PRELIMINARY ENGINEERING REPORT

GILA RIVER DIVERSION, CONVEYANCE AND STORAGE ALTERNATIVES

APRIL 2014

Prepared For:

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

Prepared By:

Bohannan Huston

Engineering
Spatial Data
Advanced Technologies

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FOR

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ABBREVIATIONS AND ACRONYMS

AF	acre-foot
AFY	acre-foot per year
AWSA	Arizona Water Settlements Act
BHI	Bohannan Huston, Inc.
BoR	Bureau of Reclamation
CBC	Concrete Box Culvert
cfs	cubic feet per second
CUFA	Consumptive Use and Forbearance Agreement
CY	Cubic Yards
DBS&A	Daniel B. Stephens and Associates
DEM	Digital Elevation Model
FWS	US Fish and Wildlife Service
GBIC	Gila Basin Irrigation Commission
gpm	gallons per minute
HEC-HMS	US Army Corps of Engineers' Hydrologic Engineering Center Hydrologic Modeling System
HGL	Hydraulic Grade Line
ISC	New Mexico Interstate Stream Commission
LCY	Loose Cubic Yards
NEPA	National Environmental Policy Act
NMAC	New Mexico Administrative Code
NMDOT	New Mexico Department of Transportation
NMED	New Mexico Environment Department
NMGRT	New Mexico Gross Receipts Tax
NOAA	National Oceanic and Atmospheric Administration
NPDES	National Pollutant Discharge Elimination System
NRCS	Natural Resources Conservation Service
NWS	National Weather Service
O&M	Operating and Maintenance
OSE	New Mexico Office of the State Engineer
OSHA	Occupational Safety and Health Administration
PER	Preliminary Engineering Report
PFWD	Plan Formulation Working Document
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
RCP	Reinforced Concrete Pipe
SCS	Soil Conservation Service
SSURGO	Soil Survey Geographic Database
STATSGO	NRCS State Soil Geographic Data
SWCA	SWCA Environmental Consultants
SWRWS	Southwest Regional Water Supply
TMB	Tunnel Boring Machine
TDH	Total Dynamic Head
USFS	US Forest Service
USGS	US Geological Survey

EXECUTIVE SUMMARY

This preliminary engineering report (PER) identifies recommended options for water diversion, conveyance, and storage along the Gila River between Turkey Creek and Cherokee Canyon, near the towns of Cliff and Gila in Grant County in southwestern New Mexico under the 2004 "Arizona Water Settlements Act" (AWSA) that allocates to New Mexico an annual average of 14,000 acre-feet (AF) of additional water from the Gila basin. The project location can be seen in Figure 1. The project planning area can be seen in Figure 2. Locations for off-stream storage in the side canyons leading into the Gila are evaluated. In addition to locations for storage, this report also identifies locations for diversions 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). The Category 1 and Category 2 diversion locations can be seen in Figure 3.

Under the terms of the AWSA, up to $128 million of funding may be available. Of this, $66 million must be used only in a four-county region in southwestern New Mexico. The four counties are Grant, Luna, Hidalgo and Catron. The rest of the money can be used anywhere in New Mexico. The New Mexico Interstate Stream Commission (ISC) is evaluating stakeholder proposals for use of water and available funding to New Mexico under the AWSA.

Bohannan Huston Inc. (BHI) was contracted by the ISC in May 2013 to develop and evaluate options for diversion, conveyance and storage of the AWSA water in the Gila valley. A major component of this effort was the assessment of the potential for storage of at least 65,000 AF 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 the releases for the health of the river and its riparian habitat. The area studied includes that reach of the Gila River between Turkey Creek to the north and Cherokee Canyon to the south.

The evaluation of side canyons along the Gila River for potential water storage was conducted as an iterative process. This report describes the iterations, criteria, and resulting recommendations which were then incorporated into alternatives. An “alternative” is comprised of a system of diversion, storage sites and conveyances from the diversion and between the impoundment areas.

The alternatives and resultant maximum system storage in the side canyons was dictated primarily by the elevation of the potential Category 2 diversions. Based upon the project scope, three Category 2 diversions were considered. The most upstream potential Category 2 diversion site is located downstream of Turkey Creek and the most downstream diversion site is located upstream of Mogollon Creek. There are three potential diversion locations, shown on Figure 3, with approximate elevations ranging from 4,770 to 4,640 that have 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 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. Alternative 1 can be seen in Figure 25. Alternative 2 can be seen in Figure 26. Alternative 3 can be seen in Figure 27.

As a result of this extensive analysis, three primary alternatives were developed. These alternatives and their approximate total project costs are presented in this report and are summarized below.

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

BHI recommends construction of Alternative 2B. It meets the objectives of the project, and it includes a minimum of 5,000 AF of storage near the top of the Cliff-Gila valley (from which most of the farmland in the valley can benefit).

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This preliminary engineering report (PER) identifies recommended options for water diversion, conveyance, and storage along the Gila River between Turkey Creek and Cherokee Canyon, near the towns of Cliff and Gila in Grant County in southwestern New Mexico under the 2004 "Arizona Water Settlements Act" (AWSA) that allocates to New Mexico an annual average of 14,000 acre-feet (AF) of additional water from the Gila basin. The general location studied is shown on Figure 1 – Vicinity Map. Locations for off-stream storage in the side canyons leading into the Gila are evaluated. In addition to locations for storage, this report also identifies locations for diversions to supply that storage, as well as locations for diversions to supply and improve the existing irrigation system in the Cliff-Gila farming valley. In this report, diversions for irrigation are referred to as Category 1 diversions and diversions of additional water beyond agricultural water rights are referred to as Category 2 diversions.

This report identifies conceptual alternatives for three existing Category 1 diversions and irrigation ditches. This report also identifies three potential Category 2 diversion locations and up to six canyons which would be dammed with intent to provide a storage capacity of at least 65,000 AF of water without harming the river and to produce a safe yield of at least 10,000 acre-feet per year (AFY). At least 5,000 AF of water storage is to be located near the top of the Cliff-Gila valley. Diversion of the AWSA water is limited by many constraints. Diversion of 99 percent of the AWSA water from the Gila River would occur on only approximately 10 percent of days and from approximately the highest 17 percent of flows. There are several constraints that must be met before New Mexico can divert the AWSA water. Two of those constraints are that no more than 140,000 acre-feet of water from the Gila River could be diverted during any ten-year period, and a maximum of 350 cfs could be diverted at any given point in time.

A. PROJECT OBJECTIVES

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

- Calculate reservoir storage volumes based on dam and water surface elevations determined with this project.
- Identify locations for Category 1 diversions.
- Identify best candidate locations for Category 2 diversions and storage.

- Identify alignments for conveyances between Category 2 diversions and storage (reservoir) sites.
- Develop cost estimates and recommendations for diversion, conveyance, and storage.
- Focus on the synergy between protecting the environment and developing water storage for future needs.
- Prepare PER summarizing analysis and recommendations.
- Develop conceptual designs for Category 1 and Category 2 diversions.
- Present findings to stakeholders.
- Present findings to Interstate Stream Commission (ISC).

B. BACKGROUND

Under the terms of the AWSA, up to $128 million of funding may be available. Of this, $66 million must be used only in a four-county region in southwestern New Mexico. The four counties are Grant, Luna, Hidalgo and Catron. The remaining funds can be used elsewhere in New Mexico.

The Consumptive Use and Forbearance Agreement (CUFA) is an agreement between New Mexico, Arizona, the US Secretary of the Interior, and senior downstream water users. The CUFA was ratified by Congress in the 2004 AWSA. As mentioned before, there are a number of constraints in the CUFA which must be met before New Mexico can divert the AWSA water. Some of those constraints are as follows: The CUFA limits diversions from the Gila River to 350 cfs at any given moment and 140,000 AF in any 10-year period. In addition, it requires New Mexico to bypass certain flows before New Mexico can divert a portion of the remainder of flows above those bypass amounts. The bypass flows range from 75.5 cfs in December to 442.5 cfs during June, July, August, and September. ISC staff models have indicated that a minimum bypass of 150 cfs (twice the median flow) ensures the Gila will maintain a greater than 150 cfs flow after any AWSA diversions. The 150 cfs bypass also provides adequate yield under the AWSA. To prevent ecological detriment, while still maintaining yield, during all modeling the ISC has applied this minimum bypass of 150 cfs on the Gila before any diversion would take place. While the CUFA does not require a 150 cfs minimum bypass, not adhering to a minimum bypass before diverting could result in New Mexico impairing existing downstream New Mexico water rights, something the ISC could not legally do as it would be contrary to New Mexico state statutes.

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 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 Category 1 diversions and the one existing Category 2 diversion that provides water for Bill Evans Lake are shown on Figure 2 – Project Planning Area. The existing Category 1 diversions, from north to south, are Upper Gila, Fort West, and Gila Farms. The only existing Category 2 diversion is owned and operated by Freeport-McMoRan Copper & Gold. Water from that diversion is pumped to Bill Evans Lake, also shown on Figure 2, from where it is pumped to the Tyrone Mine, which is located approximately 13 miles south and east of Bill Evans Lake.

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 Category 1 diversions.

III. REVIEW OF WORK BY OTHERS

BHI reviewed information and documents provided by the ISC and The Bureau of Reclamation (BoR) as part of this project. Below is a summary of what was reviewed.

A. FILES FROM BUREAU OF RECLAMATION

The BoR has previously studied this area and has a current study underway of potential projects. One CAD file was provided with potential reservoir sites and Category 2 diversions delineated. One map showing locations for irrigation ditches, siphons, and Category 1 diversions was also reviewed. The map shows the locations for the Upper Gila, Fort West, and Gila Farms irrigation diversions, ditches, and siphons.

B. GIS FILES FROM ISC

GIS files and spreadsheets related to potential storage locations were also received from ISC. Along with the GIS files, ISC provided the workflow used to calculate the volumes listed in Table III-1, below. The process used by ISC to calculate volumes was reviewed and appears reasonable. All volumes calculated by BoR, ISC and BHI are included in Table III-1. The volumes BHI calculated are different from the volumes calculated by ISC and BoR because of different processes used to calculate the volumes.

C. OTHER DATA FROM ISC

In addition to the GIS files described above, ISC provided a PowerPoint presentation on conceptual ideas for the ISC uses of the Gila Settlement water and associated funds.

Table III-1 – Volume Calculations

Location	BoR		BoR Top of Storage		ISC		BHI InRoads		BHI GIS	
	Water Surface Elevation (ft)	Volume (ac-ft)	Water Surface Elevation (ft)	Volume (ac-ft)	Water Surface Elevation (ft)	Volume (ac-ft)	Water Surface Elevation (ft)	Volume (ac-ft)	Water Surface Elevation (ft)	Volume (ac-ft)
Spar Canyon	4,760 4,720	3,133 862	4,747 4,707	1,553 39	4,754	2,400	4,755	2,512	4,755 4,750 4,745	2,376 2,000 1,725
Maldonado Canyon	4,760 4,720	4,370 909	4,707	674			4,750	3,424	4,755	3,778
Winn Canyon	4,680	5,668	4,627	1,067			4,685 4,680	7,589 6,454	4,685 4,680	7,196 6,026
Bell Canyon	4,680	2,906	4,627	2,625						
Garcia Canyon	4,720 4,680	2,554 2,358	4,667	1,824						
Northrup Canyon #1	4,720 4,680	4,932 1,240	4,667 4,627	2,802 602						
Northrup Canyon #2	4,680	2,928								
Bear Creek	4,760 4,720 4,680	67,731 28,231 9,863	4,627	7,166						
Lobo Creek			4,587	3,384						
Pope Canyon	4,680	5,472	4,587	3,146			4,640 4,600	12,857 4,534	4,640 4,630 4,625	11,926 10,490 10,046
Greenwood Canyon			4,627 4,587	29,890 13,474	4,511	30,000				
Sycamore Canyon			4,547	18,348	4,636	44,000	4,600 4,595	41,399 38,706	4,670 4,660 4,650	108,766 97,935 87,686
Dix Canyon #1	4,760 4,720 4,680	18,354 7,476 2,575	4,547	2,769			4,600 4,595	3,593 3,166	4,600 4,560	4,040 3,857
Dix Canyon #2	4,680	4,338								
Spring Canyon	4,760 4,720 4,680	12,497 5,453 2,128	4,627	2,031						
Davis Canyon			4,547	1,740	4,636	5,600	4,600	4,395	4,600 4,560	3,776 1,664
Mangas Creek	4,720 4,680	24,071 8,949	4,587	16,545						
Schoolhouse Canyon	4,760 4,720 4,680	29,661 12,736 3,738	4,587	3,116	4,636	11,800			4,630 4,625 4,615	12,346 11,487 9,032
Mogollon Canyon					4,773	15,000			4,755 4,750 4,745	11,042 9,820 8,556
Cherokee Canyon	4,760 4,720 4,680	16,626 7,034 2,749	4,547	1,325	4,636	3,800			4,600	3,129
Pancho Canyon			4,587	1,286						
Moonhull Canyon	4,760 4,720 4,680	17,122 8,100 3,599	4,547	2,573						
Patterson Canyon			4,587	711						
Road Canyon			4,507	800						
Ira Canyon			4,587	2,308						

IV. SITE VISIT OBSERVATIONS

A. INITIAL SITE VISIT

An initial site visit was conducted in May 2013 by ISC and BHI staff, and the memorandum summarizing the findings is included as Appendix A.

B. GEOTECHNICAL/GEOLOGICAL INVESTIGATION OF SIDE CANYONS

A site visit was conducted September 4 - 6, 2013 by ISC, Geo-Test, and BHI staff. As part of the trip, Spar, Maldonado, Winn, Pope, Sycamore, Dix, Davis, Cherokee, and Schoolhouse Canyons were investigated. Each canyon was walked and photographed. Photographs are included in Appendix H. The purpose of the site investigation was to evaluate each candidate canyon in the field as to its suitability for dam construction and water storage and identify any potential fatal flaws that would preclude a particular site from further consideration as a candidate site. Of particular interest were the geotechnical and geologic conditions at each canyon. The suitability of the site soils to form a dam embankment was assessed visually at each site including availability of acceptable site material to create the dam’s core to control seepage. In general, the sites were deemed suitable to excavate and create a dam pool and embankment but were found lacking in terms of suitable material to create the dam’s core. General recommendations to address this issue are included in Geo-Test’s Geology and Geotechnical report for the project. A copy of this report is included as Appendix B. Geologically, the greatest concern was the proximity of each site to the nearest fault. Geo-Test’s report notes that Schoolhouse and Cherokee Canyons have several faults that make either location a poor choice for construction of a dam. More detail with respect to the geologic conditions for each site is provided in Geo-Test’s report, attached as Appendix B.

C. GEOTECHNICAL/GEOLOGICAL INVESTIGATION OF DIVERSION LOCATIONS

A site visit to the potential Category 1 and Category 2 diversion locations was conducted October 2 - 4, 2013 by Geo-Test and BHI staff. BHI and Geo-Test staff visited the Category 1 diversion locations with Donnie Stailey, who manages the Fort West and Gila Farms diversions. BHI staff met with Jerry Woodrow, who manages the Upper Gila diversion, on our site visit held in May 2013.

Between September 10 and 18, 2013, which was two to three weeks before the diversion site visit, a historic rainfall event caused the Gila River to rise almost to a depth of

13 feet at the US Geological Survey (USGS) Gila River gaging station near Gila, which corresponds to a river flow rate of almost 30,000 cfs. The Category 1 diversions washed out in the September 2013 flooding.

During the site visit, BHI measured cross sections of the river at two of the three proposed Category 2 diversion sites and at the Upper Gila and Fort West diversion sites, where the river was relatively stable. At the uppermost proposed Category 2 diversion location, downstream from the Turkey Creek confluence, the river flow was too deep and swift to safely cross section the river. Photos and field notes, however, were taken; and the information was used in assessment of this general location for a Category 2 diversion. At the Gila Farms diversion site, it was observed that the river had moved dramatically during the recent flooding. The consensus opinion of the local irrigation managers, BHI and Geo-Test staff was that this location is not a good site for a surface diversion.

The potential Category 1 and Category 2 diversion locations can be seen on Figure 3. Geo-Test's observations on the October site visit are included in the geotechnical report, attached as Appendix B. Geo-Test noted that bedrock is probably shallow at the Category 2 diversion sites, and the depth to bedrock is critical in determining the feasibility of the Category 2 diversions. Geo-Test recommended using geophysical methods to determine the depth to bedrock. Geo-Test noted that the depth to bedrock at the Category 1 diversion sites is probably deeper than at the Category 2 diversion sites, making infiltration a viable option for the Category 1 diversions. Photographs from the October 2013 site visit are included in Appendix I.

V. EVALUATION PROCESS FOR STORAGE LOCATIONS

The evaluation of side canyons along the Gila River for potential water storage was conducted as an iterative process. This section describes the iterations, criteria, and resulting recommendations which were then incorporated into alternatives. An "alternative" is comprised of a system of diversion, storage sites and conveyances from the diversion and between the impoundment areas. The alternatives considered are described in Section VII.

A. INITIAL SCREENING

An initial screening of side canyons along the Gila River valley was conducted in order to narrow down the number of canyons for future consideration and analysis. The first part of the screening process eliminated canyons on the basis of two criteria that identified canyons that are not feasible options and should not be considered for further study. The two criteria were: 1) whether the canyon is inhabited, and 2) whether there were endangered species present based on previous environmental reports. Figure 1 shows the locations of the canyons. The initial canyon list included twenty-two of the side canyons located along the Gila River between the confluence with Turkey Creek on the north and the Bird Area on the south. Table V-1 summarizes the initial screening of the canyons, which eliminated seven canyons from further analysis, leaving fifteen canyons for further evaluation. The storage volumes shown are those that were available at the time of the initial screening, from BoR and ISC, as shown in the corresponding columns in Table III-1.

Table V-1 – Initial Screening of Canyons

Canyon		Criteria 1	Criteria 2	Evaluate?
	Largest Calculated Volume (AF)	Inhabited	Endangered Species/Habitat	
Mogollon	15,000			**Yes**
Maldonado Canyon	4,370			**Yes**
Schoolhouse Canyon	29,661			**Yes**
Spar Canyon	3,133			**Yes**
Spring Canyon	12,497		X	**No**
Winn Canyon	5,668			**Yes**
Bell Canyon	2,906	X		**No**
Cherokee Canyon	16,626			**Yes**
Dix Canyon #1	18,354			**Yes**

Canyon		Criteria 1	Criteria 2	Evaluate?
	Largest Calculated Volume (AF)	**Inhabited**	**Endangered Species/Habitat**	
Garcia Canyon	2,554			**Yes**
Bear Creek	67,731	X		**No**
Greenwood Canyon	29,890			**Yes**
Pope Canyon	5,472			**Yes**
Davis Canyon	5,600			**Yes**
Dix Canyon #2	4,338			**Yes**
Lobo Creek	3,384	X		**No**
Northrup Canyon #1	4,932			**Yes**
Northrup Canyon #2	2,928			**Yes**
Sycamore Canyon	44,000			**Yes**
Mangas Creek	24,071		X	**No**
Duck Creek	Not available	X		**No**
Dam Canyon	5,296	X		**No**

B. PRELIMINARY SCREENING MATRIX

BoR showed two different dam locations for Dix Canyon and for Northrup Canyon on their map. Hereafter, Dix Canyon and Northrup Canyon will each be considered as one canyon for the purposes of the preliminary screening matrix and the multi-criteria decision matrix, thus consolidating the fifteen viable canyons listed in Table V-1 to thirteen. To the remaining thirteen canyons, BHI applied three criteria, which were weighted according to importance, to assess and rank each canyon. The criteria were storage volume, location within the planning area, and whether a federal or state highway runs through the canyon. Canyons located closer to the north end of the planning area received a higher score because a higher storage elevation serves a greater number of users, including the majority of the currently irrigated areas of the valley. If a federal or state highway runs through the canyon, it will affect the cost of the project, as the road would have to be re-located.

The scores and weights, method for assigning scores, along with total scores calculated for each canyon are shown in Table V-2, below. A higher score is more favorable. The storage volumes shown are those from BoR or ISC calculations, except for in the case of Pope Canyon. Based on review of topographic mapping, it was noted that the canyon volume could be substantially increased if the existing highway were relocated. The

Table V-2 – Preliminary Screening Matrix

Canyon	Largest calculated volume (AF)	Criteria 1	Weight 1	Criteria 2	Weight 2	Criteria 3	Weight 3	Total Score (Weighted)	Rank (Weighted)
		Storage Volume	**5**	**Roadway**	**1**	**Location**	**3**		
Mogollon	15,000	5	5	5	1	5	3	**45**	**1**
Pope Canyon	13,955	5	5	3	1	3	3	**37**	**2**
Winn Canyon	5,668	3	5	5	1	5	3	**35**	**3**
Schoolhouse Canyon	29,661	5	5	5	1	1	3	**33**	**4**
Cherokee Canyon	16,626	5	5	5	1	1	3	**33**	**4**
Dix Canyon	18,354	5	5	5	1	1	3	**33**	**4**
Sycamore Canyon	44,000	5	5	3	1	1	3	**31**	**7**
Greenwood Canyon	29,890	5	5	0	1	1	3	**28**	**8**
Spar Canyon	3,133	1	5	5	1	5	3	**25**	**9**
Northrup Canyon	4,932	1	5	3	1	5	3	**23**	**10**
Maldonado Canyon	4,370	1	5	3	1	5	3	**23**	**10**
Davis Canyon	5,600	3	5	3	1	1	3	**21**	**12**
Garcia Canyon	2,554	0	5	5	1	5	3	**20**	**13**
Key to Scoring									
Criteria 1									
Storage Volume (AF)	**Score**								
<3,000	0								
3,000-5,000	1								
5,000 - 10,000	3								
> 10,000	5								
Criteria 2									
Existing Roadway Present?	**Score**								
No Roadway	5								
County/Local	3								
Federal	0								
Criteria 3									
Location									
Northern 3rd of plan area	5								
Middle 3rd of plan area	3								
Southern 3rd of plan area	1								

larger volume calculated under an assumption that the roadway is re-located was used in this evaluation for Pope Canyon.

Mogollon Creek was eliminated from further analysis based on potential challenges in constructing a control structure at this location to meet the terms of the CUFA. Mogollon is considered a major tributary of the Gila River.

Greenwood Canyon was eliminated due to the federal highway running through it. In addition, although Maldonado Canyon ranked lower than Northrup Canyon, it was also decided to keep and consider Maldonado Canyon due to its location at the upper part of the valley. Therefore, the eight remaining canyons for the next stage of evaluation were: Pope, Winn, Schoolhouse, Cherokee, Dix, Sycamore, Spar, and Maldonado.

C. CANYON EVALUATION MATRIX

A multi-criteria decision matrix was used to further focus the identification of preferred locations for water storage in side canyons along the Gila. The criteria were assigned a weighting as a portion of 100 percent. The weightings are based on input from ISC, BHI and Geo-Test. At this stage in the evaluation process, costs were not developed for each site; however, the evaluation matrix incorporates criteria which directly impact cost. Those criteria are: ratio of storage volume to embankment volume, accessibility with respect to conveyance (based on linear footage of conveyance to deliver water to the storage site), existing flood control dam that can remain, ratio of storage volume to borrow volume, and roadway (roadways would need to be relocated). The scoring system is summarized in Table V-3.

Table V-3 – Scoring System for Canyon Evaluation Matrix

Criteria 1		**Criteria 5**		**Criteria 9**	
Geotechnical Suitability	**Score**	**Ratio of storage volume to demand**	**Score**	**Ratio of storage volume to embankment volume**	**Score**
Most suitable	5	<6	1	<4	0
		6.0-10.0	5	>4-6	1
Unsuitable	0	>10	1	>6-8	3
		No demand	0	>8	5
Criteria 2		**Criteria 6**		**Criteria 10**	
Geological Suitability	**Score**	**Evaporation (Ratio of surface area to depth)**	**Score**	**Accessibility with respect to conveyance**	**Score**
Most suitable	5	<1	5	<100,000 ft	5
		1.0-2.0	3	100,000-125,000 ft	3
Unsuitable	0	>2.0-3.0	1	>125,000-150,000 ft	1
		>3.0	0	>150,000 ft	0
Criteria 3		**Criteria 7**		**Criteria 11**	
Storage Volume (AF)	**Score**	**Existing Flood Control Dam That Can Remain**	**Score**	**Roadway?**	**Score**
<3,000	0	Yes	5	No roadway	5
3,000-5,000	1	No	0	county/local	3
5,000 - 10,000	3			Federal	0
> 10,000	5				
Criteria 4		**Criteria 8**		**Criteria 12**	
Ratio of storage volume to borrow volume	**Score**	**Embankment Volume (CY)**	**Score**	**Accessibility of conveyance to end users**	**Score**
No borrow required	5	<1,000,000	5	Near US 180	3
>20	3	1,000,000-1,500,000	3	Near north end of valley	5
5.0-20.0	1	>1,500,000-3,000,000	1	Near south end of valley	1
<5	0	>3,000,000	0		

BHI examined the canyon rankings in coordination with consideration of the total desired storage of at least 65,000 AF, and ground truthed the results during the second field visit. Based on the minimum desired storage and the weighted scores from Table V-4, the top six ranked canyons shown below were used to develop the Alternatives.

1. Maldonado Canyon
2. Winn Canyon
3. Pope Canyon
4. Dix Canyon
5. Spar Canyon
6. Sycamore

These final six canyons are included in the recommended alternatives, described in Section VII.

D. ADDITIONAL CONSIDERATIONS

In addition to the criteria already discussed, there are several factors related to conveyance and storage that need to be considered. These include environmental investigation, river crossings, land use or right-of-way concerns, sediment control, and evaporation.

Table V-4 – Canyon Evaluation Matrix

Canyon	Criteria 1	Weight 1-AVG	Criteria 2	Weight 2-AVG	Criteria 3	Weight 3-AVG	Criteria 4	Weight 4-AVG	Criteria 5	Weight 5-AVG	Criteria 6	Weight 6-AVG	Criteria 7	Weight 7-AVG	Criteria 8	Weight 8-AVG	Criteria 9	Weight 9-AVG	Criteria 10	Weight 10-AVG	Criteria 11	Weight 11-AVG	Criteria 12	Weight 12-AVG	Total Score (Weighted)	Rank (Weighted)
	Geotechnical Suitability	16.14%	Geological Suitability	21.14%	Storage Volume	9.72%	Ratio of Storage volume to borrow volume	4.72%	Ratio of storage volume to demand	9.00%	Evaporation	7.29%	Existing Flood Control Dam That Can Remain	4.43%	Embankment Volume	5.14%	Ratio of Storage Volume to Embankment Volume	5.00%	Accessibility with respect to conveyance	7.14%	Roadway	4.57%	Accessibility of conveyance to end users	5.71%	Maximum of 60 points possible	
Spar Canyon	3.0	16.14%	2.0	21.14%	0	9.72%	1	4.72%	1	9.00%	5	7.29%	0	4.43%	5	5.14%	1	5.00%	5	7.14%	5	4.57%	5	5.71%	**31.0**	**5**
Maldonado Canyon	3.0	16.14%	4.0	21.14%	1	9.72%	5	4.72%	1	9.00%	3	7.29%	5	4.43%	5	5.14%	3	5.00%	5	7.14%	3	4.57%	5	5.71%	**40.6**	**1**
Winn Canyon	1.0	16.14%	4.0	21.14%	3	9.72%	5	4.72%	1	9.00%	1	7.29%	0	4.43%	3	5.14%	5	5.00%	5	7.14%	5	4.57%	5	5.71%	**35.7**	**2**
Pope Canyon	3.0	16.14%	2.5	21.14%	5	9.72%	5	4.72%	1	9.00%	1	7.29%	0	4.43%	1	5.14%	5	5.00%	3	7.14%	3	4.57%	3	5.71%	**32.7**	**3**
Sycamore Canyon	2.0	16.14%	4.0	21.14%	5	9.72%	3	4.72%	1	9.00%	0	7.29%	0	4.43%	0	5.14%	3	5.00%	3	7.14%	3	4.57%	3	5.71%	**30.7**	**6**
Dix Canyon	3.0	16.14%	4.0	21.14%	1	9.72%	1	4.72%	0	9.00%	5	7.29%	0	4.43%	5	5.14%	3	5.00%	1	7.14%	5	4.57%	1	5.71%	**31.2**	**4**
Cherokee Canyon	3.5	16.14%	2.0	21.14%	1	9.72%	0	4.72%	0	9.00%	5	7.29%	0	4.43%	3	5.14%	0	5.00%	0	7.14%	5	4.57%	1	5.71%	**22.7**	**8**
Schoolhouse Canyon	3.0	16.14%	0.0	21.14%	3	9.72%	0	4.72%	5	9.00%	5	7.29%	0	4.43%	0	5.14%	0	5.00%	1	7.14%	5	4.57%	1	5.71%	**23.4**	**7**

a) *Environmental Investigation for this Project*

Several studies are currently underway by the ISC to perform additional environmental investigations related to the proposed AWSA diversion and storage proposals in the Cliff-Gila Valley.

<u>Potential reservoir locations on Mangas Creek and in Spring Canyon were eliminated with the preliminary screening matrix (Table V-1) for this project based on endangered species habitat.</u>

The proposed project will be required to comply with National Environmental Policy Act (NEPA) and with United States Forest Service (USFS) requirements, which require 100 percent pedestrian surveys for cultural and biological resources and development of mitigation measures if such resources are discovered. Additional regulations to which this project is subject include the following:

- The Clean Water Act – A preliminary jurisdictional determination under Clean Water Act Section 404 – Permits to Discharge Dredged or Fill Material is needed and consultation with the US Army Corps of Engineers to determine if permitting for the project is required. If an Individual Permit is required, the ISC will obtain a Section 401 Water Quality Certification from New Mexico Environment Department (NMED).
- National Pollutant Discharge Elimination System (NPDES) Construction General Permit coverage is required to comply with Section 402(p) of the Clean Water Act. Such a permit will be required since soil disturbance activities will exceed one acre.
- Section 106 of the National Historic Preservation Act – A qualified environmental consultant must be retained to conduct a cultural resources survey.
- 40 CFR; Part 85, “Control of Air Pollution from Motor Vehicles and Motor Vehicle Engines” – All vehicles and equipment are required to comply with air quality standards. No hazardous air pollutant releases are anticipated from the construction of the proposed facilities.
- 29 CFR 1926, an Occupational Safety and Health Administration (OSHA) enforced activity, establishes standards for noise levels (85 decibels over an 8-hour, 4-day period). Noise from the construction of the proposed improvements will be temporary and will not exceed this standard.

1. RIVER CROSSINGS

In order to optimize the diversion, conveyance and storage system, some conveyances will be required to cross the river. Since the alternatives described below include pressure piping between the storage sites, it is anticipated that all river crossings will be underground and will be constructed using drilling or jacking techniques. Therefore, the river crossings will have minimal impact during construction and after construction with less significant aesthetic and environmental impacts than aerial crossings of the river.

2. LAND USE/RIGHT-OF-WAY CONCERNS

Right-of-way purchase and easements will be required for each alternative for the reservoir sites and conveyances. The three potential Category 2 diversions are all located in US Forest Service lands and as such will require a lease agreement with the US Forest Service. The alternatives described below include conveyance alignments that mostly follow the existing roads in the Gila River valley to minimize right-of-way/easement requirements by utilizing existing public right-of-way to the extent possible. Alignments along existing roadways also minimize impacts during and after construction to the natural environment while also reducing project costs. Easements for the pipelines to cross private property will be required. Alternative 1 will require 94 easements, Alternative 2 will require 91 easements, and Alternative 3 will require 102 easements. These alternatives are described in Section VII. The reservoir sites will need to be purchased from private property owners or leased from the US Forest Service, as applicable.

3. SEDIMENT CONTROL

Natural storm runoff is often laden with sediment that should be mitigated before either combining with the “clean” water held in the water supply reservoirs or discharged downstream, ultimately to the Gila River. For this purpose, BHI recommends a stormwater detention facility be constructed upstream of each water supply reservoir, designed for the 100-year storm event. These facilities will allow sediment to settle out of the stormwater before releasing the runoff into the water supply reservoirs. As water from a storm event is released from each stormwater detention facility into each water supply reservoir, an equal amount of water will be released from the water supply reservoir into the river downstream. The analysis of the stormwater detention facilities is described below.

4. EVAPORATION

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. Table V-5 below shows the six recommended canyons, with surface area and calculated evaporation per year.

Table V-5 – Annual Evaporation at Recommended Storage Sites

Canyon	Surface Area (acres)	Annual Evaporation (AF/yr)
Spar	66	330
Maldonado	120	600
Winn	275	1,375
Pope	301	1,505
Sycamore	558	2,790
Dix	94	470

VI. ENGINEERING ANALYSIS

A. DIVERSION TYPES

One of the objectives of this project is to focus on the synergy between protecting the environment and developing water storage for future needs. In addition to diverting water, diversion structures can divert a substantial number of fish, resulting in high mortality. This is especially problematic with threatened and endangered species. Additionally, the diversions must consider hydraulic design requirements. Among these are hydraulic operation – regulating flow in various river stage conditions, river geomorphology, sediment control, optimum facility location, and maintenance. Operation and management of the diversion structures will be required. The structures will be designed to allow a minimum of 150 cfs to bypass before diverting any flow and will be designed to divert no more than 350 cfs at any given moment, per the CUFA. Additional controls will be required to allow for a range of minimum bypass flows, up to 442.5 cfs during June, July, August, and September, before flow is diverted in order to comply fully with the CUFA constraints.

Temporary diversion of the river will be required during construction of the permanent diversion structures. The temporary diversion structure would be designed by the contractor to protect their work during construction. Temporary structures would be removed and the natural environment restored after construction of the permanent diversion is complete. It is likely that the contractor would divert the river to one side of the riverbed and build half the structure and then divert the river to the other side in order to complete construction.

There are methods to both divert the water and preserve the environment. Some river-friendly diversion alternatives include shallow collector wells with horizontal wells, infiltration galleries, grouted boulder weirs, low profile concrete weirs and inflatable rubber dams. The following sections briefly describe these various diversion options followed by recommendations for this project.

1. SHALLOW COLLECTOR WELLS WITH HORIZONTAL WELLS

Water could be extracted from the river through shallow collector wells with horizontal wells. This option is shown conceptually on Figure 4. The concept is to use conventional wells, potentially with infiltration trenches and/or horizontal wells, to capture river water which is filtered through the alluvium, comprising a “shallow aquifer” for the raw water supply. This shallow well collection system would not mine deep groundwater.

Increased yield from the proposed wells could occur through the use of infiltration trenches that hydraulically improve the connection between the wells and the river. An infiltration trench will reduce the drawdown in the well(s). One advantage of a shallow collector well system with horizontal wells includes the fact that it is not completely dependent on the river stage; it will operate efficiently even when the river has low flow. Construction of the wells has less impact on the river than constructing diversion weirs. Wildfires have occurred recently in the Gila watershed, upstream of the farming valley. A large amount of ash has been produced from these fires and has washed down the river and deposited at the Category 1 diversion sites. The proposed shallow well collection system would not be impacted by ash, allowing reliable water supply in the event of additional wildfires in the area. If used for the Category 2 diversions, controls would need to be included to limit diversion of flow only to periods when the river flow exceeds 150 cfs.

2. INFILTRATION GALLERIES

Water can be extracted from the river through the placement of infiltration galleries directly beneath the river bed. Basic operation of infiltration galleries begins with water infiltrating through a sand, gravel, or combination filter. After passing through the permeable filter layer, the water then flows into a series of pipes through well screens. Once water enters the pipes, either a pump or gravity flow can be used to route the water through the pipe network into the main diversion structure. The challenge in operating infiltration galleries at high flow rates is in preventing clogging of the infiltration and pipe network with river sediments. To prevent clogging of the infiltration network, regular cleaning of the upper filter layer is recommended. Sufficient velocities must be maintained within the entire system to carry incoming river sediment through the pipe network and prevent pipe network clogging. Another challenge related to infiltration galleries is river bed scour. Scour may require deep pipes, which would require locating the infiltration gallery further upstream to ensure gravity flow. This would add to the cost and increase potential to need easements for pipe connecting the infiltration gallery to the existing irrigation ditches. An example of a typical infiltration gallery can be seen in Figure 5. If used for the Category 2 diversions, controls would need to be included to limit diversion of flow only to periods when the river flow exceeds 150 cfs. To achieve up to 350 cfs, the maximum flow rate to be diverted by the Category 2 diversion would require a very large infiltration gallery and result in considerable impact on the river to construct the gallery. Further study is needed to determine the viability of an infiltration gallery considering the depth of alluvium at the diversion location and the required size (length) of infiltration pipes required for such a large diversion flow rate.

3. ROCK CROSS VANE WEIR

Water can be diverted via a rock cross vane weir with an intake structure. A cross vane weir could provide adequate head to the intake structures during low flows, provide grade control and reduce bank erosion, and provide for fish passage and enhanced fish habitat. An intake structure would be installed and would include a trash rack at the river entrance followed by a heavy duty gate, a sluice gate for sediment removal, and a secondary gate before entry to the ditch. The primary advantage of this type of weir is that natural rock is usually more visually acceptable to the public. The primary disadvantage of this type of weir is that rock structures are less permanent than concrete structures and will require more maintenance over time. Other disadvantages include moderate potential for debris clogging and sediment deposition within the headworks and moderate susceptibility to damage from large bedload material and debris. An example of a typical rock cross vane weir can be seen in Figure 6.

4. GROUTED BOULDER WEIR

Water can be diverted via a grouted boulder weir with an intake structure. The grouted boulder weir, similar to the rock cross vane weir, could provide adequate head to the intake structures during low flows, provide grade control and reduce bank erosion, and provide for fish passage and enhanced fish habitat. An intake structure would be installed and would include a trash rack at the river entrance followed by a heavy duty gate, a sluice gate for sediment removal, and a secondary gate before entry to the ditch. The primary advantage of this type of weir is that natural rock is usually more aesthetically pleasing. The grout will make this weir more permanent than a rock cross vane weir and will prevent water from passing between the boulders (causing “leakage”). While significantly more permanent than a rock cross vane weir, the grouted boulder weir structure should be considered slightly less permanent than a concrete weir. Other disadvantages include moderate potential for debris clogging and sediment deposition within the headworks and some susceptibility to damage from large bedload material and debris. An example of a typical grouted boulder weir can be seen in Figure 7.

5. LOW PROFILE CONCRETE WEIR WITH TILTED WEDGE WIRE SCREEN

A fifth option for diversions is to construct low profile concrete weirs with improved intake structures. A conceptual drawing of this type of diversion is shown in Figure 8. A photograph of this type of structure in operation can be seen below. The low profile weir is less obtrusive than a conventional concrete weir with control gates, like the existing diversion for Bill Evans Lake.

The weir has a center trough which directs the low flows toward the intake structure. A tilted wedge wire screen is recommended over this center trough. A tilted wedge wire screen is a stainless steel wedge wire, fine screen that allows water through to the trough while sediment and debris remain in the river flow. Like grouted boulder weirs, this option would also include a heavy duty gate near the river and a secondary head gate.

Major advantages of this option include superior hydraulic and sediment control and very minimal clogging potential. Typical wedge wire screen installations for river diversions have openings ranging from 0.3 to 0.5 mm. As such, river sediments larger than 0.3 or 0.5 mm, depending upon the final design size for the wedge wire screen, will not be introduced into the conveyance system downstream of the diversion. The wedge wire screen is tilted, as shown in the above photograph, and as the water flows over the screen it falls through the openings in the screen and is accelerated by gravity into the collection trough beneath the screen. Sediment and debris, larger than the wedge wire opening size, have mass and momentum that effectively cause them to continue down the face of the screen and down river. As such, the screen is self-cleaning.

Disadvantages include moderate potential for damage to the upstream weir wall from large bedload material and debris, and this type of structure could lead to concerns related to fish and aquatic passage. Bedload refers to material from the bed of the stream. However, proper design of the weir wall would minimize the damage potential, and for the Category 2 diversions, fish passage concerns would be mitigated by design of a low flow passage apart from the section with the tilted wedge wire screen to allow passage of the required minimum flow while also providing a passage for fish. A fish ladder could also be added to the structure as appropriate. Another disadvantage is that suspended particles smaller than the wedge wire opening would potentially be conveyed with the water through the screen and into the downstream conveyance system. These fine sediments would be easily flushed through the system if there is enough head and velocity, but would need to be removed at the first downstream storage reservoir, which could act as a settling basin. Further study is needed to assess sediment volumes and removal, particularly under lower flow rate conditions.

6. CONCRETE WEIR WITH CONTROL GATES

A sixth option for diversions is to construct concrete weirs with control gates. A conceptual drawing of this type of diversion is shown in Figure 9. The weir has a sluice that directs the low flows toward the intake structure. An automatic control gate could be built into the sluiceway. This gate would open and close depending on the level of the river. Like diversion options 3 and 4, this option would also include a heavy duty gate near the river and a secondary head gate. Advantages of this option include superior hydraulic control and reliability. Disadvantages are impacts on the river as this option is the most obtrusive of the options presented in this report. Additionally, this type of structure could lead to concerns related to fish and aquatic passage. A fish ladder could be provided adjacent to the divide wall on the weir side for the fish to travel either from the upstream side to the downstream side or vice versa.

7. DIVERSION CHANNEL WITH UPSTREAM VANES

A seventh option is to construct stream vanes and diversion channels in lieu of weirs. This concept is shown in Figure 10. The upstream vanes would force the river flow toward the opposite bank, where a diversion channel would direct a portion of the river flow into the intake structure. This concept is used often in areas where a diversion across the river is not acceptable for recreational, aesthetic, or regulatory reasons.

The main advantage of this option is it is the least invasive to the river, making this option likely the easiest to permit and coordinate. While this option involves minimal construction within the river, it results in a diversion intake that is more susceptible to changes in the river geomorphology, specifically degradation and aggradation that could leave the intake higher than the river flow and thus ineffective at diverting water from the river. Of the options presented herein, it is the most likely to be bypassed by the flow and is among the least reliable options presented. Other disadvantages include low to moderate potential for debris clogging, sediment deposition within the headworks, the large footprint required, and associated environmental impacts during and after construction.

8. INFLATABLE RUBBER DAMS

Another option is the use of heavy duty inflatable rubber bladders typically anchored to a concrete foundation structure crossing the river bed to control the water surface elevation in the river and divert flow to an intake structure. This concept is shown in Figure 11. The primary advantage of this option is the superior hydraulic control borne by the flexibility of adjustable height and width of the diversion dam. Through the use of a multiple bladder configuration, the dam can be raised or lowered at various locations across the river to manage flow diversion, sediment transport and fish passage. These types of dams also have proven longevity. Disadvantages include more complicated operation requirements including electricity for operation and either telemetry controls or manual controls requiring real time adjustments to provide the desired diversion flow rates. An inflatable dam would also require the construction of a power line to provide the electricity necessary for operation. A local example of an inflatable rubber dam is the Albuquerque Bernalillo County Water Utility Authority's raw water diversion dam located along the Rio Grande just downstream of Alameda Boulevard in Albuquerque, New Mexico. Operation of this particular inflatable dam requires approximately 213,000 kWh annually.

9. CONSOLIDATED CATEGORY 1 DIVERSIONS

One option for the Category 1 diversions is to use one or two diversion sites to supply water to all three irrigation ditches. The primary advantage of this option is that there will be fewer diversion structures to maintain. Some challenges related to consolidated diversions include crossing private property and costs to upsize existing conveyances.

B. DIVERSION SELECTION

Table VI-1 lists the advantages, disadvantages and comments for each diversion type.

Table VI-1 – Diversion Types, Advantages and Disadvantages

Diversion Type	Advantages	Disadvantages	Comments
Shallow Collector Wells with Horizontal Wells	Not as visible, Environmentally more friendly after construction, Operates efficiently even when the river has low flow, Construction of the wells has less impact on the river than constructing diversion weirs, 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 enhanced fish habitat, Visually pleasing.	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

Diversion Type	Advantages	Disadvantages	Comments
Inflatable Rubber Dams	Hydraulic control, Proven longevity	Complicated operations requirements	Not considered for Category 1 diversions due to complex operations requirements
Consolidated Category 1 Diversions	Fewer diversion structures to maintain, Less impact on the river	Costs to upsize conveyances, Right-of-way or easements required to connect to existing irrigation canals	Not applicable for Category 2 diversions

For the Category 2 sites, BHI recommends concrete weirs with tilted wedge wire screens. This is because these structures are the most permanent, offer superior hydraulic and sediment and debris control (with very minimal potential for clogging), and require minimal maintenance. The remote locations of the diversions make accessibility for maintenance a concern, thus diversion options requiring minimal maintenance are highly preferable. The diversion must also not divert water if the river is below 150 cfs, which is approximately twice the median flow. As shown in Figure 8, a concrete weir can be designed to fit the geometry of the river at the diversion location and provide both a permanent low flow section to pass 150 cfs and then divert water during higher flows producing a maximum diversion flow rate of 350 cfs based on the geometry of the diversion weir, tilted wedge wire screen, trough and outlet works. Additional controls by means of the outlet works gate, metering, and possibly a Supervisory Control and Data Acquisition (SCADA) system will be required to control how much flow is diverted and allow for a range of flows, above 150 cfs, to bypass in accordance with the CUFA constraints. Operations staff and an instrumentation and control system will be required. The main advantage of a concrete structure is that a concrete structure is more permanent than a rock structure. Less maintenance is anticipated with this more permanent structure. In addition, the tilted wedge wire screen allows for debris and sediment to pass over the weir while only capturing screened flow for the intake structure. The tilted wedge wire screen is very low maintenance, and if properly designed, it is self-cleaning. The diversion structure needs to be a low profile unobtrusive structure given the wild and scenic nature of the river. The Category 2 conceptual site plans are included in this report as Figures 12, 14, 16 and 17. Table VI-2 summarizes the selection process for Category 2 diversions.

Table VI-2 – 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

The Gila Basin Irrigation Commission (GBIC) Tier 2 application describes more permanent Category 1 diversion structures that would be built on the same land where earthen diversions exist today and are subject to washing out in normal river fluctuations. For the Category 1 sites, BHI recommends a grouted boulder weir for the Upper Gila diversion and the Fort West diversion. This is because the grouted boulder weir is aesthetically pleasing, while more permanent than the current earthen diversion. The grouted boulder weirs will allow fish passage and potentially enhance fish habitat. The Category 1 conceptual site plans are included in this report as Figures 19 and 20. Based on the site visit, the current diversion site for the Gila Farms ditch is not a preferred site for an above-ground diversion. The river is unstable in this area and shifted laterally and vertically during recent heavy storms such that the existing Gila Farms ditch diversion was cutoff and no longer intercepting flow from the river. There are two options for supplying water to the Gila Farms ditch:

1. Use the Fort West diversion, connecting the Gila Farms ditch to the Fort West ditch by means of a new pipe or ditch, as shown on Figure 21.
2. Install an infiltration gallery just upstream of the existing Gila Farms diversion and connect it to the existing diversion ditch, as shown on Figure 22.

BHI recommends option 1. This would reduce the number of diversions that would require regular maintenance. Table VI-3 summarizes the selection process for Category 1 diversions.

Table VI-3 – Category 1 Diversion Selection

Location	Shallow Collector Wells with Horizontal Wells		Infiltration Galleries		Rock Cross Vane Weir		Grouted Boulder Weir		Diversion Channel With Upstream Vanes	
Upper Gila	O	Operates efficiently even when river has low flow, not impacted by ash	O	High maintenance at high flow rates	X	Aesthetically pleasing, but leaks water and requires repeated maintenance	O +	Aesthetically pleasing, more permanent than current diversion method	X	Reliability
Fort West	O	Operates efficiently even when river has low flow, not impacted by ash	O	High maintenance at high flow rates	X	Aesthetically pleasing, but leaks water and requires repeated maintenance	O +	Aesthetically pleasing, requires less maintenance and re-construction than current diversion method	X	Reliability
Gila Farms	O	Not a good location for an above-ground diversion	O	Not a good location for an above-ground diversion	X	Not a good location for an above-ground diversion	X	Not a good location for an above-ground diversion	X	Not a good location for an above-ground diversion

O – suitable
X – not suitable
+ – recommended

C. CATEGORY 2 DIVERSIONS

As discussed previously, various diversion options were assessed for the Category 2 diversions, with low profile concrete weirs equipped with tilted wedge wire screens being recommended. For each of the three locations, this diversion type was developed to a conceptual level as discussed below.

1. DIVERSION 1 AT ELEVATION 4770

The highest feasible diversion location is just downstream of the confluence of Turkey Creek and the Gila River, near the end of Turkey Creek Road. This is at an elevation of 4770 feet. Turkey Creek Road would need to be improved and extended to the diversion location. Several diversion options were considered for this location. Grouted boulder or diversion channel with upstream vanes were ruled out at this location, because a more permanent structure is necessary due to limited accessibility and the need for a dependable structure that will function in higher flow rate events. Shallow collector wells with horizontal wells or an infiltration gallery are not options at this location, because the river is flowing against bedrock. Consequently, a low profile concrete weir with a tilted wedge wire screen is recommended at this location.

The proposed conceptual design for Diversion 1 is shown in Figure 12. The final dimensions and location of the diversion would be established as part of the design phase for this project and will require a detailed design survey of the diversion site. The diversion structure concept entails establishing a low flow weir and section that mimics the existing low flow, main thalweg of the river. Along with the low flow weir section, a trough and downstream pipe or channel will limit diversion of higher flows up to a total diversion of 350 cfs. The elevation of the diversion trough and tilted wedge wire screen is set such that once the flow in the river exceeds 150 cfs, water will begin to flow through the screen into the trough. The dimensions of the screen and trough are sized to convey at least 350 cfs while the outlet works gate will be used to limit the maximum flow rate to 350 cfs. Additional controls by means of the outlet works gate, metering, and a possible SCADA system will control how much flow is diverted and allow for a range of flows, above 150 cfs, to bypass in accordance with the CUFA constraints. Immediately downstream of the screened section are head gates that provide additional means to control the flow and also to shut off the diversion for maintenance or other purposes when diversion of flow is not desired.

Two locations for Diversion 1 were considered. The first, as shown in Figure 12, is the initial location based on the mapping and location of the 4770 contour at the river. As a result of our site investigation of the diversion sites, conducted from October 4 - 6 2013, an alternate location for Diversion 1 was established approximately 1,750 feet further downstream (see Figure 13). The main advantages of this alternate location are significantly reduced costs and increased constructability for the conveyance from the diversion to the first storage reservoir. The east side of the valley at the initial diversion location is formed by exposed bedrock making construction of a conveyance channel problematic and costly. If the channel were located instead on the west side of the river, an aerial river crossing further downstream would be required to supply water to Spar and Maldonado canyons on the east side of the river which is not viable given the scenic nature of the river and disruption to the aesthetics of the natural environment that would result from such a crossing. The alternate location for Diversion 1 locates the diversion just before the river valley widens and at a location where the river appears to be laterally and vertically stable. A photo of this location has been included in GeoCam Report QBP00362 in Appendix I. The conveyance channel could be constructed along the east side of the river while avoiding the bedrock slope on that side of the valley. While the elevation of the diversion would be slightly lower at this location, the length of the conveyance would be shorter, the constructability of the conveyance increased, and the overall cost of the diversion and conveyance decreased. Additionally, with this alternate location, Turkey Creek Road would not have to be extended, only improved. Consequently, if Alternative 1 (which includes Diversion 1) is selected, the alternate location for Diversion 1 as shown in Figure 13 is recommended.

2. DIVERSION 2 AT ELEVATION 4695

The general location for Diversion 2 was chosen based on the sinuosity of the river and potential to dramatically reduce the conveyance length by means of a tunnel. However, accessibility in this reach of the river for construction equipment and for maintenance is very difficult. Construction of a road from Turkey Creek Road to the diversion site, if possible, would be required. In addition, improvements to Turkey Creek Road would be required. As with Diversion 1, various diversion options were considered for this location and a concrete weir with a tilted wedge wire screen is recommended for Diversion 2 for the reasons noted previously.

The proposed conceptual design for Diversion 2 is shown in Figure 14. The final dimensions and location of the diversion would be established as part of the design phase for this project and will require a detailed design survey of the diversion site. The concept for Diversion 2 is the same as Diversion 1.

Two locations for Diversion 2 were also considered. The first as shown in Figure 14 is the initial location based on the mapping and location of the 4695 contour at the river and the location resulting in the shortest tunnel conveyance (see Figure 26). Based on our site investigation of the diversion sites, conducted from October 4 - 6 2013, an alternate location for Diversion 2 was established approximately 1,300 feet further upstream between two rock outcrops (on either side of the river) where the river turns to the south (see Figure 15). The main advantage of this alternate location is the opportunity to anchor the diversion structure into solid rock on each side of the river, creating a configuration that would be most stable and eliminate potential for the river to meander and flank the structure. Another benefit is the diversion would be slightly higher than the initial Diversion 2 location. The conveyance would be slightly longer and more expensive. However, the benefits in terms of the permanency of the diversion outweigh this minimal cost increase. Consequently, if Alternative 2 (which includes Diversion 2) is selected, the alternate location for Diversion 2 as shown in Figure 15 is recommended. The proposed conceptual design for Diversion 2 at the alternative location is shown in Figure 16.

3. DIVERSION 3 AT ELEVATION 4640

Diversion site 3 has, by far, the best accessibility of any of the three potential Category 2 diversion locations. This diversion location would also result in the least expensive conveyance to the proposed storage locations. Shallow collector wells with horizontal wells or an infiltration gallery could be used at this location. However, these types of structures require greater maintenance and have the potential for plugging and becoming non-functional. Given the critical need for the Category 2 diversions to function under all conditions, these diversion options are not recommended at this location either. A low profile concrete weir with a tilted wedge wire screen is recommended at this location. The tilted wedge wire screen is welded in place and in general is not susceptible to vandalism or theft due to its construction.

The proposed conceptual design for Diversion 3 is shown in Figure 17, and Figure 18 shows the initial location based on the mapping and location of the 4640 contour at the river. The final dimensions and location of the diversion would be established as part of the design phase for this project and will require a detailed design survey of the diversion site. The concept for Diversion 3 is the same as Diversions 1 and 2.

D. CATEGORY 1 DIVERSIONS

The GBIC Tier 2 application describes more permanent Category 1 diversion structures that would be built on the same land where earthen diversions that are subject to washing out in normal river fluctuations have been historically constructed. On the dates of our diversion site investigation, the Upper Gila and Fort West Diversions were being reconstructed, having been recently destroyed by large flow events in the river. Also, the Gila Farms Diversion had washed away and it appeared there were no plans to immediately reconstruct it. Several diversion options were assessed for the Category 1 diversions, and one was selected for development of conceptual level designs for the Upper Gila and Fort West Diversions with alternate options being explored for replacement of the function of the Gila Farms Diversion. One of the options assessed is to consolidate the three existing Category 1 diversions into one or two diversion sites to supply water to all three irrigation ditches. The primary advantage of this option is that there will be fewer diversion structures to maintain. Some challenges related to consolidated diversions include crossing private property and costs to upsize conveyances.

1. Upper Gila Diversion

The current diversion site for the Upper Gila Ditch is a good site for an above-ground diversion. Several diversion options were considered for this location. As discussed previously, a grouted boulder weir is recommended for the diversion structure at this site. The proposed conceptual design for the Upper Gila Diversion is shown in Figure 19. The final dimensions and location of the diversion would be established as part of the design phase for this project and will require a detailed design survey of the diversion site. The diversion structure concept entails establishing a more permanent grouted boulder weir that directs water to one side of the river, during low flow conditions allowing diversion of flow through a series of gates and an outlet ditch. The elevation of the grouted boulder weir would be set to allow diversion for irrigation during low flow conditions. The capacity of the diversion would be determined as part of the design phase, however, based on historic operation of the diversions would be in the range of 20 to 30 cfs. The structure would include a trash rack to prevent trash and debris from clogging the diversion. Flow control head gates and sluice gates would also be included to control the flow and sluice sediment back into the river.

2. Fort West Diversion

The current diversion site for the Fort West Ditch is a good site for an above-ground diversion. Several diversion options were considered for this location. As discussed previously, a grouted boulder weir is recommended for the diversion structure at this site as well. The proposed conceptual design for the Fort West Diversion is shown in Figure 20. The concept for the grouted boulder weir for the Fort West Diversion is the same as the Upper Gila Diversion described above.

Alternatively, the Fort West Diversion could be constructed as shown in Figure 23 to provide water to both the Fort West and Upper Gila Ditches. This option would consolidate the Upper Gila and Fort West Diversion into a single structure reducing construction and maintenance costs.

3. Gila Farms Diversion

The current diversion site for the Gila Farms Ditch is not a good site for an above-ground diversion because the geomorphology is not stable in this area and the river can migrate substantially during large storm events. This was evident at the October site visit, where it was noted the river had shifted from the east to west leaving the existing Gila Farms

Diversion disconnected from the river. Further impacts from flows and sediment from Garcia Canyon which discharges to the Gila in the middle of the Gila Farm diversion channel have the potential to adversely impact the functionality of the diversion system. Shallow collector wells w-with horizontal wells and an infiltration gallery are options for this site; however, as noted previously, concerns about maintenance of such diversion structures leads to the consideration of other options for the Gila Farms Diversion. In lieu of a diversion at the upstream end of the Gila Farms Ditch, a connection to the Fort West Ditch is proposed. As shown in Figure 24, water could be supplied to the Gila Farms Ditch by means of a connecting ditch, approximately 3,000 feet in length running from the Fort West Ditch west under Garcia Canyon to the existing Gila Farms Ditch. Easements from impacted property owners would be required with this option.

E. DIVERSION FLOW RATE AND CONVEYANCE COST-BENEFIT ANALYSIS

In order to determine the design flow rate for the conveyances, a cost-benefit analysis was conducted, as described in the following discussion. The maximum flow rate of 350 cfs is prescribed by the CUFA.

BHI evaluated both open channels and pipes, and conducted a cost-benefit analysis for conveyances. This analysis consisted of the calculation of a hydraulic profile for a system with a diversion at the highest identified location, recommended storage locations identified in the evaluation matrix, and conveyances between those storages. The analysis was completed for the following diversion flow rates: 150, 200, 250, 300, and 350 cfs. Based on the pipe sizes required from the hydraulic analysis, a cost was calculated for each flow rate. For all flow rates, a concrete lined open channel was assumed from the diversion to the first storage location. The flow rate was graphed versus the annual volume achieved (see Appendix D). The annual volumes achieved for a given flow rate were calculated in a spreadsheet based on the CUFA provided by ISC. This was compared to the cost versus diversion flow rate. The cost per acre-foot decreases with an increase in design flow rate, with the most significant drop in cost per acre-foot between 300 and 350 cfs. BHI recommends designing the diversion and conveyances to divert up to the CUFA maximum of 350 cfs. For this reason, 350 cfs was selected as the design flow rate to be used in developing the alternatives. The final design for divisions should be based on the structure withstanding high flows, up to 30,000 cfs.

The design flow for the conveyance from the diversion to the first storage is 350 cfs for each alternative. 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 all alternatives was sized using the Hazen and Williams empirical formula, as experience has shown it produces reliable results. Profiles for use in the cost-benefit analysis and 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.

F. CONVEYANCE ANALYSIS AND SIZING

For each alternative, gravity conveyances between the diversion and storage sites were evaluated. As noted previously, three alternatives were developed as part of this report. The hydraulic profile used to size the pressure piping was drawn along the conveyance alignments using BHI's statewide mapping. Because the scope of this study is high-level and conceptual in nature, the profile does not account for depth of bury or depth of water in the river. Since each pipe is open to the atmosphere at the diversion intake and at the inlet from each reservoir, there will be pressures under 20 psi in the pipelines; 20 psi is an industry standard for minimum pressures in water transmission mains. Based on the hydraulic analysis, low pressures occur only at the high end of the conveyances, which is not of concern because of the gravity flow and cyclical nature of the operation of such a system (i.e. flow is not continuous and only occurs when conditions allow diversion).

The hydraulic profiles for this analysis are based on 10 ft -12 ft contours, and an updated more detailed hydraulic analysis, using design quality mapping, will need to be completed as part of design of this project.

As noted in Section VI.A.5, if a low profile concrete weir with a tilted wedge wire screen is used for diverting the flow as recommended by this report, suspended sediment smaller than the opening size of the wedge wire will potentially be introduced into the conveyance system from the diversion to the first reservoir. This fine sediment, less than 0.3 to 0.5 mm in size, has the potential to be deposited in the conveyance particularly in under lower flow rate conditions that result in lower flow velocities in the conveyance. Any deposited sediment should easily be re-suspended by larger flows at higher velocities thus producing a self-cleaning conveyance. However, further assessment of the sediment transport conditions in the conveyance system should be completed. Refinement of the conveyance system including potential reductions in the size of the conveyance pipes may be in order to increase flow velocities to more fully maintain suspension of the sediment and as necessary provide more ability to re-suspend particles that settled out in low flow conditions.

G. TUNNEL

Subsequent to the October 2013 site visit, a tunneling contractor was consulted, due to the accessibility issues related to the diversions at elevations 4,770 and 4,695. A quote for tunneling to these two diversion points was requested from the tunneling contractor. The contractor recommended a hard-rock tunnel boring machine (TBM), due to the remoteness, the existence of brittle volcanic rock, and faults. The pipeline through the tunnel would be constructed either of a concrete pipe with grout between the pipe and the tunnel wall, or a cast-in-place concrete liner. One advantage of a tunnel over an open channel is the environmental and visual impact. A channel would have permanent impacts on the scenic nature of the Gila River valley, whereas a tunnel is out of sight, except at the entrance and exit. Another advantage of a tunnel over an open channel is that no water will be lost to evaporation through a tunnel. As discussed in Section VIII, the total cost of a tunnel is less than the total cost of a channel, given the length of the conveyance if by tunnel is considerably shorter. Due to difficult and limited accessibility, one disadvantage of a tunnel is that maintenance would be more difficult than maintenance of a channel. Another disadvantage is that site conditions during construction can be more unpredictable for a tunnel than for a channel. For example, the hardness of the rock encountered along the tunnel alignment can be different than expected, resulting in higher than budgeted costs. In addition, locations of faults and water can be difficult to predict prior to construction, also potentially increasing the cost.

H. STORAGE RESERVOIRS VOLUME AND EARTHWORK CALCULATIONS AND DESIGN CONSIDERATIONS

Storage volumes for each of the side canyons were calculated based on site specific grading analysis, related to each of the diversion location elevations described above.

The appraisal-level grading for each site was performed in Bentley InRoads V8i design software and included a 25 foot top width dam embankment with 2:1 side slopes on the upstream face and 4:1 side slopes on the downstream face of the dam, except at Dix, which has 2:1 side slopes both upstream and downstream. The preliminary geology and geotechnical report provided by Geo-Test states that the dam slopes will be analyzed for slope stability but are initially assumed to be 3:1 upstream and 2:1 downstream maximum slope. Slope stability does not take into account such factors as revegetation and seepage. In order to protect the dams from seepage, the dams are assumed to be plated with concrete on the upstream side, so a steeper slope is acceptable. The dam at Dix is assumed to be plated with concrete on both the upstream side and the downstream side. Seepage losses will need to be evaluated during future geotechnical investigations including soil sampling as reservoir sites to determine soil permeability and seepage rates. As part of these future studies, seepage control measures will be assessed and recommended. The appraisal-level grading for each canyon is included in Appendix C, along with the earthwork and water storage calculations related to the grading. Due to their size, all of these dams will be jurisdictional facilities in accordance with the New Mexico Office of the State Engineer (OSE) "Rules and Regulations Governing Dam Design, Construction and Dam Safety" NMAC 19.25.12 (Rules), and as such, will need to meet all the design standards under the Rules. From a cost perspective, most notable of these design standards and requirements is the requirement for an emergency spillway that can safely pass the probable maximum flood (PMF). This will be discussed further in Section VII.E.7. Table VI-4 lists the volumes that were calculated for the recommended canyons.

Table VI-4 – 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

I. SIDE CANYONS HYDROLOGIC ANALYSIS

Given the proposed location for the storage reservoirs within various side canyons along the Gila River, consideration of stormwater flows in these canyons must be a part of the overall analysis of these canyons as viable sites for river water storage. As noted previously, natural storm runoff is often laden with sediment that should be mitigated before combining with the "clean" water held in the proposed water supply reservoirs. As such, BHI recommends a stormwater detention facility be constructed upstream of each water supply reservoir, designed for the 100-year storm event. The recommended locations for stormwater detention facilities to detain natural stormwater runoff and trap sediment for each reservoir and for each alternative are illustrated on Figures 25 through 27. The analysis to size these ponds is described below.

1. METHODOLOGY

In order to size a detention facility, a runoff hydrograph is required to establish the runoff volume and associated detention pool size. BHI used the SCS Unit Hydrograph Method to analyze the peak runoff rates for each of the preferred side canyons. This analysis was performed using the US Army Corps of Engineers' Hydrologic Engineering Center Hydrologic Modeling System, Version 3.5 (HEC-HMS). The SCS Unit Hydrograph Method requires input of the following: drainage basin areas, runoff curve numbers, time of concentration and lag time, and a 24-hour rainfall hyetograph. The drainage basin areas and runoff curve numbers that were used are shown in Table VI-5.

2. DRAINAGE BASINS

The drainage basins were delineated using ArcGIS software, which processed a DEM (digital elevation model), produced from BHI's 2005 statewide mapping. The pixel size of the DEM used for this analysis was approximately 86 feet. This evaluation established a total of seven drainage basins that contribute runoff impacting the proposed water storage dam sites. The drainage basins ranged in size from 2 square miles for Pope Canyon to 63 square miles for Sycamore Canyon. Arroyo flow lines were also established and are shown on the drainage basin map in Appendix F.

3. RUNOFF CURVE NUMBERS

BHI used the National Land Cover dataset from USGS as well as the Soil Survey Geographic Database (SSURGO) and the Natural Resources Conservation Service (NRCS) State Soil Geographic Data (STATSGO) soil data to select curve numbers from "Urban Hydrology for Small Watersheds" (TR-55) for a large area encompassing the project site and contributing watersheds for the preferred side canyons. HEC-GeoHMS and Zonal Statistics were used to calculate an average curve number for each drainage basin. The curve numbers selected for this project, as well as the soil and land cover maps are included in Appendix F.

4. TIME OF CONCENTRATION AND LAG TIME

In accordance with the SCS Unit Hydrograph Methodology, the time of concentration for each basin was calculated using the Upland Method for the portions of each basin that lack a defined stream channel, as indicated by a blue stream line on the USGS quad map, and the Kirpich Formula for the reach of the basin that has a defined channel section. The Upland Method is used for un-gullied watersheds with overland flow and shallow concentrated flow regimes. All of the basins exhibit flow paths with a short length of overland flow and a length of shallow concentrated flow upstream of the defined stream channel. The Upland Method requires an estimate of the velocity of sheet flow and of shallow concentrated flow, which was performed using Figure 3-10 of the New Mexico Department of Transportation (NMDOT) Drainage Manual. The velocity plotted on Figure 3-10 is a function of the basin slope and land use/flow regime. The time of concentration calculations are provided in Appendix F. The lag times were calculated using the calculated time of concentration value in equation 3-99 from the NMDOT Drainage Manual, which equates the lag time to 0.6 times the time of concentration. The HEC-HMS model requires the lag time for each basin, rather than time of concentration, in order to estimate the peak flow runoff for each of the side canyons. These lag times are listed in Table VI-5.

5. 24-HOUR RAINFALL DEPTHS

The National Oceanic and Atmospheric Administration (NOAA) Precipitation Frequency Data Server Atlas 14 (NOAA-14, Volume 1, Version 5 – the most current available rainfall data) 100-year, 24-hour rainfall depth in the project area is 3.67 inches. The SCS Unit Hydrograph Method requires a hyetograph for input into the HEC-HMS model. A hyetograph was created from the peak rainfall depths for the 100-year storm using the modified NOAA-SCS rainfall distribution, which is a combination of the peak rainfall intensity defined by NOAA, with an SCS Type II-a storm rearrangement. NOAA 6-hour and 24-hour point precipitation values are used to compute rainfall intensities throughout the hypothetical storm. The hyetograph was then incorporated into the HEC-HMS model to generate the rainfall runoff. The modified NOAA-SCS rainfall distribution and NOAA precipitation data are provided in Appendix F.

6. STORMWATER DETENTION FACILITY SIZING

One of the evaluation criteria used in the storage selection process was whether there was an existing flood control dam that could remain to potentially address the need for stormwater and sediment control in the side canyons upstream of the proposed water storage reservoirs. Maldonado Canyon is the only side canyon that has such a facility that would not be engulfed by the proposed storage reservoir. The hydrograph for Maldonado was developed based on the procedures described above and routed through the as-built elevation-storage-discharge curve for the existing NRCS flood control facility in Maldonado Canyon. The NRCS flood control facility was not designed for the 100-year event. As such, the existing flood control facility cannot handle the 100-year event and will need to be replaced. The proposed stormwater detention facilities for each of the side canyons were sized in order to detain the 100-year, 24-hour storm as long as possible in order to allow the sediment to settle out, while allowing the storm volume to be released within 96 hours in accordance with the OSE Rules. The stormwater detention facility volumes and outlet configurations are listed in Table VI-5. The stormwater detention facility volume calculations are provided in Appendix F.

The drainage basin areas, runoff curve numbers, lag time, and 24-hour rainfall hyetograph described above were input into the HEC-HMS model in order to optimize the detention facility volume and outlet configuration. The peak runoff flow rates and total runoff volume calculated in the HEC-HMS model, along with the detention volume and outlet configuration are listed in Table VI-5. The HEC-HMS model output is included in Appendix F.

Table VI-5 – Hydrologic Analysis Results

Drainage Basin	Basin Area (sq mi)	Curve Number	Lag Time (min)	100-yr Peak Runoff Flow Rate (cfs)	100-yr Total Runoff Volume (AF)	Detention Volume (AF)	Outlet Configuration
Spar	4.1	80	35.7	2,900	390	220	1-48” RCP
Maldonado	2.6	80	35.2	1,860	250	150	1-30” RCP
Winn	13.8	77	108.1	3,530	1,150	470	2-72” RCP
Pope	2.0	76	35.5	1,170	160	98	1-24” RCP
Sycamore West	56.5	85	153.7	15,930	6,530	2,420	4-7’x7’ CBC
Sycamore South	3.0	86	36.8	2,820	370	210	1-54” RCP
Dix	7.1	85	62.7	4,240	820	300	4-60” RCP

Note: Results have been rounded from the results in Appendix F.

7. EMERGENCY SPILLWAYS

Due to the fact that the stormwater detention facilities will release water into the proposed reservoir sites, emergency spillways for the storage reservoirs will be required to protect the dam embankment from stormwater runoff within the side canyon. Per OSE Dam Rules and Regulations, dams are classified based on their hazard potential which in turn determines the criteria for sizing their emergency spillways. Dams with development downstream are generally classified as "high hazard" dams. For the purposes of this study, all dams will be considered high hazard dams. Failure or mis-operation would probably cause loss of human life given there is development downstream of each dam site. As a high hazard dam, the facility must be able to safely pass the runoff from the Probable Maximum Flood (PMF) resulting from the Probable Maximum Precipitation (PMP) occurring in the watershed.

The method described in "Hydrometeorological Report No. 49, Probable Maximum Precipitation Estimates, Colorado River and Great Basin Drainages" (NWS, 1984) was used to develop local storms for an average basin area of four square miles for all basins except Sycamore. Since the Sycamore drainage basin is much larger than the rest of the basins, a separate local storm was developed for Sycamore, as well as a general storm. The local storm is developed as a 6-hour duration storm which uses the most intense storm values for maximum precipitation for a one square mile basin. The general storm is developed as a 72-hour duration storm and combines both convergence and non-orographic PMP for rainfall covering larger areas. Local storms are isolated events in the Southwestern United States that threaten small areas, whereas less intense general storms occur over larger areas and are comparable to PMP storms in the Western and Northern United States. The local storm produced a higher runoff estimate than the general storm for Sycamore. As such, per OSE criteria, the local storm was used to size the spillway for Sycamore. The month of August was used for the general storm calculations, because the maximum rainfall, per NOAA, occurs during that month. Table VI-6 lists the emergency spillway sizes for each of the dams, which were sized to pass the runoff from the PMF.

Table VI-6 – Emergency Spillway Sizes

Location	Design Q (cfs)	Bottom Width (ft)	Depth (ft)
Spar	21,420	740	5
Maldonado	13,760	702	4
Winn	29,780	753	6
Pope	10,040	502	4
Sycamore	65,280	892	9
Dix	26,110	516	7

For all dams, the emergency spillway is sized for the PMF and is assumed to be concrete, 8-inches thick, with 6:1 side slopes and extending to downstream toe of dam. These assumptions form the basis for the cost assigned in the cost estimates. Costs are discussed further in Section VIII.

J. SOUTHWEST REGIONAL WATER SUPPLY

The Southwest Regional Water Supply (SWRWS) proposal, proposed by the City of Deming, relates to this project in that this system would deliver water from the Gila River to communities located east and south of the Planning Area. That proposal describes the project as comprising a system of side channel diversion structures, subsurface diversion and side channel reservoirs. This proposal includes a 5,000 AF capacity reservoir that is currently proposed to be located at Mogollon Creek. Farther downstream, on the Gila River, a subsurface diversion structure would be constructed and water conveyed to a reservoir on Mangas Creek with approximately 30,000 AF of storage capacity.

Water from the Mangas reservoir would be pumped over the continental divide to provide water to the Silver City area, mining district communities, and to the City of Deming. Potential uses of this water are municipal demand, mining and agricultural interests, and groundwater recharge in Grant and Luna Counties.

The SWRWS pipeline is proposed to be constructed along the right-of-way for Highway 180 from the Gila River to Deming. The proposed pipeline would cross both public and private lands. The proposal recommended pipe sizes of 24-inches from Mangas to Hurley and 14-inches from Hurley to Peru Hill Mill site, just north and west of Deming.

The SWRWS proposal included a map (Figure 4 of the proposal) showing land ownership, categorized as public, private, National Park Service, Department of Defense, Forest Service, or Bureau of Land Management. The number of properties that will be crossed by the pipeline was estimated based on the Grant County assessor parcel and a 20 ft offset from the road right-of-way. The Luna County parcel database was not available. Two types of properties were distinguished from one another because right-of-way easement preparation will be more costly for the rural properties. The estimated total number of properties affected for the cost estimate was 300, with 150 of those being urban. If possible, the pipeline should be located within the right-of-way for Highway 180. However, in the event that is not possible, the preliminary cost estimate for this project assumed the preparation of easement documents for all properties and the acquisition of easements from 25 property owners at an assumed unit price.

This proposal included a proposed pipeline, with total length of approximately 70 miles, comprising 185,000 linear feet of 24-inch pipe and 185,000 linear feet of 14-inch pipe. BHI created a hydraulic profile of this pipeline for an alignment along Highway 180, starting at the intersection of Highway 218 and Highway 180 (near Pope Canyon) and ending in Deming at the intersection of I-10 and Hwy 180. The total length of this alignment was measured as 410,000 linear feet, somewhat longer than listed in the proposal. For the described alignment, the elevation ranges from an elevation of 4,550 feet above mean sea level at Highway 180 at the west end, to a high point of 6,192 near Bayard and an elevation of 4,337 at Deming. This translates to a difference of 1,855 feet in elevation from high to low elevations, which has consequences as the selection of pipe material and pressure classes, and the configuration of the pumping system to deliver water from one end to another. The high point shown on the profile, also on Figure 28, occurs west of Silver City. Based on this profile, flow will be by gravity from Hurley to 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. Demands for each municipal area were estimated, loosely based on the 2005 Southwest New Mexico Regional Water Plan by Daniel B. Stephens and Associates (DBSA), Table 6-15, “Estimated Future Public Water Supply Use in the Southwest Region.” Those demands are labeled on Figure 28, which shows the pipeline alignment and a conceptual hydraulic profile.

Based on the hydraulic analysis, a 14-inch pipe size will result in excessive head losses, such that adequate pressure would not be provided at the terminus near Deming. The pipe size would optimally be a combination of 14-inch, 16-inch, and 18-inch, based on the elevation grade. For simplification for this study, and because the surface data we have used has a vertical accuracy of no greater than five feet, we have based the cost estimate on a 16-inch pipeline from Hurley to Deming.

The Tier 2 proposal includes a schematic that shows a system of five equalization tanks and booster stations along the pipeline alignment. Each booster station will require acquisition of a parcel of land for the booster pumps and tank. The first booster station would be located at the reservoir near the Gila River. The proposal cites a reservoir on Mangas Creek, but that location was eliminated by this study due to endangered species habitat. Based on the reservoirs selected for this study, the proposed reservoir at Pope Canyon would supply this regional pipeline.

The total static head that must be achieved to lift water from Pope Canyon to the high point on the pipeline is over 1,600 feet. Based on a flow of 10,000 AFY, the pump station must deliver 6,190 gallons per minute (based on 24 hour per day operation). The corresponding velocity in a 24-inch pipe for this flow is 4.4 feet per second which is generally within recommended range for transmission piping. However, BHI conducted a preliminary analysis of the pumping head requirements for a 24-inch pipe versus a 36-inch pipe. For comparison, the head loss in a 24-inch pipeline 138,000 feet in length (the distance from the start of the pipeline to the high point) at 6,190 gpm is 352 feet, while the head loss in a 36-inch pipeline of the same length is only 49 feet. The additional head requirement for the smaller size pipe has two cost implications: 1) ongoing electrical pumping costs, and 2) initial capital and recurring replacement cost of higher horsepower pumps, at each of the five proposed lift stations. Based on our high level analysis, a 36-inch pipe would increase the pipeline installation cost by approximately $7 million, but would decrease the cost of the pumps, and in addition would decrease the ongoing electrical costs and the recurring pump replacement costs. A more detailed cost-benefit analysis should be conducted at the time of design when design-level topographical data is available. For this study, a 36-inch pipeline was assumed from the storage on the Gila River to the high point along the proposed route. 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). There may be a potential to convert that energy to electricity, which would offset the electricity requirements of the booster station east of the high point.

For the purposes of preparing a preliminary cost estimate for this pipeline and the associated pumping stations, the schematic system configuration shown in the proposal was further refined. Based on a 36-inch pipeline, a water surface elevation at Pope Canyon of 4,640, and the head loss cited above, the total dynamic head required is estimated, without minor losses, as 1,967 feet. Therefore, the total lift that each booster station will have to provide is 393 feet. Based on the actual location where property is acquired, the lift for each station will change. However, for preliminary sizing purposes, five booster stations, each with 6,190 gpm at 393 feet of Total Dynamic Head (TDH) are assumed. Each station will therefore require 750 horsepower of pumping capacity.

Each booster station will require a pump station building. A configuration of three 3,100 gpm pumps is recommended which will allow flexibility in pumping rate, and standby capacity in case of a pump failure. An equalization tank with capacity of at least one hour of water supply is recommended. For the cost estimate, a tank of 375,000 gallon capacity was used.

The cost estimates are based on a 50 ft X 50 ft site, including site grading and gravel surface, chain link fence, 25 ft X 15 ft prefabricated booster station building, three horizontal split case pumps, 375,000 welded steel storage tank, and site piping as required to convey water to the tank, between the tank and booster station and discharge to the main pipeline. The Hydraulic Grade Line (HGL) analysis for the pipeline from Gila to Deming is included in Appendix E. The cost estimate for the booster station and pipeline is included in Appendix G.

VII. CONCEPTUAL ALTERNATIVES

BHI has evaluated three alternatives, which are summarized below.

A. ALTERNATIVE 1

Alternative 1 includes a river diversion at elevation 4,770, storage in all six recommended canyons, and conveyances between those storage sites (see Figure 25). The diversion site at elevation 4,770 was selected because it is just downstream of Turkey Creek, near the end of the existing Turkey Creek Road. Due to the rugged terrain and the limited available head loss between the diversion site at elevation 4,770 and the storage in Spar Canyon at elevation 4755, an open channel (rather than closed pipe) was selected and evaluated as the conveyance between the diversion at elevation 4,770 and Spar Canyon. An open channel was selected due to constructability concerns with a closed pipe. The allowable headloss between the diversion point and Spar Canyon would require a 144-inch diameter pipe. The installation of such a large pipe and the corresponding required trench width through this sinuous reach of the river would result in unacceptable disturbance of the natural environment. In addition, much of the excavation is expected to be through rock which may require blasting. In addition, there is limited access for heavy equipment.

As mentioned in Appendix B, some blasting efforts will be required for the conveyances. Geo-Test has not done a detailed analysis of rock requiring blasting. Some modifications to the alignments could be made during the design process to reduce blasting, based on detailed geotechnical/geological information. Assuming a 0.03 percent slope along the 10-mile alignment between the diversion point at an elevation of 4,770 and the water surface elevation of 4,755 in Spar Canyon, a 10 foot bottom width, 5 foot deep channel with 3 to 1 side slopes would be required to convey 350 cfs.

Pressure piping was evaluated between storage sites below Spar Canyon in Alternative 1. To provide maximum operational flexibility, a design flow rate of 350 cfs was used from the diversion to Pope Canyon. This is because the upper storage locations (Spar, Maldonado and Winn) will serve agricultural users and environmental needs of the river, while storage from Pope Canyon will potentially feed the Southwest Regional Water Supply (described below). When water for irrigation is not needed, it will be conveyed to Pope Canyon and the southernmost storage locations. For storage downstream of Pope, flow through the conveyances was assumed to be proportional to the reservoir storage volumes. Table VI-7 summarizes the conveyance lengths, their design flow rate, and their sizes for Alternative 1. The HGL analysis for Alternative 1 is included in Appendix E.

Table VI-7 – Conveyances for Alternative 1

Conveyance	Length (ft)	Design Flow Rate (cfs)	Diameter (in.)
Spar to Maldonado	6,931	350	108
Maldonado to J4	5,272	350	78
J4 to Winn	6,836	44	36
J4 to Pope	28,758	350	84
Pope to J2	12,904	258	78
J2 to Sycamore	1,369	237	54
J2 to Dix	9,262	21	36

B. ALTERNATIVE 2

Alternative 2 includes a river diversion at elevation 4,695, storage in Winn, Pope, Sycamore and Dix canyons, and conveyances between those storage sites (see Figure 26). The diversion site at elevation 4,695 was chosen because it is near a narrow ridge that could be tunneled through. As with Alternative 1, due to the rugged terrain and the limited available head loss, the initial segment of the conveyance from the diversion downstream to Junction 5, where the open channel transitions to a pipe in Box Canyon Road, was assumed to be an open channel rather than closed pipe. An example of a channel-to-pipe transition structure can be seen in the photograph below.

From Junction 5 to Junction 4 and from Junction 4 to Winn, the conveyances were assumed to be closed pipes. Pressure piping was also evaluated between storage sites below Winn Canyon in Alternative 2. Flow was apportioned to the conveyances based on the storage the conveyance is serving. Table VI-8 summarizes the conveyance lengths, their design flow rate, and their sizes for Alternative 2. The HGL analysis for Alternative 2 is included in Appendix E.

Table VI-8 – Conveyances for Alternative 2

Conveyance	Length (ft)	Design Flow Rate (cfs)	Diameter (in)
J5 to J4	10,085	350	108
J4 to Winn	6,836	38	84
J4 to Pope	28,758	350	108
Pope to J2	12,903	258	72
J2 to Sycamore	1,369	237	72
J2 to Dix	9,262	21	48

C. ALTERNATIVE 3

Alternative 3 includes a river diversion at elevation 4,640, storage in Pope, Sycamore and Dix canyons, and conveyances between those storage sites (see Figure 27). The diversion site at elevation 4,640 was chosen because it is at the top of the wide valley with easy construction and maintenance access. Alternative 3 only includes pressure piping. Table VI-9 summarizes the conveyance lengths, their design flow rate, and their sizes for Alternative 3. The HGL analysis for Alternative 3 is included in Appendix E.

Table VI-9 – Conveyances for Alternative 3

Conveyance	Length (ft)	Design Flow Rate (cfs)	Diameter (in)
Diversion to J6	61,527	350	108
J6 to Pope	2,811	54	48
J6 to J2	9,873	296	108
J2 to Sycamore	1,369	274	78
J2 to Dix	9,262	22	48

VIII. COST ESTIMATES

BHI prepared preliminary planning level cost estimates for all three alternatives. The cost estimates are included in Appendix G and are summarized in Table VIII-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 VIII-1 – Estimated Construction Costs

Alternative (per Section VII)	Construction	Non-Construction	Total
1a	$487,930,000	$71,446,000	$559,376,000
1b	$452,399,000	$66,849,000	$519,248,000
2a	$432,258,000	$63,280,000	$495,538,000
2b	$358,242,000	$53,913,000	$412,155,000
3	$307,440,000	$46,118,000	$353,558,000

A. DIVERSION

All alternatives include a diversion structure. The cost for the diversion structure is based on several assumptions. First, excavation earthwork spans the length of the structure and goes one foot deeper than the trough depth. The earthwork also spans the width of the wingwall in the streamwise direction plus two feet on either side. 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, a tilted wedge wire screen, headgates and a pedestrian bridge.

B. OPEN CHANNEL

Alternatives 1 and 2 both include a concrete lined open channel for the uppermost conveyance. The unit cost for the open channel is based on several assumptions. First, some of the channel will have 3 to 1 tie slopes and will be able to be constructed with earth. Second, some of the channel will require 2 to 1 tie slopes, which will require concrete slope paving. Third, some of the channel will require vertical structural concrete walls in order to construct the channel along the side of the valley. A template was created in Bentley InRoads V8i design software and included all three of these assumptions. This template was run along the 8.3-mile alignment between the diversion point at elevation 4,770 and Spar Canyon in order to quantify the earthwork, concrete slope paving, and structural concrete required to construct the channel. The earthwork was assumed to be 20 percent rock excavation, 20 percent rock blasting, and 60 percent conventional excavation, based on recommendations from Geo-Test. For a concrete lined channel, BHI calculated a unit price of $2,600 per linear foot. For comparison, an unlined channel was estimated to cost $2,200 per linear foot. Although a concrete lined channel is approximately 18 percent more expensive per linear foot than an unlined channel, the maintenance requirements will be less for a concrete lined channel than for an unlined channel, and no water will be lost from a lined channel through infiltration. Therefore, BHI recommends constructing a concrete lined channel for the uppermost conveyance for Alternatives 1 and 2, if a channel is to be used. Another alternative as discussed previously is the use of a tunnel for significant portions of the conveyance from the point of diversion to Spar Canyon.

C. TUNNEL

Given the high cost of an open channel conveyance from the point of diversion to Spar Canyon due to the rugged topography, an option to use a tunnel conveyance was investigated. Consultations with a tunneling contractor led to a price quote of $3,500 per linear foot for a tunnel for either Alternative 1 or Alternative 2 from the point of diversion to Spar Canyon. This price is an indication that tunnel construction can be risky, due to differing site conditions, such as fractures and water. The unit price could come down with a geotechnical baseline report, which would be part of the design of the tunnel. The geotechnical baseline report sets baseline site conditions to determine the existence of differing site conditions, such as faults, water, or hardness of the material. While the price per linear foot is significantly higher than the cost for even a lined open channel, the use of a tunnel reduces the conveyance length from the point of diversion to Spar Canyon from 8.3

miles in Alternative 1 to only 4.8 miles and from 5.3 miles in Alternative 2 to only 1.1 miles. Consequently, a tunnel option is economically superior to an open channel conveyance. The relative costs of a channel and tunnel can be seen in Table VIII-2, below.

Table VIII-2 – Relative Costs of Channel and Tunnel Alternatives

Alternative	Length (ft)	Unit Price	Total Cost of Conveyance
1a (Channel)	43,600	$2,600	$113 M
1b (Tunnel)	25,300	$3,500	$89 M
2a (Channel)	27,900	$2,600	$73 M
2b (Tunnel)	5,900	$3,500	$21 M

D. STORAGE RESERVOIRS

1. The estimated costs for the storage reservoirs are based on several assumptions, which are discussed below.
2. Earthwork has been optimized to balance the cut and fill at each reservoir site.
3. Costs for excavation and backfill are based on the total excavation at each site. The only dam site that requires import is Dix. This import is assumed to come from Sycamore and Pope, which both have excess material available.
4. 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 PMF or other overtopping situations. In addition, the emergency spillway concrete is assumed to extend down the upstream side of the dam to prevent seepage. Since the concrete is assumed to extend to the upstream toe of the dam, the dam slopes were assumed to be 2:1 on the upstream side for the purposes of balancing the earthwork. All dams have a 4:1 downstream slope, except for Dix, which has a 2:1 downstream slope to minimize import of material from other canyons.
5. 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.

E. SOUTHWEST REGIONAL WATER SUPPLY

The estimated cost of the Southwest Regional Water Supply, which consists of pipeline and booster stations along Highway 180, is $88.3 million, including non-construction costs and excluding NMGRT. The cost of the SWRWS pipeline is included in the costs shown below for each alternative.

F. 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 upstream most conveyance from the point of diversion to Spar Canyon. Under Alternative 1a, the conveyance would be an open channel, while Alternative 1b proposes this conveyance as a tunnel.

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

Table VIII-3 – Estimated Costs for Alternative 1a

Item	Cost
Diversion, Conveyance, and Reservoir Construction	$487,930,000
Design	$21,957,000
Topographic Survey	$863,000
Right-of-Way Easement Development (Pipeline + Rural)	$408,000
Permitting, Environmental & Geotechnical Investigations	$504,000
Land Acquisition Services (Reservoirs + Pipeline + Booster Stations)	$85,000
Land Acquisition (Reservoirs + Pipeline + Booster Stations)	$6,859,000
Easement Acquisition (Pipeline + Channel)	$1,736,000
Construction Observation and Management	$39,034,000
Subtotal	$559,376,000
NMGRT	**$34,611,000**
Total	**$593,987,000**

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

1. Diversion at elevation 4,770
2. Open channel from the diversion to Spar
3. Storage in Spar, Maldonado, Winn, Pope, Sycamore, and Dix Canyons
4. Pressure piping between storage sites

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

Estimated costs for Alternative 1b, including planning, design, construction, and administration/oversight are summarized in Table VIII-4, below.

Table VIII-4 – Estimated Costs for Alternative 1b

Item	Cost
Diversion, Conveyance, and Reservoir Construction	$452,399,000
Design	$20,358,000
Topographic Survey	$846,000
Right-of-Way Easement Development (Pipeline + Rural)	$408,000
Permitting, Environmental & Geotechnical Investigations	$504,000
Land Acquisition Services (Reservoirs + Pipeline + Booster Stations)	$85,000
Land Acquisition (Reservoirs + Pipeline + Booster Stations)	$6,859,000
Easement Acquisition (Pipeline)	$1,597,000
Construction Observation and Management	$36,192,000
Subtotal	$519,248,000
NMGRT	**$32,128,000**
Total	**$551,376,000**

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

1. Diversion at elevation 4,770
2. 108-inch tunnel from the diversion to Spar
3. Storage in Spar, Maldonado, Winn, Pope, Sycamore, and Dix Canyons
4. Pressure piping between storage sites

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

G. 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 upstream most conveyance from the point of diversion to Winn Canyon. Under Alternative 2a the conveyance would be an open channel transitioning to a pipe, while Alternative 2b proposes this conveyance as a tunnel.

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

Table VIII-5 – Estimated Costs for Alternative 2a

Item	Cost
Diversion, Conveyance, and Reservoir Construction	$432,258,000
Design	$19,452,000
Topographic Survey	$800,000
Right-of-Way Easement Development (Pipeline + Rural)	$406,000
Permitting, Environmental & Geotechnical Investigations	$360,000
Land Acquisition Services (Reservoirs + Pipeline + Booster Stations)	$70,000
Land Acquisition (Reservoirs + Pipeline + Booster Stations)	$5,930,000
Easement Acquisition (Pipeline + Channel)	$1,681,000
Construction Observation and Management	$34,581,000
Subtotal	$495,538,000
NMGRT	**$30,661,000**
Total	**$526,199,000**

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

1. Diversion at elevation 4,695
2. Open channel from the diversion to Junction 5
3. Storage in Winn, Pope, Sycamore, and Dix Canyons (the elevation of the diversion is too low to provide for storage in Spar and Maldonado Canyons)
4. Pressure piping between Junction 5 and the storage sites

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

Estimated costs for Alternative 2b, including planning, design, construction, and administration/oversight are summarized in Table VIII-6, below.

Table VIII-6 – Estimated Costs for Alternative 2b

Item	Cost
Diversion, Conveyance, and Reservoir Construction	$358,242,000
Design	$16,121,000
Topographic Survey	$779,000
Right-of-Way Easement Development (Pipeline + Rural)	$406,000
Permitting, Environmental & Geotechnical Investigations	$360,000
Land Acquisition Services (Reservoirs + Pipeline + Booster Stations)	$70,000
Land Acquisition (Reservoirs + Pipeline + Booster Stations)	$5,930,000
Easement Acquisition (Pipeline)	$1,588,000
Construction Observation and Management	$28,659,000
Subtotal	$412,155,000
NMGRT	**$25,502,000**
Total	**$437,657,000**

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

1. Diversion at elevation 4,695
2. 108-inch tunnel from the diversion to Junction 5
3. Storage in Winn, Pope, Sycamore, and Dix Canyons (the elevation of the diversion is too low to provide for storage in Spar and Maldonado Canyons)
4. Pressure piping between Junction 5 and the storage sites

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

H. ALTERNATIVE 3

Estimated costs for Alternative 3, including planning, design, construction, and administration/oversight are summarized in Table VIII-7, below.

Table VIII-7 – Estimated Costs for Alternative 3

Item	Cost
Diversion, Conveyance, and Reservoir Construction	$307,440,000
Design	$13,835,000
Topographic Survey	$712,000.00
Right-of-Way Easement Development (Pipeline + Rural)	$414,000
Permitting, Environmental & Geotechnical Investigations	$288,000
Land Acquisition Services (Reservoirs + Pipeline + Booster Stations)	$58,000
Land Acquisition (Reservoirs + Pipeline + Booster Stations)	$4,630,000
Easement Acquisition (Pipeline)	$1,586,000
Construction Observation and Management	$24,595,000
Subtotal	$353,558,000
NMGRT	**$21,876,000**
Total	**$375,434,000**

The cost for Alternative 3 is based on the following:

1. Diversion at elevation 4,640
2. Storage in Pope, Sycamore, and Dix Canyons (the elevation of the diversions is too low to provide for storage in Spar, Maldonado, and Winn Canyons)
3. Pressure piping between the diversion and the storage sites

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

IX. COMPARISON OF ALTERNATIVES

This report identifies three alternatives related to Category 2 diversion locations. The intent of the project is to provide a total water storage capacity of at least 65,000 acre-feet (AF) of water without harming the river to produce a safe yield of minimum 10,000 AFY. At least 5,000 AF of water storage is to be located near the top of the Cliff-Gila valley. Alternatives 1 and 2 both meet both of these criteria. Alternative 3 does not meet either of these criteria. Table IX-1 below, compares storage, cost, Operating and Maintenance (O&M) cost, and cost/AF for each alternative.

Table IX-1 – Comparison of Alternatives

Alternative	Description	Total Storage (AF)	Cost before NMGRT	Annual O&M Costs	Cost per AF
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	$2,886,000	$7,837
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	$2,846,400	$7,275
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	$2,652,160	$7,706
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	$2,570,760	$6,410
Alternative 3	Diversion at elevation 4,640, pipe from diversion to Pope Canyon, storage at Pope, Sycamore and Dix Canyons	46,406	$354 m	$2,413,060	$7,619

X. CONCLUSIONS AND RECOMMENDATIONS

A. RECOMMENDED PROJECT

This PER identifies recommended options for water diversion, conveyance, and storage along the Gila River between Turkey Creek and Cherokee Canyon, near the towns of Cliff and Gila in Grant County in southwestern New Mexico as a means of developing an additional 14,000 AF of water allocated to New Mexico under the 2004 Arizona Water Settlements Act. In addition, this PER identifies locations for Category 1 diversions to supply the existing irrigation system in the Cliff-Gila farming valley. Three alternatives related to the Category 2 diversions, the diversions which provide water to the storage reservoirs, were evaluated. Two of them, Alternatives 1 and 2, meet the objective of the project to provide a total water storage capacity of at least 65,000 AF of water without harming the river, to produce a safe yield of minimum 10,000 AFY.

BHI recommends construction of Alternative 2b. It meets the objectives of the project, and it includes a minimum of 5,000 AF of storage near the top of the Cliff-Gila valley (from which most of the farmland in the valley can benefit). To maximize the irrigable farmland in the upper valley, a gravity conveyance could be constructed to supply the Upper Gila and Fort West ditches from Winn Canyon. Assuming 0.03 percent grade from Winn Canyon, this gravity conveyance, shown on Figure 29, would connect to the existing Upper Gila ditch just to the south of Winn canyon, approximately 2.5 miles downstream of the current Upper Gila diversion site. A ditch could be constructed to the east from there, and a siphon under the river to connect to the existing Fort West ditch approximately 1.5 miles downstream of the current Fort West diversion site. Approximately 2,200 acres of irrigable land exist downstream of this area, which is approximately 93 percent of the total irrigable acreage in the valley.

The total estimated cost for the recommended project is $437 million, including environmental and geotechnical investigations, survey, design, land acquisition, construction and construction management. The future cost, in year 2024 dollars, is estimated at $556 million, assuming 27 percent inflation over the 10 year period (based on US Bureau of Labor Statistics Construction CPI Inflation Calculation www.bls.gov/data/inflation_calculator.htm).

The recommended alternative is not expected to produce negative long-term environmental impacts or conditions conducive to non-native species like tamarisk and non-native fish. Water could be stored in the reservoirs and released to feed the Gila River in dry years.

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 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, bringing them to a more detailed level similar to the diversions which were taken to a conceptual level design as part of 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 fatal flaws to the technical 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.

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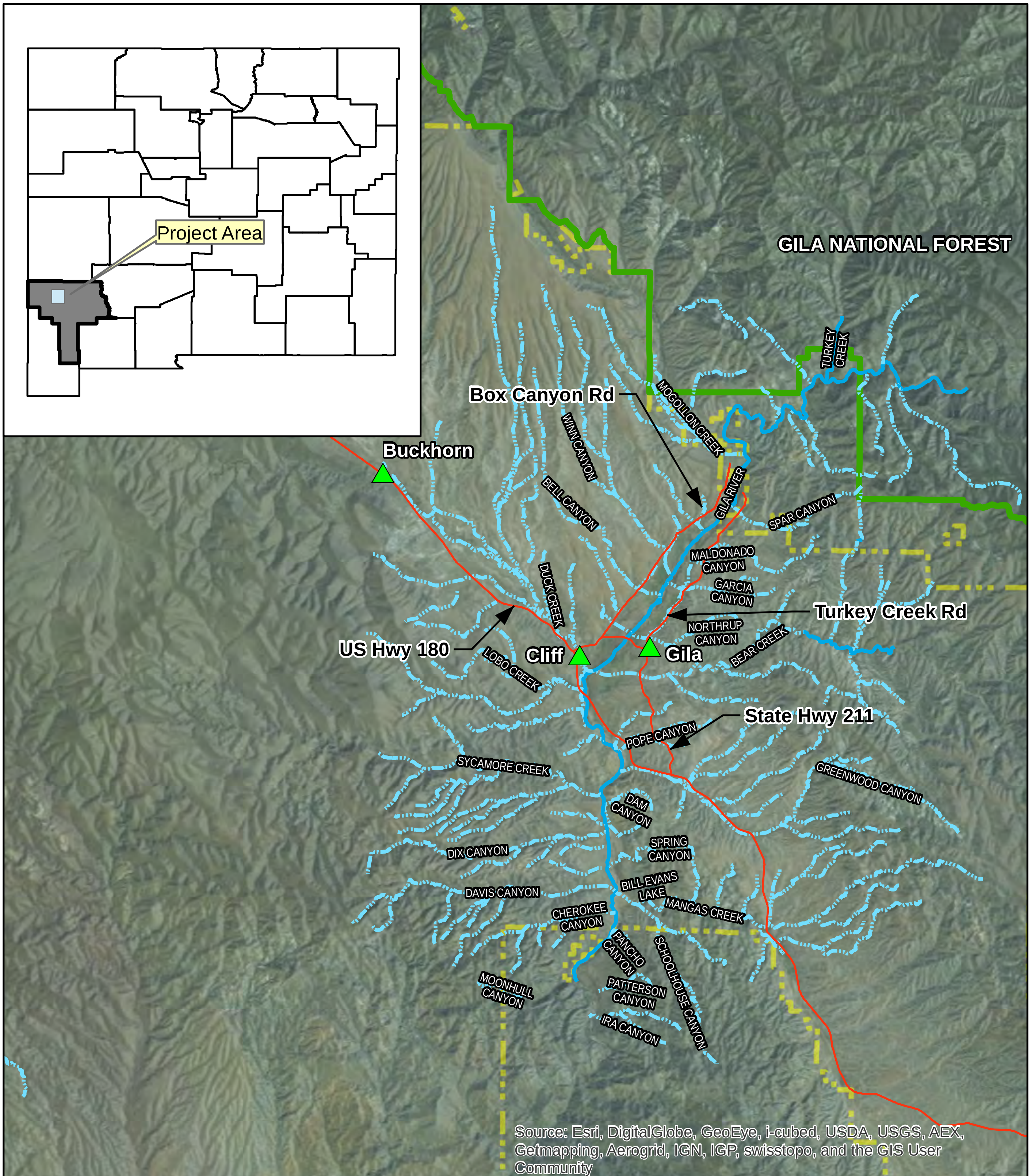
FIGURES



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GILA RIVER DIVERSION, CONVEYANCE AND STORAGE ALTERNATIVES

FIGURE 1
VICINITY MAP

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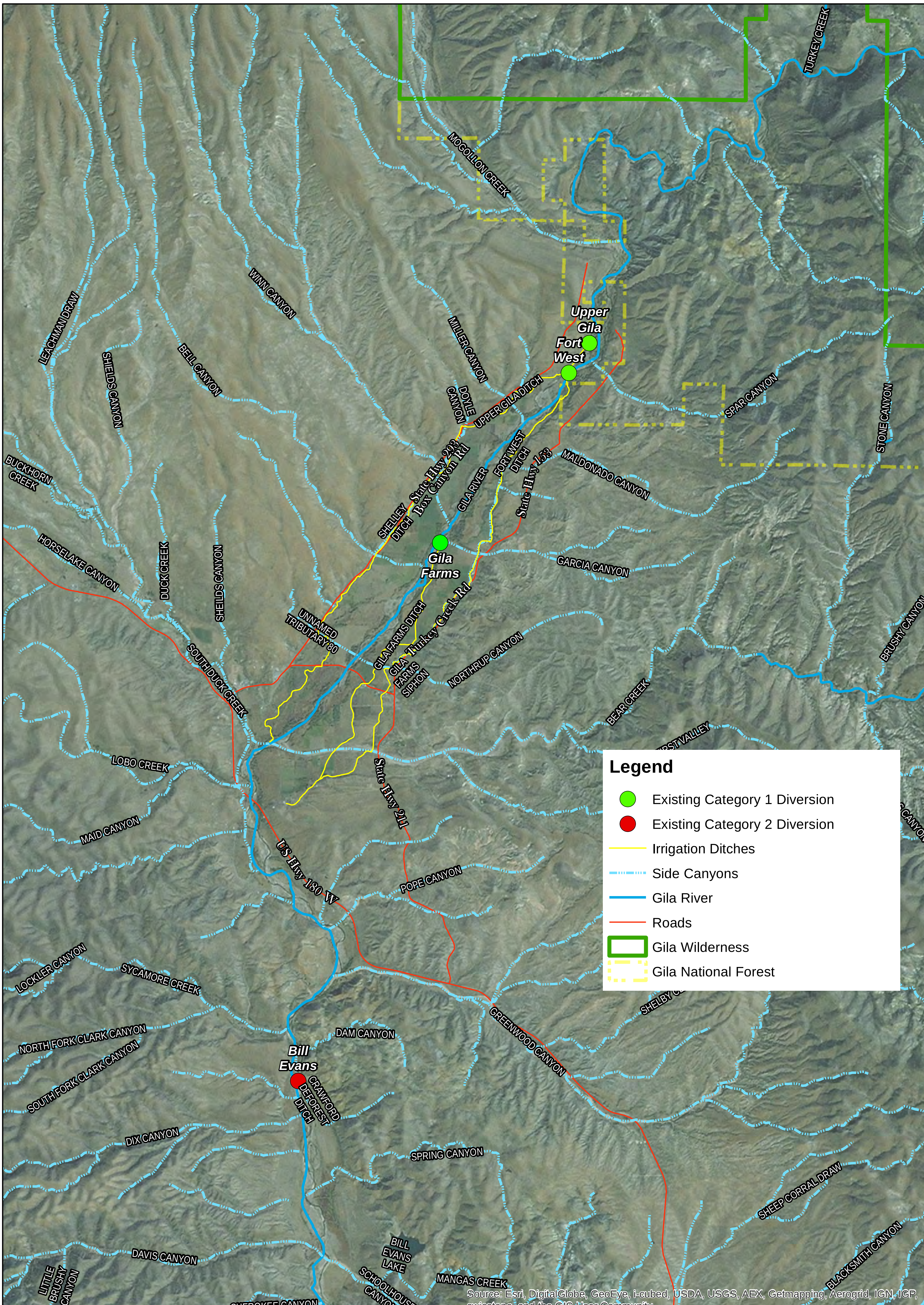
Author: ksaintlot

April, 2014



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GILA RIVER DIVERSION, CONVEYANCE AND STORAGE ALTERNATIVES

FIGURE 2
PROJECT PLANNING AREA

P:\20140055\WR\Reports\Final\Figures\PER Figures\Figure2_ProjectPlanningArea.mxd
Author: ksaintlot

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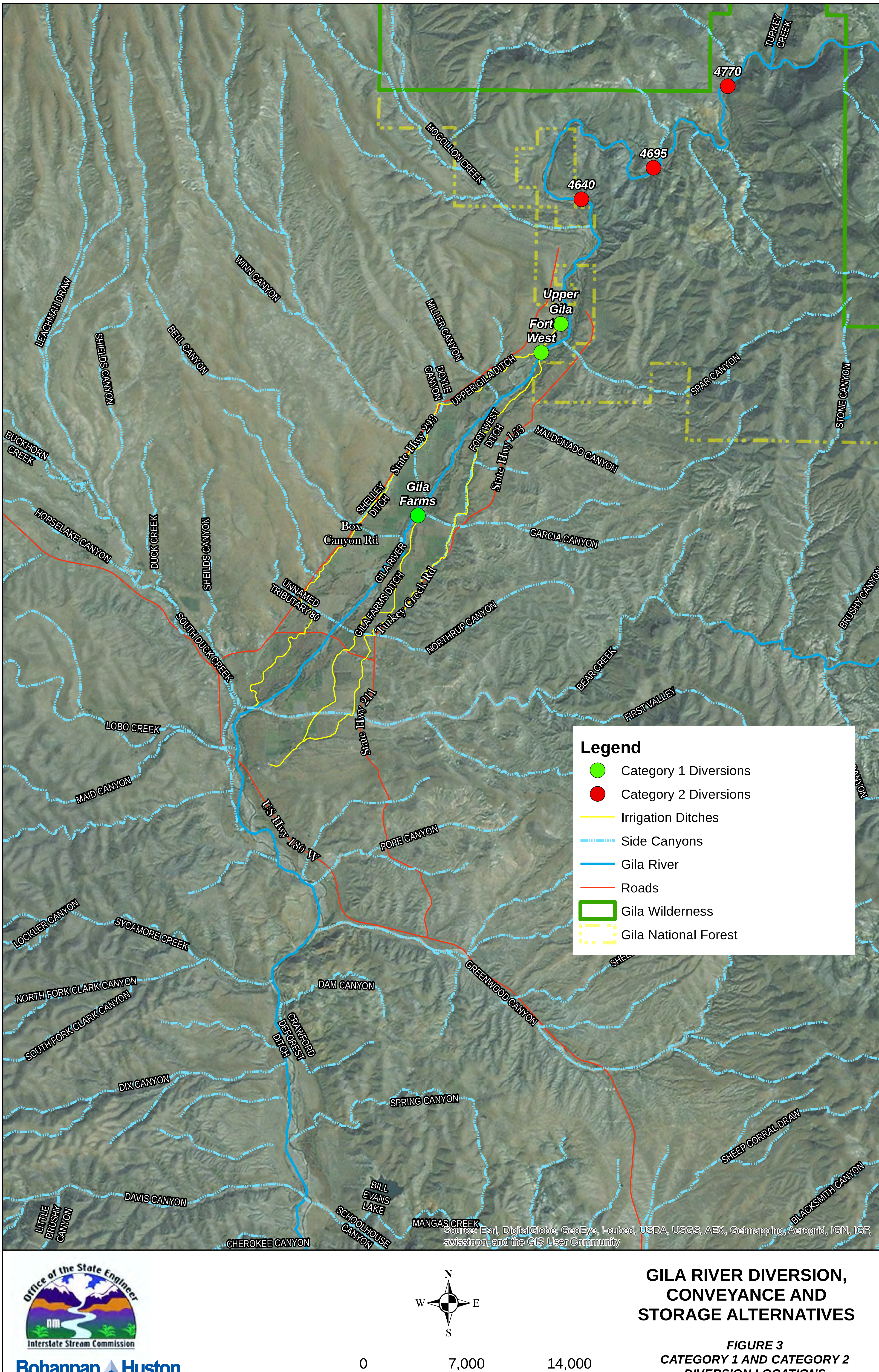

GILA RIVER DIVERSION, CONVEYANCE AND STORAGE ALTERNATIVES

FIGURE 3
CATEGORY 1 AND CATEGORY 2 DIVERSION LOCATIONS

P:\20140055\WR\Reports\Final\Figures\PER Figures\Figure3_Category1&2DiversionLocations.mxd
Author: ksaintlot

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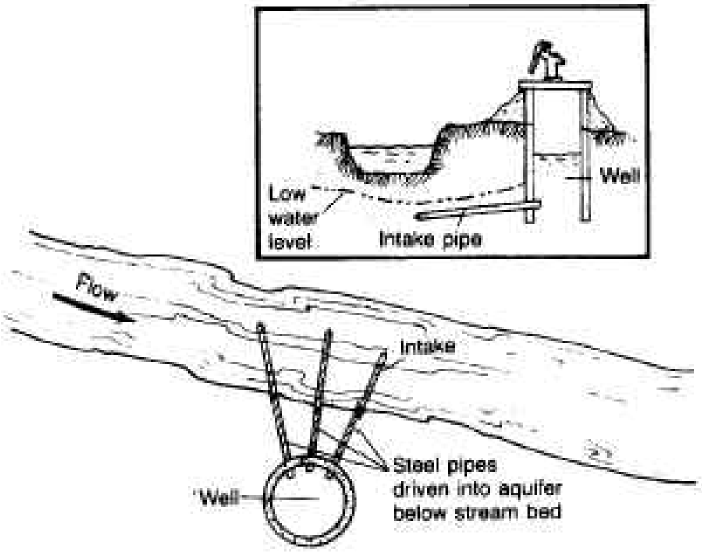
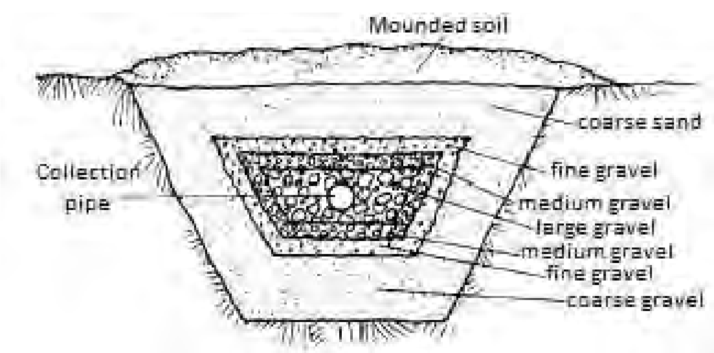
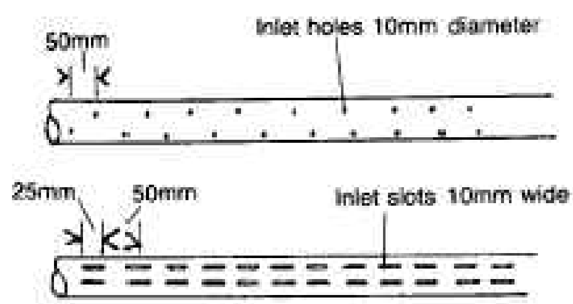


SEE TYPICAL SECTION BELOW

PLAN & CROSS SECTION VIEW
NTS



TYPICAL SECTION DETAIL
NTS



TYPICAL PIPE DETAIL
NTS

NOTES:
1. Conceptual detail adapted from "Designing Intakes for Rivers and Streams." Technical Note No. RWS1.0.3, USAID.
2. Pipe locations, size, material to be determined during final design.

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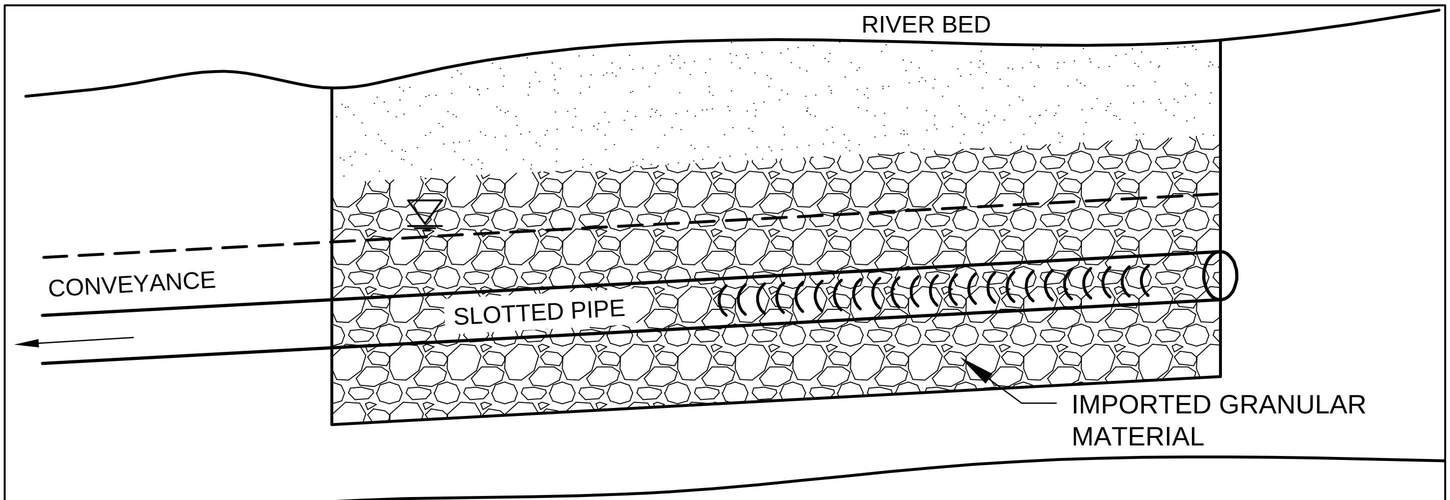
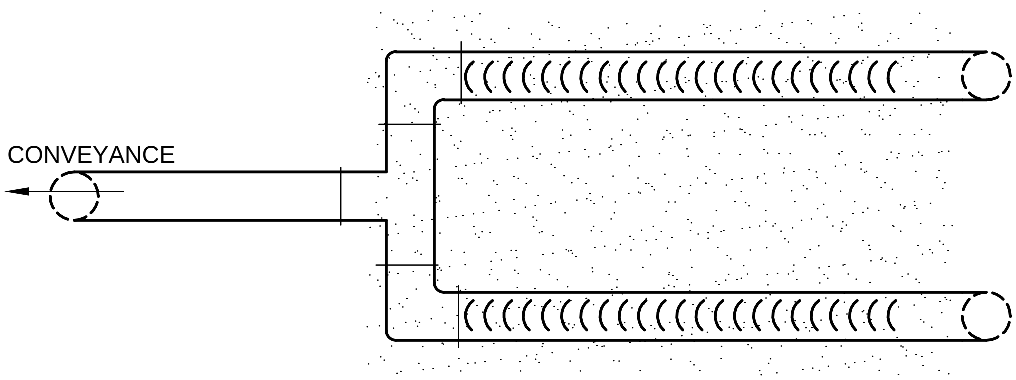
FIGURE 4
SHALLOW COLLECTOR WELLS WITH HORIZONTAL WELLS

P:\20140055\WR\Reports\Final\Figures\PER Figures\Figure4_ShallowCollectorWells.dwg
Tue, 1-Apr-2014 - 7:49:am, Plotted by: KSAINTLOT
April, 2014



PROFILE VIEW

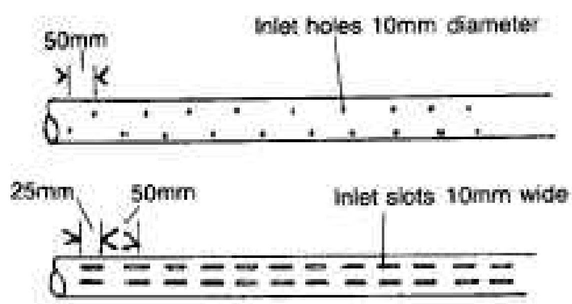
NTS



PLAN VIEW

NTS

Source: Robert Bower



TYPICAL PIPE DETAIL

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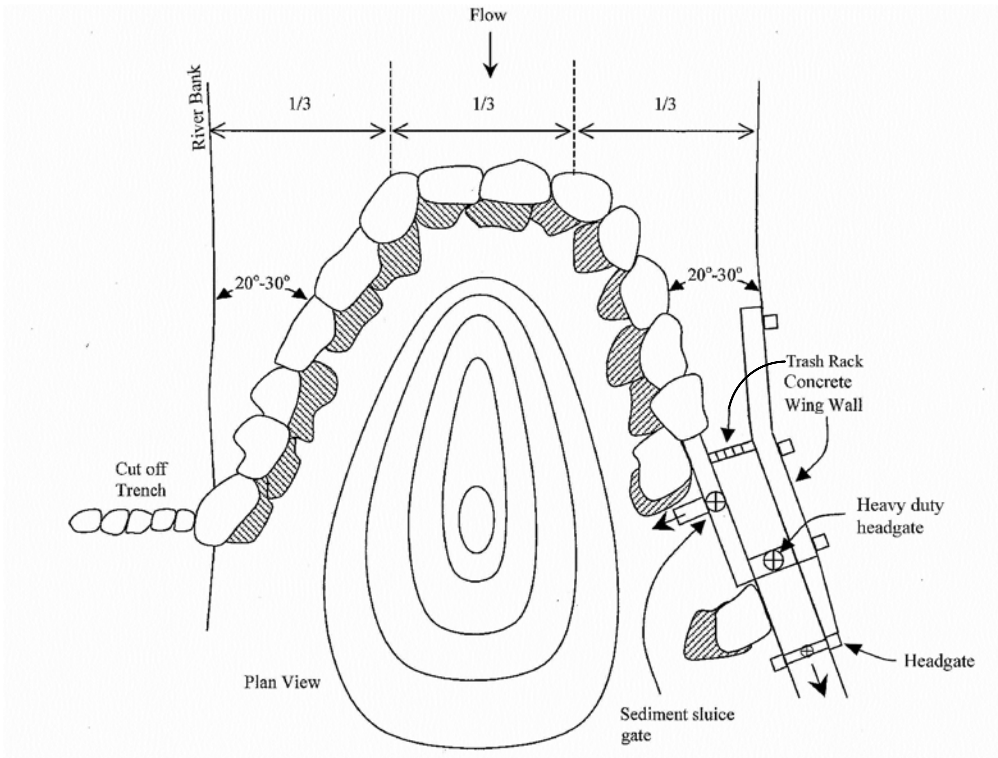
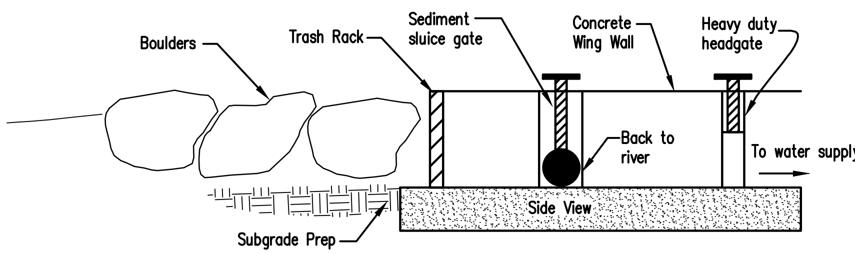
GILA RIVER DIVERSION, CONVEYANCE AND STORAGE ALTERNATIVES

FIGURE 5

TYPICAL INFILTRATION GALLERY





NOTES:

1. Conceptual detail adapted from "The Cross-Vane, W-Weir, and J-Hook Vane Structures. . .Their Description, Design, and Application for Stream Stabilization and River Restoration", D.L. Rosgen, 1998.
2. Cross section, rock elevation, and rock size specification to be determined during design.
3. Wing wall, headgates, and sediment sluice gate shown are only conceptual; location, size, and materials to be determined during final design.

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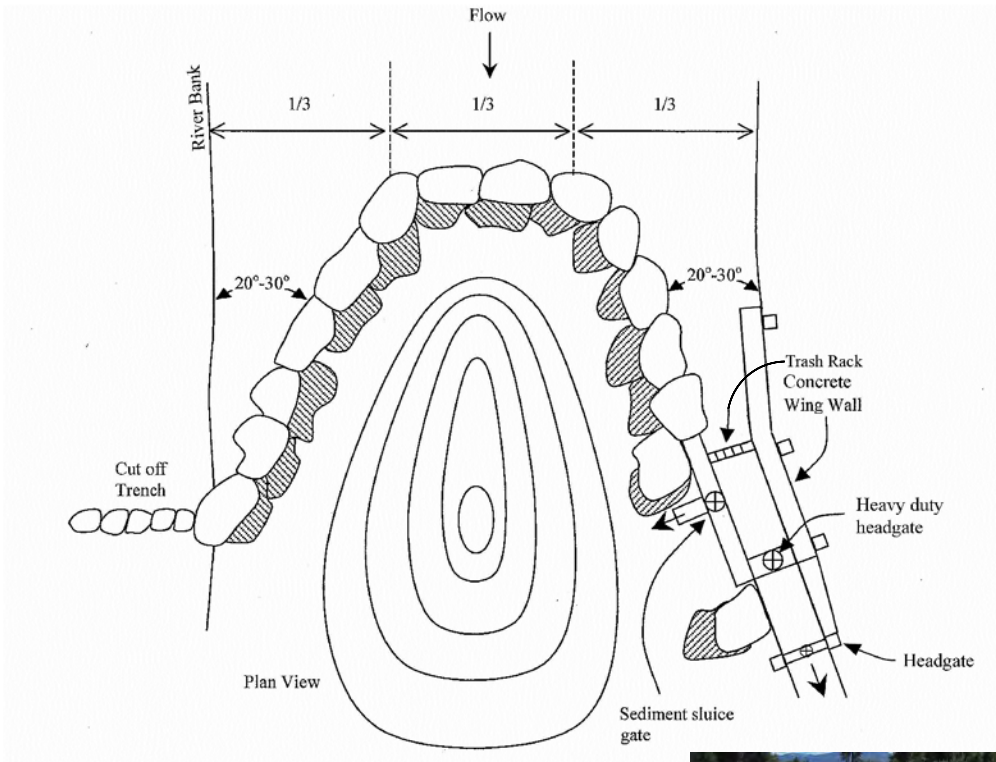
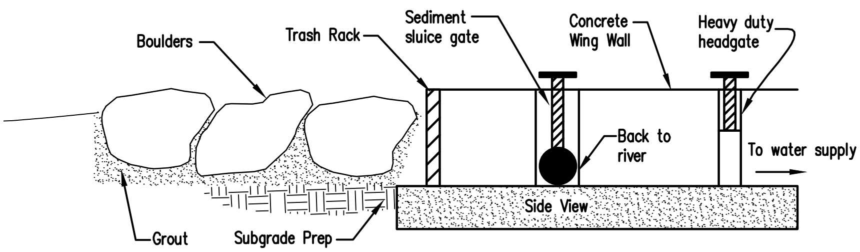
GILA RIVER DIVERSION, CONVEYANCE AND STORAGE ALTERNATIVES

FIGURE 6
TYPICAL ROCK CROSS VANE WEIR





NOTES:

1. Conceptual detail adapted from "The Cross-Vane, W-Weir, and J-Hook Vane Structures. . .Their Description, Design, and Application for Stream Stabilization and River Restoration", D.L. Rosgen, 1998.
2. Cross section, rock elevation, and rock size specification to be determined during design.
3. Wing wall, headgates, and sediment sluice gate shown are only conceptual; location, size, and materials to be determined during final design.

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GILA RIVER DIVERSION, CONVEYANCE AND STORAGE ALTERNATIVES

FIGURE 7
TYPICAL GROUTED BOULDER WEIR

P:\20140055\WR\Reports\Final\Figures\PER Figures\Figure7_TypicalGroutedBoulderWeir.dwg
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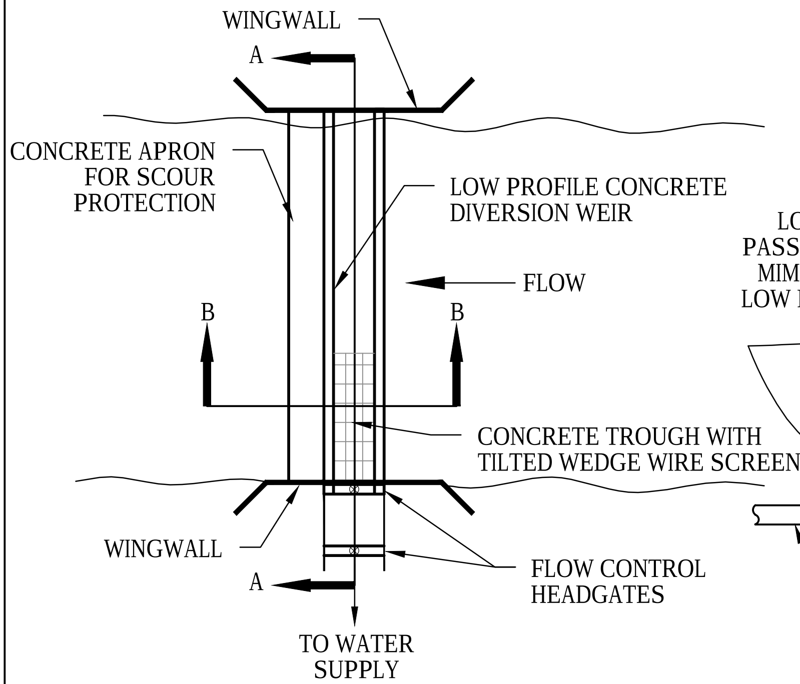
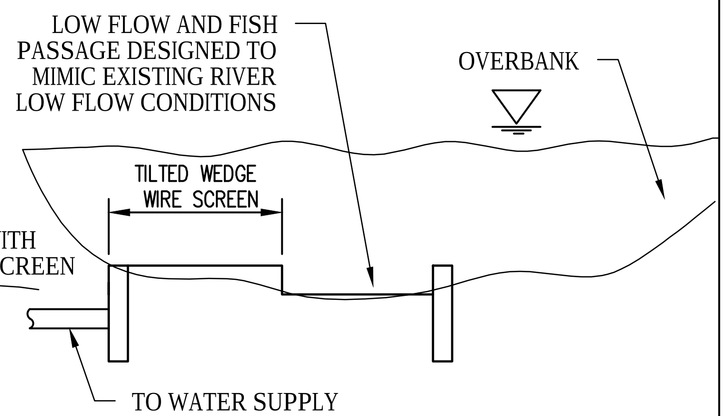


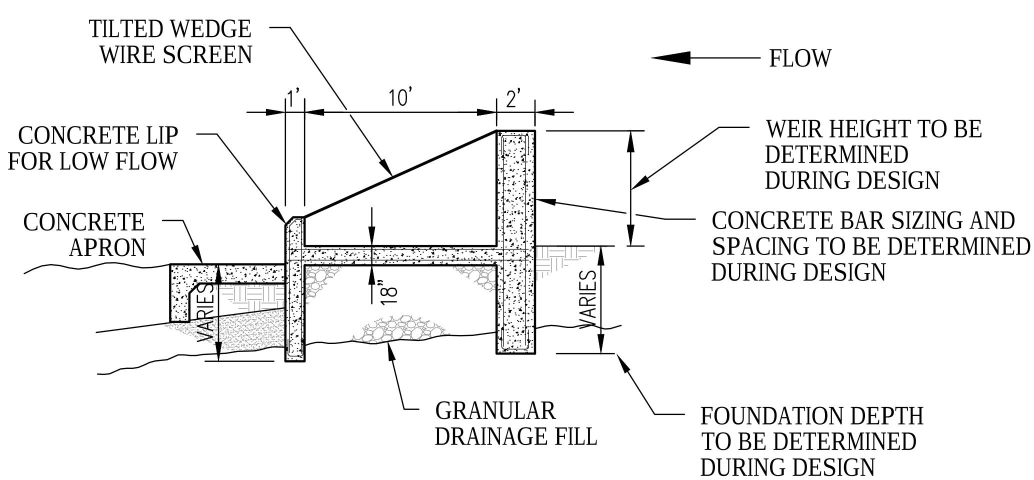


TYPICAL WEIR SECTION A

NTS

PLAN VIEW

NTS



TYPICAL WEIR SECTION B

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FIGURE 8
TYPICAL LOW PROFILE CONCRETE WEIR WITH TILTED WEDGE WIRE SCREEN

P:\20140055\WR\Reports\Final\Figures\PER Figures\Figure8_TypicalLowProfileConcreteWeir.dwg
Tue, 1-Apr-2014 - 7:52:am, Plotted by: KSAINTLOT

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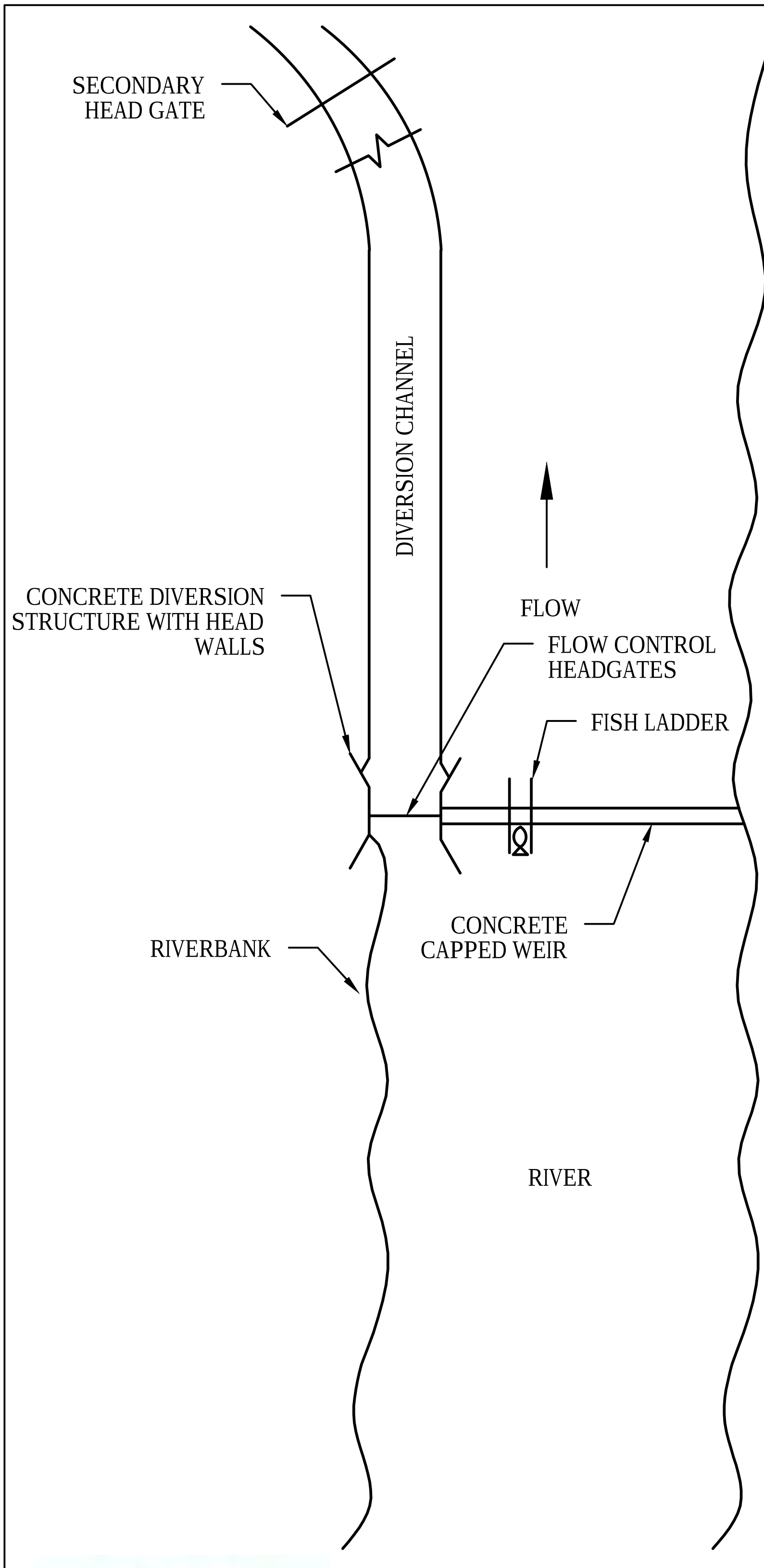

BILL EVANS DIVERSION - CONTROL GATES

BILL EVANS DIVERSION - CONCRETE CAPPED WEIR

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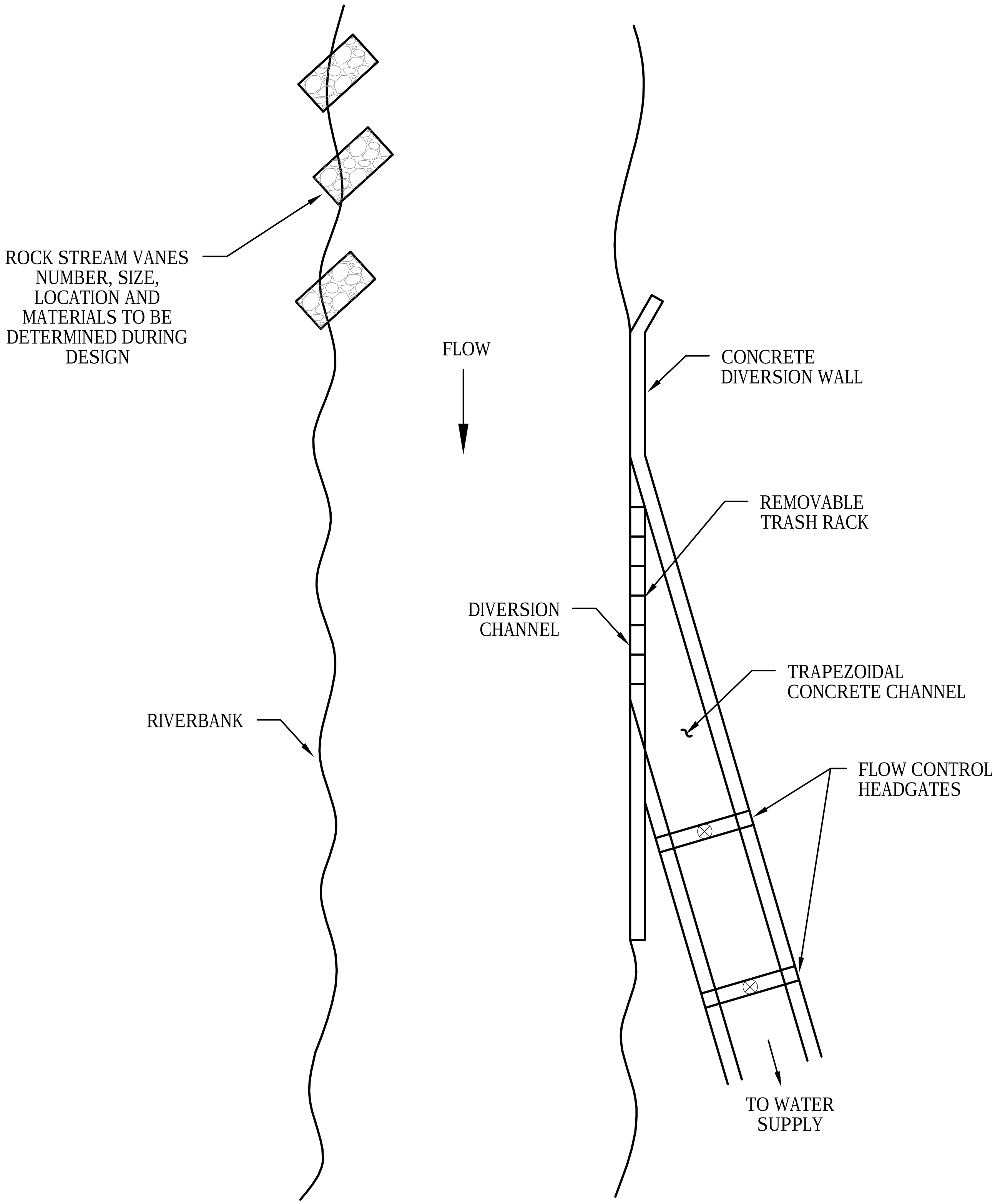
GILA RIVER DIVERSION, CONVEYANCE AND STORAGE ALTERNATIVES

FIGURE 9
TYPICAL CONCRETE DIVERSION STRUCTURE WITH CONTROL GATES

April, 2014



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GILA RIVER DIVERSION, CONVEYANCE AND STORAGE ALTERNATIVES

FIGURE 10
TYPICAL DIVERSION CHANNEL WITH STREAM VANES

P:\20140055\WR\Reports\Final\Figures\PER Figures\Figure10_DiversionWithVanes.dwg
Tue, 1-Apr-2014 - 7:53:am, Plotted by: KSAINTLOT

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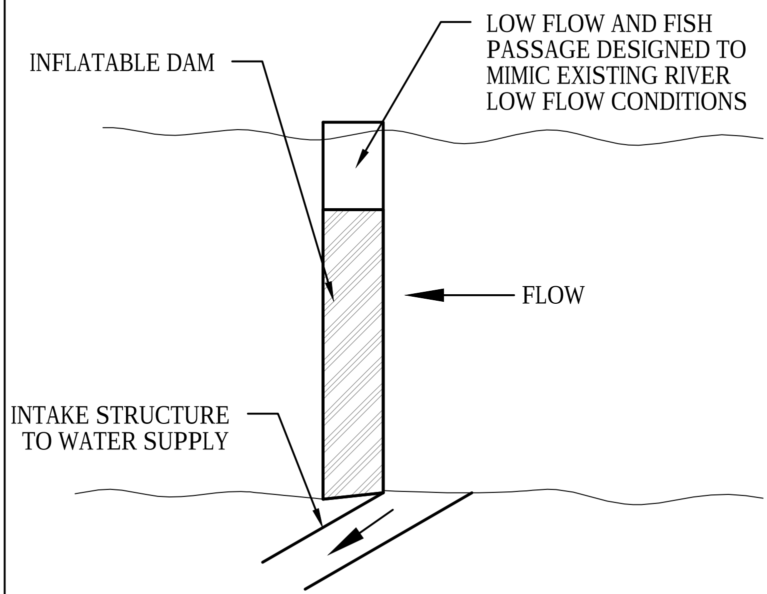
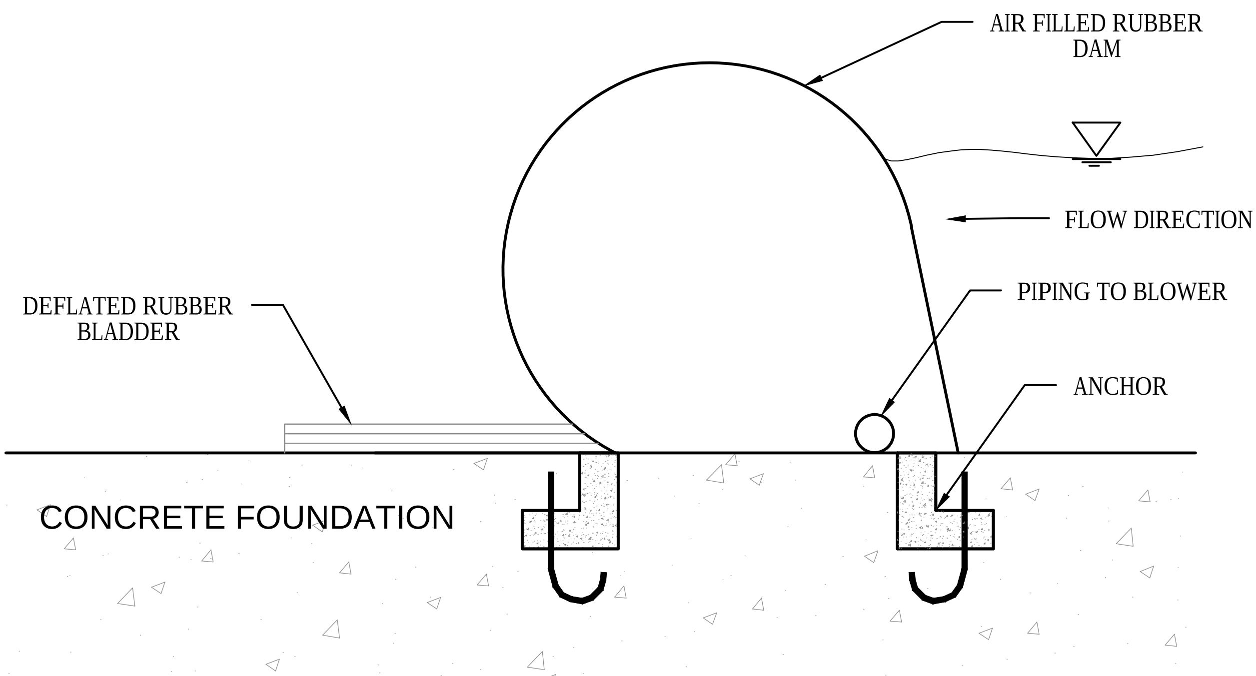


PLAN VIEW
NTS

ADJUSTABLE HEIGHT DAM
SOURCE:
WWW.ABCWUA.ORG



INFLATABLE DAM SECTION
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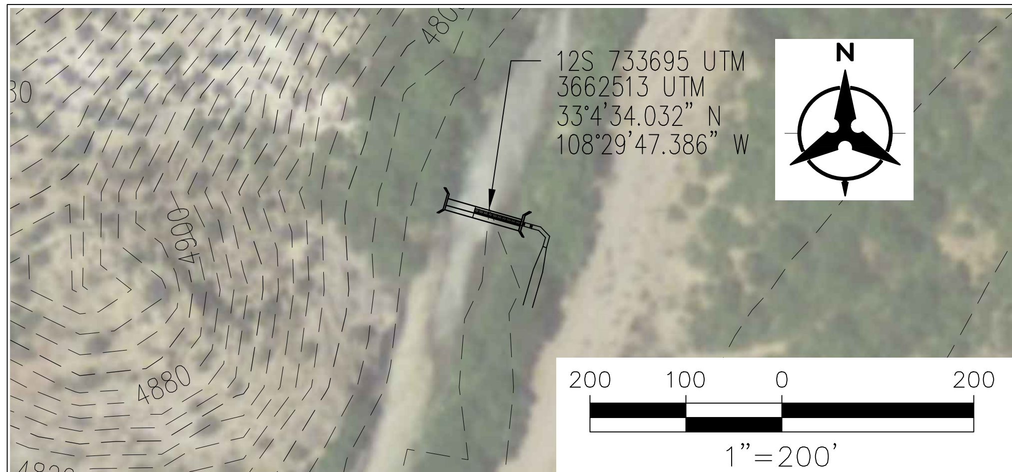
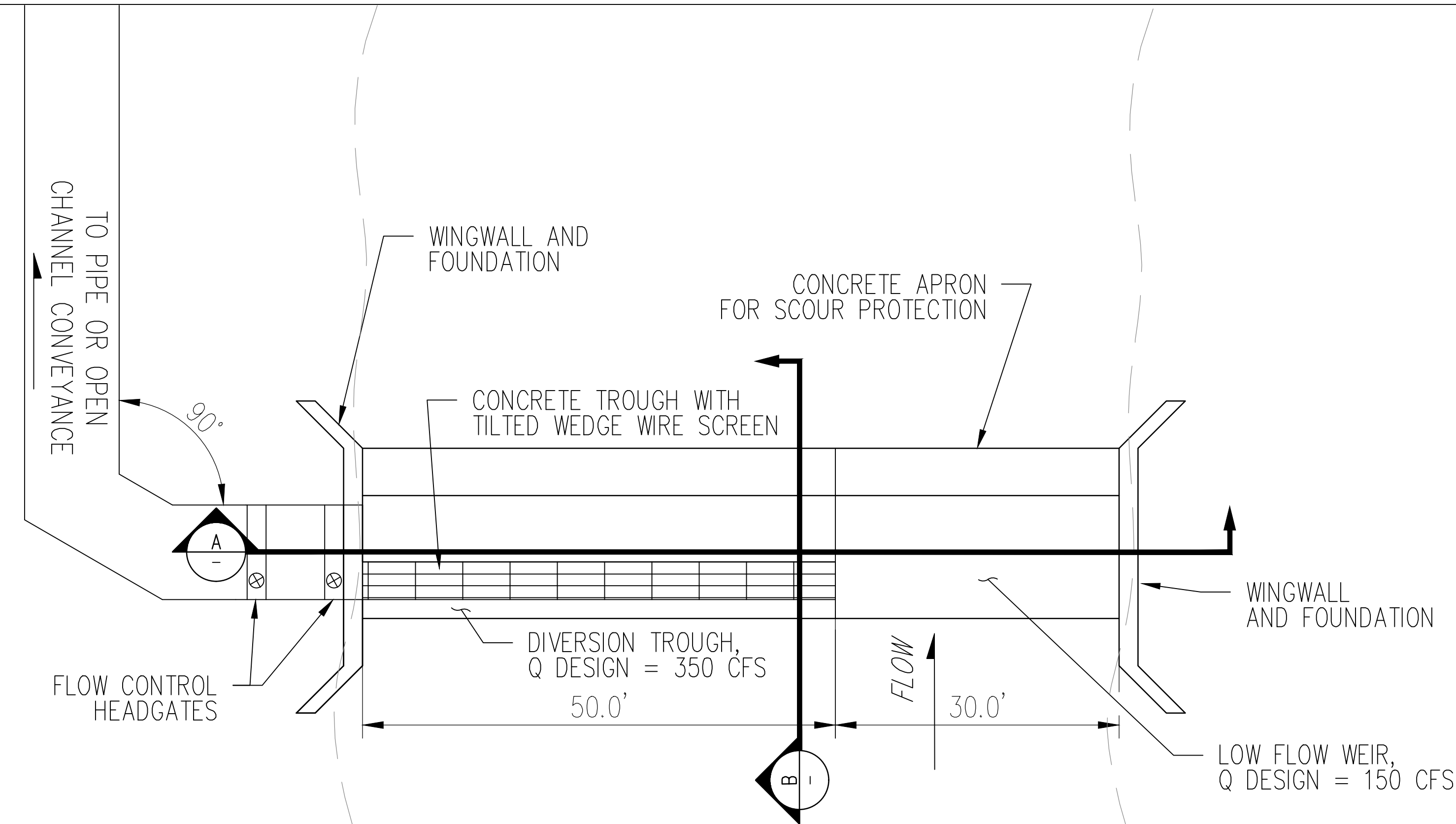
GILA RIVER DIVERSION, CONVEYANCE AND STORAGE ALTERNATIVES

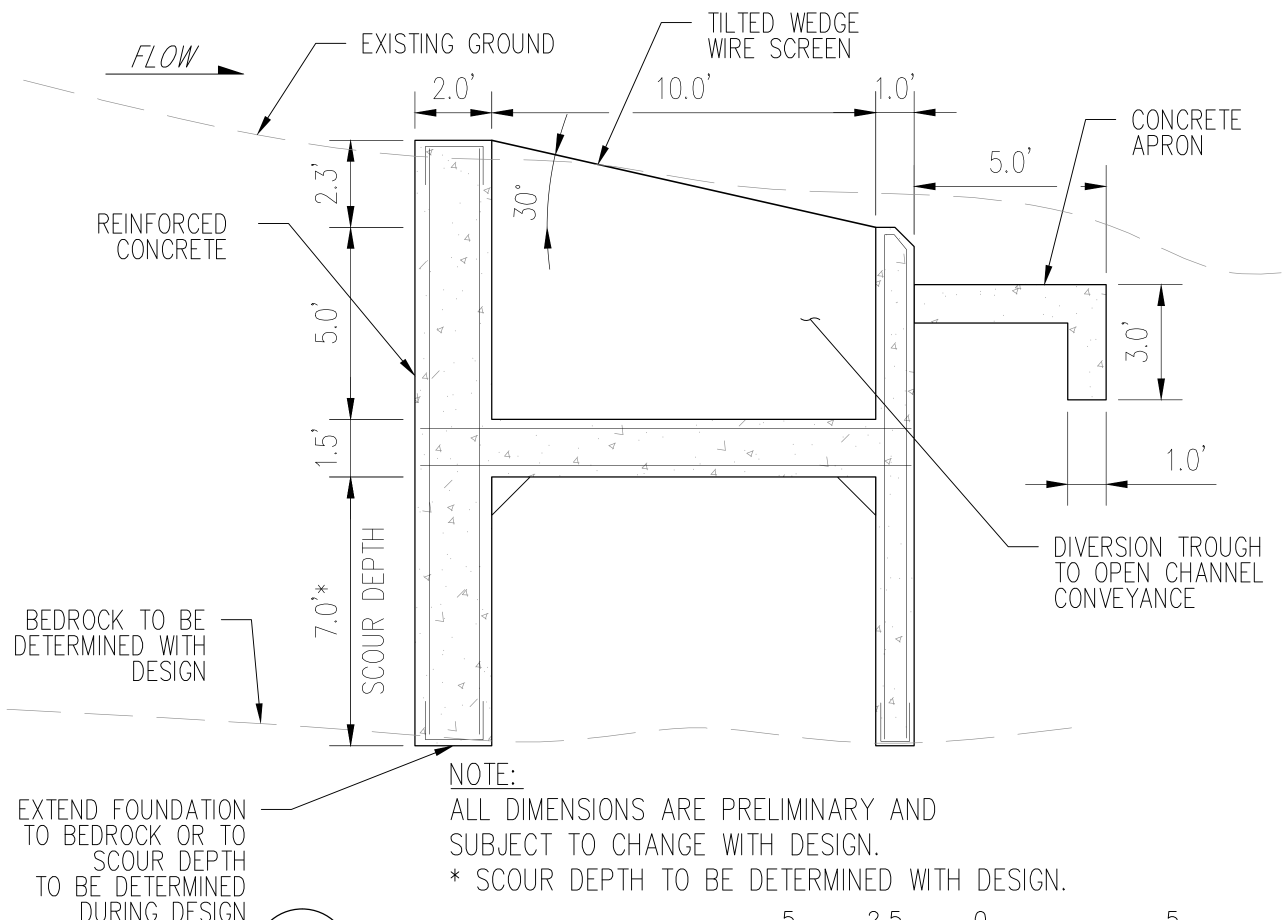
FIGURE 11
TYPICAL INFLATABLE DAM



1 PLAN VIEW
SCALE: 1" = 200'



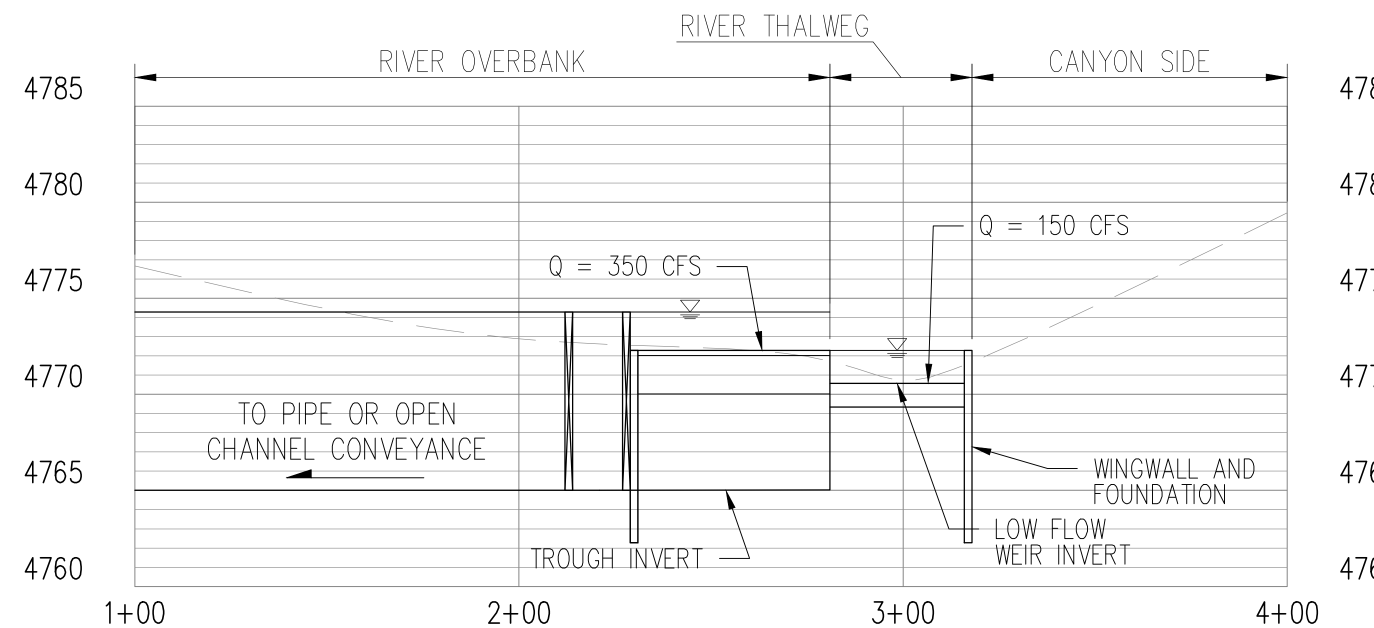
2 PLAN VIEW SCHEMATIC
SCALE: 1" = 20'



NOTE:
ALL DIMENSIONS ARE PRELIMINARY AND SUBJECT TO CHANGE WITH DESIGN.
* SCOUR DEPTH TO BE DETERMINED WITH DESIGN.

B DIVERSION TROUGH DETAIL
SCALE: 1" = 5'

5 2.5 0 5
1"=5'



A SECTION VIEW SCHEMATIC
SCALE: H:1" = 50'
V:1" = 5'

50 25 0 50
1"=50'

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FIGURE 12
DIVERSION 1 CONCEPTUAL SITE PLAN

P:\20140055\WR\Reports\Final\Figures\PER Figures\Figure12_Category 2 Site 1 (4770).dwg
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W E
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0 200 400
Feet

GILA RIVER DIVERSION, CONVEYANCE AND STORAGE ALTERNATIVES

FIGURE 13
ALTERNATE DIVERSION 1 LOCATION

P:\20140055\WR\Reports\Final\Figures\PER Figures\Figure13_AlternateD1Location.mxd
Author: ksaintlot

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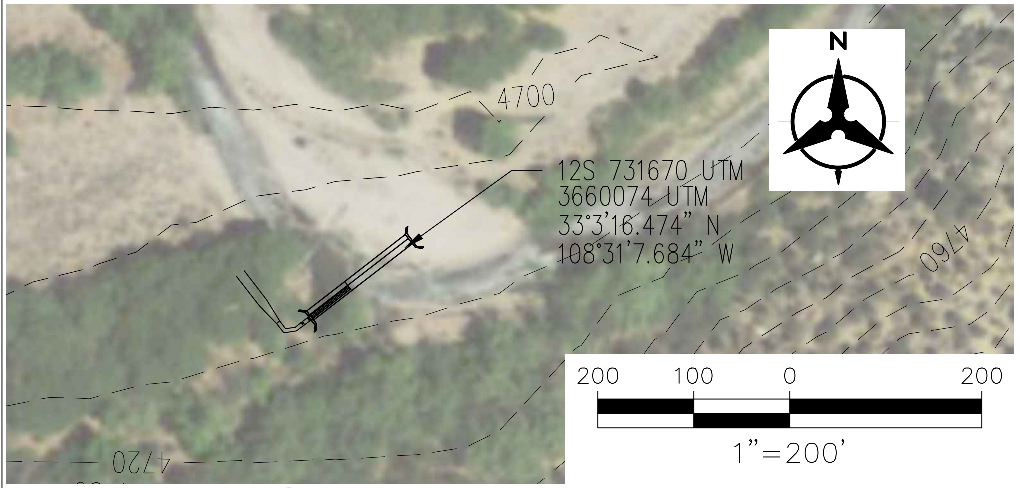
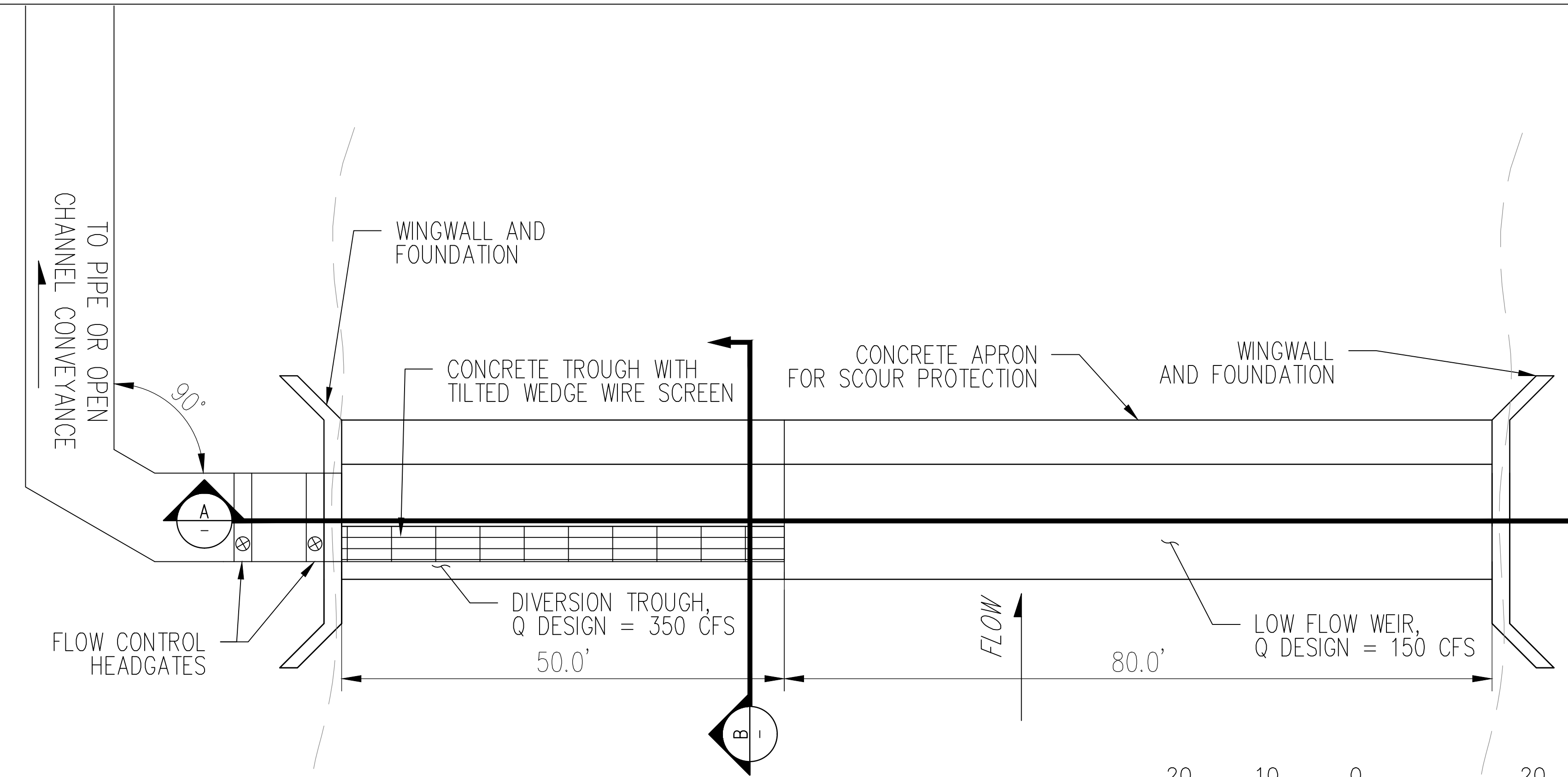
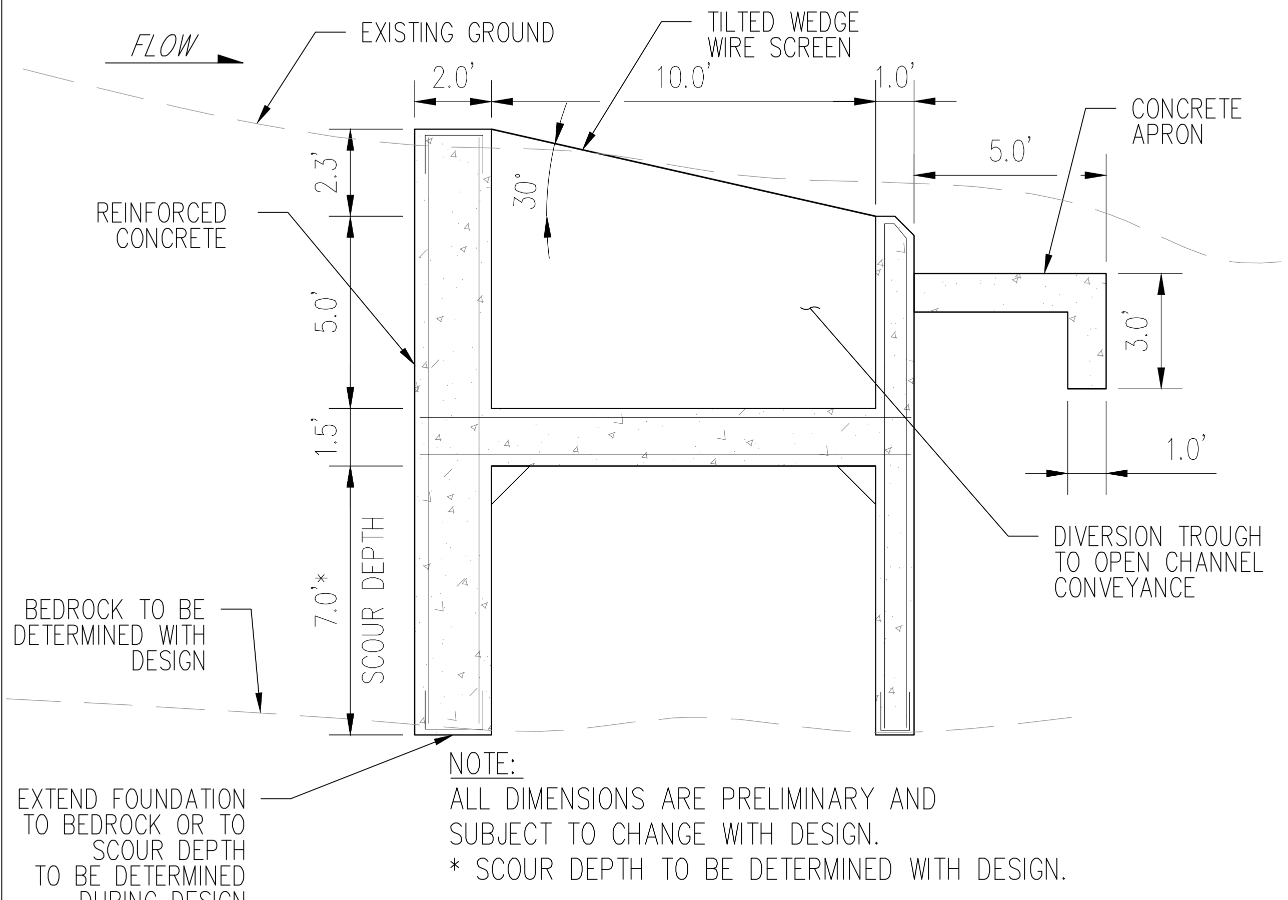


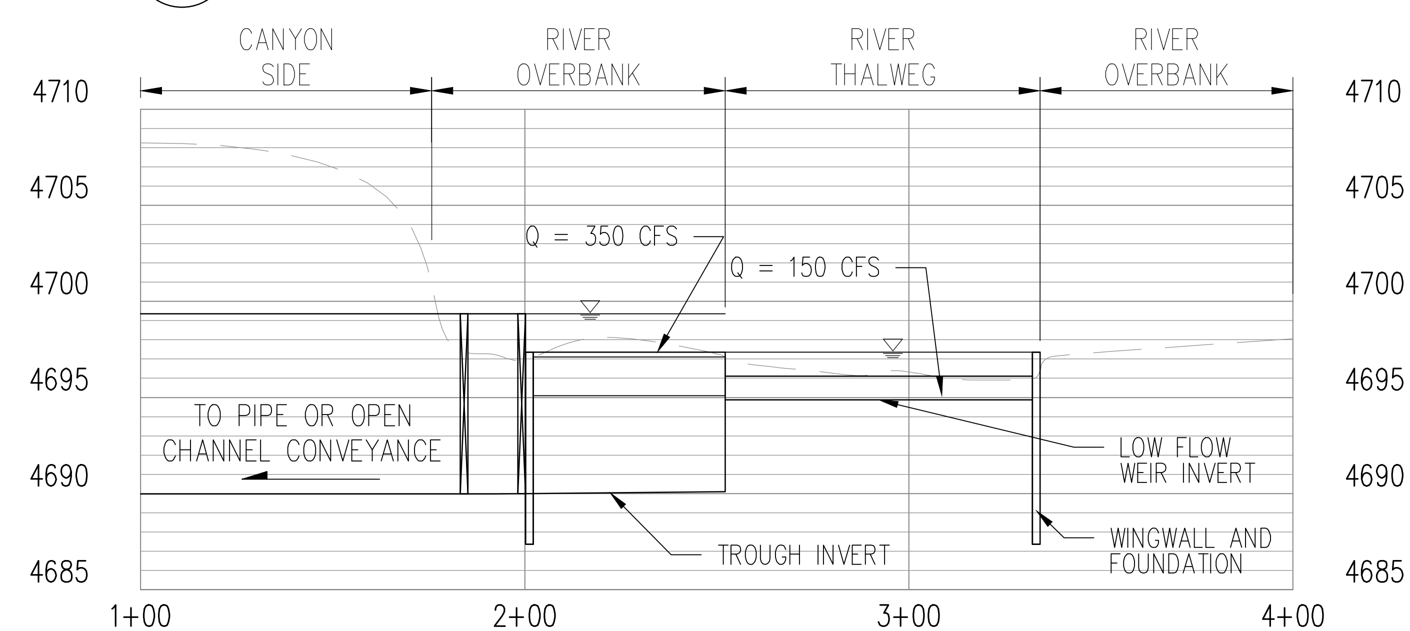
1 PLAN VIEW
SCALE: 1" = 200'



2 PLAN VIEW SCHEMATIC
SCALE: 1" = 20'



B DIVERSION TROUGH DETAIL
SCALE: 1" = 5'



A SECTION VIEW SCHEMATIC
SCALE: H:1" = 50'
V:1" = 5'

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FIGURE 14
DIVERSION 2 CONCEPTUAL SITE PLAN

P:\20140055\WR\Reports\Final\Figures\PER Figures\Figure14_Category 2 Site 2 (4695).dwg
April 01, 2014 – 8:51am
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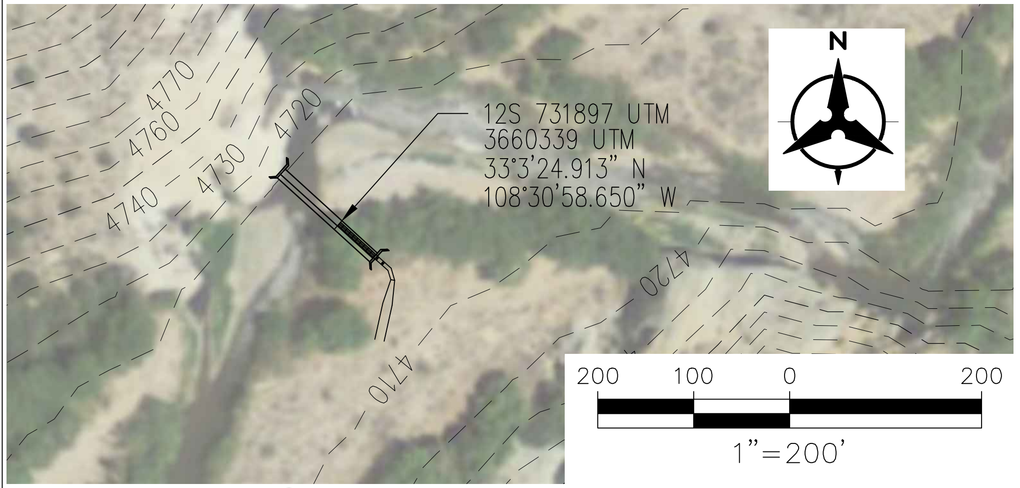
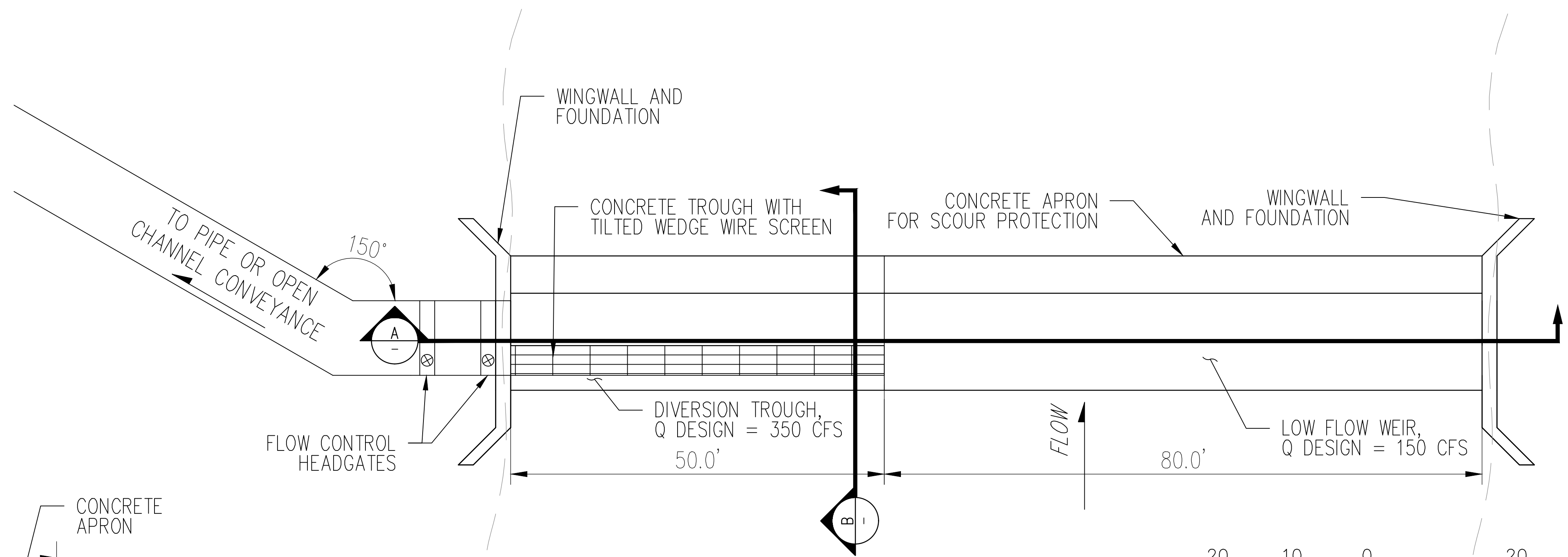
FIGURE 15
ALTERNATE DIVERSION 2 LOCATION

P:\20140055\WR\Reports\Final\Figures\PER Figures\Figure15_AlternateD2Location.mxd
Author: ksaintlot

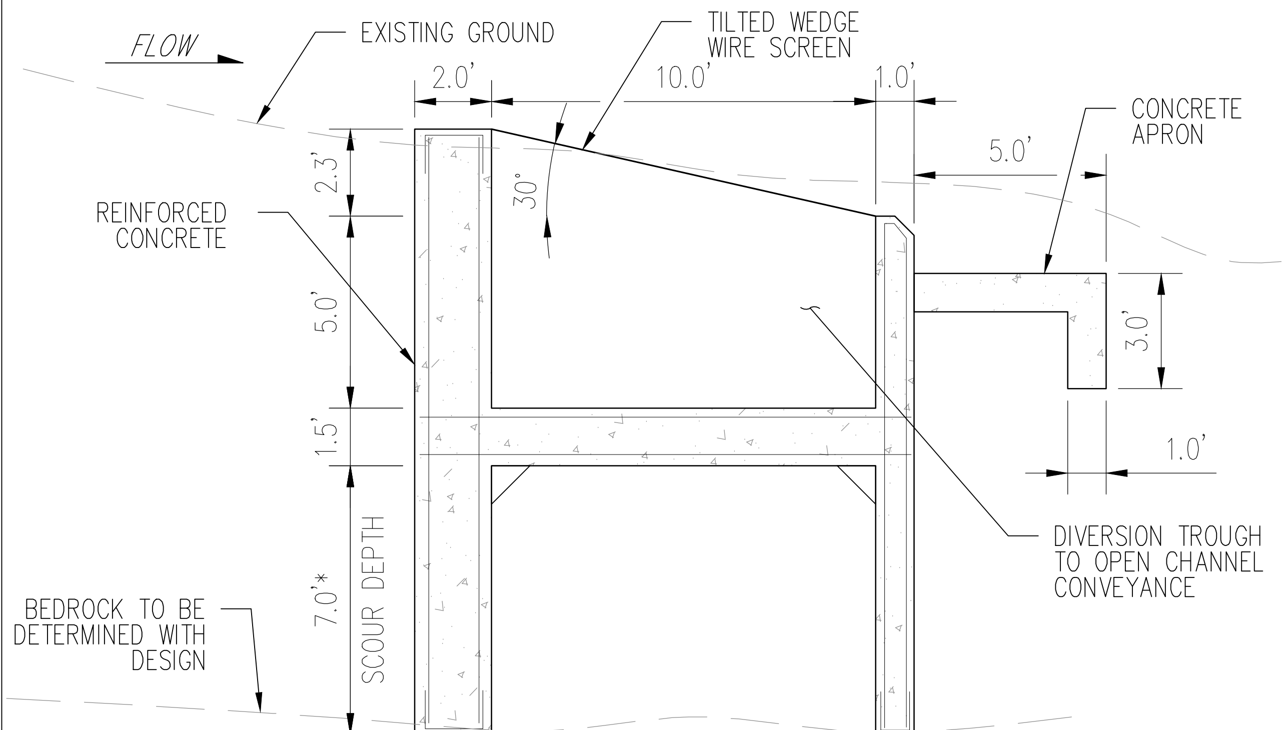
April, 2014



1 PLAN VIEW
SCALE: 1" = 200'



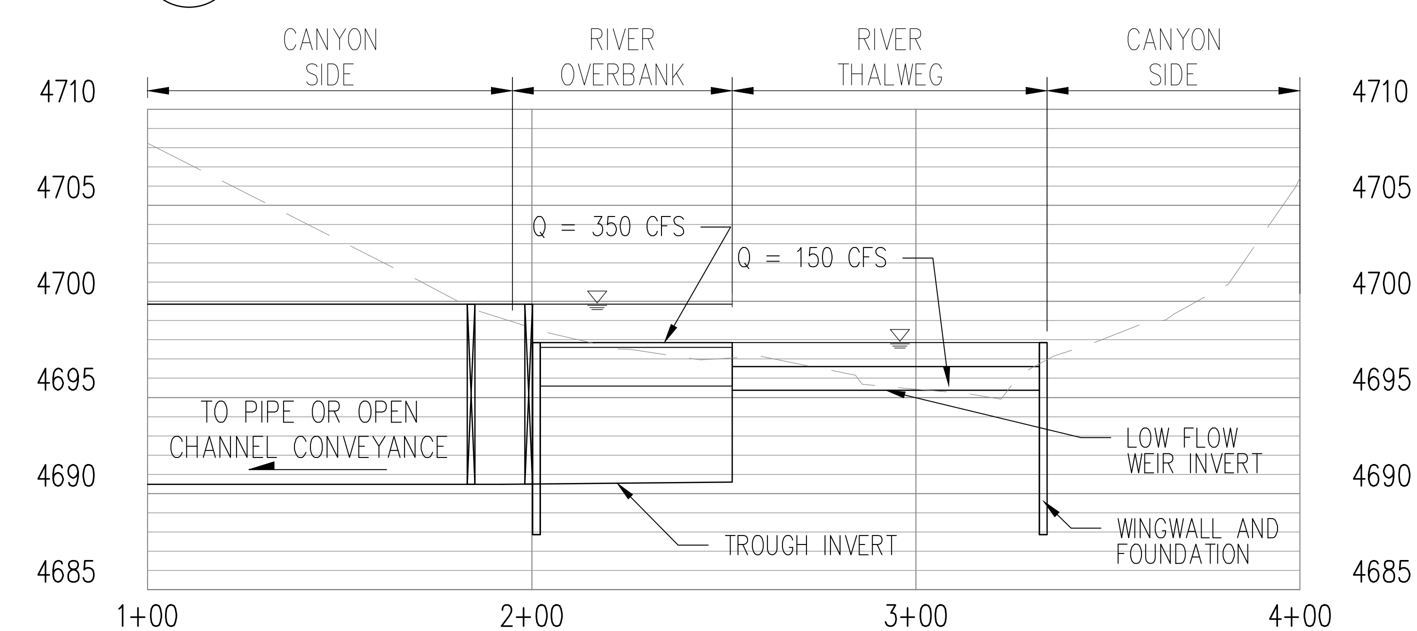
2 PLAN VIEW SCHEMATIC
SCALE: 1" = 20'



NOTE:
ALL DIMENSIONS ARE PRELIMINARY AND SUBJECT TO CHANGE WITH DESIGN.
* SCOUR DEPTH TO BE DETERMINED WITH DESIGN.

B DIVERSION TROUGH DETAIL
SCALE: 1" = 5'

5 2.5 0 5
1"=5'



A SECTION VIEW SCHEMATIC
SCALE: H:1" = 50'
V:1" = 5'

50 25 0 50
1"=50'

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FIGURE 16
ALTERNATE LOCATION DIVERSION 2 CONCEPTUAL SITE PLAN

P:\20140055\WR\Reports\Final\Figures\PER Figures\Figure16_Category 2 Site 2 Alternative (4695).dwg
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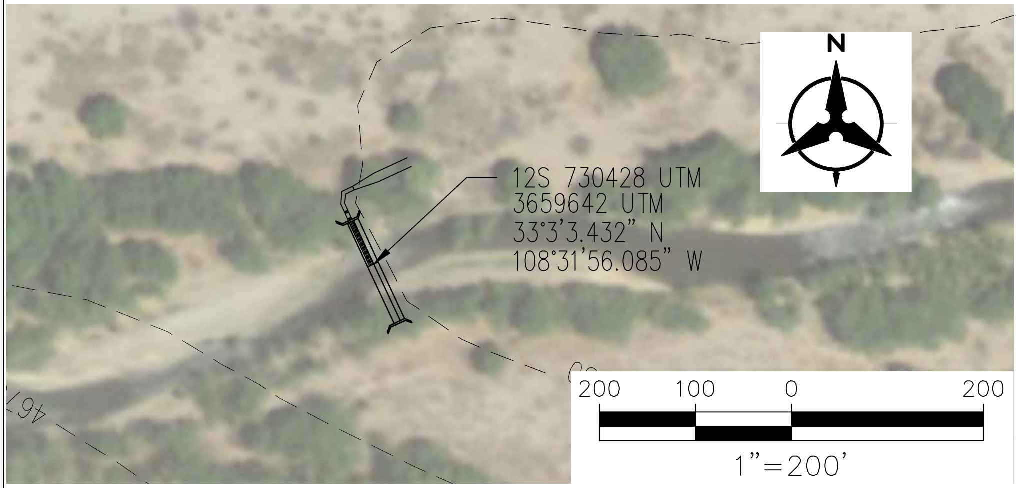
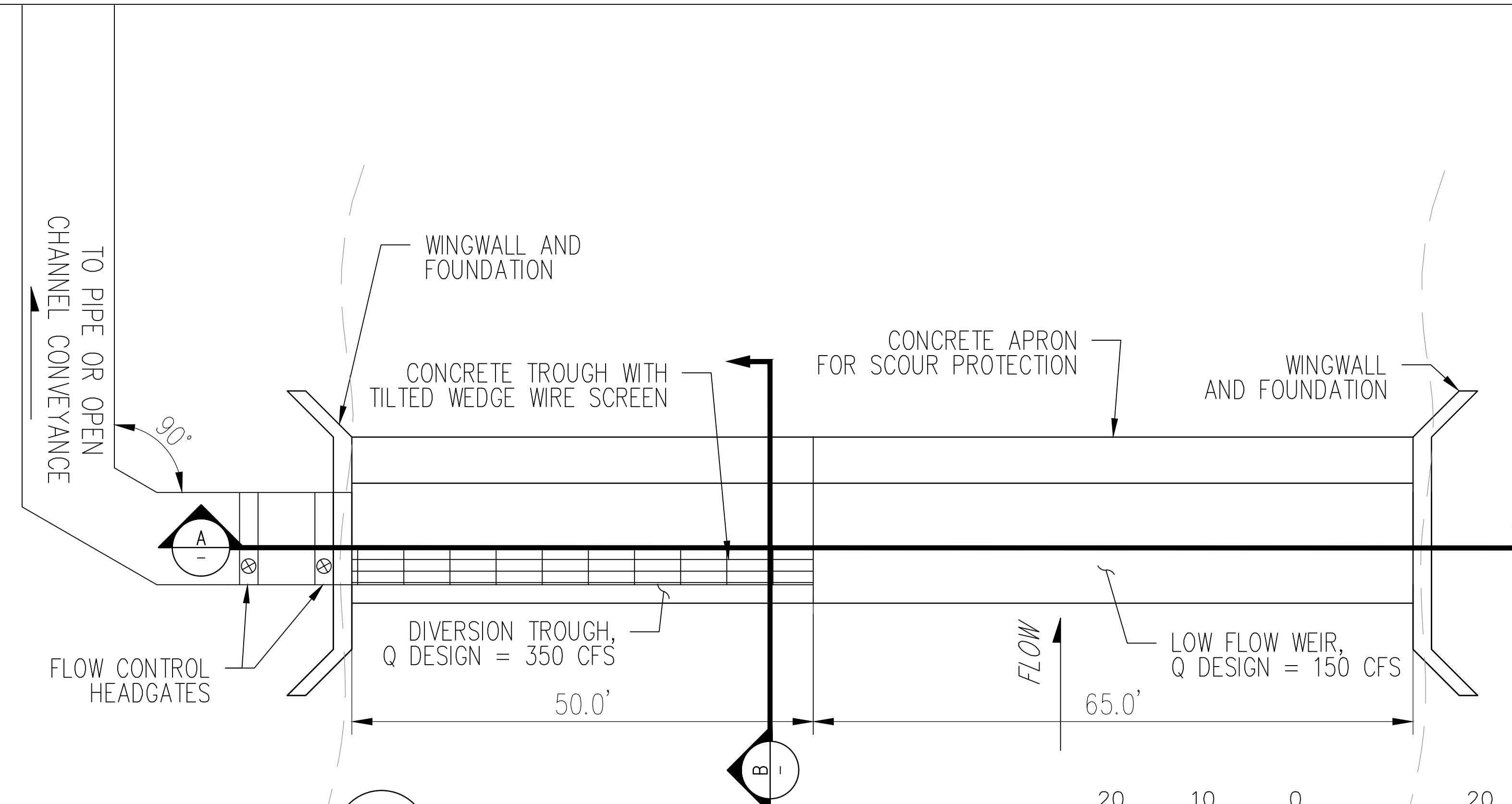
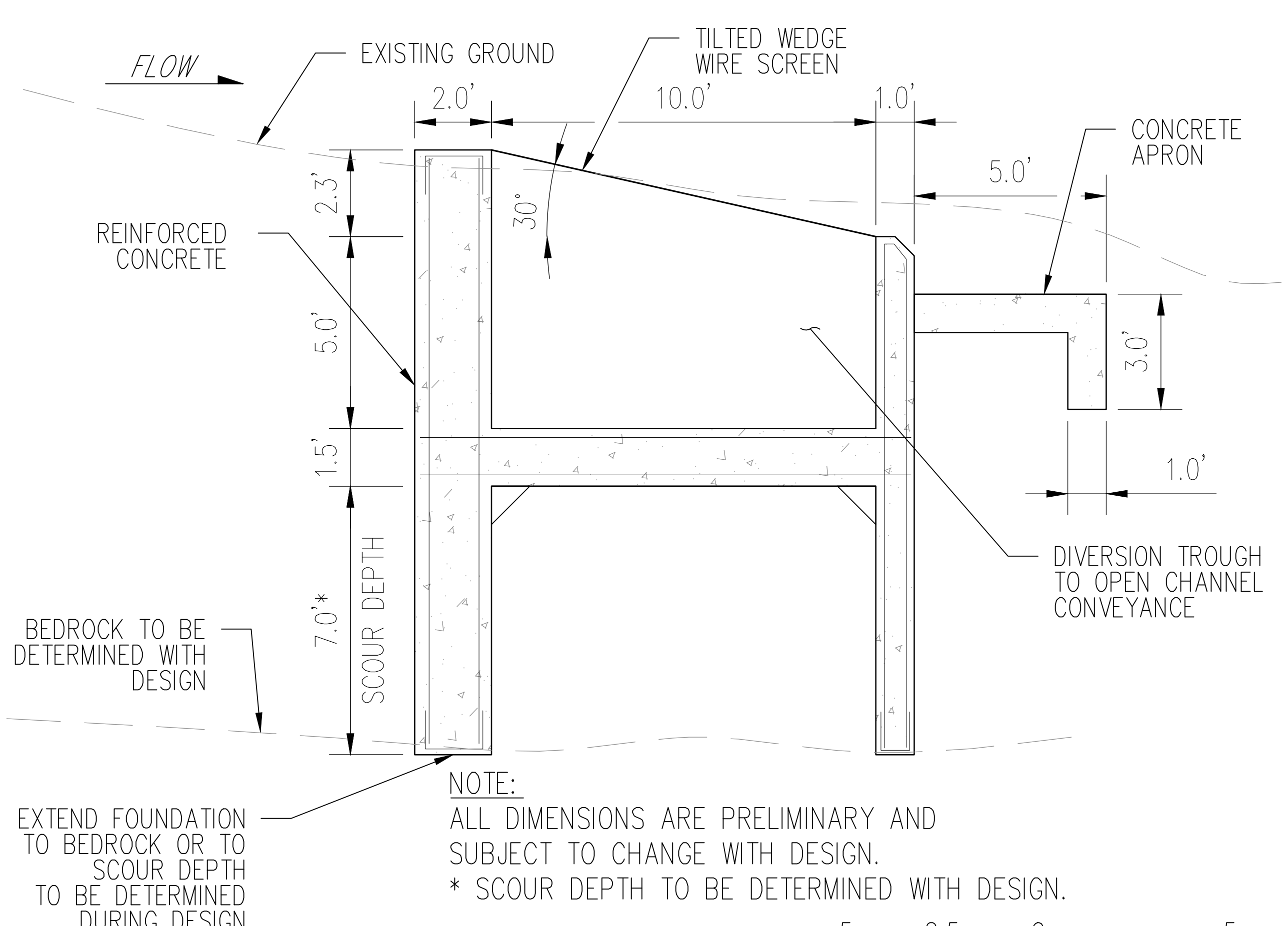


1 PLAN VIEW
SCALE: 1" = 200'



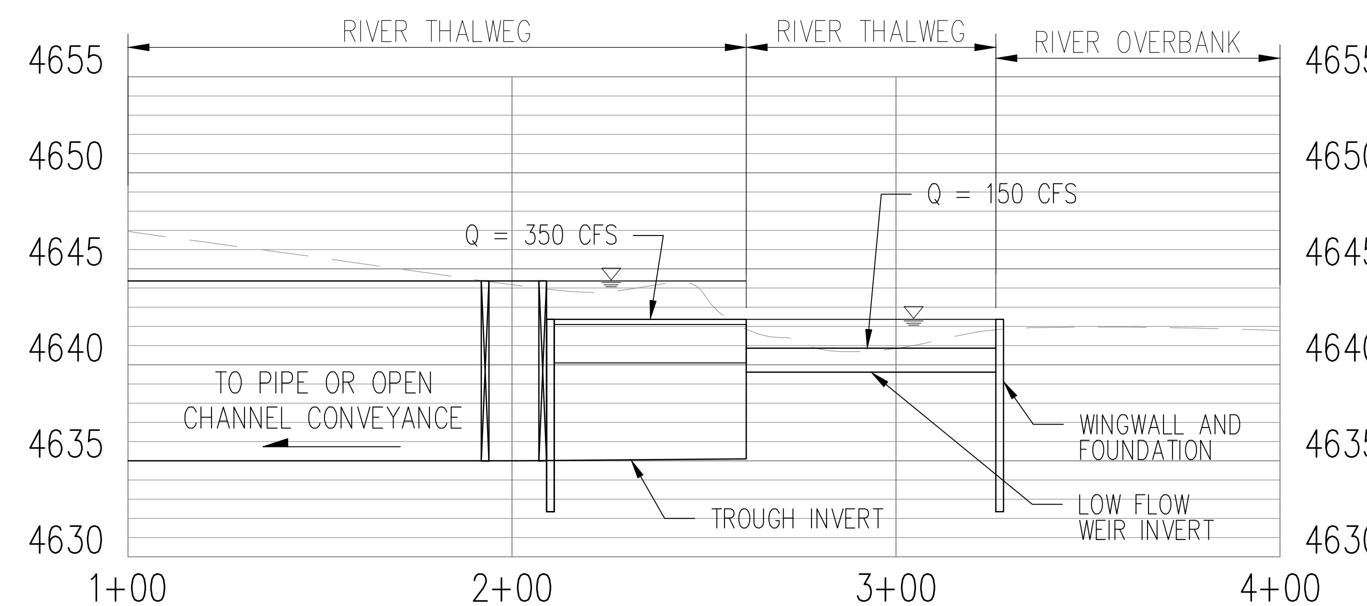
2 PLAN VIEW SCHEMATIC
SCALE: 1" = 20'



NOTE:
ALL DIMENSIONS ARE PRELIMINARY AND SUBJECT TO CHANGE WITH DESIGN.
* SCOUR DEPTH TO BE DETERMINED WITH DESIGN.

B DIVERSION TROUGH DETAIL
SCALE: 1" = 5'

5 2.5 0 5
1"=5'



A SECTION VIEW SCHEMATIC
SCALE: H:1" = 50'
V:1" = 5'

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FIGURE 17
DIVERSION 3 CONCEPTUAL SITE PLAN

P:\20140055\WR\Reports\Final\Figures\PER Figures\Figure17_Category 2 Site 3 (4640).dwg
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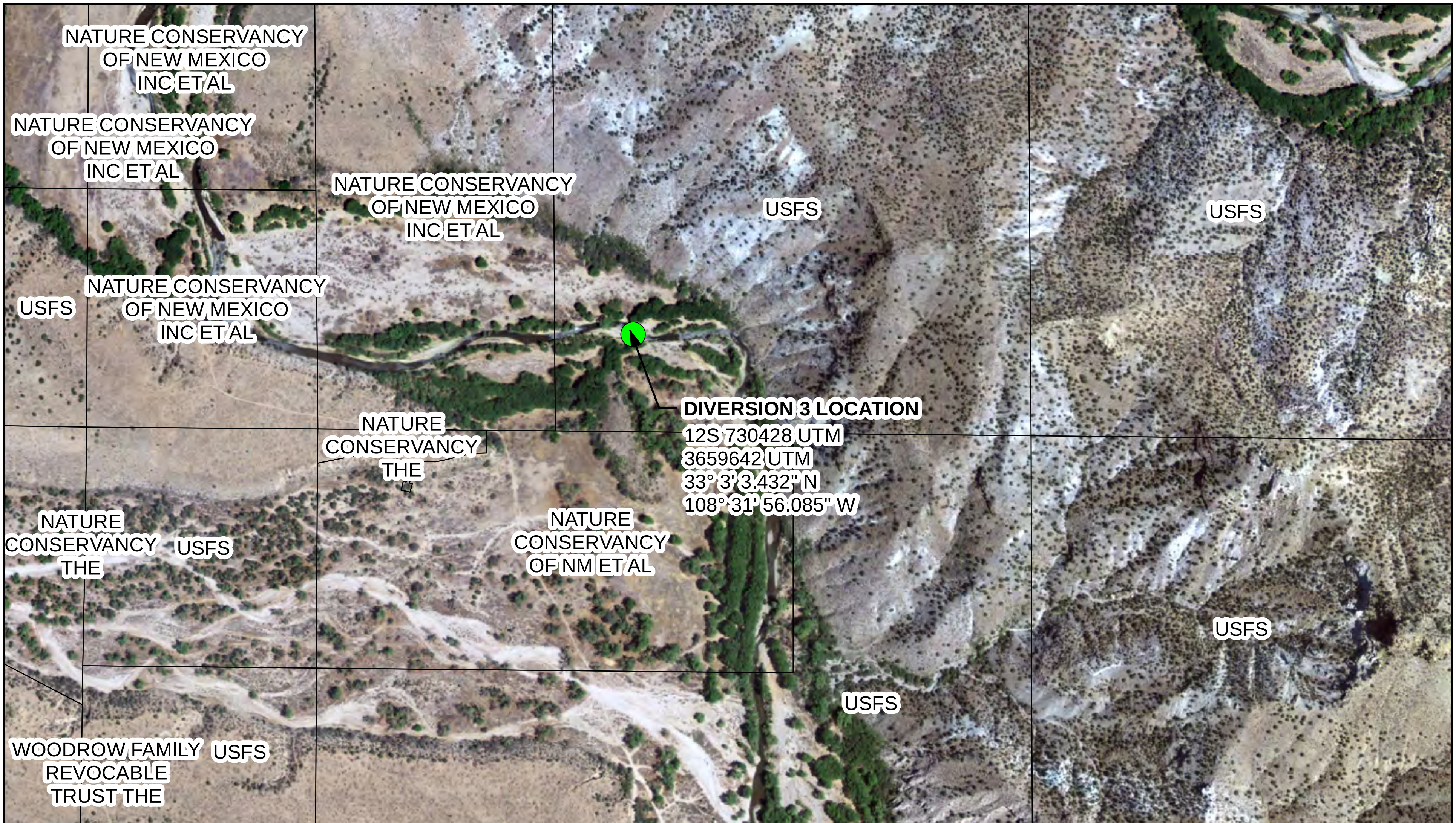

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FIGURE 18
DIVERSION 3 LOCATION

P:\20140055\WR\Reports\Final\Figures\PER Figures\Figure18_D3Location.mxd
Author: ksaintlot

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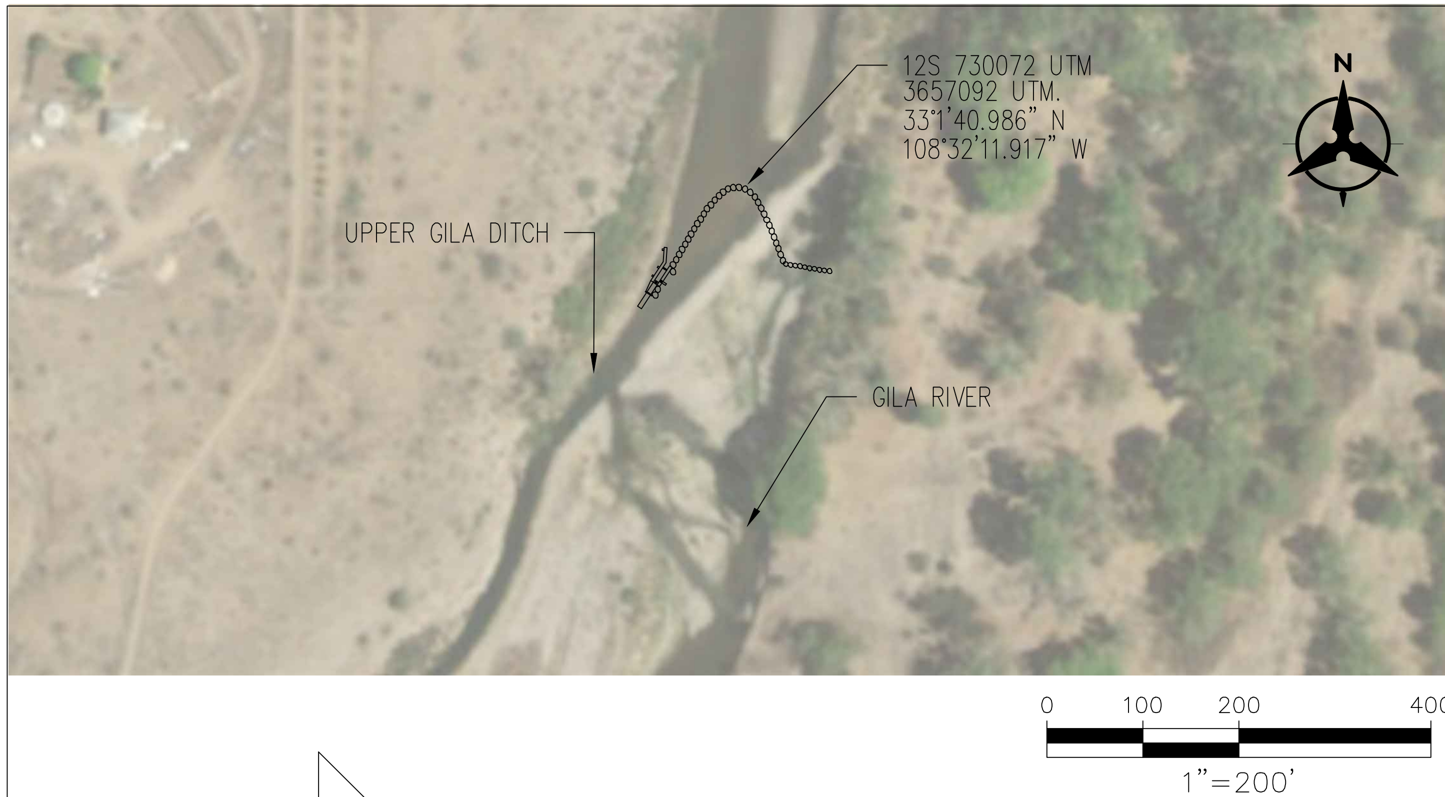
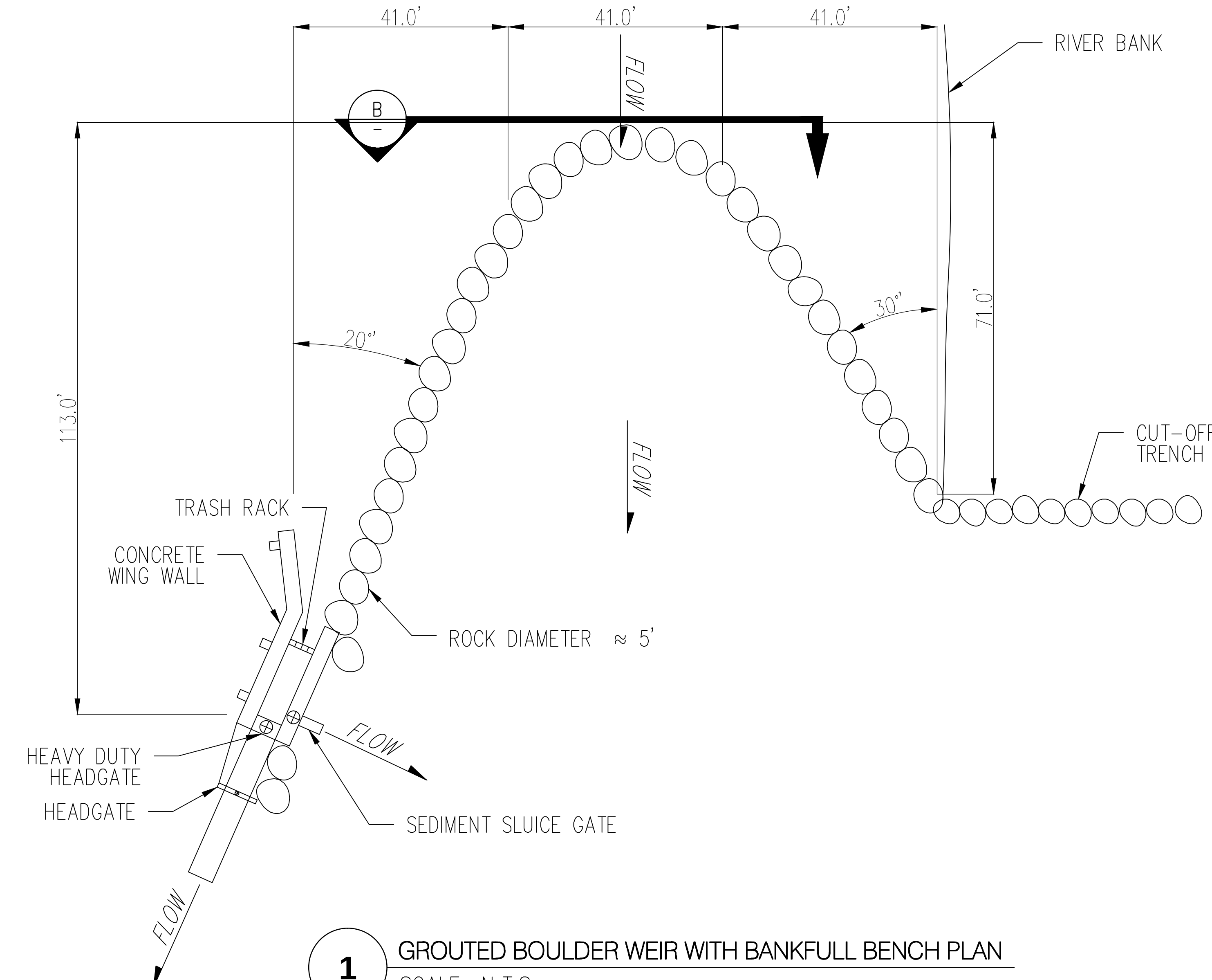
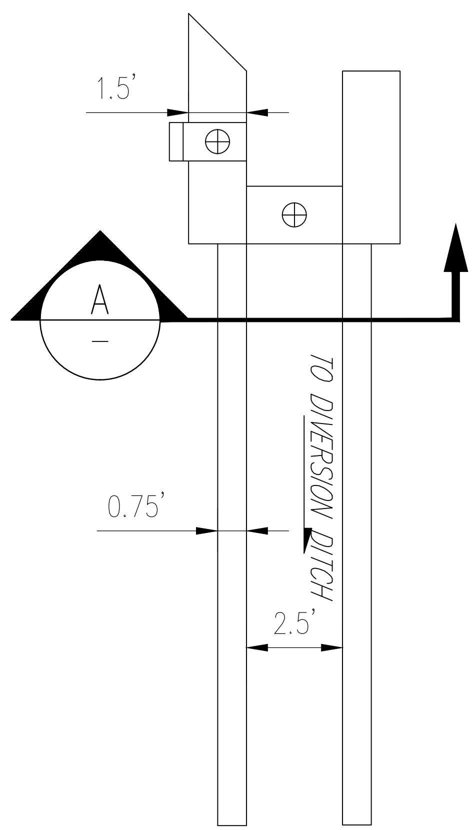
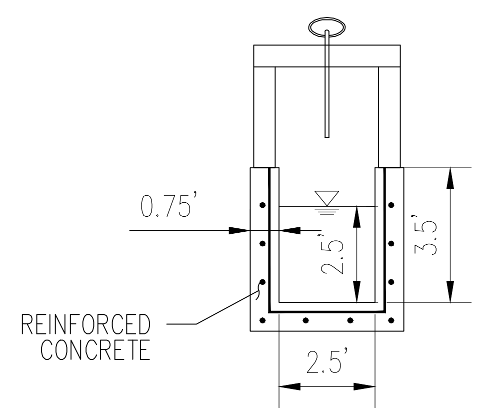




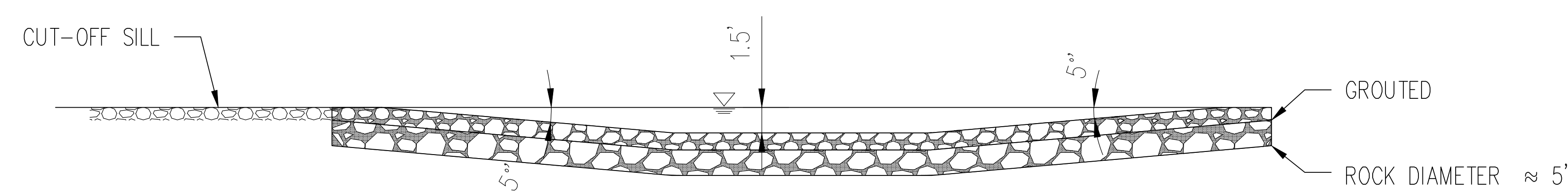
1 GROUTED BOULDER WEIR WITH BANKFULL BENCH PLAN
SCALE: N.T.S.



2 SCREW DRIVE SLUICE GATE
SCALE: 1" = 5'



A SCREW DRIVE SLUICE GATE
SCALE: 1" = 5'



B GROUTED BOULDER WEIR WITH BANKFULL BENCH PLAN SECTION
SCALE: 1" = 10'

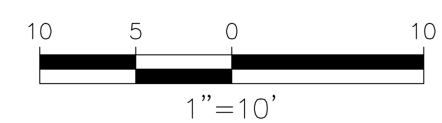


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FIGURE 19
UPPER GILA DIVERSION CONCEPTUAL SITE PLAN

P:\20140055\WR\Reports\Final\Figures\PER Figures\Figure19_Category 1 Upper Gila.dwg
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Layout

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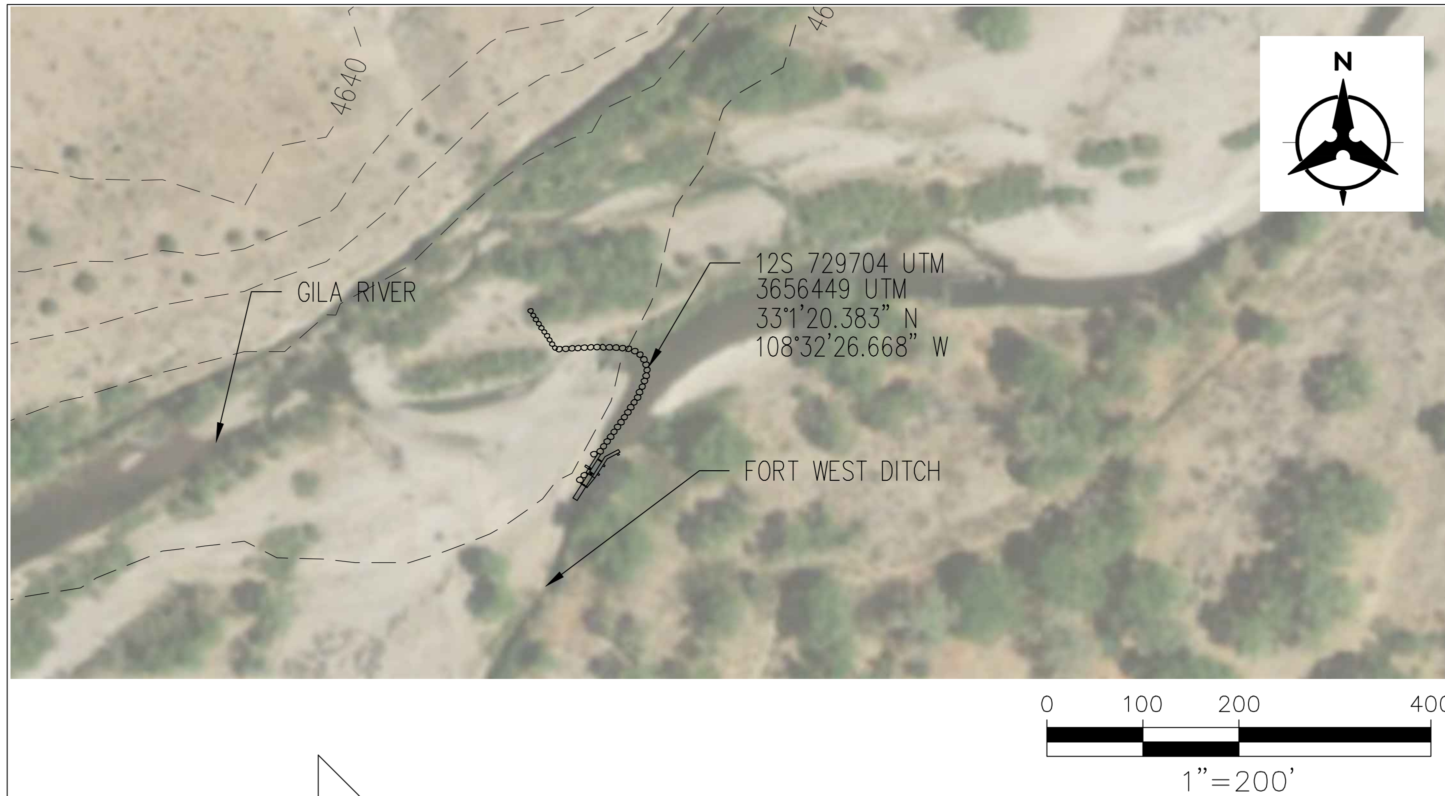
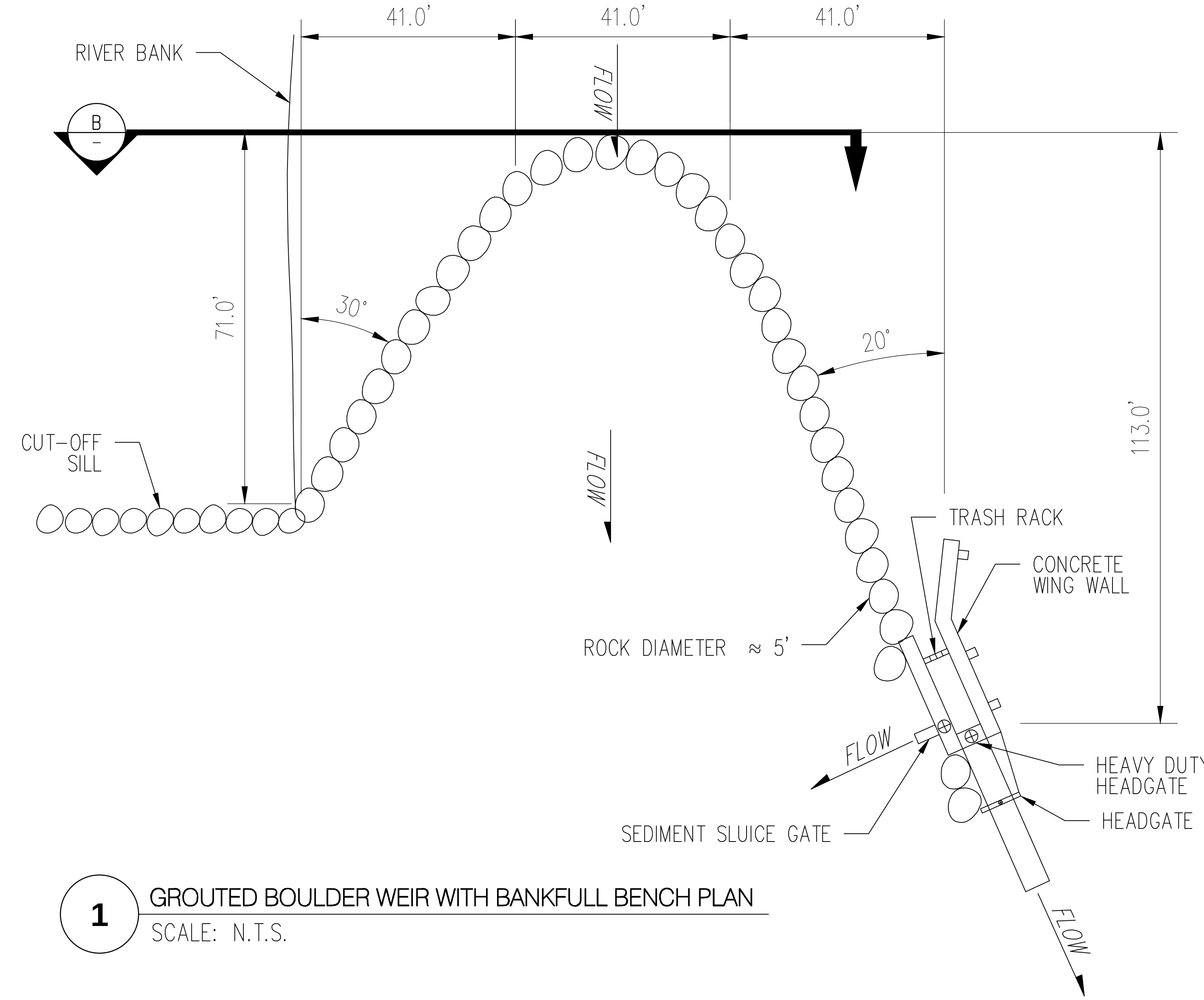
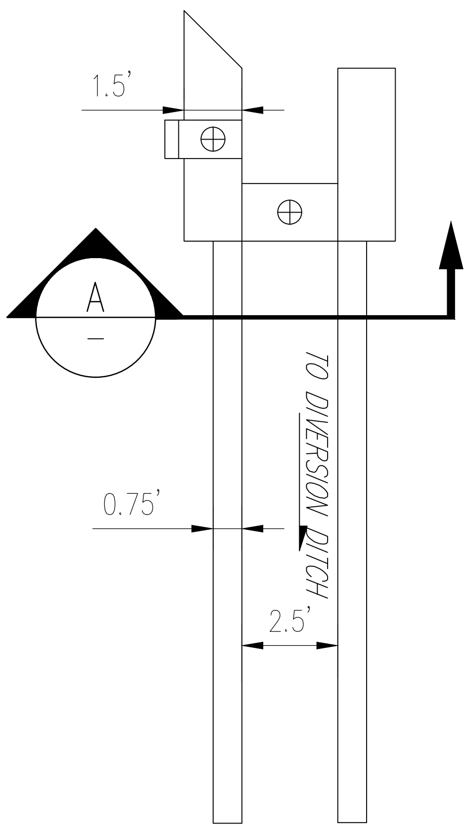
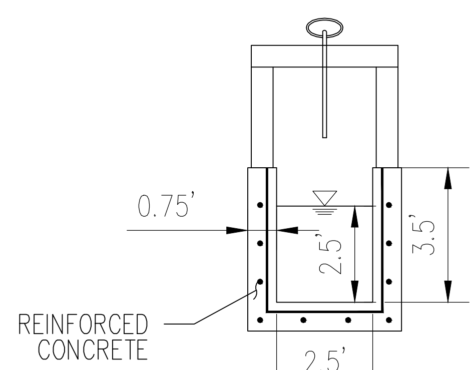




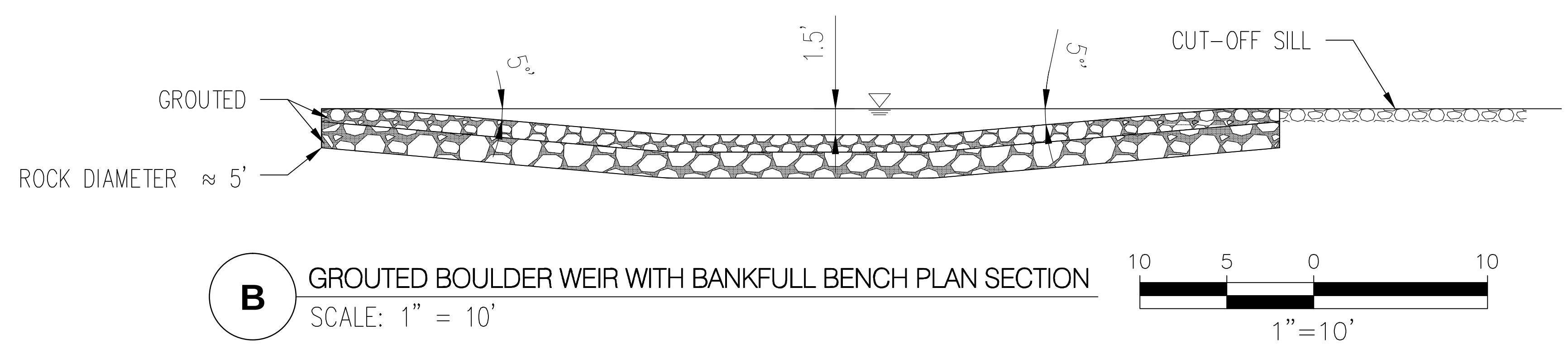
1 GROUTED BOULDER WEIR WITH BANKFULL BENCH PLAN
SCALE: N.T.S.



2 SCREW DRIVE SLUICE GATE
SCALE: 1" = 5'



A SCREW DRIVE SLUICE GATE
SCALE: 1" = 5'



B GROUTED BOULDER WEIR WITH BANKFULL BENCH PLAN SECTION
SCALE: 1" = 10'

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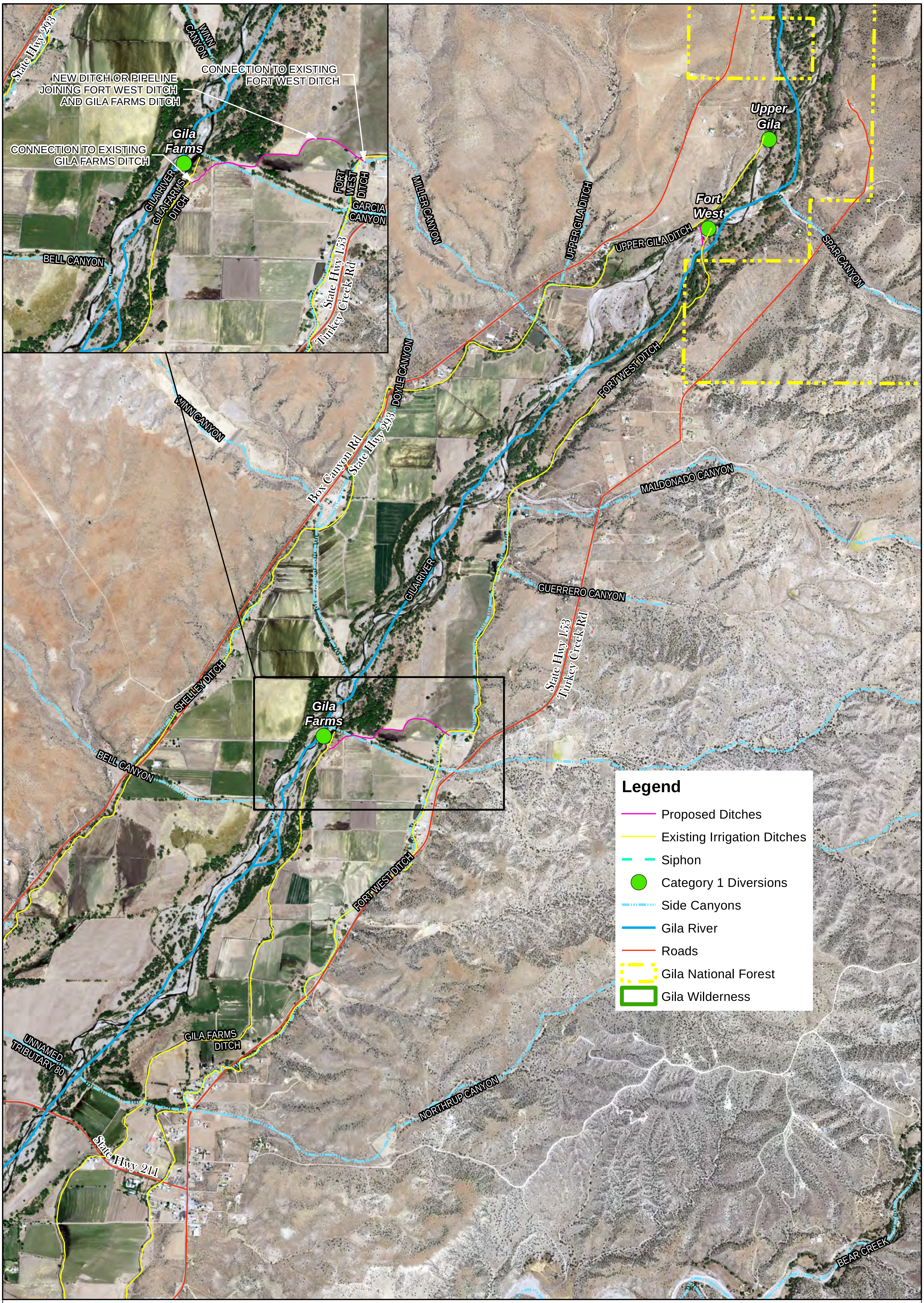
GILA RIVER DIVERSION, CONVEYANCE AND STORAGE ALTERNATIVES

FIGURE 20
FORT WEST DIVERSION
CONCEPTUAL SITE PLAN

P:\20140055\WR\Reports\Final\Figures\PER Figures\Figure20_Category 1 Fort West.dwg
April 01, 2014 - 9:02am
Layout

April, 2014



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N
W E
S

0 2,000 4,000
Feet

GILA RIVER DIVERSION, CONVEYANCE AND STORAGE ALTERNATIVES

FIGURE 21
COMBINED DIVERSION FOR UPPER GILA/FORT WEST/GILA FARMS

P:\20140055\WR\Reports\Final\Figures\PER Figures\Figure21_CombinedDiversionforUpperGilla.mxd
April, 2014
Author: ksaintlot

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Legend

- Category 1 Diversions
- Existing Irrigation Ditches
- Gila River

N W E S

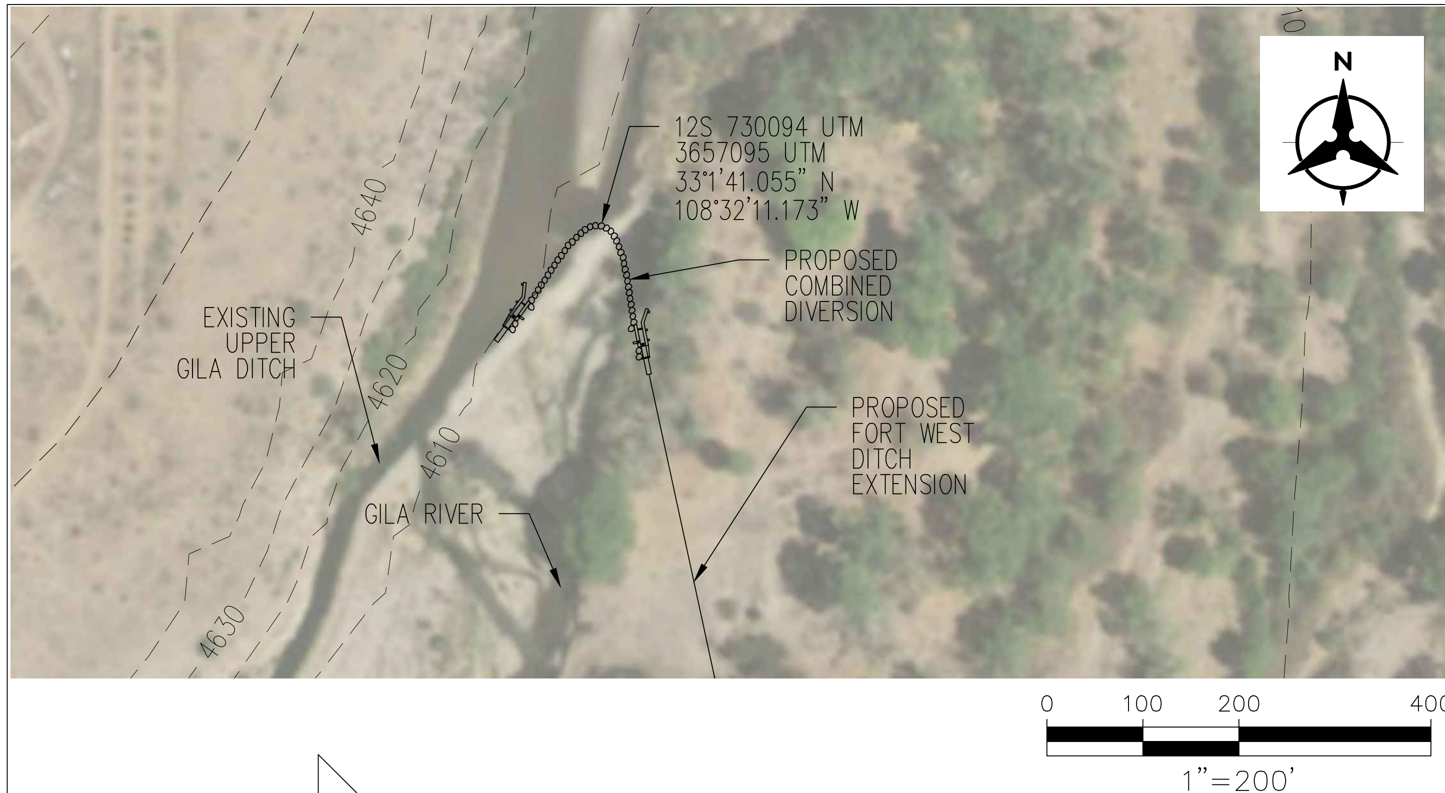
0 200 400 Feet

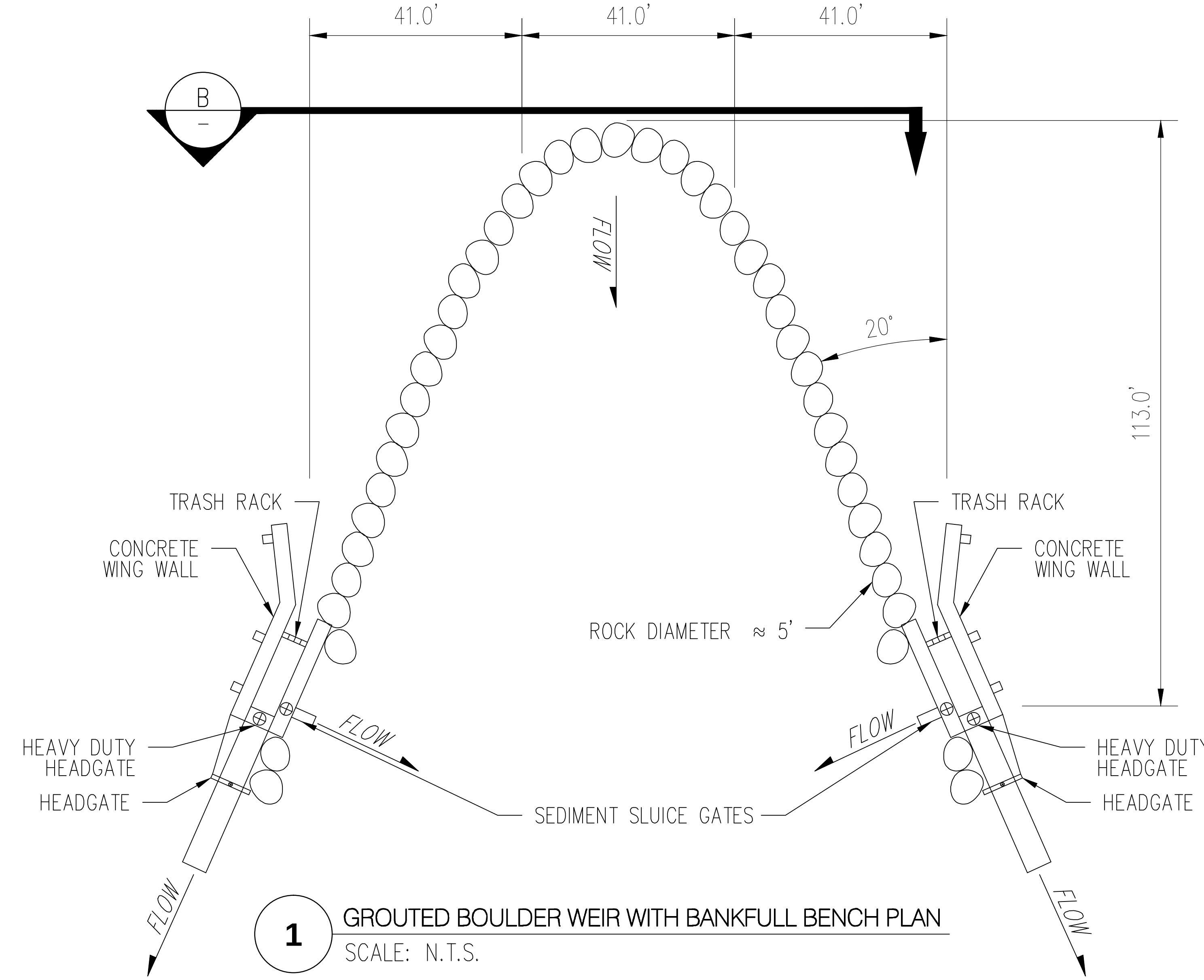
GILA RIVER DIVERSION, CONVEYANCE AND STORAGE ALTERNATIVES

FIGURE 22
GILA FARMS INFILTRATION GALLERY PLAN VIEW

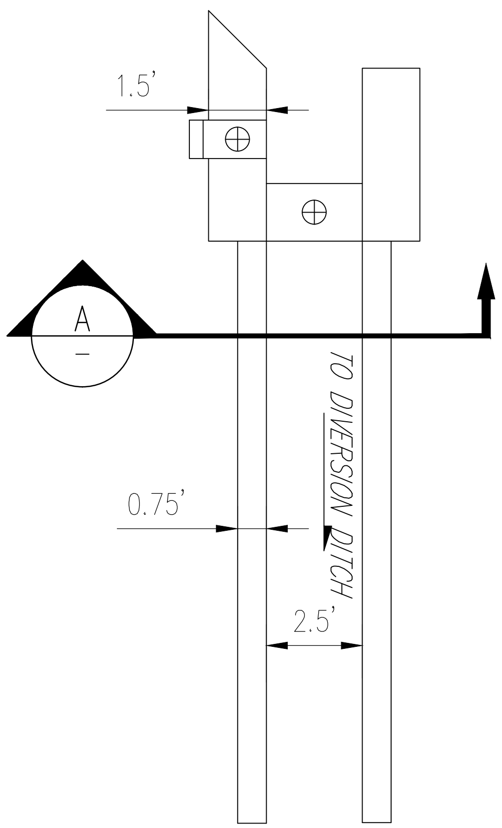
P:\20140055\WR\Reports\Final\Figures\PER Figures\Figure22_GilaFarmInfiltrationGallery.mxd
Author: ksaintlot

April, 2014

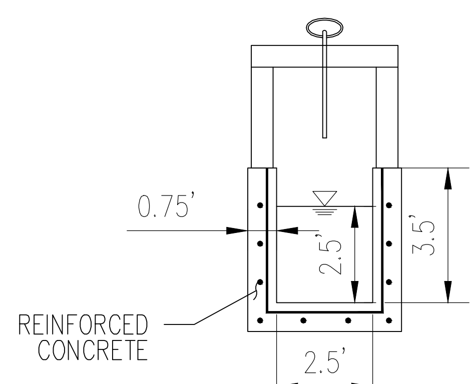




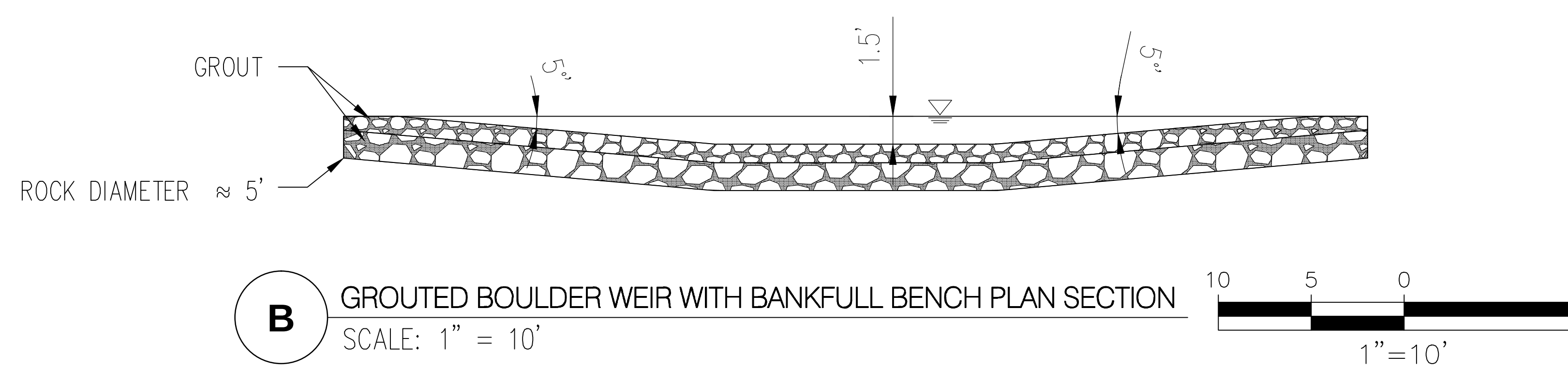
1 GROUTED BOULDER WEIR WITH BANKFULL BENCH PLAN
SCALE: N.T.S.



2 SCREW DRIVE SLUICE GATE
SCALE: 1" = 5'



A SCREW DRIVE SLUICE GATE
SCALE: 1" = 5'



B GROUTED BOULDER WEIR WITH BANKFULL BENCH PLAN SECTION
SCALE: 1" = 10'

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GILA RIVER DIVERSION, CONVEYANCE AND STORAGE ALTERNATIVES

FIGURE 23
UPPER GILA/FORT WEST COMBINED DIVERSION CONCEPTUAL SITE PLAN

P:\20140055\WR\Reports\Final\Figures\PER Figures\Figure23_Category 1 Combined Upper Gila and Fort West.dwg
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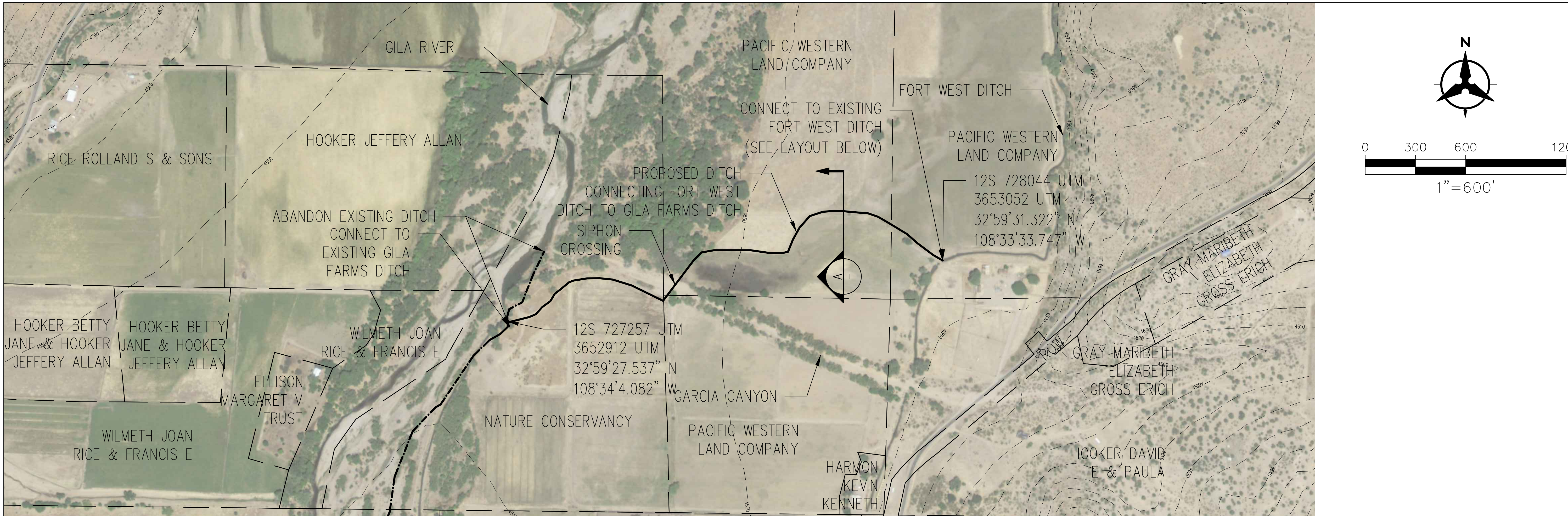
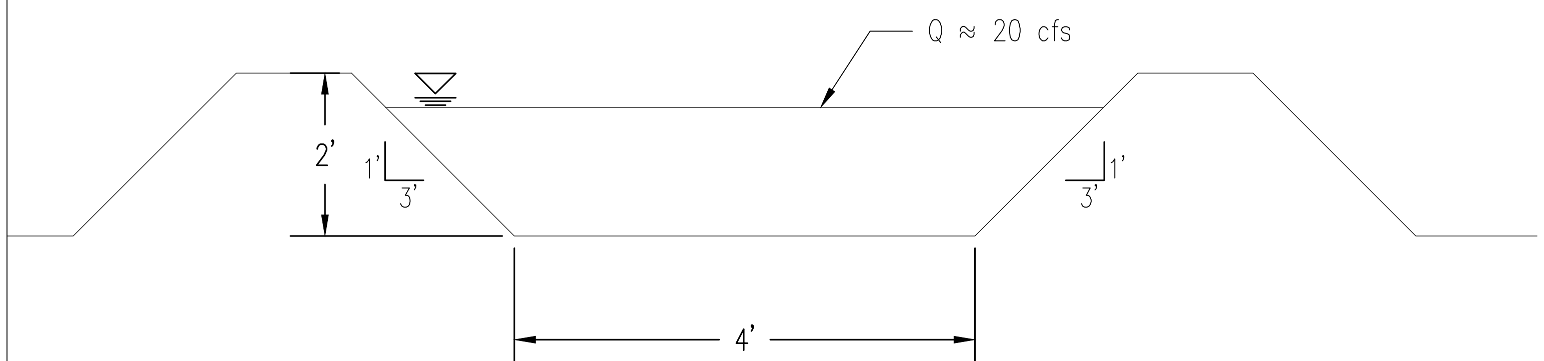
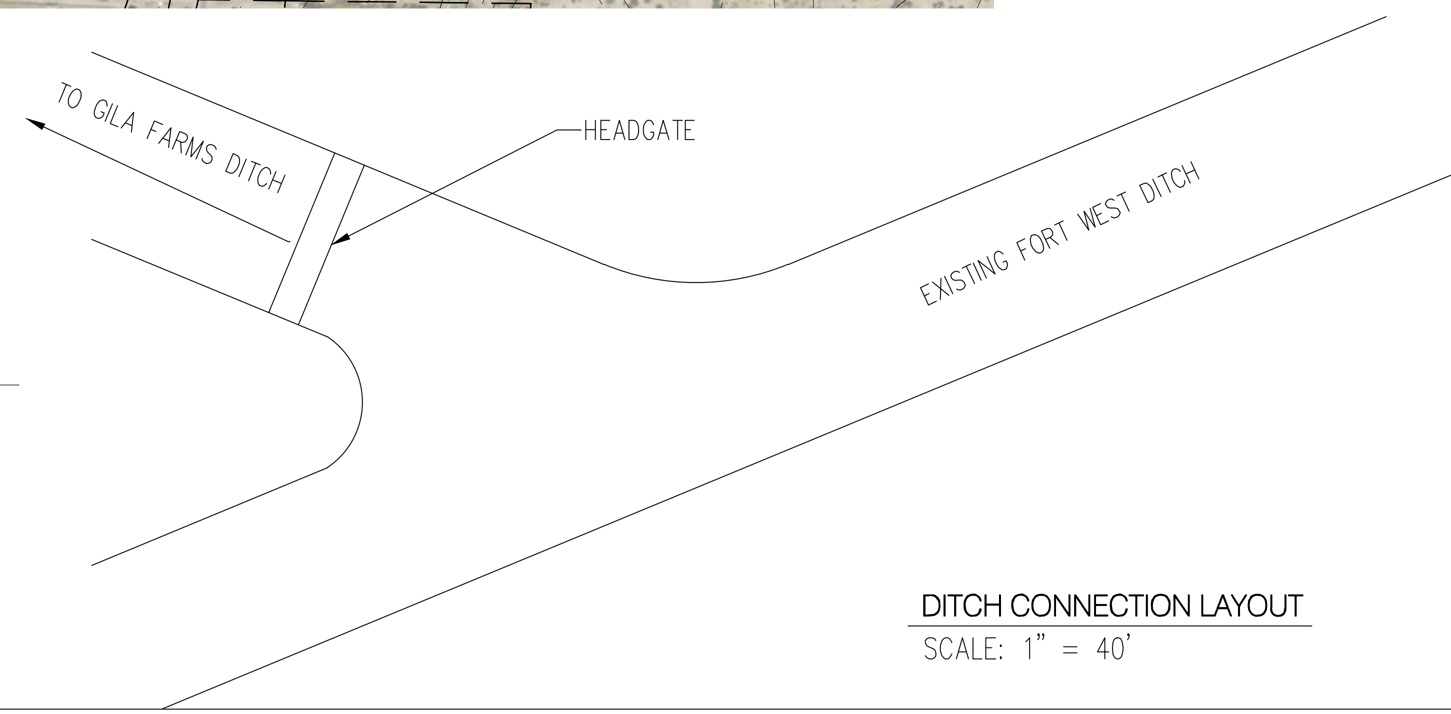
Layout

April, 2014





A DITCH CROSS SECTION
SCALE: 1" = 20'



DITCH CONNECTION LAYOUT
SCALE: 1" = 40'

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PROPERTY OWNERSHIP
PROVIDED BY GRANT
COUNTY ASSESSOR

GILA RIVER DIVERSION, CONVEYANCE AND STORAGE ALTERNATIVES

FIGURE 24
FORT WEST DITCH TO GILA FARMS DITCH CONNECTION

P:\20140055\WR\Reports\Final\Figures\PER Figures\Figure24_FortWest to Gila Farm Ditch.dwg
April 01, 2014 - 9:06am
Layout

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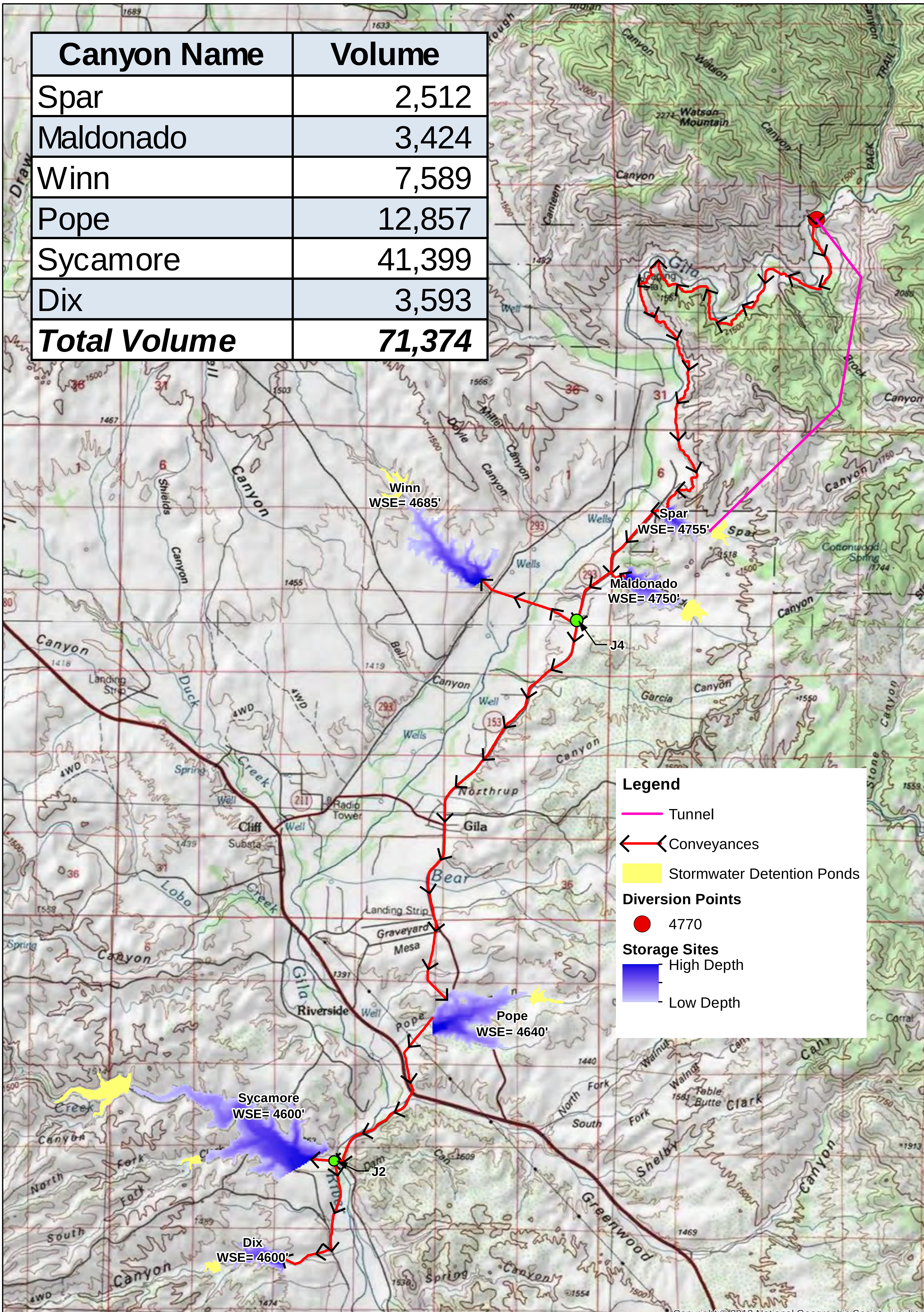
Canyon Name	Volume
Spar	2,512
Maldonado	3,424
Winn	7,589
Pope	12,857
Sycamore	41,399
Dix	3,593
Total Volume	***71,374***



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N W E S

0 6,000 12,000 Feet

GILA RIVER DIVERSION, CONVEYANCE AND STORAGE ALTERNATIVES

FIGURE 25
ALTERNATIVE 1

P:\20140055\WR\Reports\Final\Figures\PER Figures\Figure25_Alternative1.mxd
Author: rwine

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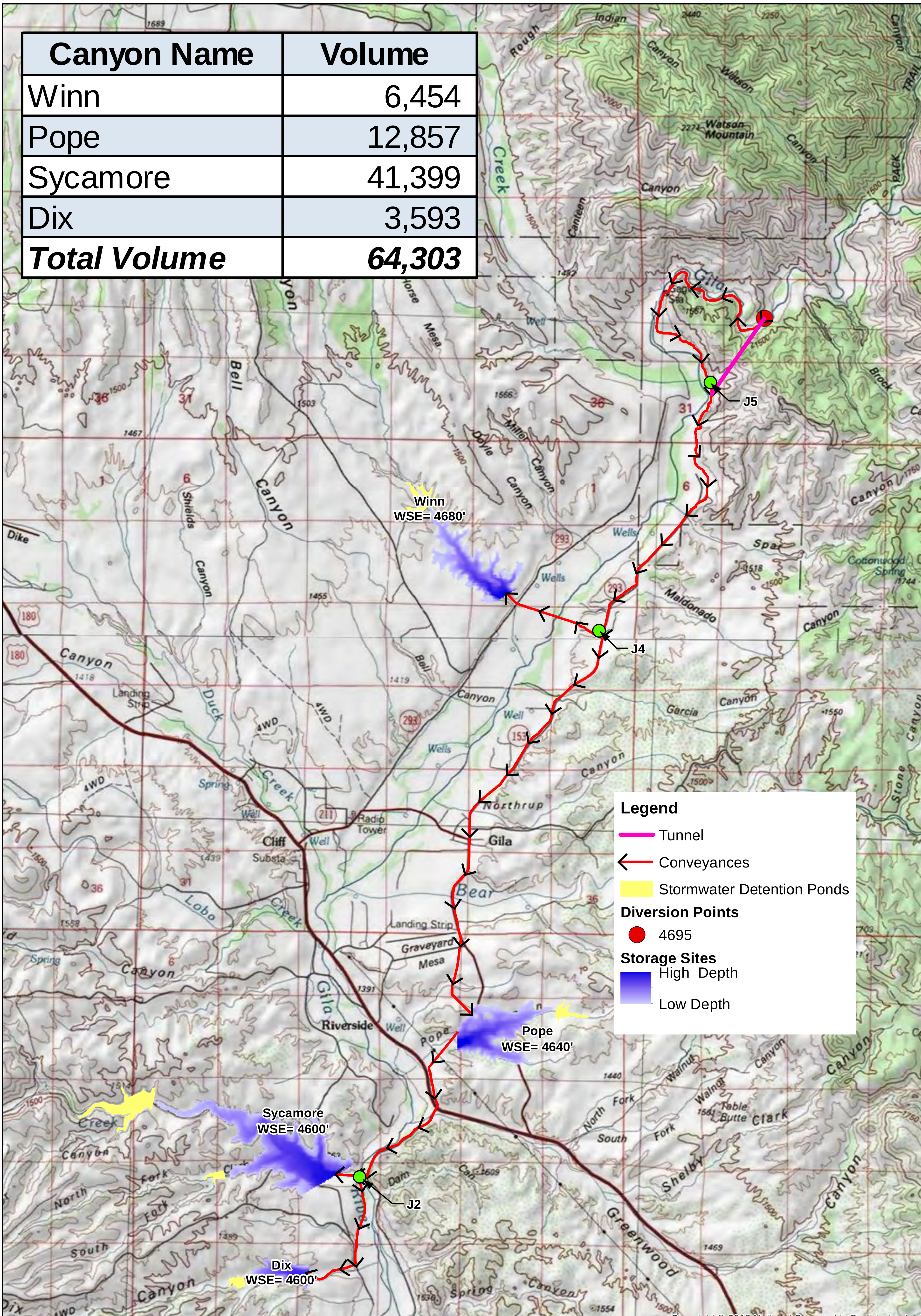
Canyon Name	Volume
Winn	6,454
Pope	12,857
Sycamore	41,399
Dix	3,593
Total Volume	***64,303***



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N W E S

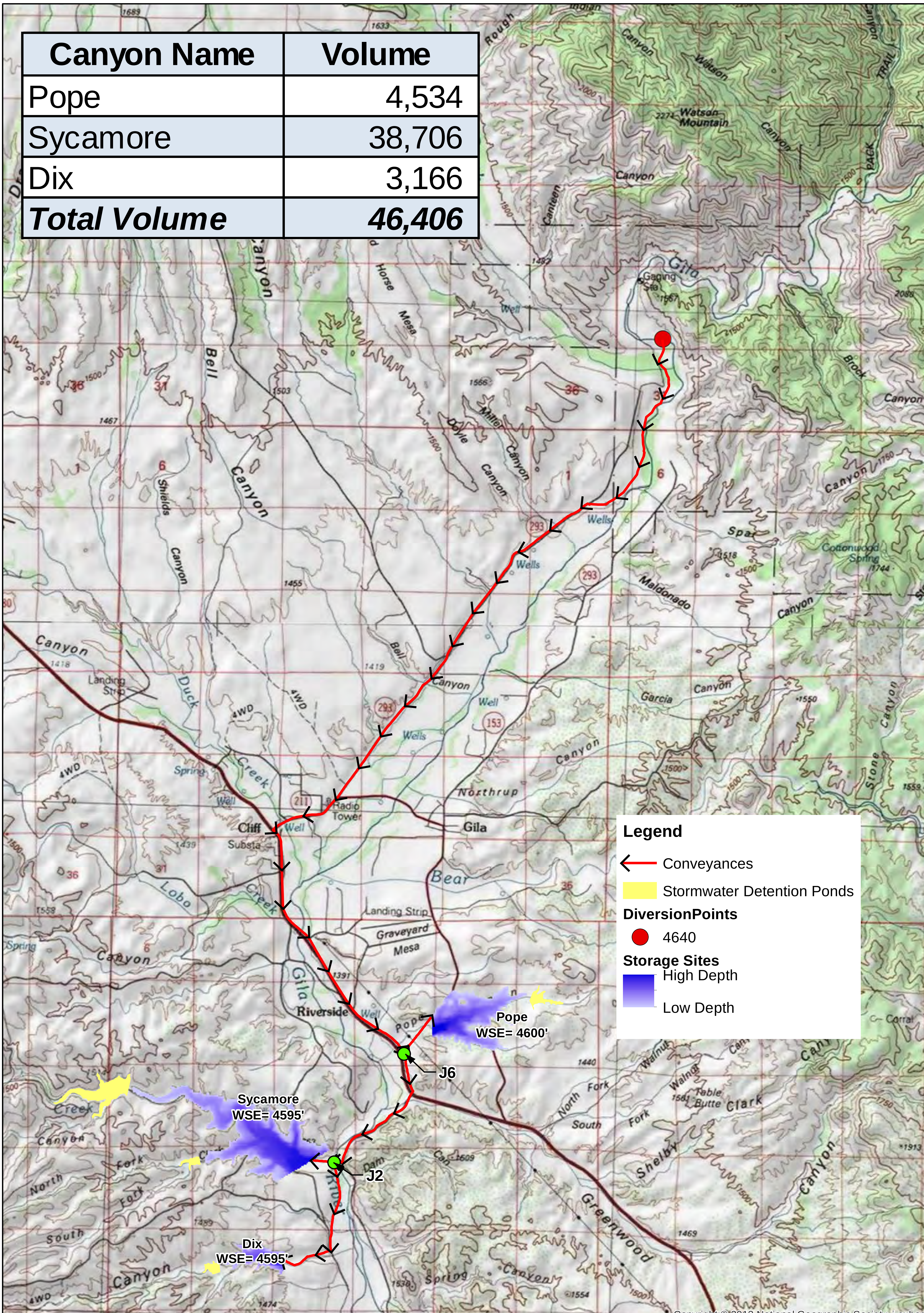
0 6,000 12,000 Feet

GILA RIVER DIVERSION, CONVEYANCE AND STORAGE ALTERNATIVES

FIGURE 26
ALTERNATIVE 2

P:\20140055\WR\Reports\Final\Figures\PER Figures\Figure26_Alternative2.mxd April, 2014
Author: rwine

Canyon Name	Volume
Pope	4,534
Sycamore	38,706
Dix	3,166
Total Volume	***46,406***



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N W E S

0 6,000 12,000 Feet

GILA RIVER DIVERSION, CONVEYANCE AND STORAGE ALTERNATIVES

FIGURE 27
ALTERNATIVE 3

P:\20140055\WR\Reports\Final\Figures\PER Figures\Figure27_Alternative3.mxd
Author: rwine

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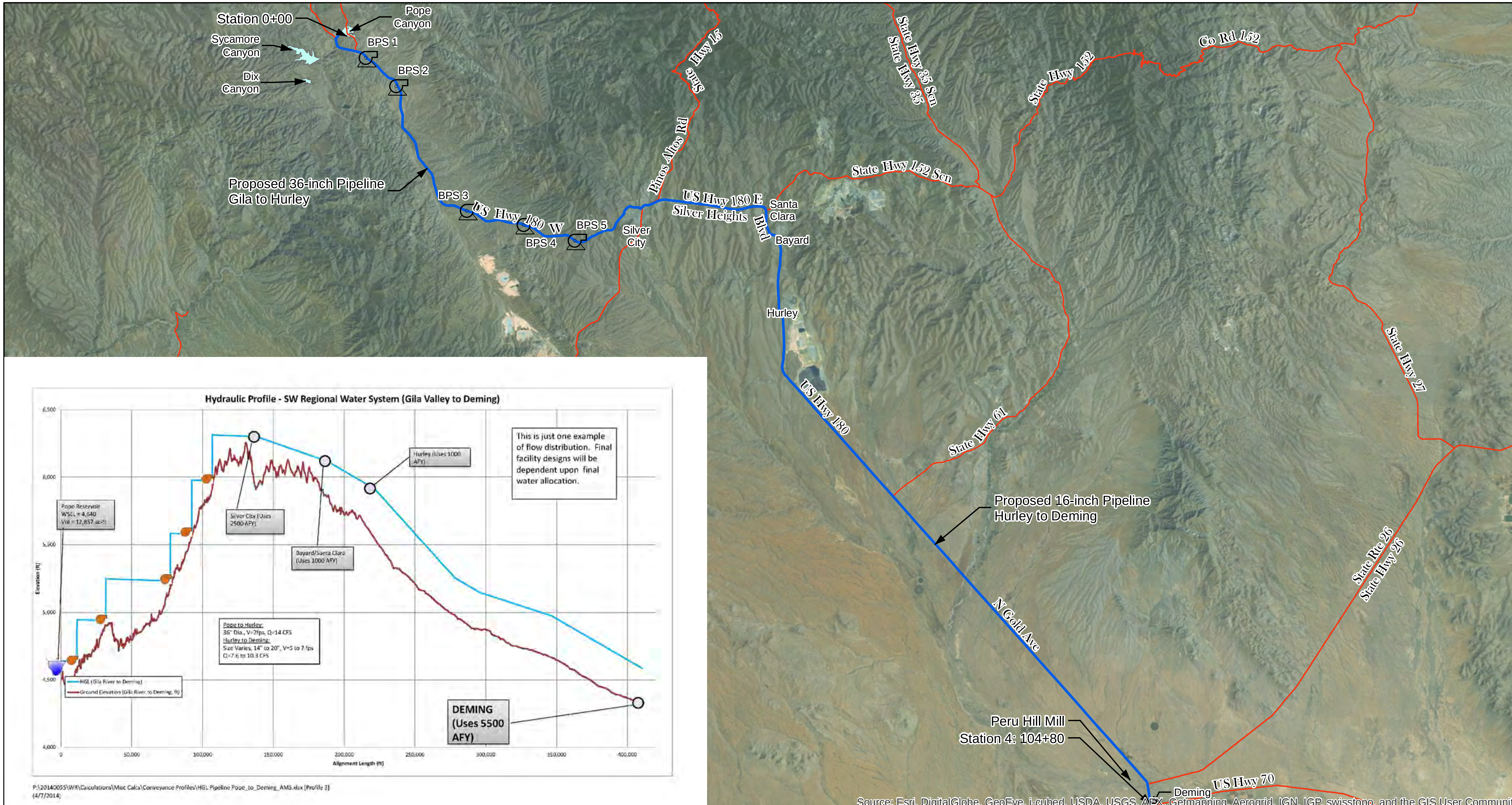

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Legend

- Booster Pump Station
- Pipeline
- Roads
- Canyon Boundaries

N
W E
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0 30,000 60,000
Feet

GILA RIVER DIVERSION, CONVEYANCE AND STORAGE ALTERNATIVES

FIGURE 28
SOUTHWEST NEW MEXICO REGIONAL WATER SUPPLY

P:\20140055\WR\Reports\Final\Figures\PER Figures\Figure28_SW_NM_RegionalWS.mxd
Author: rwine

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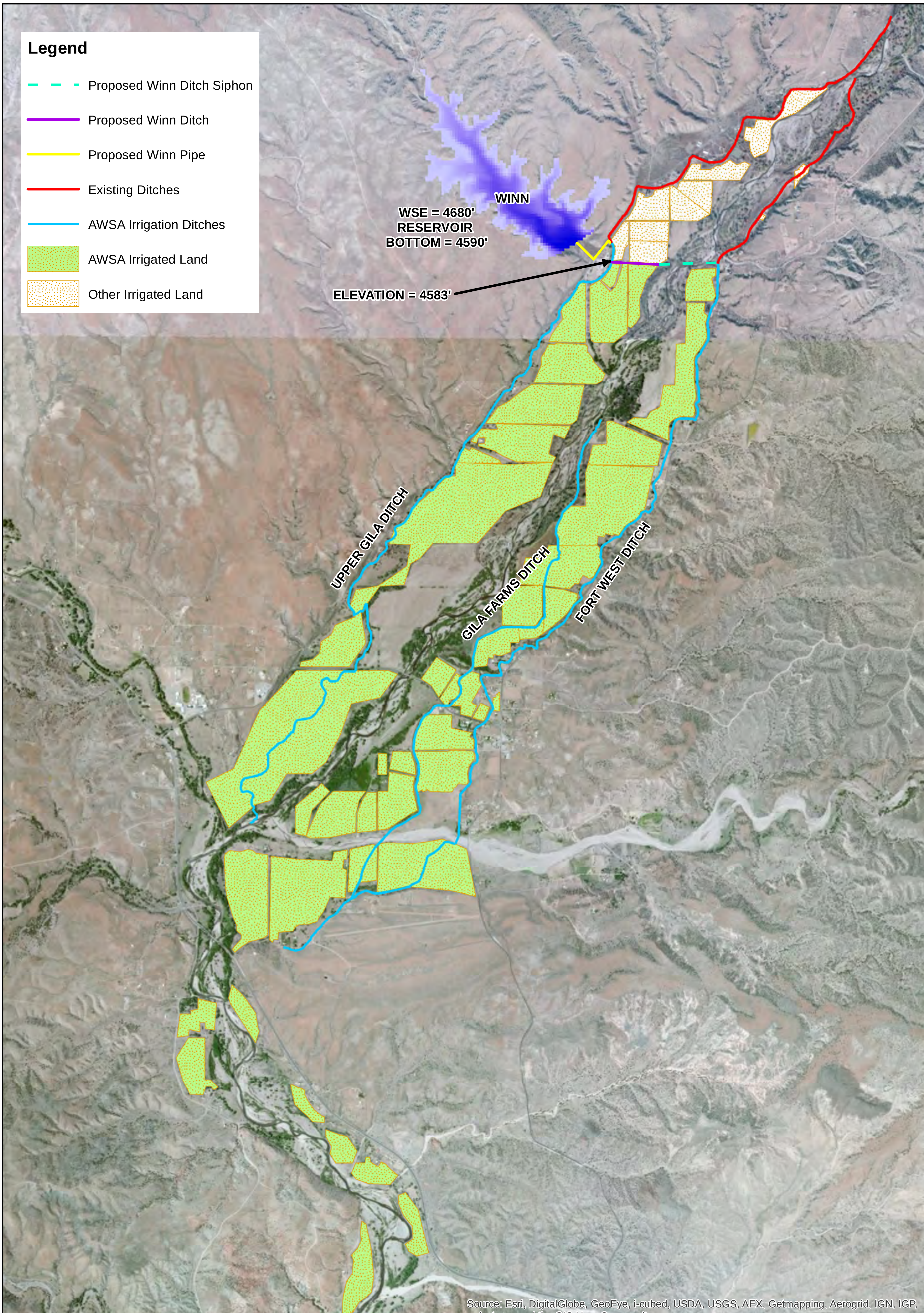

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N
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0 3,000 6,000
Feet

GILA RIVER DIVERSION, CONVEYANCE AND STORAGE ALTERNATIVES

FIGURE 29
POSSIBLE CONNECTION OF WINN CANYON TO UPPER GILA AND FORT WEST DITCHES

P:\20140055\WR\Reports\Final\Figures\PER Figures\Figure29_WinnCanyontoUpperGilaandFortWestDitches.mxd
Author: ksaintlot

April, 2014