Final Estimate of Probable Construction Cost Report

City of San Angelo Storm Water Basin

July 15, 2016

Document history and status

Revision	Date	Description	By	Review	Approved
Final	7-11-16	Comments from Ricky Dickson	Ricky Dickson	TCD	

Distribution of copies

Revision	Issue approved	Date issued	Issued to	Comments

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San Angelo Storm Water Basin

Project No:	WFXN0800
Document Title:	Final Estimate of Probable Construction Cost Report
Document No.:	Document No. 1
Revision:	1
Date:	July 15, 2016
Client Name:	City of San Angelo
Project Manager:	Larissa Knapp-Scott, CFM
Author:	Thomas C. Davies, PE
File Name:	W:\JOB\WFXN0800-SAN ANGELO STORM WATER BASIN\800 DELIVERABLES\820 Submittal\2016_07_15_Report\San Angelo Storm Water Basin Report.docx

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Executive Summary

The City of San Angelo retained Jacobs Engineering Group, Inc. of Fort Worth, Texas to prepare an Estimate of Probable Construction Cost based on the latest geotechnical investigations of the site resulting from a detailed subsurface soil boring and testing program and then followed by a series of three technical memorandums covering detailed geotechnical issues, prepared by HDR, Inc. of Austin, Texas in late 2015 and early 2016.

The updated estimates fully consider these latest geotechnical constraints and issues with the proposed Basin 2 site. The site is bounded by South Chadbourne (FM 1223) and the confluence of the Red Arroyo and the South Concho River. Most notably the site has highly porous soil layers which are usually fully saturated across the entire site. These water levels in the soil roughly reflect the water levels in the South Concho River and Red Arroyo. The average observed ground water level was approximately 1807 feet.

HDR determined in their investigations that an impermeable clay liner would need to be constructed so that the ponds would be reasonably water-tight and isolated from the ground water. The cost of the liner is directly related to the depth and corresponding volume of the proposed pond. Subsequently deeper (higher volume) ponds penetrate the ground water layer deeper and therefore require a thicker and larger clay liner to be constructed. This thicker deeper liner is required to resist the buoyancy of the liner as it sits in or protrudes into these saturated soils.

A key aspect of building a clay liner system in saturated soils is that significant dewatering must be performed to make the liner construction possible. The placement of the clay liner requires detailed grade and compaction control and dewatering of the site is therefore critical. A cost estimate for the dewatering process was prepared by HDR and is included in the detailed cost estimates.

The following summary table, Table 1, illustrates the relative cost of the various basin alternatives with respect to the liner thickness and volume of the pond.

Table 1: Options Summary Table

Basin Floor Elevation MSL	Basin Storage (ac-ft.)	Liner Thickness (ft.)	Grand Total
1797	1500	10	$44,941,631
1802	1200	5	$31,561,166
1805	1000	2	$22,480,192

These efforts were preceded by a detailed hydrologic and hydraulic modeling study of the Red Arroyo drainage basin. The following reports and memoranda are attached as Appendices to this report:

Storm Water Storage Basin Feasibility Study for UCRA – June 2013

- Summarizes the findings on the study performed to evaluate the feasibility of construction of the storm water basin and presents preliminary cost estimates of probable construction costs.

In-Field Soil Permeability Tests and Dewatering Concepts – January 2015

- Defines the required clay liner thickness at base elevations of 2, 5, and 10 feet below the water table

- Assesses the relative degree of dewatering difficulty at the three basin depths below the water table

Suitability of On-Site Soils to Construct Clay Layer – January 2016

- Assesses the characteristics of the future excavation spoils for possible re-use as liner material

Cost Estimate for Constructing Clay Layer – February 2016

- Develops a component cost estimate for the dewatering aspects of the project
- Develops a component cost estimate for the installation of the clay liner

1. Introduction

Previously the Upper Colorado River Authority (UCRA) intended to develop a site into a storm water storage basin along the Red Arroyo, a tributary of the South Concho River which extends across south-central San Angelo, just upstream of its confluence with the South Concho River east of FM 1223 and south of Avenue L in San Angelo, Texas. This basin, see Exhibit 1 at the end of this report, would be constructed to store water from rainfall events that could be used for municipal use after treatment at the nearby Lone Wolf Water Treatment Plant. The water supply potential of the storage basin can be found below in Table 2. Jacobs Engineering gave a presentation to the City of San Angelo Water Advisory Board on May 10, 2016 on the feasibility of a water storage basin on the Red Arroyo. A copy of the presentation slides are included in Appendix A.

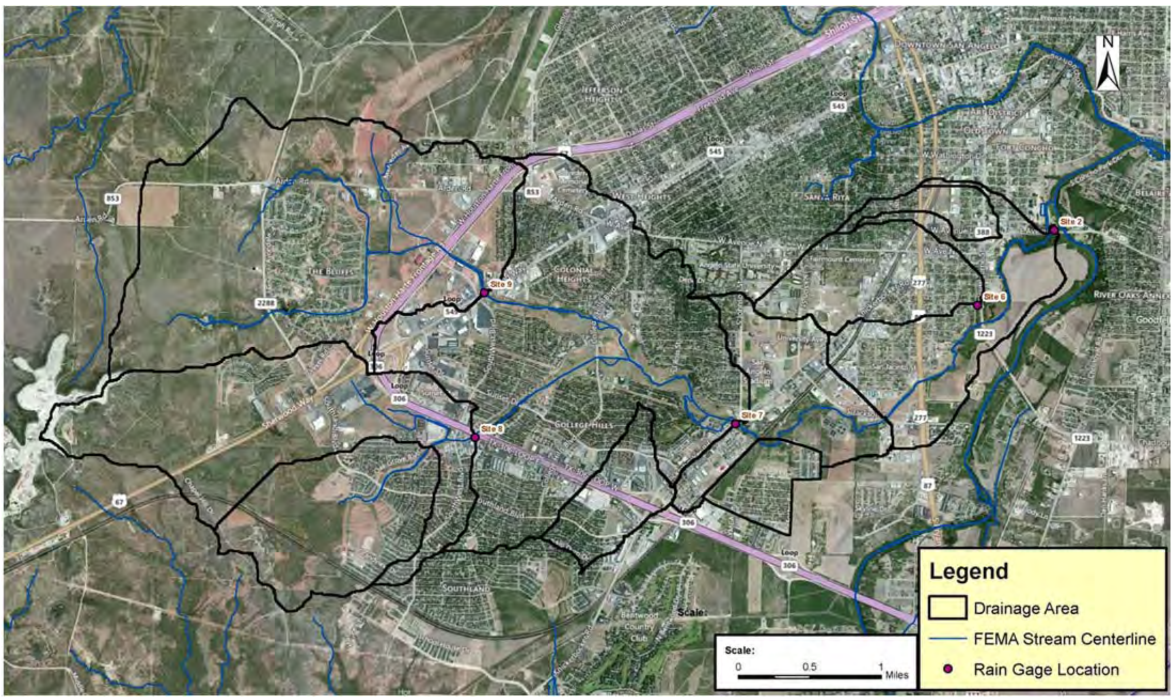

Figure 1: Red Arroyo Watershed

Following initial investigative studies by Jacobs, for UCRA, that primarily explored the hydrologic and hydraulic feasibility of the basin it was determined that additional geotechnical and constructability investigations were required to fully understand the potential for development of the site as a water storage facility. The City of San Angelo retained local geotechnical consultants, SKG Engineering, LLC, to perform soil boring and testing and HDR to explore the results the geotechnical data and fully analyze the nature of the site with respect to a significant ground water issue. Following these studies Jacobs utilized these newest specific reports and prepared probable cost estimates for construction under varying treatments for the highly perched water table at the site.

Three options for various levels of penetration into the water table at the site, including 10-foot, 5-foot and 2-foot were analyzed by HDR. Each corresponding depth of penetration into the groundwater layer requires an equal thickness of clay liner specially constructed to seal the basin off from the water bearing strata and provide a suitable counter weight to the inherent uplifting buoyant forces on the pond liner. Significant additional cost if incurred constructing these liners in water bearing layers and requires an extensive temporary dewatering system during construction. Options that include the 10-foot and 5-foot clay liners will require temporary dewatering during the construction phase while the 2-foot clay liner option does not require any de-watering. See Table 3 for a comparison of dewatering costs among the options.

Table 2: Storage Basin Water Supply Potential

	Water Supply Demand		Days of Supply - when full Volume (Ac-Ft)		
	MGD	Ac-Ft/Day	1516	1228	1044
Max Day	19.2	58.9	21	17	14
Peak Month ADF	17.97	55.2	22	18	15
Annual ADF	12.97	39.8	30	25	21

Note: Assumes 80% utilization due to water quality issues

2. Hydrologic and Hydraulic Modeling

All hydrologic and hydraulic modeling referenced in this report is from the June 2013 Storm Water Storage Basin Feasibility Study prepared for the Upper Colorado River Authority by Jacobs. The modeling done in the report was based on a model developed by the UCRA and Texas Institute for Applied Environmental Research (TIAER) at Tarleton State University. The UCRA/TIAER model was completed in February 2013 for use in the Storm Water Management Plan for the City of San Angelo. Upon receiving the model in 2013, Jacobs recommended updating the model from EPA-SWMM to XP-SWMM for better accuracy and a more stable solver. The modeling done for the June 2013 report by Jacobs was done using XP-SWMM. More information can be found in the copy of the June 2013 report in Appendix E.

3. Storage Basin System Components and Costs

3.1 Costs Overview

The following tables, Table 4, Table 5, and Table 6, contain the components and costs that make up Option 1, Option 2, and Option 3, respectively.

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Table 3: Options and Components Summary Table

Basin Floor Elevation MSL	Basin Storage ac-ft.	Liner Thickness ft.	Dewatering	Liner Earthwork	Earthwork	Intake & Overflow Structures	Pumping & Transmission	Utility Relocation	Misc	Total	Engineering and Surveying 12%	Construction Management 6%	Contingency 15%	Grand Total
1797	1500	10	$2,882,500	$ 11,295,000.00	$16,063,200	$ 550,000	$ 1,400,000	$1,000,000	$600,000	**$ 33,790,700**	$4,054,884	$2,027,442	$5,068,605	**$ 44,941,631**
1802	1200	5	$1,670,000	$ 5,795,000.00	$12,715,200	$ 550,000	$ 1,400,000	$1,000,000	$600,000	**$ 23,730,200**	$2,847,624	$1,423,812	$3,559,530	**$ 31,561,166**
1805	1000	2	$ -	$ 2,790,000.00	$10,562,400	$ 550,000	$ 1,400,000	$1,000,000	$600,000	**$ 16,902,400**	$2,028,288	$1,014,144	$2,535,360	**$ 22,480,192**

Table 4: Components and Costs of Option 1

Option 1

Red Arroyo Diversion Pond

City of San Angelo

Pond Size: 1,500 acre-feet & 10 Foot Liner

Item No.	Quantity	Unit	Item Description	Unit Price	Amount
1	2	EA	Pump, 150 HP Vertical Turbine	$185,000	$370,000
2	1	LS	Discharge piping, header, valves and miscellaneous equipment	$250,000	$250,000
3	1	LS	Pump station structure	$90,000	$90,000
4	1	LS	Electrical service to pump station	$150,000	$150,000
5	1	LS	Instrumentation and Control	$40,000	$40,000
6	2,000	LF	Pipe - Pond to WTP, 36-inch ductile iron	$250	$500,000
7	1	LS	33-Inch Water Line Relocation	$1,000,000	$1,000,000
8	2,643,600	CY	Excavation (Cut) and Haulage	$6	$15,861,600
9	16,800	CY	Embankment Excavation and Compaction (Fill)	$12	$201,600
10	1,250,000	CY	Pond Clay Liner - 10 foot thick	$9.04	$11,295,000
11	1	LS	Construction De-Watering (Clay Liner)	$2,882,500	$2,882,500
12	1	LS	Diversion Structure (dam/weir/diversion)	$350,000	$350,000
13	1	LS	Emergency Spillway and Overflow Structure	$200,000	$200,000
14	1	LS	Floodway Mitigation Cost - Land Acquisition & Construction	$400,000	$400,000
15	5,000	LF	Access Road to Pump Station	$20	$100,000
16	1	LS	Erosion Control - Total Project	$100,000	$100,000
				Subtotal	$33,790,700
				Engineering and Survey (12%)	$4,054,884
				Subtotal	$37,845,584
				Construction Management Services (6%)	$2,027,442
				Subtotal	$39,873,026
				Contingency (15%)	$5,068,605
				Total	**$44,941,631**

Table 5: Components and Costs of Option 2

Option 2

Red Arroyo Diversion Pond

City of San Angelo

Pond Size: 1,200 acre-feet & 5 Foot Liner

Item No.	Quantity	Unit	Item Description	Unit Price	Amount
1	2	EA	Pump, 150 HP Vertical Turbine	$185,000	$370,000
2	1	LS	Discharge piping, header, valves and miscellaneous equipment	$250,000	$250,000
3	1	LS	Pump station structure	$90,000	$90,000
4	1	LS	Electrical service to pump station	$150,000	$150,000
5	1	LS	Instrumentation and Control	$40,000	$40,000
6	2,000	LF	Pipe - Pond to WTP, 36-inch ductile iron	$250	$500,000
7	1	LS	33-Inch Water Line Relocation	$1,000,000	$1,000,000
8	2,088,000	CY	Excavation (Cut) and Haulage	$6	$12,528,000
9	15,600	CY	Embankment Excavation and Compaction (Fill)	$12	$187,200
10	650,000	CY	Pond Clay Liner - 5 foot thick	$8.92	$5,795,000
11	1	LS	Construction De-Watering (Clay Liner)	$1,670,000	$1,670,000
12	1	LS	Diversion Structure (dam/weir/diversion)	$350,000	$350,000
13	1	LS	Emergency Spillway and Overflow Structure	$200,000	$200,000
14	1	LS	Floodway Mitigation Cost - Land Acquisition & Construction	$400,000	$400,000
15	5,000	LF	Access Road to Pump Station	$20	$100,000
16	1	LS	Erosion Control - Total Project	$100,000	$100,000
				Subtotal	$23,730,200
				Engineering and Survey (12%)	$2,847,624
				Subtotal	$26,577,824
				Construction Management Services (6%)	$1,423,812
				Subtotal	$28,001,636
				Contingency (15%)	$3,559,530
				Total	**$31,561,166**

Table 6: Components and Costs of Option 3

Option 3
Red Arroyo Diversion Impoundment
City of San Angelo

Pond Size: 1,000 acre-feet & 2 Foot Liner

Item No.	Quantity	Unit	Item Description	Unit Price	Amount
1	2	EA	Pump, 150 HP Vertical Turbine	$185,000	$370,000
2	1	LS	Discharge piping, header, valves and miscellaneous equipment	$250,000	$250,000
3	1	LS	Pump station structure	$90,000	$90,000
4	1	LS	Electrical service to pump station	$150,000	$150,000
5	1	LS	Instrumentation and Control	$40,000	$40,000
6	2,000	LF	Pipe - Pond to WTP, 36-inch ductile iron	$250	$500,000
7	1	LS	33-Inch Water Line Relocation	$1,000,000	$1,000,000
8	1,729,200	CY	Excavation (Cut) and Haulage	$6	$10,375,200
9	15,600	CY	Embankment Excavation and Compaction (Fill)	$12	$187,200
10	300,000	CY	Pond Clay Liner - 2 foot thick	$9.30	$2,790,000
11		LS	Construction De-Watering (Clay Liner)		$0
12	1	LS	Diversion Structure (dam/weir/diversion)	$350,000	$350,000
13	1	LS	Emergency Spillway and Overflow Structure	$200,000	$200,000
14	1	LS	Floodway Mitigation Cost - Land Acquisition & Construction	$400,000	$400,000
15	5,000	LF	Access Road to Pump Station	$20	$100,000
16	1	LS	Erosion Control - Total Project	$100,000	$100,000
				Subtotal	$16,902,400
				Engineering and Survey (12%)	$2,028,288
				Subtotal	$18,930,688
				Construction Management Services (6%)	$1,014,144
				Subtotal	$19,944,832
				Contingency (15%)	$2,535,360
				Total	**$22,480,192**

3.2 Pump Station

Transferring water from the storage basin to the Lone Wolf Water Treatment Plant can be done via a gravity flow line, a gravity flow line with a pump, or entirely done with pumped flow. Jacobs recommends the use of pumped flow to convey water to the water treatment plant.

The pump station would be located in the northeastern corner of the storage basin. The structure for this pump station will be made of concrete. The estimated cost for the pump station structure is $90,000, a price developed based on the conceptual structure and estimating structural concrete work to be approximately $600 per cubic yard, then adjusted for inflation to 2016.

3.2.1 Pumps

The pump station will house two 150 HP vertical turbine pumps to provide enough head for the stored water to reach the Lone Wolf Water Treatment Plant. This pump size is estimated to be a sufficient size based on feasibility studies to pump water to the water treatment plant.

Pumps this size and type are estimated to cost $185,000 each or $370,000 for two. This price estimate comes from recent project costs adjusted for inflation to 2016.

3.2.2 Piping

Discharge piping, header, valves, and miscellaneous equipment associated with the equipment housed in the pump station will be necessary for normal operations. The specifics on the type, size, and material of all of the items listed previously would be determined in the final design of the pump station.

The total estimated cost for the discharge piping, header, valves, and miscellaneous equipment of this type is expected to be $250,000. This value was determined from comparison with recent projects where discharge piping, header, valves, and miscellaneous equipment of this type is equal to approximately 70% of pump cost, then adjusted for inflation to 2016.

3.2.3 Controls

The pump instrumentation and controls to be housed in the pump station will be determined by the client to appropriately meet their needs. An instrumentation and controls design would be included as part of the final design for the pump station.

Instrumentation and control is expected to cost $40,000, or approximately 5% of the total pump station cost, based on comparison with recent projects adjusted for inflation to 2016.

3.2.4 Electrical Service

A service line capable of providing sufficient power for the pump and accessory equipment will be required for the pump station. The cost for providing an electrical connection for the pump station is expected to be $150,000. This cost is based on recent projects where electrical is approximately 20% of the total pump station cost then adjusted for inflation to 2016.

3.2.5 Pump Station Access

The pump station is in a relatively remote location and an all-weather flex base or access road will be required to reach the pump station site for repairs and maintenance. The proposed road would connect to FM 1223 (South Chadbourne Street) to the pump station site.

Estimates for the construction of this access road put costs at $20 per running linear foot and an estimated length of 5,000 feet for a total of $100,000. The values were estimated based on recent projects then adjusted for inflation to 2016.

3.3 Pipelines - Transfer and Water Delivery

3.3.1 Transfer Pipe

Water will be transferred to the Lone Wolf Water Treatment Plant through an approximate 2,000 foot long, 36-inch diameter ductile iron pipe. See Exhibit 1.

The transfer pipe is estimated to cost $250 per linear foot, a value based on recent project bid prices adjusted for inflation to 2016. The total estimated cost for the transfer pipe is $500,000.

3.3.2 Utility Relocation

Construction of the storm water basin will require the relocation of a 33-inch water line that nearly bisects the basin into eastern and western halves. These water lines are shown on Exhibit 1.

3.4 Clay Liner and Embankment

Detailed information regarding the clay liner construction, material properties, and cost can be found in the technical memorandums in Appendix B, C, and D as prepared by HDR, Inc. of Austin, Texas. This project would require significant excavation, lining the basin to prevent seepage, and construction of an embankment. Below are some of the earthwork parameters that can be found with greater accompanying detail in Appendix B, C, and D.

Typical Excavation Depth: ~15-25 feet

Typical Existing Grade: 1,815 to 1,825 feet

Top of Perimeter Berm Around Basin: 1,822 feet

Bottom Elevation of Basin: 1,797 feet for 10 foot liner, 1802 for 5 foot liner and 1805 for 2 foot liner

Surface Area (Top): 83.3 acres

Typical Water Table Elevation: 1,807 feet

Typical Stage of South Concho River: 1,806 feet

Side slope of Excavation: 4H:1V

Basin Liner: Re-compacted Clay with added Sodium Bentonite

Table 7: Storage Volumes

Liner Thickness	Floor Elevation	Volume CY	Volume AcFt	Volume CF
10 ft	1797	2,446,516	1516	66,055,937
5 ft	1802	1,980,647	1228	53,477,475
2 ft	1805	1,684,976	1044	45,494,364

3.4.1 Clay Liner

The selection of the bottom elevation of the storm water basin determines the volume of clay liner required and costs associated with the clay liner. Excavation in this site will reach the water table in only a couple of feet requiring the clay liner to be more robust the deeper the excavation. The construction of the clay liner would progress through each of the following six steps.

According to the cost estimate provided in the memo found in Appendix B, the 10-foot thick clay liner is expected to cost $9.04 per cubic yard, require 1,250,000 cubic yards to construct, resulting in an estimated total cost of $11,295,000. The 5-foot thick clay liner has a unit cubic yard estimated cost of $8.92 and would require 650,000 cubic yards for a total estimated cost of $5,795,000. The 2-foot thick clay liner is expected to cost $9.30 per cubic yard, requiring 300,000 cubic yards, for a total estimated cost of $2,790,000.

Alternatively, as requested by the Water Advisory Board, we investigated with Roland Boehm at HDR the rough cost order of magnitude of using a vertical bentonite slurry cutoff wall instead of the clay liner of varying thicknesses as discussed above. A detailed review of the suitability of a cutoff wall was not performed as a part of this project. In general there is not adequate geotechnical soil boring data to support the detailed engineering evaluation of the depth and feasibility of the wall. Using the available information two different depths of cutoff walls were used to estimate approximate cost in general order of magnitude terms to compare to liner costs estimates. At a depth of 50-feet the wall would cost in the range of $1,500-$3,000 per linear foot of wall surrounding the entire site for a distance of approximately 10,460 linear feet for a cost in the range of $1,500 per linear foot to $3,000 per linear foot for a total in the range of $16 million to $31 million. Additionally a 75-foot deep cutoff wall was priced at $2,500 to $5,000 per linear foot for an estimated cost of $25 million to $52 million. This option must be analyzed in much more detail with added geotechnical data collected. The estimated cost alone is considerably more than the cost of the liner options. It is highly likely that in the water bearing gravel and sand materials dominant in the area that construction of a cut-off wall would not be feasible.

3.4.1.1 Dewatering

Dewatering is fully explored in the HDR technical memorandum dated in the January 12, 2016 titled "In-Field Soil Permeability Tests and Dewatering Concepts". Costs are developed for each of the three liner thickness scenarios, 10-foot, 5-foot and 2-foot thick.

As stated previously, the water table in this site is shallow and excavations for a 10-foot or 5-foot thick clay liner would have the bottom of the basin at approximately 10 feet or 5 feet below the water table, respectively. Both the 10-foot thick and 5-foot thick clay liners would require dewatering during the construction of the storm water basin; however, excavation to construct the basin that would require only a 2-foot thick clay liner would not require any dewatering.

The costs for the dewatering process are explained in more detail in the memo found in Appendix B. The cost for dewatering to allow for the construction of a 10- foot thick clay liner is $2,882,500 and the cost for the 5-foot thick liner is $1,670,000.

3.4.1.2 Basin Area Excavation

Excavating the basin to construct the storage facility requires either a 10-foot, 5-foot thick or 2-foot thick clay liner. This liner thickness will require the basin to be excavated between 18 to 23 feet, depending on the location, to accommodate the water storage volume and extra excavation to provide room for the clay liner. The total excavation for the 10-foot liner system is 1,250,000 cubic yards, 650,000 cubic yards for the 5-foot liner system, and 300,000 cubic yards the for the 2-foot liner system.

3.4.1.3 Segregating and Stockpiling Excavation Spoils

During the excavation of the basin, the removed soil will be segregated and stockpiled to be used later as component of the clay liner. Added material may be imported and is accounted for in the cost estimates as needed. Importing sodium bentonite from West Texas mines may be required to upgrade the native in-place clay soils needed to make the liner water tight.

3.4.1.4 Processing and Moisturizing Stockpiled Clay

The segregated and stockpiled soil will be either moistened or dried to the appropriate moisture content and graded to the necessary amount prior to being placed as the basin liner.

3.4.1.5 Placement and Compaction of Qualified Stockpiled Clay Materials

Placement and compaction of qualified stockpiled clay materials will be required for any selected liner option. The 10-foot thick clay liner will require 1,250,000 cubic yards, the 5-foot thick clay liner will require 650,000 cubic yards, and the 2-foot liner will require 300,000 cubic yards of clay to construct.

3.4.2 Embankment

An embankment similar to a ring dike will encircle the storm water basin. The embankment height varies from 5-10 feet around the site. The top elevation of the berm is 1,822 feet with a maximum water surface elevation in the pond of 1,820 feet set by the level of the top of the spillway structure.

3.5 Inflow and Outflow Structures

The locations of the inflow and outflow structures are shown in Figure 2.

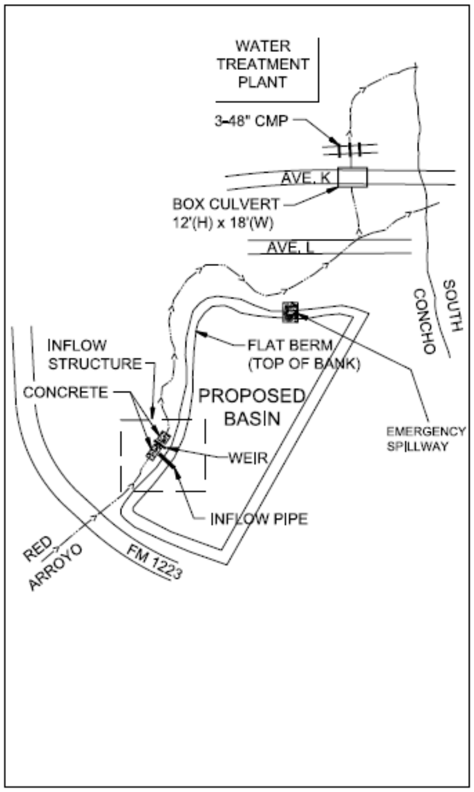


Figure 2: Location of Inflow and Outflow Structures

3.5.1 Inflow Structure

The inflow structure for the basin consists of two 48-inch parallel CMP with flap gates to prevent backflow.

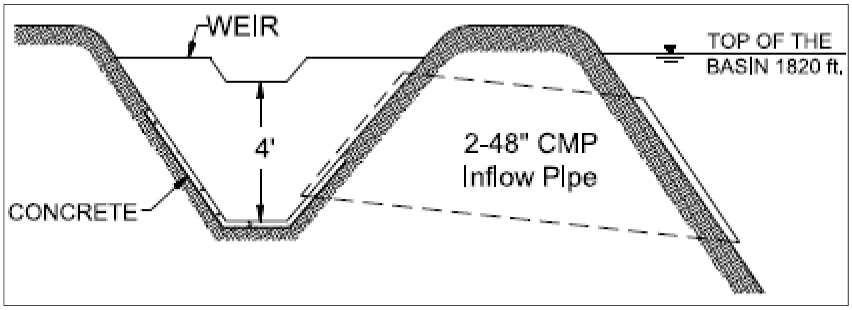


Figure 3: Profile View of Inflow Structure

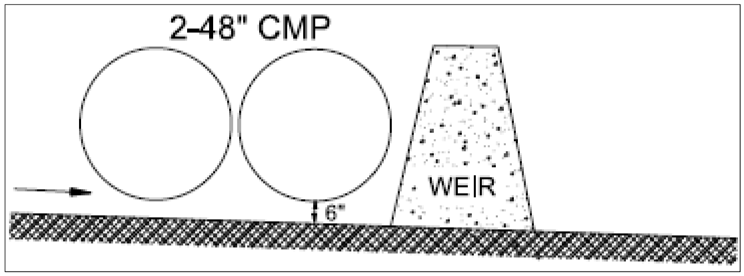


Figure 4: Longitudinal Cross-Section View of Inflow Structure

The intake/diversion structure is estimated to cost $350,000 based on comparisons with recent projects with adjustments made for inflation to 2016.

3.5.2 Emergency Spillway and Overflow Structure

Emergency spillway and overflow structure on Red Arroyo are proposed to be located near the South Concho River confluence. The emergency spillway will help to prevent water from overtopping and eroding the embankment encircling the basin which could cause its failure.

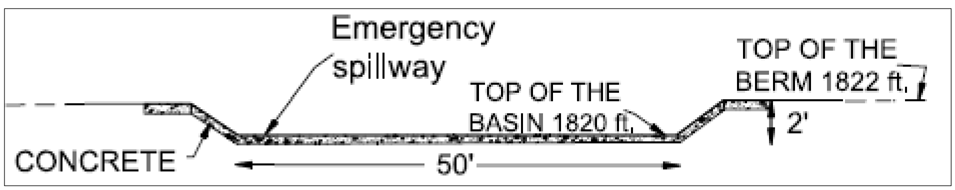


Figure 5: Profile View of Emergency Spillway

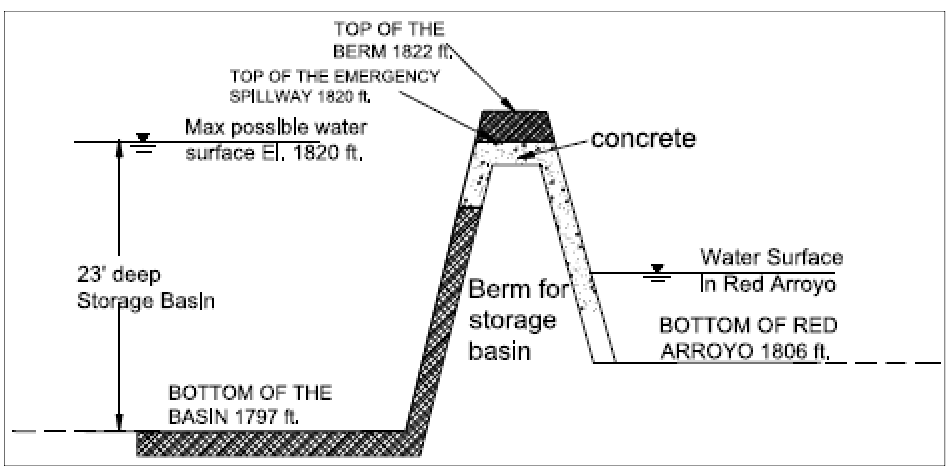


Figure 6: Longitudinal Cross-Section View of Emergency Spillway from the Storage Basin to the Red Arroyo

The emergency spillway and overflow structure has an estimated cost of $200,000 based on recent projects and adjusted for inflation to 2016.

3.6 Floodway Mitigation Cost

Two floodway mitigation areas are proposed on the west bank of Red Arroyo. The upstream floodway mitigation area is located near the basin intake structure and the downstream area is to the east of the intersection of FM 1223 and Avenue O. See Exhibit 1.

Land acquisition and construction of the two floodway mitigation areas is estimated to cost $400,000 based on recent projects and adjusted for inflation to 2016.

3.7 Erosion Control

Erosion control measures for the entire project are estimated to cost $100,000 based on recent projects and adjusted for inflation to 2016. These will include silt fencing, hay bales and other typical stormwater pollution prevention methods.

3.8 Additional Issues to Consider

3.8.1 Water Rights

Prior to construction of the storm water basin, the impact on water rights holders downstream on the South Concho River, Concho River, and Colorado River should be evaluated. Cost associated with acquiring these water rights, potential litigation or hearings, attorneys' fee or any other costs are not included in this estimate and could vary greatly depending on numerous factors.

3.8.2 Water Quality

The storm water basin is designed to allow for the first flush of water during a rainfall event to manually bypass the intake structure, preventing the intake of debris and unwanted particles found on roadways and other areas; however, the intake of undesired matter into the storm water basin remains possible and treatment, regulatory and other issues related to the water quality of the incoming water should be further explored. It is highly likely that the treatment needed would consist of micro-filtration and reverse osmosis similar to what is being studied to treat the direct reuse water supply. Should the direct reuse project not be pursued separately then strong consideration should be given to the significant added cost associated with the treatment of any stormwater collected in this or any other urban facility in San Angelo. These costs are not included in this estimate.

3.8.3 Section 404 Permitting

A permit may need to be obtained from the US Army Corps of Engineers for impacts to the waters of the US near the basin, primarily in the Red Arroyo with the construction of the intake/diversion structure. These costs are included in the Permitting and Mitigation cost estimates.

3.8.4 Property Acquisition

The City may face some difficulty in obtaining property to construct the storm water basin as the proposed site is on private property. Acquisition of the actual pond site is not included in the estimates but the acquisition of the mitigation areas is included.

3.8.5 Possible Floodplain Impacts

The 100-year floodplain covers a large percentage of the proposed location of the storm water basin. FEMA permitting in the form of initially a CLOMR and then later after construction, a LOMR would likely be required. Mitigation for flood plain impacts may also be required and is estimated at $400,000.

4. Summary

This report has covered the following six points relevant to the construction of the storm water storage basin:

1. Determine the excavation and spoil quantity for the basin scenario.
2. Determine the storage volume in the basin.
3. Determine the piping and pumping system needed to transfer water stored in the basin to the Lone Wolf Water Treatment Plant.
4. Determine the ancillary items associated with the basin needed to capture runoff, provide erosion protection, or discharge overflow to the South Concho River.
5. Determine utility relocation items associated with constructing the basin.
6. Prepare a preliminary option of probable construction cost for the items listed above.

Three options were developed for the storm water storage basin, each option required a different clay liner thickness and excavation requirements. As mentioned previously in this report, the thickness of the clay liner was directly related to the depth of the excavation, as excavation goes deeper the clay liner grows thicker. The typical excavation depth is expected to be 15-25 feet from the existing grade which ranges from 1,815 feet to 1,825 feet. The bottom of the basin would vary depending on the option selected: 1,797 feet for the 10-foot thick liner, 1,802 feet for the 5-foot thick liner, and 1,805 feet for the 2-foot thick clay liner. The estimated excavation volumes and associated costs can be found for Option 1, Option 2, and Option 3 in Table 4, Table 5, and Table 6, respectively.

The excavation and liner thickness were the drivers to the ultimate potential storage volume for the basin. The 10-foot thick liner option would have a potential storage volume of 1,516 acre-feet, the 5-foot thick liner option with 1,228 acre-feet of storage, and the 2-foot thick liner option 1,044 acre-feet of storage. A tabulated version of these numbers can be found in Table 7.

Transferring water from the storage basin to the Lone Wolf Water Treatment Plant would be accomplished through a pump and pipe system. A pump station will be built near the northeast corner of the basin to house the two 150 HP vertical turbine pumps. This size pump is expected to be a sufficient size to meet head and volume requirements in supplying water to the Lone Wolf Water Treatment Plant through a 36-inch ductile iron pipe. This pump size was determined based on previous feasibility studies to pump water to the plant. The total cost for the pumps, pump station, pipe, and miscellaneous equipment is estimated at $2,500,000. See Sections 3.2 and 3.3 for more detailed information.

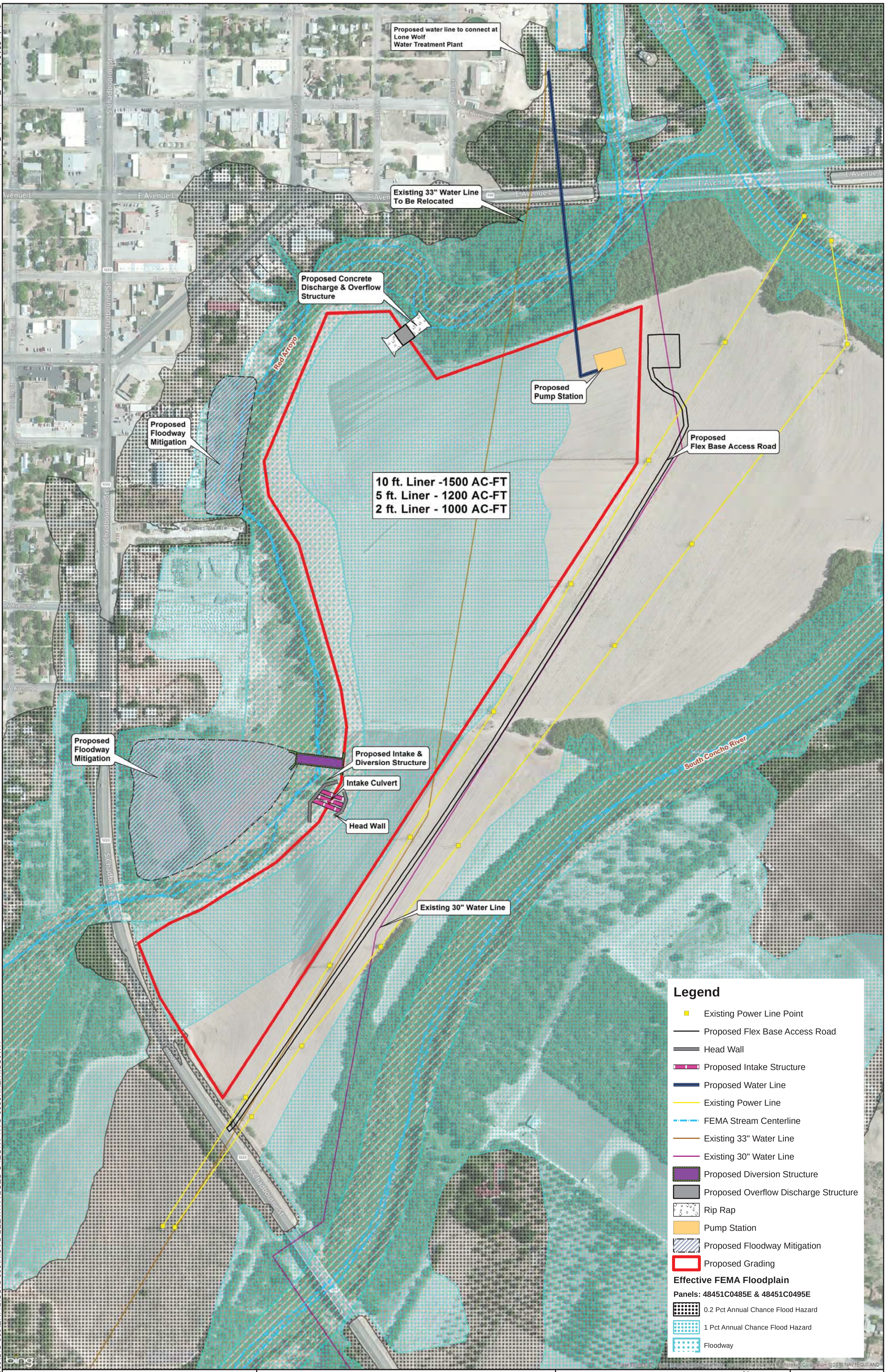
Sections 3.5 to 3.8 of this report have addressed the inflow and outflow structures, the emergency spillway and overflow structure, floodway mitigation, erosion control, and additional items such as water rights holders downstream, water quality, Section 404 permitting, property acquisition, and possible floodplain impacts.

There is currently a 33-inch waterline that nearly bisects the basin into eastern and western halves. The construction of the storm water basin will require the removal and relocation of this waterline at an estimated cost of $1,000,000.

This report serves as addressing the sixth point. This report has covered the necessary elements to construct the storm water storage basin and cost of each item.

Based on the determinations and results from the above six points, Jacobs recommends the most cost effective option for the storm water storage basin to be Option 2, the option with a 5-foot thick liner. The cost of this option is estimated to be $31,561,166. The 10-foot liner pond requires significant extra imported material which increases the cost disproportionately to the benefits of the pond.

As noted above slurry cut-off wall options greatly escalate the overall project cost by almost to more than double the current cost estimates and are not thought to be feasible in these types of water bearing sands and gravels.



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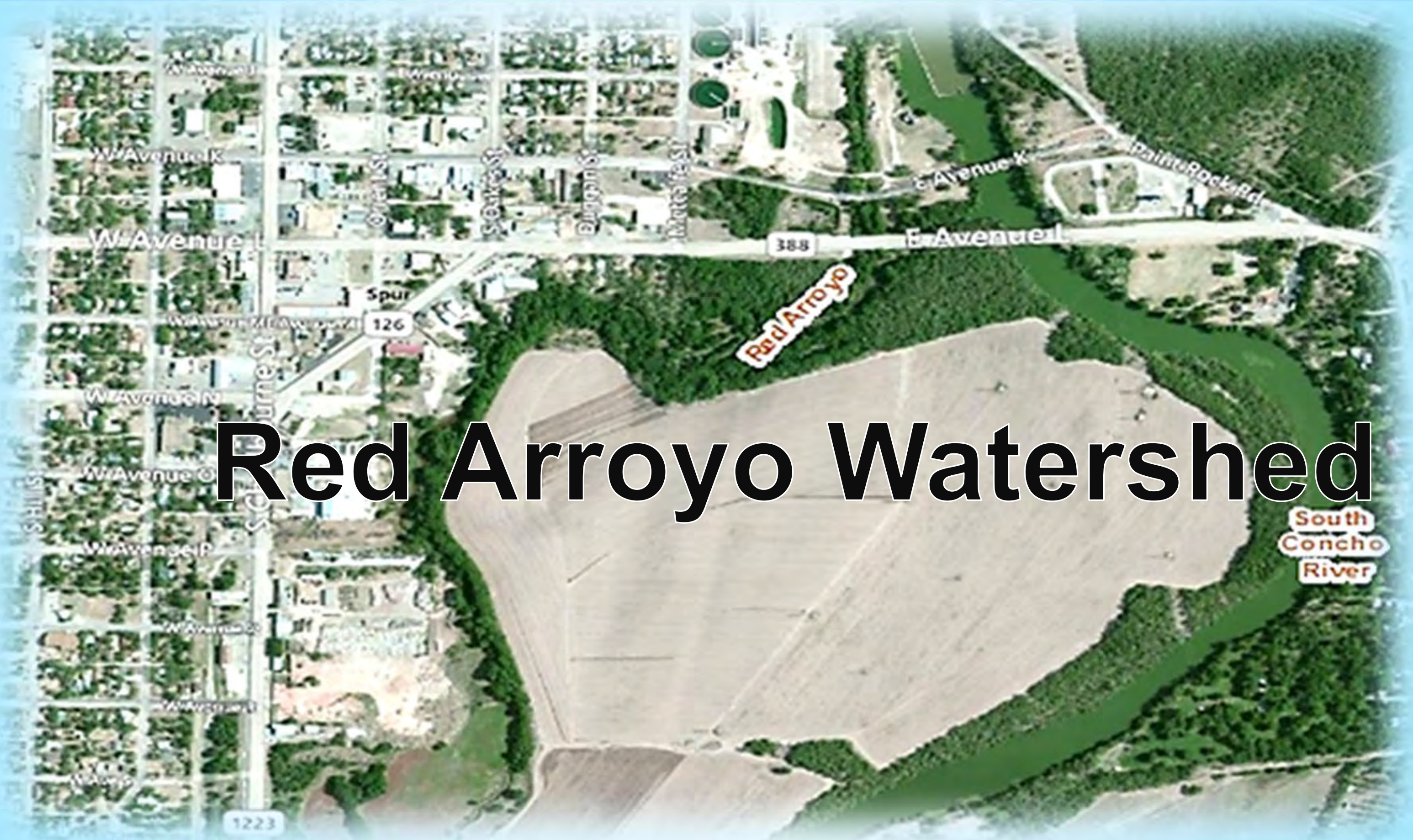
JACOBS	Red Arroyo Storm Water Storage Facility City of San Angelo, Tom Green County, Texas Jacobs Project WFXN0800	Scale: 0 200 400 Feet	N	Exhibit 1

Appendix A. Presentation to City of San Angelo Water Advisory Board – May 10, 2016

Water Advisory Board

May 2016

Red Arroyo Watershed



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Introduction

- The Upper Colorado River Authority (UCRA) intends to develop a storm water storage basin just to the east of FM 1223, south of Avenue L, and west of South Concho River for water quality, municipal water supply, and downstream release requirements in San Angelo, Texas.
- The basin will be located at the downstream end of the watershed drained by Red Arroyo, a tributary channel of South Concho River.
- This presentation summarizes the results of a feasibility study of the proposed storm water storage basin.
- If the proposal seems technically feasible and economically viable then there will be a need for a pre-design study before the system can be properly designed.

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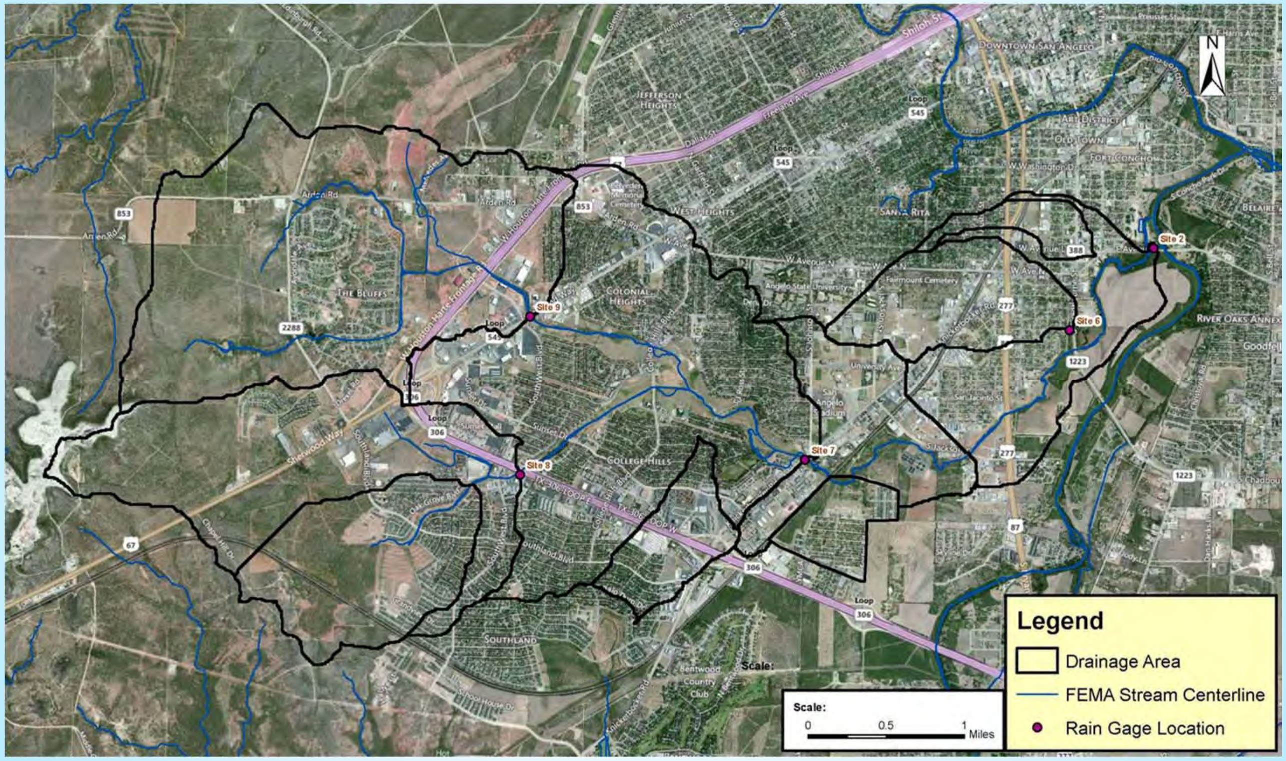
Red Arroyo Watershed



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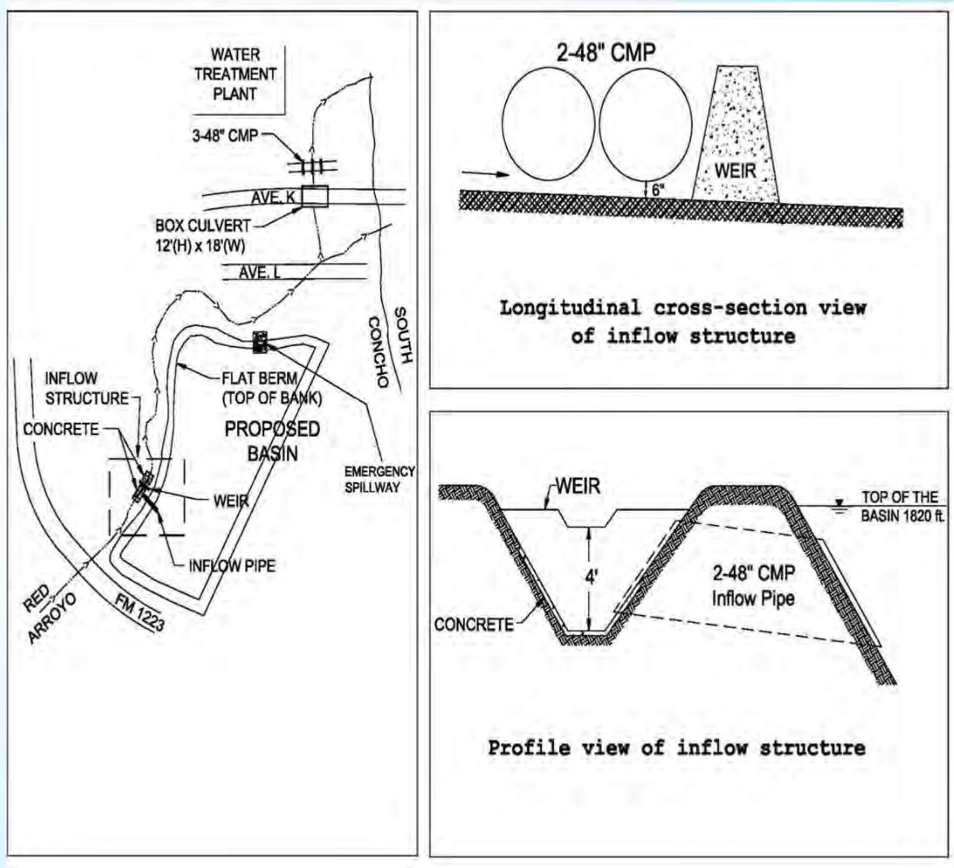
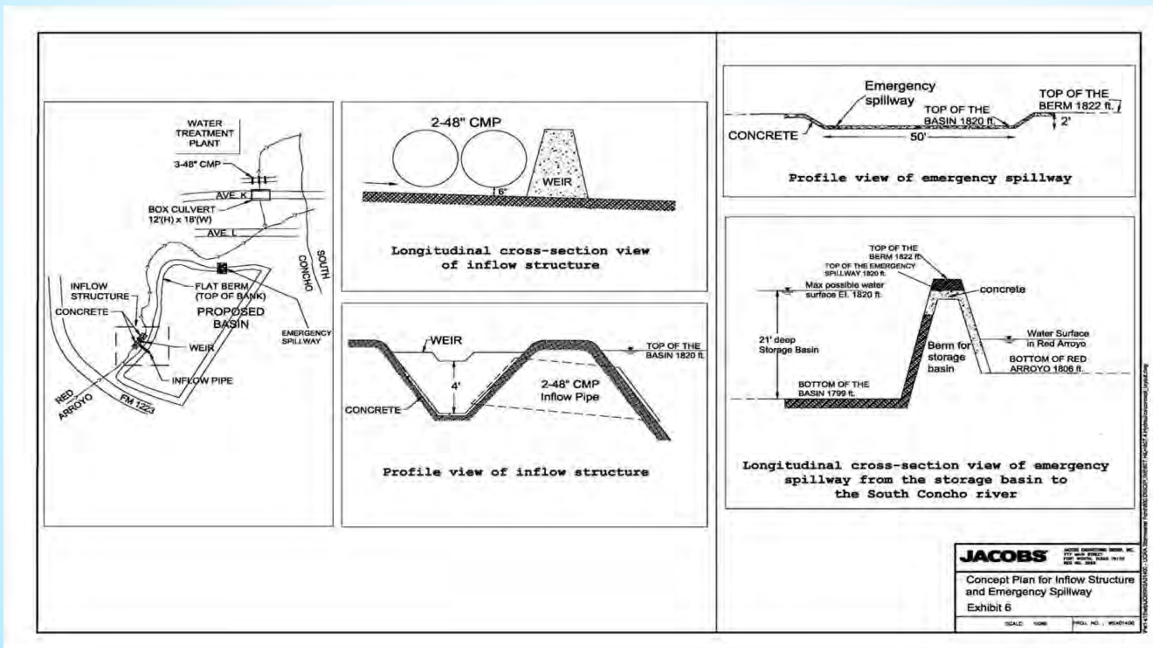
Inflow Structure

- An inflow structure is required for the water to be blocked in the channel
- Construct a concrete weir (small dam) across the channel section
- Place inflow pipes with control structures are placed immediately upstream of the weir to divert water from the channel to the basin.
- A weir at the downstream end to allow for any overflow to flow downstream
- Multiple inflow structures along the channel to capture more water for events with various magnitude



Emergency Spillway

- An emergency spillway is provided so that during extreme events when the capacity of the basin is exceeded, water can be transferred from the basin to Red Arroyo River.



Pond 2 Storage Volumes Table

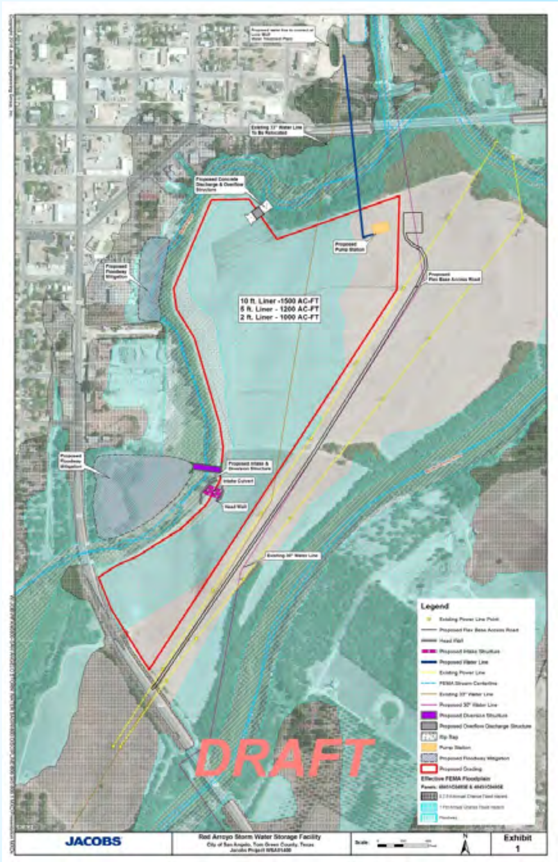
Liner Thickness	Floor Elevation	Volume CY	Volume AcFt	Volume CF
10 ft	1797	2,446,516	1516	66,055,937
5 ft	1802	1,980,647	1228	53,477,475
2 ft	1805	1,684,976	1044	45,494,364

Water Use

	Water Supply Demand		Days of Supply - when full Volume (Ac-Ft)		
	MGD	**Ac-Ft/Day**	**1516**	**1228**	**1044**
Max Day	19.2	58.9	21	17	14
Peak Month ADF	17.97	55.2	22	18	15
Annual ADF	12.97	39.8	30	25	21

Conveyance to WTP

- 3 options were evaluated for transferring the stored water to the WTP
  - Gravity flow system
  - Combined gravity flow and pump system
  - Pump flow
- Recommend a pump flow system from basin to WTP
- The basin can be emptied over a two week period with a flow rate of 20 MGD



Option 1

Red Arroyo Diversion Pond

City of San Angelo

Pond Size: 1,500 acre-feet & 10 Foot Liner

Item No.	Quantity	Unit	Item Description	Unit Price	Amount
1	2	EA	Pump, 150 HP Vertical Turbine	$185,000	$370,000
2	1	LS	Discharge piping, header, valves and miscellaneous equipment	$250,000	$250,000
3	1	LS	Pump station structure	$90,000	$90,000
4	1	LS	Electrical service to pump station	$150,000	$150,000
5	1	LS	Instrumentation and Control	$40,000	$40,000
6	2,000	LF	Pipe - Pond to WTP, 36-inch ductile iron	$250	$500,000
7	1	LS	33-Inch Water Line Relocation	$1,000,000	$1,000,000
8	2,643,600	CY	Excavation (Cut) and Haulage	$6	$15,861,600
9	16,800	CY	Embankment Excavation and Compaction (Fill)	$12	$201,600
10	1,250,000	CY	Pond Clay Liner - 10 foot thick	$9.04	$11,295,000
11	1	LS	Construction De-Watering (Clay Liner)	$2,882,500	$2,882,500
12	1	LS	Diversion Structure (dam/weir/diversion)	$350,000	$350,000
13	1	LS	Emergency Spillway and Overflow Structure	$200,000	$200,000
14	1	LS	Floodway Mitigation Cost - Land Acquisition & Construction	$400,000	$400,000
15	5,000	LF	Access Road to Pump Station	$20	$100,000
16	1	LS	Erosion Control - Total Project	$100,000	$100,000
				Subtotal	$33,790,700
				Engineering and Survey (12%)	$4,054,884
				Subtotal	$37,845,584
				Construction Management Services (6%)	$2,027,442
				Subtotal	$39,873,026
				Contingency (15%)	$5,068,605
				Total	**$44,941,631**

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Option 2

Red Arroyo Diversion Pond

City of San Angelo

Pond Size: 1,200 acre-feet & 5 Foot Liner

Item No.	Quantity	Unit	Item Description	Unit Price	Amount
1	2	EA	Pump, 150 HP Vertical Turbine	$185,000	$370,000
2	1	LS	Discharge piping, header, valves and miscellaneous equipment	$250,000	$250,000
3	1	LS	Pump station structure	$90,000	$90,000
4	1	LS	Electrical service to pump station	$150,000	$150,000
5	1	LS	Instrumentation and Control	$40,000	$40,000
6	2,000	LF	Pipe - Pond to WTP, 36-inch ductile iron	$250	$500,000
7	1	LS	33-Inch Water Line Relocation	$1,000,000	$1,000,000
8	2,088,000	CY	Excavation (Cut) and Haulage	$6	$12,528,000
9	15,600	CY	Embankment Excavation and Compaction (Fill)	$12	$187,200
10	650,000	CY	Pond Clay Liner - 5 foot thick	$8.92	$5,795,000
11	1	LS	Construction De-Watering (Clay Liner)	$1,670,000	$1,670,000
12	1	LS	Diversion Structure (dam/weir/diversion)	$350,000	$350,000
13	1	LS	Emergency Spillway and Overflow Structure	$200,000	$200,000
14	1	LS	Floodway Mitigation Cost - Land Acquisition & Construction	$400,000	$400,000
15	5,000	LF	Access Road to Pump Station	$20	$100,000
16	1	LS	Erosion Control - Total Project	$100,000	$100,000
				Subtotal	$23,730,200
				Engineering and Survey (12%)	$2,847,624
				Subtotal	$26,577,824
				Construction Management Services (6%)	$1,423,812
				Subtotal	$28,001,636
				Contingency (15%)	$3,559,530
				Total	**$31,561,166**

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Option 3
Red Arroyo Diversion Impoundment
City of San Angelo

Pond Size: 1,000 acre-feet & 2 Foot Liner

Item No.	Quantity	Unit	Item Description	Unit Price	Amount
1	2	EA	Pump, 150 HP Vertical Turbine	$185,000	$370,000
2	1	LS	Discharge piping, header, valves and miscellaneous equipment	$250,000	$250,000
3	1	LS	Pump station structure	$90,000	$90,000
4	1	LS	Electrical service to pump station	$150,000	$150,000
5	1	LS	Instrumentation and Control	$40,000	$40,000
6	2,000	LF	Pipe - Pond to WTP, 36-inch ductile iron	$250	$500,000
7	1	LS	33-Inch Water Line Relocation	$1,000,000	$1,000,000
8	1,729,200	CY	Excavation (Cut) and Haulage	$6	$10,375,200
9	15,600	CY	Embankment Excavation and Compaction (Fill)	$12	$187,200
10	300,000	CY	Pond Clay Liner - 2 foot thick	$9.30	$2,790,000
11		LS	Construction De-Watering (Clay Liner)		$0
12	1	LS	Diversion Structure (dam/weir/diversion)	$350,000	$350,000
13	1	LS	Emergency Spillway and Overflow Structure	$200,000	$200,000
14	1	LS	Floodway Mitigation Cost - Land Acquisition & Construction	$400,000	$400,000
15	5,000	LF	Access Road to Pump Station	$20	$100,000
16	1	LS	Erosion Control - Total Project	$100,000	$100,000
				Subtotal	$16,902,400
				Engineering and Survey (12%)	$2,028,288
				Subtotal	$18,930,688
				Construction Management Services (6%)	$1,014,144
				Subtotal	$19,944,832
				Contingency (15%)	$2,535,360
				Total	**$22,480,192**

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Summary Cost Table

Basin Floor Elevation MSL	Basin Storage ac-ft.	Liner Thickness ft.	Dewatering	Liner Earthwork	Earthwork	Intake & Overflow Structures	Pumping & Transmission	Utility Relocation	Misc	Total	Engineering and Surveying 12%	Construction Management 6%	Contingency 15%	Grand Total
1797	1500	10	$2,882,500	$ 11,295,000.00	$16,063,200	$ 550,000	$ 1,400,000	$1,000,000	$600,000	**$ 33,790,700**	$4,054,884	$2,027,442	$5,068,605	**$ 44,941,631**
1802	1200	5	$1,670,000	$ 5,795,000.00	$12,715,200	$ 550,000	$ 1,400,000	$1,000,000	$600,000	**$ 23,730,200**	$2,847,624	$1,423,812	$3,559,530	**$ 31,561,166**
1805	1000	2	$ -	$ 2,790,000.00	$10,562,400	$ 550,000	$ 1,400,000	$1,000,000	$600,000	**$ 16,902,400**	$2,028,288	$1,014,144	$2,535,360	**$ 22,480,192**

Additional Issues to Consider

- Water Rights Issues
- Water Quality
- Section 404 Permitting Issues for Impacts to the Waters of the US Near the Basin
- Property Acquisition
- Possible Floodplain Impacts

Conclusions

Appendix B. Cost Estimate for Constructing Clay Layer – Technical Memorandum 3

Technical Memorandum

Date: February 4, 2016

Project: Red Arroyo Stormwater Basin Study

To: Ricky Dickson – Executive Director of Public Works (City of San Angelo)

From: Rolland Boehm, PE

STATE OF TEXAS
ROLLAND GENE BOEHM
101598
LICENSED
PROFESSIONAL ENGINEER
2-4-16

Subject: **Cost Estimate for Constructing Clay Liner**

Introduction

The Upper Colorado River Authority (UCRA) and the City of San Angelo (City) are in the process of evaluating the feasibility of constructing a stormwater storage basin. The primary purpose of the basin is to provide additional water supply for the City. As proposed, the project would be located near the confluence of the Red Arroyo and South Concho River.

Background

The UCRA retained Jacobs Engineering to perform an initial feasibility level study. The study, which was issued by report (June, 2013), focused mainly on the water resources elements of the potential project. The study evaluated two fundamental options; 1) a basin area encompassing approximately 151 acres (referred to as Basin 1) and a smaller land area encompassing approximately 83 acres (referred to as Basin 2). Based on several factors, Basin 2 was selected as the preferred option. An outline of Basin 2, which lies entirely on privately owned property, is shown in Figure 1.

FIGURE 1 – Outline of Basin 2 Area

As noted in Jacobs' Feasibility Study (dated June, 2013), the basin would be filled by diverting water from the Red Arroyo during storm runoff events. Some of the key proposed features of Basin 2 are:

- Typical Excavation Depth: ~25 feet
- Typical Existing Grade: 1,815 to 1,825 ft (msl)
- Perimeter Berm Around Basin: 1,822 ft (msl
- Bottom Elevation of Basin: 1,797 ft
- Surface Area (Top): 83.3 acres
- Typical Water Table Elevation: 1,807 ft (msl)
- Typical Stage of South Concho River: 1,806
- Sideslopes of Excavation: 4H:1V
- Basin Liner: Recompacted Clay

The Jacobs' study focused largely on site hydrology and the feasibility of capturing and storing a certain amount of water runoff from Red Arroyo. A feasibility level cost estimate was provided in their study report, which included 400,000 cubic yards of clay for liner construction. Based on this volume of clay, it appears a 3-foot clay liner thickness was assumed for cost estimating purposes.

In a memo dated February 19, 2014, HDR identified several geotechnical constraints associated with developing the site for storage basin purposes. The most notable constraints included; 1) constructing the basin below the water table and 2) extending the basin into otherwise saturated granular soil. It was noted that constructing the basin within saturated granular soils would require a clay liner greater than 3 feet thick to resist hydrostatic uplift pressure (buoyancy), as well as a rather significant dewatering effort to maintain a dry excavation during construction of the liner. Given the "feasibility" nature of Jacobs' cost estimate, site dewatering was not explicitly identified as a cost component.

More recent efforts by HDR examined dewatering concepts (memo dated, January 12, 2016) and required clay thicknesses at various basin floor elevations (memo dated, January 26, 2016). The later is summarized in the following table:

Table 1 – Clay Liner Thickness vs. Depth Below Water Table

Water Table Elevation (ft)(msl)	Finished Basin Floor Elevation (ft)(msl)	Basin Depth Below Water Table (ft)	Required Clay Liner Thickness (ft)	Bottom of Basin Excavation (ft)(msl)	Basin Excavation Depths (ft)	Approximate Basin Storage Capacity (ac-ft)
1807	1805	2	2(1)	1803	12 to 17	1120
1807	1802	5	5(2)	1797	18 to 23	1319
1807	1797	10	10(3)	1787	28 to 33	1632

Notes: 1. Two-foot clay liner = 300,000 cubic yards.
2. Five-foot clay liner = 650,000 cubic yards.
3. Ten-foot clay liner = 1,250,000 cubic yards.

Purpose of Memorandum and Limitations

The purpose of this memo is to provide "feasibility" level cost estimates for constructing an uplift resistant clay liner. A total of three estimates were generated, each corresponding to a finished basin elevation noted in Table 1. For better comparativeness, the estimates include the volume of soil to be over excavated (to accommodate the clay liner thickness) and associated dewatering.

It is understood the clay liner cost estimates may be used by Jacobs' or others in developing or updating total project costs. It is not uncommon for an initial feasibility level cost estimate to be revised as more information becomes available. Ultimately these revisions may be used to better determine the overall feasibility of the project. Invariably, feasibility level cost estimates are based on concepts, as opposed to actual design. As such, a project cost estimate can change significantly during preliminary and final design phases. In addition, there are "non-design" factors that can significantly impact the actual construction costs, e.g. availability of local contractors to perform the work, difficult or undefined ground conditions, and project schedule.

Clay Liner Cost Components

The cost for installing the clay liner is a function of the bottom elevation. As the storage basin becomes larger, the area required to be lined with clay also becomes larger. In addition, the further the basin extends below the water table the thicker the liner needs to be, as well as the more robust the required dewatering effort. There are six basic construction sequences and cost components associated with the clay liner.

1. Lowering the water table below bottom excavation grades (i.e. dewatering)
2. Over-excavating the basin area to accommodate clay liner thickness
3. Segregating and stockpiling excavation spoils for reuse as clay liner material
4. Subgrade preparation
5. Processing and moisturizing stockpiled clay materials in preparation for placement
6. Placement and compaction of qualified stockpiled clay materials

Item 1 would likely be performed by a specialty dewatering contractor and require a significant amount of planning and design to successfully implement. Items 2 thru 6 can be thought of as more traditional or routine earthwork.

Cost Estimate for Temporarily Lowering Water Table

Finished Base Grade = 1797 feet

Developing the basin to elevation 1797 feet would require temporarily depressing the water table within the center of the excavation area to elevation 1787 feet or less. As noted in a previous memo (dated January, 12, 2016), this could potentially be accomplished by installing and operating a series of well points around the excavation. The wells would be connected at the surface level to a header pipe(s), which is then connected to one or more pumps. The vacuum effect created by the system's pump(s) would draw water from the well points thereby lowering the elevation of the groundwater in the area requiring dewatering. A discharge line(s) connected to the pump(s) would be the means by which the generated groundwater would be discharged to a suitable collection area, or in this case, the Concho River or Red Arroyo.

It is reasonable to assume that a dewatering contractor would install the well points in a line or row that strikes horizontally along the proposed sideslopes, possibly half way between the toe of the excavation and top of the excavation, as illustrated on Figure 1. The single row of well points would extend around the entire excavation area at equal spacing.

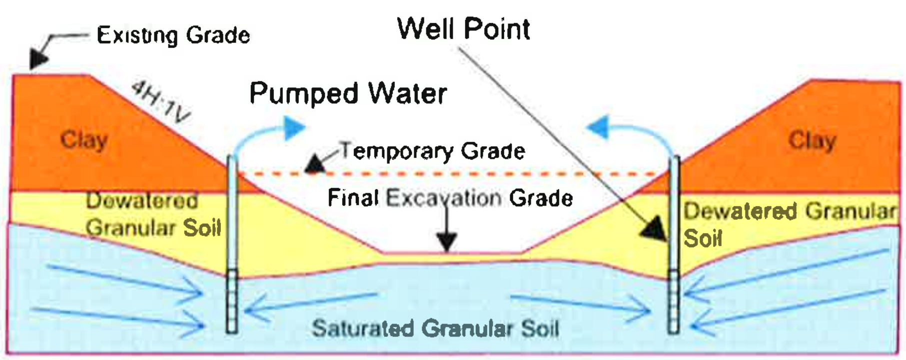

Figure 1 – Typical Dewatering Well Point Installation

Preliminarily, it is estimated the well points would need to be installed on a 3.5-foot spacing to achieve the desired drawdown within the center of the excavation. Based on a total line or row distance of 9,800 lineal feet, the total number of well point installations would be on the order of 2800.

The major cost items associated with well point system of this nature include the following:

- Total number of well points (counted as each)
- Length of vertical riser pipes (measured in vertical feet)
- Length of header pipe connecting the well points (measured in linear feet)
- Number of swing joints connecting the riser pipes to the header pipe (counted as each)
- Number of well point pumps (counted as each)
- Length of discharge pipe (measured in lineal feet)

Sizes and quantities for dewatering items were estimated based on a conceptual system layout. Table 2 provides a summary of these sizes and quantities, as well as a feasibility level cost estimate.

Table 2 – Dewatering System Items and Cost Estimate (Base El 1797 ft)

Item	Quantity	Unit Cost[1]	Extended Cost	Comment
Planning and Design	Lump Sum	$200,000	$200,000	Dewatering contractor to perform
Number of Wells	2800[2]	--	--	3.5-foot spacing
Survey Control/Staking	Lump Sum	$50,000	$50,000	Layout proposed wells
Filter Screens	2800	$36/ea	$100,800	One per well
Self Jetting tips	2800	$60/ea	$168,000	One per well
Riser Pipes	61600 vf	$10/vf	$616,000	22 feet each well
16 inch Dia. Header Pipe	9800 lf	$30/lf	$294,000	Assumes five discrete header pipes
Swing Joints w/valves	2800	$64/ea	$179,200	Includes flexible hoses
Rent Six 100 HP Electric Pumps w/Generators	6 months	$72,000/mo	$432,000	Includes one backup pump
24-Inch Diameter Discharge Piping	1500 lf	$55/lf	$82,500	Assumes three discharge locations
Pump Operator & Maintenance	6 months[2]	$60,000/mo	$360,000	Operated 24 hours per day. Assumes system will need to be operated for 6 months. Includes fuel.
Miscellaneous Items	Lump Sum	$400,000	$400,000	Includes mobilization, accessories, taxes, permit fees, and system abandonment

TOTAL = $2,882,500

Note 1: Includes installation costs, as applicable.

Note 2: Assumes well points would be installed around the entire perimeter of the basin at one time and operated continuously for 6 months (i.e. entire project area dewatered). It is possible a dewatering contractor may elect to install well points over discrete areas or sections, and complete excavation and clay liner construction in a sequential fashion. This approach, if used, would likely result in more well point installations, though less total pumping.

Finished Base Grade = 1802

Developing the basin to a finished floor elevation of 1802 feet would require temporarily depressing the water table within the center of the excavation area to elevation 1797 feet or less, which is approximately 10 feet higher than the previous scenario.

Again, it is reasonable to assume that a dewatering contractor would install the well points in a line or row that strikes horizontally along the proposed sideslopes, possibly half way between the toe of the excavation and top of the excavation. The single row of well points would extend around the entire excavation area at equal spacing.

Preliminarily, it is estimated the well points would need to be installed on a 6-foot spacing to achieve the desired drawdown within the center of the excavation. Based on a line or row distance of 9,800 lineal feet, the total number of well point installations would be on the order of 1600.

Sizes and quantities for dewatering items were estimated based on a conceptual system layout. Table 3 provides a summary of these sizes and quantities, as well as a feasibility level cost estimate.

Table 3 – Dewatering System Items and Cost Estimate (Base El 1802 ft)

Item	Quantity	Unit Cost[1]	Extended Cost	Comment
Planning and Design	Lump Sum	$125,000	$125,000	Dewatering contractor to perform
Number of Wells	1600[2]	--	--	6-foot spacing
Survey Control/Staking	Lump Sum	$30,000	$30,000	Layout proposed wells
Filter Screens	1600	$36/ea	$57,600	One per well
Self Jetting tips	1600	$60/ ea	$96,000	One per well
Riser Pipes	28800 vf	$10/vf	$288,000	18 feet each well
16 inch Dia. Header Pipe	9800 lf	$30/lf	$294,000	Assumes three discrete header pipe sections
Swing Joints w/valves	1600	$64/ea	$102,400	Includes flexible hoses
Rent Four 100 HP Electric Pumps w/Generators	4 months	$48,000/mo	$192,000	Includes one backup pump
24-Inch Diameter Discharge Piping	1000 lf	$55/lf	$55,000	Assumes two discharge locations
Pump Operator & Maintenance	4 months[2]	$45,000/mo	$180,000	Operated 24 hours per day. Assumes system will need to be operated for 4 months. Includes fuel.
Miscellaneous Items	Lump Sum	$250,000	$250,000	Includes accessories, taxes, permit fees, and system abandonment

TOTAL = $1,670,000

Note 1: Includes installation costs, as applicable.

Note 2: Assumes well points would be installed around the entire perimeter of the basin at one time and operated continuously for 4 months (i.e. entire project area dewatered). It is possible a dewatering contractor may elect to install well points over discrete areas or sections, and complete excavation and clay liner construction in a sequential fashion. This approach, if used, would likely result in more well point installations, though less total pumping.

Finished Base Grade = 1805

Developing the basin to elevation 1805 feet would require excavating the floor or base of the basin to elevation 1803 feet (to accommodate a 2-foot thick clay liner). Based on available geotechnical data, there should be a sufficient amount of in-situ clay remaining at the base of the excavation to resist hydrostatic uplift pressure, as well as significantly mitigate the amount of seepage into the excavation. However, since the excavation would still be technically below the water table (El. 1807 feet) a certain amount groundwater would still seep into the excavation, especially if seams or lens of granular material are encountered. In this case it is reasonable to assume that the amount of water that would seep into the excavation could be managed by creating low areas or sumps within the excavation, and subsequently pumping the water once it has ponded to a certain depth. It is anticipated that the cost associated with this more "passive" approach would be considered ancillary, and often included in the earthwork costs.

Cost Estimate for Earthwork

Finished Base Grade = 1797 feet

As previously noted, a finished floor elevation of 1797 feet would require constructing a 10-foot clay liner to resist the hydrostatic uplift pressures generated within the underlying granular stratum. A previous HDR memo (dated January 26, 2016) detailed certain aspects related to constructing the 10-foot clay liner. The two most significant aspects included:

1. The total required volume of in-place compacted clay would be approximately 1.25M cubic yards.
2. Excavation spoils could be used to construct the clay liner if properly segregated, tested, processed, and moisturized.

A feasibility level cost estimate for the earthwork components are provided in Table 4.

Table 4 – Earthwork Items and Cost Estimate (Base El 1797 ft)

Item	Quantity	Unit Cost	Extended Cost	Comment
Over-Excavation	1,250,000 cy	$2/cy	$2,500,000	Undercut for clay liner
Segregating and Stockpiling	1,250,000 cy	$1/cy	$1,250,000	Suitable clays segregated during general excavation
Processing and Moisturizing	1,250,000 cy	$0.50/cy	$625,000	Clay clods require processing, soil moisture may be too high or low
Amending Unsuitable Clays	250,000 cy	$1.50/cy	$375,000	Assumes insufficient suitable clays. Requiring a portion to be amended with Bentonite.
Subgrade Preparation	85 ac	$2000/ac	$170,000	Subgrade graded. Loose, soft, or wet areas reworked
Place and Compact	1,250,000 cy	$4.50/cy	$5,625,000	Haul and place in lifts
Geotechnical Testing	Lump Sum	$250,000	$250,000	Verification testing by geotechnical laboratory
Miscellaneous Items	Lump Sum	$500,000	$500,000	Includes mobilization, demobilization, cleanup, and administration.

TOTAL = $11,295,000

Finished Base Grade = 1802 feet

A finished floor elevation of 1802 feet would require constructing a 5-foot clay liner to resist the hydrostatic uplift pressures generated within the underlying granular stratum. A previous HDR memo detailed certain aspects related to constructing the 5-foot clay liner. The two most significant aspects included:

1. The total required volume of in-place compacted clay would be approximately 650,000 cubic yards.
2. Excavation spoils could be used to construct the clay liner if properly segregated, tested, processed, and moisturized.

A feasibility level cost estimate for the earthwork components are provided in Table 5.

Table 5 – Earthwork Items and Cost Estimate (Base El 1802 ft)

Item	Quantity	Unit Cost	Extended Cost	Comment
Over-Excavation	650,000 cy	$2/cy	$1,300,000	Undercut for clay liner
Segregating and Stockpiling	650,000 cy	$1/cy	$650,000	Suitable clays segregated during general excavation
Processing and Moisturizing	650,000 cy	$0.50/cy	$325,000	Clay clods require processing, soil moisture may be too high or low
Amending Unsuitable Clays	0 cy	$1.50/cy	$0	Sufficient on-site suitable clays
Subgrade Preparation	85 ac	$2000/ac	$170,000	Subgrade graded, loose, soft, or wet areas reworked
Place and Compact	650,000 cy	$4.50/cy	$2,925,000	Haul and place in lifts
Geotechnical Testing	Lump Sum	$125,000	$125,000	Verification testing by geotechnical laboratory
Miscellaneous Items	Lump Sum	$300,000	$300,000	Includes mobilization, demobilization, cleanup, and administration.

TOTAL = $5,795,000

Finished Base Grade = 1805 feet

A finished floor elevation of 1805 feet would require constructing a 2-foot clay liner, the minimum practical thickness for a functional clay liner. A previous HDR memo detailed certain aspects related to constructing the 2-foot clay liner. The two most significant aspects included:

1. The total required volume of in-place compacted clay would be approximately 300,000 cubic yards.
2. Excavation spoils could be used to construct the clay liner if properly segregated, tested, processed, and moisturized.

A feasibility level cost estimate for the earthwork components are provided in Table 6.

Table 6 – Earthwork Items and Cost Estimate (Base El 1805 ft)

Item	Quantity	Unit Cost	Extended Cost	Comment
Over-Excavation	300,000 cy	$2/cy	$600,000	Undercut for clay liner
Segregating and Stockpiling	300,000 cy	$1/cy	$300,000	Suitable clays segregated during general excavation
Processing and Moisturizing	300,000 cy	$0.50/cy	$150,000	Clay clods require processing, soil moisture may be too high or low
Amending Unsuitable Clays	0 cy	$1.50/cy	$0	Sufficient on-site suitable clays
Subgrade Preparation	85 ac	$2000/ac	$170,000	Subgrade graded, loose, soft, or wet areas reworked
Place and Compact	300,000 cy	$4.5/cy	1,350,000	Haul and place in lifts
Geotechnical Testing	Lump Sum	$70,000	$70,000	Verification testing by geotechnical laboratory
Miscellaneous Items	Lump Sum	$150,000	$150,000	Includes mobilization, demobilization, cleanup, control of water, and administration.

TOTAL = $2,790,000

Summary of Cost Estimates

The Jacobs' feasibility report provided an itemization of their assumed costs for constructing a 3-foot clay liner at basin floor elevation of 1797 feet. These costs, as well as the above newly developed costs for a 2-foot, 5-foot, and 10-foot liner are summarized in Table 7.

Table 7 – Clay Liner Costs

Basin Elevation (ft)	Basin Storage[1] (ac-ft)	Liner Thickness (ft)	Dewatering	Earthwork	Total
1797 (Jacobs')	1807[2]	3	0	$4,400,000[3][4]	$4,400,000
1797	1632[2]	10	$2,882,500	$11,295,000	$14,177,500
1802	1319	5	$1,670,000	$5,795,000	$7,465,000
1805	1120	2	0[5]	$2,790,000	$2,790,000

Notes:

1. Assumes two feet of freeboard.
2. Basin storage capacity computed by Jacobs' is approximately 12% higher than the value computed by HDR. The difference in values is attributed to different computational methods.
3. Based on 400,000 cubic yards per Jacobs' report, Table 20, Item 12.
4. Used $3/cy unit cost for 3-foot undercut excavation and $8/cy unit cost for construction of clay liner, per Jacobs' report, Table 20, Items 9 and 12.
5. Dewatering costs assumed to be ancillary to earthwork.

Discussion

As noted in the above table, the costs for developing the basin to elevation 1797 feet are sustainably higher when considering the need for a thicker clay liner and an active construction dewatering system. The computed differential between the original Jacobs' feasibly cost for constructing a 3-foot liner versus constructing the 10-foot clay liner (with dewatering) is approximately $9.8M.

The estimated costs for constructing a thinner clay liner, though at higher base elevations, decrease proportionally. As shown in Table 7, it would cost approximately $6.7M less to construct a 5-foot clay liner (set at a finished base elevation of 1802 feet) versus the 10 foot liner. Similarly it would cost approximately $11.4M less to construct a 2-foot clay liner (set at a finished base elevation 1805 feet).

Raising the base or floor elevation would not only reduce clay liner costs, but would also reduce overall general excavation costs. For instance, relative to a basin elevation 1797 feet, a basin floor elevation of 1802 feet would have approximately 505,000 cubic yards less general excavation. Similarly, there would be 826,000 cubic yards less general excavation should the finished basin floor be raised to elevation 1805 feet. At an assumed unit excavation and haul cost of $3/cy, there would be additional cost reductions of approximately $1.5M and $2.5M, respectively. However in both cases the basin storage volume is significantly impacted, as noted in Table 7.

Appendix C. Suitability of On-Site Soils to Construct Clay Layer – Technical Memorandum 2

Technical Memorandum

Date: January 26, 2016

Project: Red Arroyo Stormwater Basin Study

To: Ricky Dickson – Executive Director of Public Works (City of San Angelo)

From: Rolland Boehm, PE

STATE OF TEXAS
ROLLAND GENE BOEHM
101598
LICENSED
PROFESSIONAL ENGINEER
1-26-16

Subject: **Suitability of On-Site Soils to Construct Clay Liner**

Introduction

The Upper Colorado River Authority (UCRA) and the City of San Angelo (City) are in the process of evaluating the feasibility of constructing a stormwater storage basin. As proposed, the project is located within the City limits near the confluence of the Red Arroyo and South Concho River, as illustrated in Figure 1.

The UCRA retained Jacobs Engineering to perform an initial feasibility level study. The study, which was issued by report, focused mainly on the water resources elements of the potential project. The study evaluated two fundamental options; 1) a basin area encompassing approximately 151 acres (referred to as Basin 1) and a smaller land area encompassing approximately 83 acres (referred to as Basin 2). Based on several factors, Basin 2 was selected as the preferred option. An outline of Basin 2, which lies entirely on privately owned property, is shown in Figure 1.

FIGURE 1 – Outline of Basin 2 Area

Background

As noted in Jacobs' Feasibility Study (dated June, 2013), the basin would be filled by diverting water from the Red Arroyo during storm runoff events. Some of the key proposed features of Basin 2 are:

- Typical Excavation Depth: ~25 feet
- Typical Existing Grade: 1,815 to 1,825 ft (msl)
- Perimeter Berm Around Basin: 1,822 ft (msl)
- Bottom Elevation of Basin: 1,797 ft
- Surface Area (Top): 83.3 acres
- Typical Water Table Elevation: 1,807 ft (msl)
- Sideslopes of Excavation: 4H:1V
- Basin Liner: Recompacted Clay

There are several geotechnical constraints associated with developing the site for storage basin purposes. These were described in HDR's technical memo, dated February 19, 2014. The most notable constraints included; 1) constructing the basin below the water table and 2) extending the basin into otherwise saturated granular soil. Constructing the basin within saturated granular soils will require a significant dewatering effort, as well as a thickened clay liner to resist hydrostatic uplift pressure.

In the absence of a permanent dewatering system, the basin will tend to fill with groundwater (seepage) to approximately the water table elevation. One common engineering approach to mitigate seepage is to install a clay liner. However the clay liner will need to be thick enough to counterbalance the hydrostatic uplift pressure.

At the current proposed base grades, HDR estimated the clay liner would need to be on the order of 10 feet thick, which would require greater total excavation depths. In addition to a greater earthmoving effort, there would also be a more significant dewatering effort (since the water table needs to be further depressed during construction of the thickened clay liner).

Purpose of Memorandum

The purpose of this memorandum is to assess the suitability and availability of on-site clayey soils for constructing the clay liner. The additional costs related to construction of the clay liner and associated dewatering will be the subject of a future memorandum.

Clay Liner

Minimum Thickness

A clay liner is often constructed in water storage ponds and basins for the purpose of minimizing water losses into the natural subsurface materials or alternatively minimizing the amount of groundwater that seeps into the basin (if below the water table). However in some cases the need for a clay liner can be eliminated if the underlying subsurface is dominated by natural clayey soils or possibly other low permeable materials, e.g. intact rock. When required, clay liners are generally at least 1.5 to 2.0 feet thick. This minimum thickness tends to facilitates compaction and protects against the negative effects of wetting and drying cycles, and root penetration (from vegetation).

As noted in the previous memos, the project site has an underlying saturated granular stratum with a top elevation that would lie above the proposed floor elevation of 1797 feet. The granular stratum would essentially cause the basin to "fill up" with groundwater should a clay liner not be installed. In this case, the clay liner would need to be of sufficient thickness (or weight) to resist the hydrostatic uplift pressures that would exist within the saturated granular soil.

In a memo dated December 31, 2015, HDR estimated the required clay liner thickness (to resist buoyancy or uplift) at three different basin floor elevations. The estimated thicknesses are re-presented in Table 1.

Table 1 – Clay Liner Thickness vs. Depth Below Water Table

Water Table Elevation (ft)(msl)	Finished Basin Bottom Elevation (ft)(msl)	Basin Depth Below Water Table (ft)	Required Clay Liner Thickness (ft)	Bottom of Basin Excavation (ft)(msl)	Basin Excavation Depths(1) (ft)	Approximate Basin Storage Capacity(2) (ac-ft)
1807	1805	2	2	1803	12 to 17	1120
1807	1802	5	5	1797	18 to 23	1319
1807	1797	10	10	1787	28 to 33	1632(3)

Notes:
1. Based on existing grades ranging from El. 1815 to El. 1825.
2. Assumes 2 feet of freeboard (Pool = El.1820 feet).
3. Jacobs' Feasibility Study reported 1839 ac-ft at the same pool elevation (El. 1820). The difference in computed storage volumes (approximately 12%) is likely due to differences in computational methods. HDR used a 3-D computer model, were as Jacobs' reportedly used a modified 2-D procedure.

The clay liner thicknesses presented in the above table represent the required thickness at the base or floor of the basin to resist hydrostatic uplift pressure. It should be noted that the required clay liner thickness on the 4H:1V side slope is variable depending on the depth below the water table, though in all case a minimum 2 feet in thickness. Therefore when the base elevation is set at 1805 feet the clay liner would be 2 feet thick along both the base and side slopes.

When base elevations are set at 1802 feet and 1797 feet the required clay liner thickness on the side slopes could transition from the required base thicknesses of 5 feet or 10 feet, to the minimum liner thickness of 2 feet. The point of transition would, in this case, occur at the water table elevation (El. 1807 feet).

<u>Compaction</u>

Clay liners are typically constructed in 6-inch compacted lifts. The level of required compaction is dependent on the specific engineering criteria to be assigned during final design. In this case it is reasonable to expect the required compaction to be at least 95% of the maximum dry density as determined by the Standard Proctor Method (ASTM D 698). Soil moisture content during compaction is often specified above the optimum moisture content, as determined by ASTM D 698.

<u>Engineering Properties</u>

The desired liner permeability for water storage basins is typically on the order of 1 x 10^{-06} cm/sec. However the permeability criteria can be more stringent when water losses need to be held to an absolute minimum, or alternatively the permeability can be less stringent when the clay liner is rather thick and/or when a certain amount of water loss is acceptable.

To reliably achieve the permeability requirement, the clay liner material normally needs to have certain characteristics or criteria. The material criteria are typically specified during final design, though in this case would likely be similar to the following:

- Percent Fines (ASTM D 1140) ≥ 60%
- Liquid Limit (ASTM D 4318) ≥ 30%
- Plastic Index (ASTM D 4318) ≥ 15*
- Maximum Clod Size < 3 inches

*A Plastic Index of 15 is estimated to be lowest permissible value for obtaining a vertical permeability of 1 x 10^{-06} cm/sec.

On-Site Clayey Soils

Reuse of On-Site Spoils

Reusing on-site excavation spoils for constructing the clay liner would be the most efficient means, provided enough "quality" material can be segregated and stockpiled during mass excavation. Reusing a portion of the excavation spoils for this purpose would minimize the amount of soil that needs to be hauled off-site, while minimizing the amount of clay borrow material that must be imported. The following section discusses the suitability and availability of on-site material.

Subsurface Conditions

To date a total of 20 soil borings have been performed within or near the proposed footprint for Basin 2. Generally the borings indicate the upper 8 to 25+ feet of soil at the site consists of relatively fine grained alluvium, either Lean Clay or Silt.

The Lean Clays, which are the more prominent, are regarded as characteristically low in permeability and typically capable of satisfying the criteria for a compacted clay liner. Contrary, the Silts have a higher permeability and tend to be more difficult to compact. Highly silty soils are not considered suitable for use in constructing a clay liner. In terms of feasibility assessment only the Lean Clay soils in relatively thick strata are considered "potentially" usable as liner quality material. Table 2 summarizes the approximate thickness of the Lean Clay as encountered in the soil borings.

Table 2 – Approximate Thickness of Lean Clay

Boring	Approximate Surface Elevation (1) (ft-msl)	Approximate Thickness of Lean Clay (ft)	Approximate Bottom Elevation of Lean Clay (ft-msl)
B-1	1822	23	1799
B-2	1818	8	1810
B-3	1822	25+	<1797
B-4	1819	22	1797
B-5	1820	20+	<1800
B-6	1821	21	1800
B-7	1824	25+	<1799
B-8	1823	13	1810
B-9	1822	23	1799
B-10	1823	13	1810
B-11	1822	22	1800
B-12	1824	17	1807
B-13	1821	21	1800
B-14	1825	17	1808
B-15	1825	8	1817
MW-1	1824	9	1815
MW-2	1825	13	1812
MW-3	1817	20	1797
MW-4	1822	22	1800
MW-5	1813	16	1797
Average	***1821.6***	***17.9***	***1803.7***

Notes:
1. Surface elevation for borings B-1 through B-15 estimated from *Google Earth Pro.* Surface elevation for borings MW-1 through MW-5 based on actual survey data provided by the City.

Average Clay Thickness

As noted in the bottom row of Table 2, the average thickness of the Lean Clay as encountered at the 20 soil boring locations is approximately 18 feet, though the upper 2 feet is likely highly organic due to agricultural use.

Clay Properties and Suitability

Select samples from the 20 soil borings were tested for natural moisture content, Atterberg Limits (Liquid Limit and Plastic Index), and percent fines. The results are summarized in Table 3.

Table 2 – Laboratory Test Results (Lean Clay)

Test Parameter	Number of Samples	Low	High	Average	Preliminary Criteria
Natural Moisture Content (%)	25	11.6	26.7	17.9	--
Liquid Limit (%)	25	20[1]	46	35[1]	>30
Plastic Index	25	10[2]	25	17[2]	>20
Percent Fines (%)	26	52.3[3]	91.3	74.6[3]	>60

Notes:
1. Four of the 25 test samples did not meet the liquid limit criteria, though the average exceeded the criteria.
2. Eight of the 25 test samples did not meet the plastic index criteria, though the average exceeded the criteria.
3. Three of the 26 test samples did not meet the percent fines criteria, though the average exceeded the criteria.

Based on review of the test data, approximately 70% of the on-site Lean Clays would qualify as clay liner material.

Estimated Useable Clay Volumes

Assuming 4H:1V cut slopes and a 83.3 acre Basin 2 area, the gross estimated volume of material to a depth of 18 feet is approximately 2.15 million cubic yards. Subtracting off the upper 2 feet of topsoil, the "potentially" usable clay material would then equates to approximately 1.85 million cubic yards

Based on the laboratory testing, approximately 70% of the on-site Lean Clay appears suitable for liner construction. Multiplying the 1.85 million cubic yards of "potentially" usable clay by 70% equates to approximately 1.30 million cubic yards of "useable" clay.

Estimated Required Clay Volumes

- 2-foot clay liner = 300,000 cubic yards[1]
- 5-foot clay liner = 650,000 cubic yards[2]
- 10-foot clay liner = 1,250,000 cubic yards[3]

Notes:
1. Assumes uniform 2-foot clay liner across base and entire sideslopes.
2. Assumes 5-foot liner across entire base. Liner thickness on sideslopes transition from 5 feet thick at the toe of the slope to 2 feet thick at El 1807 feet (i.e. water table elevation), remaining constant up to El 1822 feet.
3. Assumes 10-foot liner across entire base. Liner thickness on sideslopes transition from 10 feet thick at the toe of the slope to 2 feet thick at El 1807 feet (i.e. water table elevation), remaining constant up to El 1822 feet.

Preliminary Assessment

Based on preliminary evaluation of the test data and estimated required clay volume, it appears that there would be a sufficient amount of on-site Lean Clay to construct the 2-foot and 5-foot liner. Relying strictly on the on-site Lean Clays to construct the 10-foot liner appears problematic, since the estimated required volume of 1.25 million cubic yards is approximately equal to estimated 1.30 million cubic yards of available material.

If the on-site material is to be considered for the construction of the clay liner, the material will need to be carefully separated and stockpiled to ensure quality of material is maintained. Soils samples of the stockpiled material will need to be graded by a laboratory to ensure the material meets the criteria, as determined by the engineer's final design, before placement.

Appendix D. In-Field Soil Permeability Tests and Dewatering Concepts – Technical Memorandum 1

Technical Memorandum

Date: January 12, 2015

Project: Red Arroyo Stormwater Basin Study

To: Ricky Dickson – Executive Director of Public Works (City of San Angelo)

From: Rolland Boehm, PE

STATE OF TEXAS
ROLLAND GENE BOEHM
101598
LICENSED
PROFESSIONAL ENGINEER
1-12-15

Subject: **In-Field Soil Permeability Tests and Dewatering Concepts**

Introduction

The Upper Colorado River Authority (UCRA) and the City of San Angelo (City) are in the process of evaluating the feasibility of constructing a stormwater storage basin. The primary purpose of the basin is to provide additional water supply for the City. As proposed, the project would be located near the confluence of the Red Arroyo and South Concho River, as shown on Figure 1.

FIGURE 1 – Site Location Map

Background

The UCRA retained Jacobs Engineering to perform an initial feasibility level study. The study, which was issued by report, focused mainly on the water resources elements of the potential project. The study evaluated two fundamental options; 1) a basin area encompassing approximately 151 acres (referred to as Basin 1) and a smaller land area encompassing approximately 83 acres (referred to as Basin 2). Based on several factors, Basin 2 was selected as the preferred option. An outline of Basin 2, which lies entirely on privately owned property, is shown in Figure 2.

FIGURE 2 – Outline of Basin 2 Area

As noted in Jacobs' Feasibility Study (dated June, 2013), the basin would be filled by diverting water from the Red Arroyo during storm runoff events. Some of the key proposed features of Basin 2 are:

- Typical Excavation Depth: ~25 feet
- Typical Existing Grade: 1,815 to 1,820 ft (msl)
- Perimeter Berm Around Basin: 1,822 ft (msl
- Bottom Elevation of Basin: 1,797 ft
- Water Storage (full): 1,839 ac-ft
- Surface Area (Top): 83.3 acres
- Typical Water Table Elevation: 1,807 ft (msl)
- Typical Stage of South Concho River: 1,806
- Sideslopes of Excavation: 4H:1V
- Basin Liner: Recompacted Clay

There are several geotechnical constraints associated with developing the site for storage basin purposes. These were described in HDR's technical memo, dated February 19, 2014. The most notable constraints included; 1) constructing the basin below the water table and 2) extending the basin into otherwise saturated granular soil, as illustrated in Figure 3. Constructing the basin within saturated granular soils will require a significant dewatering effort, as well as a thickened clay liner to resist hydrostatic uplift pressure.

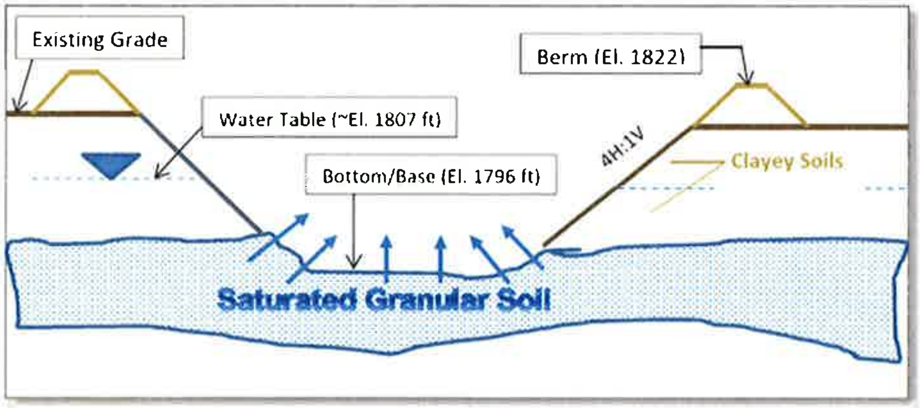

Figure 3 – Groundwater Inflows During Excavation (Conceptual)

In the absence of a permanent dewatering system, the basin will tend to fill with groundwater (seepage) to approximately the water table elevation. One remedial measure is to install a clay liner. However the clay liner will need to be thick enough to counterbalance the hydrostatic uplift pressure, as illustrated in Figure 4.

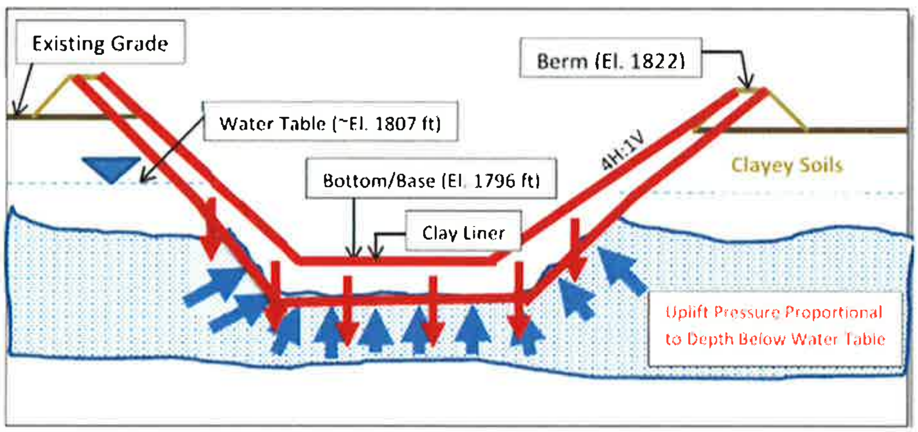

Figure 4 – Groundwater Uplift Pressure on Underside of Clay Liner (Conceptual)

At current proposed base grades, HDR estimated the clay liner would need to be on the order of 10 feet thick, which would require greater total excavation depths. In addition to a greater earthmoving effort, there would also be a more significant dewatering effort (since the water table needs to be further depressed during construction of the thickened clay liner).

Purpose of Memorandum

The City retained HDR to complete an additional feasibility study, in particular, to further evaluate the above noted geotechnical considerations. The goals of this additional study include the following:

1. Define the required clay liner thickness at base elevations of 2, 5 and ~10 feet below water table.
2. Assess the relative degree of dewatering difficulty at the three basin depths below water table.
3. Assess the characteristics of the future excavation spoils for possible re-use as liner material.
4. Develop a component cost estimate for the dewatering aspects of the project.
5. Develop a component cost estimate for installing the clay liner.

This particular memo addresses items 1 and 2.

Subsurface Investigation

Previous Exploratory Borings
A total of 15 exploratory borings were completed within the general study area between December 31, 2012 and January 3, 2013. The approximate locations for the 15 borings are illustrated on Figure 5.

The borings, which were performed by SKG Engineering, extended to depths ranging from 15 and 25 feet below ground surface (bgs), though more typically 25 feet bgs. As a point of reference, the borings typically extended 2 to 7 feet below the proposed bottom of the basin, which Jacob's study set at El. 1797 ft-msl).

Recent Exploratory Borings
Five (5) additional exploratory borings were completed for this particular study in May of 2015. The approximate locations for these five borings are also illustrated on Figure 5.

The additional borings were also completed by SKG Engineering, ranging from 26 to 30 feet bgs. All five borings were terminated due to sloughing sands and gravels. Based on actual ground survey data (provided by the City), the boring termination depths range in elevation from 1783 to 1799 feet (msl). Given a bottom elevation of 1797 ft-msl, the borings generally extend 2 feet above to 14 feet below the floor of Basin 2.

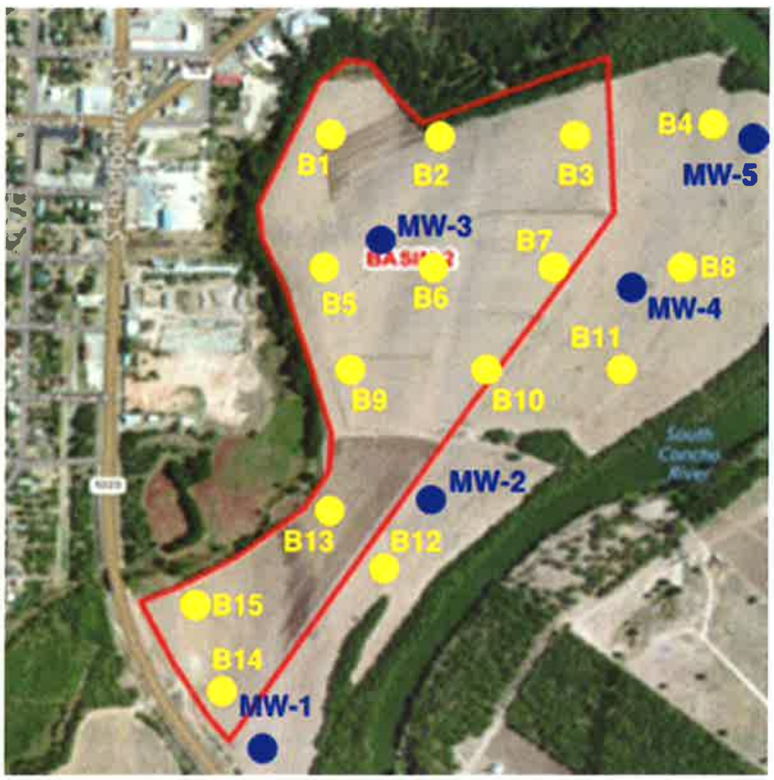

Figure 5 – Approximate Boring Locations Relative to Basin 2

Subsurface Conditions

Generally, the borings indicate the upper 13 to 25+ feet of soil at the site is relatively fine grained alluvium, consisting of Lean Clay, Sandy Lean Clay, Silt, and Sandy Silt. Exceptions occur at boring locations B-2 and MW-1, where the thickness of these finer grained deposits are only 7 feet and 9 feet, respectively.

The Lean Clays, which are the more prominent, are characteristically low in permeability and generally not capable of transmitting appreciable amounts of groundwater. Contrary, the Sandy Silts can be regarded as low to moderately permeable and capable of transmitting a certain amount of groundwater, as well as imposing hydrostatic uplift pressures on an overlying clay liner (if installed below the water table).

Materials that were encountered below the upper fine grained soils consist mostly of sands and gravels, and mixtures thereof. These soils often include a fractional amount of finer grained particles, e.g. silt and clay. Overall these "granular" soils tended to slough or "flow" during drilling, an indication they are below the water table and have little or no cohesion or unconfined shear strength.

Those granular soils with a limited amount of fine grained particles can be regarded as highly permeability and capable of transmitting large amounts of groundwater. Alternatively, granular soils with a significant amount of fines can be regarded as moderately permeability and capable of transmitting more moderate amounts of groundwater. Regardless of the fines content, all the encountered granular soils will allow enough movement of groundwater to induce hydrostatic uplift pressures on the underside of a clay liner, if installed below the water table. The estimated top elevation of the granular strata, as encountered in the 20 referenced soil borings, is summarized in the table below.

Table 1 – Approximate Elevation of Saturated Granular Strata

Boring	Approximate Surface Elevation [(1)] (ft-msl)	Approximate Elevation of Top of Granular Soils (ft-msl)	Minimum Thickness [(2)] (ft)
B-1	1822	1799	>2
B-2	1818	1800	>7
B-3	1822	<1797[(3)]	NA
B-4	1819	<1797	NA
B-5	1820	<1800[(3)]	NA
B-6	1821	1800	>4
B-7	1824	<1799[(3)]	NA
B-8	1823	1801	>12
B-9	1822	1799	>2
B-10	1823	1801	>7
B-11	1822	1800	>3
B-12	1824	1807	>8
B-13	1821	1800	>4
B-14	1825	1808	6 [(4)]
B-15	1825	<1800[(5)]	NA
MW-1	1824	1815	>17
MW-2	1825	1803	>4
MW-3	1817	1797	>10
MW-4	1822	1798	>6
MW-5	1813	1797	>14

<u>Notes</u>:

1. Surface elevation for borings B-1 through B-15 estimated from *Google Earth Pro*. Surface elevation for borings MW-1 through MW-5 based on actual survey data provided by the City.
2. In most cases the borings were terminated in granular soils, so the vertical extend and thus actual thickness is unknown.
3. Borings B-3, B-5, and B-7 did not encounter granular soils within the boring depth. Therefore the presence of any underlying granular soil at greater depths (than boring) was not documented.
4. A 2-foot layer of clayey soil was encountered underneath the granular stratum.
5. A 10-ft layer of granular soil was encountered in boring B-15 between elevation 1817 and 1807 ft-msl, but was disregarded on the basis the granular layer is partially cemented and above the water table.

Groundwater Monitoring Well Installations

Two inch diameter PVC groundwater monitoring wells were installed adjacent to each of the five (5) recently completed soil borings. The wells were installed by SKG Engineering immediately after completion of the exploratory borings. The wells were installed to depths ranging from approximately 28 to 33 feet, and in all cases included a 10-foot screen interval with a coarse sand filter pack surround. Details of the well installations were previously included in a Geotechnical Report provided by SKG Engineering, dated June, 2015.

One of the purposes for installing the monitoring wells was to obtain a more accurate measurement of the water table elevation, which had previously been estimated from soil boring observations. A second purpose was to provide a means for conducting in-field permeability tests on the granular soils.

The locations of the five wells approximately coincide with the MW boring locations identified in Figure 5.

Water Table Measurements and Discussion

Water level measurements were obtained by HDR staff on August 25th and 26th, 2015. The measurement data and corresponding water table elevation is provided in Table 1.

Table 1 – Groundwater Elevation Measurements

Monitoring Well	Ground Elevation (ft)(msl)	Water Table Elevation (ft)(msl)	Depth Below Ground (ft)
MW-1	1824.12	1806.71	17.41
MW-2	1825.12	1806.59	18.53
MW-3	1816.92	1804.34	12.58
MW-4	1821.78	1806.02	15.76
MW-5	1816.57	1804.44	12.13

As noted in Table 1, the water table measurements in the wells are reasonably consistent, ranging from 1804.34 to 1806.71. From a practical standpoint, the variation in the measurement data is rather small and possibly related to ongoing irrigation activities at the site during the summer months and regional water table gradients.

The South Concho River lies adjacent and east of the study area. The river elevation, as measured by the City on June 24, 2015, was 1804.95 feet. The measured river elevation nearly coincides with the water table elevation measurements noted in Table 1. This is expected given the close proximity of the wells to the river. Based on the soil depositional environment (alluvium) it is highly likely there is a direct hydraulic connection between the site's granular soils and the South Concho River.

It is probable the water table elevations across the site could be higher or experience greater variability during periods of wet weather, either due to increased base flow toward the river or a short term high water condition in the river. Conversely, water table elevations across the site could decrease during a lower river stage condition.

In-Field Permeability Test Results

In field permeability tests were conducted at each of the well locations by HDR staff between August 25th and August 26th, 2015. The test method that was used is often referred to as a "slug" test. In summary, a solid cylindrical tube or "slug" of known volume is inserted into the well. Immediately upon insertion there is sudden rise in the water level within well, which is in response to the inserted tube. The immediate rise in the water level within the well is directly proportional to the volume of the slug. The temporarily displaced volume of water is equal to the total volume of the submerged tube or "slug". Based a nominal 2-inch diameter well, and the length and diameter of the "slug", the temporary rise in the water level was computed to be 3.4 feet.

After a certain amount of time the increased water level within the well "fell" or dissipated back to its original elevation, which coincided with the natural ground water table, as provided in Table 1. After equilibrium was re-established the slug was rapidly removed, thus causing an instantaneous drawdown of the water level within the well. In other words, the water level within the well was temporarily 3.4 feet below the natural water table. The newly created non-equilibrium condition caused groundwater from within the surrounding saturated granular stratum to flow into the well until the water level re-established equilibrium (i.e. rose 3.4 feet). The rate or time required to re-establish equilibrium is proportional to the permeability of the granular soils that surround the well.

Since the granular soils are highly permeable, the time required to re-establish equilibrium tended to be very short, generally 30 seconds or less at all five well locations. Using the time recorded data, the permeability of the sounding soils were determined by using a well established computational method ("*The Bouwer and Rice Slug Test – An Update*", Vol. 27, No. 3 – Ground Water, May-June 1989). The permeability results at each well location are presented in Table 2.

Table 2 – In-Field Permeability Test Results

Monitoring Well	Test Zone Elevation (ft)(msl)	Primary Test Material	Permeability (cm/sec)
MW-1	1810.0 to 1800.0	Silty Gravelly Sand	3.3×10^{-2}
MW-2	1806.6 to 1798.6	Silty Gravelly Sand	2.6×10^{-2}
MW-3	1790.9 to 1800.9	Silty Gravelly Sand	5.6×10^{-3}
MW-4	1794.1 to 1804.1	Silty Gravelly Sand	6.8×10^{-3}
MW-5	1783.5 to 1793.5	Silty Sandy Gravel	1.2×10^{-2}

<u>Discussion</u>

The in-field permeability results range from 0.0056 to 0.033 cm/sec or approximately 16 to 94 ft/day. The given range is indicative and consistent with literature values for the granular soil type(s) that were identified during completion of the companion exploratory boring.

Qualitatively, the reported soil permeability values are moderate to high. This is supported by an existing on-site irrigation well that appears to be capable of meeting most or all of the supplemental water requirements for growing crops.

Dewatering the moderate to highly permeable soils could prove to be technically challenging, depending on the required dewatering depth, project area, and uniformity/thickness of the saturated granular layer.

Clay Liner Thickness

As previously noted, a clay liner installed below the water table will realize buoyant or hydrostatic uplift pressures. The magnitude of the uplift pressure on the underside of the clay will be proportional to the depth it is placed below the surface of the water table. To avoid uplift or heave failure, the clay liner will need to impose a counterbalance pressure on the excavation subgrade that is equal to or greater than the hydrostatic uplift pressure. Therefore, the deeper the basin is below the water table, the thicker (or "heavier") the clay liner will need to be to resist the uplift pressure. Table 3 presents the required clay thickness at various depths below the water table, assuming a natural groundwater elevation of 1806 feet (msl).

Table 3 – Clay Liner Thickness vs. Depth Below Water Table

Water Table Elevation (ft)(msl)	Finished Basin Bottom Elevation (ft)(msl)	Basin Depth Below Water Table (ft)	Required Clay Liner Thickness (ft)	Bottom of Basin Excavation (ft)(msl)	Basin Excavation Depths(1) (ft)
1807	1805	2	2	1803	12 to 17
1807	1802	5	5	1797	18 to 23
1807	1797	10	10	1787	28 to 33

Notes:

1. Based on existing grades ranging from El. 1815 to El. 1820.

Dewatering Concepts

Based on Table 3, the maximum depth of excavation would be 28 to 33 feet below existing grade, or approximately 18 feet below the water table elevation. Constructing the clay liner below the water table will require a certain amount of seepage control to maintain a "dry" excavation.

As the excavation gets close to or into the saturated granular stratum the complexity or degree of difficulty in maintaining a dry excavation will increase. Even if the excavation does not extend into the saturated granular soils, any overlying native cohesive soil could be subject to heave or uplift failure, as illustrated in Figure 6.

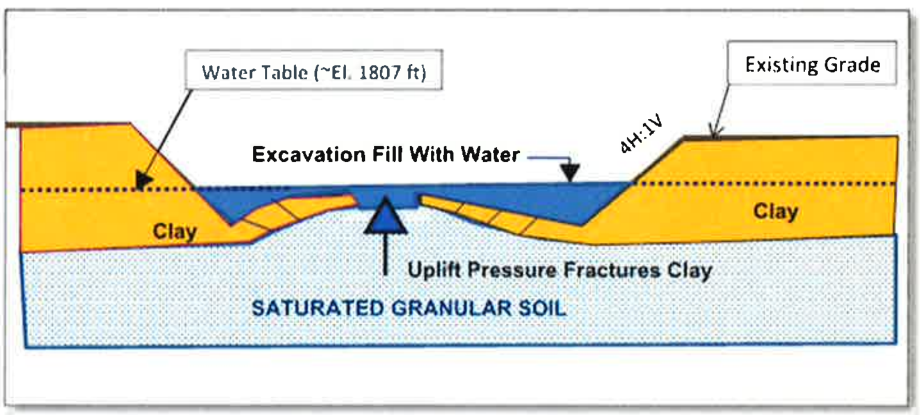

Figure 6 – Basal Heave Failure During Excavation

Fundamental Methods

There are two fundamental methods for controlling groundwater during excavation, those methods which keep water out, or those methods which depend on its control by drainage processes. Chemical grout, cement grout, steel sheet piles, or slurry walls are methods that often serve to keep out most of the water. However, in this particular case, their use would almost certainly be cost prohibitive. Therefore control of groundwater by drainage methods are considered applicable for this study.

Drainage methods include the collection of seepage from sumps located within the excavation. In soils of relatively low permeability, e.g. clay soils, the use of collection sump are normally the most feasible, since the volume of seepage would be relatively small. Conversely, non-cohesive or granular soils often require a more active dewatering system, e.g. well points or vacuum wells.

Active Dewatering Systems

Active dewatering systems have the following main purposes:

1. Intercepting seepage before it enters the excavation and interferes with the work.
2. Improving the stability of slopes, thus preventing sloughing or slope failures.
3. Preventing the bottoms of excavations from having excessive hydrostatic pressure.
4. Drying up the soils to be excavated so they can be more properly compacted.

The three most commonly used active dewatering methods include:

- Well Points
- Pumping Wells
- Vacuum Wells

Well Points

A well point usually is a small diameter tube or pipe fitted with plastic or metal screens which permit water to enter without the loss of adjacent soil. They often are equipped with metal points which permit them to be driven or jetted into soil formations. Well points are most successfully used in sand and silty sand. They are usually installed in a line or ring surrounding the excavation and are connected through a manifold to a suction pump which extracts seepage to lower the water table in the area to be excavated.

The required spacing, usually between 3 and 12 feet, depends on the soil type and the amount the groundwater table must be lowered. Well points are often used for dewatering excavations that do not require deep lowering of the water table. If the water table or hydrostatic head must be lowered by more than 20 feet, the maximum effective lift of suction pumps, then two or more stages of well points, or deep pumping wells would be required.

Pumping Wells

Large diameter "production" style wells (equipped with submersible pumps) are often used to significantly lower the water table or hydrostatic head, especially when the area of concern is wide spread or expansive. In some cases, pumping wells may be the only viable option if the water bearing stratum is deep or the water table must be lowered by more than 20 feet.

The required spacing for pumping wells is highly dependent upon the permeability and thickness of the target stratum, its proximity to a lake or river, and the diameter of individual pumping wells.

Vacuum Wells

Vacuum wells can be used to strengthened fine-grained soils that would otherwise have the potential to liquefy or lose their shear strength due to an improper balance between the hydrostatic water pressures within the soil pores and the overall confining pressure. Vacuum wells are similar to a well point, though installed in a borehole with a surrounding sand filter. A vacuum or educator pump is connected to the well and used to reduce the pore pressures within the surrounding fine-grained soils (e.g. silts). This reduction in pore pressure ultimately stabilizes and strengthens the soil.

Initial Assessment of Available Active Dewatering Systems

Well Points

The maximum anticipated lowering of the water table at the Basin 2 site is on the order of 18 feet, indicating a single stage well point operation is technically feasible. Well point spacing on the order of 5 to 10 feet around the perimeter of the excavation area may be required depending on the water table lowering requirement. It is plausible that additional well points could be required within the central portions of the excavation, depending on the total thickness of the saturated granular stratum and/or its uniformity.

Pumping Wells

The use of pumping wells at the subject site may have some applicability, given that the maximum excavation depth is estimated to be 18 feet below the water table. In this case, the use of pumping wells versus well points would depend on hydrogeological factors and parameters that were not defined as part of this study. However as a general rule, pumping wells are most applicable when lowering the water table by more than 18 feet.

Vacuum Wells

Vacuum wells would be relatively ineffective in the moderate to highly permeable granular stratum that prevails across the site.

Appendix E. Storm Water Storage Basin Feasibility Study for UCRA – June 2013

Storm Water Storage Basin Feasibility Study

Red Arroyo Watershed, San Angelo

June 2013

WSA01400

Prepared for:

Upper Colorado River Authority
512 Orient Street
San Angelo, TX 76903

JACOBS

Jacobs Engineering Group Inc. Reg. No. F-2966 777 Main Street
Fort Worth, Texas 76102 817-735-6000

Jacobs

Storm Water Storage Basin Feasibility Study

Red Arroyo Watershed, San Angelo

6/24/2013

STATE OF TEXAS
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92709
LICENSED
PROFESSIONAL ENGINEER

B. Mukhopadhyay
6/24/2013

TABLE OF CONTENT

Executive Summary

- Upper Colorado River Authority (UCRA) proposes to construct a storm water storage basin near the confluence of Red Arroyo and the South Concho River. The purpose of this municipal basin will be to treat storm water runoff from Red Arroyo for subsequent downstream utilization or delivery of the stored water to the water treatment plant (Lone Wolf Reservoir) located about half a mile north west of the proposed storage basin.
- This report summarizes the findings to evaluate the feasibility of construction of the storm water storage basin.
- UCRA and the Texas Institute for Applied Environmental Research (TIAER) at the Tarleton State University recently completed (February 2013) a study to establish a Storm Water Management Plan for the City of San Angelo (COSA). This study developed a hydrologic model of the Red Arroyo Watershed. The model was developed using EPA-SWMM computer program.
- Jacobs submitted an interim report to UCRA on April 26th, 2013. Key elements of the interim report were:
  - Recommendation to update and refine the hydrologic model developed by TIAER.
  - A detailed rainfall analysis identifying the 95th percentile rainfall depth. Analysis of the rainfall is a crucial step in evaluating an optimal storage capacity for the proposed storm water storage basin. Optimum storage for the storm water storage basin can be assessed by simulating the hydrologic model with a rainfall event which is obtained by statistical frequency analysis of historical rainfall data.
- Following submittal of the interim report, a conference call was set up to discuss project goals. Key elements of the conference call are as follow:
  - Storm water basin volume of 1500 to 2500 acre-feet has been set as the target storage capacity
  - A status update of the hydrologic model
  - Stream flow hydrographs estimated by UCRA will be used for modeling the hydraulic functionality of the storage basin as an integral component of the feasibility analysis.
  - Stream flow hydrographs for two events namely the August 2011 event and the January 2012 event will be used for hydraulic feasibility analysis.
- Jacobs utilized XP-SWMM for the hydraulic feasibility analysis. Multiple models were created for this purpose. In general, the models of Red Arroyo start from upstream of FM 1223 and extend all the way to the outfalls at South Concho River. Red Arroyo gets diverted towards an open channel just north of Ave L, flowing through a 18'(W) X 12' (H) box culvert under Ave K with the channel ultimately outfalling to South Concho River downstream of the Lone Wolf dam. The main stem of Red Arroyo outfalls to South Concho River just north of Ave L.
- Two basin scenarios have been evaluated. Basin 1 provides 2861 acre-feet of storage volume and Basin 2 provides 1839 acre-feet of storage volume. Any of this storage basins will be adequate to capture runoff volumes for the frequent rainfall events that occur 95% of the times. Water from the Red Arroyo will be diverted to the basin utilizing inflow structures. More than 99% of runoff volumes resulting from frequent storms will be captured. For rare events, up to 87-90% of runoff volumes are expected to be captured by the proposed system. For safety, an emergency spillway has also been included as a part of the proposed system. However, for the modeled events, the water surface in the basin did not reach the elevation of the emergency spillway.
- A pump system is recommended to be utilized to draw the captured water from the basin and delivered to the Lone Wolf Water Treatment Plant.
- The probable cost of construction of the storage basin and the associated infrastructure to withdraw and convey water is anticipated to be $70.6 million for Basin 1 (2861 acre-feet).
- The probable cost of construction of the storage basin and the associated infrastructure to withdraw and convey water is anticipated to be $20.4 million for Basin 2 (1839 acre-feet).

- The concept of a storm water storage basin to supply water to the water treatment plant is technically feasible and economically viable. Comparing hydraulic efficiency and cost of construction of the two basins, Basin 2 is the preferred alternative.

Background

The Upper Colorado River Authority (UCRA) selected Jacobs Engineering Group, Inc. (Jacobs) to provide engineering services for the evaluation of feasibility of a proposed storm water storage basin and if it the proposal seems technically feasible and economically viable then to design the system. The project will be conducted in two phases- Phase 1 and Phase 2.

Phase 1 comprises of the following tasks:

- Task 1- Site exploration (includes geotechnical services)
- Task 2- Water Quality Data Analysis
- Task 3- Interim Report
- Task 4- Project Goal Meeting
- Task 5- Alternative Project Analysis (includes development of the hydrologic model of the Red Arroyo watershed, evaluation of alternatives to discharge water to the treatment plant, and preparation of preliminary opinion of probable cost)
- Task 6- Final memorandum

After completion of Phase 1 tasks, Phase 2 will be authorized and will involve design and production of construction documents for the storm water storage basin. Exhibit 1 shows the location of the proposed storm water storage basin.

Basis of Design

UCRA and the Texas Institute for Applied Environmental Research (TIAER) at the Tarleton State University recently completed (February 2013) a study to establish a Storm Water Management Plan for the City of San Angelo (COSA). This study developed a hydrologic model of the Red Arroyo Watershed. The model was developed using EPA-SWMM computer program. Jacobs planned to use this hydrologic model after verification as a starting point for the present investigation which involves modeling of the hydraulics of the proposed system.

Data Collection

Table 1 lists the various data collected by Jacobs during Phase 1.

Table 1: Data Collection Summary

Description	Source	Format
UCRA Storm water management plan for COSA	UCRA	PDF document
Hydrologic model	TIAER/ UCRA	EPA-SWMM
Land Use	TIAER	GIS shapefile
Impervious Area	TIAER	GIS shapefile
Soil Data	TIAER	GIS shapefile
Sub-basin delineation	TIAER	GIS shapefile
Gage/ monitoring station locations	TIAER/UCRA	PDF Map
1' topographic contours	COSA	GIS shapefile
Rainfall data	UCRA	Microsoft access database
Stage data	UCRA	Microsoft access database
Stream center line	FEMA DFIRM	GIS Shapefile
Historical rainfall data for Mathis Field (1949-2012)	NOAA	Comma separated variable (csv)

Evaluation of current hydrologic model

As part of the Storm Water Management Plan for the COSA, TIAER developed a hydrologic model of the Red Arroyo watershed using EPA-SWMM computer program. The drainage area of the Red Arroyo watershed is approximately 15 square miles. The model is set up to estimate runoff rates for frequency-based design storms with 1-year and 5-year return periods. In addition to the design storms, the model also estimates runoff rates for four relatively recent storm events in San Angelo as noted in Table 2.

Table 2: Rainfall Events used by TIAER in EPA SWMM model

Event Name	Rainfall Event Duration	Storm Total Precipitation
August 2011 Event	08/13/1106:00 AM to 08/17/11 19:00 PM	3.74 inches
October 2011 Event	10/08/11 08:00 AM to 10/11/11 17:00 PM	2.71 inches
January 2012 Event	01/24/12 14:30 PM to 01/30/12 02:00 AM	1.67 inches
March 2012 Event	03/08/12 22:00 to 03/10/12 09:15 AM	0.74 inches

TIAER attempted to calibrate the hydrologic model using the data for the events identified in Table 2. UCRA operates 5 gages (measuring rainfall depths and water surface elevations/stages) in the Red Arroyo Watershed. The gages are identified as Site 2, Site 6, Site 7, Site 8 and Site 9. According to the TIAER report[1], uncertainty in the measured rainfall depths has led to either under or over estimation of the peak flow rates and total discharge volumes for the four rainfall events.

After communicating with the authors of the TIAER report, our understanding is that the calibration process was focused only on estimated peak flow rates and discharge volumes, and not on stage heights in the channels. UCRA used the stage data and cross-sectional geometry data at the gage locations to estimate the flow rates using Manning's equation for the four storm events. The authors of the TIAER hydrologic model indicated that the flow rates provided by UCRA were adjusted by modifying the Manning's roughness coefficient based on field observations. The adjusted flow hydrographs were subsequently used by TIAER for calibration. The subbasin equivalent width parameter was also modified to calibrate the model to the adjusted flow rates.

[1] Appendix A of the Storm Water Management Plan for the City of San Angelo

The kinematic wave model approximates the overland surface of a sub-basin as a rectangular plane. Thus, the TIAER EPA-SWMM model utilizes a sub basin equivalent width parameter to estimate an overland flow length (Equation 1) as an input to the calculation of time of concentration or travel time according to kinematic wave model of overland flow (Equation 2).

$$Overlandflowlength(L_o) = \frac{\text{Area}}{\text{Subbasinequivalentwidth}}$$
(Eqn 1)

$$Time\ of\ Concentration\ \ (T_c) = \left(\frac{L_0}{a \times i^{(m-1)}}\right)^{\frac{1}{m}}$$
(Eqn 2)

Where T_c= travel time (time of concentration) in second
L_o= maximum overland flow length (ft) as given in Equation 1
a and *m* are kinematic wave parameters.
The parameter, *a* is given as
$a = (1/N)*S_o^{1/2}$
Where,
N = Manning's roughness coefficient for overland surface
i = rainfall intensity for a desired frequency (in/hr) at the time of concentration
S_o= Overland slope (ft/ft)
The parameter *m* is constant (*m* = 1.67).

Salient features of the TIAER EPA-SWMM model input-

- Rainfall losses are estimated using Green-Ampts method which uses *three* parameters
- A sub-basin equivalent width and slope are specified for each sub-basin, EPA-SWMM uses these parameters for estimating the time of concentration using kinematic wave theory. In this method, a sub-basin is approximated as a rectangular area and computation of time of concentration requires *five* parameters
- Varying rainfall hyetograph inputs to the sub-watersheds based on 5 rain gages
- Hydraulic routing of reaches assume uniform geometry of natural channels

The primary purpose of the TAIER hydrologic modeling effort was directed towards establishing a model framework for estimation of peak flow rates and discharge volumes for implementation of Best Management Practices (BMPs). The BMPs address storm water quality and quantity issues in the Red Arroyo watershed.

Updating the EPA-SWMM model to XP-SWMM

Although the EPA-SWMM program has been widely used by many engineers, it does not utilize the most robust hydrologic and hydraulic modeling platforms for the numerical algorithms (an explicit finite difference scheme) that are implemented in its computational engine. Therefore, for the purpose of accurately estimating the hydraulic functionality of the inflow and outflow structures in the proposed storm water storage basin, Jacobs recommended in the scope of work to update the EPA-SWMM model to XP-SWMM. The advantage of using a platform like XP-SWMM is that the hydrology of the watershed, the hydraulics of the channel, the storage basin, and the inflow and outflow structures can be simulated simultaneously. In addition the finite difference algorithms implemented in XP-SWMM provide better accuracy and a more stable numerical solver.

Model conversion

- Translation of the model to the new platform required review to ensure all input parameters are identical in both model versions:
    - Spill crest elevations
    - Channel geometries and parameters
    - Sub basin width

Updating an existing model to another platform presents challenges. Jacobs converted the EPA SWMM model to XP SWMM platform and reviewed the converted XP SWMM model to ensure that all elements and parameters in both model versions were identical. After conversion, we found that there were various differences in parameter values between these two models such as spill crest elevations, channel geometries and sub-basin widths. Subsequently, these parameters have been corrected in the XP SWMM model to match the values that are present in the EPA SWMM model.

Comparison of EPA-SWMM and XP-SWMM model Output

XP SWMM model has been simulated using the same rainfall events used by TIAER in EPA SWMM model as summarized in Table 2. Apart from the events summarized in Table 2, frequency-based design storms with 2-year and 5-year return periods have also been simulated in EPA SWMM and XP SWMM models.

The results of peak flow rates, total discharge volumes, and time to peak are summarized in Table 3, Table 4, and Table 5 respectively. These comparisons are made at Site 2 which is immediately downstream of the location of the proposed storm water storage basin. Exhibit 2 shows the location of Site 2 in relation to the proposed storm water storage basin.

As noted earlier, UCRA used the stage data at the gage locations and the cross-sectional geometries at the gage location to estimate the flow rates using Manning's equation. The peak flow estimated by UCRA is summarized in Table 3 and the total discharge volumes from the measured flows are tabulated in Table 4. The authors of the TIAER hydrologic model indicated that the flow rates provided by UCRA were adjusted by modifying the Manning's roughness coefficient on the basis of field observations. The adjusted flow hydrographs were subsequently used by TIAER for calibration. The sub-basin equivalent width parameter was modified to calibrate to the adjusted flow rates. Communications with UCRA personnel during the course of the present investigation indicated the observed flow rates are reasonably accurate and can be used with confidence. Thus, for subsequent discussions, comparisons are made between the simulated values and the observed values estimated by UCRA (see footnote 3).

Table 3: Peak Flow Rates at Site 2

	PEAK FLOW (in cfs) at Site 2			
Rainfall Event Modeled	**Measured - from UCRA[2]**	**Measured-revised by TIAER[3]**	**EPA SWMM Simulated[4]**	**XP SWMM Simulated[5]**
August 2011 Event	2092	1255	2203	5717
October 2011 Event	1390	834	465	1314
January 2012 Event	685	411	170	685
March 2012 Event	429	257	109	448

From Table 3, it can be noted that in case of the EPA SWMM results, the peak dischargesfor three of the events are underestimated with the exception being the August 2011 event. The XP SWMM results are also underestimated, albeit to lesser extents compared to the EPA SWMM results, with the exception being the August 2011 and March 2012 events. The overestimation of the peak discharge value for the March 2012 event is minor but for the August 2011 event, it is quite significant. However, UCRA has informed Jacobs that part of the outflow during August 2011 event was diverted upstream of the culvert outlet. Hence the observed discharge was perhaps lower than actual.

Table 4 shows the observed time of peak discharge and that calculated by both EPA SWMM and XP SWMM models for each of the storm events modeled. As shown in Table 4, the peak discharge in

[2] Estimate provided by UCRA to TIAER
[3] Estimate revised by TIAER based on field adjustments of Manning's *n* parameter. However, a subsequent discussion with UCRA reveals that the estimates made by UCRA are reasonably accurate and there is really no valid reason to make the corrections to these values.
[4] Estimate from the EPA SWMM hydrologic model by TIAER
[5] Estimate from the converted TIAER hydrologic model executed using XP-SWMM

the XP SWMM simulations occurs sooner than the observed time for all of the events analyzed. The same is true for EPA SWMM simulations except for the March 2012 event simulation when the time of peak discharge occurs 150 minutes later than the observed time of peak discharge. Compared to the XP-SWMM simulation results, the time difference between the observed and calculated time of peak discharge in the EPA SWMM models is less, with the exception of the August 2011 event.

Table 4: Time of Peak at Site 2

Rainfall Event Modeled	Time to Peak at Site 2		
	Observed - from UCRA[2]	EPA SWMM Simulated[4]	XP SWMM Simulated[5]
August 2011 Event	8/13/11 1:35 PM	8/13/11 1:05 PM	8/13/11 1:15 PM
October 2011 Event	10/8/11 11:40 PM	10/8/11 8:00 PM	10/8/11 3:29 PM
January 2012 Event	1/25/12 8:35 AM	1/25/12 7:30 AM	1/25/12 3:09 AM
March 2012 Event	3/9/12 3:20 PM	3/9/12 5:49 PM	3/9/12 10:09 AM

Table 5 gives the total volumes of runoff estimated by UCRA, TIAER, and those from the calculated runoff hydrographs in EPA SWMM and XP SWMM models.

Table 5: Total Discharge Volumes at Site 2

Rainfall Event Modeled	TOTAL DISCHARGE VOLUME (in ac-ft) at Site 2			
	Measured - from UCRA[2]	Measured-revised by TIAER[3]	EPA SWMM Simulated[4]	XP SWMM Simulated[5]
August 2011 Event	1881	1129	1047	1385
October 2011 Event	1483	894	461	731
January 2012 Event	1007	604	316	470
March 2012 Event	292	175	126	215

From Table 5, it can be noted that the calculated volumes of total runoff for each of the simulated events as given by both the EPA SWMM and the XP SWMM models, are significantly less than those estimated from flows observed by UCRA.

Besides the peak flow rate, the total discharge volume is also governed by the shape of the hydrograph which in turn is controlled by the watershed characteristics as well as rainfall patterns and distributions. The observed and calculated hydrographs for the four storm events analyzed are shown in Figures 1, 2, 3, and 4. These figures also illustrate the variability in the flow hydrographs with respect to rainfall patterns as shown by the rainfall hyetographs for the events.

For the modeled storm events, the shapes of the calculated hydrographs are significantly different from those derived from observed flows (Figs. 1 - 4). This observation suggests that the physical and hydrologic characteristics of the sub-basins and the watershed are not captured accurately in the models. The calculated hydrographs have sharper peaks with steeper rising and falling limbs compared to those derived from observations. This observation further indicates that the model parameters do not accurately represent the lag time and characteristics of the rainfall loss and runoff generation in the watershed. Table 6 shows all of the physical and hydrological parameters of the sub-basins that are input to both EPA SWMM and XP SWMM models. These are the parameters that have been derived by TIAER for development of the hydrologic model of Red Arroyo Watershed. From the discussion presented above it is evident that when these multiple sets of parameters are used, the hydrologic response of Red Arroyo Watershed cannot be modeled with reasonable accuracy.

As shown in Table 6, there are five parameters that control the values of time of concentration or watershed lag time, three parameters that govern the characteristics of rainfall losses, and five parameters that define characteristics of runoff generation. Accurate estimation of these parameters requires a large body of data obtained through observations over a long period of time and their refinements in the process of calibration need to be established by computational algorithms such as Monte Carlo simulations.

Application of kinematic wave theory in SWMM modeling environment requires approximation of a sub-basin by using a rectangular planar area. This in turn requires sub-basin areas to be small enough so that such an approximation can be made. As shown in Table 6, the sub-basin areas are too large to perform these approximations with a reasonable validity. Similarly, by using a single value for certain parameters for each of the sub-basins, an assumption is made that over such large areas, there is no spatial variations in that set of model parameters. Such an assumption is not valid in reality.

From the observations made above, it can be stated that in order to develop a calibrated hydrologic model of Red Arroyo Watershed using the same methodology adopted by TIAER, the watershed must be divided into a large number of smaller sub-basins and for each of such smaller sub-basins, all of the model parameters must be evaluated on the basis of field observations, soil type, land use, and land cover. Alternatively, a hydrologic method can be adopted that uses a smaller set of model parameters that can be reasonably lumped over a relatively large area tolerant to spatial variation within reasonable limits.

Table 6 : Hydrologic Parameters Summary Table

		Surface characteristics*														
		Impervious Area			Pervious area		Green-Ampt parameters**			Basin characteristics*				Kinematic wave parameters***		
Subbasin ID	XP SWMM Node ID	Depression storage (inches)	Manning's surface roughness "N"	Zero detention (%)	Depression storage (inches)	Manning's surface Roughness "N"	Suction head (inches)	Initial moisture deficit	Saturated hydraulic conductivity (inches/hour)	Area (acres)	Percent imperviousness (%)	Width (ft.)	Slope	Conveyance factor (a) for Impervious Area	Conveyance factor (a) for Pervious Area	m
20	J16	0.1	0.024	25	0.3	0.8	6.3012	0.4636	0.0717	673.95	26.9	58714	0.01	4.2	0.1	1.67
21	J20	0.1	0.024	25	0.3	0.8	7.315	0.501	4.74	110.78	42.89	9651	0.01	4.2	0.1	1.67
24	J18	0.2	0.024	25	0.3	0.8	8.27	0.479	0.26	445.74	36.03	194166	0.01	4.2	0.1	1.67
25	J3	0.2	0.024	25	0.3	0.8	7.2343	0.501	0.83	2300.11	39.76	200386	0.01	4.2	0.1	1.67
28	J8	0.2	0.2	25	0.3	0.8	6.0935	0.4644	0.46	2140	54.68	186436	0.01	0.5	0.1	1.67
29	J14	0.1	0.024	25	0.3	0.8	7.796	0.501	4.74	581.71	45.27	50678	0.01	4.2	0.1	1.67
30	J4	0.1	0.024	12.5	0.3	0.8	6.3639	0.398	0.06	1271.24	23.81	110751	0.005	2.9	0.1	1.67
31	J9	0.2	0.011	25	0.3	0.05	7.8292	0.4639	0.26	265.95	72.88	23169	0.01	9.1	2.0	1.67
32	J5	0.1	0.024	12.5	0.3	0.8	5.5591	0.398	1.18	791.62	15.02	68966	0.005	2.9	0.1	1.67
33	J12	0.1	0.024	25	0.3	0.8	7.468	0.501	4.74	182.06	26.31	15861	0.01	4.2	0.1	1.67

Rainfall Analysis

Analysis of the rainfall is a crucial step in evaluation of an optimal storage capacity for the proposed storm water storage basin. Optimum storage for the storm water storage basin can be assessed by simulating the hydrologic model with a rainfall event which is obtained by statistical frequency analysis of historical rainfall data.

The historical rainfall data obtained from UCRA is for a two-year period from 2010 to 2012. The two-year period of record for rainfall data is not sufficient to perform a statistical analysis to determine a rainfall depth with a certain probability of occurrence. Therefore, other sources were researched to find rainfall data with a longer period of record to perform a meaningful frequency analysis. The National Oceanic and Atmospheric Administration (NOAA) operate weather stations throughout the nation that record daily and/or hourly rainfall patterns. A NOAA rain-gage station, Mathis Field (See Exhibit 1) is located close to the proposed basin site location.

The Mathis Field rain-gage station has hourly precipitation records from 1949 to present. We have used the set of rainfall data covering a period of 63 years (1949 - 2012) from this rain-gage station to conduct a frequency analysis. The objective of this analysis is to identify a rainfall depth that can be utilized in the hydrologic and hydraulic modeling to optimize the volumetric discharge that exceeds the volumetric discharges that can result from frequent events. This rainfall depth is identified as the 95^{th} percentile of the rainfall depths that are given in a period of record. Because the 95^{th} percentile rainfall depth represents a precipitation amount thatis not exceeded by 95% of the rainfall events that occurred during the period of record. Federal guidelines established in the Section 438 of the Energy Independence and Security Act (EISA) recommend that at least a 20 to 30-year period of rainfall record is to be used to establish the 95^{th} percentile rainfall depth.

Figure 5 shows the daily rainfall depths at the Mathis Field rain-gage station for the 63 year period. During this period, there were 2,047 days (out of 23,009 days) on which 24-hour total rainfall depths were greater than 0.1". The events that produced rainfall depths less than 0.1" were not counted as rain events since such events did not generate any significant runoff. Based on this observation, the probability of having a rainy day in this area is only 0.09 [P(r)]. This indicates the general aridity of the area and requires appropriate sizing of the storage basin so that a minimum pool level can be maintained in the basin for most of the year.

Figure 6 is the cumulative frequency spectrum of 24-hour rainfall depths based on the recorded 2047 days of daily rainfall data. As shown in Figure 9, the 95^{th} percentile 24-hour rainfall depth is estimated to be 1.67". This implies that only 5% of the events in the 2047 days exceeded 1.67" of 24-hour rainfall.

Out of the 2,047 rainy days with 24-hour rainfall depths exceeding 0.1" only 32 days had 24-hour rainfall depths in the range of 1.67" ± 0.1". Thus, the probability of occurrence of a rainfall event that would produce a 24-hour rainfall depth of 1.67" ± 0.1" is 0.01. In other words, such a rainfall depth generates a runoff which has only 1% chance of occurrence in a given year. Thus, this rainfall depth can be used to assess an optimum storage volume for the proposed storm water detention basin. Such a storage volume will be adequate to capture the runoff generated from frequent rain events and is also expected to capture those resulting from events with larger magnitudes and rarer frequencies.

Once the events for which 24-hour rainfall depths are close to 1.67" are identified, it is necessary to select one or two particular events that can be used in hydrologic modeling. This selection is based on the rainfall patterns exhibited by these events. The rainfall patterns of the events that are similar to the one established for this region are the best candidates for event-based hydrologic modeling.

The City of San Angelo lies within the zone of Type II rainfall distribution pattern as described in the Technical Release No. 55 (Urban Hydrology for Small Watersheds) developed by the Natural Resources Conservation Services (NRCS). As shown in Figure 7, time distribution of the rainfall from the May 3, 2005 storm event closely matches with the NRCS Type II storm hyetograph. The rainfall mass curve of this event is also similar to the mass curve of Type II rainfall pattern, as shown in Figure 8. From this comparative analysis, May 3, 2005 storm event has been selected to be used in the hydrologic modeling which in turn will be used in the hydraulic modeling to assess the optimum capacity of the storm water storage basin.

It should be noted at this point that the 24-hour rainfall depth that has been selected in this investigation is less than the corresponding rainfall depths with 2- and 5-year return periods (Table 7) as derived from frequency-based design storm data given in the Technical Paper 40 (Rainfall Frequency Atlas of the United States for durations from 30 minutes to 24 hours and return periods from 1 to 100 years) developed by the United States Department of Commerce (1961).

Table 7: Frequency Based Design Rainfall Depths and 95th Percentile Rainfall Depth

95th percentile rainfall depth in 24-hour period	2- Yr 24-hour frequency based design rainfall depth	5- Yr 24-hour frequency based design rainfall depth[6]
1.67”	2.37”	3.43”

At the time of preparation of this report, the scope of work for Jacobs does not include development of a new hydrologic model using the May 3, 2005 storm event and more appropriate methodology as discussed above. Rather, the stream flow hydrographs derived from the January 2012 event with a total rainfall depth of 1.67” and the August 2011 event with a total rainfall depth of 3.74” are utilized for the feasibility analysis of the detention basin through hydraulic analyses and the subsequent feasibility analysis to use stored water for supply to the water treatment plant.

Hydraulics

To simulate the movement of water though Red Arroyo and the functionality of the proposed storm water storage basin, a hydraulic model was created using XP-SWMM. A fully dynamic routing methodology was selected in XP-SWMM to predict the magnitudes, volumes, and temporal patterns of the flows as those are translated down the channel. The data needed to create a model for an open channel are- channel geometry and flow rates.

The channel geometry is represented by a node- link system. A node can represent a junction or a storage element, and a link represents any element that conveys water including but not limited to channel sections, orifices, weirs, culverts, pumps etc. The hydrographs and associated flow rates estimated by UCRA for the August 2011 and the January 2012 events (see Table 2) at Site 2 are utilized for the hydraulic analysis as the current hydrologic model for the Red Arroyo watershed created by TIAER was not updated (see discussion above).

Model domain

The hydraulic model of Red Arroyo starts from downstream of FM 1223 and extends all the way to the outfall at South Concho River. The main channel of Red Arroyo flows under Ave L and towards an 18’ (W) X 12’ (H) box culvert under Ave K with the channel ultimately outfalling to South Concho River downstream of the Lone Wolf Dam (Outfall 1). A portion of Red Arroyo naturally diverts flows towards South Concho just north of Ave L (Outfall 2) upstream of the Lone Wolf Dam. The limit of the hydraulic model is shown in Exhibit 3.

[6] City of San Angelo Storm Water Design Manual

Existing channel cross-sections have been cut utilizing 2-foot topographic contours provided by the City of San Angelo. No survey was carried out during this phase of the investigation. Geometric elements of existing hydraulic structures (box culvert under Ave K and pipes immediately downstream of Ave K) have been provided to Jacobs by UCRA.

To evaluate the feasibility of constructing a storage basin, the following model scenarios were evaluated-

- Existing conditions
- Proposed conditions by inclusion of diversion structures, storage area, inflow structures, and emergency spillway

Boundary conditions

To estimate water surface elevations in a conduit resulting from inflows, a hydraulic model needs upstream and downstream boundary conditions to be specified. The types of downstream boundary conditions that can be specified in XP-SWMM are:

- Free outfall: The water surface elevation of the receiving waters is low enough so that a backwater effect from the downstream boundary can be disregarded. The water surface elevation at the conduit (open channel or closed pipe) at the free outfall is taken as the minimum of critical or normal depth.
- Fixed backwater: The water surface elevation at the receiving water is specified and is held constant. This water surface elevation controls the water surface elevations in the conveyance conduits.
- Varying backwater: A time-dependent backwater condition is specified at the outfall. The time varying backwater specified is that of the receiving water body.

Since there are two outfall locations- Outfall 1 and Outfall 2, we evaluated the appropriate boundary conditions to be specified at these two locations as noted below.

- Outfall 1: A free outfall is specified since there is a 15’ drop from the Red Arroyo channel invert (1804’) to the water surface elevation of South Concho River past the Lone Wolf Dam. Critical depth at the Red Arroyo at this location was specified as the downstream boundary condition.
- Outfall 2: A backwater elevation of 1806’ was specified as the fixed backwater. The backwater elevation is set at the normal pool elevation (1806’) as obtained from the stream profile of South Concho River. See Exhibit 4 for the stream profile. Red Arroyo channel invert is at 1806’. In addition to specifying a fixed backwater, critical depth was specified to be calculated at the start of the computation. XP-SWMM compares the value of the computed critical depth with the specified backwater elevation, and selects the larger of the two.

The upstream boundary conditions at the upstream location of FM 1223 are given by the inflow hydrographs for the two storm events modeled. These inflow hydrographs are those obtained by UCRA at Site 2.

Existing condition

The existing conditions of Red Arroyo are modeled to establish a base condition to evaluate water surface elevations along the channel during the August 2011 and the January 2012 events. Establishing a base condition will allow conceptualization of the proposed inflow structure and comparisons to be made for the water surface elevations between the existing and proposed conditions at critical locations along the stream to evaluate potential impacts of the proposed storm water storage basin. The model geometry of the Red Arroyo is shown in Exhibit 5 as the node-link system. Each link between the nodes represents the natural section of the channel that has been generated using the 2’ topographic contours. The stream flow hydrographs have been assigned to the most upstream node of the model as shown in Exhibit 5. The water surface profiles for both the storm events have been calculated by selecting the tailwater that is higher of the fixed backwater and the computed critical depth. Table 8 lists the flow rates along channel. Table 9 lists the water surface elevations at critical locations along the channel.

Table 8: Flow rates along the channel

Storm event	Flow rate at most upstream node (cfs)	Flow rate upstream of Ave L	Flow rate in Red Arroyo (Outfall 1)	Flow rate in the diversion (Outfall 2)
January 2012 (1.67" STP[7])	685	652	0	652
August 2011 (3.74" STP)	2789	2692	123	2569

Table 9: Water surface elevations along the channel

Storm event	WSE US of FM 1223 (ft) (Top of road:1834')	WSE DS of FM 1223 (ft) (Top of road:1834')	WSE US of Ave L (Top of road:1828')	WSE DS of Ave L (Top of road:1828')	WSE US of Ave K (Top of road:1819')	WSE DS of Ave K (Top of road:1819')
January 2012 (1.67" STP)	1819.65	1818.43	1809.02	1807.86		
August 2011 (3.74" STP)	1822.54	1820.64	1810.98	1809.55	1808.95	1808.86

Proposed condition

The proposed conditions include a storm water storage basin to capture the runoff and reuse the water by supplying it as downstream releases or to the water treatment plant. To achieve this, an inflow structure is required for the water to be blocked in the channel and then diverted into a storage basin. Blocking the water can be achieved by constructing a concrete inflow structure across the channel section. The proposed concrete inflow structure has a weir at its downstream end to allow for any overflow to flow downstream. Obstructing the flow in the channel by a concrete structure will immediately result in the water surface elevation upstream of the structure to rise. To ensure diversion of the blocked water, inflow pipes are placed immediately upstream of the weir. The inflow pipes carry the water from the channel into the storage basin by gravity. Exhibit 6A and Exhibit 6B show the concept plan for the inflow structure with its essential elements. In addition to the inflow structure, there needs to be an emergency spillway through which water will overflow to South Concho River or Red Arroyo near Avenue L in case the capacity of the storage basin is exceeded during extreme storm events. Exhibit 6A and Exhibit 6B also shows the concept plan for such an emergency spillway.

The proposed conditions hydraulic model consists of the Red Arroyo channel, two outfalls, the inflow structure, and the emergency spillway as described above. The boundary conditions are same as those used in the existing conditions model. Through an iterative process the height and length of the weir and the size of the inflow pipes are established. The iterative process is essential to optimize the dimensions of the components of the inflow structure to ensure that the inflow structure does not have any negative impact upstream by raising the water surface elevations to affect critical infrastructure or to cause property damage.

The storm water basin volume desired by UCRA is approximately 1500 to 2500 acre-feet. Using the topographic contours, two separate detention basin shapes have been approximated to be able to

[7] STP: Storm total precipitation

provide the volume desired. The preliminary basins are identified as Basin 1 (providing approximately 2800 acre-feet of storage volume) and Basin 2 (providing approximately 1800 acre-feet of storage volume) for the remainder of this report. These two basins represent two different options.

Basin 1 has a depth of 23’. The conceptual basin is designed with 4:1 side slopes. The bottom of the basin is at an elevation of 1799’ and the top of the basin is at an elevation of 1822’ (matching existing topography). The water in the basin is stored up to elevation 1820’. UCRA anticipates drawing water out of the basin to elevation 1801’, leaving 2 foot of water in the basin to promote vegetation and aquatic life. An emergency overflow has been designed to allow any water over 1820’ to flow out to South Concho River. The basin's hydraulic functionality is modeled in XP-SWMM by utilizing a storage node and assigning a depth and surface area relationship. The depth, surface area and cumulative storage for the preliminary basin, known as the storage-elevation data, are given in Table 10.

Table 10: Storage-elevation data- for Basin 1 (bottom elevation 1799’)

Stage (ft)	Elevation (ft) (from NAVD)	Surface Area (ac)	Incremental volume calculated using conic method (ac-ft)	Cumulative volume /storage (ac-ft)
23	1822	150.7	150.1	3160
22	1821	149.5	149.0	3010
21	**1820**	**148.5**	**147.9**	**2861**
20	1819	147.3	146.7	2713
19	1818	146.1	145.5	2566
18	1817	144.9	144.3	2420
17	1816	143.8	143.2	2276
16	1815	142.6	142.0	2133
15	1814	141.4	140.8	1991
14	1813	140.2	139.7	1850
13	1812	139.1	138.5	1711
12	1811	137.9	137.3	1572
11	1810	136.7	136.2	1435
10	1809	135.6	135.0	1299
9	1808	134.4	133.9	1164
8	1807	133.3	132.7	1030
7	1806	132.1	131.6	897
6	1805	131.0	130.4	765
5	1804	129.8	129.3	635
4	1803	128.7	128.1	506
3	1802	127.6	127.0	378
2	1801	126.4	125.9	251
1	1800	125.3	124.7	125
0	1799	124.2	0.0	0

Basin 2 has a depth of 25'. The conceptual basin is designed with 4:1 side slopes. The bottom of the basin is at an elevation of 1797' and the top of the basin is at an elevation of 1822' (matching existing topography). The water in the basin is stored up to elevation 1820'. UCRA anticipates drawing water out of the basin to elevation 1799', leaving 2 foot of water in the basin to promote vegetation and aquatic life. An emergency overflow has been designed to allow any water over 1820' to flow out to Red Arroyo near Avenue L. The basin's hydraulic functionality is modeled in XP-SWMM by utilizing a storage node and assigning a depth and surface area relationship. The depth, surface area and cumulative storage for the preliminary basin, known as the storage-elevation data, are given in Table 11.

Table 11: Storage-elevation data- for Basin 2 (bottom elevation 1797')

Stage (ft)	Elevation (ft) (from NAVD)	Surface Area (ac)	Incremental volume calculated using conic method (ac-ft)	Cumulative volume /storage (ac-ft)
25	1822	83.31	83.19	2,005.31
24	1821	83.07	82.94	1,922.12
23	**1820**	**82.82**	**82.69**	**1,839.18**
22	1819	82.57	82.44	1,756.49
21	1818	82.32	82.20	1,674.04
20	1817	82.07	81.95	1,591.85
19	1816	81.82	81.70	1,509.90
18	1815	81.58	81.45	1,428.20
17	1814	81.33	81.20	1,346.74
16	1813	81.08	80.96	1,265.54
15	1812	80.83	80.71	1,184.58
14	1811	80.58	80.46	1,103.87
13	1810	80.34	80.21	1,023.41
12	1809	80.09	79.96	943.20
11	1808	79.84	79.72	863.24
10	1807	79.59	79.47	783.52
9	1806	79.34	79.22	704.05
8	1805	79.10	78.97	624.83
7	1804	78.85	78.72	545.86
6	1803	78.60	78.48	467.14
5	1802	78.35	78.23	388.66
4	1801	78.10	77.98	310.43
3	1800	77.86	77.73	232.45
2	1799	77.61	77.48	154.72
1	1798	77.36	77.24	77.24
0	1797	77.11	0.00	0.00

<u>Proposed condition- January 2012 event (1.67" STP)</u>

By iterative process, an inflow structure has been designed that will be able to divert approximately 91% of the peak flow and approximately 99% of the total volume into the conceptualized detention basins. Geometric elements of the inflow structure are as follows:

- Height of the weir at the downstream end of the concrete structure from channel flow line: 4' at elevation of 1818'
- Weir opening on top of the concrete structure: 50'
- Inflow pipes immediately upstream of the weir: Twin 48" RCP
- Height of inflow pipes above channel bottom: 6-inches above elevation 1814'

<u>Proposed condition- August 2011 event (3.74" STP)</u>

Utilizing the inflow structure designed for the January 2012 event, the basin and channel hydraulics were evaluated for the August 2011 event. The inflow structure is able to divert about 48% of the peak flow and about 61% of the total storm volume into the proposed basin. Since with just one inflow structure a significant volume of the total inflow was not being captured in the basin, another inflow structure was placed downstream to ensure that the proposed basin could capture more of the runoff volumes. Installation of two control structures will ensure that about 80% of the peak flow and about 76% to 87% of the total runoff volumes are diverted into the basin. Additional inflow structures could be installed along the channel if higher capture efficiency is desired. Geometric elements of both the inflow structures are identical and identified as follows:

- Height of the weir from channel flow line: 4' at elevation of 1818'
- Weir opening on top of the concrete structure: 50'
- Inflow pipes immediately upstream of the concrete structure: Twin 48" RCP
- Height of inflow pipes above channel bottom: 6-inches above elevation 1814'

It is recommended that UCRA considers construction of two control structures at a minimum to maximize the volume that can potentially be available in the basin.

The water surface elevation in the channel and the storm water storage basin along with pertinent volumes with the inflow structure are provided in Table 12 through Table 14 for Basin 1 and in Table 14 through Table 17 for Basin 2. The stage time curves representing the varying water surface elevations in the basins are shown in Exhibit 7 for the two basins.

Table 12: Proposed condition hydraulic results for the Jan 2012 event with <u>ONE</u> inflow structure in Basin 1

	Peak inflow (cfs)	Routed peak flow just U/S of inflow structure (cfs)	Peak inflow through pipe(cfs)	% of peak flow entering basin (using the routed flow U/S of inflow pipe)	% of total flow volume entering basin (using the routed flow U/S of inflow pipe)	Max stage in pond (ft)	Storage volume corresponding to max stage (ac-ft)	Freeboard (ft) from top of pond (1820 ft)	Overflow over the weir (cfs)	% of flow over weir (using the routed flow U/S of inflow structure)	WSE just D/S of FM 1223 (ft) (Top of road:1834')	WSE just U/S of FM 1223 (ft) (Top of road:1834')	Overflow from the emergency spillway (cfs)
Channel hydraulics with 1.67" STP	685	569.44	517.3	90.8%	98.9%	1806.61	974.16	13.39	51.65	9.1%	1818.7	1819.8	0

Table 13: Proposed condition hydraulic results for the August 2011 event with <u>ONE</u> inflow structure in Basin 1

	Peak inflow (cfs)	Routed peak flow just U/S of inflow pipe (cfs)	Peak inflow through pipe(cfs)	% of peak flow entering basin (using the routed flow U/S of inflow pipe)	% of total flow volume entering basin (using the routed flow U/S of inflow pipe)	Max stage in pond (ft)	Storage volume corresponding to max stage (ac-ft)	Freeboard (ft) from top of pond (1820 ft)	Overflow over the weir (cfs)	% of flow over weir (using the routed flow U/S of inflow pipe)	WSE just D/S of FM 1223 (ft) (Top of road:1834')	WSE just U/S of FM 1223 (ft) (Top of road:1834')	Overflow from the emergency spillway (cfs)
Channel hydraulics with adjusted 3.74" STP	2789	2268.85	1081.64	47.7%	60.6%	1809.65	1381.13	10.35	1184.93	52.2%	1824.11	1824.39	

Table 14: Proposed condition hydraulic results for the August 2011 event with <u>TWO</u> inflow structures in Basin 1

	Peak inflow (cfs)	Routed peak flow just U/S of inflow pipe (cfs)	Peak inflow through pipe(cfs)	% of peak flow entering basin (using the routed flow U/S of inflow pipe)	% of total flow volume entering basin (using the routed flow U/S of inflow pipe)	Max stage in pond (ft)	Storage volume corresponding to max stage (ac-ft)	Freeboard (ft) from top of pond (1820 ft)	Overflow over the weir (cfs)	% of flow over weir (using the routed flow U/S of inflow pipe)	WSE just D/S of FM 1223 (ft) (Top of road:1834')	WSE just U/S of FM 1223 (ft) (Top of road:1834')	Overflow from the emergency spillway (cfs)
Channel hydraulics with 3.74" STP	2789	2210	1772	80%	87%	1814.01	2013	5.96	433.6	20%	1824.51	1824.72	0

Table 15: Proposed condition hydraulic results for the Jan 2012 event with ONE inflow structure in Basin 2

	Peak inflow (cfs)	Routed peak flow just U/S of inflow structure (cfs)	Peak inflow through pipe(cfs)	% of peak flow entering basin (using the routed flow U/S of inflow pipe)	% of total flow volume entering basin (using the routed flow U/S of inflow pipe)	Max stage in pond (ft)	Storage volume corresponding to max stage (ac-ft)	Freeboard (ft) from top of pond (1820 ft)	Overflow over the weir (cfs)	% of flow over weir (using the routed flow U/S of inflow structure)	WSE just D/S of FM 1223 (ft) (Top of road:1834')	WSE just U/S of FM 1223 (ft) (Top of road:1834')	Overflow from the emergency spillway (cfs)
Channel hydraulics with 1.67" STP	685	569.44	517.3	90.8%	98.9%	1809.4	981	10.6	51.65	9.2%	1818.7	1819.8	0

Table 16: Proposed condition hydraulic results for the August 2011 event with ONE inflow structure in Basin 2

	Peak inflow (cfs)	Routed peak flow just U/S of inflow pipe (cfs)	Peak inflow through pipe(cfs)	% of peak flow entering basin (using the routed flow U/S of inflow pipe)	% of total flow volume entering basin (using the routed flow U/S of inflow pipe)	Max stage in pond (ft)	Storage volume corresponding to max stage (ac-ft)	Freeboard (ft) from top of pond (1820 ft)	Overflow over the weir (cfs)	% of flow over weir (using the routed flow U/S of inflow pipe)	WSE just D/S of FM 1223 (ft) (Top of road:1834')	WSE just U/S of FM 1223 (ft) (Top of road:1834')	Overflow from the emergency spillway (cfs)
Channel hydraulics with adjusted 3.74" STP	2789	2268.85	1081.64	47.8%	60%	1814.1	1358	5.9	1184.93	52.2%	1824.12	1824.39	0

Table 17: Proposed condition hydraulic results for the August 2011 event with TWO inflow structure in Basin 2

	Peak inflow (cfs)	Routed peak flow just U/S of inflow pipe (cfs)	Peak inflow through pipe(cfs)	% of peak flow entering basin (using the routed flow U/S of inflow pipe)	% of total flow volume entering basin (using the routed flow U/S of inflow pipe)	Max stage in pond (ft)	Storage volume corresponding to max stage (ac-ft)	Freeboard (ft) from top of pond (1820 ft)	Overflow over the weir (cfs)	% of flow over weir (using the routed flow U/S of inflow pipe)	WSE just D/S of FM 1223 (ft) (Top of road:1834')	WSE just U/S of FM 1223 (ft) (Top of road:1834')	Overflow from the emergency spillway (cfs)
Channel hydraulics with 3.74" STP	2789	2210.39	1772.03	80.2%	75.8%	1818.2	1686	1.8	433.61	19.8%	1824.13	1824.39	0

Water Supply Feasibility

The water captured by the inflow structure will be stored in the storage basin for water supply or for downstream release. It is anticipated that in a year with average rainfall, 11,500 acre-feet of water will be captured in the basin. The water stored in the basin will be conveyed to the Lone Wolf Reservoir water treatment plant located about half a mile north west of the proposed storage basin. The stored water will start to be released approximately 2 days after a storm event. We have evaluated multiple scenarios to recommend a feasible option. The following scenarios have been evaluated:

- Option 1: Gravity flow the water- An outflow pipe from the basin will be manually operated by a gate which will release the water upstream of the culvert under Ave K. The water will then be carried down the Red Arroyo. Before the outfall of Red Arroyo, a control structure will need to be constructed to allow for the water to be pumped to the treatment plant.

- Option 2: Combined gravity and pump flow- An outflow pipe from the basin will gravity flow till permissible by the elevation in the basin, and then the rest will be pumped from a clear-well pump station.

- Option 3: Pump flow- The stored water is pumped directly from the basin to a receiving point within the water treatment plant.

Table 18 discusses the merits and limitations of each scenario.

Table 18: Water supply options

Scenario	Merits	Limitations
Gravity flow (Option 1, See Exhibit 8A)	• Construction cost will be nominal	• For smaller storm events, the water surface in the basin may not be high enough to allow gravity flow. The storm total precipitation for the Jan 2012 event is similar to the 95th percentile storm event (1.67”), and the pond fills up about 7.6’. Based on the elevation, potentially only 6-inches of water can gravity flow out of the basin. • For larger storm events such as the Aug 2011 event, a significant volume of water will remain that cannot be withdrawn by gravity. • Water quality can be an issue as water flows through an open channel to get to the

Scenario	Merits	Limitations
		treatment plant
Combined gravity flow and pump (Option 2, See Exhibit 8B)	• Smaller pump station than the pump station in Option 3.	• Inefficient since the gravity system will not operate continually. • Greater chances of system failure • Construction cost for two separate systems to carry the water from the basin. • Water quality can potentially be an issue as water flows through an open channel to get to the treatment plant
Pump flow (Option 3, See Exhibit 8C)	• Reliable flow rates under varying pool levels in the storage basin can be designed to withdraw the water from the pond to the treatment plant. • Water quality issues can be minimized.	• Of all the three options, construction cost will likely be the highest for this option.

Based on the evaluation presented above, we recommend a pump system to directly draw the stored water from the basin to the water treatment plant. The specifics of the pump system are outlined below:

- Two 150-HP pumps to provide approximately 30 - 35' of total dynamic head (to pump water from an elevation of 1801' to 1822' including head loses due to bends, friction, appurtenances, etc.)
- 2000' linear foot of 36" ductile iron pipe to carry the water from the basin to the receiving units of the treatment plant

The preliminary system to transfer the stored water assumes that water is withdrawn from the basin over a two week period with a flow rate of 20 million gallons per day (MGD).

Environmental Factors and Utility Coordination

Based on preliminary research, UCRA may need to identify any potential wetland locations along Red Arroyo. Utilizing the Environmental Protection Agency's (EPA) website (nepaassittool.epa.gov) some wetland locations along Red Arroyo have been identified as shown in Exhibit 9. Even though in the inflow structure proposed above will allow some water to bypass the weir and hence will maintain some flows, it will be necessary to determine whether there is any requirements for "environmental flows" for Red Arroyo.

A 33 inch water line is in place across the proposed basin location. Additionally a 30 inch water line is proposed to be constructed in the near future. The alignments of the existing and proposed water lines with respect to the two basin scenarios are shown in Exhibit 10 and Exhibit 11. Coordination will have to be carried out to ensure that constructions of the basin and the water line do not create conflicts with each other.

Preliminary Opinion of Probable Cost

The preliminary cost estimate to construct Basin 1 to provide a storage volume of 2860 acre-feet and the infrastructure for transferring the captured water to the treatment plant is approximately **$70.6 million**. The cost estimate assumes that the high powered and low powered electrical lines running across the site will be relocated as part of the construction. The single largest contributing factor to the cost is the excavation cost at 60% of the total estimated cost. The excavation cost is high to accommodate haulage of the spoil from the site. Items included in the cost estimate are listed in Table 19. The basin configuration is shown in Exhibit 10.

Table 19: Probable cost estimate for a 2860 ac-ft storage basin (Basin 1)

Item No.	Quantity	Unit	Item Description	Unit Price	Amount
1	2	EA	Pump, 150 HP Vertical Turbine	$150,000	$300,000
2	1	LS	Discharge piping, header, valves and miscellaneous equipment	$200,000	$200,000
3	1	LS	Pump station structure	$75,000	$75,000
4	1	LS	Electrical service to pump station	$150,000	$150,000
5	1	LS	Instrumentation and Control	$40,000	$40,000
6	2,000	LF	Pipe, 36-inch ductile iron	$200	$400,000
7	200	LF	Inflow pipes, 48-inch CMP	$160	$32,000
8	1	LS	Weir and Emergency Spillway	$100,000	$100,000
9	4,615,745	CY	Excavation and haulage	$10	$46,157,450
10	1	LS	33-Inch Water Line Relocation	$1,000,000	$1,000,000
11	1	LS	High power voltage line relocation	$3,000,000	$3,000,000
12	1	LS	Low power voltage line relocation	$1,000,000	$1,000,000
13	2	EA	Inflow structure (dam and weir)	$50,000	$100,000
14	405,000	CY	Detention pond clay liner	$8	$3,240,000
15	1,000	SY	Rip-rap for inflow structure	$10	$10,000
16	4	EA	Headwall-wingwall for inflow pipes	$5,000	$20,000
				Subtotal	$55,830,000
				Engineering and Survey (10%)	$5,583,000
				Subtotal	$61,413,000
				Contingency (15%)	$9,211,950
				Total	**$70,624,950**

The preliminary cost estimate to construct Basin 2 to provide a storage volume of 1839 acre-feet and the infrastructure for transferring the captured water to the treatment plant is approximately **$20.4 million**. Significant reduction in the excavation cost is achieved by disposing the spoil on the land east of the proposed Basin 2 configuration as shown in Exhibit 11. Items included in the cost estimate are listed in Table 20.

Table 20: Probable cost estimate for a 1820 ac-ft storage basin (Basin 2)

Item No.	Quantity	Unit	Item Description	Unit Price	Amount
1	2	EA	Pump, 150 HP Vertical Turbine	$150,000	$300,000
2	1	LS	Discharge piping, header, valves and miscellaneous equipment	$200,000	$200,000
3	1	LS	Pump station structure	$75,000	$75,000
4	1	LS	Electrical service to pump station	$150,000	$150,000
5	1	LS	Instrumentation and Control	$40,000	$40,000
6	2,000	LF	Pipe, 36-inch ductile iron	$200	$400,000
7	600	LF	Pipe, 48-inch CMP	$160	$96,000
8	1	LS	Weir and Emergency Spillway	$100,000	$100,000
9	3,484,800	CY	Excavation and haulage	$3	$10,454,400
10	1	LS	33-Inch Water Line Relocation	$1,000,000	$1,000,000
11	2	EA	Inflow structure (dam and weir)	$50,000	$100,000
12	400,000	CY	Detention pond clay liner	$8	$3,200,000
13	1,000	SY	Rip-rap for inflow structure	$10	$10,000
14	4	EA	Headwall-wingwall for inflow pipes	$5,000	$20,000
				Subtotal	$16,150,000
				Engineering and Survey (10%)	$1,615,000
				Subtotal	$17,765,000
				Contingency (15%)	$2,665,000
				Total	**$20,430,000**

Recommendations for further analyses

Jacobs would like to request UCRA to consider the following recommendations for further analyses

- Change the hydrologic modeling method
    - Adopt the rainfall loss and transformation methods that use a small number of model parameters so that the model can be reasonably calibrated. We recommend using the hydrologic method developed by the Natural Resources Conservation Service (NRCS). Update the reach routing methodology from the current uniform channels to

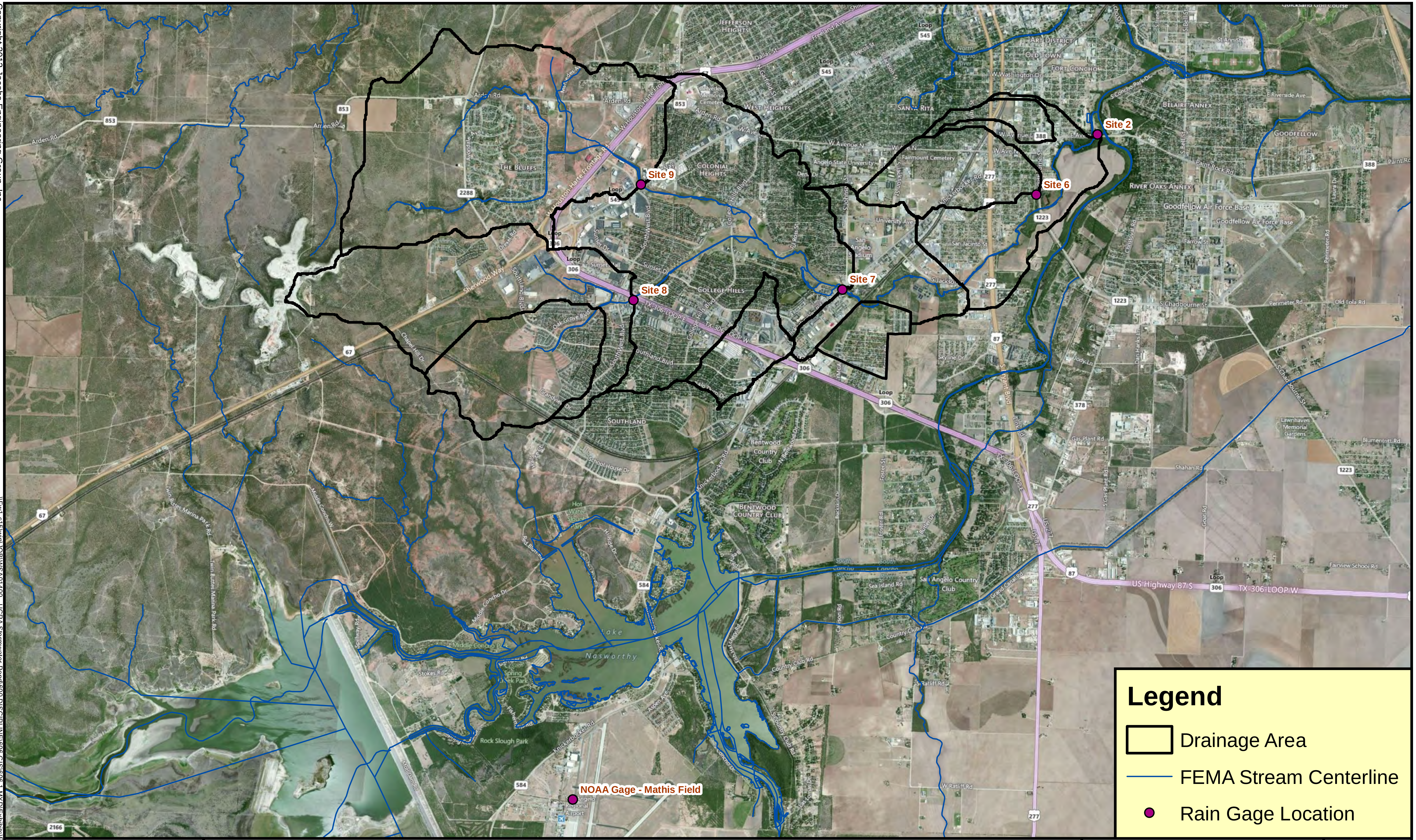
using actual cross-sections. A review of the available topographic data reveals that the channels are not uniform, and updating from the uniform channels will potentially identify the actual storage volume in the channel and its effect in the peak flow attenuation.

- Obtain detailed channel survey along Red Arroyo to enable creating of a more accurate hydraulic model of the Red Arroyo. An accurate model will help in optimizing the inflow structures.
- Develop more comprehensive hydraulic model including the proposed outflow structures and conveyance to the water treatment plant.
- Carry out a reservoir balance to evaluate the transfer rate from the storage basin to the water treatment plant.
- Determine environmental flow requirements.

The results of the analysis described above can be parts of the pre-design engineering study.

EXHIBITS

EXHIBIT 1



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Drainage Area Map of Red Arroyo Watershed

UCRA Stormwater Basin
City of San Angelo, Texas
Jacobs Project No. WSA01400

Source: ESRI® Data (2013)

Scale: 0 3,000 6,000 Feet

Exhibit: 1

\\Fw1-s15\wp\JOBS\WSA01400 - UCRA Stormwater Pond\600 DISCIPLINE\600 GIS\606.1 MXD\Exhibits\E1_DA_Map_11X17.MXD

EXHIBIT 2

JACOBS

Location Map

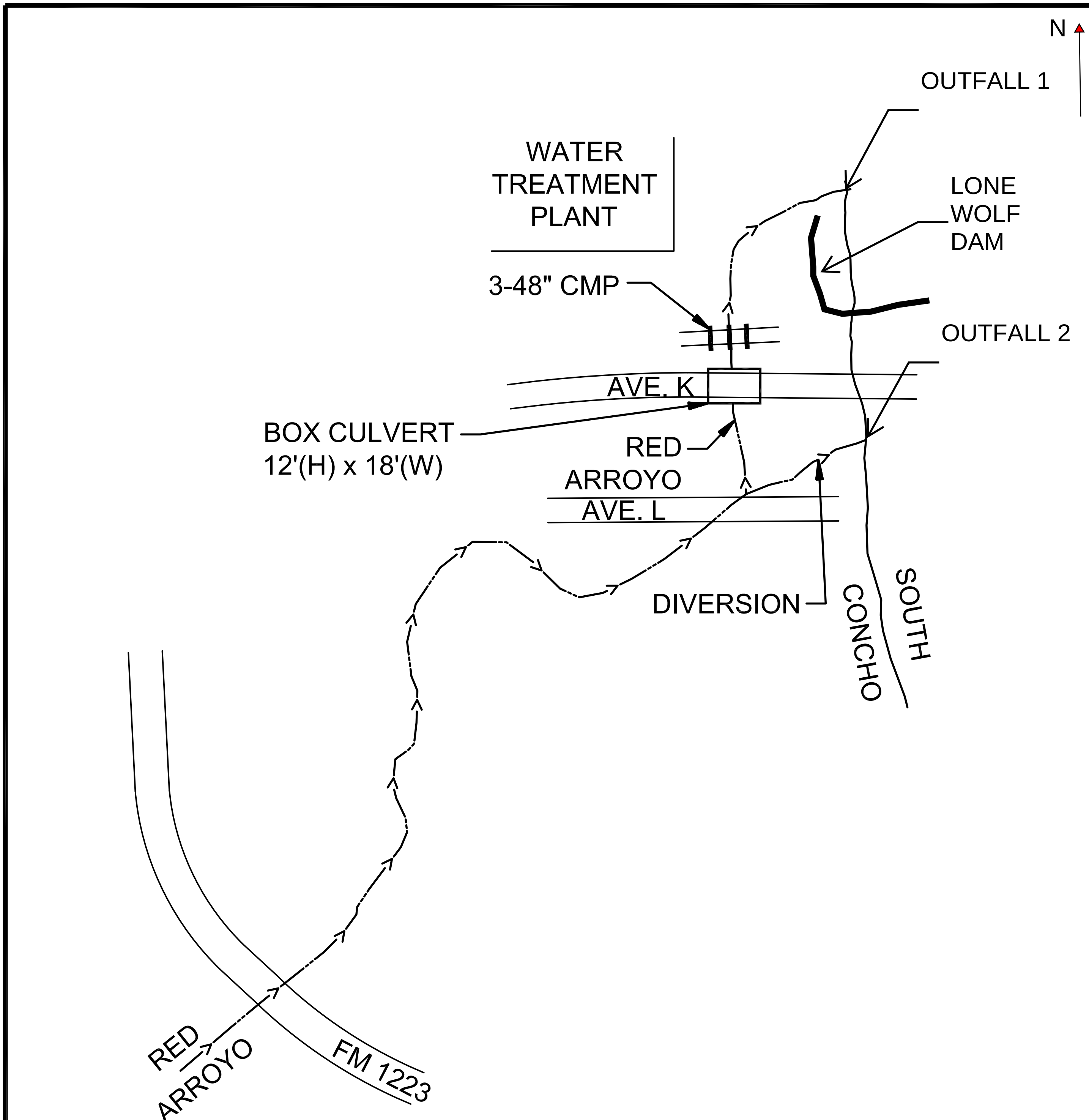
UCRA Stormwater Basin
City of San Angelo, Texas
Jacobs Project No. WSA01400

Source: ESRI® Data (2013)

Scale: 0 1,000 Feet

Exhibit 2

EXHIBIT 3



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5995 ROGERDALE ROAD
HOUSTON, TEXAS 77072
281.351.6000

HYDRAULIC MODEL DOMAIN

EXHIBIT 3

SCALE: NONE	PROJ. NO.

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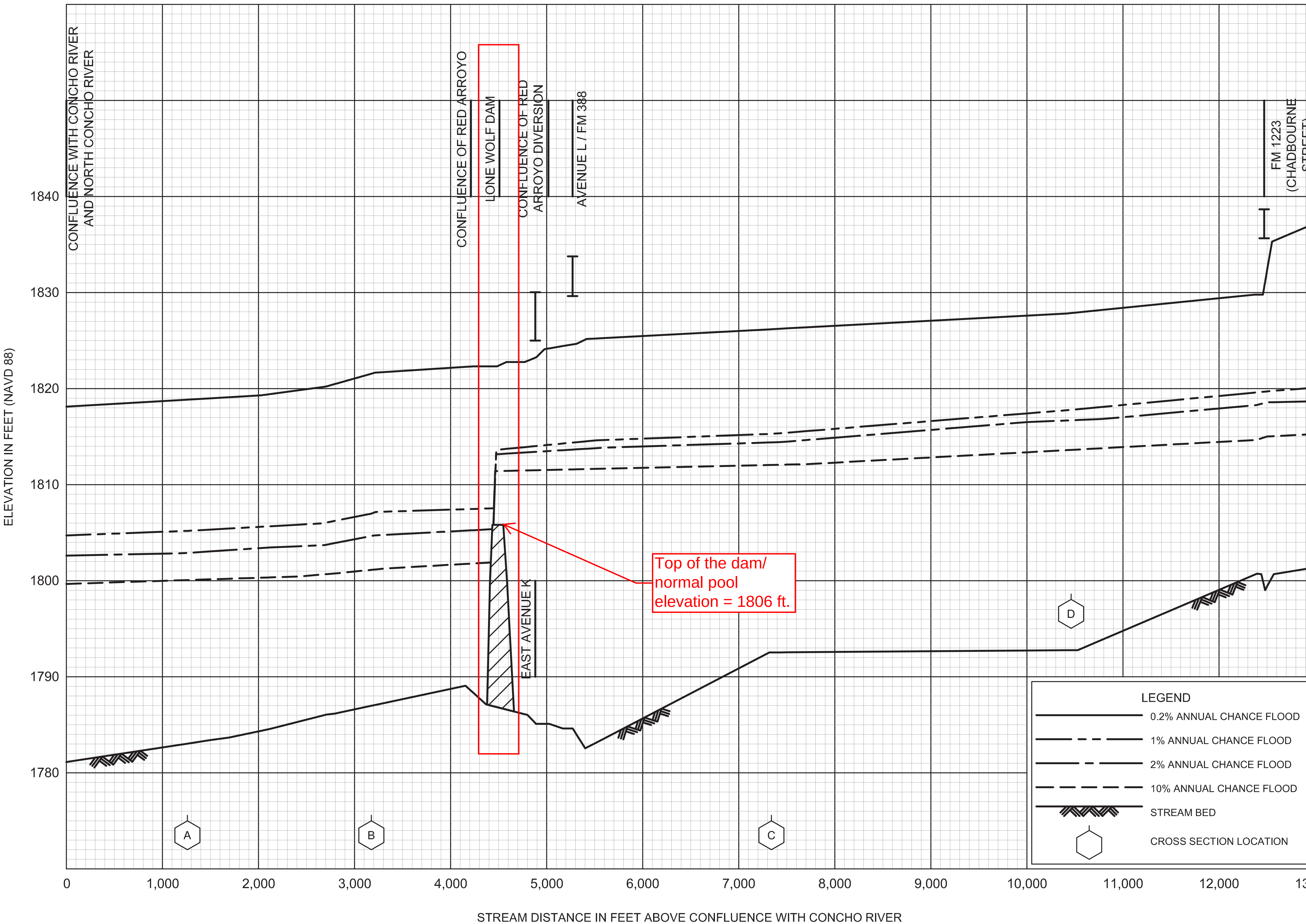
EXHIBIT 4



FLOOD PROFILES

SOUTH CONCHO RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

TOM GREEN COUNTY, TX

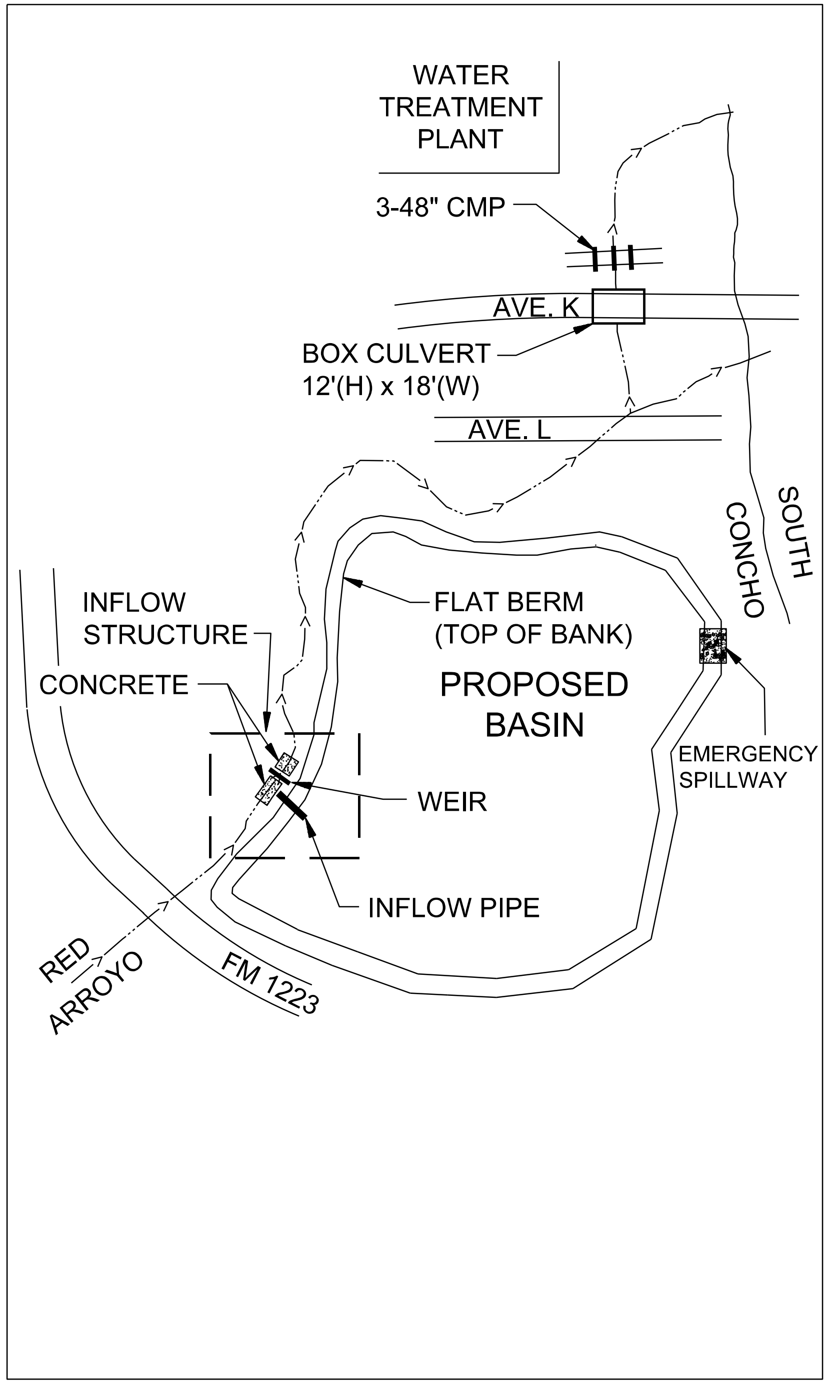
AND INCORPORATED AREAS

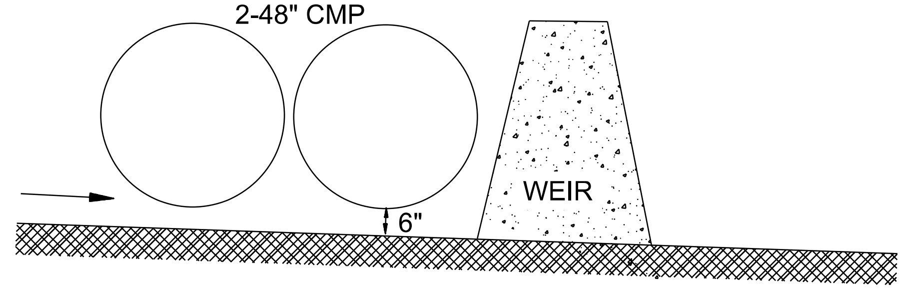
104P

EXHIBIT 5

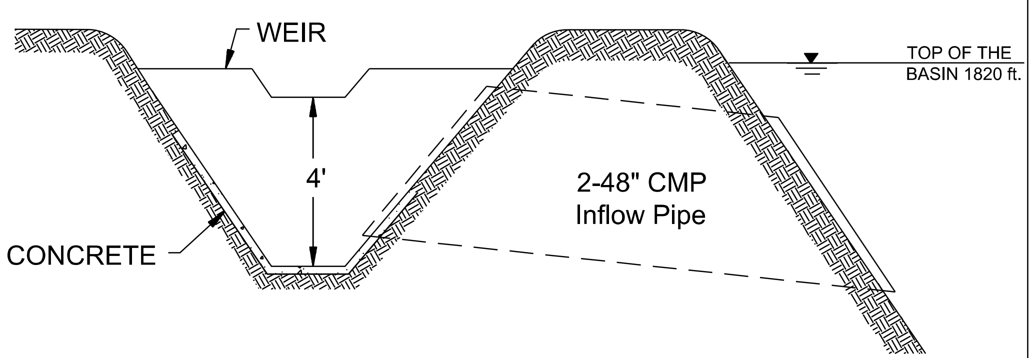

N
Outfall 1
Outfall 2
Stream flow hydrograph entered at this node
W Avenue K
W Avenue L
W Avenue N
W Avenue O
W Avenue P
S Chadbourne St
Paint Rock Rd
South Concho River

EXHIBIT 6

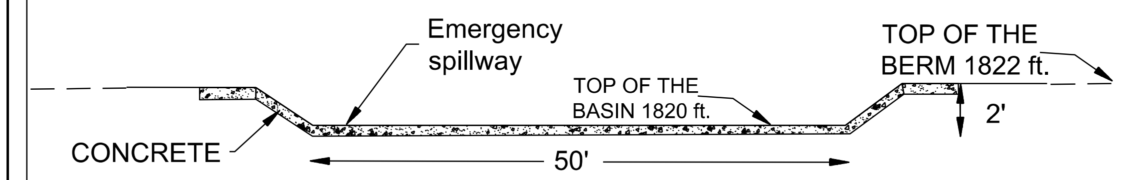




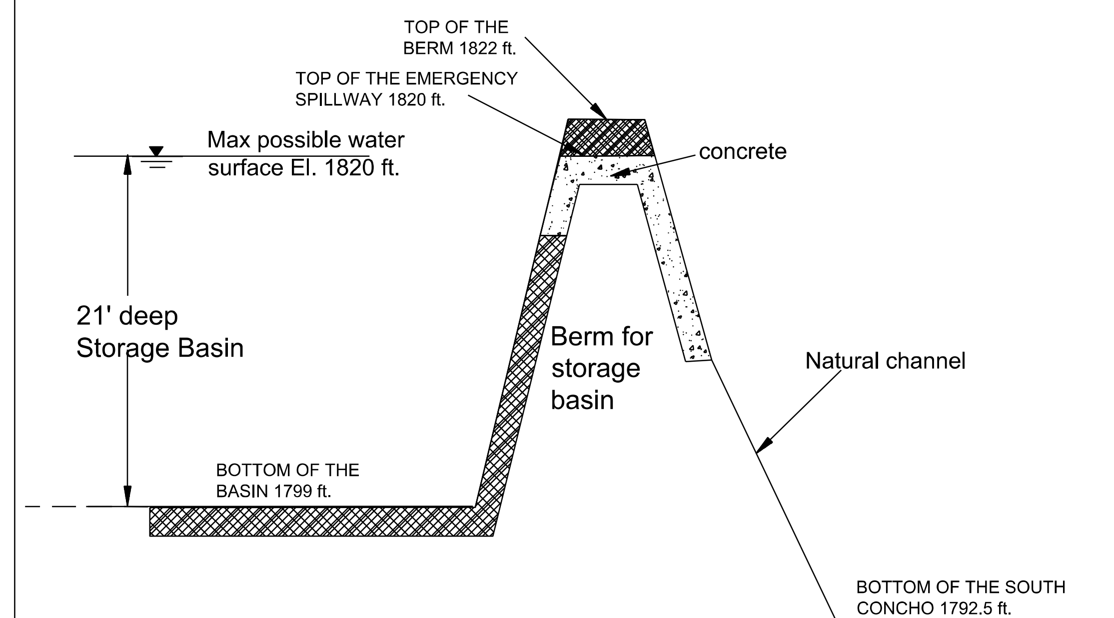
Longitudinal cross-section view of inflow structure



Profile view of inflow structure



Profile view of emergency spillway



Longitudinal cross-section view of emergency spillway from the storage basin to the South Concho river

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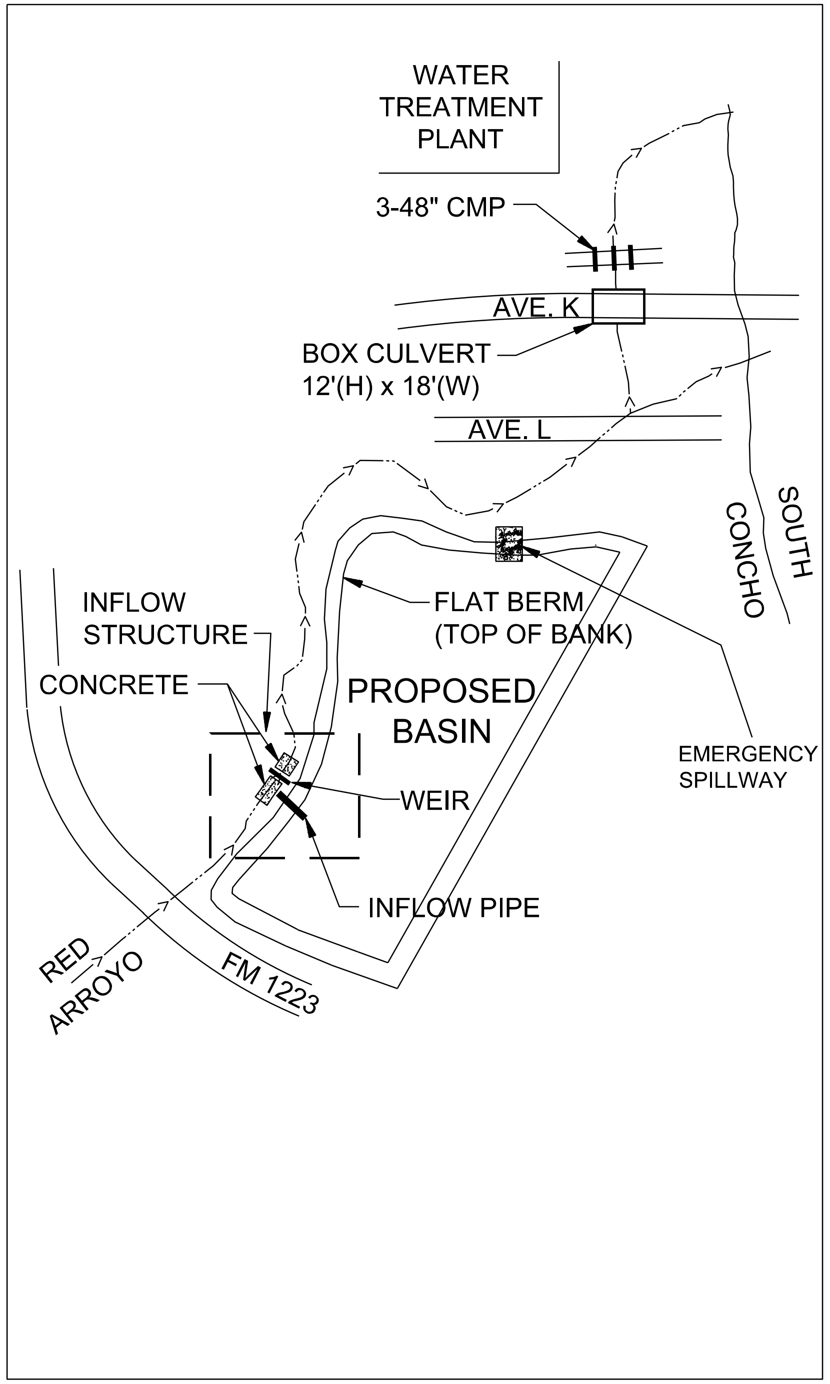
REG NO. 2966

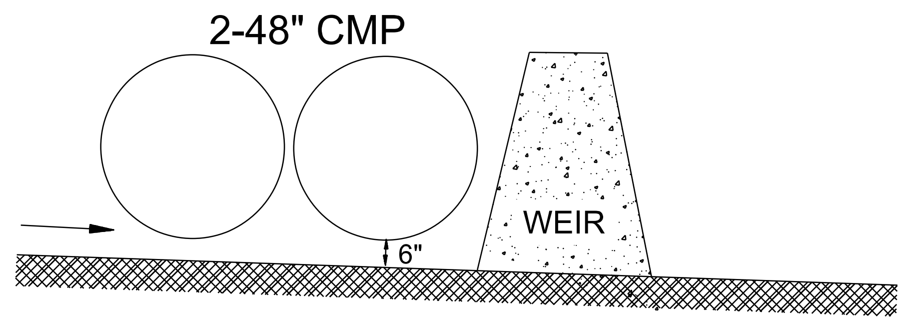
Concept Plan for Inflow Structure and Emergency Spillway

Exhibit 6 A

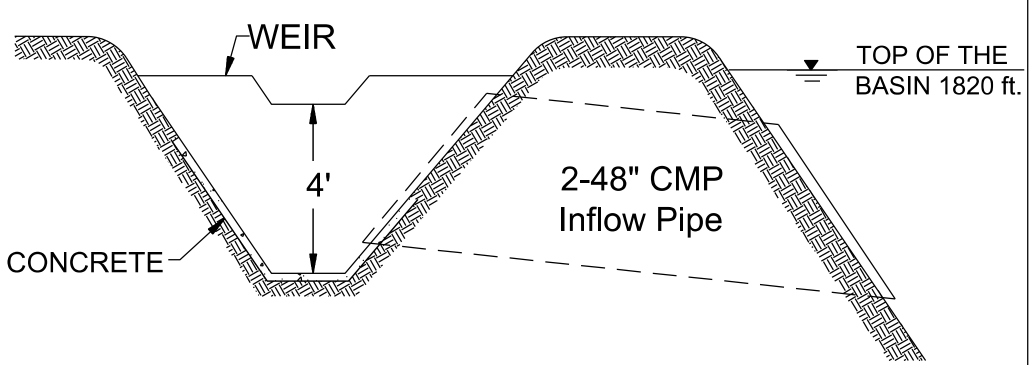
SCALE: NONE | PROJ. NO. : WSA01400

\\Fw1-s15\wip\JOB\WSA01400 - UCRA Stormwater Pond\600 DISCIPLINE\607 H&H\607.4 Hydraulics\concept_layout.dwg

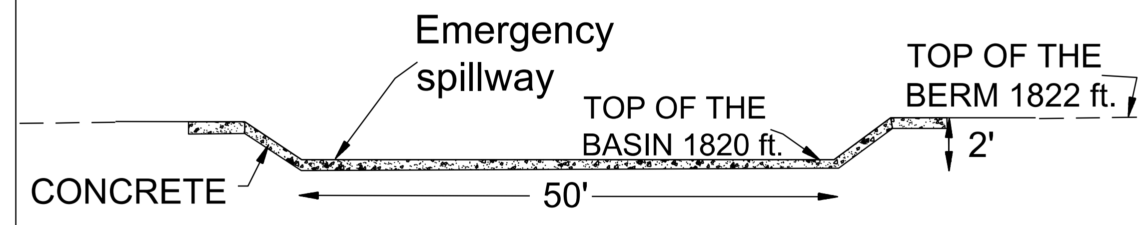




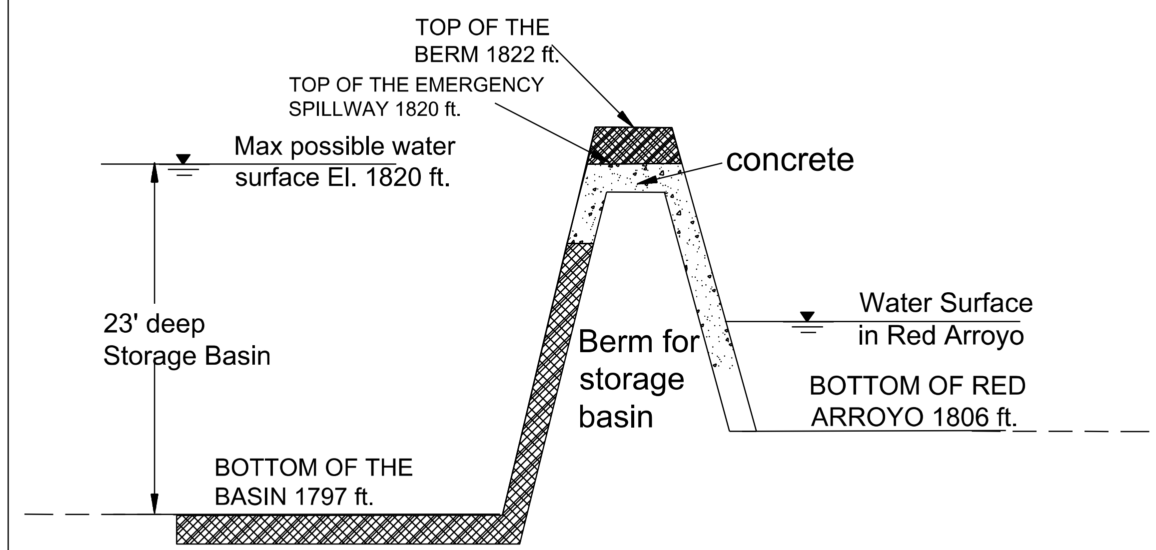
Longitudinal cross-section view of inflow structure



Profile view of inflow structure



Profile view of emergency spillway



Longitudinal cross-section view of emergency spillway from the storage basin to the South Concho river

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REG NO. 2966

Concept Plan for Inflow Structure and Emergency Spillway

Exhibit 6B

SCALE: NONE | PROJ. NO. : WSA01400

\Fw1-s15\wip\JOB\WSA01400 - UCRA Stormwater Pond\600 DISCIPLINE\607 H&H\607.4 Hydraulics\concept_layout.dwg

EXHIBIT 7

Exhibit 7A : Basin 1 - stage time curve for one inflow structure with January 2012 storm - 1.67 in STP

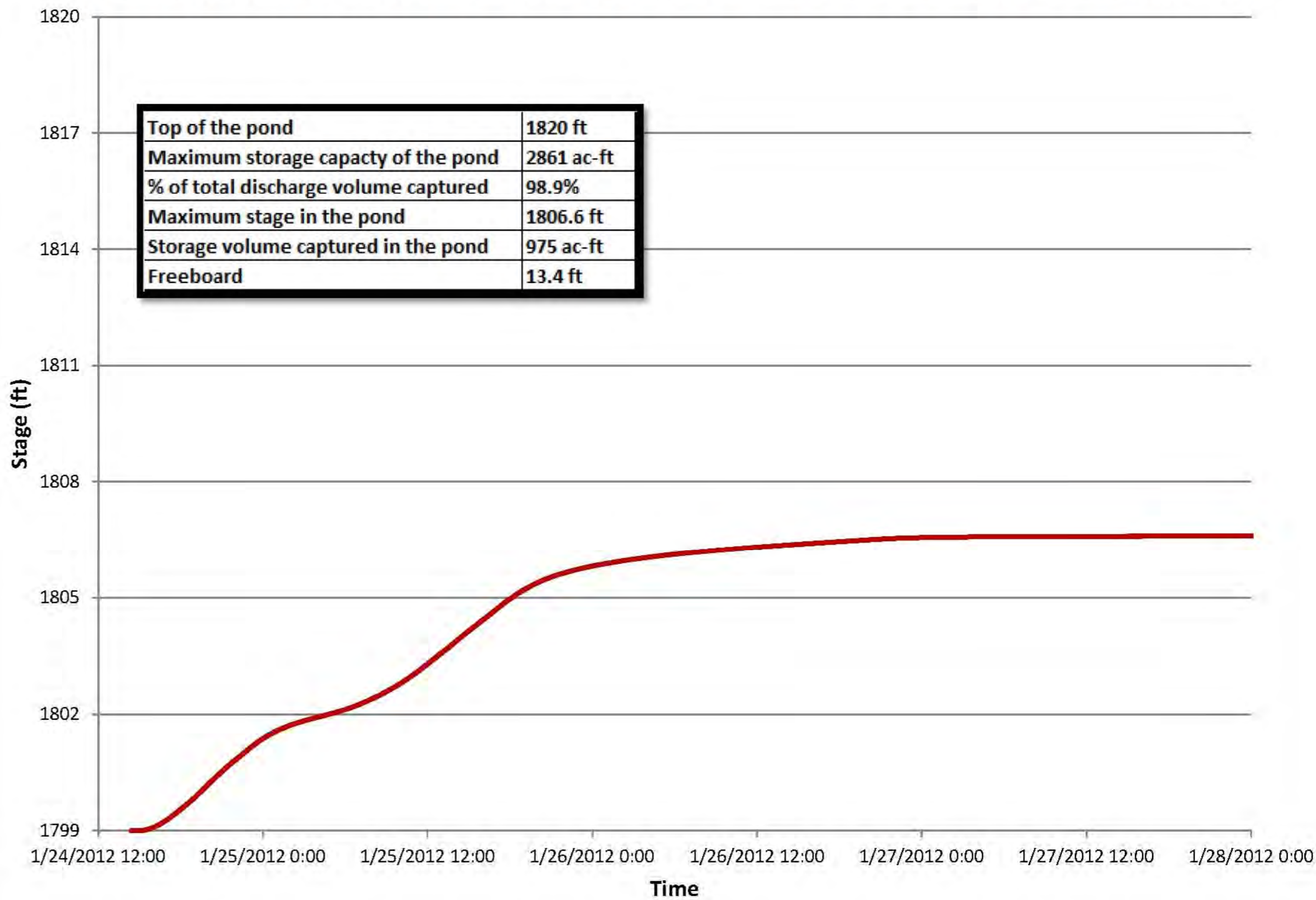
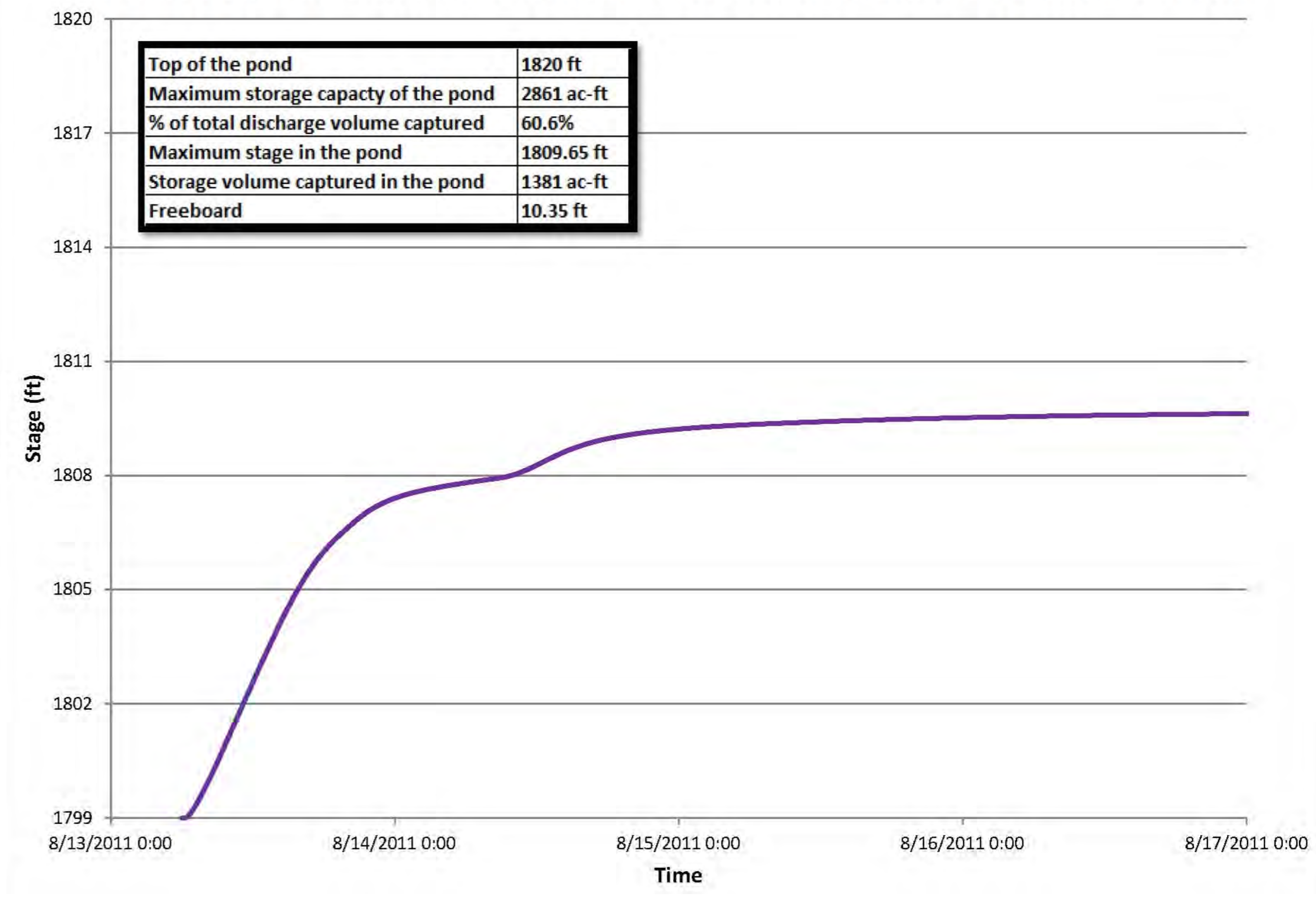


Exhibit 7B : Basin 1 - stage time curve for one inflow structure with August 2011 storm - 3.74 in STP



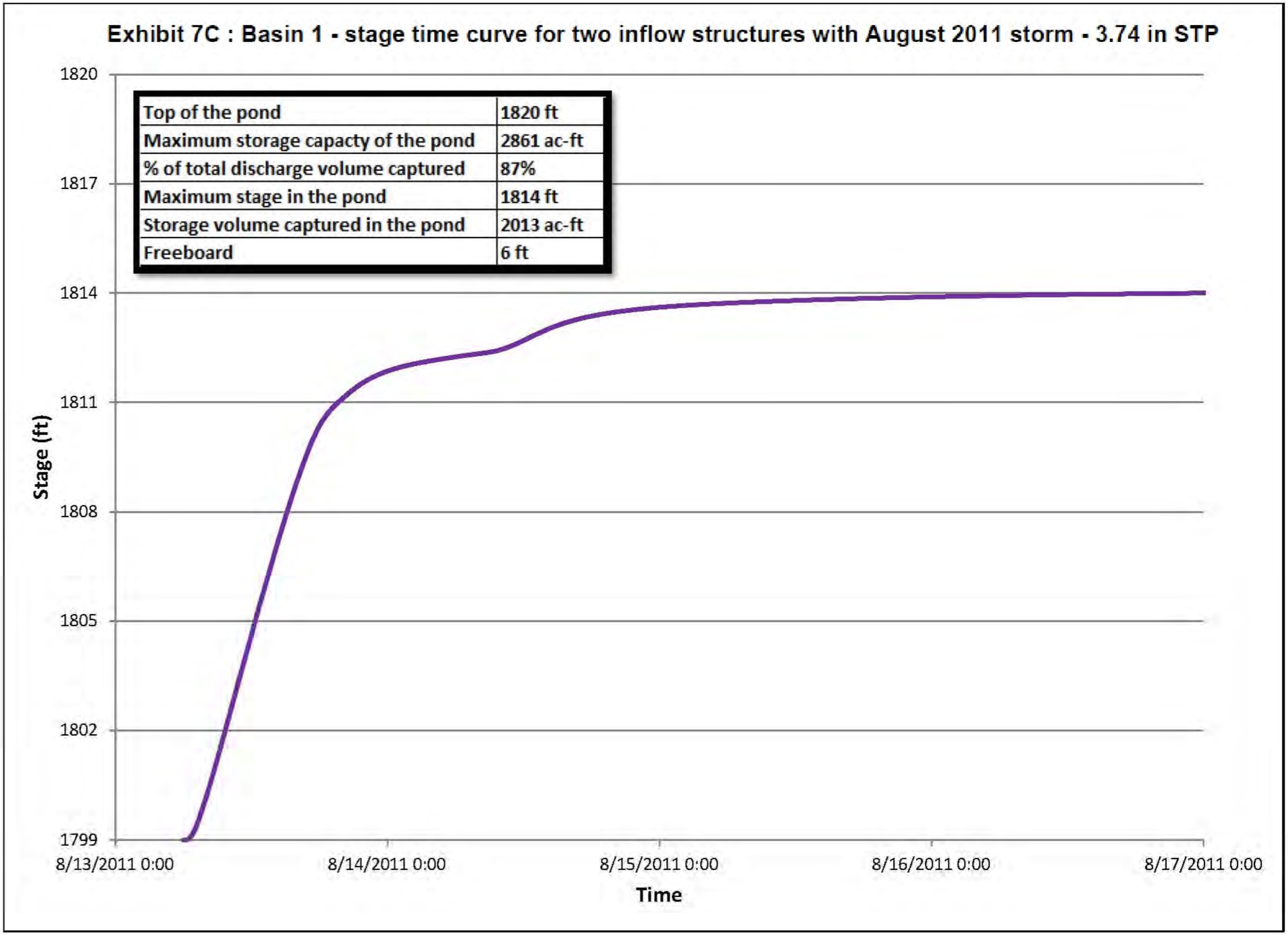
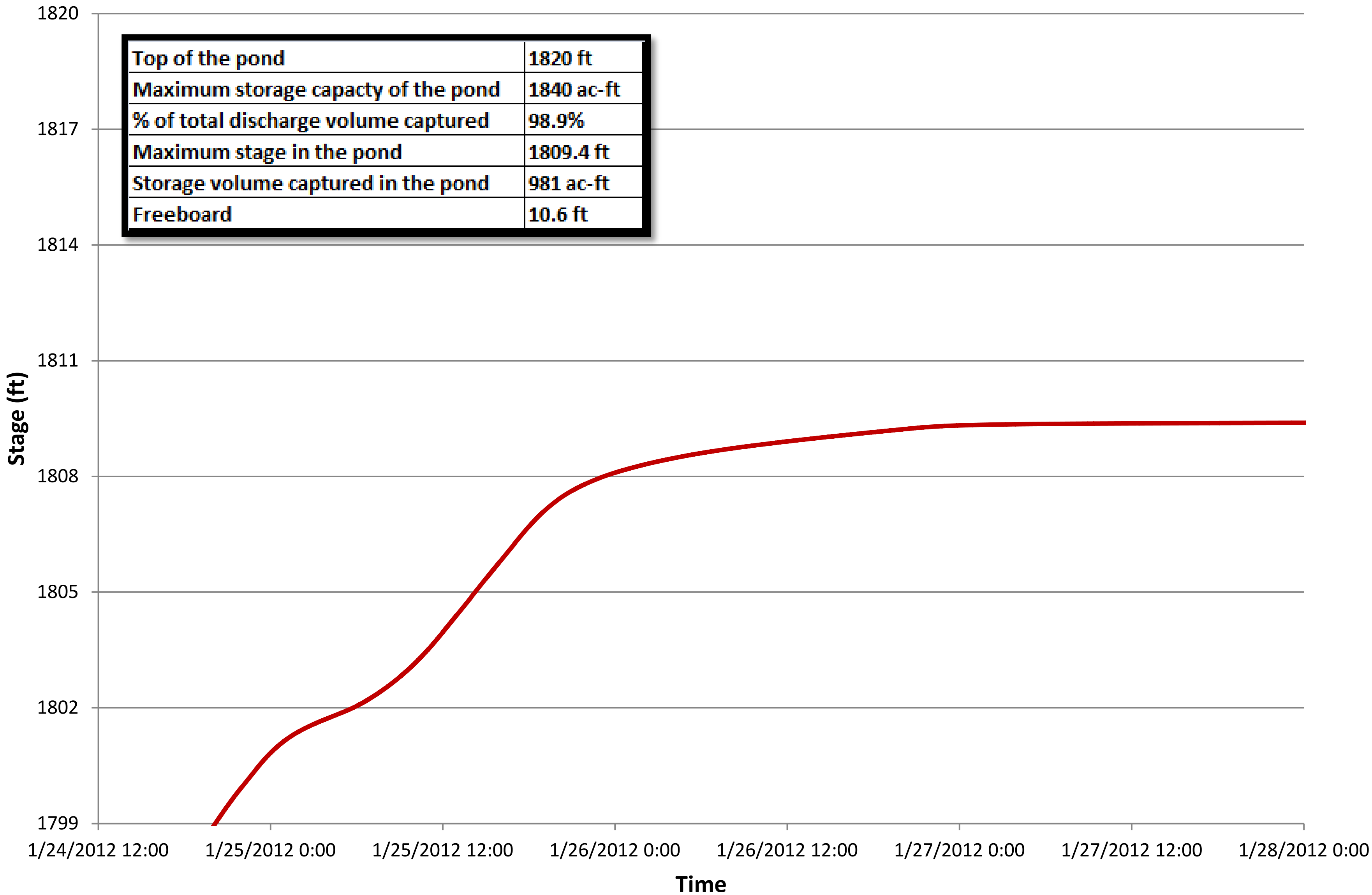


Exhibit 7C : Basin 1 - stage time curve for two inflow structures with August 2011 storm - 3.74 in STP
Top of the pond | 1820 ft
Maximum storage capacty of the pond | 2861 ac-ft
% of total discharge volume captured | 87%
Maximum stage in the pond | 1814 ft
Storage volume captured in the pond | 2013 ac-ft
Freeboard | 6 ft
Stage (ft)
1820
1817
1814
1811
1808
1805
1802
1799
8/13/2011 0:00
8/14/2011 0:00
8/15/2011 0:00
8/16/2011 0:00
8/17/2011 0:00
Time

Exhibit 7D : Basin 2 - stage time curve for one inflow structure with January 2012 storm - 1.67 in STP



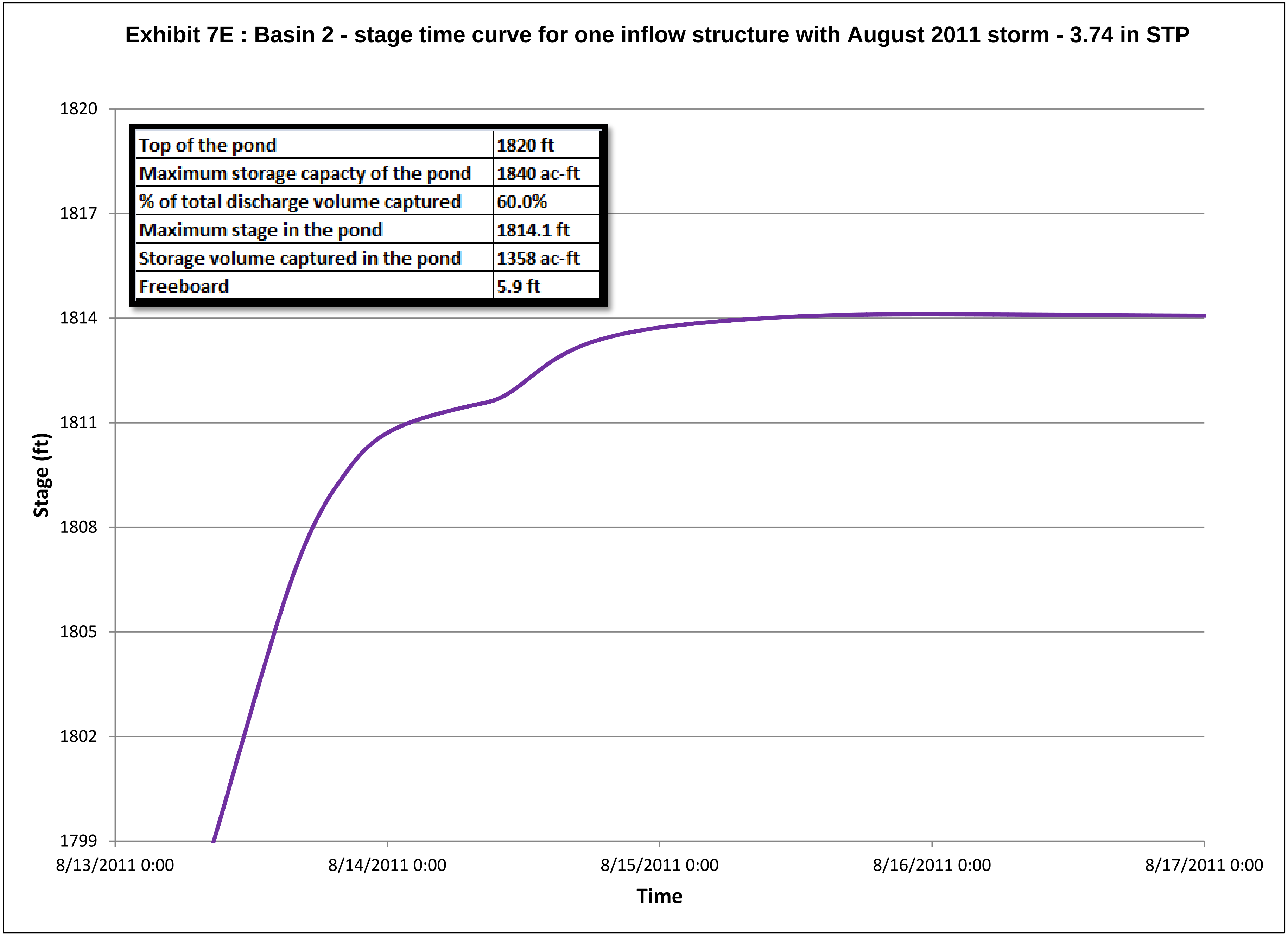
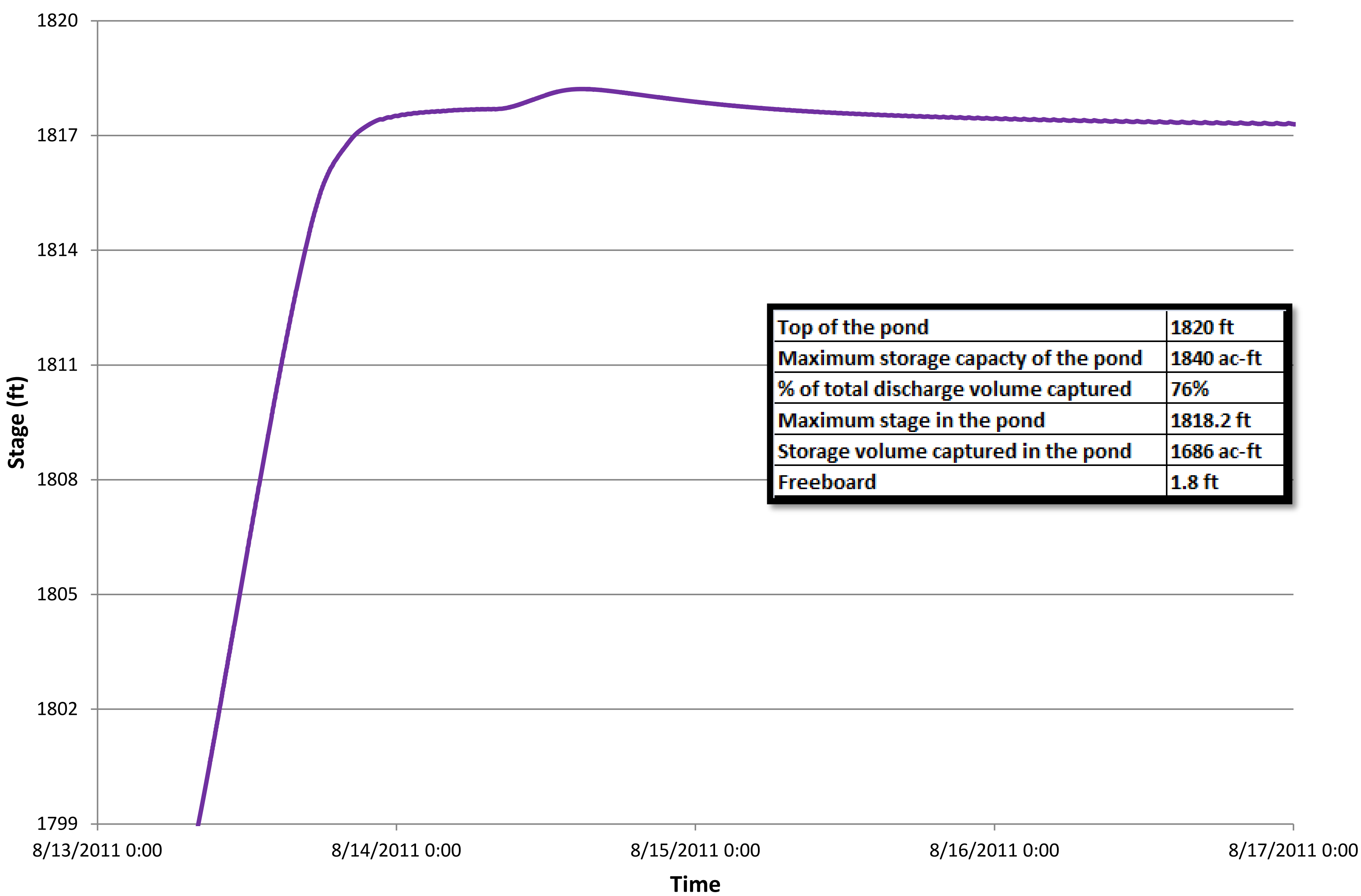


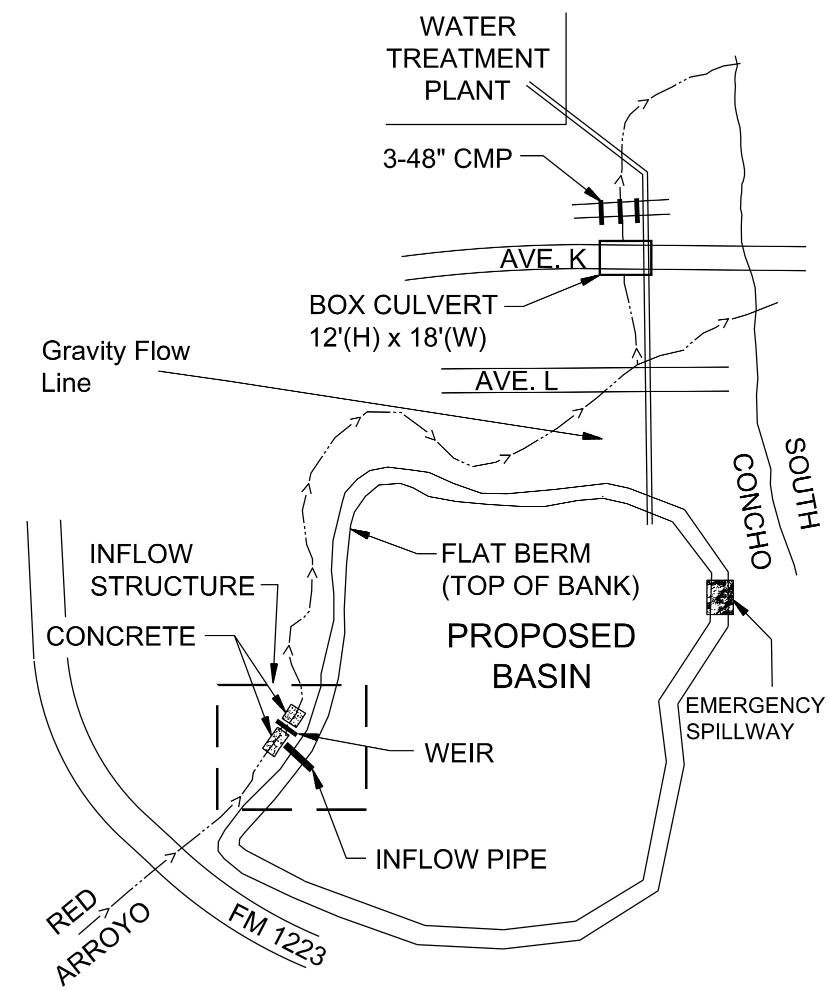
Exhibit 7E : Basin 2 - stage time curve for one inflow structure with August 2011 storm - 3.74 in STP
Top of the pond | 1820 ft
Maximum storage capacty of the pond | 1840 ac-ft
% of total discharge volume captured | 60.0%
Maximum stage in the pond | 1814.1 ft
Storage volume captured in the pond | 1358 ac-ft
Freeboard | 5.9 ft
Stage (ft)
1820
1817
1814
1811
1808
1805
1802
1799
8/13/2011 0:00
8/14/2011 0:00
8/15/2011 0:00
8/16/2011 0:00
8/17/2011 0:00
Time

Exhibit 7F : Basin 2 - stage time curve for two inflow structures with August 2011 storm - 3.74 in STP

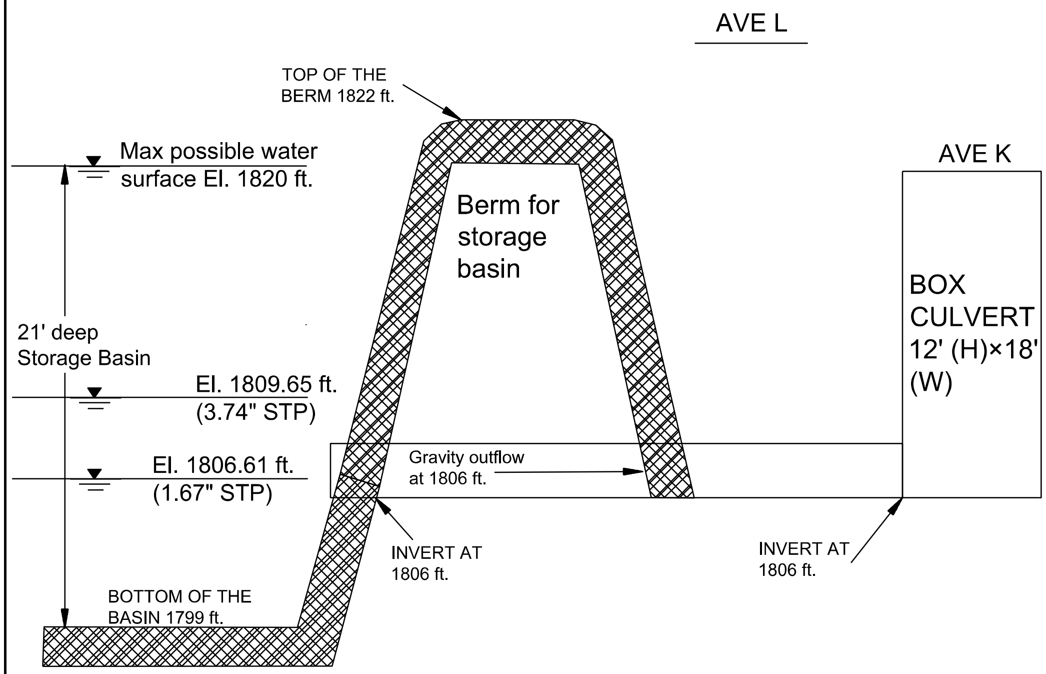


Top of the pond	1820 ft
Maximum storage capacty of the pond	1840 ac-ft
% of total discharge volume captured	76%
Maximum stage in the pond	1818.2 ft
Storage volume captured in the pond	1686 ac-ft
Freeboard	1.8 ft

EXHIBIT 8



PLAN VIEW



PROFILE VIEW

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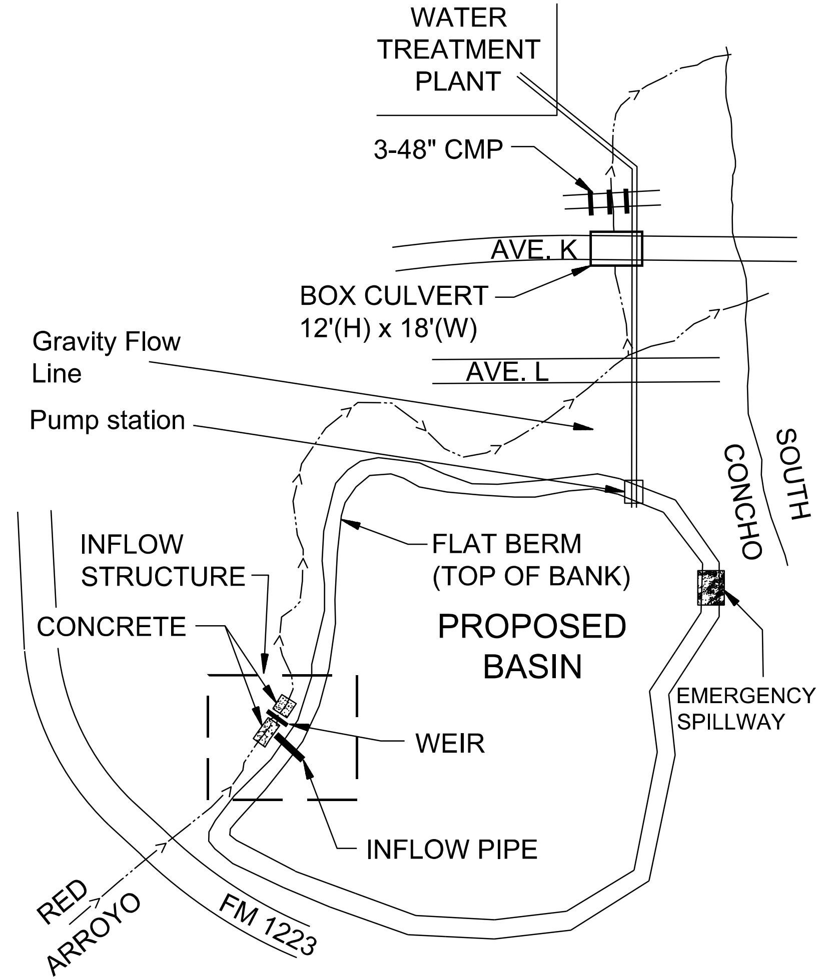
FORT WORTH, TEXAS 76102
REG NO. 2966

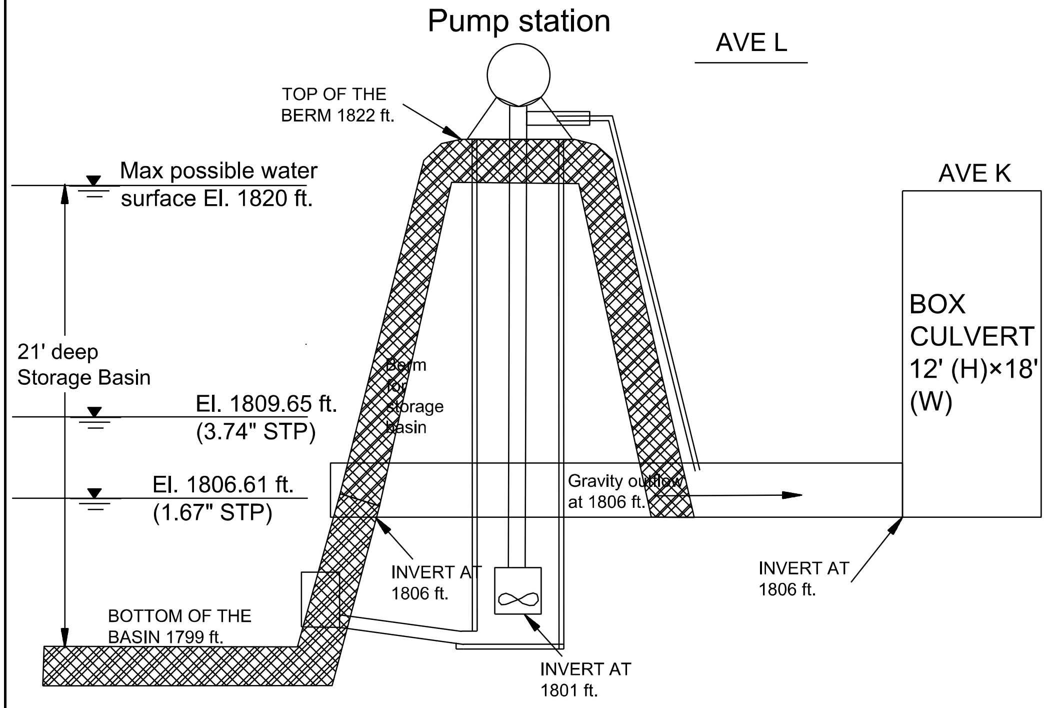
Water Supply Feasibility Option 1 - Concept Plan
Exhibit 8A

SCALE: NONE	PROJ. NO. : WSA01400

\Fw1-s15\wip\JOB\WSA01400 - UCRA Stormwater Pond\600 DISCIPLINE\607 H&H\607.4 Hydraulics\concept_layout.dwg



PLAN VIEW



PROFILE VIEW

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FORT WORTH, TEXAS 76102
REG NO. 2966

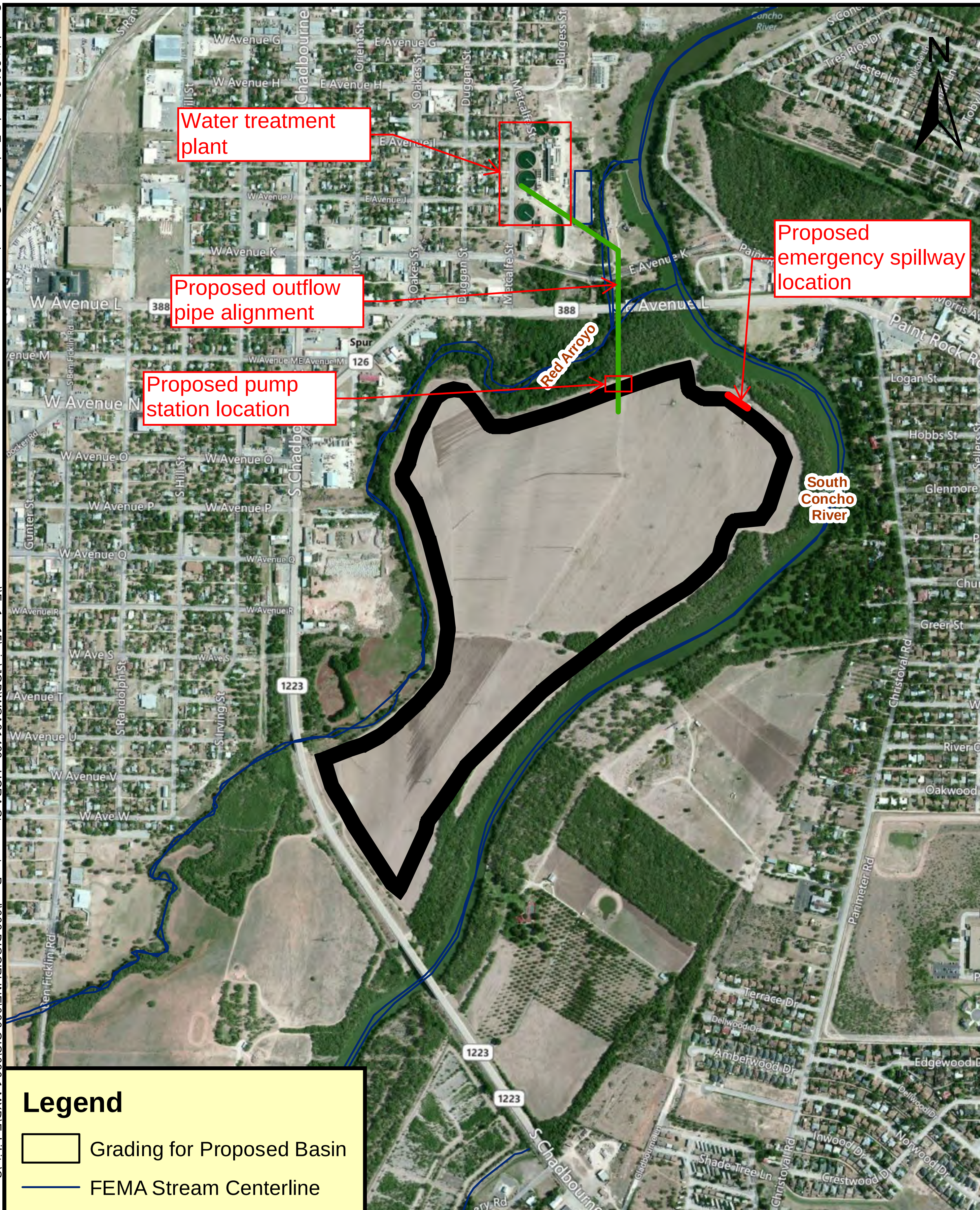
Water Supply Feasibility Option 2 - Concept Plan

Exhibit 8B

SCALE: NONE

PROJ. NO. : WSA01400

\Fw1-s15\wip\JOB\WSA01400 - UCRA Stormwater Pond\600 DISCIPLINE\607 H&H\607.4 Hydraulics\concept_layout.dwg



JACOBS	**Plan view of emergency spillway and outflow structure** UCRA Stormwater Pond City of San Angelo, Texas Jacobs Project No. WSA01400	**Source:** ESRI® Data (2013) **Scale:** 0 1,000 Feet	**Exhibit** 8C

\\Fw1-s15\wip\JOB\WSA01400 - UCRA Stormwater Pond\600 DISCIPLINE\606 GIS\606.1 MXD\Exhibits\Scenario_1_Map_8x11.mxd

EXHIBIT 9

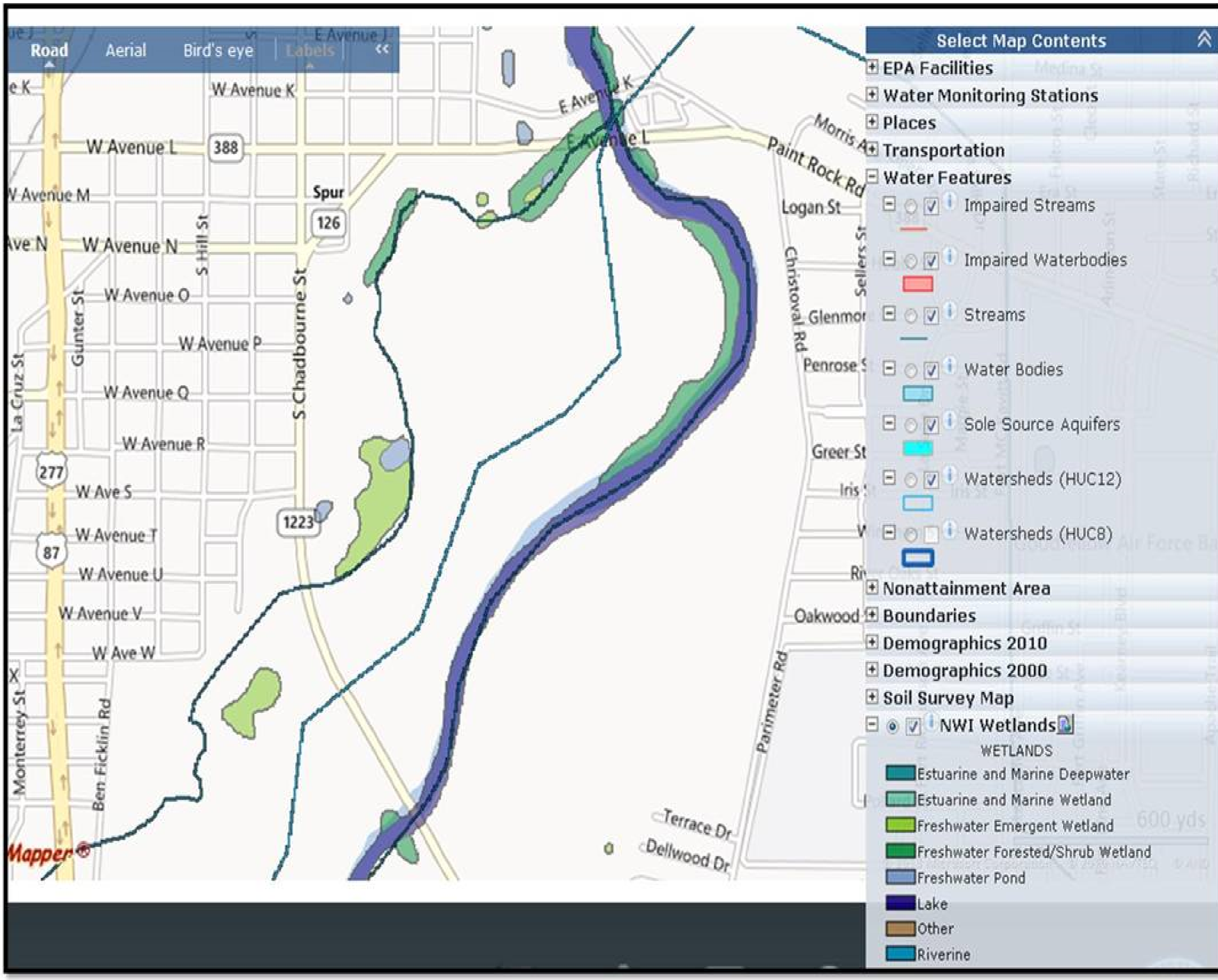
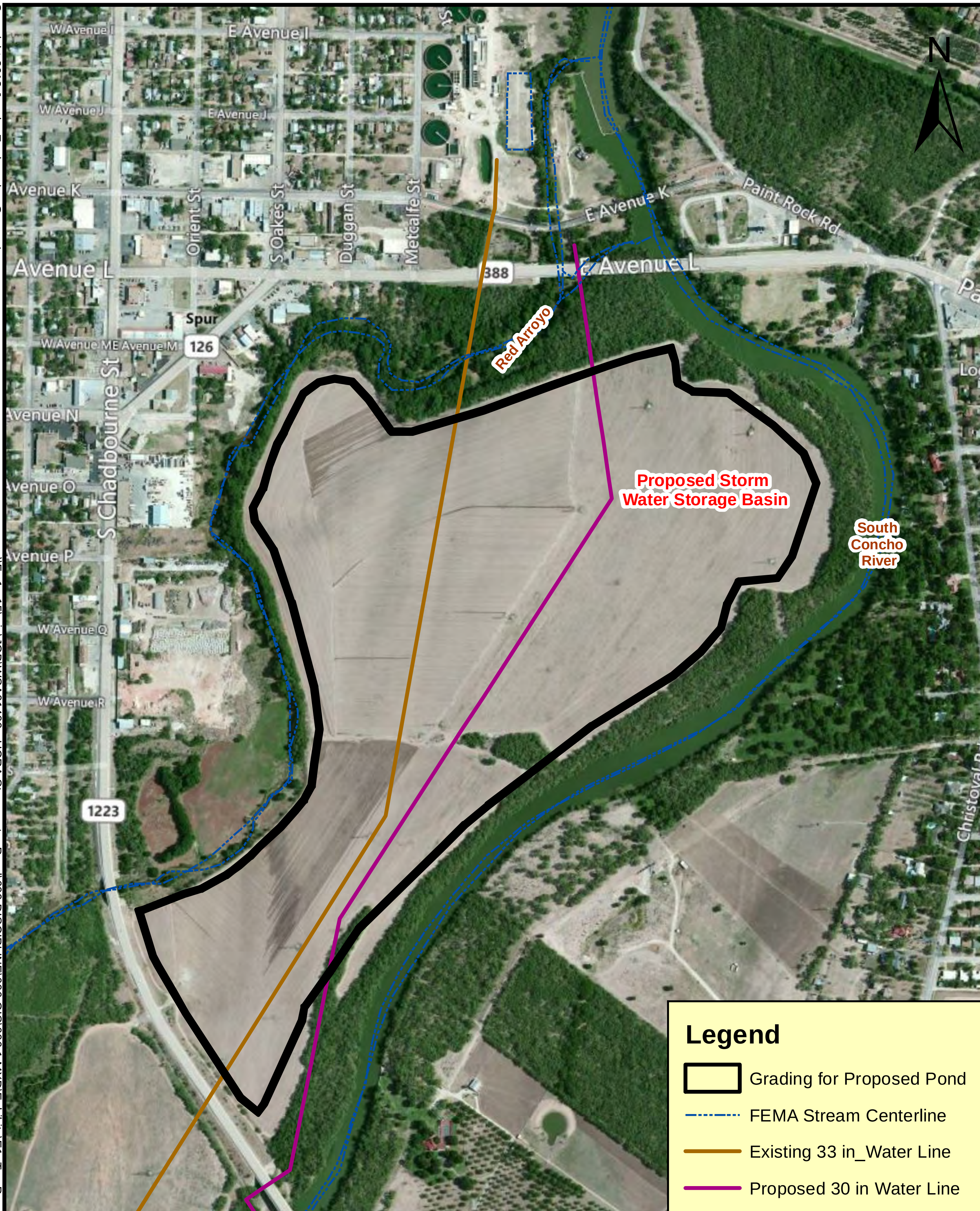


EXHIBIT 7: WETLANDS FROM EPA WEBSITE

EXHIBIT 10



Location of water lines and Basin-1

UCRA Stormwater Basin
City of San Angelo, Texas
Jacobs Project No. WSA01400

Source: ESRI® Data (2013)

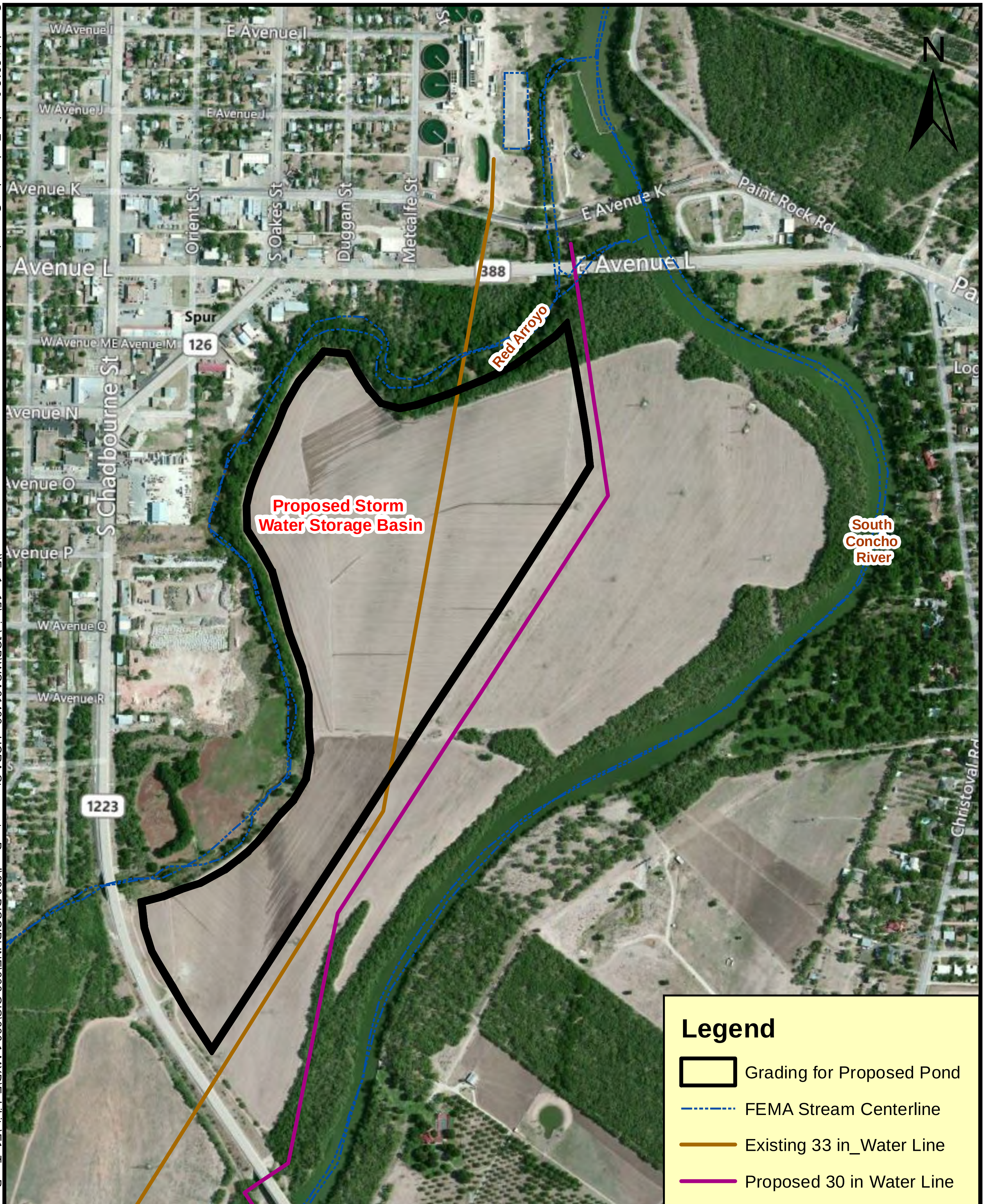
Scale: 0 300 600 Feet

Exhibit 10

JACOBS®

\\Fw1-s15\wip\JOB\WSA01400 - UCRA Stormwater Pond\600 DISCIPLINE\606 GIS\606.1 MXD\Exhibits\E1_TwoPondScenario_8x11.mxd

EXHIBIT 11



\\Fw1-s15\wip\JOB\WSA01400 - UCRA Stormwater Pond\600 DISCIPLINE\606 GIS\606.1 MXD\Exhibits\E1_TwoPondScenario_8x11.mxd

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Location of water lines and Basin-2

UCRA Stormwater Basin
City of San Angelo, Texas
Jacobs Project No. WSA01400

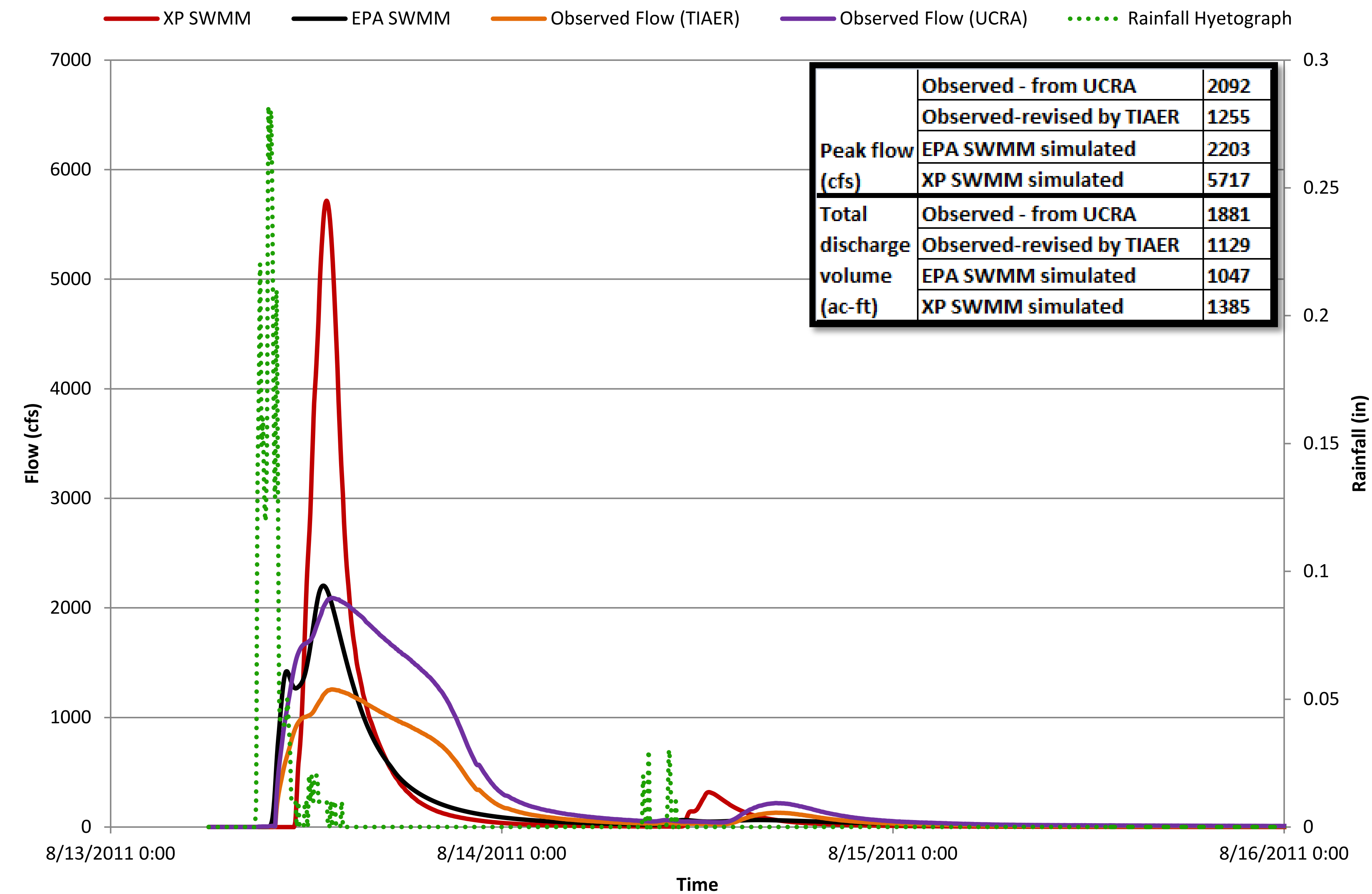
Source: ESRI® Data (2013)

Scale: 0 300 600 Feet

Exhibit 11

FIGURES

Figure 1 : Outflow hydrograph at site 2 for August 2011 storm from UCRA observed data (3.74 in total rainfall depth)



Peak flow (cfs)	Observed - from UCRA	2092
	Observed-revised by TIAER	1255
	EPA SWMM simulated	2203
	XP SWMM simulated	5717
Total discharge volume (ac-ft)	Observed - from UCRA	1881
	Observed-revised by TIAER	1129
	EPA SWMM simulated	1047
	XP SWMM simulated	1385

Figure 2 : Outflow hydrograph at site 2 for October 2011 storm from UCRA observed data (2.71 in total rainfall depth)

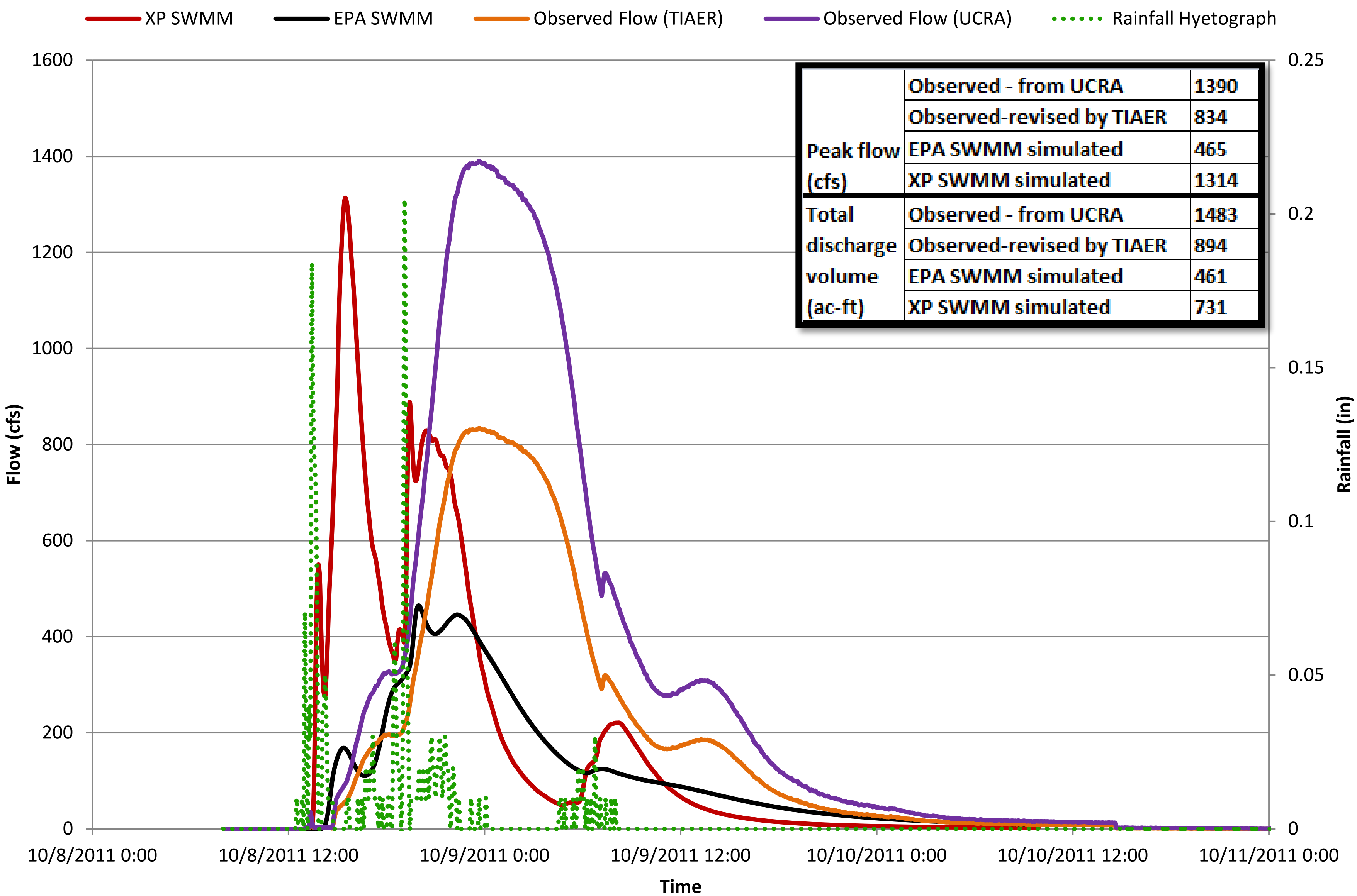


Figure 3 : Outflow hydrograph at site 2 for January 2012 storm from UCRA observed data (1.67 in total rainfall depth)

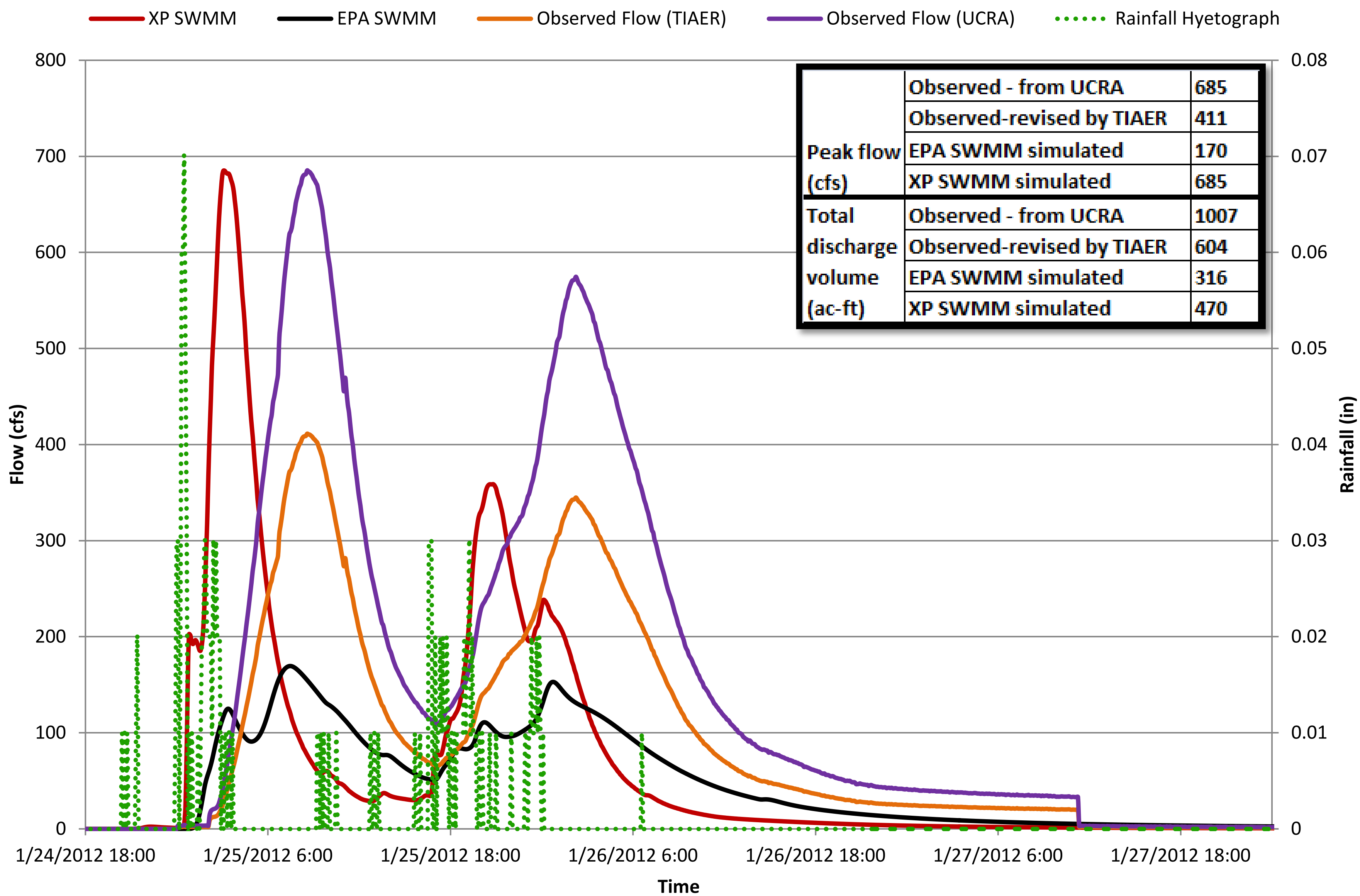
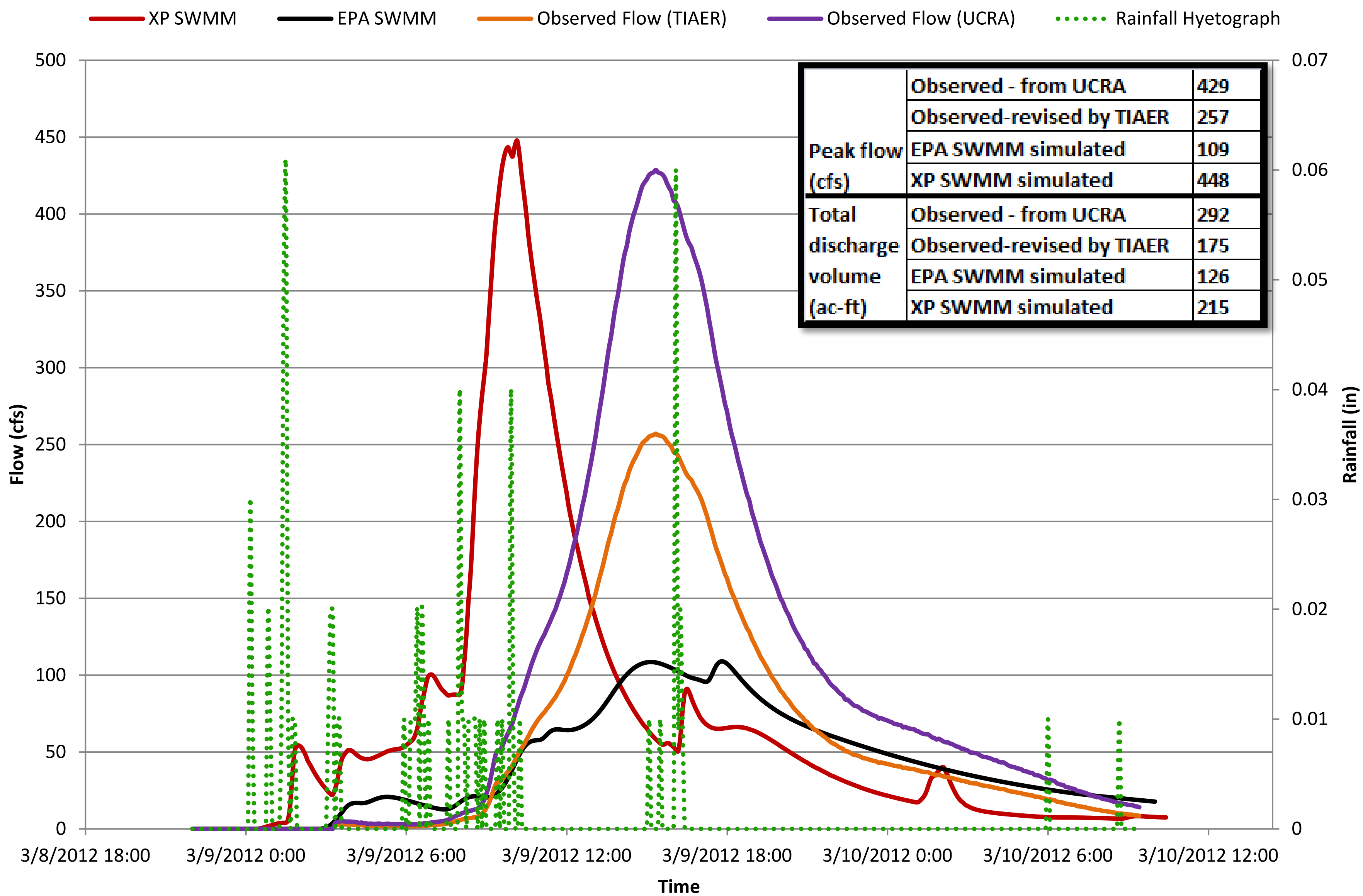


Figure 4 : Outflow hydrograph at site 2 for March 2012 storm from UCRA observed data (0.74 in total rainfall depth)



Peak flow (cfs)	Observed - from UCRA	429
	Observed-revised by TIAER	257
	EPA SWMM simulated	109
	XP SWMM simulated	448
Total discharge volume (ac-ft)	Observed - from UCRA	292
	Observed-revised by TIAER	175
	EPA SWMM simulated	126
	XP SWMM simulated	215

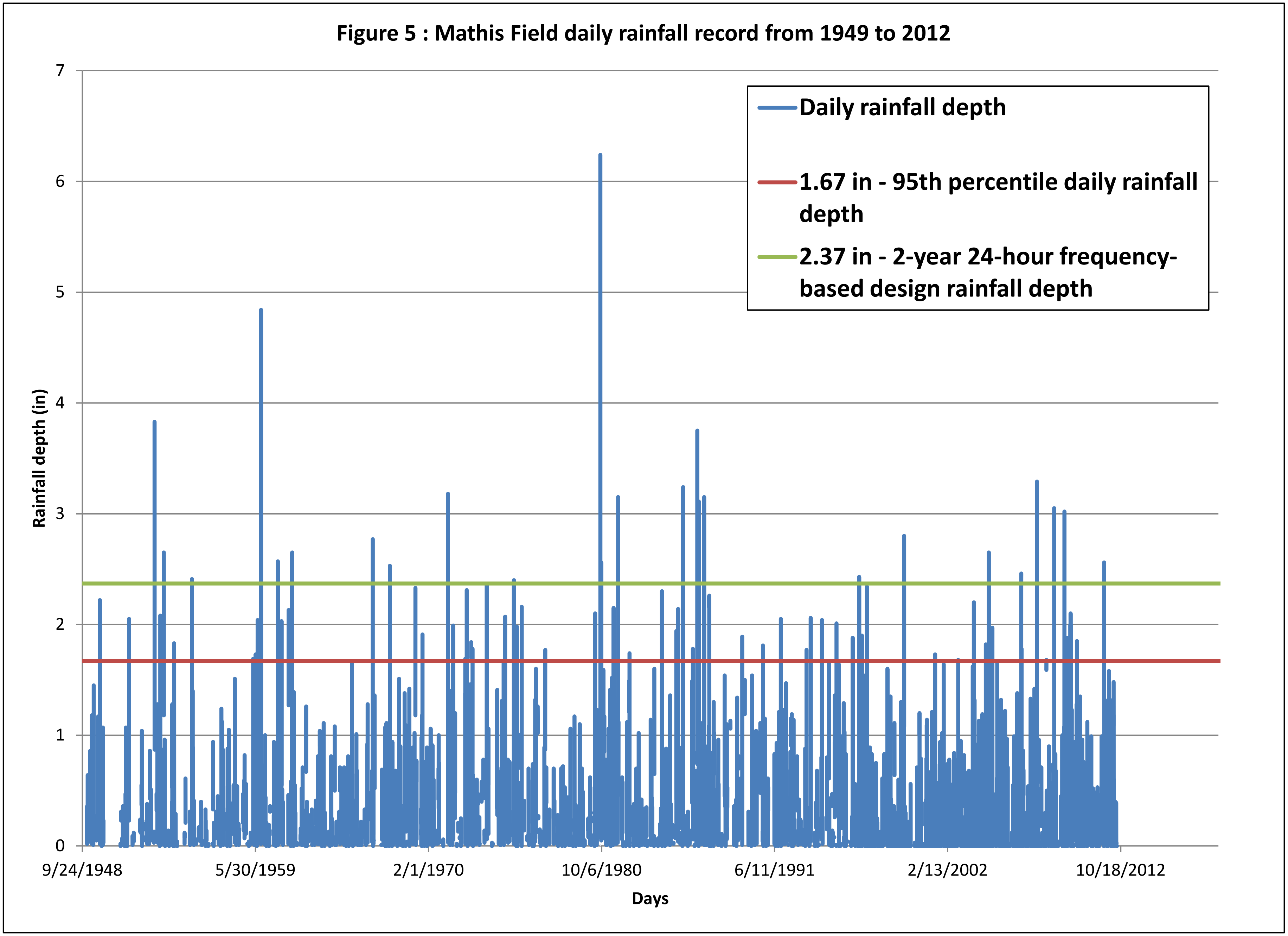


Figure 5 : Mathis Field daily rainfall record from 1949 to 2012
Daily rainfall depth
1.67 in - 95th percentile daily rainfall depth
2.37 in - 2-year 24-hour frequency-based design rainfall depth
Rainfall depth (in)
Days
0
1
2
3
4
5
6
7
9/24/1948
5/30/1959
2/1/1970
10/6/1980
6/11/1991
2/13/2002
10/18/2012

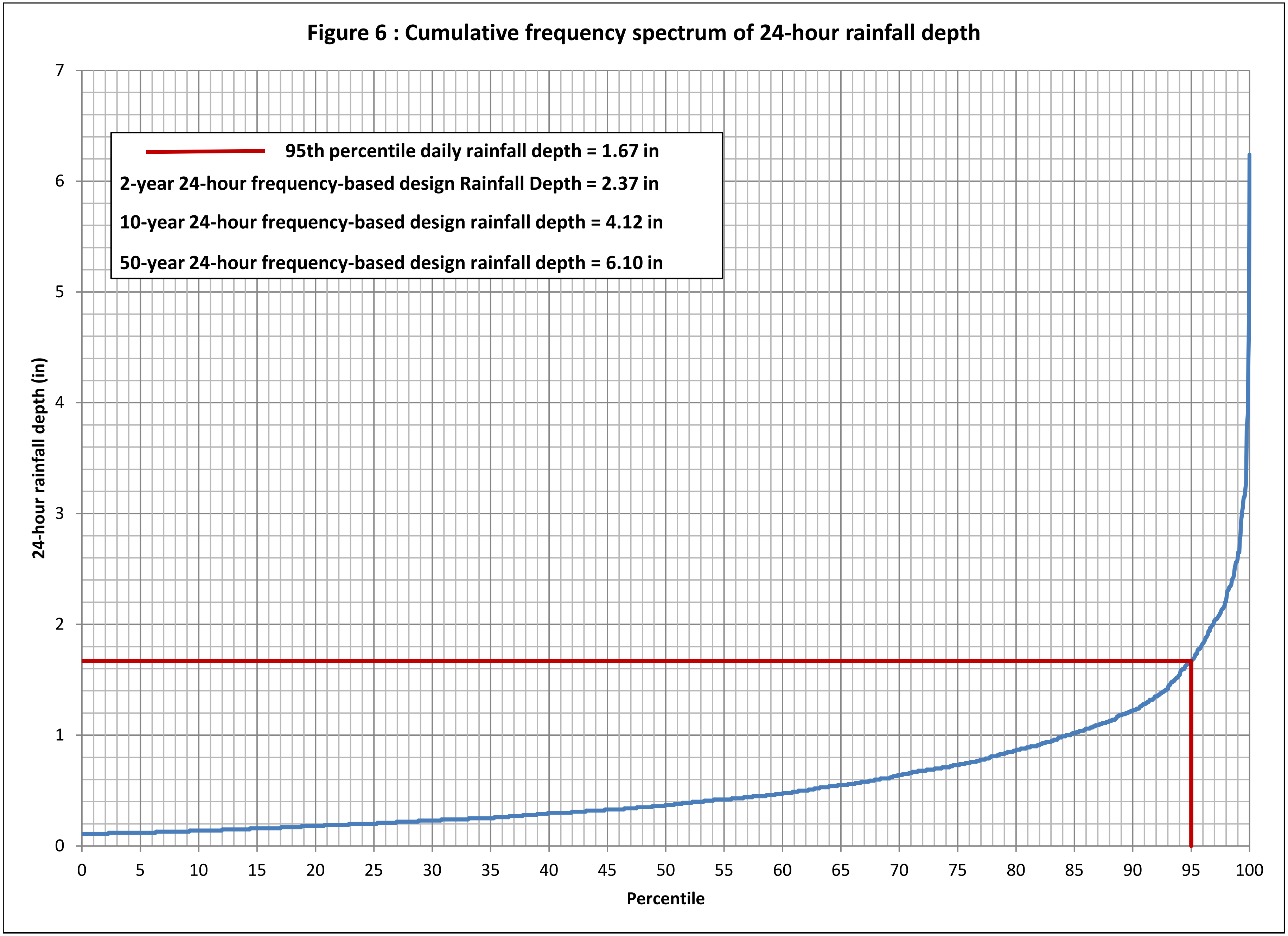


Figure 6 : Cumulative frequency spectrum of 24-hour rainfall depth
95th percentile daily rainfall depth = 1.67 in
2-year 24-hour frequency-based design Rainfall Depth = 2.37 in
10-year 24-hour frequency-based design rainfall depth = 4.12 in
50-year 24-hour frequency-based design rainfall depth = 6.10 in
24-hour rainfall depth (in)
0
1
2
3
4
5
6
7
Percentile
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
85
90
95
100

Figure 7 : NRCS 24-hour Type II rainfall hyetograph compared to selected historical storm events

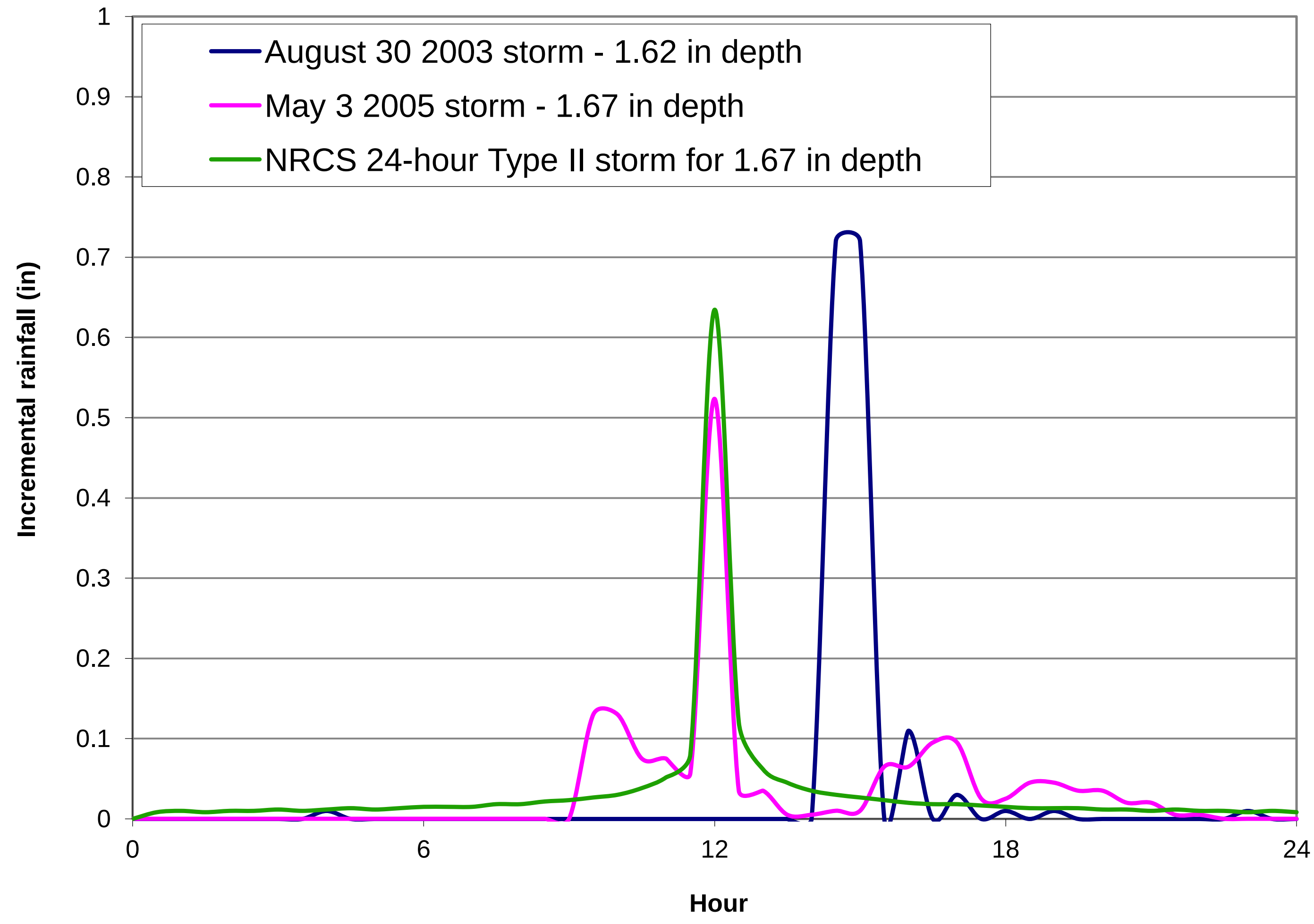


Figure 8 : NRCS 24-hour Type II rainfall mass curve compared to the observed mass curves of selected historical storm events