,340	1,908	3,523,667	404
4,570	18,739,563	101,833,903	2,338	3,988,353	458
4,575	21,234,988	123,068,891	2,825	4,516,452	518
4,580	24,026,690	147,095,581	3,377	5,092,537	585
4,585	26,774,009	173,869,590	3,991	5,577,167	640
4,590	28,860,605	202,730,195	4,654	5,962,300	684
4,595	30,668,131	233,398,327	5,358	6,307,355	724
4,600	32,430,551	265,828,878	6,103	6,660,245	764
4,605	34,172,255	300,001,133	6,887	7,010,972	805
4,610	35,902,752	335,903,885	7,711	7,349,090	844
4,615	37,541,398	373,445,282	8,573	7,663,173	880
4,616	6,921,387	380,366,669	8,732	7,717,531	886

Table V-10 – ACAP Table for Sycamore

Sycamore Canyon					
Elevation	**Incremental Volume (cu ft)**	**Cumulative Volume (cu ft)**	**Acre-Feet**	**Surface Area (sq ft)**	**Annual Evaporation (AF)**
4,435	20,050	20,050	0	39,966	5
4,440	1,482,148	1,502,199	34	627,093	72
4,445	5,011,252	6,513,451	150	1,397,560	160
4,450	9,137,877	15,651,328	359	2,271,598	261
4,455	13,481,781	29,133,109	669	3,138,367	360
4,460	17,723,886	46,856,994	1,076	3,926,926	451
4,465	21,569,540	68,426,534	1,571	4,716,793	541
4,470	25,751,213	94,177,748	2,162	5,579,188	640
4,475	29,928,812	124,106,560	2,849	6,368,944	731
4,480	33,694,445	157,801,005	3,623	7,116,295	817
4,485	37,620,526	195,421,531	4,486	7,948,521	912
4,490	42,070,741	237,492,272	5,452	8,906,871	1,022
4,495	47,028,625	284,520,897	6,532	9,859,397	1,132
4,500	51,116,186	335,637,083	7,705	10,546,214	1,211
4,505	54,002,666	389,639,749	8,945	11,028,920	1,266
4,510	56,197,581	445,837,331	10,235	11,457,612	1,315
4,515	58,593,779	504,431,110	11,580	12,003,597	1,378
4,520	61,576,897	566,008,007	12,994	12,632,471	1,450
4,525	64,753,982	630,761,989	14,480	13,269,405	1,523
4,530	67,931,219	698,693,208	16,040	13,876,156	1,593
4,535	70,691,561	769,384,769	17,663	14,394,179	1,652
4,540	73,196,691	842,581,460	19,343	14,880,939	1,708
4,545	75,615,072	918,196,532	21,079	15,371,021	1,764
4,550	78,120,363	996,316,896	22,872	15,877,212	1,822
4,555	80,841,782	1,077,158,678	24,728	16,454,564	1,889
4,560	83,620,426	1,160,779,105	26,648	16,998,143	1,951
4,565	86,435,877	1,247,214,982	28,632	17,570,364	2,017
4,570	89,331,512	1,336,546,494	30,683	18,180,617	2,087
4,575	92,473,938	1,429,020,432	32,806	18,792,850	2,157
4,580	95,341,635	1,524,362,067	34,995	19,338,317	2,220
4,585	98,003,009	1,622,365,076	37,244	19,863,193	2,280
4,590	100,708,112	1,723,073,188	39,556	20,425,335	2,345
4,595	103,588,249	1,826,661,437	41,934	21,010,037	2,412
4,600	104,363,336	1,931,024,773	44,330	21,581,261	2,477

D. PUMPING OPTIONS

Pumping options were evaluated for three separate and distinct functions:

1. Pumping from Winn Canyon to irrigate properties that could not be served by gravity from Winn if the reservoir was less than 20 percent full. The purpose of the Winn Canyon pump is to supply water to irrigated properties in the far north end of the irrigated Cliff-Gila Valley. Water from Winn Canyon could also be discharged to the river, just downstream of the Upper Gila Diversion, to supply the reach that periodically dries up between the Ft. West and Gila Farms diversions. Two separate options were evaluated: the first system capable of maintaining up to a minimum 10 cfs return to the river at the Upper Gila Ditch, and secondly a system capable of maintaining up to a 50 cfs return to the proposed irrigation diversion.
2. Pumping from the Gila River to Spar Canyon reservoir to maximize reservoir storage capacity at a rate up to 350 cfs maximum. The purpose of the Gila River to Spar Canyon pump is to eliminate the gravity diversion and conveyance constraints which limit the storage capacity in Spar Canyon. In a pumping scenario, storage at Spar Canyon is dictated by site topography, rather than by the elevation of the gravity diversion.
3. Pumping from Pope Reservoir southeast over the Continental Divide to supply irrigation water to Silver City, Bayard, Hurley and Deming, also known as the "Southwest Regional Water Supply" or SWRWS at a rate of 14 cfs maximum. This will require multiple pump stations in series to be used to lift to the maximum elevation necessary at the Continental Divide. The proposed Grant County Reservoir located near Bayard will enable the system flexibility in managing water storage volumes throughout the year in combination with Pope Reservoir.

1. PUMPING FROM WINN CANYON TO UPSTREAM IRRIGATION DIVERSION

It has been assumed farmers served by the Upper Gila and Fort West Ditches may need up to 60 cfs during the irrigation season from April 15 through September 15. Historically, on 102 days between 1936 and 2013, the river flow was less than 20 cfs (this is less than 0.4 percent of the days). The minimum flow recorded was 10 cfs. (The river flow was less than 30 cfs on 1355 days, or 4.9 percent of the days). Using this historical data as a basis, BHI evaluated two options to determine their viability: (1) a Winn Reservoir delivery system capable of maintaining the minimum 10 cfs supplemental flow to the river at Upper Gila Ditch either by gravity flow controlled by a flow control valve, or if the elevation of Winn

Reservoir is too low, the flow rate would be boosted up to 10 cfs by an in-line pump station; and (2) a much larger Winn Reservoir delivery system capable of delivering a 50 cfs supplemental flow, or essentially the full irrigation requirement above and beyond the historical minimum flow of 10 cfs to the Upper Gila Ditch and the Fort West Ditch either by gravity flow controlled by a hydraulic flow control valve, or if the Winn Reservoir elevation is too low, the flow rate would be boosted up to 50 cfs by an in-line pump station.

In both of these above scenarios, a local programmable controller will be required to monitor flow in the river, flow in the Winn Reservoir outlet pipe, level in Winn Reservoir, and the local farmer weekly irrigation schedule. Based on this instrumentation input to the controller it will determine how to control the automatic hydraulic flow control valve, or operate an in-line variable speed pump to maintain continuous water delivery according to the irrigation schedule assuming water is available in Winn Reservoir above the minimum level. BHI has evaluated the resulting required pump capacity, ancillary flow control equipment, and required piping alignments for the purpose of preparing a 10 percent appraisal level cost estimate.

a) *Design Criteria*

For the purposes of the pumping analysis, the minimum allowable water surface elevation in Winn Canyon is assumed to be 4,612 ft, which corresponds to approximately 10 ft of water in the reservoir. The maximum water surface elevation is 4,685 ft. The ground elevation at the discharge point from the pipeline into the Upper Gila Ditch is 4,612 ft, based on the alignment and profile shown in Figure 11. The hydraulic analysis was based on USGS elevation data, from which a high point is shown at an elevation of 4,634 ft; however, the irrigation ditch flows downhill with a uniform slope, so HGL elevations are not based on that high point constraint, but rather on the discharge elevation of 4,612.

b) *Capacity*

For the first scenario of achieving a reservoir discharge capacity of 10 cfs (4,488 gpm), a 24-inch diameter pipe size was selected based on the profile, which exhibits a maximum velocity of 5 fps and acceptable headloss based on the ground profile and pipe length of 15,640 ft. Pipe friction losses are estimated at 25 ft. Based on a profile along the irrigation ditch with an assumed slope of 0.25 percent, and an assumed efficiency of 75 percent, a 38 brake horsepower (bhp) pump attached to a 50 hp rated motor (nominal industry rating of motor not brake horsepower) will be required.

The pump will be on a variable speed drive allowing the pump to supplement flows up to 10 cfs, when needed during the irrigation season from April 15 through September 15. The available storage in Winn Canyon identified in Section V.B.2.c) above is 10,713 AF at a WSEL of 4,685 ft. Assuming that there is sufficient reservoir storage volume from April 15 through September 15, it allows gravity flow within the 24 inch pipeline to maintain the 10 cfs flow rate requirement, which would be maintained by an automatic flow control valve. If the reservoir level drops to an elevation of approximately 4,640 feet, the in-line pump will need to begin operation with varying speed to supplement the flow and gradually increase power to maintain 10 cfs. Based on a 38 bhp pump, and if five months of continuous pumping at 10 cfs is required, the maximum power consumption would be approximately 102,000 kWh per year.

Similarly, for a reservoir discharge capacity of 50 cfs (22,442 gpm), a 42-inch diameter pipe size was selected based on the profile, with a maximum velocity of 5 fps, and acceptable headloss based on the ground profile and pipe length of 15,640 ft. Pipe friction losses are estimated at 24 ft. Based on a profile along the irrigation ditch with an assumed slope of 0.25 percent, and an assumed efficiency of 75 percent, a 234 brake horsepower (bhp) pump attached to a 300 hp rated motor will be required. The reservoir can maintain a 50 cfs flow by gravity within a 42 inch pipeline until the level drops to an elevation of 4640 ft. For reservoir levels below this, the in-line pump station will need to begin operation with varying speed to supplement the flow and gradually increase power to maintain a 50 cfs flow. Based on a 234 bhp pump and five months of continuous pumping at 50 cfs, the maximum power consumption would be approximately 628,830 kWh per year. An added benefit of constructing a pipeline to handle a larger flow scenario of 50 cfs is that the resulting 42 inch pipeline would enable a 10 cfs flow to easily discharge from the reservoir nearly all the way to its lowest point of 4612 ft without the need for an in-line pump station to operate.

c) *Alignments*

The alignment for the irrigation pipeline from Winn Canyon to the Upper Gila ditch follows Box Canyon Road to the northeast for approximately 4200 ft before heading east following the Upper Gila Ditch to the proposed outlet, upstream of all land currently irrigated with water from the Upper Gila Ditch. The pipeline would discharge into the river just upstream of the diversion structure with appropriate energy dissipation and erosion control

measures. It is assumed that easements will be required for the pipeline. Figure 11 shows the proposed alignment and the associated profile.

d) *Cost Estimate*

The cost estimate for the 10 cfs irrigation pump station option at Winn Canyon includes a 15,640 LF 24 inch pipeline, pump station site improvements, one 4,500 gpm pump with a 50 hp rated motor (nominal industry rating of motor not brake horsepower) and variable speed drive, control valves, pump controller and SCADA system, protective metal building and other items, including easements. The alternative option cost estimate for the larger 50 cfs irrigation pump station at Winn Canyon includes a 15,640 LF 42 inch pipeline, pump station site improvements, one 22,400 gpm pump with a 300 hp rated motor (nominal industry rating of motor not brake horsepower) and variable speed drive, control valves, pump controller and SCADA system, protective metal building and other items, including easements. The cost estimates for both options are included in Appendix J and summarized in Section VIII.

2. PUMPING FROM GILA RIVER TO SPAR RESERVOIR

In order to maximize storage near the top of the Cliff-Gila farming valley, BHI has evaluated pumping from the river, at a maximum rate of 350 cfs, from a nearby diversion location along the Gila River to Spar Canyon.

a) *Conceptual Plan for Pump Station*

The purpose of this pump is to pump water from the river to storage. There is a similar existing diversion and pumping station to storage in the project area for Bill Evans Lake, which serves the Freeport McMoran mine (there is a second pump station from the lake to the mine). Figure 12 shows the proposed diversion location at an elevation of 4,590 ft, pump station site, and corresponding reservoir in Spar Canyon at an elevation of 4,960 ft. Based on the revised dam location at Spar Canyon to maximize storage capacity, the static lift is 350 feet at the maximum water surface level.

b) *Capacity*

Pumping will not begin until all the requirements of the AWSA are met. Then, the pumping rates could vary anywhere between 0 and 350 cfs. Pumping capacities of 100, 150, and 350 cfs (45,000, 67,500, 157,500 gpm or 65, 97, and 226 MGD) were evaluated to optimize the pipe and pump sizes. Assuming four parallel 60-inch pipelines, approximately

20,100 bhp would be required to pump water at a rate of 350 cfs from the Gila River to the revised storage location for Spar Canyon. A total of seven pumps would be required. Three of the pipelines would each have single pumps, with motors rated at 5,900 hp that would come on in step fashion as higher flows are available to be pumped from the river. The fourth pipeline would have four smaller pumps with motors each rated at 1,500 hp and would be staged to run specifically during the smaller flow ranges available from the river (below 100 cfs). Assuming all pumps are operational, producing a maximum of 350 cfs, running continuously for 24 hours a day until 65,000 AF of storage is achieved (filling all the reservoirs from empty) the pumps will run for approximately 94 days. Therefore, a peak energy consumption of 39.8 million kWh would be necessary to power the Spar pump station for an initial filling of Spar Reservoir or in a wet year following a dry period.

Based on the AWSA, the maximum instantaneous diversion rate is 350 cfs. For illustration purposes, Table V-11 summarizes the number of days pumping would theoretically occur over two 5-year periods while adhering to the AWSA constraints. These two periods were selected because they were the wettest periods in the daily flow data from October 1936 to October 2001.

Table V-11 – AWSA Pumping Days for Two 5-Year Periods

5 yr Period Start Date	0 cfs	25 cfs	26-50 cfs	51-100 cfs	101-150 cfs	151-200 cfs	201-250 cfs	251-300 cfs	301-<350 cfs	350 cfs
10/1/1936	1528	2	24	40	24	29	37	26	19	84
2/18/1958	1446	52	45	65	64	26	30	35	4	58

For the period beginning October 1, 1936, pumping would have occurred on 290 days (or roughly 15 percent of the time) at an average rate of 215 cfs. Similarly for the period beginning February 18, 1958, pumping would have occurred on 379 days (about 20 percent of the time) at an average rate of 152 cfs.

c) *Alignments*

The alignment for the 350 cfs pipeline from the Spar pump station to the maximized Spar Canyon reservoir begins with a diversion structure in the river that diverts and routes flow to the pump station. The diversion structure is assumed to be a low profile concrete weir with a tilted wedge-wire screen similar to the one discussed in Section V.A.2. From the pump station, the pipe alignment is routed northeast to the east side of Turkey Creek Road

where it then follows the road then east up Spar Canyon to the maximized reservoir location. Figure 12 shows the proposed alignment and the associated profile based on the USGS data available.

Figure 13 shows the conceptual site plan for the Spar pump station. This conceptual pump station is based on vertical turbine type pump assemblies. The station pump suction clearwell structure would be at the valley floor elevation with a pump motor room floor fifteen feet above the maximum flood stage elevation. The diversion structure pipeline will terminate at the deep clearwell adjacent to and partially constructed as part of a protective building substructure that the deep vertical pumps would suction from. All motor housings would be inside the protective metal building for weather protection and servicing. All surge control systems including tanks and control valves and flow and pressure instruments would be on the 60-inch horizontal discharge pipe prior to exiting the building and extending underground to the reservoir. Table V-12 defines the concept pump schedule.

Table V-12 – Spar Station Pump Schedule

Pump	Flow Rate (gpm)	Motor (hp)	Motor Voltage	Starter
Spar #1	10,000	1,500	4,160 VAC	Medium Voltage Soft Start
Spar #2	10,000	1,500	4,160 VAC	Medium Voltage Soft Start
Spar #3	10,000	1,500	4,160 VAC	Medium Voltage Soft Start
Spar #4	10,000	1,500	4,160 VAC	Medium Voltage Soft Start
Spar #5	40,000	5,900	4,160 VAC	Medium Voltage Soft Start
Spar #6	40,000	5,900	4,160 VAC	Medium Voltage Soft Start
Spar #7	40,000	5,900	4,160 VAC	Medium Voltage Soft Start

d) *Cost Estimate*

The cost estimate for the Spar pump station includes the diversion structure, pipeline to station, underground pump clearwell, protective metal buildings for pump motors and electrical systems, discharge pipelines, control valves and surge tanks, power distribution, SCADA control systems with instrumentation, electrical substation and miscellaneous other items. The cost estimates are included in Appendix J and summarized in Section IX.

The utility source supply from the Public Service Company of New Mexico (PNM) will be a substantial investment on their part, and a cost sharing agreement will need to be negotiated for this power delivery. PNM has prepared a high level cost estimate for the project with the information provided to them by BHI. It includes approximately 35 miles of 69 kV high voltage transmission line to bring power to the Spar pump station area and the other local SWRWS booster pump stations along US Highway 180. At that point, the 69 kV

transmission level voltage will be stepped down to the PNM distribution level voltage of 12.47 kV at two utility owned, high voltage substations. The distribution voltage will be fed to customer owned, medium voltage substations at Spar pump station and each SWRWS pump station where the PNM distribution voltage will be stepped down to the motor voltage of 4,160 volts and building service voltage of 480 volts. The costs related to the customer owned substations are outline in the cost estimates for each pump station. The costs of the PNM transmission lines, substations and distribution lines are summarized in Table V-13.

Table V-13 – PNM Power Extension Costs

Item	Cost
New 69 kV Overhead Transmission Line, Approx. 35 miles	$17,500,000
New PNM Substation near Spar Pump Station, 69 kV to 12.47 kV	$4,600,000
New PNM Substation for SWRWS Pump Stations, 69 kV to 12.47 kV	$4,600,000
Distribution Feeder Additions and Improvements	$9,500,000
Total for Electrical Utility Work	$36,200,000

If a second power line is desired to provide redundancy in case of a catastrophic failure of one power line or substation, PNM outlines the following total project cost estimate:

Table V-14 – PNM Redundant Utility Extension Costs

Item	Cost
Second New 69 kV Overhead Transmission Line, Approx. 35 miles	$17,500,000
Second New PNM Substation near Spar Pump Station, 69 kV to 12.47 kV	$2,600,000
Second New PNM Substation near Spar Pump Station, 69 kV to 12.47 kV	$2,600,000
Transmission Breaker	$1,000,000
Total for Electrical Utility Work for Redundant System	$23,700,000
Total For Project With Redundant System	$59,900,000

BHI does not recommend providing power redundancy. Therefore, the costs included in Appendix J for the Spar pump station are based on the costs in Table V-13.

e) *Re-assessing Storages and Conveyances Downstream of Spar*

The design flow rate for the conveyance to Spar reservoir is 350 cfs under any scenario. Where flow is split, design flows were assigned based on a proportion of storage volume served by the conveyance. The design flow rates for each conveyance are given in tables in the following discussion of each alternative.

All pressure piping for both gravity and pumping options was sized using the Hazen and Williams empirical formula. Profiles for use in developing and evaluating the alternatives were drawn along the conveyance alignments. Based on the grading that is required to accommodate a liner at Spar, the revised storage at Spar based on pumping is

46,037 AF. The total storage achieved in Spar, Winn, and Pope Reservoirs is 65,482 AF, just over the required 65,000 AF. As such, Sycamore is not needed under the pumping scenario.

3. PUMPING FROM POPE RESERVOIR TO SOUTHWEST REGIONAL WATER SUPPLY PROJECT

As part of the proposed Southwest Regional Water Supply Project, up to 10,000 acre-feet (AF) of water could potentially be pumped over the Continental Divide to the communities in the Mimbres Basin (Silver City, Santa Clara, Bayard, Hurley and Deming). The intent of this system is to make this water volume available to these communities when it is possible to do so within the operational constraints of the AWSA diversion, conveyance and storage system. Pope and Grant County Reservoirs should be utilized to enable allocations to be delivered during peak demands as best as possible. Careful use of the Grant County reservoir volume will allow an operations team the ability to deliver flow rates to each community at a higher demand peaking factor in the summer as would be expected.

BHI has evaluated the resulting required pump capacity, ancillary flow control equipment, and required piping alignments for the purpose of preparing a 10 percent appraisal level cost estimate.

a) *Design Criteria*

For the purposes of the pumping analysis, the minimum operating water surface elevation in Pope Reservoir is assumed to be 4,530 ft which corresponds to approximately 10 ft of water remaining in the reservoir. The maximum reservoir water surface elevation is 4,615 ft. The hydraulic analysis was based on USGS elevation data, which shows a high point at an elevation of 6,256 ft at the Continental Divide west of Silver City, resulting in a maximum lift of 1,726 ft. This static lift was broken down into five steps each at approximately 350 ft of lift, not including the dynamic friction. The intent of this value was to maintain the pipe system to operate at or below 200 psig and allow for 100 psig surge.

b) *Capacity*

To pump up to 10,000 AF within a year, this equates to up to 6,200 gpm continuous pumping 24 hours a day for up to 365 days. A 36 inch pipeline was chosen for the pump lifting section up over the Continental Divide to Silver City from the Pope Reservoir through a series of five booster pump stations to allow for a minimal friction loss at 2 ft/s to keep the duty points manageable. At each of the five pump stations, a storage tank will be used to break the hydraulic grade line and simplify operations and maintenance, as well as allowing

for a surge relief point during pump shutdown. A sixth tank is expected to also break the hydraulic grade line at the high point near Silver City. Due to the expected fall in elevation and additional energy available, along with the recognition that 1,500 gpm has been delivered to Silver City, reducing the flow to 4,700 gpm allows the pipeline size to be decreased to 24 inches past the Silver City connection point. This 24 inch pipeline would then terminate at the Grant County Reservoir. This reservoir will enable the operations of the system to manage stored volumes throughout the year, from a peaking use in the summer to storing additional volume in the winter when consumption may not be as high. From the Grant County Reservoir a separate 24-inch pipeline would then descend to the Bayard and Hurley connection points for delivery and ultimately to the Deming connection point.

Each of the five booster pump stations would be operating at a duty point of maximum 14 cfs (6,200 gpm), with a total dynamic head (TDH) of approximately 365 feet. Each station would exhibit two continuous pumps each rated at 3,100 gpm, with an assumed efficiency of 75 percent, each generating 391 bhp and attached to a 500 hp rated motor. Based on a total of 782 bhp and up to twelve months of continuous pumping at up to 14 cfs, the maximum power consumption would be approximately 5,380,000 kWh per year.

c) *Alignments*

The alignment for the SWRWS pipeline from Pope Reservoir extends adjacent to NM Highway 180 for the entire length to Deming. It is assumed that easements will be required for the pipeline parallel to the NMDOT right-of-way. This alignment will most likely need to alter in the vicinity of Silver City to by-pass the downtown area and skirt the community on the southern edge extending towards Grant County Reservoir. Figure 14 shows the proposed alignment and the associated profile.

d) *Cost Estimate*

The cost estimate for the system is based on 138,000 LF of 36 inch pipeline and 272,000 LF of 24 inch pipeline all rated for 330 psig, five 14 cfs booster pump stations with dual pumps with adjacent staging water storage tanks, control valves, pump controllers with radio telemetry SCADA system, protective metal buildings and other miscellaneous items, including right-of-way easements and land costs. The cost estimates for the SWRWS system is included in Appendix J and summarized in Section IX.

E. ANALYSIS OF SEDIMENT CONTROL METHODS

1. SEDIMENT CONVEYED THROUGH THE DIVERSION STRUCTURE AND INTO STORAGE

Tetra Tech analyzed the sediment delivery capacity to the diversion headworks and the capacity of the tunnel to Spar. The sediment delivery to Spar was used to estimate the sediment removal in the O&M costs.

a) *Sediment Load Delivery to the Tunnel*

The proposed tilted wedge-wire screen diversion provides an effective means of screening all but the finer sediment particles preventing them from entering into the tunnel and downstream storage and conveyance system. With a screen opening size of 0.5 mm, particles larger than this size cannot enter the diversion headworks and tunnel. As part of Tetra Tech's sediment transport analysis, they estimated the bed load, suspended sand load and suspended fine sediment (silt and clay) load entering the tunnel. Their analysis conservatively assumed that all of the bed load that passes the diversion weir and is finer than 0.5 mm will pass through the screen. For the suspended sand load and the suspended fine sediment load, it was assumed that the suspended sediment concentration in the diverted flow passing through the screen will be the same as the suspended sediment concentration in the river. Based on the suspended sediment rating curves discussed earlier in Section III.F, the total volume of sediment delivered to the tunnel to Spar was then estimated for four representative water years and is summarized in Table 5.2 of the Tetra Tech report (Appendix D) and in Table V-15 below.

Table V-15 – Summary of Annual Sediment Load Delivered to Tunnel for Selected Water Years

Water Year	Year Type	Number of Operational Days for Diversion	Sediment Delivery (CY)	Sediment Delivery (AF)
1989	Dry	2	325	0.2
1993	Wet	25	29282	18.1
1998	Average	72	35410	21.9
2005	Wet	119	71328	44.2

To convert the sediment load rates into volumes, Tetra Tech used a specific weight of 93 pounds per cubic foot that reflects the unit weight of the deposited sediment with voids. This value does not reflect any compaction other than that which would be caused by free flowing water. The reason for the decrease in sediment delivery between the average year

and wet year (1993) lies in the number of days when the diversion is operational, which decreases from 72 days in the average year to 25 days under the wet year (1993). In 2005, the number of days the diversion would have been operational is 119, resulting in relatively large sediment delivery volumes. The amount of sediment delivered to the tunnel is relatively small and manageable as discussed further in the following sections. As indicated below in Section V.E.1.b) and in Appendix D, there are certain tailwater conditions in Spar Reservoir that could result in sediment deposition in the pipeline. The analysis indicates that deposition would only occur during certain river discharge-diversion discharge-tailwater elevation conditions, and that deposition in the pipeline would not occur at diversion discharges of less than 280 cfs even under the highest tailwater condition. The system is currently envisioned and will be designed and operated to keep the tailwater elevation below an elevation that would lead to sediment deposition in the pipeline. With a reduced tailwater elevation, free-surface flow conditions would occur along the pipe for lower diversion discharges, and the analysis indicates that the sediment-transport capacity exceeds the sediment supply over these discharges, so no deposition would occur at the lower discharges. At higher discharge rates greater than 280 cfs, even with a reduced tailwater condition, pressure flow conditions will occur in the upstream portion of the pipe. However, the analysis indicates that the available energy gradient would be sufficiently larger than the piezometric headline gradient of the water/sediment mixture, so no deposition would occur at the higher discharges as well.

b) *Sediment Transport Capacity of the Tunnel*

Given the above understanding of the sediment delivered to the tunnel, further analysis was completed regarding the tunnel's sediment transport capacity and the potential for sediment deposition in the tunnel. Tetra Tech completed this analysis by estimating the hydraulic conditions in the tunnel for various flow rates and headwater (water surface elevation at the screen intake) and tailwater (water surface elevation at the tunnel outlet) conditions. Their analysis was based on two conservatively high tailwater conditions that would result in full pipe flow, corresponding with a tailwater elevation of 4724 (top of Spar Reservoir) and 4722.2 (top of the tunnel outlet). The results from Tetra Tech's analysis indicate that for the high tailwater condition (water surface elevation of 4724 in Spar Reservoir), deposition in the tunnel would occur over the range of river discharges between 370 and 1,070 cfs (corresponding to diversion discharges of between 220 and 350 cfs), as shown in Figure 5.20 in Appendix D. Under the moderate tailwater condition (water surface

elevation of 4722.2 ft), deposition would occur over the range of river discharges between 440 and 670 cfs (corresponding to only diversion discharges of between 290 and 350 cfs).

The above analysis assumed that the tailwater conditions in Spar Reservoir would be conservatively high (a tailwater elevation of 4724 (top of Spar Reservoir) and 4722.2 (top of the tunnel outlet)), resulting in submerged outlet conditions that limit the sediment transport capacity of the tunnel. Further analysis was therefore completed for non-submerged outlet conditions by assuming that the tailwater in Spar Reservoir was at 4713.2 ft, corresponding to the invert of the tunnel outlet in the Tetra Tech analysis. For diversion discharges below 280 cfs, free surface conditions occur in the tunnel throughout the length of the tunnel, so the sediment-transport capacity of the tunnel was estimated using a sediment-transport formula for open-channel conditions. The resulting transport capacity is higher than the estimated sediment supply, so no deposition would occur (Figure 5.21 in Appendix D). For diversion discharges above 280 cfs, when at least the upstream portion of the pipe is full, Tetra Tech repeated the analysis for pipe-full conditions discussed in the previous paragraph. This analysis indicated that the available energy gradient that is based on the assumed headwater and tailwater elevations (with a tailwater of 4713.2 rather than 4722.2 or 4724) is greater than the piezometric headline gradient of the water/sediment mixture, so no deposition would occur (Figure 5.20 in Appendix D). Consequently, with a maximum tailwater of 4713.2 at the tunnel outlet, sediment deposition will not occur over the range of flows from 0 to 350 cfs to be diverted.

c) *Potential for Consolidation of Deposits*

At the current time, the details regarding system operations are not known, so it is not possible to determine how significant sediment deposition in the tunnel would be under high tailwater conditions (water surface in Spar Reservoir or 4722.2 or 4724) if allowed. Consequently, the recommended design approach is to design and operate the system to create lower tailwater conditions (4713.2) in Spar Reservoir. To that end, further analysis of Spar with respect to storage volume and sediment settling/trapping conservatively assumes a maximum water surface of 4713. If during design of the project it becomes desirable to increase the water surface elevation at Spar thereby raising the downstream tailwater and creating the potential for sediment deposition in the tunnel, further analysis of the potential effects of this sediment deposition using established methods noted in Appendix D should be completed.

2. POTENTIAL DAMAGE TO THE DIVERSION STRUCTURE

There is potential for damage to the upstream weir wall from large bedload material and debris. Potential damage points include the screen itself and the concrete structure supporting the screen and forming the trough that collects and conveys the flow through the screen to the tunnel. To address these concerns, BHI has done additional research and analysis. The wedge-wire screens are attached to support frames which can be designed to withstand impact loads of varying magnitudes depending on the diversion structure orientation, hydrostatic pressure above the screen, and the expected sediment load. Based on Tetra Tech's analysis, the 100-year event, commonly used as the design standard for civil infrastructure, will produce a velocity in the river at the diversion of about 17 fps which will mobilize particles with a grain size of approximately 325 mm (13 inches) or less. This means in any given year, there is a 1 percent probability that boulders up to 13 inches in diameter will be mobilized at the proposed location of the diversion structure. The shear load produced on the structure by that size of material only requires two inches of concrete to resist a direct impact hit by a 13 to 14 inch boulder. Assuming the wall is a minimum of 8 inches thick; it provides four times the necessary shear strength. The shear load calculation is included in Appendix G.

Given the proposed configuration of the diversion structure, the screens (located on the downstream side of the structure) will not be subject to direct impacts by boulders or other large debris transported in the flow. Buoyant forces will cause the boulders or other large debris to instead roll down or float over the screen, exhibiting minimal force on the screen and support frame. The capacity of the screen includes a 1.5 factor of safety against clogging.

F. ADDITIONAL CONSIDERATION

Based upon direction from the ISC, the Alternatives examined in the April 2014 PER were based upon a gravity only diversion. Pumping of water to supply the storage reservoirs was not included. As part of this PER, an alternative was added to pump water from the Gila River to Spar Reservoir. These two diversion and conveyance approaches form the basis for alternatives that effectively bracket the potential engineering solutions for diversion, storage, and conveyance. Based upon the analysis completed to date, a solution that combines these two approaches may be the most feasible approach. More specifically, a surface diversion and gravity conveyance to Spar Reservoir combined with a much smaller pump station from Spar to a second larger Spar Reservoir to be located upstream in

Spar Canyon might be a means of taking the best from both approaches (See Figure 15). It is possible, depending upon operating procedures and storage requirements, that this third approach could eliminate the need for Sycamore Reservoir and associated conveyance pipes as Alternative 3 does, but at a much more affordable cost by using the storage at Spar, and possibly Maldonado, to allow for the required pumps to be drastically reduced in size. This would reduce both upfront capital costs for a pump station while also reducing the high operation and maintenance costs associated with the large river-to-Spar pump station. Maldonado could be constructed adjacent to Spar and connected with an equalization line to greatly increase the effective storage for the Spar pump station thereby further reducing the cost of the pump station. This third combined gravity and pumping approach would require additional engineering analysis to verify and determine infrastructure requirements and costs but has the potential to be a more optimal configuration than either gravity or pumping alone approaches.

VI. SYSTEM ALTERNATIVES

From the above additional site investigation and broad based analyses, BHI has developed and evaluated three alternatives, which are summarized below.

A. ALTERNATIVE 1

Alternative 1 includes a surface river diversion at Site 6, storage in Spar, Winn, Pope, and Sycamore canyons, and conveyances between those storage sites (see Figure 16). Alternative 1 also includes an irrigation pump station from Winn to the top of the Cliff-Gila farming valley. The diversion Site 6 was selected on the basis of reasons described in Section V.A.1 above. Pressure piping was evaluated between all storage sites below Spar Canyon in Alternative 1. For all storage sites downstream of Spar, flow through the conveyances was assumed to be proportional to the reservoir storage volumes. The design flow from J4 to Pope is based on the storage at Sycamore and Pope added together, since Sycamore is downstream of Pope. The design flow from J4 to Winn is based on the storage at Winn only. Table VI-1 summarizes the storage volume and each reservoir and the overall total storage for the Alternative. Table VI-2 summarizes the conveyance lengths, their design flow rate, and their sizes for Alternative 1. The HGL analysis for Alternative 1 is included in Appendix G.

Table VI-1 – Storage Reservoirs for Alternative 1

Canyon	Storage Volume (AF)
Spar	1,642
Winn	10,713
Pope	8,732
Sycamore	44,330
TOTAL	65,417

Table VI-2 – Conveyances for Alternative 1

Conveyance	Length (ft)	Design Flow Rate (cfs)	Diameter (in)
Spar to J4	12,200	350	78
J4 to Winn	6,840	75	48
J4 to Pope	28,760	275	72
Pope to Sycamore	14,270	209	60

B. ALTERNATIVE 2

Alternative 2 includes a combined subsurface river diversion along two miles of river above Site 6, a surface river diversion at Site 6, storage in Spar, Winn, Pope, and Sycamore canyons, and conveyances between those storage sites (see Figure 17). Alternative 2 also includes an irrigation pump station from Winn to the top of the Cliff-Gila farming valley. Pressure piping was evaluated between all storage sites below Spar Canyon in Alternative 2. For all storage sites downstream of Spar, flow through the conveyances was assumed to be proportional to the reservoir storage volumes. The design flow from J4 to Pope is based on the storage at Sycamore and Pope added together, since Sycamore is downstream of Pope. The design flow from J4 to Winn is based on the storage at Winn only. Table VI-3 summarizes the storage volume and each reservoir and the overall total storage for the Alternative. Table VI-4 summarizes the conveyance lengths, their design flow rate, and their sizes for Alternative 2. The HGL analysis for Alternative 2 is included in Appendix G.

Table VI-3 – Storage Reservoirs for Alternative 2

Canyon	Storage Volume (AF)
Spar	1,642
Winn	10,713
Pope	8,732
Sycamore	44,330
TOTAL	65,417

Table VI-4 – Conveyances for Alternative 2

Conveyance	Length (ft)	Design Flow Rate (cfs)	Diameter (in)
Spar to J4	12,200	350	78
J4 to Winn	6,840	75	48
J4 to Pope	28,760	275	72
Pope to Sycamore	14,270	209	60

C. ALTERNATIVE 3

Alternative 3 includes a surface river diversion near Spar Canyon at an elevation of 4,590, a pump station to Spar Canyon, storage in Spar, Winn, and Pope Canyons, and conveyances between those storage sites (see Figure 18). Alternative 3 does not include an irrigation pump station, as it is assumed that water could be released from Spar to the river to meet both agricultural and environmental needs. Pressure piping was evaluated between all storage sites below Spar Canyon in Alternative 3. For all storage sites

downstream of Spar, flow through the conveyances was assumed to be proportional to the reservoir storage volumes. The design flow from J4 to Pope is based on the storage at Pope. The design flow from J4 to Winn is based on the storage at Winn. Table VI-5 summarizes the storage volume and each reservoir and the overall total storage for the Alternative. Table VI-6 summarizes the conveyance lengths, their design flow rate, and their sizes for Alternative 3. The HGL analysis for Alternative 3 is included in Appendix G.

Table VI-5 – Storage Reservoirs for Alternative 3

Canyon	Storage Volume (AF)
Spar	46,037
Winn	10,713
Pope	8,732
TOTAL	65,482

Table VI-6 – Conveyances for Alternative 3

Conveyance	Length (ft)	Design Flow Rate (cfs)	Diameter (in)
Diversion to Spar	6,194	350	4x60
Spar to J4	14,060	350	60
J4 to Winn	6,840	186	48
J4 to Pope	28,760	164	54

VII. SOLAR ARRAY OPTIONS FOR PUMPING

A. INTRODUCTION

The assessment of the production of solar electricity in sufficient quantities to power electric pump motors to pump water has been studied and addressed for the following scenarios:

- From the Alternative 3 surface diversion point near the confluence with Spar Canyon to Winn and Spar Canyon reservoirs
- From Winn Canyon Reservoir to the furthest upstream irrigable properties
- From Pope reservoir over the continental divide with five identically sized booster pump stations

BHI subcontracted with Consolidated Solar Technologies, LLC (CST) to perform preliminary solar power investigations for this project. The following is CST's objective:

- Estimate the configuration and costs of a photovoltaic (PV) solar array for diversion and conveyance purposes.

The locations of the pumping operations are within the territory served by PNM. Consequently, the regulations, rate structures, and contract terms of PNM apply to this project and will affect any decisions made from this report. A discussion of PNM's rate structures and contract riders relevant to pumping operations is included in this section.

B. SCENARIOS

In evaluating the solar array sizes for each pumping situation, CST did not pursue the option of pumping operations powered purely from solar energy. They offer the concept of solar arrays that are connected to utility power distribution system (grid) for the purposes of decreasing the electric load drawn from the power utility company and selling excess electricity generated by solar to the utility company in an effort to offset the power consumption of the pumps. This is because stand-alone, or off-grid, solar arrays must employ techniques to store enough energy to supply power to run the system during times of low or no solar exposure like nighttime or during a rain storm. The most common storage method is the use of batteries. Batteries store energy in chemical form and release the energy in electric form, and energy is lost during this process. Batteries used for power applications are typically lead acid type, which require regular maintenance, and are capable of a limited number of charge/discharge cycles. Because of the chemicals used to build lead acid batteries and store energy, NM state law has banned landfilling or

incineration of lead acid batteries. This means that a disposal program will need to be designed and in place before any battery banks are put into service. An emergency cleanup and containment plan to deal with the environmental impact of leaking cells or a fire at the battery bank will also need to be developed.

Other energy storage methods are currently under development for utility use, such as heating salt to a molten state for later energy release, or using solar energy to produce hydrogen from water to burn for energy. These methods are still in early development and remain prohibitively expensive.

A third theoretical methodology of expanding the pumping system and solar arrays to have dedicated pumps use additional water and power during the day when power is available to lift a volume to a storage reservoir, and then use that volume source to run back through a hydropower unit to extract power and continue to run at night. This method would necessitate a large and complex expansion of the system, causing even more volume of water to be extracted during the day and greatly complicate the process.

Another issue is enabling large pump motors to start. When an electric motor is started, a large amount of electric current is drawn by the motor from the power source for a brief period of time. The solar arrays and related equipment will not be able to provide for this motor inrush current, and additional storage methods and extra capacity will need to be employed that will greatly increase the cost, complexity and reliability of the power system, merely for the capability of starting the pump motors.

In order to power the pumps without the use of utility power, the amount of power necessary to run the pumps during the day and simultaneously charge the batteries, or lift additional water to store as energy, to enable the pumps to run during the night, the solar arrays described by CST would need to be at least three times the sizes shown in the tables at the end of Appendix H.

CST calculated the individual pump station array sizes based on calculated brake horsepower (bhp) necessary for each station. This does not account for motor inefficiencies. BHI calculated the required annual power requirements based on motor horsepower in kWh per year, which are summarized in Table VII-1, below.

C. DISCUSSION OF FINDINGS

Pursuant to the Renewable Energy Act (REA) and New Mexico Public Regulation Commission Rule 572, PNM procures renewable energy certificates (RECs) from New Mexico renewable generation facilities to meet the requirements of the REA and Rule 572.

When PNM is seeking RECs, they issue a Request for Proposals (RFP) for interconnection of utility scale renewable projects. Two types of proposals are generated, Power Purchase Agreements (PPA), and turnkey proposals. A PPA relates to a renewable energy facility that is owned and operated by a third party, and connected to PNM's transmission system for the purpose of providing power. This type of agreement requires a contract to be negotiated at the time of bid between the operator and PNM regarding how the facility will be operated and the power purchase price. A turnkey proposal relates to a facility built by a third party but turned over to PNM upon completion. The proposals are accepted based on the lowest bid for the wholesale of energy, the distance to a PNM transmission line, and proximity to a high utility load. The latest bid for a utility scale solar array accepted by PNM is a turnkey operation for $70 per megawatt hour.

It is the recommendation of CST to build one large grid-tied solar array to offset the power consumption of all of the pump stations. A single monolithic array would need to produce 71,000,000 kWh annually to cover the projected requirement of all pump stations, would require approximately 90 to 150 acres, and would cost $60 to $80 million depending on the technology chosen (see Appendix H). This approach will not only require the cooperation of PNM, but must also coincide with an RFP by PNM and the generation of a winning bid proposal.

PNM does allow for a utility interconnection (grid-tie) of up to one megawatt of customer owned renewable energy generation at a customer owned service, if the equipment is located on the customer owned premises in what is known as "net metering." For billing purposes, net metering is the net power consumption as measured at the customer's electric meter. Special electric meters are used that not only record the flow of power from the power utility, but when excess power is generated by the customer, the meter will also record the amount of that power and the customer will be credited. Regarding the continuously operating pump stations, a credit will not be seen as much as the electric bill simply being lowered and during much of PNM's peak time, 8 a.m. to 8 p.m. on weekdays. Per PNM rate structure 32, "Solar Renewable Energy Certificate Purchase Programs" (Appendix I), solar panel array size is limited to 120 percent of average annual consumption. This limitation would only affect the smaller irrigation pump station at Winn Canyon, limiting the array size to approximately 16 kilowatts for the 10 cfs pumping option for the irrigation pump station and approximately 100 kilowatts for the 50 cfs pumping option for the irrigation pump station. These array sizes are based on 120 percent of the average

annual consumption estimates of 113,392 kilowatt hours annually for the 10 cfs irrigation pumping option and 698,256 kilowatt hours annually for the 50 cfs irrigation pumping option.

Due to the costs and complexities related to energy storage, the scenario of pumping operations powered purely from solar energy is not recommended. Additionally, the restrictions imposed by PNM on customer owned generation facilities hinder the recommendation of a monolithic array. Instead, a system of co-located solar arrays that are interconnected to the utility power distribution system (grid) for the purposes of lessening the customer load and selling excess electricity generated by solar to the utility company in an effort to offset the power consumption of the pumps is recommended.

CST discusses two solar PV panel technology alternatives in their report, conventional solar panels, and higher efficiency, investment grade panels. Conventional solar panels are about 15 percent efficient and the estimated cost per installed megawatt is estimated by CST to be $2,191,000. The high efficiency panels are about 22 percent efficient. This is due to a more advanced design that is detailed on page 10 of Appendix H. The interpolated cost per installed megawatt of high efficiency panels from the table titled “Sunpower Panels – Fixed Tilt” is $2,741,000. These interpolations use the fixed tilt monolithic array cost estimates for conventional panels and high efficiency Sunpower panels as a basis by dividing the array size into the cost to find the price per installed kilowatt, and multiplying that number by one thousand to find the price per installed megawatt.

The estimated land use per megawatt is also outlined by CST in Appendix H. The high efficiency panels require a smaller physical array size, requiring about 28 to 40 percent less area than required by a conventional panel array. Due to the smaller array footprint and longer warranty, high efficiency panels are recommended over conventional panels.

Another technology discussed by CST is single axis or East-West solar tracking. Solar trackers increase the amount of power produced by following the sun throughout the day by about 20 percent relative to fixed position solar arrays. While tracking devices are generally known to add cost to a small scale solar panel installation, on large scale projects they actually decrease the total cost of installation by approximately 12 percent since more power is generated from a smaller number of solar panels. They do, however, require more physical installation space. For a conventional panel installation, a tracking array requires about 27 percent more land; for a high efficiency panel tracking installation, a tracking array requires about 16 percent increase in land use. The land purchase price will have to be known before an accurate cost analysis of tracking vs. fixed position solar panel arrays can be completed.

While the lower installation cost associated with the solar tracking technology may seem attractive, due to their complexity of moving parts, electronic control and power consumption, they offer a potential for failure and require a more rigorous maintenance program. The utility scale trackers proposed for use by CST have an extendable warranty up to 10 years, while some high efficiency panels have a warranty of 25 years, so a possible replacement of the tracking system could occur long before the end of life of the panels. For these reasons, fixed tilt arrays are recommended.

D. POWER CONSUMPTION AND UTILITY COSTS

PNM offers different rate structures for different ways that power is consumed. For customers continuously consuming more than 8000 kW, rate structure 5B "Large Service" applies. While the Spar pump station will consume approximately 18,000 kW, rate structure 5B stipulates that the minimum amount of power be consumed continuously. Since the Spar pump station will operate only part of a year, PNM rate structure 11B (Appendix I), "Water and Sewage Pumping Service-Time-Of-Use Rate," is applicable.

Per rate structure 11B, PNM energy charges are $0.19 per kWh for summertime (June, July and August) peak times, $0.123 per kWh for the rest of the year peak times, and $0.037 per kWh for year round off peak times. Rate structure 11B also provides for a customer charge per bill of $491.60 and a fuel surcharge of $0.021 per kWh.

Throughout the year while the Spar pump station is not in operation, the power from a one megawatt solar array built at Spar pump station could be purchased by PNM at their avoided energy cost of about $0.03 per kWh ($30 per megawatt hour).

Table VII-1 summarizes the power consumption costs for Spar pump station, with and without net metering.

Table VII-1 – Spar Pump Station Power Consumption Costs

Total Pump Motor HP	Total Days of 24 hr Station Operation	kWh/Year for Pumping	Daily Energy Cost (Summer)	Daily Energy Cost (Sept-May)	Yearly Energy cost Without Net Metering	Yearly Energy cost With Net Metering
23,700	94	38,265,560	$46,000	$36,000	$4,316,000	$4,166,000

Rate schedule 11B is only applicable to loads greater than 2,500 kW. While the SWRWS booster pump stations are expected to operate continuously throughout the year, each station will only be consuming about 550 kW. For purposes of estimating costs, PNM has conceptualized one high voltage substation to serve the five SWRWS booster pump

stations. Thereby, it may be possible to have the five pump stations on one service and utilize rate structure 11B. However, this would require the owner of the pump stations to build and maintain the electrical distribution system from the PNM high voltage substation to each of the pump stations. If each SWRWS booster pump station is metered independently, PNM rate structure 4B is applicable. Table VII-2 was prepared using the values from PNM rate structure 4B and a one megawatt solar array built at each station for net metering.

Table VII-2 – SWRWS Booster Pump Stations Power Consumption Costs

Rate Structure 4B	Total Pump Motor HP	Total Days of 24 hr Station Operation per Year	kWh per Year for Pumping	Daily Energy Cost (Summer)	Daily Energy Cost (Sept-May)	Yearly Energy cost Without Net Metering	Yearly Energy cost With Net Metering
1 Station	1,000	365	5,379,516	$1,400	$1,200	$450,000	$290,000
5 Stations	5,000	1825	26,897,580	$7,000	$6,000	$2,250,000	$1,450,000

There are several rate structure options potentially available for the irrigation pump station. Due to the part year operation, the different options will have to be studied and an agreement will have to be reached with PNM. The rate structure 2B "Small Power Service Time-Of-Use Rate" was used to calculate the costs outlined in Table VII-3. The table is based on 90 days of summertime pricing, 60 days of regular year pricing, and solar arrays for offsetting costs at the meter during pumping operations and selling power to PNM during periods of non-operation.

Table VII-3 – Irrigation Pump Station Power Consumption Costs

Total Pump Motor HP	Total Days of 24 hr Station Operation	kWh/Year for Pumping	Daily Energy Cost (Summer)	Daily Energy Cost (Sept-May)	Yearly Energy cost Without Net Metering	Yearly Energy cost With Net Metering
50	150	113,392	$110	$100	$15,600	$12,300
300	150	698,256	$700	$600	$95,200	$74,500

An evaluation of the financial viability of co-located solar arrays at each pump station was generated to outline the capital costs for installing the arrays, the annual maintenance costs and the annual amount that can be expected to be earned or saved from the generation of solar power. The capital costs described in Table VII-4 are comprised of fixed tilt high efficiency panels described in Appendix H, with a 6 ft chain link and barbed wire fence enclosing the arrays, priced at $35 per linear foot and a raw land and site work cost

estimated to be $8,000 per acre. The one megawatt arrays are estimated to be five acres and the smaller arrays to be located at the irrigation pump station are estimated to be about ¼ of an acre. The analyses were computed using a National Renewable Energy Laboratory renewable energy payback calculator. The following financial parameters were used to estimate the payback periods shown in Table VII-4: Inflation Rate 2 percent, project life of 25 years corresponding to the warranty of the panels, a debt ratio of 20 percent, and a debt interest rate of 4 percent and a debt term of 20 years. A payback period of less than 20 years is considered acceptable. BHI recommends solar power at all pump stations that achieve payback periods of less than 20 years.

Table VII-4 – Costs and Savings from Solar Arrays

Station Name	Array Size	Capital Cost @ $2,741 / installed kW	Annual Maintenance @ $25 / kW	Annual Solar Generation Earned	Potential Payback period in Years	Recommended
Spar Pump Station	1 MW	$2,811,000	$25,000	$150,000	20	No
SWRWS Booster Pump Station 1	1 MW	$2,811,000	$25,000	$160,500	18.5	Yes
SWRWS Booster Pump Station 2	1 MW	$2,811,000	$25,000	$160,500	18.5	Yes
SWRWS Booster Pump Station 3	1 MW	$2,811,000	$25,000	$160,500	18.5	Yes
SWRWS Booster Pump Station 4	1 MW	$2,811,000	$25,000	$160,500	18.5	Yes
SWRWS Booster Pump Station 5	1 MW	$2,811,000	$25,000	$160,500	18.5	Yes
10 cfs Winn Irrigation Pump Station	16 kW	$55,000	$400	$3,200	20.2	No
50 cfs Winn Irrigation Pump Station	100 kW	$271,000	$2,500	$20,000	16.6	Yes

Judging from the expense of installation and length of payback period, the use of solar arrays for offsetting the cost of water pumping at Spar pump station is not recommended. This is due to the high usage of power and the short pumping season. The SWRWS booster station arrays have payback periods of less than 20 years, so the use of solar power for offsetting pumping costs at those locations is recommended. For the 10 cfs pumping option at the irrigation pump station, the payback period for the array will be more than 20 years. Therefore, a solar array for the 10 cfs pumping option at the irrigation pump station is not recommended. For the 50 cfs pumping option at the irrigation pump station, the payback period for the array will be less than 20 years. Therefore, a solar array is recommended for the 50 cfs pumping option at the irrigation pump station.

VIII. HYDROELECTRIC ENERGY RECOVERY

A. INTRODUCTION

The power necessary to pump water from the Pope Reservoir over the Continental Divide to the Grant County Reservoir will require approximately 28,660,000 kilowatt hours per year at a cost of about $2.36 million per year (without net metering). In an attempt to recover some of the energy expended to pump this water to a higher elevation, hydroelectric turbines could be installed along the pipeline to recapture this energy as the pipeline descends from the Grant County Reservoir to Deming. The HGL from the Grant County Reservoir down to Deming was analyzed to understand the viability of this approach.

A 36 inch pipeline was chosen for the pump lifting section up to Silver City from the Pope Reservoir. Previously, a 16 inch pipeline had been chosen to throttle or manage the pressure within the ratings of the pipeline based on a friction loss at maximum flow as the pipeline descended from Silver City to Deming. Due to the difficulties of managing pressure in this manner, using friction loss and not being able to recover energy led to the decision to increase the pipe diameter to 24 inches to allow energy recovery with a hydroturbine rather than losing it to heat generation to the water body. The HGL is included in Appendix G.

B. ENERGY RECOVERY

Along the 55 mile length of 24-inch pipeline from the Grant County Reservoir to Deming, the available pressure head allows for three hydroelectric turbines to be placed in line and still provide enough residual pressure in Deming for the water to be used for multiple purposes without the need for another booster pump station. The locations of the turbines are dictated by the pressure available in the pipe. A turbine could be placed at locations where the pressure head available is approximately 200 psi. This would also limit the pressure within the pipeline to an acceptable level. Further investigation would be needed to determine the exact location of each turbine. Each turbine would pass about 3000 gpm and be installed with a pressure reducing valve (PRV) in parallel that would pass the remainder of the estimated total 3,400 gpm flow, as well as act as a bypass if maintenance is required on a turbine. The PRVs will also modulate pressure as necessary as flow rates change throughout the year depending upon the demand.

The first two stations would operate at 195 psi and provide 187.5 kW of electrical output. The third station would operate at 78 psi and provide 72 kW of electrical output. For a continuous maximum flow of 3000 gpm, 24 hours a day, 365 days a year, this equates to

just under four megawatt hours per year of total electrical power output from the three turbines.

The HGL for the SWRWS pipeline, including the booster pump stations and the hydro turbines, is included in Appendix G. The estimated construction cost of the three turbine units, including necessary appurtenances for operation, is described in Appendix J. A small heated metal building will be necessary to house each of the units, although maintenance on the units will be minimal.

C. DISCUSSION OF POWER CONNECTION

PNM does not allow customers to build standalone power generation plants for connection to the PNM electrical distribution system, such as these hydropower turbines. Per PNM rate structure 12, Cogeneration and Small Power Production, the customer owned generation facility must be on or near the site where the customer is consuming power. In order to qualify for rate structure 12, the facility must meet the Federal Energy Regulatory Commission (FERC) Rules and Regulations 18 CFR Part 292 Subpart B, Sections 292.201 through 292.207. If the hydroelectricity cannot be consumed onsite for other purposes, contract negotiations will have to be initiated with PNM to allow for a variance to Rate Structure 12. If PNM allows for interconnection to their distribution grid for the purposes of buying the power recovered from the pipeline, it can be assumed that they will purchase it at their avoided cost rate of about $0.03 per kilowatt hour. Assuming the turbines are operating under full pressure and flow rate, 24 hours a day, the turbines will generate $117,000 per year. The life of a turbine in continuous operation is estimated to be ten years, the same length as a pump. Although this payback period for these system installations appears to be longer than 10 years, we do recommend these facilities be included if PNM will allow the interconnection to recover the available energy. The cost estimates for the hydroelectric turbine systems are included in Appendix J.

IX. COST ESTIMATES

BHI prepared preliminary planning level cost estimates for all three alternatives. The cost estimates are included in Appendix J and are summarized in Table IX-1, below. Costs shown in the table are before New Mexico Gross Receipts Tax (NMGRT). Some unit costs are based on BHI's construction cost library of bid tabulations, as well as published unit cost data from the City of Albuquerque and New Mexico Department of Transportation. The pipeline unit costs are based on consultation with contractors experienced with installation of large diameter pipes in remote areas. Estimated construction costs for all alternatives include the costs for relocating NM 211 to accommodate the reservoir in proposed Pope Canyon and for relocating McCauley Road to accommodate the proposed reservoir in Sycamore Canyon. The portion of NM 211 south of Pope Canyon was assumed to be relocated to connect to US 180 approximately two miles to the west of the current intersection. The portion of McCauley Road crossing Sycamore Canyon was assumed to be relocated to the east to cross the embankment across Sycamore Canyon and reconnect to the existing McCauley Road to the north and to the south of Sycamore Canyon. Relocating both NM 211 and McCauley Road along the dam embankments at Sycamore and Pope Canyons will require consideration of alternate routes for emergency planning purposes.

Table IX-1 – Estimated Construction Costs

Alternative (per Section VI)	Construction	Non-Construction	Total
1a	$581,341,000	$88,445,000	$669,786,000
1b	$583,522,000	$88,759,000	$672,281,000
2a	$604,438,000	$93,527,000	$697,965,000
2b	$606,622,000	$93,841,000	$700,463,000
3	$581,400,000	$81,104,000	$662,504,000

A. SURFACE DIVERSION

All alternatives include a surface diversion structure. The cost for the diversion structure is based on several assumptions. First, excavation earthwork spans the length of the structure and goes 1 foot deeper than the trough depth. Also, rock excavation is estimated at 30 percent of total excavation and traditional excavation at 70 percent of total excavation. Finally, the diversion structure cost includes such items as reinforced concrete, wingwalls, an apron, support piles driven to bedrock, a tilted wedge-wire screen, headgates and a pedestrian bridge for maintenance that will extend the length of the screen.

B. SUBSURFACE DIVERSION

Alternative 2 includes a subsurface diversion structure. The cost for the subsurface diversion structure is based on several assumptions. First, a temporary construction cofferdam and dewatering will be required at each diversion point along the infiltration gallery. Also, trenching will be required for each diversion point. Finally, the subsurface diversion structure cost includes such items as perforated pipe, blank casing for cleanouts and access ports, 24 inch to 66 inch collector pipes, and headgates.

C. TUNNEL

The estimated cost for the tunnel proposed in Alternatives 1 and 2 is based on consultation with a tunneling contractor that was documented in the April 2014 PER.

D. STORAGE RESERVOIRS

The estimated costs for the storage reservoirs are based on several assumptions, which are discussed below.

1. All dams will require a black, 60 mil, HDPE liner, or equivalent (described in more detail in Section IV.C.3 above). In addition to the HDPE liner at each dam, a 12-inch thick compacted clay subgrade is included. The cost estimate for Spar Canyon also includes a cost for a soil cement grade control slab across the bottom of the dam pool.
2. Floodpool grading will be required before placement of the impermeable liner. Earthwork has been optimized to better balance the cut and fill at each reservoir site. A balanced earthwork approach should be taken to facilitate potential construction phasing. Costs for excavation and backfill are based on the total excavation at each site. The only dam that requires import is Sycamore. This import is assumed to come from Winn and Pope, which both have excess material available. The grading could be refined during design to reduce the import required for construction of the dam at Sycamore and to reduce the excess earth generated from excavation at Winn and Pope.
3. A concrete emergency spillway is assumed to extend down the downstream side of each dam to convey flow over the top of the dam in the event of the Probable Maximum Flood (PMF) or other overtopping situations. In addition, the emergency spillway concrete is assumed to extend 5 feet down the upstream

side of the dam, overlapping the HDPE liner. The dam slopes were assumed to be 2:1 on the upstream side and 4:1 on the downstream side.

4. The cost for each reservoir site includes a cost for an upstream stormwater detention facility. The stormwater detention facility costs are based on a graph of dam storage vs. cost per acre-foot for previously constructed detention facilities, in 2014 dollars.

E. SPAR CANYON RESERVOIR CONCEPTUAL OUTLET WORKS

The cost for the Spar Canyon Reservoir conceptual outlet works includes reinforced concrete, wire-enclosed riprap at the orifice inlets, 72 inch pipe to convey 90 cfs to the Gila River, 30 inch pipe connecting the 90 cfs orifice inlets to the 90 cfs outlet pipe to the Gila River, and a ported riser for the 350 cfs outlet.

F. CONVEYANCE PIPES BETWEEN STORAGE RESERVOIRS

The estimated costs for conveyances between storage reservoirs are based on assumptions that were documented in the April 2014 PER, including expected rock trenching and excavation, as well as traditional excavation.

G. IRRIGATION PUMP STATION AT WINN CANYON

The cost for the 10 cfs irrigation pump station at Winn Canyon is based on several assumptions. The cost includes a 24 inch pipeline from Winn Canyon to the Upper Gila diversion site. Also included is a cast in place concrete clearwell, a 30 ft x 30 ft x 15 ft metal building, one 4,500 gpm pump, 50 hp motor, VFD pump control and SCADA system.

Alternatively, the cost for a 50 cfs irrigation pump station at Winn Canyon is based on a 42 inch pipeline from Winn Canyon to the Upper Gila diversion site. Also included is a cast in place concrete clearwell, a 30 ft x 30 ft x 15 ft metal building, one 22,500 gpm pump, 300 hp motor, VFD pump control and SCADA system. The cost for the 50 cfs irrigation pump station at Winn Canyon also includes a 100 kW solar array.

H. SPAR PUMP STATION

The cost for the 350 cfs pump station to Spar Canyon includes the following:

1. 108 inch pipeline from the diversion structure to the pump station
2. Four parallel 60 inch pipelines from the pump station to the storage dam
3. 180 ft x 30 ft x 20 ft cast in place concrete clear well
4. 150 ft x 80 ft x 30 ft pre-engineered metal building to house the pumps

5. 120 ft x 60 ft x 25 ft bridge crane system
6. 40 ft x 80 ft x 20 ft electrical/control building
7. Electrical components
8. Three 40,000 gpm pumps, 5,900 hp motors
9. Four 10,000 gpm pumps, 1,500 hp motors
10. Pump control and SCADA system
11. PNM substation and power line extension

I. SOUTHWEST REGIONAL WATER SUPPLY BOOSTER PUMP STATIONS

The cost for the SWRWS pipeline has been updated from the April 2014 PER to include the following items for each booster pump station:

1. 50 ft x 30 ft x 20 ft pre-engineered metal building
2. Two 3,100 gpm pumps, 500 hp motors
3. Electrical components
4. Pump control and SCADA system
5. PNM Power Extension
6. 1 MW solar array

J. ALTERNATIVE 1

For the purposes of the cost estimate, Alternative 1 was split into two alternatives: 1a and 1b. The alternatives are identical except for the irrigation pump station. Under Alternative 1a, the irrigation pump station would have a capacity of 10 cfs, while Alternative 1b proposes this pump station as a 50 cfs pump station.

Estimated costs for Alternative 1a, including planning, design, construction, and administration/oversight are summarized in Table IX-2, below.

Table IX-2 – Cost Estimate for Alternative 1a

Item	Cost
Diversion, Conveyance, and Reservoir Construction	$581,341,000
Design	$28,647,000
Topographic Survey	$858,000
Right-of-Way Easement Development (Pipeline + Rural)	$439,000
Permitting, Environmental & Geotechnical Investigations	$5,000,000
Land Acquisition Services (Reservoirs + Pipeline + Booster Stations)	$90,000
Land Acquisition (Reservoirs + Pipeline + Booster Stations)	$6,672,000
Easement Acquisition (Pipeline + Channel)	$232,000
Construction Observation and Management	$46,507,000
Subtotal	$669,786,000
NMGRT	**$41,443,000**
Total	**$711,229,000**

The cost for Alternative 1a is based on the following:

1. 350 cfs gravity surface diversion at Site 6
2. Tunnel from the diversion to Spar
3. Storage in Spar, Winn, Pope and Sycamore Canyons
4. Pressure piping between storage sites
5. 10 cfs irrigation pump station at Winn Canyon
6. Pumping from Pope Canyon over the Continental Divide using SWRWS pipeline

In addition to the capital costs listed above, Alternative 1a will have annual operations and maintenance costs, which are included in Appendix J.

Estimated costs for Alternative 1b, including planning, design, construction, and administration/oversight are summarized in Table IX-3, below.

Table IX-3 – Cost Estimate for Alternative 1b

Item	Cost
Diversion, Conveyance, and Reservoir Construction	$583,522,000
Design	$28,786,000
Topographic Survey	$858,000
Right-of-Way Easement Development (Pipeline + Rural)	$439,000
Permitting, Environmental & Geotechnical Investigations	$5,000,000
Land Acquisition Services (Reservoirs + Pipeline + Booster Stations)	$90,000
Land Acquisition (Reservoirs + Pipeline + Booster Stations)	$6,672,000
Easement Acquisition (Pipeline + Channel)	$232,000
Construction Observation and Management	$46,682,000
Subtotal	$672,281,000
NMGRT	**$41,597,000**
Total	**$713,878,000**

The cost for Alternative 1b is based on the following:

1. 350 cfs gravity surface diversion at Site 6
2. Tunnel from the diversion to Spar
3. Storage in Spar, Winn, Pope and Sycamore Canyons
4. Pressure piping between storage sites
5. 50 cfs irrigation pump station at Winn Canyon
6. Pumping from Pope Canyon over the Continental Divide using SWRWS pipeline

In addition to the capital costs listed above, Alternative 1b will have annual operations and maintenance costs, which are included in Appendix J.

K. ALTERNATIVE 2

For the purposes of the cost estimate, Alternative 2 was split into two alternatives: 2a and 2b. The alternatives are identical except for the irrigation pump station. Under Alternative 2a, the irrigation pump station would have a capacity of 10 cfs, while Alternative 2b proposes this pump station as a 50 cfs pump station.

Estimated costs for Alternative 2a, including planning, design, construction, and administration/oversight are summarized in Table IX-4, below.

Table IX-4 – Cost Estimate for Alternative 2a

Item	Cost
Diversion, Conveyance, and Reservoir Construction	$604,438,000
Design	$31,881,000
Topographic Survey	$858,000
Right-of-Way Easement Development (Pipeline + Rural)	$439,000
Permitting, Environmental & Geotechnical Investigations	$5,000,000
Land Acquisition Services (Reservoirs + Pipeline + Booster Stations)	$90,000
Land Acquisition (Reservoirs + Pipeline + Booster Stations)	$6,672,000
Easement Acquisition (Pipeline)	$232,000
Construction Observation and Management	$48,355,000
Subtotal	$697,965,000
NMGRT	**$43,187,000**
Total	**$741,152,000**

The cost for Alternative 2a is based on the following:

1. 150 cfs gravity subsurface diversion between Turkey Creek and Site 6
2. 200 cfs gravity surface diversion at Site 6
3. Tunnel from the diversion to Spar
4. Storage in Spar, Winn, Pope and Sycamore Canyons
5. Pressure piping between storage sites
6. 10 cfs irrigation pump station at Winn Canyon
7. Pumping from Pope Canyon over the Continental Divide using SWRWS pipeline

In addition to the capital costs listed above, Alternative 2a will have annual operations and maintenance costs, which are included in Appendix J.

Estimated costs for Alternative 2b, including planning, design, construction, and administration/oversight are summarized in Table IX-5, below.

Table IX-5 – Cost Estimate for Alternative 2b

Item	Cost
Diversion, Conveyance, and Reservoir Construction	$606,622,000
Design	$32,020,000
Topographic Survey	$858,000
Right-of-Way Easement Development (Pipeline + Rural)	$439,000
Permitting, Environmental & Geotechnical Investigations	$5,000,000
Land Acquisition Services (Reservoirs + Pipeline + Booster Stations)	$90,000
Land Acquisition (Reservoirs + Pipeline + Booster Stations)	$6,672,000
Easement Acquisition (Pipeline)	$232,000
Construction Observation and Management	$48,530,000
Subtotal	$700,463,000
NMGRT	**$43,341,000**
Total	**$743,804,000**

The cost for Alternative 2b is based on the following:

1. 150 cfs gravity subsurface diversion between Turkey Creek and Site 6
2. 200 cfs gravity surface diversion at Site 6
3. Tunnel from the diversion to Spar
4. Storage in Spar, Winn, Pope and Sycamore Canyons
5. Pressure piping between storage sites
6. 50 cfs irrigation pump station at Winn Canyon
7. Pumping from Pope Canyon over the Continental Divide using SWRWS pipeline

In addition to the capital costs listed above, Alternative 2b will have annual operations and maintenance costs, which are included in Appendix J.

L. ALTERNATIVE 3

Estimated costs for Alternative 3, including planning, design, construction, and administration/oversight are summarized in Table IX-6, below.

Table IX-6 – Cost Estimate for Alternative 3

Item	Cost
Diversion, Conveyance, and Reservoir Construction	$581,400,000
Design	$22,523,000
Topographic Survey	$778,000
Right-of-Way Easement Development (Pipeline + Rural)	$415,000
Permitting, Environmental & Geotechnical Investigations	$5,000,000
Land Acquisition Services (Reservoirs + Pipeline + Booster Stations)	$63,000
Land Acquisition (Reservoirs + Pipeline + Booster Stations)	$5,598,000
Easement Acquisition (Pipeline)	$215,000
Construction Observation and Management	$46,512,000
Subtotal	$662,504,000
NMGRT	**$40,992,000**
Total	**$703,496,000**

The cost for Alternative 3 is based on the following:

1. 350 cfs pump station near Spar Canyon
2. Storage in Spar, Winn, and Pope Canyons
3. Pressure piping between storage sites
4. Pumping from Pope Canyon over the Continental Divide using SWRWS pipeline

In addition to the capital costs listed above, Alternative 3 will have annual operations and maintenance costs, which are included in Appendix J.

M. COMPARISON OF ALTERNATIVES

This Phase II PER identifies three alternatives related to diversion and storage of AWSA water in the side canyons along the Cliff-Gila Valley. The intent of the project is to provide a total water storage capacity of at least 65,000 AF. Table IX-7 below compares construction cost, annual O&M cost, and cost per AF for each alternative. The present value of the annual O&M costs is based on the assumption that the O&M costs increase by 3 percent every year. The present value is calculated for a 1.5 percent interest rate for 20 years. The cost estimates for all alternatives are included in Appendix J.

Table IX-7 – Comparison of Alternatives

Alternative	Description	Capital Cost before NMGRT	Annual O&M Costs	Present Value of O&M Costs	Total Capital and O&M Costs	Total Storage (AF)	Cost per AF
Alternative 1a	Surface diversion at Site 6, tunnel from diversion to Spar Canyon, storage at Spar, Winn, Pope, and Sycamore Canyons, 10 cfs irrigation pump station at Winn Canyon	$669,786,000	$3,156,900	$41,065,000	$710,851,000	65,417	$10,866
Alternative 1b	Surface diversion at Site 6, tunnel from diversion to Spar Canyon, storage at Spar, Winn, Pope, and Sycamore Canyons, 50 cfs irrigation pump station at Winn Canyon	$672,281,000	$3,223,400	$41,930,000	$714,211,000	65,417	$10,918
Alternative 2a	Subsurface diversion for 150 cfs above Site 6 combined with surface diversion for 200 cfs at Site 6, tunnel from diversion to Spar Canyon, storage at Spar, Winn, Pope, and Sycamore Canyons, 10 cfs irrigation pump station at Winn Canyon	$697,965,000	$3,181,900	$41,390,000	$739,355,000	65,417	$11,302
Alternative 2b	Subsurface diversion for 150 cfs above Site 6 combined with surface diversion for 200 cfs at Site 6, tunnel from diversion to Spar Canyon, storage at Spar, Winn, Pope, and Sycamore Canyons, 50 cfs irrigation pump station at Winn Canyon	$700,463,000	$3,249,400	$42,268,000	$742,731,000	65,417	$11,354
Alternative 3	Diversion at elevation 4,640 (approximately 2,000 feet downstream of the confluence of Spar Canyon with the Gila River), pumping from diversion to Spar Canyon, storage at Spar, Winn and Pope Canyons	$662,504,000	$8,030,300	$104,458,000	$766,962,000	65,482	$11,713

X. CONCLUSION AND NEXT STEPS

A. CONCLUSION

This PER recommends one new diversion location, refines analysis of potential storage at Spar and Winn canyons, summarizes a new geomorphic review, an additional geophysical/geotechnical field campaign, and analyzes pumping options and sediment control measures. All of the alternatives meet the objective of the project to provide a total water storage capacity of at least 65,000 AF to produce a safe yield of approximately 10,000 AFY.

During this phase of work, BHI was able to refine some of the assumptions that form the basis for the cost estimates presented in Section IX above. The seismic survey at the diversion site, as well as the bathymetric survey by Tetra Tech, provided more detailed information on which to base the quantities and costs for the diversion structure. One major difference between this analysis and the April 2014 PER is the recommendation for liners at each of the reservoir sites. These liners and the associated earthwork increased the cost of each dam appreciably. However, due to some refinement of the assumptions that form the basis of the hydraulic profiles, the sizes, and therefore costs of some of the conveyances, are smaller than what was proposed in the April 2014 PER.

BHI investigated the price of a radial collector well system on the Gila River. High-level cost estimates for a radial collector well system on the Gila River were on the order of $100,000,000. As discussed in Section V.A.2 above, more detailed investigation is required if a radial collector well system remains a viable alternative on the Gila River. BHI has developed the three alternatives for diversion, conveyance, and storage of Gila River water. As described herein, each alternative meets the project objectives including the ability to provide a total water storage capacity of at least 65,000 AF to produce a safe yield of approximately 10,000 AFY with a minimum of 5,000 AF of storage near the top of the Cliff-Gila valley (from which releases to mitigate drying and all of the farmland in the valley can benefit).

B. NEXT STEPS

For this project to continue, the next steps would consist of more detailed environmental, geological, and geotechnical investigations and conceptual design of a selected Alternative. The environmental investigation should include field surveys, jurisdictional determination, and permitting. Geological and geotechnical investigations

should include physical sampling of subsurface conditions at the reservoir sites and diversion site in order to provide criteria for the conceptual design. The conceptual design would refine the conveyances and storage from the appraisal level concepts presented in this report. This would enable development of more certain project cost estimates for budgeting and funding purposes.

Provided the environmental and geotechnical investigations and conceptual design do not uncover any serious technical challenges to the viability of the project, the next steps would be to secure permitting, easements, or land acquisition of the reservoir sites, pipelines, and booster stations.

Once permitting, clearances and right-of-way or easements have been secured, design survey and preliminary and final design can commence, followed by construction and commissioning. The design and construction, while forming the bulk of the cost of the project, will likely require less time in the schedule than the pre-design (investigations, permits, and conceptual design) activities.

XI. REFERENCES

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"Buckman Direct Diversion Project Raw Water Facilities Construction Plans," CH2MHill, October 24, 2008.

"City of Albuquerque North I-25 Reclamation/Reuse System Nonpotable Surface Water Reclamation Project Alameda Reuse Diversion Facilities Record Drawings," CH2MHill, April 6, 2005.

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"National Elevation Dataset," USGS.

"New Mexico Consumptive Use and Forbearance Agreement among the Gila River Indian Community, San Carlos Irrigation and Drainage District, the United States, Franklin Irrigation District, Gila Valley Irrigation District, Phelps Dodge Corporation, the Secretary of the Interior, and Other Parties Located in the Upper Valley of the Gila River," October 31, 2005.

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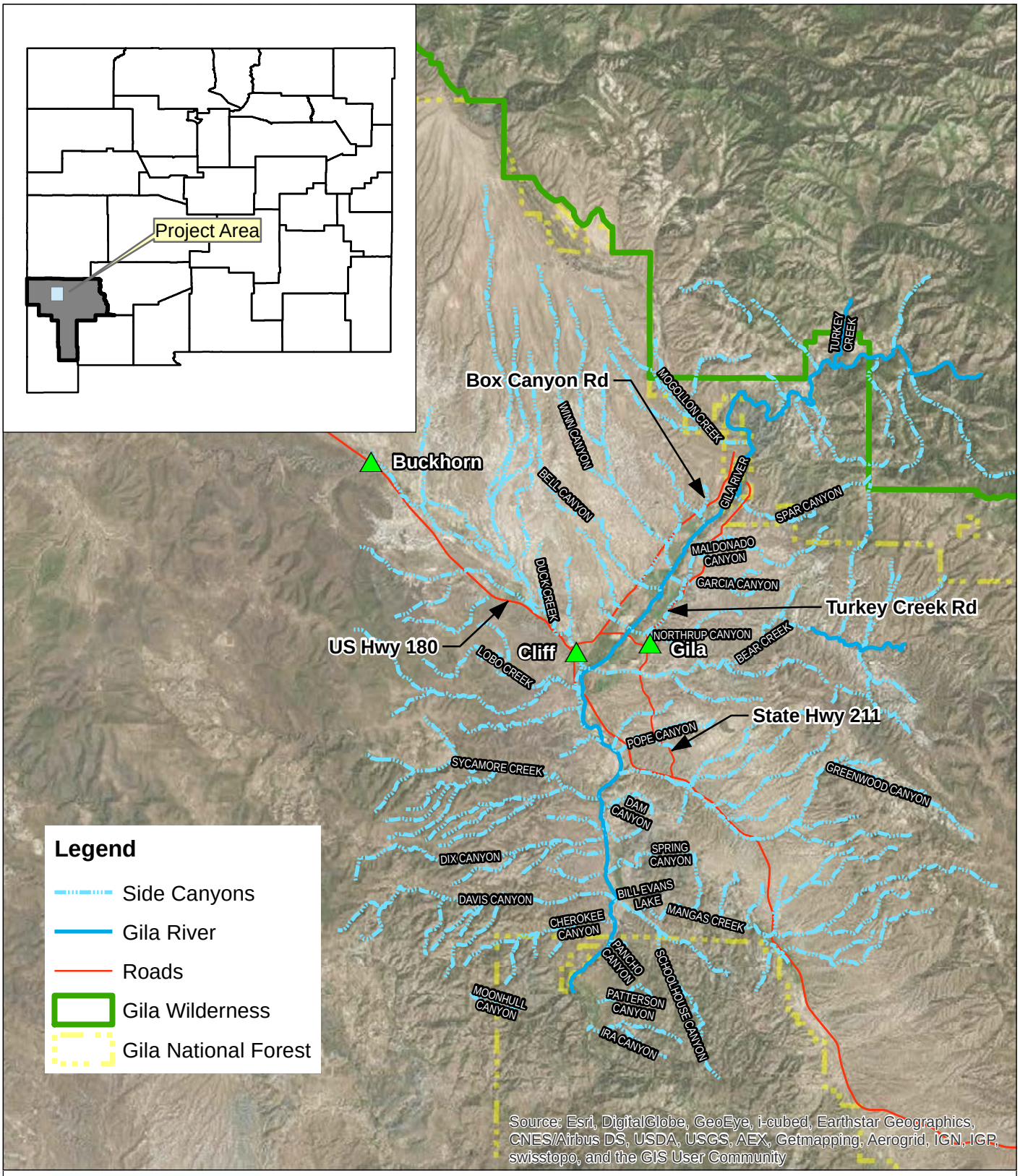
"Sediment Basin Sizing," California State Water Resources Control Board, March 18, 2008.

"Upper Gila River Fluvial Geomorphology Study," US Department of the Interior, Bureau of Reclamation, 2001-2004.

FIGURES

FIGURE 1 – VICINITY MAP



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1 inch = 20,000 feet

GILA PHASE II ENGINEERING EVALUATION

FIGURE 1
VICINITY MAP

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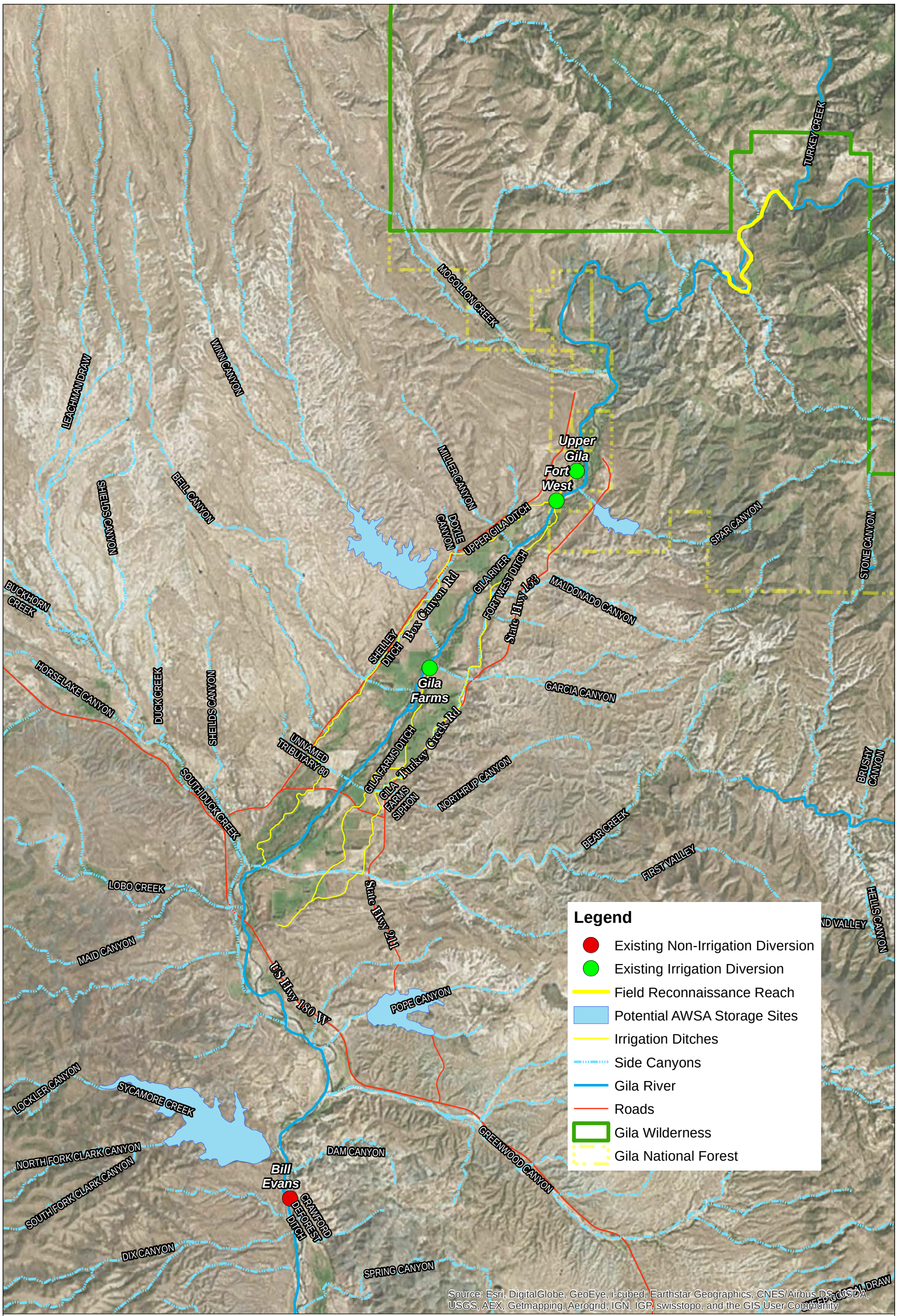
Author: gmaynard

November 2014

FIGURE 2 – PROJECT AREA



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GILA PHASE II
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FIGURE 2
PROJECT AREA

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Author: gmaynard

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FIGURE 3 – POTENTIAL AWSA DIVERSION LOCATIONS

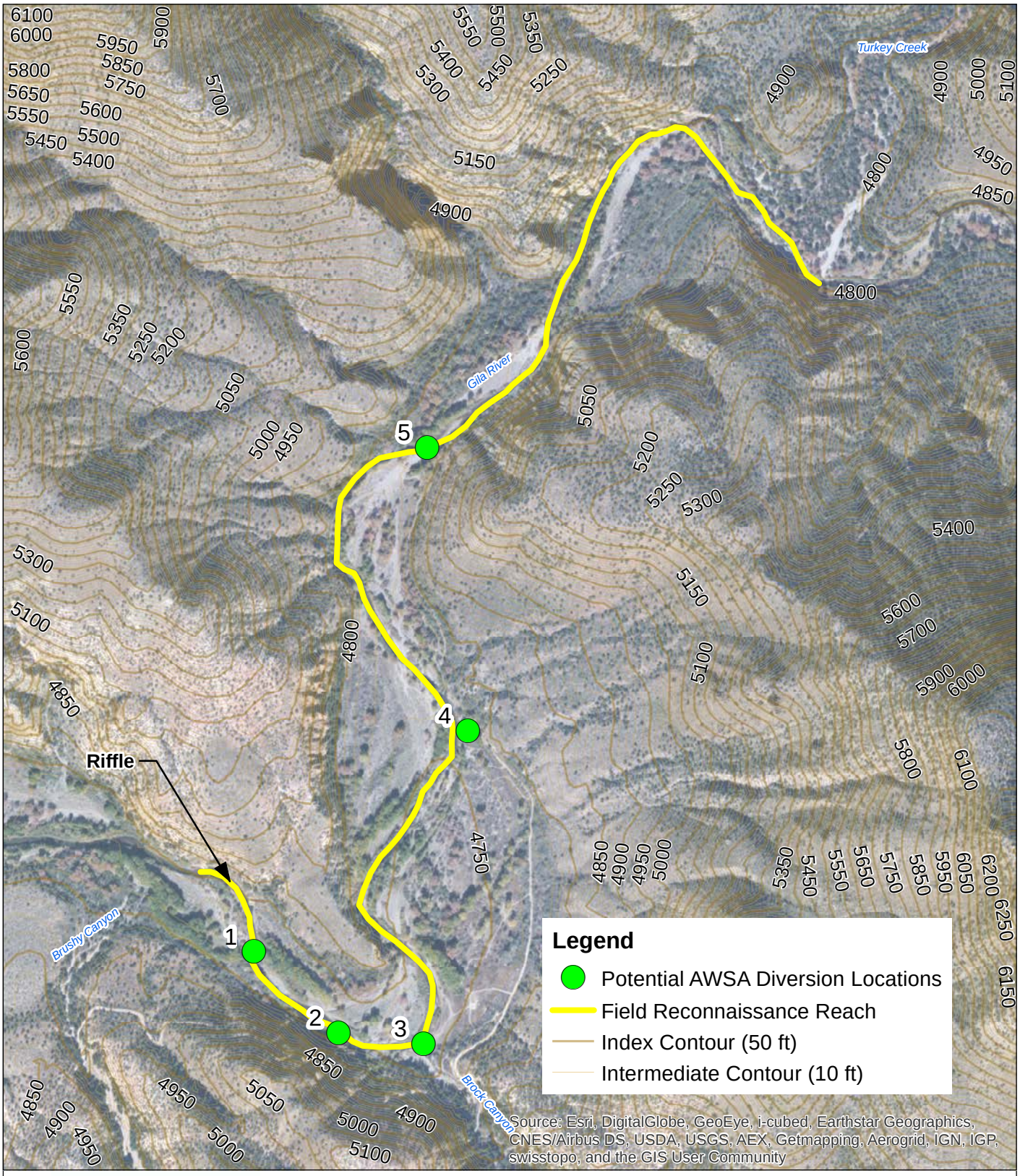

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FIGURE 3
POTENTIAL AWSA DIVERSION LOCATIONS

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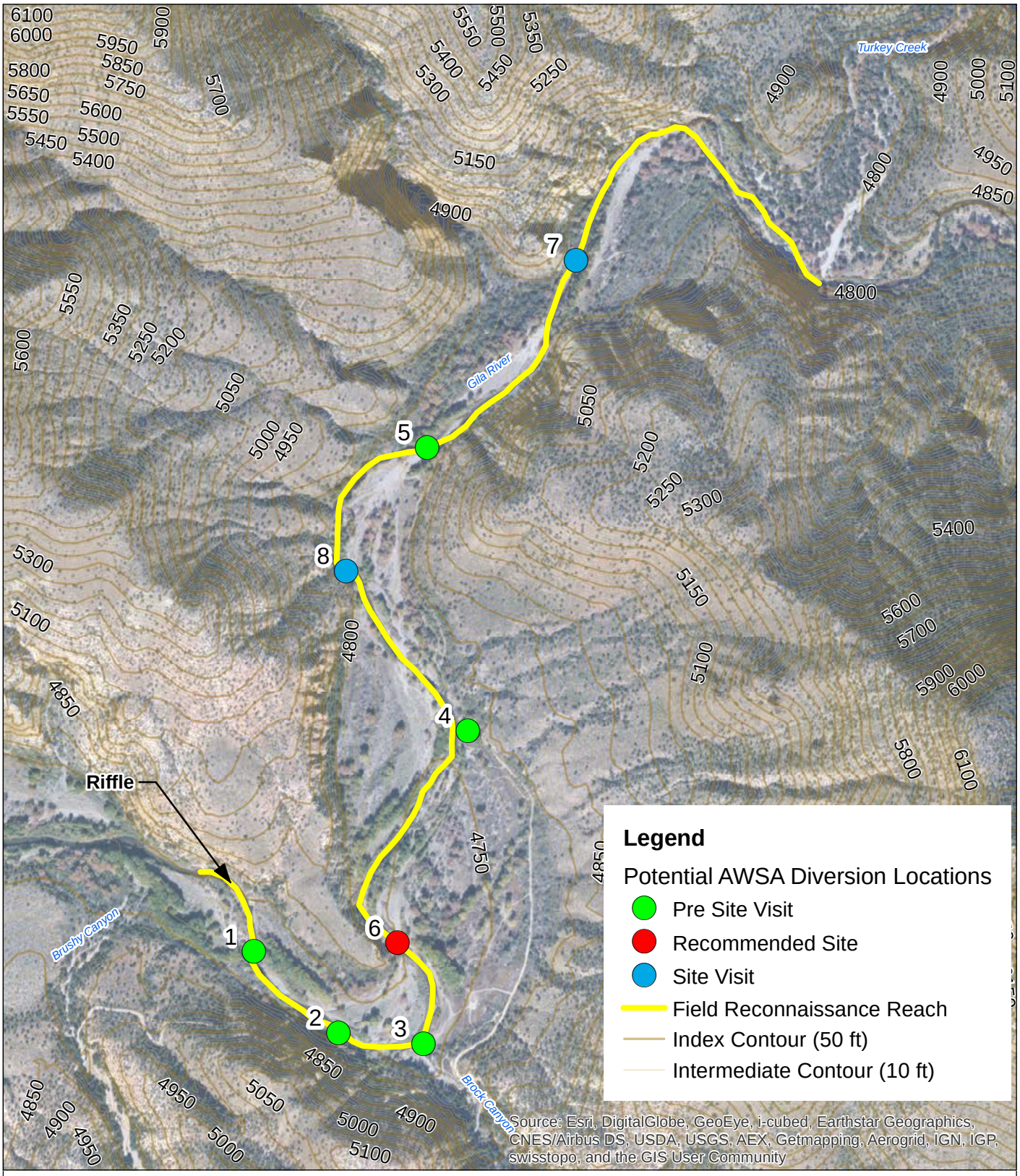
FIGURE 4 – ADDITIONAL POTENTIAL AWSA DIVERSION LOCATIONS



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GILA PHASE II ENGINEERING EVALUATION

FIGURE 4
ADDITIONAL POTENTIAL AWSA DIVERSION LOCATIONS

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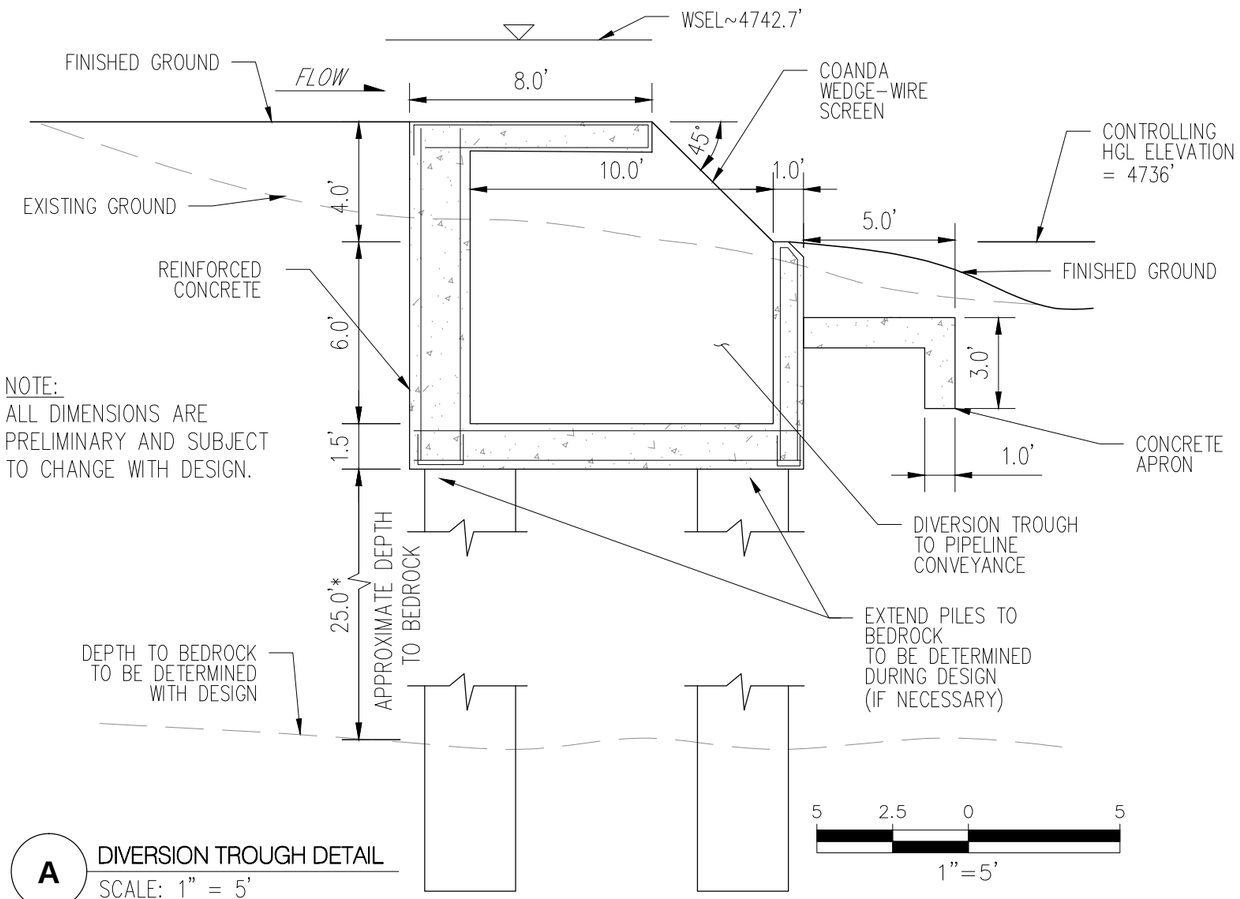
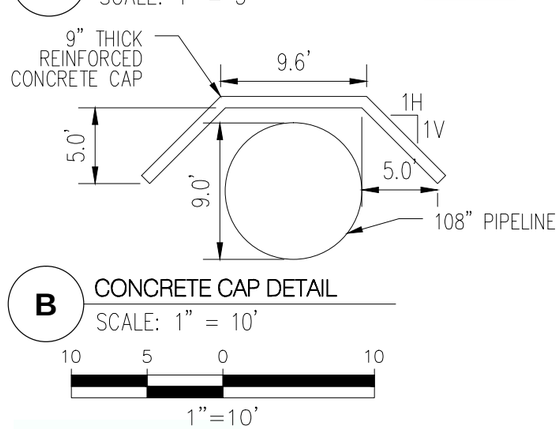
Author: gmaynard

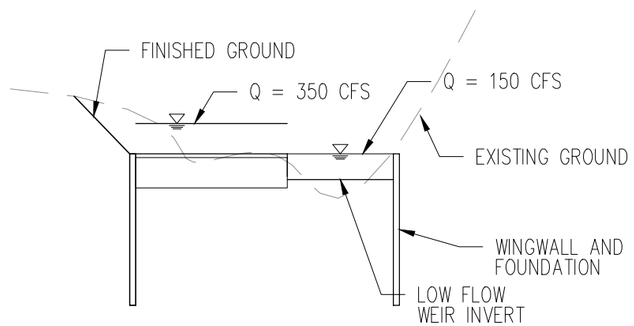
FIGURE 5 – CONFIGURATION OF LOW PROFILE CONCRETE WEIR WITH TILTED WEDGE-WIRE SCREEN



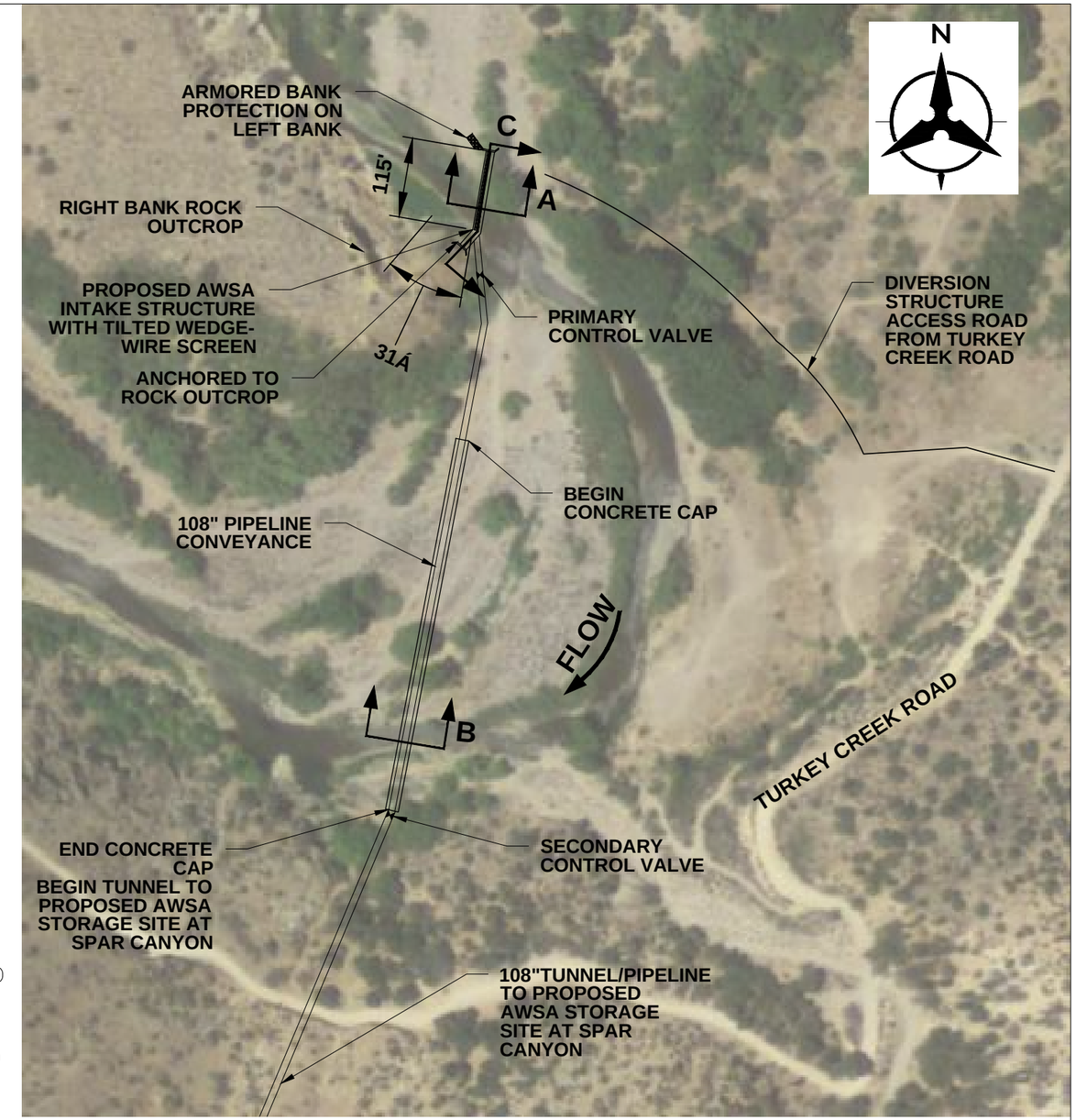
A DIVERSION TROUGH DETAIL
SCALE: 1" = 5'



B CONCRETE CAP DETAIL
SCALE: 1" = 10'



C SECTION VIEW SCHEMATIC



1 PLAN VIEW SCHEMATIC
SCALE: 1" = 200'

200 100 0 200
1"=200'

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GILA PHASE II

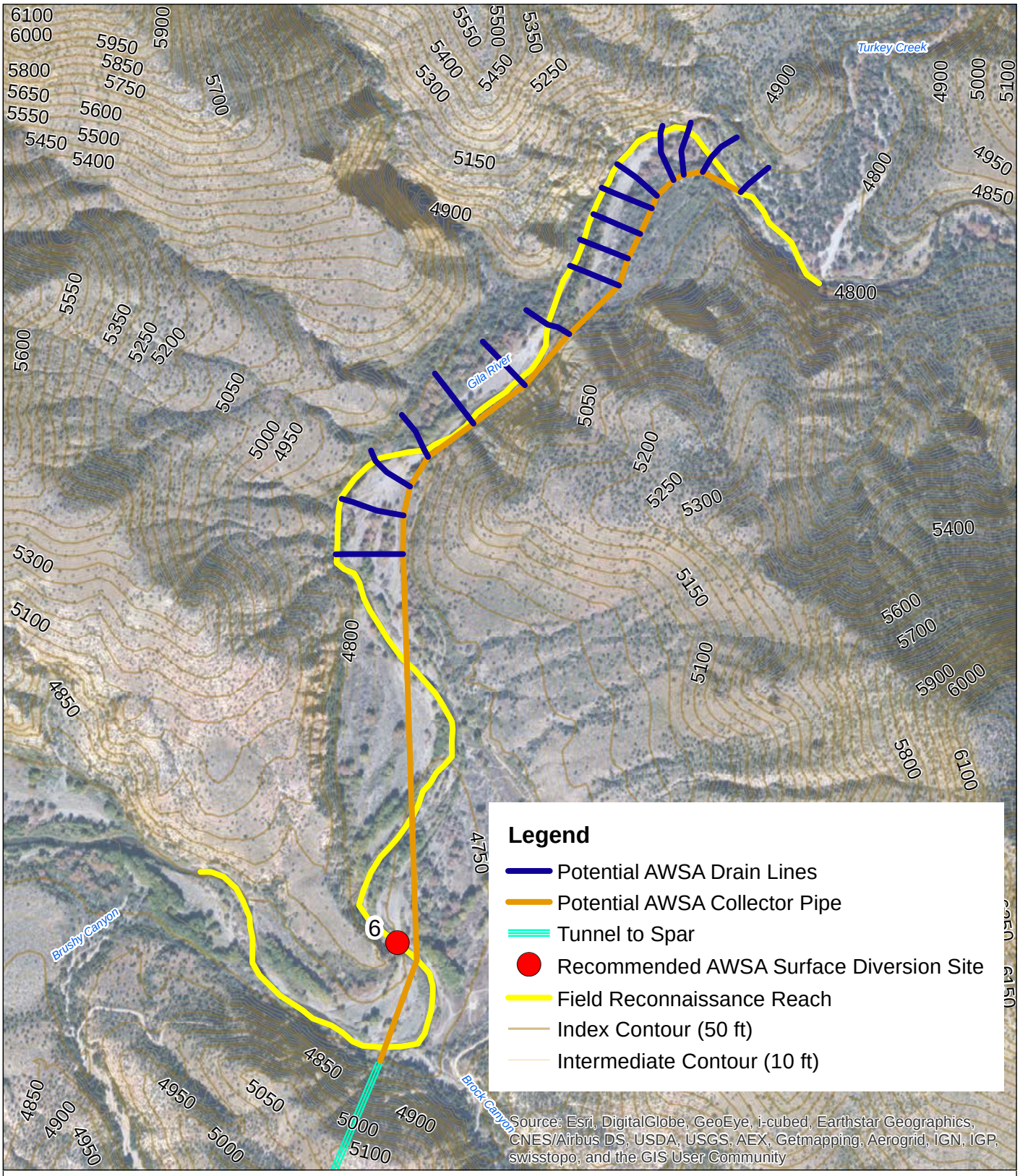
ENGINEERING EVALUATION

FIGURE 5
CONFIGURATION OF LOW PROFILE CONCRETE WEIR WITH TILTED WEDGE- WIRE SCREEN

P:\20150063\WR\Reports\Figures\Figure5_Configuration of Low Profile Concrete Weir with Tilted Wedge Wire Screen.dwg
November 05, 2014 – 2:57pm
Figure5

November 2014

FIGURE 6 – INFILTRATION GALLERY CONFIGURATION



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GILA PHASE II ENGINEERING EVALUATION

FIGURE 6
INFILTRATION GALLERY CONFIGURATION

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Author: gmaynard

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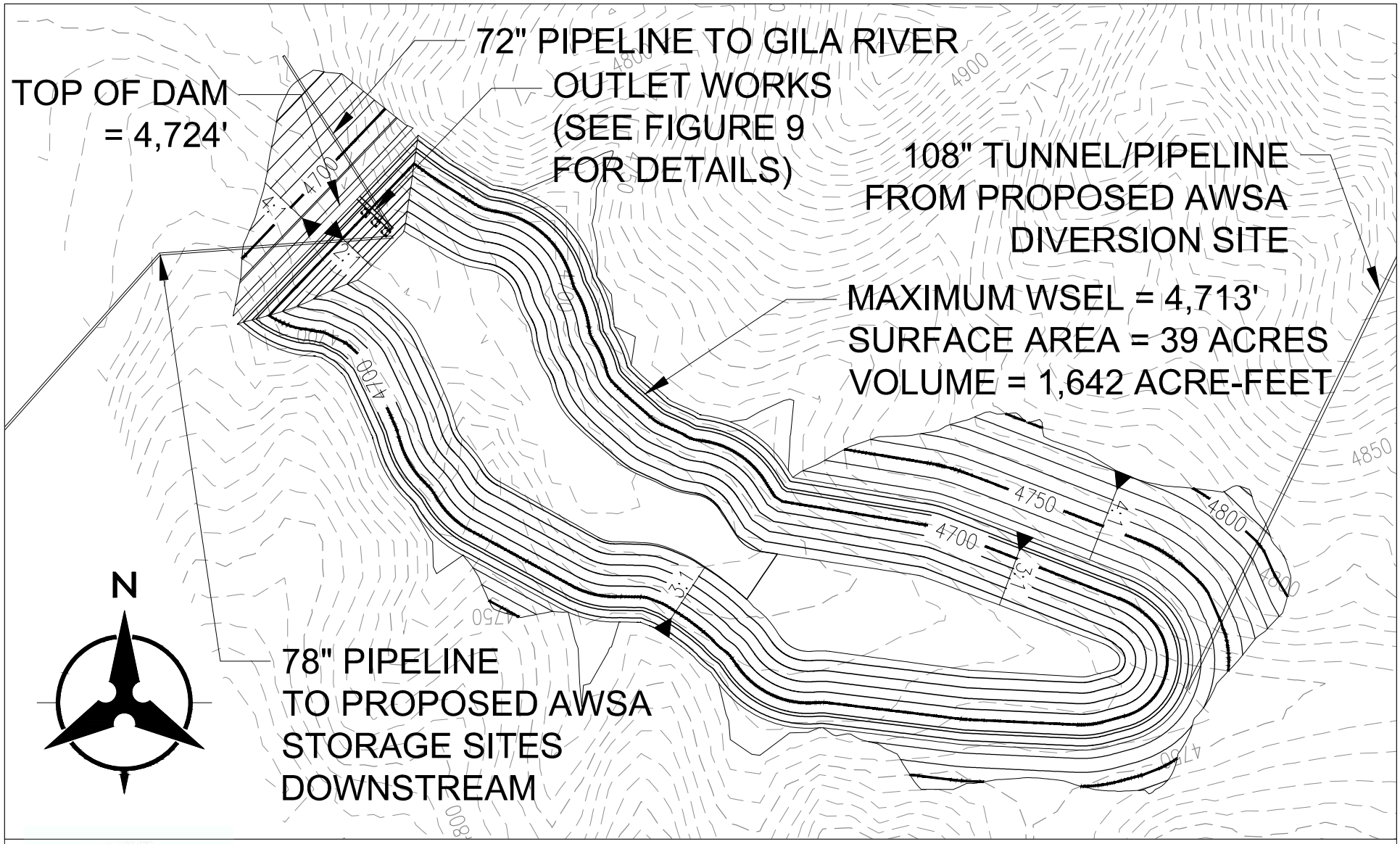
FIGURE 7 – SPAR CANYON REVISED STORAGE



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0 200 400 800
1"=400'

GILA PHASE II
ENGINEERING EVALUATION

FIGURE 7
SPAR ALTERNATIVE 1
CONCEPTUAL GRADING PLAN
WSE = 4,713'

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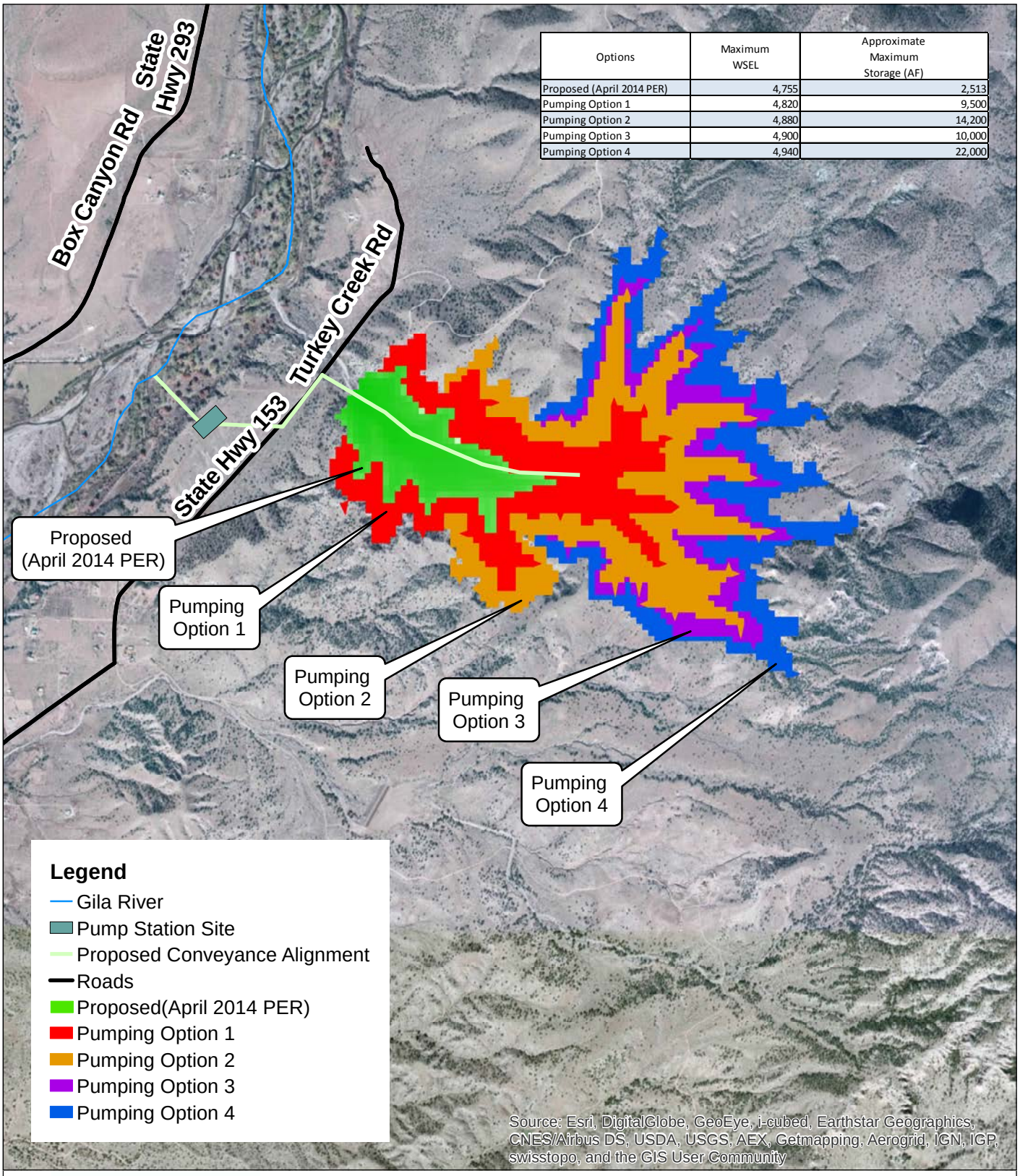
November 05, 2014 - 2:57pm
Figure7_Spar Canyon Revised Storage

November 2014

FIGURE 8 – SPAR CANYON REVISED STORAGE BASED ON PUMPING FROM THE GILA RIVER



Options	Maximum WSEL	Approximate Maximum Storage (AF)
Proposed (April 2014 PER)	4,755	2,513
Pumping Option 1	4,820	9,500
Pumping Option 2	4,880	14,200
Pumping Option 3	4,900	10,000
Pumping Option 4	4,940	22,000

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GILA PHASE II ENGINEERING EVALUATION

Figure 8
SPAR CANYON
BASED ON PUMPING
FROM GILA RIVER

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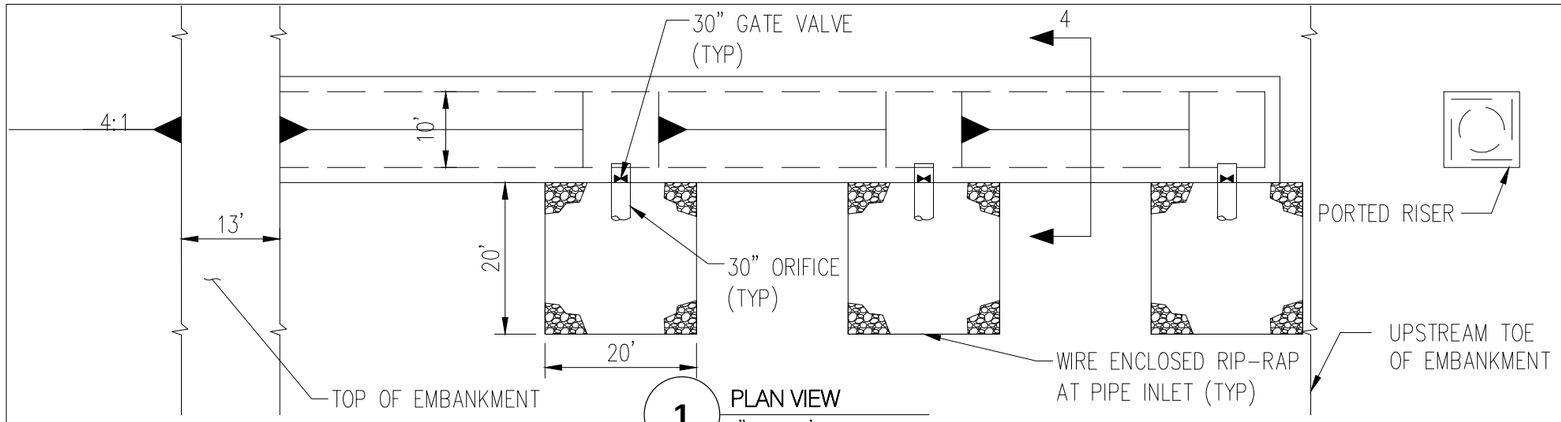
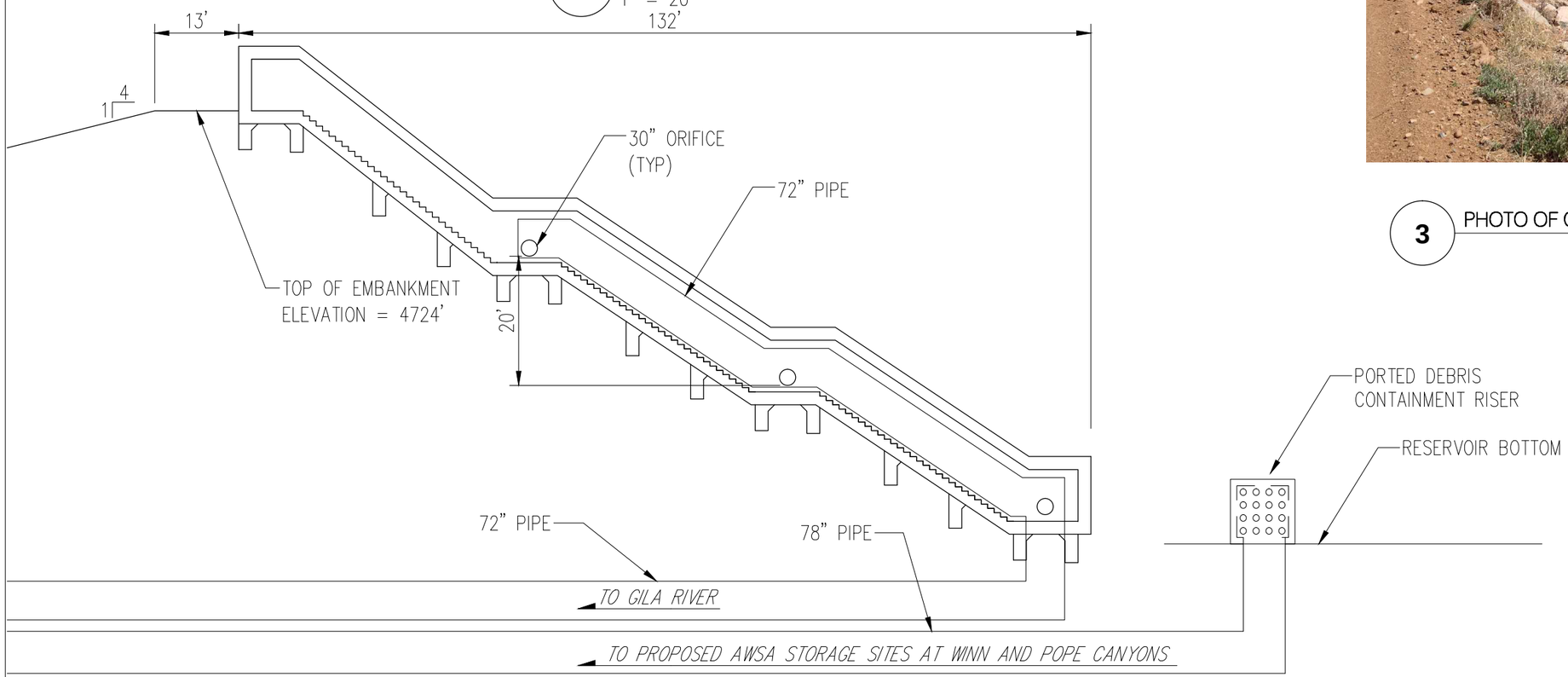
Author: gmaynard

November 2014

FIGURE 9 – SPAR CANYON RESERVOIR OUTLET WORKS CONCEPTUAL PLAN

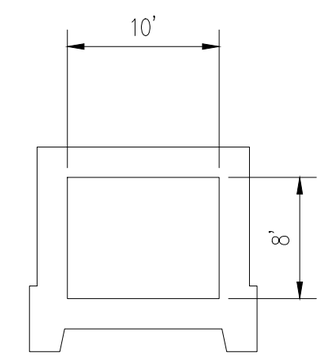


1 PLAN VIEW
1" = 20'



2 PROFILE VIEW
1" = 20'

3 PHOTO OF OUTLET STRUCTURE CONCEPT



4 TYPICAL SECTION
1" = 10'

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GILA PHASE II

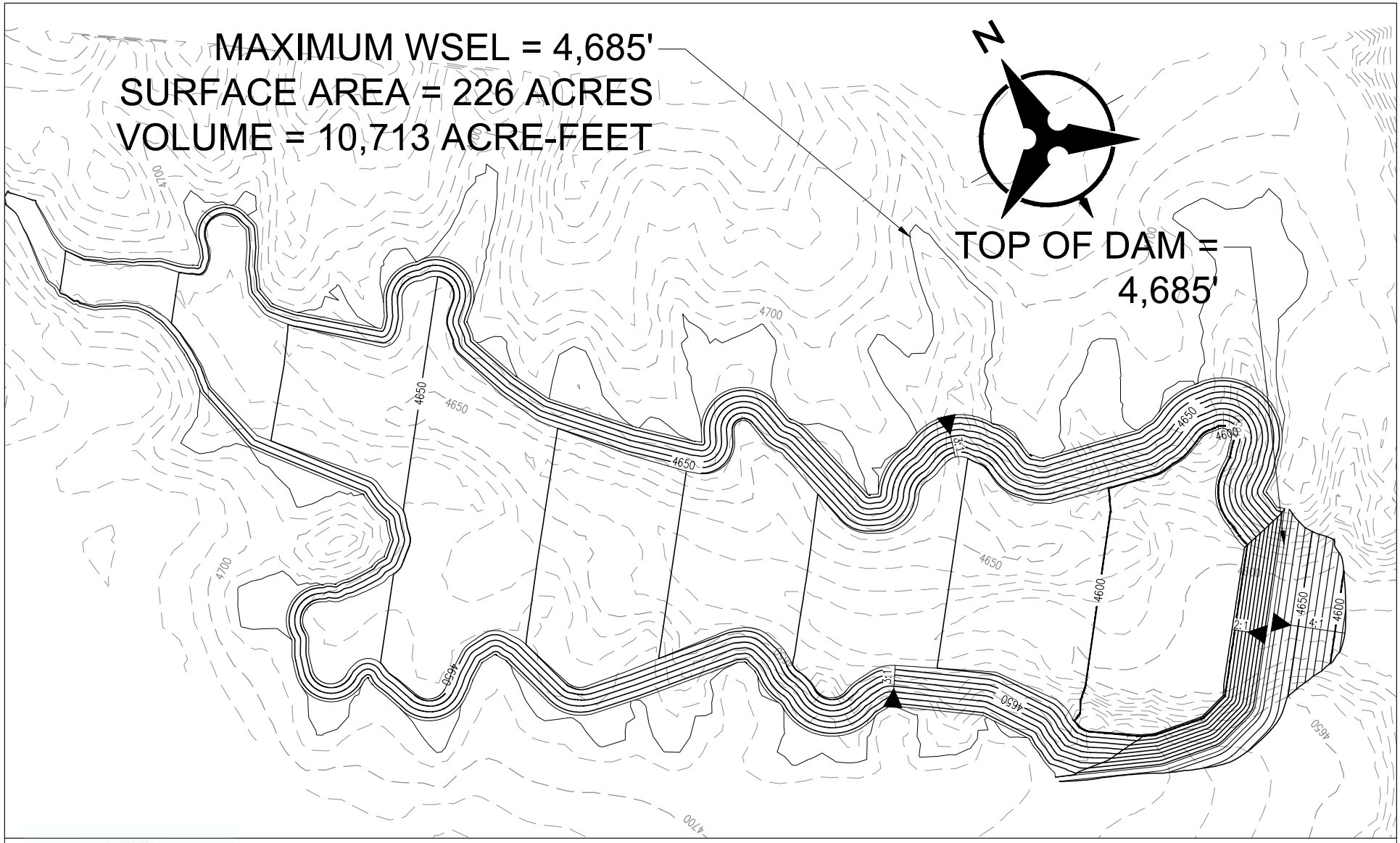
ENGINEERING EVALUATION

FIGURE 9
SPAR CANYON RESERVOIR
OUTLET WORKS CONCEPTUAL PLAN

P:\20150063\WR\Reports\Figures\Figure9_Conceptual Site Plan for Spar Canyon Reservoir Outlet.dwg
November 05, 2014 – 2:59pm
Conceptual Site Plan for Spar Pump Station

November 2014

FIGURE 10 – WINN CANYON REVISED STORAGE



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0 325 750 1400
1"=750'

GILA PHASE II
ENGINEERING EVALUATION

FIGURE 10
WINN CANYON
CONCEPTUAL GRADING PLAN
WSE = 4,685'

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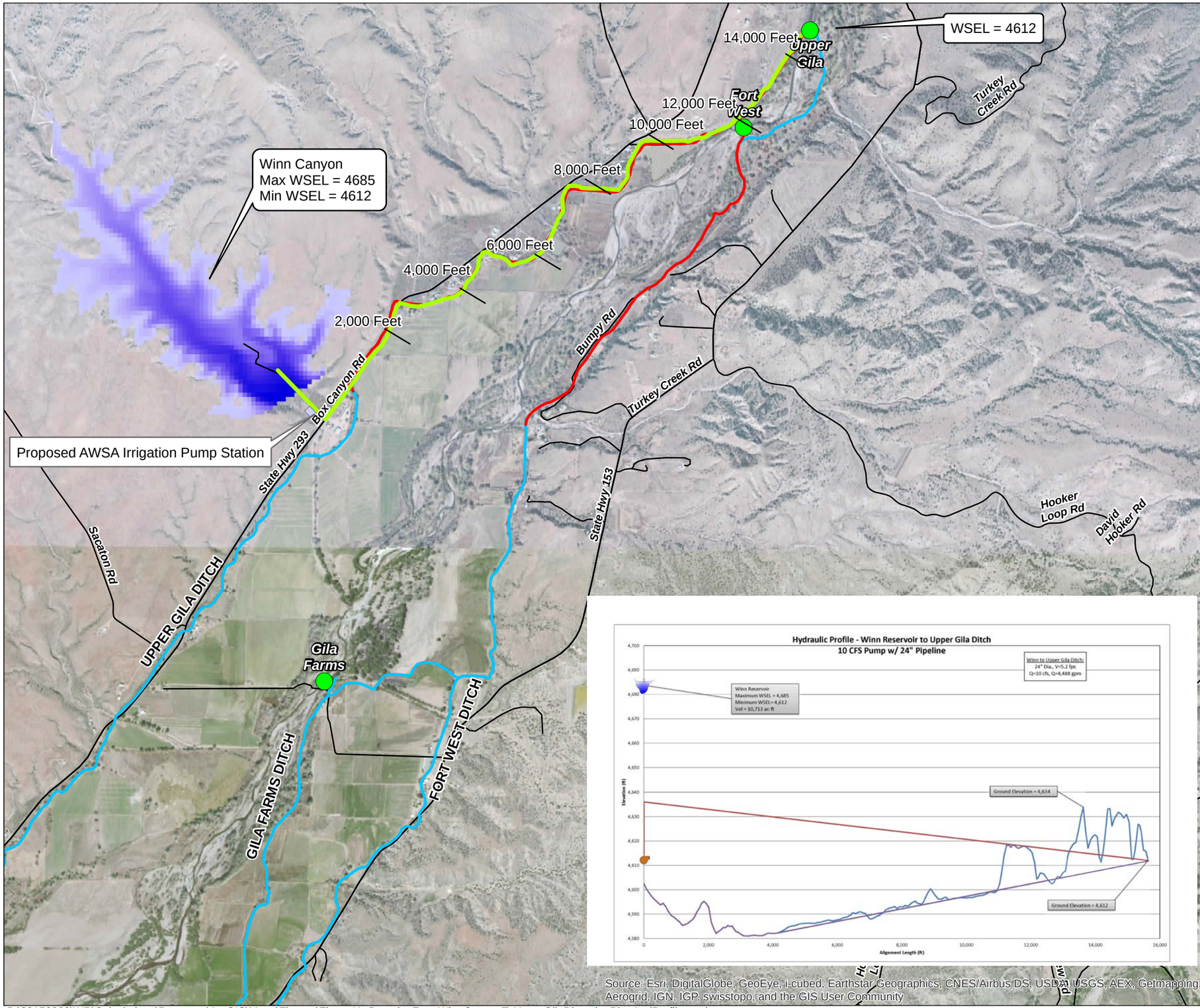
November 05, 2014 – 3:00pm
Figure 10 Winn Canyon Revised Storage

November 2014

FIGURE 11 – PUMPING FROM WINN CANYON TO UPPER GILA DIVERSION PLAN AND PROFILE

GILA PHASE II
ENGINEERING EVALUATION

FIGURE 11
PUMPING FROM WINN CANYON
TO UPPER GILA DIVERSION
PLAN AND PROFILE



Legend

- Proposed AWSA Irrigation Pump Station
- Existing Non-Irrigation Diversion
- Existing Irrigation Diversion
- Proposed AWSA Irrigation Pipeline Alignment
- Existing Ditches
- AWSA Irrigation Ditches
- Roads

Proposed AWSA Storage Site at Winn Canyon

High Depth

Low Depth

N
W E
S

0 2,000 4,000 Feet

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P:\20150063\WR\Calculations\Programs\ArcGIS\Maps\Figures\Figure11_PumpingFromWinnCanyonToUpperGilaDiversionPlanAndProfile.mxd
Author: gmaynard

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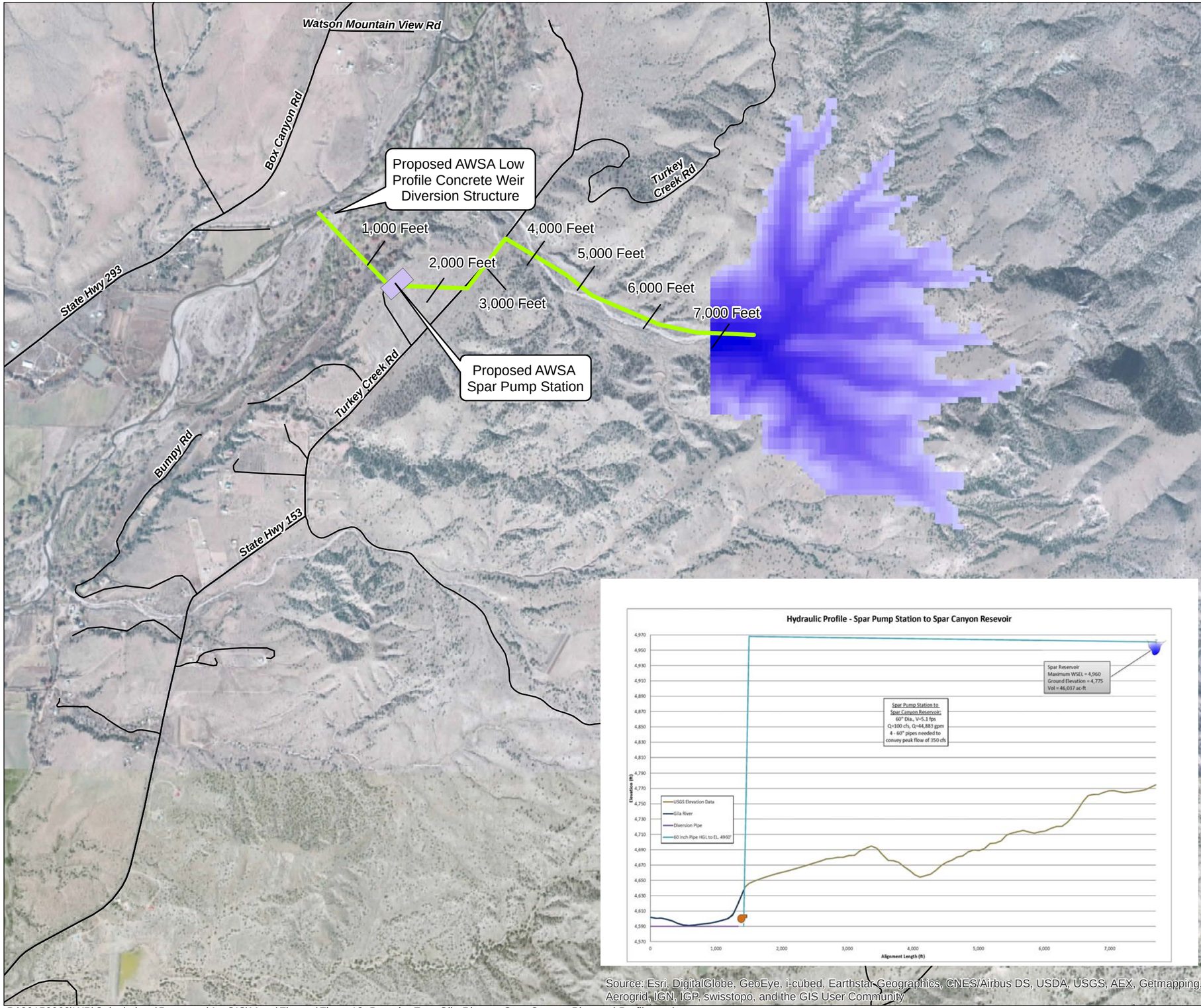
FIGURE 12 – PUMPING FROM GILA RIVER TO SPAR CANYON PLAN AND PROFILE

GILA PHASE II
ENGINEERING EVALUATION

FIGURE 12
PUMPING FROM GILA RIVER
TO SPAR CANYON PLAN AND PROFILE



Legend

- Proposed AWSA Spar Pump Station
- Proposed AWSA Spar Conveyance Alignment
- Roads

Proposed AWSA Storage Site at Spar Canyon

- High Depth
- Low Depth

N
W E
S

0 1,500 3,000
Feet

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P:\20150063\WR\Calculations\Programs\ArcGIS\Maps\Figures\Figure12_PumpingFromGila River to Spar Canyon Plan and Profile.mxd
Author: gmaynard

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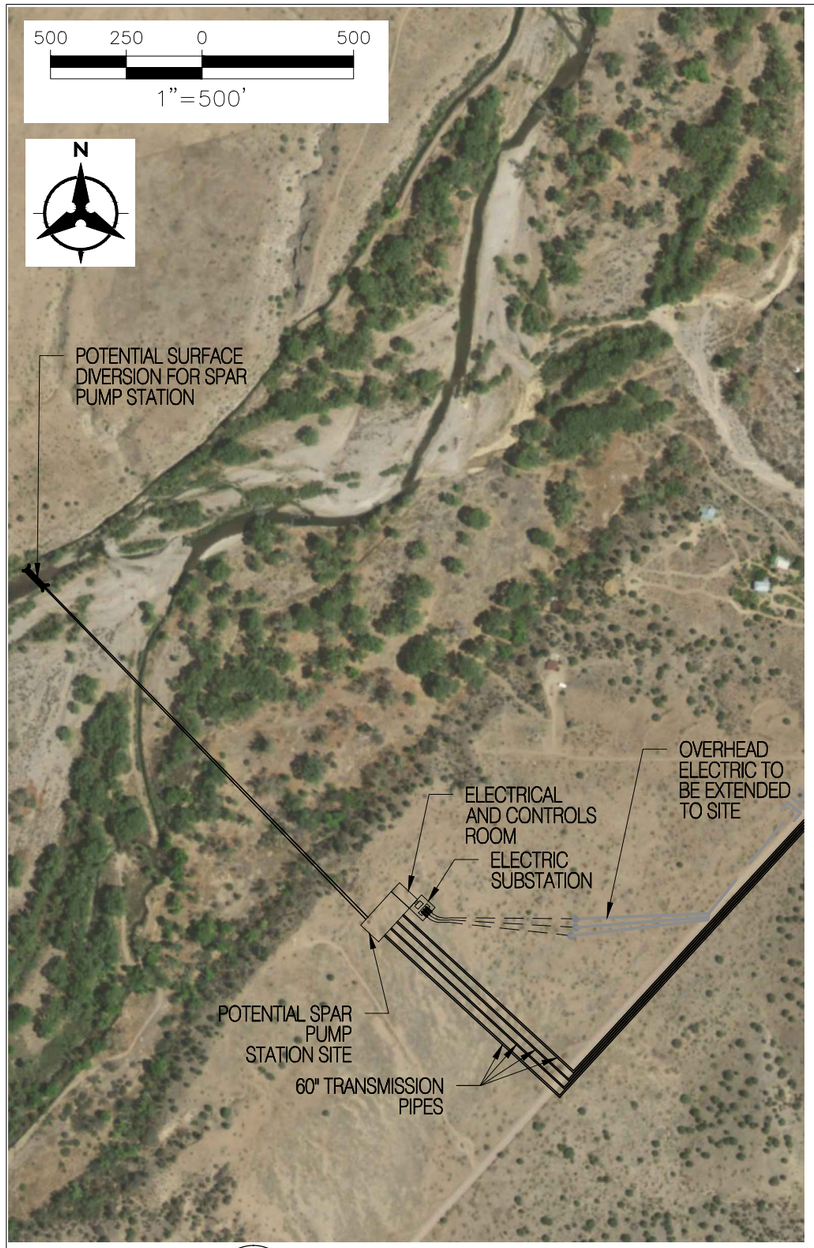
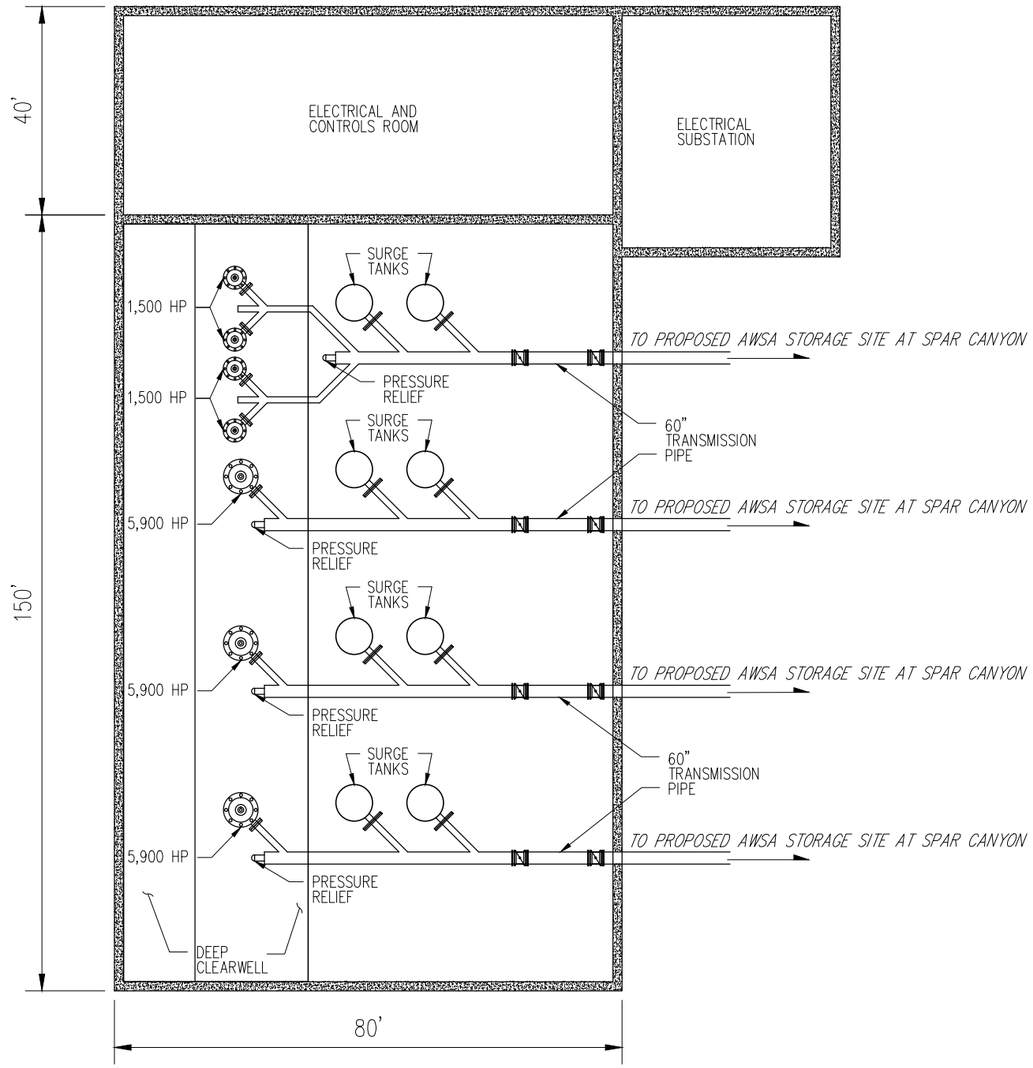
FIGURE 13 – CONCEPTUAL SITE PLAN FOR SPAR PUMP STATION



1 PLAN VIEW
SCALE: 1" = 500'



2 PLAN VIEW PUMP ROOM CONCEPTUAL SITE PLAN

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GILA PHASE II

ENGINEERING EVALUATION

FIGURE 13
CONCEPTUAL SITE PLAN
FOR SPAR PUMP STATION

P:\20150063\WR\Reports\Figures\Figure13_Conceptual Site Plan for Spar Pump Station.dwg
November 05, 2014 - 3:05pm
Conceptual Site Plan for Spar Pump Station

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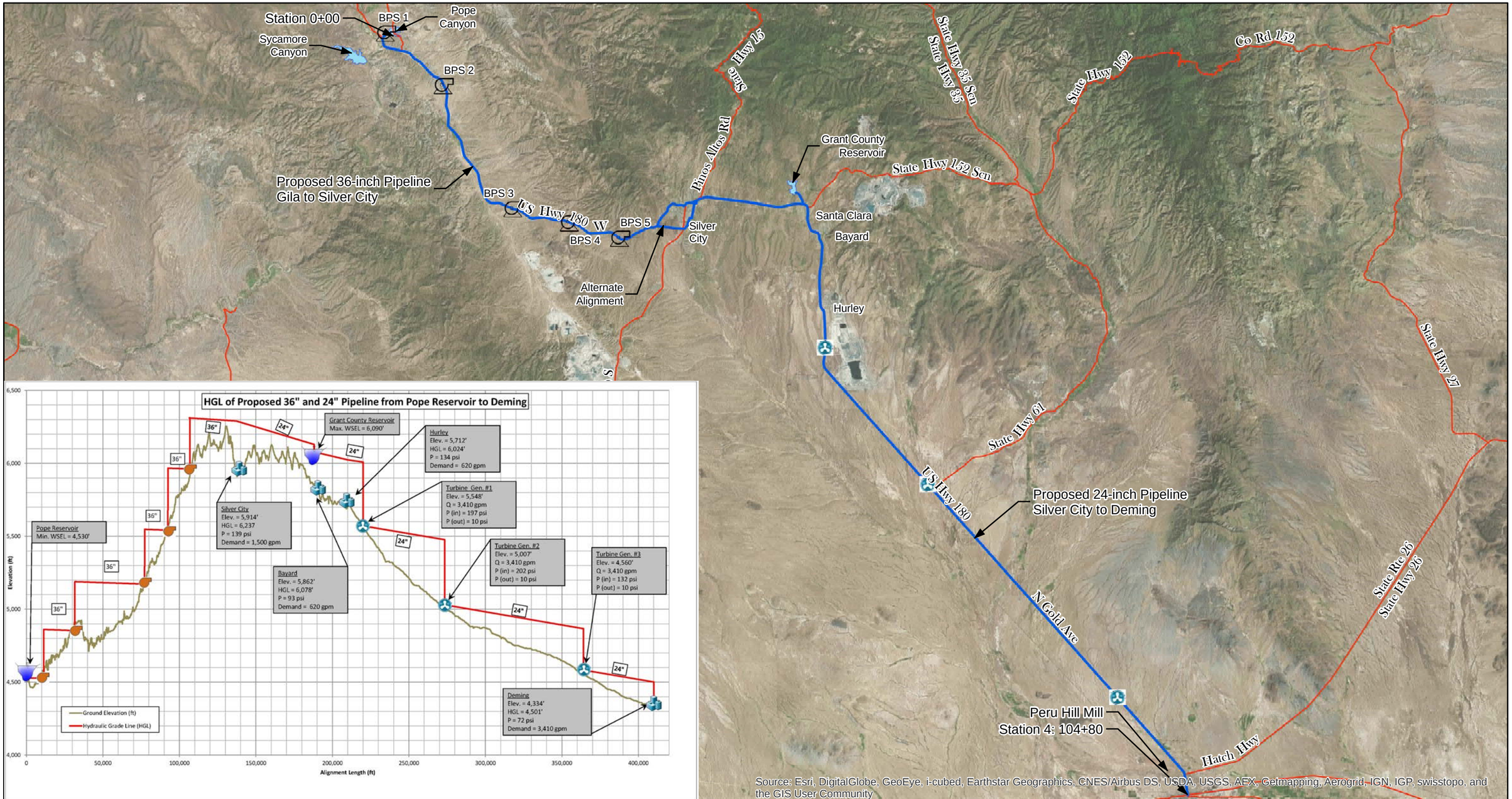
FIGURE 14 – SOUTHWEST NEW MEXICO REGIONAL WATER SUPPLY



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Legend

- HydroelectricTurbines
- Booster Pump Station
- Pipeline
- Roads
- Potential AWSA Storage Sites

N
W E
S

0 30,000 60,000 Feet

GILA PHASE II ENGINEERING EVALUATION

FIGURE 14
SOUTHWEST NEW MEXICO REGIONAL WATER SUPPLY

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Author: gmaynard

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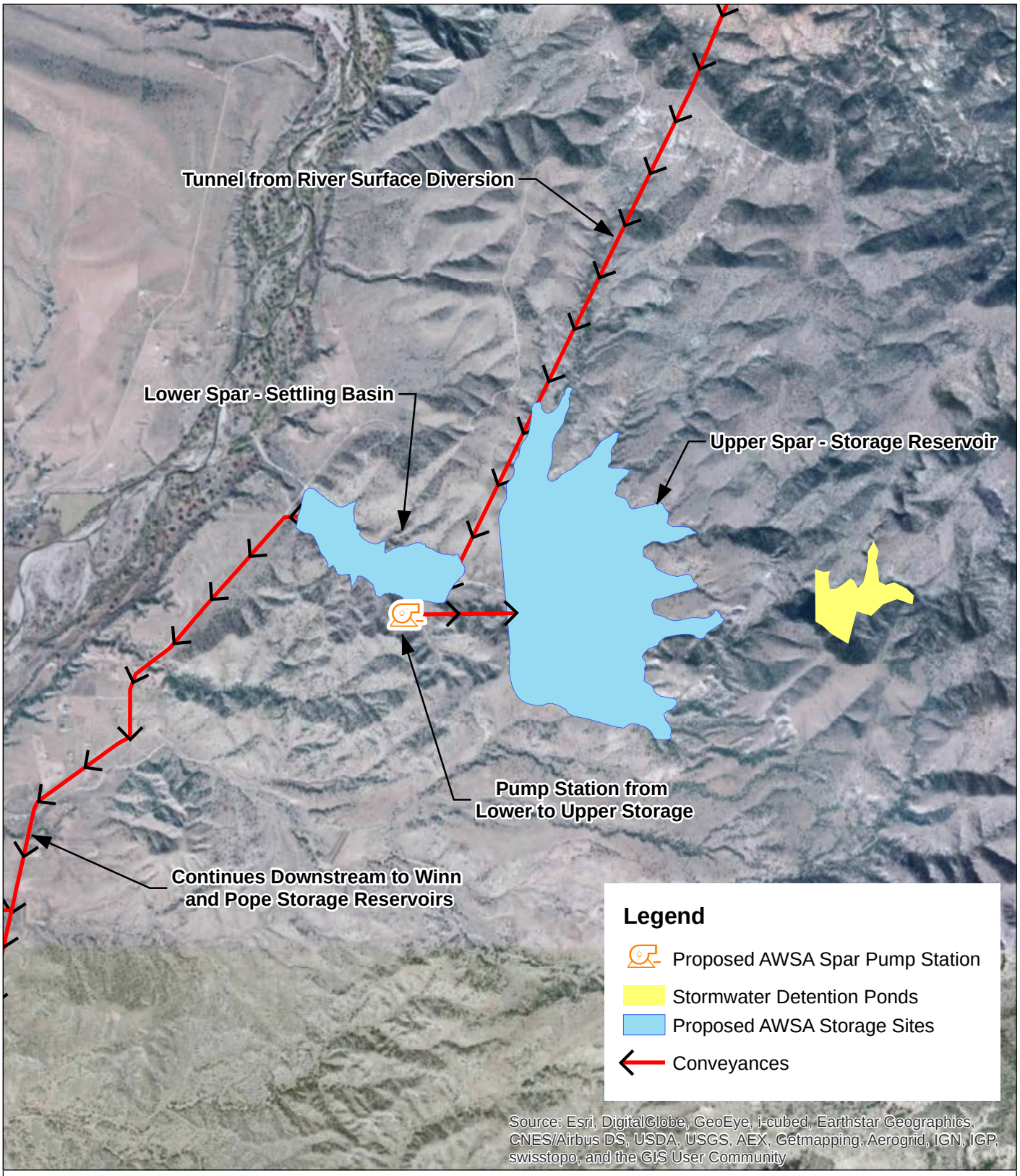
FIGURE 15 – COMBINED GRAVITY AND PUMPING SCENARIO



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N
W E
S

0 1,250 2,500 5,000
Feet

GILA PHASE II ENGINEERING EVALUATION

FIGURE 15
COMBINED GRAVITY AND PUMPING SCENARIO

P:\20150063\WR\Calculations\Programs\ArcGIS\Maps\Figures\Figure15_Combined_Gravity.mxd
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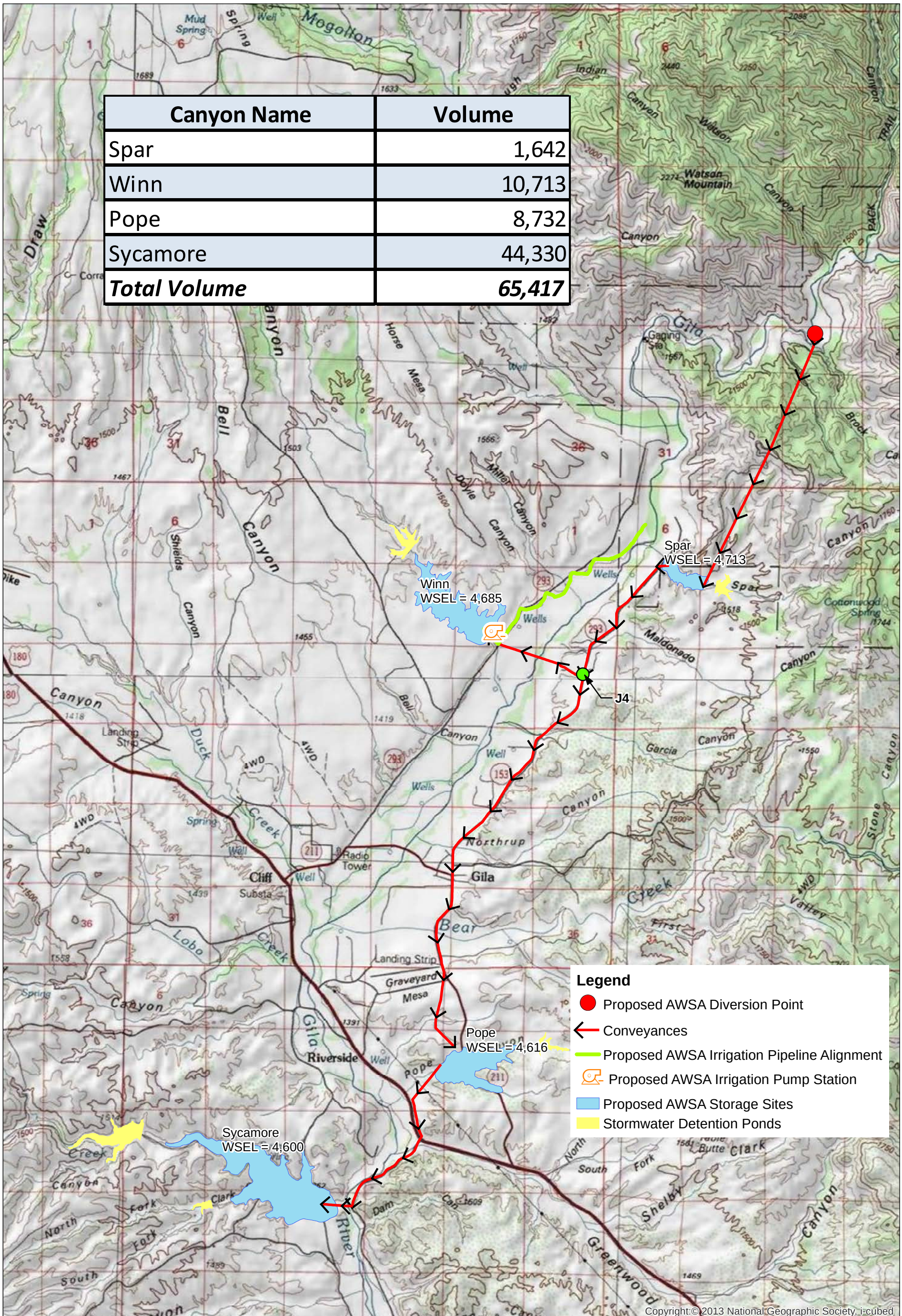
FIGURE 16 – ALTERNATIVE 1

Canyon Name	Volume
Spar	1,642
Winn	10,713
Pope	8,732
Sycamore	44,330
Total Volume	***65,417***



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N
W E
S

0 3,000 6,000 12,000 Feet

GILA PHASE II
ENGINEERING EVALUATION

FIGURE 16
ALTERNATIVE 1

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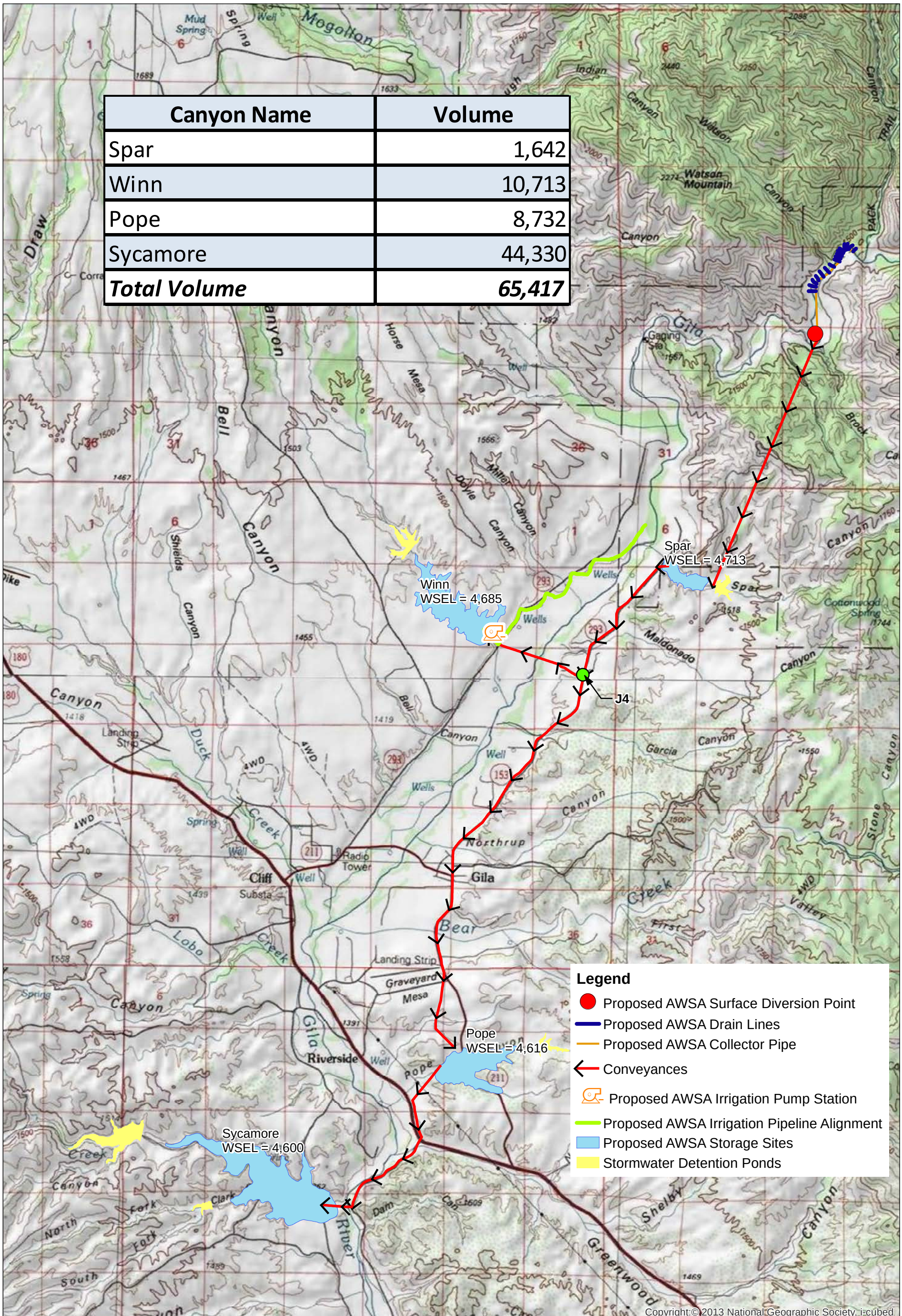
FIGURE 17 – ALTERNATIVE 2

Canyon Name	Volume
Spar	1,642
Winn	10,713
Pope	8,732
Sycamore	44,330
Total Volume	***65,417***



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N
W E
S

0 3,000 6,000 12,000 Feet

GILA PHASE II
ENGINEERING EVALUATION

FIGURE 17
ALTERNATIVE 2

P:\20150063\WR\Calculations\Programs\ArcGIS\Maps\Figures\Figure17_Alternative2.mxd
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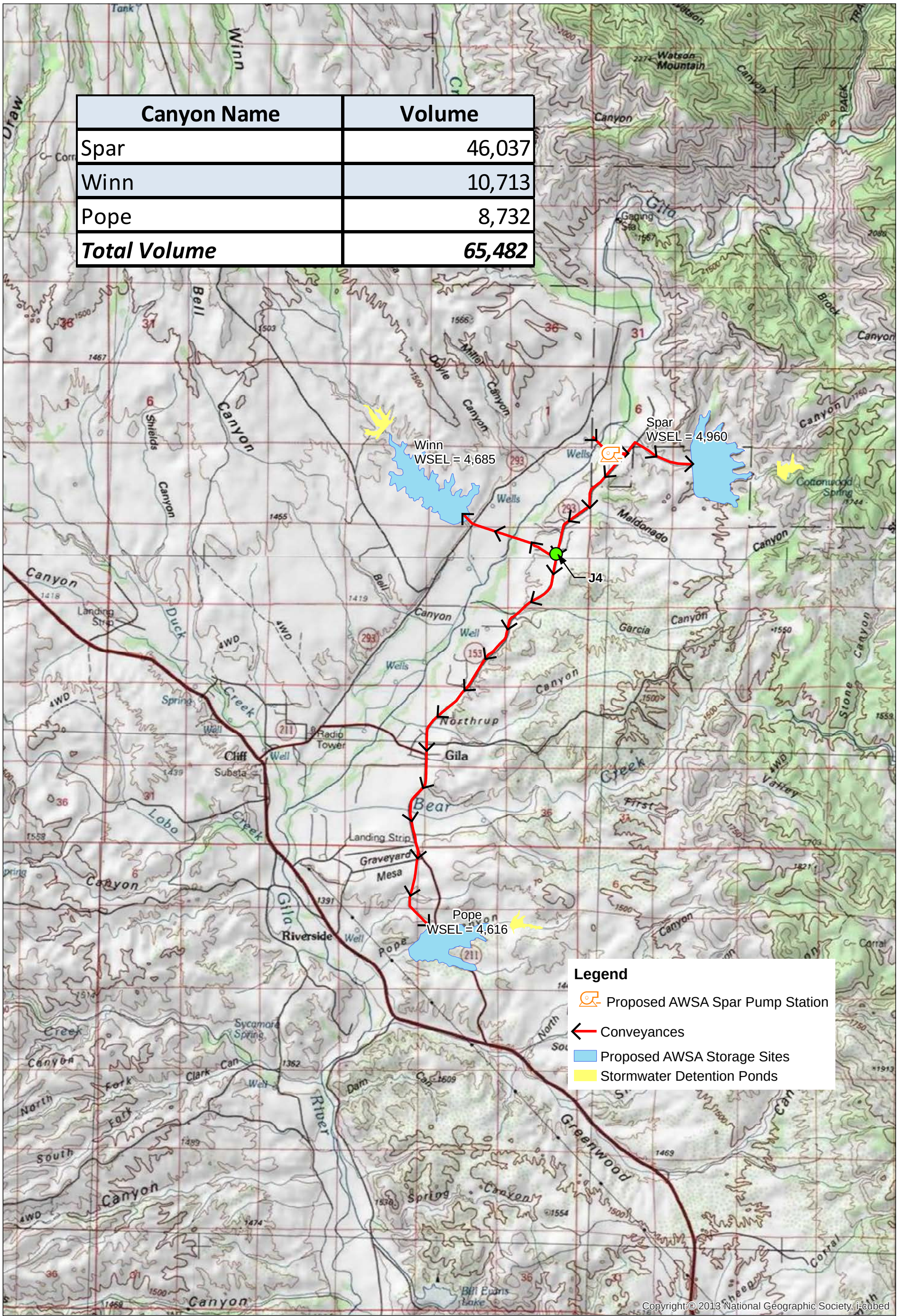
FIGURE 18 – ALTERNATIVE 3

Canyon Name	Volume
Spar	46,037
Winn	10,713
Pope	8,732
Total Volume	***65,482***



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N
W E
S

0 3,000 6,000 12,000 Feet

GILA PHASE II
ENGINEERING EVALUATION

FIGURE 18
ALTERNATIVE 3

P:\20150063\WR\Calculations\Programs\ArcGIS\Maps\Figures\Figure18_Alternative3.mxd
Author: gmaynard

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