Appendix K
Water Supply Yield Analysis

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Executive Summary

The analyses and results presented in this report were performed in support of the Aquilla Lake Reallocation Study, an investigation of the feasibility of raising the top of conservation pool elevation, thus increasing water supply yield by reallocating a portion of existing flood storage space to conservation storage space. This report relates the expected hydrologic effects of three respective increments of reallocation on water supply yield, pool elevation frequency and duration, and dam discharge frequency and duration, as compared to the same parameters for existing top of conservation pool elevation. The three alternative increments of reallocation considered are represented by raising the top of conservation pool from existing elevation 537.5 feet to elevations 540.0 feet, 542.0 feet, or 544.0 feet, respectively. An additional study objective was the comparison of calculated yields produced by the SWD legacy reservoir system simulation program, SUPER, versus its replacement, RiverWare.

The hydrologic effects of the reallocation alternatives were determined based on results of a 1939-2009 period of record simulation of operation of the Brazos River Basin system of reservoirs, of which Aquilla Lake is a part. Summary results for critical water supply yield (the constant rate of water supply withdrawal that could be supported through the simulated drought of record) are presented in Table 1. These critical yield values, produced by a RiverWare system simulation, are about 0.1 cfs (less than one-half percent) less than corresponding values produced by a SUPER system simulation.

Table 1 - Critical Yield Analysis Results

Alternative	Top of Conservation Pool Elevation [ft]	Critical Yield [cfs]	Critical Yield [mgd]	Increase in Critical Yield [%]
Existing Condition	537.5	22.7	14.7	NA
Alt. #1	540.0	24.5	15.8	8%
Alt. #2	542.0	26.1	16.9	15%
Alt. #3	544.0	27.9	18.0	23%
Note: Yields are hydrologic yields based on USACE mythology.				

The maximum effect on pool elevation-frequency was associated with Alternative #3, resulting in changes ranging from a 4.5-foot increase in the 2-yr pool elevation (from 542.0 to 546.5) to a 3.2-foot increase in the 500-yr pool elevation. Detailed results for a range of frequencies are discussed further in Section 4 and presented Table 7.

The change in the pool elevation-duration relationship associated with a proposed alternative versus the existing condition was almost directly related to the proposed increase in top of conservation pool elevation. For example, the effect of raising the top of conservation pool 4.5 feet, as proposed for Alternative #2, may be expected to increase the pool elevation associated with any given duration by about 4.50 feet. The pool elevation-duration curves for the existing condition and proposed alternatives are shown on Plate 12.

The magnitude of total dam discharge associated with any alternative was increased versus the existing condition only for discharges with a return period greater than about 300 years. The existing condition 500-yr discharge of 2,950 cfs, all through the controlled outlet works, may be expected to increase to 5,700 cfs for Alternative #1; 8,550 cfs for Alternative #2; and 11,050 cfs for Alternative #3; with the increased magnitude of discharge attributable to overflow of the emergency uncontrolled spillway.

The effect of the reallocation alternatives on discharge-duration is insignificant for discharges greater than 500 cfs. The increase in the duration of lesser discharges ranges from a minimum of about 2% for a 500 cfs discharge to a maximum of about 7% for a 100 cfs discharge, with the increase in duration increasing with the magnitude of reallocation.

Wind wave run-up analyses were performed for each reallocation alternative to support geotechnical analyses of erosion protection requirements for the upstream face of the dam embankment. Maximum expected wave height, which does not vary measurably over the range in top of conservation pool elevations encompassed by the proposed alternatives, was determined to be about 3.8 feet. Wave run-up was determined to be near zero due to the combined attenuating effects of very shallow water at the toe of the dam and a relatively flat, rip-rap covered slope.

1.0 Introduction

1.1 Purpose of Study

Aquilla Lake is one of a system of many reservoirs in the Brazos Basin. The water supply storage in this system is administered by the Brazos River Authority (BRA), an agency of the State of Texas. As BRA looks at future water demands in the system, they anticipate a need for increased water supply from Aquilla Lake. This study examines the effects of reallocating flood storage to conservation storage for use as water supply. The objective of the simulations was to evaluate the increase in yield between the existing condition with a top of conservation pool of 537.5 feet and three alternative top of conservation pools of 540, 542, and 544 feet. An additional study objective was to assess the difference in calculated yields between the SWD legacy reservoir system simulation program, SUPER, and its replacement, RiverWare.

As part of the reallocation study for Aquilla Lake, this report documents a 1939-2009 period of record reservoir system simulation analysis based on current Water Control Plans of Regulation for the Brazos River Basin reservoirs. The objective of the analysis was to provide updated Aquilla Lake pool elevation-frequency, pool elevation-duration, total discharge-frequency, and total discharge-duration curves for existing conditions and each of the three alternatives. System simulations of long periods of record provide a means of modeling the expected response of the current system to a wide range of hydrologic events occurring both pre-project and post-project, thereby providing a basis for better definition of expected frequency and duration relationships than could be had based solely on post-construction observed data.

1.2 Project Purpose and Authorization

The primary purposes of the Aquilla Lake project are flood control, water supply, and recreation. Congressional authority for the construction of Aquilla Lake, a unit in the plan of improvement of the Brazos River Basin, Texas, was outlined in the Flood Control Act approved 13 August 1968.

1.3 Project Location and Description

Aquilla Dam is located at river mile 23.3 on Aquilla Creek, Brazos River Basin, Texas in Hill County, approximately three miles east of Aquilla, Texas. The project consists of a rolled earth filled embankment with an impervious core, an uncontrolled broad-crested weir spillway, and a gated outlet works.

1.4 Basin Description

The headwaters of Aquilla Creek begin southeast of Cleburne in southwestern Johnson County in Texas and flows 54 miles south-southeast to its confluence with the Brazos River. The river drops from an elevation of about 850 feet at its source to 478 feet at the Aquilla Dam site. Aquilla Creek continues to drop to elevation 380 feet at its confluence with the Brazos River. The average streambed slope from headwater to the confluence is about 8.5 feet per mile.

The principal tributaries contributing to Aquilla Creek are Cottonwood, Little Aquilla, Hackberry, Cobb, and Alligator Creeks. Cobb Creek and Alligator Creek are located downstream of Aquilla Lake.

1.5 Climatology

The climate in the Aquilla Creek watershed is subtropical with cool winters and hot humid summers. Maritime tropical air masses from the Gulf of Mexico play a dominant role in the climate from late spring through early fall, while polar air masses determine the winter climate. Warm seasonal rainfall is largely the result of thunderstorm activity, with amounts varying considerably in both intensity and location.

This area experiences a continental type of climate, characterized by a wide range between annual extremes of temperature. Cold, high pressure air masses from the northwestern polar regions and continental highlands cause occasional snowfall and freezing temperatures.

Because of the preponderance of maritime tropical air, heavy showers of short duration may occur at any time during the year. The mean annual precipitation over the Aquilla Creek watershed is approximately 35 inches.

2.0 System Simulation Model

2.1 Model Design

RiverWare 6.0.4, developed by the Center of Advanced Decision Support for Water and Environmental Systems (CADSWES), University of Colorado, has been approved for use in support of USACE studies and incorporated into the Corps Water Management System (CWMS) for real time operations. RiverWare was used to model the Brazos River Basin and simulate system reservoir operations for the 1939–2009 period of record. Obtaining sound results at any location in the Brazos River Basin requires simulation of the entire system because reservoir regulation decisions depend on the status of other reservoirs in the system and on downstream conditions. Although the discussion in this section includes specific information only for Aquilla Lake, the methods and procedures discussed generally apply for all elements of the model.

The same principles apply to SUPER, the Southwest Division's (SWD) reservoir system simulation legacy software. SUPER was also used to simulate the system reservoir operations for the 1939-2009 period of record for the entire Brazos Basin.

RiverWare and SUPER are multi-purpose models that simulate both conservation and flood control operations on a daily time step basis (the smallest time step for which historic hydrologic data is commonly available). The basic hydrologic input data includes uncontrolled reservoir inflow (total reservoir inflow minus routed releases from upstream reservoirs) and downstream uncontrolled flows accumulated at respective downstream control points in the modeled area. A basin description file describes reservoir physical characteristics, downstream control point flood flow constraints, water supply requirements, and low flow requirements. Computational control points are defined for each reservoir and each downstream point of interest. For each time step of a simulation, the change in reservoir storage is computed by subtracting the sum of water supply withdrawals and dam releases made in accordance with the water control plan of regulation from the sum of uncontrolled inflow and routed releases from upstream reservoirs. Analogously, computed flow at a downstream control point consists of accumulated uncontrolled flow between a control point and the next upstream control point, combined with routed flow from the next upstream control point. Detailed program documentation for RiverWare is available on the CADSWES website (see 6.0 – References).

2.2 Hydrologic Input Development
Hydrologic input consists of daily uncontrolled reservoir inflow and uncontrolled flow at each downstream control point for the area between a given control point and the next upstream control point, be it a stream station or a reservoir. Hydrologic input was developed from observed daily stream flow, precipitation and evaporation data, and computed daily reservoir inflow. Records of mean daily stream flow were obtained from U.S. Geological Survey (USGS) published data and USACE records of observed flow. The USGS daily mean stream flow data for each gauging station is available via the internet (see 6.0 – References). USACE records of observed stream flow and computed reservoir inflow are available upon request from the Fort Worth District Water Management Section. Precipitation and evaporation data were obtained from published National Oceanic and Atmospheric Administration (NOAA) Climatological Data Annual Summaries.

2.2.1 Post Project Data
Development of hydrologic input is simplified for the portions of a simulated period of record for which computed reservoir inflow and observed stream flow are available at all control points. Reservoir total inflow is computed and recorded daily as a part of normal reservoir regulation procedures based on observed change in reservoir storage, dam releases, direct lake withdrawals, and evaporation data collected at or near the reservoir site. Daily uncontrolled inflow to a reservoir for use in a system simulation is computed by subtracting routed releases from upstream reservoirs, if any, from the reservoir total inflow. Daily uncontrolled flow at a downstream control point is computed by subtracting flow routed from the next upstream control point from observed flow at the downstream control point and adding known withdrawal from the reach.

2.2.2 Pre-project Data
Records of observed stream flow, evaporation, and precipitation at nearby stations are used to develop the required reservoir and downstream control point hydrologic input for the portions of a simulated period of record for which computed reservoir inflow and/or observed stream flow are not available.

Pre-project uncontrolled reservoir inflow is estimated by analyzing observed flow at nearby stream stations and adjusting the results to reflect the difference in drainage area between the observed flow station and the uncontrolled area upstream of the reservoir. Since this estimated record of uncontrolled reservoir inflow does not reflect precipitation on or evaporation from the lake surface, an additional “net evaporation” hydrologic input record is required to support simulation computations for the pre-project portion of a simulated period of record. Net evaporation is computed as evaporation minus precipitation, based on the closest weather stations for which data is available. Net evaporation for a given day, which may be a positive or negative quantity, is applied to the lake surface area during the simulation to compute the contribution of rainfall and evaporation to change in reservoir storage. Uncontrolled flow at a downstream control point for pre-gauge and/or significant periods of missing record is analogously estimated by analyzing observed flow at a nearby stream station and adjusting the results to reflect the difference in drainage area between the observed flow station and the uncontrolled area between the control point of interest and the next upstream control point.

To illustrate, the development of the Aquilla Lake period of record daily inflows was broken into several time periods dictated by data availability at nearby stations. The Jan1939-Sep1979 period inflows were developed by multiplying the Aquilla Creek near Aquilla gauging station flows by a drainage area ratio factor since the available data was collected at a point where the drainage area was 308 sq-mi versus the 255 sq-mi area upstream of Aquilla Dam. The USGS Aquilla Creek above Aquilla gauging station was used to fill in the data from Oct1979-Apr1982 and the CORPS Aquilla Creek above Aquilla gauging station was used to fill in the data from May1982-Apr1983. For the period May1983-Dec2009, during which Aquilla Lake was operational, uncontrolled inflow to the lake was based on USACE records of computed daily total inflow. The available data records used for development of the record of uncontrolled inflow to Aquilla Lake are shown schematically on Plate 1. The referenced gauging stations are listed in Table 2.

Table 2 - Example Gauging Stations for Hydrologic Data

Gauge Station	Agency	Basin Area [sq-mi]	Date Range for Data
Aquilla Lake above Aquilla	CORPS	255.0	May83-Dec09
Aquilla Creek above Aquilla	USGS	255.0	OCT79-APR82/JUN01-DEC09
Aquilla Creek above Aquilla	CORPS	255.0	MAY82-MAY01
Aquilla Creek near Aquilla	USGS	308.0	JAN39-APR01

2.3 Reservoir Elevation-Storage Relationship

The most recent *Volumetric and Sedimentation Survey for Aquilla Lake* (see 6.0 – References) was completed by the Texas Water Development Board (TWDB) in March 2008, shown in Plate 2. The results from this study indicate Aquilla Lake has a total reservoir capacity of 44,577 acre-ft at top of conservation pool elevation 537.5 feet. An additional 15,073 acre-ft of water can be stored between top of conservation pool elevation and 542.0 feet, for a total reservoir capacity of 59,650 acre-ft at elevation 542.0 feet. Comparisons of capacities at conservation pool elevation derived from current and previous surveys suggest Aquilla Lake looses between 84 acre-ft per year and 218 acre-ft per year of conservation storage space. That is equivalent to 0.33 to 0.85 acre-ft per square mile of drainage area.

2.4 Routing Methods

The RiverWare step response routing method used in USACE simulation models for river reach routing is based on linear routing coefficients developed from Muskingum routing coefficients. This concept was developed and is utilized in SUPER. The method was programmed into RiverWare using C++. The method states that given a daily flow volume entering a routing reach on Day 1, a series of daily coefficients designate the percentage of the flow volume leaving the reach on Day 1, Day 2, Day 3, etc. The sum of the coefficients equals one.

The level pool routing method is used for reservoir routing.

2.5 Water Control Plan

The Corps' Brazos River Basin flood control projects are operated as a system with the primary goal of minimizing downstream flood damages. Flood control releases from Aquilla Dam are coordinated with releases from other lakes in the Brazos River Basin. Flood waters stored in the nine projects operated by the Corps of Engineers are released as soon as downstream channel capacity is available. The lake levels are lowered to their conservation pools at the earliest possible date in order to provide available flood storage for future flood events. Controlled releases from Aquilla Lake are made at a rate that, when combined with flows from downstream areas, will not exceed the controlling channel capacities shown in Table 3.

Table 3 - Key Downstream Control Points

River Channel and USGS Gauging Station	Control Capacity (cfs)
Aquilla Creek above Aquilla	3,000
Brazos River near Aquilla and Aquilla Creek above Aquilla	25,000
Brazos River near Waco	60,000
Brazos River near Hempstead	60,000
Brazos River at Richmond	60,000

As a flood event develops and the lake elevation is forecasted to rise between elevations 537.5 feet and 538.0 feet, a minimum release of 25 cfs is made, provided the controlling downstream flows shown in Table 3 will not be exceeded. Analogously, a minimum release of 50 cfs is made for a forecasted lake elevation between 538.0 feet and 538.5 feet; a minimum release of 100 cfs is made for a forecasted lake elevation between 538.5 feet and 539.0 feet; a minimum release of 500 cfs is made for a forecasted lake elevation between 539.0 feet and 540.5 feet. As the lake elevation rises above 540.5 feet and is forecasted to continue to rise, the rate of release, subject to downstream controls, is increased to 3000 cfs.

If the lake elevation is forecast to rise above spillway crest elevation 564.5 feet, releases when combined with spillway discharges should not exceed the control capacities at the downstream control points. By approximately elevation 565.7 feet, spillway discharge is 3,000 cfs and all gates are closed. As the elevation rises higher, downstream controls are necessarily exceeded.

2.6 Water Supply

The Brazos River Authority (BRA) has an Aquilla Lake water supply storage contract dated 5 April, 1976 for 33,600 acre-ft below elevation 537.5 feet. The remaining 18,800 acre-ft below conservation pool is allocated for sediment storage. Presently, BRA has activated 25,493 acre-ft (75.87%) of the total water supply contract as of 2012.

The Cities of Cleburne and Aquilla pump directly from Aquilla Lake on an as needed basis. The two cities share the intake structure on Aquilla Lake. The amount of water pumped daily by each city is reported to the Fort Worth District, Water Management Section at the end of the Fiscal Year for an annual report. Releases from the outlet works are made to supply water for withdrawals by downstream users. BRA makes telephone requests to the Water Management Section for water supply releases.

2.7 Model Calibration and Verification

The nature and purpose of system simulation modeling limits the portion of long period of record simulation results that may be compared to observed data for purposes of model calibration and verification. The observed record inherently reflects changes in system response to hydrologic events over time due to the addition of storage reservoirs, storage reallocations in existing reservoirs, changes in reservoir water control plans of regulation, or changes in the magnitude of water supply withdrawals. System simulation results reflect strict adherence to the rules prescribed in the reservoir water control plans of regulation, and dam releases that reflect optimal use of downstream channel capacity via some degree of perfect forecast knowledge. The observed record reflects the effects of human judgment with regard to meeting overall water control plan objectives and estimating the timing and magnitude of dam releases based on imperfect forecast knowledge. Simulation results are therefore compared to observed data only for that portion of the period of record for which observed data may be expected to reflect the current state of the system, and matching results on a daily basis is not expected. The comparisons of reservoir pool elevation, dam release volume, and flow volume at downstream control points are made to verify that simulation results generally reflect observed trends and extremes. The calibration process was only performed for the existing condition. Once the satisfactory calibration was met with the existing conditions, the resulting model was used for the alternatives with their equivalent Top of Conservation Pool elevations.

SUPER was SWD's approved software for system simulation modeling prior to implementing RiverWare. The legacy SUPER models for respective river basins were calibrated and verified as described above. Implementation of RiverWare included duplication of SUPER computational logic within the RiverWare program and calibration of RiverWare basin models to the point that RiverWare results closely matched SUPER results given identical input data sets. RiverWare was subsequently approved as the reservoir system simulation program used for SWD studies.

2.8 Watershed Model

The computer program used for hydrologic watershed modeling was the USACE, Southwestern Division Watershed Model (WSM). The input to this program describes the sub-area boundaries with map x-y coordinates. The program has the capability to look up previously digitized HMR51 PMP charts simply from an input storm center referenced by latitude and longitude. The HMR51 depth-area duration data is then, as one WSM program option, transposed into an elliptical storm pattern similar to that given in EM 1110-2-1411 for the Standard Project Storm (SPS). It should be noted that the EM 1110-2-1411 SPS transposition is an "all area" storm. That is, it does not have storm size as a parameter as does "Hydrometeorological Report No. 52, Application of Probable Maximum Precipitation Estimates - United States East of 105th Meridian"(HMR 52) derived storm patterns. This means for the SPS, that the elliptical pattern of hypothetical storm rainfall, if integrated outward from the center of the storm, will have an average hypothetical storm rainfall at any of the isohyets equivalent to the input average over area hypothetical storm rainfall. Conversely, HMR52 derived storms will only have an average over area depth of Probable Maximum Precipitation (PMP) proportion at the selected storm size. All larger or smaller elliptical isohyets will have less than the input average over area hypothetical rainfall depths. The WSM also has the option for using HMR52 procedures but those procedures are applicable to development of the Probable Maximum Flood (PMF) whereas EM 1110-2-1411 procedures are applicable to the development of the Standard Project Flood (SPF).

3.0 Critical Period Yield Analysis

3.1 Critical Period Yield Analysis Methods

Critical period yield is the constant rate of water supply withdrawal that can be supported through the simulated drought of record, which is the “critical period” for purposes of water supply yield analysis. The study’s purpose is to execute a critical period yield study and document the computations by the two reservoir system simulation computer programs SUPER and RiverWare for current sedimentation conditions for Aquilla Lake. Simulations were developed for the existing top of conservation elevation and each of the three proposed alternatives.

3.1.1 Program Setup

In SUPER and RiverWare, the models for the Brazos River Basin were updated with current area capacity tables and set up to calculate the yield through an iterative process. An initial yield value is entered in the program and the model then simulates the period of record using this demand on the reservoir. The process is repeated iteratively until the yield/demand value which exhausts the water supply storage is determined.

3.2 Critical Period Yield Analysis

For these analyses, the bottom of the conservation pool for Aquilla Lake was set at the lowest outlet invert elevation of 503 feet in all simulations. The results from RiverWare and SUPER for the critical dependable yields were approximately the same. RiverWare calculated a lower yield by 0.1 cfs for each alternative, which is more likely due to the difference in the internal coding of each program. The date of the maximum drawdown occurred on the same day for both programs for each conditional run. For each condition, the time from full to refill, the critical drawdown period, varied slightly between the two programs with RiverWare computing a shorter time than SUPER. Table 4 and table 5 display the results of RiverWare and SUPER, respectively.

Table 4 – RiverWare

Top of Conservation Pool Alternative	Elevation at Top of Conservation (feet)	Conservation Pool Capacity (acre-ft)	Critical Period Yield		Critical Period Begin Date	Date of Maximum Drawdown	Date Conservation Pool Refilled
			(cfs)	(ac-ft/yr)			
Existing	537.50	44,577	22.7	16,445	21Jun1953	31Mar1957	26Apr1957
Alt # 1	540.00	52,659	24.5	17,749	17Jun1953	31Mar1957	27Apr1957
Alt # 2	542.00	59,650	26.1	18,908	13Jun1953	31Mar1957	01May1957
Alt # 3	544.00	68,144	27.9	20,213	9Jun1953	31Mar1957	10May1957
(1) Bottom of conservation pool at elevation 503.00							
(2) Leakage of 0.0 cfs assumed during the simulation.							
Note: Yields are hydrologic yields based on USACE mythology.							

Table 5 SUPER

Top of Conservation Pool Alternative	Elevation at Top of Conservation (feet)	Conservation Pool Capacity (acre-ft)	Critical Period Yield		Critical Period Begin Date	Date of Maximum Drawdown	Date Conservation Pool Refilled
			(cfs)	(ac-ft/yr)			
Existing	537.50	44,577	22.8	16,482	17Jun1953	31Mar1957	26Apr1957
Alt # 1	540.00	52,659	24.6	17,825	14Jun1953	31Mar1957	27Apr1957
Alt # 2	542.00	59,650	26.2	18,985	11Jun1953	31Mar1957	02May1957
Alt # 3	544.00	68,144	28.0	20,306	6Jun1953	31Mar1957	11May 1957
(1) Bottom of conservation pool at elevation 503.00							
(2) Leakage of 0.0 cfs assumed during the simulation.							
Note: Yields are hydrologic yields based on USACE mythology.							

3.3 Critical Period Yield versus Water Supply Contract Yield

The critical period yield for Aquilla Lake was computed based on the assumption of use of all the storage space currently available (as per latest survey) between the top of the conservation pool and the lowest invert of the outlet works. This value represents a best estimate of the average continuous rate of withdrawal a water supply user might expect to sustain during occurrence of a future drought of similar magnitude and characteristics if the user had unlimited contract rights to the same storage space, and the user's withdrawal facilities supported withdrawals down to the lowest invert of the outlet works. The yield actually available to a given water supply user during the course of a future drought of similar proportions and characteristics will vary due to the uncertainties involved in estimating critical period yield and the volume of contracted storage space versus the volume of storage space used in calculating the critical period yield. Because yield cannot be guaranteed, water supply contracts have historically been executed on the basis of storage space in a reservoir, not expected critical period yield. Water supply contracts may refer to an expected critical period yield value associated with a given volume of contracted storage space, but yield is not warranted.

The expected critical period yield that is available to a water supply user, henceforth referred to as "water supply contract yield", can be estimated by multiplying the critical period yield associated with a given top of conservation pool elevation by the ratio of contracted storage space to total storage space used in the yield study. For example, the Brazos River Authority

(BRA) has contracted 33,600 acre-ft of Aquilla Lake conservation storage below top of conservation pool EL 537.5 FT. Based on results of the latest yield study available at the time of contract execution, this 33,600 ac-ft of storage space was expected to provide a critical period yield of about 15 cfs. Based on results of the current yield study (see Table 3), the yield associated with the same storage space is expected to be about 17 cfs [(33600/44577) X 22.7].

It is important to note the yield-to-storage ratio varies with the top of conservation pool assumed for the yield analysis, and typically decreases with increase in elevation of top of conservation pool. This may largely be attributed to the increase in evaporation loss with increase in pool surface area. While expected critical period yield increases with conservation pool capacity, the yield per unit volume of storage space actually decreases. For example, from Table 3 we see an expected critical period yield of 22.7 cfs provided by 44,577 ac-ft of storage space below EL 537.5 FT, for a yield-to-storage ration of 0.0005 cfs/ac-ft (22.7/44577). Analogously, for top of conservation pool EL 544.0 FT we see an expected critical period yield of 27.9 cfs provided by 68,144 ac-ft of storage space below EL 544.0 FT, for a yield-to-storage ration of 0.0004 cfs/ac-ft (27.9/68144). The expected yield is higher for the larger storage space but the efficiency of the storage has decreased.

4.0 Frequency and Duration Analyses

4.1 Aquilla Lake Frequency Procedures

The need for future water supply being the basis for a reallocation/pool raise, a water supply demand equivalent to the critical period yields associated with the existing condition conservation storage volume, and respective proposed alternative conservation storage volumes, were used for the corresponding period of record system simulation runs on the assumption of full use of available storage.

Frequency analyses for the existing condition and respective proposed alternatives were initially developed on the elevation output data from the existing condition simulation run. Although the period of record covers 71 years, it appeared that there were no historically rainfall events greater than the 50 year inflow volume-duration-frequency hydrograph developed in the Design Memorandum No.1. The Aquilla Lake's Design Memorandum No.1 has a 50 year frequency elevation of 556.0 feet with an initial elevation of 542.0 feet**,** based on an inflow volume-duration-frequency study. However**,** the simulated period of record for alternative 2 (542.0 feet) resulted in a maximum elevation of 554.5 feet. To better confirm that historically significant events were unlikely to have occurred during the period of record, an inflow volume duration frequency analysis was performed on the period of record hydrologic inflows. The 100 year inflow from the analysis revealed a peak of approximately 23,000 cfs. Aquilla Lake's Design Memorandum No.1 has a peak inflow of 48,500 cfs for the 50 year frequency inflow.

To establish more confidence in the frequency curves, further statistical analyses were developed. Additional inflow volume duration frequencies, joint probability elevation frequency, and hypothetical storms were run to produce plotting position points to add better definition to the less frequent events. These additional points were then plotted together with the period of record elevation points to provide the basis for the pool elevation frequency curves for the existing condition run and the three respective proposed alternatives.

4.2 Log Pearson Type III Peak Discharge Frequency Curve

An un-regulated peak discharge frequency curve was developed at Aquilla Dam based on the square root of the drainage area ratio of the Aquilla Dam to the drainage area of the Aquilla gauge on Aquilla Creek. The frequency curve at the Aquilla gauge on Aquilla Creek was developed using the computer program HEC-FFA. HEC-FFA utilizes the procedures outlined in "Guidelines for Determining Flood Flow Frequency, "Bulletin # 17B" by the Interagency Advisory Committee on Water Data. The data selected for evaluation at the Aquilla gauge was 44 annual peaks observed before the Aquilla Dam was constructed. The four largest peak discharges obtained from USGS records are shown in Table 6. The resulting frequency curve from HEC-FFA is shown on Plate 3. Table 7 shows the Aquilla gauge peak discharge frequency data adjusted to the Aquilla Dam site.

Table 6 - Aquilla Creek near Aquilla, Texas

Drainage Area = 308 sq mi					
Four greatest peaks in the systematic record					
Year	**Month**	**Day**	**Peak Discharge (cfs)**	**Average Discharge (cfs)**	**Ratio PeakQ/AverageQ**
1968	5	10	40200	18800	2.14
1969	5	7	32600	16400	1.99
1976	7	4	27200	13100	2.08
1981	6	16	53300	27000	1.97
				Average	2.04

Table 7 - Aquilla Dam Peak Discharge Frequency Adjusted from Aquilla Gauge near Aquilla, TX

	Aquilla Gauge Drainage Area = 308 sq mi	**Aquilla Dam Drainage Area = 255 sq mi**
Percent Chance Exceedance	**Computed Peak Frequency (cfs)**	**Discharge Frequency (Aquilla Gauge x 0.91) (cfs)**
0.2	174000	158340
0.5	122000	111020
1	92000	83720
2	68300	62153
5	44500	40495
10	31100	28301
20	20600	18746
50	10100	9191
80	5480	4987
90	4120	3749
95	3310	3012
99	2290	2084

4.3 Volume Duration Frequency Analyses

Annual high inflow volume-duration frequency curves were developed for Aquilla Lake from the inflows from SUPER simulation run B10X01. This was the latest SUPER run available for analysis and comparison purposes. This was accomplished with a computer program named LOWFREQ (LOWFREQ performs both low flow and high flow analysis). These inflow volume-duration frequency curves were developed for each duration, 1 through 10 days. The data developed is then used by the program as the basis for the construction of balanced hypothetical inflow floods of selected probabilities. The program establishes a smooth hydrograph curve through each balanced daily hydrograph and 1-hour ordinates are obtained while maintaining the daily volumes. The same type analysis was also performed for the computed total local daily flow hydrograph for the Mouth of Aquilla Creek control point to obtain balanced local flow hydrographs with 1-hour ordinates. These balanced 1 hour ordinate hydrographs, for various frequencies, both for Aquilla Lake and the Mouth of Aquilla Creek, along with the Aquilla Lake regulation plan, the elevation capacity curve, the spillway rating curve, and the May elevation duration curve from run B10X01 were input to a computer program named RESPROB. The May elevation duration curve was selected as providing an appropriate antecedent condition for the historically most likely maximum inflow period. This process was performed for two skew values – a skew that fit the data (a negative skew) and a zero skew. Plates 4-7 show the results of the Aquilla Inflow max annual duration frequencies with a skew to fit the data (a negative skew) and zero skew, and respective Aquilla inflow volume duration frequencies.

4.4 Joint Probability Frequency Analyses

The primary purpose of the RESPROB program is to provide a basis for the extrapolation of reservoir elevation frequency curves above the part of the relationship that is defined by period of record data. This program makes routings of the hypothetical frequency floods through the lake for multiple initial lake level conditions. The routings may be made in either a normal or emergency operations mode. The program is given some exact knowledge of the future inflows in normal operations followed by a normal recession to simulate normal operations with a forecast. In emergency operations mode, only the current period, no future data, is known then followed by a normal recession. The program routes the flood, based on its forecasted knowledge, to minimize the maximum release, while taking into account the minimum release (the regulating plan and the induced surcharge curve), the maximum release constraint defined by the spillway rating curve, and the maximum allowed flood release vs. reservoir elevation function. The downstream local flow hydrograph and the channel capacity at that location are also considered in the operation. A simultaneous downstream event on the un-controlled area of the same frequency as the reservoir inflow hydrograph is assumed there. The set of maximum lake elevation vs. probability of the inflow flood values generated by the program for each initial pool level define a conditional probability curve for that initial level. There are then multiple conditional reservoir elevation probability curves depending on the number of initial lake elevations considered. The probability of each initial elevation is taken from the reservoir elevation duration curve by approximating the fraction of time that elevation prevails. The total probability at a given reservoir elevation is determined by summing all the conditional probability curves, the product of the conditional probability at that elevation, and the probability of the initial pool. This is based on the use of the total probability theorem. In this study two separate estimates of joint probability pool elevation were made. One estimate was based on a skew coefficient for the inflow volume duration frequency analysis that fit the data. The second estimate was based on a skew coefficient of zero which appeared more reasonable when compared to the peak discharge frequency curve. Plate 8 graphs the two elevation joint probability curves.

4.5 Determination of Hypothetical Un-regulated Peak Discharge

Three storm sizes were selected, the 35%, 42% and 50% of the HMR51 PMP. The initial step was to find the critical center and major axis orientation for each of these storms. The critical center and orientation are those that produce the maximum discharge at the Aquilla Dam. A utility program named OPTIMIZ was used to find the critical centering. OPTIMIZ repetitively executed the un-regulated condition WSM until the critical storm center was found. Initially a coarse storm center grid is used. Each time the critical center is found, the program automatically halves the grid size until the critical center and orientation have been found on a one mile grid and the orientation is to the nearest one degree. Once the three critical centers were found, the WSM was executed for the three storms critically centered to obtain detailed output for each. The three hypothetical peaks along with the un-regulated frequency curves are shown on Plate 9 for the Aquilla Dam. The plotting positions for the hypothetical peaks were selected so that they would plot on the un-regulated Aquilla Dam peak discharge frequency curve.

4.6 Determination of Hypothetical Regulated Peak Discharge

The critical centers that produce the maximum elevation in Aquilla Lake for each of the selected storm sizes for the un-regulated conditions in WMS were used during the execution of the regulated conditions in WMS. The regulated hypothetical peaks for each initial condition were plotted along with the period of record simulated maximum annual peaks from SUPER run B10X01. The hypothetical peaks were plotted at probabilities corresponding to the plotting positions determined for the regulated peaks at Aquilla Dam. These plots are shown on Plate 10.

4.7 Aquilla Lake Frequency Analyses

The Median/Chegodayev's formula was used to determine the frequency plotting positions of the annual peak pool elevations. The peak pool elevation associated with the previously modeled Probable Maximum Flood (PMF) event was plotted at the 0.01 exceedance probability to guide the upper end of a graphical best fit curve used to determine the pool elevations for less frequent events.

The pool elevation frequencies shown in Table 8 are extracted from the plotted points and best fit curve shown on Plate 11. It can be seen that the expected return period for exceeding spillway crest elevation 564.5 feet is more than a 100 year frequency event for all simulated runs.

In accordance with the water control plan of regulation, the downstream control limit for Aquilla Lake is 3,000 cfs. Spillway discharge equals 3,000 cfs at about pool elevation 565.7 feet. Thus the potential for controlling dam discharge so as not to exceed the downstream control limit exists for pool elevations up to 565.7 feet. Allowable dam discharge for pool elevations up to 565.7 feet is determined as a function of pool elevation and available downstream channel capacity. For pool elevations between spillway crest elevation 564.5 feet and 565.7 feet total dam discharge may include both gated releases and uncontrolled spillway discharge up to a maximum of 3,000 cfs. For lake elevations above 565.7 feet, total dam discharge equals spillway discharge, and the frequency associated with spillway discharge is the same as for the pool elevation required to produce the discharge. It can be seen on Plate 11 the exceedance probability of pool elevation 565.7 feet is more than a 100 year frequency event in all simulated runs.

The Aquilla Lake annual peak total discharge frequency plot is shown on Plate 13. Plotting positions for discharges greater than 3,000 cfs are not shown because, as explained above, the frequencies for discharges greater than 3,000 cfs equal the frequency of the pool elevation required to produce the discharge over the spillway. It should be noted that for all three alternatives, the discharge frequency plots vary little from the existing conditions. In all the runs, no spill release was made during the period of record. Table 9 shows the total discharge frequencies for Aquilla Lake. Table 10 separates out the spillway discharge frequency from the total discharge frequency.

Table 8 - Aquilla Lake Elevation Frequency

Staring Elev. Return Period (years)	537.5 Elevation (feet)	540.0 Elevation (feet)	542.0 Elevation (feet)	544.0 Elevation (feet)
2	542.0	543.5	545.2	546.5
5	546.5	547.8	549.0	550.2
10	548.5	549.5	550.5	551.5
25	552.0	552.8	553.5	554.5
50	554.0	555.0	556.0	557.0
100	557.0	558.0	559.0	560.0
300	563.0	564.0	565.0	565.5
500	564.5	566.5	567.2	567.7
PMF	552.14			

Table 9 - Aquilla Lake Total Discharge Frequency

Staring Elev. Return Period (years)	537.5 Discharge (cfs)	540.0 Discharge (cfs)	542.0 Discharge (cfs)	544.0 Discharge (cfs)
2	2,250	2,300	2,350	2,400
5	2,400	2,450	2,500	2,525
10	2,450	2,500	2,550	2,575
25	2,550	2,600	2,650	2,650
50	2,600	2,700	2,725	2,750
100	2,750	2,775	2,800	2,825
300	2,900	2,950	3,000	3,000
500	2,950	5,700	8,550	11,050

Table K10 - Aquilla Lake Spillway Frequency

Staring Elev. Return Period (years)	537.5 Discharge (cfs)	540.0 Discharge (cfs)	542.0 Discharge (cfs)	544.0 Discharge (cfs)
2	0	0	0	0
5	0	0	0	0
10	0	0	0	0
25	0	0	0	0
50	0	0	0	0
100	0	0	0	0
300	0	0	1,000	2,400
500	0	5,700	8,550	11,050

4.8 Aquilla Lake Duration Analyses

The annual pool elevation-duration relationship was developed by ranking the simulation daily pool elevation results for the entire 71 year period of record in descending order, and computing the percent of time the elevation was equaled or exceeded as rank divided by the total number of daily values. Annual duration reflects the percent of time a given pool elevation was equaled or exceeded over the entire period of record, with no regard to date of occurrence or time of year. A plot of the annual pool elevation-duration curve is shown on Plate 12. It can be seen that top of conservation pool elevation 537.5 feet was equaled or exceeded about 40 percent of the time and that alternative 2 top of conservation pool elevation 542.0 feet was equaled or exceeded about 35 percent of the time. Top of flood pool and spillway crest elevation 556.0 feet was equaled or exceeded less than one percent of the time for all simulation runs.

5.0 Wind Wave Run-up Analyses

5.1 Wind Wave Run-up Analysis

Wind wave run-up analyses were performed for each reallocation alternative to support geotechnical analyses of erosion protection requirements for the upstream face of the dam embankment. The wind wave run-up heights for Aquilla Dam were determined in accordance with the method set forth in Engineer Technical Letter No. 1110-2-305 dated 16 February 1984, subject: "Determining Sheltered Water Wave Characteristics". The wind wave run-up analysis for Aquilla Lake was calculated using the Southwestern Division's program *Wave Characteristics, Wave Run-up, and Wind Setup Computational Model*. This program, written in January 1985, computes the wave run-up and wind setup based on the requirements described in ETL 1110-2-305.

5.2 Effective Wave Fetch

The extent of the wave growth in sheltered water bodies for a given wind speed and direction depends on the fetch or generating area. The method discussed in ETL 1110-2-305 consists of superimposing radials on a base map of the water body. The radials are constructed in such a manner that they emanate from the dam site where wave information is needed and extend across the water area until they intersect the shoreline. The angle between any two adjacent radials is 3 degrees. A minimum of 9 radials are required to define the wave fetch which covers a total angle of 24 degrees. The method involves taking the 9 longest sequential radial lengths that can occur at a particular site. As many as 90 radials can be entered, however it was determined that 13 radials covered the area adequately. The effective wave fetch is computed by taking a simple average of the critical fetch lengths. The average wave fetch determined from the program was 2.23, 2.25, and 2.30 miles for the alternatives 1, 2, and 3.

Surface waves induced by wind are classified in accordance to their lengths and the depths over which they travel, or specifically by the ratio d/L. A wave is said to be shallow water wave if d/L is less than 1/25. The average depth of the lake along the longest radial line was calculated by measuring the depths along the line at 20 increments and was calculated to be 23.0 feet. Aquilla Lake is considered to have a shallow water wave since the average depth is 23 feet over a distance of 2.25 miles.

5.3 Design Wind

The one-percent chance fastest mile wind speed and 1-hour wind speeds for the Aquilla Lake freeboard study were determined in accordance with ETL 1110-2-305 using the regional map with iso-lines for the fastest mile and 1-hour wind speed. Plate 22 is the regional map from ETL 1110-2-305. The iso-lines were developed from wind observations from 53 stations within and adjacent to the SWD area. Based on the observed wind speed data and a series of statistics the 1 percent chance fastest mile and 1-hour wind speeds were developed for the SWD area. For the Aquilla study the fastest mile and the 1-hour wind speeds were determined to be 73 and 52, respectively. The program computed a design wind speed of 61 miles per hour and a duration of 37 minutes. In addition the program computes a design wave height for each alternative shown in Table 11.

Table 11 - Design Wave Height

Alternative	Pool Elevation (N.G.V.D.)	Design Wave Height (ft)
1	540	3.8
2	542	3.8
3	544	3.9

5.4 Additional Required Input

Additional information was needed to calculate the wave run-up. The vertical height above the still water level that a wave will run up the face of a structure depends on several factors. These factors are identified as the structure shape and roughness, water depth at the toe of the structure, bottom slope in front of the structure, and the characteristics of the waves impinging on the structure.

The radials were measured from Aquilla Dam between station 66+35 and station 93+20. The embankment slope at this location at the elevation of the alternatives has a slope of 1 on 8. It was also assumed that riprap would be located across the embankment at the elevation, so the study considered this a riprap slope. The depth of the toe was very shallow for the alternatives at this location of the dam. The depths ranged from 2 to 4 feet below the water surface.

A run-up value of 0.0 feet was computed for all alternatives based on the input described above. With the shallow depths at the toe of the embankment, the flat slopes, and riprap placed along the selected alternative elevation, the run-up is determined to be negligible.

6.0 Sedimentation Analysis

6.1 Purpose of Sedimentation Analysis

According to the *2011 Brazos G Regional Water Plan prepared by Brazos G Regional Water Planning Group*, the 2060 local demand for Lake Aquilla is approximately 11,400 acre-ft. Each alternative requires an evaluation of the 2060 yield to determine whether each alternative will supply adequate yield to support the predicted 2060 demand for Lake Aquilla. In order to accomplish this analysis the 2008 elevation-area-capacity (EAC) table developed by the Texas Water Development Board (TWDB) for Lake Aquilla must be projected forward to 2060. A sedimentation study was performed that calculated the sedimentation rate and distribution of the sediment throughout Lake Aquilla's EAC table.

6.2 Sedimentation Rate

Sedimentation rate is a value that varies from year to year as it is affected by a variety of factors like meteorological effects and farming practices. The original EAC table was estimated by calculating the area using the Conic Method from 10-foot contour lines. The TWDB has conducted three bathymetric surveys on Lake Aquilla: October 1995, April 2002, and April 2008. The TWDB developed an EAC table from each survey and from the changes between each EAC table a sedimentation rate was calculated. Since the sedimentation rate can vary to a large degree, to best analyze whether the three alternatives will produce enough yield to meet the Lake Aquilla 2060 local demand, a sensitivity analysis of the sedimentation rates was conducted. Table 12 shows the different sedimentation rates calculated from each of the surveys developed for Lake Aquilla.

Table 12 - Sedimentation Rates

Survey	Volume Comparisons (acre-ft)				
Nov 1973	52,400				52,400
Oct 1995	46,896	46,896		46,896	
Apr 2002		45,151	45,151		
Apr 2008			44,566	44,566	44,566
Change in Volume (acre-ft)	5,504	1,745	585	2,330	7,834
Number of Years Between Surveys	23	6.5	6	12.5	36
Sedimentation Rate (acre-ft/year)	239.3	268.5	97.5	186.4	217.6
Sedimentation Rate (acre-ft/year)/drainage area	0.95	1.07	0.39	0.74	0.86

Design Memorandum No.1 Hydrology, states the expected sedimentation rate for 100 years is 25,700 acre-ft or 257 acre-ft per year. Depending on which comparison of EAC tables is used, the sedimentation rate ranges from 97 to 269 acre-ft per year. For the sensitivity analysis, the sedimentation rate had an upper and lower bound set to 257 and 186 acre-ft per year. The 2002 survey appears to create an over and under estimate of the amount of sedimentation of Lake Aquilla. Since these high and low sedimentation rates calculated between 1995 and 2002, and

2002 and 2008, are an effect of the 2002 survey, taking the difference between the 1995 and 2008 surveys provides a more smooth sedimentation rate over the period. The original survey was not used for the calculations as this was a rough estimate from the topography.

6.3 Sedimentation Distribution

To develop the Lake Aquilla 2006 EAC table, the total sedimentation accumulated over 52 years ranging from 2008 (TWDB survey) and 2060 was calculated by multiplying the lower and upper bound sedimentation rates by 52. These totals were then applied to the 2008 TWDB EAC table to produce a 2060 EAC table. With these modified EAC tables, 2060 Lake Aquilla yield analyses were performed to calculate a range of yields to determine whether the alternative pool reallocations would supply the required 2060 demand for Lake Aquilla.

The 52 year accumulated sedimentation amounts for the lower and upper bounds were 9,672 to 13,364 acre-ft. Following the Engineering Manual EM 1110-2-4000 Engineering and Design – Sedimentation Investigations of Rivers and Reservoirs, these accumulated sedimentation amounts were applied to the 2008 TWDB EAC table. It should also be noted that these procedures are outlined in the Bureau of Reclamation's Erosion and Sedimentation Manual 2006. This manual supplied supplemental explanation and examples that helped provide guidance for calculating the distribution of sediment throughout the 2060 EAC table.

There are multiple factors that affect the distribution of sediment in a reservoir: reservoir size and shape, sediment quantities and characteristics, sediment sources, reservoir regulation practice, and magnitude, frequency, and sequence of hydrologic events. These factors were evaluated as part of the analysis and helped determine the distribution of sediment for Lake Aquilla. In addition there are multiple empirical methods outlined in EM 1110-2-4000 for calculating the distribution. The empirical area-reduction method developed for the Bureau of Reclamation in 1958 was selected as the method for Lake Aquilla. The method recognizes that the distribution of sediment depends upon: (1) the manner in which the reservoir is to be operated; (2) the size of deposited sediment particle; (3) the shape of the reservoir; and (4) the volume of sediment deposited in the reservoir. The shape of the reservoir was adopted as the major criterion for development of empirically derived design curves for use in distributing sediment. Plate 23 represents the design curves developed from the 30 reservoirs studied as part of the development of the empirical area-reduction method.

The Lake Aquilla reservoir type was determined by plotting reservoir depth verse reservoir capacity. The reservoir type is the adjustment to the empirical area reduction method to include a correction for the reservoir shape. Plate 23 shows the calculations to determine the reservoir type. The lower and upper curves on this plate were computed for two different reservoir types, and the entire curve was also calculated. The slope of the line, m, was determined to be 3.35 for the entire curve which was a Type II – Flood-plain foothill reservoir. This calculation coupled with the moderate drawdown on Lake Aquilla due to the operations and demands, determined that for the analysis, reservoir type II would be used to determine the capacity. Plate 24 shows the four reservoir type curves from EM 1110-2-4000.

The next step was to determine the elevation of sediment deposited at the dam. The formula to calculate directly the elevation of sediment deposited at the dam in EM 1110-2-4000 is $h'(p) = S - V(ph) / HA(ph)$ where:

h'(p) = a function of the reservoir and its anticipated sediment storage.

S = total sediment inflow in acre-ft.

V(ph) = reservoir capacity in acre-ft at a given elevation.
H = height of the dam in feet.
A(ph) = reservoir area in acres at a given elevation.

Table 13 shows the results from the lower bound sedimentation rate calculations using the formula listed above. These results were plotted against the reservoir type curves given relative depth (p) versus h'(p). Plate 25 shows the plotted curves and the intersection point that determined the elevation of sediment deposited at the dam. The P_0 curve intersected the reservoir type II curve at approximately 0.1 relative depth (p). Given the 0.1 relative depth for P_0, the additional feet of sediment deposited to the bottom elevation is calculated by multiplying P_0 by the height of the dam (58ft). This comes to 5.8 feet of sediment deposited at the dam. With a bottom elevation at Lake Aquilla of 498, the elevation of sediment deposited at the dam was approximately 504 feet. The same procedure was then followed for the upper bound sedimentation rate.

Table 13 - Direct Determination of Elevation of Sediment Deposited at the Dam

Empirical Area-Reduction Method Lower Bound Sedimentation Rate						
Elevation (ft)	**p**	**V(pH)**	**A (ph)**	**S-V(ph)**	**HA(ph)**	**h'(p)**
500	0.034483	8	9	9664	522	18.5134
501	0.051724	20	18	9652	1044	9.2452
502	0.068966	51	43	9621	2494	3.8577
503	0.086207	106	73	9566	4234	2.2593
504	0.103448	198	113	9474	6554	1.4455
505	0.12069	334	157	9338	9106	1.0255
510	0.206897	1734	412	7938	23896	0.3322
515	0.293103	4682	779	4990	45182	0.1104
520	0.379310	9478	1166	194	67628	0.0029
P_0 = 0.1		H = 58 ft		S = 9,672 acre-ft		
P_0*H = 5.8 ft						
Bottom Elevation = 498 ft						
Elevation of sediment deposited at dam = 504 ft						

The last step for the Empirical area-reduction method was to compute the sediment deposition. This determined how the sediment was deposited throughout the EAC table. At the end of this computation Lake Aquilla has a 2060 EAC table for the lower and upper bound sedimentation rates. Table 14 has the original and computed results of Lake Aquilla EAC for the lower bound.

The relative depth (p) was computed by taking the difference between the incremental elevations and the original bottom depth, 498 ft, and dividing by the depth (H), 44 ft. As an example the first value for (p), 0.1364, was calculated by p=(504-498)ft/44ft. It should be noted that the depth (H) used for these calculations originates from taking the difference between elevation 542 and 498 ft. Elevation 542 was selected for this analysis rather than the top of flood pool 556 because of additional sedimentation studies performed by Fort District in January 1981 and the current calculated pool elevation duration curve. These studies of reservoirs in the area showed that sedimentation deposition was mostly in the conservation pool with only a small amount depositing in the first few feet of the flood pool. Lake Aquilla's watershed falls between Lake Whitney's watershed which record 85% of the sediment deposited in the conservation pool and Lake Navarro Mill's watershed which recorded 105% sedimentation in the conservation pool. In addition, elevation 542 is reached approximately 2-5% of the time according to the pool elevation duration curves. Based on this information, the sediment deposition was applied to elevations 542 ft and below.

The formula for calculating the relative sediment area (a) was found in Chapter 2 of the Bureau of Reclamation's Erosion and Reservoir Sedimentation document. The reservoir type II formula used was $a = 2.487 * p^{0.57} * (1-p)^{0.41}$. Once the (a) Type II column was computed, the sediment area was calculated. First the relative distribution coefficient (K) was calculated and is shown at the bottom of Table 14. The relative distribution coefficient (K) is the reservoir area divided by the relative area at the elevation of the sediment deposited at the dam, 504 ft. The sediment area column was calculated by multiplying (K) by the incremental relative sediment area (a) for each elevation. As an example, for elevation 505 the sediment area was equal to (K)*a or 150.2*0.812 which was 122 acres. The equivalent sediment volume was found by using the end area average method.

Within the sediment deposition computation, there is a mathematical check to insure that correct sedimentation total volume was being applied to the capacity table. The accumulated sediment volume column should equal the total sediment volume. When the volumes were not equal, then a new relative distribution coefficient was calculated. The new (K2) coefficient was determined by multiplying the original (K) by the ratio of the total sediment volume and the accumulated volume. With the new distribution coefficient (K2) the sediment area, volume, and accumulated volume columns were recalculated. The original and redeveloped sediment area, volume, and accumulated volumes are found in Table 14.

The final 2060 EAC table for Lake Aquilla developed from the lower bound sedimentation rate sediment volume is shown in Table 14. The revised area was calculated by subtracting the sediment area from the original area at each elevation. Similarly the revised volume was calculated by subtracting the sediment volume from the original volume at each elevation. The increments that calculated to a negative were set to zero as this was an indication that that portion of the EAC table was filled by sedimentation.

Table 14 - Empirical Area-Reduction Method Sediment Deposition Computations

Lake Aquilla Lower Bound Sedimentation Rate											
S = 9,672 acre-ft				a = relative sediment area				Reservoir Type II Formula for a			
H = 44 feet				p = relative depth of reservoir measured from the bottom				a = (2.487*p^0.57)*(1-p)^0.41			
							Revised Sediment Deposition based on revised K2			**2060 EAC Table**	
Elev (ft)	**Original Area (acres)**	**Relative Depth (p)**	**a Type II**	**Sediment Area (acres)**	**Sediment Volume (ac-ft)**	**Accumulated Sediment Volume (ac-ft)**	**Sediment Area 2 (acres)**	**Sediment Volume 2 (ac-ft)**	**Accumulated Sediment 2 Volume (ac-ft)**	**Revised Area (acres)**	**Revised Capacity (ac-ft)**
504	113	0.1364	0.752	113		198	172		198	0	0
505	157	0.1591	0.812	122	118	316	186	179	377	0	0
506	205	0.1818	0.867	130	126	442	198	192	569	7	0
507	252	0.2045	0.916	138	134	576	210	204	773	42	0
508	303	0.2273	0.962	144	141	717	220	215	988	83	0
509	356	0.2500	1.003	151	148	864	230	225	1213	126	138
510	412	0.2727	1.041	156	154	1018	238	234	1447	174	287
511	480	0.2955	1.075	162	159	1177	246	242	1689	234	491
512	544	0.3182	1.107	166	164	1341	253	250	1939	291	752
513	619	0.3409	1.135	171	168	1509	260	257	2196	359	1073
514	712	0.3636	1.161	174	172	1681	266	263	2459	446	1477
515	779	0.3864	1.184	178	176	1857	271	268	2727	508	1955
516	844	0.4091	1.204	181	179	2037	276	273	3000	568	2494
517	912	0.4318	1.222	184	182	2219	280	278	3278	632	3092
518	993	0.4545	1.238	186	185	2404	283	282	3560	710	3760
519	1086	0.4773	1.250	188	187	2591	286	285	3845	800	4515
520	1166	0.5000	1.261	189	189	2779	289	287	4132	877	5355
521	1236	0.5227	1.269	191	190	2969	290	290	4422	946	6267
522	1308	0.5455	1.274	191	191	3160	292	291	4713	1016	7248
523	1379	0.5682	1.277	192	192	3352	292	292	5005	1087	8299
524	1451	0.5909	1.277	192	192	3544	292	292	5297	1159	9420
525	1553	0.6136	1.275	192	192	3736	292	292	5589	1261	10629
526	1661	0.6364	1.270	191	191	3927	291	291	5881	1370	11944
527	1755	0.6591	1.261	189	190	4117	289	290	6170	1466	13363
528	1866	0.6818	1.250	188	189	4305	286	288	6458	1580	14884
529	1982	0.7045	1.236	186	187	4492	283	285	6742	1699	16527
530	2089	0.7273	1.218	183	184	4676	279	281	7023	1810	18282
531	2191	0.7500	1.196	180	181	4858	274	276	7299	1917	20145
532	2319	0.7727	1.170	176	178	5035	268	271	7570	2051	22128
533	2460	0.7955	1.139	171	173	5209	261	264	7834	2199	24251
534	2595	0.8182	1.103	166	168	5377	252	257	8091	2343	26524
535	2733	0.8409	1.060	159	162	5539	243	248	8339	2490	28937
536	2892	0.8636	1.011	152	156	5695	231	237	8576	2661	31513
537	3017	0.8864	0.952	143	147	5842	218	225	8800	2799	34245
538	3105	0.9091	0.881	132	138	5980	202	210	9010	2903	37099
539	3288	0.9318	0.794	119	126	6106	182	192	9202	3106	40119
540	3388	0.9545	0.682	102	111	6217	156	169	9371	3232	43288
541	3493	0.9773	0.520	78	90	6307	119	138	9509	3374	46591
542	3613	1.0000	0.000	0	39	6346	0	60	9568	3613	50082
543	4246	1.0227							9672	4246	54125
544	4448	1.0455								4448	58472
545	4655	1.0682								4655	63024
546	4867	1.0909								4867	67785
547	5083	1.1136								5083	72760
548	5305	1.1364								5305	77954
549	5530	1.1591								5530	83371
550	5761	1.1818								5761	89017
551	5959	1.2045								5959	94877
552	6160	1.2273								6160	100936
553	6365	1.2500								6365	107199
554	6573	1.2727								6573	113668
555	6784	1.2955								6784	120346
											Continued

Table 14 - Empirical Area-Reduction Method Sediment Deposition Computations (Continued)

Elev (ft)	Original Area (acres)	Relative Depth (p)	a Type II	Sediment Area (acres)	Sediment Volume (ac-ft)	Accumulated Sediment Volume (ac-ft)	Sediment Area 2 (acres)	Sediment Volume 2 (ac-ft)	Accumulated Sediment 2 Volume (ac-ft)	Revised Area (acres)	Revised Capacity (ac-ft)
556	6999	1.3182								6999	127238
557	7217	1.3409								7217	134346
558	7438	1.3636								7438	141673
559	7663	1.3864								7663	149224
560	7891	1.4091								7891	157001
561	8127	1.4318								8127	165010
562	8367	1.4545								8367	173257
563	8611	1.4773								8611	181746
564	8858	1.5000								8858	190480
565	9108	1.5227								9108	199463
566	9362	1.5455								9362	208698
567	9619	1.5682								9619	218189
568	9880	1.5909								9880	227938
569	10140	1.6136								10140	237948
570	10410	1.6364								10410	248223
571	10920	1.6591								10920	258888
572	11440	1.6818								11440	270068
573	11970	1.7045								11970	281773
574	12510	1.7273								12510	294013
575	13060	1.7500								13060	306798
576	13630	1.7727								13630	320143
577	14200	1.7955								14200	334058
578	14790	1.8182								14790	348553
579	15400	1.8409								15400	363648
580	16010	1.8636								16010	379353
581	16630	1.8864								16630	395673
K = relative distribution coefficient						K = 113 / .752 = 150.2					
K2 = K * (total sediment volume / accumulated volume)						K2 = 150.2 * (9672 / 6346) = 228.9					

6.4 Application of 2060 EAC Table

Given the upper and lower bound 2060 EAC tables, the Lake Aquilla 2060 yield was calculated to answer whether the reservoir would be able to meet the 2060 water supply demands. According to the *2011 Brazos G Regional Water Plan*, the 2060 local demand for Lake Aquilla was approximately 11,400 acre-ft. The 2060 EAC tables were input into the RiverWare model to evaluate what annual volume Lake Aquilla could yield over the period of record. A yield analysis for USACE purposes calculates the hydrologic yield for a reservoir which does not include any additional demand. However a yield analysis for a Texas state agency like the Brazos River Authority, has to evaluate the yield of a reservoir after all the senior water rights have been met, or in other words an available yield. The question of whether the 2060 demand would be met falls back to a yield analysis for BRA which does look at the yield after the senior water rights have been met. To accomplish this, BRA supplied the water supply demands for the senior water rights that come out of Lake Aquilla to the Fort Worth District, and this time series of flows were applied to the RiverWare model. Table 15 shows the results from the yield analyses that apply the upper and lower bound sedimentation rate EAC tables and the senior water right demands. The existing top of conservation pool, 537.5 ft, was also performed to evaluate water supply availability yield for 2060.

Table 15 - Lake Aquilla Water Supply Yield

Top of Conservation Pool Elevation Alternatives	Current EAC Table	Lower Bound Sedimentation Rate	Upper Bound Sedimentation Rate
	2008	**2060**	**2060**
Feet	Acre-ft	Acre-ft	Acre-ft
Current (537.5')	13,000	11,200	11,000
Alternative 1 (2.5’)	14,300	12,100	11,500
Alternative 2 (4.5’)	15,400	13,400	12,500
Alternative 3 (6.5’)	16,800	14,600	13,700

7.0 References

1. *Aquilla Water Control Manual*, *Appendix D of Trinity Master Manual*, U.S. Army Corps of Engineers, Fort Worth District, April 1997.

2. *Volumetric and Sedimentation Survey of Aquilla Lake*, Texas Water Development Board, April 2009.

3. http://cadswes.colorado.edu/PDF/RiverWare/documentation/index.html, Center of Advanced Decision Support for Water and Environmental Systems.

4. http://waterdata.usgs.gov, U.S. Geological Survey.

Daily Hydrologic Data for Brazos River Basin from Aquilla Creek to Bosque River

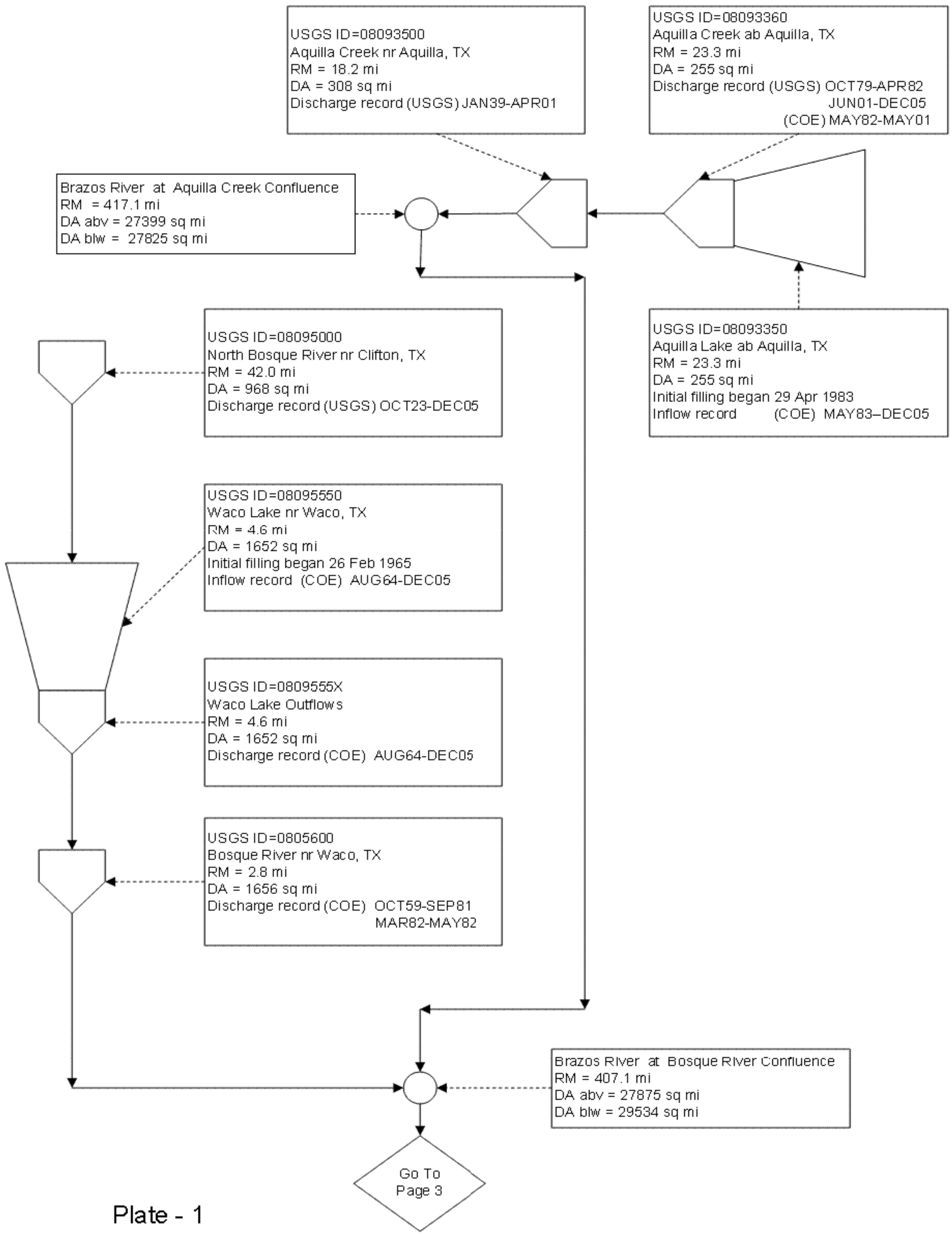


Plate - 1

Table 1					
Aquilla Lake Elevation-Area-Capacity Table					
(Based on 2008 survey)					
Elevation (feet)	**Area (acres)**	**Capacity (acre-feet)**	**Elevation (feet)**	**Area (acres)**	**Capacity (acre-feet)**
496	0	0	539	3288	49321
497	0	0	540	3388	52659
498	0	0	541	3493	56100
499	4	2	542	3613	59650
500	9	8	543	4246	63797
501	18	20	544	4448	68144
502	43	51	545	4655	72696
503	73	106	546	4867	77457
504	113	198	547	5083	82432
505	157	334	548	5305	87626
506	205	515	549	5530	93043
507	252	744	550	5761	98689
508	303	1021	551	5959	104549
509	356	1351	552	6160	110608
510	412	1734	553	6365	116871
511	480	2180	554	6573	123340
512	544	2691	555	6784	130018
513	619	3269	556	6999	136910
514	712	3936	557	7217	144018
515	779	4682	558	7438	151345
516	844	5494	559	7663	158896
517	912	6370	560	7891	166673
518	993	7320	561	8127	174682
519	1086	8360	562	8367	182929
520	1166	9487	563	8611	191418
521	1236	10689	564	8858	200152
522	1308	11961	565	9108	209135
523	1379	13304	566	9362	218370
524	1451	14717	567	9619	227861
525	1553	16218	568	9880	237610
526	1661	17825	569	10140	247620
527	1755	19533	570	10410	257895
528	1866	21342	571	10920	268560
529	1982	23269	572	11440	279740
530	2089	25305	573	11970	291445
531	2191	27444	574	12510	303685
532	2319	29698	575	13060	316470
533	2460	32085	576	13630	329815
534	2595	34615	577	14200	343730
535	2733	37276	578	14790	358225
536	2892	40089	579	15400	373320
537	3017	43045	580	16010	389025
538	3105	46109	581	16630	405345

Plate - 2

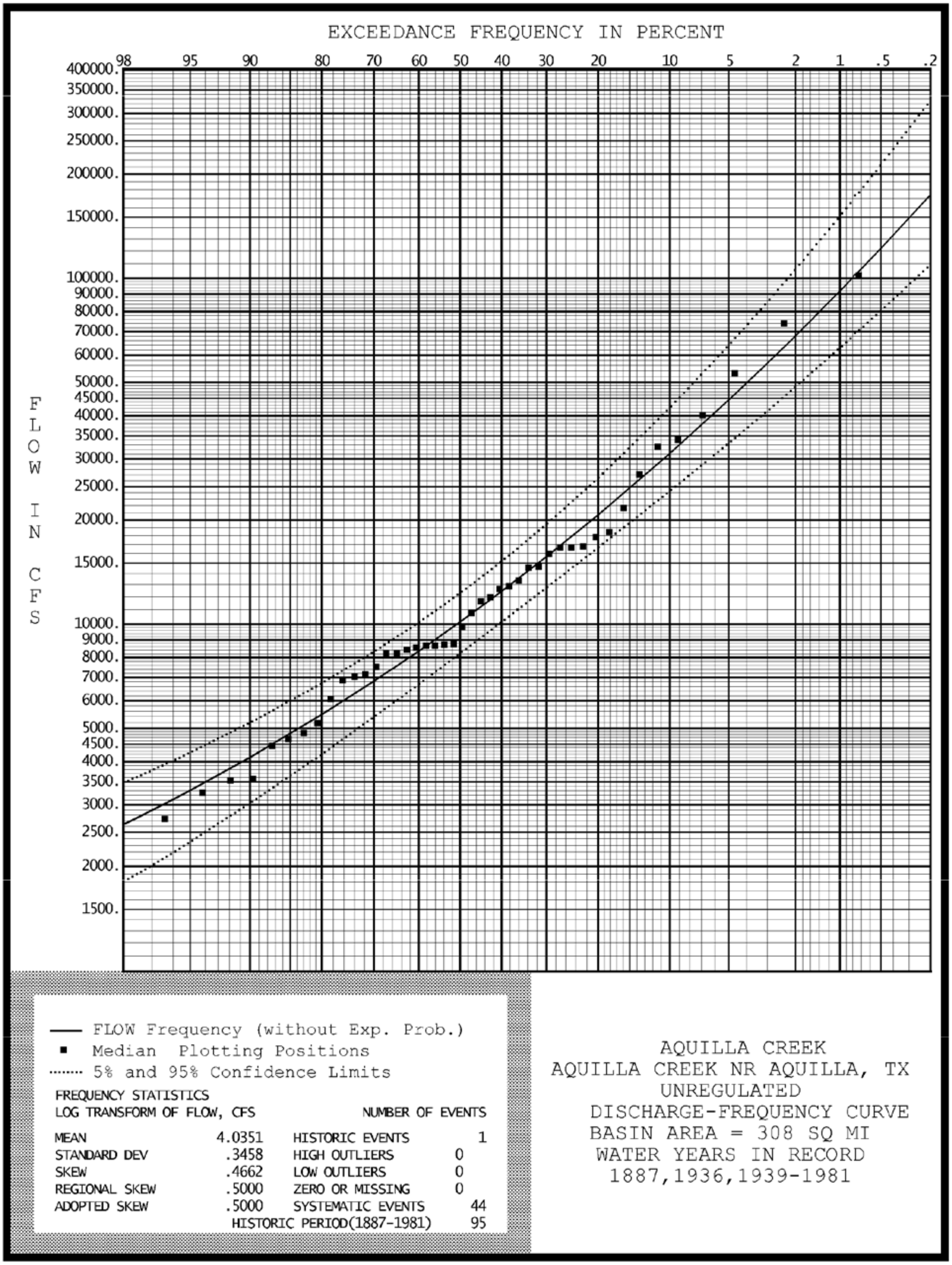
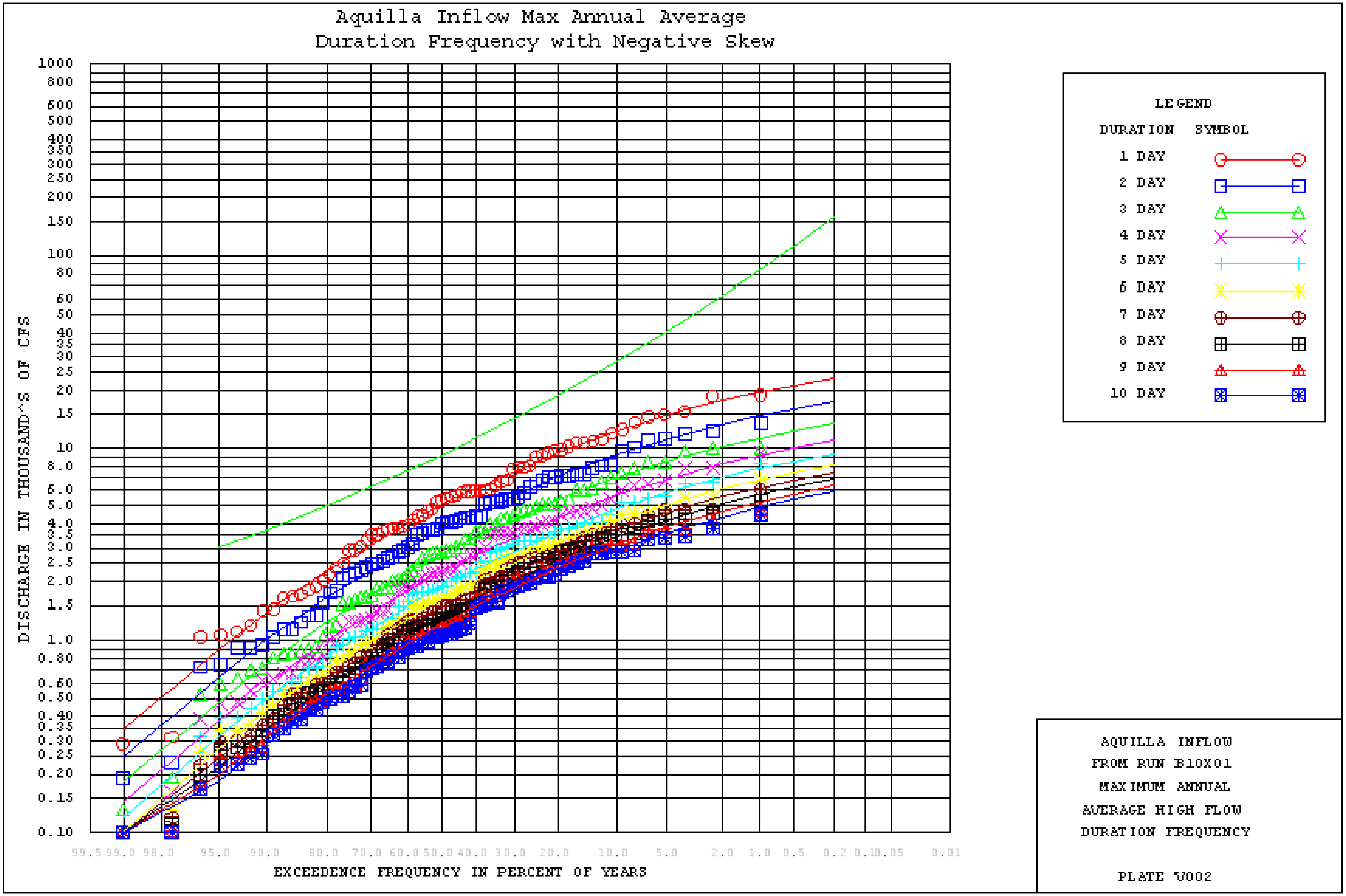

EXCEEDANCE FREQUENCY IN PERCENT
98 95 90 80 70 60 50 40 30 20 10 5 2 1 .5 .2
FLOW IN CFS
400000. 350000. 300000. 250000. 200000. 150000. 100000. 90000. 80000. 70000. 60000. 50000. 45000. 40000. 35000. 30000. 25000. 20000. 15000. 10000. 9000. 8000. 7000. 6000. 5000. 4500. 4000. 3500. 3000. 2500. 2000. 1500.
FLOW Frequency (without Exp. Prob.)
Median Plotting Positions
5% and 95% Confidence Limits
FREQUENCY STATISTICS
LOG TRANSFORM OF FLOW, CFS
NUMBER OF EVENTS
MEAN 4.0351
STANDARD DEV .3458
SKEW .4662
REGIONAL SKEW .5000
ADOPTED SKEW .5000
HISTORIC EVENTS 1
HIGH OUTLIERS 0
LOW OUTLIERS 0
ZERO OR MISSING 0
SYSTEMATIC EVENTS 44
HISTORIC PERIOD(1887-1981) 95
AQUILLA CREEK
AQUILLA CREEK NR AQUILLA, TX
UNREGULATED
DISCHARGE-FREQUENCY CURVE
BASIN AREA = 308 SQ MI
WATER YEARS IN RECORD
1887,1936,1939-1981

Plate - 3



Aquilla Inflow Max Annual Average
Duration Frequency with Negative Skew
DISCHARGE IN THOUSAND^S OF CFS
1000
800
600
500
400
350
300
250
200
150
100
80
60
50
40
35
30
25
20
15
10
8.0
6.0
5.0
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.80
0.60
0.50
0.40
0.35
0.30
0.25
0.20
0.15
0.10
99.5 99.0 98.0 95.0 90.0 80.0 70.0 60.0 50.0 40.0 30.0 20.0 10.0 5.0 2.0 1.0 0.5 0.2 0.1 0.05 0.01
EXCEEDENCE FREQUENCY IN PERCENT OF YEARS
LEGEND
DURATION SYMBOL
1 DAY
2 DAY
3 DAY
4 DAY
5 DAY
6 DAY
7 DAY
8 DAY
9 DAY
10 DAY
AQUILLA INFLOW
FROM RUN B10X01
MAXIMUM ANNUAL
AVERAGE HIGH FLOW
DURATION FREQUENCY
PLATE V002

Plate - 4

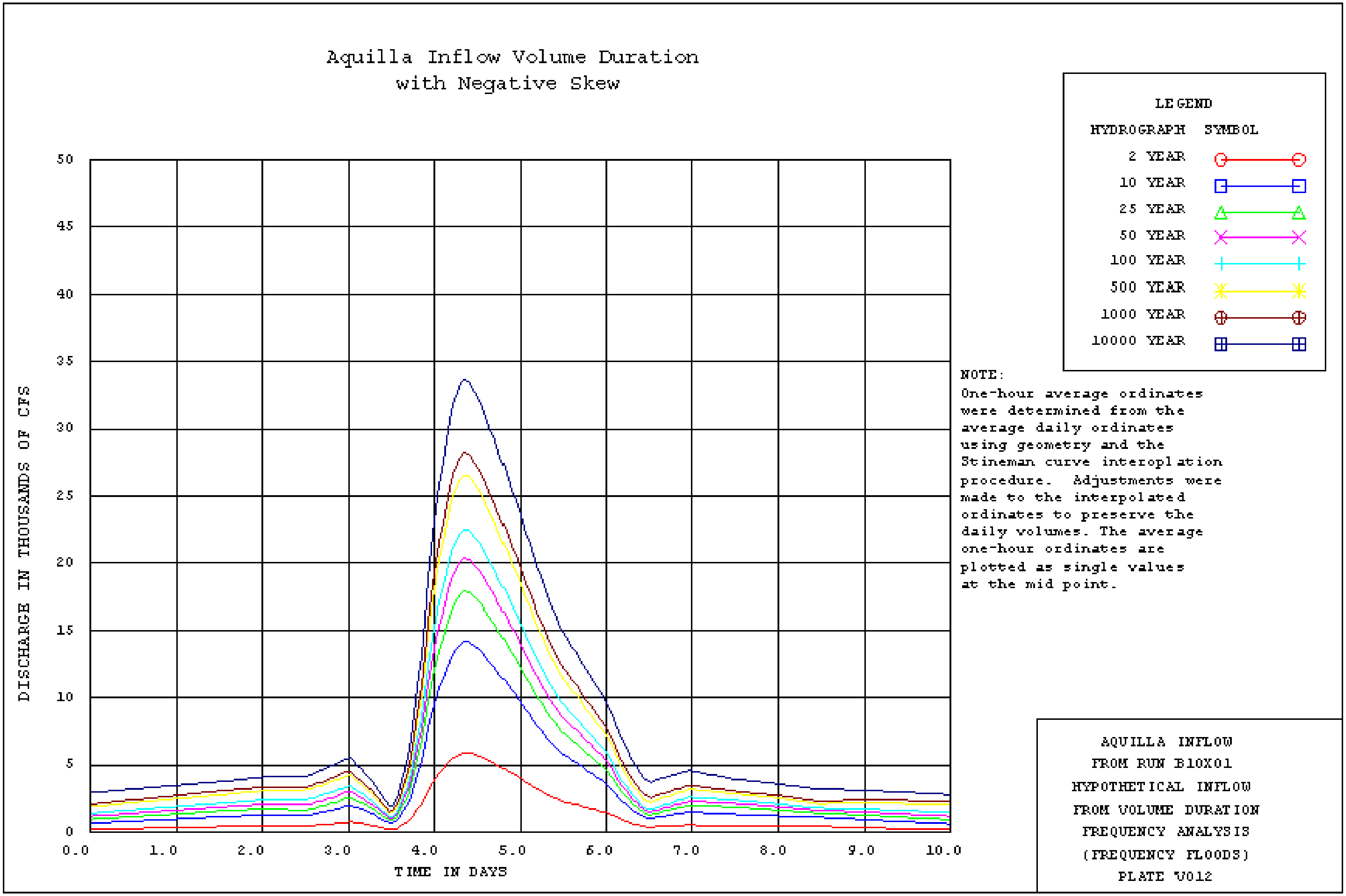
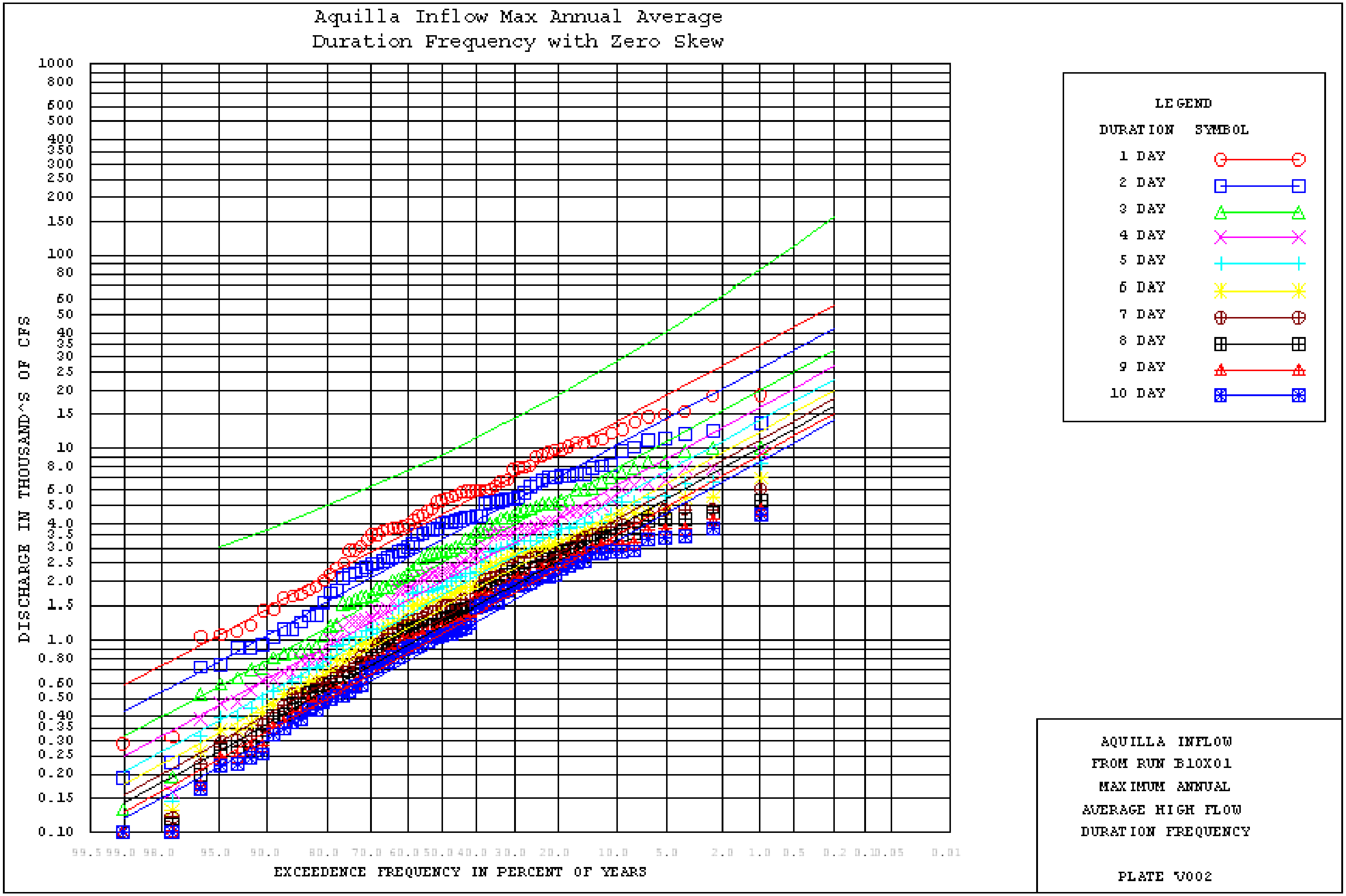

Aquilla Inflow Volume Duration
with Negative Skew
LEGEND
HYDROGRAPH SYMBOL
2 YEAR
10 YEAR
25 YEAR
50 YEAR
100 YEAR
500 YEAR
1000 YEAR
10000 YEAR
NOTE:
One-hour average ordinates were determined from the average daily ordinates using geometry and the Stineman curve interoplation procedure. Adjustments were made to the interpolated ordinates to preserve the daily volumes. The average one-hour ordinates are plotted as single values at the mid point.
DISCHARGE IN THOUSANDS OF CFS
0
5
10
15
20
25
30
35
40
45
50
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
TIME IN DAYS
AQUILLA INFLOW
FROM RUN B10X01
HYPOTHETICAL INFLOW
FROM VOLUME DURATION
FREQUENCY ANALYSIS
(FREQUENCY FLOODS)
PLATE V012

Plate - 5



Aquilla Inflow Max Annual Average
Duration Frequency with Zero Skew
DISCHARGE IN THOUSAND^S OF CFS
EXCEEDENCE FREQUENCY IN PERCENT OF YEARS
LEGEND
DURATION SYMBOL
1 DAY
2 DAY
3 DAY
4 DAY
5 DAY
6 DAY
7 DAY
8 DAY
9 DAY
10 DAY
AQUILLA INFLOW
FROM RUN B10X01
MAXIMUM ANNUAL
AVERAGE HIGH FLOW
DURATION FREQUENCY
PLATE V002

Plate - 6

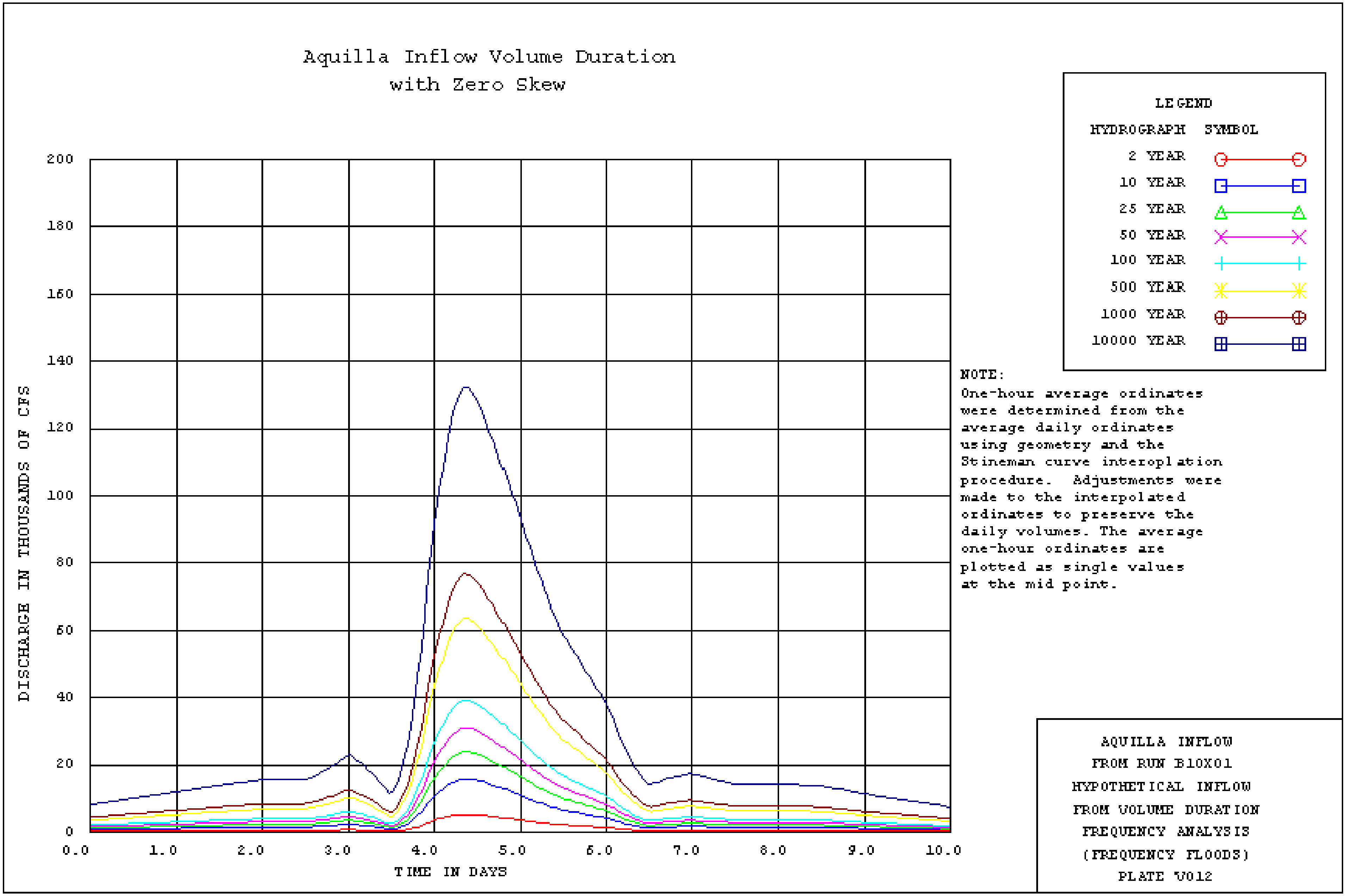
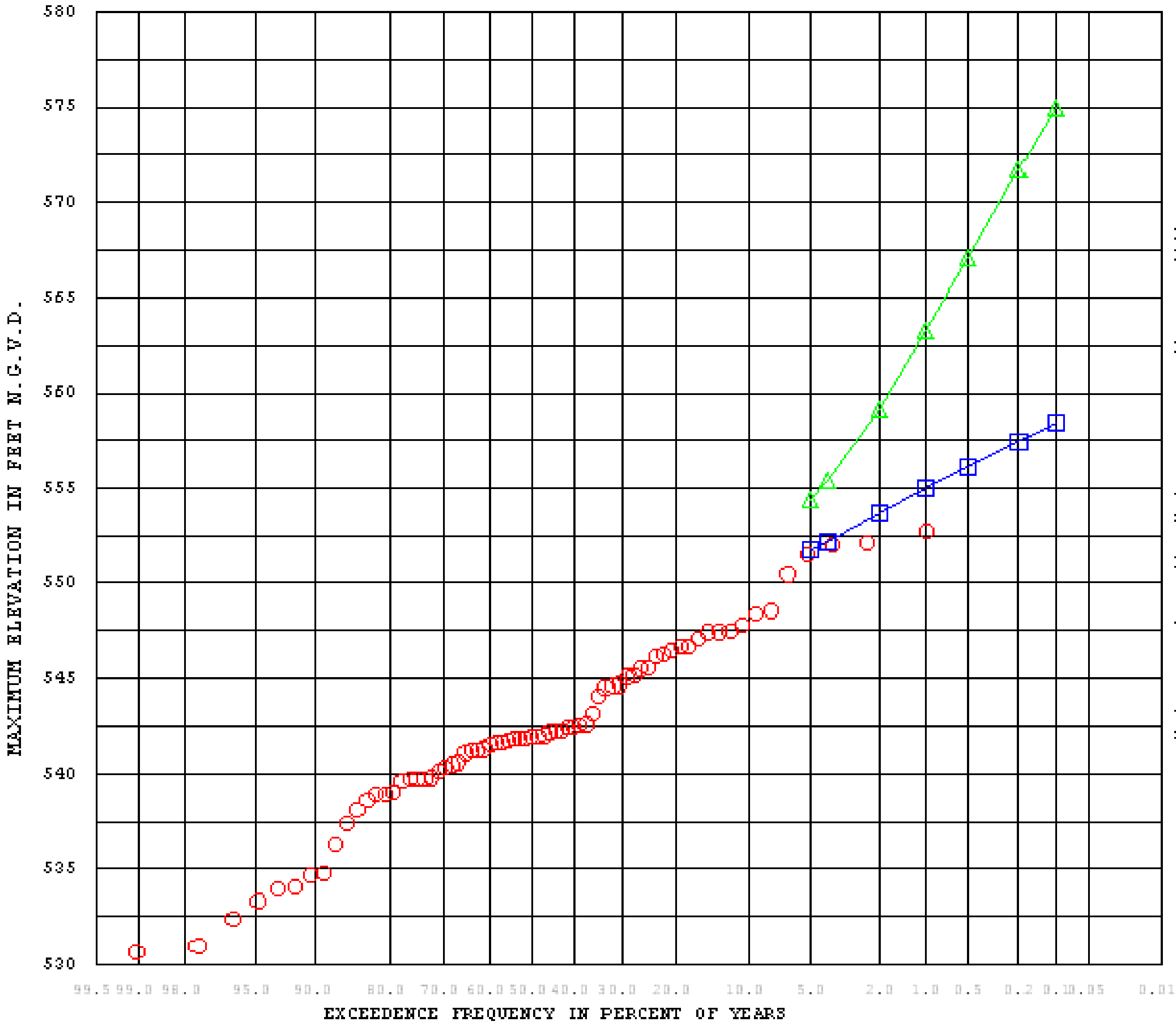

Aquilla Inflow Volume Duration
with Zero Skew
DISCHARGE IN THOUSANDS OF CFS
200
180
160
140
120
100
80
60
40
20
0
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
TIME IN DAYS
LEGEND
HYDROGRAPH SYMBOL
2 YEAR
10 YEAR
25 YEAR
50 YEAR
100 YEAR
500 YEAR
1000 YEAR
10000 YEAR
NOTE:
One-hour average ordinates were determined from the average daily ordinates using geometry and the Stineman curve interoplation procedure. Adjustments were made to the interpolated ordinates to preserve the daily volumes. The average one-hour ordinates are plotted as single values at the mid point.
AQUILLA INFLOW
FROM RUN B10X01
HYPOTHETICAL INFLOW
FROM VOLUME DURATION
FREQUENCY ANALYSIS
(FREQUENCY FLOODS)
PLATE V012

Plate -7



LEGEND

RUN NO.	SYMBOL
B10X01	
Curve # 1	
Curve # 2	

NOTE:
B10X01 = regulated daily simulation (1939-2009)

Curve # 1 is the joint (total) probability elevation frequeny curve based on inflow volume statistics (skew to fit data) and the elevation duration curve from run B10X01 using a 1 hour perfect inflow forecast followed by a normal recession with a down stream control at mouth of Aquilla Creek.
Curve # 2 is the joint (total) probability elevation frequeny curve based on inflow volume statistics (skew = 0.0) and the full elevation duration curve from run B10X01 using a 1 hour perfect inflow forecast followed by a normal recession with a down stream control at mouth of Aquilla Creek.

BRAZOS BASIN
HYDROLOGIC STUDIES
AQUILLA LAKE
ELEVATION FREQUENCY
DAILY SIMULATION FOR
PERIOD 1939-2009
PLATE B001

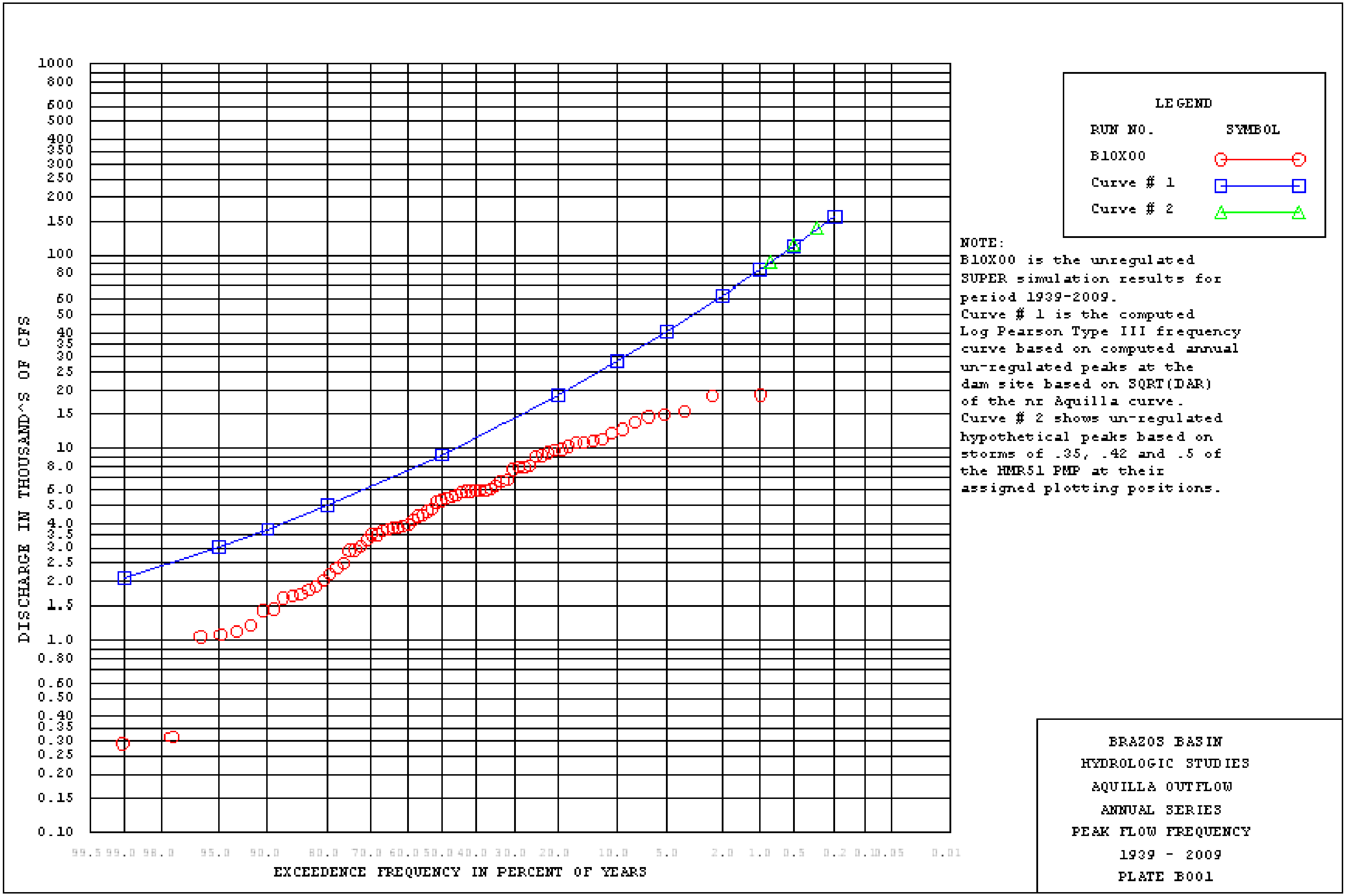

LEGEND
RUN NO.
SYMBOL
B10X00
Curve # 1
Curve # 2
NOTE:
B10X00 is the unregulated
SUPER simulation results for
period 1939-2009.
Curve # 1 is the computed
Log Pearson Type III frequency
curve based on computed annual
un-regulated peaks at the
dam site based on SQRT(DAR)
of the nr Aquilla curve.
Curve # 2 shows un-regulated
hypothetical peaks based on
storms of .35, .42 and .5 of
the HMR51 PMP at their
assigned plotting positions.
DISCHARGE IN THOUSAND^S OF CFS
EXCEEDENCE FREQUENCY IN PERCENT OF YEARS
BRAZOS BASIN
HYDROLOGIC STUDIES
AQUILLA OUTFLOW
ANNUAL SERIES
PEAK FLOW FREQUENCY
1939 - 2009
PLATE B001

Plate - 9

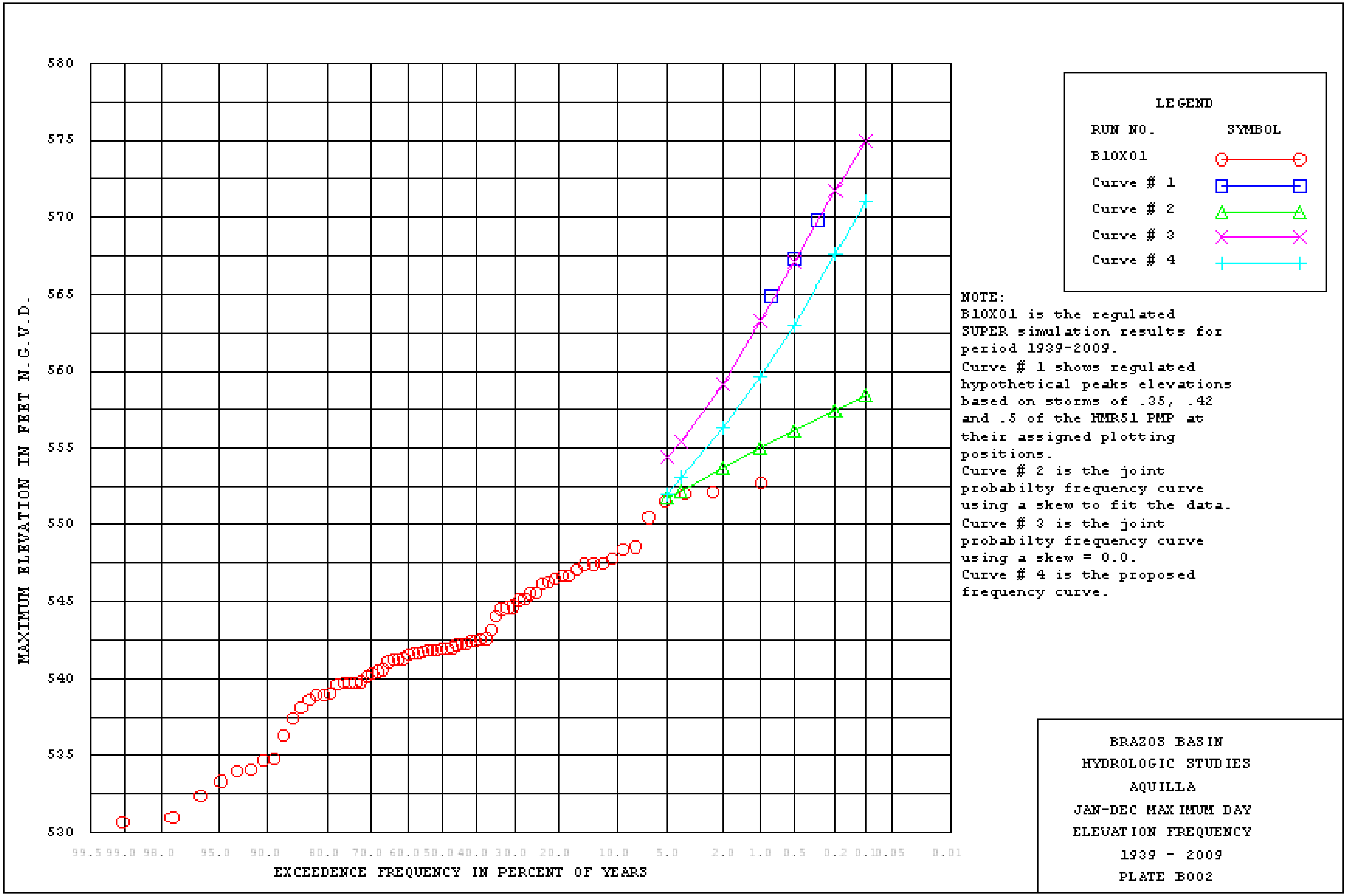

MAXIMUM ELEVATION IN FEET N.G.V.D.
580
575
570
565
560
555
550
545
540
535
530
99.5 99.0 98.0 95.0 90.0 80.0 70.0 60.0 50.0 40.0 30.0 20.0 10.0 5.0 2.0 1.0 0.5 0.2 0.1 0.05 0.01
EXCEEDENCE FREQUENCY IN PERCENT OF YEARS
LEGEND
RUN NO. SYMBOL
B10X01
Curve # 1
Curve # 2
Curve # 3
Curve # 4
NOTE:
B10X01 is the regulated
SUPER simulation results for
period 1939-2009.
Curve # 1 shows regulated
hypothetical peaks elevations
based on storms of .35, .42
and .5 of the HMR51 PMP at
their assigned plotting
positions.
Curve # 2 is the joint
probabilty frequency curve
using a skew to fit the data.
Curve # 3 is the joint
probabilty frequency curve
using a skew = 0.0.
Curve # 4 is the proposed
frequency curve.
BRAZOS BASIN
HYDROLOGIC STUDIES
AQUILLA
JAN-DEC MAXIMUM DAY
ELEVATION FREQUENCY
1939 - 2009
PLATE B002

Plate -10

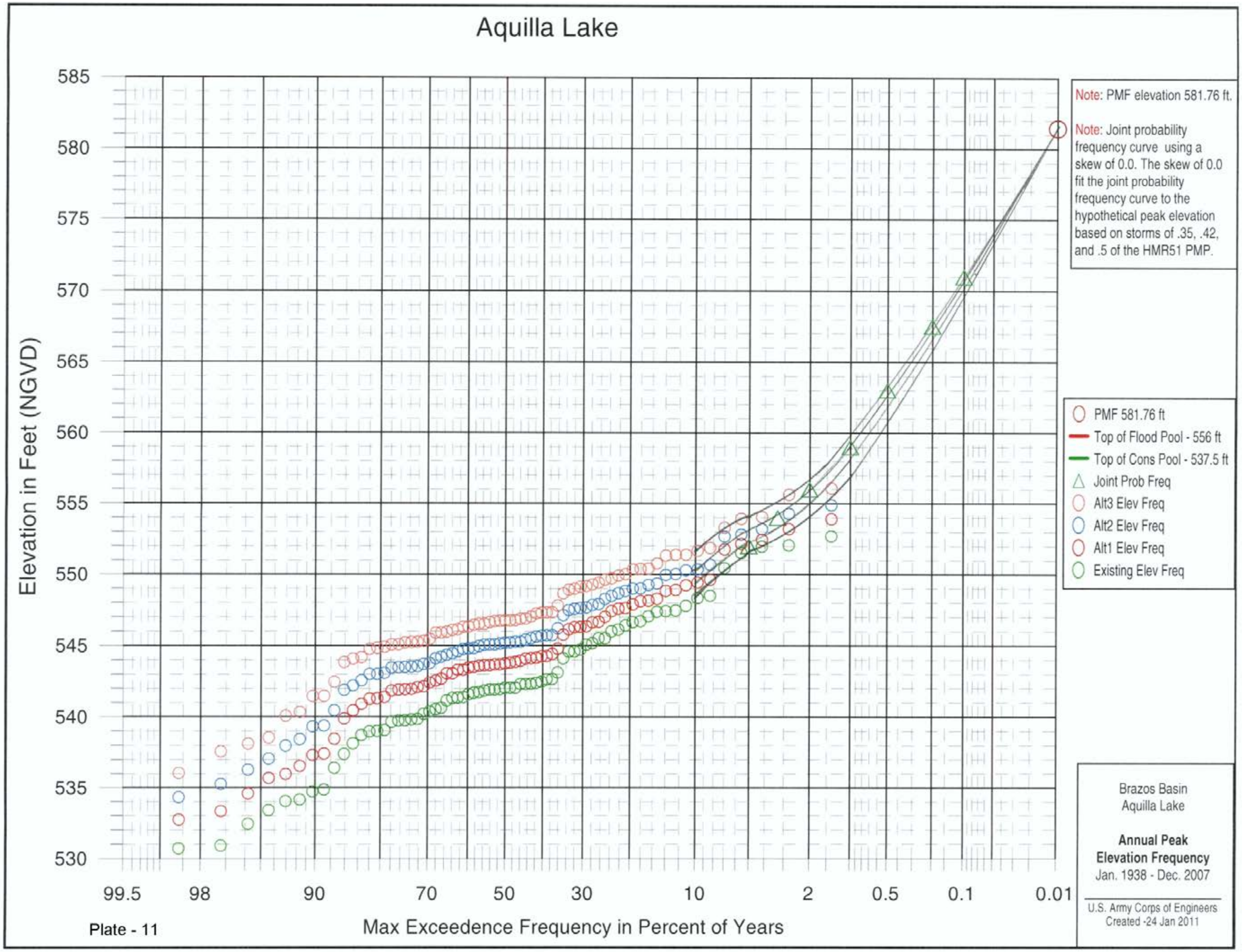
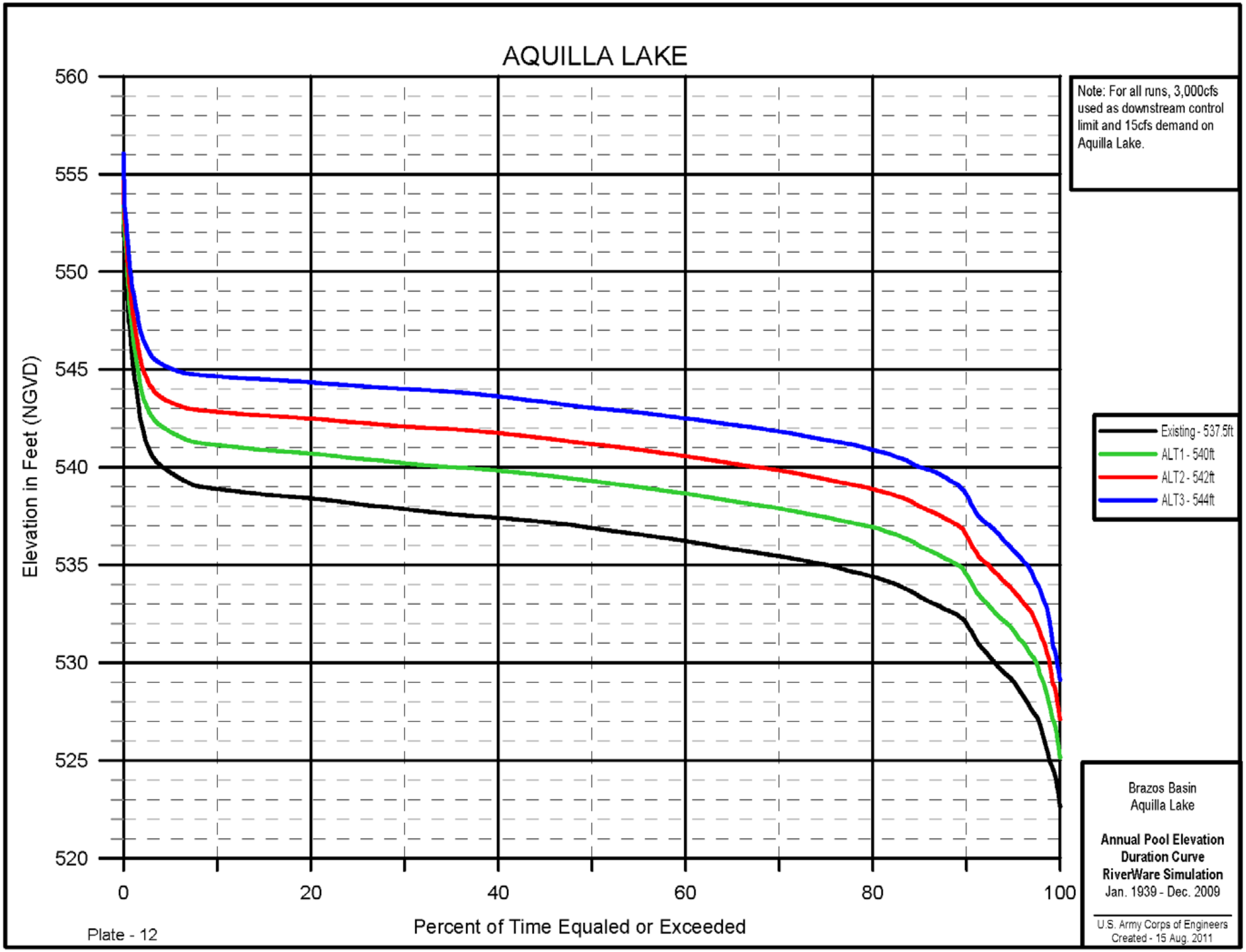
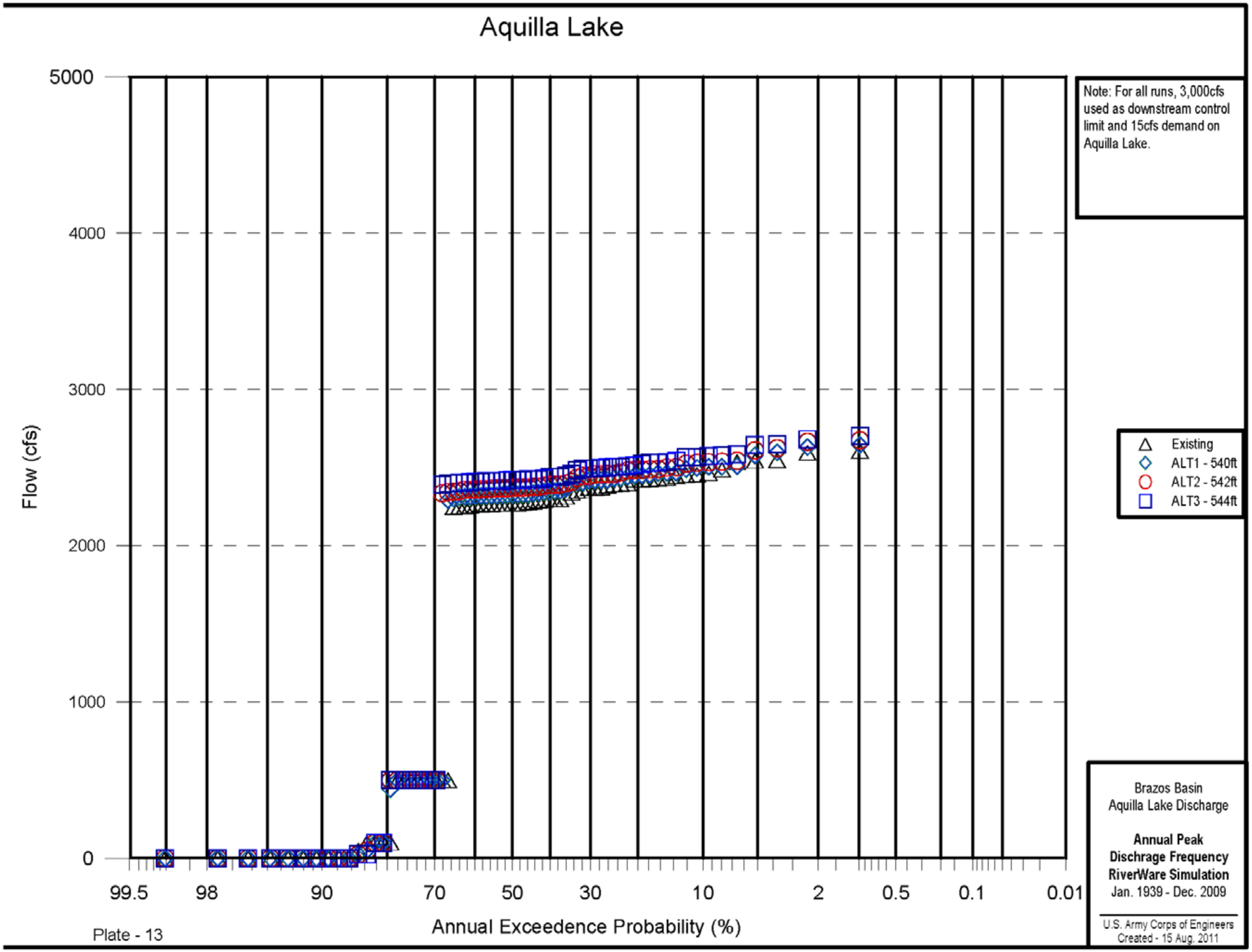

Aquilla Lake
Elevation in Feet (NGVD)
585
580
575
570
565
560
555
550
545
540
535
530
99.5
98
90
70
50
30
10
2
0.5
0.1
0.01
Max Exceedence Frequency in Percent of Years
Note: PMF elevation 581.76 ft.
Note: Joint probability frequency curve using a skew of 0.0. The skew of 0.0 fit the joint probability frequency curve to the hypothetical peak elevation based on storms of .35, .42, and .5 of the HMR51 PMP.
PMF 581.76 ft
Top of Flood Pool - 556 ft
Top of Cons Pool - 537.5 ft
Joint Prob Freq
Alt3 Elev Freq
Alt2 Elev Freq
Alt1 Elev Freq
Existing Elev Freq
Brazos Basin
Aquilla Lake
Annual Peak
Elevation Frequency
Jan. 1938 - Dec. 2007
U.S. Army Corps of Engineers
Created -24 Jan 2011

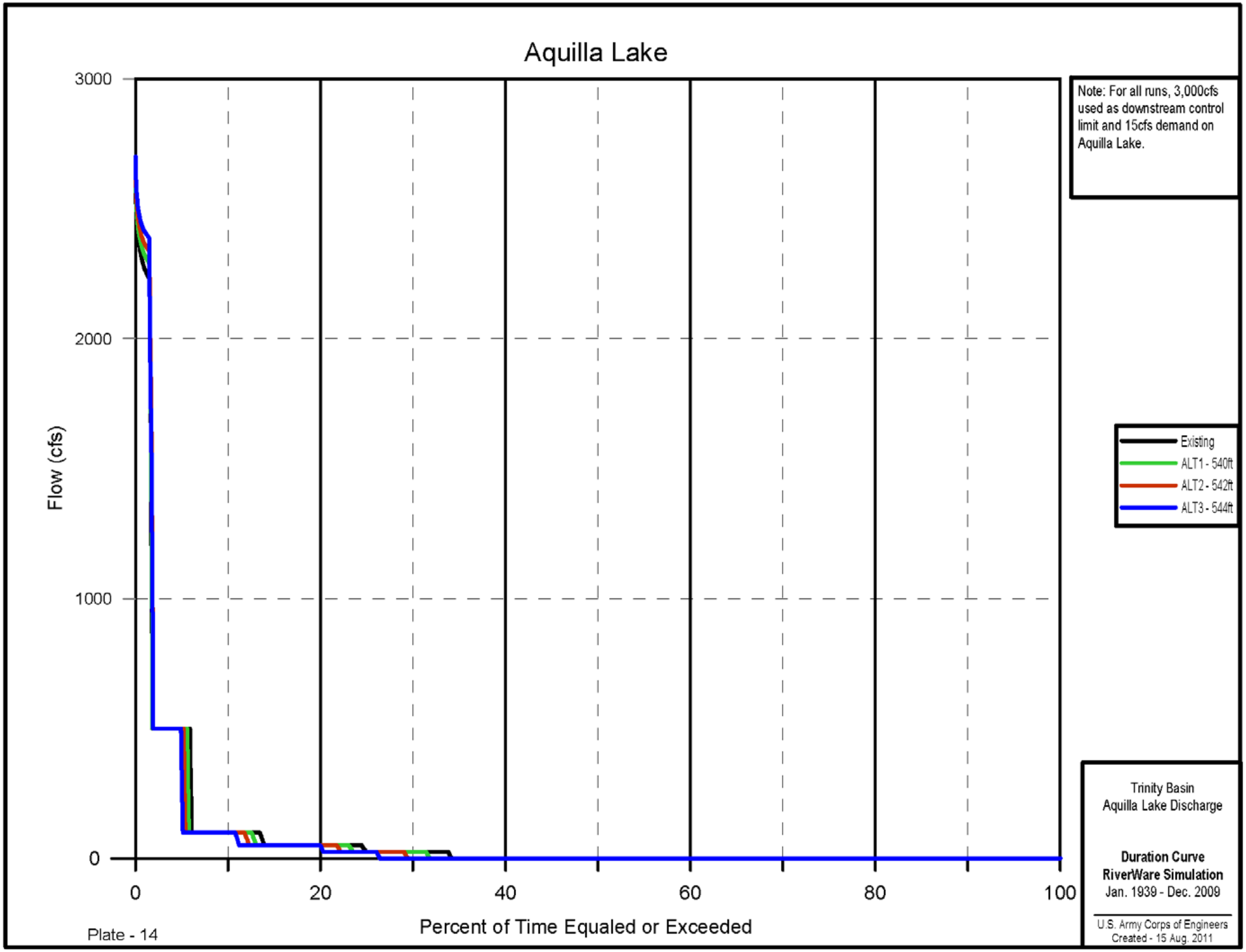
Plate - 11

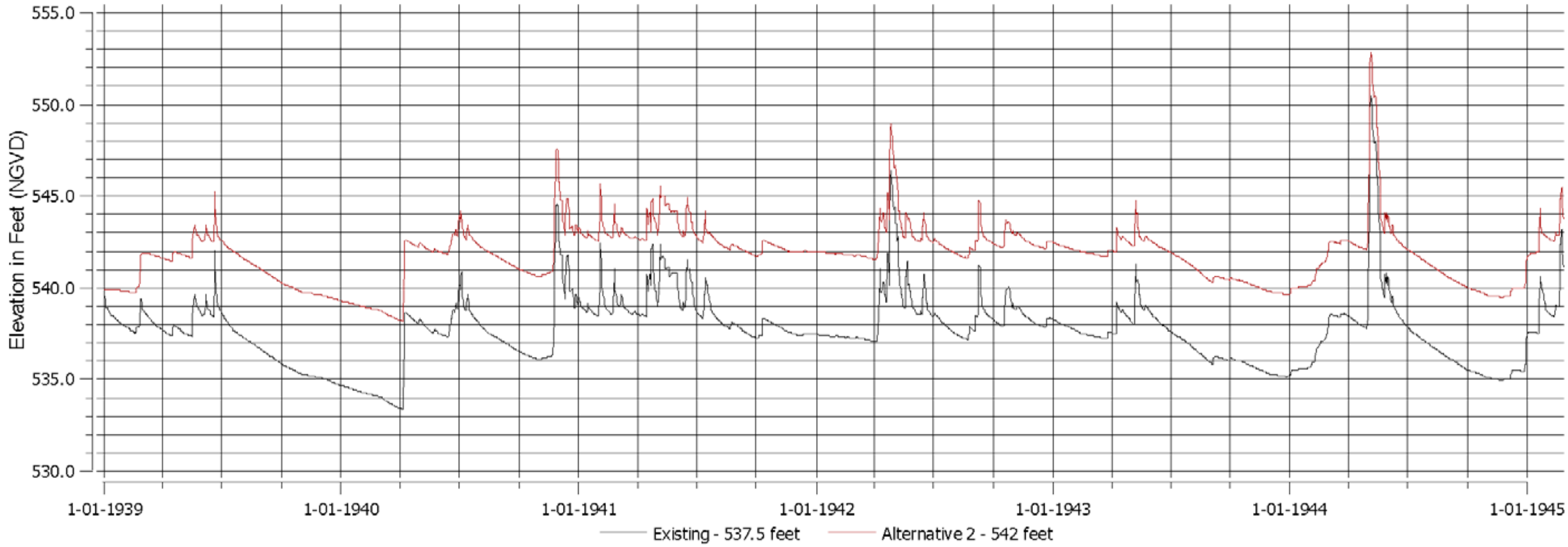


AQUILLA LAKE
Note: For all runs, 3,000cfs used as downstream control limit and 15cfs demand on Aquilla Lake.
Existing - 537.5ft
ALT1 - 540ft
ALT2 - 542ft
ALT3 - 544ft
Elevation in Feet (NGVD)
560
555
550
545
540
535
530
525
520
0
20
40
60
80
100
Percent of Time Equaled or Exceeded
Brazos Basin
Aquilla Lake
Annual Pool Elevation
Duration Curve
RiverWare Simulation
Jan. 1939 - Dec. 2009
U.S. Army Corps of Engineers
Created - 15 Aug. 2011

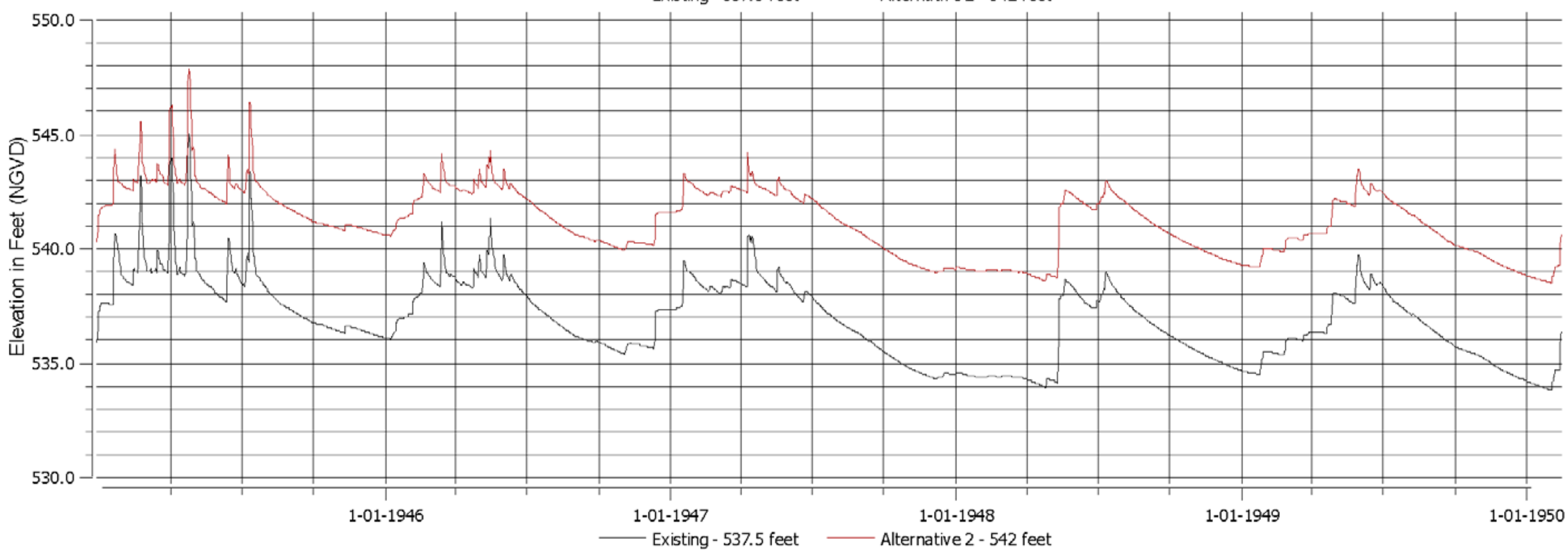
Plate - 12


Aquilla Lake
Flow (cfs)
5000
4000
3000
2000
1000
0
99.5
98
90
70
50
30
10
2
0.5
0.1
0.01
Annual Exceedence Probability (%)
Plate - 13
Note: For all runs, 3,000cfs used as downstream control limit and 15cfs demand on Aquilla Lake.
Existing
ALT1 - 540ft
ALT2 - 542ft
ALT3 - 544ft
Brazos Basin
Aquilla Lake Discharge
Annual Peak
Dischrage Frequency
RiverWare Simulation
Jan. 1939 - Dec. 2009
U.S. Army Corps of Engineers
Created - 15 Aug. 2011


Aquilla Lake
3000
2000
1000
0
Flow (cfs)
0
20
40
60
80
100
Percent of Time Equaled or Exceeded
Note: For all runs, 3,000cfs used as downstream control limit and 15cfs demand on Aquilla Lake.
Existing
ALT1 - 540ft
ALT2 - 542ft
ALT3 - 544ft
Trinity Basin
Aquilla Lake Discharge
Duration Curve
RiverWare Simulation
Jan. 1939 - Dec. 2009
U.S. Army Corps of Engineers
Created - 15 Aug. 2011

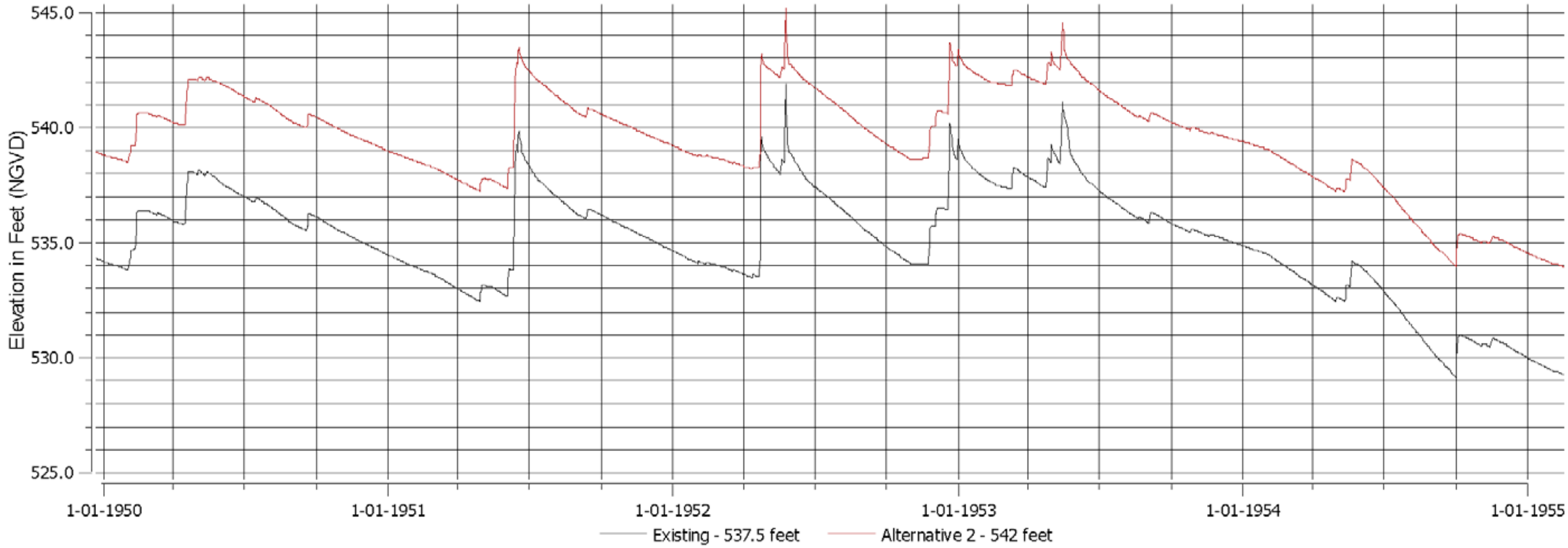


Elevation in Feet (NGVD)
555.0
550.0
545.0
540.0
535.0
530.0
1-01-1939
1-01-1940
1-01-1941
1-01-1942
1-01-1943
1-01-1944
1-01-1945
Existing - 537.5 feet
Alternative 2 - 542 feet

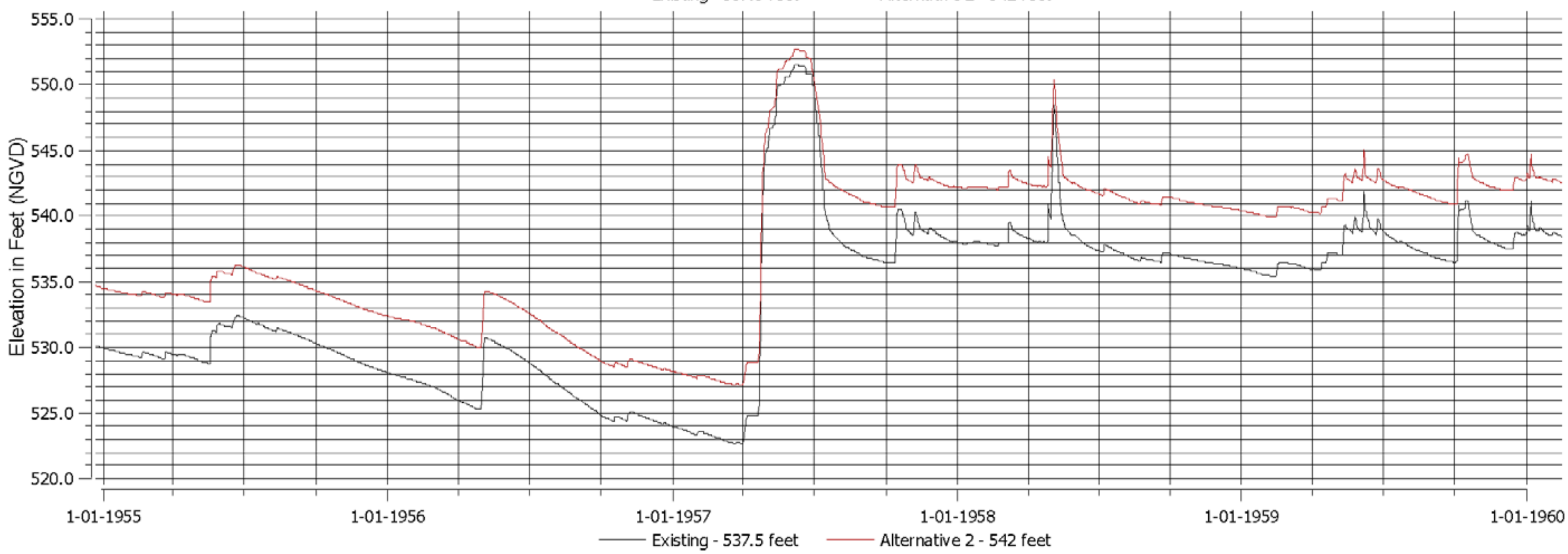


Elevation in Feet (NGVD)
550.0
545.0
540.0
535.0
530.0
1-01-1946
1-01-1947
1-01-1948
1-01-1949
1-01-1950
Existing - 537.5 feet
Alternative 2 - 542 feet

Plate - 15

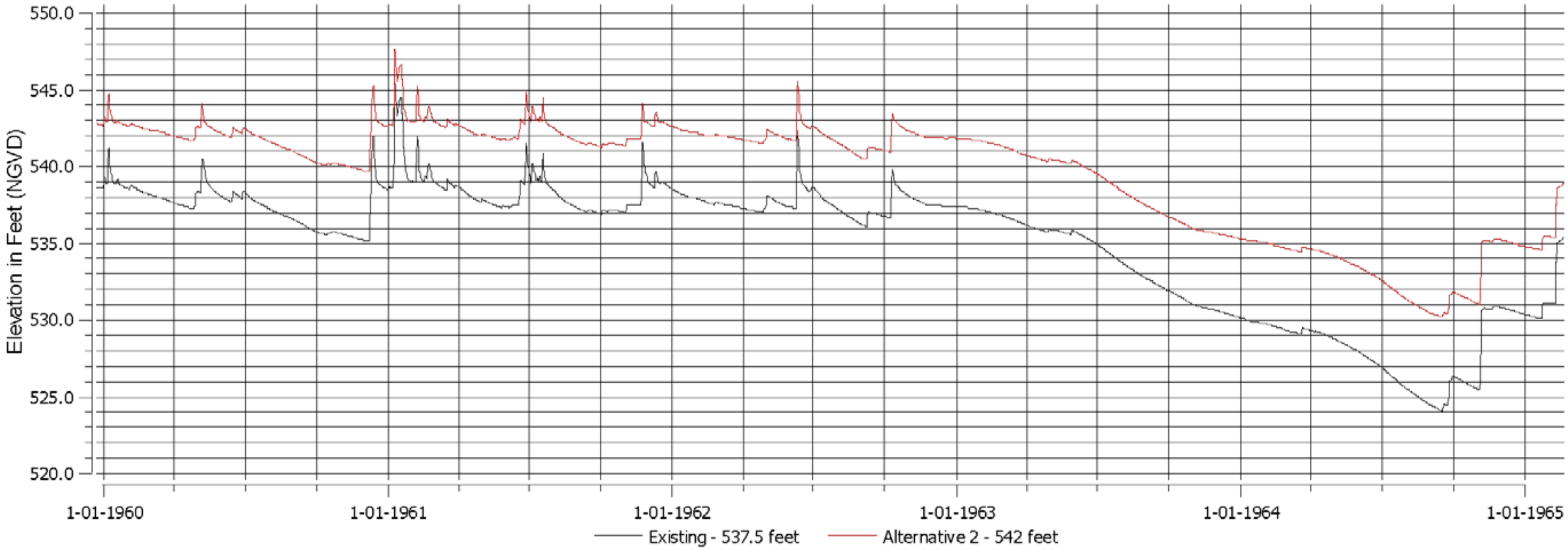


545.0
540.0
535.0
530.0
525.0
Elevation in Feet (NGVD)
1-01-1950
1-01-1951
1-01-1952
1-01-1953
1-01-1954
1-01-1955
Existing - 537.5 feet
Alternative 2 - 542 feet



555.0
550.0
545.0
540.0
535.0
530.0
525.0
520.0
Elevation in Feet (NGVD)
1-01-1955
1-01-1956
1-01-1957
1-01-1958
1-01-1959
1-01-1960
Existing - 537.5 feet
Alternative 2 - 542 feet

Plate - 16


550.0
545.0
540.0
535.0
530.0
525.0
520.0
Elevation in Feet (NGVD)
1-01-1960
1-01-1961
1-01-1962
1-01-1963
1-01-1964
1-01-1965
Existing - 537.5 feet
Alternative 2 - 542 feet

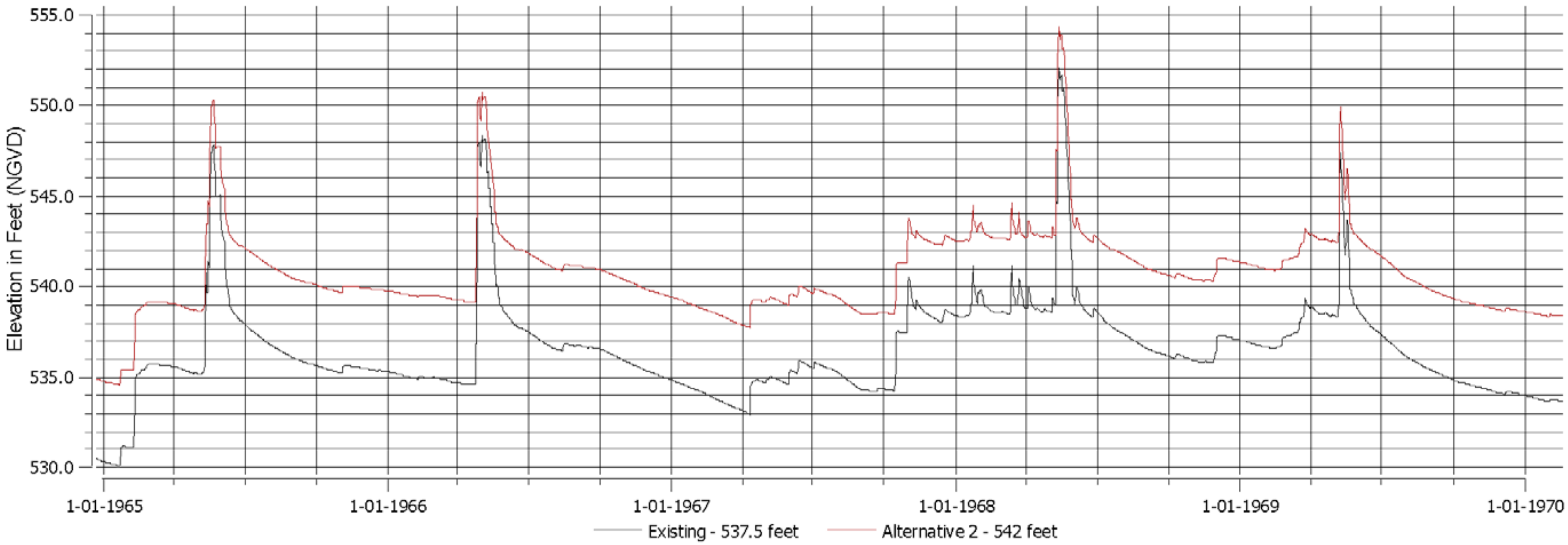
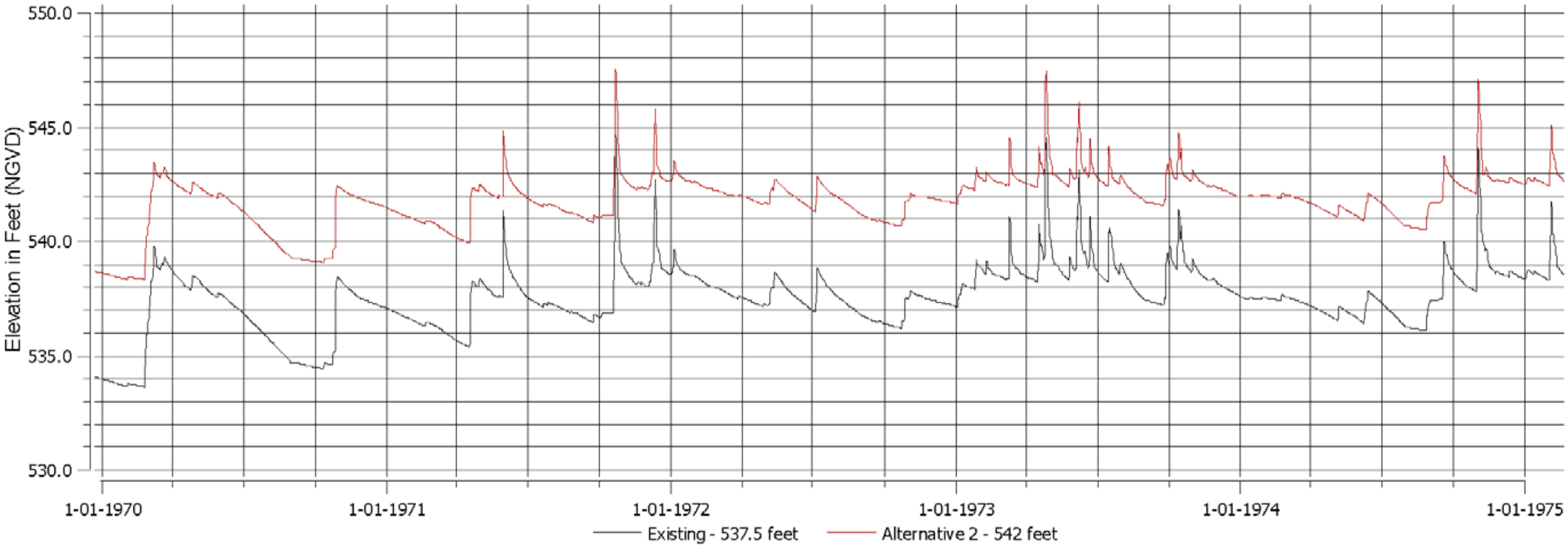
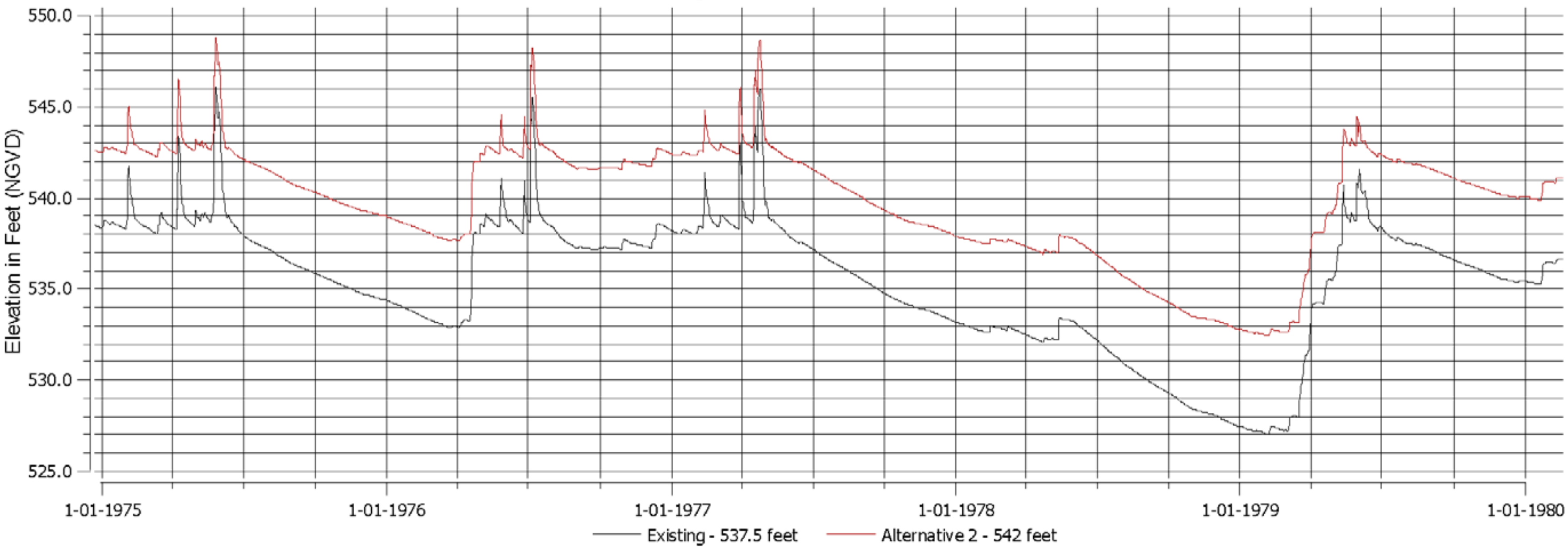

555.0
550.0
545.0
540.0
535.0
530.0
Elevation in Feet (NGVD)
1-01-1965
1-01-1966
1-01-1967
1-01-1968
1-01-1969
1-01-1970
Existing - 537.5 feet
Alternative 2 - 542 feet

Plate - 17

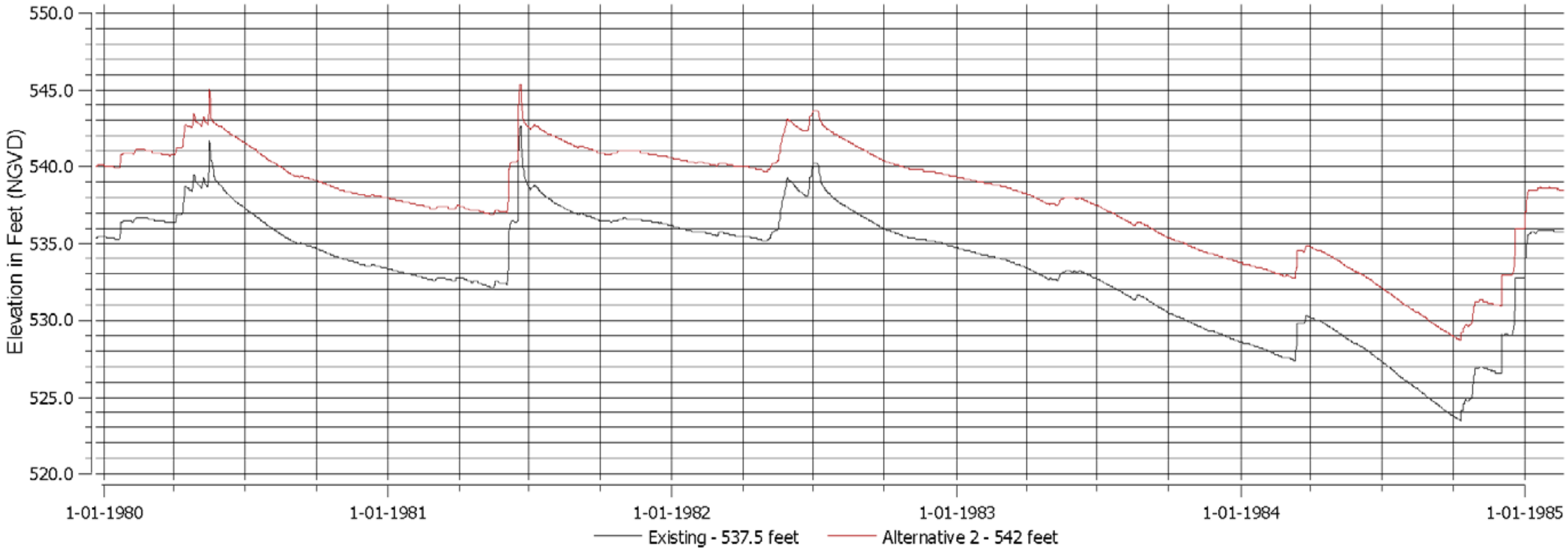


550.0
545.0
540.0
535.0
530.0
Elevation in Feet (NGVD)
1-01-1970
1-01-1971
1-01-1972
1-01-1973
1-01-1974
1-01-1975
Existing - 537.5 feet
Alternative 2 - 542 feet



550.0
545.0
540.0
535.0
530.0
525.0
Elevation in Feet (NGVD)
1-01-1975
1-01-1976
1-01-1977
1-01-1978
1-01-1979
1-01-1980
Existing - 537.5 feet
Alternative 2 - 542 feet

Plate - 18


550.0
545.0
540.0
535.0
530.0
525.0
520.0
Elevation in Feet (NGVD)
1-01-1980
1-01-1981
1-01-1982
1-01-1983
1-01-1984
1-01-1985
Existing - 537.5 feet
Alternative 2 - 542 feet

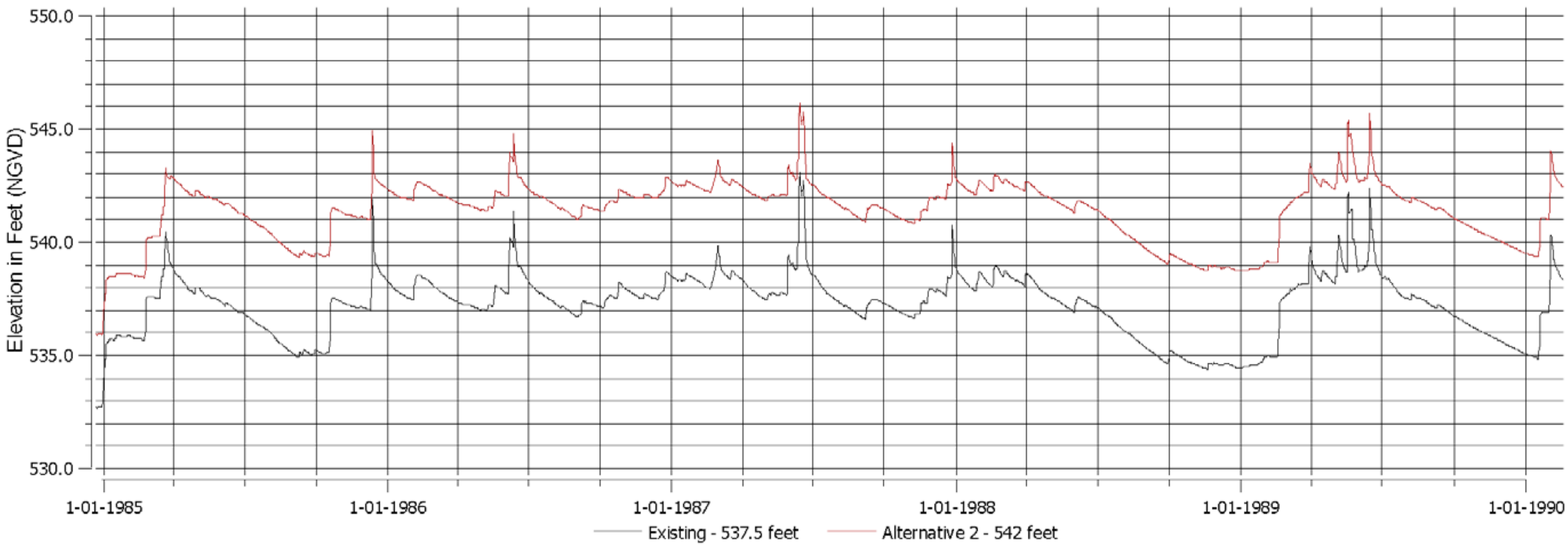
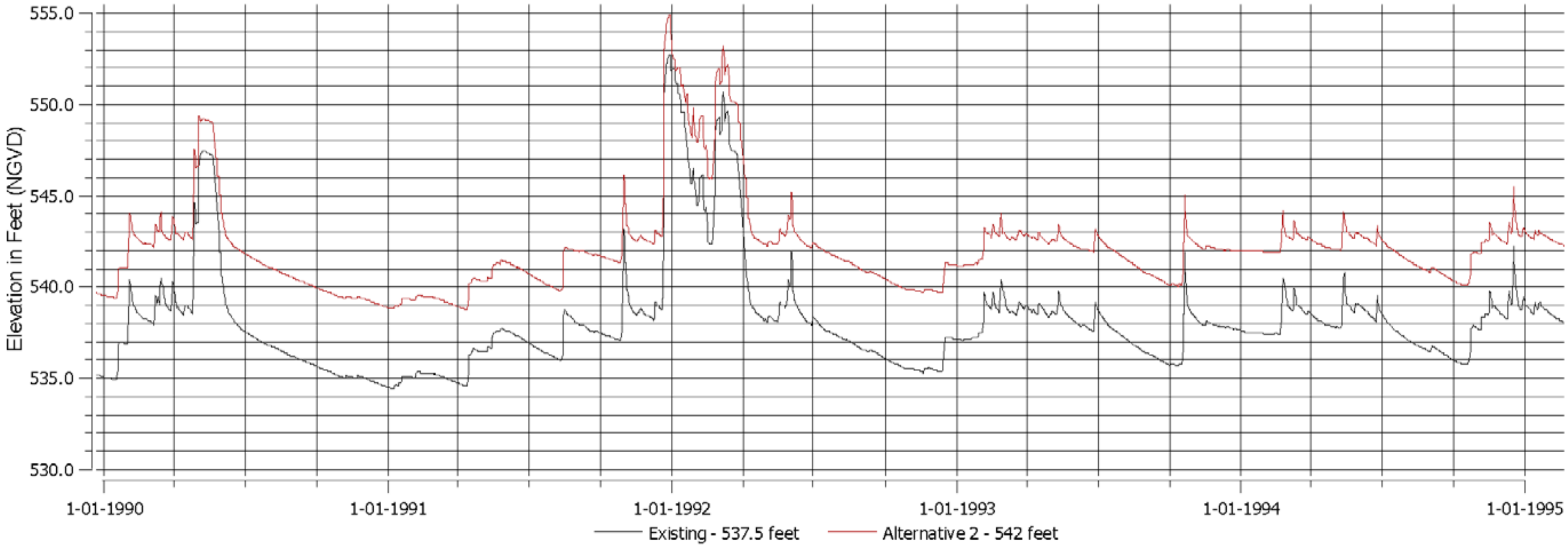
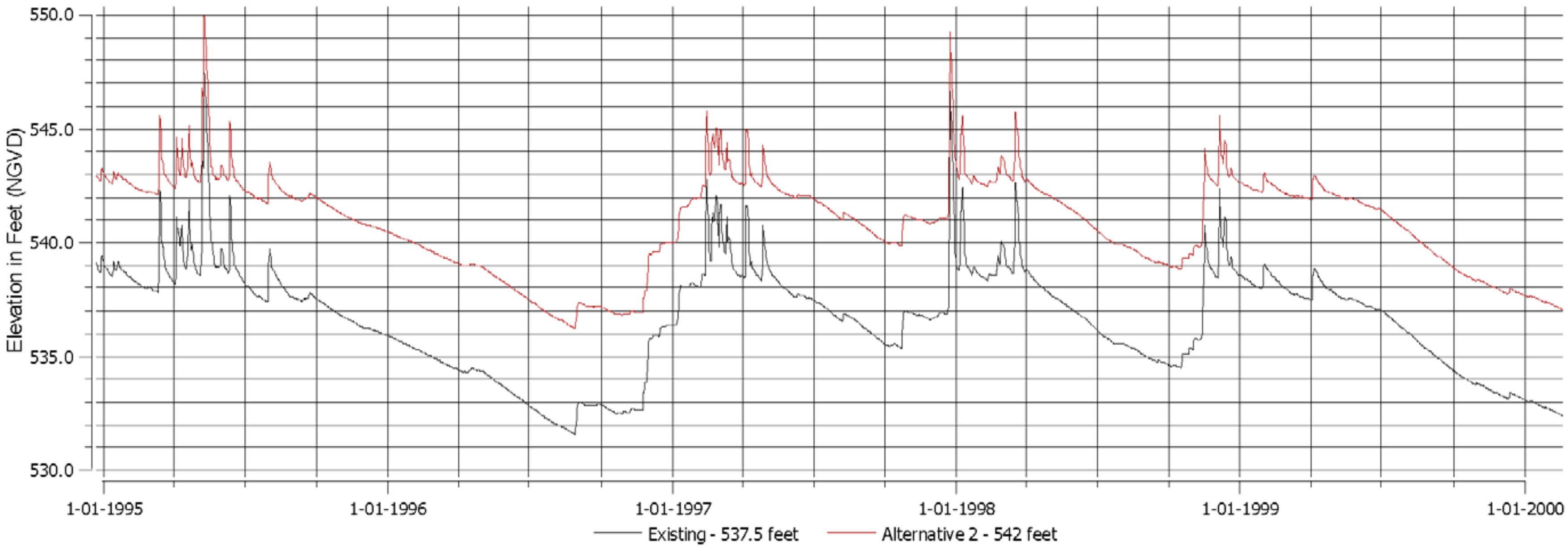

550.0
545.0
540.0
535.0
530.0
Elevation in Feet (NGVD)
1-01-1985
1-01-1986
1-01-1987
1-01-1988
1-01-1989
1-01-1990
Existing - 537.5 feet
Alternative 2 - 542 feet

Plate - 19

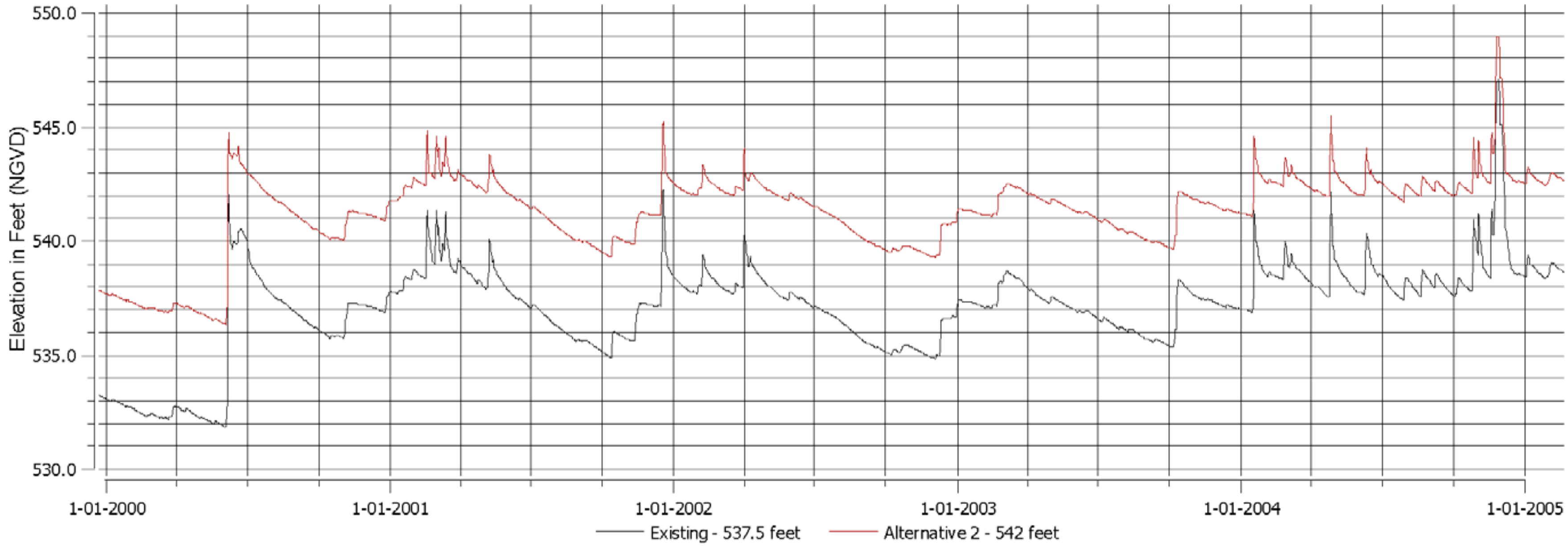


555.0
550.0
545.0
540.0
535.0
530.0
Elevation in Feet (NGVD)
1-01-1990
1-01-1991
1-01-1992
1-01-1993
1-01-1994
1-01-1995
Existing - 537.5 feet
Alternative 2 - 542 feet



550.0
545.0
540.0
535.0
530.0
Elevation in Feet (NGVD)
1-01-1995
1-01-1996
1-01-1997
1-01-1998
1-01-1999
1-01-2000
Existing - 537.5 feet
Alternative 2 - 542 feet

Plate - 20


550.0
545.0
540.0
535.0
530.0
Elevation in Feet (NGVD)
1-01-2000
1-01-2001
1-01-2002
1-01-2003
1-01-2004
1-01-2005
Existing - 537.5 feet
Alternative 2 - 542 feet

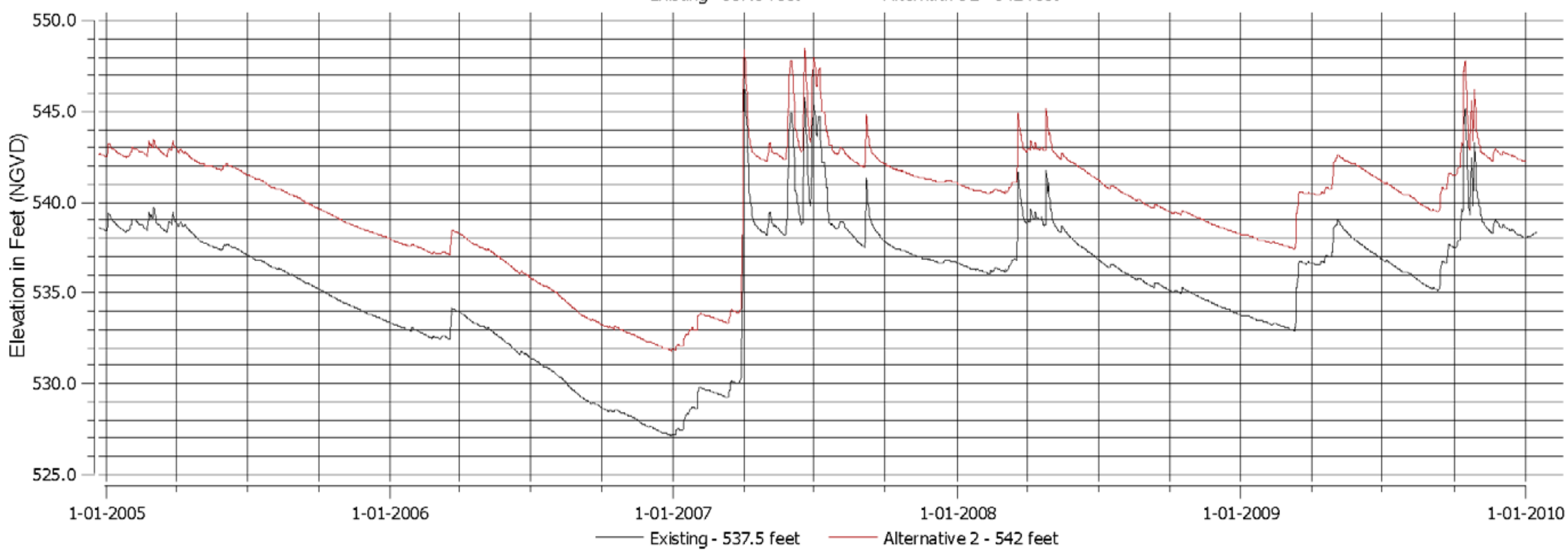
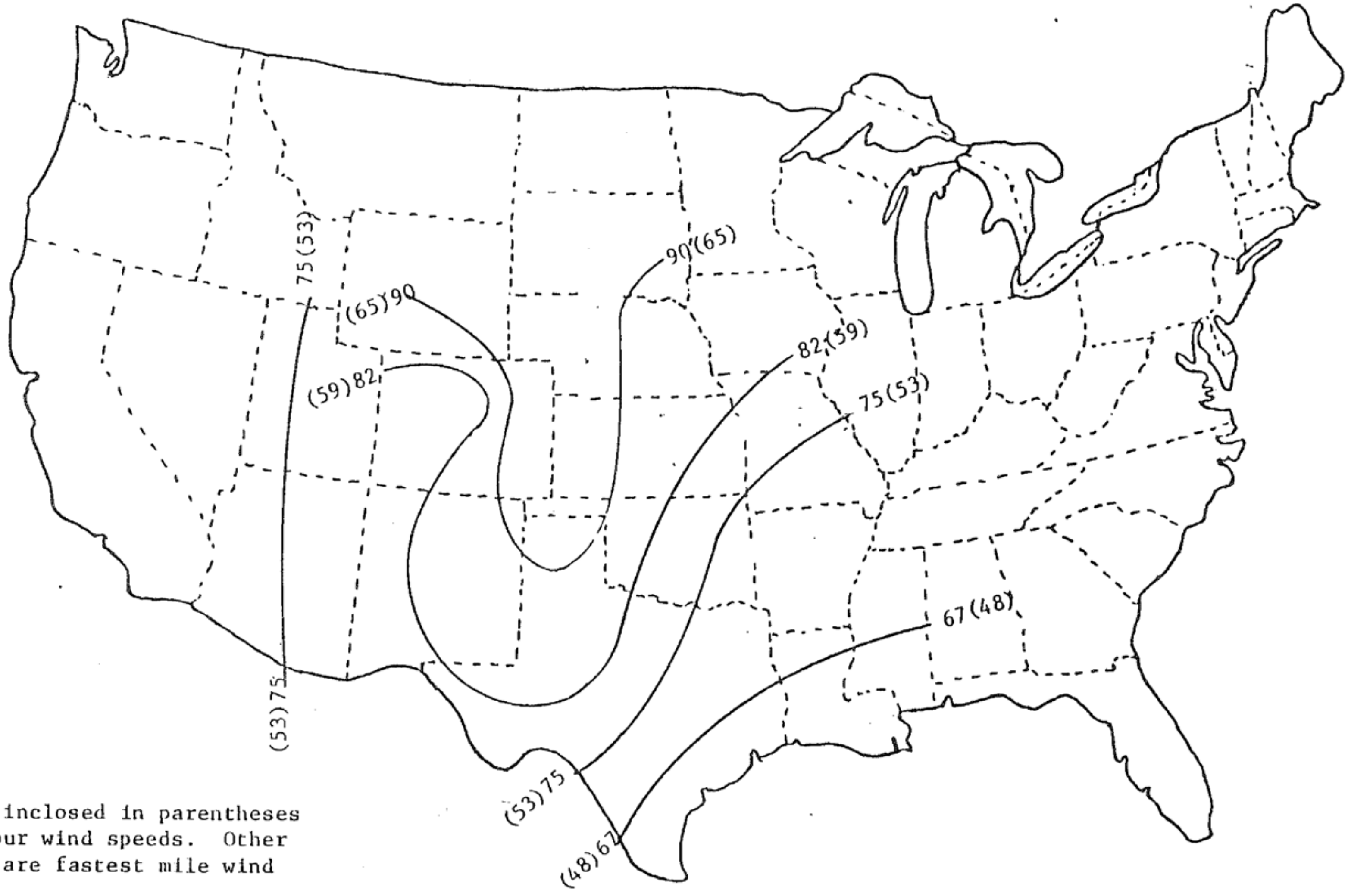
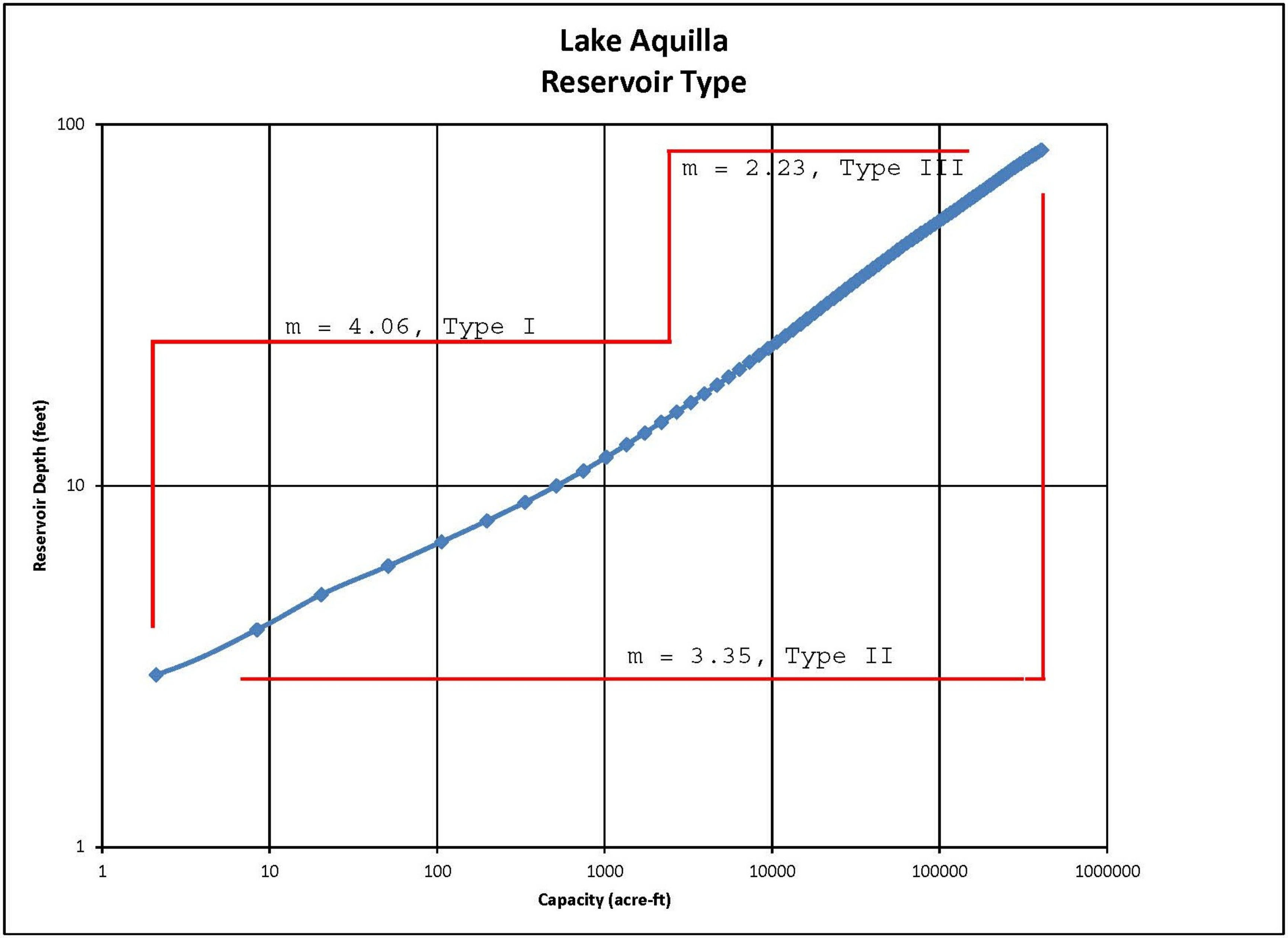

550.0
545.0
540.0
535.0
530.0
525.0
Elevation in Feet (NGVD)
1-01-2005
1-01-2006
1-01-2007
1-01-2008
1-01-2009
1-01-2010
Existing - 537.5 feet
Alternative 2 - 542 feet

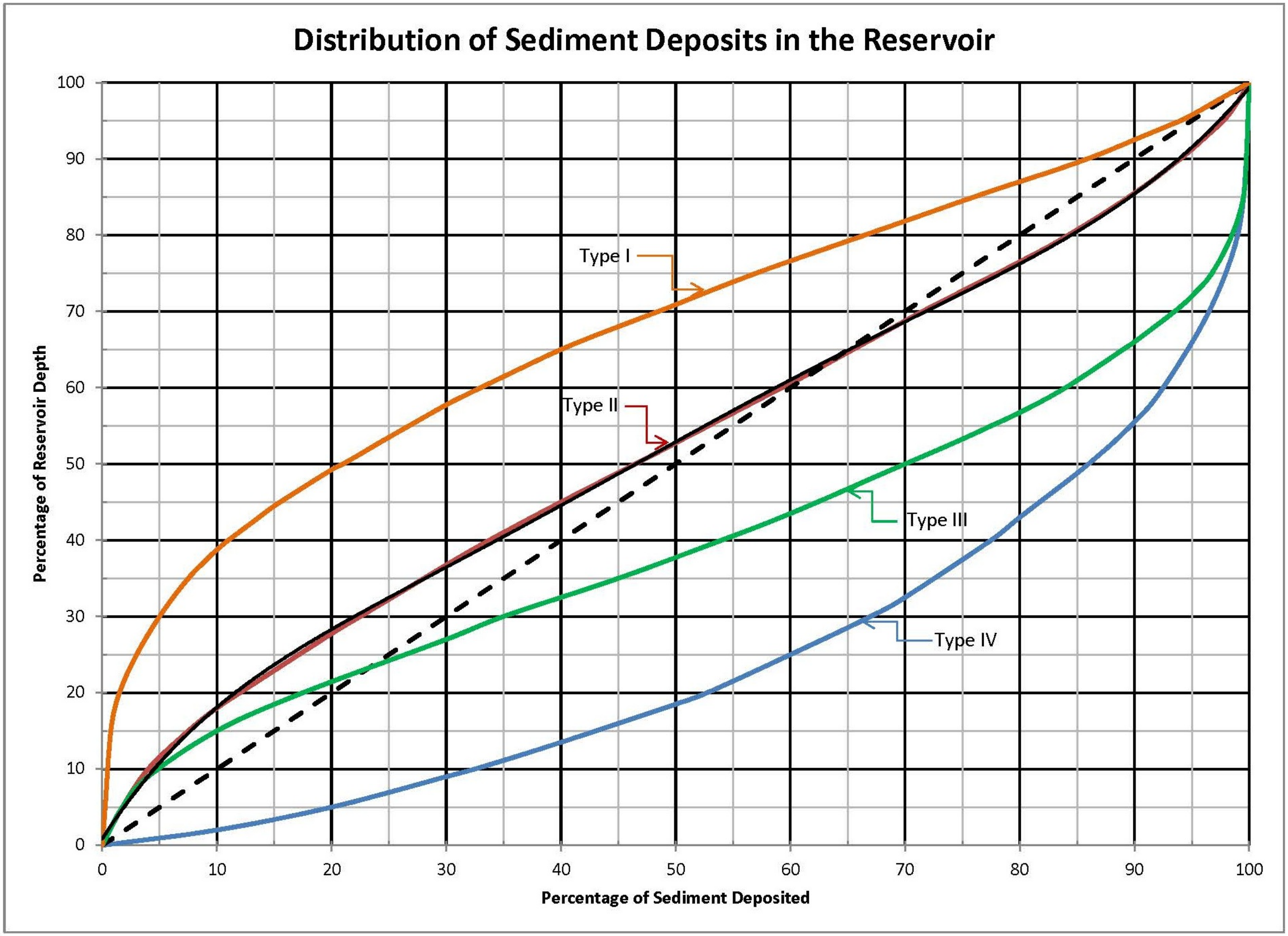
Plate - 21



NOTE:
Numbers inclosed in parentheses are 1-hour wind speeds. Other numbers are fastest mile wind speeds.

FIGURE 3: ONE PERCENT CHANCE WINDS


Lake Aquilla
Reservoir Type
m = 2.23, Type III
m = 4.06, Type I
m = 3.35, Type II
100
10
1
Reservoir Depth (feet)
1
10
100
1000
10000
100000
1000000
Capacity (acre-ft)


Distribution of Sediment Deposits in the Reservoir
Percentage of Reservoir Depth
Percentage of Sediment Deposited
0
10
20
30
40
50
60
70
80
90
100
Type I
Type II
Type III
Type IV

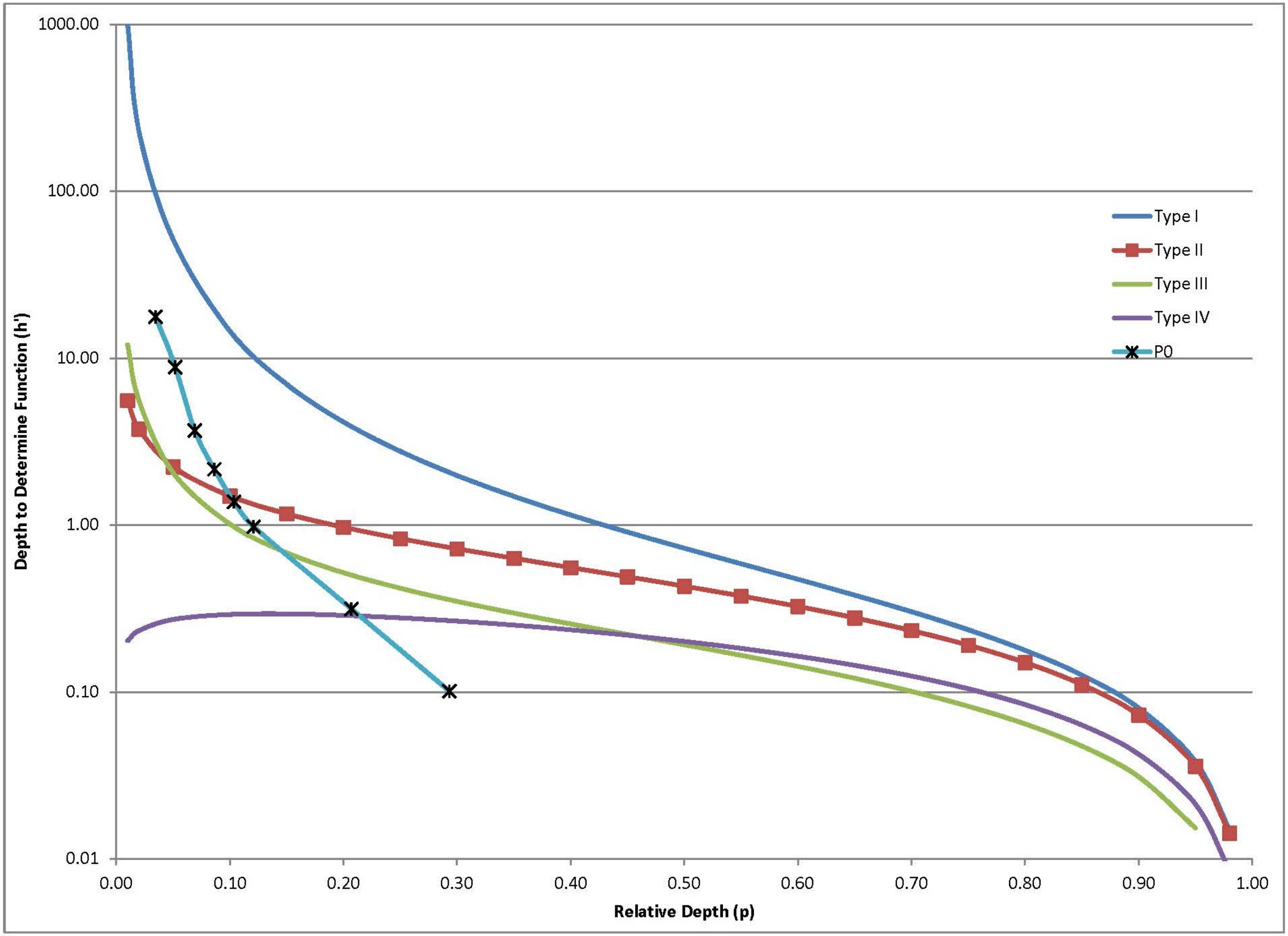

1000.00
100.00
10.00
1.00
0.10
0.01
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00
Depth to Determine Function (h')
Relative Depth (p)
Type I
Type II
Type III
Type IV
P0

Plate - 25