Appendix D

Hydrology and Hydraulics for Spillway and Flood Plain Analysis

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INTRODUCTION

In order to insure that future water supply needs are met, the Brazos River Authority (BRA) requested that the U.S. Army Corps of Engineers (USACE) perform a systems assessment of the USACE lakes in the Brazos River Basin to determine potential water availability as a function of changes in conservation and flood control storage in each of the lakes (reallocation). In Phase I of the Brazos Systems Assessment, a period of record analysis was done for each of the lakes assuming several different conservation pool elevations for each lake. Dependable yield curves were computed for each lake. The yield curves, analysis and recommendations were published in the Brazos Systems Assessment Interim Feasibility Study report for Phase I. Based on the report, the Project Delivery Team (PDT), which included BRA, recommended that the study proceed to Phase II, with Aquilla Lake being the chosen project to analyze in detail with respect to a change in conservation and flood control storage (raising the top of conservation pool).

The Hydrology and Hydraulics Branch of the Fort Worth District, US Army Corps of Engineers participated in the Phase II analysis, and provided hydrologic and hydraulic data results to the other members of the PDT to the determine the consequences of a pool raise with respect to each one's area of expertise.

The first major task in the hydrologic and hydraulic (H&H) analysis was to determine the dependable yield for the existing conservation pool and each of the three proposed alternative conservation pools based on the historical period of record. For the Phase II analysis, the existing period of record model was updated to include the most recent volumetric and inflow data for Aquilla Lake. The yield analysis provides an indication of how much water would be available for conservation use for each of the pool alternatives. The yield analysis was done by the Reservoir Control Section which is part of the Hydrology and Hydraulics Branch. The three pool alternatives analyzed in detail were chosen by the PDT. The task also included determining what effects the pool raises would have on reservoir control operations. This part of the analysis, along with the yield study, is described in Appendix K. The pool elevations analyzed were 537.5 (existing), 540.0, 542.0, and 544.0 feet.

The second major task in the hydrologic and hydraulic analysis was to determine what effect, if any, proposed changes in conservation storage and corresponding decreases in available flood control storage would have on the frequency and extent of downstream and upstream flooding. This part of the analysis was done by the Hydrology and Hydraulic Design Section. It also included determining the effects that more frequent emergency spillway overtopping might have on spillway erosion and stability. This part of the analysis was done in conjunction with the Geotechnical Section and is described in more detail in the Geotechnical Appendix.

Appendix D, Hydrology and Hydraulics for Spillway and Flood Plain Analysis, covers the hydrologic and hydraulic analysis for the alternative selection part of Phase II. Additional analysis will be added in the future for the selected alternative once that selection is made, adequately vetted, and agreed to by the project delivery team and sponsor.

HYDROLOGIC AND HYDRAULIC ANALYSIS

The H&H analysis requirements were:

1) Determine changes in maximum upstream and downstream flood plain elevations for different frequency events for each conservation pool alternative.
2) Provide water surface elevations and flooding durations for downstream damage computations for each alternative for economic analysis.
3) Provide downstream delineation maps for each alternative to show changes in flood frequencies resulting from more frequent spillway overtopping.
4) Provide upstream delineations maps for the selected alternative to show changes in frequency flooding resulting from increased conservation storage and decreased flood control storage.
5) Compute enveloping curves to determine upstream extent of effects of conservation pool and flood pool reallocation for selected alternative.
6) Provide spillway frequency hydrographs for detailed SITES analysis of potential spillway erosion and associated risk of failure for each alternative.

HYDROLOGIC ANALYSIS

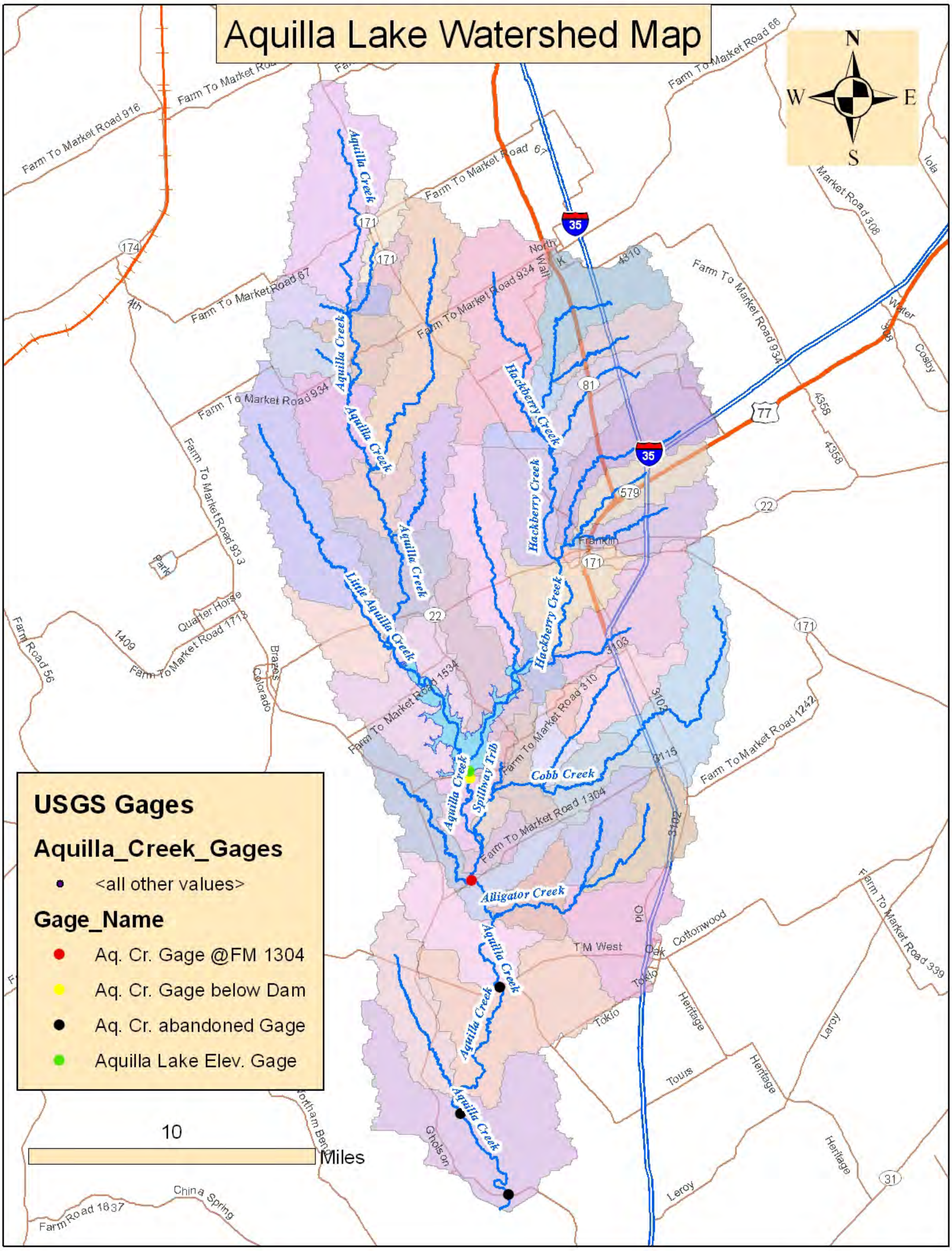
Watershed Description

The largest portion of the Aquilla Creek watershed is located in Hill County, Texas, with small portions in McLennan and Johnson Counties. The Aquilla Creek watershed is located approximately 25 miles north of Waco, Texas, and just southwest of Hillsboro, Texas. Aquilla Lake is located near the center of the watershed, with Aquilla Lake Dam being located at the confluence of Hackberry Creek with Aquilla Creek. The total drainage area of this watershed is 407.93 square miles and the drainage area at Aquilla Lake Dam is 254.52 square miles. The drainage area calculations are a result of a Geographic Information Systems (GIS) analysis process and are not official. These drainage areas are calculated and used within the HEC-HMS model for hydrologic calculations. The official Aquilla Lake drainage area is 255 square miles. Elevations within this watershed range from about 860 to 385 feet National Geodetic Vertical Datum (NGVD). The Aquilla Creek watershed drains in a southerly direction. The average stream slope for the upper reach of Aquilla Creek (above Aquilla Lake) is about 17.6 feet per stream mile and 17.8 feet per stream mile for Hackberry Creek. The average stream slope for lower Aquilla Creek (below Lake Aquilla Dam) is about 15.5 feet per stream mile.

This watershed includes five major tributaries to Aquilla Creek including: Hackberry Creek (128.96 square miles-includes Little Hackberry Creek), Little Aquilla Creek (25.28 square miles), Little Hackberry Creek (26.55 square miles), Cobb Creek (39.46 square miles) and Alligator Creek (30.93 square miles), plus several smaller tributaries.

The watershed is mostly a rural watershed with scattered development and a few small towns. Interstate Highway 35W runs through the eastern portion of the watershed. A general watershed map with streams and subbasins is shown in Figure 1.

Figure 1 - Watershed Map



USGS Gage Sites

The USGS currently maintains one stream flow gage (site number 08093360) and one stage gage (site number 08093350) recorder on Aquilla Lake. Records start in 1980 and continue to the present for site 08093360. The drainage area at these gages is about 255 square miles. Closure for Aquilla Lake Dam began in March 1982 and the dam was completed in January 1983. A USGS gage at FM 1304 (site 08093500) located downstream of Aquilla Lake Dam was in operation from 1936 to 2001. The gaged drainage area was about 308 square miles prior to construction. The location of all USGS gage sites is shown on the Aquilla Lake Watershed Map.

Flood History

Major flood events occurred on the Aquilla Creek watershed in August 1887, January 1936, April 1942, May 1944, May 1958, January 1961, May 1965, April 1966, May 1968, July 1976, June 1981, December 1997, June 2000, March 2007, June 2007, and October 2009.

The topography of the Aquilla Creek watershed, the character of the soil, and the nature of the rainfall in the area are conducive to rapid runoff and sharp-crested hydrographs. Such floods occur frequently at almost any time of the year. Based on historical and recorded flood data, the maximum known flood in the vicinity of the gaging station on Aquilla Creek near Aquilla, Texas, occurred in August 1887. The stage for that flood was estimated to be 34 feet; however, the discharge was not determined. The flood of September 27, 1936 was the highest subsequent to 1887, and reached a stage of 33 feet. The peak discharge for this flood was 74,200 cfs as determined by the USGS by indirect measurement. The May 1968 flood, with a peak discharge of 40,200 cfs, reached a maximum stage of 30.3 feet. Additional annual peak flows from 1936 to 1980 are listed in Table 1.

Stream Gage Frequency Analysis

A frequency analysis was prepared using the HEC-SSP version 2.0 computer program for the peak annual flow in cfs for water years 1936 to 1981 for USGS gage site 08093500 at FM 1304 which is located downstream of the Aquilla Lake Dam . This is for the period of record prior to the closure of Aquilla Lake Dam. This analysis was done to determine reasonable peak frequency inflow discharges for Aquilla Lake at Aquilla Lake Dam. Therefore, the gage records after the closure of Aquilla Lake Dam could not be used because this would create a mixed record. Table 1 is a list of the Annual Peak flows that were analyzed. Figure 2 is a discharge frequency graph based on the same data. The frequency discharges were developed using Bulletin 17B discharge frequency analysis guidelines. Bulletin 17B was authored by the Interagency Advisory Committee on Water Data of which the Corps of Engineers is a member, and describes the methodology adopted by the Corps of Engineers for frequency analysis for stream gage data. The algorithms used in HEC-SSP are based on Bulletin 17B. The resultant discharge frequency values were used to establish target frequency inflow values for Aquilla Lake. The confidence limits provide a measure of the uncertainty of the computed frequency discharges. The 5% and 95% bands are standards normally used for frequency analysis and provide a range of values that cover 90% of the values from the given gage data set. The wider the range of values between 5% and 95%, the more uncertainty there is in the computed discharges.

Table 1 - Annual Peak Flows – Aquilla Creek USGS Gage FM 1304

USGS Gage 08093500 Aquilla Creek near Aquilla, Tx D.A. 308 Square Miles						
Abandoned in 2001						
Year	Date	Annual Peak Discharge (CFS)		Year	Date	Annual Peak Discharge (CFS)
1	2-Jan-1936	74,200		23	6-Oct-1959	8,240
2	20-Jun-1939	9,860		24	9-Jan-1961	16,700
3	7-Apr-1940	8,690		25	10-Jun-1962	13,400
4	3-Feb-1941	8,560		26	10-Oct-1962	8,770
5	26-Apr-1942	16,000		27	25-Sep-1964	2,470
6	9-Apr-1943	6,910		28	17-May-1965	17,900
7	3-May-1944	34,200		29	27-Apr-1966	21,700
8	12-Jul-1945	14,700		30	12-Apr-1967	4,450
9	14-Mar-1946	7,060		31	11-May-1968	40,200
10	10-Apr-1947	8,440		32	8-May-1969	32,600
11	12-May-1948	8,260		33	26-Feb-1970	3,260
12	28-May-1949	3,540		34	31-May-1971	11,700
13	18-Apr-1950	3,580		35	21-Oct-1971	16,800
14	13-Jun-1951	8,690		36	25-Apr-1973	12,700
15	25-May-1952	12,000		37	14-Oct-1973	5,180
16	20-Dec-1952	12,900		38	1-Nov-1974	16,700
17	20-May-1954	4,660		39	5-Jul-1976	27,200
18	20-May-1955	4,850		40	28-Mar-1977	14,600
19	2-May-1956	7,550		41	13-May-1978	2,740
20	24-Apr-1957	10,800		42	30-May-1979	7,190
21	4-May-1958	18,500		43	17-May-1980	8,750

Figure 2 - Discharge Frequency Graph – Aquilla Creek USGS Gage FM 1304

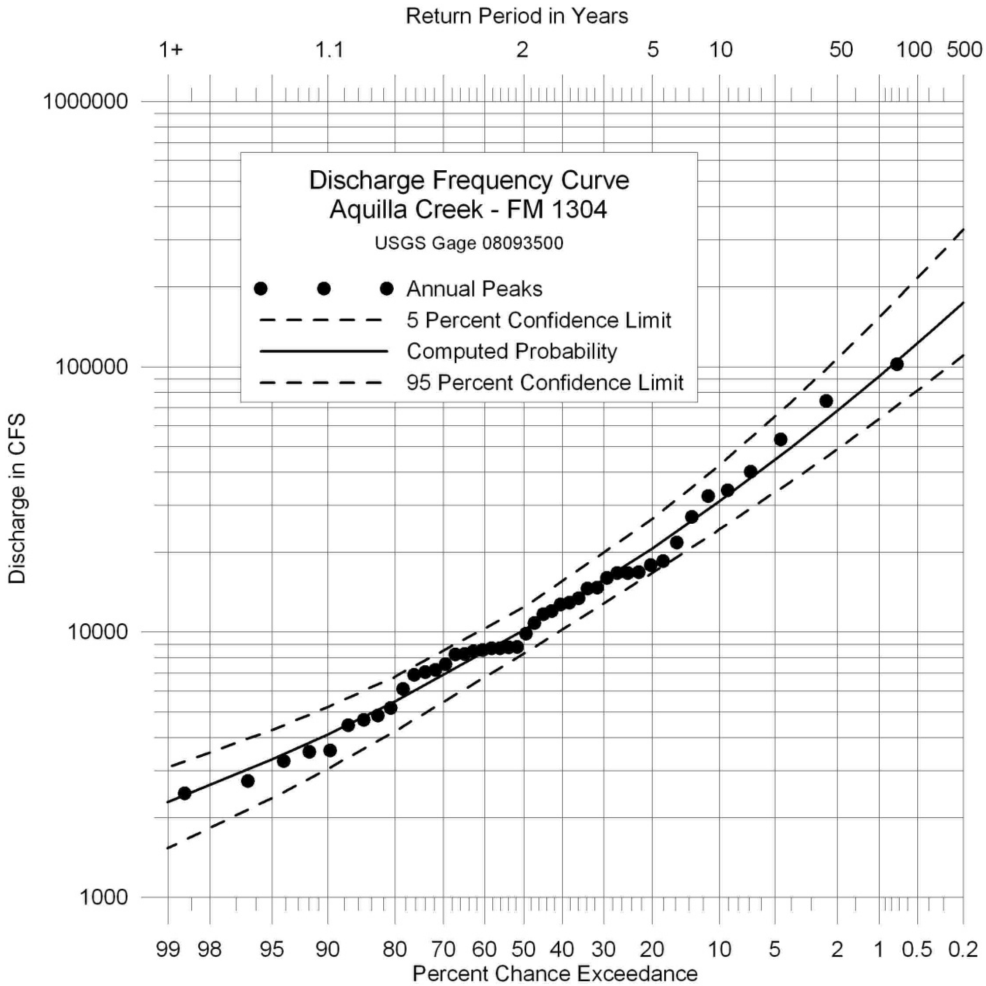


Table 2 is a tabular representation from the frequency analysis shown in Figure 2 (computed and expected probability) for the peak annual flows for water years 1936 to 1981 for USGS Gage 08093500 at FM 1304. HEC-SSP calculates both expected and computed probability. The expected probability adjustment is an attempt to adjust the computed discharges for the uncertainty caused from the shortness of the record; however, these were not used for this study, and are shown for informational purposes only. The discharges at the Aquilla Lake dam site shown in the table are based on computed probability flows from the gage site projected upstream to the dam site using the square root of the drainage area ratio. A projected discharge based on a simple drainage area ratio times the computed probability flows is also shown in Table 2. The use of either ratio is accepted hydrologic practice for projecting peak flows to a different location from the gage. Both sets of projected peak flows were used as part of the process for calibrating the HEC-HMS model results at the dam. When calibrated, the HEC-HMS computed peak flows for the 2-, 5-, 10-, 50-, and 100-year return periods were within the range (both square root of the drainage area and simple ratio of drainage area) of projected values from the gage analysis. The 25-year computed peak discharge value was 0.8% higher, the 250-year value was 0.8% lower, and the 500-year value was about 12% lower.

Table 2 - Computed Probability Discharges for Aquilla Creek USGS Gage at FM 1304 and Dam Site

Flood Return Period	Percent Chance Exceedance	Computed Probability	Expected Probability	Based on Square Root Drainage Area Ratio	Based on Drainage Area Ratio
		USGS Gage - FM 1304		Gage Flows Projected Upstream to Dam Site (Computed Probability)	
(Years)		Discharge (cfs)	Discharge (cfs)	Discharge (cfs)	Discharge (cfs)
500	0.2	174,243	221,567	158,490	144,161
250	0.4	133,572	161,388	121,496	110,512
100	1.0	92,027	105,109	83,707	76,139
50	2.0	68,276	75,193	62,103	56,489
25	4.0	49,627	52,986	45,140	41,059
10	10.0	31,094	32,180	28,283	25,726
5	20.0	20,637	20,986	18,771	17,074
2	50.0	10,148	10,148	9,231	8,396
	80.0	5,481	5,417	4,985	4,535
	90.0	4,116	4,031	3,744	3,405
	95.0	3,307	3,204	3,008	2,736
	99.0	2,286	2,159	2,079	1,891
USGS Gage - D.A.= 307.63 Sq. Mi. **Dam Site - D.A. = 254.52 Sq. Mi.**					

Hydrologic Model Development

A watershed runoff model was developed utilizing HEC-HMS, version 3.5, software. Geo-HMS running on an Environmental Services and Resources Institute (ESRI) ArcGIS 9.3 base was used to delineate the subbasins based on a 10 meter ArcView Grid file (Digital Elevation Model (DEM)) which was developed from the USGS 10 meter DEM data.

<u>Stream Lines and Subbasins</u>

For the Aquilla Creek watershed study, the stream lines (flow paths) were developed from digital USGS 7-1/2 minute topographic data, digital aerial photographs, and from detailed mapping data where it was available. Geo-HMS was used to generate the subbasin parameters of drainage area, stream length, stream length from the subbasin outflow point to the subbasin centroid, and stream slope.

The Aquilla Creek watershed was subdivided into 54 subbasins, requiring the development of routing data for approximately 25 reaches, in order that discharges could be computed for all locations required to develop profiles for the study area. Subbasins were created for all tributaries being studied in detail, for all major tributaries, and at or near gage locations. With the model calibration done, the relatively large number of subbasins used for the watershed model would tend to provide results that would more accurately reflect flooding expected in the watershed. The subbasins and junctions were defined to obtain detailed flow information (flood hydrographs) at all points of interest in the watershed. These points included the confluence of Aquilla and Hackberry Creeks with all tributaries and Aquilla Lake. A 15 minute computation time interval was used in the model to provide adequate detail (shaping) of the unit hydrograph applied at the smaller subbasins in the analysis. A 15 minute time interval is needed to accurately capture the peak flow value because of the relatively quick flood response time occurring on Aquilla Creek and Hackberry Creek into Aquilla Lake. Although, the overall study analysis is normally not significantly affected by the computational time interval used in the analysis, the actual peak flow may not be captured if a longer time interval is used.

Rainfall

Theoretical point rainfall data was developed using the National Weather Service (NWS) Technical Paper No. 40 (TP40) and blended with the National Weather Service's Hydro 35 for rainfall durations up to one hour. The 0.4% and 0.2% annual chance exceedance (ACE) rainfall, commonly referred to as the 250-year and 500-year rainfall, were computed by extrapolation of this data. The hypothetical precipitation array is presented in Table 3.

Table 3 - Point Rainfall Depths (inches)

	Annual Exceedance Probability							
	50%	20%	10%	4%	2%	1%	0.4%	0.2%
	Recurrence Interval (years)							
Storm Duration	2	5	10	25	50	100	250	500
5 minutes	0.50	0.58	0.65	0.74	0.82	0.89	0.99	1.06
15 minutes	1.08	1.25	1.38	1.58	1.73	1.89	2.10	2.25
1 hour	2.00	2.49	2.83	3.33	3.72	4.10	4.63	5.02
2 hours	2.37	3.08	3.55	4.20	4.72	5.22	5.88	6.41
3 hours	2.59	3.42	4.04	4.81	5.31	5.87	6.62	7.23
6 hours	3.07	4.07	4.83	5.73	6.44	7.23	8.09	8.88
12 hours	3.54	4.85	5.76	6.75	7.68	8.67	9.78	10.75
24 hours	4.12	5.55	6.70	7.83	8.86	9.93	11.24	12.34

Figure 15 from the NWS TP40, Depth-Area-Duration curves, was used to adjust the point rainfall to representative average values over the contributing watershed size at each point of interest for the 2-year through 100-year rainfall values. The NWS TP40, Figure 15 adjustments are a built-in function and are made in HEC-HMS based on the drainage area specified by the user. This adjustment is one of the standard methods used to adjust point rainfall values to represent average values for larger areas. A 24-hour storm duration for each of the frequency related storm events was used. The most intense rainfall was centered at the middle of the storm. The 24-hour storm was used to account for all the runoff-producing rainfall. The 24-hour duration storm is the standard storm pattern being used for watersheds of this size. A test of a 48 and 96 hour duration storm (rainfall values from NWS TP 49) for the 100-year flood at Lake Aquilla Dam was made and there was only a 0.03% difference in computed peak discharges with the storm centered over the twelfth hour and a maximum of about 3% when centered at the middle of the storm. Therefore, the difference was considered insignificant and the 24-hour storm was used for this analysis.

Initial Abstractions and Infiltration Rates

The rainfall loss values were assumed to vary with both soil type and the frequency of each storm event. Soil type for each subbasin was assessed by digitally overlaying the watershed map and the Natural Resources Conservation Service (NRCS) SSURGO digital soils map. The SSURGO digital soils maps were developed to cover each county. The Aquilla Creek watershed covers portions of Hill, McLennan and Jones Counties, Texas. The number of acres of each soil type for each subbasin was computed using the ArcMap Interpolate tool with the digital watershed and SSURGO soils maps. A lookup table was created for each county with soil type names and the percent sand determined using the Fort Worth District method of determining percent sand in watersheds based on the soil permeability rates. These values were further adjusted based on the moisture holding capacity of the first 48 inches of the soil. The table created from the ArcMap interpolate tool was then joined to the lookup table and sorted by subbasin. This combined table included the subbasin names, soil type names, acres for each soil type, and the assigned percent sand. The combined table was then imported into an Excel spreadsheet, and an average percent sand for each subbasin was computed.

The deficit and constant loss rate method in HEC-HMS was used to compute losses. Loss rates were based on the Fort Worth District standard default initial and constant loss rates, which vary for each frequency flood event for 100% sand and 0% sand (0% being the same as clay). The final, or adopted, loss rates used for the frequency storms in the HEC-HMS model were computed for each subbasin using the average percent sand values, and were further adjusted to better fit the frequency analysis results. These updated loss rates also compared favorably to the loss rates required to calibrate the five storms described in the model calibration section. It should be noted that initial and constant loss rates are the first and third variables in the deficit and constant loss rate method. The method contains an additional term which is the maximum deficit, which is used for calibration of storms lasting more than 8 to 10 days in length. Consequently, the same loss rates can be used for 24-hour frequency storms as well as calibration storms lasting several weeks. Tables 4A and 4B show the minimum and maximum loss rates for the frequency based storms and for each of the calibration storms.

Table 4A – Minimum and Maximum Loss Rates for Frequency Storms

Adopted Loss Rates			
Description	**Initial Deficit (Inches)**	**Maximum Storage (Inches)**	**Constant Rate (Inches/Hour)**
2 - Year			
Min	1.3359	2.0859	0.1656
Max	1.9588	2.7088	0.2279
5 - Year			
Min	1.4374	2.1874	0.1857
Max	2.1215	2.8715	0.2541
10 - Year			
Min	1.3765	2.1265	0.1736
Max	2.0239	2.7739	0.2384
25 - Year			
Min	0.9609	1.7109	0.1209
Max	1.3884	2.1384	0.1566
50 - Year			
Min	0.7453	1.4953	0.0679
Max	0.9151	1.6651	0.1046
100 - Year			
Min	0.3144	1.0644	0.0359
Max	0.3908	1.1408	0.0726
250 - Year			
Min	0.1258	0.8758	0.0209
Max	0.1563	0.9063	0.0576
500 - Year			
Min	0.0377	0.7877	0.0139
Max	0.0469	0.7969	0.0506

Table 4B – Minimum and Maximum Loss Rates for Calibration Storms

Loss Rates from Storm Calibration Runs			
	Initial Deficit (Inches)	**Maximum Storage (Inches)**	**Constant Rate (Inches/Hour)**
Dec 97-Jan 98	**(slightly greater than the 50-Year loss rates)**		
Min	0.6811	1.4311	0.0659
Max	0.8625	1.6125	0.1026
May-June 2000	**(slightly greater than the 50-Year loss rates)**		
Min	0.6811	1.4311	0.0659
Max	0.8625	1.6125	0.1026
Mar-Apr 2007	**(slightly less than the 25-Year loss rates)**		
Min	0.9609	1.7109	0.1209
Max	1.3884	2.6384	0.1576
May-Jul 2007	**(slightly less than the 25-Year loss rates)**		
Min	0.9609	1.7109	0.1209
Max	1.3884	2.6384	0.1576
Oct-Nov 2009	**(between 25 and 50-Year loss rates)**		
Min	0.8247	1.5747	0.0934
Max	1.1087	1.8587	0.1301

Land Use

Land use was determined from the TNRIS 2006 and 2008 digital aerial photographs for McLennan and Hill Counties, Texas, by overlaying the subbasin map over the aerial photographs. The watershed lies mostly in a rural area with a few small communities and a few roads. The values for percent urbanization and percent impervious for each land use type were based on Fort Worth methodology described in "Effects of Urbanization on Various Frequency Peak Discharges" by Paul K. Rodman, October 1977.

Urbanization and Imperviousness

Values of percent urbanization and percent imperviousness were developed for each subbasin. Urbanization is the percentage of a subbasin that has been developed and improved with channelization and/or a storm collection network. It affects the Snyder's unit hydrograph lag time (t_p). Imperviousness is the percentage of a subbasin that is covered with impervious material and is hydraulically connected to the drainage network. It affects the volume of rainfall lost through interception and infiltration.

The urbanization and imperviousness values for each subbasin are based on the land use mentioned above. Each land use was assigned a value for urbanization and imperviousness and net values for each subbasin were derived by weighting the land uses within each subbasin. Urbanization and imperviousness values for each subbasin are presented in Table 5.

Development of Unit Hydrographs

Snyder's unit hydrograph method was used for consistency with previous studies in the region. The adopted unit hydrograph peaking coefficient, (CP640) value of 550 was determined by review of previous studies in the area and the five storm calibrations. The peak discharges and hydrograph shapes fit the recorded results reasonably well with a value of CP640 = 550 (c_p = 0.8594). The CP640 value affects the shape of the unit hydrograph peak, mostly for the upstream areas of the watershed model. The higher the CP640 value, the higher the peak would be. For the Aquilla Creek Watershed area, a CP640 value of 550 would fall in the range expected because of the stream slope and relatively quick peaking of the storms that have occurred. Also, because of the relatively large number of subbasins required for the study area, the CP640 value would have a minimal effect on the computed peak flows for the area of interest. Snyder's unit hydrographs were developed for each subbasin based on the specific physical measurements generated by Geo-HMS. These measurements are used in the standard equations for determining Snyder's lag time (t_p) which is a function of the length of the major stream (L), the distance from the subbasin outflow point to the location of the subbasin centroid (L_{ca}), the weighted slope (S_{st}) of the major stream that shows the best representation of the valley slope, and the percent urbanization. The Snyder's unit hydrograph lag time (t_p) was calculated for each subbasin using methodology described in the following reports.

1. "Synthetic Hydrograph Relationships, Trinity River Tributaries, Fort Worth-Dallas Urban Area" by T.L. Nelson, 1970.
2. "Effects of Urbanization on Various Frequency Peak Discharges" by Paul K. Rodman, October 1977.

The equation to calculate t_p is as follows:

$$\log(t_p) = 0.3833\log(L*Lca/(S_{st}^{0.5})) + (\%Sand/100*(\log1.81-\log.92)+\log.92) - (BW*\%Urban./100)$$

Where: t_p = Snyder's lag time
L = longest stream length within subbasin (miles)
Lca = distance along stream from subbasin centroid to outlet (miles)
S_{st} = stream slope over reach between 10% and 85 % of L
Sand = percentage sand (0 percent = all clayey and 100 percent = all sandy)
BW = log (t_p) bandwidth between 0% and 100% urbanization
Urban = percentage urbanization factor

Snyder's unit hydrograph lag times for each subbasin are presented in Table 5.

Table 5 – Aquilla Creek Watershed Unit hydrograph Data

					Table 5 : Unit Hydrograph Data						
Area	**Description**	**Area**	**L**	**Lca**	**U/S Elev.**	**D/S Elev.**	**Slope**	**Urban**	**% Sand**	**% Imp.**	**Tp**
Name		**Sq. Miles**	**feet**	**feet**	**85% feet**	**10% feet**	**Ft./Mi**	**(%)**			**hours**
AlCr-01	Alligator Creek	8.018	38,937	20,044	639.50	510.12	23.39	8	32.48	4.5	0.600
AlCr-02	Alligator Creek	11.684	31,843	15,623	609.40	490.04	26.39	7	24.44	4	0.630
AlCr-03	Alligator Creek	5.226	34,415	13,611	559.94	460.98	20.24	5	50.82	3	0.872
AlCrTr-01	Alligator Creek Trib	6.003	35,077	17,307	617.16	510.18	21.47	5	26.01	3	0.566
AqCr-01	Aquilla Creek	19.119	61,162	31,875	769.62	655.36	13.15	12	87.38	7	0.383
AqCr-02	Aquilla Creek	0.012	1,206	567	653.84	649.98	22.52	5	101.30	3	0.409
AqCr-03	Aquilla Creek	4.911	24,736	12,103	672.38	620.10	14.88	6	72.69	4	0.409
AqCr-04	Aquilla Creek	8.237	29,773	10,890	687.80	593.56	22.28	5	125.26	3	0.693
AqCr-05	Aquilla Creek	5.307	25,239	10,761	667.42	569.29	27.37	6	105.55	4	0.695
AqCr-06	Aquilla Creek	8.781	45,856	22,645	615.74	537.76	11.97	5	90.06	3	0.737
AqCr-07	Aquilla Creek	16.120	56,613	29,082	605.85	537.63	8.48	28	44.84	24.5	0.539
AqCr-08	Aquilla Creek	3.016	24,817	10,575	540.19	469.80	19.97	5	25.43	3	0.686
AqCr-09	Aquilla Creek	2.395	18,525	8,131	589.68	460.87	48.95	5	51.03	3	0.891
AqCr-10	Aquilla Creek	4.442	37,929	18,695	575.04	449.23	23.35	5	44.32	3	0.905
AqCr-11	Aquilla Creek	10.494	43,944	18,613	541.33	439.25	16.35	5	36.11	3	0.890
AqCr-12	Aquilla Creek	20.304	55,060	28,001	520.54	415.84	13.39	7	36.23	4	1.012
AqCr-13	Aquilla Creek	24.130	77,618	32,156	507.31	386.88	10.92	5	43.90	3	1.261
AqCrTr-1-01	Aquilla Creek Trib-1	3.195	21,356	12,451	767.20	656.23	36.58	5	103.53	3	0.425
AqCrTr-2-01	Aquilla Creek Trib-2	2.693	26,110	14,331	787.56	688.16	26.80	7	57.10	4	0.258
AqCrTr-2-02	Aquilla Creek Trib-2	1.350	14,286	6,910	721.43	649.84	35.28	5	79.37	3	0.366
AqCrTr-3-01	Aquilla Creek Trib-3	4.128	24,253	12,983	749.60	629.38	34.90	5	124.56	3	0.572
CobCr-01	Cobb Creek	15.395	67,567	31,245	759.37	597.14	16.90	7	41.27	4	0.399

					Table 5 : Unit Hydrograph Data						
Area	**Description**	**Area**	**L**	**Lca**	**U/S Elev.**	**D/S Elev.**	**Slope**	**Urban**	**% Sand**	**% Imp.**	**Tp**
Name		**Sq. Miles**	**feet**	**feet**	**85% feet**	**10% feet**	**Ft./Mi**	**(%)**			**hours**
CobCr-02	Cobb Creek	5.706	37,209	16,333	616.11	509.99	20.08	5	13.43	3	0.520
CobCr-03	Cobb Creek	4.214	28,052	6,946	606.92	496.03	27.83	5	45.87	3	0.701
CobCr-04	Cobb Creek	1.721	19,771	10,678	517.52	469.35	17.15	5	43.42	3	0.764
CobCrTr-01	Cobb Creek Trib	6.898	37,164	15,280	669.35	551.96	22.24	6	11.15	4	0.409
CobCrTr-02	Cobb Creek Trib	2.608	22,654	8,195	599.66	512.84	26.98	5	7.13	3	0.485
ColCr-01	Coleman Creek	10.617	45,681	25,228	757.85	619.08	21.39	10	20.24	6	0.302
ColCr-02	Coleman Creek	1.522	19,034	9,564	639.81	588.04	19.15	6	34.00	4	0.38
ColCrTr-01	Coleman Creek Trib	5.251	41,443	21,749	779.63	619.93	27.13	6	29.10	4	0.323
CotCr-01	Cottonwood Creek	19.013	77,108	41,283	730.38	600.94	11.82	6	32.68	4	0.362
DHCr-01	Dead Horse Creek	8.233	55,979	34,036	565.32	480.22	10.70	7	23.97	4	0.648
HacCr-01	Hackberry Creek	19.952	59,962	29,277	683.26	588.26	11.15	8	21.64	4.5	0.354
HacCr-02	Hackberry Creek	1.724	18,507	9,610	670.41	577.47	35.35	6	17.90	4	0.366
HacCr-03	Hackberry Creek	9.523	43,432	13,879	644.15	559.58	13.71	6	24.09	4	0.420
HacCr-04	Hackberry Creek	9.123	37,729	17,543	627.66	537.63	16.80	15	14.15	13	0.444
HacCr-05	Hackberry Creek	2.578	21,012	5,547	590.08	537.63	17.57	15	29.16	13	0.480
HacCr-06	Hackberry Creek	10.838	58,850	28,194	650.37	537.63	13.49	20	44.95	18	0.555
HacCrTr-01	Hackberry Creek Trib	11.829	45,673	20,442	700.93	537.63	25.17	8	26.38	4.5	0.503
HoBr-01	Horne Branch	5.823	42,521	23,619	689.33	577.34	18.54	5	31.83	3	0.407
JaBr-01	Jacks Branch	10.606	41,622	30,928	640.90	537.63	17.47	7	31.05	4	0.502
LAqCr-01	Little Aquilla Creek	18.521	70,509	34,310	707.15	574.56	13.24	5	84.54	3	0.597
LAqCr-02	Little Aquilla Creek	6.761	32,076	13,296	667.66	540.22	27.97	8	68.00	4.5	0.645
LHacCr-01	Little Hackberry Creek	12.177	65,746	34,542	759.66	569.65	20.35	6	31.18	4	0.441
LHacCr-02	Little Hackberry Creek	0.493	8,591	4,290	618.64	554.48	52.57	45	33.54	25	0.454
LovCr-01	Lovelace Creek	8.848	48,689	22,473	753.66	592.22	23.34	7	21.32	4	0.358

					Table 5 : Unit Hydrograph Data						
Area	**Description**	**Area**	**L**	**Lca**	**U/S Elev.**	**D/S Elev.**	**Slope**	**Urban**	**% Sand**	**% Imp.**	**Tp**
Name		**Sq. Miles**	**feet**	**feet**	**85% feet**	**10% feet**	**Ft./Mi**	**(%)**			**hours**
PatBr-01	Patten Branch	10.001	48,906	24,931	548.98	426.73	17.60	5	66.07	3	1.183
PecCr-01	Pecan Creek	6.609	39,297	22,441	740.00	578.32	28.96	15	33.99	11	0.422
PecCr-02	Pecan Creek	0.235	6,163	4,224	582.24	559.03	26.51	18	12.71	15	0.375
PecCrTr-01	Pecan Creek Trib	7.033	39,905	20,632	734.13	572.65	28.49	7	22.41	4	0.403
SpilTr-01	Spillway Trib	0.699	8,943	3,949	622.44	560.62	48.66	5	61.80	3	0.531
SpilTr-02	Spillway Trib	0.578	10,455	6,661	558.24	498.50	40.23	6	47.55	4	0.677
Tr_AqCrTr-2	Trib of Aquilla Creek Trib-2	1.593	14,420	7,027	760.49	687.09	35.84	5	43.13	3	0.233
Tr_CobCrTr-01	Trib of Cobb Creek Trib	1.646	14,654	7,996	611.69	543.89	32.57	5	3.11	3	0.393
	Total	407.933									

Routing Procedures

The modified Puls routing method was used for all routing reaches. The valley storage versus discharge relationships were derived from backwater analyses using HEC-RAS, version 4.0. A more detailed description of the hydraulic modeling process is presented in the Hydraulic Analysis section.

Development of Discharge-Frequency Relationships

The precipitation-runoff process for the watershed was modeled using the USACE HEC-HMS, version 3.5, watershed program model. The Snyder's unit hydrograph at each subbasin was applied to each block of excess rainfall to develop the hypothetical flood hydrographs. These hydrographs were combined and then routed downstream. Discharges for the 50-, 20-, 10-, 4-, 2-, 1-, 0.4, and 0.2 percent annual chance exceedance storms and the Standard Project Flood are presented in Table 6. The discharges were used for the HEC-RAS hydraulic modeling. These standard frequency-related events are more commonly known as those having recurrence intervals of 2, 5, 10, 25, 50, 100, 250, and 500 years, respectively. The names of the hydrologic element are based on the adjacent subbasins (example: JAqCr-01b is discharge at the Junction (J) for drainage area AqCr-01 (AqCr = Aquilla Creek) and the b would be below the tributary junction). The subbasin numbers (num. after the -) are numbers from upstream to downstream for each stream or tributary studied. The hydrologic element (also used in the HEC-HMS model) along with the location description column should identify the location of the discharge computation and tie the location to the HEC-HMS model developed.

Model Calibration

Calibration and verification of the HEC-HMS model was accomplished through a series of Aquilla Lake flood hydrograph reproduction runs for the significant flood events of December 1997–January 1998, May-June 2000, March-April 2007, May-July 2007, and May-June 2009. NWS NexRad gridded rainfall data, with a 1-hour time interval, was applied in these simulation runs. Initial abstractions and infiltration rates were adjusted slightly, in order to best represent impacts of the variations in antecedent surface soil conditions during each of these runoff events. Successful calibration of the temporal distribution (timing) of the flood hydrographs was achieved by adjustment of upstream lag times. For four of the five simulated events, the HEC-HMS modeling results produce hydrographs that very closely parallel the observed hydrographs, both in terms of timing and magnitude. Simulated peak stages on Aquilla Lake, for both the primary and lesser flood peaks during each event, matched the observed values within 0.8 foot. The simulated primary peaks are actually within 0.2 foot. The one exception to the success of these flood hydrograph reproduction efforts relates to the May-June 2000 event. It appears that the NexRad data did not capture the temporal and spatial nature of that storm event. Figures 2A through 2D show pool elevation hydrograph results from four of the five calibration storms.

Figure 2A – Pool Elevation Hydrograph from HMS Calibration

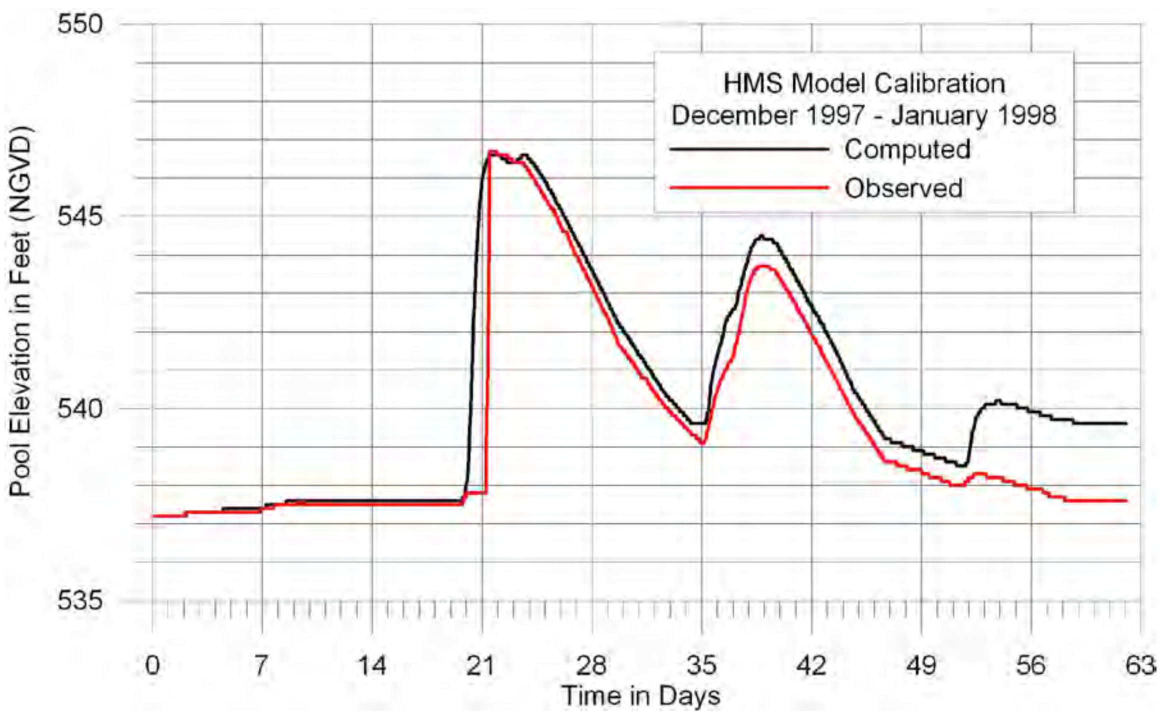


Figure 2B – Pool Elevation Hydrograph from HMS Calibration

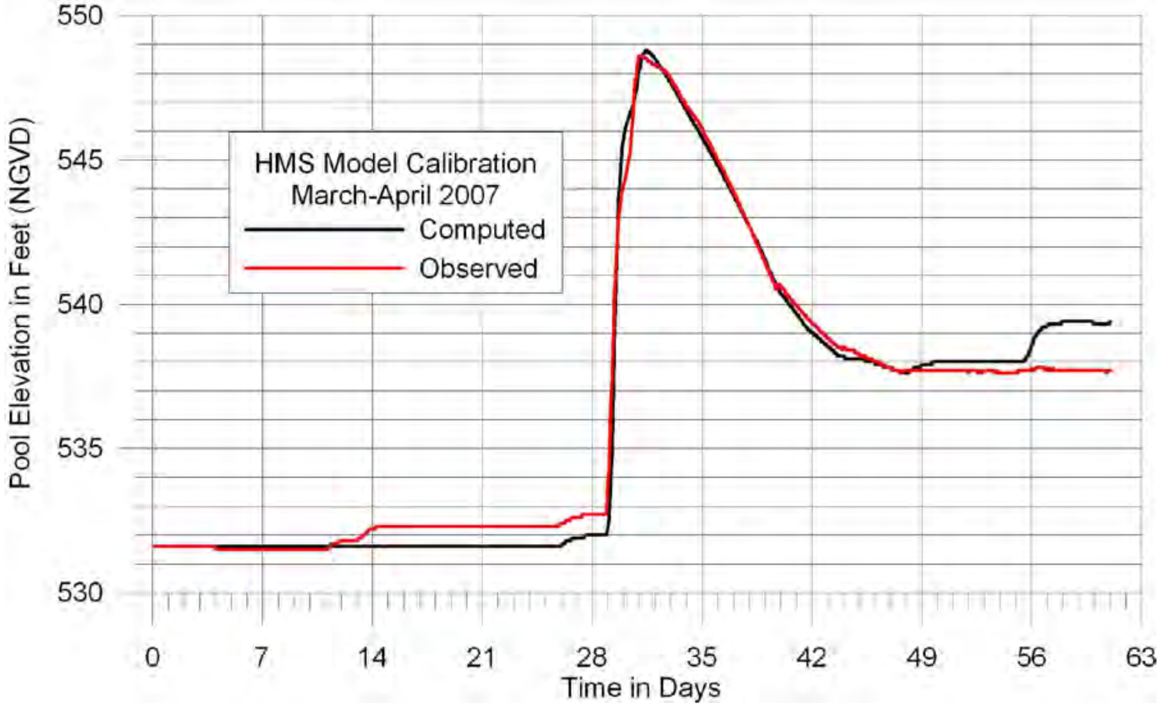


Figure 2C – Pool Elevation Hydrograph from HMS Calibration

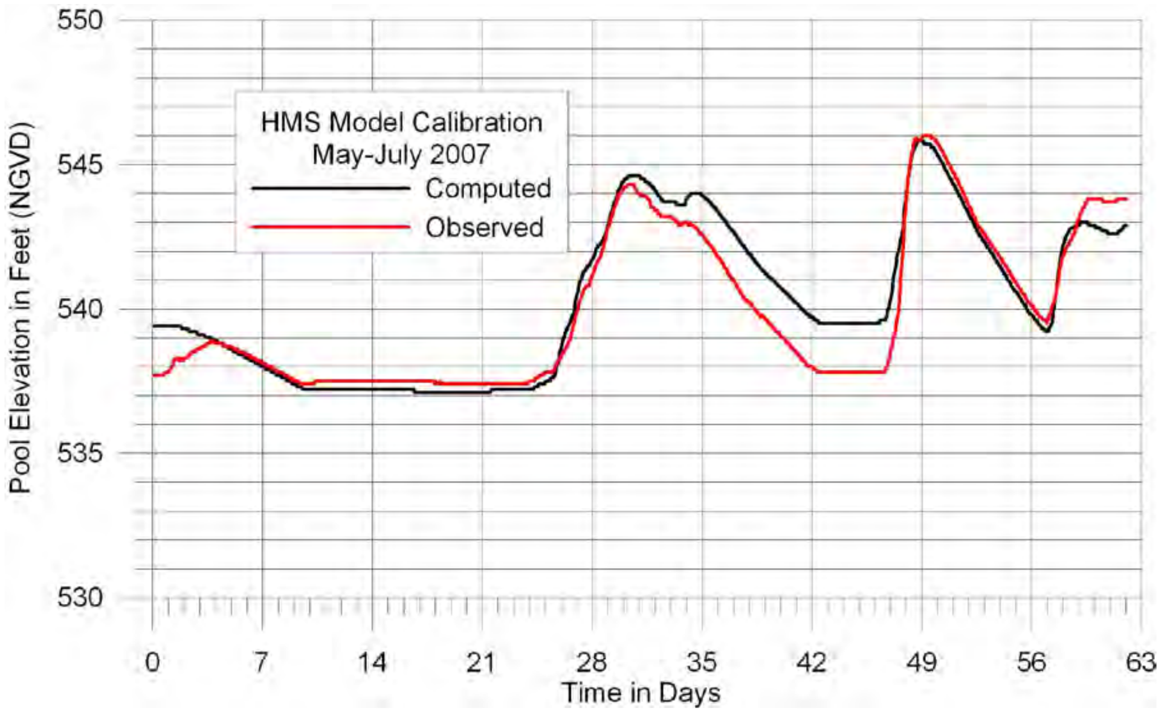
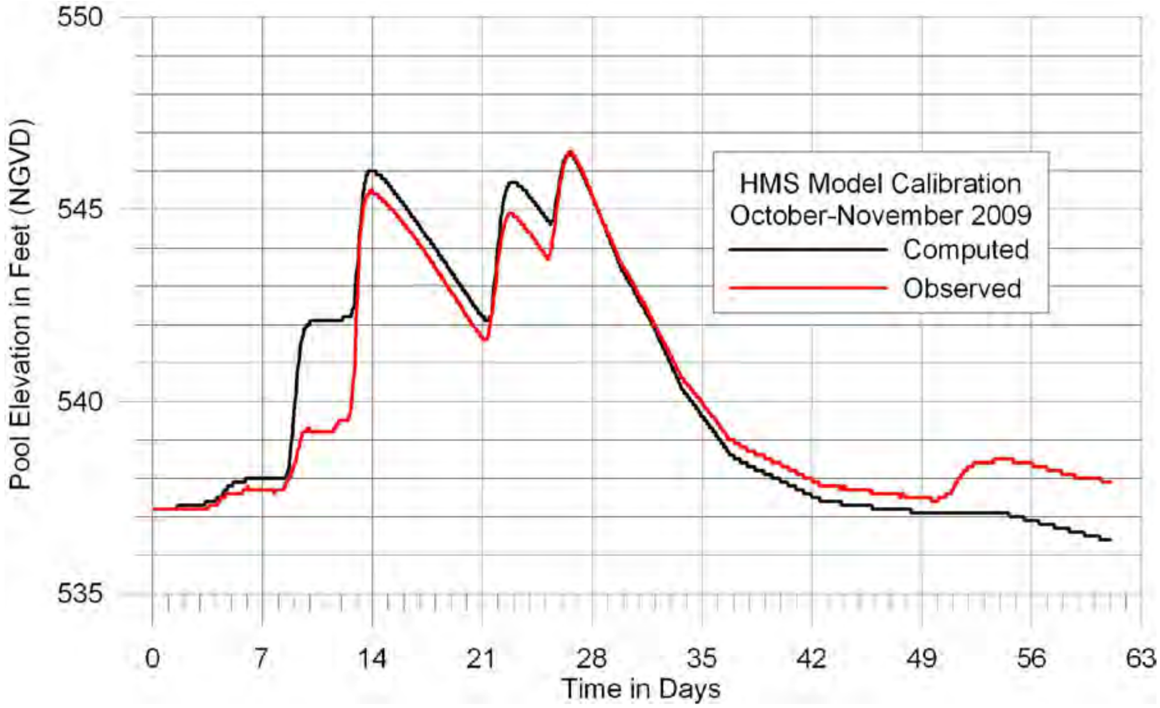


Figure 2D – Pool Elevation Hydrograph from HMS Calibration



Model Observations – Spillway vs. Cobb Creek Flows

For without project conditions and all alternatives analyzed, the computed peak pool elevations on Aquilla Lake for all frequency flood events that are less than or equal to the 250-year event are lower than the emergency spillway crest elevation. Consequently, for these events, all flooding on Aquilla Creek downstream from the dam is a result of runoff from the lower watershed and Cobb Creek. The greatest runoff contribution near the dam comes from Cobb Creek.

Table 7 and Figure 3 show peak pool elevations for 0.2, 0.4, 1, 2, 4, 10, 20, and 50 percent Annual Chance Exceedance (ACE) events for existing conditions, and each of the 3 alternatives. These are based on the period of record analysis. For existing conditions, spillway overtopping starts at around the 500-year event. For the largest pool reallocation alternative, it starts at around the 250-year event. The spillway discharges into the Cobb Creek Spillway Tributary and then into lower Cobb Creek. Cobb Creek flows into Aquilla Creek below the dam. It is not until around the 500-year event that the spillway peak flow exceeds the 500-year peak flow from the Upper Cobb Creek watershed, and controls the maximum extent of downstream flooding. When compared to existing conditions, the increased spillway use between the 250- and 500-year events does not affect the maximum extent of downstream flooding because Cobb Creek and the local runoff area peak about 35 hours before the spillway peaks. Since it may affect the duration of flooding for these events, stage-hydrograph data were included in the data provided for the economic damage calculations.

Table 6 – Aquilla Creek Watershed – Frequency Discharge Data Summary

				Table 6 Summary of Discharges							
Stream	Hydrologic	Drainage		Return Period (Years)							
Location Description	Element	Area	2	5	10	25	50	100	250	500	SPF
		(Sq. Mi.)		Discharge (cfs)							
Aquilla Creek											
Above Aquilla Creek Trib-1	AqCr-01	19.12	1,831	3,991	6,009	8,950	11,053	12,882	15036	17,001	22,683
Below Aquilla Creek Trib-1	JAqCr-01b	22.31	1,922	4,301	6,550	9,791	12,176	14,300	16808	19,006	25,869
Below Aquilla Creek Trib-2	JAqCr-02b	27.96	2,663	6,089	9,210	13,741	16,964	19,817	23659	26,802	36,246
Above Aquilla Creek Trib-3	JAqCr-03a	32.87	2,455	5,703	8,816	13,437	16,937	20,015	24263	27,512	37,612
Below Aquilla Creek Trib-3	JAqCr-03b	37.00	2,413	5,642	8,751	13,536	17,285	20,628	25165	28,540	39,633
Above Cottonwood Creek	JAqCr-04a	45.24	2,188	5,282	8,303	13,005	16,980	20,550	25482	29,035	40,968
Below Cottonwood Creek	JAqCr-04b	64.25	2,787	6,206	9,650	16,145	21,477	27,476	35482	40,523	64,601
Above Horne Branch	JAqCr-05a	69.56	2,602	5,978	9,288	15,602	21,390	27,673	35899	41,173	65,796
Below Horne Branch	JAqCr-05b	75.38	2,571	6,002	9,832	15,686	21,817	28,372	36959	42,415	68,098
Above Little Aquilla Creek	JAqCr-06a	84.16	2,296	5,724	9,214	15,397	21,698	28,505	37488	43,082	71,328
Below Little Aquilla Creek	JAqCr-06b	109.44	3,015	7,951	13,165	21,961	29,284	36,410	49130	56,919	88,838
Above Hackberry Creek in Aquilla Lake	JAqCr-7L	125.56	2,930	7,906	13,132	21,920	29,431	36,550	50062	57,984	90,868
Aquilla Lake Dam Inflow	Aquilla Lake	254.52	8,951	18,307	28,080	45,509	60,540	79,117	109655	126,488	195,083
	Peak Lake Elev.	254.52	542.0	546.5	548.5	552.0	554.0	557.0	562.1	566.0	573.0
	Outlet	254.52	2,276	2,366	2,448	2,600	2,621	2,650	2,695	2,699	2,700
	Spillway	254.52	0	0	0	0	0	0	0	3,828	59,341
Above Cobb Creek	JAqCr-08a	257.54	3,514	4,559	5,465	6,258	6,859	7,402	8,087	8,866	10,450
Below Cobb Creek	JAqCr-08b	297.00	5,605	9,436	12,709	17,088	18,703	22,988	30,933	34,958	62,088
Above Dead Horse Creek	JAqCr-09a	299.40	5,303	9,353	12,629	17,021	18,633	23,108	31,208	35,254	62,172

				Table 6 Summary of Discharges								
Stream	Hydrologic	Drainage		Return Period (Years)								
Location Description	Element	Area	2	5	10	25	50	100	250	500	SPF
		(Sq. Mi.)		Discharge (cfs)							
USGS Gage 08093500	JAqCr-09b	307.63	5,303	9,384	13,452	18,740	20,805	25,685	34,858	39,448	62,535
Above Alligator Creek	JAqCr-10a	312.07	4,875	8,905	12,654	18,358	20,701	25,696	35,107	39,758	62,688
Below Alligator Creek	JAqCr-10b	343.01	8,551	15,333	22,064	30,697	34,073	44,587	63,089	71,551	108,643
	JAqCr-11	353.50	7,698	14,140	20,902	30,323	34,462	45,063	64,730	73,810	114,281
Above Patten Branch	JAqCr-12a	373.80	7,062	12,887	18,757	28,521	33,854	44,524	65,985	75,471	119,500
Below Patten Branch	JAqCr-12b	383.80	7,062	12,887	18,757	28,521	34,232	45,278	67,487	77,310	123,136
Confluence with Brazos River	JAqCr-13	407.93	6,553	11,847	17,100	26,561	33,167	44,424	67,165	77,927	126,596
Patten Branch	PatBr-01	10.00	1,447	3,038	4,467	6,403	7,626	8,776	9,984	10,246	16,335
Alligator Creek											
Above Alligator Creek Trib-01	AlCr-01	8.02	2,201	4,173	5,761	7,566	8,747	9,890	11,164	12,612	17,473
Below Alligator Creek Trib-01	JAlCr-01b	14.02	3,934	7,439	10,239	13,432	15,523	17,538	20,191	22,816	33,194
	JAlCr-02	25.71	6,191	12,239	16,917	22,475	26,214	29,788	35,584	40,239	57,490
At Confluence with Aquilla Creek	JAlCr-03	30.93	5,360	11,346	16,279	22,292	26,227	30,186	36,571	41,435	61,483
Alligator Creek Trib-01	AlCrTr-01	6.00	1,891	3,546	4,836	6,250	7,201	8,116	9,094	10,277	15,730
Dead Horse Creek	DHCr-01	8.23	1,564	2,964	4,148	5,657	6,670	7,645	8,642	9,760	11,002
Cobb Creek											
Headwater	CobCr-01	15.40	2,459	4,868	6,971	9,749	11,579	13,319	15,361	17,355	23,103
Above Cobb Creek Trib-01	JCobCr-02a	21.10	2,221	4,586	6,614	9,522	11,644	13,654	16,083	18,208	24,381
Below Cobb Creek Trib-01	JCobCr-02b	32.25	3,975	7,812	10,774	14,150	17,745	21,450	26,106	29,778	45,200
Above Spillway Trib	JCobCr-03a	36.47	3,721	7,583	10,811	14,710	18,342	22,427	27,511	31,303	49,523
Below Spillway Trib	JCobCr-03b	37.74	3,750	7,644	10,943	15,236	18,607	22,804	27,990	31,838	61,904

				Table 6 Summary of Discharges							
Stream	Hydrologic	Drainage		Return Period (Years)							
Location Description	Element	Area	2	5	10	25	50	100	250	500	SPF
		(Sq. Mi.)		Discharge (cfs)							
At Confluence with Aquilla Creek	JCobCr-04	39.46	3,433	7,300	10,586	14,899	18,500	22,656	28,029	31,926	61,947
Spillway Trib											
Headwater	SpilTr-01	0.70	476	903	1,187	1,454	1,652	1,838	2,078	2,256	2,531
Adjacent to Aquilla Lake Spillway	JSpilTr-01	0.70	476	903	1,187	1,454	1,652	1,838	2,078	3,828	59,370
At Confluence with Cobb Creek	JSpilTr-02	1.28	474	1,071	1,500	1,993	2,312	2,612	3,002	3,828	59,380
Cobb Creek Trib-01											
Above Trib of Cobb Creek Trib	CobCrTr-01	6.90	2,690	4,859	6,412	8,076	9,279	10,423	11,698	13,226	17,248
Below Trib of Cobb Creek Trib	JCobCrTr-01b	8.54	3,282	5,850	7,728	9,845	11,336	12,743	14,385	16,269	21,248
At Confluence with Cobb Creek	JCobCrTr-02	11.15	2,975	5,549	7,613	9,881	11,484	13,117	15,044	17,051	23,620
Trib of Cobb Creek Trib-01	Tr_CobCrTr-01	1.65	1,233	2,061	2,484	3,037	3,461	3,856	4,378	4,759	5,666
Hackberry Creek											
Above Coleman Creek	HacCr-01	19.95	3,785	7,167	10,000	13,551	15,978	18,308	21,356	24,119	35,232
Below Coleman Creek	JHacCr-02a	37.34	6,548	13,065	18,455	25,007	29,385	33,876	41,298	46,671	58,620
Below Lovelace Creek	JHacCr-02b	47.91	6,181	12,726	18,244	25,745	31,104	36,097	44,871	50,869	67,249
Above Little Hackberry Creek	JHacCr-03a	57.44	5,545	11,773	17,310	24,779	30,605	35,932	45,349	51,537	68,215
Below Little Hackberry Creek	JHacCr-03b	83.98	6,120	12,307	17,325	25,760	33,540	40,521	52,656	59,959	84,184
Above Hackberry Creek Trib-01	JHacCr-04a	93.11	4,865	10,901	16,771	25,467	33,522	40,698	53,022	60,521	85,724
Below Hackberry Creek Trib-01	JHacCr-04b	104.94	5,011	10,788	16,614	25,279	33,705	41,298	54,039	61,737	87,882
Above Jacks Branch	JHacCr-05a	107.51	5,031	10,710	16,544	25,237	33,707	41,375	54,174	61,901	88,285
Below Jacks Branch	JHacCr-05b	118.12	6,639	13,117	18,235	25,111	33,972	42,133	55,687	63,674	94,833

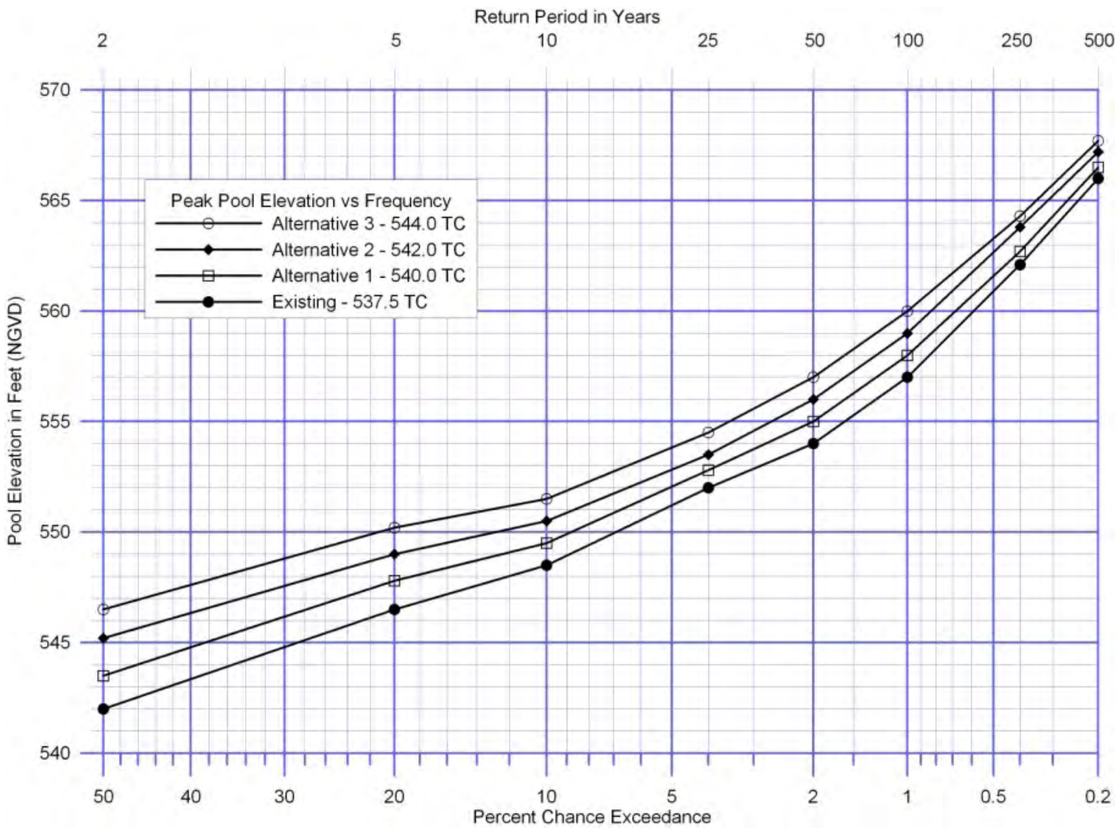
				Table 6 Summary of Discharges							
Stream	Hydrologic	Drainage		Return Period (Years)							
Location Description	Element	Area	2	5	10	25	50	100	250	500	SPF
		(Sq. Mi.)		Discharge (cfs)							
At Confluence with Aquilla Creek	JHacCr-06L	128.96	8,111	15,913	22,283	29,949	35,390	44,448	60,392	69,533	112,699
Jacks Branch	JaBr-01	10.61	2,318	4,442	6,210	8,359	9,763	11,119	12,657	14,296	22,274
Hackberry Creek Trib-1	HacCrTr-01	11.83	3,238	6,098	8,373	10,962	12,681	14,343	16,392	18,517	21,242
Little Hackberry Creek											
Above Pecan Creek	LHacCr-01	12.18	2,208	4,253	6,006	8,258	9,746	11,176	12,776	14,431	15,461
Below Pecan Creek	JLHacCr-01b	26.05	5,785	11,050	15,273	20,401	23,841	27,121	32,235	36,410	37,901
At Confluence with Hackberry Creek	JLHacCr-02b	26.55	5,674	10,871	15,130	20,235	23,689	26,973	32,059	36,264	37,993
Pecan Creek											
Above Pecan Creek Trib-1	PecCr-01	6.61	1,968	3,619	4,936	6,422	7,397	8,345	9,372	10,588	10,584
Below Pecan Creek Trib-1	JPecCr-01b	13.64	4,088	7,574	10,348	13,450	15,506	17,490	20,106	22,718	22,827
At Confluence with Little Hackberry Creek	JPecCr-02b	13.88	3,969	7,458	10,214	13,267	15,329	17,312	19,910	22,491	22,844
Pecan Creek Trib-01	PecCrTr-01	7.03	2,261	4,206	5,709	7,355	8,473	9,547	10,734	12,130	12,243
Lovelace Creek	LovCr-01	8.85	2,448	4,576	6,268	8,203	9,492	10,738	12,155	13,731	14,775
Coleman Creek									,		
Above Coleman Creek Trib-01	ColCr-01	10.62	2,893	5,340	7,299	9,562	11,071	12,532	14,264	16,112	16,468
Below Coleman Creek Trib-01	JColCr-01b	15.87	4,199	7,879	10,815	14,188	16,435	18,607	21,526	24,316	24,179
At Confluence with Hackberry Creek	JColCr-02	17.39	3,911	7,522	10,427	13,756	16,079	18,410	21,423	24,185	24,704
Coleman Creek Trib-01	ColCrTr-01	5.25	1,457	2,748	3,784	4,966	5,742	6,493	7,262	8,204	7,711
Little Aquilla Creek											
Confluence with Aquilla Creek	JLAqCr-02	25.28	1,480	3,236	4,980	7,672	9,936	11,874	14,193	16,068	17,526

				Table 6 Summary of Discharges							
Stream	Hydrologic	Drainage		Return Period (Years)							
Location Description	Element	Area	2	5	10	25	50	100	250	500	SPF
		(Sq. Mi.)		Discharge (cfs)							
Horne Branch	HoBr-01	5.82	1,421	2,725	3,798	5,064	5,884	6,677	7,488	8,459	12,922
Cottonwood Creek	CotCr-01	19.01	2,581	5,024	7,169	10,110	12,205	14,144	16,464	18,595	28,983
Aquilla Creek Trib-3	AqCrTr-3-01	4.13	5,51	1,374	2,160	3,246	3,824	4,363	4,876	5,517	6,742
Aquilla Creek Trib-2											
Above Trib of Aquilla Creek Trib-2	AqCrTr-2-01	2.69	7,64	1,525	2,146	2,843	3,275	3,691	4,087	4,620	5,794
Below Trib of Aquilla Creek Trib-2	JAqCrTr-2-01b	4.29	1,445	2,776	3,774	4,965	5,728	6,446	7,157	8,103	9,909
At Confluence with Aquilla Creek	JAqCrTr-2-02	5.64	985	2,172	3,148	4,429	5,340	6,123	6,902	7,828	9,356
Trib of Aquilla Creek Trib-2	Tr_AqCrTr-2	1.59	881	1,616	2,132	2,644	3,028	3,382	3,687	4,186	4,758
Aquilla Creek Trib-1	AqCrTr-1-01	3.20	608	1,397	2,121	3,021	3,499	3,962	4,405	4,982	5,793

Table 7 – Peak Pool Elevations for Alternative Analysis

Flood Return Period	Percent Chance Exceedance	Existing	Alternative 1	Alternative 2	Alternative 3
		Top of Conservation Pool (feet)			
		537.5	**540.0**	**542.0**	**544.0**
(Years)		Corresponding Peak Pool Elevations (feet)			
500	0.2	566.0	566.5	567.2	567.7
250	0.4	562.1	562.7	563.8	564.3
100	1.0	557.0	558.0	559.0	560.0
50	2.0	554.0	555.0	556.0	557.0
25	4.0	552.0	552.8	553.5	554.5
10	10.0	548.5	549.5	550.5	551.5
5	20.0	546.5	547.8	549.0	550.2
2	50.0	542.0	543.5	545.2	546.5

Figure 3 - Peak Pool Elevations for Alternative Analysis



Future Watershed Runoff Conditions

The potential for future increases in the hypothetical flood peak discharges over the flood damage reduction economics analysis period (50 years) was considered to be insignificant for this watershed. The majority of the Aquilla Creek Watershed is rural with only scattered developed areas and relatively small towns. The anticipated urbanization effects (over the next 50 years) within this headwater area were deemed insignificant from a hydrologic analysis perspective. In order to significantly change the runoff potential, major dense residential or commercial development and stream channelization would have to occur. Therefore, the effect on future discharges will be insignificant.

PRELIMINARY HYDRAULIC ANALYSIS

A preliminary hydraulic analysis, based on existing hydraulic and hydrologic models, was performed at the beginning of Phase II. Its purposes were to ascertain whether or not the proposed pool raise would cause a significant increase in downstream damages and whether or not the pool raise would pose a threat to dam safety. If either were the case, the PDT would decide whether or not to proceed with a more detailed multi-disciplined analysis as outlined in the Project Management Plan for Phase II.

Background

In Phase I it was determined that raising the top of conservation pool results in increased frequency of spillway overtopping. More frequent spillway overtopping can potentially increase flood damage downstream from the dam over the life of the project. If this is significant, the flood control purpose of Aquilla Lake will be compromised. This would factor into a decision by the PDT whether or not to proceed with the detailed multi-disciplined analysis.

In order to proceed with a pool raise, all matters of dam safety must first be addressed. For Aquilla Lake, this includes determining if raising the top of conservation pool would significantly increase the risk of spillway damage, or possibly even spillway failure, due to erosion. If this were the case, a decision would be made on whether to proceed with the more detailed multi-disciplined analysis.

Analysis Tools

No new hydrology from either HEC-1 or HEC-HMS (Hydrologic Modeling System) was developed for the Aquilla Creek watershed for the preliminary analysis,. Lake pool elevations came from the frequency analysis developed for Phase I of the Brazos System Assessment Study. These were based on the period of record up through 2007. Updated values were not yet developed when the preliminary analysis was done. The period of record analysis and pool frequencies developed for Phase II were used in the detailed hydraulic analysis. The Phase I elevations were used to determine peak outflow discharges for Aquilla Lake based on the most up-to-date spillway rating curve for the lake. For the hydraulic analysis a HEC-River Analysis System (RAS) model was built from several existing LRD-1 backwater models developed during the design phase of the dam in the 1970's. These models spanned the reach of Aquilla Creek from the Brazos River upstream to the dam, Cobb Creek from its confluence with Aquilla Creek upstream to the Spillway Tributary, and the Spillway Tributary from Cobb Creek to the lake. Aquilla Creek and Hackberry Creek upstream from the dam were also modeled. The converted model was not geo-referenced. The old LRD-1 models were based on surveyed valley sections, additional channel sections that were extended with USGS 7-1/2 minute topo data, and as-built bridge plans. The Spillway Tributary model was based on design plans and more detailed 2-foot contour topographic mapping. The original Manning's roughness coefficients were based on field investigations prior to construction and examinations of aerial photographs from the 1980's. For the current study these were re-evaluated using more recent aerial photography and site visits (See detailed hydraulic analysis for discussion). The spillway cross-sections were updated to reflect as-built geometry. Once converted, the model was recalibrated to the United States Geological Survey (USGS) gage at FM 1304. Two other computer models were used in the preliminary engineering analysis. RiverWare was used in the period of record analysis to determine the changes in pool frequency elevations, and SITES was used to estimate erosion and head-cutting in the spillway section. Descriptions of the two models, how they were used, and how the results were interpreted can be found in the Reservoir Control Appendix and the Geotechnical Appendix.

Downstream Flooding Analysis

The preliminary analysis indicated that the increase in extent of downstream flooding due to raising the top of conservation pool for the different alternatives would not warrant terminating the study. Aquilla Dam has a perched spillway at elevation 564.5 with the top of flood control pool at 556.0. Under existing conditions, the top of conservation pool is 537.5 and the return period for the spillway is between 400 and 500 years (see Figure 3 - Peak Pool Elevations for Alternative Analysis). With the largest pool raise alternative, top of conservation pool at 544.0, it decreases the return period to 250 years. For events that do not result in spillway overtopping, the lake outflow is limited to 3,000 cfs, the maximum release through the outlet works. Any flooding downstream is the result of local runoff and runoff from the Cobb Creek sub-basin. For ~~most~~ events that result in spillway overtopping, maximum flooding downstream is caused by the peak from Cobb Creek rather than the peak flow out of the lake. Consequently, nothing in the preliminary flood analysis indicated that the study should be terminated. The relationship between peak flows from Cobb Creek versus spillway flow is discussed in more detail in the hydrology part of this appendix.

Spillway Erosion Analysis

From the period of record analysis performed in Phase I, it was known that the frequency of emergency spillway use for the Aquilla Dam would be increased if the top of conservation pool were raised. The original design put the overtopping return period at about the 500 year event. During the dam’s design phase, a detailed analysis of potential spillway erosion was not deemed necessary because of the infrequent use. Aquilla was designed in the 1970’s and built in the early 1980’s. At that time no Fort Worth District spillways had experienced major erosion, as would later be demonstrated at Grapevine and Canyon Dams, and to some extent at Lewisville. During the 1970’s there was not the emphasis on risk of failure and resulting consequences that we have today. The preliminary hydraulic analysis was part of the erosion analysis to determine if the spillway would fail, or be significantly damaged, during the passage of hydrologic events great enough to cause spillway flows. The spillway erosion analysis was done jointly by the Hydrology and Hydraulic Design Section and the Geotechnical Section of the Fort Worth District. The analysis is described in this appendix and also in the Geotechnical appendix.

The Aquilla spillway is a broad crested weir located in a trapezoidal grass-lined earthen cut adjacent to the left abutment of the dam. The actual crest consists of a 3-feet wide, 1200-feet long, horizontal concrete sill at elevation 564.5 with 4.5 horizontal to 1 vertical side slopes up to elevation 577.5, and then a grassy slope up to natural ground. The crest is 8.5 feet above the top of flood control pool which is at elevation 556.0. The existing overtopping return period is 500 years. With the maximum proposed pool rise (Top Conservation Pool at 544.0), the return period is decreased to about 250 years (See Period of Record Analysis described in the Reservoir Control Appendix). Based on USACE experience with similar spillway designs, some erosion or damage would be expected during spillway events. With such large return periods it was reasoned that the increase in annualized spillway damage repair cost would be very small when compared to the benefits of more available water supply. In the past, additional analysis of potential spillway erosion would not be considered necessary; however, because of the current emphasis on dam safety, additional analysis is now required.

Hydraulic parameters from routing the Probable Maximum Flood (PMF) through the spillway were computed using the converted HEC-RAS model. The PMF was used because it was the only large flood event for which inflow and outflow hydrographs had been computed and were available at the time of the preliminary analysis. If the analysis indicated that the spillway could pass this event without failure, then it could also successfully pass the more frequent events which are described in this appendix. The PMF was used to estimate erosion rates for more frequent flood events. The pool rises quickly during a PMF and drops slowly on the recession side of the hydrograph. The assumption was made that for more frequent floods the pool would rise rapidly and then would recede at the same rate as the PMF from any corresponding pool elevation. This assumption allows us to estimate the duration of erosive discharge rates for floods other than the PMF if we know the peak pool elevations.

The PMF spillway hydrographs from a 2009 study were used to develop a time dependent series of water surface profiles through the spillway cut. From these, velocity duration and tractive force relationships were developed for use with the SITES spillway erosion computer program in the geotechnical analysis of the spillway. SITES (origin of name unknown) was developed by the Natural Resources Conservation Service (NRCS) for evaluating potential spillway and embankment erosion. The computer program has been adopted by the Corps of Engineers for this type of analysis.

Tractive force is the pull of water on a wetted surface, and is an important parameter used in erosion analysis. It is a function of the weight of water, the surface area over which is acts, and the slope of the surface. Tractive force is one of the output parameters available in HEC-RAS where it is referred to as shear stress. Critical, or permissible, tractive force is the maximum unit tractive force in lbs/sq ft that will not cause serious erosion. Figure 5 shows the orientation of the spillway with respect to the dam, and the approximate location of the area of initial erosion. Figure 6 shows the HEC-RAS model cross-section alignment through the spillway for section 900. Section 2200 is the upstream side of the spillway crest. Figure 7 is a cross-section plot for section 900 and section 2200. Computed velocity and shear values are shown in Figures 6, 7, and 8 for cross-section 900. This is where head-cutting is most likely to start. The topsoil in the spillway is stiff clay which has a critical tractive force of around 0.26 lb/sq ft. With a grass cover, it increases to 4.0 lb/sq ft. (This is based on Department of Transportation Hydraulic Engineering Circular No. 15, Third Edition, Chapter 4.) From Figure 8 it can be seen that for pool elevations above 568.5, which is 4.0 feet above the spillway crest, erosion of the topsoil is likely to occur. The underlying material is sandstone. It is not significantly affected by the tractive force, but instead fails as a result of head cutting initiated at the downstream end near section 900. This is described in detail in the SITES analysis which is included in the Geotechnical appendix. The results of that analysis indicate that although there will be some spillway damage caused by head cutting, spillway failure is not likely to occur during the passage of the PMF or lesser events.

Figure 5- Aquilla Dam - Plan View

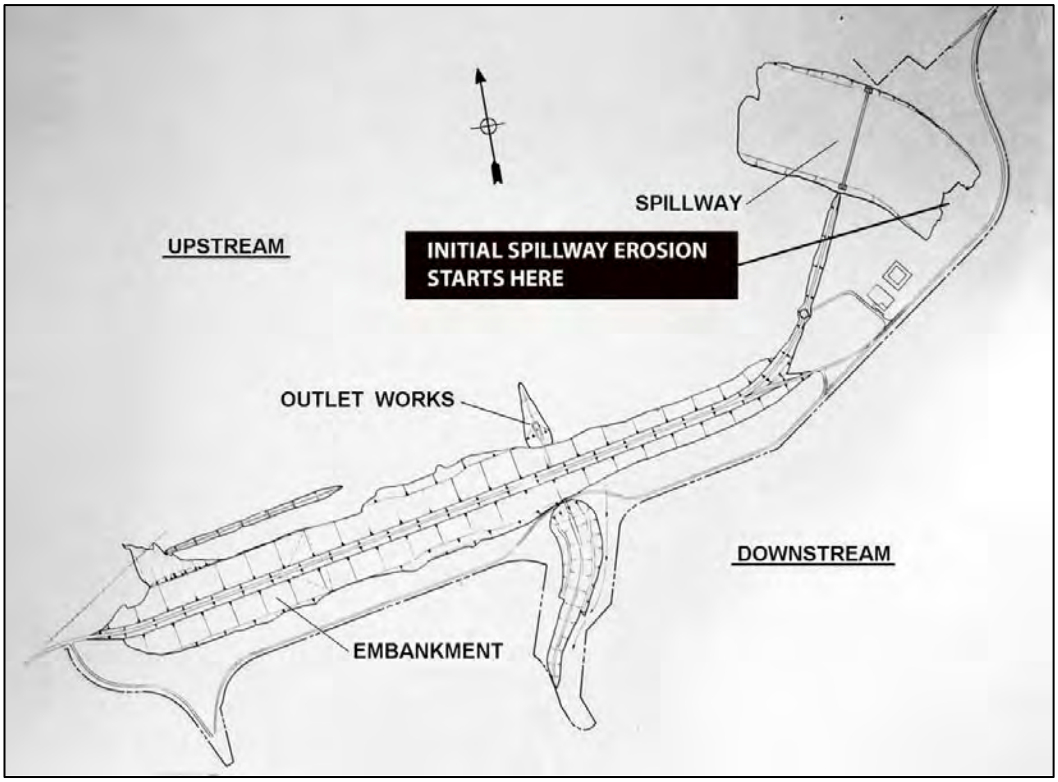


Figure 6 – Spillway Cross-section Layout

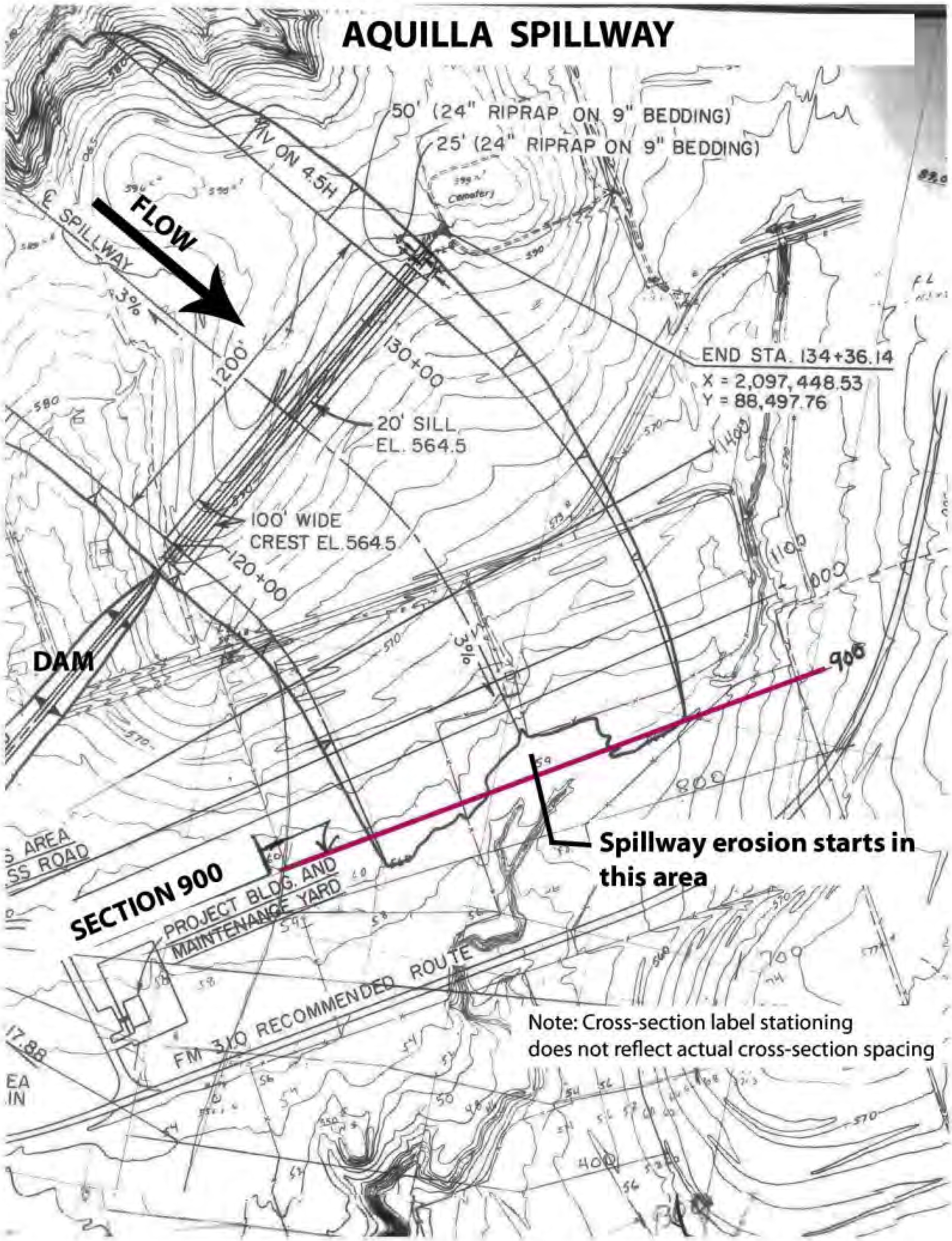


Figure 7 – HEC-RAS Cross-sections – Spillway Crest and Downstream End

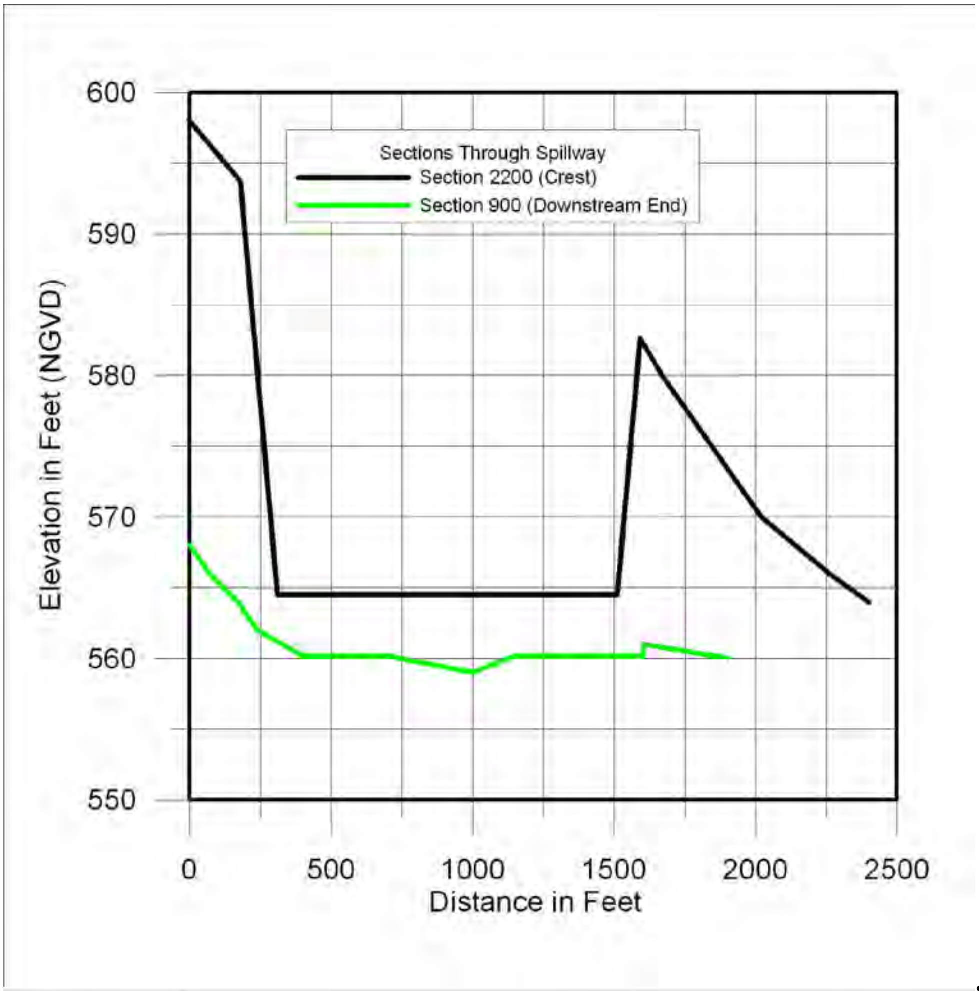


Figure 8 – Pool Elevation vs. Tractive Force for PMF

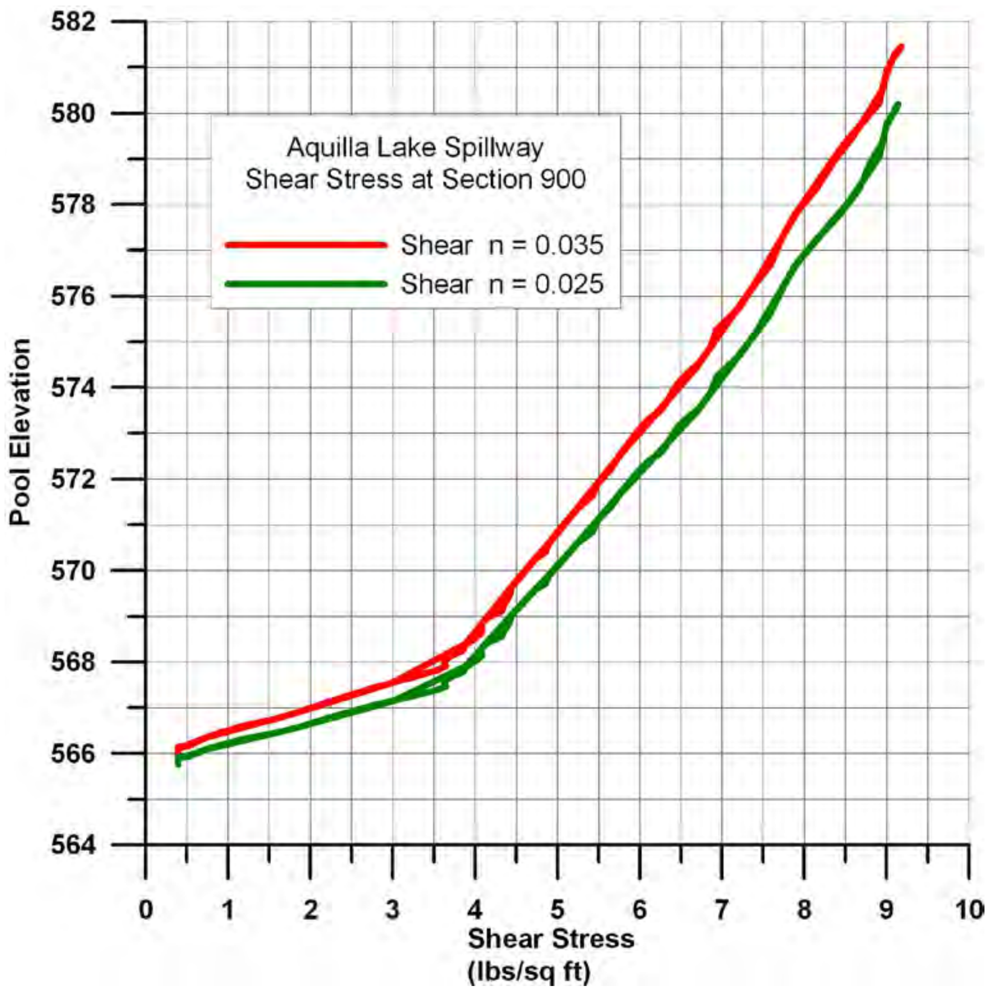


Figure 9 – Tractive Force vs. Time for PMF

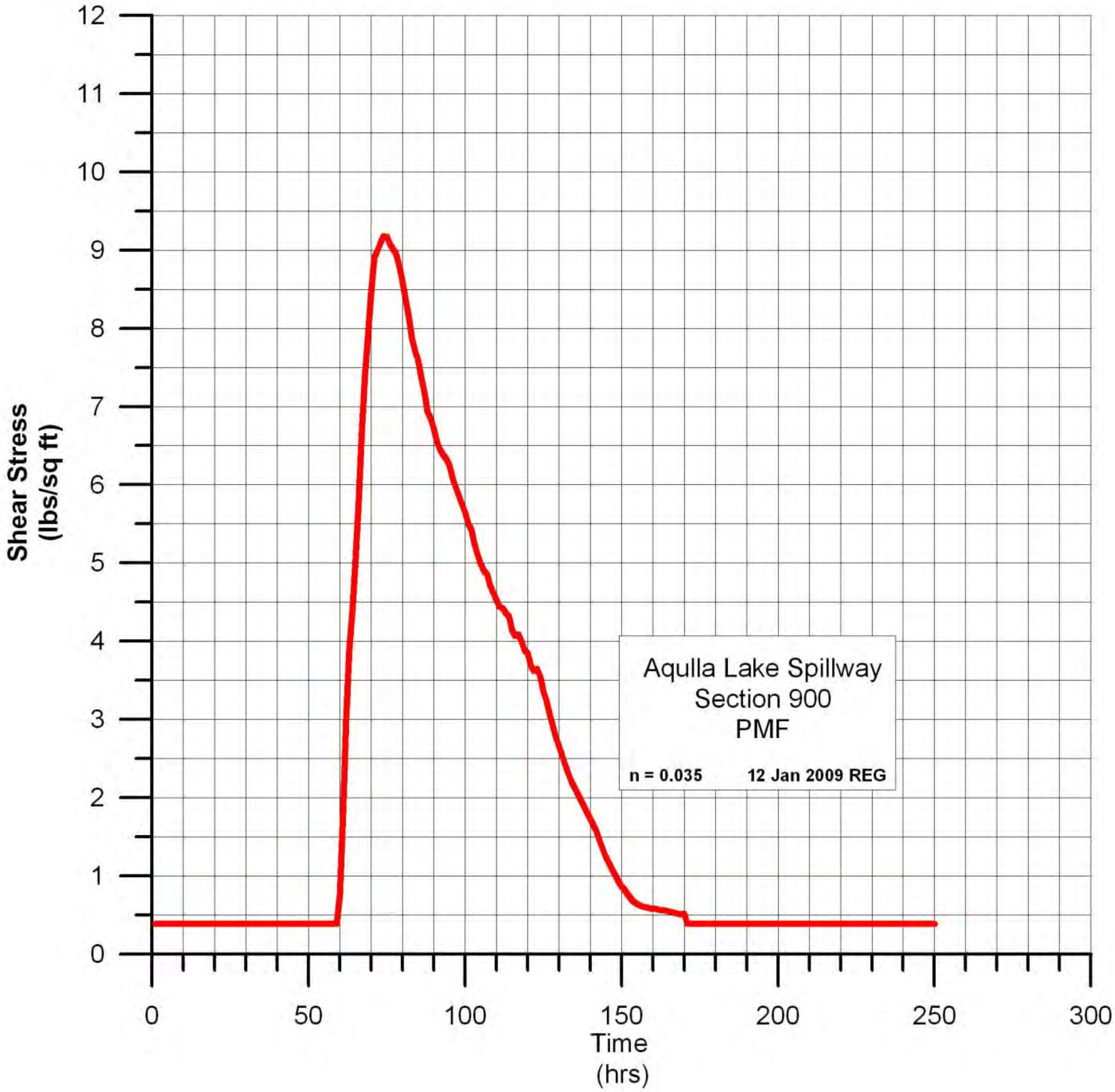
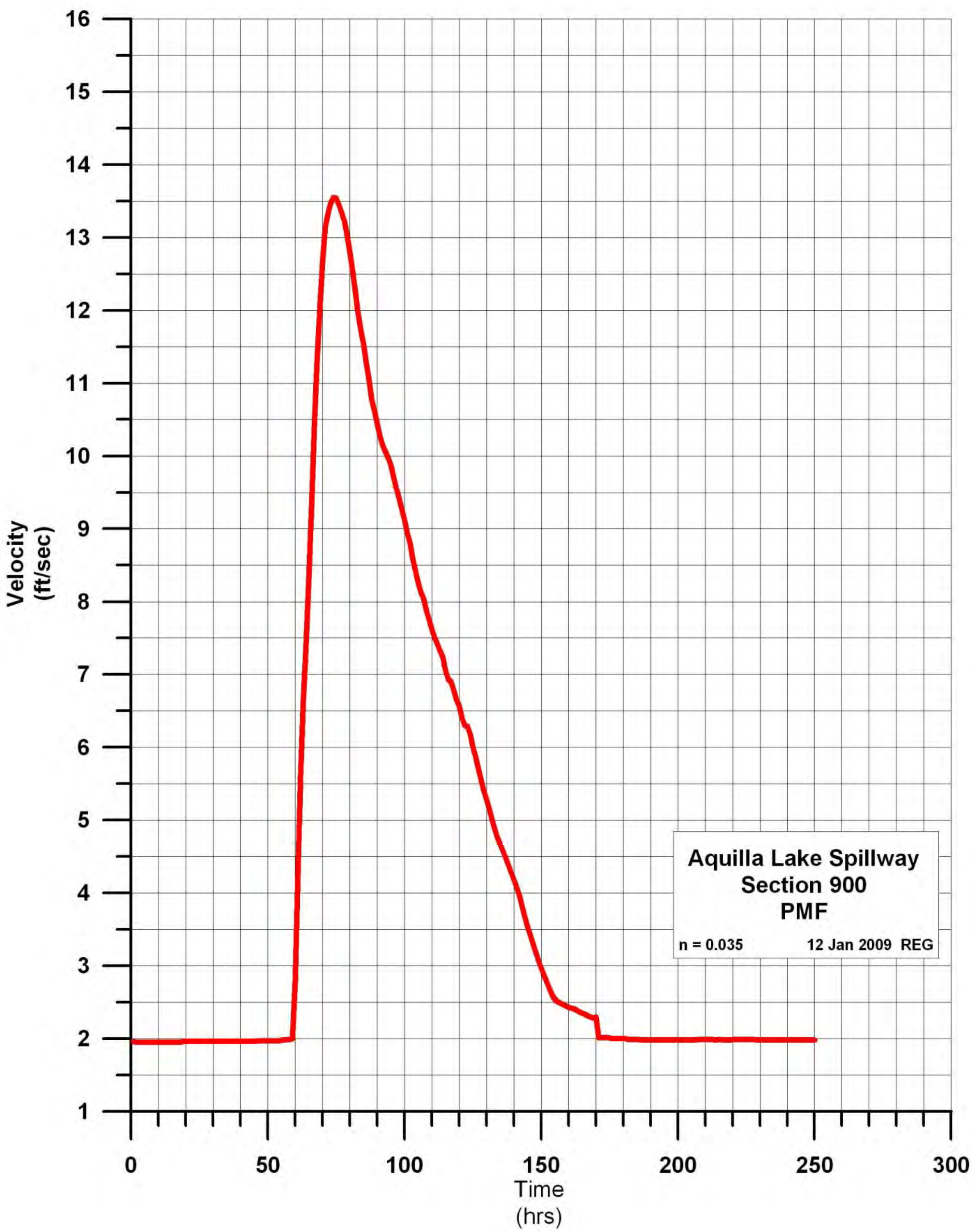


Figure 10 – Velocity vs. Time for PMF



This information was incorporated in the preliminary erosion analysis described in the Geotechnical Appendix. The preliminary erosion analysis indicated that although the concrete spillway sill would probably fail during a PMF scenario, the spillway approach channel would be unlikely to head cut all the way back to the lake. Even with increased frequency of use, raising the top of conservation pool would impose very little risk for a complete spillway failure as long as repairs were made following each major flow event. This additional risk was not significant enough to terminate the study.

DETAILED HYDRAULIC ANALYSIS

Although the converted LRD-1 model was sufficient to determine that increased downstream flooding would not be a problem, and that spillway erosion was unlikely to fail the dam, the converted model could not provide the necessary level of detail needed for a detailed economic analysis. In addition there was not an existing hydrologic model that could supply the frequency discharges necessary for such an analysis. Consequently, the analysis would require the development of both hydraulic and hydrologic models to do any type of frequency based flood plain analysis either upstream or downstream from the dam. The hydrologic model development is described in the first part of this appendix.

For the detailed hydraulic analysis, a new, geo-referenced, HEC-RAS water surface profile model was developed for the Aquilla Creek watershed both downstream and upstream from the Aquilla Lake Dam. Aquilla Creek, Hackberry Creek, Cobb Creek, the Spillway Tributary, Little Aquilla Creek, Patten Branch, Country Club Creek, and several smaller tributaries were modeled. Flood plain and channel geometry was developed from 10 meter USGS digital elevation model (DEM) data sets, surveyed channel and valley cross-section data from the original Aquilla Dam construction project, and 2009 TWDB volumetric survey data for Aquilla Lake. The primary development tools were HEC-RAS version 4.1, Geo-RAS version 4.3, and ESRI ArcMap version 9.3. Frequency discharges for the alternative analysis used in the HEC-RAS model were developed with HEC-HMS with input from the RiverWare modeling for the lake discussed in the

Appendix K. Profiles and delineations were computed for the 2-, 5-, 10-, 25-, 50-, 100-, 250-, and 500-year floods or the 50, 20, 10, 4, 2, 1, 0.4, and 0.2 percent Annual Chance Exceedance (ACE) events for existing conditions, and each of the 3 alternatives. Overall, 28 set of profiles and delineations were compiled for the study. This information was used as part of the input for the economic analysis. At a later date, the 1000- and 2000-year flood profiles were also computed to be used in the economic analysis.

HEC-RAS Model

The HEC-RAS model was developed using 10 meter USGS DEM data sets and surveyed cross-section data. DEM's for the area of interest were imported into ArcMap Version 9.3 and converted to a digital terrain model (DTM). For the lake reach of Aquilla and Hackberry Creeks, the Geo-RAS DTM was further refined for the lake reach with additional xyz data points from the 2009 Texas Water Development Board volumetric study. HEC's Geo-RAS plug-in was used to layout stream centerlines and cross-section cut lines on the DTM. Initially, cross-section cut lines were laid out at locations where there were surveyed valley and channel data from the original LRD-1 backwater model. The surveyed sections were usually more than a mile apart. Consequently, additional cross-sections were added to better define the valley shape, and also to reduce the distance between computational points. Existing surveyed cross-section data were available for Aquilla Creek, Cobb Creek, Hackberry Creek, and Little Aquilla Creek. No new surveys were done for this study. For the smaller tributaries, where there was no survey data, cross-section geometry was based solely on 10 meter DEM data. For the required level of detail in the study, and the flood frequencies for which one could expect to have significant damages, it was felt that the major tributaries were the main contributors to any changes in the flood plains for with and without project conditions. Consequently, not as much detail on the smaller tributaries was required – only enough to adequately develop storage routing data for the HEC-HMS modeling. Approximately 112 miles of stream were modeled upstream and downstream from the dam with over 450 cross-sections. Figure 11 shows the general layout of the HEC-RAS cross-sections.

The Geo-RAS DEM cross-sections were exported to HEC-RAS where they became the base geometry file. Ten meter DEM's, although adequate for developing flood plain cross-section geometry, did not contain enough resolution to model the channel section in sufficient detail for the backwater analysis. Consequently, the channel geometry in the cross-sections was replaced with either surveyed or interpolated surveyed channel cross-section data. An additional geometry file was built from surveyed valley and channel cross-section data. Reach lengths matched the reach lengths in the Geo-RAS generated file. Interpolated sections were then added so that there was a corresponding cross-section in the second file for each cross-section in the Geo-RAS generated file. Most of these were channel sections only. Using the visual cross-section editor in HEC-RAS, the sections from the second file were overlaid on the first file at their corresponding locations, and the channel and overbank geometry from the first file was then replaced with the geometry from the second.

Cross-section spacing, on average, ranged between 500 and 1,800 feet. Spacing was based on hydraulic modeling requirements needed to represent channel and flood plain geometry, requirements for hydrologic storage volume computations, and requirements for GIS flood plain delineation mapping. Although modeled cross-sections were located at roadway crossings and included roadway embankments, no bridge decks or piers were modeled. The flood plain for Aquilla Creek and its tributaries is agricultural, and bridges are spaced miles apart. Therefore, detailed bridge modeling would have only minimal effect on the extent of the flood plain near the road crossings. Because the model will be used to determine changes in the extent and depth of backwater effects in the shallow upstream reach of the lake due to a pool raise, the lake itself was modeled with cross-sections. This was also a requirement for delineating the flood plain in ArcMap.

Manning's Roughness Values

Aerial photos from Google Maps and Microsoft Bing, and on-site visits were used to estimate Manning's roughness values for the new model. Roughness values for the original model (LRD-1) were based on aerial photography from the 1970's and site visits. The model was calibrated to the USGS gage on the downstream side of FM 1304 bridge. The new HEC-RAS model was calibrated to the same gage. Manning's roughness values ranged from 0.022 for the lake bottom to 0.075 for the channel, and 0.045 to 0.07 for the overbank areas. The flood plain use is primarily agricultural, with woody vegetation adjacent to the creek channels. None of the study alternatives incorporated any channel improvements or anything that would warrant changing channel or overbank roughness values in the HEC-RAS models.

Starting Water Surface Elevations

The HEC-RAS model has two control sections for starting water surface elevations – one for the reach downstream from the dam and one for the reach upstream. Control sections are at locations where the water surface is either known or can be determined, calculated or estimated without regard for downstream flow conditions. Consequently, for backwater models, these are good places to start the computations. These are typically places where there are weirs, USGS stream gages, rating curves, or estimates of water surface slopes. For the downstream control section, a short reach of the Brazos River was modeled both upstream and downstream from the mouth of Aquilla Creek. Since there was no rating curve available for the Brazos River near the downstream end of the model reach, the model used a slope-area start with an estimated slope of 0.0001 ft/ft at the downstream end of the Brazos reach. This was based on an estimate of the bottom slope of the Brazos River in this reach. The Aquilla Creek confluence was modeled as a junction. Junction inflows were the peak discharge on Aquilla Creek and the regulated control point discharge from Lake Whitney on the Brazos River. The flow downstream from the junction was the sum of the two upstream flows.

The second control section for starting water surface elevations was just upstream from Aquilla Dam. Here the different frequency peak pool elevations for each alternative were used for starting water surface elevations. For the alternative selection process, peak inflows were used. No effort was made to analyze coincident discharge and pool elevation, and how it affects the maximum water surface elevations profiles. This analysis will be performed for the selected alternatives and existing conditions.

Flood Plain Delineations

Flood plain delineations for Aquilla Creek and its tributaries downstream from the dam were developed from the HEC-RAS water surface profiles using the tools in ArcMap. Delineations were created for each of the pool raise alternatives for each set of computed frequency discharges. This information along with stage hydrograph data were used in the economic analysis (see Economics Appendix) to assess changes in downstream flood damages resulting from each alternative. Figure 12 is a delineation of the 500 year event showing the existing flood plain and the Alternative 2 flood plain downstream from the dam. This particular delineation was chosen for presentation because it shows that even with flow over the spillway, increases in downstream flooding are not easily discernible. For events where there is no flow over the spillway, the downstream floodplain does not change at all.

Figure 12 also shows the HEC-RAS flood plain delineation upstream from the dam. It should be noted that a flat peak pool elevation from the period of record analysis, rather than the HEC-RAS delineations, were used in the alternative selection analysis by the Project Delivery Team (PDT) to determine the effects of raising the conservation pool upstream from the dam. The period of record elevations were available early on in the study, whereas the HEC-RAS and HEC-HMS model results were not available until much later. Waiting for their completion would have seriously impacted the study schedule, but would not have affected the relative ranking of the alternatives. For the map scale used in Figure 12, the difference in flood plain extent for a flat pool versus a HEC-RAS delineation would not be discernible, and the area of Aquilla and Hackberry Creeks in the upstream reaches of the lake, where the differences occur, would not been large enough to change the relative rankings of the alternatives.

Automated flood plain delineations were also developed for the 2-, 5-, 10-, 25-, 50-, 100-, 250-, 500-, and 2000-year flood events for existing conditions (top of conservation pool = 537.5 feet NGVD), for the 4.5 foot pool raise (top of conservation pool = 542.0 feet NGVD), and for the 6.5 foot pool raise (top of conservation pool = 544.0 feet NGVD) on Aquilla Creek above the Lake Aquilla Dam and for Hackberry Creek. For each frequency and condition analyzed, delineations for the maximum pool and peak inflow profiles depict both the Lake and up-stream reaches. Shape files were developed by combining the maximum flood and peak inflow delineations to represent the maximum flood plain delineations for the 50-Year flood for existing conditions and the 4.5 foot and the 6.5 foot pool raise conditions. These delineations were used to determine that envelope curves for the small tributaries of Aquilla and Hackberry Creeks in the Lake area are not required. It was also determined that the pool raises on these small tributaries would not exceed the already acquired flowage easements for Aquilla Lake.

AQUILLA LAKE ENVELOPE CURVES FOR THE PROPOSED 4.5 AND 6.5 FOOT POOL RAISE CONDITIONS

Enveloping curves were developed to show how far upstream the lake dominates the water surface profiles for a particular flood event (backwater effect). An envelope curve is a curve that connects the high points of intersection of pre-project and post-project water surface profiles. An explanation of enveloping curves and their use is found in EM 1110-2-1420, Engineering and Design - Hydrologic Engineering Requirements for Reservoirs, dated 31 Oct 97. Additional Hydrologic (HEC-HMS) and Hydraulic (HEC-RAS) analyses were prepared so the envelope curves could be developed for the 4.5 foot and the 6.5 foot pool raise conditions. The Aquilla Lake project was designed for the 50-year pool, therefore discharges were determined for the 50-year flood for the maximum pool, peak inflow, discharges at the time of the peak inflow for both Aquilla and Hackberry Creeks, and 3 additional intervening points in time for existing conditions and for the proposed 4.5 and 6.5 foot pool raise conditions. Steady-state HEC-RAS profiles for each of the above listed conditions were prepared for existing conditions and for the 4.5 foot and the 6.5 foot pool raise conditions. The envelope curve points for these conditions were determined by plotting the intersection of the profile for each of the above points in time on the lake inflow hydrograph for the existing conditions profile and the corresponding profile for each of the pool raise conditions. The lowest point on the envelope curves is the point where the flood pool elevation for the lake intersects the stream invert. The resultant envelope curves depict the location of the maximum limit of the increase in flooding stage caused by the lake with the pool raises of 4.5 and 6.5 feet. See Figures 13 – 16 for the envelope curves for both pool raise conditions on Aquilla and Hackberry Creeks. The envelope curves for all pool raise conditions are contained within existing acquired flowage easements.

HEC-RAS WATER SURFACE PROFILES

Water surface profiles were plotted for the 2-, 5-, 10-, 25-, 50-, 100-, 250-, 500-, and 2000-year flood events for existing conditions (top of conservation pool = 537.5 feet NGVD), for the 4.5 foot pool raise (top of conservation pool = 542.0 feet NGVD), and for the 6.5 foot pool raise (top of conservation pool = 544.0 feet NGVD) on Aquilla Creek above the Lake Aquilla Dam and for Hackberry Creek. See Figures 17-22 for the water surface profiles.

Individual water surface profiles were plotted with the existing, 4.5 foot pool raise, and 6.5 foot pool raise conditions for the 10-, 25-, 50-, 100-, and 500-year flood events for Aquilla Creek and Hackberry Creek above the Aquilla Lake Dam. These profiles can be used to determine the approximate difference in water surface elevation for existing conditions and the 4.5 foot and 6.5 foot pool raise conditions. See Figures 23-32 for these profiles.

CONCLUSIONS AND RECOMMENDATIONS

The hydrologic and hydraulic analysis indicates that raising the top of conservation pool at Aquilla Lake from 537.5 to 540.0, 542.0, or 544.0 will not significantly impact flood risk management downstream from the dam. Even with increased operation of the emergency spillway, the maximum extent and depth of flooding downstream from the dam would still be the result of runoff from the downstream watershed for events less than a 500-year return period. There may be some increased spillway maintenance. However, with spills starting around the 250-year event rather than the 500-year event, the average annual cost increase would be quite small. Based on this analysis, any one of the alternatives is acceptable and one of the alternatives should be selected for implementation.

Figure 11 – HEC-RAS Cross-section Locations

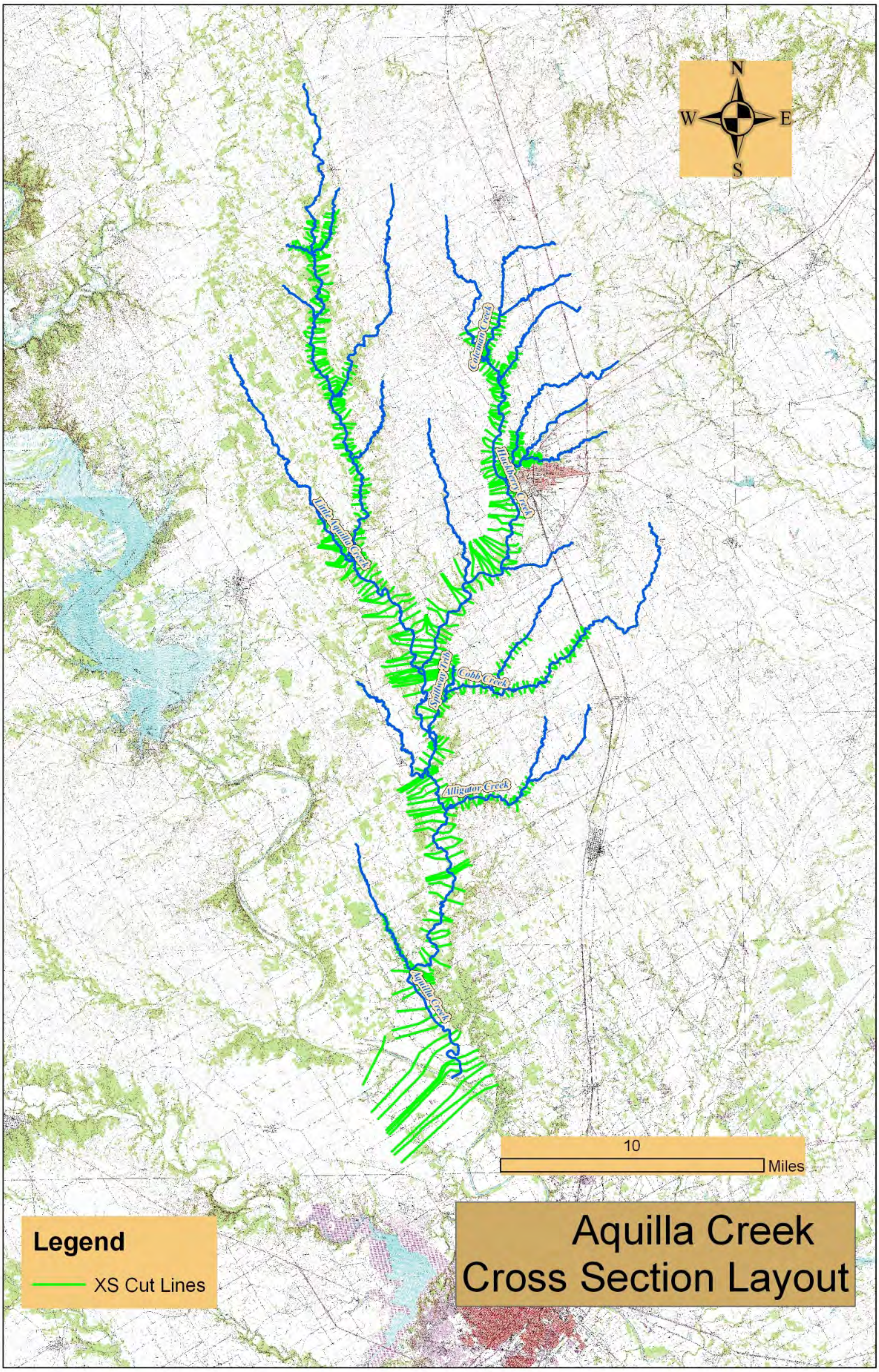


Figure 12 – Flood Plain Delineation

Aquilla Floodplain and Roads

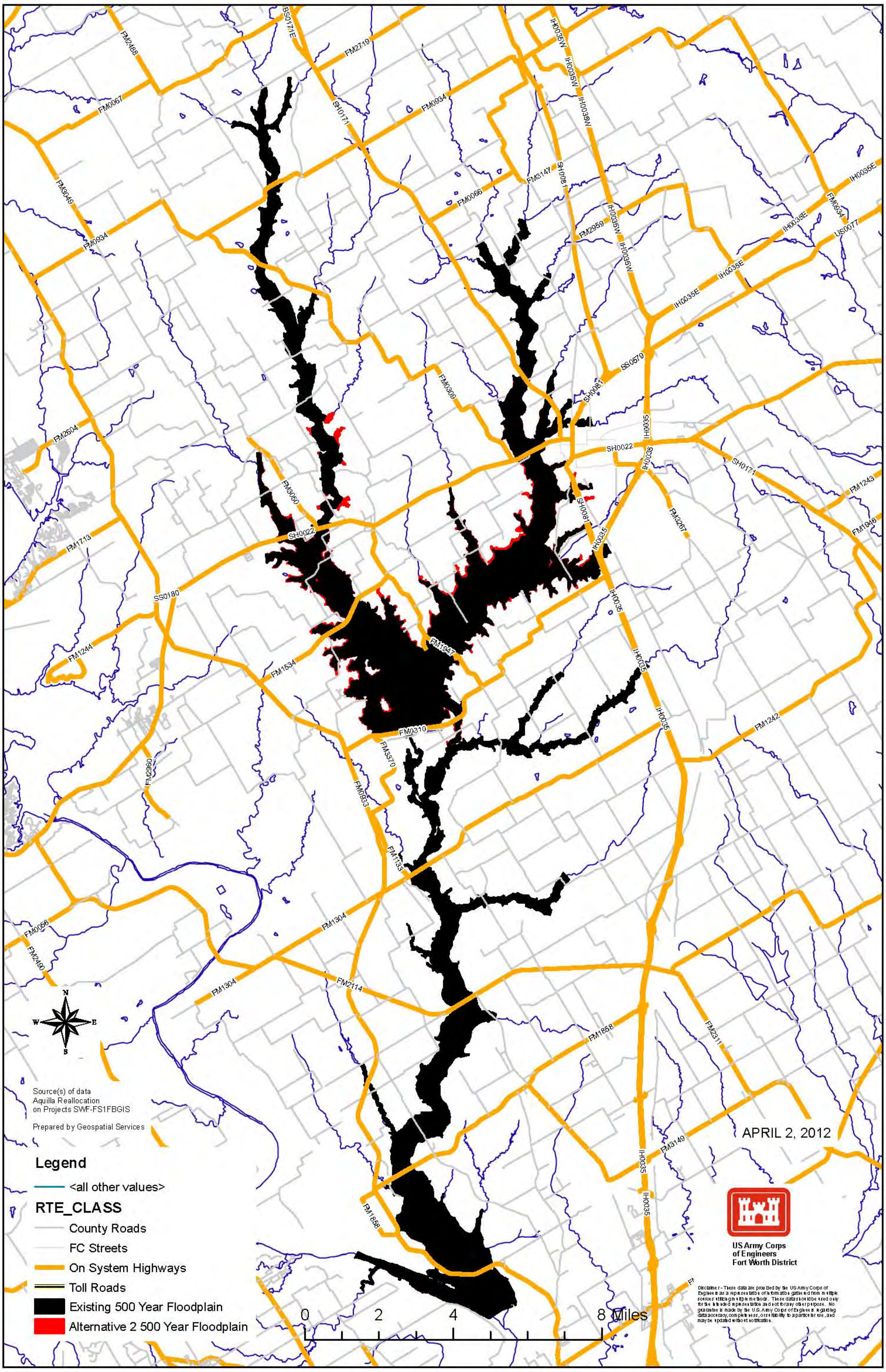
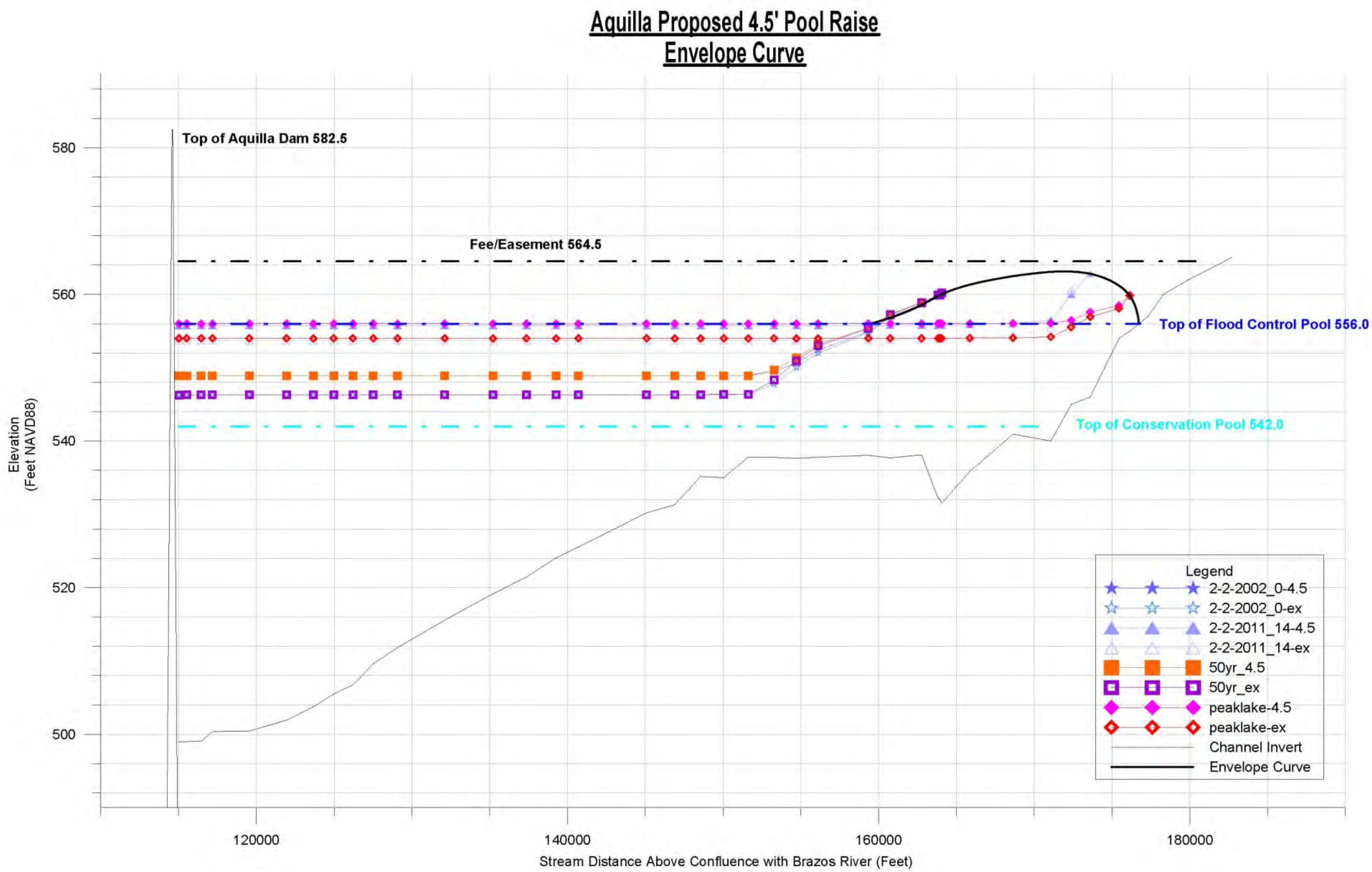
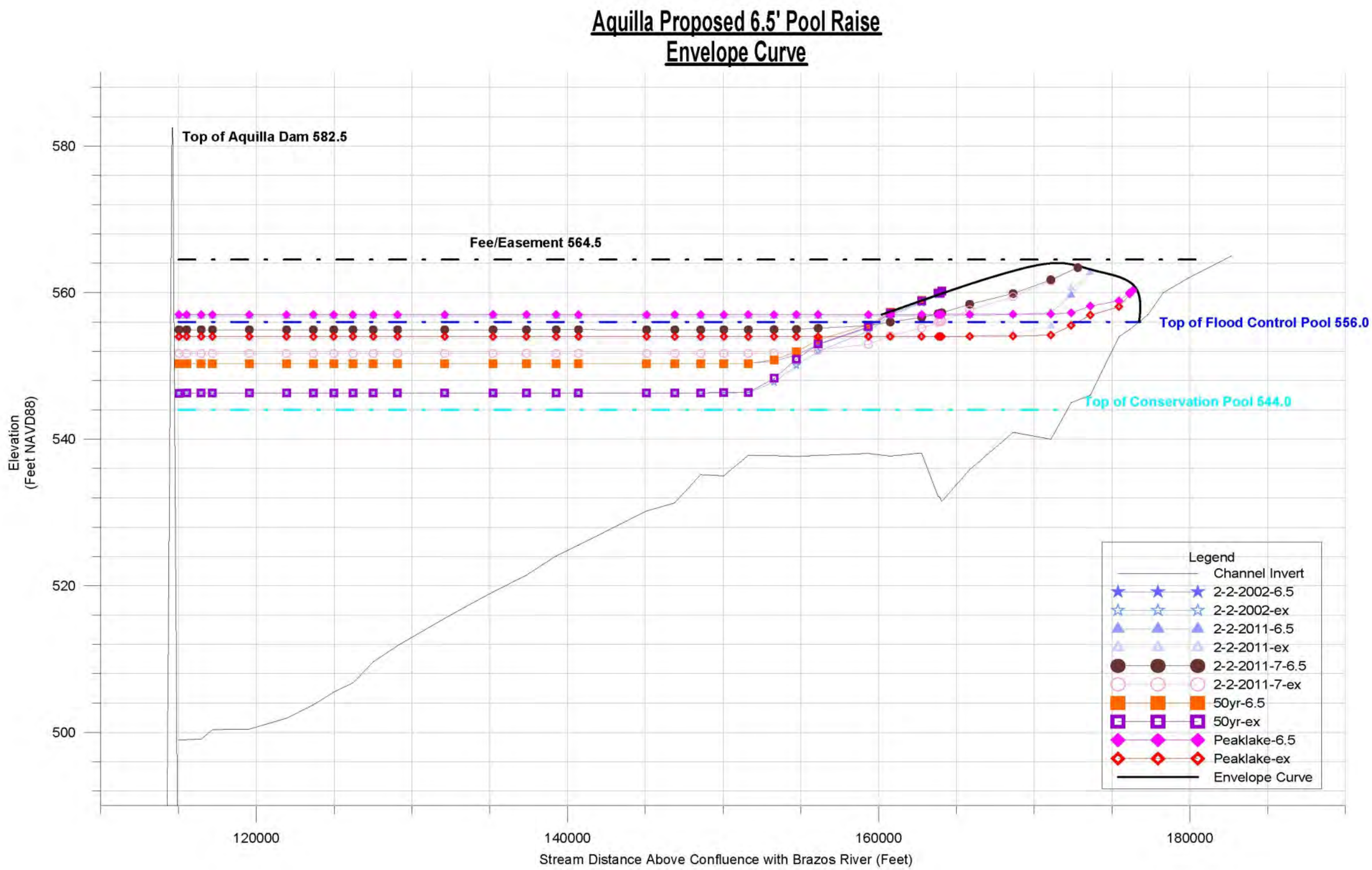


Figure 13


Aquilla Proposed 4.5' Pool Raise
Envelope Curve
Top of Aquilla Dam 582.5
Fee/Easement 564.5
Top of Flood Control Pool 556.0
Top of Conservation Pool 542.0
580
560
540
520
500
Elevation
(Feet NAVD88)
120000
140000
160000
180000
Stream Distance Above Confluence with Brazos River (Feet)
Legend
2-2-2002_0-4.5
2-2-2002_0-ex
2-2-2011_14-4.5
2-2-2011_14-ex
50yr_4.5
50yr_ex
peaklake-4.5
peaklake-ex
Channel Invert
Envelope Curve



Aquilla Proposed 6.5' Pool Raise
Envelope Curve
Top of Aquilla Dam 582.5
Fee/Easement 564.5
Top of Flood Control Pool 556.0
Top of Conservation Pool 544.0
580
560
540
520
500
Elevation
(Feet NAVD88)
120000
140000
160000
180000
Stream Distance Above Confluence with Brazos River (Feet)
Legend
Channel Invert
2-2-2002-6.5
2-2-2002-ex
2-2-2011-6.5
2-2-2011-ex
2-2-2011-7-6.5
2-2-2011-7-ex
50yr-6.5
50yr-ex
Peaklake-6.5
Peaklake-ex
Envelope Curve

Figure 14

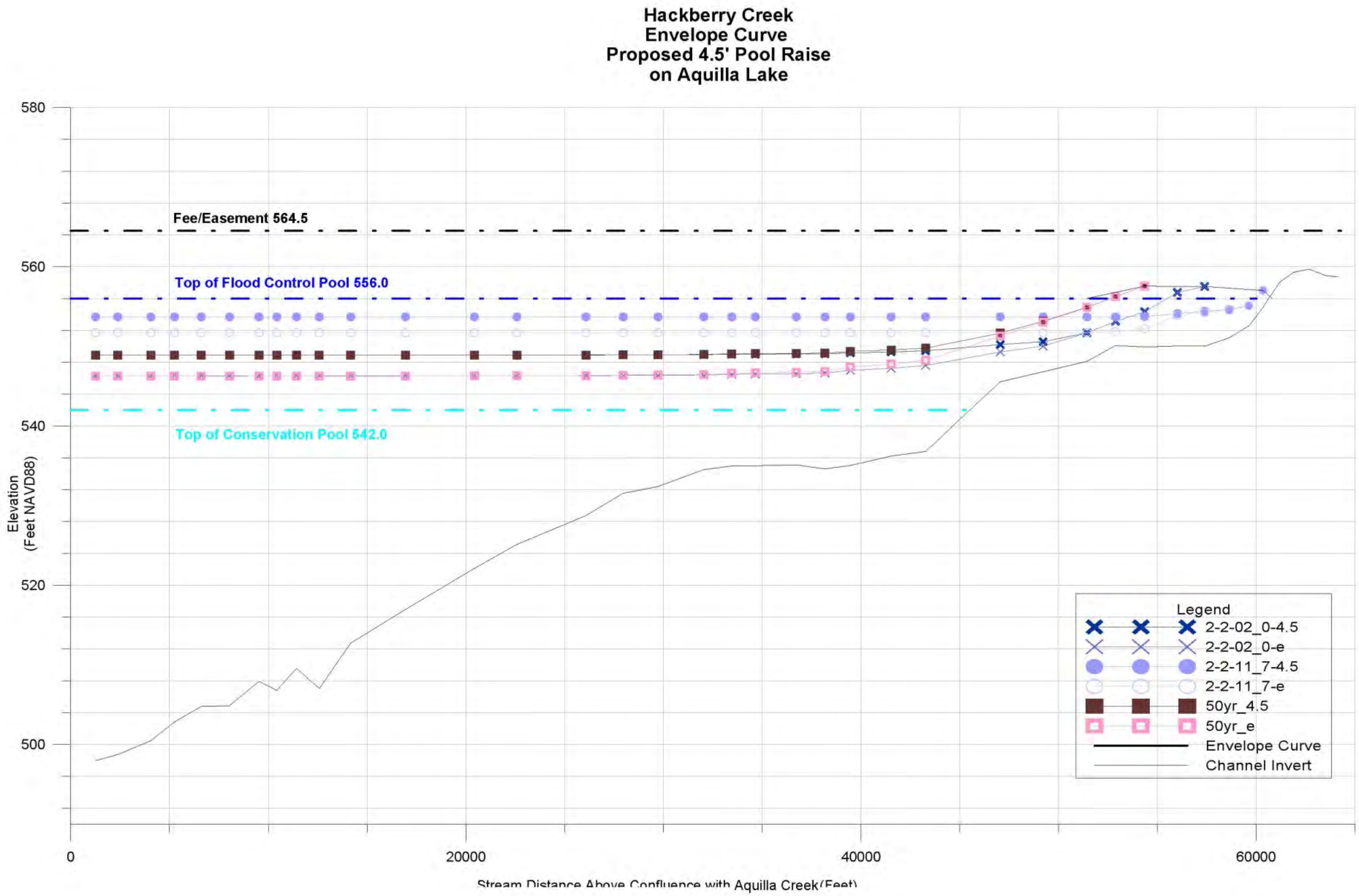

Hackberry Creek
Envelope Curve
Proposed 4.5' Pool Raise
on Aquilla Lake
Fee/Easement 564.5
Top of Flood Control Pool 556.0
Top of Conservation Pool 542.0
Elevation (Feet NAVD88)
580
560
540
520
500
0
20000
40000
60000
Stream Distance Above Confluence with Aquilla Creek(Feet)
Legend
2-2-02_0-4.5
2-2-02_0-e
2-2-11_7-4.5
2-2-11_7-e
50yr_4.5
50yr_e
Envelope Curve
Channel Invert

Figure 15

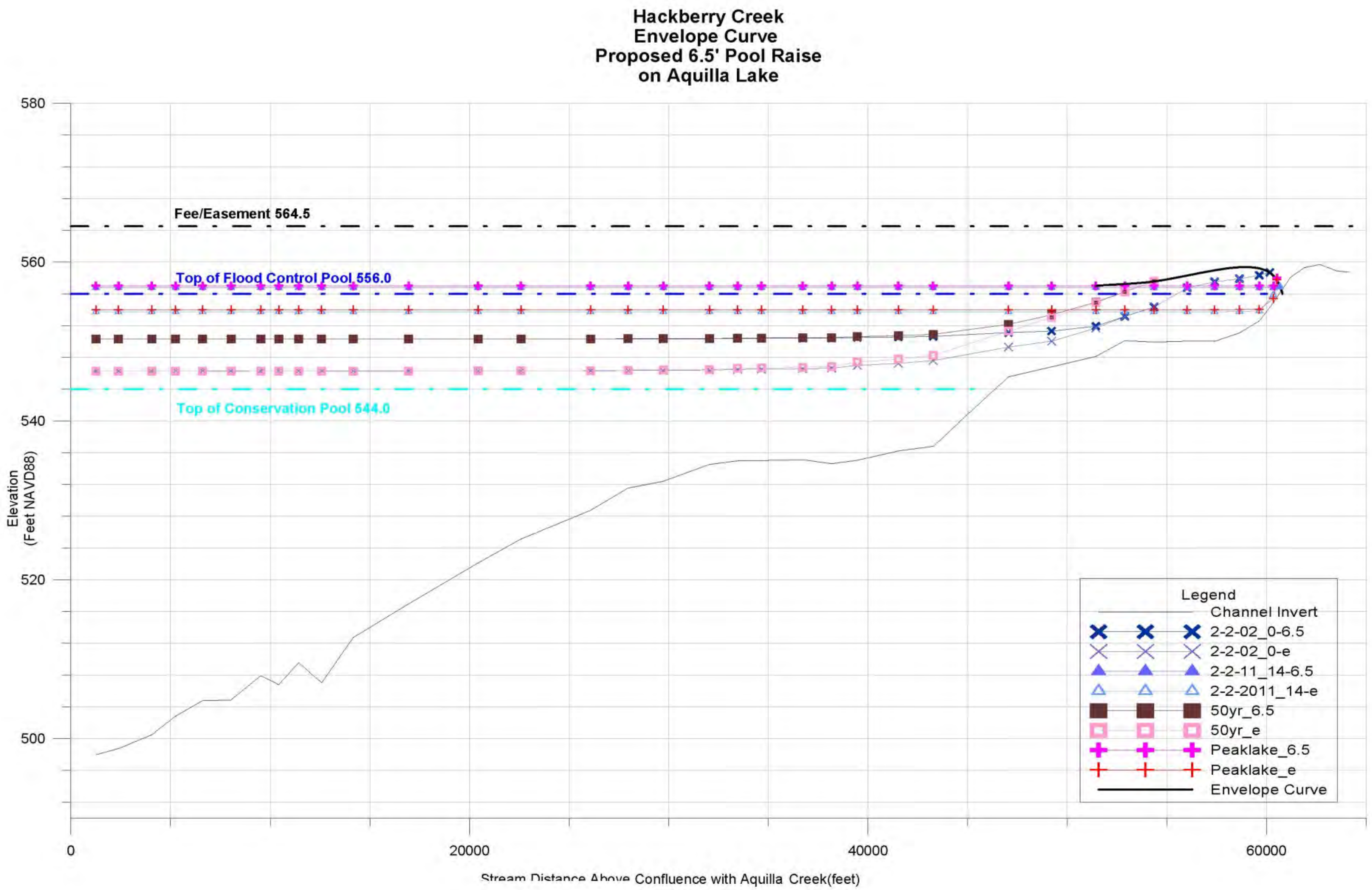
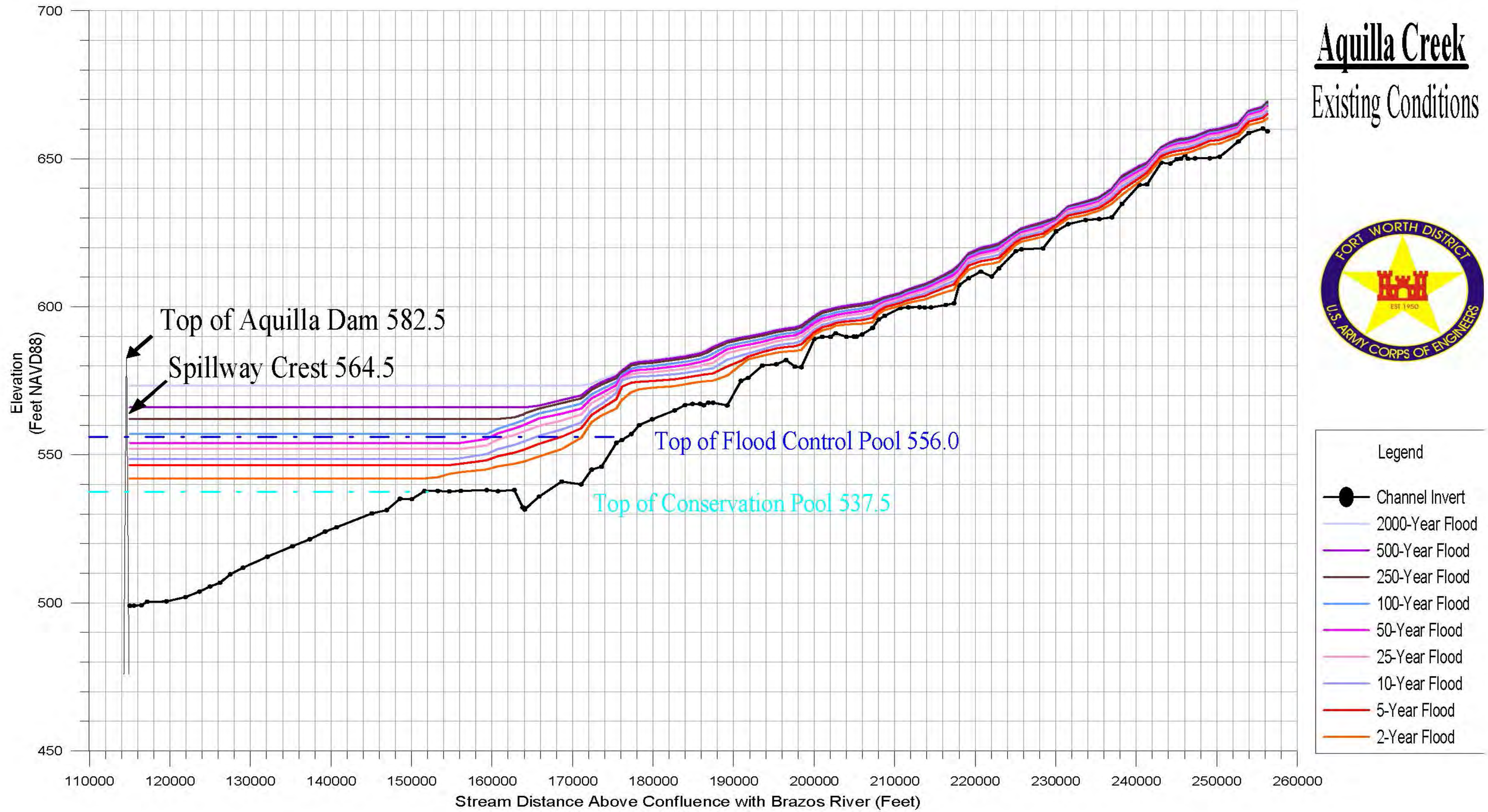

Hackberry Creek
Envelope Curve
Proposed 6.5' Pool Raise
on Aquilla Lake
Fee/Easement 564.5
Top of Flood Control Pool 556.0
Top of Conservation Pool 544.0
Elevation
(Feet NAVD88)
580
560
540
520
500
0
20000
40000
60000
Stream Distance Above Confluence with Aquilla Creek(feet)
Legend
Channel Invert
2-2-02_0-6.5
2-2-02_0-e
2-2-11_14-6.5
2-2-2011_14-e
50yr_6.5
50yr_e
Peaklake_6.5
Peaklake_e
Envelope Curve

Figure 16



Aquilla Creek
Existing Conditions
Top of Aquilla Dam 582.5
Spillway Crest 564.5
Top of Flood Control Pool 556.0
Top of Conservation Pool 537.5
Elevation (Feet NAVD88)
Stream Distance Above Confluence with Brazos River (Feet)
700
650
600
550
500
450
110000
120000
130000
140000
150000
160000
170000
180000
190000
200000
210000
220000
230000
240000
250000
260000
FORT WORTH DISTRICT
U.S. ARMY CORPS OF ENGINEERS
Legend
Channel Invert
2000-Year Flood
500-Year Flood
250-Year Flood
100-Year Flood
50-Year Flood
25-Year Flood
10-Year Flood
5-Year Flood
2-Year Flood

Figure 17

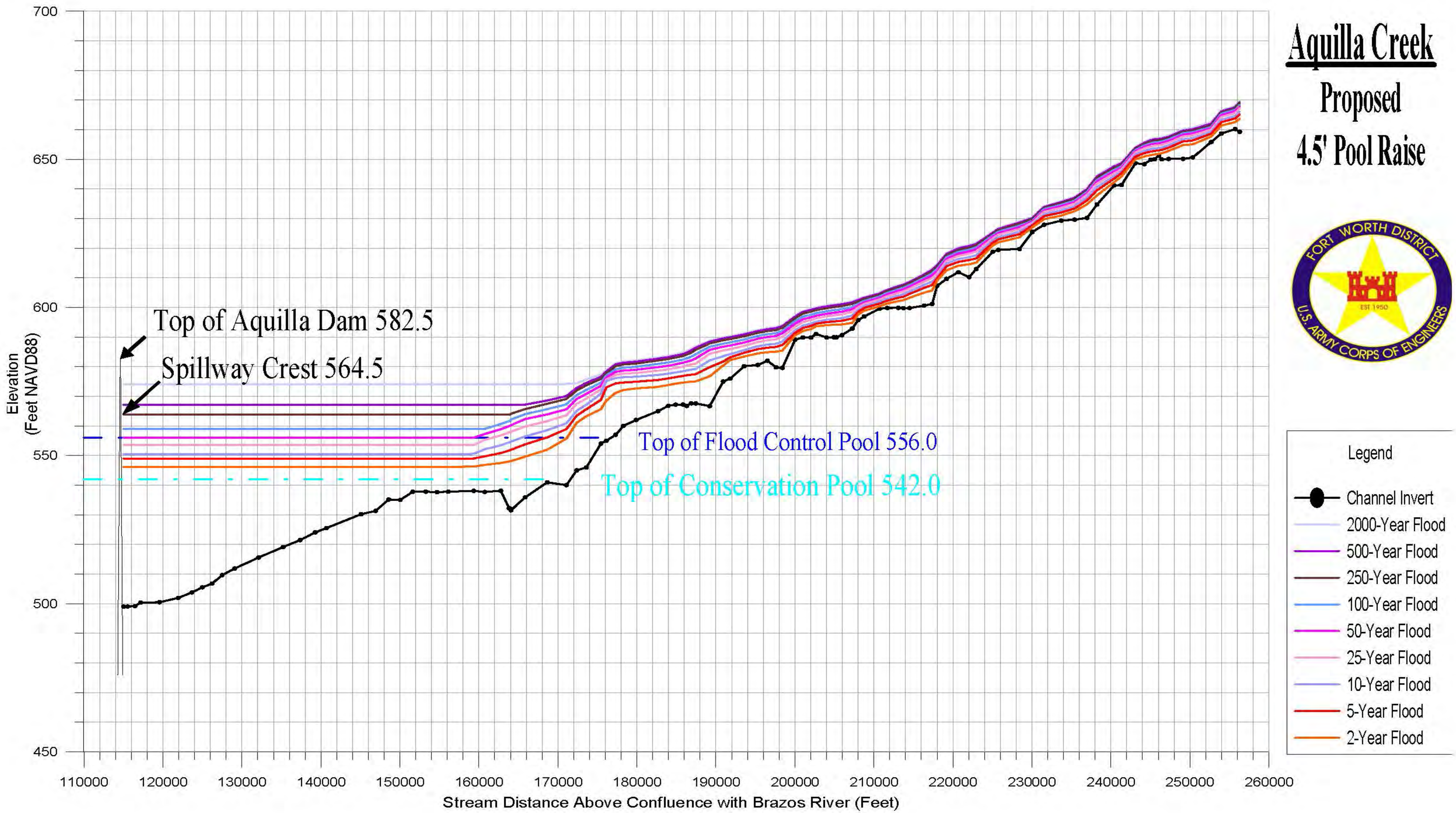
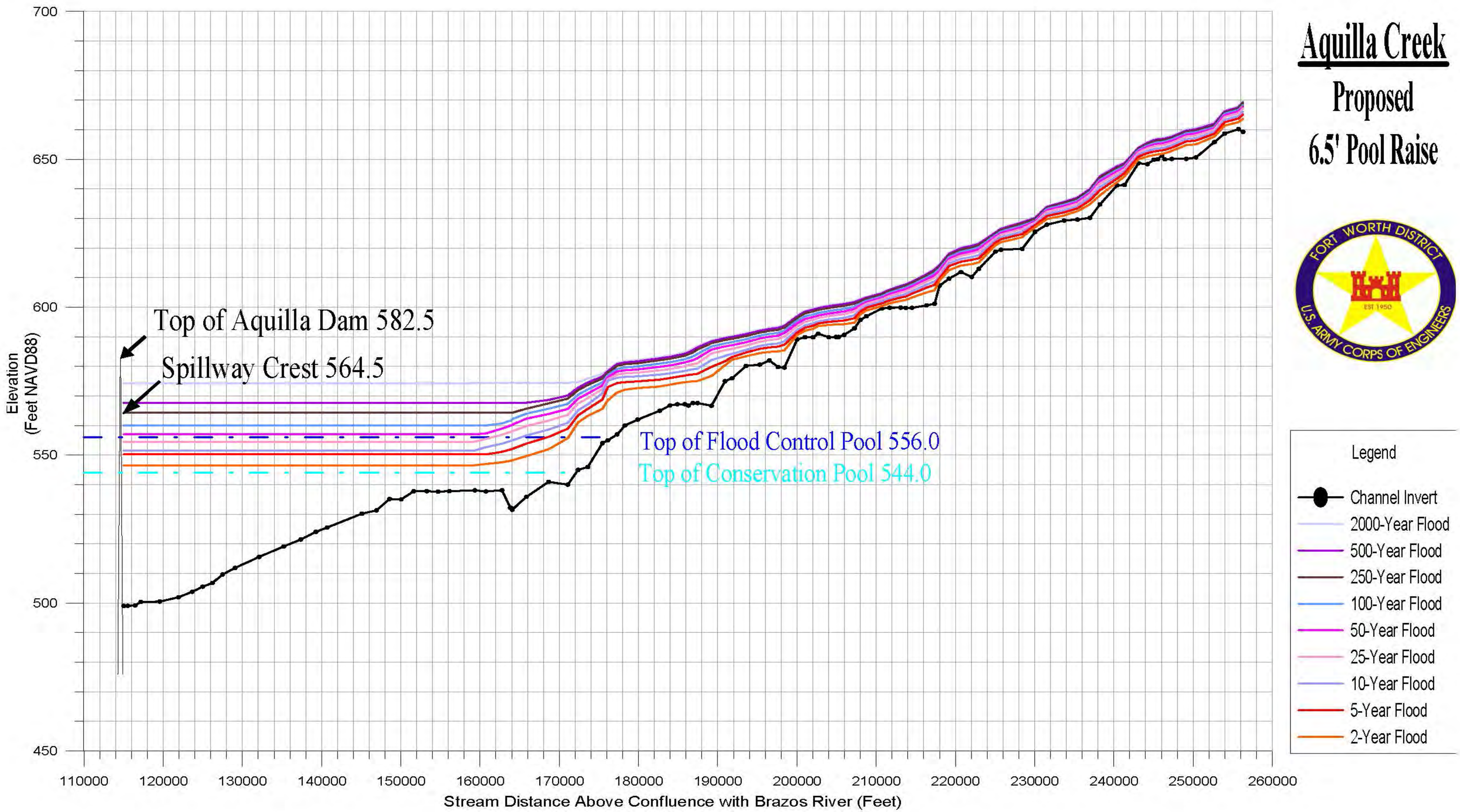

Aquilla Creek
Proposed
4.5' Pool Raise
Top of Aquilla Dam 582.5
Spillway Crest 564.5
Top of Flood Control Pool 556.0
Top of Conservation Pool 542.0
Elevation
(Feet NAVD88)
700
650
600
550
500
450
110000
120000
130000
140000
150000
160000
170000
180000
190000
200000
210000
220000
230000
240000
250000
260000
Stream Distance Above Confluence with Brazos River (Feet)
Legend
Channel Invert
2000-Year Flood
500-Year Flood
250-Year Flood
100-Year Flood
50-Year Flood
25-Year Flood
10-Year Flood
5-Year Flood
2-Year Flood

Figure 18



Aquilla Creek
Proposed
6.5' Pool Raise
Top of Aquilla Dam 582.5
Spillway Crest 564.5
Top of Flood Control Pool 556.0
Top of Conservation Pool 544.0
Elevation (Feet NAVD88)
700
650
600
550
500
450
110000
120000
130000
140000
150000
160000
170000
180000
190000
200000
210000
220000
230000
240000
250000
260000
Stream Distance Above Confluence with Brazos River (Feet)
Legend
Channel Invert
2000-Year Flood
500-Year Flood
250-Year Flood
100-Year Flood
50-Year Flood
25-Year Flood
10-Year Flood
5-Year Flood
2-Year Flood
FORT WORTH DISTRICT
U.S. ARMY CORPS OF ENGINEERS
EST 1950

Figure 19

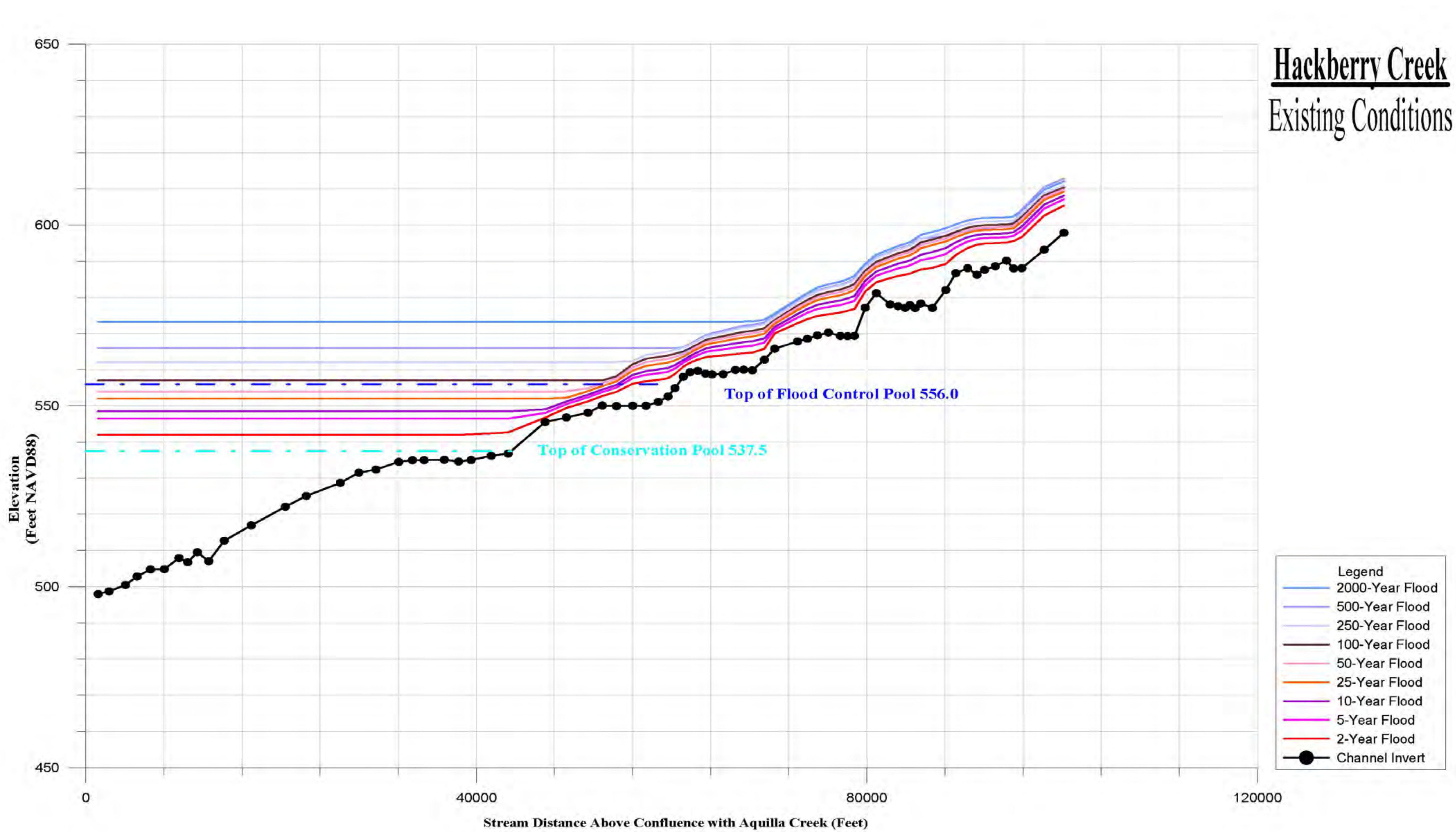

Hackberry Creek
Existing Conditions
Elevation (Feet NAVD88)
650
600
550
500
450
0
40000
80000
120000
Stream Distance Above Confluence with Aquilla Creek (Feet)
Top of Flood Control Pool 556.0
Top of Conservation Pool 537.5
Legend
2000-Year Flood
500-Year Flood
250-Year Flood
100-Year Flood
50-Year Flood
25-Year Flood
10-Year Flood
5-Year Flood
2-Year Flood
Channel Invert

Figure 20

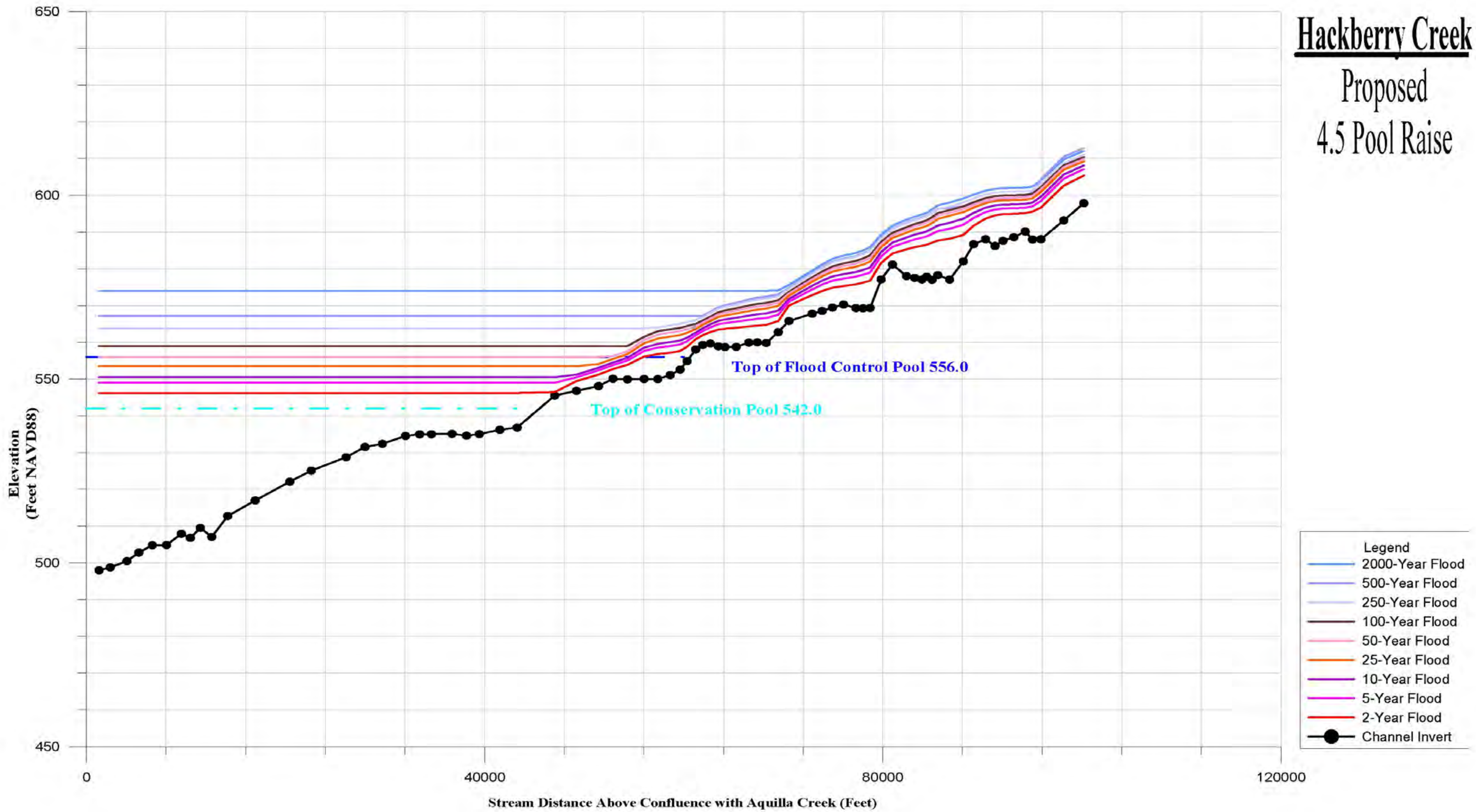

Hackberry Creek
Proposed
4.5 Pool Raise
Top of Flood Control Pool 556.0
Top of Conservation Pool 542.0
650
600
550
500
450
0
40000
80000
120000
Elevation
(Feet NAVD88)
Stream Distance Above Confluence with Aquilla Creek (Feet)
Legend
2000-Year Flood
500-Year Flood
250-Year Flood
100-Year Flood
50-Year Flood
25-Year Flood
10-Year Flood
5-Year Flood
2-Year Flood
Channel Invert

Figure 21

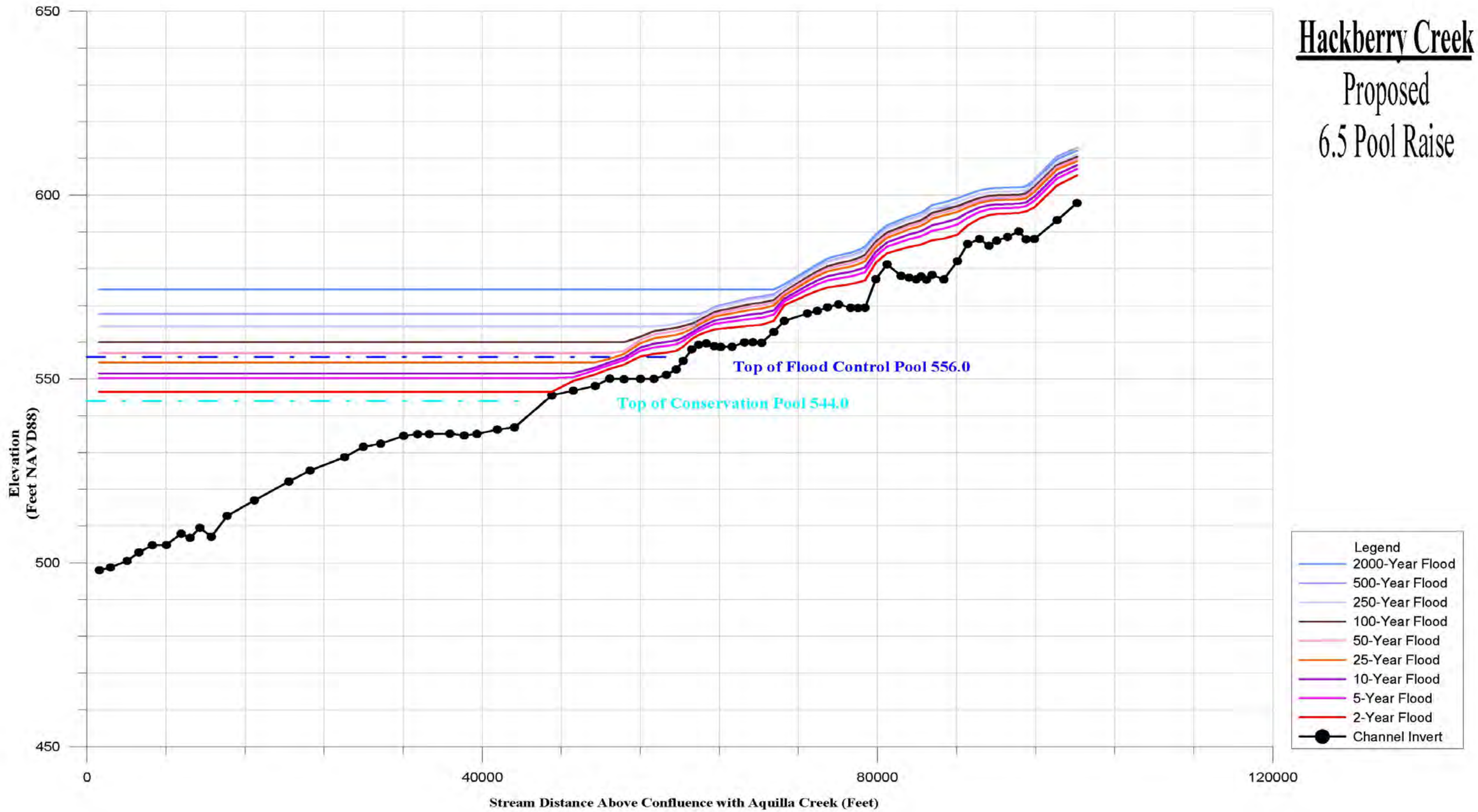
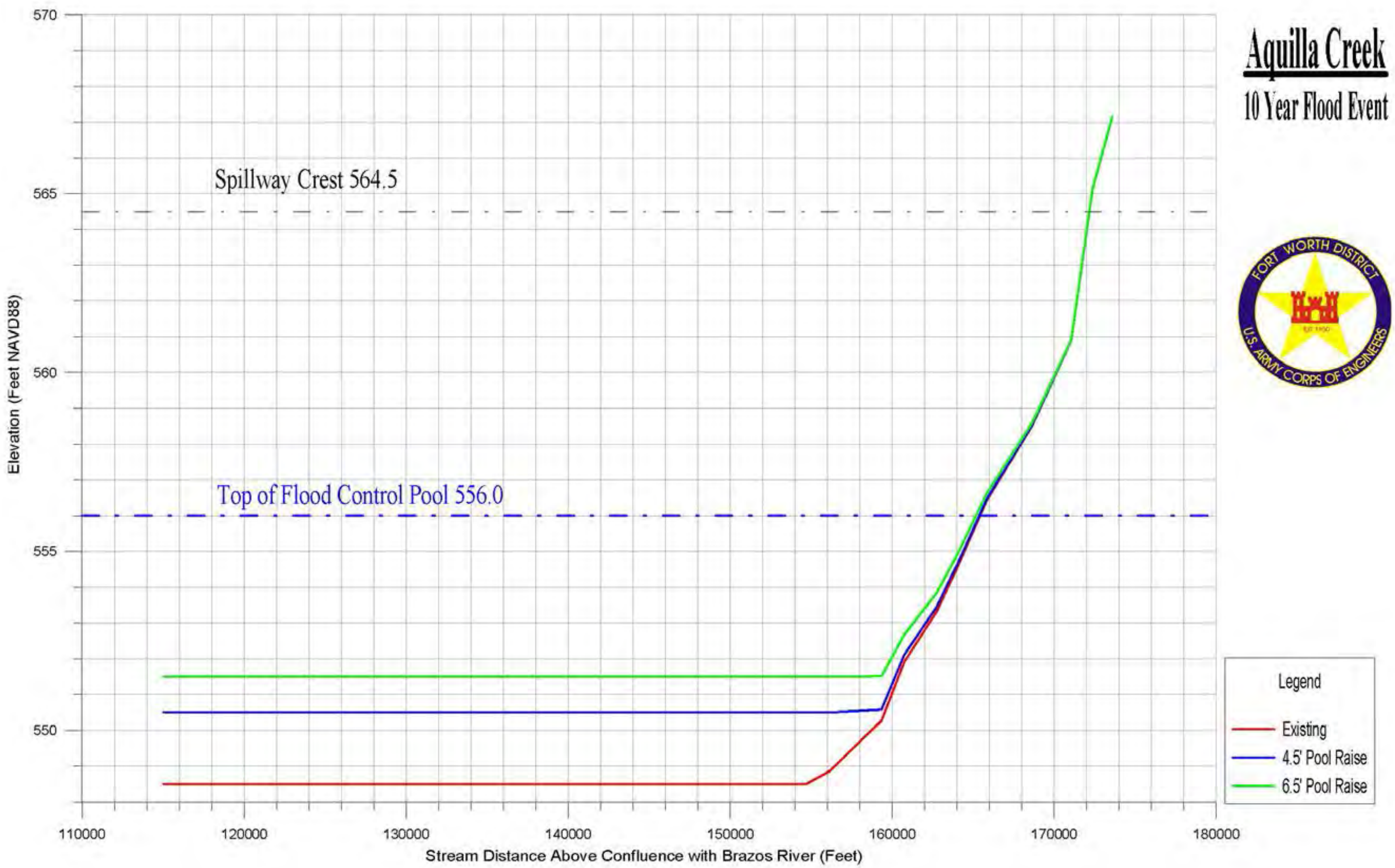

Hackberry Creek
Proposed
6.5 Pool Raise
Top of Flood Control Pool 556.0
Top of Conservation Pool 544.0
Elevation
(Feet NAVD88)
650
600
550
500
450
0
40000
80000
120000
Stream Distance Above Confluence with Aquilla Creek (Feet)
Legend
2000-Year Flood
500-Year Flood
250-Year Flood
100-Year Flood
50-Year Flood
25-Year Flood
10-Year Flood
5-Year Flood
2-Year Flood
Channel Invert

Figure 22



Aquilla Creek
10 Year Flood Event
Spillway Crest 564.5
Top of Flood Control Pool 556.0
FORT WORTH DISTRICT
U.S. ARMY CORPS OF ENGINEERS
Legend
Existing
4.5' Pool Raise
6.5' Pool Raise
Elevation (Feet NAVD88)
570
565
560
555
550
110000
120000
130000
140000
150000
160000
170000
180000
Stream Distance Above Confluence with Brazos River (Feet)

Figure 23

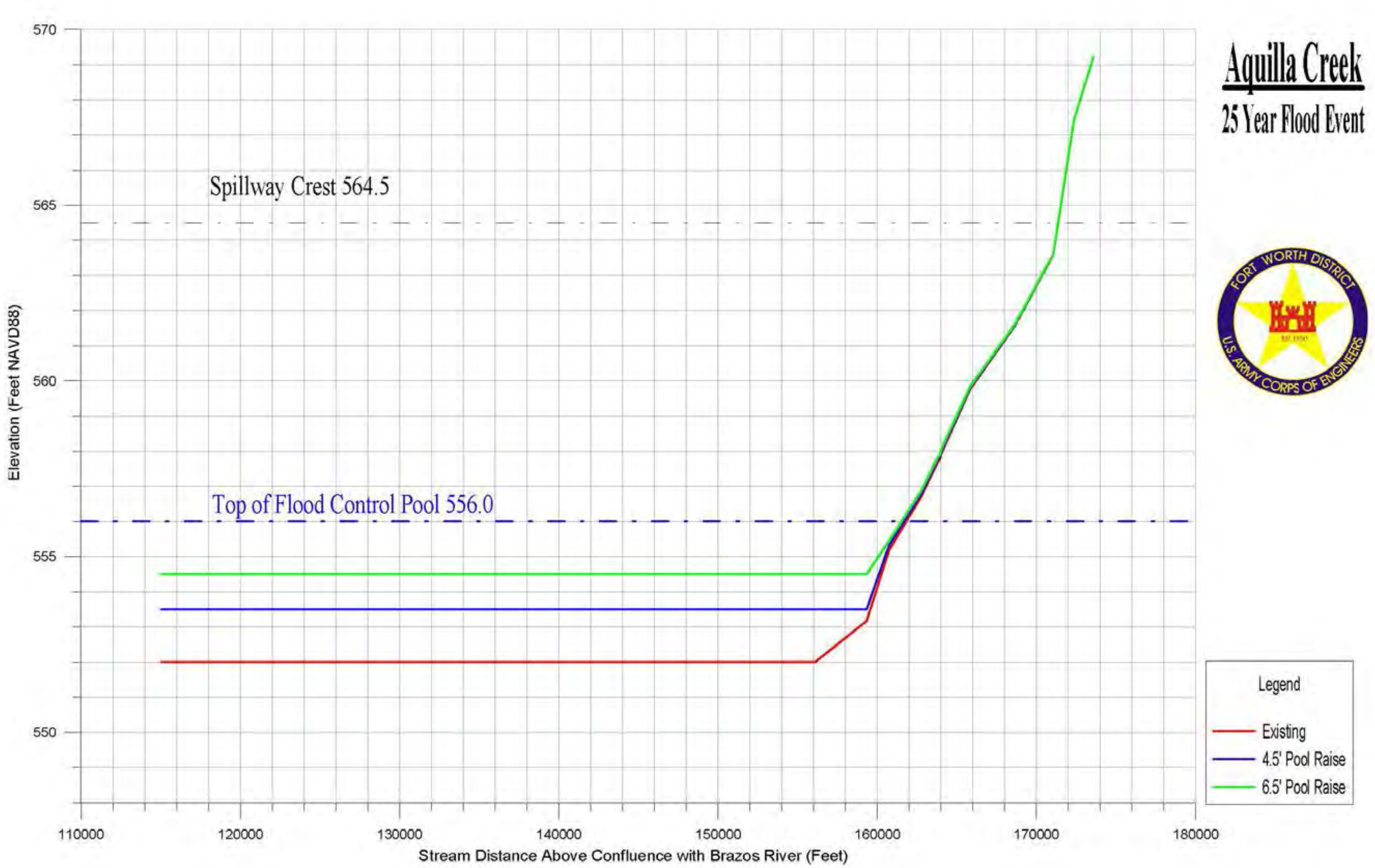

Aquilla Creek
25 Year Flood Event
Spillway Crest 564.5
Top of Flood Control Pool 556.0
FORT WORTH DISTRICT
U.S. ARMY CORPS OF ENGINEERS
Elevation (Feet NAVD88)
570
565
560
555
550
110000
120000
130000
140000
150000
160000
170000
180000
Stream Distance Above Confluence with Brazos River (Feet)
Legend
Existing
4.5' Pool Raise
6.5' Pool Raise

Figure 24

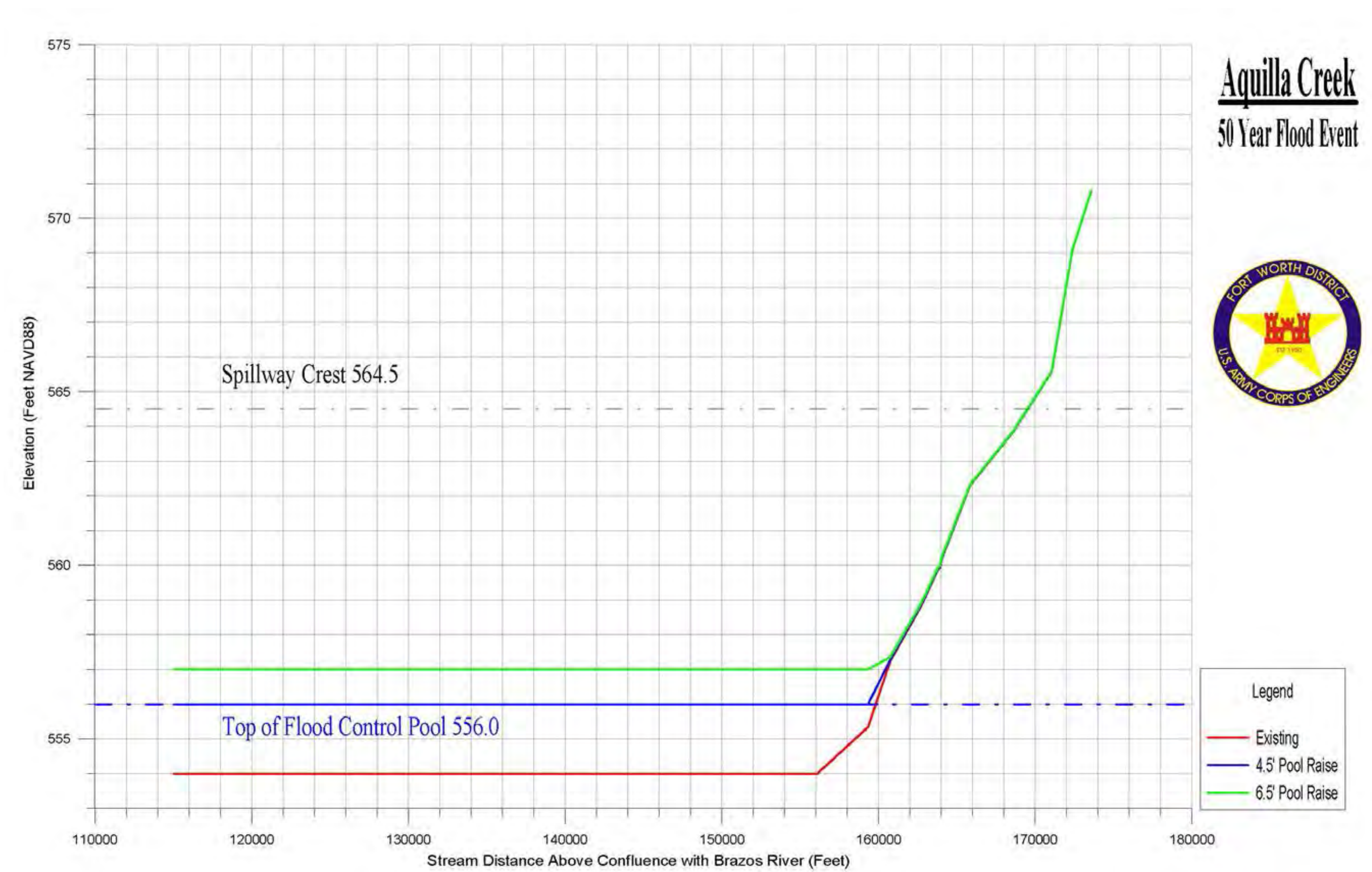

Aquilla Creek
50 Year Flood Event
FORT WORTH DISTRICT
U.S. ARMY CORPS OF ENGINEERS
Spillway Crest 564.5
Top of Flood Control Pool 556.0
Legend
Existing
4.5' Pool Raise
6.5' Pool Raise
Elevation (Feet NAVD88)
575
570
565
560
555
110000
120000
130000
140000
150000
160000
170000
180000
Stream Distance Above Confluence with Brazos River (Feet)

Figure 25

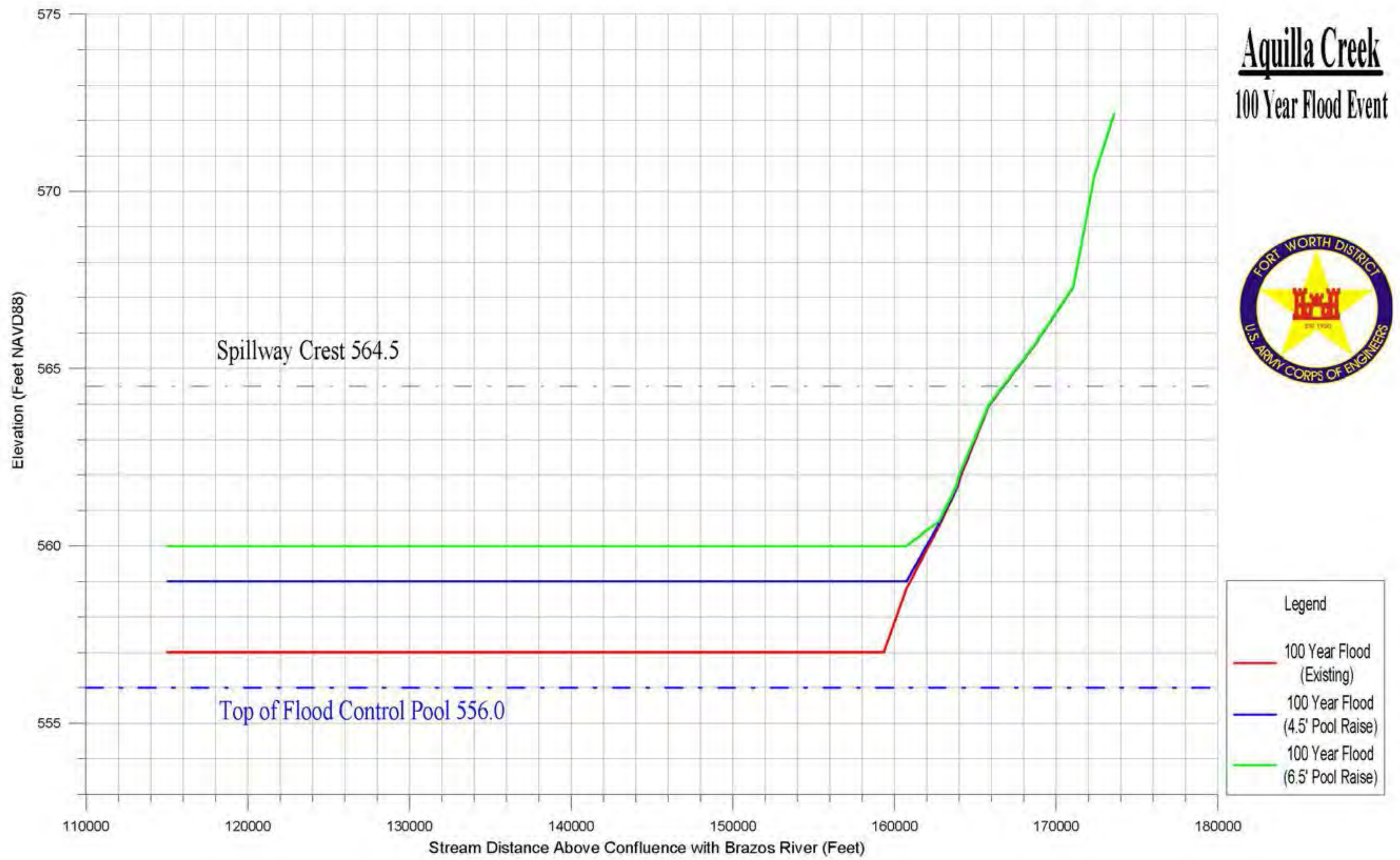

Aquilla Creek
100 Year Flood Event
Spillway Crest 564.5
Top of Flood Control Pool 556.0
Elevation (Feet NAVD88)
Stream Distance Above Confluence with Brazos River (Feet)
575
570
565
560
555
110000
120000
130000
140000
150000
160000
170000
180000
Legend
100 Year Flood (Existing)
100 Year Flood (4.5' Pool Raise)
100 Year Flood (6.5' Pool Raise)
FORT WORTH DISTRICT
U.S. ARMY CORPS OF ENGINEERS

Figure 26

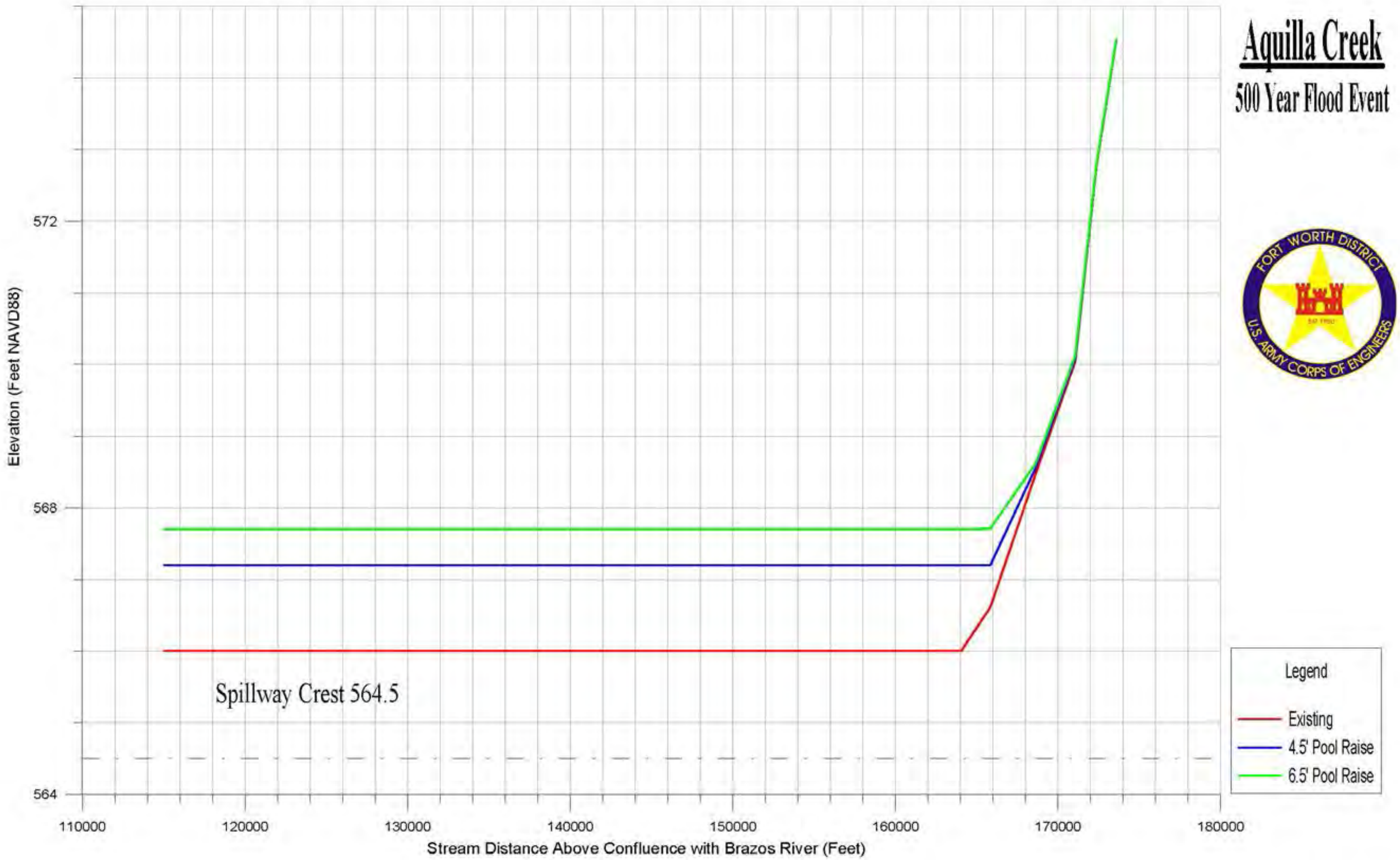

Aquilla Creek
500 Year Flood Event
FORT WORTH DISTRICT
U.S. ARMY CORPS OF ENGINEERS
Elevation (Feet NAVD88)
572
568
564
Spillway Crest 564.5
Legend
Existing
4.5' Pool Raise
6.5' Pool Raise
110000
120000
130000
140000
150000
160000
170000
180000
Stream Distance Above Confluence with Brazos River (Feet)

Figure 27

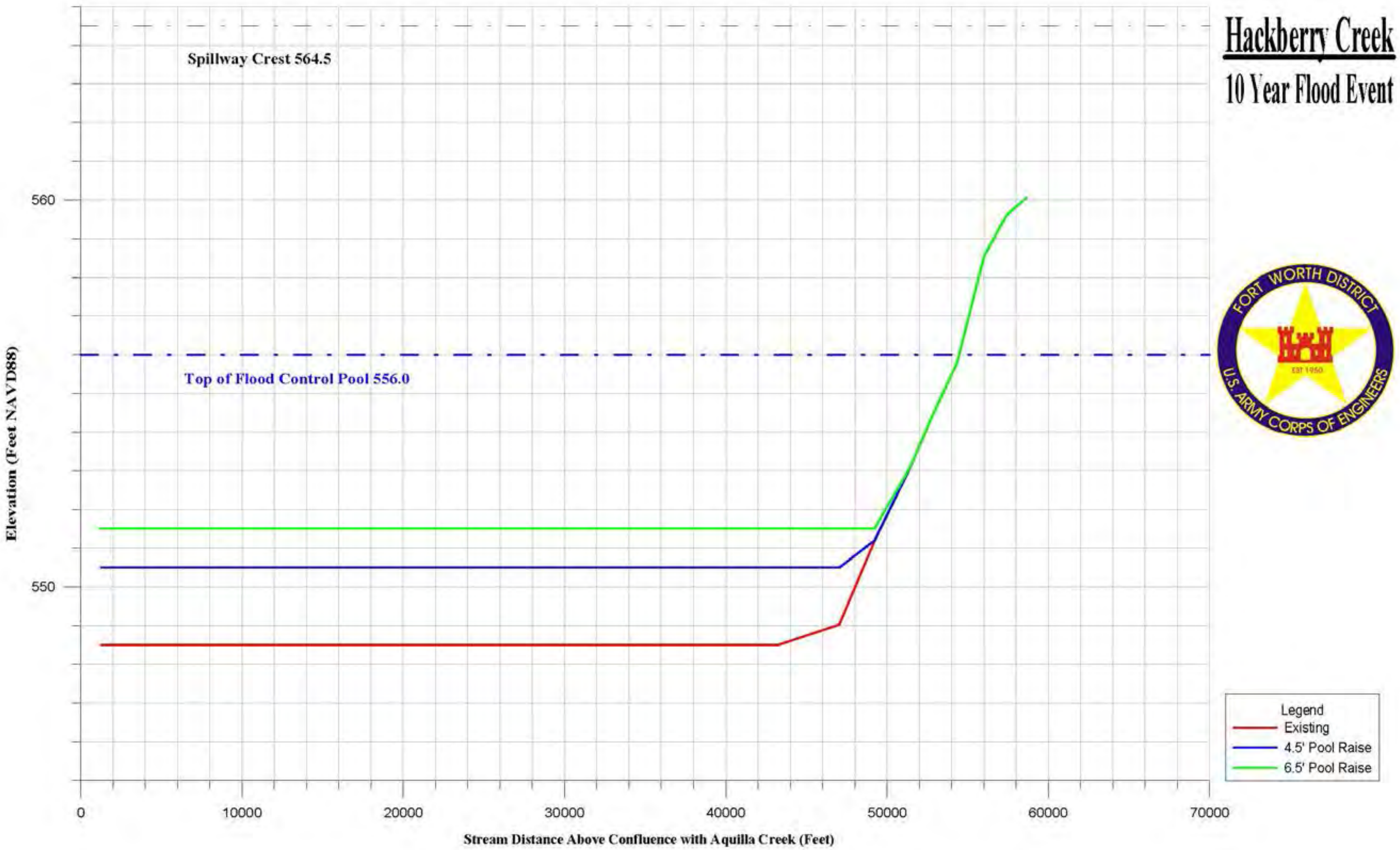

Hackberry Creek
10 Year Flood Event
Spillway Crest 564.5
Top of Flood Control Pool 556.0
560
550
Elevation (Feet NAVD88)
0
10000
20000
30000
40000
50000
60000
70000
Stream Distance Above Confluence with Aquilla Creek (Feet)
FORT WORTH DISTRICT
U.S. ARMY CORPS OF ENGINEERS
Legend
Existing
4.5' Pool Raise
6.5' Pool Raise

Figure 28

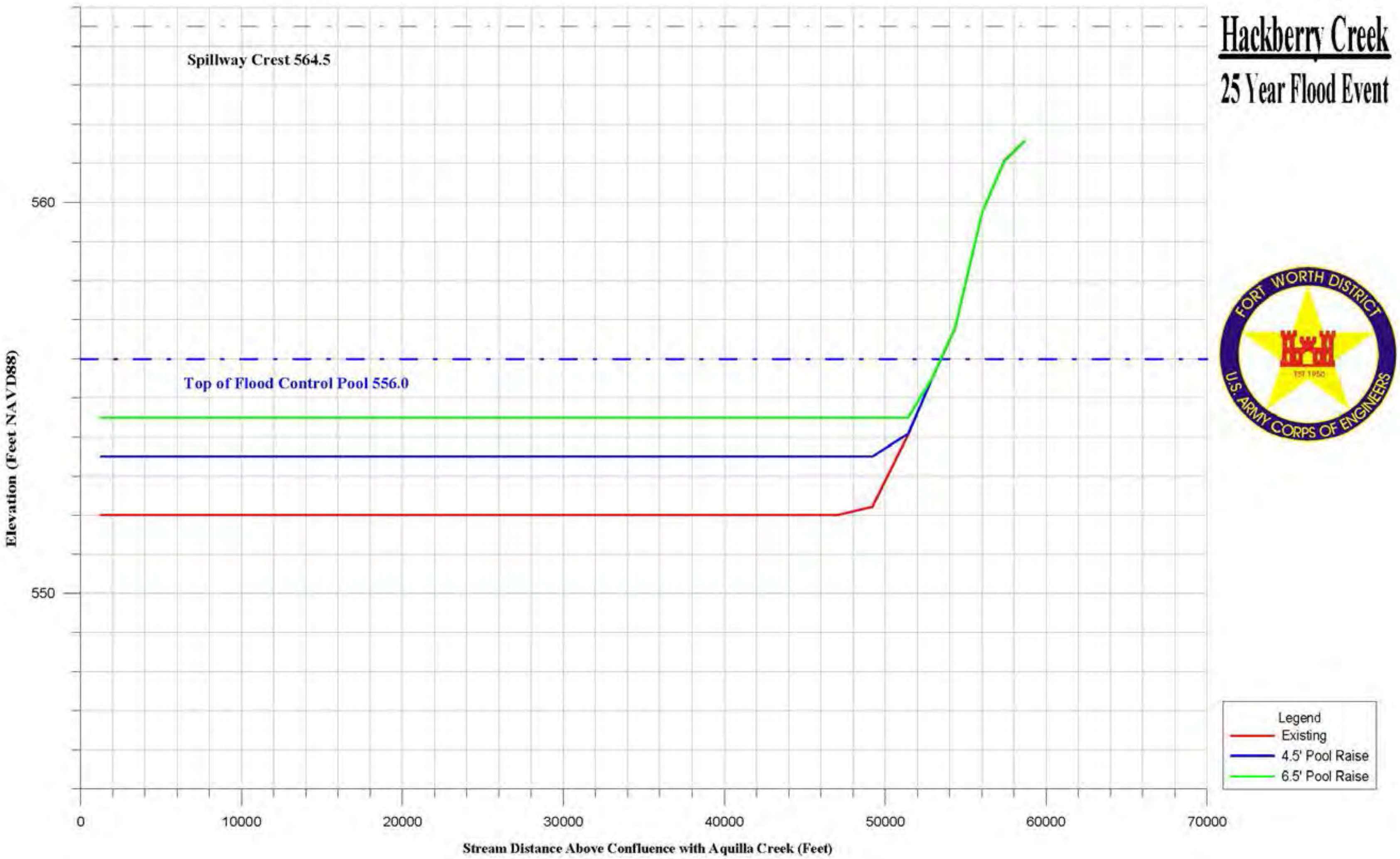

Hackberry Creek
25 Year Flood Event
Spillway Crest 564.5
Top of Flood Control Pool 556.0
Elevation (Feet NAVD88)
560
550
0
10000
20000
30000
40000
50000
60000
70000
Stream Distance Above Confluence with Aquilla Creek (Feet)
Legend
Existing
4.5' Pool Raise
6.5' Pool Raise
FORT WORTH DISTRICT
U.S. ARMY CORPS OF ENGINEERS

Figure 29

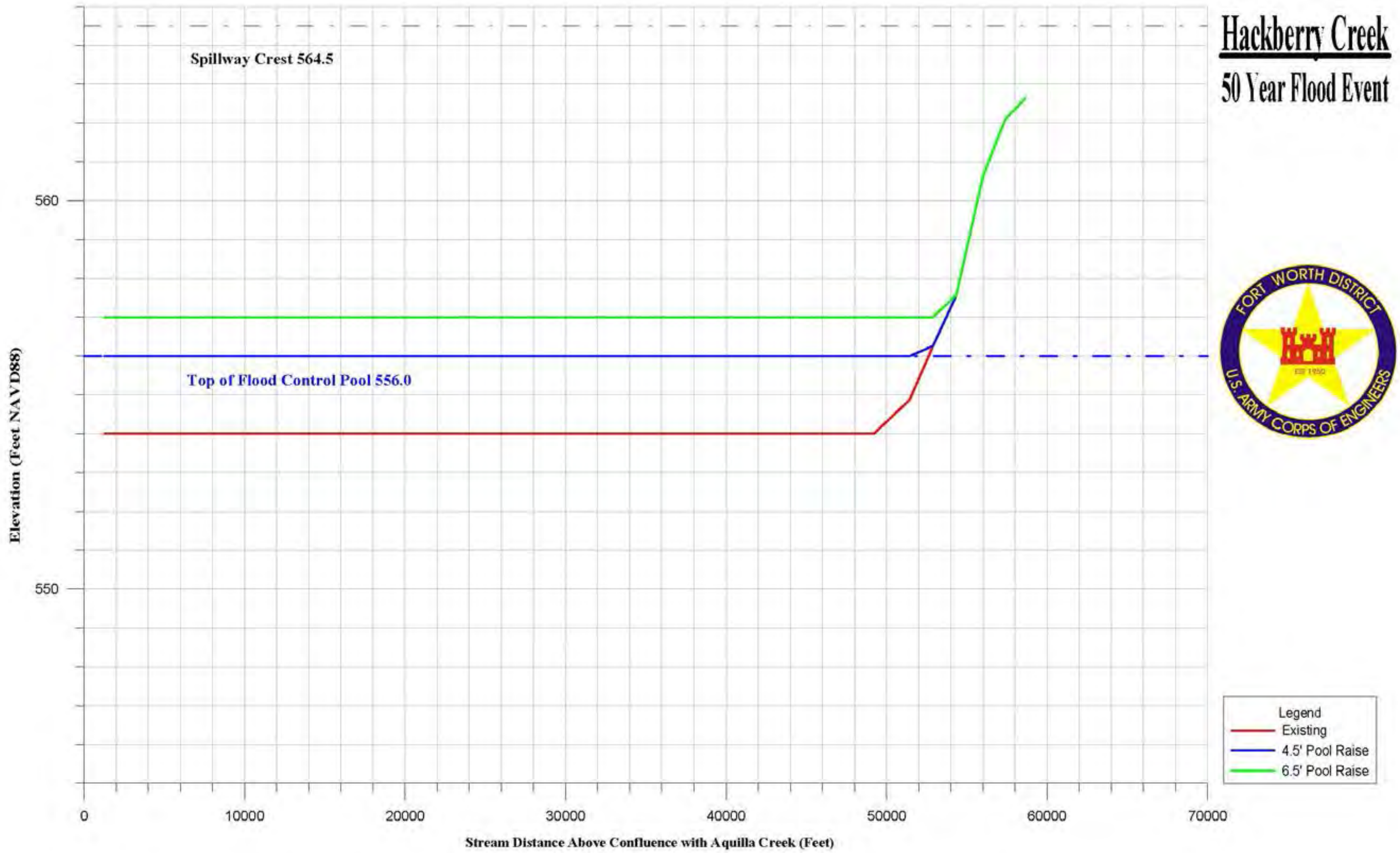

Hackberry Creek
50 Year Flood Event
Spillway Crest 564.5
Top of Flood Control Pool 556.0
Elevation (Feet NAVD88)
560
550
0
10000
20000
30000
40000
50000
60000
70000
Stream Distance Above Confluence with Aquilla Creek (Feet)
Legend
Existing
4.5' Pool Raise
6.5' Pool Raise
FORT WORTH DISTRICT
U.S. ARMY CORPS OF ENGINEERS

Figure 30

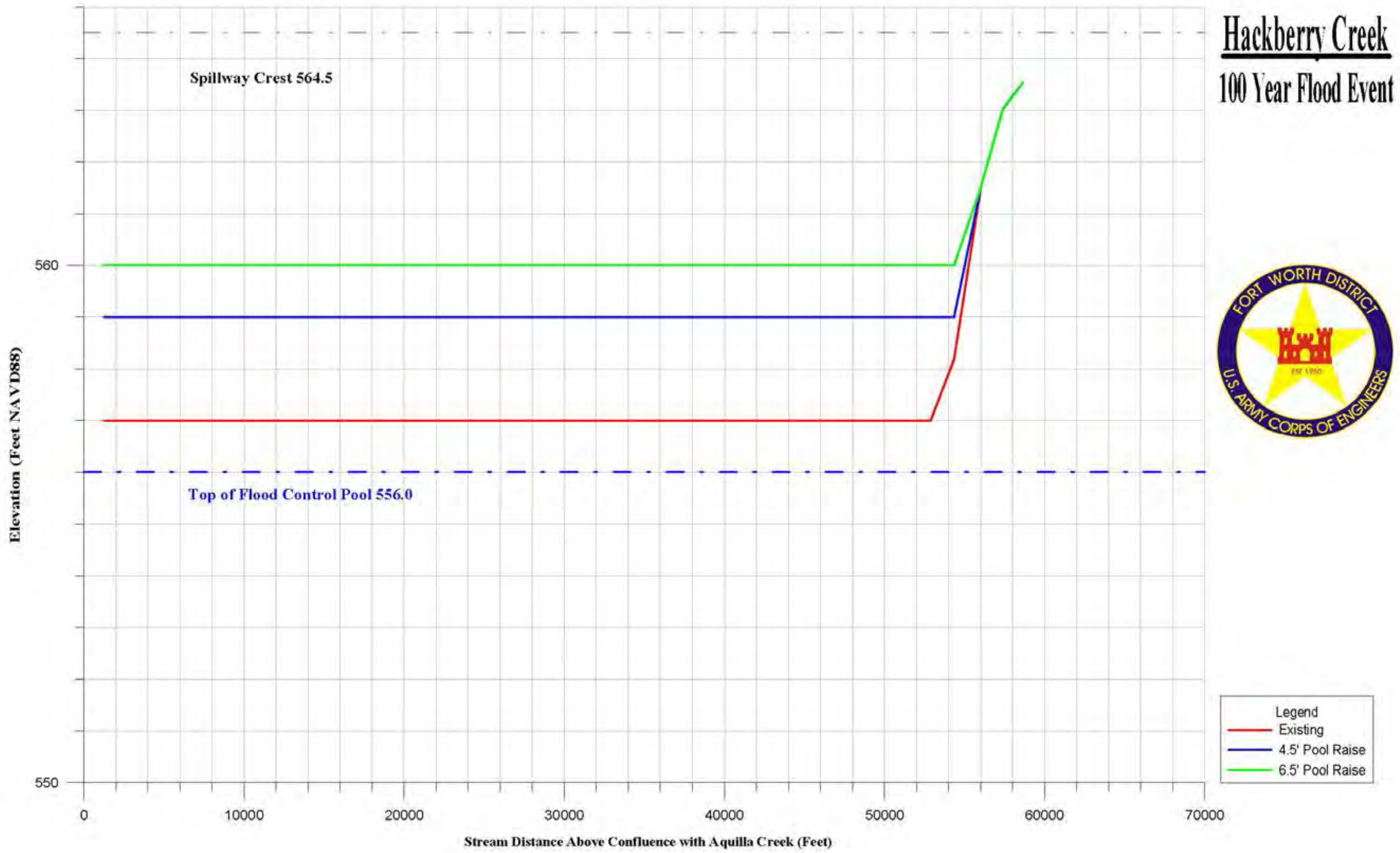

Hackberry Creek
100 Year Flood Event
Spillway Crest 564.5
Top of Flood Control Pool 556.0
FORT WORTH DISTRICT
U.S. ARMY CORPS OF ENGINEERS
Legend
Existing
4.5' Pool Raise
6.5' Pool Raise
560
550
0
10000
20000
30000
40000
50000
60000
70000
Elevation (Feet NAVD88)
Stream Distance Above Confluence with Aquilla Creek (Feet)

Figure 31

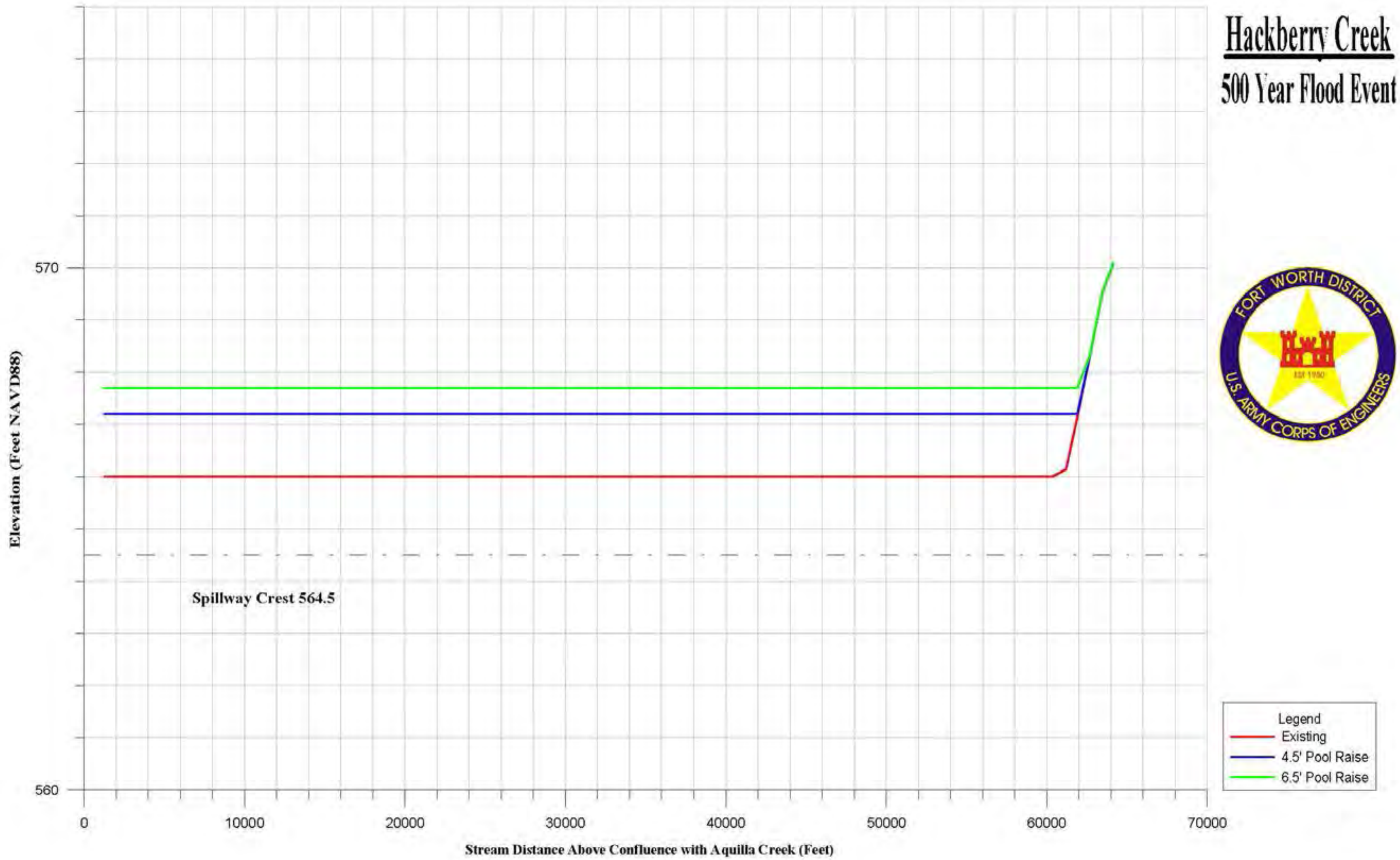

Hackberry Creek
500 Year Flood Event
570
560
Elevation (Feet NAVD88)
Spillway Crest 564.5
0
10000
20000
30000
40000
50000
60000
70000
Stream Distance Above Confluence with Aquilla Creek (Feet)
FORT WORTH DISTRICT
U.S. ARMY CORPS OF ENGINEERS
Legend
Existing
4.5' Pool Raise
6.5' Pool Raise

Figure 32