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 **©COPYRIGHT 2013**

Jacobs

Storm Water Storage Basin Feasibility Study

Red Arroyo Watershed, San Angelo

6/24/2013

STATE OF TEXAS
BISWAJIT MUKHOPADHYAY
92709
LICENSED
PROFESSIONAL ENGINEER

B. Mukhopadhyay 6/24/2013

TABLE OF CONTENT

EXECUTIVE SUMMARY **1**
BACKGROUND **2**
BASIS OF DESIGN **2**
DATA COLLECTION **2**
EVALUATION OF CURRENT HYDROLOGIC MODEL **3**
Updating the EPA-SWMM model to XP-SWMM 4
Comparison of EPA-SWMM and XP-SWMM model Output 5
RAINFALL ANALYSIS **9**
HYDRAULICS **10**
Model domain 10
Boundary conditions 11
Existing condition 11
Proposed condition 12
WATER SUPPLY FEASIBILITY **18**
ENVIRONMENTAL FACTORS AND UTILITY COORDINATION **19**
PRELIMINARY OPINION OF PROBABLE COST **20**
RECOMMENDATIONS FOR FURTHER ANALYSES **21**

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

Rainfall Event Modeled	PEAK FLOW (in cfs) at Site 2			
	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*					Green-Ampt parameters**			Basin characteristics*				Kinematic wave parameters***		
		Impervious Area			Pervious area											
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 95th percentile of the rainfall depths that are given in a period of record. Because the 95th 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 95th 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 95th 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

Proposed condition- January 2012 event (1.67" STP)

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'

Proposed condition- August 2011 event (3.74" STP)

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 ONE 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 ONE 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 TWO 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.

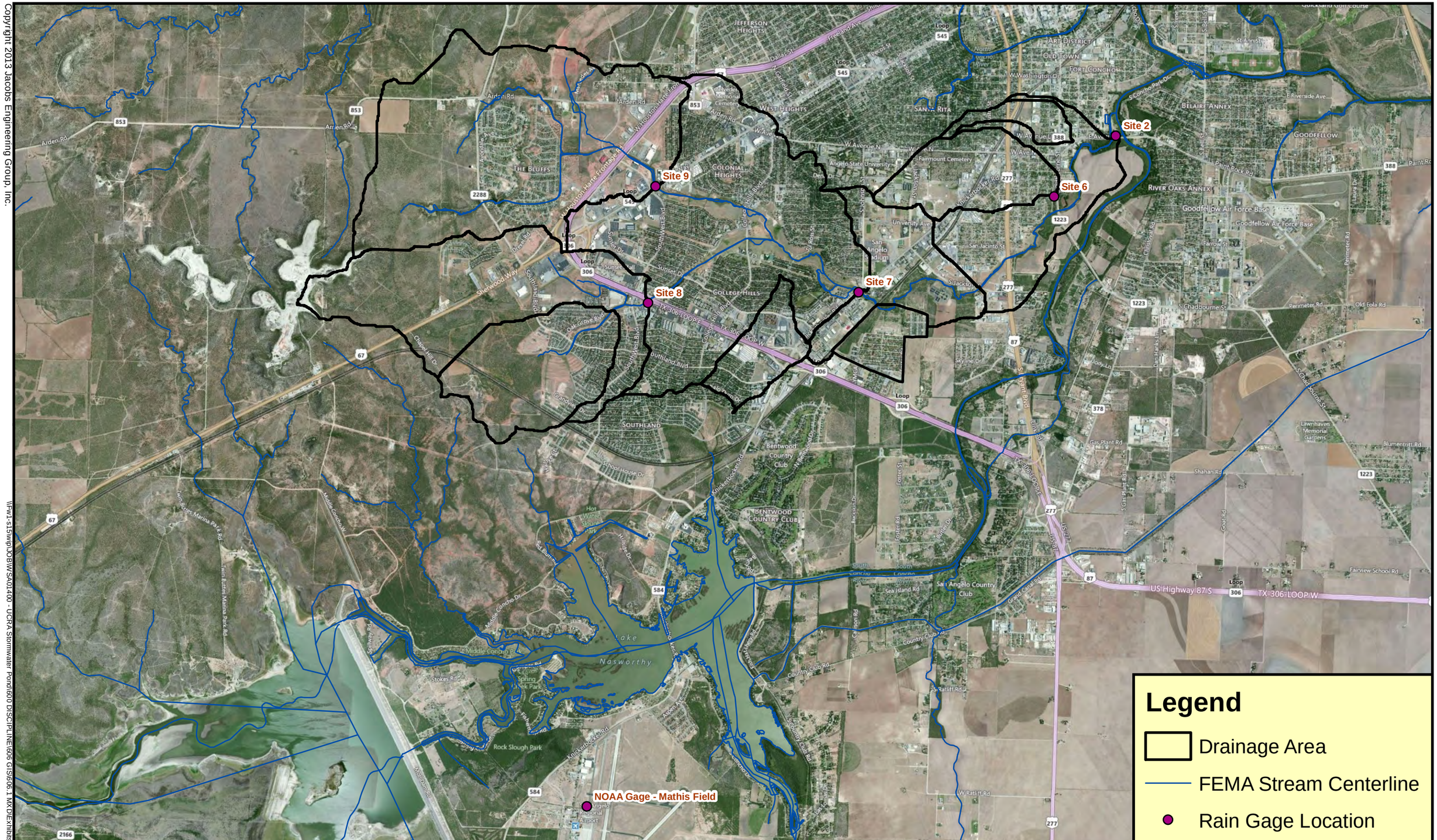
- Obtain detailed channel survey along Red Arroyo to enable creating of a more accurate hydraulic model of the Red Arroyo. An accurate model will help in optimizing the inflow structures.
- Develop more comprehensive hydraulic model including the proposed outflow structures and conveyance to the water 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

Copyright 2013 Jacobs Engineering Group, Inc.



JACOBS

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-s15wip\JOBS\WSA01400 - UCRA Stormwater Pond\600 DISCIPLINE\606 GIS\606.1 MXD\Exhibits\E1_DA_Map_11X17.MXD

EXHIBIT 2

Copyright 2013 Jacobs Engineering Group, Inc.

\\Fw1-s15\wip\JOB\WSA01400 - UCRA Stormwater Pond\600 DISCIPLINE\606 GIS\606.1 MXD\Exhibits\E2_LocationMap_8x11.mxd

JACOBS®

Location Map

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

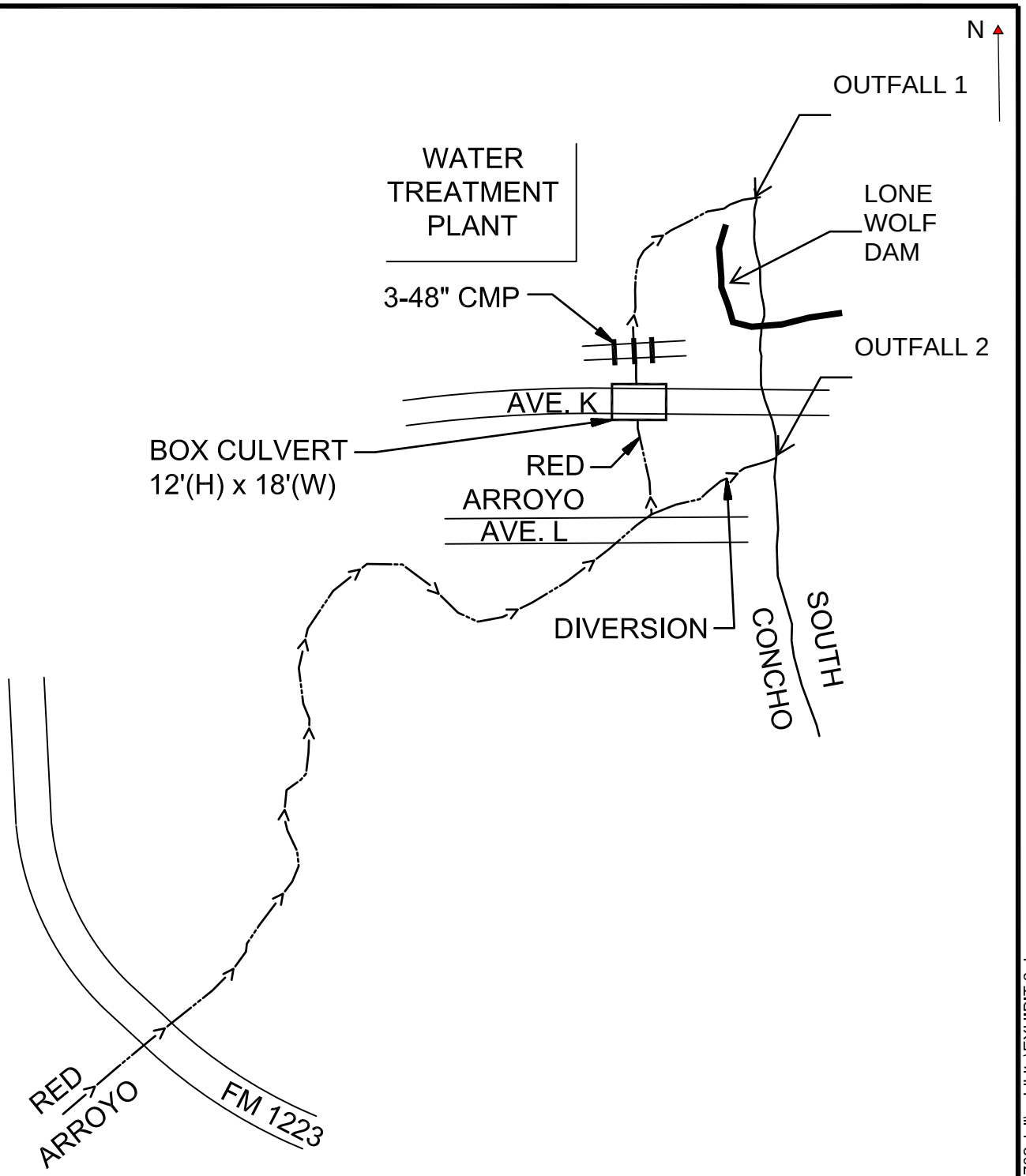
Source: ESRI® Data (2013)

Scale: 0 1,000 Feet

Exhibit

2

EXHIBIT 3



JACOBS™

JACOBS ENGINEERING GROUP, INC.
5995 ROGERDALE ROAD
HOUSTON, TEXAS 77072
281.351.6000

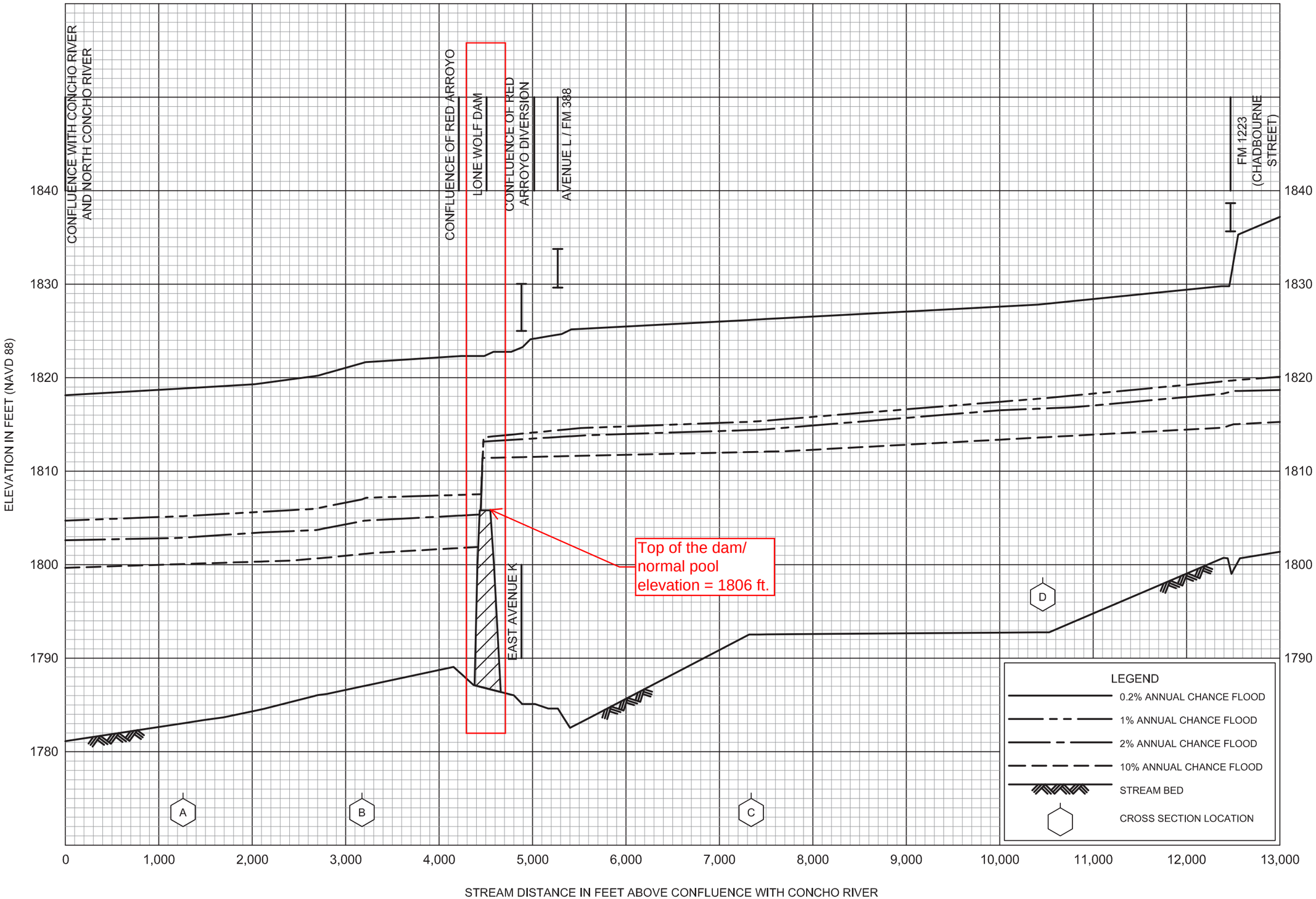
HYDRAULIC MODEL DOMAIN

EXHIBIT 3

SCALE: NONE	PROJ. NO.

G:\NAI\Projects\SHELL\Franklin\700CADD\702civil\exhibits\EXHIBIT 3.dwg

EXHIBIT 4



FLOOD PROFILES

SOUTH CONCHO RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

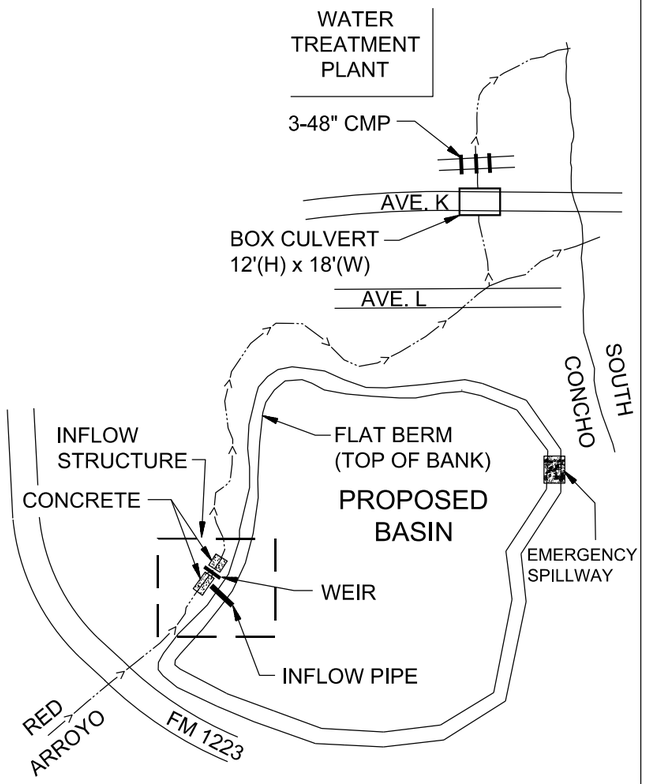
TOM GREEN COUNTY, TX

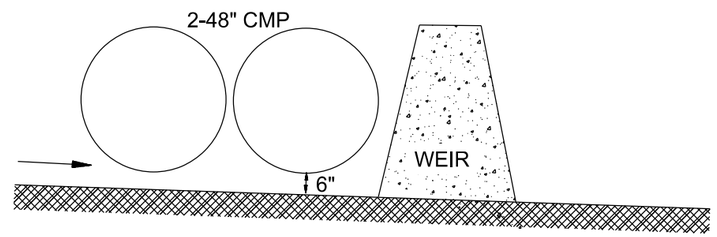
AND INCORPORATED AREAS

EXHIBIT 5

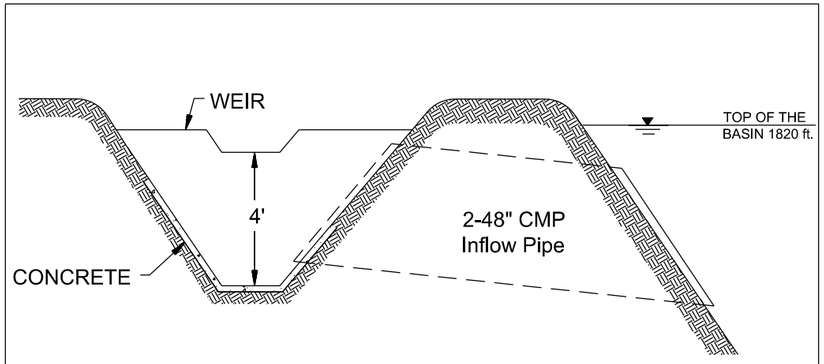
N
Outfall 1
Outfall 2
Stream flow hydrograph entered at this node
W Avenue I
E Avenue I
W Avenue J
E Avenue J
W Avenue K
E Avenue K
Paint Rock Rd
W Avenue L
E Avenue L
388
Spur
126
W Avenue M
E Avenue M
W Avenue N
W Avenue O
W Avenue P
W Avenue Q
W Avenue R
W Ave S
1223
S Chadbourne St
Orient St
S Oakes St
Duggan St
Metcalfe St
South Concho River
RA1
RA9
RA6
RA5
RAD3
Chnl_XS_9
Chnl_XS_8
Chnl_XS_6
Chnl_XS_5
Chnl_XS_4
Chnl_XS_3
Chnl_XS_2
Chnl_XS_1
A1
A2
A3
A4
A5
A6
A7
A8
A9
A10
A11
Bridge XS2
Bridge XS1

EXHIBIT 6

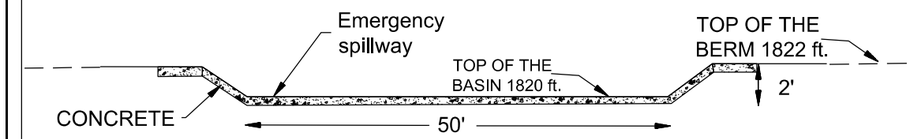




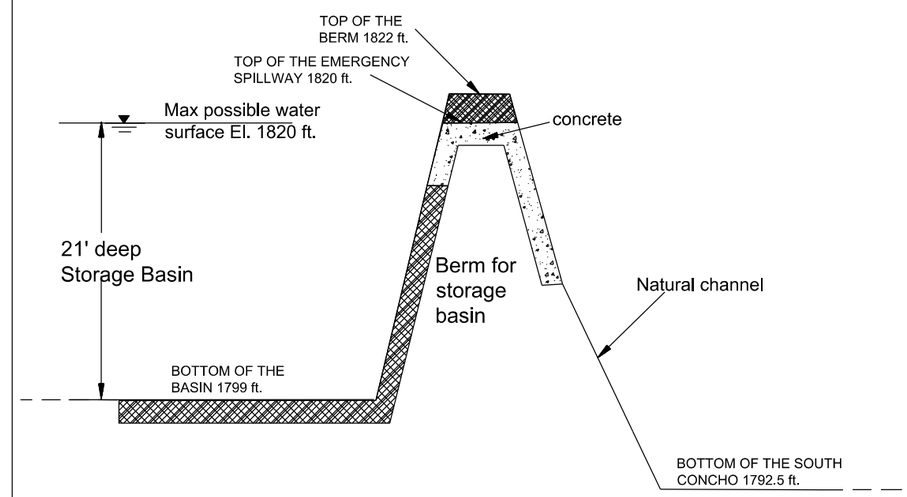
Longitudinal cross-section view of inflow structure



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

JACOBS

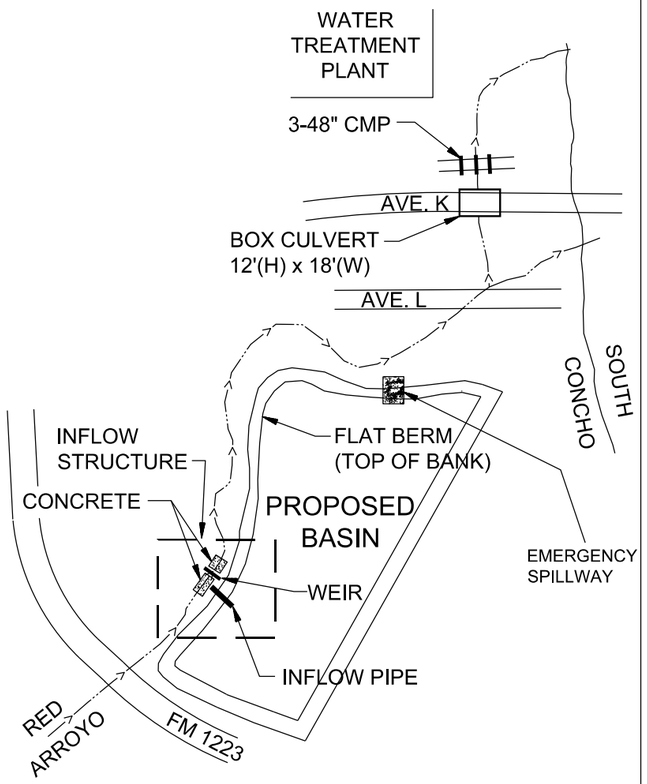
JACOBS ENGINEERING GROUP, INC.
777 MAIN STREET
FORT WORTH, TEXAS 76102
REG NO. 2966

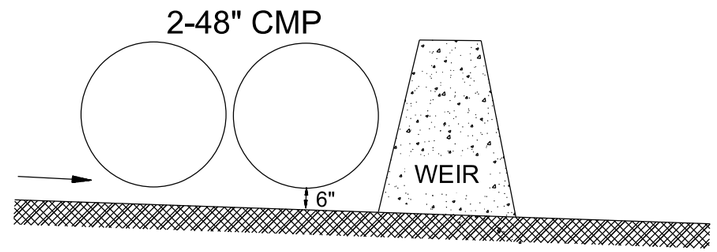
Concept Plan for Inflow Structure and Emergency Spillway

Exhibit 6A

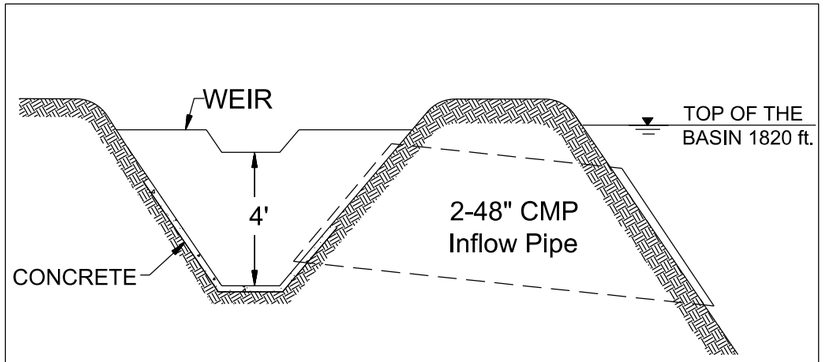
SCALE: NONE | PROJ. NO. : WSA01400

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

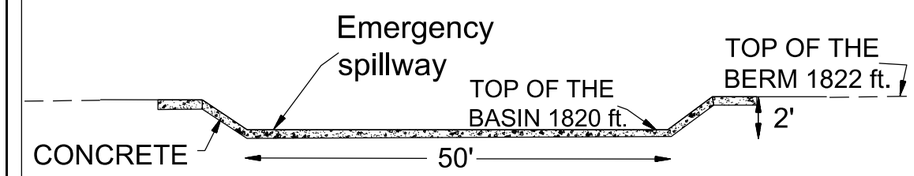




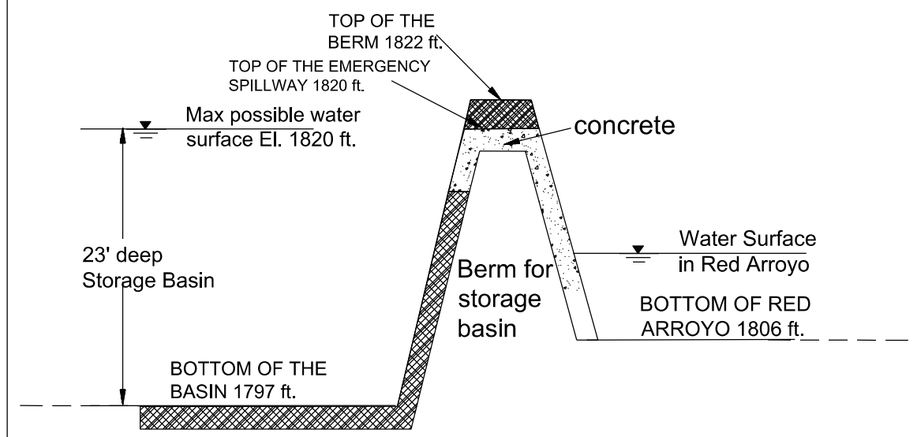
Longitudinal cross-section view of inflow structure



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

JACOBS

JACOBS ENGINEERING GROUP, INC.
777 MAIN STREET
FORT WORTH, TEXAS 76102
REG NO. 2966

Concept Plan for Inflow Structure and Emergency Spillway

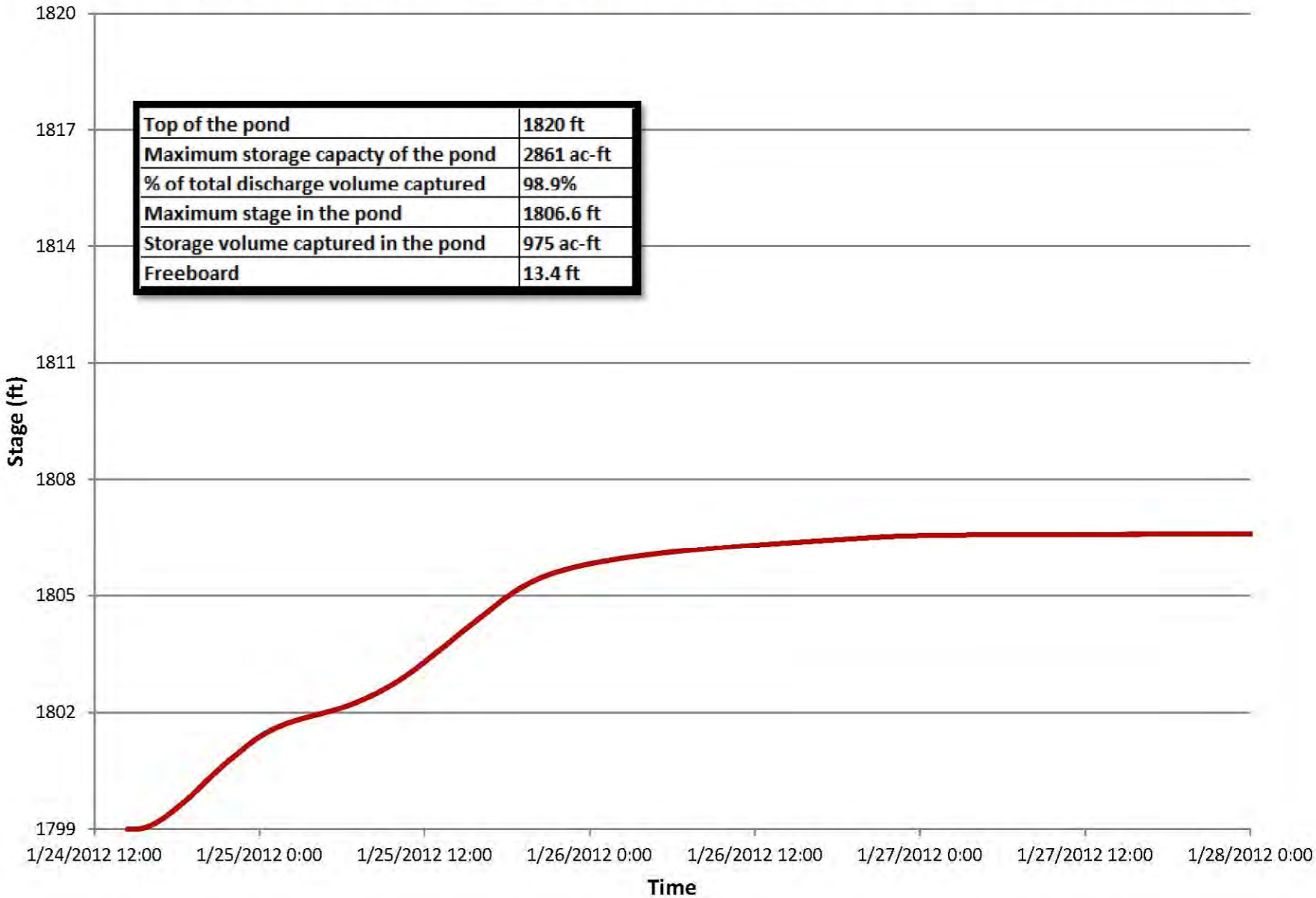
Exhibit 6B

SCALE: NONE | PROJ. NO. : WSA01400

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

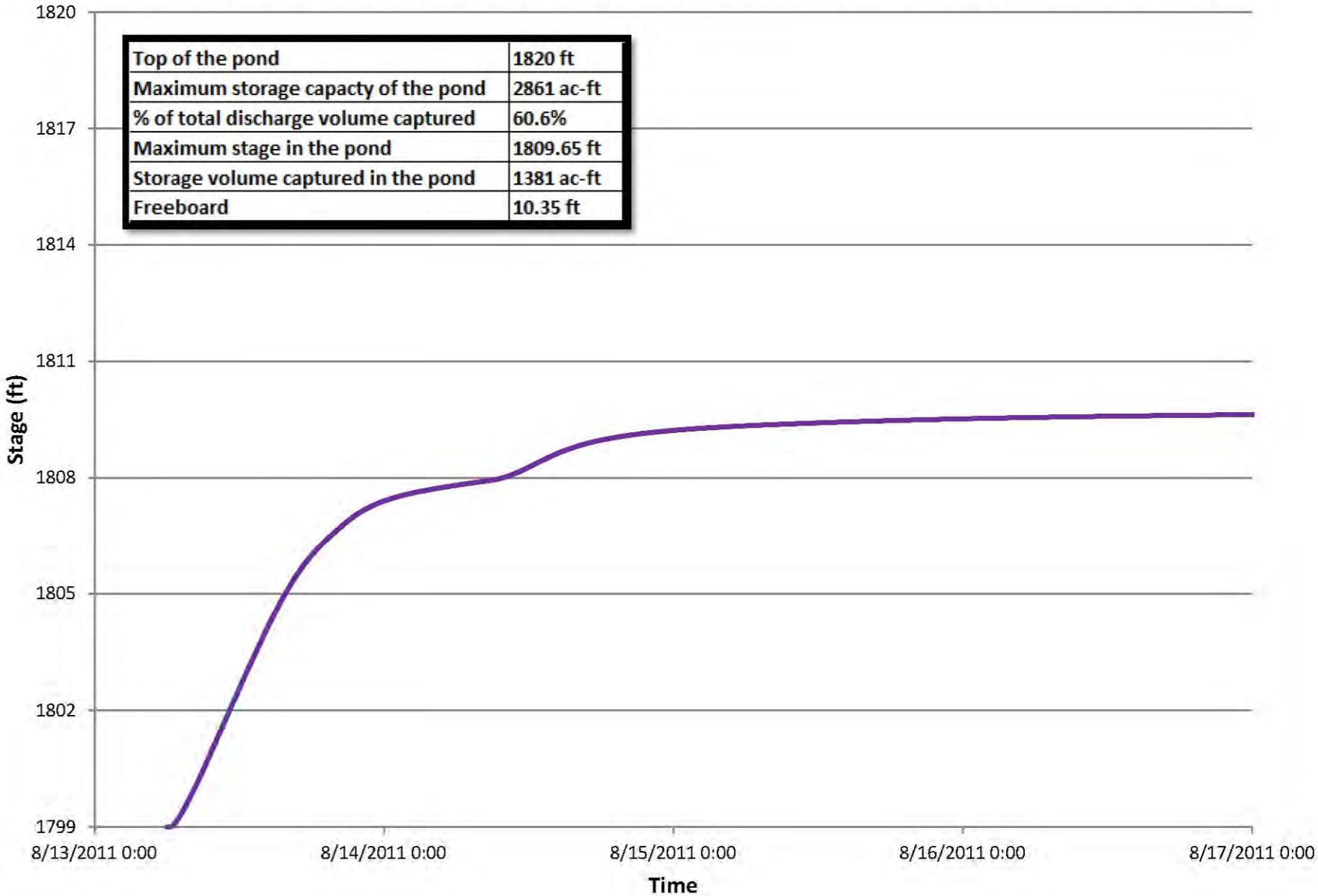
EXHIBIT 7

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



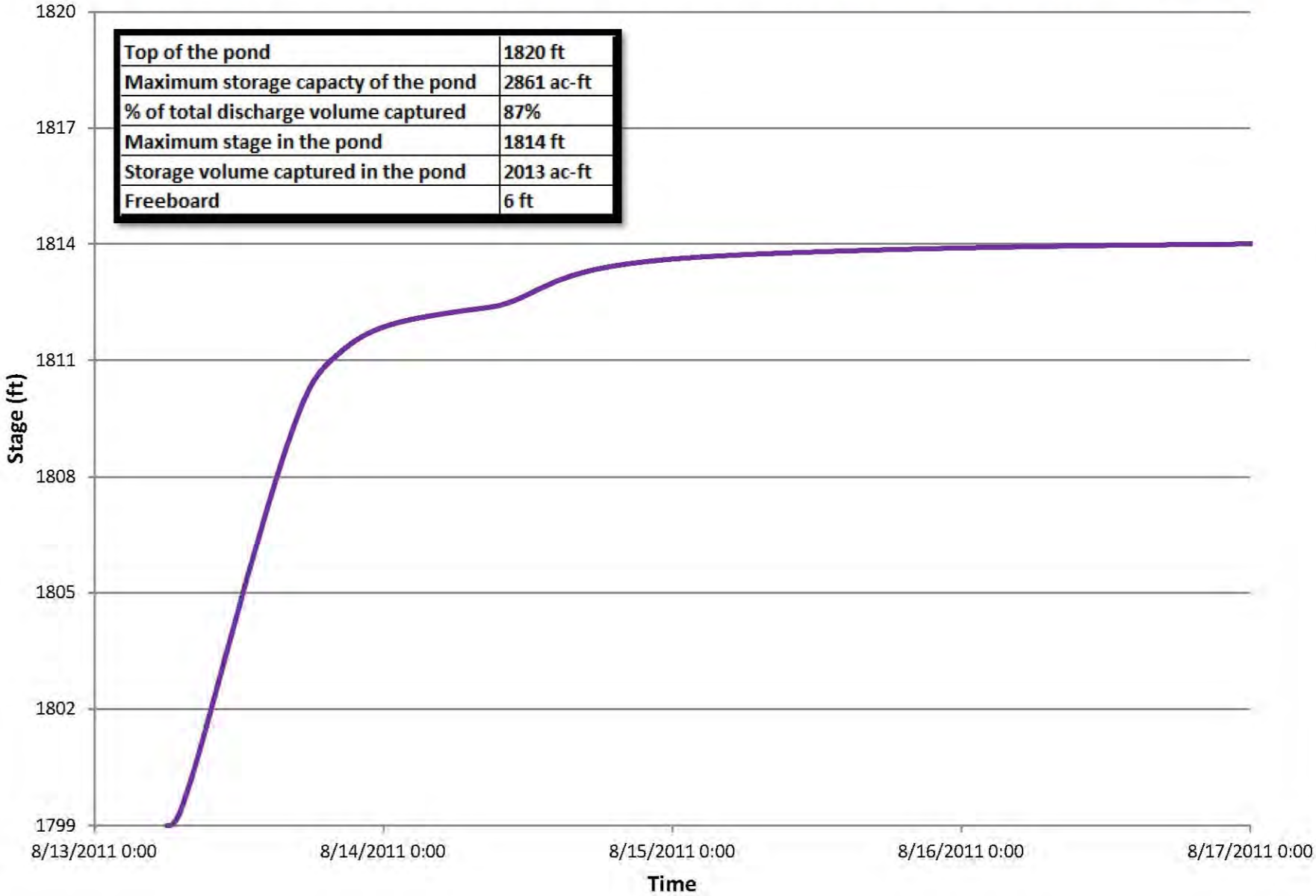
Top of the pond	1820 ft
Maximum storage capacty of the pond	2861 ac-ft
% of total discharge volume captured	98.9%
Maximum stage in the pond	1806.6 ft
Storage volume captured in the pond	975 ac-ft
Freeboard	13.4 ft

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



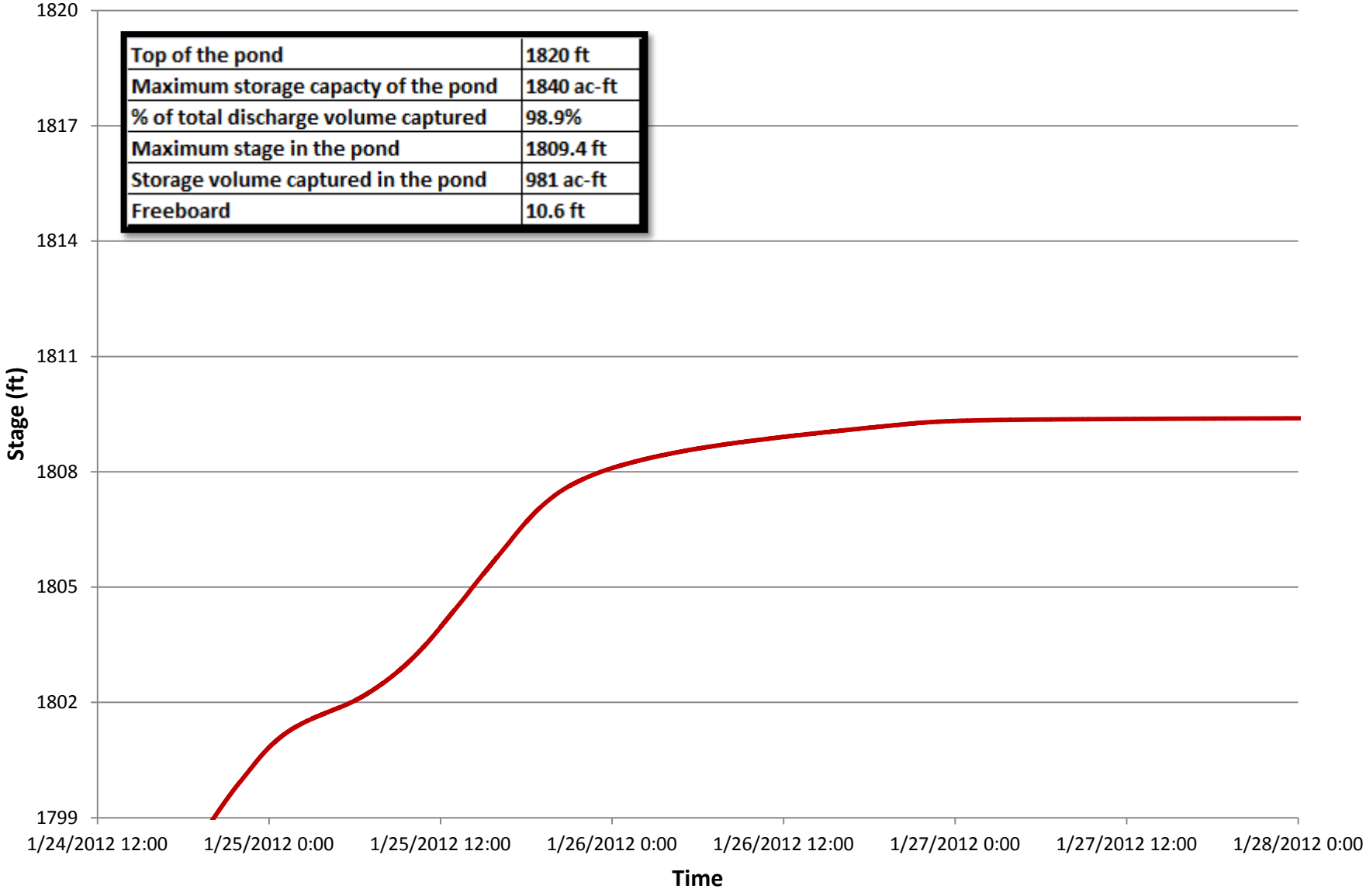
Top of the pond	1820 ft
Maximum storage capacty of the pond	2861 ac-ft
% of total discharge volume captured	60.6%
Maximum stage in the pond	1809.65 ft
Storage volume captured in the pond	1381 ac-ft
Freeboard	10.35 ft

Exhibit 7C : Basin 1 - stage time curve for two inflow structures with August 2011 storm - 3.74 in STP



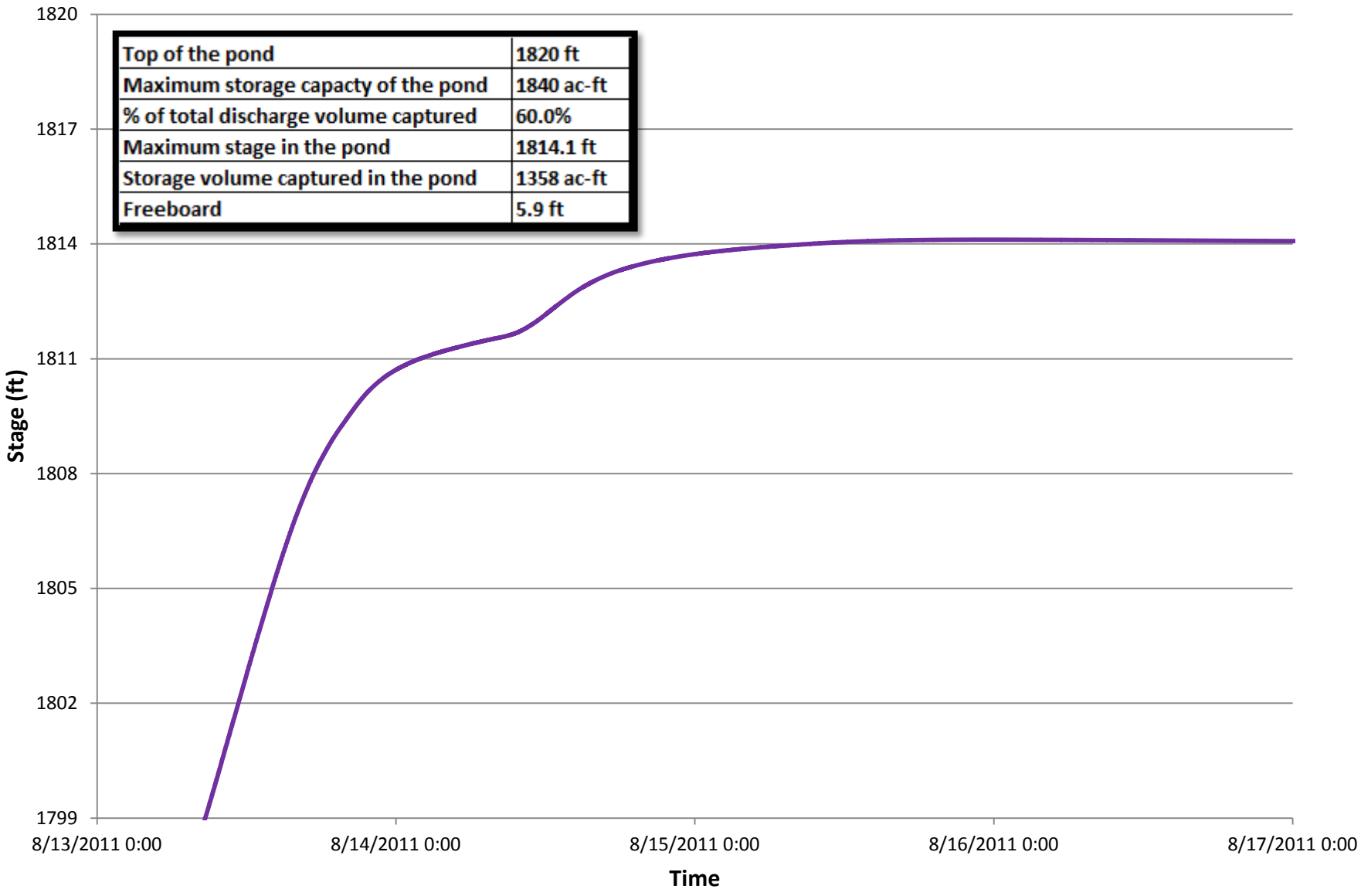
Top of the pond	1820 ft
Maximum storage capacty of the pond	2861 ac-ft
% of total discharge volume captured	87%
Maximum stage in the pond	1814 ft
Storage volume captured in the pond	2013 ac-ft
Freeboard	6 ft

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



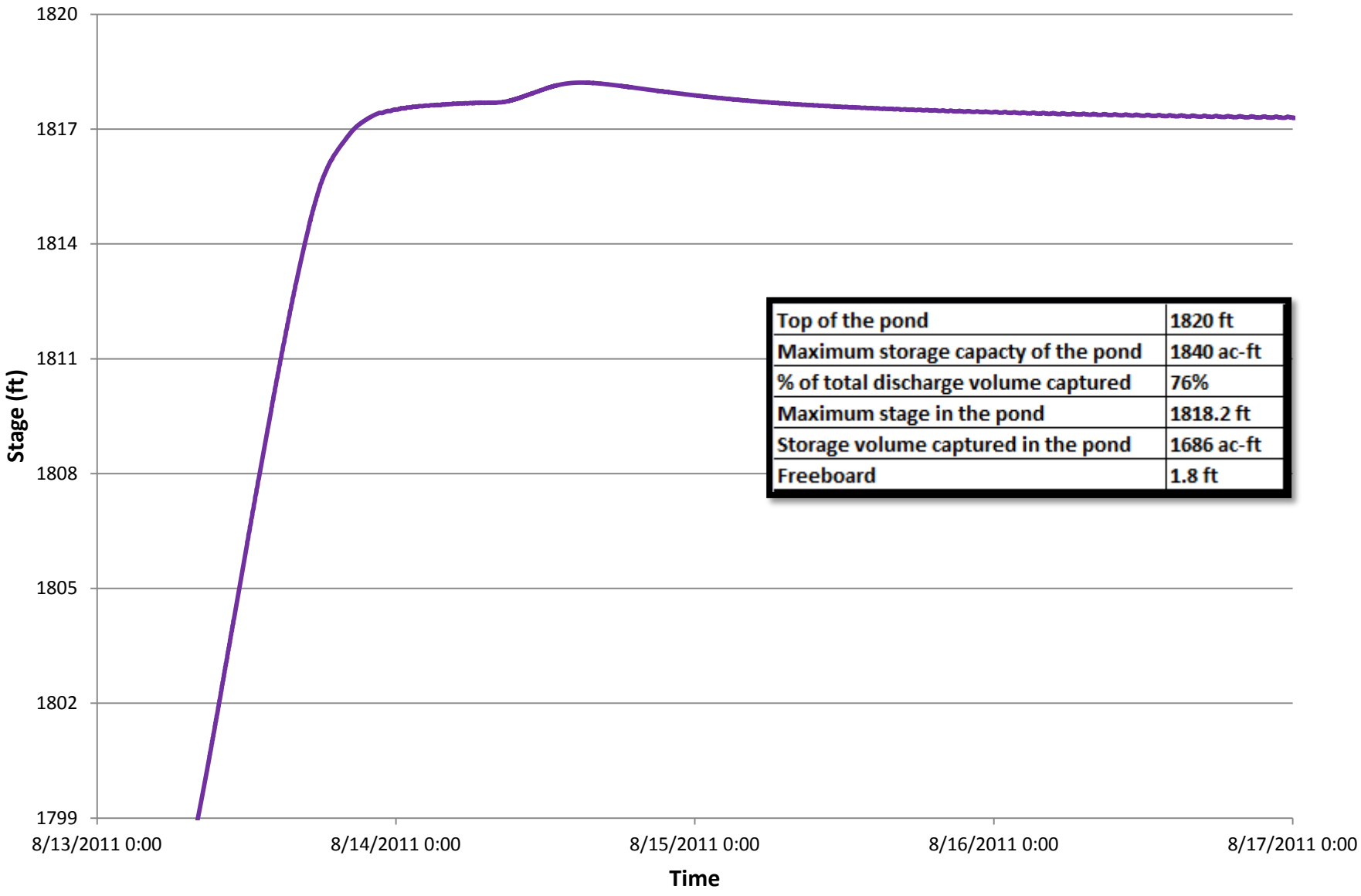
Top of the pond	1820 ft
Maximum storage capacty of the pond	1840 ac-ft
% of total discharge volume captured	98.9%
Maximum stage in the pond	1809.4 ft
Storage volume captured in the pond	981 ac-ft
Freeboard	10.6 ft

Exhibit 7E : Basin 2 - stage time curve for one inflow structure with August 2011 storm - 3.74 in STP



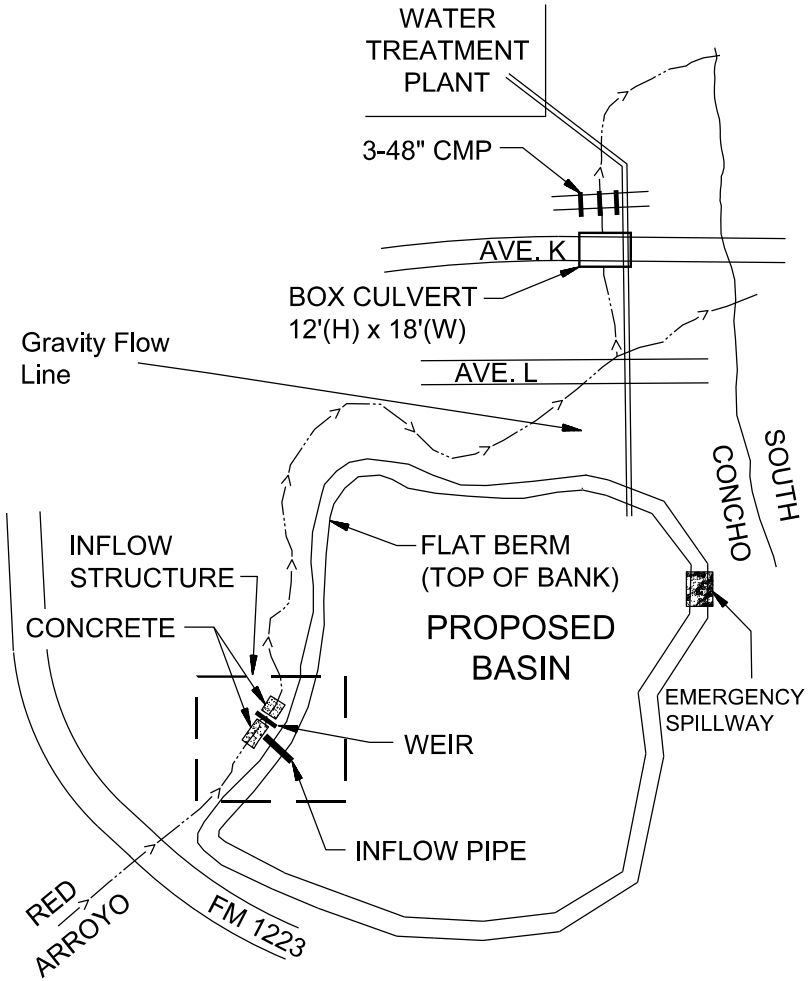
Top of the pond	1820 ft
Maximum storage capacty of the pond	1840 ac-ft
% of total discharge volume captured	60.0%
Maximum stage in the pond	1814.1 ft
Storage volume captured in the pond	1358 ac-ft
Freeboard	5.9 ft

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

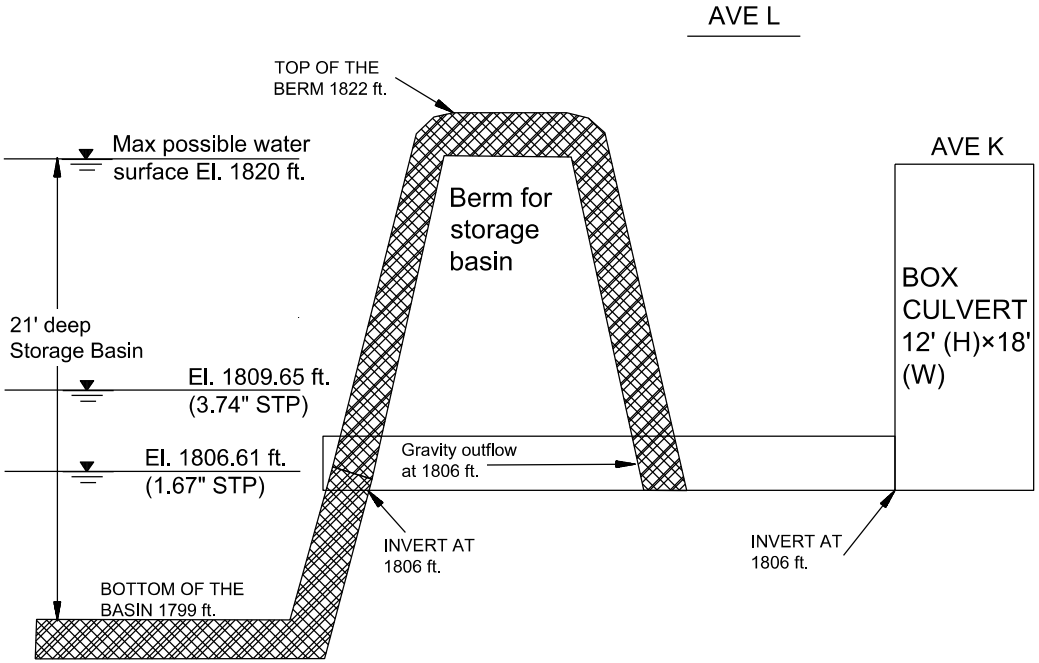


Top of the pond	1820 ft
Maximum storage capacty of the pond	1840 ac-ft
% of total discharge volume captured	76%
Maximum stage in the pond	1818.2 ft
Storage volume captured in the pond	1686 ac-ft
Freeboard	1.8 ft

EXHIBIT 8



PLAN VIEW



PROFILE VIEW

JACOBS

JACOBS ENGINEERING GROUP, INC.
777 MAIN STREET
FORT WORTH, TEXAS 76102
REG NO. 2966

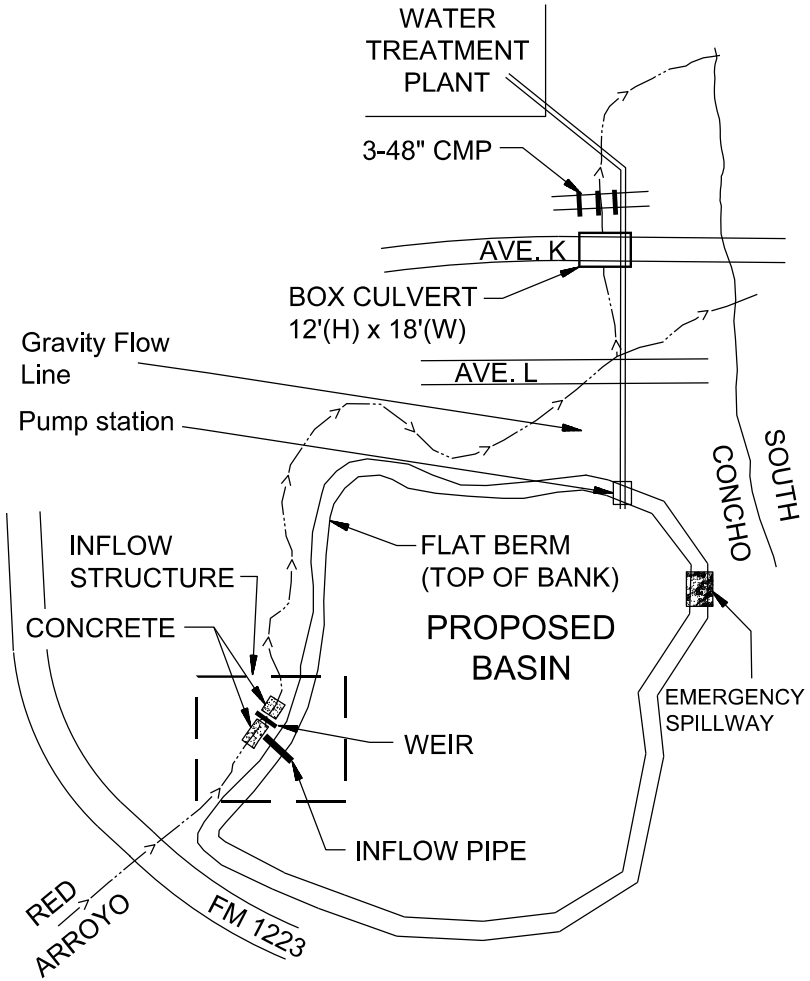
Water Supply Feasibility Option 1 - Concept Plan

Exhibit 8A

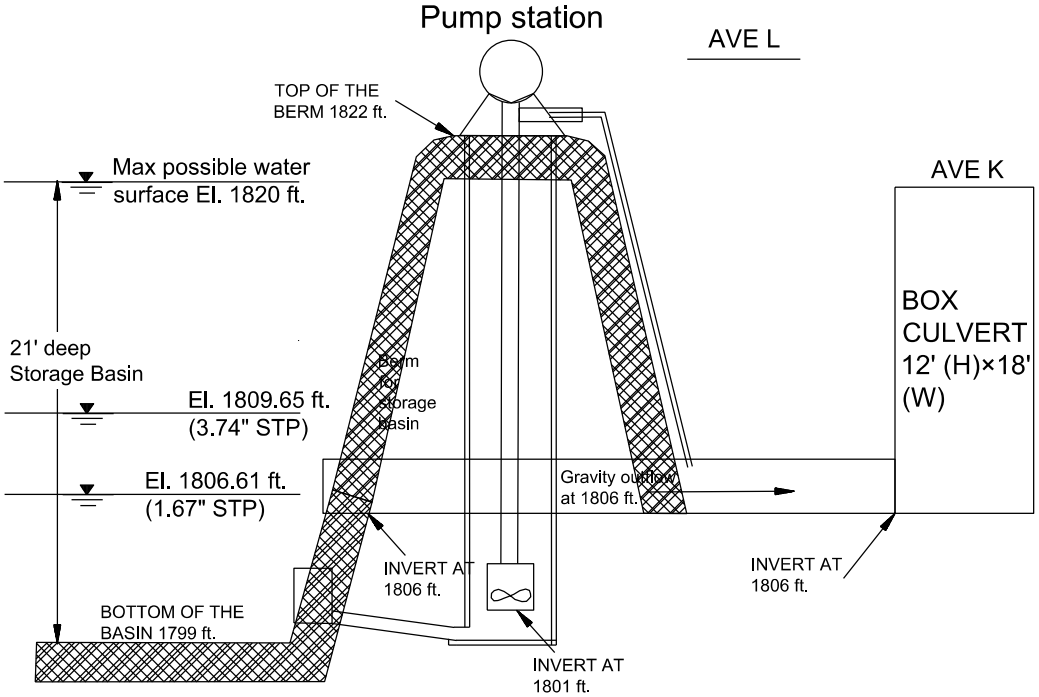
SCALE: NONE

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

JACOBS

JACOBS ENGINEERING GROUP, INC.
777 MAIN STREET
FORT WORTH, TEXAS 76102
REG NO. 2966

Water Supply Feasibility Option 2 - Concept Plan

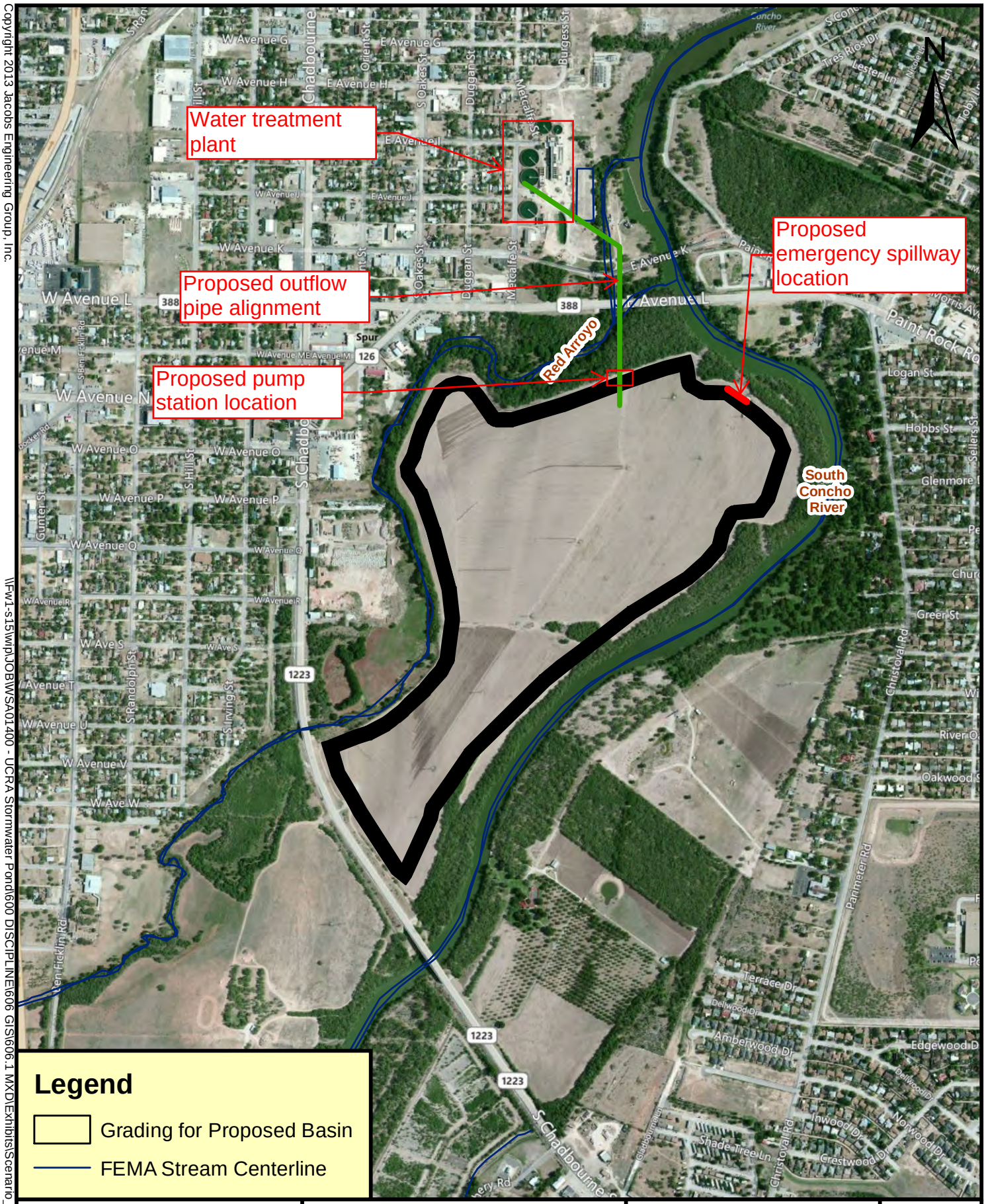
Exhibit 8B

SCALE: NONE

PROJ. NO. : WSA01400

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

Copyright 2013 Jacobs Engineering Group, Inc.



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

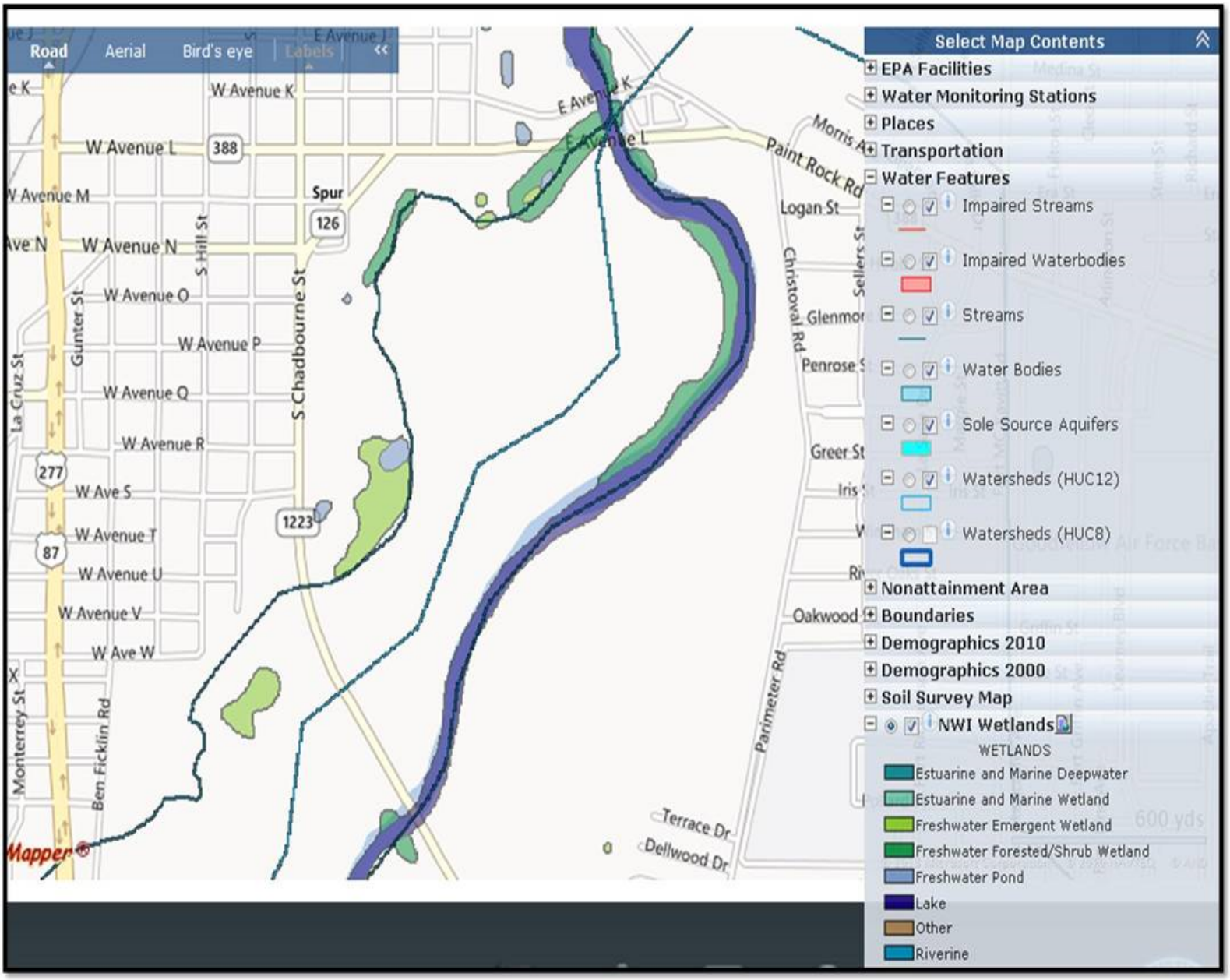
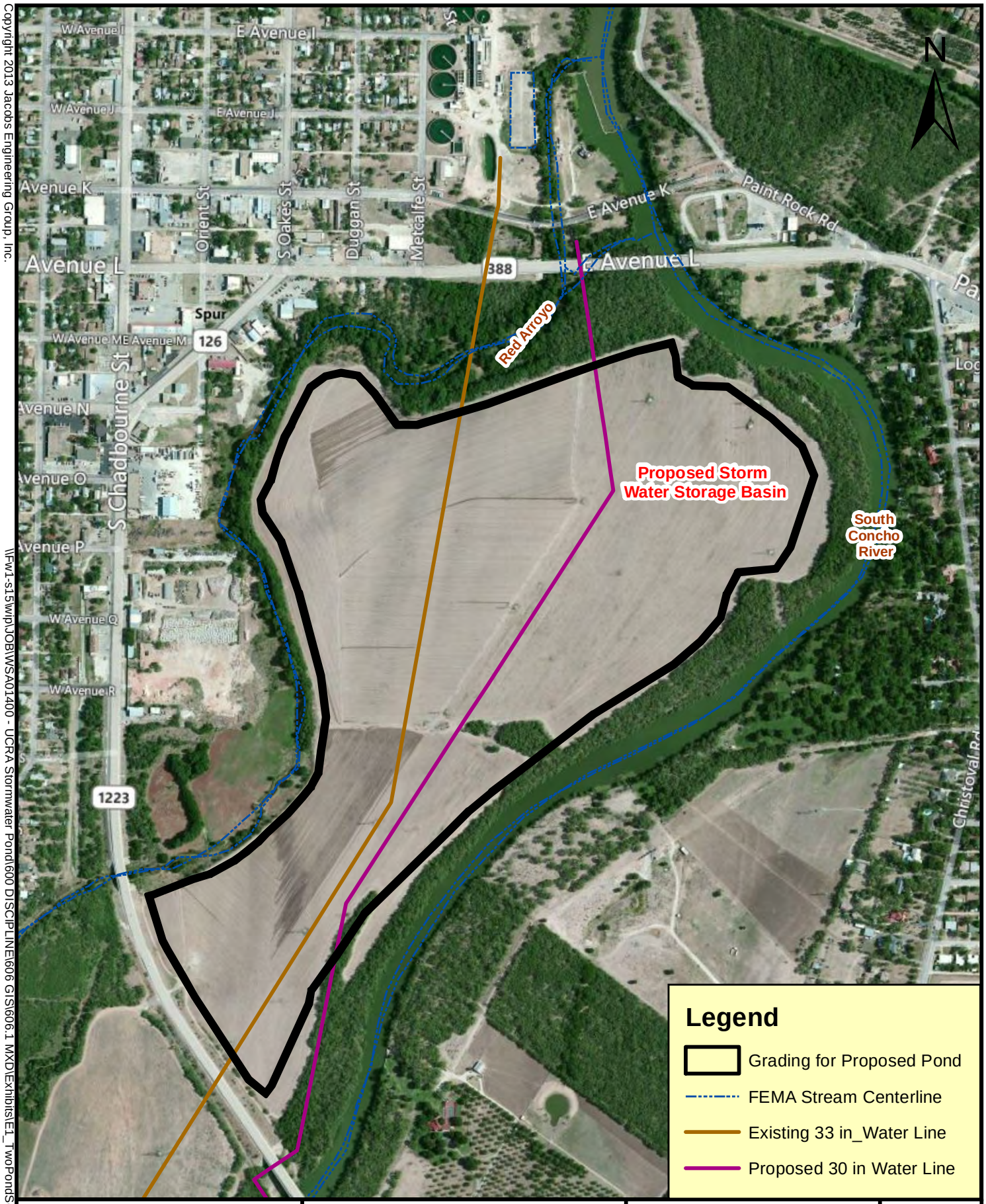


EXHIBIT 7: WETLANDS FROM EPA WEBSITE

EXHIBIT 10

Copyright 2013 Jacobs Engineering Group, Inc.



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

JACOBS®

Location of water lines and Basin-1

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

Source: ESRI® Data (2013)

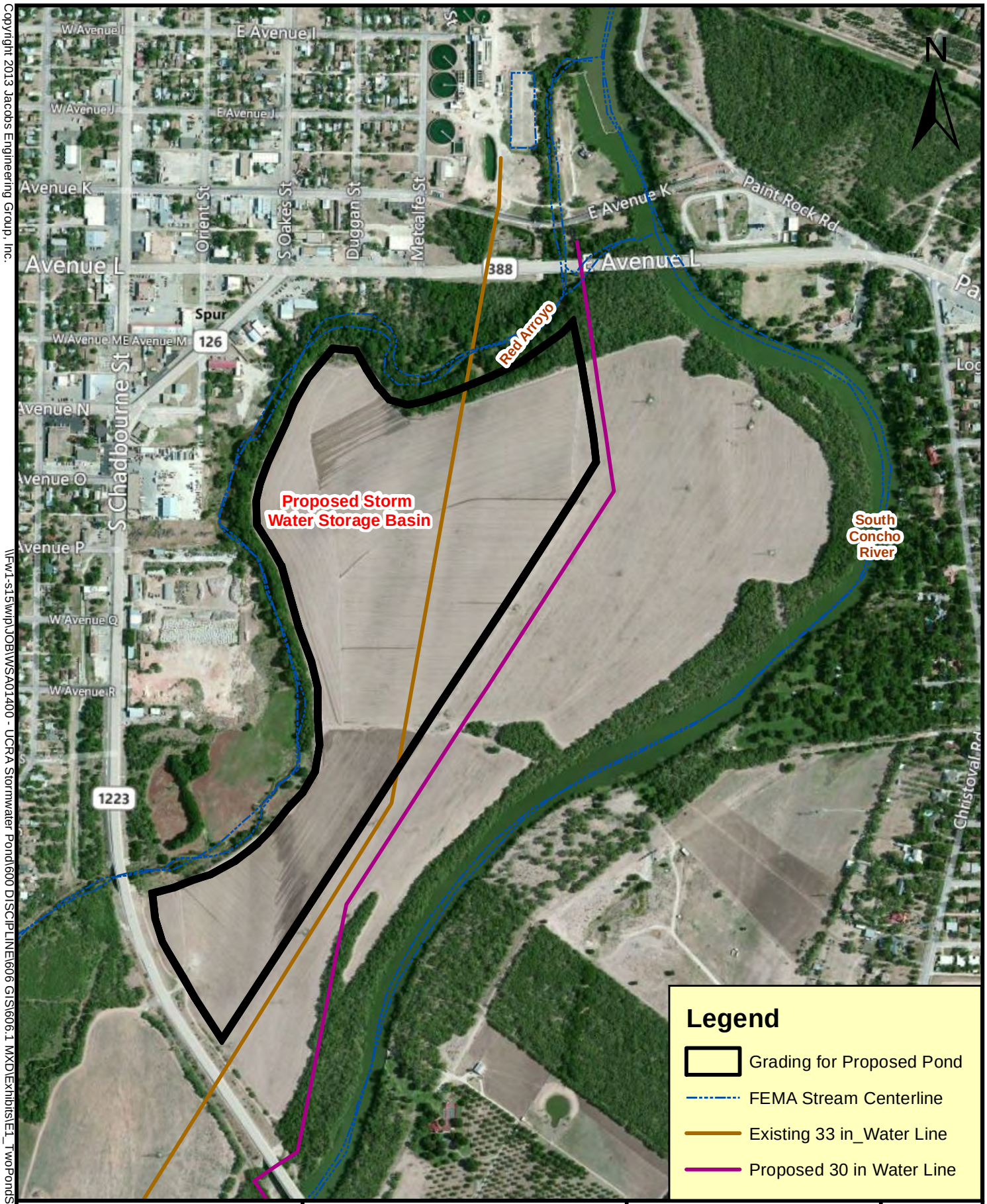
Scale: 0 300 600 Feet

Exhibit

10

EXHIBIT 11

Copyright 2013 Jacobs Engineering Group, Inc.



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

JACOBS®

Location of water lines and Basin-2

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

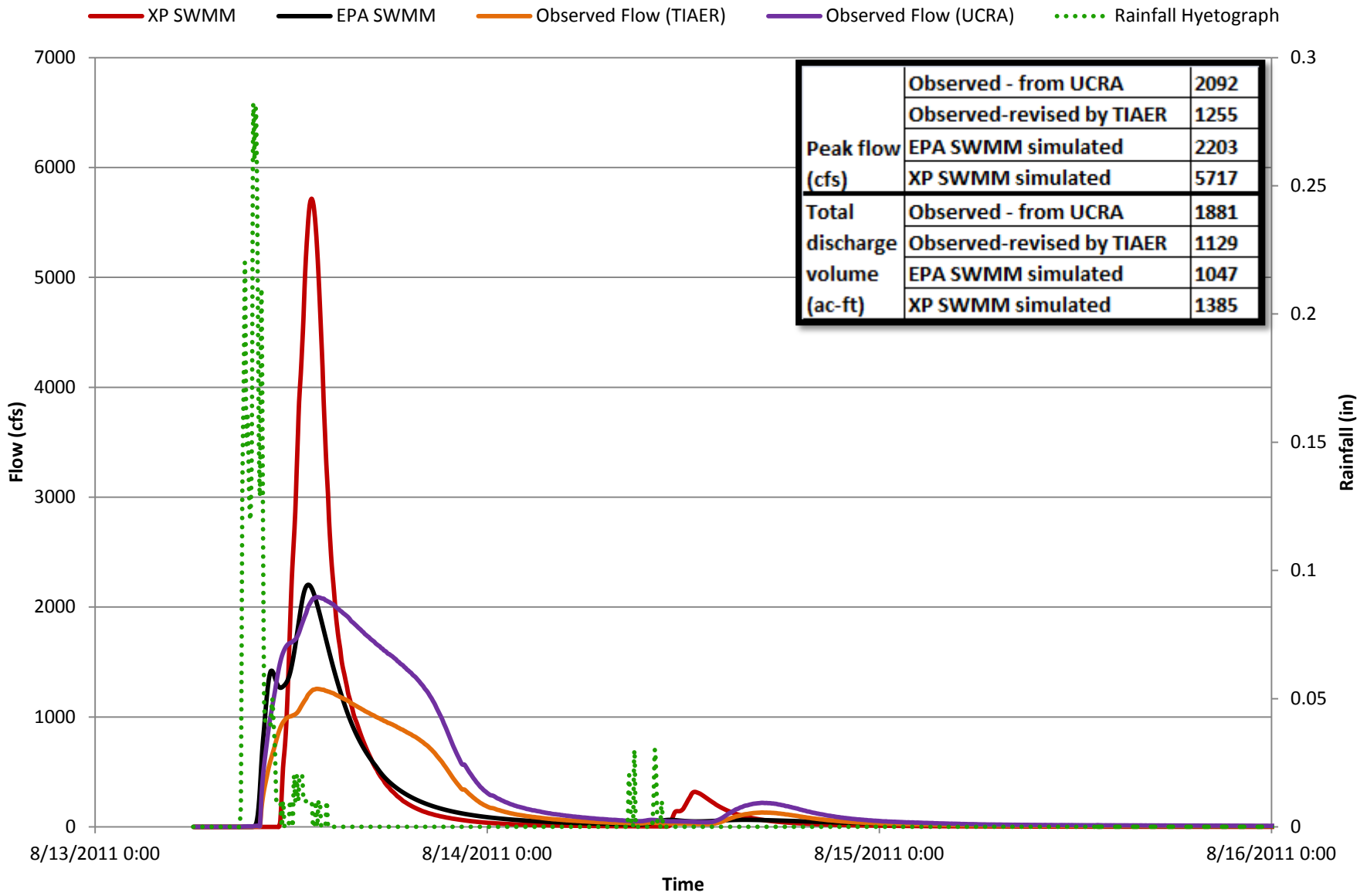
Source: ESRI® Data (2013)

Scale: 0 300 600 Feet

Exhibit 11

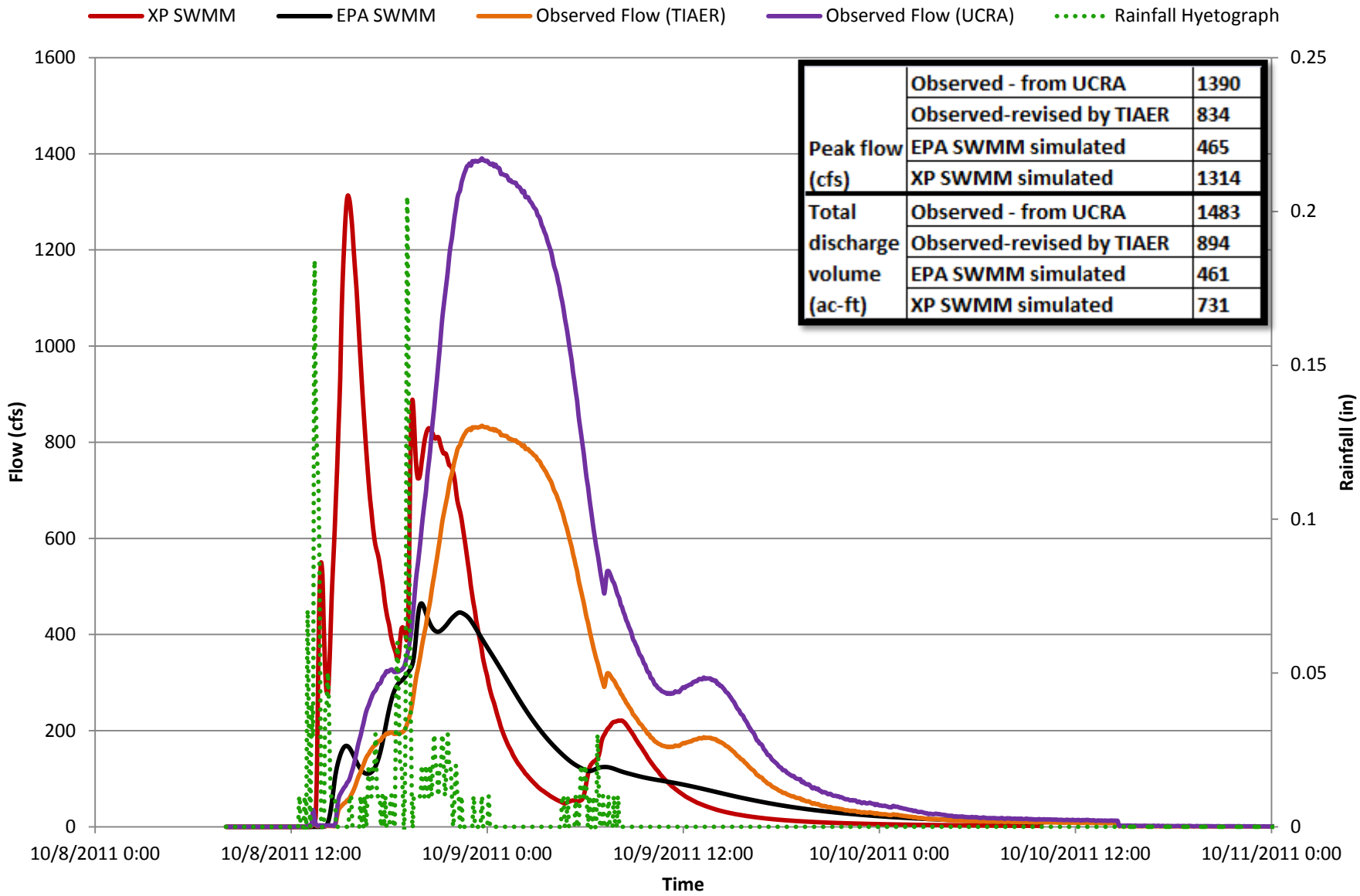
FIGURES

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



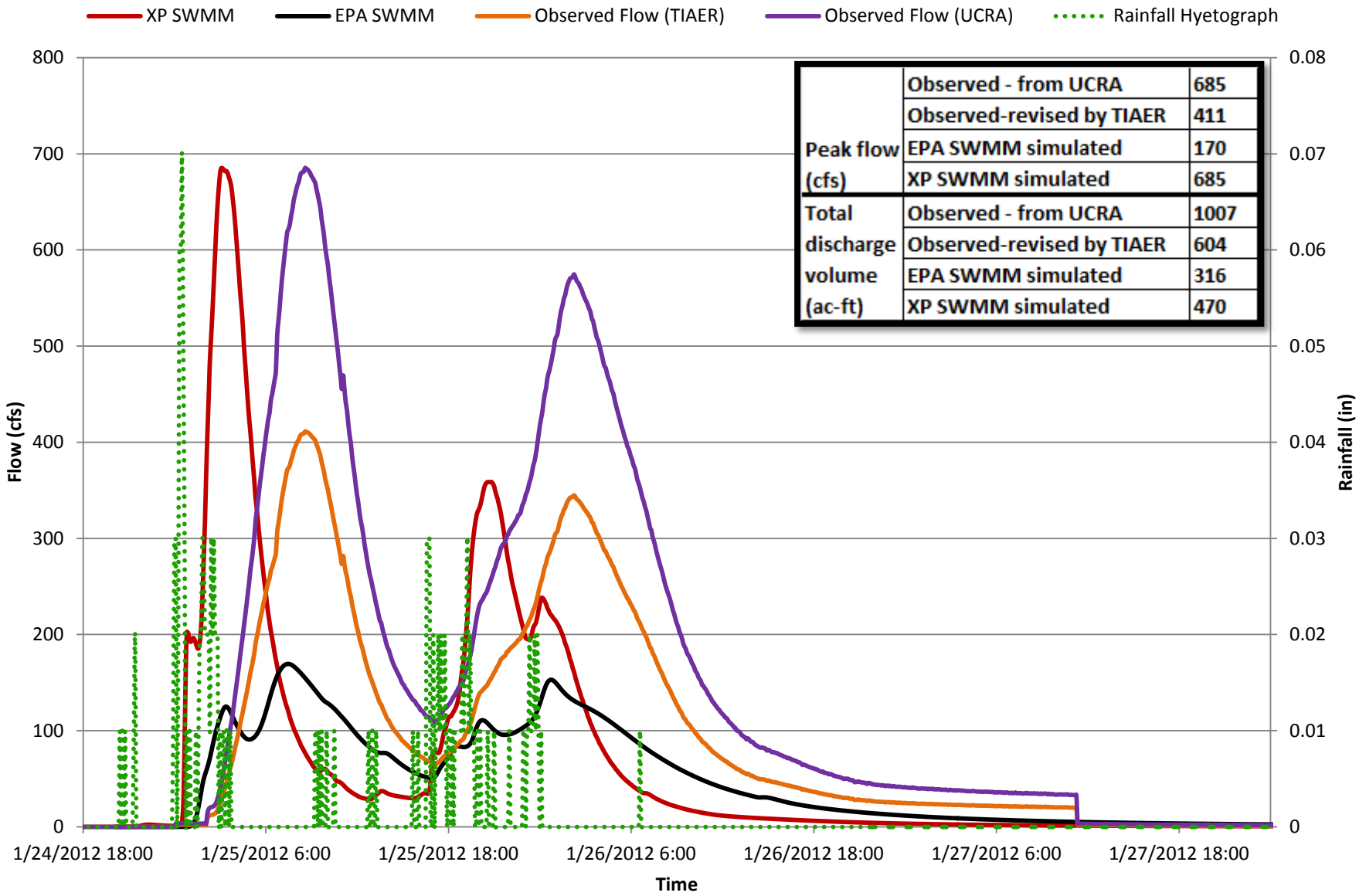
Peak flow (cfs)	Observed - from UCRA	2092
	Observed-revised by TIAER	1255
	EPA SWMM simulated	2203
	XP SWMM simulated	5717
Total discharge volume (ac-ft)	Observed - from UCRA	1881
	Observed-revised by TIAER	1129
	EPA SWMM simulated	1047
	XP SWMM simulated	1385

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



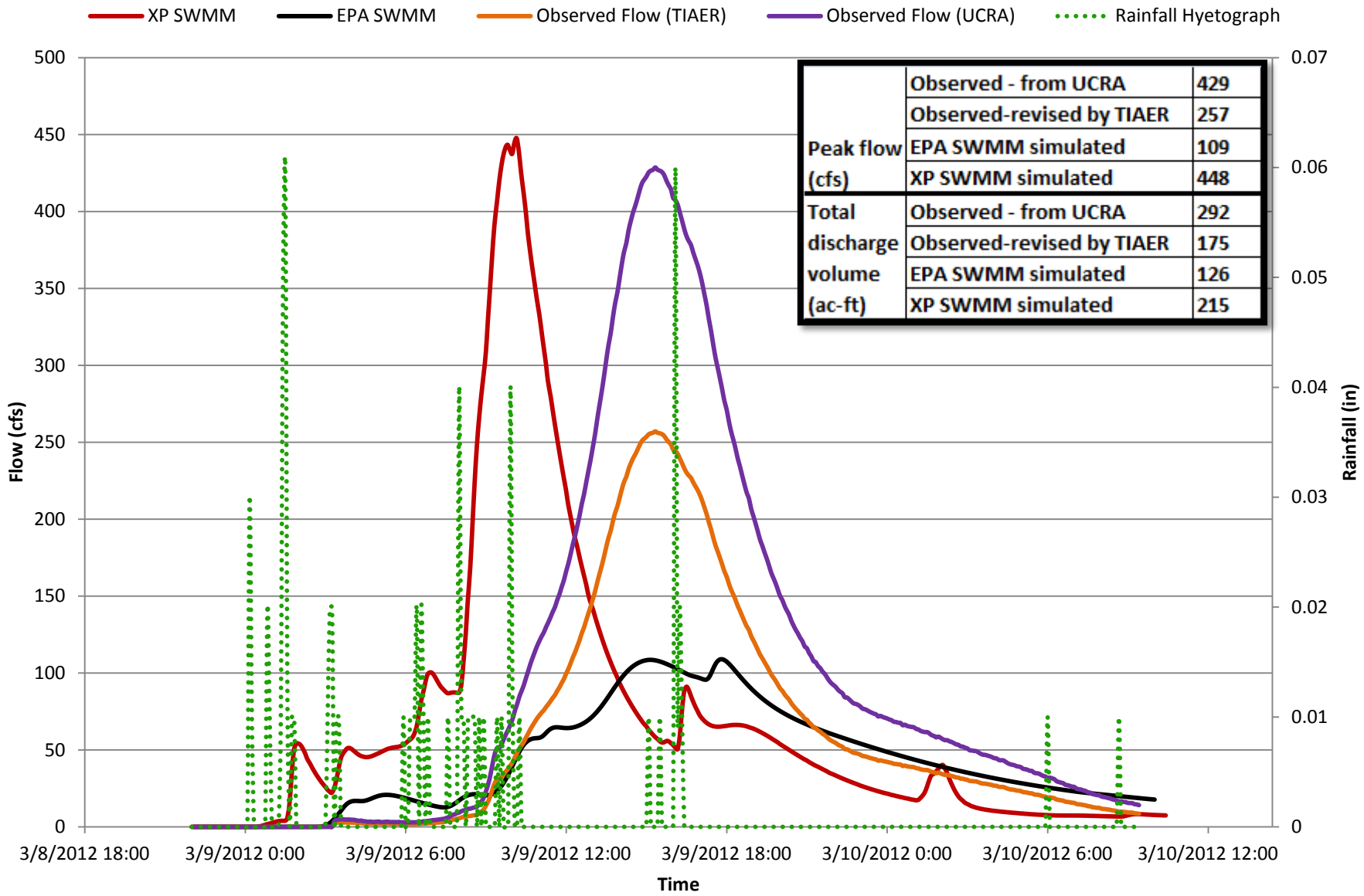
Peak flow (cfs)	Observed - from UCRA	1390
	Observed-revised by TIAER	834
	EPA SWMM simulated	465
	XP SWMM simulated	1314
Total discharge volume (ac-ft)	Observed - from UCRA	1483
	Observed-revised by TIAER	894
	EPA SWMM simulated	461
	XP SWMM simulated	731

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



Peak flow (cfs)	Observed - from UCRA	685
	Observed-revised by TIAER	411
	EPA SWMM simulated	170
	XP SWMM simulated	685
Total discharge volume (ac-ft)	Observed - from UCRA	1007
	Observed-revised by TIAER	604
	EPA SWMM simulated	316
	XP SWMM simulated	470

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

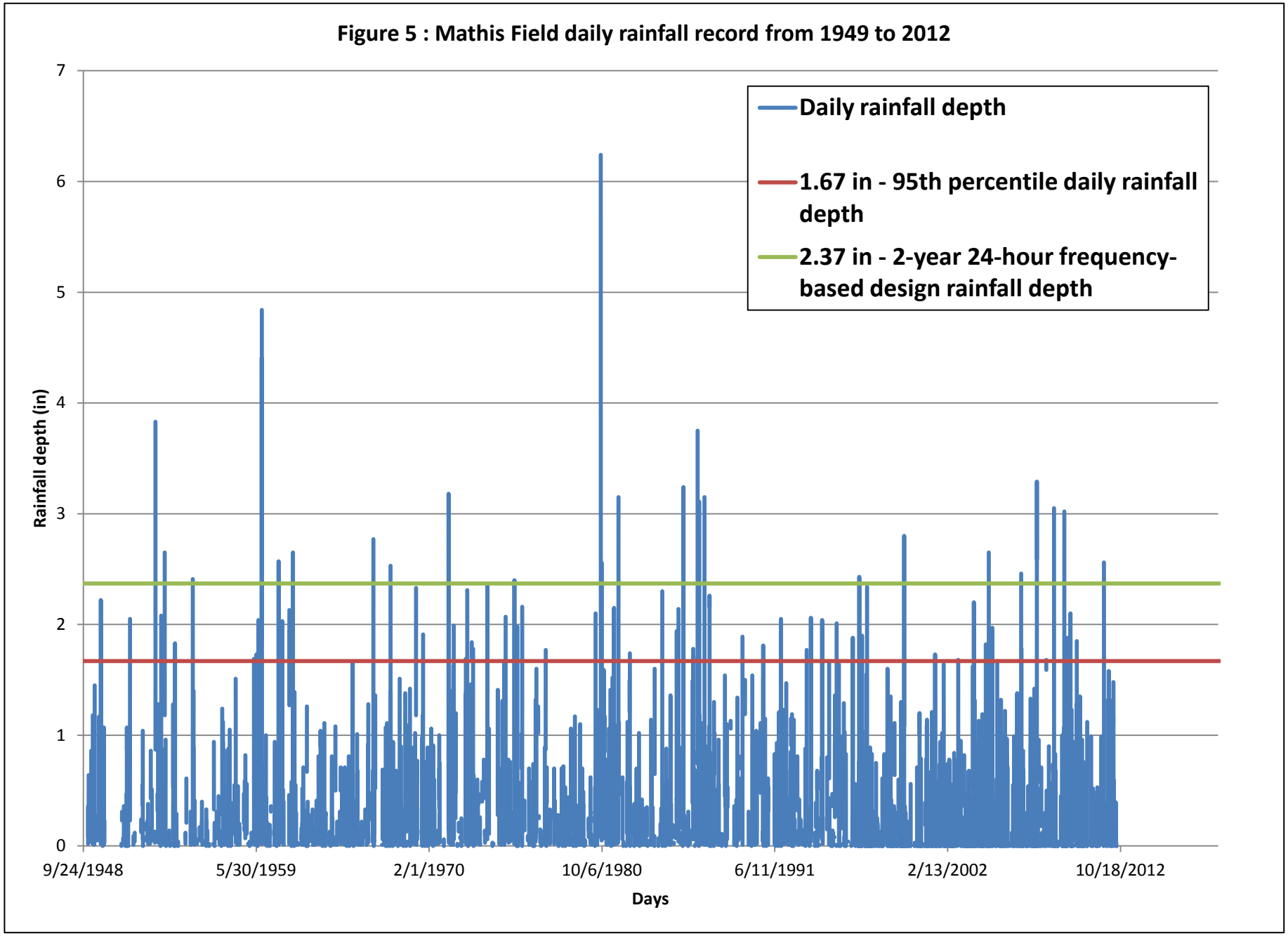


Figure 5 : Mathis Field daily rainfall record from 1949 to 2012
Daily rainfall depth
1.67 in - 95th percentile daily rainfall depth
2.37 in - 2-year 24-hour frequency-based design rainfall depth
Rainfall depth (in)
0
1
2
3
4
5
6
7
9/24/1948
5/30/1959
2/1/1970
10/6/1980
6/11/1991
2/13/2002
10/18/2012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

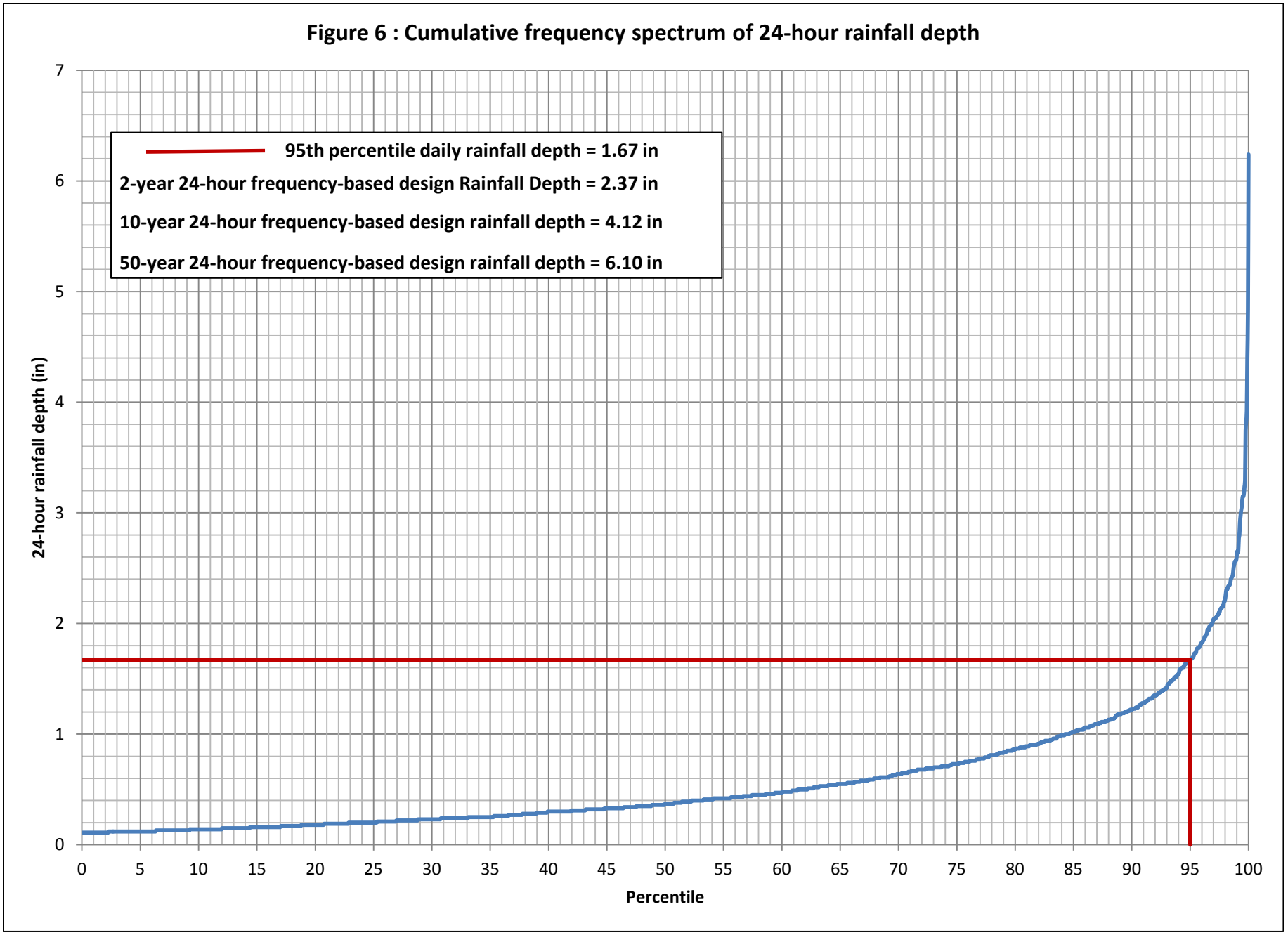


Figure 6 : Cumulative frequency spectrum of 24-hour rainfall depth
95th percentile daily rainfall depth = 1.67 in
2-year 24-hour frequency-based design Rainfall Depth = 2.37 in
10-year 24-hour frequency-based design rainfall depth = 4.12 in
50-year 24-hour frequency-based design rainfall depth = 6.10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

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

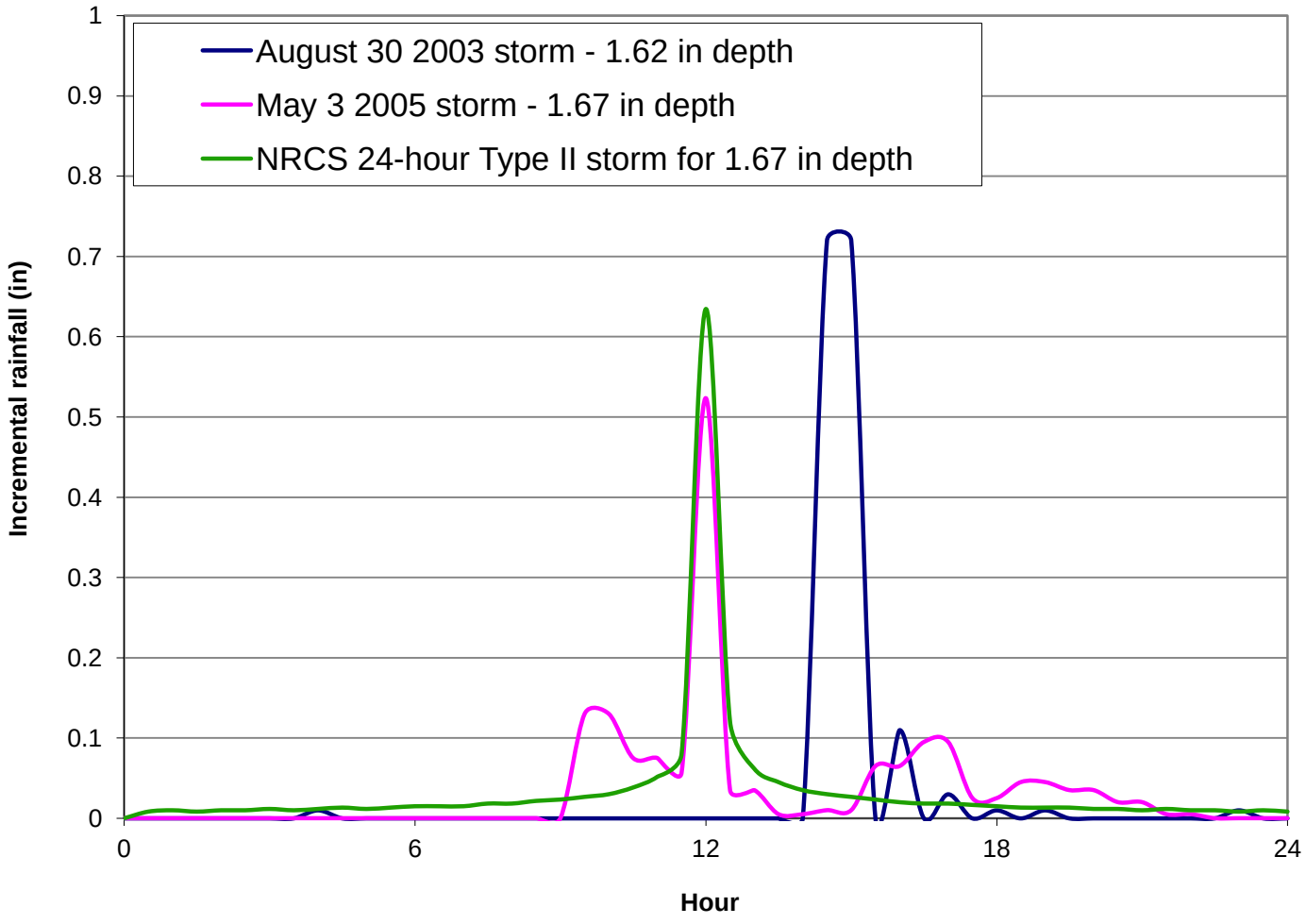
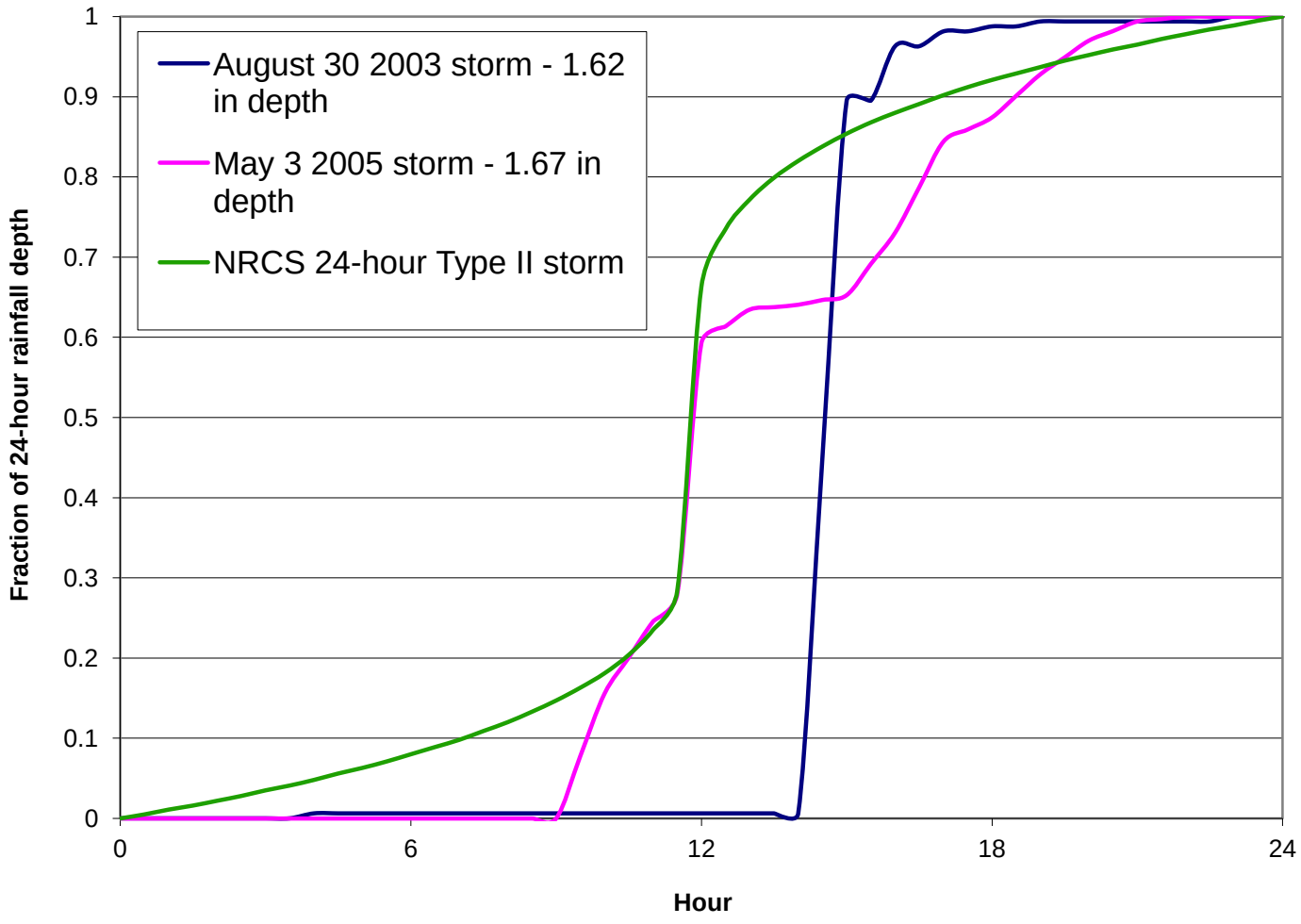



Figure 8 : NRCS 24-hour Type II rainfall mass curve compared to the observed mass curves of selected historical storm events