## Section 10 – Lubbock's Next Water Supply – Year 2012 – Lake Alan Henry

### Content

a. 2006-07 Preliminary Engineering for Infrastructure
b. Original Feasibility Study
c. State Water Permit
d. Prior and Current Yield information
e. 2005 Volumetric study
f. 2005 Dam inspection

### Summary

The John T. Montford Dam is located about 60 miles southeast of Lubbock in Kent and Garza Counties. The Dam, which impounds Lake Alan Henry, was developed in cooperation with the Brazos River Authority. Plans for construction were completed and sealed by Freese & Nichols, Inc. on October 19, 1990, and then approved by the Texas Water Commission on November 21, 1990. Construction of the Dam was completed in October 1993, and impoundment of Lake Alan Henry began in November 1993. The dam is a zoned earthfill embankment with a slurry wall cutoff, a maximum height of about 140 feet above the original stream bed, a length of approximately 4,150 feet, a top dam elevation of 2,263 feet mean sea level (ft-msl), and a conservation pool elevation of 2,220 ft-msl.

In order to take advantage of Lake Alan Henry as a water supply, a water transmission line, water treatment facilities, and related infrastructure must be constructed. The City initiated the Preliminary Engineering for this water transmission line and related infrastructure in August of 2006. The project will include design, sizing and location recommendations for a 60 mile water transmission line, for 2 to 3 pump stations, and for a water treatment facility. In addition, plans to connect the new water source to the City's existing water distribution system will be developed.

By about June of 2007, the City will be able to select a route for the water transmission line and locations for the pump stations and water treatment facility. With these locations designated, the City will be ready to acquire right-of-way and property for these portions of the project. Once the Preliminary Engineering is complete, the City will be ready to begin Final Design of the infrastructure improvements. Once final design is complete, the City will be prepared to bid out the project for construction. The goal is to have the project completed by 2012.

Lake Alan Henry has a permit to divert and use 35,000 acre-feet of water annually. The original yield for the lake was 32,000 acre-feet of water annually. That amount has reduced to a 2004 projection of 22,500 acre-feet annually. In September of 2005, the Texas Water Development Board completed a volumetric study for Lake Alan Henry and determined that the Lake's volume is actually 18% less than that originally projected. The study indicated that the storage volume of Lake Alan Henry is 94,808 acre-feet instead of the 115,937 acre-feet originally projected. The 2007 safe yield for Lake Alan Henry was estimated by HDR Engineering, Inc. to be 19,000 acre-feet per annum.

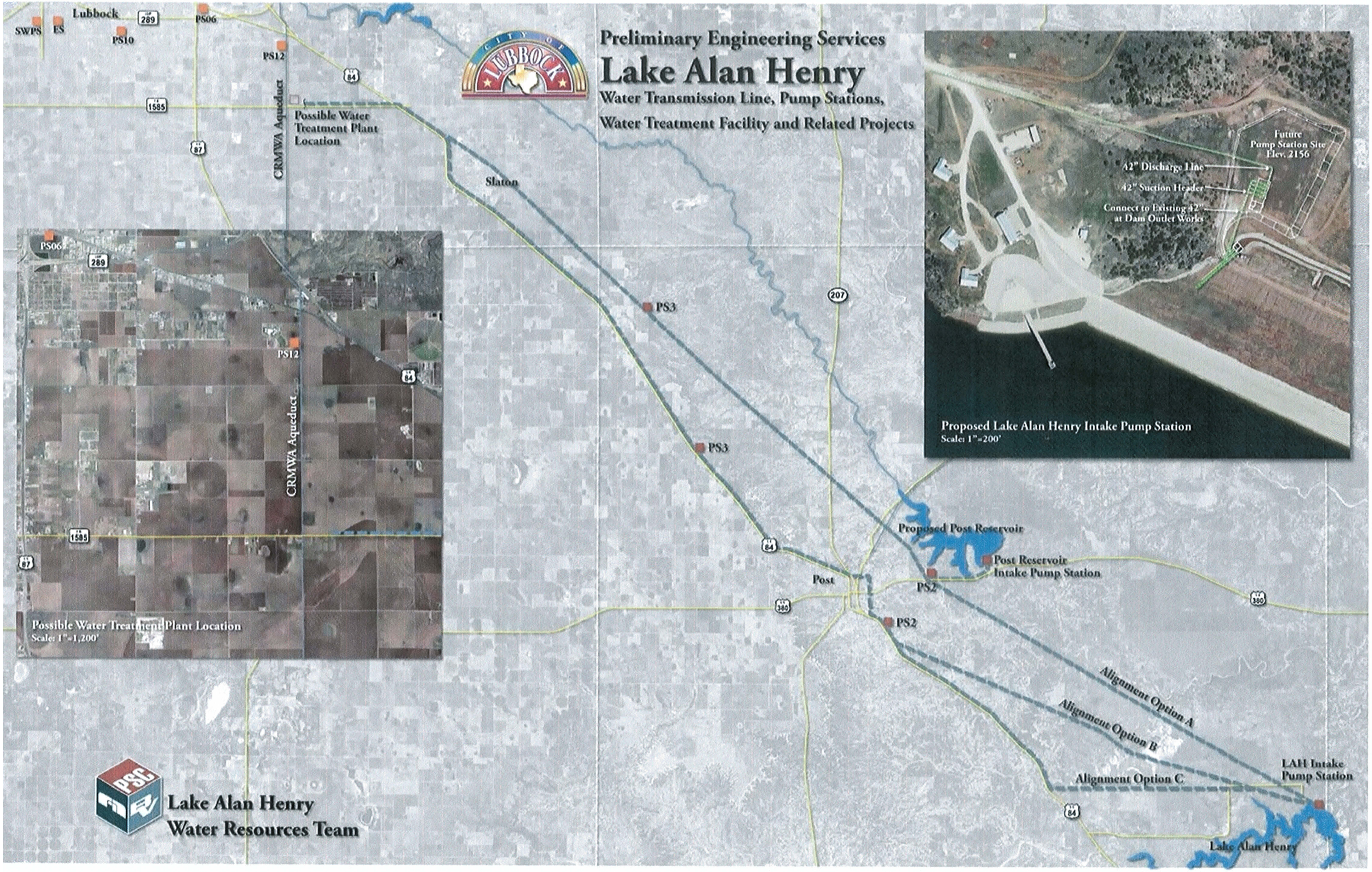
Because of the decrease in yield projections, the Lubbock Water Advisory Commission has recommended that the City supplement Lake Alan Henry water with water from other sources. Lake Alan Henry water supplies can be supplemented by the use of developed waters originating from storm water and wastewater effluent.

Since Lake Alan Henry receives its water from the South Fork tributaries of the Double Mountain Fork of the Brazos River, one logical alternative is to supplement Lake Alan Henry with waters from the North Fork. The additional watershed could help provide additional water supplies that the transmission line could deliver to Lubbock and area communities.

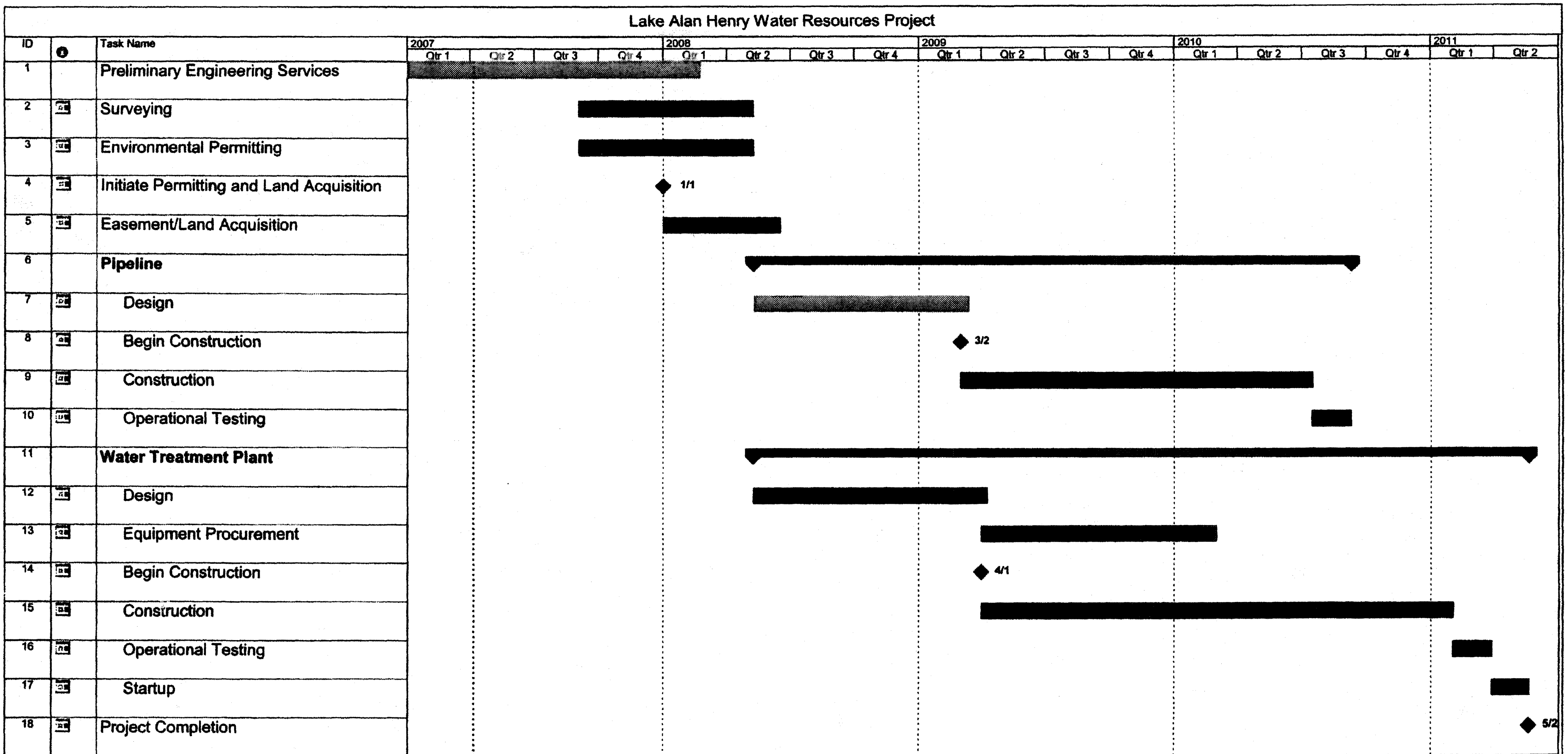
Developed waters of the City could be discharged into either the North Fork or the South Fork to supplement Lake Alan Henry. A North Fork option could be accomplished through the proposed Post Reservoir and would capture both developed storm water and wastewater effluent. A South Fork option would not capture developed storm water since it is discharged into the North Fork. Without the addition of an approximately 40 mile pipeline, a South Fork option could only capture about half of the current volumes of wastewater effluent for reuse.

# Section 10 – Lubbock's Next Water Supply – Year 2012 - Lake Alan Henry

a. 2007-07 Preliminary Engineering for Lake Alan Henry



Preliminary Engineering Services
Lake Alan Henry
Water Transmission Line, Pump Stations,
Water Treatment Facility and Related Projects
City of Lubbock
Lubbock
SWPS
ES
PS10
PS06
PS12
289
84
1585
87
CRMWA Aqueduct
Possible Water Treatment Plant Location
Slaton
PS3
PS3
207
Proposed Post Reservoir
Post Reservoir Intake Pump Station
Post
PS2
PS2
380
380
Alignment Option A
Alignment Option B
Alignment Option C
LAH Intake Pump Station
Lake Alan Henry
Possible Water Treatment Plant Location
Scale: 1"=1,200'
Future Pump Station Site Elev. 2156
42" Discharge Line
42" Suction Header
Connect to Existing 42" at Dam Outlet Works
Proposed Lake Alan Henry Intake Pump Station
Scale: 1"=200'
Lake Alan Henry Water Resources Team



Lake Alan Henry Water Resources Project
ID
Task Name
2007
2008
2009
2010
2011
Qtr 1
Qtr 2
Qtr 3
Qtr 4
1 Preliminary Engineering Services
2 Surveying
3 Environmental Permitting
4 Initiate Permitting and Land Acquisition
1/1
5 Easement/Land Acquisition
6 Pipeline
7 Design
8 Begin Construction
3/2
9 Construction
10 Operational Testing
11 Water Treatment Plant
12 Design
13 Equipment Procurement
14 Begin Construction
4/1
15 Construction
16 Operational Testing
17 Startup
18 Project Completion
5/2

# Section 10 – Lubbock's Next Water Supply – Year 2012 - Lake Alan Henry

## b. Original Feasibility Study

LUBBOCK, TEXAS

FEASIBILITY REPORT ON JUSTICEBURG RESERVOIR

1978

## 1. INTRODUCTION

In 1971 Freese and Nichols prepared a Report on Water Supply (1) for the City of Lubbock, in which the probable long-range water requirements of the City were projected and potential sources of additional future supply were evaluated. Comparison of several alternative sources led to the recommendation that Lubbock consider development of new surface water supplies from the Post Reservoir site on the North Fork of the Double Mountain Fork of the Brazos River and the Justiceburg Reservoir Site on the South Fork of the Double Mountain Fork. It was estimated that the two reservoirs, operated in conjunction with a moderate volume of regulating storage at the proposed Canyon Lake 8, could add as much as 80 MGD (million gallons per day) to the peak-day capability of the Lubbock water system and would provide approximately 40,000 acre-feet per year of additional annual yield. Figure 1.1 is a vicinity map, showing the locations of these facilities and their relationship to the City of Lubbock.

In May of 1975, Freese and Nichols was asked to prepare a supplemental report (2), in which the basic findings of the 1971 study were reviewed and up-dated. The 1975 investigation, like the earlier study, indicated the potential of the combined Post-Justiceburg sources. The

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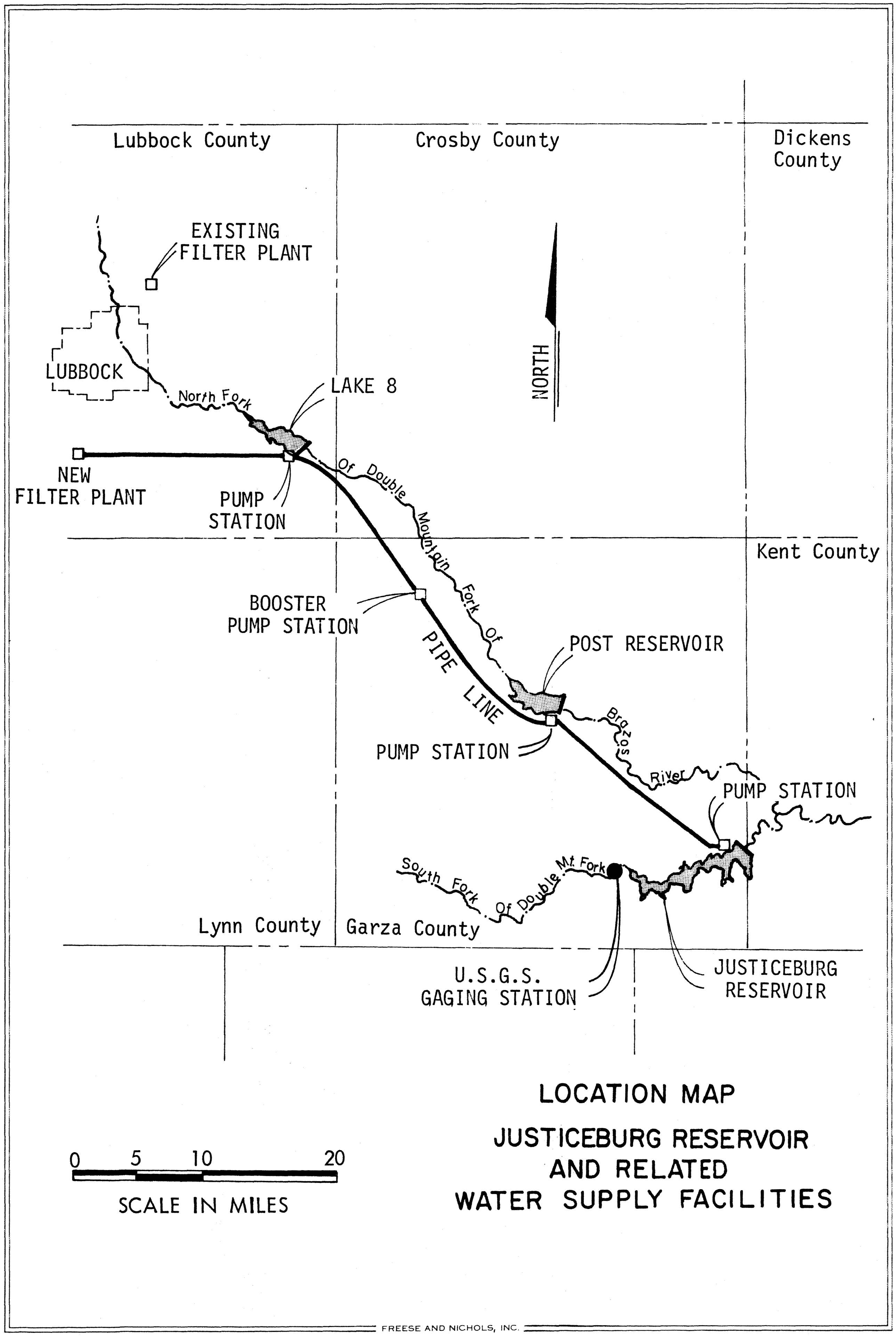
(1, 2) Numbers in parentheses match references listed in Appendix A.



Lubbock County
Crosby County
Dickens County
EXISTING FILTER PLANT
LUBBOCK
North Fork
LAKE 8
NORTH
NEW FILTER PLANT
PUMP STATION
Of Double Mountain Fork Of Brazos River
Kent County
BOOSTER PUMP STATION
PIPE LINE
POST RESERVOIR
PUMP STATION
PUMP STATION
South Fork Of Double Mt Fork
Lynn County
Garza County
U.S.G.S. GAGING STATION
JUSTICEBURG RESERVOIR
LOCATION MAP
JUSTICEBURG RESERVOIR AND RELATED WATER SUPPLY FACILITIES
0 5 10 20
SCALE IN MILES
FREESE AND NICHOLS, INC.

FIGURE 1.1

report also emphasized the need for field testing of the water quality and for preliminary geotechnical studies, to confirm the basic feasibility of the Justiceburg Site.

In August of 1975, Lubbock authorized Freese and Nichols to proceed with additional, more detailed studies relating to the Justiceburg project. At that same time, the City approved a program of field investigations on and near the Justiceburg site by Mason-Johnston and Associates, Inc., a firm of geotechnical consultants experienced in dam foundation work. The City also instructed Freese and Nichols to enter into agreement with the U. S. Geological Survey and the Texas Water Development Board for establishment and operation of a chemical quality monitoring station at the U. S. Highway 84 bridge on the South Fork of the Double Mountain Fork at Justiceburg.

The results of the geotechnical and water quality studies are described in this report, along with an evaluation of the reservoir yield in the light of the most recent hydrologic data, plus preliminary design analysis of the dam and spillway. As set forth in the scope of work for the assignment, the items covered include the following:

a. Review of the latest available hydrologic data
b. Determination of reservoir storage requirements
c. Reservoir operation studies and estimates of yield
d. Design flood analysis and evaluation of spillway requirements
e. Basic dam and spillway design
f. Water quality routings.

## 2. JUSTICEBURG RESERVOIR SITE

### Watershed Characteristics

The Justiceburg Reservoir site is located on the South Fork of the Double Mountain Fork of the Brazos River at the eastern edge of Garza County, approximately 60 miles southeast of Lubbock. Figure 2.1 is a map of the watershed and surrounding areas.

The watershed is about 35 miles long in a generally east-west direction and varies in width from around 6 miles to as much as 15 miles. The average north-south dimension is about 11 miles. It is predominantly ranch land, with little cultivated agriculture. The topography is rugged, with steep slopes and pronounced relief. Scores of earthen tanks and small ponds are located throughout the watershed, some natural and others man-made. There is a significant amount of oil field activity. Oil wells, oil storage tanks or pipelines are indicated on all of the topographic quadrangle maps for the area.

The normal annual rainfall is about 18 inches at the western end of the watershed, increasing to approximately 19 inches at the eastern end. Most of the streams are often dry, and it is not unusual for the main channel of the South Fork itself to go without flow for several months at a time. During periods of significant rainfall (during thunderstorms, for example), the runoff rates tend to be relatively high. In the 17-year span of flow records at the Justiceburg gaging station, a flood peak of nearly 50,000 cubic feet per second (cfs) has been observed on one occasion (May 1969), and floods of more than 30,000 cfs have occurred several times. Based on high water marks observed at Justiceburg in 1955, before the U.S.G.S. station was established, it is

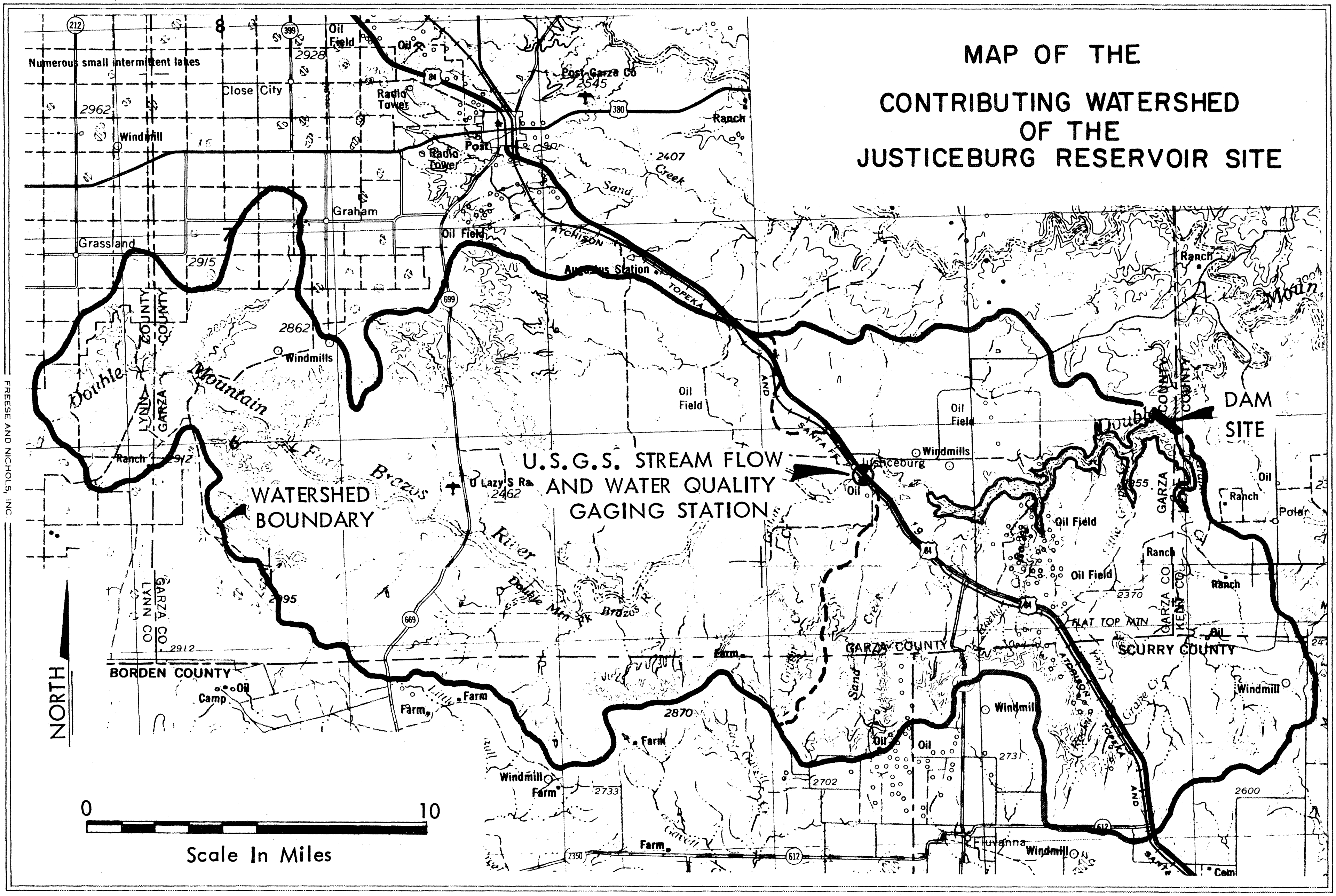



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FIGURE 2.1

known that a flood peak in September of that year reached a level six feet above the flood of May 1969, which would indicate a flow considerably in excess of 50,000 cfs.

Thus, the stream often experiences low flow or no flow at all but is also subject to unusually heavy runoff intensities when there is storm rainfall. There is normally very little runoff during the six months from November through April, and there is generally heavy runoff in one or more of the other six months of the year. May is the most probable month of high runoff amounts.

Water quality measurements at the Justiceburg gaging station indicate significant amounts of oil field brine contamination during times of low runoff. Dissolved chemical concentrations observed at the gage vary widely with the rate of flow, ranging from good quality conditions for high flows to poor quality when the flow is small. The analyses show sodium chloride to be the main dissolved chemical component. Because of its obvious importance when related to a potential source of municipal supply, the question of water quality has received close attention in the present investigation. Section 4 of this report deals with water quality in further detail.

## Geotechnical Investigation and Site Selection

A program of core borings was begun by Mason-Johnston and Associates in the summer of 1976, to evaluate the suitability of foundation conditions for construction of a dam. The borings were not restricted to a single location, but instead covered three potential sites along the channel of the stream. Two initial borings were made in July 1976 at a point designated as Site 5, close to where the South Fork flows from

Garza County into Kent County. Site 5 had been the preferred choice at the time of the 1971 study (1) and was the basis of the preliminary evaluations done at that time. In the first borings, gypsum deposits were encountered at elevations which made the site obviously undesirable, and further exploration was not thought to be justified at Site 5.

Early in August 1976, the exploratory work was moved upstream to a point identified as Site 4A, where three more borings were made. Gypsum deposits were also encountered at Site 4A, but in that instance they were at a greater depth below the stream bed, and it was concluded that they would be less of a problem than would the gypsum layers found at Site 5. On the other hand, portions of the canyon walls at Site 4A were found to consist of a pervious formation of sand, sandstone, shale and gravel, in which it would be relatively difficult to prevent loss of water due to seepage. In a report on the initial series of borings (3), Mason-Johnston and Associates described the results and projected the probable subsurface conditions at a third location, identified as Site 6, located between Site 5 and Site 4A at the point where Grape Creek flows into the South Fork. It was tentatively concluded from correlation of the data obtained at Sites 4A and 5 that Site 6 might have a suitable depth of cover over the soluble gypsum deposits beneath the stream bed and at the same time might have impervious abutments to a height sufficient to avoid serious seepage problems. In the report, Mason-Johnston stated: "Considering the anticipated problems that could evolve from the pervious caprock and the soluble gypsum deposits, it appears that selection of Site 6 is the only viable alternative. It is therefore recommended that a core boring program be undertaken to prove the subsurface conditions at Site No. 6."

The additional exploration was approved by the City of Lubbock, and in December of 1976 Mason-Johnston made four core borings at Site 6, followed by a group of seven auger borings to evaluate the quality of embankment materials available within the reservoir area. Infiltration tests were made at intervals during the core borings. Core samples were collected in the field and later tested for strength and other engineering characteristics in the Mason-Johnston laboratory. Preliminary analyses of embankment stability and potential seepage loss were made, based on the results of the field and laboratory testing.

Two of the core borings at Site 6 were in the bottom of the canyon, along the banks of the stream. One of these (Boring No. 2) may have coincided with a refilled former channel, since it reflected sand to a depth of 60 feet. No clear-cut gypsum formations were encountered in the total depth of 85 feet in Boring No. 2. The other boring in the valley bottom (Boring No. 3) encountered gypsum at a depth of 53 feet, or approximately 45 feet below the top of the primary geologic strata at that location. Borings No. 1 and No. 4 were on the abutments at either side of the canyon. In those holes, the more pervious materials similar to those encountered at Site 4A were found to extend down to elevations slightly above elevation 2220, which is the level shown herein as the recommended top of conservation storage.

The final report of Mason-Johnston and Associates on the initial geotechnical work at Site 6 (4) is reproduced in full as Appendix B to this report. The Mason-Johnston study concluded that Site 6 is acceptable for the proposed dam construction. It also emphasized the requirement for more detailed geotechnical exploration during design, to obtain better definition of matters such as the required depth of cutoff trench and the best sources of embankment materials.

On the basis of the results of the geotechnical work, the choice was narrowed to Site 6, and the other alternative sites were not given further consideration. Throughout this report, references to the Justiceburg Reservoir or the Justiceburg Dam should be understood as relating to Site 6 unless specifically indicated otherwise. Figure 2.2 is a layout map of the dam and reservoir at this site.

## Area and Capacity Characteristics

The surface area acreage and storage capacity characteristics of the Justiceburg Reservoir site are summarized in Table 2.1 and shown graphically in Figure 2.3. These values were derived by planimeter measurements from U.S.G.S. topographic quadrangle maps entitled Justiceburg and Justiceburg SE, which have a scale of one inch to 2,000 feet and a contour interval of 20 feet.

Due to the steep canyon walls, the water surface area will be unusually small in relationship to the storage volume. This condition makes the reservoir site hydrologically efficient, in the sense that it results in a low rate of surface evaporation loss per acre-foot of storage.

Because of the permeable materials in the upper levels of the abutments, with the associated problems of seepage control, it was concluded that the highest feasible level of normal conservation storage is about elevation 2220. At that elevation, the storage volume of the lake

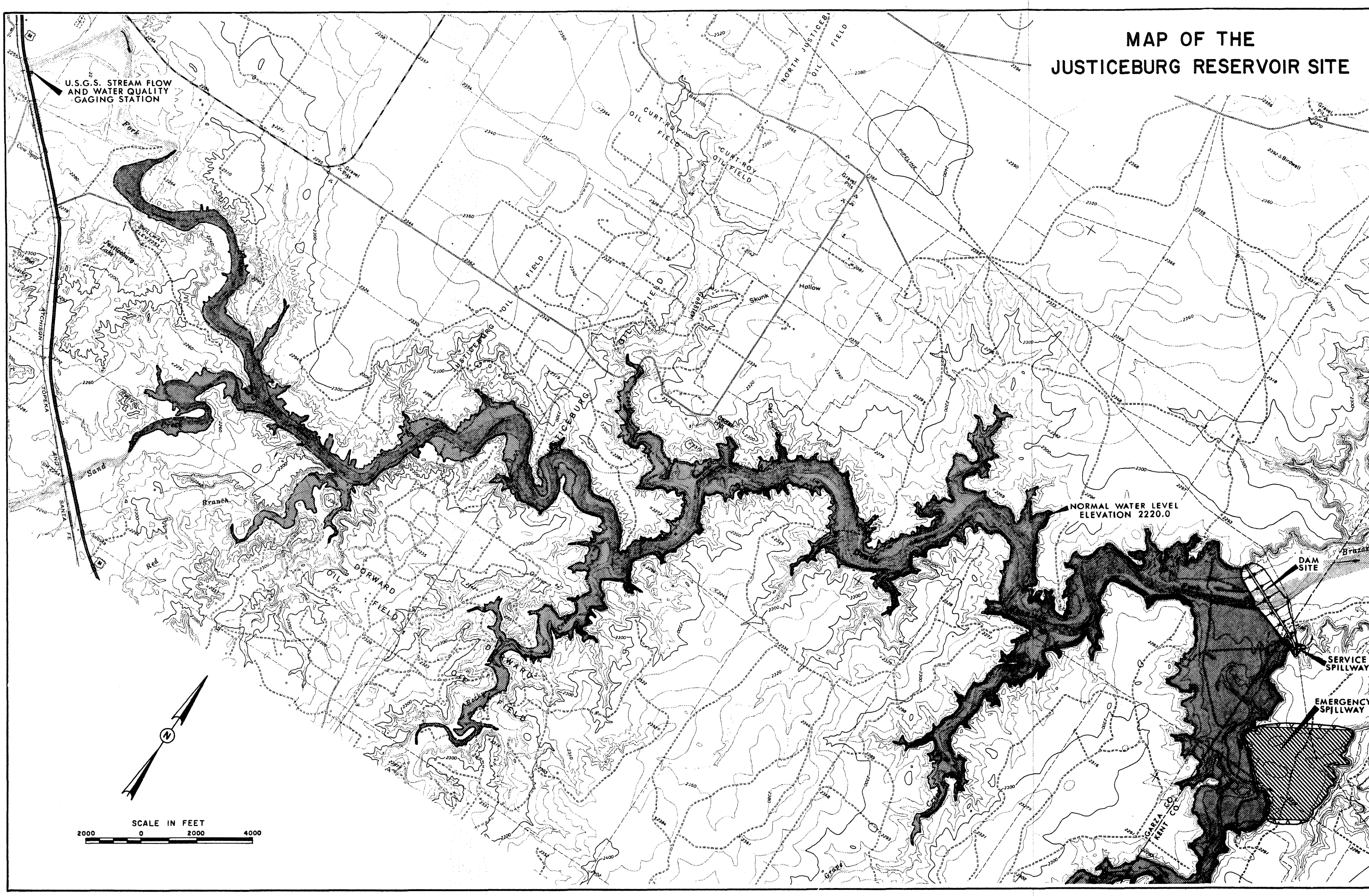
MAP OF THE
JUSTICEBURG RESERVOIR SITE
U.S.G.S. STREAM FLOW
AND WATER QUALITY
GAGING STATION
NORMAL WATER LEVEL
ELEVATION 2220.0
DAM
SITE
SERVICE
SPILLWAY
EMERGENCY
SPILLWAY
SCALE IN FEET
2000 0 2000 4000
CURT-ROY
OIL
FIELD
DORWARD
OIL
FIELD
Skunk
Hollow
Sand
Red
Branch
Fork
GARZA CO.
KENT CO.

FIGURE 2.2

## Table 2.1

## Justiceburg Reservoir
## Area and Capacity Characteristics

| Elev. | 0 | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 2110 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 16 | 19 | Acres |
| | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 8 | 26 | Ac-Ft |
| 2120 | 22 | 27 | 32 | 39 | 46 | 50 | 54 | 69 | 84 | 96 | Acres |
| | 46 | 71 | 100 | 136 | 178 | 226 | 278 | 340 | 416 | 506 | Ac-Ft |
| 2130 | 108 | 119 | 129 | 145 | 161 | 176 | 191 | 202 | 213 | 233 | Acres |
| | 608 | 721 | 845 | 982 | 1,135 | 1,304 | 1,487 | 1,684 | 1,891 | 2,164 | Ac-Ft |
| 2140 | 253 | 279 | 305 | 329 | 353 | 377 | 400 | 426 | 452 | 479 | Acres |
| | 2,407 | 2,673 | 2,965 | 3,282 | 3,623 | 3,988 | 4,377 | 4,790 | 5,229 | 5,694 | Ac-Ft |
| 2150 | 506 | 530 | 554 | 578 | 602 | 630 | 659 | 687 | 714 | 739 | Acres |
| | 6,187 | 6,705 | 7,247 | 7,813 | 8,403 | 9,019 | 9,663 | 10,336 | 11,037 | 11,763 | Ac-Ft |
| 2160 | 765 | 789 | 813 | 842 | 870 | 901 | 932 | 964 | 996 | 1,021 | Acres |
| | 12,515 | 13,292 | 14,093 | 14,921 | 15,777 | 16,662 | 15,579 | 18,527 | 19,507 | 20,515 | Ac-Ft |
| 2170 | 1,046 | 1,074 | 1,102 | 1,131 | 1,160 | 1,186 | 1,212 | 1,242 | 1,272 | 1,301 | Acres |
| | 21,549 | 22,609 | 23,697 | 24,813 | 25,959 | 27,132 | 28,331 | 29,558 | 30,815 | 32,101 | Ac-Ft |
| 2180 | 1,330 | 1,367 | 1,404 | 1,437 | 1,471 | 1,506 | 1,541 | 1,577 | 1,612 | 1,647 | Acres |
| | 33,417 | 34,765 | 36,151 | 37,571 | 39,025 | 40,514 | 42,037 | 43,596 | 45,191 | 46,820 | Ac-Ft |
| 2190 | 1,682 | 1,716 | 1,751 | 1,786 | 1,820 | 1,855 | 1,891 | 1,928 | 1,965 | 2,005 | Acres |
| | 48,485 | 50,184 | 51,917 | 53,686 | 55,489 | 57,326 | 59,199 | 61,109 | 63,055 | 65,040 | Ac-Ft |

Table 2.1 (Continued)

| Elev. | 0 | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 2200 | 2,045 | 2,080 | 2,114 | 2,151 | 2,188 | 2,232 | 2,276 | 2,315 | 2,355 | 2,396 | Acres |
| | 67,065 | 69,128 | 71,225 | 73,357 | 75,527 | 77,737 | 79,991 | 82,286 | 84,621 | 86,997 | Ac-Ft |
| 2210 | 2,437 | 2,473 | 2,509 | 2,559 | 2,608 | 2,654 | 2,700 | 2,742 | 2,784 | 2,834 | Acres |
| | 89,414 | 91,869 | 94,360 | 96,894 | 99,477 | 102,108 | 104,785 | 107,506 | 110,269 | 113,078 | Ac-Ft |
| 2220 | 2,884 | 2,950 | 3,017 | 3,075 | 3,133 | 3,197 | 3,261 | 3,340 | 3,418 | 3,504 | Acres |
| | 115,937 | 118,854 | 121,838 | 124,884 | 127,988 | 131,153 | 134,382 | 137,683 | 141,062 | 144,523 | Ac-Ft |
| 2230 | 3,589 | 3,676 | 3,763 | 3,869 | 3,975 | 4,094 | 4,214 | 4,338 | 4,461 | 4,622 | Acres |
| | 148,069 | 151,702 | 155,421 | 159,237 | 163,159 | 167,194 | 171,348 | 175,624 | 180,024 | 184,565 | Ac-Ft |
| 2240 | 4,784 | | | | | | | | | | Acres |
| | 189,268 | | | | | | | | | | Ac-Ft |

73357

72600

7560 Ac/ft at this storage

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# JUSTICEBURG RESERVOIR SITE AREA AND CAPACITY CHARACTERISTICS

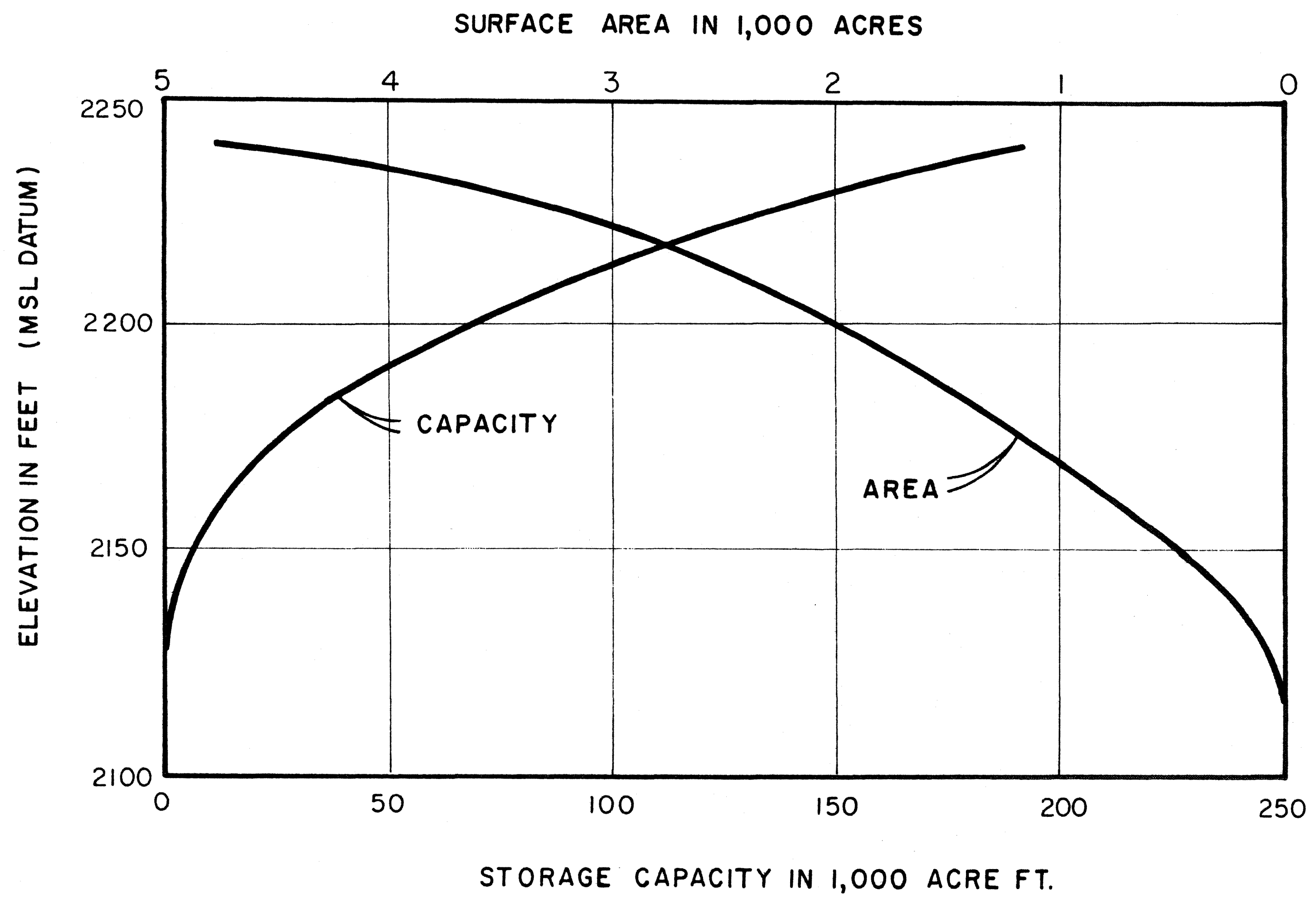


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FIGURE 2.3

would be 115,937 acre-feet, and the surface area would be 2,884 acres.

## Contributing Drainage Area

The contributing drainage area at the site is 394 square miles, based on measurements from topographic quadrangle maps of the U. S. Geological Survey and the published value for the contributing area above the U.S.G.S. gaging station at Justiceburg (5). In addition to the contributing watershed, there is a non-contributing area in excess of 1,200 square miles which is technically part of the South Fork drainage but which lies above the Caprock, on the High Plains. Except on some lands immediately adjoining the rim of the Caprock, nearly all surface runoff originating on the High Plains is caught and held by local playa lakes, and little or no flow from this region can be expected to reach the Justiceburg Reservoir.

The contributing area was delineated on the U.S.G.S. topographic maps, and the size of the area was determined by planimeter measurements. The value (244 square miles) which the Geological Survey gives for the net area contributing runoff to the Justiceburg gaging station (5) was found to be in close agreement with the independent measurements and was therefore adopted for use in the analysis. The additional contributing area between the Justiceburg gage and the dam site was measured as 150 square miles. This amount, added to the 244 square miles above the gaging station, resulted in the total of 394 square miles for the Justiceburg Reservoir. For purposes of evaluating the water supply yield of the project, only this contributing area was counted, and it was assumed that no runoff would be derived from the larger, non-contributing area above the Caprock.

## Runoff

Estimates of historical runoff experienced at the proposed dam site during the years 1940 through 1978 were derived from the recorded stream flows on the Double Mountain Fork as collected and published by the U. S. Geological Survey (5). The methodology and criteria of the runoff estimates are explained more fully in Appendix C. Since November of 1961, the U.S.G.S has made continuous measurements of flows on the South Fork at Justiceburg, and those records constitute the primary data for evaluating the historical runoff at the reservoir site. For a number of years before installation of the Justiceburg gage, records were kept at another station, on the Double Mountain Fork near Aspermont, which is also still in service at the present time. Through correlation of the records at the Justiceburg and Aspermont stations during years when both have been in operation concurrently, it was possible to develop relationships for estimating the Justiceburg Reservoir inflows during the earlier years (1940-1961) based on the Aspermont gage flows.

Runoff data are also available for the Colorado River watershed above Lake J. B. Thomas, which adjoins the Justiceburg watershed to the south. Comparisons of flows estimated for the two drainage areas showed close correspondence of the relative amounts of runoff during the period since the Justiceburg gage went into service and during the drouth period of the 1950's, which is the critical period of record for many areas of the State. This agreement with flows derived independently from a different set of gaging station records on the neighboring watershed served as a good check of the over-all validity of the Justiceburg estimates for the period before the U.S.G.S. started stream flow measurements at the Justiceburg gage.

The average amount of runoff at the Justiceburg Reservoir site during the 39-year period from 1940 through 1978 was 47,012 acre-feet per year. The minimum estimated flow in any calendar year was 7,620 acre-feet, in 1956. The maximum was 213,410 acre-feet, in 1940. It was found that the critical drouth conditions for the site were not in the earlier drouth period but in recent years, from October 1972 through April 1978. The average rate of runoff in that interval of time was 13,536 acre-feet per year, or about 29% of the long-term average rate of flow.

The fact that the critical period of record falls in the years 1972 through 1978 is significant for several reasons. First, it means that the definitive drouth conditions for purposes of predicting the dependable yield occurred after the Justiceburg gaging station was in service, and thus the yield estimates are based on the most reliable part of the body of runoff data. Also, it points up the importance of continued collection of data at the Justiceburg gage. Probably as late as one year ago, it would not have been apparent from available information that conditions other than those of the 1940's and 1950's would be critical. And, finally, the very recent critical drouth will include and reflect any basic changes in the runoff characteristics of the watershed that may have taken place over the years.

## Future Runoff Depletions

It is generally recognized that ongoing soil and water conservation programs on farms and ranches in some areas of Texas can be expected to have corollary effects on the normal runoff characteristics of the land.

On many West Texas streams, including the South Fork of the Double Mountain Fork of the Brazos, the predicted impact of modern agricultural practices is to produce a modest but noticeable decrease in the volume of runoff resulting from a given amount of rainfall. The most authoritative investigation of this relationship was made by the U. S. Bureau of Reclamation as part of the work of the U. S. Study Commission for Texas in the 1960's (6). In that analysis, it was indicated that the Double Mountain Fork watershed would experience runoff depletions averaging approximately 10.3 acre-feet per year per square mile as of the year 1975 (i.e., the annual runoff per square mile as of 1975 would average that much less than it would have under totally natural conditions, uninfluenced by the works of man), and the process of change was predicted to continue through the year 2010. The incremental depletions between 1975 and 2010 were estimated at 4.0 acre-feet per year per square mile, leading to aggregate depletions of 14.3 acre-feet per square mile per year as of 2010 when contrasted with completely natural conditions.

As mentioned previously, the critical drouth period of record was found to have occurred from 1972 through 1978, and the year 1975 was approximately the midpoint of the critical period. Thus, from the standpoint of dependable reservoir yield, any runoff depletions experienced prior to 1975 were reflected in the historical runoff as actually observed at the Justiceburg gaging station. Depletions subsequent to 1975 should be allowed for in predicting future yields through the year 2010 or later. Based on Reference (6), the potential runoff depletion effect after 1975 for a drainage area of 394 square miles in this area would be approximately 1,600 acre-feet per year.

## Evaporation

Monthly depths of net evaporation loss from the reservoir surface were derived from Texas Water Development Board Report 64, which is a compilation of historical net evaporation values throughout the State (7). Although the original Water Development Board study covered only the period from 1940 through 1965, data for the next ten years (1966-1975) have subsequently been prepared as supplemental material, available from the Board in the form of computer printouts. For the most recent years, beginning with 1976, the evaporation estimates for this study were based on published records of the Texas Agricultural Experiment Station System (8). Details of the net evaporation estimates are given in Appendix C, along with a tabulation of the resulting monthly quantities. The average annual depth of net evaporation loss from a reservoir at the Justiceburg site would be approximately 4.57 feet, and yearly extremes would range from as little as 1.71 feet to as much as 7.44 feet, based on the historical conditions for the 39 years from 1940 through 1978.

## Sedimentation

Most of the contributing watershed of the Justiceburg Reservoir lies in the region known as the rolling plains. The silt content of runoff from this land resource area tends to be relatively high, and significant amounts of sediment would be deposited in the reservoir due to impoundment of the runoff. Bulletin 5912 of the Texas Board of Water Engineers (9) is a study of rates of siltation in Texas, prepared by the U. S. Soil Conservation Service, giving probable sedimentation rates in acre-feet per year per square mile of drainage area for watersheds of various sizes throughout the State. For an area of 394 square miles on the South Fork of the Double Mountain Fork, the predicted long-range average rate of siltation is 494 acre-feet per year. Over a period of 50 years, that rate of sediment accumulation would diminish the reservoir storage capacity by 24,700 acre-feet.

## Conflicts

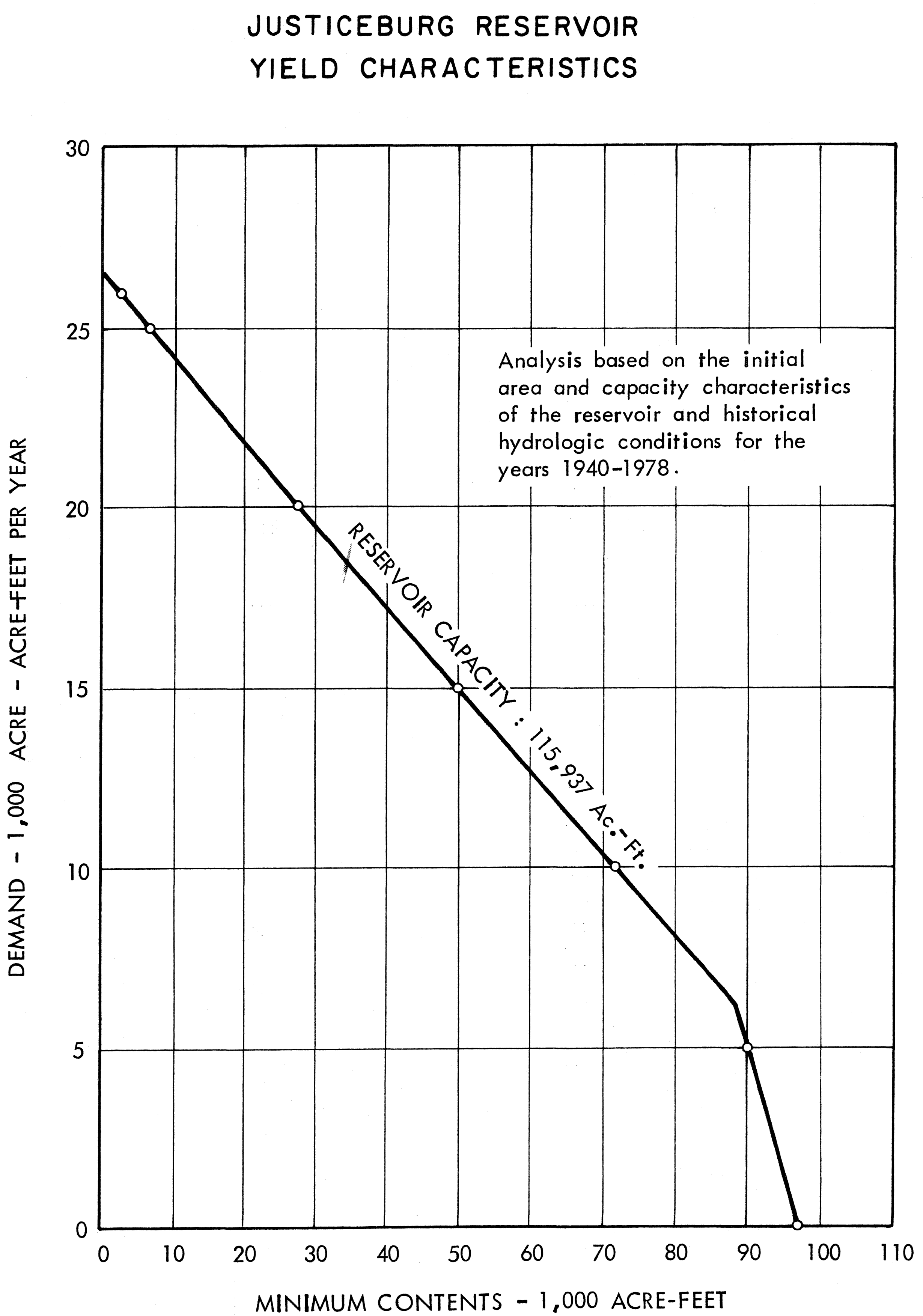
There are few existing man-made improvements in the proposed reservoir area. Several oil wells would be inundated by the lake and would need to be raised above the water level. There are apparently no houses or other structures which would be affected. A few unimproved dirt roads pass through the canyon and cross the streams within the reservoir. There is one 8-inch oil pipeline which crosses the South Fork immediately downstream from the dam and cuts through the south abutment in the area which would be used for the spillways. The pipeline will need to be lowered and protected. Other than the oil wells and the pipeline, there are no known conflicts which would require adjustment or involve added cost.

## 3. RESERVOIR YIELD

The runoff and evaporation data described in the preceding section were used as input for a series of computer simulations of reservoir performance with varying rates of water supply withdrawal. Basically, the computer runs fell into two categories: (a) studies in which the demand rate remained the same regardless of reservoir content and (b) studies which assumed a variable demand, depending on the amount of water stored in the lake from month to month. Most of the analyses were of the first type, in which the annual rate of withdrawal remained constant throughout the study period for each individual computer run. The results of these simulations are summarized in Appendix D. They are also presented graphically in Figure 3.1, which is a plot showing the minimum reservoir storage content amounts that would have been experienced historically for a range of demand rates at the Justiceburg site with a conservation capacity of 115,937 acre-feet.

From Figure 3.1 it is possible to determine how the proposed reservoir at the Justiceburg site would have performed if it had been in operation from 1940 through 1978 and if the water supply demand had been any given amount from zero up to the rate of withdrawal which would have emptied the reservoir at the low point of the critical drouth of record. Similar analyses were also made for other capacities over a span from 50,000 acre-feet to 130,000 acre-feet. The results of these studies for other reservoir sizes are summarized in Table D-1 of Appendix D.

Depending on a number of factors, it is often prudent to assume that a water supply reservoir would not be emptied completely, even



JUSTICEBURG RESERVOIR
YIELD CHARACTERISTICS
Analysis based on the initial area and capacity characteristics of the reservoir and historical hydrologic conditions for the years 1940-1978.
RESERVOIR CAPACITY : 115,937 Ac.-Ft.
DEMAND - 1,000 ACRE - ACRE-FEET PER YEAR
MINIMUM CONTENTS - 1,000 ACRE-FEET
0
5
10
15
20
25
30
0
10
20
30
40
50
60
70
80
90
100
110




FIGURE 3.1

during a critical drouth, but that there would always be some water left in storage. This approach makes allowance for the fact that it is generally difficult to remove the last few acre-feet from the bottom of the lake and also recognizes the possible deterioration of water quality that may occur when the content falls close to zero. In cases where complete dependability of the supply is an important factor, a moderate volume of storage might also be assumed to remain in storage at the point of maximum drawdown as a factor of safety, to cover the possible occurrence of future drouth conditions more severe than any reflected in the available records. For the Justiceburg project, such safety factor is not necessary, however, because Lubbock has the backup of a major ground water source. The computer model studies show how the lake would have behaved during the worst drouth of the last 40 years. In the event of still worse drouth conditions at some future date, Lubbock would be able to increase ground water pumpage from the Sand Hills well field on a temporary basis in order to relieve part of the load on the surface water system.

Table 3.1 reflects the estimated yields of the Justiceburg Reservoir with various storage capacities, before siltation. Yields are shown based on complete use of the storage and also for drawdown to a minimum content of 2,000 acre-feet. The complete drawdown criterion is generally the basis on which yields are evaluated by the Texas Department of Water Resources for purposes of determining water rights. The assumption that approximately 2,000 acre-feet would be left unused in the bottom of the lake corresponds more closely to what might be expected in actual operation of the project. At that point, based on the initial area and

Table 3.1

Estimated Yields of the Justiceburg Reservoir
For Various Storage Capacity Volumes

| Reservoir Capacity (Ac-Ft) | Yield With Complete Drawdown (Ac-Ft/Yr) | Yield With Minimum Content of 2,000 Ac-Ft (Ac-Ft/Yr) |
|---|---|---|
| 50,000 | 17,800 | 17,300 |
| 60,000 | 19,100 | 18,600 |
| 70,000 | 20,500 | 20,000 |
| 90,000 | 23,000 | 22,500 |
| 110,000 | 25,800 | 25,300 |
| 130,000 | 27,900 | 27,500 |

capacity characteristics before sedimentation, the remaining water would be approximately 22 feet deep at the dam.

Throughout most of the range of capacities in Table 3.1, the yield of the project is shown to increase steadily with increasing storage volume. For each additional 1,000 acre-feet of storage, there is a gain in yield of approximately 135 acre-feet per year, up to a capacity of about 120,000 acre-feet. Beyond that point, there is noticeably less benefit from additional capacity. From the standpoint of project yield, there would be justification for a capacity of 120,000 acre-feet plus allowance of 24,700 acre-feet for 50 years of sedimentation, or a total of 144,700 acre-feet.

Due to the geologic conditions, as mentioned previously, it may not be feasible to impound water above about elevation 2220, at which level the initial capacity before sedimentation would be 115,937 acre-feet. Judging from the initial core borings at the site (4), it apparently would be difficult to avoid undue seepage losses through the relatively porous formations in the abutments at higher elevations. Based on the preliminary geotechnical and hydraulic results, it was concluded that elevation 2220 is the highest feasible conservation level at the Justiceburg site and that 115,937 acre-feet is the optimum capacity that can be developed when all factors are considered.

Table 3.2 is a summary of an operation study for the Justiceburg Reservoir with a capacity of 115,937 acre-feet and a demand of 26,100 acre-feet per year, based on the 39 years of historical hydrologic conditions from 1940 through 1978. In order to make the results more readily understandable, the summary is presented in terms of annual totals, although the computer studies were carried out in one-month time increments. The full printout of this particular run is also reproduced in Appendix D, as an example of the details of the computer analyses. The critical drouth period was found to extend from October 1972 through April of 1978. The minimum reservoir content, at the end of April 1978, was 2,162 acre-feet, which corresponds to about elevation 2139.

In actual operation of the Justiceburg project, Lubbock would be able to take water from the lake on an overdraft basis much of the time, using the surface water supply in excess of the firm yield rate. That mode of operation would make maximum use of the renewable surface water resource while conserving as much as possible of the non-renewable ground water supply from the Sand Hills well field. During critically dry years, when the surface reservoirs are at lower levels, the ground water use can be increased temporarily to ease the demand on the surface system. Since the drouth conditions are experienced only occasionally, the overdraft

Table 3.2

Summary of Justiceburg Reservoir Operation Study
With Constant Annual Withdrawal Rate
- Quantities in Acre-Feet -

| | Evapora-Loss | Demand | Inflow | Spills | End-of-Year Content |
|---|---|---|---|---|---|
| Start | | | | | 115,937 |
| 1940 | 13,768 | 26,100 | 34,200 | 0 | 110,269 |
| 1941 | 4,887 | 26,100 | 213,410 | 178,845 | 113,847 |
| 1942 | 9,460 | 26,100 | 41,360 | 6,485 | 113,162 |
| 1943 | 13,997 | 26,100 | 12,570 | 0 | 85,635 |
| 1944 | 8,349 | 26,100 | 13,400 | 0 | 64,586 |
| 1945 | 8,627 | 26,100 | 30,170 | 0 | 60,029 |
| 1946 | 8,432 | 26,100 | 29,670 | 0 | 55,167 |
| 1947 | 11,549 | 26,100 | 55,470 | 0 | 72,988 |
| 1948 | 12,214 | 26,100 | 40,310 | 0 | 74,984 |
| 1949 | 7,612 | 26,100 | 35,260 | 0 | 76,532 |
| 1950 | 10,365 | 26,100 | 52,310 | 0 | 92,377 |
| 1951 | 12,618 | 26,100 | 18,660 | 0 | 72,319 |
| 1952 | 12,190 | 26,100 | 8,030 | 0 | 42,059 |
| 1953 | 8,469 | 26,100 | 37,690 | 0 | 45,180 |
| 1954 | 13,015 | 26,100 | 50,670 | 0 | 56,735 |
| 1955 | 13,543 | 26,100 | 197,380 | 104,219 | 110,253 |
| 1956 | 18,434 | 26,100 | 7,620 | 0 | 73,339 |
| 1957 | 10,739 | 26,100 | 120,470 | 44,035 | 112,935 |
| 1958 | 10,393 | 26,100 | 27,760 | 3,452 | 100,750 |
| 1959 | 11,328 | 26,100 | 77,130 | 27,365 | 113,087 |
| 1960 | 9,694 | 26,100 | 99,390 | 63,375 | 113,308 |
| 1961 | 10,073 | 26,100 | 65,690 | 38,590 | 104,235 |
| 1962 | 11,131 | 26,100 | 52,610 | 11,422 | 108,192 |
| 1963 | 11,715 | 26,100 | 57,660 | 28,298 | 99,739 |
| 1964 | 12,607 | 26,100 | 10,180 | 0 | 71,212 |
| 1965 | 10,715 | 26,100 | 42,680 | 0 | 77,077 |
| 1966 | 7,506 | 26,100 | 19,900 | 0 | 63,371 |
| 1967 | 9,823 | 26,100 | 83,180 | 6,065 | 104,563 |
| 1968 | 7,478 | 26,100 | 14,550 | 0 | 85,535 |
| 1969 | 7,936 | 26,100 | 74,660 | 13,057 | 113,102 |
| 1970 | 13,176 | 26,100 | 16,720 | 0 | 90,546 |
| 1971 | 10,738 | 26,100 | 37,350 | 0 | 91,058 |
| 1972 | 9,152 | 26,100 | 55,360 | 2,503 | 108,663 |
| 1973 | 12,230 | 26,100 | 10,000 | 0 | 80,333 |
| 1974 | 9,987 | 26,100 | 9,630 | 0 | 53,876 |
| 1975 | 7,246 | 26,100 | 22,570 | 0 | 43,100 |
| 1976 | 5,765 | 26,100 | 15,650 | 0 | 26,885 |
| 1977 | 5,147 | 26,100 | 15,740 | 0 | 11,378 |
| 1978 | 3,092 | 26,100 | 26,390 | 0 | 8,576 |
| Avg. | 10,133 | 26,100 | 47,012 | 13,531 | |

Note: Minimum content = 2,162 acre-feet, at end of April 1978.

operation would lead to an appreciable over-all gain in the amount of surface water supplied and a corresponding saving in total ground water pumpage. The fact that the system is based on a combination of significant amounts of both surface water and ground water makes it feasible to operate in that manner, taking full advantage of the surface supply and extending the useful life of the well field. Lubbock has followed a comparable procedure in recent years with respect to the Canadian River supply, utilizing as much as possible of the Lake Meredith water so as to lighten the load on the Sand Hills, and the same basic approach would be indicated for the Justiceburg Reservoir.

Table 3.3 shows a summary of one possible mode of overdraft operation, in which the rate of withdrawal was raised to 35,000 acre-feet per year when the lake contained more than 60,000 acre-feet of storage. Between 60,000 and 30,000 acre-feet of lake content, the demand was at the rate of 25,000 acre-feet per year. And, below 30,000 acre-feet of storage content, the demand was decreased to 20,000 acre-feet per year. It can be seen that during 30 of the 39 years of the study period it would have been possible to take water from the reservoir at an average rate greater than the 26,100 acre-feet per year dependable yield. In four other years (1945, 1946, 1956 and 1974), the supply would have been very little less than 26,100 acre-feet. Only in five years (1953, 1975, 1976, 1977 and 1978) out of the 39-year period would the available supply have fallen below 90% of the firm yield amount. The average supply made available for the 39 years under this method of operation would be 30,209 acre-feet per year, about 16% more than the firm yield rate of 26,100 acre-feet per year. Table 3.4 is a comparison of the

Table 3.3

Summary of Justiceburg Reservoir Operation Study With Variable Demand Based on Reservoir Content
- Quantities in Acre-Feet -

| | Evapora-Loss | Demand | Inflow | Spills | End-of-Year Content |
|---|---|---|---|---|---|
| Start | | | | | 115,937 |
| 1940 | 13,393 | 35,000 | 34,200 | 0 | 101,744 |
| 1941 | 4,831 | 35,000 | 213,410 | 162,991 | 112,332 |
| 1942 | 9,182 | 35,000 | 41,360 | 0 | 109,510 |
| 1943 | 13,284 | 35,000 | 12,570 | 0 | 73,796 |
| 1944 | 7,267 | 30,810 | 13,400 | 0 | 49,119 |
| 1945 | 7,232 | 25,000 | 30,170 | 0 | 47,057 |
| 1946 | 7,089 | 25,000 | 29,670 | 0 | 44,638 |
| 1947 | 10,471 | 30,020 | 55,470 | 0 | 59,617 |
| 1948 | 10,885 | 29,200 | 40,310 | 0 | 59,842 |
| 1949 | 6,650 | 30,050 | 35,260 | 0 | 58,402 |
| 1950 | 8,892 | 30,870 | 52,310 | 0 | 70,950 |
| 1951 | 10,309 | 29,960 | 18,660 | 0 | 49,341 |
| 1952 | 9,207 | 24,170 | 8,030 | 0 | 23,994 |
| 1953 | 6,289 | 20,840 | 37,690 | 0 | 34,555 |
| 1954 | 11,696 | 26,690 | 50,670 | 0 | 46,839 |
| 1955 | 13,032 | 30,870 | 197,380 | 91,572 | 108,745 |
| 1956 | 17,670 | 35,000 | 7,620 | 0 | 63,695 |
| 1957 | 10,514 | 34,180 | 120,470 | 28,131 | 111,340 |
| 1958 | 10,164 | 35,000 | 27,760 | 0 | 93,936 |
| 1959 | 11,044 | 35,000 | 77,130 | 15,637 | 109,385 |
| 1960 | 9,495 | 35,000 | 99,390 | 52,489 | 111,791 |
| 1961 | 9,935 | 35,000 | 65,690 | 32,025 | 100,521 |
| 1962 | 10,692 | 35,000 | 52,610 | 1,505 | 105,934 |
| 1963 | 11,537 | 35,000 | 57,660 | 21,739 | 95,318 |
| 1964 | 11,797 | 35,000 | 10,180 | 0 | 58,701 |
| 1965 | 9,570 | 30,870 | 42,680 | 0 | 60,941 |
| 1966 | 6,381 | 27,530 | 19,900 | 0 | 46,930 |
| 1967 | 8,809 | 30,050 | 83,180 | 0 | 91,251 |
| 1968 | 6,546 | 35,000 | 14,550 | 0 | 64,255 |
| 1969 | 6,836 | 32,480 | 74,660 | 0 | 99,599 |
| 1970 | 11,699 | 35,000 | 16,720 | 0 | 69,620 |
| 1971 | 8,882 | 29,990 | 37,350 | 0 | 68,098 |
| 1972 | 7,575 | 30,810 | 55,360 | 0 | 85,073 |
| 1973 | 9,872 | 33,330 | 10,000 | 0 | 51,871 |
| 1974 | 7,223 | 25,000 | 9,630 | 0 | 29,278 |
| 1975 | 5,067 | 20,420 | 22,570 | 0 | 26,361 |
| 1976 | 4,381 | 20,000 | 15,650 | 0 | 17,630 |
| 1977 | 4,324 | 20,000 | 15,740 | 0 | 9,046 |
| 1978 | 3,210 | 20,000 | 26,390 | 0 | 12,226 |
| Avg. | 9,049 | 30,209 | 47,012 | 10,413 | |

Note: Minimum content = 1,927 acre-feet, at end of April 1978.

Table 3.4

Comparison of Firm Yield Operation and Overdraft Operation

- Values in Acre-Feet per Year -

| | | Firm Yield Operation | Overdraft Operation |
|---|---|---|---|
| Annual withdrawals: | Maximum | 23.30 MGD 26,100 | 31,25 MGD 35,000 |
| | Minimum | 26,100 | 20,000 |
| | Average | 26,100 | 30,209 |
| Average evaporative loss: | | 10,133 | 9,049 |
| Average annual spills: | | 13,531 | 10,413 |

firm yield and overdraft runs, showing the gain in usable supply and its relationship to the corresponding decreases in evaporative loss and spills due to use of the extra water made available in years of normal or above-normal runoff.

Obviously, there are many different combinations of operating rules which might reasonably be adopted for operation in the variable-demand mode. The rules governing the relationship of demand to reservoir content which were followed in the study shown by Table 3.3 are not unique, and a number of other sets of similar guidelines might well be equally suitable or perhaps even more effective. The method of operation shown here is generally representative of the potential benefits available from a realistic amount of overdraft demand, and it should be viewed as a typical example rather than the only possible option.

Table 3.5 shows the estimated firm yields and the potential supply available from operation with variable rates of demand (a) when the reservoir is first filled and (b) after 50 years of siltation. Allowance is also made in the 50-year values for the predicted impact of future soil

## Table 3.5

### Estimated Amounts of Water Supply Yield Available From the Justiceburg Reservoir

| | Acre-Feet per Year | Equivalent MGD |
|---|---|---|
| Initial firm yield | 26,100 | 23.3 |
| Initial average yield available from variable-demand operation | 30,200 | 26.9 |
| Firm yield after 50 years of siltation and after estimated future runoff depletions | 20,600 | 18.4 |
| Average yield available from variable-demand operation after 50 years of siltation and after estimated future runoff depletions | 27,000 | 24.1 |

Note: All quantities are reounded to the nearest 100 acre-feet per year and the nearest 0.1 MGD.

and water conservation activities on the watershed, which are expected to deplete the average annual runoff by about 1,600 acre-feet per year.

One point of uncertainty should be noted with respect to the yield values shown in the tables of this section. It is possible that the critical drouth could extend into 1979 or later, depending on what rainfall and runoff events occur in the next few months. Based on data available to date, the critical period ended in April of 1978, and there was enough runoff in May and subsequent months to refill the lake to a moderate extent. If there is a normal amount of runoff in 1979, the critical drouth period would not be extended. On the other hand, failure to experience the usual volumes of runoff next spring could cause a lengthening of the critical period and thus a decrease in the firm yield values. The average amounts of supply estimated to be available from operation in the variable-demand mode would also be affected if 1979 should prove to be a year of low runoff. However, the variable-demand yields are, in effect, the average results over the full period of hydrologic records, and they would not be changed as noticeably as would the firm yield estimates due to the occurrence of another dry year in 1979.

# Section 10 – Lubbock's Next Water Supply – Year 2012
# - Lake Alan Henry

## c. State Water Permit

Tom, 12/28/05
FYI, CL

# TEXAS COMMISSION ON ENVIRONMENTAL QUALITY

*Protecting Texas by Reducing and Preventing Pollution*

December 19, 2005

RECEIVED
DEC 28 2005
BY:

Mr. Ches Carthel, Chief Engineer
City of Lubbock
P. O. Box 2000
Lubbock, Texas 79457

Re: Permit No. 4146A (Application No. 4155A)
Impoundment of 115,937 acre-feet of water in Lake Alan Henry on the South Fork of the Double Mountain Fork of the Brazos River for (1) non-consumptive recreational purposes in Garza and Kent Counties and (2) diversion from said lake and use of 35,000 acre-feet of water per annum for municipal purposes and secondary use of 21,000 acre-feet of said 35,000 acre-feet of water per annum for irrigation of 10,000 acres in Lubbock and Lynn Counties, Brazos River Basin, Texas

Dear Mr. Carthel:

Based on an Agreement To Transfer Lake Alan Henry dated July 14, 2005; and Deed And Assignment Without Warranty And Bill Of Sale dated August 16, 2005 submitted by Messrs. Timothy W. Jahn and Mike McClendon of Brazos River Authority along with the Change of Ownership form and the related $100.00 fee, we are changing our records to reflect the City of Lubbock, a Texas home rule municipal corporation, as the owner of the referenced permit.

If we can be of any assistance in the future, please do not hesitate to contact us.

Very truly yours,

Mohan Reddy

Mohan A. Reddy
Water Rights Permitting & Availability Section--**MC 160**--(Please use this code as part of my address)
Water Supply Division
512/239-4611

cc: Mr. Mike McClendon
Government & Customer Relations Manager
4600 Cobbs Drive
P. O. Box 7555
Waco, Texas 76714-7555

PERMIT TO
APPROPRIATE STATE WATER

APPLICATION NO. 4155 PERMIT NO. 4146 TYPE: Section 11.121

| | | | |
|---|---|---|---|
| Permittee : | City of Lubbock | Address : | P. O. Box 2000<br>Lubbock, Texas 79457 |
| Received : | August 21, 1981 | Filed : | October 5, 1981 |
| Granted : | August 6, 1984 | Counties : | Lubbock, Lynn, Garza and Kent |
| Watercourse: | South Fork of the Double Mountain Fork of the Brazos River, tributary of the Brazos River | Watershed: | Brazos River Basin |

WHEREAS, the Texas Water Commission finds that jurisdiction of the application is established; and

WHEREAS, a public hearing has been held and specific findings of fact and conclusions of law were adopted in the form of a Commission order, as required by law;

NOW, THEREFORE, this permit to appropriate and use State water is issued to City of Lubbock, subject to the following terms and conditions:

1. IMPOUNDMENT

Permittee is authorized to construct, and before acquiring any rights hereunder shall construct, a dam and reservoir on the South Fork of the Double Mountain Fork of the Brazos River and impound therein not to exceed 115,937 acre-feet of water. The dam and spillways will be located in the Houston and Great Northern Railroad Company Survey No. 55, Abstract No. 120, Kent County, and Abstract No. 810, Garza County; Houston and Great Northern Railroad Company Survey No. 57, Abstract No. 121, Kent County, and Abstract No. 811, Garza County; H. T. Cornelius Survey No. 56, Abstract No. 466, Kent County, and Abstract No. 802, Garza County; Houston and Great Northern Railroad Company Survey No. 69, Abstract No. 57, Garza County and Will Williams Survey No. 70, Abstract No. 988, Garza County, Texas.

Station 0+00 on the centerline of the dam will be S 29° 30' W, 5000 feet from the northwest corner of the Houston and Great Northern Railroad Company Survey No. 55, Abstract No. 120, Kent County, and Abstract No. 810, Garza County, Texas.

2. USE

(a) Permittee is authorized to divert and use not to exceed 35,000 acre-feet of water per annum from the reservoir for municipal purposes.

(b) Permittee is authorized to make secondary use of not to exceed 21,000 acre-feet of water per annum (treated sewage effluent) out of the maximum 35,000 acre-feet of water diverted for municipal purposes to irrigate 10,000 acres of land in Lubbock and Lynn Counties, Texas.

(c) Permittee is authorized to use the impounded water for nonconsumptive recreational purposes.

(d) Permittee is authorized to divert and use not to exceed 200 acre-feet of water per annum for five years from the South Fork of the Double Mountain Fork of the Brazos River for construction of the dam and reservoir.

3. DIVERSION

(a) Point of Diversion: On the north shore of the reservoir, at a point S 43° W, 6500 feet from the northwest corner of the Houston and Great Northern Railroad Company Survey No. 55, Abstract No. 120, Kent County, and Abstract No. 810, Garza County, Texas.

(b) Maximum Rate: 69.6 cfs (31,200 gpm).

4. TIME LIMITATIONS

Construction of the dam and related facilities herein authorized shall be in accordance with plans approved by the Executive Director and shall be commenced within two years and completed within five years from the date of issuance of this permit. Failure to commence and/or complete construction of the dam and related facilities within the period stated shall cause this permit to expire and become null and void, unless permittee applies for an extension of time to commence and/or complete construction prior to the respective deadlines for commence and completion, and the application is subsequently granted.

This permit is issued subject to all superior and senior water rights in the Brazos River Basin.

Permittee agrees to be bound by the terms, conditions and provisions contained herein and such agreement is a condition precedent to the granting of this permit.

All other matters requested in the application which are not specifically granted by this permit are denied.

This permit is issued subject to the Rules of the Texas Department of Water Resources and to the right of continual supervision of State water resources exercised by the Department.

TEXAS WATER COMMISSION

/s/ Paul Hopkins
Paul Hopkins, Chairman

/s/ Lee B. M. Biggart
Lee B. M. Biggart, Commissioner

/s/ Ralph Roming
Ralph Roming, Commissioner

Date Issued:

September 25, 1984

Attest:

/s/ Mary Ann Hefner
Mary Ann Hefner, Chief Clerk

# Section 10 – Lubbock's Next Water Supply – Year 2012 - Lake Alan Henry

## d. Prior and Current Yield Information

**Yield History for Lake Alan Henry**
City of Lubbock
July 13, 2007

2007 – HDR - Safe Yield – 19,000 AF

2007 – HDR - Firm Yield – 22,200 AF

2004 – Feese and Nichols – Average Yield - 22,200 AF (maintain minimum elevation of 2,185)

2003 – Freese and Nichols – Firm Yield - 22,500 AF

2003 – Freese and Nichols – Maintain Recreation Level Yield - 16,500 AF

1992 – Geraghty & Miller, Inc. – Firm Yield – 27,400 AF

1984 – State of Texas permit process – Firm Yield - 24,750 AF (no document, but referenced by Freese and Nichols in 2003)

1978 – Freese & Nichols – Firm Yield – 26,100 AF

1975 – Freese & Nichols – Firm Yield – 28,500 AF

1971 – Freese & Nichols – Firm Yield – 40,000 AF for both Post and Justiceburg (Lake Alan Henry

1971 – Freese & Nichols – Firm Yield – 30,000 AF

**From:** "Dunn, David" <David.Dunn@hdrinc.com>
**To:** "Thomas Adams" <TAdams@mail.ci.lubbock.tx.us>
**Date:** 4/11/2007 11:04:50 AM
**Subject:** Lake Alan Henry yield

Tom,

We have completed our update of the yield analysis for Lake Alan Henry. We obtained some very non-intuitive results and had to verify what is happening. We will summarize in a memorandum, but here is the information in a nutshell:

1. Freese and Nichols (FNI) completed an analysis of the yield of Lake Alan Henry in 2003, which extended the hydrology through December 2002. We were able to re-create the 2003 firm yield analysis by FNI within 370 acft/yr. The 2003 FNI analysis produced an estimated firm yield of 22,500 acft/yr. Our recent analysis produces an estimated firm yield of 22,870 acft/yr. These yields are based on the original elevation-area-capacity data for LAH, and a storage capacity of 115,937 acft. We obtained the same critical drought period, which begins in October 1972 and extends through April 1999. The length of the drought is key to the analysis, which will be explained later.

2. The FNI analysis extended the period of record through 2002. However, the reservoir does not refill by that time, so the estimated yield could have been low since the drought was not "broken" by the end of 2002. Our analysis indicates that the reservoir would refill by June 2005, but would be within 730 acft of full in January 2005. The critical period does not change with the updated hydrology.

3. Using the updated elevation-area-capacity data recently provided by the TWDB and a reduced storage capacity of 94,808 acft, the yield is reduced from 22,870 acft/yr to 22,200 acft/yr, or a reduction of only 670 acft/yr. This is the result that is not intuitively obvious. One would normally assume that a reduction of 21,129 acft in reservoir storage (18 percent of the original) would reduce the firm yield by more than 670 acft/yr. The reason is that the critical drought period is so long (almost 27 years) that larger evaporation from a moderately larger reservoir surface area at the original capacity effectively uses up the additional 21,129 acft of storage. By the time the two simulations reach October 1998, the storage amounts in the reservoirs are essentially equal. In short, at the larger original capacity, the reservoir loses water to evaporation at a higher rate and this negates any benefit the larger storage volume would apparently provide. If the critical drought period were not so long, this would not be the case.

4. Storage in the reservoir during the critical period is sensitive to small changes in the estimate. Reducing the yield estimate by a few hundred acft/yr results in a minimum storage during the critical drought period that is several thousand acft.

5. Due to the extended nature of the new critical drought, it would be best to depend on a safe yield supply from the reservoir rather than a firm yield supply. Additionally, it would be prudent to explore the concept of using a safe yield supply that leaves a longer than 1-year supply in storage during the critical drought month, say an 18-month or

2-year supply. The one-year safe yield of the reservoir is about 19,000 acft/yr.

I hope you find this information useful. We will have a formal memorandum to you shortly.

Regards,

David

David D. Dunn, P.E.
Vice President/Project Manager
HDR | ONE COMPANY | Many Solutions
4401 West Gate Boulevard, Suite 400 | Austin, TX | 78745
Phone: 512.912.5136 | Fax: 512.912.5158 | Email: David.Dunn@hdrinc.com

**CC:** "Lemonds, Paula Jo" <Paula.Lemonds@hdrinc.com>

[redacted]

2004

Freese and Nichols, Inc. Engineers Environmental Scientists Architects

4055 International Plaza, Suite 200 Fort Worth, Texas 76109 817 735-7300 817 735-7491 fax www.freese.com

September 8, 2004

Mr. Chester Carthel
City of Lubbock
Fax: (806) 775-3344

Dear Mr Carthel

Per your request, I am sending the water availability results for a diversion of 25,000 acre-feet per year from Lake Alan Henry. We used the modified version of the Brazos Water Availability Model of the Brazos Basin (Brazos WAM) that was used by Freese and Nichols in 2002 for a previous evaluation of the firm yield of the Lake. This modified Brazos WAM extends the hydrology through 2002 and allows Lake Alan Henry to impound all inflows. (The unmodified TCEQ Brazos WAM has a period of record from 1940 to 1997 and assumes priority appropriation, which means that Lake Alan Henry must pass inflows for senior water rights). We considered two scenarios: One with no minimum storage, and another leaving a minimum storage at elevation 2185 feet.

With no minimum elevation, the diversion of 25,000 acre-feet per year is met 92% of the years. The average diversion is 24,400 acre-feet per year with a minimum annual diversion of 16,065 acre-feet. Attachments 1 thorough 3 show the storage trace, the annual diversion, and the annual reliability curve for the diversion. The annual reliability curve represents the frequency at which the annual diversion is equaled or exceeded.

With a minimum elevation at 2185 feet, the total 25,000 acre-feet are diverted in 75% of the years, the average diversion is 22,200 acre-feet per year, and the minimum annual diversion is 145 acre-feet. These results are summarized in Attachments 4 though 6.

Please contact me at 817-735-7292 or at aas@freese.com if you have further questions.

Best regards,

Andres Salazar, PhD

2005 Richard ~~Casan~~
~~firm Anton~~ B.

# DRAFT MEMORANDUM TO FILE

From: Thomas C. Gooch, P.E., and Andres A. Salazar, Ph.D., Freese and Nichols

To: Martin Rochelle

Date: March 19, 2003 File: LGB03164:\T\Memorandum.doc

Project: LGB-03164,

Subject: Analysis of the Yield of Lake Alan Henry

## INTRODUCTION

In March of 2003, Lloyd, Gosselink, Blevins, Rochelle, Baldwin, and Townsend, P.C., hired Freese and Nichols to develop an analysis of the yield of Lake Alan Henry, a lake located on the South Fork of the Double Mountain Fork of the Brazos River in Garza and Kent Counties, Texas. Texas water right permit 4146, held by the Brazos River Authority, authorizes the impoundment of 115,937 acre-feet in Lake Alan Henry and the use of up to 35,000 acre-feet per year from the reservoir. The BRA has contracted to provide water from the lake to the City of Lubbock.

The permitted diversion of 35,000 acre-feet per year is in excess of the reliable supply from the lake. During the water right permit hearings on Lake Alan Henry, its firm yield was established as 24,750 acre-feet per year, based on impounding all inflow to the lake and on the initial area and capacity characteristics of the lake. (The firm yield is the amount of water than could be supplied without shortage for the entire period of record. The firm yield of a lake is based on historical conditions. Therefore, the firm yield can be reduced if there is a drought worse than any in the historical record.)

Since there has been an on-going drought in the upper Brazos River Basin, where Lake Alan Henry is located, Freese and Nichols was asked to extend the period of record for Lake Alan Henry to include recent years and determine the firm yield of the project based on hydrologic data from 1940 through 2002.

## BRAZOS BASIN WATER AVAILABILITY MODEL

The Texas Natural Resource Conservation Commission has been developing Water Availability Models for all river basins in Texas. The purpose of the models is determine the reliable supply available to existing water rights and establish a basis for modeling future applications for water rights. The Water Availability Model for the Brazos Basin (Brazos WAM) was developed for TCEQ by HDR, assisted by Freese and Nichols, Crespo Consulting, Inc., and Densmore and DuFrain Consulting [(1)]. The Brazos WAM used hydrology from 1940 through 1997.

---

[(1)] Superscripted numbers in parentheses match references listed in Appendix A.

2003



Like all of the Water Availability Models developed for TCEQ, the Brazos WAM followed certain basic assumptions:

- Water rights were modeled based on the water right documents, without regard for side agreements not included in the water rights.
- Water rights were allowed to use water strictly in priority order. Thus, no water right could impound or divert flow in a month unless all downstream senior water rights were fully satisfied.
- The strict priority doctrine was applied even in cases like that of Lake Alan Henry, where most water released for downstream water rights would be consumed by channel losses.

As a result of these basic assumptions, the Brazos WAM modeled substantial releases of inflow from Lake Alan Henry to satisfy downstream water rights, including the Brazos River Authority's Possum Kingdom Lake. With these releases of inflow, the firm yield for Lake Alan Henry in the Brazos WAM was 9,595 acre-feet per year. This is an underestimate of the reliable supply from the reservoir for several reasons:

- The City of Lubbock has reached an agreement with the Brazos River Authority regarding the impact of Lake Alan Henry on the yield of Possum Kingdom Lake. This agreement is assumed to allow Lake Alan Henry to impound inflows without regard to the senior water rights in Possum Kingdom Lake.
- Several of the run-of-the-river water rights between Lake Alan Henry and Possum Kingdom Lake are permits for the diversion of underflow from the stream. This means that the water rights are authorized to pump river-related groundwater from the alluvium near the river. Such water rights do not require a continuous flow in the stream, since the alluvium acts to store water during floods for later diversion. (The Brazos WAM assumes that releases of inflow would be made to maintain a continuous streamflow for these rights. In my opinion, this assumption is overly conservative.)
- The Brazos WAM bases inflow to Lake Alan Henry on a combination of flows at the USGS gage on the South Fork of the Double Mountain Fork of the Brazos River near Justiceburg, just upstream from the dam, and flows at the USGS gage on the Double Mountain Fork near Aspermont, considerably downstream from the dam. During the water right hearing for the project, testimony established that the flow characteristics of the watershed between the Justiceburg gage and the dam are similar to those above the gage. For this reason, we believe that the inflows to Lake Alan Henry should be based on flows at the Justiceburg gage. This is discussed in greater detail in Appendix B.

Based on the discussion above, we believe that it is more appropriate to look at the firm yield of Lake Alan Henry holding all inflow and using inflows based on flow at the Justiceburg gage. This gives a yield for Lake Alan Henry of 23,800 acre-feet per year.

The critical period of low flows that determines the yield is from October of 1972 through March of 1997.

## EXTENSION OF PERIOD OF RECORD THROUGH 2002

The analysis of Lake Alan Henry yield holding all inflow and using hydrology from 1940 through 1997 indicates that the reservoir would be extremely low at the end of 1997. This would make the project vulnerable to a reduction in yield if there were low flows in the years immediately following. In order to check the impact of the actual hydrologic conditions on project yield, we extended the data available for the WAM to include 1998 through 2002, so that the total period of record was 1940 through 2002. The methodology for extending the hydrology is discussed in Appendix C.

With the extended hydrologic period, the yield of Lake Alan Henry holding all inflow is reduced slightly, to 22,500 acre-feet per year. The critical period of low inflow that determines the yield extends from October of 1972 through April of 1999. Figure 1 shows how the storage in Lake Alan Henry would vary over time if it were operated to retain all inflow with a constant demand of 22,500 acre-feet per year. Figure 1 shows that the reservoir would not be full at the end of the analysis (at the end of 2002), but that it would have over 50,000 acre-feet in storage. Thus the reservoir would be vulnerable to a reduced yield if low flows continue for an extended period in 2003 and after. A short period of low flows in 2003 would not reduce the yield because diversions could be met for a time from the water in storage.

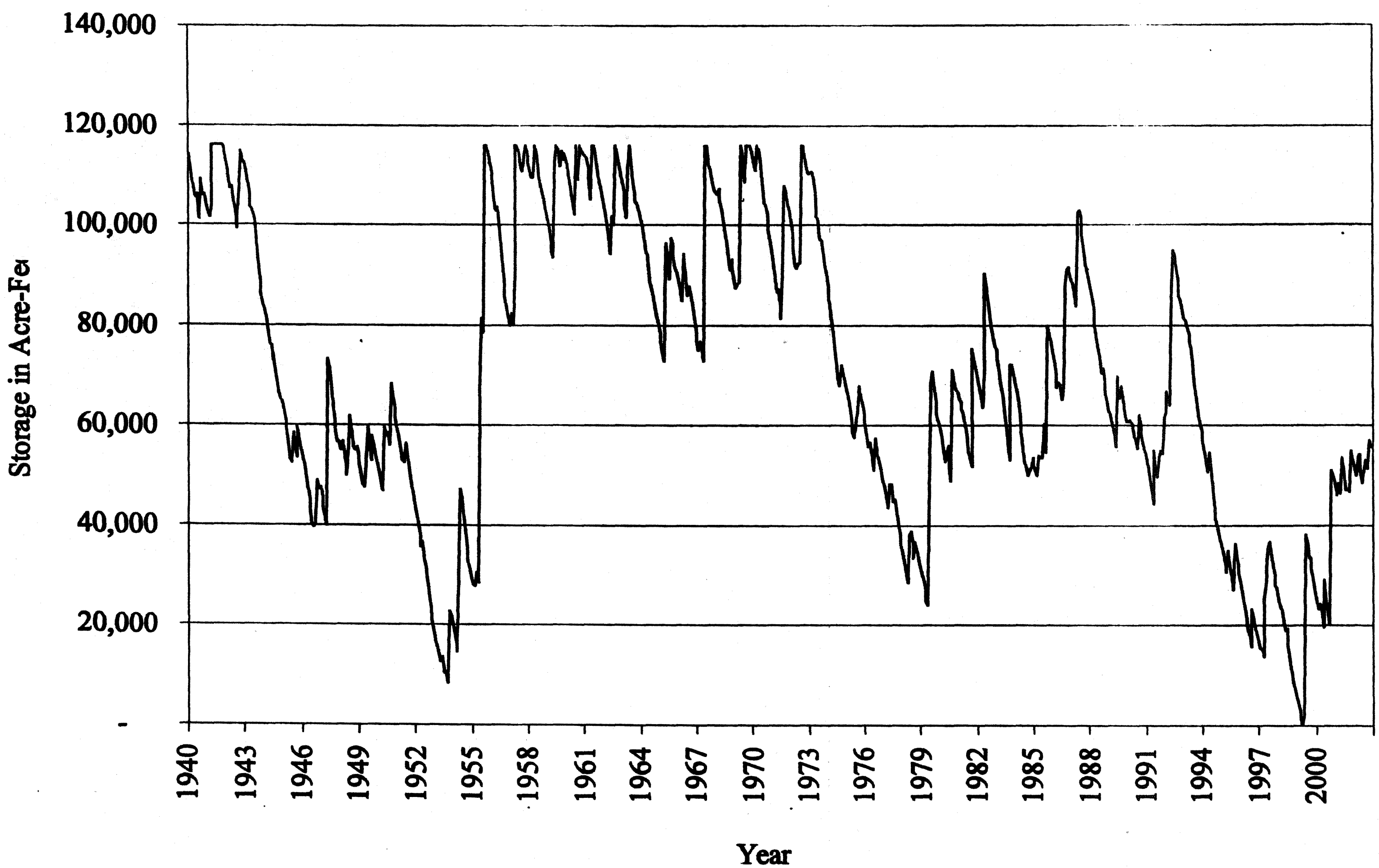
## YIELD WITH A MINIMUM POOL ELEVATION OF 2185

Freese and Nichols was asked to determine the impact on the yield of Lake Alan Henry of maintaining a minimum elevation in the lake of 2185 feet above sea level. Maintaining this minimum elevation (equivalent to slightly over 40,000 acre-feet of storage in the reservoir) would reduce the yield of the project to 16,500 acre-feet per year impounding all inflow. Figure 2 shows how the storage in Lake Alan Henry would vary over time with a constant demand of 16,500 acre-feet per year.

## IMPACT OF POSSIBLE RELEASES FOR DOWNSTREAM WATER RIGHTS

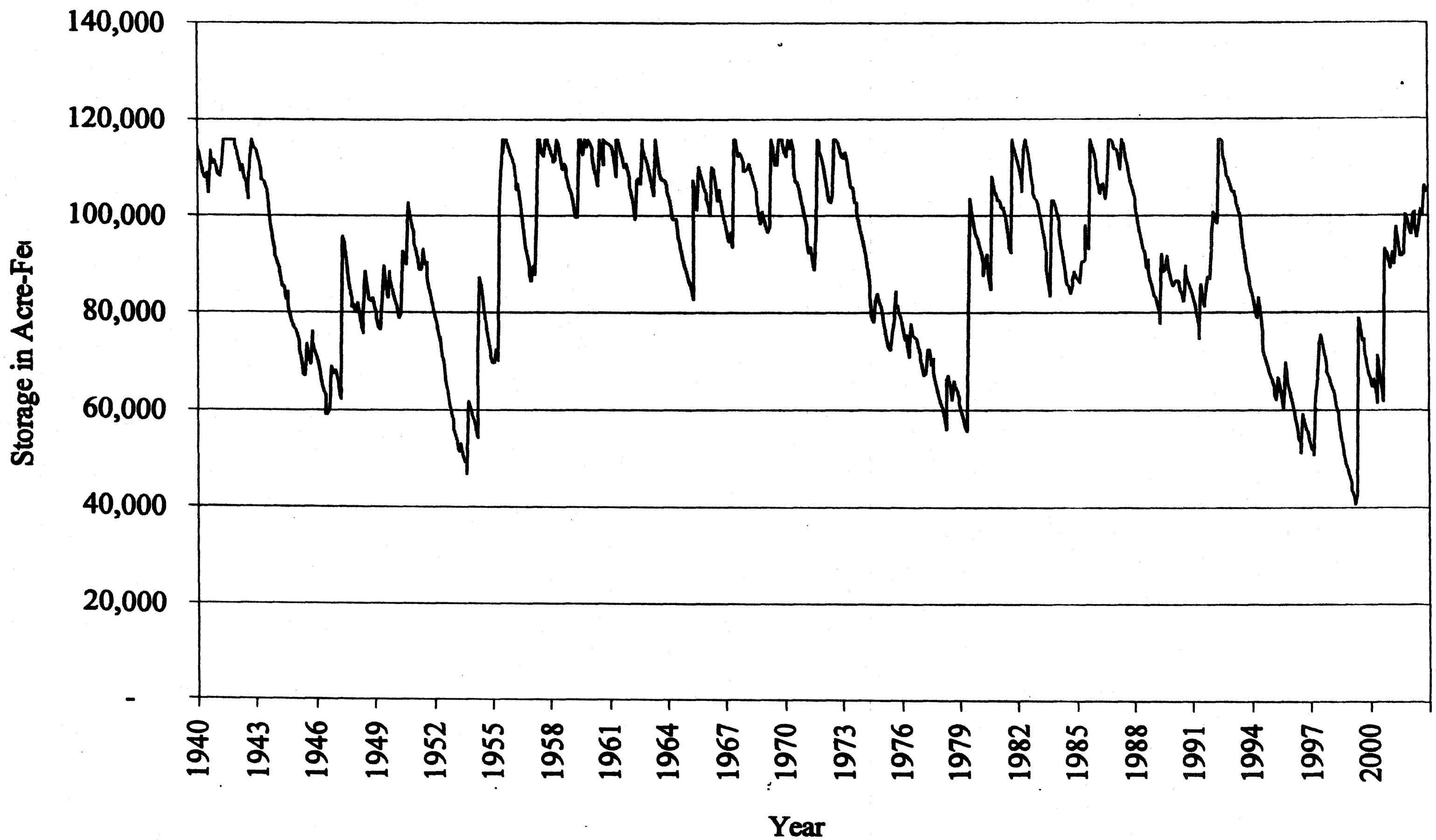
As discussed in the section on the Brazos WAM, releases of inflow to satisfy downstream water rights could have an impact on the yield of Lake Alan Henry. Table 1 lists water rights on the Double Mountain Fork of the Brazos River and the Brazos River between Lake Alan Henry and Possum Kingdom Lake [2]. The table lists underflow water rights, which are authorized to pump groundwater from the alluvium near the stream channel, and surface water rights. Our analysis of the possible impact of releases of inflow for downstream water rights is based on the following assumptions:

**Figure 1**
**Storage in Lake Alan Henry from 1940 through 2002 with Diversions of 22,500 Acre-Feet per Year**




2003

**Figure 2**
**Storage in Lake Alan Henry from 1940 through 2002 with Diversions of 16,500 Acre-Feet per Year**
**(Minimum Pool Elevation of 2185 Feet)**




2003

**Table 1**
**List of Water Rights between Lake Alan Henry and Possum Kingdom Lake**

| WR Number | Owner Name | Amount (Ac-Ft/Yr) | Priority | Stream Name | County |
|---|---|---|---|---|---|
| | | **Underflow Water Rights** | | | |
| 3718 | OCCIDENTAL PERMIAN LTD | 3,525 | 03/05/1958 | DBL MTN FRK BRAZOS RIVER | Kent |
| 3718 | OCCIDENTAL PERMIAN LTD | 2,375 | 07/22/1969 | DBL MTN FRK BRAZOS RIVER | Kent |
| 3719 | KERR-MCGEE OIL & GAS ONSHORE LLC | 165 | 06/24/1968 | DBL MTN FRK BRAZOS RIVER | Fisher |
| 3722 | KERR-MCGEE OIL & GAS ONSHORE LLC | 565 | 07/03/1972 | DBL MTN FRK BRAZOS RIVER | Stonewall |
| 5282 | CITATION 1994 INVEST LTD PART | 235 | 02/02/1990 | DBL MTN FRK BRAZOS RIVER | Stonewall |
| 5435 | PLAINS PETROLEUM OPERATING CO | 235 | 11/05/1992 | BRAZOS RIVER | Knox |
| *Total senior to Alan Henry* | | *6,630* | | | |
| *Total underflow water rights:* | | *7,100* | | | |
| | | **Surface Water Rights** | | | |
| 3717 | BALDRIDGE FAMILY LAND TX PARTN | 420 | 08/31/1951 | DBL MTN FRK BRAZOS RIVER | Kent |
| 3724 | DON W DAVIS | 1,016 | 08/31/1955 | DBL MTN FRK BRAZOS RIVER | Haskell |
| 3453 | PITCOCK BROTHERS READY-MIX | 100 | 12/19/1960 | BRAZOS RIVER | Young |
| 5692 | ZEBRA INVESTMENTS INC | 67 | 07/19/2000 | DBL MTN FRK BRAZOS RIVER | Stonewall |
| *Total senior to Alan Henry* | | *1,536* | | | |
| *Total surface water rights:* | | *1,603* | | | |
| **Total Senior to Alan Henry** | | **8,166** | | | |
| **Total** | | **8,703** | | | |

- The existing agreement between the Brazos River Authority and Lubbock makes it unnecessary to release inflows to satisfy the senior water rights in Lake Possum Kingdom. (This agreement should be reviewed by an attorney to confirm that this is a reasonable assumption.)
- It is unnecessary to release inflows to satisfy senior water rights downstream from Possum Kingdom Lake because of the extremely limited impact of releases on flows below Possum Kingdom. (The Brazos WAM estimates channel losses

between Lake Alan Henry and Possum Kingdom Lake as 84 percent of upstream flows.)

- It is unnecessary to release inflows for underflow water rights because the releases are not needed to maintain the availability of these rights.

With these assumptions, inflows would at most be released to meet senior surface water rights between Lake Alan Henry and Possum Kingdom Lake, which total 1,536 acre-feet per year.

Releasing inflows as needed to meet these water rights would reduce the yield of Lake Alan Henry by 550 acre-feet per year. It is certainly possible that releases for these water rights would not be required because they are accustomed to intermittent water availability. It could also be argued that release of water from Lake Alan Henry for these downstream rights would be wasteful because of the channel losses between the lake and the water rights.

**SUMMARY**

1. Considering hydrology from 1940 through 2002, the firm yield of Lake Alan Henry holding all inflow is 22,500 acre-feet per year with no minimum pool elevation.

2. Considering hydrology from 1940 through 2002, the firm yield of Lake Alan Henry holding all inflow is 16,500 acre-feet per year with a minimum pool elevation of 2185 feet above sea level.

3. Releasing inflows as needed for diversions by senior water rights between Lake Alan Henry and Possum Kingdom Lake would reduce the yield of Lake Alan Henry by 550 acre-feet per year. This assumes that:
    - Releases are not made for Possum Kingdom Lake.
    - Releases are not made for water rights downstream from Possum Kingdom Lake.
    - Releases are not made for underflow diversion water rights.

4. The agreement between Lubbock and the Brazos River Authority should be reviewed by an attorney to assure the validity of the assumption that releases of inflow for Possum Kingdom Lake are not needed.

2003

# APPENDIX A

## REFERENCES


1. HDR Engineering Inc. *Water Availability in the Brazos River Basin and San Jacinto-Brazos Coastal River Basin,* prepared for the Texas Commission of Environmental Quality, December 2001.

2. Texas Commission of Environmental Quality (TCEQ). Water rights database, available online at http://www.tceq.state.tx.us/permitting/waterperm/wrpa/permits.html#databases.

3. United States Geological Survey (USGS). Daily streamflow data for Texas. Available on line at http://waterdata.usgs.gov/tx/nwis/discharge.

4. Texas Commission of Environmental Quality (TCEQ). Records of historical diversion and return flows. Provided by TCEQ Central Records.

5. National Oceanic and Atmospheric Administration (NOAA). Climatological Data for Texas. Volumes 106 and 107. 2001-2002.

6. Texas Water Development Board (TWDB). Evaporation/Precipitation Data for Texas. Available on line at http://hyper20.twdb.state.tx.us/Evaporation/evap.html

2003

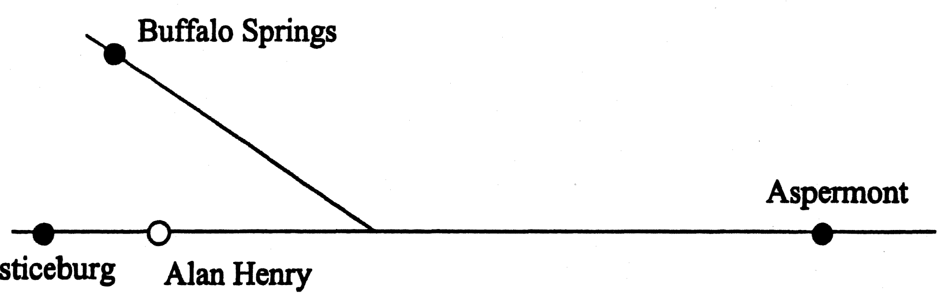
# APPENDIX B

## USE OF JUSTICEBURG GAGE FOR LAKE ALAN HENRY INFLOWS

The Brazos WAM uses the drainage area ratio method to find the inflow to Lake Alan Henry. The incremental flow per square mile between several upstream gages and one downstream gage is assumed to remain constant in that part of the basin. The gages used in the WAM to estimate the flow are found in Table B-1.

**Table B-1. Gages Used in the WAM to Estimate Lake Alan Henry Inflows**

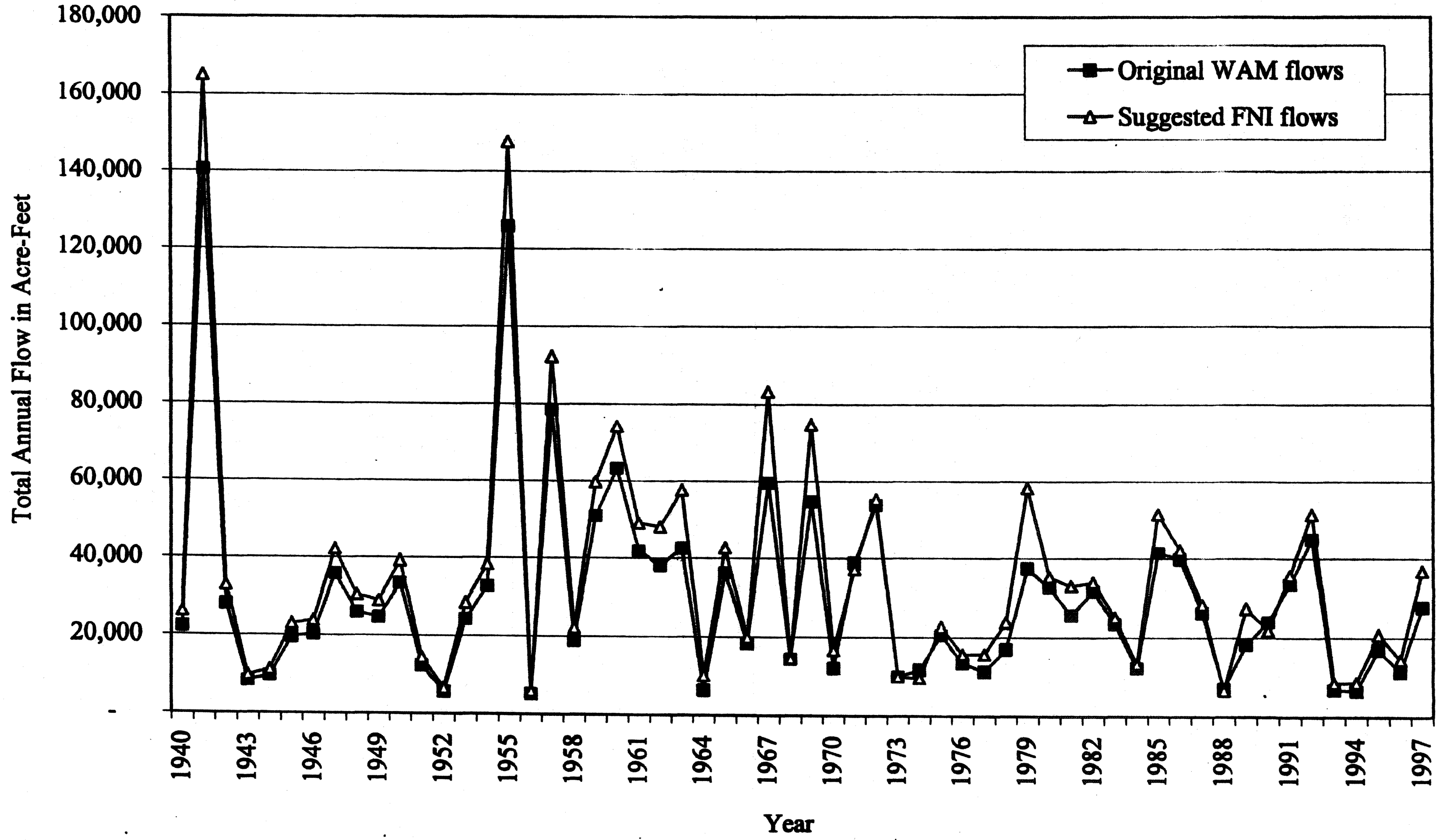
| Stream Name | Gage Number | WAM Contributing Drainage Area (Square Miles) | USGS Contributing Drainage Area (Square Miles) |
|---|---|---|---|
| *Upstream:* | | | |
| Double Mountain Fork Brazos River at Justiceburg | USGS 08080500 | 265 | 244 |
| Buffalo Springs Lake near Buffalo | USGS 08079550 | 245 | 236 |
| *Downstream* | | | |
| Double Mountain Fork Brazos River near Aspermont | USGS 08080500 | 1,891 | 1,864 |
| *Drainage area to estimate flow* | | | |
| Alan Henry | | 408 | 395 |




Lake Alan Henry is significantly closer to the gage at Justiceburg than any of the other gages. Thus, we believe that the flow in Alan Henry should be proportional to the flow at Justiceburg. The ratio between the flows is the ratio between the USGS total drainage areas:

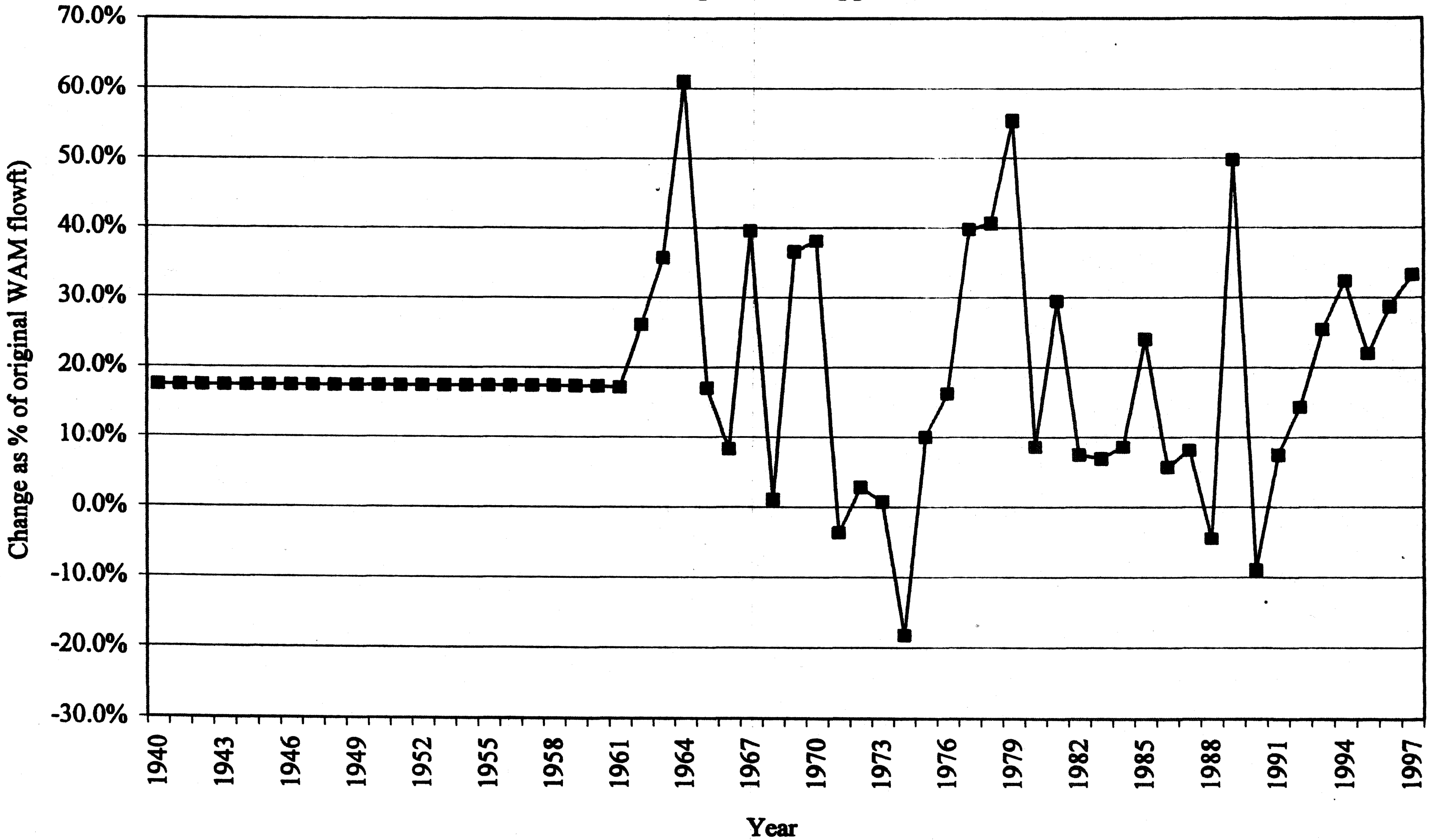
Flow Alan Henry = (395/244) Justiceburg, or Flow Alan Henry = 1.619 * Justiceburg

**Figure B.1**
**Comparison of Annual Natural Flow in Lake Alan Henry using the Current WAM Approach and the Proposed FNI Aapproach**




2003

**Figure B.2**
**Percentage of Change in Natural Flow in Lake Alan Henry between the Current WAM Approach and the Proposed FNI Approach**




2003

2003

Figures B.1 and B.2 show the impact of this change on inflows to Lake Alan Henry. The yield holding all inflows with WAM estimated inflows is 20,250 acre-feet per year. Using the recommended approach, the yield changes to 23,800 acre-feet per year.

2003

# APPENDIX C

## EXTENDING HYDROLOGIC DATA TO INCLUDE 1998 THROUGH 2002

Inflows for Lake Alan Henry was extended to include 1998 through 2002 by developing naturalized flow data for the Justiceburg USGS gage based on flows at the gage [3] adjusted for diversions by upstream water rights. There are three water rights upstream of the Justiceburg USGS gage [1].

| | |
|---|---|
| CA- 3713: | 140 acre-feet per year |
| Permit 5359: | 200 acre-feet per year |
| CA-3714: | 63 acre-feet per year |

The only record available from TCEQ for 1998-2002 water use for these rights was for Permit 5934 during the year 1998 [4]. TCEQ does not have records of more recent years or any other record for the other two water rights upstream of the Justiceburg gage.

According to TCEQ records [4], CA 3713 and CA 3714 historically have had no reported consumption. Reported water use for Permit 5394 started in 1995 with about one third of the authorized amount. Consumption from 1996 through 1998 was between 137 and 160 acre-feet per year, or 80% of the permit. To develop naturalized flows for 1998 through 2002, the consumption for Permit 5394 was assumed to be equal to the average of the last three years with reported diversions.

Evaporation rates were also extended using TWDB and NOAA climatological data [5, 6]. TWDB records end in 2000. NOAA data of pan evaporation (adjusted with the pan coefficient factor) and monthly precipitation were used to estimate net evaporation for the period 2001-2002.

2003

# LAKE ALAN HENRY AVAILABLITY MODEL (WAM)
## Freese and Nichols, Inc.
## March 2003

Executive Summary Drafted April 2005
By Ches Carthel

**Background.** Planning for another surface water supply for Lubbock began in the late 1960's not long after Lake Meredith was built. Freese & Nichols, Inc. (FNI) prepared a report in 1971 that recommended the construction of Lake Alan Henry (LAH)(then called the Justiceburg Reservoir) in about 1990. This 1971 report estimated the LAH Firm Annual Yield (FAY) at about 32,000 acre-feet/year (AF/yr). The Firm Annual Yield (FAY) is the amount of water that could be supplied without shortage for the entire period of record. Since the FAY is based on historical record it can vary depending on the length of records available.

In 1978, FNI performed a detailed feasibility analysis of the Justiceburg Reservoir site. They concluded that a FAY of 26,100 AF/yr was possible. During the permit hearings for LAH, it was established that the FAY was 24,750 AF/yr. The approved permit (no. 4146) allowed a maximum diversion of 35,000 AF/yr.

**Report Summary.** The analysis performed by FNI in 2003 determined that the FAY for LAH is about 22,500 AF/yr. They performed an additional analysis to estimate the FAY should the minimum lake elevation be held at 2185 feet (67 feet depth). This elevation was chosen to maintain a certain amount of water in the lake for recreation. FNI's analysis indicated this restriction would reduce the FAY to 16,500 AF/yr.

The report identified senior water rights downstream of LAH that total 1,536 AF/yr. FNI estimated that releases from LAH to meet these water rights would reduce the FAY by 550 AF/yr. However, it was FNI's opinion that releases would probably not be required because the existing senior water rights are accustomed to intermittent water availability.

The report also identified three senior water rights in the watershed upstream of LAH that total 403 AF/yr. However, it was also noted that only one permit had reported any use.

1992



**Lake Meredith Supply**

Supplies from Lake Meredith were relied upon for the entire planning period in all runs, at 80% allocation of the contract amount, or approximately 30,500 acre-feet/year. The full amount of supply available under the contract with the City of Pampa was also relied upon in all runs for the period.

New surface water supplies that could be developed under contracts with other cities that share in the Lake Meredith supply was dropped from further consideration as a potential supply. This decision was based on the fact that the pipeline from Lake Meredith was operating at full capacity during peak demand periods when such additional supplies would be needed. While not retained for further evaluation in this study and not used in the recommended alternative, this potential supply should be considered by the City as a means of firming up surface water supplies during periods when deliveries would be less than pipeline capacity. In addition, this alternative could become a viable supply beyond the 50-year planning horizon.

**Lake Alan Henry Projected Supply**

Supplies from Lake Alan Henry were used in several runs and in two modes; delivery of 80% of the estimated safe yield of 27,400 acre-feet/year (a supply of 21,900 acre-feet/year was used in the runs) and a staged delivery. For those runs where Lake Alan Henry supplies were relied upon, full deliveries were not required until late in the period leading to ground water overdraft as demands "ramped up" to full yield. When the deliveries were staged, Lake Alan Henry supplies were brought in earlier in the planning period. These runs assumed that the transmission line would be constructed at full capacity but the pumping station and water treatment plant would be built in two modules. This later approach essentially eliminated overdraft and provided better utilization of both surface water and ground water.

1978

Table 3.5

Estimated Amounts of Water Supply Yield Available From the Justiceburg Reservoir

| | Acre-Feet per Year | Equivalent MGD |
|---|---|---|
| Initial firm yield | 26,100 | 23.3 |
| Initial average yield available from variable-demand operation | 30,200 | 26.9 |
| Firm yield after 50 years of siltation and after estimated future runoff depletions | 20,600 | 18.4 |
| Average yield available from variable-demand operation after 50 years of siltation and after estimated future runoff depletions | 27,000 | 24.1 |

Note: All quantities are reounded to the nearest 100 acre-feet per year and the nearest 0.1 MGD.

and water conservation activities on the watershed, which are expected to deplete the average annual runoff by about 1,600 acre-feet per year.

One point of uncertainty should be noted with respect to the yield values shown in the tables of this section. It is possible that the critical drouth could extend into 1979 or later, depending on what rainfall and runoff events occur in the next few months. Based on data available to date, the critical period ended in April of 1978, and there was enough runoff in May and subsequent months to refill the lake to a moderate extent. If there is a normal amount of runoff in 1979, the critical drouth period would not be extended. On the other hand, failure to experience the usual volumes of runoff next spring could cause a lengthening of the critical period and thus a decrease in the firm yield values. The average amounts of supply estimated to be available from

1975

## Table 5.1

### Main Features of Proposed New Surface Water Supply

#### Post Reservoir Site

| | |
|---|---|
| Conservation capacity | 57,420 Ac-Ft |
| Surface area when full | 2,283 Acres |
| Water surface elevation at top of conservation storage | Elev. 2430 |
| Maximum depth of normal storage | 68 Feet |
| Average depth when full | 25.2 Feet |
| Total contributing drainage area | 568 Sq.Mi. |
| Estimated annual yield rate | 9.6 MGD |

#### Justiceburg Reservoir Site

| | |
|---|---|
| Conservation capacity | 133,390 Ac-Ft |
| Surface area when full | 3,123 Acres |
| Water surface elevation at top of conservation storage | Elev. 2220 |
| Maximum depth of normal storage | 104 Feet |
| Average depth when full | 42.7 Feet |
| Total contributing drainage area | 428 Sq.Mi. |
| Estimated annual yield rate | 28.5 MGD |

#### Lake 8 Reservoir Site

| | |
|---|---|
| Normal capacity when full | 49,930 Ac-Ft |
| Surface area when full | 1,680 Acres |
| Water surface elevation when full | Elev. 2921 |
| Maximum depth of normal storage | 73 Feet |
| Average depth when full | 29.8 Feet |
| Total contributing drainage area | 408 Sq.Mi. |

#### Transmission System

| | | |
|---|---|---|
| Pipeline diameters: | Justiceburg Res. to Post Res. | 42 Inches |
| | Post Res. to Lake 8 | 48 Inches |
| | Lake 8 to filter plant | 60 Inches |
| Pipeline distances: | Justiceburg Res. to Post Res. | 17.0 Miles |
| | Post Res. to Lake 8 | 27.3 Miles |
| | Lake 8 to filter plant | 15.7 Miles |
| Proposed capacity: | Justiceburg Res. to Post Res. | 40 MGD |
| | Post Res. to Lake 8 | 50 MGD |
| | Lake 8 to filter plant | 80 MGD |

1971

concentrations. Most of the contamination under these circumstances is from sodium chloride, which suggests the presence of oil well brine. There is appreciable oil activity on the watershed, and it is apparent that some salt water was reaching the watercourse at the time of the measurements. This kind of problem can ususally be cured by more careful oil field operation, and conditions may already be improving due to tighter State regulation of brine disposal methods. Prior to development of the Justiceburg site as a municipal supply, more definitive quality data would be needed, but the information available at this point encourages the belief that the bulk of the runoff would be satisfactory and that the water in the reservoir would be of acceptable chemical composition.

The Reynolds Bend Reservoir would be by far the biggest of those considered herein. The contributing watershed above the Reynolds Bend site is large enough to support a major project, with more yield than could be obtained from any of the other alternatives; it was therefore investigated in detail, although it is the farthest away from Lubbock. However, water quality studies indicated essentially unfavorable prospects for municipal usage from Reynolds Bend, with the concentration of total dissolved solids in the lake frequently exceeding 1,000 milligrams per liter and ranging upward to a maximum of nearly 3,900 milligrams per liter.

Of the several alternatives, the most promising surface water prospects are the Justiceburg site and the Post site. Together, they would provide over 40,000 acre-feet per year of added supply, and they are closer to Lubbock than the other surface water sources. In order to

## 11. Estimates of Cost

To handle the water requirements predicted for the year 2020, new sources must be developed which will furnish approximately 140 MGD of peak daily demand. On an annual basis, the additional supply (or supplies) should provide at least 40,000 acre-feet per year and preferably as much as 60,000 acre-feet per year. Of the various possibilities discussed in the preceding sections, there are two combinations which will meet these goals and which are clearly superior to other available options. The two most promising alternatives are summarized in Table 11.1. One is based primarily on surface water from the Justiceburg and Post Reservoirs and the other on ground water from Hartley County. Both of them also involve ground water from the Eastern Sand Hills area.

Table 11.1

Summary of Most Likely Alternatives
To Meet Additional Requirements Through the Year 2020

| | | Peak Daily Rate In MGD | Annual Supply In Ac-Ft/Yr |
|---|---|---|---|
| **Alternative No. 1** | | | |
| Justiceburg Reservoir | | 40 | 30,000 |
| Post Reservoir | | 10 | 10,000 |
| Peaking storage in Canyon Lake 8 | | 30 | - |
| Additional raw water terminal storage at Lubbock | | 20 | - |
| Eastern Sand Hills wells | | 40 | 20,000 |
| | Total | 140 | 60,000 |
| **Alternative No. 2** | | | |
| Hartley County wells | | 80 | 40,000 |
| Eastern Sand Hills wells | | 50 | 20,000 |
| Additional covered terminal storage at Lubbock | | 10 | - |
| | Total | 140 | 60,000 |

# Section 10 – Lubbock's Next Water Supply – Year 2012 - Lake Alan Henry

## e. 2005 Volumetric Study

# Volumetric Survey of ALAN HENRY RESERVOIR

## July 2005 Survey

Prepared by:

The Texas Water Development Board

September 2006

## Executive Summary

In March of 2005, the Texas Water Development Board (TWDB) entered into agreement with the Brazos River Authority, for the purpose of performing a volumetric survey of Alan Henry Reservoir while the reservoir was near the top of the conservation pool elevation. This information was converted into updated Elevation-Volume and Elevation-Area Tables. The original design information for Alan Henry Reservoir is unavailable; therefore, the TWDB 2005 results are compared to the impoundment rights allowed by Permit to Appropriate State Water No. 4146. In addition, the TWDB established twenty-two sediment range lines to track sedimentation in the reservoir.

**The results of the TWDB 2005 Survey indicate Alan Henry Reservoir has a volume of 94,808 acre-feet and encompasses 2,741 acres at conservation pool elevation, 2,220.0 ft above msl.** Original reservoir volume, as per Permit to Appropriate State Water No. 4146 granted in 1984, was 115,937 acre-feet. This indicates the reservoir has experienced an 18% decrease in volume, or 21,129 acre-feet loss, since it was designed. The BRA states that the area of Lake Alan Henry is 2,884 acres at conservation pool elevation. The TWDB 2005 survey indicates a 5%, or 143 acre, loss in surface area at the conservation pool elevation.

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## Appendices

## Alan Henry Reservoir General Information

Alan Henry Reservoir is located in Garza and Kent Counties on the South Fork of the Double Mountain Fork of the Brazos River. See Figure 1, below.

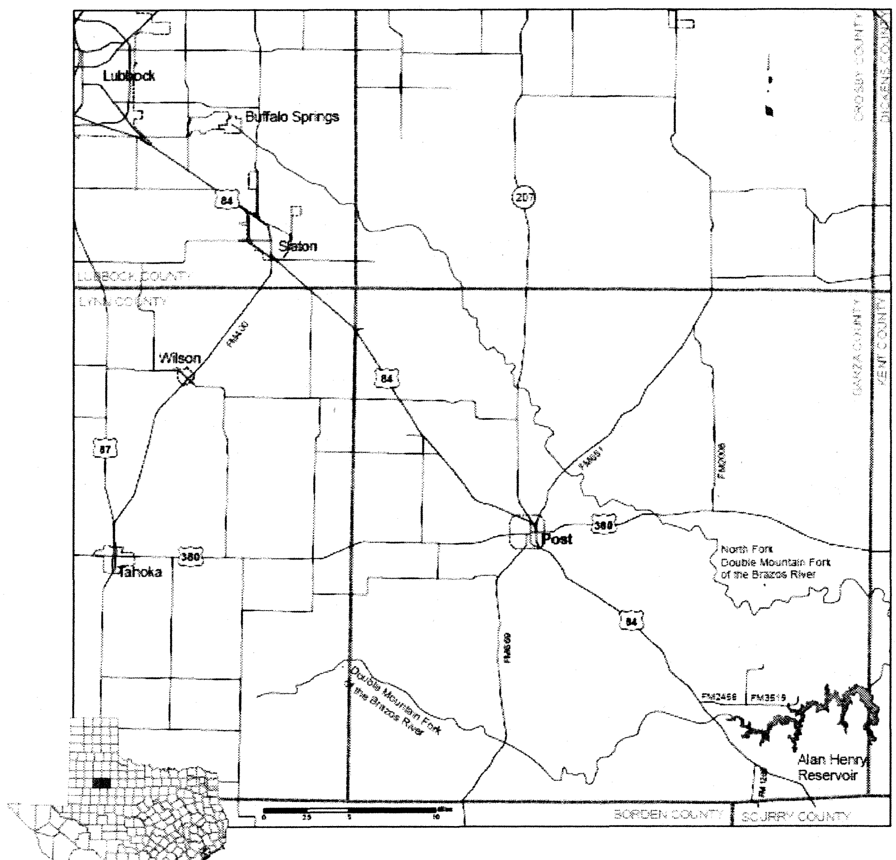



**Figure 1**. Location Map: Alan Henry Reservoir

Planning for the John T. Montford Dam and Alan Henry Reservoir began in the 1960's when city leaders realized that if the population of the City of Lubbock continued to grow as projected; the city would need another source for water. The application was granted and design work completed in the 1980's. Construction of the dam began in 1991, and was completed in October of 1993[1,2]. Currently, the City of Lubbock obtains 80% of its drinking water from Lake Meredith, north of Amarillo, and the other 20% from two ground water well fields in Bailey County (Muleshoe Area) and Roberts County (Pampa Area) that draw from the Ogallala Aquifer. Lake Alan Henry is a tertiary drinking water supply for future use.[1]

The City of Lubbock is located 65 miles Northwest of Alan Henry Reservoir, and is approximately 1,000 ft higher in elevation. Therefore, for Lubbock to use Alan Henry

Reservoir, the city needs three pump stations to take the water uphill to the city, a 65-mile pipeline to carry the water, and a new treatment plant to blend the Lake Alan Henry water with Bailey County well water. The treatment plant will be located in southwest Lubbock.[1]

Garza County and the majority of Alan Henry Reservoir are located within the Llano Estacado Regional Water Planning Group (LERWPG), Region O. LERWPG is a planning body only and does not hold any implementation authority. In the January 2006 Regional Water Plan, approved by the TWDB, there are two water management strategies involving Lake Alan Henry. The first is as a future water supply for the City of Lubbock. The second is to supply water to areas in close proximity to the lake under the jurisdiction of the Lake Alan Henry Water Supply District. The Lake Alan Henry Water Supply District was created through legislation enacted during the 78th Texas Legislative Session, 2003, for the purpose of supplying water from the lake to developing areas adjacent to and near the lake. Voters of the service area confirmed the District in 2004. The City of Lubbock, a wholesale water provider, and the Lake Alan Henry Water Supply District are currently in the process of negotiating a contract to supply water to the District. Figure 2 is a map of the Region O strategy and Lake Alan Henry Water Supply District Project.[3]

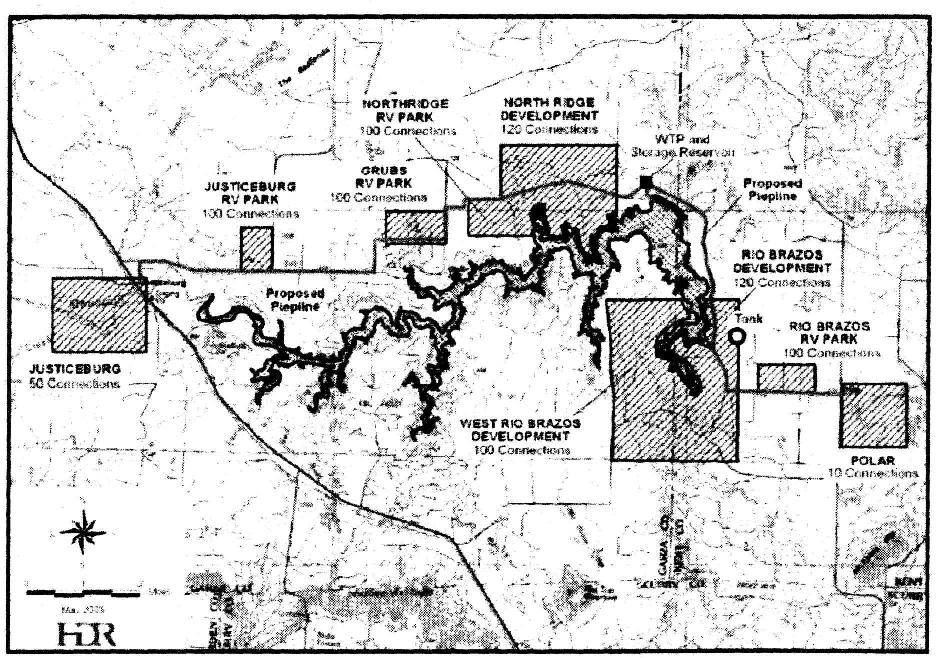



**Figure 2**. Lake Alan Henry Water Supply District Project Map, from the Region O Water Plan.[3]

Alan Henry Reservoir was built by the Brazos River Authority (BRA)[4] and operated by the BRA until 2005, when ownership and operation of the dam and reservoir became the responsibility of the City of Lubbock.[5] Water rights for Lake Alan Henry are as follows:

• **Permit to Appropriate State Water No. 4146**, granted August 6,1984, authorized the City of Lubbock to construct a dam and reservoir on the South Fork of the Double Mountain Fork of the Brazos River and impound therein not to exceed 115,937 acre-feet of water. The permit authorizes the City of Lubbock to divert and use not to exceed 35,000 acre-feet of water per annum from the reservoir for municipal purposes at a maximum diversion rate of 69.6 cfs. The City of Lubbock is also authorized to make secondary use of not to exceed 21,000 acre-feet of water per annum (treated sewage effluent) out of the maximum 35,000 acre-feet of water diverted for municipal purposes to irrigate 10,000 acres of land in Lubbock and Lynn Counties, Texas. In addition the permit authorizes the City of Lubbock to use the impounded water for non-consumptive recreational purposes.

• **Amendment to Water Use Permit No. 4146A**, granted May 2, 2005, recognizes that the Brazos River Authority (BRA) owns Permit No. 4146 with all the rights discussed above. The Amendment deletes the diversion point authorized by Permit 4146 and adds a diversion point at the existing diversion works of the dam, and adds a diversion segment on the north shore of Lake Alan Henry which includes the entire shoreline of the Sam Wahl Recreation Area in Garza County. The Amendment also requires the owner to implement water conservation plans.

• **Texas Commission on Environmental Quality (TCEQ) interoffice memorandum** dated December 19, 2005, from the Water Rights Permitting & Availability Section, Water Supply Division. This memorandum documents the change of ownership of Permit No. 4146A from the BRA to the City of Lubbock, a Texas home rule municipal corporation, by Agreement to Transfer Lake Alan Henry dated July 14, 2005; and Deed and Assignment Without Warranty and Bill of Sale dated August 16, 2005. The complete certificates and permits are on file in the Records Division of the TCEQ.

The following table is a list of pertinent data about the John T. Montford Dam and Alan Henry Reservoir.[1,6]

**Table 1: Pertinent Data for the John T. Montford Dam and Alan Henry Reservoir**

**Owner**:
City of Lubbock
**Operator**:
City of Lubbock
**River Miles from Gulf**: 1,056
**Contributing drainage area (sq. miles)**: 394
**Top of Conservation Pool Elevation**: 2,220.0 ft above msl
**Construction Facts**
**Composition**: 6.5 Million cubic yards of soil, clay, and soil-cement
**Height of Dam**: 138 ft
**Crest Elevation/ Top of Dam**: 2,263 ft above msl
**Length of Dam**: 3,600 ft
**Width of Dam**: 1,000 ft wide at the base
**Service Spillway (Concrete)**: Designed to pass 15.6 million gallons per minute
**Emergency Spillway (Earthen)**: Designed to pass 211 million gallons per minute

## Volumetric Survey of Alan Henry Reservoir

### Introduction

In March of 2005, the Texas Water Development Board entered into agreement with the Brazos River Authority, for the purpose of performing a volumetric survey of Alan Henry Reservoir while the reservoir was near the top of the conservation pool elevation. This information was converted into updated Elevation-Volume and Elevation-Area Tables. Original design information is unavailable, therefore, the TWDB Survey results are compared to the permitted impoundment capacity in Permit to Appropriate State Water No. 4146 and new Sediment Range Lines have been established by the TWDB throughout Alan Henry Reservoir to track future sedimentation.

### Bathymetric Survey

Bathymetric data collection for Alan Henry Reservoir occurred between July 7th and July 9th of 2005, while the water surface elevation was slightly below the conservation pool elevation of 2,220.0 ft above mean sea level (msl). The water surface elevation varied between 2,219.42 ft and 2,219.46 ft above msl during the TWDB survey. The survey team used one shallow water boat equipped with a depth sounder, velocity profiler, and integrated Differential Global Positioning System (DGPS) equipment to navigate along pre-planned range lines spaced approximately 500 feet apart in a perpendicular fashion to the original stream channel. During the 2005 survey, the team navigated over 129 miles of range lines and collected approximately 70,000 data points. Figure 3 shows the data points collected during the TWDB 2005 survey.

The depth sounder was calibrated each day using the velocity profiler to measure the speed of sound in the water column and a weighted tape or stadia rod to verify the depth reading. The average speed of sound through the water column varied between 4,858 and 4,913 feet per second during the 2005 survey.

## Datum

The vertical datum used during this survey is that used by the United States Geological Survey (USGS) for the reservoir elevation gauge USGS 08079700 Lk Alan Henry Res nr Justiceburg, TX.[7] The datum for this gauge is reported as National Geodetic Vertical Datum 1929 (NGVD29) or mean sea level (msl), thus elevations reported here are in feet (ft) above msl. Volume and area calculations in this report are referenced to water levels provided by the USGS gauge. The horizontal datum used for this report is NAD83 State Plane Texas North Central Zone.

## Survey Results

The results of the TWDB 2005 Survey indicate Alan Henry Reservoir has a volume of 94,808 acre-feet and encompasses 2,741 acres at conservation pool elevation, 2,220.0 ft above msl. This indicates the reservoir has experienced an 18% decrease in volume, or 21,129 acre-feet loss, when compared to the original reservoir volume of 115,937 acre-feet, as given in Permit to Appropriate State Water No. 4146. The BRA states that the area of Lake Alan Henry is 2,884 acres at conservation pool elevation.[6] The TWDB 2005 survey indicates a 5%, or 143 acre, reduction in surface area at the conservation pool elevation. Due to the likely differences in the methodologies used to calculate the reservoir's capacity between 1984 and 2005, comparison of these values is not recommended and is presented here for informational purposes only.[8] The TWDB considers the 2005 survey to be a significant improvement over previous methods and recommends that the same methodology be used to resurvey Alan Henry Reservoir in 5 to 10 years.

## Data Processing

### Model Boundary

The reservoir boundary was digitized from aerial photographs using Environmental Systems Research Institute's (ESRI) ArcGIS 9.1 software. The aerial photographs, or digital orthophoto quadrangle images (DOQs), used for Alan Henry Reservoir were Justiceburg and Justiceburg SE. These images were photographed on October 18, 2004. At the time of the photographs the water surface elevation measured 2,220.2 ft above msl, just above the conservation pool elevation. At the scale of the photographs, the difference between 2,220.0 ft and 2,220.2 ft is indiscernible; therefore the boundary was digitized at the land water interface from the photos, and assigned the conservation pool elevation of 2,220 ft.

The United States Department of Agriculture, Farm Service Agency's, Aerial Photography Field Office (APFO), National Agriculture Imagery Program (NAIP) acquires the photographic imagery during the agricultural growing seasons in the continental U.S.[9] The imagery resides in the public domain and can be downloaded from the Texas Natural Resources Information System (TNRIS) website at http://www.tnris.state.tx.us/. For more information visit the APFO website at http://www.apfo.usda.gov/NAIP.html or contact TNRIS.

### Triangular Irregular Network (TIN) Model

Upon completion of data collection, the raw data files were edited using HydroEdit, an automated editing routine developed by the TWDB, to remove any data anomalies. The water surface elevations for each respective day are applied and the depths are converted to corresponding bathymetric elevations, exported, and converted to a shapefile using ArcCatalog. The ArcGIS 3D Analyst Extension is then used to create a Triangular Irregular Network (TIN) model of the bathymetry based on the sounding shapefile and the reservoir boundary files. The ArcGIS 3D Analyst Extension uses

Delaunay's criteria for triangulation to place a triangle between three non-uniformly spaced points, including vertices of the lines in the reservoir boundary file.[10] The Alan Henry Reservoir TIN Model was enhanced through the use of a Self-Similar Interpolation routine developed by the TWDB. See the following section on Self-Similar Interpolation and the Shallow Area Problem for more information.

Using Arc/Info software, volumes and areas are calculated from the TIN Model for the entire lake at one-tenth of a foot intervals, from elevation 2,140.8 ft to elevation 2,220.0 ft. The Elevation-Volume and Elevation-Area Tables, updated for 2005, are presented in Appendices A and B, respectively. An Elevation-Volume graph and an Elevation- Area graph are presented in Appendices C and D, respectively.

The TIN Model was interpolated and averaged using a cell size of 10 ft and converted to a raster. The raster was used to produce Figure 4, an Elevation Relief Map representing the topography of the reservoir bottom, Figure 5, a map showing shaded depth ranges for Alan Henry Reservoir, and Figure 6, a 10-ft contour map.

**Self-Similar Interpolation and the Shallow Area Problem**

A limitation of the Delaunay method for triangulation in the TIN Model results in artificially-curved contour lines extending into the reservoir where the reservoir walls are steep and the reservoir is relatively narrow. These curved contours are likely a poor representation of the true reservoir bathymetry in these areas. To ameliorate this problem, a Self-Similar Interpolation routine (developed by the TWDB) was used to interpolate the bathymetry in between many 500ft-spaced survey lines to increase the density of points input into the TIN Model. The increased point density alters the mean triangle shape from long and skinny to more equilateral, thus providing better representations of reservoir topography.[11] In areas where obvious geomorphic features indicate a high-probability of cross-section shape changes (e.g. incoming tributaries, significant widening/narrowing of channel, etc.), this self-similar assumption is not likely to be valid; therefore, self-similar interpolation was not used in areas of Alan Henry Reservoir where a high probability of change between cross-sections exists.[11] Figure 7 shows the resulting point density after the Self-Similar Interpolation routine was employed. The area interpolated equals 36.5% of the reservoir area (at conservation pool elevation).

Another limitation of the Delaunay method of TIN generation involves the calculation of areas and volumes in sections of the reservoir that were too shallow for bathymetric data collection by boat. This "shallow area problem," as identified by the TWDB, is corrected using the HydroEdit interpolation routines developed by the TWDB. The Delaunay triangulation method, within ArcGIS, creates large flat triangles throughout these un-surveyed areas for which each corner of the triangle lies on the reservoir boundary. These triangles do not suggests any change in slope along the boundary and are assigned zero depths, causing an artificial spike in the elevation-area graphs at the last elevation interval for which reservoir areas are calculated. To correct this, the HydroEdit software program linearly interpolates elevations along connecting lines between the reservoir boundary vertices and their closest sounding points. These interpolated data points are used in conjunction with the surveyed sounding points and the Self-Similar Interpolated points to generate the TIN model. The additional data points result in a model with a more realistic representation of the reservoir bathymetry.[11] Figure 8 shows the resulting point density after the HydroEdit "Shallow Area Problem" routine was employed.

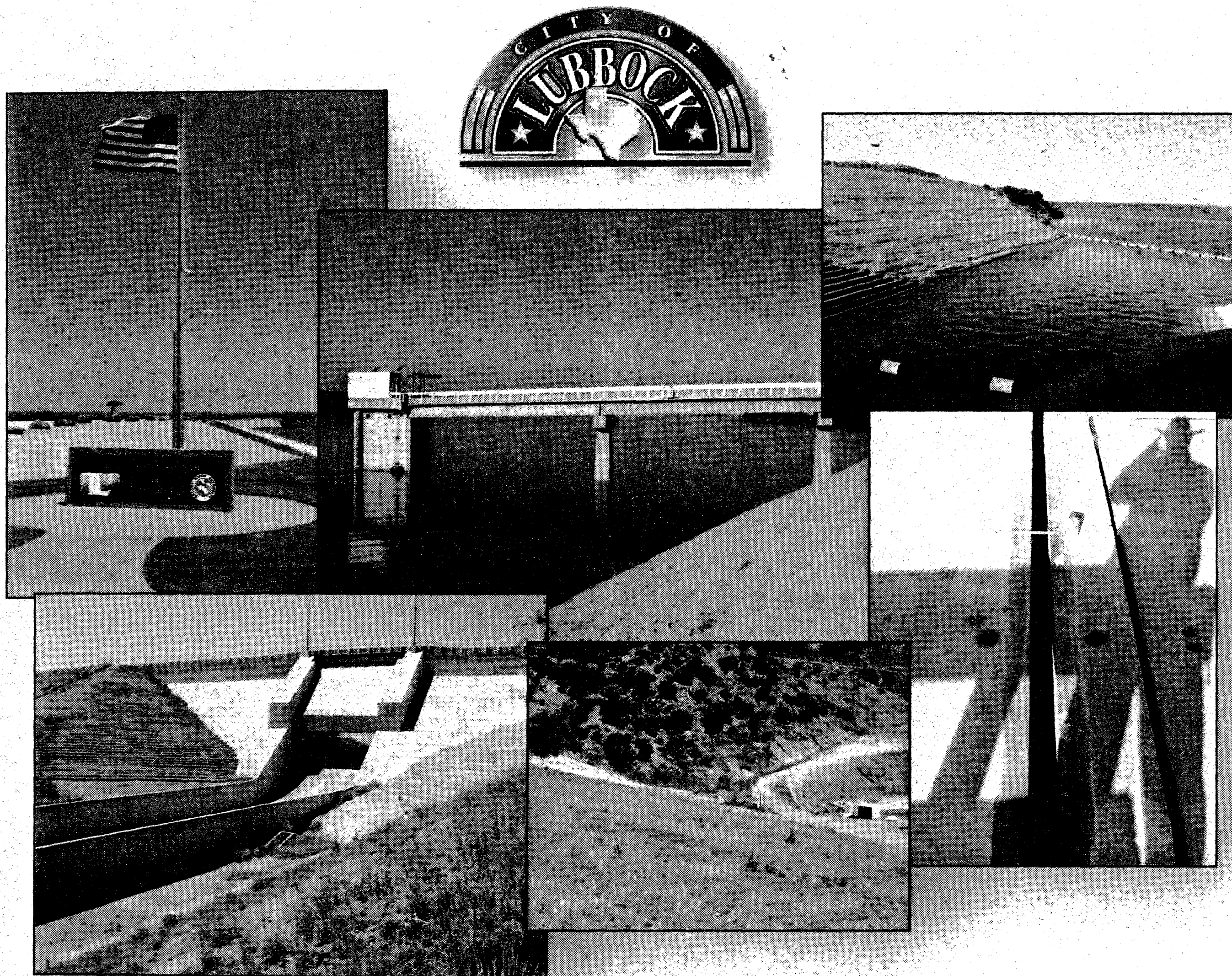
**Sediment Range Lines**

Information for the original design, including range lines, was unavailable. Therefore, the TWDB established twenty-two Sediment Range Lines in Alan Henry Reservoir to track sedimentation in the reservoir. Using ArcGIS, the TWDB staff established sediment range lines near the confluences of each stream, the main channel of the lake, and in bends in the main lake channel where water velocities would slow and drop any sediment load. The Sediment Range Line cross-sectional plots presented in Appendix E were extracted from the TIN Model. Appendix E also contains a map displaying the location of the range lines and a Table listing the endpoint coordinates of each line.

# Section 10 – Lubbock's Next Water Supply – Year 2012
# - Lake Alan Henry

## f. 2005 Dam Inspection

# John T. Montford Dam (Lake Alan Henry) Inspection Report




August 2005

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# Executive Summary

On June 21-22, 2005, HDR Engineering, Inc. (HDR) inspected the John T. Montford Dam, which impounds Lake Alan Henry, one of the City of Lubbock's (City) planned water supply sources. The Brazos River Authority (BRA) currently manages the dam and related project facilities. In mid-August 2005, ownership and management of the project will transfer to the City of Lubbock. The purpose of this inspection, prior to the transfer of the project, was to determine if conditions exist that could threaten the safety of the dam or lead to major capital expenditures or increased operation and maintenance costs.

The dam and appurtenant structures are generally in good condition and no conditions were observed that would constitute an immediate threat to the dam's safety. The dam and appurtenant structures appear to be performing as anticipated by the design with two exceptions. These include: (1) movements at the access bridge to the intake structure, and (2) the lack of a healthy stand of grass on the downstream slope of the dam for protection against erosion by surface runoff. Both of these issues will require continued vigilant monitoring and maintenance, and may ultimately require capital expenditures to correct or improve.

The BRA retained a geotechnical engineer in late 2003 to investigate and evaluate the movements at the abutment and pier supports for the access bridge to the intake structure. Movements are thought to be the result of expansion of shale in the foundation due to an increase in the moisture content as the lake filled. This is certainly plausible, and the relatively higher movements at the abutment and first bridge span could be the result of excavating approximately 25 feet of overburden from the original hill to create the bowl-shaped lower parking area. This same movement is suspected to be the cause of wide cracks in the concrete slope paving under the bridge and transverse cracks in the soil cement armoring along each side of this structure.

To date the foundation movements have resulted in two points of structural distress. First, the bridge girders are dragging outward on the bearing pads at the abutment. The stress concentrations at this location have resulted in spalling of the concrete at the bottom edge of two of the girders. Reinforcing steel is exposed at the bottom of the outer girder on the north side. The second point of distress is located where the bridge meets the intake tower. On the left side (looking upstream), the bridge has made contact with the tower and a small piece of concrete on the parapet wall has spalled off. This reportedly occurred several years ago and the condition has not changed since. Continued monitoring of the bridge and tower will be required and a decision may need to be made at some point regarding the need for modifications.

A substantial amount of resources have been expended trying to establish a healthy stand of grass on the downstream slope of the dam. The BRA project staff has done a commendable job to improve the conditions of the slope and sprinkler system over the last two years. However, much work remains to be done and there are several obstacles to overcome. After nearly 13 years of effort, it may be time to consider abandoning the maintenance-intensive sprinkler system and explore alternate methods for erosion protection of the slope. This could result in a large initial capital expenditure, but the alternatives would need to be weighed against the costs to continue operating and maintaining the sprinkler system, repairing erosion gullies, fixing damage from feral hogs, seeding repaired areas, and mowing.

The pneumatic instrumentation used to monitor the dam's behavior is reaching the end of its service life. The original six base plate settlement devices no longer function. Several of the piezometers have stopped working. This type of instrumentation is especially critical for monitoring the dam during construction, during initial filling of the reservoir, and for several years thereafter. The instrumentation appears to have provided data sufficient to indicate that the critical elements of the dam, such as the core, slurry trench cutoff wall, and internal drainage systems have performed as anticipated by the design. A detailed analysis of the historic data from pneumatic piezometers that have stopped working should be undertaken to determine if simple open riser piezometers should be installed in some of those areas where valuable data were being collected. This analysis should also take into account locations of existing seepage.

A total of 10 drain outlets were installed along the downstream side of the dam to convey water collected by drainage systems inside the embankment. These outlets are monitored monthly and, to date, only three of them are flowing. There is also seepage emerging from the left (looking downstream) abutment near the contact with the embankment in the vicinity of the outlet works stilling basin. A portion of this seepage is collected and measured. The total amount of seepage being collected and measured at these four locations has been approximately 10 gallons per minute with the lake at the conservation pool elevation. This amount is very small for a dam of this size, which provides further evidence that critical internal elements are functioning properly.

The six inclinometers and 24 surface reference monuments on the dam indicate normal behavior of the embankment. Total settlement to date is just over 8 inches at the maximum point, which was predicted during the design.

The maximum lake level to date was approximately 6 feet above the crest of the service spillway. The spillway and its stilling basin reportedly functioned as anticipated by the design and physical model testing. During the inspection, seepage appeared to be emerging from two transverse construction joints in the 3% chute slab nearest to the spillway crest. These joints are at the locations of two sets of pneumatic piezometers that are not being monitored because the read out box for one (spillway Sta. 11+00) has not been located and the other (spillway Sta. 13+00) no longer functions. A determination on the need for piezometers in these locations should be made in light of the seepage observed.

BRA project staff recently installed a continuous floating boom type barrier across the upstream end of the spillway approach channel. Boaters were reportedly getting too close to the structure when it was operating and were at risk of getting swept through by the current. The new barrier system should provide an additional measure of safety. The original buoy and cable system, which was installed closer to the spillway crest, remains in place.

The emergency spillway was designed to operate during extreme floods in excess of the 100-year event. Since completion, small to medium mesquite trees and other woody vegetation and brush have grown throughout the spillway. This vegetation should be cleared because it will restrict flow and may reduce the spillway's capacity to pass large floods.

The present level of operation, maintenance and monitoring being performed by the BRA is appropriate for the size and nature of the project. As the dam and appurtenant structures age, maintenance and repairs to items such as the gates and gate operating system and the various locations of soil cement armoring will likely increase. Continued maintenance and repairs to the sprinkler system and erosion gullies in the downstream slope of the dam are inevitable, and may increase as the system ages. The City may want to consider alternative slope protection solutions that would reduce the long-term maintenance requirements. The addition of a water supply pump station in the future will increase the operation and maintenance activities at the project. Changes to the instrumentation-monitoring schedule are recommended, but will not substantially decrease the current level of effort provided by the BRA project staff.

One of the best tools for monitoring the performance and safety of a dam is vigilant visual observations by the same individual(s) over time. This is especially important during and immediately after major flood events that can subject the dam and appurtenant structures to loading conditions larger than those previously experienced. On-site personnel are the first to evaluate a situation and determine whether or not the Emergency Action Plan should be initiated. Having a history of what constitutes "normal" behavior greatly enhances the response to events that could jeopardize the dam's safety.

Extensive photographic documentation was obtained during the inspection. A CD containing all of the photographs taken during the inspection and a photograph log accompanies this report. This report will have the most value for the City if similar inspections are performed on a regular basis to compare future conditions at the project with the documentation provided herein.

# Section 1
# Project Description

The John T. Montford Dam is located in Kent and Garza Counties, Texas, approximately 60 miles southeast of Lubbock, on the Double Mountain Fork of the Brazos River (Figure 1-1). The dam impounds Lake Alan Henry, which is one of the City of Lubbock's planned water supply sources. The Brazos River Authority (BRA) presently manages the dam and related project facilities. In mid-August 2005, ownership and management of the project will transfer to the City of Lubbock. The purpose of this inspection, prior to the transfer of the project, was to determine if any conditions exist that could threaten the safety of the dam or lead to major capital expenditures or increased operation and maintenance costs.

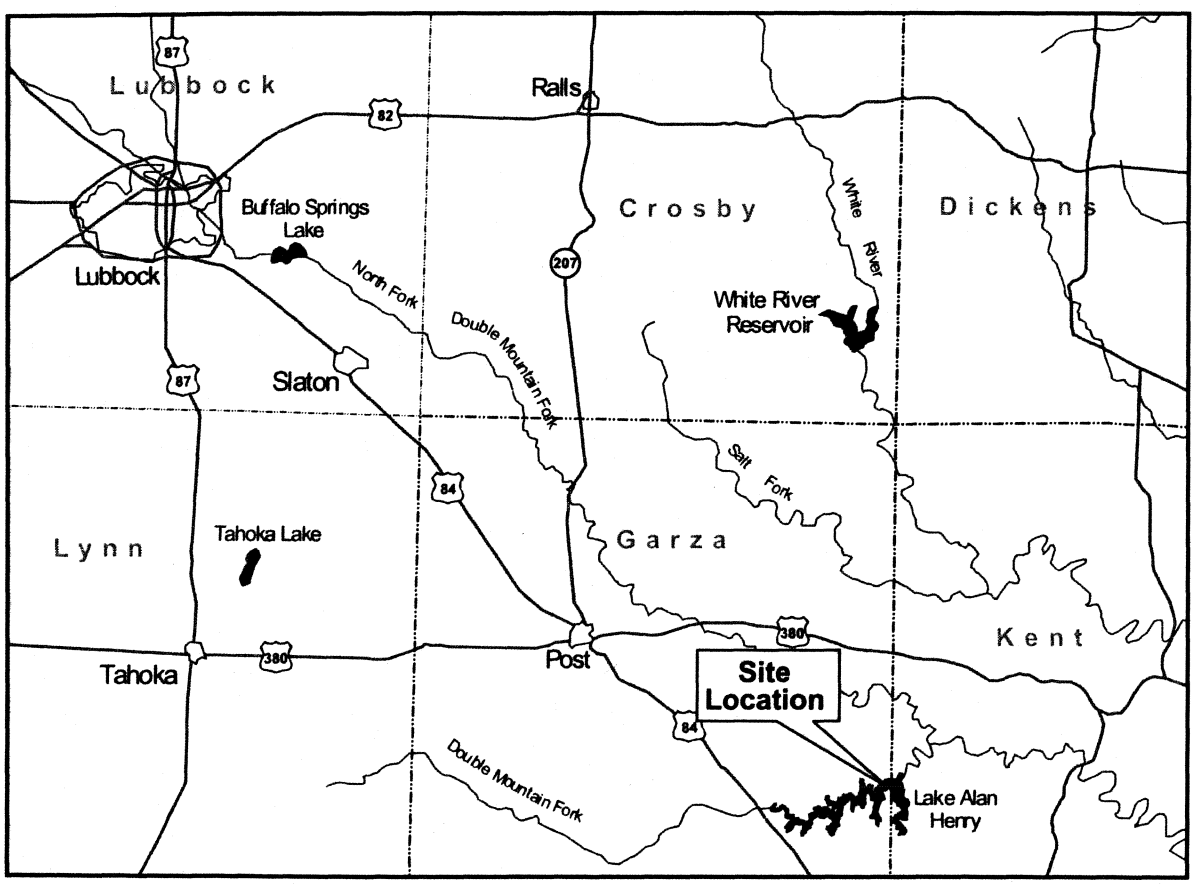


*Figure 1-1. Project Location Map*

Freese & Nichols, Inc., Fort Worth, Texas, designed the dam and appurtenant structures, and plans for construction were sealed on October 19, 1990. The Texas Water Commission approved the construction plans on November 21, 1990. Construction began in March 1991 and was completed in October 1993. Impoundment of Lake Alan Henry began in November 1993. The Record drawings ("as-builts") were sealed by the project engineer on May 19, 1994. The dam is identified as Inventory No. TX06464 by the Texas Commission on Environmental Quality (TCEQ) Dam Safety Program. A site plan showing the major features of the project is provided in Figure 1-2.

The John T. Montford Dam is a zoned earthfill embankment with a slurry wall cutoff. The dam has a maximum height of about 140 feet above the original streambed and a length of approximately 4,150 feet. The top of dam elevation is 2,263 feet mean sea level (ft-msl) and the conservation pool elevation is 2,220 ft-msl. At conservation pool, the lake has a surface area of 2,884 acres and a permitted capacity of 115,937 acre-feet.

Appurtenant structures consist of a concrete service spillway, earth/rock-cut emergency spillway, and outlet works system. The service spillway is a 40-foot wide, uncontrolled, fixed-gate ogee with a hydraulic jump stilling basin to dissipate the flow energy. The overflow crest (elevation 2,220 ft-msl) and stilling basin floor (elevation 2,105 ft-msl) are connected by a 559-foot long concrete chute. The spillway was designed to pass the 100-year flood before the emergency spillway engages. The 1,700-foot wide, earth/rock cut emergency spillway has a crest elevation of 2,240 ft-msl and is located approximately one mile south of the service spillway and right (looking downstream) abutment of the dam. The combined spillway system was designed to safely pass the probable maximum flood (PMF) without overtopping the dam.

The outlet works system consists of a 124-foot tall, dual-chamber concrete intake structure, a 42-inch diameter water supply conduit, a 30-inch diameter discharge conduit, and a combined impact-style concrete outlet structure. The intake tower is located upstream of the dam's left abutment. This location reportedly allowed for the base of the structure to be founded in the shale of the left abutment. The top of the intake tower is at elevation 2,245 ft-msl, five feet above the emergency spillway crest. The tower is connected to shore at the left abutment by a 14-foot wide bridge that has three spans of 73.8 feet each. The concrete bridge girders and deck were designed for H-15 truck loading so that cranes could be used to service equipment at the intake structure.

The two outlet conduits exit the tower at elevation 2,140 ft-msl and pass through the dam foundation askew to the dam centerline along the base of the left abutment before terminating at the outlet structure. The 42-inch outlet conduit will be used for releasing water to a future pump station that is to be located downstream of the left abutment. The chamber for this conduit has three sluice gates (each 54- by 96-inch) to withdraw water from various elevations depending on the lake level and water quality. The 42-inch conduit is presently used to supply water to an irrigation system in the downstream slope of the dam and a small on-site water treatment unit. A 42-inch square sluice gate on the upstream end of the 42-inch conduit is typically open, but can be closed to dewater the conduit if necessary. The 30-inch outlet conduit provides for controlled releases from the lake should that be desired. One 54- by 96-inch sluice gate, near the base of the tower, provides water to the chamber for the 30-inch conduit. Two sluice gates, a 12- inch and a 14-inch, are arranged in a head works at the upstream end of the conduit to provide a range of flow releases by using either or both sluice gates. It should be noted, however, that there are no low-flow release requirements for the project. This conduit can also be dewatered, if necessary, by closing both sluice gates. All sluice gates are operated via hydraulically controlled actuators housed in a 6- by 17-foot concrete masonry building constructed on top of the intake tower.

The two chambers in the tower are connected by a service manway through the center partition wall at elevation 2,236 ft-msl and by a chain-operated 6-inch diameter plug valve at elevation 2,174 ft-msl. The plug valve is used for transferring water between the chambers to reduce the opening head pressure on the lake-side gates. The tower has been designed so that either or both chambers can be drained with the lake at conservation pool elevation 2,220 ft-msl without overstressing or floating the structure due to buoyancy.

# Section 4
# Summary and Recommendations

## 4.1 Summary of Conditions

On June 21-22, 2005, HDR Engineering, Inc. (HDR) inspected the John T. Montford Dam, which impounds Lake Alan Henry, one of the City of Lubbock's (City) planned water supply sources. The purpose of the inspection, prior to the transfer of the project from the Brazos River Authority (BRA) to the City in mid-August, was to determine if any conditions exist that could threaten the safety of the dam or lead to major capital expenditures or increased operation and maintenance costs.

The dam and appurtenant structures are generally in good condition and no conditions were observed that would constitute an immediate threat to the dam's safety. The dam and appurtenant structures appear to be performing as expected with two exceptions. These include: (1) movements at the access bridge to the intake structure, and (2) the lack of a healthy stand of grass on the downstream slope of the dam for protection against erosion by surface runoff.

The BRA retained a geotechnical engineer in late 2003 to investigate and evaluate the movements at the abutment and pier supports for the access bridge to the intake structure. Movements are thought to be the result of expansion of shale in the foundation due to an increase in the moisture content as the lake filled. To date the foundation movements have resulted in two points of structural distress. First, the bridge girders are dragging outward on the bearing pads at the abutment. The stress concentrations at this location have resulted in spalling of the concrete at the bottom edge of two of the girders. Reinforcing steel is exposed at the bottom of the outer girder on the north side. The second point of distress is located where the bridge meets the intake tower. On the left side (looking upstream), the bridge has made contact with the tower and a small piece of concrete on the parapet wall has spalled off. This reportedly occurred several years ago and the condition has not changed since. This same movement is suspected to be the cause of wide cracks in the concrete slope paving under the bridge and transverse cracks in the soil cement armoring along each side of this structure.

A substantial effort has been made trying to develop a healthy stand of grass on the downstream slope of the dam for erosion protection. Project staff have done a commendable job to improve the conditions of the slope and sprinkler system over the last two years. Time will tell if the recent work to regrade the slope, repair the sprinkler system, and hydro-seed the slope with Bermuda grass will be successful. If not, the City may want to consider abandoning the maintenance-intensive sprinkler system and explore alternate non-vegetative methods for erosion protection of the slope to reduce the long-term maintenance requirements. The capital cost of alternative slope protection measures would need to be weighed against the costs to continue operating and maintaining the sprinkler system, repairing erosion gullies, fixing damage from feral hogs, seeding repaired areas, and mowing.

The pneumatic instrumentation used to monitor the dam's behavior is reaching the end of its service life. The original six base plate settlement devices no longer function. Several of the piezometers have stopped working. This type of instrumentation is especially critical for monitoring the dam during construction, during initial filling of the reservoir, and for several years thereafter. The instrumentation appears to have provided data sufficient to indicate that the critical elements of the dam, such as the core, slurry trench cutoff wall, and internal drainage systems have performed as anticipated by the design. An analysis of data from pneumatic piezometers that have stopped working should be undertaken to determine if simple open risers should be installed in some of those areas where valuable data were being collected.

A total of 10 drain outlets were installed along the downstream side of the dam to convey water collected by drainage systems inside the embankment. These outlets are monitored monthly and, to date, only three of them are flowing. There is also seepage emerging from the left (looking downstream) abutment near the contact with the embankment in the vicinity of the outlet works stilling basin. Only a portion (roughly half) of this seepage is collected and measured. The total amount of seepage being measured at these four locations has been approximately 10 gallons per minute with the lake at the conservation pool elevation. This amount is very small for a dam of this size, which provides further evidence that critical internal elements are functioning properly.

The six inclinometers and 24 survey monuments on the dam indicate normal consolidation behavior of the embankment. Total settlement to date is just over 8 inches at the maximum point, which was predicted during the design.

The maximum lake level to date was approximately 6 feet above the crest of the service spillway. The spillway and its stilling basin reportedly functioned as anticipated by the design and physical model testing. During the inspection, seepage appeared to be emerging from two transverse construction joints in the 3% chute slab nearest to the spillway crest. These joints are at the locations of two sets of pneumatic piezometers that are not being monitored because the read out box for one (spillway Sta. 11+00) has not been located and the other (spillway Sta. 13+00) no longer functions. A determination on the need for repairing these piezometers or installing new open riser type monitoring wells should be made in light of the seepage observed.

BRA project staff recently installed a continuous floating boom type barrier across the upstream end of the spillway approach channel. Boaters were reportedly getting too close to the structure when it was operating and were at risk of getting swept through by the current. The new barrier system should provide an additional measure of safety. The original buoy and cable system, which was installed closer to the spillway crest, remains in place.

The emergency spillway was designed to operate during extreme floods in excess of the 100-year event. Since completion, small to medium mesquite trees and other woody vegetation and brush have grown throughout the spillway. This vegetation should be cleared to maintain the original design capacity of the spillway.

The present level of operation, maintenance and monitoring being performed by the BRA is appropriate for the size and nature of the project. As the dam and appurtenant structures age, maintenance and repairs to items such as the gates and gate operating system and the various locations of soil cement armoring will likely increase. Continued maintenance and repairs to the sprinkler system and erosion gullies in the downstream slope of the dam are inevitable, and may increase as the system ages. The addition of a water supply pump station in the future will increase the operation and maintenance activities at the project. Changes to the instrumentation-monitoring schedule are recommended, but will not substantially decrease the current level of effort provided by the BRA project staff.

## 4.2 Recommendations

### 4.2.1 Embankment

(1) Visually monitor scarp-shaped crack in upstream soil cement slope protection near Sta. 5+00 and approximately elevation 2228 ft-msl. Monitor monthly while reading piezometers.

(2) Repair erosion holes and cracks in the soil cement with lean concrete in accordance with the project O&M manual.

(3) Spray vegetation on the upstream slope with a suitable herbicide to prevent roots from degrading the soil cement.

(4) Continue efforts to repair erosion gullies and the sprinkler system in the downstream face of the dam. Time will tell if the hydro-seeding currently being performed will be successful. Repair leaky sprinkler heads, if possible, to avoid mistaking wet areas for seepage through the dam. If the leaky heads are not repairable, then mark them with flagging or some other means.

(5) Repair erosion along uphill side of access road that traverses the downstream face of the dam. Form a new drainage ditch and line it using lean concrete with fiber mesh reinforcement in lieu of wire or rebar. Provide several rows of staggered impact blocks across the new concrete ditch near the bottom to dissipate flow energy and prevent scour at the end of the ditch.

(6) Continue efforts to reduce the feral hog population.

(7) Collect all seepage emerging from the left abutment in existing ditch along upstream side of access road and install a V-notch weir across the ditch adjacent to inlet for culvert beneath access road. Monitor the turbidity and flow rate of this seepage monthly.

(8) Clean out sediment and vegetation from in and around the finger drain outlets to make access safer and more convenient.

(9) Correct identification of the finger drain outlet structure at Sta. 30+00.

### 4.2.2 Outlet Works System

(1) Perform internal inspections of intake tower and both outlet conduits, and an underwater inspection of gates on the intake tower a couple years prior to utilizing the lake for water supply.

(2) Repaint metal items exhibiting rust on intake tower.

(3) Replace missing bird screen on intake tower vent hole.

(4) Arrange for a service call to diagnose and correct problem with gate controllers.

(5) Replace rubber seals on 12- and 14-inch sluice gates to reduce leakage.

(6) Repair section of outlet works discharge channel where soil cement lining has failed. Lean concrete with fiber mesh reinforcing could be used in lieu of soil cement. Gabion mattresses may also be a viable alternative if a source of 3- to 5-inch rock is available close to the site.

(7) Clear sediment and vegetation from end of outlet works discharge channel and grade channel to drain towards original river channel.

### 4.2.3 Service Spillway

(1) Determine if water observed at two transverse construction joints below the ogee crest is from seepage through the joints.

(2) Examine vertical construction joint in soil cement at right side slope of approach channel following significant flood events.

(3) Remove sediment and vegetation from spillway stilling basin and discharge channel. Reestablish drainage between end of discharge channel and original river channel.

### 4.2.4 Emergency Spillway

(1) Clear and grub trees and brush growing in channel bottom, and then either burn on site or haul off, as local regulations permit.

### 4.2.5 Instrumentation

(1) A detailed analysis of data from pneumatic piezometers that have stopped working should be undertaken to determine if simple open riser piezometers should be installed in some of those areas where valuable data were being collected. This analysis should also take into account locations of existing seepage.

(2) If water observed at transverse joints in spillway chute is determined to be seepage, consider repairing existing pneumatic piezometers or installing new open risers at spillway stations 11+00 and 13+00.

(3) Continue monthly monitoring of remaining pneumatic piezometers, open risers, and seepage.

(4) Continue quarterly monitoring of surface markers on intake tower, access bridge and its abutment, and service spillway walls. Given the distress to the concrete bridge

girders at the access bridge abutment, consider reading the surface markers monthly. Seek advice from structural engineer.

(5) Change frequency of inclinometer readings from quarterly to yearly.

(6) Retain Clear Fork Surveying to perform another survey of the monuments located throughout the project before the end of 2006, or sooner if dictated by conditions at the access bridge

### 4.2.6 Access and Security

(1) Continue to maintain gravel access road to the project to ensure all-weather access.

(2) If vandalism becomes a problem as use of the public facilities and lake increases, consider installing fake surveillance cameras in selected areas such as the intake tower and spillway bridge.