RJH CONSULTANTS, INC.

GEOTECHNICAL AND
WATER RESOURCES ENGINEERING

FEASIBILITY EVALUATION

ARKANSAS STORAGE FACILITY, PHASE 1 PROJECT

PUEBLO COUNTY, COLORADO

Submitted to
Two Rivers Water Company
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FEASIBILITY STUDY APPROVAL
Pursuant to Colorado Revised Statutes 37-60-121 &122, and in accordance with policies adopted by the Board, the CWCB staff has determined this Feasibility Study meets all applicable requirements for approval.
Signed
Date 5/14/13

April 2013
Project 13101

COLORADO LICENSED
ROBERT J. HUZJAK
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4/11/13
PROFESSIONAL ENGINEER

Robert J. Huzjak, P.E.
Project Manager

This Feasibility Evaluation has been prepared in accordance with the CWCB *Water Project Loan Program Guidelines*, Revised January 2006.

The Owner and other consultants working directly with the Owner have prepared work in the following outline sections and this work is presented in an italic font:

- *Section 1 - Introduction*
- *Section 2 - Sponsor*
- *Section 3 - Water Rights and Water Demands*
- *Section 4 - Analysis of Alternatives*
- *Section 10 - Impacts*
- *Section 12 - Financial Plan*

RJH has prepared work in the following outline sections and this work is presented in a non-italic font:

- Section 1 - Introduction
- Section 5 - Selected Alternative
- Section 6 - Subsurface Investigation and Site Stratigraphy
- Section 7 - Typical Embankment Section, Seepage and Stability Analysis
- Section 8 - Hydrologic Evaluation
- Section 9 - Hydraulic Structures
- Section 11 - Opinion of Probable Project Cost
- Section 13 - Implementation Schedule
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SECTION 1 - INTRODUCTION

1.1 Purpose

The purpose of this loan application is to seek $9,990,000 in supplemental debt financing from the Colorado Water Conservation Board (CWCB) to compliment Two Rivers Water & Farming Company's (Two Rivers) $12,533,793 of equity capital to be used for development of Phase I (4,110 ac-ft) of an estimated 25,000 ac-ft multi phase gravel pit reservoir storage facility located adjacent to the Arkansas River just east of the confluence of Fountain Creek and the Arkansas River (Arkansas Storage Facility or ASF).

The ASF will be 100 percent "gravity in" via the Excelsior Ditch and approximately 80 percent "gravity out" to the Arkansas River and consist of approximately 72 percent surface storage and 28 percent alluvial storage. The ASF will immediately facilitate a viable rotational farm-fallowing program between farmers and municipal and industrial water users in the Arkansas River Basin located on the southern Front Range of Colorado.

As a precondition to closing on the CWCB loan and as part of Phase I of the ASF, Two Rivers will use a portion of its $12,533,793 in equity capital to purchase a majority interest (53.77 percent or 1,792 shares) of the Excelsior Irrigation Company and approximately 323 acres of land which is capable of developing into approximately 13,000 ac-ft of storage space to provide the CWCB with a first Deed of Trust as collateral for the CWCB Loan. Below is a table that describes how the money will be used, the cost per ac-ft, and the source of funds for the remaining project costs.

ASF Phase 1 Project	***Use of Proceeds***	
Acre Feet	*4,110*	
Excelsior Ditch	*$140*	*$3,500,000*
Property Acquisitions	*$216*	*$5,407,500*
Construction	*$3,052*	*$12,543,731*
Finance Cost	*$261*	*$1,072,562*
Total	*$3,669*	*$22,523,793*
Source of Funds	***Total***	
Two Rivers, Equity Capital	*56 percent*	*$12,533,793*
CWCB, Debt	*44 percent*	*$9,990,000*
Total	*100 percent*	*$22,523,793*

1.2 Objectives

The objectives of this report are to present the feasibility-level design of the selected alternative, an opinion of probable costs, and a proposed implementation schedule for the Arkansas Storage Facility, Phase 1 for use in obtaining a CWCB loan. The selected feasibility-level alternative and cost opinion presented in this report were developed to enable an evaluation of technical, environmental permitting, and associated cost. The selected alternative will be refined during final design based on analyses specific to the selected alternative. These specific analyses will likely result in modifications to the concepts presented in this report. Supporting calculations for the alternative included in final design will be developed and presented in future design reports.

1.3 Scope of Work

RJH Performed the following scope of work:

- Developed a map of the Project that included proposed and existing components, floodplains, and inundation zones.
- Developed feasibility-level sizes and layouts for the cutoff, reservoir, dam, outlet works, spillway, and inlet facilities. Layouts were primarily developed based on judgment, general design criteria, and experience. Specific analyses were performed only to support the relative size of facilities that could not be based on observation, experience, and judgment.
- Prepared a preliminary elevation-capacity curve.
- Performed wave run-up analysis to select the required freeboard height.
- Performed preliminary hydrology analysis for use in developing an approximation of the inflow design flood (IDF). Used this IDF and performed reservoir routing and peak discharge to support selection of the required spillway size.
- Described previous field investigations, methodology, site geology and geologic mapping, availability of construction materials, and subsurface investigations.
- Identified probable coordination required with federal, state, and local agencies.
- Estimated quantities of primary materials required for construction and prepared an overall opinion of probable project costs to construct the Project.
- Developed a Project schedule for design and construction of the dam and appurtenant facilities.

- Performed a preliminary permitting assessment to identify major technical permits and permitting issues for the Project. Identified anticipated required technical permits (i.e., local, state, federal).

1.4 Project Personnel

The following personnel from RJH are responsible for the work contained in this report:

Project Manager	Robert J. Huzjak, P.E.
Project Engineer	Michael L. Graber, P.E.
Hydraulic Engineer	G. George Slovensky, P.E.
Geotechnical/Structural Engineer	Jennifer D. Forbes, P.E.

SECTION 2 - SPONSOR

Two Rivers has developed and operates a revolutionary new private farming and water business model suitable for arid regions in the southwestern United States whereby the Company synergistically integrates irrigated farming and wholesale water distribution into one company, utilizing a practice of rotational farm fallowing. Rotational farm fallowing, as it applies to water, is a best methods farm practice whereby portions of farm acreage are temporarily fallowed in cyclic rotation to give soil an opportunity to reconstitute itself. As a result of fallowing, an increment of irrigation water can be made available for municipal use without permanently drying up irrigated farmland. Collaborative rotational farm fallowing agreements between farmers and municipalities can make surplus irrigation water available for urban use during droughts and, conversely, make surplus urban water available for irrigation during relatively wet periods. The Company produces and markets high value vegetable and fodder crops on its irrigated farmland and provides wholesale water distribution through farm fallowing agreements in its initial area of focus on the Arkansas River and its tributaries on the southern Front Range of Colorado. Two Rivers is a for-profit enterprise that is publicly traded on the OTC QB exchange under the symbol TURV. Our financial information and other disclosures are filed with the United States Securities Exchange Commission (www.sec.gov).

The Company owns a 95 percent interest in the Huerfano Cucharas Irrigation Company, which was purchased in 2010. In 2011, the Company purchased the Orlando Reservoir and Butte Valley water rights. In 2012, the Company purchased farmland along the Bessemer Ditch and Dionisio Farms and Produce, a produce business in Pueblo County. The Company currently has the capacity to store 15,000 acre-feet of water; when the reservoirs are fully restored and conditional rights are made absolute, the water rights are in excess of 64,000 acre-feet of water annually. Two Rivers' surface water rights total more than 50 cubic feet per second of stream flow, which historically yields 15,000 acre-feet of water annually.

SECTION 3 - WATER RIGHTS AND WATER DEMANDS

3.1 Water Rights and Water Demands

The first phase of ASF development associated with the Excelsior Ditch provides 4,110 acre-feet (ac-ft) of storage. The sources of water that could be placed into storage could include (a) water rights associated with the Excelsior Ditch, (b) water from the Two Rivers lease with the Pueblo Board of Water Works (PBWW), and/or (c) storage of PBWW and/or Colorado Springs Utilities (CSU) reusable return flows associated with the Restoration of Yield (ROY) as part of the Pueblo Flow Management Program (PFMP).

3.2 Excelsior Ditch

The Excelsior Ditch diverts water from the Arkansas River east of Pueblo, and downstream from the confluence of Fountain Creek and the Arkansas River, and has a decreed water right with an appropriation date of December 1861 (CA2535), which allows the Excelsior Ditch to divert up to 60 cubic feet per second (cfs) of water when this right is in priority. The Colorado Decision Support System (CDSS) database was queried regarding the historic diversions under the Excelsior Ditch, and Table 1 summarizes the monthly and annual diversions under the Excelsior Ditch for the period 1911-2011. As these diversion records indicate, the Excelsior Ditch has diverted an average of 4,223 ac-ft of water annually, with a maximum annual water diversion of 10,953 ac-ft. A graph of the annual historic diversions under the Excelsior Ditch is shown in Figure 1 of the engineering report.

The two owners of the Excelsior Ditch water rights are Stonewall Springs Quarry, LLC/Stonewall Water, LLC (Stonewall) and the Arkansas Groundwater Users Association (AGUA). Excelsior Ditch water is generally being used during the irrigation season for both direct use for irrigation and as a source for augmentation. Figure 2 shows the historic distribution of Excelsior Ditch water by month. When AGUA uses Excelsior Ditch water for augmentation, AGUA can either release water directly back to the river or the water can be sent to recharge ponds to cover lagged depletions during the non-irrigation season. As such, surface storage would be valuable to allow irrigation water to be stored to make late-season irrigation water more consistently available and to provide a means for storing augmentation water that can be released to offset lagged depletions during the non-irrigation season. As Figure 1 shows, while the historic ditch diversions have been 4,223 ac-ft per year (ac-ft/yr), the volume of *water available under the Excelsior Ditch has varied considerably from year to year. In addition, Figure 2 shows the average monthly distribution of diversions associated with the Excelsior Ditch, which indicates a relatively skewed distribution that could be designed to better-fit irrigation demands if surface storage is available.*

3.3 PBWW Lease

Two Rivers currently has a water lease agreement with PBWW for the delivery of up to 500 ac-ft/yr of totally consumable water from PBWW's water supply sources (Appendix A). Since this water would be released from Pueblo Reservoir, the water could be re-diverted at the Excelsior Ditch headgate and routed to storage for subsequent use under this lease. While this lease is for a term of 5 years (April 2012—April 2017), there is the potential for extension of the lease to provide water under a longer term. Given PBWW's multiple uses for its water rights, this water could be a supplemental source for AGUA.

3.4 ROY Reusable Return Flows

Under the PFMP, a ROY project was developed to provide facilities that would allow recovery of a portion of the yield lost related to participation in the PFMP. While part of the ROY program has been storage in Holbrook Reservoir, which is located a significant distance downstream on the Arkansas River (between Rocky Ford and La Junta), storage in constructed reservoirs that can be fed by the Excelsior Ditch would provide more efficient means for the restoration of yield than the utilization of Holbrook Reservoir. Both PBWW and CSU have water reclamation facilities, which are upstream of the Excelsior Ditch headgate, and both of these facilities discharge reusable effluent. Since it is difficult to continuously exchange water from downstream of the confluence of Fountain Creek back to Pueblo Reservoir, the means to facilitate restoration of yield is by storing water downstream when the exchange cannot be effectuated and then releasing water from storage when exchange potential exists to move water into Pueblo Reservoir. Not only would water storage from the Excelsior Ditch facilitate restoration of water yield by improving the exchange capacity into Pueblo Reservoir, but there will also be less transit loss in moving water from the PBWW and CSU water reclamation facilities to the Excelsior Ditch, as opposed to sending this water approximately another 45 miles downstream to the Holbrook Reservoir, the current downstream storage component in the ROY program.

In summary, there are a number of water rights that are potentially available for storage in a facility that can be gravity-fed from the Excelsior Ditch, and the demands associated with this storage would include (a) continued irrigation use, (b) use for augmentation *releases, and (c) use for restoration of yield under the PFMP through an exchange process.*

Table 1

Historic Excelsior Ditch Diversions[1)]

(acre-feet)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
1911	0.0	0.0	0.0	0.0	89.3	3272.8	2935.6	83.3	0.0	412.6	0.0	0.0	6793.5
1912	0.0	0.0	0.0	0.0	892.6	3451.3	1864.5	495.9	0.0	0.0	0.0	0.0	6704.2
1913	0.0	0.0	0.0	0.0	0.0	1567.4	456.2	238.0	0.0	0.0	0.0	0.0	2261.6
1914	0.0	0.0	0.0	0.0	2618.2	912.4	1567.0	1150.4	0.0	555.4	0.0	0.0	6803.4
1915	0.0	0.0	0.0	0.0	674.4	1398.4	1330.9	1223.8	119.0	0.0	39.7	0.0	4786.2
1916	0.0	0.0	0.0	0.0	386.8	1507.5	476.0	714.1	166.6	0.0	825.1	148.8	4224.9
1917	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
1918	0.0	0.0	0.0	0.0	0.0	2340.5	1632.4	0.0	39.7	0.0	0.0	0.0	4012.6
1919	0.0	0.0	0.0	0.0	89.3	1041.3	416.5	476.0	1507.5	1229.8	0.0	0.0	4760.4
1920	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
1921	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
1922	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
1923	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
1924	0.0	0.0	0.0	1045.3	1259.5	1273.4	366.9	0.0	0.0	0.0	0.0	0.0	3945.2
1925	0.0	315.4	0.0	41.7	422.5	238.0	486.0	827.1	107.1	0.0	0.0	234.1	2671.8
1926	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
1927	1061.2	39.7	952.1	922.3	374.9	1608.6	1352.7	741.8	115.0	428.4	412.6	952.1	8961.5
1928	753.7	386.8	196.4	240.0	684.3	2124.3	973.9	714.1	420.5	0.0	0.0	297.5	6791.5
1929	614.9	555.4	317.4	595.1	444.3	1400.4	1116.7	361.0	815.2	416.5	515.7	614.9	7767.4
1930	307.4	452.2	259.8	1803.0	158.7	1116.7	1110.8	1265.5	1285.3	698.2	456.2	406.6	9320.5
1931	158.7	827.1	267.8	430.4	688.3	563.3	835.1	493.9	196.4	396.7	801.3	991.8	6650.7
1932	1094.9	660.5	0.0	285.6	242.0	1213.9	884.6	337.2	198.4	0.0	0.0	932.2	5849.3
1933	1051.3	79.3	0.0	59.5	561.3	2542.8	1317.0	1100.8	188.4	0.0	0.0	0.0	6900.6
1934	420.5	218.2	1047.3	684.3	505.8	337.2	430.4	363.8	144.8	0.0	0.0	277.7	4430.0
1935	0.0	0.0	0.0	0.0	610.9	2300.9	1344.8	1013.6	107.1	0.0	0.0	0.0	5377.3
1936	807.3	0.0	138.8	515.7	1096.9	1158.4	505.8	1011.6	337.2	0.0	0.0	39.7	5611.3
1937	694.2	1182.2	1229.8	595.1	749.8	688.3	376.9	628.8	335.2	13.9	0.0	605.0	7098.9
1938	0.0	0.0	0.0	59.5	317.4	3812.3	922.3	991.8	569.3	0.0	0.0	119.0	6791.5
1939	0.0	0.0	0.0	452.2	839.0	515.7	585.1	376.9	79.3	0.0	0.0	0.0	2848.3
1940	0.0	0.0	0.0	0.0	289.6	267.8	491.9	614.9	307.4	9.9	0.0	0.0	1981.5
1941	0.0	0.0	0.0	79.3	2663.8	1993.4	1947.8	997.7	489.4	208.3	0.0	0.0	8379.8
1942	0.0	0.0	107.1	1325.0	1582.8	3149.8	1647.5	977.5	228.1	0.0	0.0	0.0	9017.8
1943	0.0	238.0	674.4	595.1	456.2	1481.7	952.1	733.9	456.2	0.0	0.0	0.0	5587.5
1944	0.0	0.0	0.0	0.0	0.0	3340.2	1765.3	847.0	704.1	0.0	0.0	0.0	6656.6
1945	0.0	0.0	99.2	297.5	287.6	501.8	690.3	1136.5	803.3	0.0	0.0	0.0	3816.3
1946	0.0	0.0	152.7	297.5	287.6	1249.6	714.1	535.5	0.0	0.0	0.0	0.0	3237.1
1947	0.0	0.0	0.0	188.4	2509.1	2122.3	3153.8	2162.0	297.5	297.5	0.0	0.0	10730.7
1948	0.0	0.0	638.7	2628.1	1537.2	2995.1	1864.5	1110.8	178.5	0.0	0.0	0.0	10952.9
1949	0.0	0.0	0.0	297.5	505.8	1995.4	5478.4	228.1	138.8	0.0	0.0	0.0	8644.1
1950	0.0	0.0	0.0	9.9	257.9	991.8	684.3	287.6	0.0	0.0	0.0	0.0	2231.4
1951	0.0	0.0	0.0	188.4	349.1	1634.4	543.5	674.4	148.8	0.0	0.0	0.0	3538.6
1952	555.4	1150.4	0.0	289.6	406.6	1725.6	515.7	357.0	0.0	0.0	0.0	0.0	5000.4
1953	803.3	0.0	0.0	0.0	79.3	1757.4	1418.2	694.2	297.5	0.0	0.0	386.8	5436.8
1954	595.1	0.0	0.0	0.0	0.0	99.2	0.0	138.8	0.0	0.0	0.0	386.8	1219.9
1955	0.0	0.0	0.0	0.0	267.8	1071.1	178.5	1110.8	0.0	0.0	0.0	0.0	2628.1
1956	0.0	178.5	9.9	0.0	277.7	666.5	208.3	59.5	0.0	0.0	0.0	0.0	1400.4
1957	0.0	0.0	0.0	0.0	654.6	934.2	1666.1	1428.1	0.0	0.0	0.0	0.0	4683.0
1958	0.0	0.0	0.0	829.1	1539.2	2459.5	365.0	238.0	0.0	0.0	0.0	0.0	5430.8
1959	0.0	0.0	1045.3	0.0	0.0	1081.0	0.0	0.0	0.0	674.4	714.1	1229.8	4744.5
1960	0.0	0.0	138.8	753.7	0.0	1338.9	0.0	0.0	0.0	0.0	1190.1	0.0	3421.5
1961	0.0	238.0	366.9	0.0	277.7	872.7	158.7	476.0	317.4	0.0	0.0	0.0	2707.5
1962	0.0	0.0	0.0	436.4	1071.1	1358.7	1666.1	39.7	0.0	0.0	1190.1	357.0	6119.1
1963	0.0	0.0	0.0	0.0	0.0	0.0	39.7	158.7	39.7	0.0	0.0	0.0	238.0
1964	0.0	0.0	0.0	0.0	0.0	452.2	9.9	9.9	0.0	0.0	0.0	0.0	472.1
1965	0.0	0.0	0.0	0.0	79.3	654.6	2769.0	684.3	476.0	220.2	0.0	0.0	4883.4
1966	0.0	0.0	0.0	0.0	83.3	79.3	357.0	420.5	19.8	0.0	75.4	0.0	1035.4
1967	0.0	0.0	0.0	0.0	385.0	778.8	575.2	0.0	0.0	0.0	0.0	0.0	1739.0
1968	0.0	0.0	0.0	0.0	0.0	2711.4	158.7	1504.5	0.0	0.0	0.0	0.0	4374.6

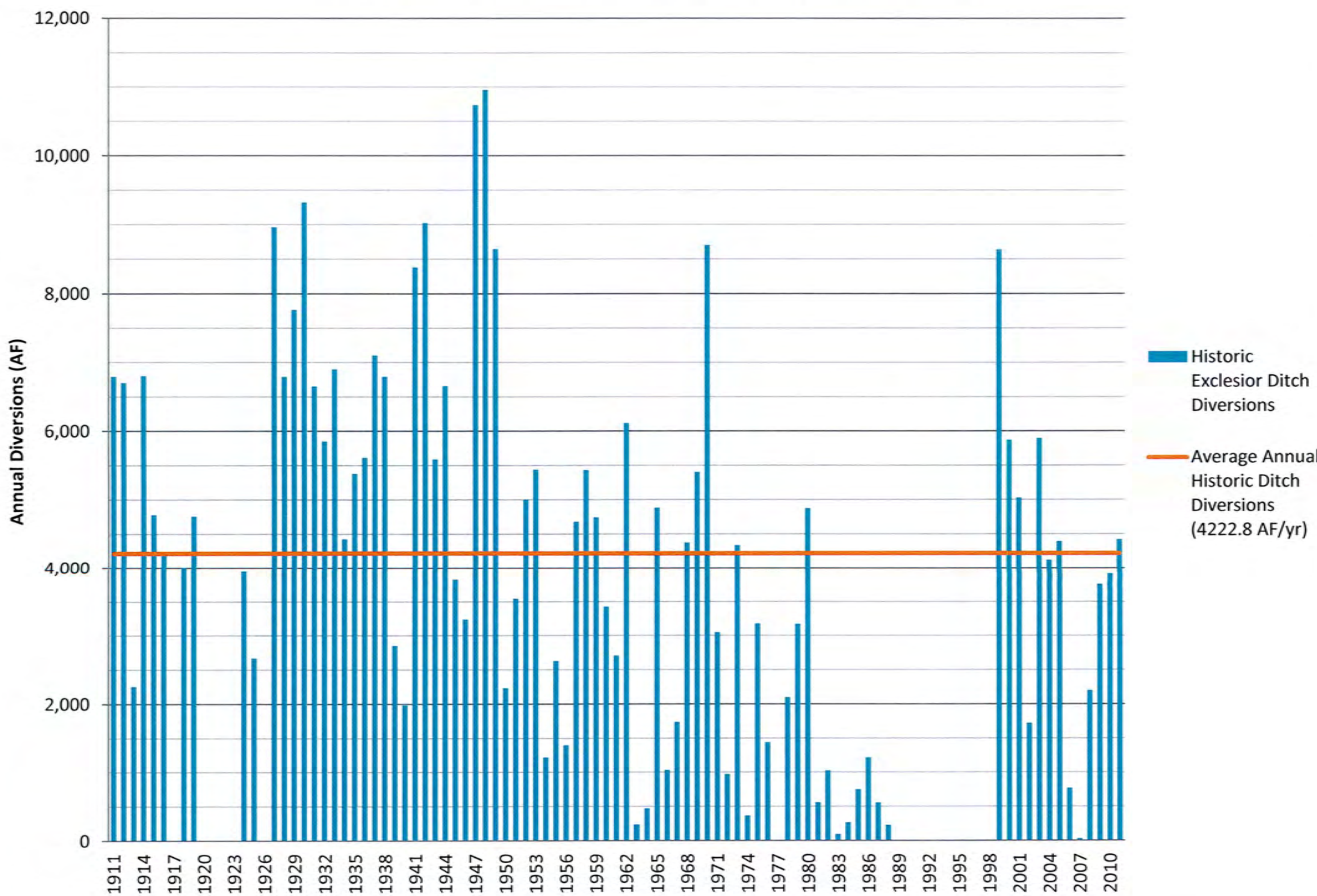
Historic Excelsior Ditch Diversions[1]
(acre-feet)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
1969	0.0	0.0	0.0	0.0	1540.2	3074.4	748.8	39.7	0.0	0.0	0.0	0.0	5403.1
1970	0.0	0.0	0.0	63.5	2474.6	2673.8	1889.1	121.8	197.4	1282.1	0.0	0.0	8702.2
1971	0.0	0.0	0.0	0.0	45.0	1170.5	445.1	0.0	0.0	0.0	807.1	574.6	3042.3
1972	0.0	0.0	0.0	0.0	50.6	882.9	0.0	0.0	39.7	0.0	0.0	0.0	973.1
1973	0.0	0.0	0.0	0.0	978.4	1463.5	1688.0	177.9	31.5	0.0	0.0	0.0	4339.3
1974	0.0	0.0	0.0	0.0	164.6	188.8	13.2	0.0	0.0	0.0	0.0	0.0	366.7
1975	0.0	0.0	0.0	0.0	0.0	1410.0	1237.7	470.1	59.5	0.0	0.0	0.0	3177.2
1976	0.0	0.0	0.0	0.0	237.0	548.1	243.7	413.2	0.0	0.0	0.0	0.0	1442.0
1977	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
1978	0.0	0.0	0.0	0.0	0.0	1630.6	450.3	17.0	0.0	0.0	0.0	0.0	2097.9
1979	0.0	0.0	0.0	0.0	113.6	1876.8	1180.2	0.0	0.0	0.0	0.0	0.0	3170.5
1980	0.0	0.0	0.0	56.5	1192.3	2326.2	770.2	527.4	0.0	0.0	0.0	0.0	4872.7
1981	0.0	0.0	0.0	0.0	0.0	50.6	0.0	510.5	0.0	0.0	0.0	0.0	561.1
1982	0.0	0.0	0.0	0.0	0.0	420.5	277.7	327.3	0.0	0.0	0.0	0.0	1025.5
1983	0.0	0.0	0.0	0.0	0.0	0.0	95.2	0.0	0.0	0.0	0.0	0.0	95.2
1984	0.0	0.0	0.0	0.0	0.0	0.0	0.0	162.6	101.2	0.0	0.0	0.0	263.8
1985	0.0	0.0	0.0	0.0	0.0	0.0	208.3	216.2	123.0	59.5	138.8	0.0	745.8
1986	0.0	0.0	0.0	249.9	47.6	357.0	333.2	226.1	0.0	0.0	0.0	0.0	1213.9
1987	0.0	0.0	0.0	0.0	472.1	83.3	0.0	0.0	0.0	0.0	0.0	0.0	555.4
1988	0.0	0.0	0.0	0.0	0.0	226.1	0.0	0.0	0.0	0.0	0.0	0.0	226.1
1989	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
1990	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
1991	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
1992	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
1993	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
1994	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
1995	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
1996	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
1997	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
1998	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND	ND
1999	0.0	0.0	536.6	199.5	1459.8	2549.9	1727.1	967.2	483.3	511.1	203.0	0.0	8637.4
2000	354.1	458.4	372.2	326.2	1299.7	1236.5	371.0	393.4	219.1	177.0	525.3	139.1	5871.8
2001	0.0	0.0	675.6	178.3	1709.1	1964.4	125.3	0.0	164.8	0.0	0.0	212.6	5030.0
2002	0.0	620.0	636.9	0.0	0.0	0.0	48.5	0.0	0.0	0.0	416.6	0.0	1722.0
2003	0.0	897.3	1052.2	119.8	316.7	1777.6	0.0	0.0	0.0	0.0	598.7	1134.7	5897.0
2004	0.0	769.0	660.2	155.5	267.4	196.2	133.3	51.0	0.0	0.0	739.2	1153.5	4125.2
2005	530.0	817.8	686.6	76.5	592.0	1009.5	0.0	0.0	0.0	0.0	155.3	530.0	4397.7
2006	0.0	0.0	0.0	0.0	105.2	436.1	116.7	0.0	0.0	108.6	0.0	0.0	766.4
2007	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	33.4	0.0	0.0	33.4
2008	0.0	0.0	326.2	562.1	181.1	700.9	199.5	26.8	51.1	120.6	33.2	0.0	2201.5
2009	0.0	0.0	28.4	613.5	1069.3	1224.5	572.9	0.0	90.7	0.0	160.4	0.0	3759.8
2010	0.0	0.0	61.0	841.2	760.7	1657.3	384.0	0.0	0.0	0.0	203.1	0.0	3907.2
2011	0.0	0.0	354.0	537.6	63.4	2417.6	905.9	104.6	16.0	25.1	0.0	0.0	4424.3

Average[2]	116.7	120.1	155.1	237.2	532.1	1306.0	803.2	440.3	157.3	93.8	121.4	139.6	4222.8
Max	1094.9	1182.2	1229.8	2628.1	2663.8	3812.3	5478.4	2162.0	1507.5	1282.1	1190.1	1229.8	10952.9
Min	0	0	0	0	0	0	0	0	0	0	0	0	33.442
% Dist:[3]	3%	3%	4%	6%	13%	31%	19%	10%	4%	2%	3%	3%	100%

1) ND = No data in the CDSS database
2) Average diversion by month for the period of record.
3) Percent distribution by month for the period of record.

Figure 1: Historic Excelsior Ditch Diversions



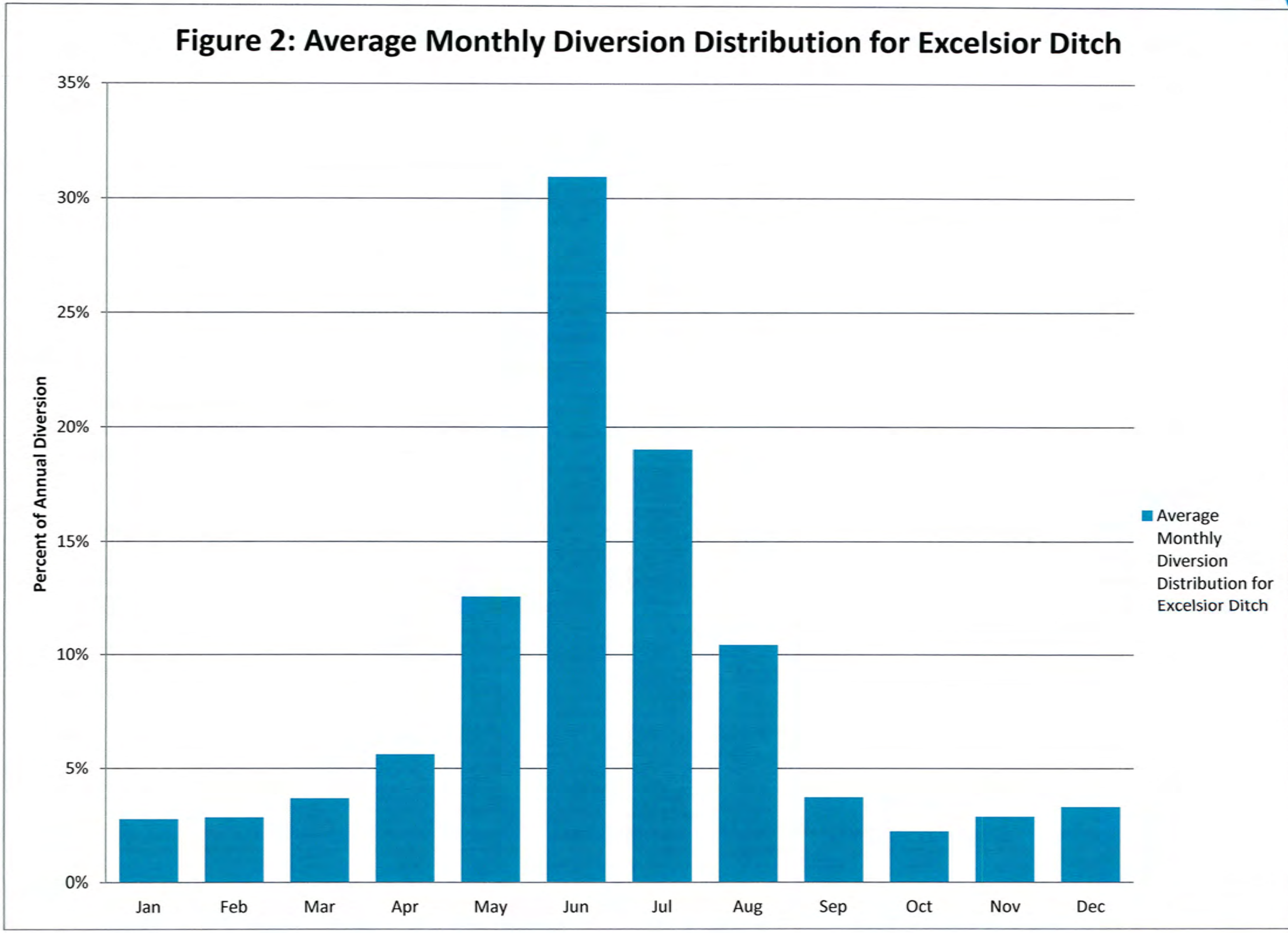


Figure 2: Average Monthly Diversion Distribution for Excelsior Ditch
35%
30%
25%
20%
15%
10%
5%
0%
Percent of Annual Diversion
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec
Average Monthly Diversion Distribution for Excelsior Ditch

SECTION 4 - ANALYSIS OF ALTERNATIVES

4.1 Enlarge Pueblo Reservoir

The principal potential alternatives to the proposed water storage described in this application would be the potential to expand Pueblo Reservoir and/or continue to use Holbrook Reservoir. The expansion of Pueblo Reservoir is not considered to be a practicable alternative, given that a recent Water Court Application by Southeastern Colorado Water Conservancy District to enlarge this reservoir was recently withdrawn, and the re-regulation of Pueblo Reservoir by CSU under the Southern Delivery System Project took approximately 8 years just to re-regulate uses in the reservoir—not to enlarge it. In addition, the water sources that would help with the restoration of yield under the PFMP could not be reliably stored in Pueblo Reservoir, since these flows are generated downstream of the reservoir and an exchange potential only occasionally exists to move these waters into Pueblo Reservoir. For all of these reasons, Pueblo Reservoir is not a good candidate site for additional water storage.

4.2 Holbrook Reservoir

Holbrook Reservoir is currently being used as part of the ROY project; however, there are significant transit losses associated with delivering this water to a point downstream of Rocky Ford and then potentially exchanging that water back up the river. In addition, Holbrook Reservoir would not satisfy the needs of the current users of the Excelsior Ditch for direct irrigation water, and for augmentation water to offset out-of-priority depletions from the pumping of AGUA's wells. As such, the proposed reservoir in this application is ideally situated for a number of purposes and is superior to other storage alternatives in this reach of the Arkansas River.

SECTION 5 - SELECTED ALTERNATIVE

5.1 Site Description and General Project Location

The proposed Arkansas Storage Facility, Phase 1 will be located in eastern Pueblo County, approximately 10 miles east of Pueblo, Colorado, immediately south of US Highway 50 and approximately 1/4 mile north of the Arkansas River in Section 34, Range 63 West, Township 20 South of the Sixth Principal Meridian. The site topography is flat and gently sloping south toward the Arkansas River channel. The future reservoir site is currently being used to produce blue grass sod, which is irrigated by a single center pivot irrigation system. The proposed project location (Site) is located on lands historically irrigated by the Excelsior Ditch, which has a diversion point approximately three miles to the west on the north bank of the Arkansas River. The general site location and plan of key project components is shown on Figure 1.

5.2 Arkansas Storage Facility, Phase 1 East Reservoir Project Description and Conceptual Plan

The Arkansas Storage Facility, Phase 1 gravel pit reservoir will develop water storage above and below the existing ground surface by storing water in the granular alluvial soils, excavating a basin into the native soil profile, and constructing an embankment dam. The embankment dam will be a ring dam and will vary in height from a few feet on the north end to approximately 30 feet on the south end. The dam embankment will have 3.5 horizontal to 1 vertical (H:V) upstream and 3H:1V downstream slopes. A soil-bentonite cutoff wall will extend along the dam centerline, completely encompassing the reservoir, from the existing ground elevation through the alluvial foundation and into the claystone bedrock to provide foundation seepage control and isolate the water within the reservoir and cutoff wall from the groundwater. The upstream embankment slope will have soil-cement erosion protection from the dam crest down to the existing ground elevation. The downstream slope will be seeded with native grass. Reservoir seepage through the embankment will be intercepted and safely conveyed with an embankment chimney drain, finger drains, and a toe drain. The embankment will be approximately 10,970 feet long, creating a reservoir surface area of approximately 152 acres at a normal maximum pool of Elevation (El.) 4548.0. If most of the soils above the granular alluvium are removed, the bottom of the excavated reservoir would be at about El. 4528 and the bottom of the alluvial storage would be at about El. 4503.0. The facility would have total above and below ground storage capacity of approximately 4,110 acre-feet (ac-ft). Approximately 2,910 ac-ft would be reservoir storage and about 1,200 ac-ft would be alluvial storage.

The reservoir will receive water from the Excelsior Ditch. The Excelsior Ditch receives water from the Arkansas River through an existing diversion structure, approximately 4 miles west of the Project site. A new gated diversion structure located on the north side of the reservoir will divert flows from the Excelsior Ditch, which is an open channel, into a 60-inch-diameter buried pipeline that will flow into the new reservoir. A concrete rundown chute at the discharge end of the buried delivery pipeline and energy dissipation structure will safely convey the diverted water into the reservoir. A combined vertical pipe spillway and gated outlet structure will deliver reservoir releases to the Arkansas River through a 48-inch-diameter buried pipeline that will discharge into an energy dissipation structure. A well will be used to remove water stored in the alluvium and convey that water to the outlet works. A general plan of the primary project components is shown on Figure 2.

5.2.1 Elevation-Capacity Table

The elevation-storage capacity for the reservoir is provided on Figure 3. Active reservoir storage is storage that can be released directly to the Arkansas River through the gated outlet works. Active storage is approximately 2,910 ac-ft. The alluvial storage capacity, which is the porous space in the alluvium, is approximately 1,200 ac-ft. The total reservoir storage for the current project configuration is 4,110 ac-ft. An additional storage volume of about 2,000 ac-ft would be available in the future if all of the alluvium is removed to the top of bedrock.

SECTION 6 - SUBSURFACE INVESTIGATIONS AND SITE STRATIGRAPHY

6.1 Kleinfelder Investigation

Kleinfelder drilled 11 borings around May of 2007 to identify the general stratigraphy at two proposed reservoir sites. The borings were advanced to the top of bedrock using hollow-stem auger and SPT sampling methods. No samples were recovered or tests performed.

Four of the 11 borings were drilled within the reservoir footprint. The upper soils were described as clay with average thickness of 9.8 feet overlying narrowly graded sands and sands with silt with an average thickness of about 25 feet. The bedrock was described as Pierre Shale and was encountered at an average depth of 34.6 feet below the ground surface.

Groundwater was measured during drilling at an average depth of about 11.8 feet below ground surface. The elevation of the ground surface at the borings was not recorded.

A summary of the Kleinfelder Investigation is included in Appendix A.

6.2 RJH Investigation

RJH performed a geotechnical investigation at the site between August 11 and September 12, 2008. Fourteen exploratory borings were advanced and eight test pits were excavated within or near the reservoir footprint.

The borings were drilled by High Plains Drilling Company of Aurora, Colorado, under subcontract to RJH. The purposes of the borings were to identify the subsurface stratigraphy and collect rock core samples for classification and laboratory testing. An RJH engineer was onsite during drilling operations to photograph recovered samples, visually classify samples, and prepare field logs for each borehole. The approximate locations of the borings are shown on Figure 4. A stick log representation of the primary materials encountered in each boring is presented on a generalized centerline profile of the dam embankment on Figures 6, 7, and 8.

Test pits were excavated by Southwest Farms, Inc. of Pueblo, Colorado, who was the owner of the property. An RJH engineer was onsite during excavation to visually classify the soil and rock, to collect samples, prepare field logs for each test pit, and record relevant excavation information. The purposes of the test pit explorations were to observe surficial soil that could potentially be used as borrow material for the dam, observe the contact between clayey and granular materials, and obtain samples for laboratory testing. Test pit locations are shown on Figure 4.

6.2.1 Groundwater

Groundwater was encountered in all borings and ranged between 14.0 and 29.0 feet below the ground surface. The average elevation of the water table during the time of drilling was estimated to be about El. 4521.7. Based on the information from the borings, the groundwater generally has a gradient of 0.4 to 0.5 percent to the southeast toward the Arkansas River.

6.2.2 Drilling/Sampling Methods

Borings were drilled using a CME 75 truck-mounted drill rig to advance 4 1/4-inch hollow-stem augers through overburden materials to competent rock. Once rock was encountered NQ-size wireline continuous rock coring equipment utilizing water as the drilling fluid was used to advance the borings. Core runs were generally 5 feet long. Borings ranged from 46 to 62 feet deep. Samples suitable for laboratory testing were placed in a Ziploc bag and taped in compression to allow for transportation. Upon completion of drilling, a piezometer was installed or the borehole was backfilled with cuttings.

Soil and bedrock samples were obtained using either a standard split-spoon, California sampler, rock coring, or bulk sampling methods.

6.2.3 Excavation Methods

The test pits were excavated with a CAT 416C backhoe with a 3-foot-wide bucket. Total depths ranged from 7.0 to 15.5 feet below the ground surface and were generally about 4 feet wide and 12 to 18 feet long. Each test pit was backfilled with the excavated material upon completion. Each test pit was marked with a flagged lath and was subsequently surveyed by Edward-James Surveying, Inc.

6.2.4 Logging Method

For the borings, soil samples were generally classified according to ASTM D 2488 (visual-manual method) and rock cores were generally classified according to the U.S. Bureau of Reclamation (USBR) *Engineering Geology Field Manual* (USBR, 2001). Following laboratory testing, laboratory index test results were compared to field classifications and, if appropriate, logs were modified according to ASTM D 2487 (Unified Soil Classification System). Lithology between recovered samples is interpreted. As appropriate, RJH field staff utilized indirect observations (e.g., drill chatter, drilling resistance, screening drill fluid, etc.) to interpret unobserved lithology.

For the test pits, stratigraphy changes were measured from the middle of the trench (length wise). For each different type of material identified during excavation, the field engineer recorded the classification and depth interval, and placed a representative sample in a large bulk sample bag.

6.2.5 Laboratory and Field Testing

Laboratory testing was performed on selected samples of soil and bedrock obtained in borings and test pits. The following properties were evaluated: index, strength, moisture-density, and permeability. RJH engaged Advanced Terra Testing, Inc. of Lakewood Colorado and Hepworth-Pawlak Geotechnical, Inc. of Parker, Colorado to perform the testing.

Laboratory testing included:

- Four moisture content tests per ASTM D 2216
- Fourteen Atterberg limits tests per ASTM D 4318
- Seventeen grain-size analyses per ASTM D 6913
- Two hydrometer tests per ASTM D 422
- Two dry unit weight tests per ASTM D 2937
- Two consolidated-undrained tests per ASTM D 4767
- Two unconfined compression tests per ASTM D 2166
- Five unconfined compression tests per ASTM D 7012, Method C
- Two standard Proctor compaction tests per ASTM D 698

- Two remolded back pressure permeability tests on specimens of proposed embankment material per ASTM D 5084

A summary of the test results are included in Appendix B.

Field tests included standard penetration resistance tests in the overburden soils and bedrock, pocket penetrometer tests on clayey standard split-spoon and California samples, and hydraulic conductivity tests (Packer tests) in competent bedrock.

6.3 Site Geology

The Site is located in the Colorado Piedmont Section of the Great Plains physiographic province. Bedrock below the Site consists of Pierre Shale, which is an Upper Cretaceous sedimentary rock formation. Surficial soils at the site consist of Post-Piney Creek alluvial deposits consisting primarily of clayey to sandy materials with occasional gravel deposits. The site generally slopes to the south toward the Arkansas River. Each of the geologic units are briefly described in the following sections.

6.3.1 Topsoil

Topsoil is present across entire Site at the ground surface and extends between approximately 0.5 and 1.0 foot deep. Topsoil consists of mostly clayey soils with occasional organics and contains between 5 and 15 percent fine sand. It is characterized as being soft to hard, light to dark brown, and dry to moist.

6.3.2 Post-Piney Alluvium

Alluvium underlies topsoil throughout the Site. The thickness of alluvium encountered ranged from about 45 feet in the northeast corner to about 30 feet in the southwest corner. The thickness of alluvium in the Kleinfelder borings ranged from about 12 to 33 feet. The alluvium consists of clay, clay with sand, sandy clay, clayey sand, sand with silt, sand, sand with silt and gravel, and gravel with sand and silt. Generally, the alluvium is divided into clayey and granular materials. Clayey deposits generally consist of medium to high plasticity fines with sand contents ranging between 1 and 50 percent. From the two samples tested, the liquid limit ranged from 38 to 62 with an average of 46 and the plasticity index ranged from 19 to 46 with an average of 29. Clayey materials were classified as being very soft to hard, and dry to moist.

Granular deposits are relatively well distributed between fine, medium, and coarse. The granular alluvium contains between less than 5 percent to about 60 percent gravel, with gravels grading from fine to coarse and typically shaped from sub-rounded to rounded. Gravel contents generally increase with depth. Standard Penetration Test (SPT) N-values range between 1 and 65 with an average of 19, which is representative of medium dense soils. SPT N-values generally increase with depth. Densities within the coarse deposits are estimated to range from very loose to dense. The granular deposits were described as typically moist to wet.

6.3.3 Pierre Shale

The claystone and sandy claystone is generally thinly bedded to laminated and varies between moderately weathered to fresh with a distinct weathering pattern that decreases with depth. The claystone ranges from intensely to slightly fractured with higher concentrations of fractures near the top of the unit. Generally, joints were tight to open, slightly rough to rough, with no to moderately thick infilling. Hydraulic conductivity measured using Packer permeability tests range from 1.3 x 10^{-3} cm/sec to 1 x 10^{-7} cm/sec or less. Calcite seams ranging between 1/2 and 1 ½ inches were observed in various borings. Calcite tends to be moderately hard and react strongly with hydrochloric acid.

6.4 Availability of Construction Materials

Based on subsurface investigation, testing, and analysis, the Site appears suitable for the construction of the proposed reservoir. In addition, materials required for the construction of the embankment fill, soil-cement slope protection, soil-bentonite cutoff wall, and embankment drain system are available on the Site. The clayey alluvium overburden would be the primary material used to form the embankment. The selected materials needed for the filter drains and soil-cement could be processed from the alluvial sand and gravels under or adjacent to the reservoir.

6.5 Mineral Resources

Gravel and sands are present under the overburden at the Site. The Project work only includes removing the overburden soils and only the sands and gravels required for construction of the soil-cement slope protection, soil-bentonite cutoff wall, and the embankment drain system. The reservoir would preserve the soil and gravel resource for the future when there is an economic need for the materials.

6.6 Reservoir Excavation

Approximately 2,229,000 cubic yards of overburden will be excavated from the reservoir to develop the reservoir bottom shown on Figure 5. Alluvium materials above El. 4528 will be removed at the southwest part of the reservoir to provide hydraulic connectivity between the reservoir and the sand and gravel. The area of removal is shown on Figure 5. About 4,300 cubic yards of alluvium above the invert of the outlet pipe is not planned to be removed initially. This material may be removed during future phases of Project development. Excavated overburden materials not needed for embankment construction will be stockpiled on private property to the east and immediately adjacent to the Site. The material has commercial value and may be incorporated into future projects.

SECTION 7 - TYPICAL EMBANKMENT SECTION, SEEPAGE AND STABILITY ANALYSIS

7.1 General

The Arkansas Storage Facility, Phase 1 embankment dam will be a ring dam and will vary in height from a few feet on the north side and along Excelsior Ditch to approximately 30 feet on the south side. A soil-bentonite cutoff wall will extend along the dam centerline from the existing ground elevation and into the claystone bedrock. The upstream embankment slope will have soil-cement erosion protection from the dam crest down to the existing ground elevation. A drainage layer will be constructed under the soil-cement to reduce uplift pressures. The downstream slope will be seeded with native grass.

Reservoir seepage through the embankment will be intercepted and safely conveyed with an embankment chimney drain, finger drains, and a toe drain. The embankment will be approximately 10,970 feet long, creating a reservoir surface area of approximately 152 acres at a normal maximum pool at El. 4548.0.

A plan of the embankment is shown on Figure 5; a centerline profile on Figures 6, 7, and 8; and typical sections on Figures 9 and 10.

7.2 Typical Embankment Section

Seepage and slope stability analyses were performed for one representative maximum cross section along the southern end of the reservoir and for one representative section at the north part of the dam at the existing Excelsior Ditch. The analyses were performed based on a conservative long-term future condition when the sands and gravels are removed to top of bedrock. The embankment crest was modeled at El. 4553.0 with a crest width of 15.0 feet for both cross sections. The embankment upstream and downstream slopes were modeled as 3H:1V, which is the steepest slope anticipated and meets the State Engineer Office's (SEO's) requirements for dam safety. The embankment slopes were modified early in concept development and the upstream slope modeled at 3H:1V is steeper than the selected slope of 3.5H:1V. Therefore, the slope stability analysis is conservative and the actual upstream slope stability factor of safety for the proposed cross section is greater than computed. The upstream toe elevations were set to top of bedrock and downstream toe elevations were set to top of existing ground, which vary for each cross section. Both cross sections extend 350 feet into the reservoir bottom upstream of the embankment and extend 200 feet downstream of the embankment.

The Arkansas River is located approximately 600 to 1,000 feet downstream of the south section embankment toe and a far field boundary condition was input into the model to represent the estimated total head in the river. This cross section is estimated to be the most critical section along the dam perimeter.

7.3 Slope Stability

Two-dimensional stability analyses were performed to document the slope stability factor of safety for the embankment geometry. Critical failure surfaces and minimum safety factors were computed using Spencer's Method, which considers moment and force equilibrium. We used the computer program Slope/W© 2007 to perform the iterative task of locating the critical failure surfaces and calculating the minimum safety factors.

Slope stability analyses were performed for the north and south sections of the embankment dam for steady state seepage and rapid drawdown conditions. The rapid drawdown load condition represents a condition where the reservoir is rapidly lowered from maximum normal pool. The preliminary rapid drawdown analyses are based on a conservative assumption that drawdown is instantaneous and that drainage (pore pressure dissipation) does not occur in fine grained materials.

Material properties were selected for the analyses based on laboratory testing of materials sampled from the Site and engineering judgment. The material properties are presented in Table 7.1. Conservative material properties were selected at this stage of design. It is probable that once additional data is collected, the material properties will change. Table 7.2 provides a summary of the results of the stability analyses.

TABLE 7.1
SELECTED PRELIMINARY MATERIAL PROPERTIES

Material Properties	Anticipated USCS Classifications	Unit Weights: Moist γ_m (pcf)	Unit Weights: Sat. γ_{sat} (pcf)	Strength Properties: c' (psf)	Strength Properties: ϕ' (deg)	Strength Properties: S_u (psf)	Seepage Analysis Properties: Hydraulic Conductivity k_h (cm/s)	Seepage Analysis Properties: Hydraulic Conductivity k_h (ft/s)	Seepage Analysis Properties: Anisotropy Radio k_v/k_h
Embankment Fill	CL, SC	122	128	0	25	1,000	2.5 x 10^{-6}	8.2 x 10^{-8}	1/9
Clayey Alluvium	CL, SC, SM, SH	120	125	0	28	500	2.5 x 10^{-6}	8.2 x 10^{-8}	1/5
Sand and Gravel Alluvium	SP, SC, SM, SW, GM, GC, SP-SM, SW-SM, GW-GM, GP-GM, GP-GC	122	132	0	30	--	1.0 x 10^{-3}	3.3 x 0^{-5}	1/2
Bedrock		135	140	1,200	25	5,000	5.0 x 10^{-4} 1.0 x 10^{-4} 1.0 x 10^{-5}	1.6 x 10^{-5} 3.3 x 10^{-6} 3.3 x 10^{-7}	1/4
Sand Drain	SP, SW, GW, GP	--	115	0	30	--	1.0 x 10^{-2}	3.3 x 0^{-4}	1/2
Soil-Bentonite Cutoff Wall		--	125	0	28	500	1.0 x 10^{-7}	3.3 x 0^{-9}	1/1

TABLE 7.2
SLOPE STABILITY ANALYSIS SUMMARY

Condition	Recommend Minimum Factor of Safety(1)	Computed Factor of Safety: Max Section	Computed Factor of Safety: North Section
Steady State Seepage	1.5	1.7	1.8
Rapid Drawdown	1.2	1.3	1.2

Note:
1. Recommended minimum factor of safety for peak strength based on SEO *Rules and Regulations for Dam Safety and Dam Construction* (SEO, 2007).

Computer outputs showing embankment sections, materials properties, and critical failure surfaces are presented in Appendix C.

7.4 Seepage Analysis

Seepage analyses were performed to identify the probable location of the phreatic surface in the embankment and to estimate the expected seepage quantities through the foundation and embankment to support design of the internal drainage systems. The seepage analyses were performed using the two-dimensional, finite-element program GeoStudio 2007, specifically, the SEEP/W module.

The permeability of the various materials were based on field and laboratory test results, empirical correlations, and experience. Anisotropy of the various embankment and foundation materials were developed based on engineering judgment. RJH developed hydraulic conductivity functions based on Soil-Water Characteristic Curves presented in the SEEP/W manuals and the design values for saturated permeability. The far field boundaries were set to the normal pool upstream of the dam embankment. Downstream of the dam embankment, the far field boundaries were set to the existing groundwater levels based on drilling records for the north section and to the Arkansas River water level for the maximum embankment section.

Generally, the bedrock along the proposed centerline of the cutoff wall can be divided into three distinct zones based upon Packer test results, fracturing, weathering, and hardness characteristics of the bedrock. Based on the data, bedrock along the dam alignment is characterized as 4 to 7 feet of intensely to moderately fractured rock with closely to moderately spaced fractures being slightly open to open, slightly rough to rough, with no to moderately thick infilling. Weathering is described as moderately to slightly weathered with a typical hardness of H7. Bedrock below about 7 feet is characterized as being moderately to slightly fractured with slightly open fractures with no filling. Weathering is described as being moderately to slightly weathered with hardness ranging between H7 and H6. Recorded Packer test results range between 2.1×10^{-4} and 1.0×10^{-7} cm/s. Hydraulic conductivities tend to decrease with hardness. The estimated hydraulic conductivities for the three layers are provided in Table 7.1.

A soil-bentonite cutoff wall will be constructed from the existing ground and into bedrock. The depth of penetration into bedrock will vary based on the variation of rock conditions. For this preliminary analysis the soil-bentonite cutoff wall was modeled as 3.0 feet wide and 5.0 feet into bedrock.

A chimney drain will be constructed downstream of the dam embankment centerline. The preliminary width of 6.0 feet was selected based on constructability. Based on the results from the seepage analysis at the maximum embankment section, the capacity of the drain is estimated to be about 4,000 times the amount required to carry flow into the drain. Seepage into the chimney drain will be conveyed through finger drains to the dam embankment toe, which will be constructed at a spacing required to carry the anticipated flow amount.

Using the results from the seepage analyses, we estimated total losses through the dam and dam foundation by taking the results from the two-dimensional seepage analysis times the approximate length of the dam, which is conservative. Annual losses could be up to about 107 gallons per minute (gpm) or 170 ac-ft per year (ac-ft/yr) if the reservoir is maintained at normal pool for the entire year. These amounts are for seepage through the dam embankment and foundation only and do not include evaporation losses.

The results of the seepage analysis are provided in Appendix D.

Additionally, a model was preformed to estimate inflow into an empty reservoir excavated down to the top of bedrock to compare with lining criteria for gravel pits. The criteria used were from the State Engineer *Guidelines for Lining Criteria for Gravel Pits* (SEO, 1999). The maximum section (south section) was used for the analysis with the upstream boundary condition set as an estimated Arkansas River water elevation. The downstream (reservoir) boundary condition was conservatively set to the top of bedrock. The maximum allowed inflow to meet the design standard is estimated to be about 118 gpm. An inflow of about 111 gpm is estimated based on the model output, which meets the Design Standard and provides a safety factor of 3 on the Performance Standard.

7.5 Slope Protection

The interior slope of the dam will need to be protected from erosion caused by wave action. Typically on reservoirs with an above and below ground storage volume, the upstream slope above original ground surface (the dam) is protected against erosion. For this Project, soil-cement was selected because it is a proven and effective slope protection method, it can be manufactured using the on-site granular alluvial soils, and because there is not a close source of riprap. This thickness of soil-cement varies based on many factors. For this feasibility design we have selected a thickness (normal to the slope) of 12 inches, based on experience and recommendations developed by the Portland Cement Association (PCA).

SECTION 8 - HYDROLOGIC EVALUATION

8.1 100-Year Floodplain

A portion of the proposed embankment is located within the Federal Emergency Management Agency (FEMA) regulatory 100-year floodplain for the Arkansas River. Floodplain mapping for this reach of the Project area is designated FEMA Zone A which denotes "approximate" mapping. No flood discharges, hydraulic modeling, or base flood elevations are published for Zone A areas. FEMA floodplain limits for the Project area are illustrated on Figure 2. Evaluating the impact of the proposed reservoir on flood elevations requires that a 100-year discharge for the tributary 6,250 square mile drainage area of the Arkansas River upstream of the Site be estimated. Detailed hydrologic and hydraulic modeling will need to be performed during design to establish base flood elevations. The required analyses and modeling for the Site have been initiated but not completed at this time. Completion of these analyses and floodplain permitting are expected to be a first step in the next phase of design. Significant coordination with FEMA and the local floodplain administrator is anticipated for establishment of an appropriate flood discharge.

Generally, increases in flood levels would require property or flood easement acquisition for impacted areas and any impacts to insurable structures would have to be eliminated through acquisition and removal, raising, or moving. Because land use upstream of the proposed Site is agricultural with very few structures in the floodplain, small increases in regulatory flood levels are unlikely to have large implications. If future analyses indicate the proposed reservoir would cause an unacceptable rise in base flood elevations, modest adjustments to reservoir limits could be made to reduce impacts.

Floodplain permitting requirements will depend in part on how large an impact (if any) the proposed reservoir has on regulatory flood levels. Likely permitting requirements are described in Section 8.1.

8.2 Approximate Dam Breach Flow and Estimated Dam Hazard Classification

The reservoir is expected to be classified as a Small, Low-Hazard dam in accordance with requirements set forth in the State Engineer *Rules and Regulations for Dam Safety and Dam Construction* (SEO, 2007). A small, low-hazard classification is appropriate because the maximum dam height will be less than 50 feet and above grade storage is less than 4,000 ac-ft. Breach flows from the dam would flow directly into the Arkansas River channel to the south of the reservoir. The estimated peak breach flow from the maximum reservoir section is on the order of 27,400 cubic feet per second (cfs). The estimated 100-year flood flow in the Arkansas River based on the current FEMA flood limits at the Site is approximately 57,700 cfs. The breach flow from the maximum section of the dam would be easily contained within the 100-year floodplain of the Arkansas River but would exceed the existing channel capacity of the river, which is estimated to be approximately 6,000 cfs. Only minor flooding beyond the river channel would occur and the breach flows would quickly attenuate because of the broad and flat floodplain downstream of the dam and reservoir.

8.3 Wave Run-Up Analysis

A wave run-up analysis was performed to evaluate the amount of minimum freeboard required by SEO guidelines. The analysis was performed to evaluate long-term conditions with a reservoir bottom set at top of bedrock. Guidance from the U.S. Army Corps of Engineers and the U.S. Bureau of Reclamation was used for the analysis.

The total wave run-up is estimated to be about 3.8 feet, which is less than the minimum freeboard required by SEO of 5.0 feet. It might be possible in future stages of design to reduce the freeboard to 4 feet.

8.4 Inflow Design Flood and Reservoir Routing

Given a classification of a small, low-hazard dam, the IDF required by SEO regulations will be the 100-year storm. The reservoir will be an off-channel structure with no tributary inflow area. A total embankment ranging in height from about 5 to 21 feet above existing ground will surround the entire reservoir isolating it from flood flows in the Arkansas River and runoff from local watersheds. Runoff from local watersheds historically tributary to the Site is currently intercepted by Highway 50 and passed under the roadway in several culverts. Discharge from these culverts is in turn intercepted by the Excelsior Ditch with any flows exceeding the ditch capacity overtopping the ditch bank and sheet flowing down to the Arkansas River. The reservoir embankment may require erosion protection in limited areas but no off-site flows will enter the reservoir.

Under these conditions, the 100-year inflow design event for the proposed reservoir will consist only of the volume of water that falls on the interior of the reservoir during the 100-year rainfall event. Rainfall depths for the 100-year event are available in the *Precipitation-Frequency Atlas of the Western United States, Volume III, Colorado* (NWS, 1973). Based on this reference, the 100-year, 24-hour precipitation depth for the

Site is 4.6 inches. Freeboard for the reservoir will be at least 4 feet (currently set at 5 feet) and the 100-year event would therefore be easily stored within the reservoir. An ungated outlet structure (spillway) will maintain reservoir pool between El. 4548.0 and El. 4548.4 and will allow a slow release of any 100-year event. Information on the ungated spillway is provided Section 9.

SECTION 9 - HYDRAULIC STRUCTURES

9.1 Reservoir Inlet Structures

Water would be supplied to the proposed reservoir by the existing Excelsior Ditch. The ditch and feasibility-level locations for water intake structures are shown on Figure 5. Reservoir intake structures would consist of a ditch check, an intake structure with a gate for flow control, pipeline to the reservoir, an outlet structure, rundown, and an energy dissipater. Reservoir inlet structures are illustrated on Figures 11 and 12 and described in the following paragraphs.

The reservoir intake would consist of a gated diversion structure on the Excelsior Ditch. A trashrack has been included to reduce the potential to plug the inlet pipe and convey debris into the reservoir. The trashrack was sized to provide a 2 foot per second (fps) velocity through the trashrack with 30 percent plugging and a 6-inch bar spacing. The intake face and trashracks would be vertical to provide easier installation and removal, limit debris accumulation, and provide access from the top for manual cleaning if necessary. Internal concrete divider walls would provide support for the trashracks and openings or orifices between the walls could be added during the design phase to equalize inflows over the trashrack face. Inflow equalization would help limit debris accumulation on the trashracks, plugging potential, and debris quantities delivered to the reservoir.

A 5-foot-diameter pipe was selected to convey water from the intake structure to the reservoir. For the expected low head levels in the ditch at the intake structure, the pipe flow is inlet controlled and a tapered inlet transition from the intake structure was used to improve hydraulic capacity. The inlet pipeline is expected to be precast concrete pipe with gasketed joints.

The intake pipeline would discharge to a concrete rundown structure on the interior of the reservoir embankment. A level, elevated concrete crest at the top of the rundown would provide for spreading and even distribution of flow across the rundown. Maximum velocities on the rundown would be less than 35 fps. The concrete rundown channel would discharge to a concrete stilling basin depressed below the reservoir bottom. The stilling basin would only be required when filling the reservoir from an empty condition. Once a substantial water depth is reached, the pool itself would provide energy dissipation.

The ditch check structure would be located a short distance downstream of the intake structure. The check would be used to regulate the volume of flow through the ditch to enable all or a portion of the flow to be diverted to the reservoir. The current concept includes two overshot gates. Overshot gates were selected because they provide the advantage of opening from the top down and when full open, the gates are close to the channel invert. This allows the gate to pass or flush surface debris when partially open and, when lowered fully, to flush bottom sediments. Locating the check structure a short distance downstream of the intake for the reservoir should limit debris accumulation on the intake trashracks.

9.2 Reservoir Outlet Structures

The outlet works concept consists of a low-level intake for release of Project flows to the Arkansas River and an ungated service/emergency spillway. Discharges from these facilities would be delivered to the Arkansas River by a steel pipe that terminates in an impact basin near the river. The outlet works concept is illustrated on Figure 13 and described in the following paragraphs.

The low-level intake would consist of concrete intake box with trashrack and hydraulically actuated sluice gate delivering water to a 54-inch-diameter steel pipeline. The trashrack has been sized to provide a 2 fps velocity through the trashrack assuming 50 percent plugging and 6-inch bar spacing. A 54-inch-diameter sluice gate has been included as a “guard gate.” This gate will typically be positioned either fully open or fully closed and not used for flow control. When delivering Project water, the sluice gate would generally be in the full-open position. Closure of the gate would allow dewatering, inspection, and required maintenance of the entire downstream pipe and facilities.

Water from the low-level intake would be delivered through the 54-inch pipeline to a valve and spillway tower near the dam crest. The tower would contain a hydraulically actuated, knife gate valve to provide flow control for Project discharges. A hydraulic power unit to operate this valve and the guard gate would be located within a valve house on top of the tower. The tower would also function as a service and emergency spillway. A 10-foot-wide ungated overflow crest with trashrack would be located on the face of the tower. The crest elevation of the spillway would be set at the maximum normal water surface at El. 4548.0. The spillway would provide for controlled release of the 100-year design storm event and any more routine storm events that result in reservoir levels exceeding the maximum normal water surface.

Project water releases and any spillway flows would enter the 54-inch steel pipe at the bottom of the tower for delivery to the Arkansas River. The 54-inch-diameter steel outlet works pipe would provide a discharge capacity of 150 cfs or greater for reservoir levels above approximately El. 4538. Below this reservoir storage elevation, discharge capacity would gradually decrease. Outlet works capacity would be reduced to about 100 cfs at a storage elevation at El. 4534.0 and to 50 cfs at about El. 4532. These discharges assume low flow conditions and a water surface elevation in the Arkansas River at El. 4528.0. At higher river flows, outlet works capacities would be less.

A USBR impact-type basin for energy dissipation would be located at the discharge point to the river. The discharge point is located on a bend to reduce pipe length. Although the river channel appears to be reasonably stable, the stability of the channel and need for any type of erosion protection needs to be evaluated in future stages of design. The outlet works outlet would be constructed so that excavation is not required below the normal high water line of the Aransas River.

9.3 Alluvial Well

An alluvial well would be constructed on the south part of the reservoir as shown on Figure 2. The well would consist of 24-inch-diameter casing with a 40 horsepower motor capable of pumping about 700 to 1,000 gpm. The water will be removed and conveyed to the outlet works tower.

SECTION 10 - IMPACTS

Generally, this water storage project will provide a positive impact to regional water resource development, as described in the previous sections. Since the water will be diverted at an existing diversion structure on the Arkansas River, we do not believe that there will be any Federal nexus related to Section 404 permitting under the Clean Water Act. Since the water being stored is likely to be serving existing uses, we do not believe there will be any significant change in land use other than the actual building of the reservoir. In terms of water quality, the ROY project can actually provide higher-quality water to PBWW and CSU water users, based on lower-quality water being captured at the proposed storage facility and then exchanged upstream with higher-quality water at Pueblo Reservoir.

SECTION 11 - OPINION OF PROBABLE PROJECT COST

This opinion of probable costs are based on the feasibility-level design concepts presented in this report. Quantities were estimated for the major construction items. The lump sum item prices are based on qualitative estimates of the work required and the corresponding cost. Estimated unit prices and costs for the primary work items were derived from the following sources:

- Published and non-published bid price data for similar work from similar projects.
- R.S. Means Heavy Construction Cost Data for 2013.
- Manufacturer’s budgetary price quotes.
- Our previous experience and engineering judgment.

It is our opinion that this cost opinion represents a Class 4 level estimate as defined by the Association for the Advancement of Cost Estimating (AACE). This level is appropriate for a study or feasibility phase where the design engineering is between 1 and 15 percent complete. The reliability of this level of estimate according to the AACE should be considered to be between about minus 15 to 30 percent and plus 20 to 50 percent.

The sum of the listed items for each category is defined for this study as the “Base Construction Subtotal” (BCS). The sum of the BCS, mobilization, bonds, insurance is defined as the “Direct Construction Cost” (DCC). For this study an allowance of 2.5 percent of the BCS was included to account for the construction contractor’s costs for mobilization, bonds, and insurance.

The Opinion of Probable Project Costs (OPPC) is the sum of the DCC, construction contingencies, and engineering and administration costs. For this Project, the OPPC includes an allowance of 40 percent for contingencies, engineering, and administration as follows:

- 20 percent of the DCC to account for construction contingencies. This allowance will decrease as project development progresses toward more detailed levels of design.
- 9 percent of the DCC for design engineering.
- 9 percent of the DCC for construction engineering.
- 2 percent of the DCC for legal fees, permitting, and owner administration.

A summary of quantities and our OPPC is presented in Table 11.1. Costs are presented in 2013 dollars.

This opinion of probable construction costs (OPCC) is based on professional opinion of the costs to construct the Project as described in this report. Actual costs would be affected by a number of factors beyond current control, such as supply and demand for the types of construction required at the time of bidding and in the Project vicinity, changes in material supplier costs, changes in labor rates, the competitiveness of contractors and suppliers, changes in applicable regulatory requirements, and changes in design standards and concepts. Therefore, conditions and factors that arise as Project development proceeds through construction may result in construction costs that differ from the estimates documented in this report.

Much of the cost for this Project is earthwork and the cost for earthwork is highly sensitive to fuel costs. If fuel costs change significantly in the next few years the cost of the Project could be directly impacted. RJH has not attempted to predict changes in future fuel prices to develop this OPPC.

TABLE 11.1
OPINION OF PROBABLE PROJECT COST

Item	Quantity	Unit	Unit Price ($)	Totals ($)
Dam Embankment Components				**5,165,200**
Soil-Bentonite Slurry Wall				**2,126,250**
Mobilization	1	LS	75,000.00	75,000
Soil-Bentonite Design Mix	1	EA	15,000.00	15,000
Cutoff Wall	452,500	SF	4.50	2,036,250
Dam Components				**3,038,950**
Embankment Fill	315,000	CY	2.00	630,000
Soil-Cement Slope Protection	27,000	CY	47.00	1,269,000
Drainage Layer	18,000	CY	25.00	450,000
Sand (Chimney)	21,000	CY	26.00	546,000
Sand (Trenched)	460	CY	25.00	11,500
Gravel	260	CY	30.00	7,800
Drain Pipe (8-inch-diameter)	8,310	LF	15.00	124,650
Inlet Works Facilities				**283,070**
Ditch Check Structure				**110,170**
Canal Diversion Structure				**57,100**
Intake Pipeline				**45,920**
Reservoir Outlet and Rundown				**69,880**
Outlet Works Facilities				**913,800**
Low level Inlet Structure				**87,800**
Steel Outlet Works Pipeline				**496,000**
Valve House				**154,500**
Flow Monitoring Vault				**31,500**
Energy Dissipater				**144,000**
Alluvial Well				**200,000**
Base Construction Subtotal (BCS)				**6,549,220**
Mob/Demob (15% of BCC)				98,238
Bonds/Insurance (1% of BCC)				65,492
Direct Construction Cost (DCC)				**6,712,951**
Construction Contingencies (20% of DCC)				1,342,590
Final Design and Engineering (9% of DCC)				604,166
Construction Engineering (9% of DCC)				604,166
Owner Administration (1% of DCC)				67,130
Permitting (1% of DCC)				67,130
Excess Overburden Removal	1,913,000	CY	1.60	3,060,800
Opinion of Probable Project Cost				**12,458,931**
Cost Per Acre-Foot of Total Storage				**3,031**
Cost Per Acre-Foot of Total Storage (not including Alluvial Storage)				**4,281**

SECTION 12 - FINANCIAL PLAN

The total project cost is estimated at $22,523,793 with 44% LTV financing, this creates a loan request of $9,900,000. Since we are constructing this storage to use in our agricultural operations, we are requesting a 10 year amortization at 1.25% annual interest rate.

We plan to fund our 56% plus other costs through the Company's cash on hand and availability of other cash resources through the equity markets. The table below is a summary.

TABLE 12.1

PROJECT FUNDING, SUMMARY FINANCIAL INFORMATION AND REPAYMENT

	2013	2014	2015	2016	2017
Farm Revenue	1,950,624	8,089,182	20,134,375	29,520,319	38,785,388
Farm Cost of Revenue	1,170,374	4,061,473	9,540,999	13,265,445	16,622,549
Farming Gross Margin	780,250	4,027,709	10,593,376	16,254,874	22,162,839
Debt service:					
CWCB - New Loan	-	(92,813)	(1,059,330)	(1,059,330)	(1,059,330)
Cash Flow After CWCB New Loan and Before G&A	780,250	3,934,896	9,534,046	15,195,544	21,103,508

As a precondition to closing on the CWCB loan and as part of Phase I of the ASF, Two Rivers will use a portion of its $12,533,793 in equity capital to purchase a majority interest (53.77% or 1,792 shares) of the Excelsior Irrigation Company and approximately 323 acres of land which is capable of developing into approximately 13,000 ac-ft of storage space to provide the CWCB with a first Deed of Trust as collateral for the CWCB Loan.

SECTION 13 - IMPLEMENTATION SCHEDULE

The Two Rivers Water Company plans to construct the Arkansas Storage Facility, Phase 1 as soon as practical. A proposed implementation schedule is presented in Table 13.1.

TABLE 13.1
PROJECT IMPLEMENTATION SCHEDULE

Item	Schedule Date
Loan Application and Feasibility Study to CWCB	April 2013
Feasibility Study Reviewed and Approved by CWCB	May 2013
Funding Approved by CWCB Board	May 2013
Final Project Engineering Design Started	June 2013
Permitting Started	June 2013
Project Design Completed	April 2014
Designs, Plans, and Specifications Submitted to SEO	May 2014
Flood Plan Permits Obtained	November 2013
Other Permits Obtained	February 2014
SEO Approves Project	July 2014
Bidding and Procurement	June 2014
Project Construction Started	August 2014
Project Construction Completed	August 2015

13.1 Permitting and Institutional Feasibility

Permitting from and coordination with a number of governmental agencies will be required to construct the Project. Following is a listing of the agencies and the anticipated permits that will be required.

13.1.1 Floodplain Permitting

Floodplain permitting requirements for the Project will consist of one or more of the following elements depending on the degree of impact:

- **Local Floodplain Development Permit**. Issued by Pueblo County and required in all cases regardless of the level of impact (or lack of) because a portion of the Project is within a FEMA mapped floodplain.
- **FEMA Conditional Letter of Map Revision (CLOMR)**. Typical FEMA regulations for Approximate A Zones require a CLOMR application only if it is determined that a rise greater than 1 foot will occur.

13.1.2 State Engineers Office

The dam and reservoir must be designed and constructed in accordance with the SEO *Rules and Regulations for Dam Safety and Dam Construction* (SEO, 2007). Review and approval of Project designs, plans, specifications, and construction by the SEO will be required.

Design, construction, and performance standards will need to be considered in accordance with *State Engineer Guidelines for Lining Criteria for Gravel Pits* (SEO, 1999). A Gravel Pit Well Permit will also be required.

13.1.3 Pueblo County

A county Special Use and Land Use 1041 permit will be required to construct the Project. The Pueblo County Planning Department will review the Project application and the County Commissioners will approve the permits based on recommendation of the Planning Department staff. The Pueblo County Planning Department staff will route the Special Use Permit application to all local and state agencies that might have comments on the Project.

SECTION 14 - LIMITATIONS

The information presented in this report is suitable for feasibility design purposes only. The information in this report is based primarily on data obtained from review of existing documents, data, and studies for the subject site. Significant additional data is needed to refine the concepts in this report. Also, the nature and extent of variations between specific subsurface data may not become evident until future phases of exploration and construction. Timely and comprehensive observation and evaluation of actual subsurface conditions, supported by appropriate field and laboratory testing, will be critical during future design and construction phases. Variations in the subsurface profile described herein should be anticipated.

RJH has endeavored to conduct our professional services for this Project in a manner consistent with a level of care and skill ordinarily exercised by members of the engineering profession currently practicing in Colorado under similar conditions as this project. RJH makes no other warranty, expressed or implied.

Opinions of Probable Project Costs presented in this report are based on our professional opinion of the cost to construct the Project as described in this report. The estimated costs are based on the sources of information described herein, and our knowledge of current construction cost conditions in the locality of the Project. Actual Project construction costs are affected by a number of factors beyond our control. Therefore, conditions and factors that arise as Project development proceeds through design and construction may result in construction costs that differ from the estimates documented in this report.

This report has been prepared for use by Two Rivers Water Company and for exclusive application to the Arkansas Storage Facility, Phase 1 Project.

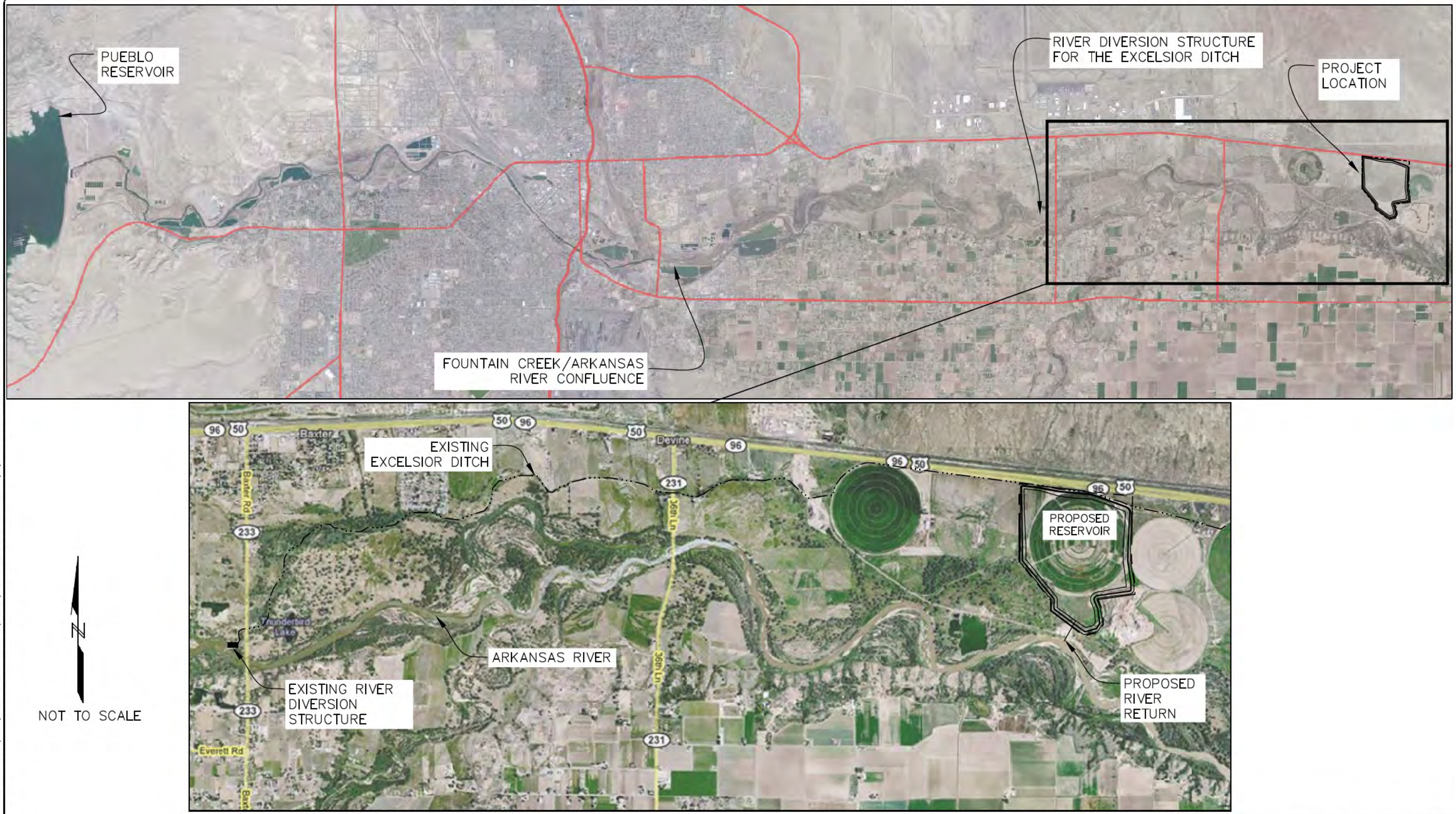
SECTION 15 - REFERENCES

Colorado Office of the State Engineer (SEO) (2007). *Rules and Regulations for Dam Safety and Dam Construction*.

Colorado Office of the State Engineer (SEO) (1999). *Guidelines for Lining Criteria for Gravel Pits*.

National Weather Service (NWS) (1973). *Precipitation-Frequency Atlas of the Western United States, Volume III, Colorado*.

U.S. Bureau of Reclamation (USBR) (2001). *Engineering Geology Field Manual.*



Two Rivers WATER & FARMING

RJH CONSULTANTS, INC.

ARKANSAS STORAGE FACILITY, PHASE 1 PUEBLO COUNTY, COLORADO	PROJECT LOCATION MAP AND KEY PROJECT COMPONENTS	
PROJECT NO. 13101	April 2013	**Figure 1**

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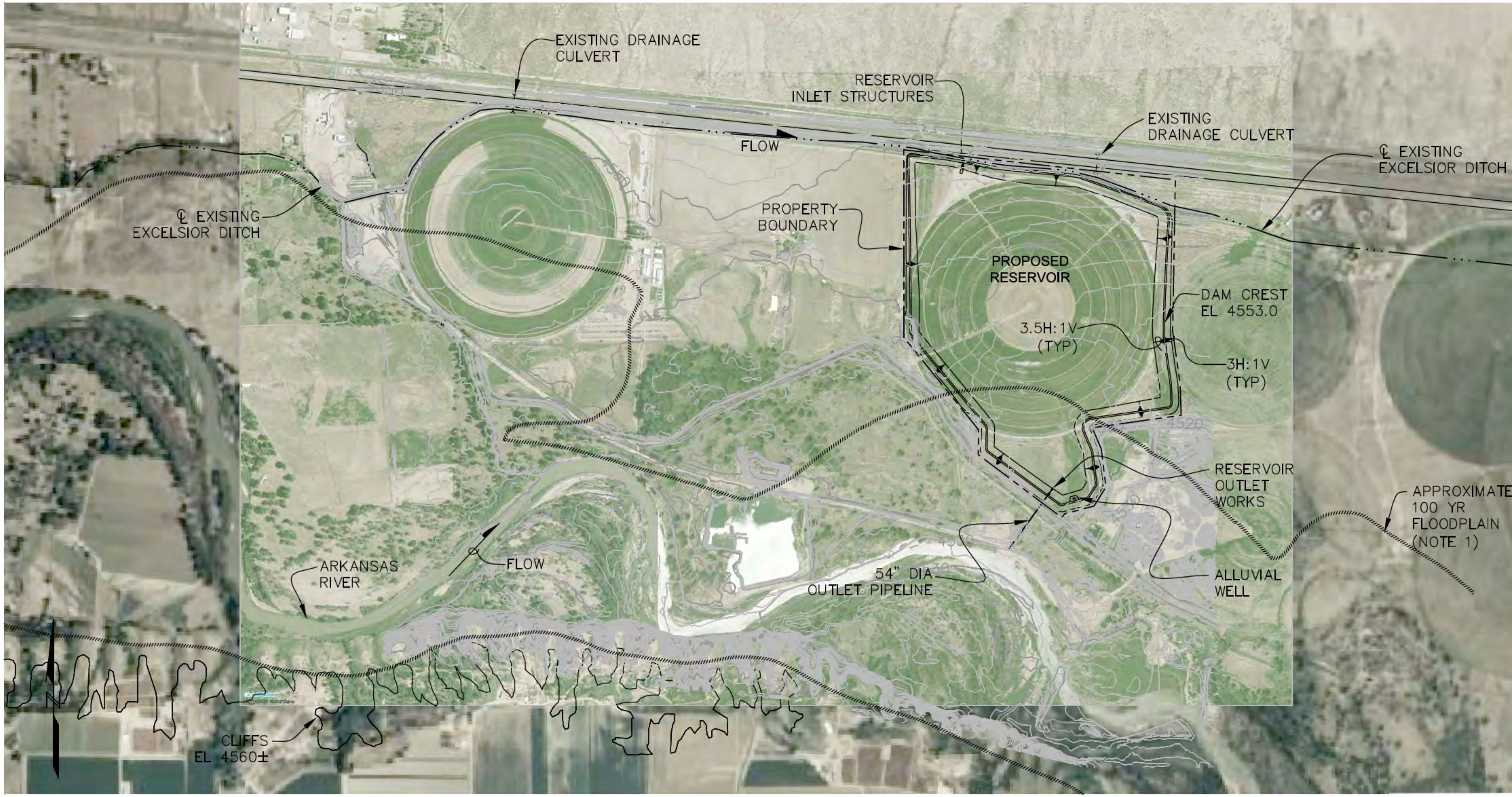

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NOTES:

1. 100–YEAR FLOODPLAIN DELINEATION BASED ON FEMA FLOOD MAP DETERMINED IN THE FLOOD INSURANCE STUDY BY APPROXIMATE METHODS. DELINEATION MAY BE SUBJECT TO CHANGE ONCE DETAILED ANALYSIS IS PERFORMED.

Two Rivers WATER & FARMING	RJH CONSULTANTS, INC.	ARKANSAS STORAGE FACILITY, PHASE 1 PUEBLO COUNTY, COLORADO	GENERAL SITE PLAN
		PROJECT NO. 13101	April 2013 **Figure 2**

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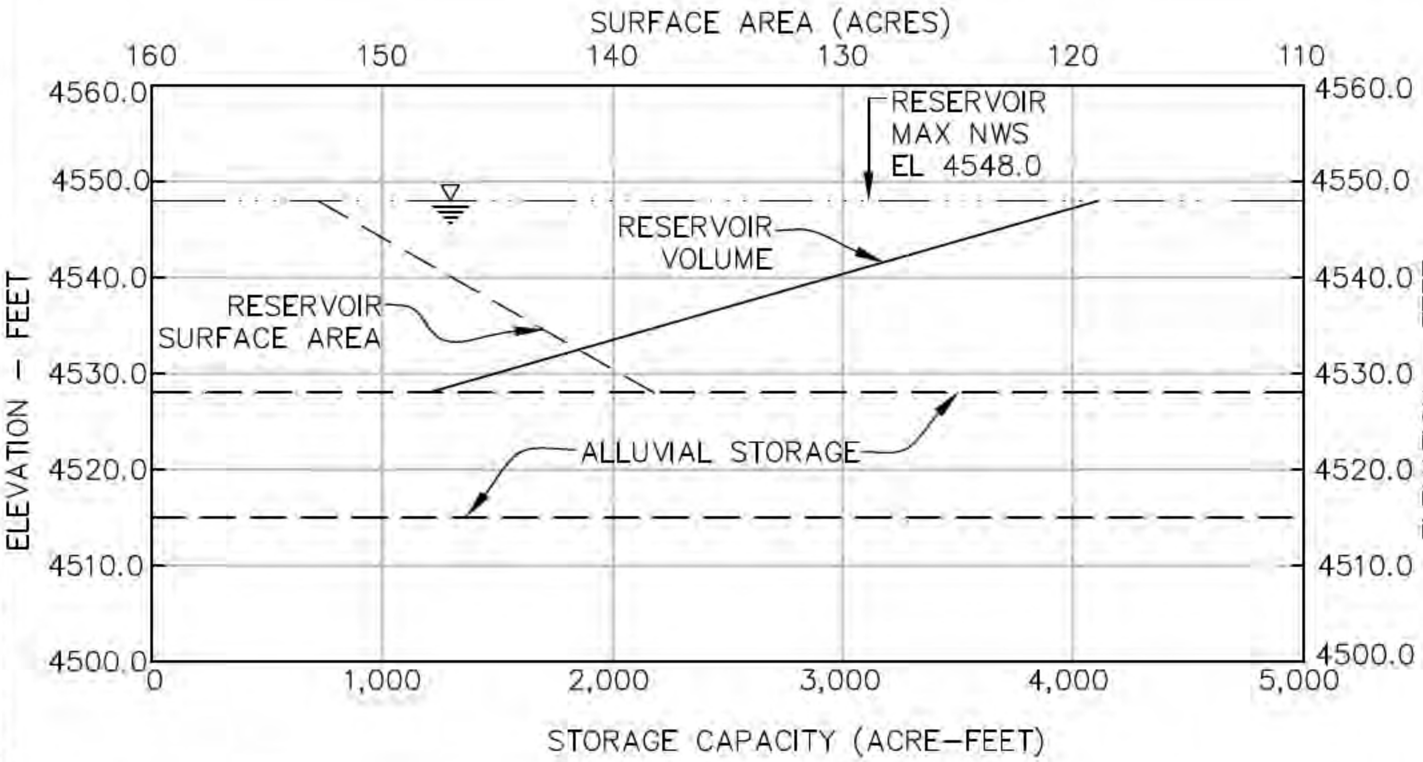

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Two Rivers Water & Farming

RJH CONSULTANTS, INC.

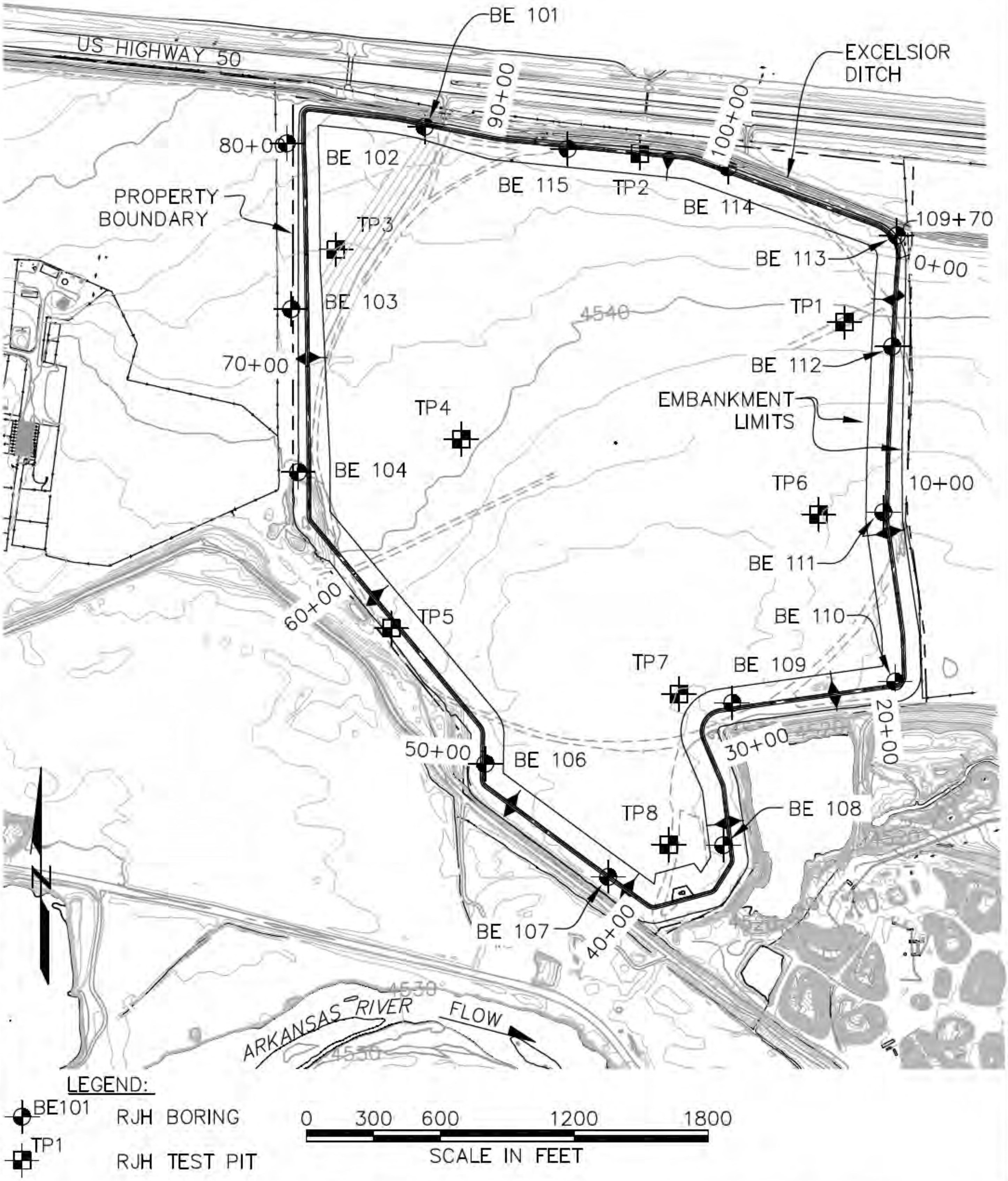
ARKANSAS STORAGE FACILITY, PHASE 1
PUEBLO COUNTY, COLORADO

ELEVATION-AREA-CAPACITY CURVE

PROJECT NO. 13101

April 2013

Figure 3



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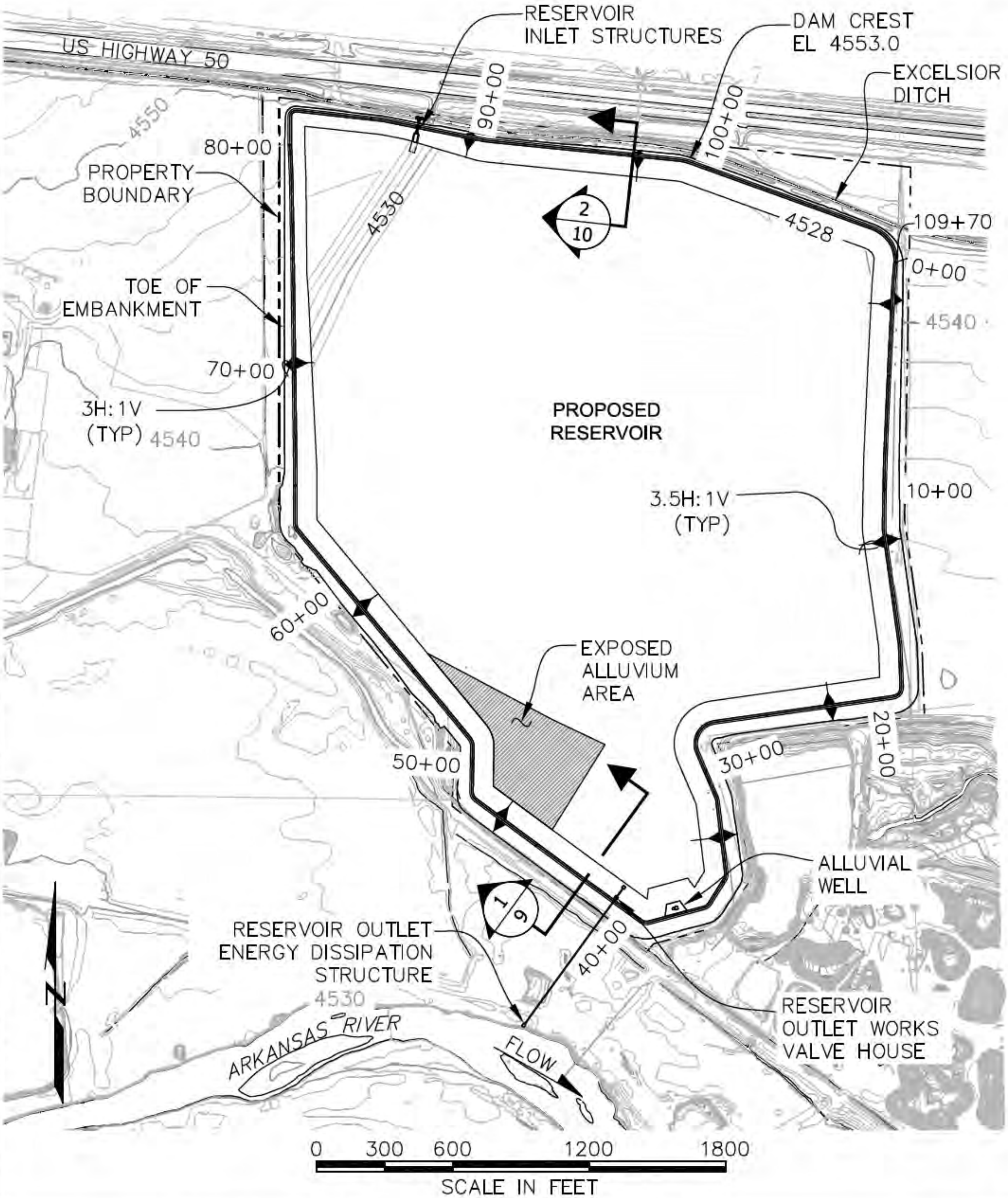
Two Rivers WATER & FARMING	RJH CONSULTANTS, INC.	ARKANSAS STORAGE FACILITY, PHASE 1 PUEBLO COUNTY, COLORADO	PLAN OF RJH BORINGS AND TEST PITS
		PROJECT NO. 13101	April 2013 Figure 4



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Two Rivers Water & Farming

RJH Consultants, Inc.

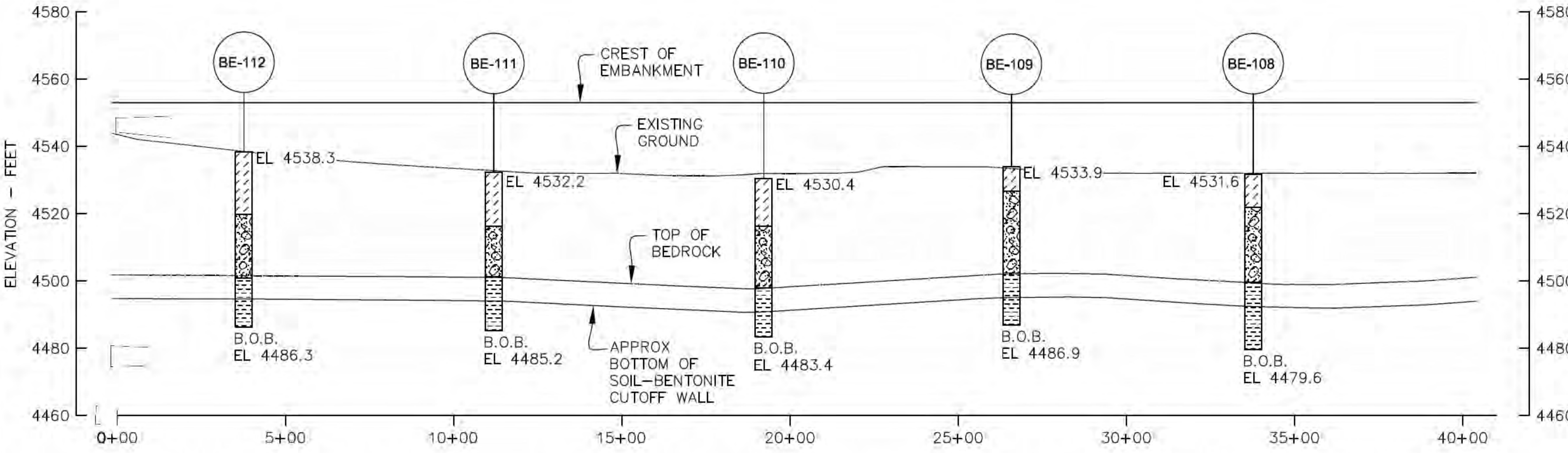
ARKANSAS STORAGE FACILITY, PHASE 1
PUEBLO COUNTY, COLORADO

GENERAL PLAN OF MODIFICATIONS

PROJECT NO. 13101

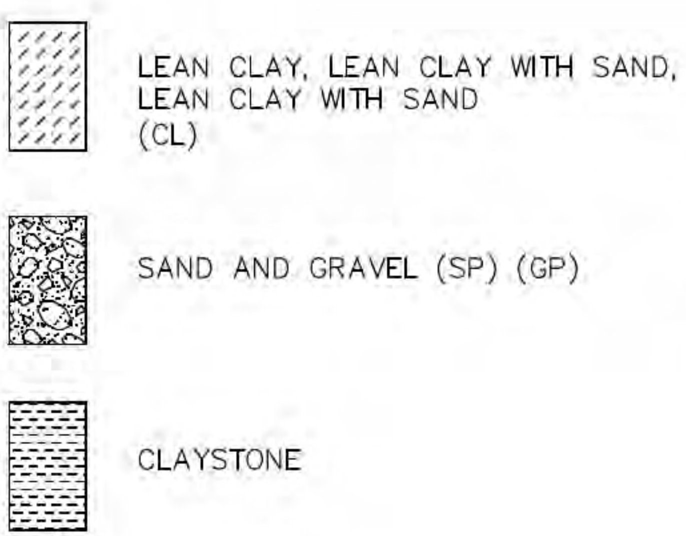
April 2013

Figure 5



PROFILE
STA 0+00 - 40+00

LEGEND:



Two Rivers Water & Farming	RJH Consultants, Inc.	ARKANSAS STORAGE FACILITY, PHASE 1 PUEBLO COUNTY, COLORADO	DAM EMBANKMENT CENTERLINE PROFILE 1 OF 3
		PROJECT NO. 13101	April 2013 Figure 6

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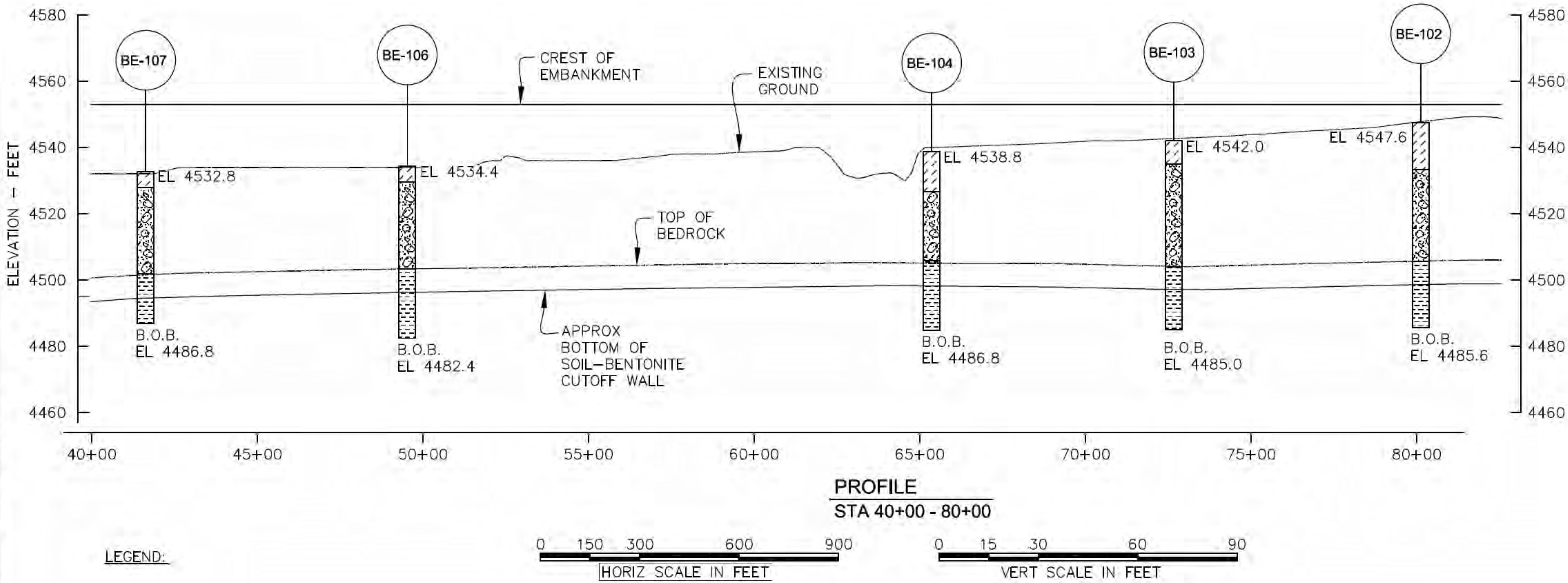

PROFILE
STA 40+00 - 80+00

LEGEND:

LEAN CLAY, LEAN CLAY WITH SAND, LEAN CLAY WITH SAND (CL)

SAND AND GRAVEL (SP) (GP)

CLAYSTONE

Two Rivers WATER & FARMING

RJH CONSULTANTS, INC.

ARKANSAS STORAGE FACILITY, PHASE 1 PUEBLO COUNTY, COLORADO	DAM EMBANKMENT CENTERLINE PROFILE 2 OF 3
PROJECT NO. 13101	April 2013 Figure 7

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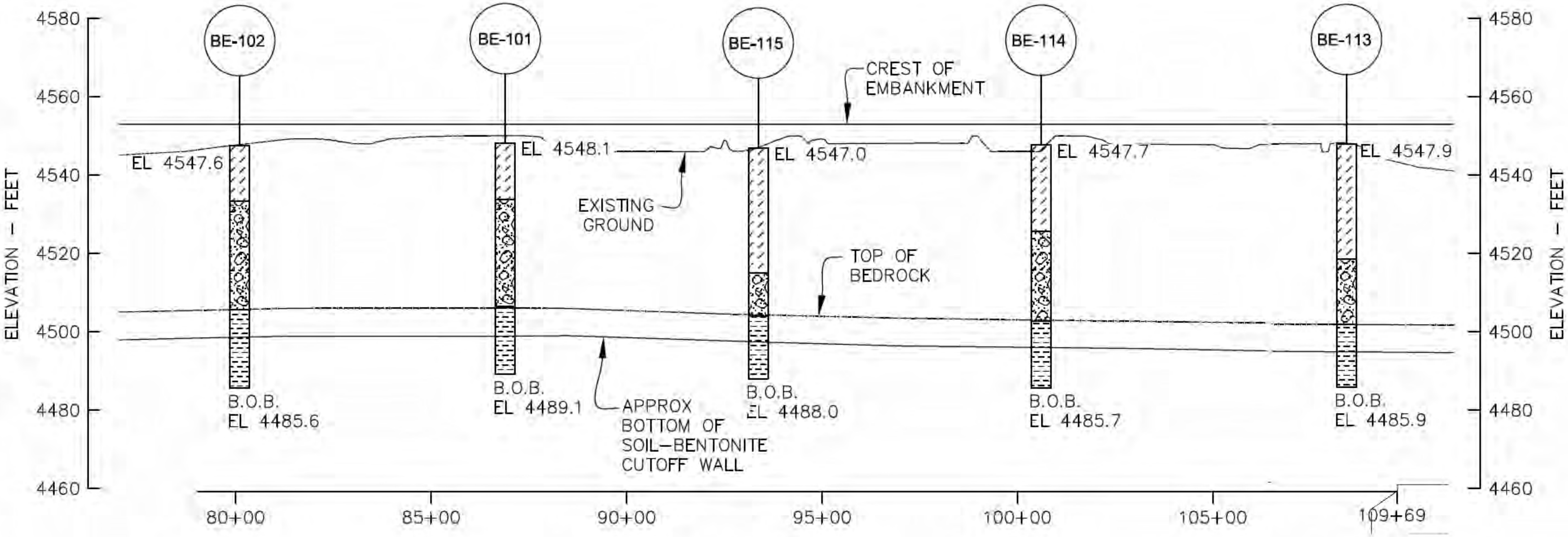

PROFILE
STA 80+00 - 109+69

LEGEND:

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0 15 30 60 90
VERT SCALE IN FEET

LEAN CLAY, LEAN CLAY WITH SAND, LEAN CLAY WITH SAND (CL)

SAND AND GRAVEL (SP) (GP)

CLAYSTONE

Two Rivers WATER & FARMING

RJH CONSULTANTS, INC.

ARKANSAS STORAGE FACILITY, PHASE 1
PUEBLO COUNTY, COLORADO

DAM EMBANKMENT CENTERLINE PROFILE 3 OF 3

PROJECT NO. 13101

April 2013

Figure 8

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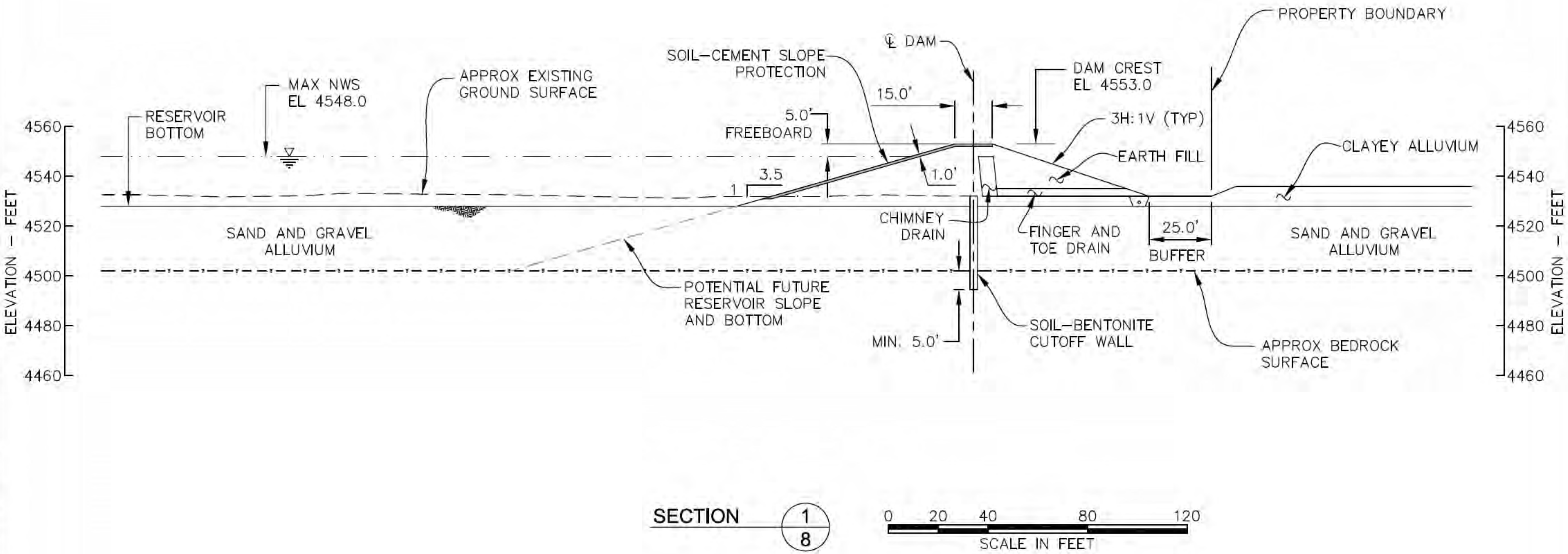

Two Rivers Water & Farming	RJH Consultants, Inc.	ARKANSAS STORAGE FACILITY, PHASE 1 PUEBLO COUNTY, COLORADO	TYPICAL MAXIMUM DAM EMBANKMENT SECTION
		PROJECT NO. 13101	April 2013 Figure 9

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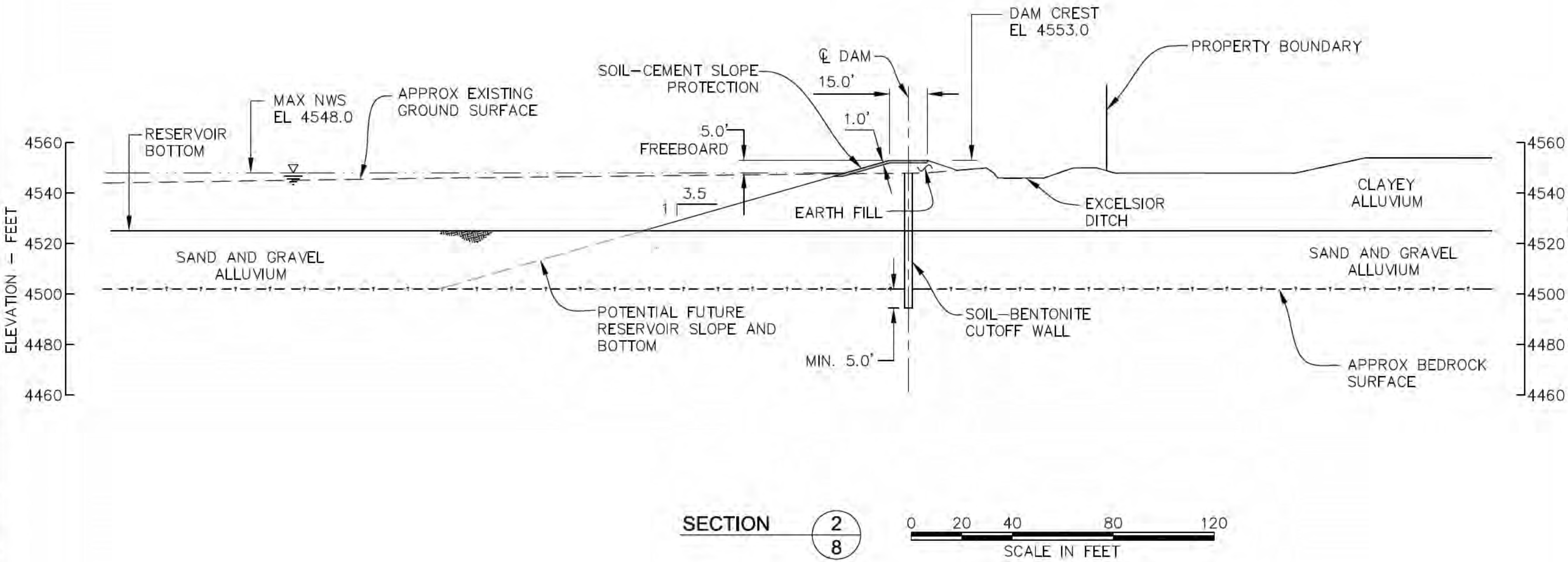

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Two Rivers Water & Farming

RJH Consultants, Inc.

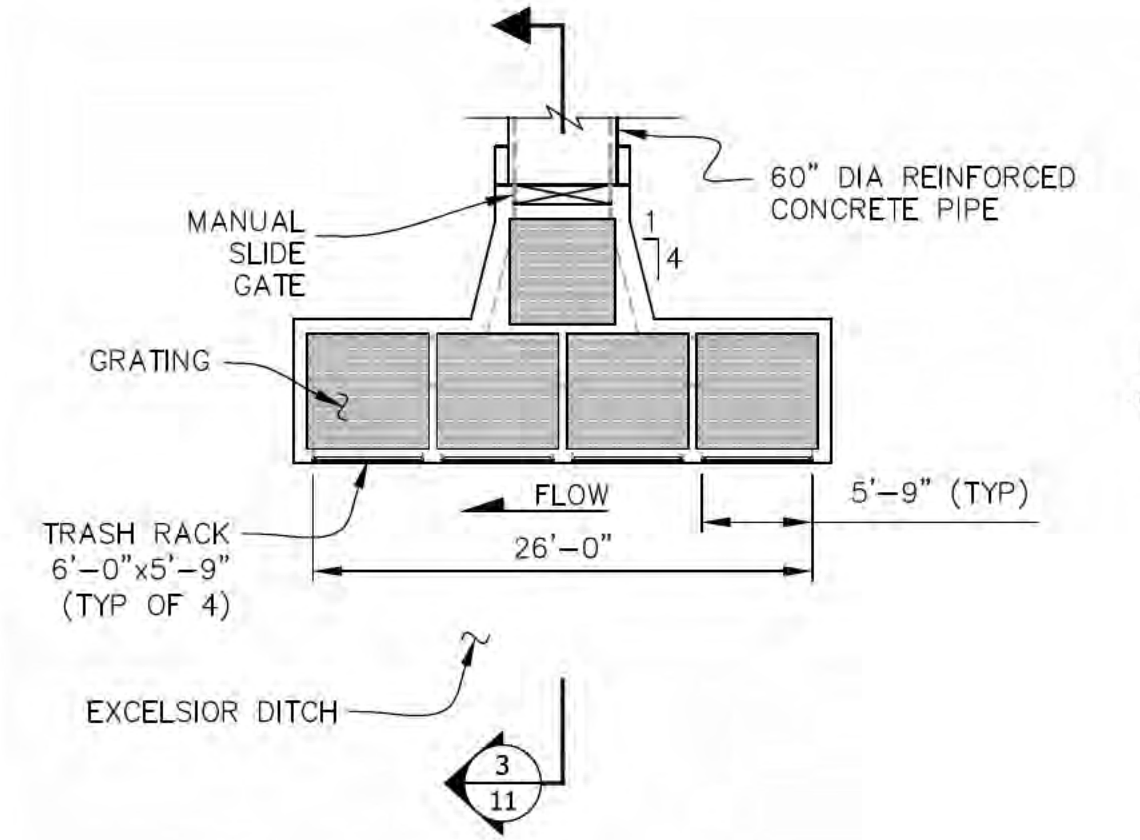
ARKANSAS STORAGE FACILITY, PHASE 1
PUEBLO COUNTY, COLORADO

PROJECT NO. 13101

NORTH DAM EMBANKMENT SECTION

April 2013

Figure 10



PLAN
DITCH INTAKE STRUCTURE

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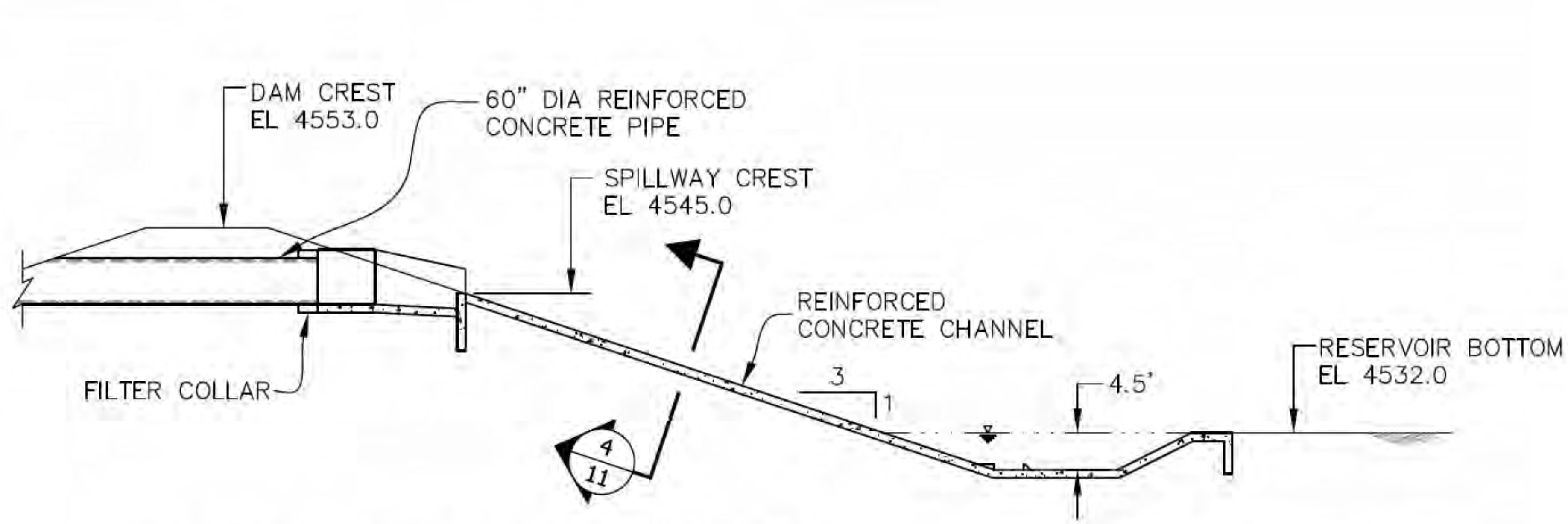
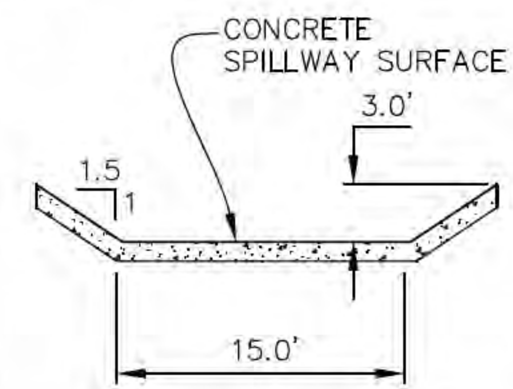


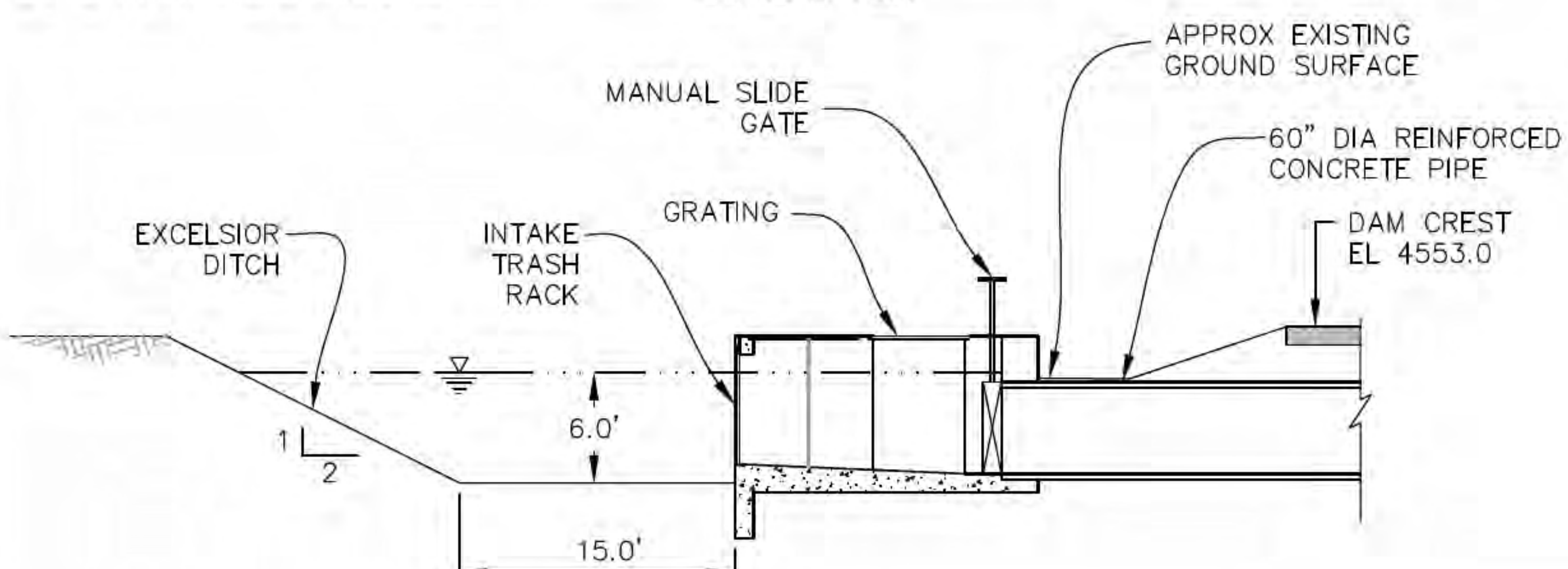
RESERVOIR INLET RUNDOWN STRUCTURE
DAM EMBANKMENT

0 10 20 40 60
SCALE IN FEET



SECTION 4/11
RUNDOWN

0 5 10 20 30
SCALE IN FEET



SECTION 3/11

0 5 10 20 30
SCALE IN FEET

NOTE:

1. RAILING NOT SHOWN.

Two Rivers Water & Farming

RJH Consultants, Inc.

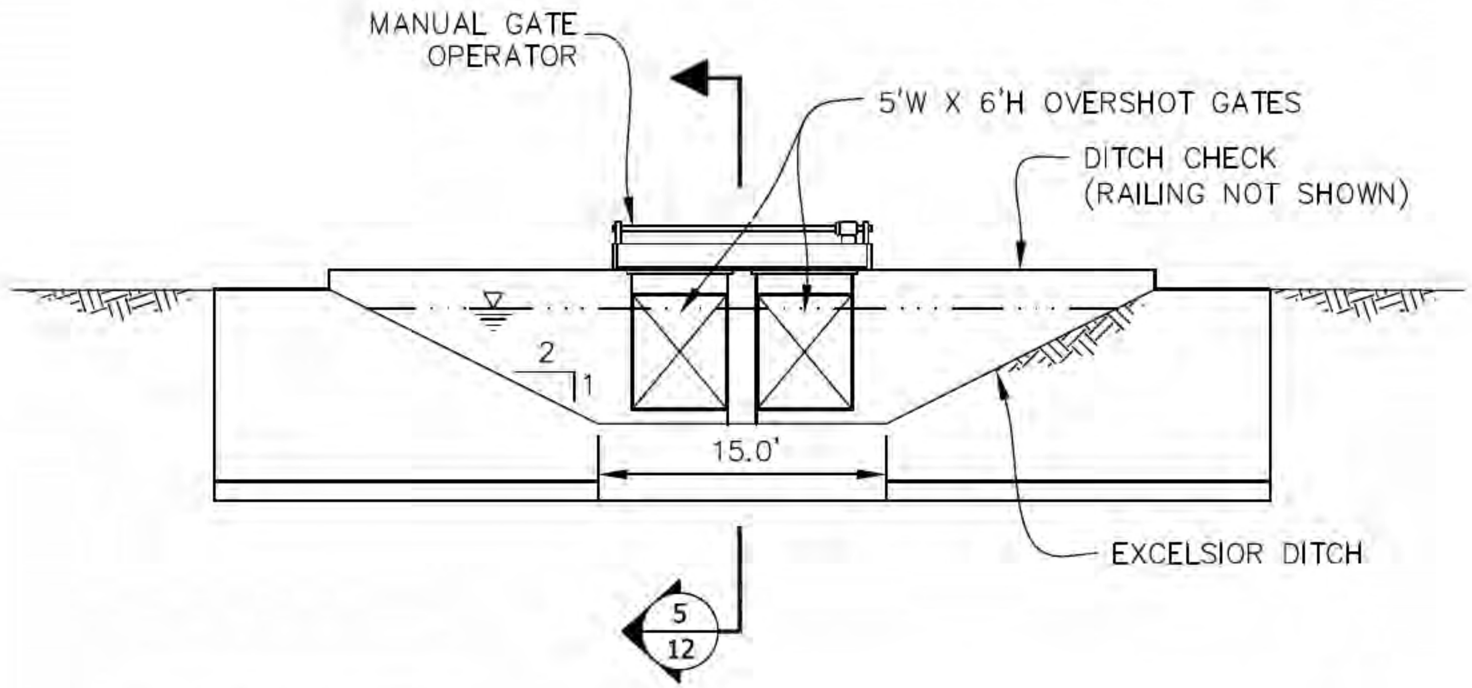
ARKANSAS STORAGE FACILITY, PHASE 1
PUEBLO COUNTY, COLORADO

PROJECT NO. 13101

RESERVOIR INLET STRUCTURES

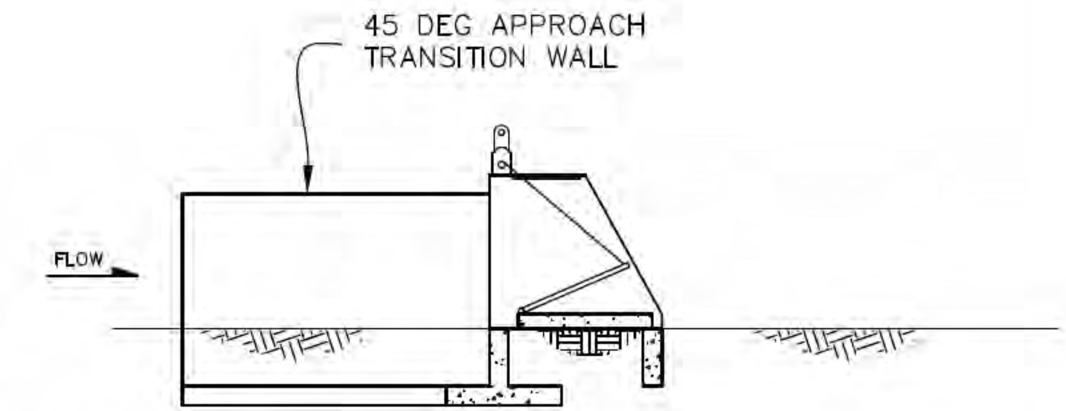
April 2013

Figure 11



FRONT ELEVATION
DITCH CHECK STRUCTURE

0 5 10 20 30
SCALE IN FEET

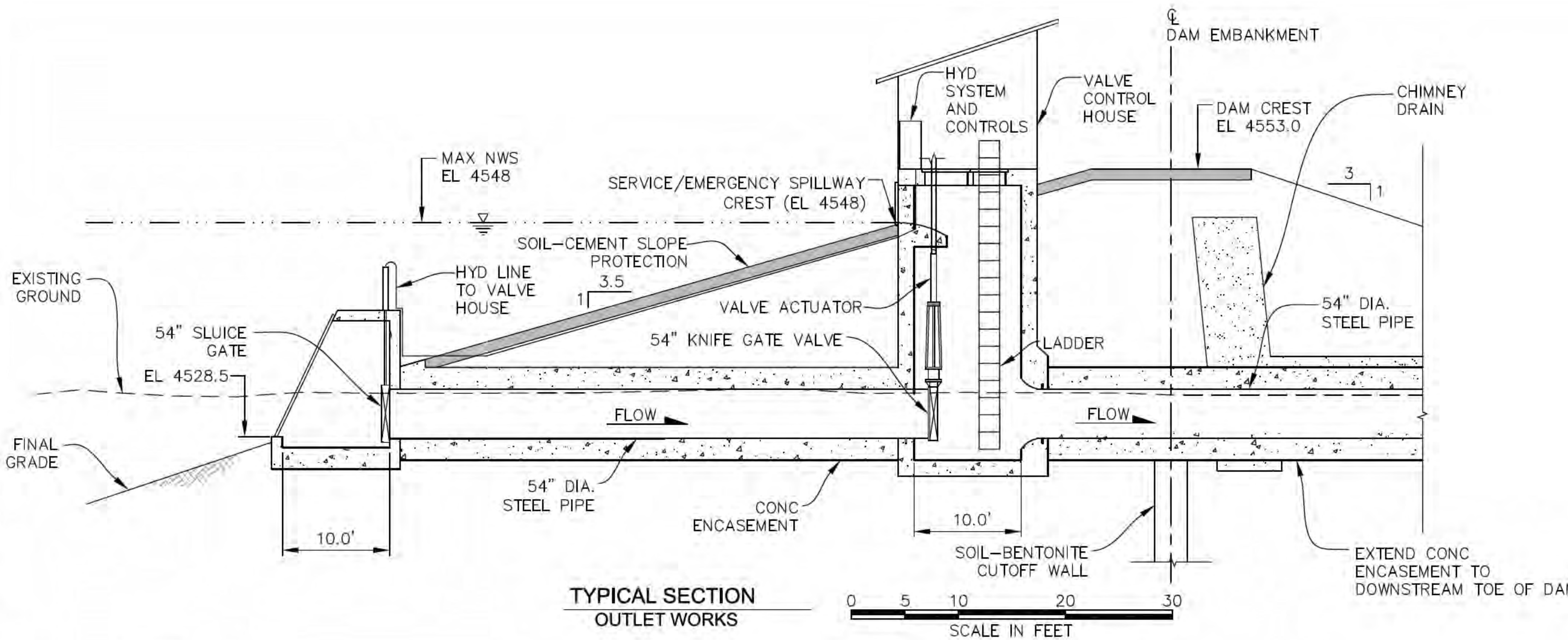


SECTION 5/12

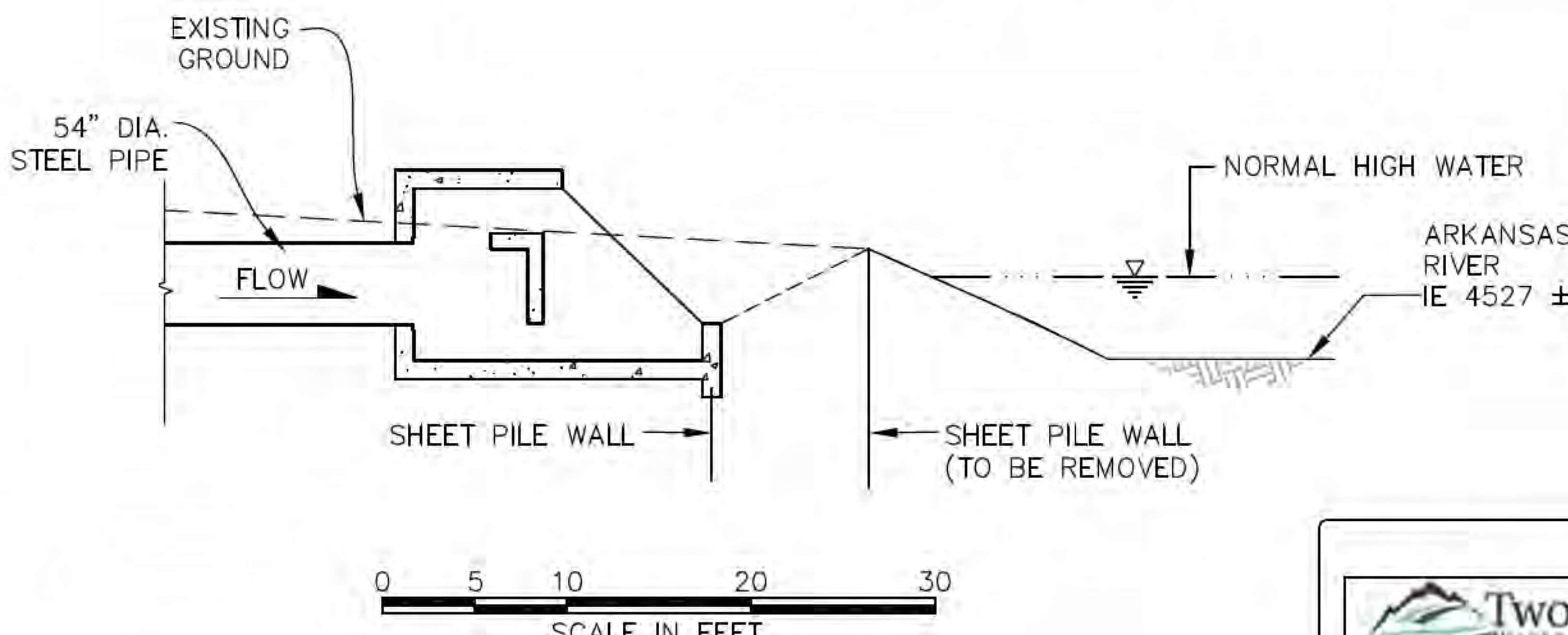
0 5 10 20 30
SCALE IN FEET

Two Rivers Water & Farming	RJH Consultants, Inc.	ARKANSAS STORAGE FACILITY, PHASE 1 PUEBLO COUNTY, COLORADO	DITCH CHECK STRUCTURE
		PROJECT NO. 13101	April 2013 **Figure 12**

P:\13101 - BLUE GRASS RESERVOIR\CAD\FEAS_FIGS_EAST_PIT\FINAL\13101_EP_FIGURE_11-12.DWG 4/8/2013 12:06 PM



TYPICAL SECTION
OUTLET WORKS



Two Rivers Water & Farming	RJH Consultants, Inc.	ARKANSAS STORAGE FACILITY, PHASE 1 PUEBLO COUNTY, COLORADO	OUTLET WORKS STRUCTURES
		PROJECT NO. 13101	April 2013 **Figure 13**

P:\13101 – BLUE GRASS RESERVOIR\CAD\FEAS_FIGS_EAST_PIT\FINAL\13101_EP_FIGURE_13.DWG 4/8/2013 12:06 PM

APPENDIX A

KLEINFELDER GEOTECHNICAL INVESTIGATION

May 21, 2007
Project No:

Southwest Farms, Inc.
33601 United Avenue
Pueblo, Colorado 81001

Attention: Mr. John Sliman

Subject : Subsurface exploration
Sections 34 & 35, T-20-W, R-63-W of the 6 P.M.
Pueblo County, Colorado

Dear Mr. Sliman:

Kleinfelder, Inc. has completed the initial exploration borings at above referenced site. The exploration included the drilling of eleven exploration borings to bedrock and logging of layers encountered.

Test boring information includes location, thickness of overburden, sand & gravel alluvium thickness, depth to bedrock and groundwater levels are attached. Should you have any questions regarding this investigation or need additional information or services, please contact this office at (719) 546-1150.

Respectfully submitted,
KLEINFELDER, INC.

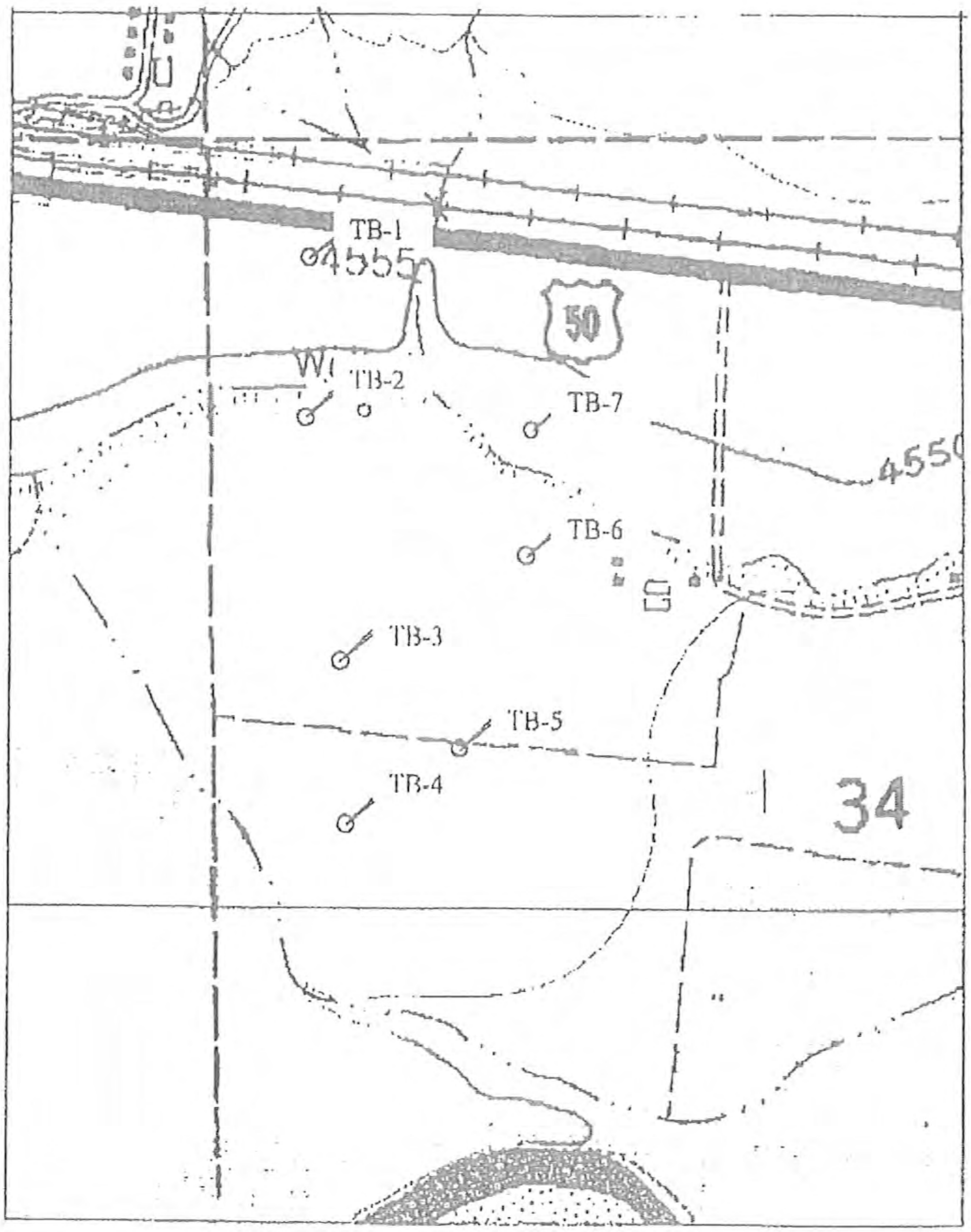
Larry J. Frank
Senior Professional

Project: Southwest Farms, Inc.

Project No:

Test Boring Location Drawing

Section 34, T-20-S, R-63-W of the 6th P.M., Pueblo County, Colorado

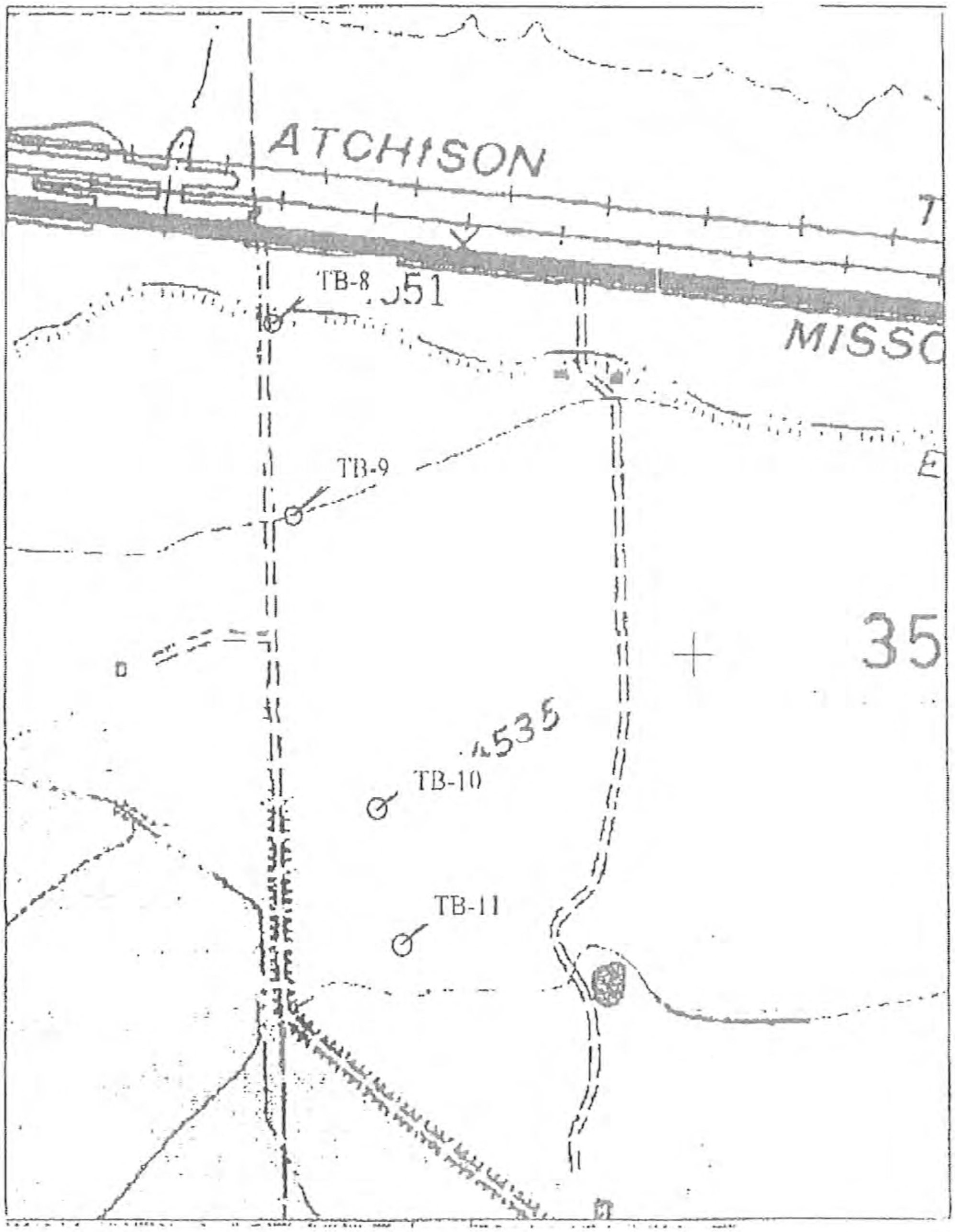


Project: Southwest Farms. Inc.

Project No:

Test Boring Location Drawing

Section 35, T-20-S, R-63-W of the 6th P.M., Pueblo County, Colorado



Project: Southwest Farms, Inc. **Date:** 18-May-07

Project No.

Test Boring No.	GPS Cordinates		Overburden		Sand & Gravel Alluvium		Bedrock		Groundwater
	North	West	Thickness - ft.	Soil Type	Thickness - ft.	Soil Type	Depth-ft.	Formation	Depth -ft.
WEST 1	38° 16.351'	104° 26.401'	14	CL	21.5*	SP-SP/SM	35.5	Pierre Shale	14.5
2	38° 16.246'	104° 26.404'	4	CL	28	SP-SP/SM	32	Pierre Shale	6.5
3	38° 16.083'	104° 26.374'	5	CL/SC	23.5	SP-SP/SM	28.5	Pierre Shale	6
4	38° 15.977'	104° 26.368'	7	CL	21	SP-SP/SM	28	Pierre Shale	6.5
5	38° 16.026'	104° 26.267'	9	CL/ML	19	SP-SP/SM	28	Pierre Shale	7
6	38° 16.154'	104° 26.208'	8	CL	23.5	SP-SP/SM	31.5	Pierre Shale	8.5
7	38° 16.237'	104° 26.204'	14.5	CL	21.5	SP-SP/SM	36	Pierre Shale	14.5
EAST 8	38° 16.223'	104° 26.370'	9	CL	31	SP-SP/SM	40	Pierre Shale	16
9	38° 16.094'	104° 26.354'	6.5	CL	33	SP-SP/SM	39.5	Pierre Shale	14
10	38° 15.898'	104° 26.280'	17	CL	12	SP-SP/SM	29	Pierre Shale	8.5
11	38° 15.808'	104° 26.259'	6.5	CL	23.5	SP-SP/SM	30	Pierre Shale	8.5
			100.5 / 11 = 9.14		257.5 / 11 = 23.41		358 / 11 = 32.5		
		EAST AVG	9.75'	(CL)	24.9'	SP-SP/SM	TOTAL DEPTH = 34.6' (AVG)		

Notes: All depths taken from the existing ground surface
Groundwater depth was at time of drilling
* Lean clay layer 3 ft thick and 3 ft below top of alluvium

GW = 11.75'

OB $\frac{425 \times 9 \times 43560}{27}$ = 6,171,000 cy > [illegible]
35,000/[illegible]

S&G $\frac{425 \times 23.41 \times 43560}{20}$ = 21,669,[illegible] tons x 1.5 = 32,500,000
[illegible]

APPENDIX B

SUMMARY OF LABORATORY TEST RESULTS

Table 1
Summary of Index Test Results

Boring/Test Pit ID	Sample ID[1]	Sample Depth Interval (ft)	General Material Description	Natural Moisture Content (%)	Dry Unit Weight (lb/ft^3)	Atterberg Limits: Liquid Limit, LL (%)	Atterberg Limits: Plastic Index, PI (%)	Gradation: % Gravel (+No. 4)	Gradation: % Sand (-No. 4 to +No. 200)	Gradation: % Fines (-No. 200)
ALLUVIUM										
BE-104	BU-1	0.0-4.0	Clayey Sand with Gravel			43	19	28	33	39
BE-104	S-6	24.0-25.5	Narrowly Graded Sand with Gravel					40	58	2
BE-108	S-6	24.2-24.9	Widely Graded Sand with Gravel					47	51	2
BE-110	BU-1	0.0-4.0	Lean Clay with Sand			43	26	2	20	78
BE-110	S-6	24.6-25.5	Widely Graded Gravel with Silt and Sand					58	35	7
BE-110	S-7	29.0-30.4	Widely Graded Sand with Silt					0	95	5
BE-111	BU-1	0.0-4.0	Lean Clay			47	29	0	4	96
BE-113	CA-3	9.52-9.85	Fat Clay	20.1	106	50	32			
BE-114	S-5	19.0-20.5	Fat Clay			62	46	0	1	99
BE-114	S-8	34.0-34.9	Widely Graded Sand					5	95	0
BE-115	CA-4	14.33-14.67	Lean Clay with Sand	21	104	42	25			
TP-3	BU-1	1.0-5.0	Sandy Lean Clay	20.9		38	24	0	7	93
Composite-1	1	1.0-10.5	Lean Clay	23		42	29	36	2	62
PIERRE SHALE										
BE-104	R	37.9-38.2	Claystone			31	12	0	8	92
BE-104[3]	R	38.3-38.9	Claystone			31	12	0	8	92
BE-106	R	37.0-38.3	Sandy Claystone			29	13	0	47	53
BE-110	R	35.5-36.4	Sandy Claystone			27	11	0	26	74
BE-111	R	35.6-36.5	Sandy Claystone			28	11	0	11	89
BE-114	R	52.2-52.9	Claystone			29	13	0	9	91

Notes:

1. BU = Bulk, CA = California Sampler, S = Standard Split-Spoon Sampler, R = Core Run.
2. Composite-1 consists of a mixture of TP-1, BU-1, 1.0-10.5 ft. and TP-6, BU-1, 1.0-3.0 ft.
3. Sample combined with BE-104 (37.9-38.2).

Table 2
Summary of Strength, Compaction, and Permeability Results

Boring/Test Pit ID	Sample ID[(1)]	Sample Depth Interval (ft)	General Material Description	Effective Strength		Total Strength		Unconfined Compressive Strength (psf)	Compaction		Permeability (cm/s)
				Φ' (deg.)	c' (psf)	Φ_T (deg.)	c_T (psf)		Optimum Moisture Content (%)	Max. Dry Density (pcf)	
ALLUVIUM											
BE-113	CA-3	9.52-9.85	Lean Clay					10,800			
BE-115	CA-4	14.33-14.67	Lean Clay with Sand					3,900			
TP-3	BU-1	1.0-5.0	Sandy Lean Clay	24.3	228				17.9	109.2	1.1 E-07
Composite-1[(2)]	1	1.0-10.5	Lean Clay	25.4	192				18.1	108.4	4.9 E-06
PIERRE SHALE											
BE-104	R	37.3-37.9	Claystone					7,810[(3)]			
BE-106	R	38.3-38.9	Sandy Claystone					123,000			
BE-110	R	35.5-36.4	Sandy Claystone					79,700			
BE-111	R	35.6-36.5	Sandy Claystone					65,600			
BE-114	R	52.2-52.9	Claystone					92,300			

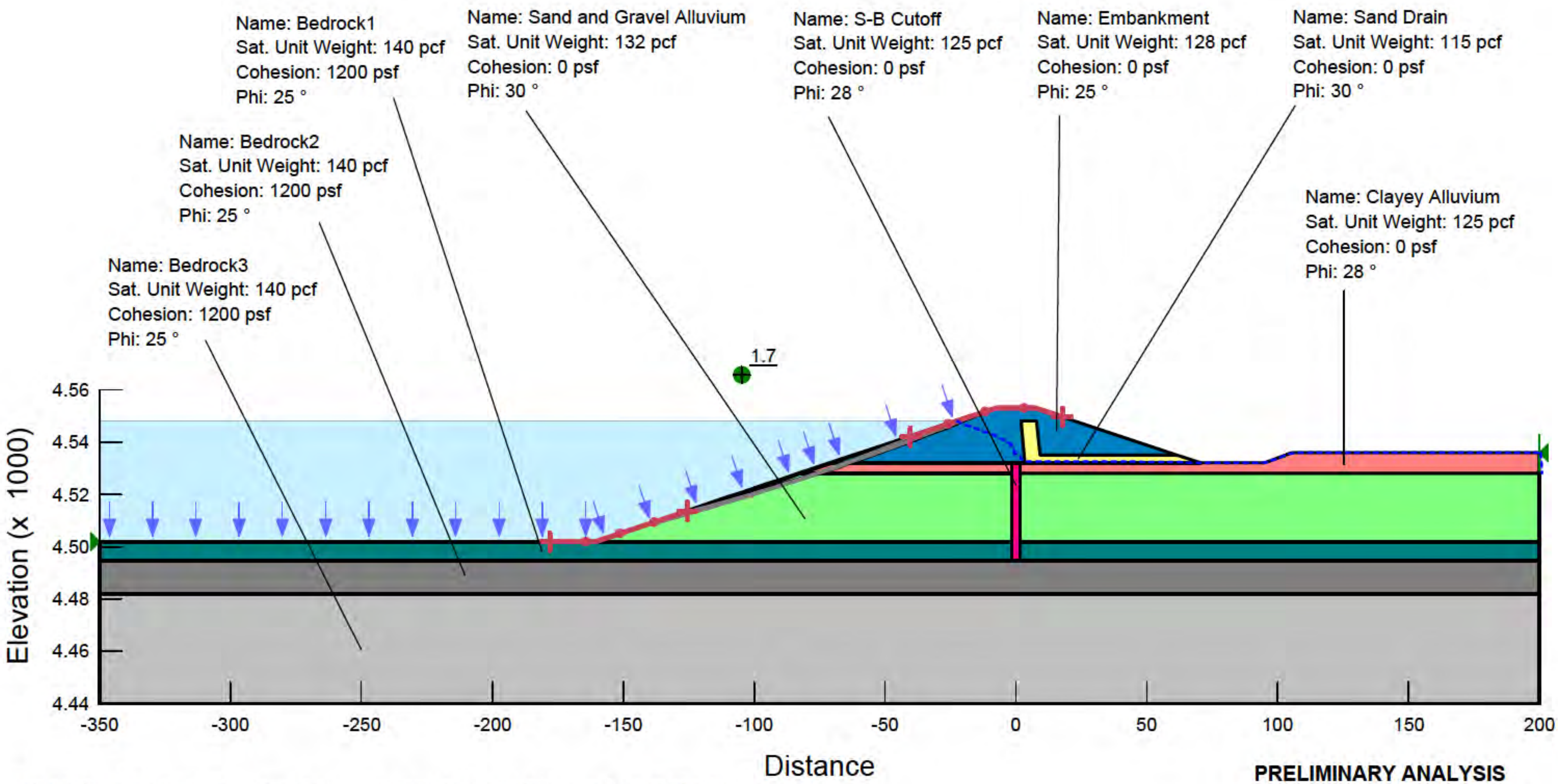
Notes:

1. BU = Bulk, CA = California Sampler, R = Core Run.
2. Composite-1 consists of a mixture of TP-1, BU-1, 1.0-10.5 ft. and TP-6, BU-1, 1.0-3.0 ft.
3. Sample fractured prior to testing.

APPENDIX C

SLOPE STABILITY ANALYSES

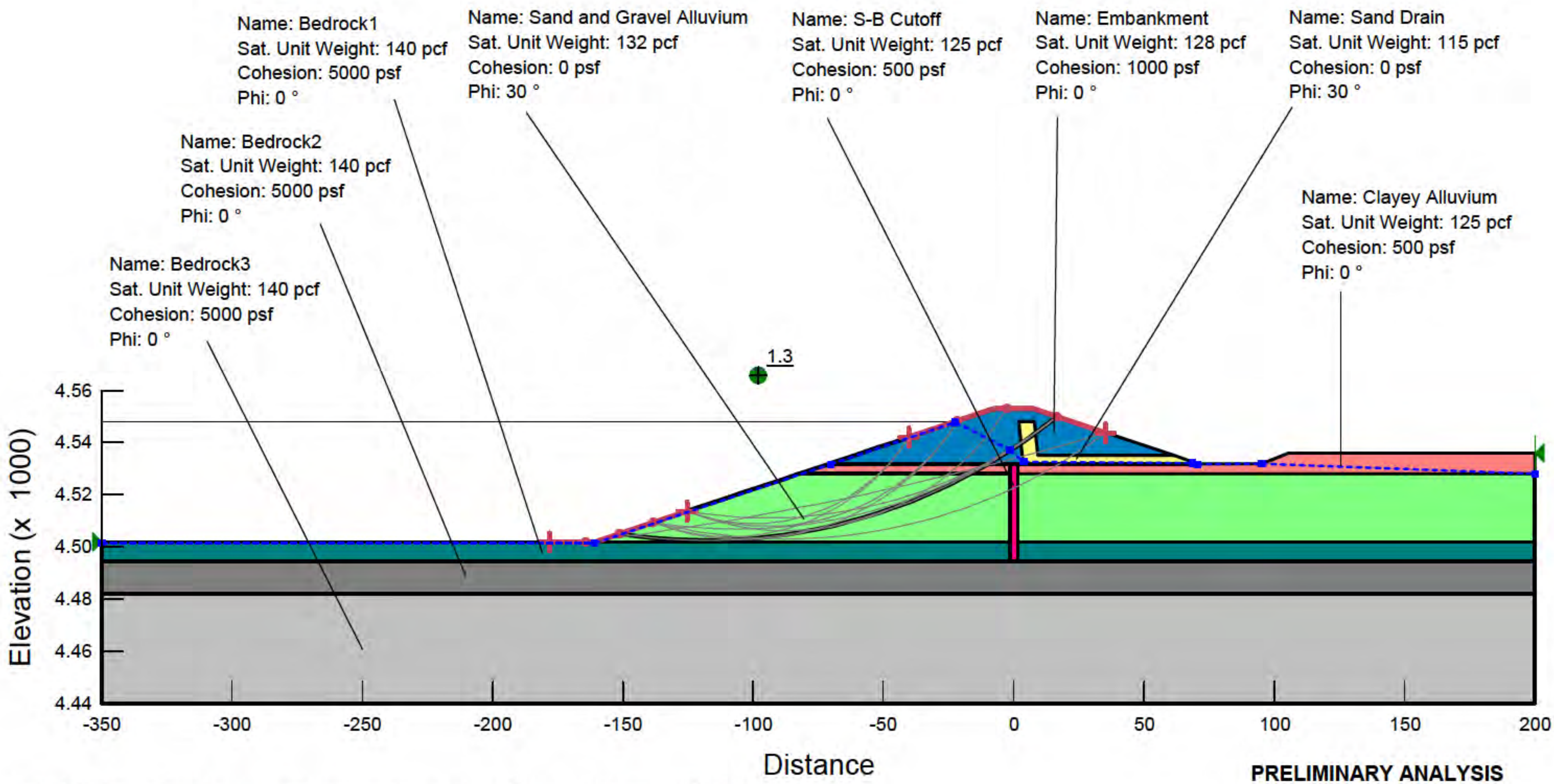
13101 Blue Grass Reservoir
Slope Stability, Steady State Seepage
Max Embankment Section (South)



PRELIMINARY ANALYSIS

P:\13101 - Blue Grass Reservoir\Engineering\Geotechnical\Seepage-Stability\13101_South_SS.gsz

13101 Blue Grass Reservoir
Slope Stability, Rapid Draw Down
Max Embankment Section (South)



PRELIMINARY ANALYSIS

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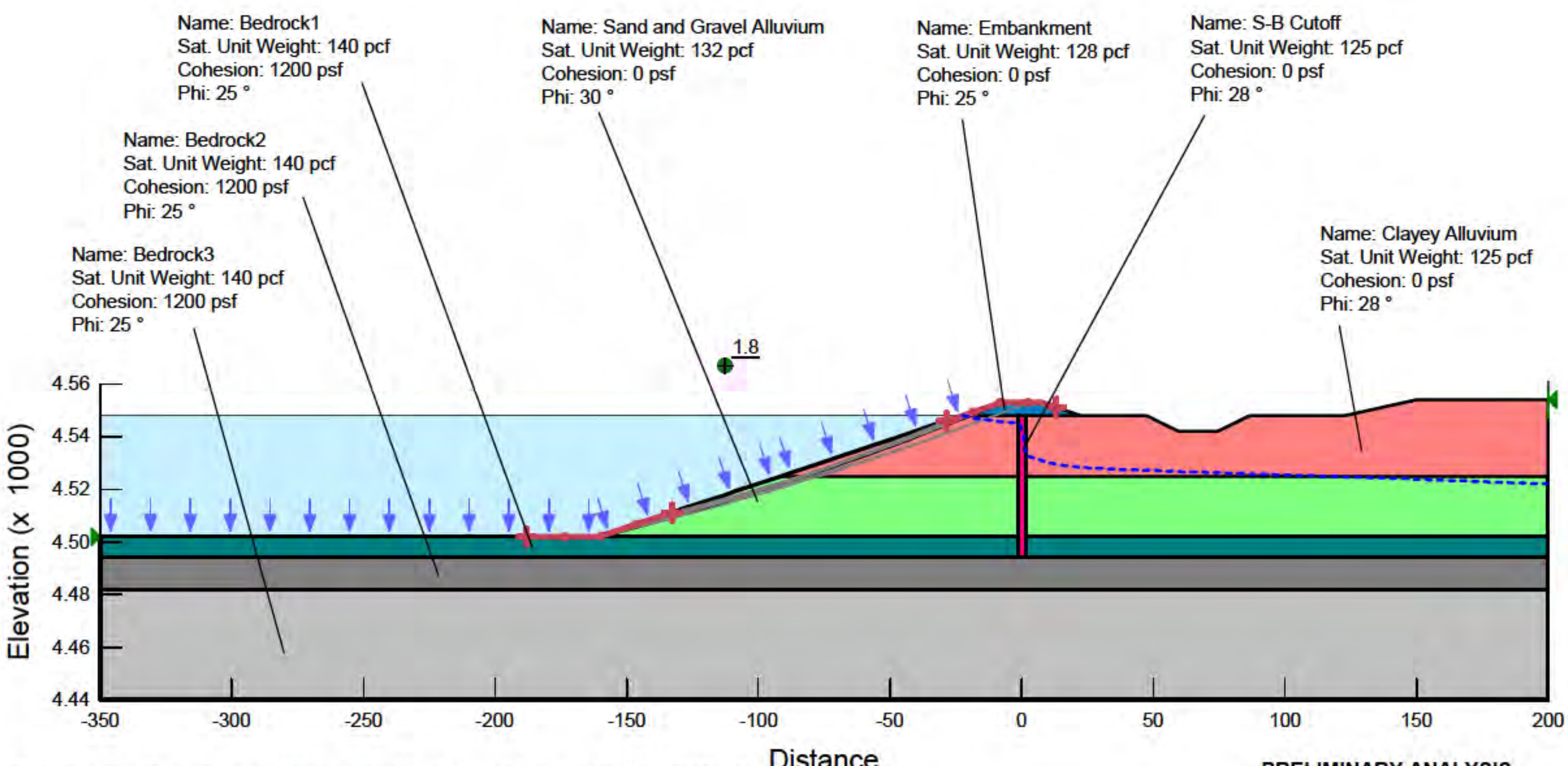
13101 Blue Grass Reservoir
Slope Stability, Steady State Seepage
North Section



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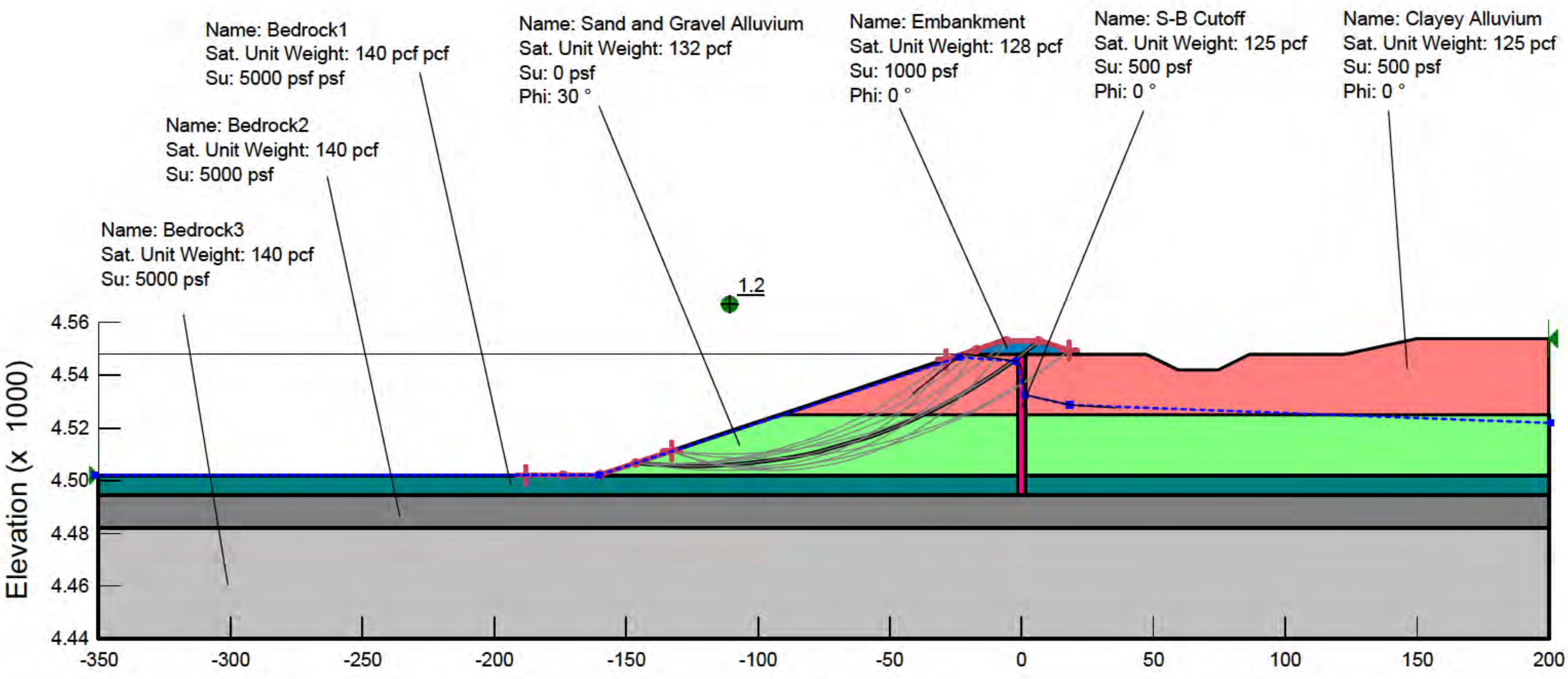
PRELIMINARY ANALYSIS

13101 Blue Grass Reservoir
Slope Stability, Rapid Drawdown
North Section



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PRELIMINARY ANALYSIS

APPENDIX D

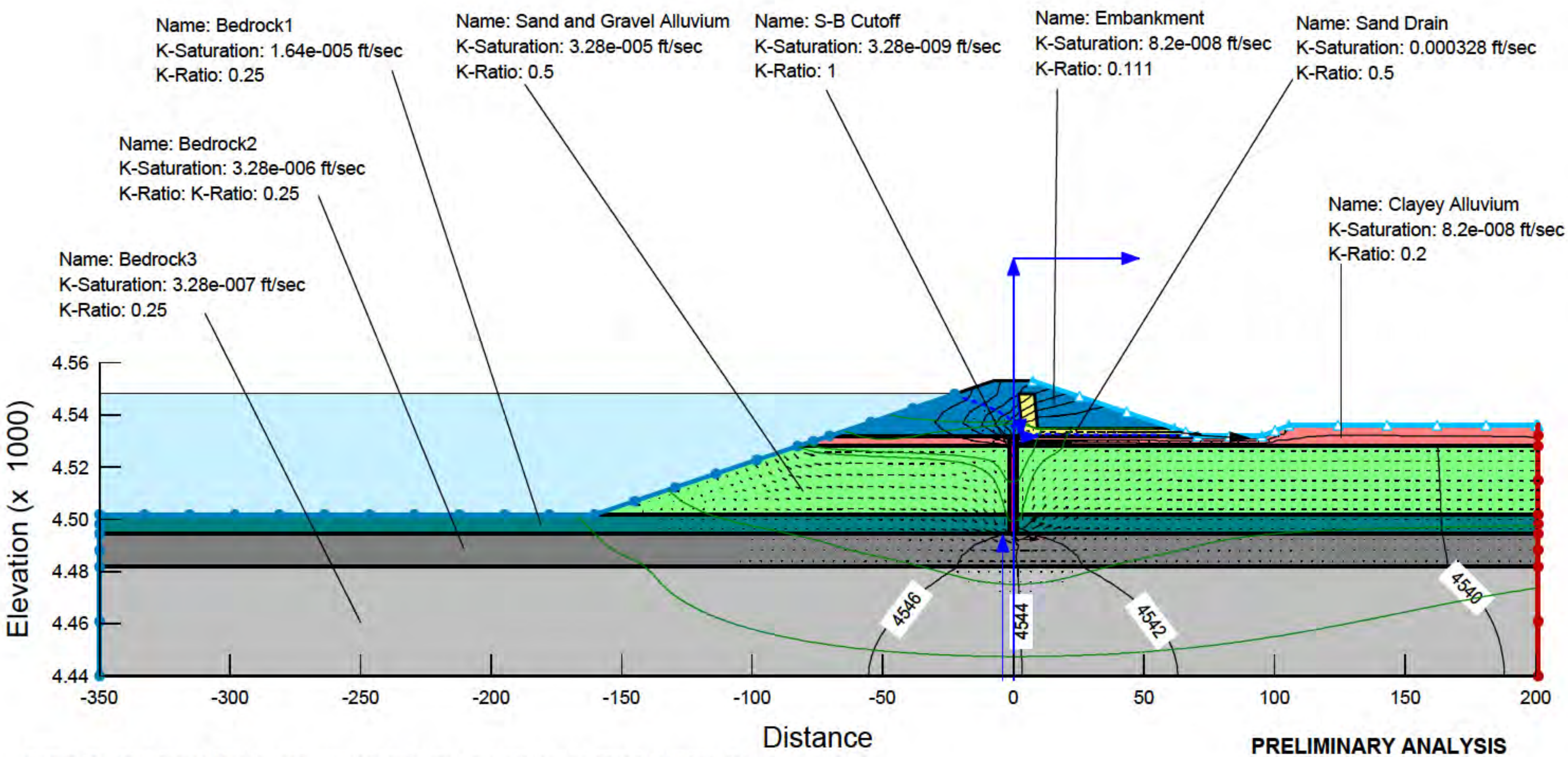
SEEPAGE ANALYSES

13101 Blue Grass Reservoir
Steady State Analysis
Max Embankment Section (South)

Total Seepage: 1.1486083e-005 ft³/sec

Seepage Into Drain: 7.4880899e-007 ft³/sec

Seepage Below Cutoff: 4.0767185e-006 ft³/sec

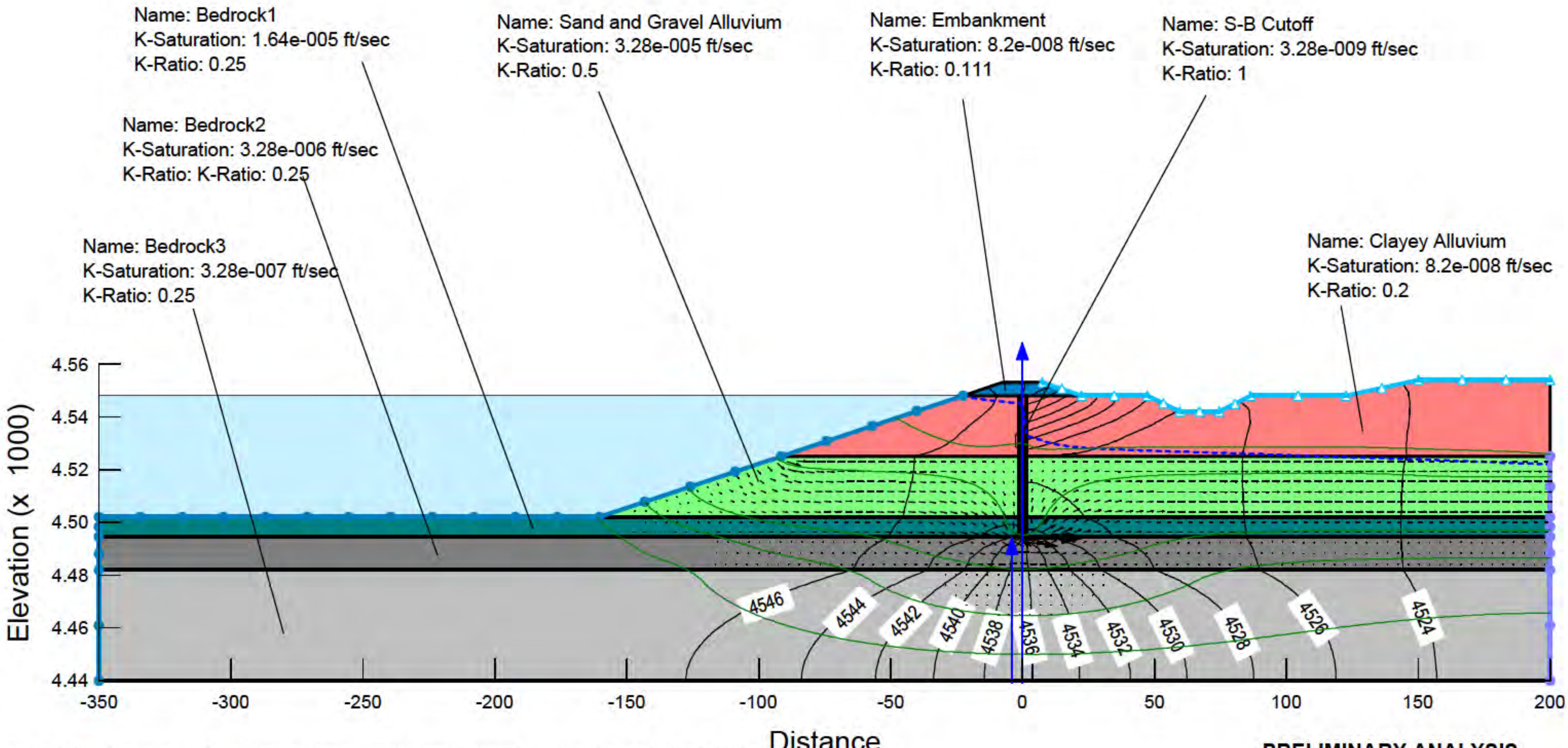


PRELIMINARY ANALYSIS

P:\13101 - Blue Grass Reservoir\Engineering\Geotechnical\Seepage-Stability\13101_South_SS.gsz

13101 Blue Grass Reservoir
Steady State Analysis
North Section

Total Seepage: 3.1277109e-005 ft³/sec
Seepage Below Cutoff: 1.2482826e-005 ft³/sec



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PRELIMINARY ANALYSIS

APPENDIX E

SUMMARY OF OPERATIONS AND DEBT SERVICE

Two Rivers Water & Farming Company
Summary of Operations and CWCB Debt Service

	12 mo 2013	12 mo 2014	12 Mo 2015	12 mo 2016	12 mo 2017
Farm Revenue	1,950,624	8,089,182	20,134,375	29,520,319	38,785,388
Farm Cost of Revenue	1,170,374	4,061,473	9,540,999	13,265,445	16,622,549
Farming Gross Margin	780,250	4,027,709	10,593,376	16,254,874	22,162,839
Debt service:					
CWCB - New Loan	-	(92,813)	(1,059,330)	(1,059,330)	(1,059,330)
Cash Flow After CWCB New Loan and Before G&A	780,250	3,934,896	9,534,046	15,195,544	21,103,508

CWCB - new loan	
Principal	$ 9,900,000
Interest rate	1.250%
Life (years)	10
Yearly payment	$ (1,059,330)