APPENDIX 1

CONSULTANT'S REPORT ON WATER DEVELOPMENT

USF200002251

REPORT ON WATER DEVELOPMENT IN THE FT. HUACHUCA AREA, ARIZONA

Prepared For

CORPS OF ENGINEERS
DEPARTMENT OF ARMY
LOS ANGELES DISTRICT
LOS ANGELES, CALIFORNIA

Registered Professional Engineer (Mining)
CERTIFICATE EXPIRES
JOHN WILLIAM HARSHBARGER
DEC. 31, 1974
ARIZONA U.S.A.

Registered Geologist
CERTIFICATE EXPIRES
John W. Harshbarger
DEC. 31, 1974
ARIZONA, U. S. A.

March 15, 1974

HARSHBARGER AND ASSOCIATES

CONSULTANTS IN HYDROGEOLOGY

1525 EAST KLEINDALE ROAD
TUCSON, ARIZONA 85719

APPENDIX I

USF200002252

REPORT ON WATER SUPPLY

FORT HUACHUCA AND VICINITY, ARIZONA

A P P E N D I X E S

NO.	TITLE
1	CONSULTANT'S REPORT ON WATER DEVELOPMENT
2	ARIZONA WATER COMMISSION'S STATUS REPORT OF A STUDY OF THE ADEQUACY OF THE WATER SUPPLY OF THE FORT HUACHUCA AREA
3	CONCEPT DESIGN REPORT FOR PROPOSED WATER SYSTEM EXPANSION
4	TEST WELL DRILLING AND STUDY OF HYDROGEOLOGIC CONDITIONS

USF200002253

CONTENTS

Page

Introduction..... 1
Purpose and Scope..... 1
Previous Investigations..... 1
Location and Physical Features..... 2
Hydrogeology..... 4
Geologic Boundaries..... 4
Alluvial Deposits..... 4
Pantano Formation..... 5
Lower Basin Fill..... 5
Upper Basin Fill..... 6
Floodplain Alluvium..... 6
Hydrologic System..... 7
Groundwater..... 9
Perched Mountain Front Groundwater..... 9
Regional Aquifer..... 9
Floodplain Aquifer..... 10
Review and Comment on Digital Model Analysis Prepared by Arizona Water Commission..... 12
Model Input Data..... 13
Coefficients of Transmissivity and Storage..... 13
Natural Recharge and Discharge..... 14
Water Levels and Historic Pumpage..... 15
Model Verification..... 15
Prediction of Future Water Levels..... 16
Summary..... 21
Potential Water Supply..... 23
Availability of Groundwater..... 23
Long-term Effects of Proposed East Range Well Field... 24
Summary and Conclusions..... 30
References..... 33

USF200002254

LIST OF PLATES

Page

Plate 1. Reconnaissance geology, location of selected wells and springs, irrigated areas, and water-level contours, 1968, in the upper San Pedro basin, Arizona. (Adapted from: Roeske and Werrell, Plate 1, 1973, U.S. Geol. Survey)..... in pocket

2. Water level contour map of Fort Huachuca, Arizona..... in pocket

LIST OF TABLES

Table 1. Summary of alternative water demands for total study area (Arizona Water Commission, 1974)..... 18

2. Conditions for digital model runs (Arizona Water Commission, 1974)..... 19

3. Results of digital model simulation runs (Arizona Water Commission, 1974)..... 20

4. Aquifer parameters calculated from pump tests of East Range test wells (from aquifer test data furnished by Arizona Water Commission and Corps of Engineers, 1973)..... 25

5. Summary of predicted drawdown in the proposed East Range well field, 2060..... 27

USF200002255

REPORT ON WATER DEVELOPMENT IN THE FT. HUACHUCA AREA, ARIZONA

INTRODUCTION

PURPOSE AND SCOPE

The purpose of this report is to review technical data collected by other agencies concerning the hydrogeology of the Fort Huachuca area, upper San Pedro River basin, Arizona; and to provide an independent opinion to the United States Army Corps of Engineers concerning the overall availability of additional groundwater supply, the magnitude of possible water development, and the effects of such development on the hydrogeologic system.

No field work was conducted by Harshbarger and Associates for this study. The analysis presented herein is based solely on data submitted to this office by the Corps of Engineers, the Arizona Water Commission, and from published reports.

PREVIOUS INVESTIGATIONS

A comprehensive investigation of the water resources of the Fort Huachuca Military Reservation and adjacent areas was conducted by the United States Geological Survey from 1959 to 1963, and a report describing the geology, hydrology, and availability of water in the area was published by Brown and others, (1966).

USF200002256

A basic data report on the hydrologic conditions in the San Pedro River valley has been prepared by the U.S. Geological Survey and published by the Arizona Water Commission (Roeske and Werrell, 1973).

A consultant's report concerning the water system at Fort Huachuca prepared by Blanton and Co., Architects and Engineers (1973), and a summary report of groundwater supply conditions for Fort Huachuca prepared by the Geology and Concrete Section, Foundation and Materials Branch, Sacramento District of the Corps of Engineers (1970) were provided by the Corps of Engineers. Data pertinent to the test drilling on the East Range of Fort Huachuca Military Reservation are contained in two reports prepared by the Corps of Engineers (1972, 1974).

LOCATION AND PHYSICAL FEATURES

Fort Huachuca is located on the left flank of the San Pedro Basin in Cochise County, southeastern Arizona, about 16 miles north of the international boundary (Plate 1). The San Pedro River flows northward from its headwaters about 25 miles south of the international border in Sonora, Mexico, and enters Arizona just south of Palominas. The area of interest is bounded on the south by the international boundary, on the west by the Huachuca Mountains, on the north by Babocomari River and on the east by the San Pedro River.

The San Pedro River enters the United States at an altitude of 4,275 feet above mean sea level and joins with Babocomari River near Fairbank at an altitude of approximately 3,800 feet. The peaks of the Huachuca Mountains to the west have elevations exceeding 9,000 feet.

The Fort Huachuca Military Reservation is located on the northeastern flank of the Huachuca Mountains between the 4,700 and 4,900 foot elevations. The land surface slopes gently northeastward from the Huachuca Mountains toward

USF200002257

the Babocomari and San Pedro Rivers. The southern and southwestern boundaries of Fort Huachuca follow the crest of the Huachuca Mountains (Plate 2). The western boundary extends from the northern end of the range north to the Babocomari River to near the San Pedro River at Charleston. The irregular eastern and southeastern boundaries are shown on Plate 2.

USF200002258

HYDROGEOLOGY

GEOLOGIC BOUNDARIES

The San Pedro River valley lies in a major structural depression formed by northwest-trending mountain blocks. The Huachuca Mountains comprise a faulted complex of granite, carbonate rocks, conglomerate, and claystone ranging in age from Precambrian to Cretaceous (Brown and others, 1966). These crystalline and indurated rocks form the western boundary of the alluvial basin, and also crop out in the Charleston - Fairbank area along the San Pedro River (Plate 1). Volcanic rocks of Cretaceous and early Tertiary age are believed to comprise the basement rock in the San Pedro River valley. This complex of igneous rocks and indurated sedimentary rocks has been intensely faulted and fractured by tectonic activity which attained a maximum during the Miocene period. Although the numerous fractures and faults provide a limited volume of space for the storage of water, there are no significantly extensive areas that comprise major aquifers for water development. In general, rainfall and snowfall in the mountains produce runoff which infiltrates into the more permeable alluvial materials on the flanks of the valley and along the drainage channels. The mountains receive from two to three times as much precipitation as the valley floor, and water moves down the topographic gradient along numerous fractures. Consequently, large amounts of the groundwater recharged into the alluvial sequence in the San Pedro Basin originates along the mountain front.

ALLUVIAL DEPOSITS

The alluvial deposits in the structural trough provide a porous media for the storage of groundwater. These deposits are a heterogeneous mixture of conglomerate, sandstone, gravel,

USF200002259

sand, silt, and clay. The subdivision of the alluvial deposits, as used in this report, is as follows (Brown and others, 1966): (1) Pantano (?) Formation (Brennan; 1957, 1962); (2) lower basin fill; (3) upper basin fill; and (4) floodplain alluvium. In addition, numerous thin terrace gravel deposits are widely distributed over the area.

PANTANO FORMATION: The Pantano formation comprises a semi-consolidated brownish-red to brownish-gray conglomerate that crops out along the fringes of the Huachuca Mountains from Lyle Canyon on the north to the Barchas Ranch on the east. The matrix of the conglomerate ranges from grit to coarse sandstone; and fragments are pebbles, cobbles, and boulders which include welded tuff, andesite, shale, siltstone, granite, limestone, and quartzite. Bedding is tabular to lenticular, with cross-bedding uncommon. The Pantano is slightly to moderately-well cemented, with corresponding low to moderate permeability.

The Pantano formation was encountered by all of the test wells drilled by the Corps of Engineers (Plate 2), and was completely penetrated by Test Well No. 8. In Test Wells 3, 4, and 5 the Pantano is coarse grained, consisting of sandy gravel and gravelly sand, with small amounts of interstitial clay. In Test Wells 6, 7, 8, and 9, the Pantano is fine to medium grained, angular sand containing small amounts of brown clay (Corps of Engineers, 1974).

LOWER BASIN FILL: The lower basin fill unit was deposited after the development of basin-and-range topography and comprises interbedded brown sandy clay, silty sand, and sandy gravel with scattered cobbles and boulders (Corps of Engineers, 1972). The lower basin fill unconformably overlies the Pantano formation and is about 235 feet thick in the Fort Huachuca well field, but may thicken toward the San Pedro River. Test wells drilled

by the Corps of Engineers penetrated 110 to 200 feet of this unit. Cementation is variable; in a few exposures the entire unit is strongly cemented, but others are weakly cemented. Generally, the gravel beds are poorly sorted, and individual beds or layers contain a mixture of silt, sand, pebbles, cobbles, and boulders. The sandstone beds are also poorly sorted; they range from fine to very coarse grained and contain varying amounts of pebbles. The variability in size, sorting, and degree of cementation of the materials in this unit creates an aquifer with permeability ranging from poor to good. Yields of wells in the Sierra Vista-Fort Huachuca area range from 130 to 900 gpm (gallons per minute) and average 430 gpm (Roeske and Werrell, 1972).

UPPER BASIN FILL: The upper basin fill overlies the lower basin fill and consists of weakly cemented and compacted clay, gravel, sand and silt. The unit is poorly exposed and forms steep slopes where it is capped by well-cemented terrace deposits, but erodes to a badland topography in places not protected by these terrace deposits (Brown and others, 1966). It grades from a very permeable fan gravel near the mouth of streams issuing from the Huachuca Mountains to relatively impermeable silt and limey clay containing several sandstone beds in the central part of the basin near Charleston. The upper unit is as much as 620 feet thick in the Fort Huachuca well field, about 250 feet thick in Test Wells Nos. 3, 4, and 5, and only 10 feet thick along the San Pedro River.

FLOODPLAIN ALLUVIUM: The sediments overlying the upper and lower basin fill consist of Pleistocene and Recent alluvial deposits comprising floodplain, channel-fill, and terrace deposits. The last major period of downcutting and erosion formed the present surface of the floodplains along the upper San Pedro River and its tributaries. Terrace deposits extend from the base of the mountains to the floodplains of the San Pedro

USF200002261

and Babocomari Rivers. These deposits are a poorly-sorted mixture of gravel, sand, and clay and may be as much as 50 to 100 feet thick near the center of the basin. This material is very permeable; but, as much of it overlies the regional water table, it does not provide significant groundwater storage.

Stream alluvium is the youngest terrace deposit and is as much as 50 feet thick along the San Pedro River, where it overlies the lower unit of basin fill and along the Babocomari River, where it rests on the upper unit of basin fill. The alluvium is a very permeable mixture of sand and gravel and forms a productive aquifer along the Babocomari River and parts of the San Pedro River. The stream alluvium of these rivers occupies the lowest topographic position in the basin.

HYDROLOGIC SYSTEM

The ultimate source of all groundwater and surface water is from precipitation on the land surface. The mean annual precipitation is 16.5 inches at Fort Huachuca (elevation 5,000 feet) and 11.4 inches at Fairbank (Green and Sellers, 1964). Annual average precipitation in the Huachuca Mountains is about 26 inches. Precipitation on the land surface infiltrates into the soil, unless the infiltration capacity of the soil is exceeded. If the infiltration capacity of the soil is exceeded, the excess precipitation becomes runoff and flows overland to stream channels. Significant amounts of water infiltrate to the groundwater reservoir in certain parts of the San Pedro basin. The processes by which infiltration occur are : 1) direct infiltration from rainfall into the underlying materials; 2) infiltration from runoff that originates in the mountains and seeps into the alluvial materials adjacent to the mountain front; and 3) infiltration from streamflow along the channel course.

USF200002262

The magnitude of direct infiltration from rainfall is not considered to be significant. The major sources of replenishment to the groundwater reservoir are mountain front recharge and channel infiltration.

The upper and lower basin fill units are recharged along the mountain fronts and along the upper reaches of washes where water flows from the mountain canyons onto the valley slopes. Streamflow in washes originating near the mountain front seldom reaches the San Pedro River, except during torrential summer storms or exceptionally prolonged cyclonic winter precipitation. A portion of the streamflow infiltrates into the permeable material underlying the stream channels and percolates to the groundwater reservoir, but much of the surface flow emerging from the mountain canyons is lost by evapotranspiration. The groundwater reservoir occurs in the subsurface and the water level gradient generally parallels the topographic gradient, except near pumping centers. Groundwater emerges as base flow in the San Pedro River and to a minor extent in the Babocomari River, where it is again subject to evapotranspiration loss.

The floodplain alluvium along the San Pedro River is recharged, in part, by groundwater from the basin fill units. This groundwater is forced to the surface by a groundwater barrier formed by consolidated volcanic and sedimentary rocks which crop out near the Charleston gaging station (Plate 1). Groundwater discharge to the river channel thus maintains a short reach of perennial flow at the gaging station.

An additional source of groundwater input, to the area of interest, is groundwater underflow northward along the axis of the San Pedro River valley. The total underflow has been estimated to be 3,400 acre-feet per year at the international

USF200002263

border and 2,800 acre-feet per year at Palominas (U.S. Geological Survey). The difference of 600 acre-feet is lost through evapotranspiration by crops and phreatophytes.

GROUNDWATER

A water level contour map (Plate 2) of the Fort Huachuca area was prepared, based on data from the Corps of Engineers, the Arizona Water Commission, and Roeske and Werrell (1973). The data indicate the presence of a perched mountain front groundwater body and a regional groundwater body.

PERCHED MOUNTAIN FRONT GROUNDWATER: The perched groundwater body extends northwestward from the vicinity of Carr Canyon toward the Fort Huachuca Military Reservation boundary, and extends northeastward toward the San Pedro River. This occurrence is indicated by unusually high water levels in wells in the area where the steep groundwater gradient is approximately 300 feet per mile. In contrast, the average gradient in the regional groundwater aquifer is about 25 feet per mile (Plate 2).

This perched groundwater body probably receives recharge from streams and springs which drain the adjacent portion of the Huachuca Mountains. Downward leakage from this perched zone recharges the regional groundwater body in limited amounts.

REGIONAL AQUIFER: The regional aquifer comprises the saturated portions of the upper and lower units of basin fill, and the Pantano formation. The water level contour map indicates that water enters these units as mountain front recharge from the Huachuca Mountains and moves in a northeasterly direction toward the Babocomari and San Pedro Rivers (Plate 2). The water level contours comprise prominent lobes pointing in an upstream direction in both of these drainages, indicating that groundwater discharges into the drainage channels. These two rivers act as drains for effluent flow from the groundwater system.

USF200002264

Two significant cones of depression have developed in the area due to pumping in the Fort Huachuca - Sierra Vista area and the Huachuca Vista area. The depression cone in the Fort Huachuca - Sierra Vista area is centered about the military post well field and appears to extend for approximately 4 miles, elongated in a northwest-southeast direction, along the mountain front. The cone of depression is approximately 1.5 miles wide.

The depression cone in the Huachuca Vista area extends approximately 3 miles along the Babocomari River, elongated in a southwest-northeast direction along the river channel. Heavy pumping in the Huachuca Vista area has apparently reversed the direction of groundwater flow along the reach of the Babocomari River for several miles downstream from Huachuca Vista. Groundwater that formerly followed the Babocomari River to the northeast has been diverted to the southwest and enters the depression cone around Huachuca Vista.

A zone of high transmissivity appears to exist in the regional aquifer extending northeast from Test Well No. 3 to Test Well No. 7 and proposed wells F and G (Plate 2). The presence of such a zone is indicated by distortion of the 4100- and 4150-foot water-level contours in an up-gradient (southwestward) direction. Test Well No. 7 appears to tap this zone and exhibits a transmissivity of 70,000 gpd/ft (gallons per day per foot). In contrast, Test Well No. 9, which is located approximately 2 miles southeast of the zone of high transmissivity, exhibited a transmissivity of only 13,000 gpd/ft.

FLOODPLAIN AQUIFER: The water level contours indicate that the floodplain alluvium receives recharge from groundwater underflow in the basin fill deposits of the regional aquifer system, as well as from streamflow. In some reaches, groundwater

USF200002265

is discharged from the stream alluvium and comprises base flow to the Babocomari and San Pedro Rivers (Brown and others, 1966). Yields of wells, producing water from this unit in the Benson-St. David area, range from 200 to 1,800 gpm (gallons per minute) and average 770 gpm. Yields of wells range from 150 to 2,800 gpm and average 1,000 gpm in the Palominas-Hereford area. Part of this water is derived from valley-fill deposits as well as floodplain alluvium (Roeske and Werrell, 1973).

USF200002266

REVIEW AND COMMENT ON DIGITAL MODEL ANALYSIS
PREPARED BY ARIZONA WATER COMMISSION

The Arizona Water Commission has constructed a digital model to evaluate the response of the regional water level in the Fort Huachuca area to future proposed water developments. The model consists of a network or grid or rectangles superimposed on the study area, with appropriate physical aquifer characteristics assigned to each node of the network based on the available data. The digital model is calibrated by adjusting the aquifer parameters and boundary conditions of the model until a known water level configuration existing at a certain time for the aquifer can be simulated with an acceptable magnitude of error. The model is verified for a different time period than that used for calibration, which consists of simulating a known water level configuration resulting from a certain pumpage stress imposed on the aquifer. This verification procedure confirms that the values of aquifer parameters resulting from adjustment to a particular set of pumpage conditions can be extrapolated to other time periods and for various rates of water withdrawal.

When the model is calibrated and verified to simulate known water level configurations in the aquifer with an acceptable level of error, various proposed pumpage schemes can be imposed on the system to predict future water level configurations.

The digital model constructed by the Arizona Water Commission is bounded on the west by the bedrock-alluvial contact of the Huachuca Mountains, and on the east by the perennial San Pedro River. The northern and southern boundaries are north of the Babocomari River and south of the international border, respectively.

USF200002267

The northern and southern boundaries are assumed to be no-flow boundaries, and are arbitrarily located far enough from the pumping centers to insure little or no effect on simulated water level declines.

MODEL INPUT DATA

Input data for the digital model consist of transmissivity and storage coefficient values assigned to each model node, natural recharge and discharge estimates, amount and distribution of pumpage from the aquifer, and initial water level measurements.

COEFFICIENTS OF TRANSMISSIVITY AND STORAGE: Transmissivity data for the digital model were obtained from 16 aquifer tests in the study area (Arizona Water Commission, 1974). Two of these aquifer tests were conducted in the southern part of the study area and 14 tests were conducted in the central part of the study area. No aquifer test data were available from the northern part of the study area. Reliability of data from these aquifer tests varies in accordance with testing conditions and procedures.

The values for transmissivity used in the digital model (February 1974) appear to be of reasonable magnitude, except in the area of the post well field. Transmissivity values assigned to nodes surrounding the post well field are apparently too large, as indicated by the results of simulation runs (Arizona Water Commission, 1974).

Two aquifer tests in the southern and central portion of the study area (Plate 2) provided data adequate to determine the coefficient of storage of the aquifer. These data were supplemented by specific yields calculated from drillers' logs. As a result, storage coefficients of 0.05 and 0.10 were assigned to large areas of the model (Arizona Water Commission, 1974, Figure 4). Due to delayed drainage conditions in the upper

USF200002268

portion of the aquifer, long-term storage coefficients in the regional aquifer probably range from at least 0.10 to as high as 0.15. Recognizing this condition, the Arizona Water Commission assigned a storage coefficient of 0.12 to all model nodes for one set of simulation runs. This value of storage coefficient is believed to be the most reasonable.

NATURAL RECHARGE AND DISCHARGE: Data pertaining to the annual volume of recharge received by the aquifer in the study area has been developed by the U.S. Geological Survey, the Arizona Water Commission, and Harshbarger and Associates. Natural recharge of about 11,500 acre-feet per year is received by the entire study area, primarily along the mountain fronts where streams enter the basin sediments. This recharge was initially distributed to the model based on runoff characteristics along the mountain fronts, and adjusted during the verification process. Mountain-front recharge along the Huachuca Mountains is a portion of this total, and amounts to 3,950 acre-feet which is distributed along the western boundary of the model in the Fort Huachuca area (Arizona Water Commission, 1974).

In addition, groundwater underflow enters the study area from the south. Approximately 3,400 acre-feet per year of underflow enters the study area at the international boundary. The southern boundary of the digital model is a no-flow boundary and does not consider any recharge from this source.

Natural discharge is assumed to equal natural recharge under pre-pumping conditions, and occurs as groundwater discharge to the San Pedro and Babocomari Rivers, underflow out of the study area, and evapotranspiration by phreatophytes along the Babocomari and San Pedro Rivers. In the digital model, this discharge is represented by constant head conditions along the San Pedro River and the lower reach of the Babocomari River. The model adds or subtracts water at these nodes so that the water level elevation remains constant.

USF200002269

WATER LEVELS AND HISTORIC PUMPAGE: An unpublished 1941 U.S. Geological Survey water-level contour map of the study area was used as a basis for the steady-state calibration of the model. This map is assumed to represent pre-pumping steady-state conditions in the aquifer, when natural recharge was balanced by natural discharge.

Historic pumpage for the period 1940 to 1960 was estimated based on population statistics furnished by the Museum Director at Fort Huachuca for the period 1940 to 1954. Water use was calculated based on an average 120 gallons per capita per day. Population statistics for the fort were not available for the period 1954-1967, but pumping records were available for 1960-1970. The average annual water use was computed for the 1960-1967 period and assumed to be the same for the 1954-1960 period. Average annual pumpage data for the 1960-1973 period was calculated from data supplied by the Corps of Engineers.

Greater uncertainty exists concerning non-military pumpage in the study area. Prior to 1954, civilian pumpage in the area was assumed to be negligible. For the period 1954 to 1960, the average annual pumpage was assumed to be one-half of the pumpage for the year 1960. Water use for the period 1960-1973 was obtained from records of water companies serving the area.

MODEL VERIFICATION

The verification procedure comprised testing the ability of the model to simulate changes in the actual water level configuration resulting from pumping. The model was first calibrated by adjusting the aquifer parameters and boundary conditions until the known 1941 water level configuration was approximately reproduced by the model. This water level configuration was then used as the initial condition to simulate the results of groundwater withdrawals for the period 1941-1961 and 1962-1973. Simulated water levels were then compared to

USF200002270

measured water levels, and if the error in simulated water levels was not acceptable, the aquifer parameters used in the model were adjusted. A new set of initial 1941 water levels was then generated to serve as the initial condition for another test run. This process was repeated until the difference between simulated and measured water levels was acceptable.

The 1973 water level simulation, which is based on the final generated set of 1941 water levels, did not agree with measured 1973 water levels. This lack of agreement was caused by inherent imprecision in the modeling process, and the limited data used to construct the 1941 water level contour map (Arizona Water Commission, 1974). In the area of the existing Fort Huachuca well field, the difference between simulated and measured water levels was as much as 30 feet, while along the Babocomari River this difference was as much as 75 feet.

Due to time constraints which limited further experimentation, and because some portions of the simulated 1941 water level configuration were close to the measured configuration, nonsteady state calibration and projection runs were executed using the simulated 1941 water levels. The 1973 water levels simulated on this basis differed as much as 50 feet from measured water levels near the existing post well field. Agreement east of this area was considered good.

Simulation of water level declines since 1961 duplicate the measured decline in the post well field within one to five feet, due to the adequacy of pumpage records for this period (Arizona Water Commission, 1974).

PREDICTION OF FUTURE WATER LEVELS

Simulation of the long term effects on the aquifer in the Fort Huachuca area due to projected demands for groundwater

USF200002271

was based on four alternative population levels for the area for the years 1980 to 2030. Projected water demands were obtained by applying an average daily per capita consumption of 200 gallons to the military population, and 160 gallons to the civilian population. These alternative water demands are summarized in Table 1. Alternatives I and IV were chosen for use in the model in order to bracket the range of possible future water demands in the area. Four model runs were executed using Alternatives I and IV for two different storage coefficient configurations, as shown in Table 2. Pumpage from the civilian sector was distributed according to present patterns of pumpage and population growth, while future pumpage from Fort Huachuca was distributed in the East Range area according to the well field design supplied by the Sacramento District, U.S. Corps of Engineers (January, 1974).

Water level declines in the Fort Huachuca area were simulated for the various alternatives for the period 1974 to 2060. The results of these simulations are summarized in Table 3, and clearly illustrate the importance of the coefficient of storage in predicting long-term drawdown effects. A maximum drawdown of 80 feet is predicted using storage coefficients of 0.05 and 0.10 distributed over the model in run 1; but if a storage coefficient of 0.12 is assigned to all nodes, the predicted drawdown is reduced to 50 feet. This latter value is believed to be more realistic. Maximum drawdown in the area occurs under the conditions of run 3, which utilizes the maximum water demand projection and the storage coefficient configuration of 0.05 and 0.10.

It should be recognized that the drawdowns predicted by the digital model are approximations of the actual drawdown, and are subject to errors within the input data as well as some error inherent in the modeling process. The long-term value of the storage coefficient is of primary importance in this

USF200002272

TABLE 1.--SUMMARY OF ALTERNATIVE WATER DEMANDS FOR TOTAL STUDY AREA, ACRE-FEET PER YEAR (FROM ARIZONA WATER COMMISSION, 1974)

YEAR	1980	2000	2030
Alternative I	7,000	8,600	10,800
Alternative II	5,100	6,600	8,900
Alternative III	8,000	14,700	16,900
Alternative IV	8,000	19,000	21,300

USF200002273

TABLE 2.--CONDITIONS FOR DIGITAL MODEL RUNS
(FROM ARIZONA WATER COMMISSION, 1974)

RUN	WATER DEMAND	STORAGE COEFFICIENT
1	Alternative I	Nodes either 0.05 or 0.10 as shown in Figure 4, Arizona Water Commission report, February, 25, 1974.
2	Alternative I	All nodes 0.12
3	Alternative IV	Nodes either 0.05 or 0.10 as shown in Figure 4, Arizona Water Commission report, February, 25, 1974.
4	Alternative IV	All nodes 0.12

USF200002274

TABLE 3.--RESULTS OF DIGITAL MODEL SIMULATION RUNS
(ARIZONA WATER COMMISSION, 1974)

RUN	COEFFICIENT OF STORAGE	MAXIMUM WATER LEVEL DECLINE, FEET	AVERAGE RATE OF DECLINE, FEET/YEAR
1	0.05 and 0.10	80	0.9
2	0.12	50	0.6
3	0.05 and 0.10	160	1.8
4	0.12	110	1.2

USF200002275

analysis, and the reliability of future simulation runs is directly dependent on the real value of the coefficient of storage for the area.

The results of each simulation run indicate that the volume of groundwater presently discharged from the study area as evapotranspiration and subsurface underflow would be reduced by 20 to 50 percent, depending on the magnitude of pumpage and storage coefficient (Arizona Water Commission, 1974). This water will be diverted toward the pumping centers and will decrease the magnitude of depletion from groundwater storage.

SUMMARY

Verification of the digital model is hampered by a lack of pumping records for the area prior to 1960, and does not duplicate measured 1973 water levels if verification is extended to 1941. The model does duplicate water level declines in the area of the post well field from 1961 to 1973, for which time annual pumpage data are available (Arizona Water Commission, 1974).

The results of four simulation runs indicate that the maximum water level decline in the Fort Huachuca area will range between 50 and 160 feet for Alternatives I and IV (Arizona Water Commission, 1974). The average rate of decline ranged from 0.6 to 1.8 feet per year. The simulation runs illustrate that the value of the storage coefficient in the area is very important in determining the drawdown. Variations up to 50 feet in the predicted drawdown result from relatively small changes in the storage coefficient.

Both the magnitude and rate of water level declines for the period 1974-2060, simulated by the digital model are modest by Arizona standards. With at least 1,000 feet of saturated thickness of aquifer in the proposed East Range well field, the maximum water level decline predicted by the model is considered to be acceptable.

USF200002276

This indicates that the groundwater supply would be adequate to meet the maximum projected demand (Arizona Water Commission, 1974).

When additional pump test data from the area southeast of the military reservation become available, the digital model will be refined. These data will be used to adjust the coefficients of transmissivity and storage assigned to the model in the civilian sector southeast of the fort. Consequently, there should be an improvement of agreement between simulated and measured water levels in certain areas of the model. However, the range of predicted drawdowns in the aquifer will not change significantly.

USF200002277

POTENTIAL WATER SUPPLY

AVAILABILITY OF GROUNDWATER

Groundwater in the northern San Pedro Basin occurs in the Pantano formation, the upper and lower basin fill units, and the floodplain alluvium. Calculations by the U. S. Geological Survey, to estimate the volume of recoverable water in these sediments, were based on estimates of the aquifer thickness and specific yield. Appropriate estimates of specific yield for the various water-bearing zones were based on drill-cuttings, geophysical logs, and aquifer pump tests. Aquifer thickness commonly is determined from geologic well logs, but in the Fort Huachuca area (Plate 2) most wells do not penetrate the complete aquifer thickness. The U. S. Geological Survey estimated the saturated thickness of the aquifer to be approximately 1,000 feet in the San Pedro Basin southeast of Fort Huachuca, but the actual saturated aquifer thickness may be several hundred feet greater.

Calculations by the U. S. Geological Survey indicate about 13 million acre-feet of recoverable groundwater in storage in the aquifer at depths to 1,200 feet in the area north of the international boundary, west of the San Pedro River, east of the Huachuca Mountains, and south of the Babocomari River (Corps of Engineers, 1973).

Under alternative water demand I (Table 1), the total volume of water required in the study area by military and civilian users for the period 1974 to 2060 is estimated to be about 810 thousand acre-feet. The total volume of recoverable water in storage in the study area (Plate 2) is conservatively estimated to be more than 15 times greater than the total projected water demand to the year 2060. Under alternative water demand IV (Table 1), the total volume of water required in the study area by military and civilian

USF200002278

users for the period 1974 to 2060 is estimated to be 1,560,000 acre-feet. The total volume of recoverable water in storage in the study area is conservatively estimated to be more than 8 times greater than this total projected water demand. Only a portion of the total projected water demand will be withdrawn from groundwater storage. Significant amounts of groundwater are presently lost from the area of interest via evapotranspiration by phreatophytes, groundwater discharge to surface flow which leaves the area via the San Pedro River, and subsurface underflow. A decline in the water table by pumpage from wells would result in interception of some of the groundwater lost via evapotranspiration; under this condition, the magnitude of groundwater depleted from storage in the aquifer would be reduced.

LONG-TERM EFFECTS OF PROPOSED EAST RANGE WELL FIELD

The northeastern portion of the Fort Huachuca Military Reservation, bounded on the north by the Babocomari River and on the east by the San Pedro River, is termed the East Range and is the area (Plate 2) proposed for future groundwater development (Corps of Engineers, 1972, 1974). A well field configuration proposed by the Sacramento Office, Corps of Engineers, was reviewed and subsequently modified by Harshbarger and Associates. The modified well field configuration is included in the supplemental report issued by the Corps of Engineers (January 1974) and is shown on Plate 2.

Assessment of the long-term effects of the proposed East Range well field is based on analyses of pumping test data provided by the Sacramento District, Corps of Engineers, for four test wells drilled in the area. Values of the coefficient of transmissivity (T) and storage (S) computed for the aquifer in the area of the proposed well field are shown in Table 4. T ranges from 13,000 gpd/ft to 70,000 gpd/ft. It is believed that the T in the proposed well field will average approximately 60,000 gpd/ft, and this value was used in the prediction analysis.

USF200002279

TABLE 4.--AQUIFER PARAMETERS CALCULATED FROM PUMP TESTS OF EAST RANGE TEST WELLS

(FROM AQUIFER TEST DATA FURNISHED BY ARIZONA WATER COMMISSION AND CORPS OF ENGINEERS, 1973)

TEST WELL	TRANSMISSIVITY (T) GPD/FT*	STORAGE (S) COEFFICIENT
6	45,000	-
7	70,000	-
8	20,000	0.10
9	13,000	-

*Gallons per day per foot of aquifer, at 100 percent gradient

Harshbarger and Associates
1974

USF200002280

Only data from Test Well No. 8 were adequate to define a value of S, which will increase with time as pumping proceeds due to delayed drainage conditions in the aquifer. Values for S of 0.12 and 0.15 were used in the prediction analysis.

In order to ascertain the effects on the aquifer of pumpage from the proposed well field, the theoretical drawdown of the water level in the aquifer was computed for the point of maximum drawdown in the well field, which is hereafter referred to as the centroid. The centroid for the proposed well field is located approximately 0.8 mile north of proposed Well A (Plate 2).

It was assumed that the eleven new wells in the proposed well field will be pumped at an annual average rate of 11 mgd (million gallons per day), or at a continuous rate of approximately 700 gpm. These average annual values (rounded) are based on the estimated water requirement for a post population of 50,000. The drawdown at the centroid of the proposed well field in the year 2060, after 80 years of pumping, is approximately 60 feet (Table 5).

The geometry of the proposed well field indicates that the greatest water level drawdown would occur in proposed Well A, due to interference from the other wells. Drawdown at Well A would be greater than that computed for the centroid of the well field, primarily due to head losses resulting from groundwater entering the well through perforated or louvered casing. The computed drawdown in Well A is based on the assumption that the well would have an efficiency of 60 percent. The calculated drawdown in Well A after 80 years of pumping is approximately 100 feet (Table 5).

Table 5 also compares the results of the model simulation runs with the calculations based on the pumping test data. Estimates of the drawdown in the proposed East Range well field are in the same order of magnitude for the digital model and

USF200002281

TABLE 5.--SUMMARY OF PREDICTED DRAWDOWN IN THE PROPOSED EAST RANGE WELL FIELD, 2060

PUMP TEST ANALYSES:	DRAWDOWN IN WELL FIELD (FEET)	DRAWDOWN IN CRITICAL WELL A (FEET)
S=0.12 Q=12,000 AF/yr T=60,000 gpd/ft	62 /1	105 /1
S=0.15 Q=12,000 AF/yr T=60,000 gpd/ft	60 /1	100 /1
DIGITAL MODEL SIMULATION: /2		
RUN 1, S=0.05 and 0.10	80	-
RUN 2, S=0.12	50	-
RUN 3, S=0.05 and 0.10	160	-
RUN 4, S=0.12	110	-

/1 Military pumpage only

/2 Total pumpage; military plus civilian (Table 1)

USF200002282

the pump test analysis. In view of the character and representation of the aquifer coefficients implicit for each method of the analysis, a comparison of results from the two methods indicates there is good agreement.

Interference from other groundwater users in the area, notably the towns of Sierra Vista and Huachuca Vista, would increase the calculated drawdowns in the proposed well field. Future population increases in these areas probably will parallel growth of the fort, and the existing cones of depression in these two communities will probably continue to expand. Proposed Wells C, E, and J will experience the greatest interference effects with the Huachuca Vista depression cone, and Proposed Well B will experience the greatest interference effect with the Sierra Vista depression cone (Plate 2). The magnitude of these interference effects is difficult to predict due to the uncertainty of future pumping demands in these areas.

Calculations concerning the areal extent of the depression cone that would be created by the pumpage from the proposed East Range well field indicate that several hydrologic conditions may be encountered. Extensive phreatophyte growth along the Babocomari River presently transpires approximately 2,250 acre-feet of groundwater per year (Harshbarger and Associates, in preparation). Significant lowering of the water table in the reach southeast of Huachuca Vista by proposed Wells J and K could divert a large part of this groundwater now lost to evapotranspiration, which in effect would constitute a recharge source for these wells. This recharge boundary would probably stabilize the depression cone that would be created in the area by proposed Wells J and K.

A possible barrier boundary exists approximately 8,000 feet northeast of proposed Well H in section 30 of T. 20 S., R. 21 E., where igneous rocks protrude from beneath the alluvial sediments (Brown and others, 1966). These igneous rocks would function as impermeable barriers to groundwater flow, and their presence would accelerate the expected drawdown in wells located near them.

USF200002283

Proposed Well H is located nearest this suspected boundary, and would probably experience the largest effect. Under previously assumed conditions for a pumping period of 80 years, the expected drawdown in proposed Well H would probably be increased about 5 feet due to this barrier boundary.

It is believed the probable effect of the proposed East Range well field on streamflow in the San Pedro River between Charleston and Fairbank would be insignificant. The available data indicate that after 80 years of pumping under the proposed conditions, the drawdown in the aquifer near the San Pedro River would not be discernible. The San Pedro River and the extensive phreatophyte growth along the river floodplain are sources of potential recharge to the East Range well field. The depression cone developed by the proposed East Range well field might divert some groundwater from the floodplain alluvium that is presently used for transpiration by phreatophytes, but would not cause significant infiltration of water in the channel of the San Pedro River.

USF200002284

SUMMARY AND CONCLUSIONS

Available water level data indicate the presence of a perched mountain front groundwater body and a regional water-table groundwater body. The perched groundwater body occurs in a narrow zone extending northwestward along the mountain front from the vicinity of Carr Canyon toward the Fort Huachuca Military Reservation. The limited extent of the perched groundwater body precludes its consideration as a sustained water source. The regional groundwater body comprises the saturated portions of the Pantano formation, upper and lower units of basin fill, and floodplain alluvium. Water level contours on the regional aquifer indicate that the Babocomari and San Pedro Rivers act as drains for the groundwater system. Two significant depression cones have developed in the regional aquifer centered around the post well field in the Fort Huachuca-Sierra Vista area, and the Huachuca Vista area. A zone of high transmissivity appears to exist in the regional aquifer extending northeast from Test Well No. 3 to Test Well No. 7 and proposed Wells F and G.

Results of the digital model analysis indicate that the drawdown resulting from projected water demands of both military and civilian users in the Fort Huachuca area will range from 50 to 160 feet, depending on the future population level and the storage coefficient assumed for the aquifer. Results of calculations based on aquifer tests in the East Range indicate that the drawdown in the proposed well field for an assumed post population of 50,000 will be about 60 feet after 80 years of pumping (1980-2060), with the maximum drawdown of approximately 100 feet occurring in proposed Well A, assuming an average well efficiency of 60 percent. The digital model incorporates interference effects from projected civilian water demands, which were not considered in the pump test analysis, and indicates a drawdown of 110 feet for the same military population level (50,000) and storage coefficient (S=0.12, run 4, Table 5).

USF200002285

These predicted water level declines are within the same order of magnitude, and in consideration on the character of the input data for each analytical method, the agreement is believed to be reasonably good.

Assessment of the long-term effects of the proposed East Range well field is based on analyses of pumping test data provided by the Sacramento District, Corps of Engineers (Table 4), which indicate that the transmissivity of the aquifer in the East Range well field ranges from 13,000 to 70,000 gpd/ft.

It is estimated that the interference effects between the proposed East Range well field and the existing depression cones around Sierra Vista and Huachuca Vista would probably not exceed 20 feet of drawdown over an 80 year period. The available data indicate that 80 years of pumping in the proposed East Range well field will produce no discernible drawdown in the vicinity of the San Pedro River.

The total volume of recoverable groundwater in storage in the Fort Huachuca area is conservatively estimated to be 15 times larger than the projected water demand under Alternate I (Table 1) for the period 1974 to 2060, and 8 times larger than the projected water demand under Alternate IV (Table 1). A portion of the projected water demand would be satisfied by salvage of groundwater that is presently lost to evapotranspiration and subsurface underflow in the area of interest.

Review of hydrogeologic conditions and available quantitative data in the Fort Huachuca area indicates the following conclusions:

1. Adequate volumes of recoverable groundwater are present in the regional aquifer to satisfy the maxiumum projected water demand. It is conservatively estimated that the volume of recoverable groundwater in storage in the regional aquifer is 8 to 15 times greater than the total projected water demand.

USF200002286

2. Projected water requirements for a military population of 50,000 could be satisfied by construction of the proposed well field on the East Range. Drawdown in the proposed well field after 80 years of pumping would be on the order of 60 to 110 feet.

3. Future interference effects with civilian groundwater users in the area are of acceptable magnitude with a proper management plan. The depression cone developed by the proposed East Range well field would not cause significant infiltration of water in the channel of the San Pedro River.

4. Future refinement of the digital model will improve the agreement between simulated and measured water levels in some areas. The magnitude of predicted drawdown in the regional aquifer will not be significantly affected by these future model refinements.

USF200002287

REFERENCES

Arizona Water Commission, Preliminary report on the water supply of the Fort Huachuca area, Arizona. February 25, 1974. 52 p.

Blanton and Co., Architects and Engineers, Tucson, Arizona, Investigation and recommendations for upgrading the water system at Fort Huachuca, Arizona: Prepared for Department of Army, Sacramento District, Corps of Engineers, February, 1973.

Brennan, D. J., 1957, Geological reconnaissance of Cienega Gap, Pima County, Arizona (abs.): Dissertation Abstracts, v. 17, no. 7, p. 1533.

Brennan, D. J., 1962, Tertiary sedimentary rocks and structures of the Cienega Gap area, Pima County, Arizona: Arizona Geol. Soc. Digest, v. 5, p. 45-57.

Brown, S. G., Davidson, E. S., Kister, L. R., and Thomsen, B. W., 1966, Water resources of Fort Huachuca military reservation, southeastern Arizona: U. S. Geol. Survey Water-Supply Paper 1819-D.

Corps of Engineers, Summary of ground water supply conditions, Fort Huachuca, Arizona. July, 1970.

Corps of Engineers, Sacramento District. Final report, test well drilling and study of hydrogeologic conditions, Fort Huachuca, Cochise County, Arizona. April, 1972.

Corps of Engineers. Interim report on water supply for Fort Huachuca and the surrounding areas. 20 April 1973.

Corps of Engineers, Sacramento District. Supplemental report, test well drilling and study of hydrogeologic conditions, Fort Huachuca, Cochise County, Arizona. January, 1974.

Green, C. R. and Sellers, W. D., 1964, Arizona climate: Univ. Arizona Press, Tucson, 503 p.

Roeske, R. H. and W. L. Werrell, 1973, Hydrologic conditions in the San Pedro River Valley, Arizona, 1971: Arizona Water Commission Bulletin 4. Prepared by the U. S. Geol. Survey.

U. S. Geological Survey, 1973, in Interim report on water supply for Fort Huachuca and the surrounding areas, 20 April, 1973.

USF200002288

APPENDIX 2

ARIZONA WATER COMMISSION'S REPORT ON WATER SUPPLY

USF200002289

Status Report of a Study of the Adequacy of the Water Supply of the Fort Huachuca Area, Arizona

March 18, 1974

Arizona Water Commission
222 North Central Avenue, Suite 800
Phoenix, Arizona 85004

Appendix 2

USF200002290

INTRODUCTION

The water supply for the Fort Huachuca area is primarily obtained from groundwater sources, although part of the Fort's water supply comes from springs in the nearby Huachuca Mountains. Groundwater began to be extensively pumped in the early 1940's for domestic and commercial uses. From 1940 to 1973 a total of more than 34,000 acre-feet of groundwater was removed from storage in the Fort Huachuca area resulting in a maximum water-level decline of almost 60 feet. Annual pumpage varied considerably from year to year during this period. As a result of these declines concern has been expressed about the future of the water supply for the Fort and surrounding area.

The Arizona Water Commission began a study of the groundwater resources of the upper San Pedro River Valley in late 1972 to: (1) determine the adequacy of the water supply for proposed subdivisions, (2) assist local governments in their water resource planning efforts, and (3) provide input data to the State Water Plan. Subsequently, the Los Angeles District, U. S. Corps of Engineers requested the Commission to prepare a special report evaluating the adequacy of Fort Huachuca's water supply based upon the Commission's regional studies. This report was prepared in response to that request.

Purpose and Scope

The purpose of this study was to determine the adequacy of the water supply of the upper San Pedro River Valley in general and of the Fort in particular. Computer groundwater modeling techniques were used to simulate the groundwater basin because this allowed evaluation of long

USF200002291

term effects under a variety of basin conditions and water demands.

This report summarizes the study method, presents results of the model studies in the Fort Huachuca area to date, and presents conclusions regarding the adequacy of the Fort's water supply.

Present Status of the Regional Study

The Arizona Water Commission is continuing the study of the water resources of the upper San Pedro River Valley. Additional data are expected to become available in portions of the study area outside of the Fort Huachuca area which will enable the computer model to be further refined. Additional reports will be prepared covering the entire study area when the model has been refined to the fullest possible extent using all data that can reasonably be expected to become available.

Water Supply Adequacy

Unlike those areas in the United States possessing an adequate surface water supply, the mining of groundwater in Arizona is a commonly accepted practice. Thus, the mining of groundwater in Arizona is considered to be an adequate means of supplying water if the supply so derived falls within normal practice in Arizona.

Unlike some other areas in the United States, the mining of groundwater in many parts of Arizona does not necessarily lead to disaster. In these areas, although the annual renewable supply may be small relative to the demand, the volumes of water in storage are truly vast and, with proper management and use, can provide a sustained yield for extended periods of time. In fact, no water supplies in these areas have failed in nearly thirty years of intensive groundwater mining.

USF200002292

The evaluation of the adequacy of Fort Huachuca's water supply is presented in this report based upon the above guidelines and with due consideration for those aspects unique to the Fort.

Location

The Fort Huachuca area, which includes the Fort itself and the adjacent communities of Sierra Vista and Huachuca City, is located in the San Pedro River Valley in southeastern Arizona. The main Fort facilities and the adjacent communities lie on a gently sloping alluvial plain at the eastern foot of the Huachuca Mountains and at the western edge of the San Pedro River Valley. The area is within the Basin and Range Physiographic Province and the upper San Pedro groundwater basin.

Geohydrology

Runoff from the mountains is the primary source of recharge to the water-bearing sediments in the Fort Huachuca area. Only a small portion of the runoff reaches the groundwater reservoir where it moves eastward downgradient toward the San Pedro River and then northward along the trough of the valley. Some of the groundwater appears as surface flow in the San Pedro River and in the lower portion of the Babocomari River.

The aquifer system is considered to have been in approximate equilibrium prior to 1940, that is, inflow was equal to outflow. Some groundwater was being pumped for domestic and stock purposes but not enough to significantly change the water levels. Increasing pumpage after 1940 resulted in significant water level declines in the Fort area.

USF200002293

conditions on the areas of prime interest within the study area.

The model solves the mathematical equations describing groundwater flow using a finite difference solution technique to analyze the changes that occur between a group of interconnected nodes of finite size that have been assigned appropriate physical characteristics. The node system used for this study is shown on Figure 2 with each square or rectangle representing a node. The model was programmed for water table conditions.

The modeling procedure consists of three steps: (1) collection and preparation of model input data, (2) calibration of model, and (3) projection of future water-level declines.

MODEL INPUT DATA

The first step in modeling procedure is the collection and preparation of data that defines the overall hydrologic system. For this study input data to the model included: (1) transmissivity and storage coefficient values which describe the physical characteristics of the aquifer, (2) natural recharge and discharge, (3) water level elevations, and (4) the amount and distribution of pumpage.

Transmissivity

Transmissivity (T) is the ability of an aquifer to transmit water, and a measure of this ability can be determined from aquifer tests conducted in a pumped well. This study used transmissivity data obtained from 16 aquifer tests; three of these tests were conducted by the Arizona Water Commission staff in 1973, two were conducted by the U. S. Geological Survey in 1958 and 1960, eight were conducted the U.S.

USF200002294

Corps of Engineers in 1971 and 1973, one was conducted by the U. S. Bureau of Reclamation in 1966, and two were conducted by private consulting firms in 1973.

The values of transmissivity from these tests ranged from 10,000 gpd/ft. to 260,000 gpd/ft.[1/] Two tests were conducted in the southern part of the study area and 14 tests in the central part of the study area. No aquifer tests were available from the northern part of the study area. Data from these tests were of quite variable reliability because testing conditions were sometimes less than desirable. The most complete data were from work done by the U. S. Corps of Engineers in the East Range of Fort Huachuca.

Initial values of transmissivity assigned to the model were based on all of these tests, and were subsequently adjusted during the calibration step. The final magnitude and distribution of T values for the Fort Huachuca area as used for this report are shown on Figure 3.

Storage Coefficient

The storage coefficient (S) of an aquifer is the volume of water released from storage in a one square foot vertical column when the water table or piezometric surface declines one foot.

Only two aquifer tests with sufficient data to determine reasonably good values of the storage coefficient were available in the study area. An aquifer test in the southern part of the study area gave an S value of .05 and a test in the central part yielded an S value of .10. These data were supplemented by calculating the specific

---

1/ The T value of 260,000 gpd/ft. as reported in USGS WSP 1819-D is apparently much larger than the actual value because the type of analysis used did not take delayed drainage into account. After personal discussion with the author of the report a T value of 100,000 gpd/ft. was selected as being more realistic.

USF200002295

yield[2] of the water-bearing sediments in the study area from available water-well drillers's logs. These calculations were based on methods used in U. S. Geological Survey Water-Supply Paper 1469. In a water-table aquifer, specific yield is approximately equal to the storage coefficient.

Values ranging generally from 5 to 10 percent were obtained fro specific yield calculations. A set of storage coefficient values based on these calculations were assigned to the model nodes in the Fort Huachuca area as shown on Figure 4 and not changed. However, due to the delayed drainage characteristics of the aquifer, it was felt that after several years of pumping the storage coefficient values could be somewhat higher than those shown on Figure 4. Therefore, another set of storage coefficient values were assumed by assigning an S value of .12 to all nodes in the model. This value was assumed on the basis of average long term storage coefficients determined for nearby alluvial basins. It was felt that using these two sets of storage coefficient values in the model would allow a bracketing of the possible range of storage coefficients and, for purposes of this report, permit evaluation of the impact of this range on the adequacy of water supply.

Natural Recharge and Discharge

Data developed by the U. S. Geological Survey, the consulting firm of Harshbarger and Associates, and by the Water Commission indicated about 11,500 acre-feet of natural recharge to the aquifer is

---

2/ Specific yield is the ratio, in percent, of the volume of water a given mass of saturated rock or soil will yield by gravity to the volume of that mass.

USF200002296

received annually by the entire study area. Recharge to the aquifer occurs primarily along the mountain fronts where streams enter the valley. Natural discharge, which is assumed to equal natural recharge under pre-pumping conditions, occurs as inflow to the San Pedro and Babocomari Rivers, groundwater underflow, and evapotranspiration.

The natural recharge of about 11,500 acre-feet was initially distributed to the model by the Commission based upon estimated run-off conditions along the mountain fronts, and subsequently adjusted during the calibration process. Figure 4 shows the final magnitude and distribution of recharge for the Fort Huachuca area as used in the model.

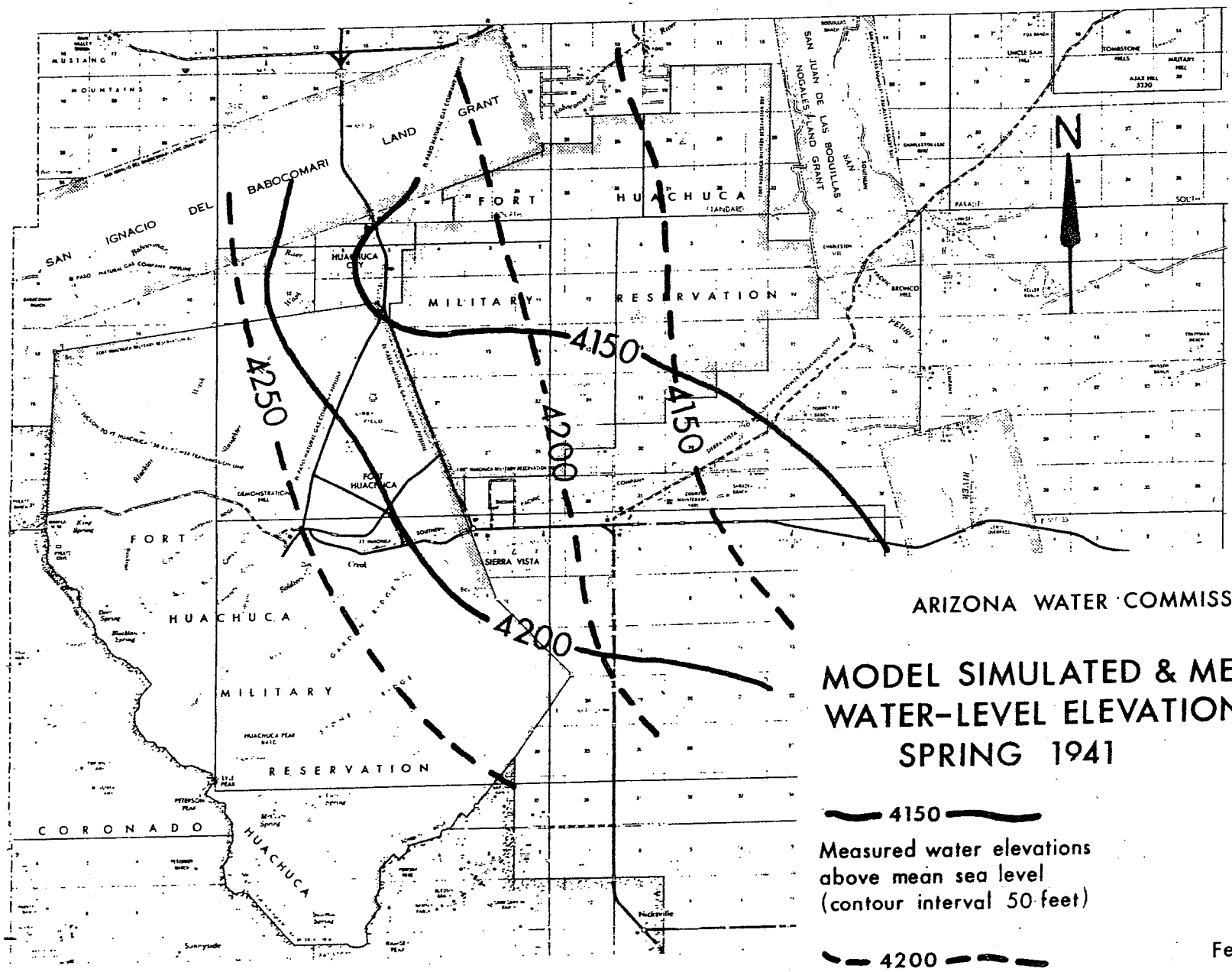
Water-Level Elevations

An unpublished 1941 water-level map of the study area was obtained from the U. S. Geological Survey files in Tucson. The map is based upon limited data but is the earliest map available with depth to water and groundwater elevation information. The groundwater elevations given on this map were based upon old and somewhat inaccurate topographic maps. Using the 1941 depth-to-water data and current topographic maps, the groundwater elevations were recalculated and the water-level elevation contours redrawn as shown on Figure 5. This modified version of the 1941 map was used as a basis for the steady-state or equilibrium condition calibration.

The 1961 water-level map in U. S. Geological Survey Water-Supply Paper 1819-D was used as a check point to provide partial verification during the 1941-1973 period.

In order to update the 1961 map, water-level measurement data

USF200002297



ARIZONA WATER COMMISSION

MODEL SIMULATED & MEASURED WATER-LEVEL ELEVATIONS, SPRING 1941

4150
Measured water elevations above mean sea level (contour interval 50 feet)

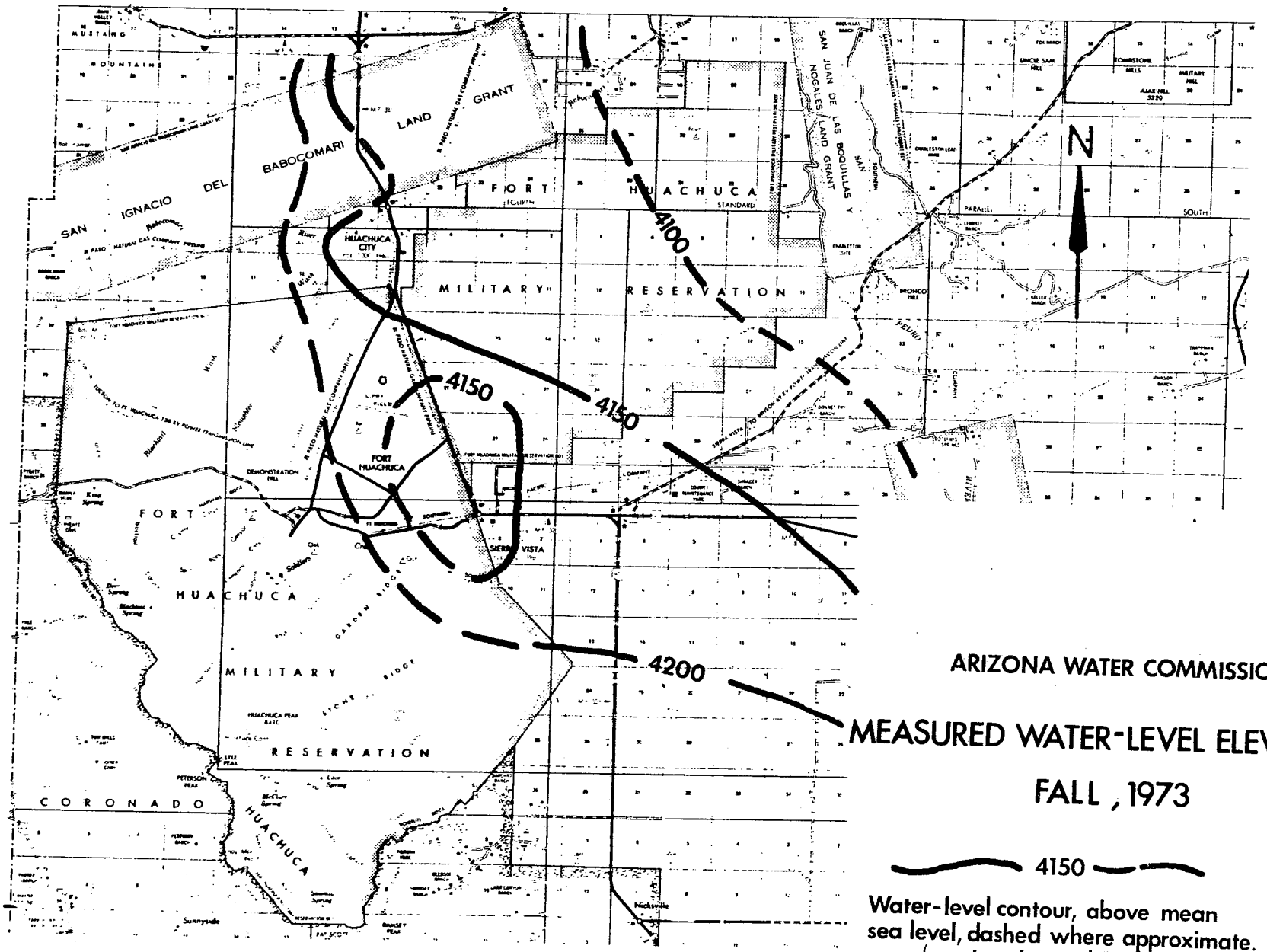
4200
Simulated water elevations above mean sea level (contour interval 50 feet)

Feb. 1974

SOURCE: Measured elevations modified from unpublished U.S.G.S. data

FIGURE 5

USF200002298



ARIZONA WATER COMMISSION

MEASURED WATER-LEVEL ELEVATIONS

FALL, 1973

4150

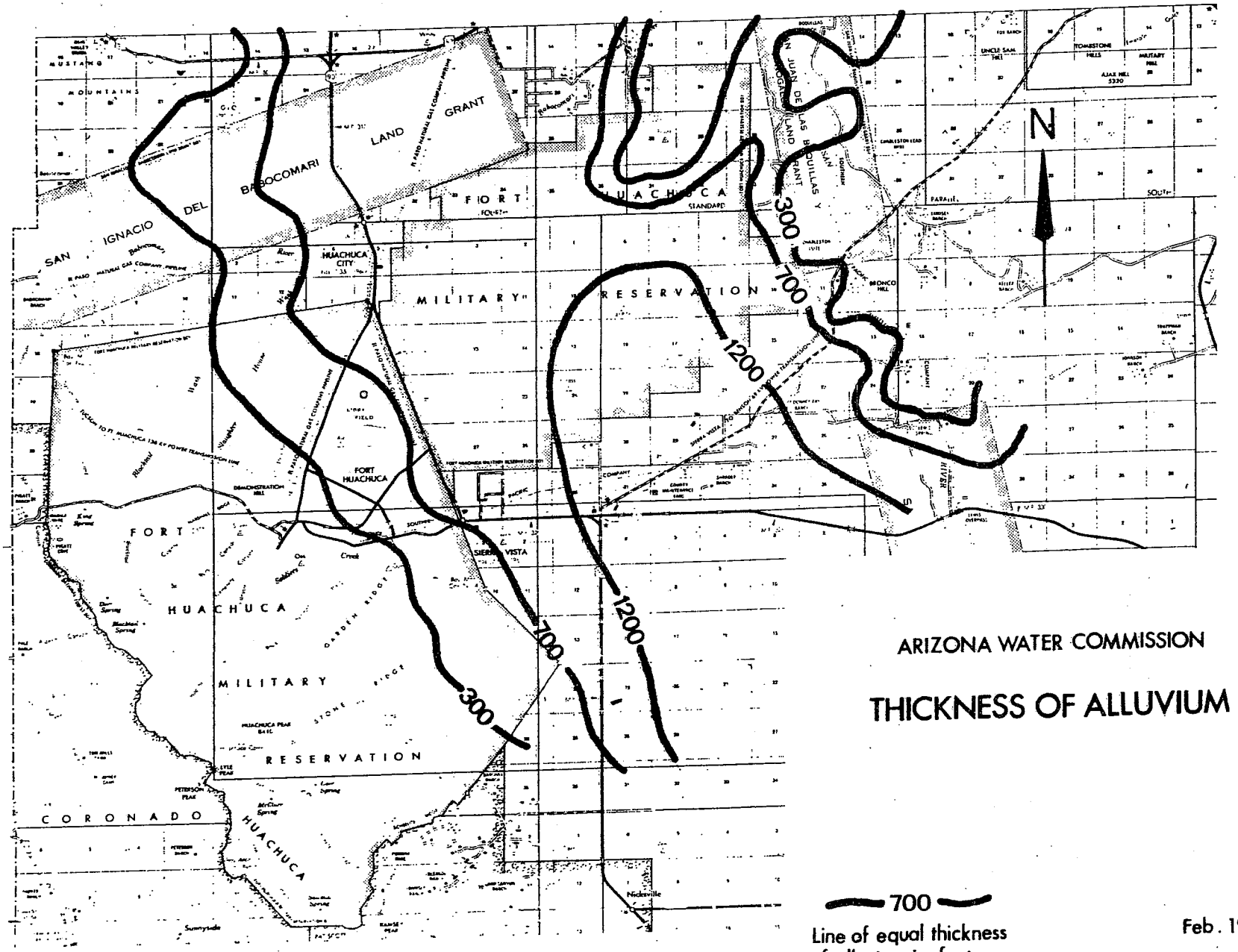
Water-level contour, above mean sea level, dashed where approximate. (contour interval 50 feet)

SOURCE: Arizona Water Commission and U.S. Corps of Engineers.

Feb, 1974

FIGURE 6

USF200002299



ARIZONA WATER COMMISSION

THICKNESS OF ALLUVIUM

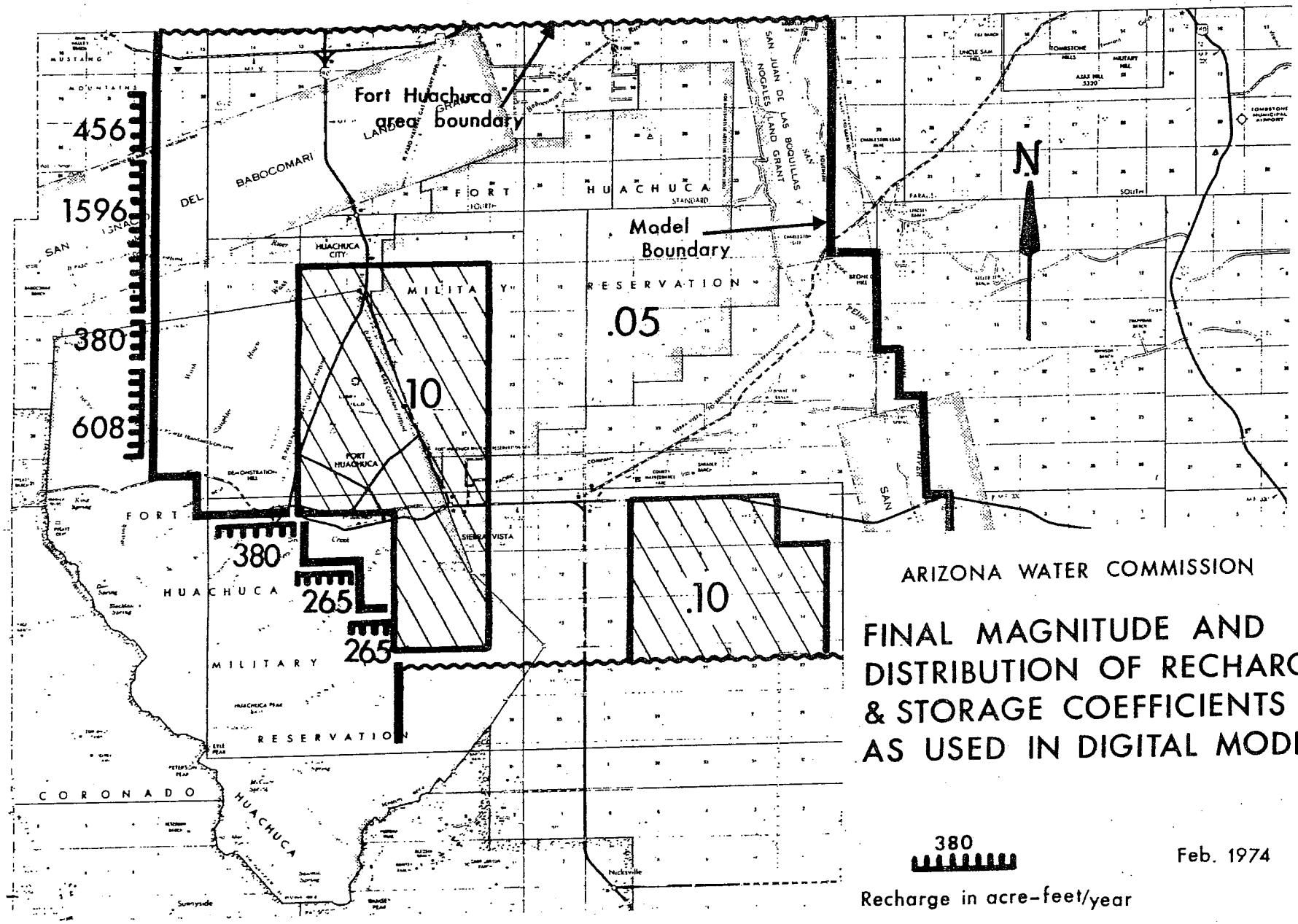
700
Line of equal thickness of alluvium in feet.

Feb. 1974

SOURCE: From unpublished U.S.G.S. data.

FIGURE 1

USF200002300



ARIZONA WATER COMMISSION

FINAL MAGNITUDE AND DISTRIBUTION OF RECHARGE & STORAGE COEFFICIENTS AS USED IN DIGITAL MODEL

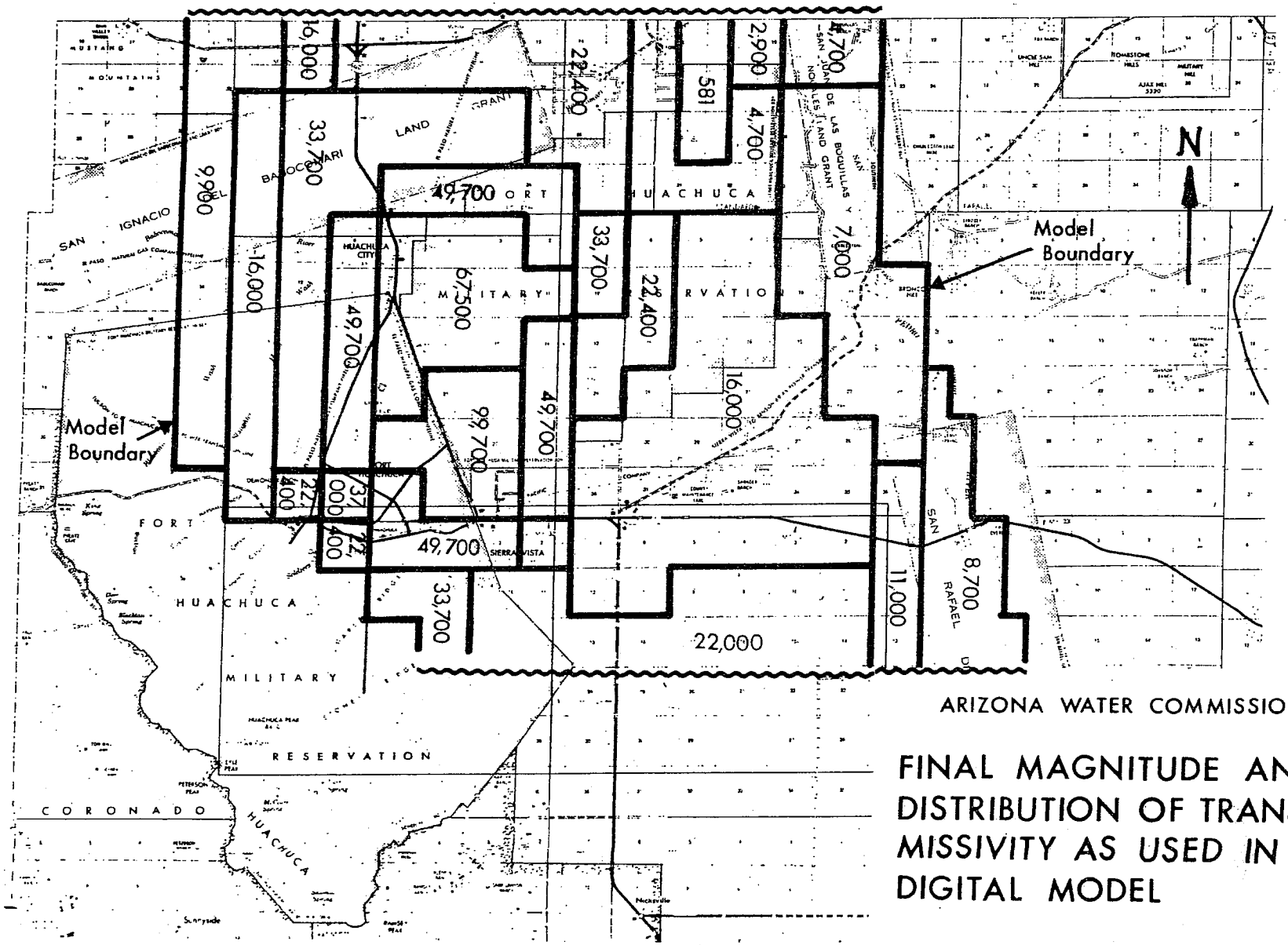
380

Recharge in acre-feet/year

Feb. 1974

FIGURE 4

USF200002301



ARIZONA WATER COMMISSION

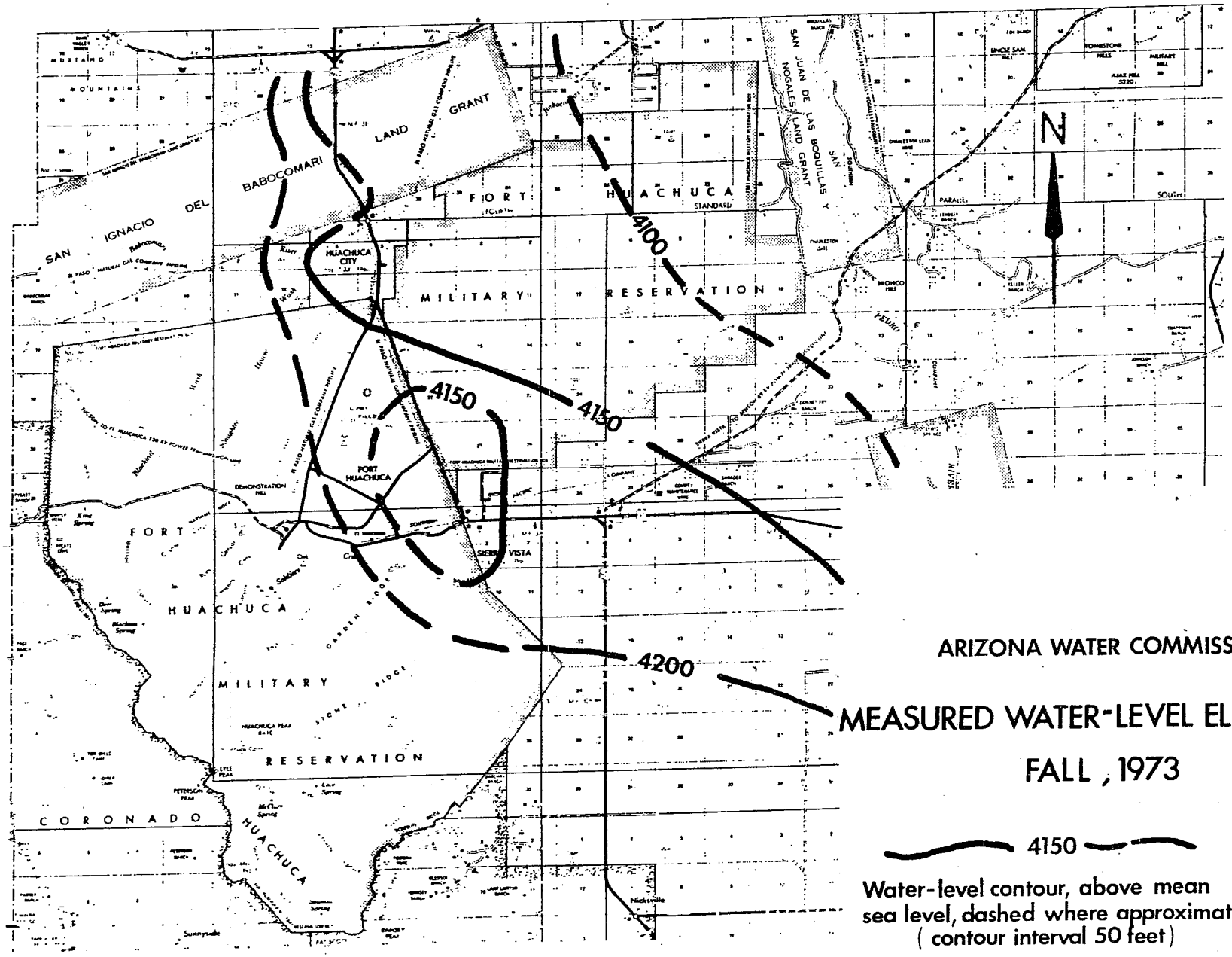
FINAL MAGNITUDE AND DISTRIBUTION OF TRANSMISSIVITY AS USED IN DIGITAL MODEL

NOTE: Transmissivity values are given in gpd/ft.

Feb. 1974

FIGURE 3

USF200002302



ARIZONA WATER COMMISSION

MEASURED WATER-LEVEL ELEVATIONS FALL, 1973

4150

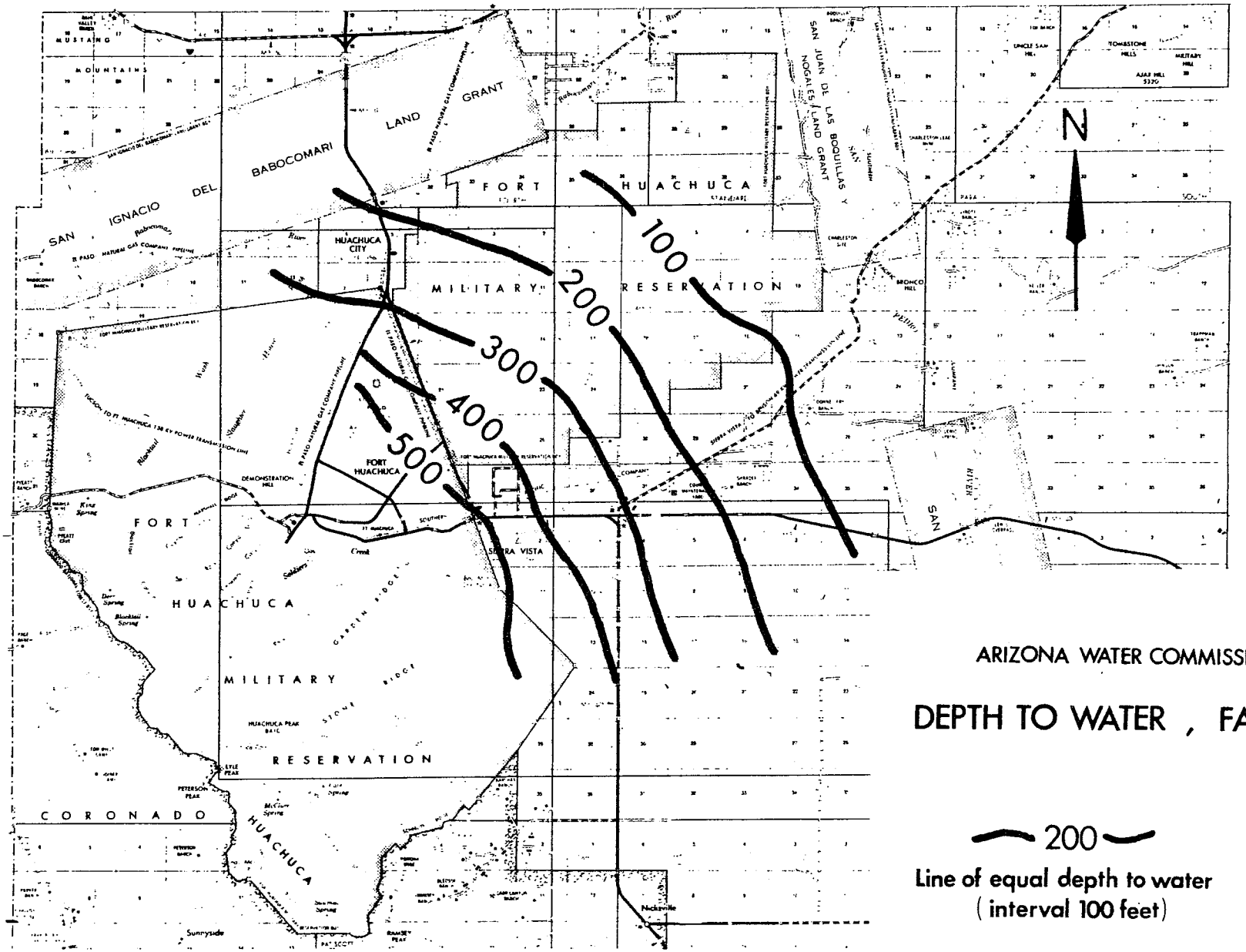
Water-level contour, above mean sea level, dashed where approximate. (contour interval 50 feet)

SOURCE: Arizona Water Commission and U.S. Corps of Engineers.

Feb, 1974

FIGURE 6

USF200002303



ARIZONA WATER COMMISSION

DEPTH TO WATER , FALL 1973

— 200 —

Line of equal depth to water
(interval 100 feet)

Feb. 1974

SOURCE : From Arizona Water Commission
and US Corps of Engineers
1973 water level data

FIGURE 7

USF200002304

C. From 1946 to 1951, the Fort was closed and only a small crew of civilian employees were based on the Fort for fire protection and custodian maintenance.

D. In 1951, the Fort was occupied by the Air Force and later in the year by certain Army units. The combined Air Force-Army strength consisted of 10,300 troops and support personnel for the years 1951 and 1952.

E. Effective May 1, 1953, the Fort was placed in an inactive status. Post population at this time consisted of two officers and fifty civilian personnel.

F. An average annual daily per capita consumption of 120 gallons was used to estimate annual water use at Fort Huachuca in the 1940-1954 period. The 120 gpd/capita was a Water Commission estimate based on the low level of development in the area during this period.

G. From 1954 to 1967, the Fort was designated as the Army Electronic Proving Ground. There were no figures available relating to the number of personnel assigned to the Fort in this period. However, water use data for the 1960 to 1970 period were available. Since the mission of the Fort between 1954 and 1960 was essentially the same as for the period 1960-1967, average annual water use for the 1960-1967 period was computed and applied to the 1954-1960 period.

The following table shows the estimated annual pumpage developed from the above data and assumptions for Fort Huachuca between 1940 and 1960.

USF200002305

Year	Fort Population	Annual Pumpage (AF)
1940	5,500	740
1941	5,500	740
1942	40,500	5,440
1943	40,500	5,440
1944	40,500	5,440
1945	20,250	2,720
1946-50	INACTIVE	UNKNOWN, ASSUMED NEGLIGIBLE
1951	10,300	1,380
1952	10,300	1,380
1953	INACTIVE	UNKNOWN, ASSUMED NEGLIGIBLE
1954-60	NOT AVAILABLE	2,580

Average annual pumpage for the 1960-1973 period was calculated from Fort Huachuca pumpage data provided by the Corps of Engineers and is as follows:

Year	Average Annual Pumpage (AF)
1960-67	2,600
1967-73	3,300

Prior to 1954, water use in the Fort Huachuca area off of the military reservation was considered to be negligible for modeling purposes. Water use data are not available for the 1954-1960 period, so it was assumed that the average pumpage during that period was about one-half of the pumpage for the year 1960. The water use for the 1960-1973 period was developed from available records of the private water companies serving the area. The estimated average annual

USF200002306

pumpage for the 1954-1973 period for the off-post portion of the Fort Huachuca area is shown on the following table:

Year	Average Annual Pumpage (AF)
1954-60	300
1960-68	900
1968-73	1,200

CALIBRATION OF MODEL

Model calibration using historic water levels, recharge, pumpage, and discharge is a prerequisite to use of the model for projection of future conditions. To the extent possible verification was also conducted to determine if the model would adequately simulate future water-level declines in response to projected pumpage. In addition, these steps allow the adjustment and refinement of the data, point out the sensitivity of the model to data inaccuracies, and help determine areas where more data is needed.

The model was structured with a constant-head boundary on the eastern side, by no-flow boundaries on north and south sides, and by a recharge boundary on the western side. The constant-head boundary can be a line source or a line sink depending on what is necessary to hold the prescribed nodes at a predetermined head. The model removes or adds enough water to such nodes that the water elevation remains constant. In the case of this model the San Pedro River was assumed to be a constant-head boundary along the entire eastern boundary of the model. The model nodes along the Babocomari River were also assumed to be constant head from the San Pedro River to a point four

USF200002307

miles west of the confluence. The constant-head nodes are shown on Figure 2.

For modeling purposes the north and south boundaries were assumed to be no-flow boundaries although the aquifer actually extends across both of them. These boundaries were described as such because when the system was in a steady state condition the water-level contours were essentially perpendicular to both boundaries. When pumping begins to cause water-level changes near these boundaries, the contours will not remain perpendicular. In the real system, these changes would induce flow to enter the modeled area from outside. In the case of a no-flow boundary, however, the boundary would create an image well effect causing the simulated drawdowns to be greater than they actually would be.

For this model, it would have been better to have made the north and south boundaries constant-head, but there was not enough data in the boundary areas to adequately define the situation. Instead, the boundaries were located far enough from the pumping areas to insure little or no effect on the simulated well field declines.

The western boundary is a recharge boundary with fixed amounts of recharge assigned to boundary nodes. These amounts were obtained by using the estimates of total recharge and dividing it based on the drainage area feeding a given node and the average rainfall occurring in that drainage area.

The calibration step utilized both steady-state and non-steady-state conditions. The steady-state condition was used to describe the water levels prior to 1940 when the aquifer was assumed to be under natural conditions, i.e., natural recharge is equal to natural discharge and both are constant. In obtaining this condition the storage coefficients for all nodes were set equal to zero. This constraint

USF200002308

forced the model to change water-level elevations in all nodes except constant-head nodes until a configuration was obtained that balanced the inflow with the outflow for all nodes. This simulated steady-state configuration was used as a starting point for the non-steady-state model to determine if the model would duplicate changes that had occurred since 1940. If the non-steady-state model did not close well with the measured water levels the aquifer parameters such as T and recharge were changed in given nodes and new steady-state contours were simulated.

The final simulated 1941 steady-state water-level configuration is shown on Figure 5 along with the measured 1941 water levels. As can be seen, the simulated levels do not match the measured levels. Part of the lack of fit is caused by inaccuracies in the model itself while part of the problem is due to the limited well data used to draw the 1941 contour map. It is also possible that some of the wells along the Babocomari River used for contouring purposes were not representative of the regional water table, but were actually representing perched water tables. In addition, the lack of precise ground surface elevations at individual well sites introduces an error of about five feet because of the necessity of interpolating surface elevations from a topographic map. The simulation of a water-level configuration with the same shape as the measured one was also made difficult by a finite amount of computer storage which limited the number of nodes in the model. In order to locate the northern boundary of the model a sufficient distance from the Fort to make boundary effects negligible and still keep the width of a nodal column in the

USF200002309

Fort area to a maximum of 1.5 miles, it was necessary to make nodes north of the Fort Huachuca area as wide as 3 miles. These large nodes, especially when they are constant head, make it difficult to perform fine adjustments to the model. Table 1 compares simulated water-level elevations at various nodes around the Fort with measured levels. The maximum difference in these nodes is 30 feet, which is quite close considering the probable accuracy of the measured 1941 levels. The difference between the two levels may be as much as 75 feet in nodes along the Babocomari River indicating that the parameters are in error there, that the nodes are too large, or to a combination of these. For modeling purposes, the area of the Babocomari River was assumed to have a low transmissivity; however, after reviewing data that the Bureau of Reclamation obtained from drill holes at Charleston Dam site, it appears that the T may be greater than presently assumed. In addition, the Babocomari River may operate as a constant-head boundary approximately a mile farther west than assumed at this stage.

The simulated 1941 configuration as shown on Figure 5 was used in this report because of time constraints which limited further experimentation. Because portions of the configuration were close to the actual, it was decided to continue with the non-steady-state calibration and the projection runs. When time permits further changes will be made to the model. These changes include decreasing the size of the nodes in the northern part of the study areas now that additional computer storage has become available, incorporating additional data which will be collected later in the year, and experimenting with changes in transmissivity values and constant-head elevations along

USF200002310

the lower reach of the Babocomari River. A better match between measured and simulated 1941 water levels may be obtained as a result of these model changes.

Table 1

Comparison of simulated 1941 water-level elevations with the measured levels (in feet above mean sea level) at selected nodes. 3/

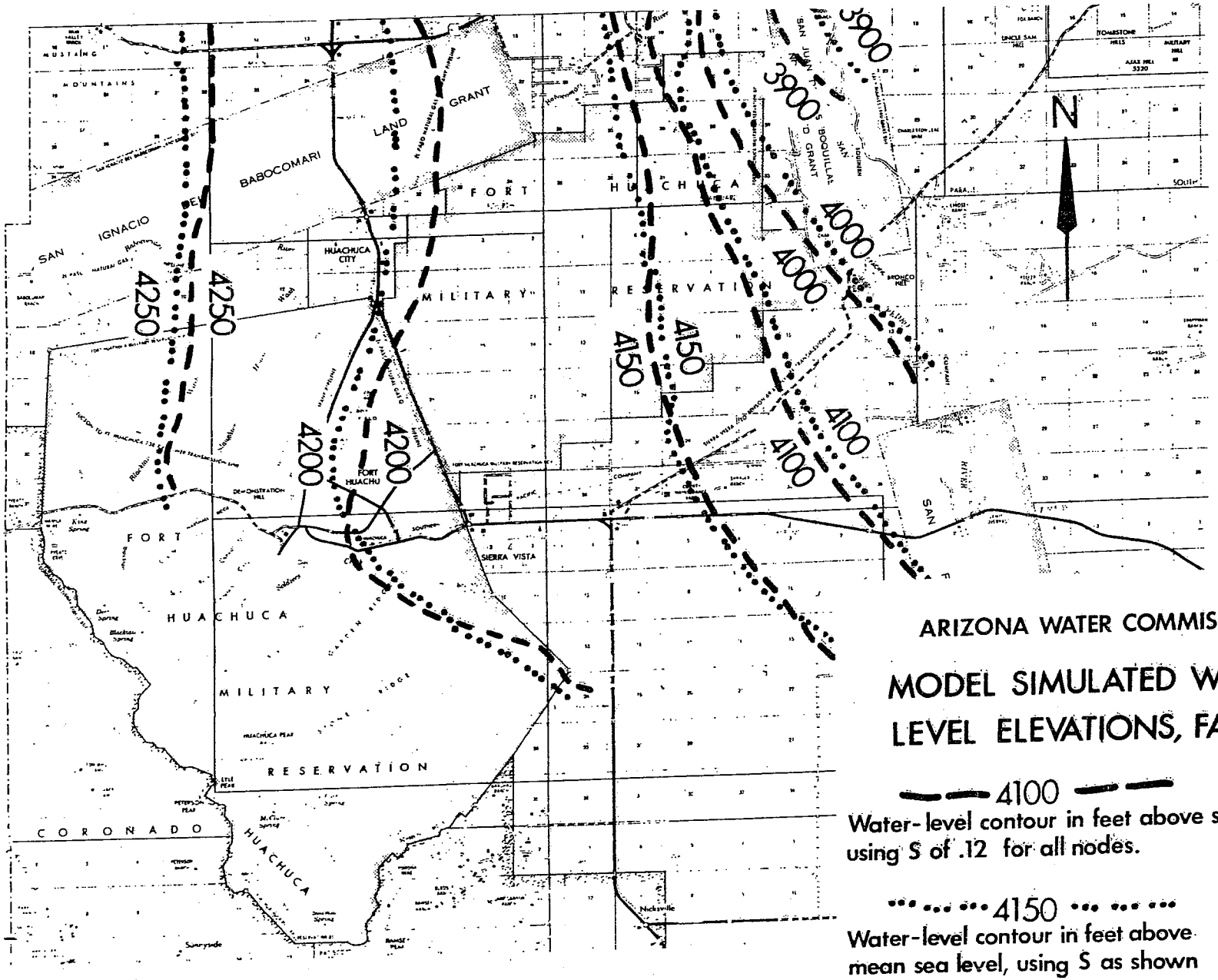
node		simulated	measured
row	column	1941 water level	1941 water level
13	13	4180	4175
13	14	4185	4185
14	13	4195	4175
14	14	4200	4185
15	13	4210	4180
15	14	4210	4190
16	13	4210	4185
16	14	4215	4195
17	13	4215	4195
17	14	4220	4190

3/ Rounded to nearest five feet.

USF200002311

Starting with the simulated 1941 water-level values with the system in a steady-state condition, the effects of pumpage within the modeled area were simulated for the periods 1941 to 1961 and 1962 to 1973. These periods were delineated because the pumping data prior to 1961 were based entirely on estimates while those after 1961 were based on existing records of pumpage. The simulated 1961 water levels were looked at but were not compared to the measured 1961 water levels node by node because the 1941 levels used as a base were in error, and the pumpage estimates were questionable. It was felt that if the simulated water levels did compare to the measured it would be a coincidence. The simulated 1961 water levels were used to calculate the simulated changes between 1962 and 1973 which were used as the basis for determining how well the model was calibrated. Two different storage coefficient configurations were used to test the sensitivity of the model output to variations in these parameters. Figure 8 shows the simulated 1973 water levels obtained from these two runs and, as can be seen, there is little difference in resulting water levels. Table 2 compares the measured 1973 water levels with the simulated 1973 levels. The agreement is good east of the Fort but differences became larger near the Fort with as much as a 50 foot difference in water levels. The difference between the two simulated water levels was only five feet in all nodes which indicates that using two different storage coefficient configurations has not significantly effected the water levels up to 1973. However, the difference between these two simulated water levels will become larger at a given node as pumpage continues into the future.

USF200002312



ARIZONA WATER COMMISSION

MODEL SIMULATED WATER – LEVEL ELEVATIONS, FALL 1973

— — 4100 — —
Water-level contour in feet above sea level, using S of .12 for all nodes.

••• ••• ••• 4150 ••• ••• •••
Water-level contour in feet above mean sea level, using S as shown on Figure 4

Feb, 1974

FIGURE 8

USF200002313

Table 2

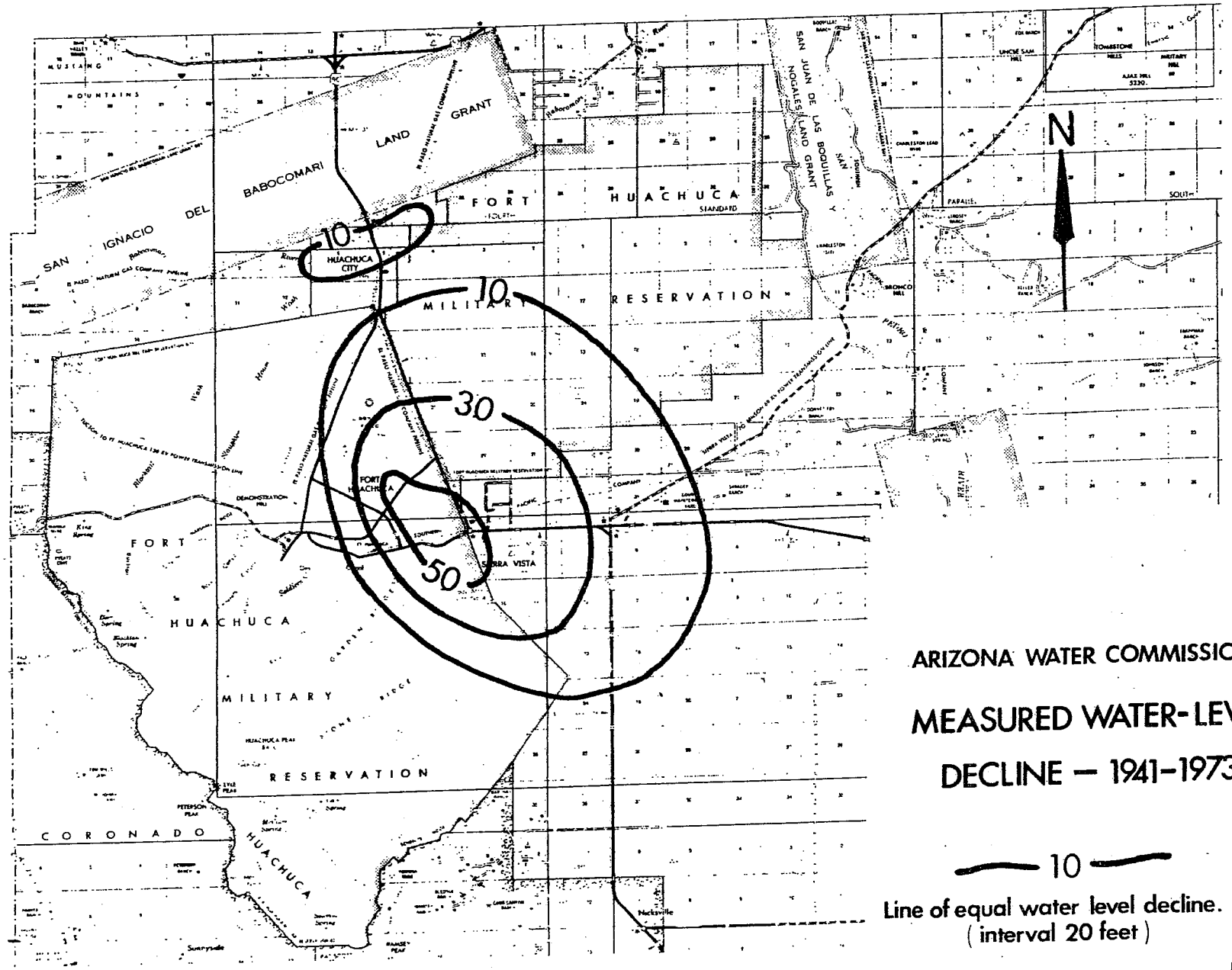
Comparison of measured 1973 water-levels with simulated levels in feet above mean sea level. 4/

node		measured 1973 water-level	simulated 1973 water levels	
Row	Column		storage coef. as on Fig. 4	storage coef. all - 0.12
13	13	4160	4165	4170
13	14	4165	4170	4175
14	13	4165	4175	4180
14	14	4170	4175	4185
15	13	4160	4175	4185
15	14	4165	4180	4185
16	13	4135	4180	4185
16	14	4140	4185	4190
17	13	4150	4180	4185
17	14	4145	4180	4185

The measured water-level changes from 1941 to 1973 (Figure 9) were used as a check on the closeness of calibration. The actual maximum declines in the center of the Fort's well field are 20-25 feet greater than the simulated declines shown on Figures 10 and 11, while the declines a few miles from the Fort compare quite well. The difference in the center of the well field could have been caused by too low a pumpage estimate for the period between 1940 and 1961 or by

4/ Rounded to nearest five feet.

USF200002314



ARIZONA WATER COMMISSION

MEASURED WATER-LEVEL DECLINE — 1941-1973

10

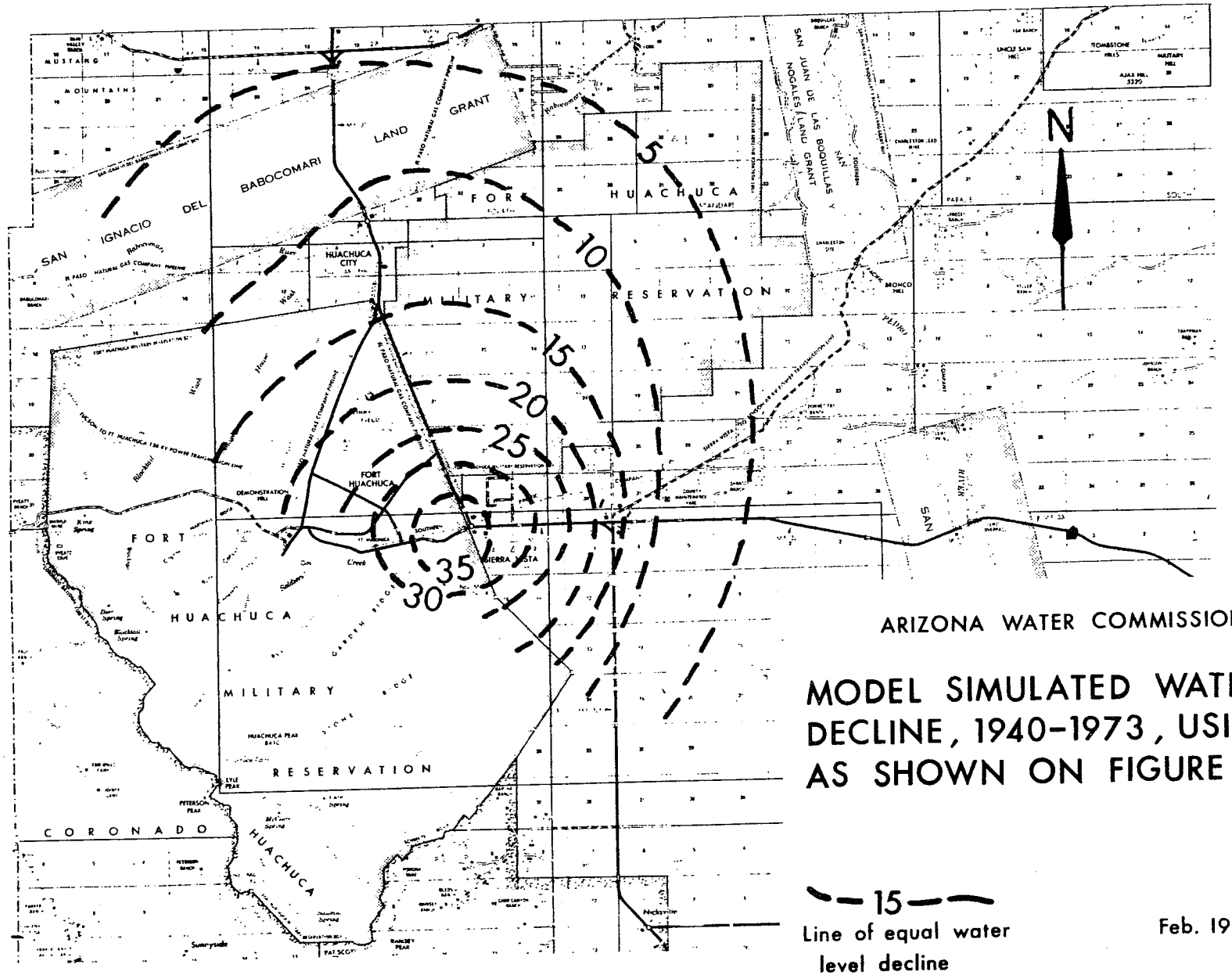
Line of equal water level decline.
(interval 20 feet)

Feb. 1974

SOURCE : By AWC from 1941 - 1973 water - level measurements.

FIGURE 9

USF200002315



ARIZONA WATER COMMISSION

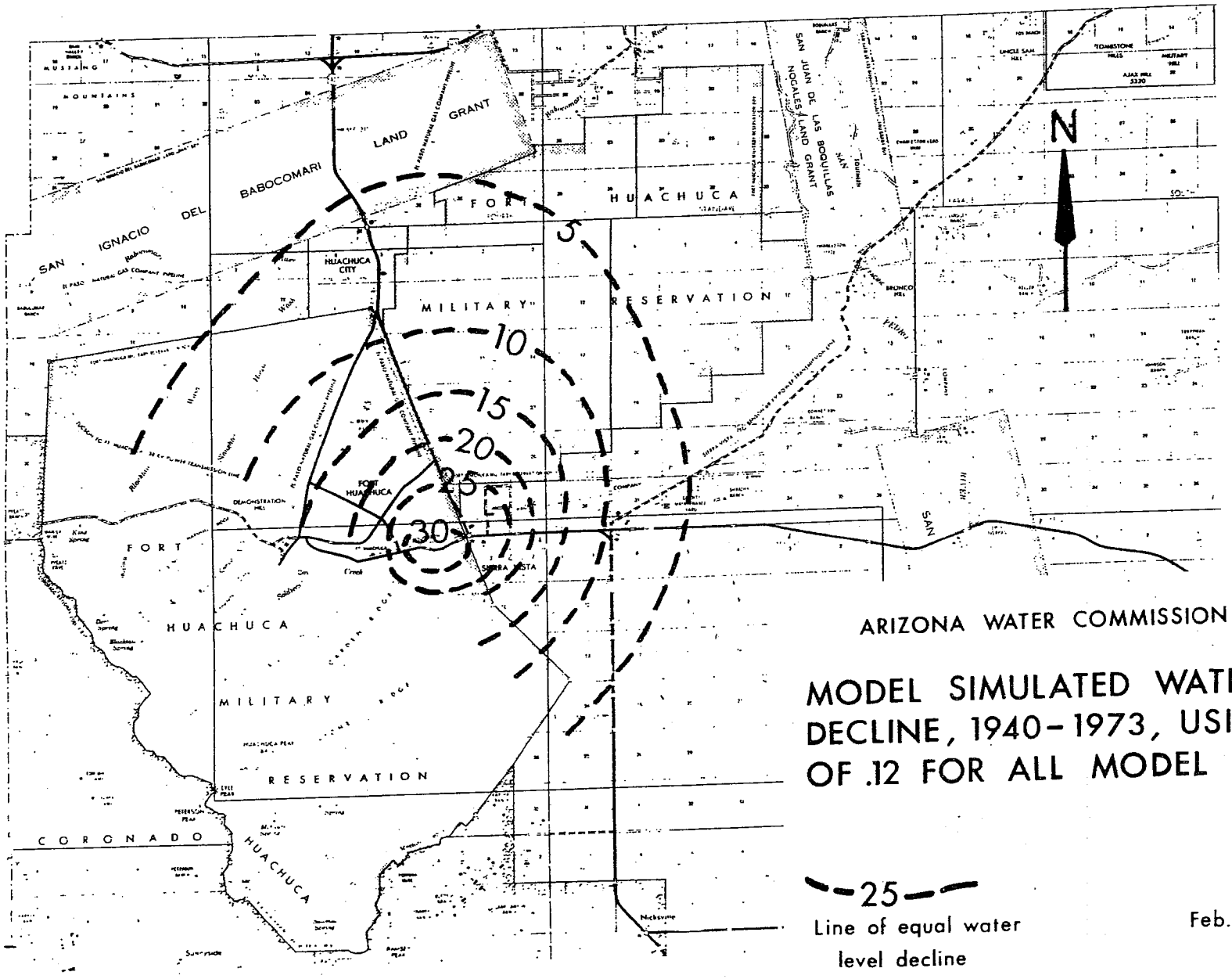
MODEL SIMULATED WATER LEVEL DECLINE, 1940-1973, USING S AS SHOWN ON FIGURE 4

15
Line of equal water level decline (interval 5 feet)

Feb. 1974

FIGURE 10

USF200002316



ARIZONA WATER COMMISSION

MODEL SIMULATED WATER LEVEL DECLINE, 1940-1973, USING S OF .12 FOR ALL MODEL NODES

25 Line of equal water level decline (interval 5 feet)

Feb. 1974

FIGURE 11

USF200002317

too high a transmissivity being used for the present well field area as indicated by the presence of the large cone of influence and the reversal of gradient that has occurred. In addition, the coarseness of node size could have resulted in the center of the well field not having simulated values as large as those observed. When historic pumpage is assigned to a model node, it is distributed over the entire area of the node. The well field falls in three nodes, thus the model operated as if the well field had an area of three square miles when, in actuality, all of the wells fall within an area that is slightly larger than one square mile.

Table 3 compares the average measured water-level declines with the average simulated declines for some selected nodes in the Fort's well field for 1941-1973. Table 4 compares the declines from 1961 to 1973 for the same nodes.

Table 3

Comparison of measured water-level declines and simulated declines in feet for selected nodes for the period 1941-1973.

node		average measured decline 1941 - 1973	Simulated average decline 1941-1973	
row	column		storage coef. as on Fig. 4	storage coef. all - 0.12
16	11	30	22	16
16	12	40	27	21
16	13	45	32	25
16	14	50	32	25
17	11	32	23	17
17	12	42	29	23
17	13	53	35	29
17	14	52	38	31

USF200002318

Table 4

Comparison of measured water-level declines and simulated declines in feet for selected nodes for the period 1961-1973.

node		average measured decline 1961 - 1973	Simulated average decline 1961-1973	
row	column		storage coef. as on Fig. 4	storage coef. all - 0.12
16	11	10	10	7
16	12	20	13	10
16	13	15	15	12
16	14	10	15	11
17	11	15	11	9
17	12	20	13	10
17	13	15	14	12
17	14	15	16	12

The preceding two tables indicate that the model is calibrated better for the 1961-1973 period than for the 1941-1973 period. This would seem to indicate that the estimated pumpages between 1940 and 1961 at the Fort itself are not as large as actually occurred. Also, the transmissivity values that were assigned to the nodes surrounding the well field were apparently too high as indicated by the fact that simulated water-level declines are less than the measured declines for 1961 to 1973 for nodes on the edge of the well field.

In summary, it has not been possible to date to closely duplicate the measured 1941 and 1973 water levels with model simulated water levels. The 1973 measured and simulated water levels deviate by as much as 40 feet. This deviation is due the difference between the

USF200002319

measured and simulated 1941 water levels, the poor estimate of pumpage between 1941 and 1961, and to errors in the hydrologic parameters used. However, it has been possible to duplicate within a few feet the measured 1961 to 1973 water-level declines with model simulated declines in the Fort Huachuca area. This indicates that the storage coefficients modeled reflect the actual values fairly well. Therefore, some confidence can be placed in the model to predict at least relative changes in water levels in response to future pumpage.

PROJECTION OF FUTURE WATER-LEVEL DECLINES

Projection of the long term effects on the groundwater reservoir in the Fort Huachuca area resulting from future demands for groundwater was the last step in the simulation modeling procedure. This section of the report provides the basis for estimating four possible levels of future groundwater pumpage, considers the impact of the Charleston Dam Project on future water demands, describes the projection runs using two pumpage estimates, and presents the results of the projections runs.

Future Pumpage

Four alternative population levels for the Fort Huachuca area for the 1980-2030 period were considered. Alternative I is based upon the 1972 OBERS[5/] population projections for water resources subarea 1504.[6/] The OBERS projections were disaggregated to the county level and subsequently to the study area based upon the 1970 census distribution of population as follows:

5/ Office of Business Economics (OBE), U.S. Department of Commerce and the Economic Research Service (ERS), U.S. Department of Agriculture.

6/ A multi-county delineation consisting of Cochise, Santa Cruz, Pima and Pinal Counties.

USF200002320

	1970	1980	2000	2030
Sub-area 1504	----	577,300	743,600	928,600
Cochise County	61,900	69,300	88,500	105,900
Study Area	22,000	33,500	42,500	55,000

The projected population data in Alternative I for Fort Huachuca assumes a projected post population based on the present DA stationing plan resulting in a night-time population of 20,500 persons.

Alternative I

	1980	2000	2030
Fort Huachuca Area	30,600	31,200	31,800
Fort Reservation Only	20,500	20,500	20,500
Remainder of Study Area	2,900	11,300	23,200
Total Study Area	33,500	42,500	55,000

The remaining three alternatives used population levels at Fort Huachuca that were specified by the Corps of Engineers in order to assess the effects of both lower and higher levels of demand than Alternative I on the aquifer system, and are as follows:

Alternative II

	1980	2000	2030
Fort Huachuca Area	22,100	22,700	23,300
Fort Reservation Only	12,000	12,000	12,000
Remainder of Study Area	2,900	11,300	23,200
Total Study Area	25,000	34,000	46,500

USF200002321

Alternative III

	1980	2000	2030
Fort Huachuca Area	35,100	51,800	52,200
Fort Reservation Only	25,000	40,000	40,000
Remainder of Study Area	2,900	20,200	31,800
Total Study Area	38,000	72,000	84,000

Alternative IV

	1980	2000	2030
Fort Huachuca Area	35,100	61,800	62,200
Fort Reservation Only	25,000	50,000	50,000
Remainder of Study Area	2,900	32,200	44,300
Total Study Area	38,000	94,000	106,500

Projected annual water demands for Fort Huachuca and the remainder of the study area were developed by applying average daily per capita water use to the projected population levels. Projected water demands for Fort Huachuca were computed on the basis of an average daily per capita consumption of 200 gallons and the assumption that golf course and parade ground irrigation would be accomplished using sewage effluent. All other projected water demands in the study area are based upon an average daily per capita consumption of 160 gallons. The daily consumption levels at Fort Huachuca are reflective of the historically higher water consumption levels experienced at the Fort relative to the surrounding area. A summary of estimated annual water use in acre-feet for each alternative level of population in the study area are as follows:

USF200002322

USF20000232 3

Alternative I

	1980	2000	2030
Fort Huachuca Area	6,400	6,500	6,600
Fort Reservation Only	4,600	4,600	4,600
Remainder of Study Area	500	2,100	4,300
Total Study Area	7,000	8,600	10,800

Alternative II

	1980	2000	2030
Fort Huachuca Area	4,500	4,600	4,600
Fort Reservation Only	2,700	2,700	2,700
Remainder of Study Area	500	2,000	4,300
Total Study Area	5,100	6,600	8,900

Alternative III

	1980	2000	2030
Fort Huachuca Area	7,500	11,000	11,000
Fort Reservation Only	5,600	9,000	9,000
Remainder of Study Area	500	3,700	5,900
Total Study Area	8,000	14,700	16,900

Alternative IV

	1980	2000	2030
Fort Huachuca Area	7,500	13,300	13,300
Fort Reservation Only	5,600	11,200	11,200
Remainder of Study Area	500	5,800	8,000
Total Study Area	8,000	19,000	21,300

Charleston Dam Project

The Charleston Dam project is a feature of the authorized Central Arizona Project, and plans call for its construction concurrently with the main portion of the project. The Dam will be located just downstream from the old townsite of Charleston. Studies by the Bureau of Reclamation indicate the project will provide an average of 12,000 acre-feet per year of new water yield.

Studies by the Arizona Water Commission indicate that when Charleston Dam is finished it could supply all the projected demands in the Fort Huachuca area except for Alternative IV. In the case of this level of population, all but approximately 1000 acre-feet/year could be supplied by the project.

Projection Runs

For this report, the alternatives that were modeled were chosen to bracket the possible future populations. Alternative I was used because present growth rates indicate that it is the most probable population projection. Alternative IV was used because it has the highest level of development at the Fort that the Army considered possible, and would have the greatest impact on the groundwater reservoir of any of the alternatives considered.

Because of the possible range in the distribution and magnitude of storage coefficients in the modeled area, it was decided to use two sets of S values for each alternative as explained earlier. All other model parameters were not varied because there is more certainty as to their value and because the variation of S has the most direct

USF200002324

effect on long-term drawdowns in a situation where groundwater is being mined. For example, if there was no inflow to or outflow from a basin, the average drawdown would be decreased by one-half if the storage coefficient were doubled.

The model runs made and the assumptions used for each are as follows:

Run	Population Level	Storage Coefficient
1	Alternative I	Nodes either 0.05 or 0.10 as shown in figure 4
2	Alternative I	All nodes at 0.12
3	Alternative IV	Nodes either 0.05 or 0.10 as shown in figure 4
4	Alternative IV	All nodes at 0.12

It was assumed that any increase of personnel at the Fort would not take place before 1980. For this reason, all runs assume the same annual pumpage for the first seven years of the projection (1974-1980) as used for 1968-1973. For the remaining three periods, (1980-2000, 2000-2030, and 2030-2060), the average pumpage for the period was assumed to be the same as the pumpage at the beginning of the period. This assumption requires about seven percent less water than would be pumped if the average of the beginning and ending values for each period had been used.

The total water demands of each alternative were distributed within the model to selected nodes using present patterns of pumpage and population growth as a guide, except in the case of the Fort itself. The pumpage from the Fort was assigned using a schedule distributing

USF200002325

the pumpage between the existing well field and the proposed East Range well field as supplied by the Sacramento District, U. S. Corps of Engineers.

Results of Simulation Runs

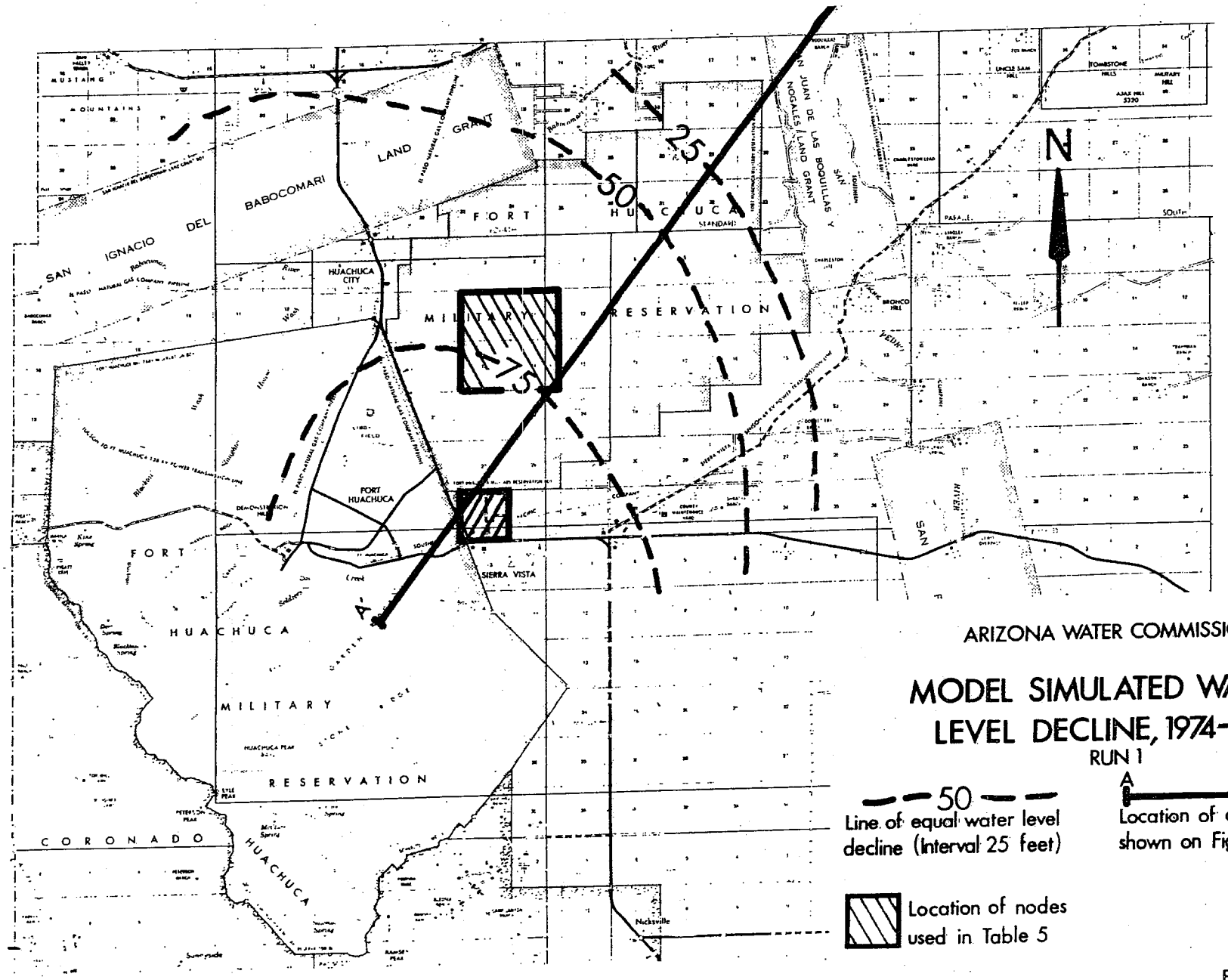
Figure 12 shows the water-level declines between 1974-2060 in the Fort Huachuca area for Run 1. The maximum decline was about 80 feet which is an average decline of 0.9 feet/year over the period. Results indicate that the amount of water presently being used by phreatophytes and being discharged to the rivers from the aquifer system would be reduced by about 30 percent.

Figure 13 shows the water-level declines over the period for Run 2. The declines are less than those for Run 1 because of the difference in storage coefficient which makes more water available for each foot of drawdown experienced. Thus, with the same demand, the maximum drawdown in a well field as well as the size of the cone of depression is reduced. The maximum decline over the period was 53 feet which is an average decline of 0.6 feet/year. Discharge by the aquifer system was reduced by only about 20 percent because of the larger storage coefficient.

Figure 14 shows the water-level declines over the period for Run 3 with an average maximum decline of 1.8 feet/year. This run affects the aquifer discharge by about 50 percent.

Figure 15 shows the water-level declines over the period for Run 4. Because of the larger storage coefficient, the declines are less than Run 3 with the maximum decline having an average rate of

USF200002326



ARIZONA WATER COMMISSION

MODEL SIMULATED WATER-LEVEL DECLINE, 1974-2060

RUN 1

50 — Line of equal water level decline (Interval 25 feet)

A — A' Location of cross section shown on Figure 16

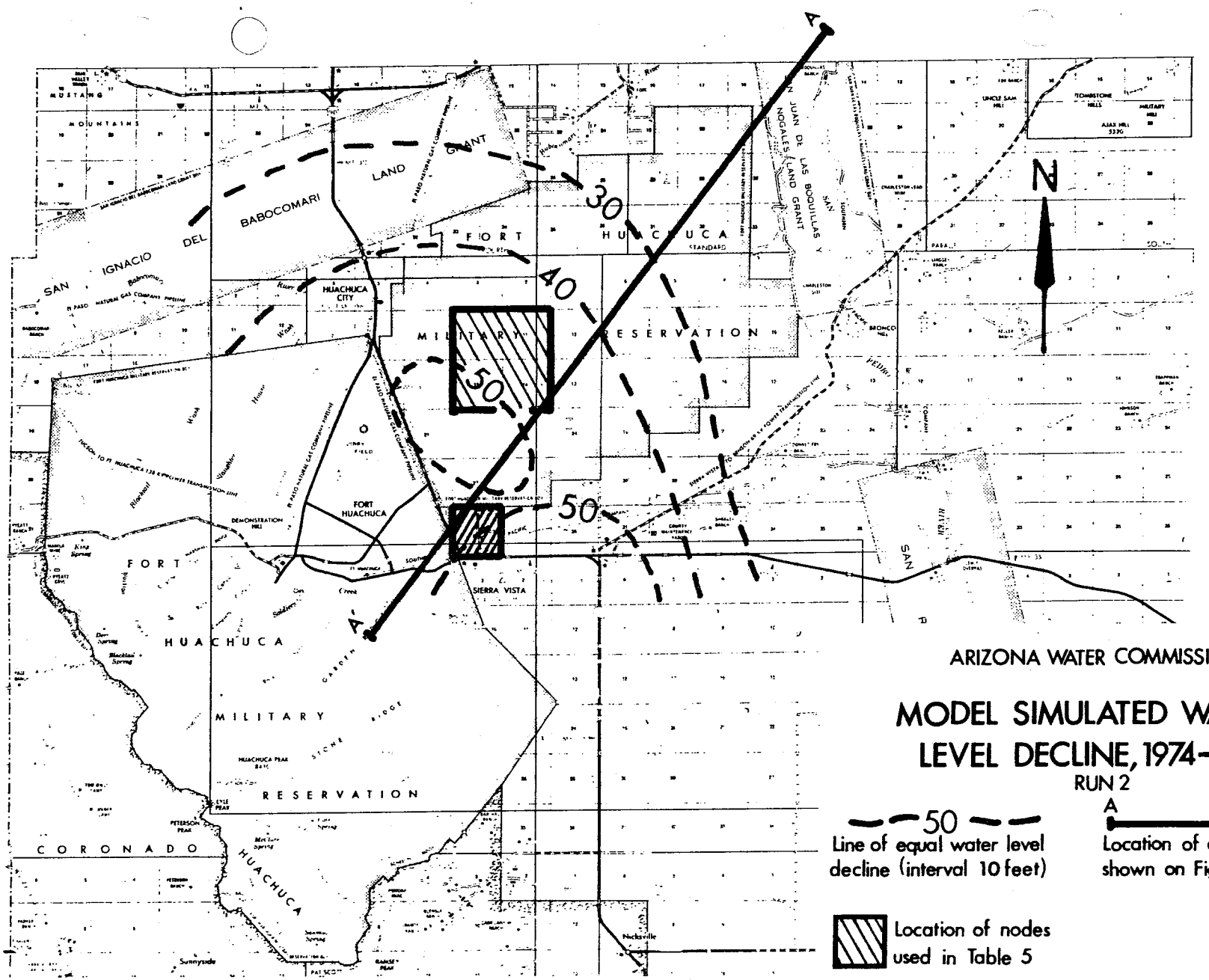
Location of nodes used in Table 5

Location of node used for Figure 17

Feb. 1974

FIGURE 12

USF200002327



ARIZONA WATER COMMISSION

MODEL SIMULATED WATER—LEVEL DECLINE, 1974-2060

RUN 2

50 — Line of equal water level decline (interval 10 feet)

A — A' — Location of cross section shown on Figure 16

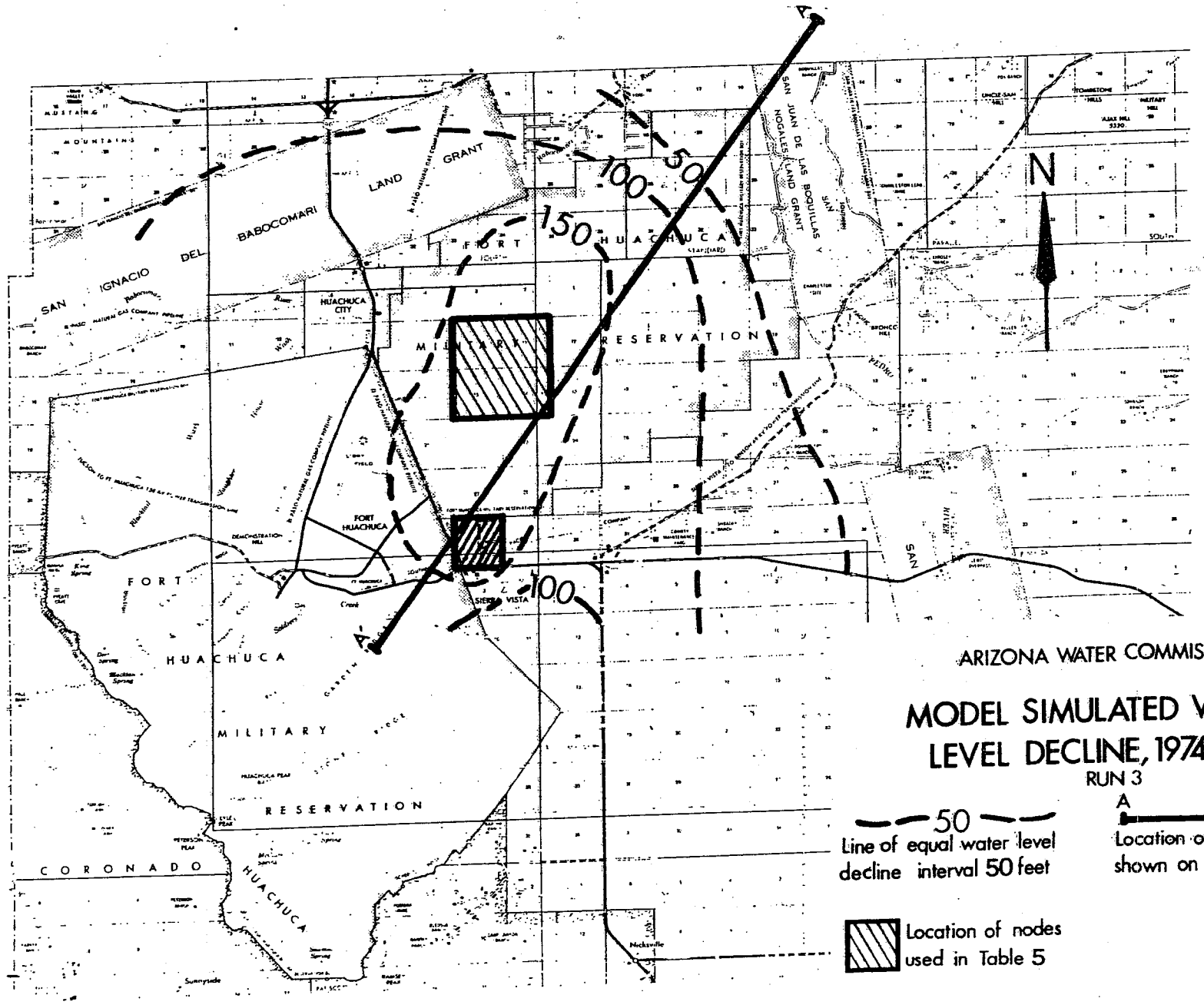
Location of nodes used in Table 5

Location of node used for Figure 17

Feb. 1974

FIGURE 13

USF200002328



ARIZONA WATER COMMISSION

MODEL SIMULATED WATER-LEVEL DECLINE, 1974-2060

RUN 3

50 — Line of equal water level decline interval 50 feet

A — A' Location of cross section shown on Figure 16

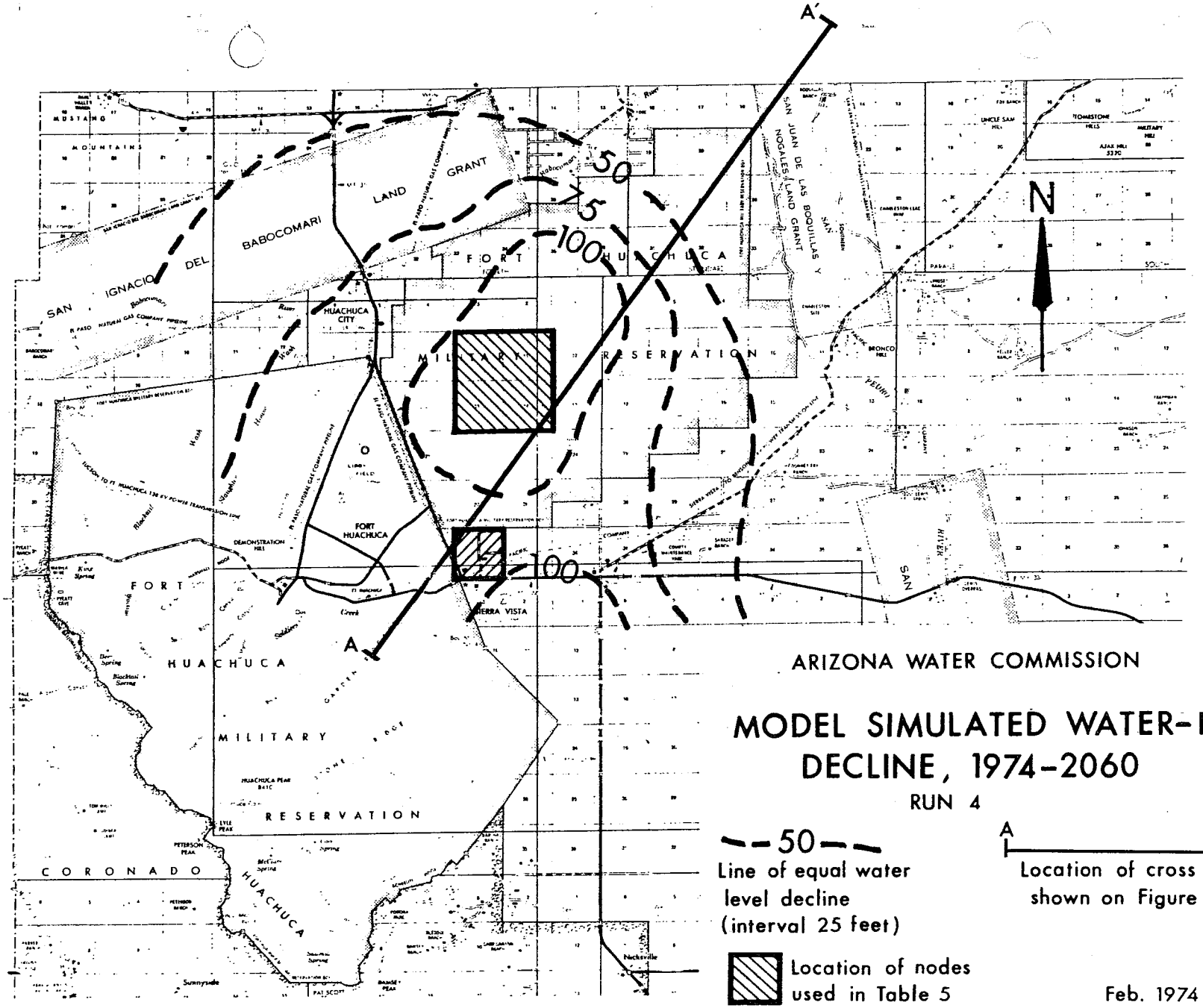
Location of nodes used in Table 5

Location of node used for Figure 17

Feb. 1974

FIGURE 14

USF200002329



ARIZONA WATER COMMISSION

MODEL SIMULATED WATER-LEVEL DECLINE, 1974-2060

RUN 4

--50-- Line of equal water level decline (interval 25 feet)

A — A' Location of cross section shown on Figure 16

Location of nodes used in Table 5

Feb. 1974

Location of node used for Figure 17

FIGURE 15

USF200002330

1.2 feet/year. In addition, the amount of water leaving the aquifer as discharge has only been reduced by about 35 percent.

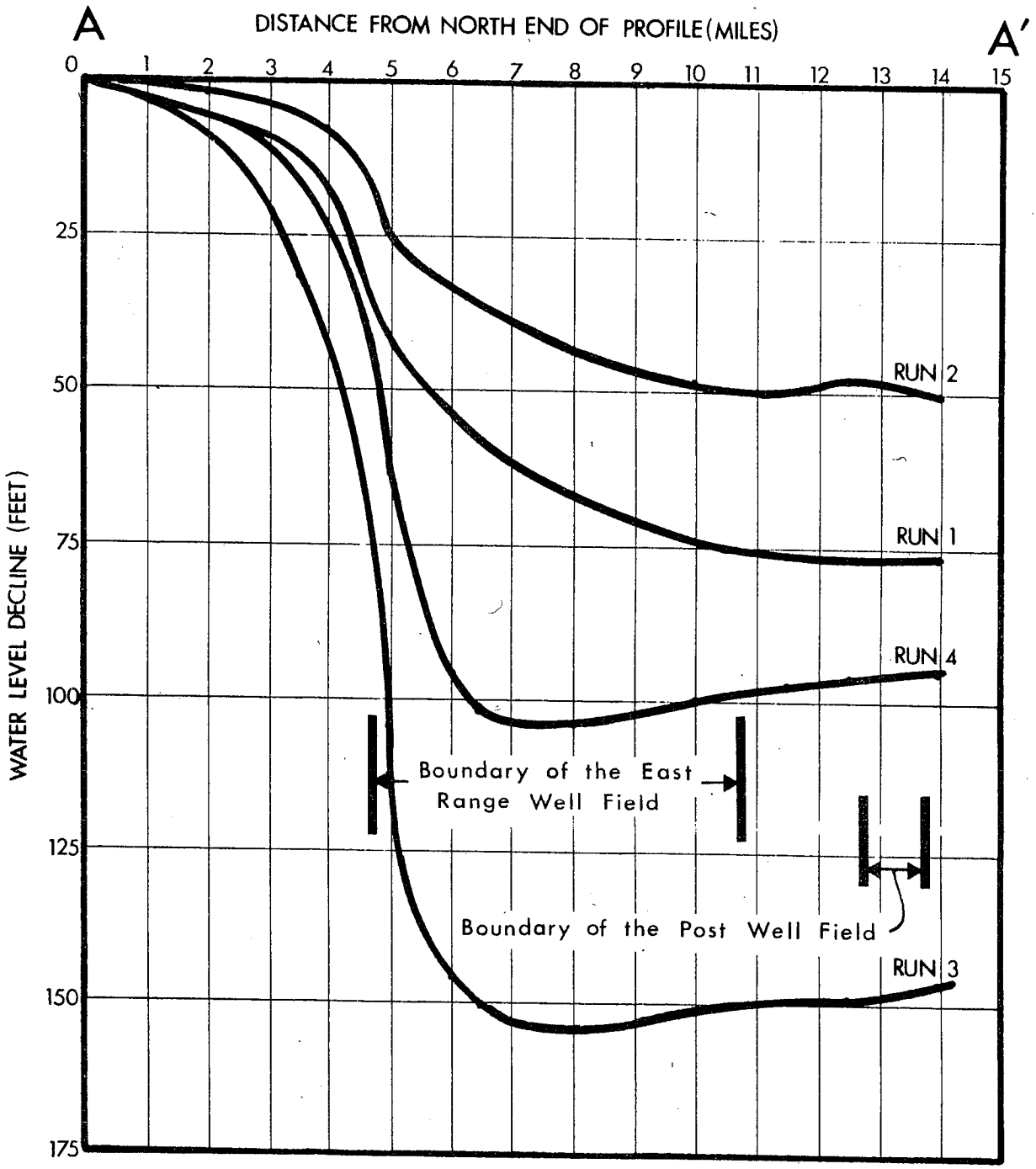
Figure 16 shows water-level profiles along section A-A' at the end of the projection period for all runs. The approximate boundaries of the Fort's present well field and their proposed East Range well field are also shown. All runs show the effect of the large pumpages in the Fort's present well field, which is unfavorably located close to an impermeable groundwater boundary — the Huachuca Mountains. The effect of the difference in storage coefficients is clearly evident in comparing Runs 1 and 2 which have a maximum difference of 25 feet for the same demand. The importance of the storage coefficient is even greater at higher demands as can be seen by the 50 foot difference between Runs 3 and 4 with the maximum demand. The effect of heavy pumping in the East Range to supply the Alternative IV demand is indicated by the shifting of the lowest point of the profile to the northeast.

Figure 17 is a comparison of declines for each run from 1974 to 2060 for a model node just east of the present well field and lying between the two well fields. As shown in the figure, the rate of decline in each run becomes relatively constant for the last 40 years.

SUMMARY

Because of concern over the adequacy of the water supply for Fort Huachuca, the Los Angeles District of the Corps of Engineers requested that the Arizona Water Commission evaluate the adequacy of the Fort's water supply based upon regional studies that the

USF200002331



ARIZONA WATER COMMISSION

PROJECTED WATER-LEVEL DECLINES IN FORT HUACHUCA WELL FIELDS

DURING THE PERIOD 1974-2060

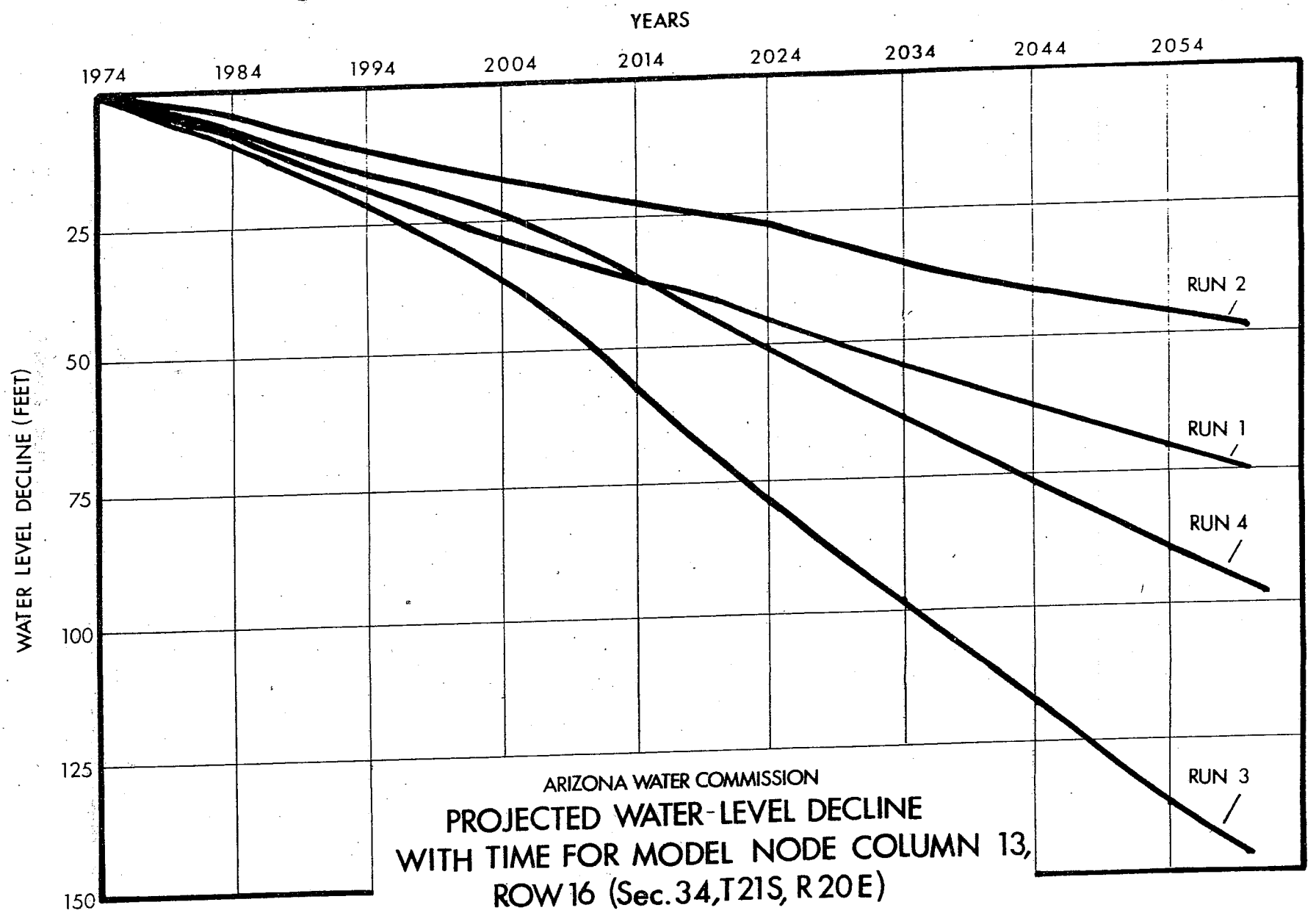
(Location of cross-section shown on Figures 12 – 15)

Feb 1974

FIGURE 16

USF200002332

USF200002333



ARIZONA WATER COMMISSION

PROJECTED WATER-LEVEL DECLINE WITH TIME FOR MODEL NODE COLUMN 13, ROW 16 (Sec. 34, T21S, R20E)

(Location of node shown on Figures 12–15)

Feb, 1974

FIGURE 17

Commission had begun. The Commission had decided that, because of the size of the area and the importance of long-term effects, a digital model could be used to good advantage to simulate the response to future water demands and would facilitate comparative analysis of inaccuracies in input data and population projections. Further, the digital model allows refinement of the data through simulation of known conditions, and locations of areas where additional data are needed.

Data within the study area are scanty and lacking in many areas but, fortunately, the greatest concentration of fairly reliable data is in the vicinity of the Fort itself. Because very little storage coefficient data were available for the study area, it was necessary to use two sets of S configurations in order to allow a bracketing of the possible range of storage coefficients.

The verification of the model was hampered by the lack of available pumpage data around the Fort prior to 1960 as well as the lack of aquifer data, especially west of the present Fort well field. The model does not duplicate the actual 1973 water levels. It does, however, come within one to five feet of duplicating the decline in the Post well field from 1961 to 1973. This close duplication of measured water-level changes indicates that, in the vicinity of the Fort, the model can give reasonable projections of water-level changes resulting from future water demands.

The results of the four model runs indicate that the drawdown will range between 50 and 160 feet for population growth Alternatives I and IV. The average maximum decline ranged from 0.5 to 1.8 feet/year. The effect of the storage coefficient on drawdown is quite important, giving a 50 foot difference in water levels between the

USF200002334

two runs for Alternative IV water level demands.

Table 5 shows a comparison of the average declines for all model runs for a four nodal area in the East Range well field. The location of these four nodes is shown on Figures 12-15.

Table 5

Comparison of average declines in feet for four nodes in the East Range well field.

Year	Water level Decline (ft.) Run 1	Difference in Decline Between Run 1 and Runs 2,3, & 4. Run 2	Run 3	Run 4
1980	5	-2	0	-2
1990	20	-6	+2	-5
2000	30	-10	+2	-8
2010	38	-13	+27	+6
2020	46	-16	+40	+12
2030	53	-18	+52	+18
2040	61	-22	+62	+22
2050	68	-24	+72	+27
2060	75	-26	+81	+29

In Table 5 the total model-simulated water-level decline is given for Run 1. The difference in the decline between Run 1 and the decline for each of the other model runs is shown as a plus or minus value for the remainder of the runs.

Table 5 points out the importance of the storage coefficient in effecting the magnitude of the declines. Runs 3 and 4 have the

USF200002335

same pumpage demand but different values of S. The average amount of decline is much greater for Run 3 than it is for Run 4, which has the higher storage coefficient. For both Alternatives I and IV the use of the higher storage coefficient configuration may result in water-level declines being different by as much as 150 percent. Additional aquifer tests would be desirable in the study area in order to obtain a better understanding of the actual storage coefficient values. Continued data collection over a period of several years will also enable some refinement to be made to the storage coefficient estimates.

In general, the simulated declines over the projection period are modest by Arizona standards. However, it is probable that the large withdrawals for Alternatives III and IV from the Fort's present well field, as specified in the model, would dewater portions of the aquifer underlying this well field as well as portions of the aquifer underlying Sierra Vista by the end of 2060. The severity of this impact would depend upon the actual thickness of the aquifer in these areas. It would be possible in actual operations to moderate this impact by shifting pumpage to the East Range well field where a greater thickness of saturated alluvium is available. With the simulated declines resulting from the largest water demands used in this study there would be 500-800 feet of saturated thickness remaining under the East Range well field at the end of 2060.

The model predicts reductions in the aquifer discharge to the rivers ranging from 20 percent to about 50 percent for the four runs. This would reduce base flows as well as and probably reduce the water supply available to phreatophytic vegetation along portions of the San Pedro and Babocomari Rivers. The most severe reduction

USF200002336

in discharge occurred in Run 3 which reflects the highest water demand and the lower storage coefficient configuration.

Future improvements of the model will include some adjustments of the T distribution north of the Fort's well field in addition to a sensitivity study on the effect of changing the estimated early pumpages. In addition, sensitivity studies will be conducted regarding the effect on the model of changing the transmissivity near the confluence of the San Pedro and the Babocomari Rivers and to explore the data inputs necessary to force the model to fit the observed water level data.

CONCLUSIONS

The conclusions presented in this report regarding the groundwater supplies of the Fort Huachuca area are drawn from model studies that will continue to be perfected and upgraded with the addition of new data that should improve the accuracy of predictions. The conclusions have been arrived at, and therfore must be judged, considering the accuracy of the predicted impacts vis a vis the safety factors of remaining saturated thickness and operational water management options available to the area. Ths following conclusions have been drawn:

1. The digital model of the groundwater reservoir in the Fort Huachuca area has, primarily due to time constraints, not yet been verified to the degree that permits unequivocable reliance. Nevertheless, it is concluded that the model as presently developed is able to give a reasonable prediction of the range of possible effects of the future demands for water on the groundwater resource.

USF200002337

2. On the basis of studies to date, it is evident that the effects of the projected groundwater demands for all demand levels considered have a large impact on the groundwater reservoir. The impacts of the Alternative I population levels are within the range of experience elsewhere in Arizona, and within the ability of the reservoir to physically provide a water supply. Therefore, the conclusion of the Arizona Water Commission is that the groundwater supply of the Fort Huachuca area is adequate for Alternative I, the most probable level of population growth for the Fort Huachuca area. Although not included in the projections, it can be concluded that the water supply is also adequate for Alternative II, a lower population level in which the Fort population is projected to remain at its current level.

3. The studies to date also indicate the impacts of the large withdrawals for Alternative population levels III and IV from the Fort's present well field on the water resource are unacceptably severe as the aquifer underlying this well field as well as that under the adjacent portion of Sierra Vista probably would be dewatered by 2060. However, the impact in this area can be relieved through a water management option that would place a greater share of the demand on the proposed East Range well field where greater groundwater supplies are available. Thus, it is preliminarily concluded that an ade-

USF200002338

quate water supply could be developed for the area for Alternatives III and IV from the groundwater resources available to Fort Huachuca.

4. Based on studies by the Bureau of Reclamation the Arizona Water Commission concludes that the authorized Charleston Dam project could supply sufficient water to meet all of the projected water demands in the Fort Huachuca area with no further depletion of the groundwater supplies except for Alternative IV. For this population level all but about 1000 acre-feet/year could be supplied by the Project; this withdrawal would have minimal impact on the groundwater resource.

5. It is preliminarily concluded that effects of the projected groundwater pumpage for all population levels would reduce base flows as well as possibly reduce the water supply available to phreatophytic vegetation along portions of the San Pedro and Babocomari Rivers.

USF200002339

SELECTED REFERENCES

Brown, S. G., et al, 1966, Water resources of the Fort Huachuca Military Reservation, southeastern Arizona: U. S. Geol. Survey Water-Supply Paper 1819-D, 57p.

Davis, G. H., et al, 1959, Ground-Water conditions and storage capacity in the San Joaquin Valley, Calif., U. S. Geol. Survey Water-Supply Paper 1469, p.p. 199-214.

Hayes, P. T., and Raup, R. B., 1968, Geologic map of the Huachuca and Mustang Mountains, southeastern Arizona: U. S. Geol. Survey Misc. Geol. Inv. Map I-509.

Pinder, G. F., An iterative digital model for aquifer evaluation: U. S. Geol. Survey training manual, p.44.

Roeske, R. H., and Werrell, W. L., Hydrologic conditions in the San Pedro River Valley, Arizona: Arizona Water Comm. Bull. 4, 76 p.

U. S. Geological Survey, 1941, Contours and ground-water surface, Fort Huachuca area, Arizona: unpublished map compiled by the U. S. Geol. Survey Ground Water Division, Tucson, Arizona.

USF200002340

APPENDIX 3

CONCEPT DESIGN REPORT FOR PROPOSED WATER SYSTEM EXPANSION

USF200002341

Page No.

III. BUDGET COST ESTIMATES (continued)

7. Alternative Ib(4) Optional Solution 4 36

Alternative II 37
Alternative IIIa 38

PLATE "A" Existing Water System

PLATE "B" Water Wells and Transmission Lines

PLATE "C" New Power Transmission Lines

PART II

(See Index at end of PART I
following PLATE "C" of PART I)

USF200002342

CONCEPT DESIGN REPORT

for

PROPOSED WATER SYSTEM EXPANSION

Fort Huachuca, Arizona

Department of the Army
Sacramento District, Corps of Engineers
650 Capitol Mall
Sacramento, California 95814

January 1974

prepared by

BLANTON & CO.
ARCHITECTS AND ENGINEERS
Tucson, Arizona

Appendix 3

USF200002343

The City of Sierra Vista, Arizona is adjacent to the easterly boundary of the Post. Some existing Post wells are located near this boundary. In evaluating optional approaches for providing an adequate water system for the Post, the effects that the various optional methods will have on the water systems in the City will be considered.

3. Background

Fort Huachuca was established prior to the beginning of this century but was reestablished on 1 February 1954 and is now serving in several capacities. It is now under the control of the U.S. Army Communications Command. The present population is about 12,000. PART II of this report entitled "Investigation and Recommendations for Upgrading the Water System at Fort Huachuca, Arizona" dated February 1973 describes historical and present conditions at the Post.

Although the present population appears to be stable, the environmental conditions are quite desirable for many types of future military development, testing, training and research missions. The Post contains in excess of 75 square miles. This large area allows for extensive additions of both working and residential facilities as needs may require.

Proposed new water wells on the East Range would have an estimated supply capacity of 11,500 gpm. Information on the locations shown on Plate "B" and capacities of the wells are based on the most recent hydrogeological information made available through the Sacramento District Office.

PART II of this report describes the present water system conditions. It contains alternative construction items but not estimated costs for satisfying present needs. The budget cost estimates for these conditions are provided in this PART I.

Although PART II shows that a daily maximum of about 400 gallons of water per person is used, the daily average in the maximum use month is 330 gallons per capita per day (gpcd) and

USF200002344

PREFACE

This CONCEPT DESIGN REPORT consists of two parts.

PART I was completed in January 1974 and addresses itself basically to three Alternatives or Conditions of population of Fort Huachuca for which an adequate water system should be provided. There are subheadings or optional approaches under the basic Alternatives.

Budget Cost Estimates are presented for all Alternatives and the options in PART I.

PART II was completed in February 1973 and consists of an in-depth evaluation of existing water supply, storage and distribution facilities at Fort Huachuca together with conclusions and recommendations of steps needed to adequately serve the present strength or population of Fort Huachuca.

No Budget Cost Estimates are contained in PART II.

A tabular summary of all significant information is presented on page 4 of PART I.

USF200002345

I N D E X

Page No.

PART I

Preface -i-

I. INTRODUCTION 1

1. Purpose 1
2. Considerations 1
3. Background 2

TABLE I Summary of Water System Alternatives and Budget Estimates 4

II. CONCEPT DESIGN ALTERNATIVES 5

1. General 5
2. Alternative I - Present Population in 1974 Using Existing Wells 5
3. Alternative Ia - Present Population in 1977 Add Test Well #5 7
   a. Optional Solution 1 7
   b. Optional Solution 2 9
4. Alternative Ib - Present Population in 1977 Other Modifications 9
   a. Optional Solution 1 10
   b. Optional Solution 2 11
   c. Optional Solution 3 12
   d. Optional Solution 4 14
5. Alternative II - 25,000 Population in Year 1980 17
6. Alternative IIIa - 50,000 Population in Year 2000 22
7. Alternative IIIb - 50,000 Population in Year 2030 27

TABLE II - Well Pump Information 29

III. BUDGET COST ESTIMATES

1. Alternative I 30
2. Alternative Ia(1) 31
3. Alternative Ia(2) 32
4. Alternative Ib(1) Optional Solution 1 33
5. Alternative Ib(2) Optional Solution 2 34
6. Alternative Ib(3) Optional Solution 3 35

Registered Professional Engineer (Civil) CERTIFICATE EXPIRES HENRY L. ROHRIG DEC. 31, 19 74 ARIZONA, U.S.A.

USF200002346

CONCEPT DESIGN REPORT

for

PROPOSED WATER SYSTEM EXPANSION

Fort Huachuca, Arizona

PART I

Registered Professional Engineer (Civil)
CERTIFICATE EXPIRES
HENRY L. ROHRIG
DEC. 31, 1974
ARIZONA, U.S.A.

USF200002347

I. INTRODUCTION

1. Purpose

The purpose of PART I of this report is to evaluate the existing potable water supply and distribution facilities at the Post and consider various alternatives and budgeting costs for adequately serving the Post under existing and possible future conditions.

A tabulation of these Alternatives, a brief outline of work to be accomplished and Budget Cost Estimates for them are shown in TABLE I on page 4.

2. Considerations

Three basic design populations are considered in this PART I; an existing population of 12,000; an assumed year 1980 population of 25,000; an assumed year 2000 population of 50,000. Methods for providing water systems to serve the various populations are described under Roman Numerals I for existing population, II for 25,000 population and III for 50,000 population. The Roman Numerals are followed by small letters to reflect variations within these alternatives, and by (1), (2) etc. to reflect optional solutions to the variations in the alternatives. Budget estimates on page 4 reflect costs for Alternative I either in year 1974 or 1977. Costs for II and IIIa indicate additional costs after 1977.

Discussions with Post personnel resulted in siting of the assumed increases in population for the Post as is delineated on Plate "A" and Plate "B".

Plate "B" indicates the concept location and sizes of additional water transmission mains necessary to utilize the water developed from new deep wells.

Plate "C" indicates the electrical power system necessary to energize the new deep well pumps.

USF200002348

the normal daily average is 220 gpcd. This figure is higher than is usually thought to be normal. A figure of 300 gpcd is considered to be a reasonable quantity for estimating future water use during the 1970's only.

These per capita uses are based on present potable water use conditions whereby the golf course and Chaffee Field are irrigated with wastewater treatment plant effluent. The 300 gpcd figure will be used through the year 1980 when it is estimated that the Post population may be 25,000. In years beyond year 1980 there should be more conservation and reuse of water so that facilities for serving the estimated additional 25,000 population in year 2000 will be based on a more normal 200 gpcd figure.

The possibility of using natural gas or oil fueled engines for driving pumping equipment at new wells in lieu of electric motors was considered. However, because of the limitations on the supply and availability of gas and oil, it is felt that electric power would be a more realistic energy source.

PLATES "A" "B" and "C" follow the narrative portion of PART I of this report. The clearance lines along the airport runways have been developed by the Post. There is to be no construction of residential or operations facilities closer than 3000' from the runway. In the next 1500 foot strip, only facilities such as storage and other types of installations not normally occupied by full time personnel are to be utilized. The shaded areas for future populations are the general locations that the Post personnel felt would be developed for housing and use by the additional population.

PLATE "A" shows the existing water system skeleton and PLATES "B" and "C" show additional water well and water transmission facilities tied into the existing water system and the power lines to operate the pumps.

USF200002349

II. CONCEPT DESIGN ALTERNATIVES

1. General

The discussions herein will describe investigations and optional solutions for providing the potable water requirements to the Post under present conditions and under the estimated future population conditions. The discussion of the various design concepts as outlined in TABLE I on the preceding page will provide descriptions of each of the alternatives. TABLE II on page 29 indicates sizes and horsepower for recommended new well pumps. Detailed estimated costs of all alternative conditions are shown on pages 30 through 37.

2. Alternative I - Present Population in 1974 Using Existing Wells

The concept for this Alternative will utilize only the existing water sources that are now in service. As now operated the equipment delivers about 3090 gallons per minute (gpm). This is not sufficient to deliver the 4.613 million gallons per day (mgd) presently used during the month of May even when pumping is continuous for the entire 24 hours. If the existing wells were provided with new pumps discharging at maximum capacity directly into the distribution system, they would all need to operate at least 20.1 hours daily for a month or more to supply the monthly maximum demand of 4.613 mgd. If changes are made in the existing wells and pumping equipment as listed below, the required 4.613 mgd could be delivered by operating all pumps 22.2 hours per day. Neither of these situations are satisfactory because they do not allow for any emergency breakdowns of pumping equipment as recommended by TM-5-813-2. However, since the design concept changes in presently inadequate equipment can supply the demand in 22.2 hours, the major expenditure for new well pumps in wells other than #1 and #2 cannot be justified for the 2.1 hours less pumping per day.

USF200002350

The following items in PART II pages 48 through 52 of the report would be complied with:

a. Construct as described (Pressure zone changes).

c. Construct as described (Booster pumps from Zone "C" to Zone "B").

d. The existing pumping system would be replaced with new submersible pumps in well Nos. 1 and 2 that discharge directly into the transmission main to the 1-1/2 million gallon reservoir. (10 inch) 27 stage - 200 horsepower - 600 gpm each).

e. Construct as described (New 1.5 million gallon reservoir).

g. Construct as described (Add to and integrate all pump controls).

h. Construct as described (Redevelop Well No. 5)

i. Construct as described (Plug abandoned water lines)

k. Construct as described (Valve changes).

The reasons for omitting items b, f, j, l, m and n, which were additional recommendations of PART II, are as follows: Item b is the recommendation to include Test Well #5 to the existing system. This recommendation will now be included in Alternative Ia. Items f and j refer to changes in the present situation at Well #1 and #2, that were eliminated by Item d. Item i does not need to be added until wastewater use for irrigation of other areas is approved by the Medical Department. Item m, the development of additional wells, are to be included in other Alternatives. Item n refers to equipment changes that would not necessitate immediate capital expenditures.

USF200002351

3. Alternative Ia -
Present Population in 1977- Add Test Well #5

a. Ia(1) Optional Solution 1 - Minimum Requirements

PART II indicates that pumping and other operating equipment should be added to Test Well #5, which currently exists only as a drilled, cased and capped well. Then a water transmission line would be constructed to connect Test Well #5 to the existing 18 inch line at Libby Field. Also, new pumping equipment with a combined capacity of 1200 gpm would be placed in well Nos. 1 and 2. This would increase the delivery to the system by 160 gpm. The total pump delivery would then be 3090 gpm + 1500 gpm + 160 gpm = 4750 gpm. Army TM-5-813-2 requires that the pumps furnish the required water demands by operating a maximum of 16 hours daily. The pumping facilities would then deliver the 4.613 mgd in 16.2 hours. Good engineering design practice for a water supply from wells should provide for the system to deliver the demand when the largest well is temporarily out of service. Therefore, with Test Well #5 pump out of service, the total pumping rate would be 3250 gpm which would supply the 4.613 mgd requirement in 23.7 hours. These pumping rates are close enough to the Army manual to be considered satisfactory.

The following items in PART II on pages 48 through 52 of the report would be complied with:

a. Construct as described (Pressure zone changes).

b. Construct as described (Add Test Well #5 pumping equipment and transmission main).

c. Construct as described (Booster pumps from Zone "C" to Zone "B").

USF200002352

d. The existing pumping system would be replaced with new submersible pumps in well Nos. 1 and 2 that discharge 1200 gpm directly into the transmission main to the 1-1/2 million gallon reservoir.

e. Construct as described (New 1.5 million gallon reservoir).

f. Check and repair switch gear on well and booster pumps at well Nos. 3, 4, and 6.

g. Construct as described (Add to and integrate all pump controls).

i. Construct as described (Plug abandoned water lines).

k. Construct as described (Valve changes).

The reasons for the omission of items h, j, l, m, and n are the same as those explained for Alternative I, on page 6. Consideration was given to either abandoning or rehabilitating existing well No. 5 which is described under item h. However, if abandoned, there would be insufficient water supply to satisfy the criteria. The chance of success with rehabilitation is somewhat questionable. It is therefore thought best to continue its present operation.

In looking to the future, as will be described later in Alternative II of this report for future population, the transmission line under Item b. above will need to be larger than an 18 inch to carry future flows from 4 other wells in the same general area. Sizes shown on lines on PLATE "B" for Ia are for serving Test Well #5 only. Sizes determined in Alternative II indicate that a larger line should be constructed under Alternative Ia in order to save the expense of constructing another line for transporting water from any future wells in the area.

USF200002353

Electric power and pump control lines would also be constructed from the central transformer station near the Administration Building to the site of Test Well #5. They would be of such size and number to serve all future equipment that may be installed in the well field. However, in order to compare costs for minimum equipment against those large enough to serve estimated future well pumps, budget costs for the required smaller water and electric lines are determined separately.

b. Ia(2) Optional Solution 2 - Optional Solution 1 Including Sizing for Future Requirements.

This solution is the same as Solution 1 above except that the water transmission line is sized larger to serve future well pumps. The electric power line is larger for serving future pumps and begins at the central transformer station instead of using a section of smaller existing line near the station.

4. Alternative Ib - Present Population in 1977 - Other Modifications

Alternative I and Alternative Ia discussed minimum modifications to the existing water system for providing a potable water supply for the present Post population expected through 1976. Alternative I will not meet the standards in TM 5-813-2.

This section of the report will discuss other options or possible methods of serving the same existing population. The options would improve the water system so that it meets the requirement that the well field produce the required daily demand for water in approximately 16 hours. In addition, the well field would be capable of delivering the maximum daily demand while the highest capacity well pump is temporarily out of service. The remaining wells would be pumped more than the desired 16 hours to compensate for the well taken out of service but this would be only a temporary situation.

USF200002354

The existing pumping facilities are composed of boosting equipment at each well site. If the well pumps have a 75% efficiency and the booster pumps have a 75% efficiency, the overall efficiency is about 56%. This does not include losses from extra starting of booster pumps due to unavoidable imbalance of the volume delivery of the two pumping systems. Therefore, power costs could be cut about 30% by removing and replacing all existing well pumping systems and replacing them with new well pumps discharging directly into the water transmission system. However, this would entail capital expenditures greater than would be justified since present equipment is in good enough operating condition to last another 20 years. This should be the case at wells Nos. 3, 4, 5, and 6.

The following discussions are possibilities for modifications and additions that can provide economical and adequate water supply facilities other than those described under Alternatives I and Ia. The items a, c, e, g, and i in PART II pages 48 through 52 would be necessary under all options.

a. Ib(1) Optional Solution 1

Well No. 5 would be abandoned because of its original poor specific capacity and its continued dropping of capacity. (1958 test, 14 gpm/ft. drawdown; recent 6.2 gpm/ft. drawdown).

Wells No. 1 and 2 would be provided with new pumps discharging directly into the water transmission line to the 1.5 million gallon reservoir. The existing booster equipment would be detached. As explained on p. 14 of PART II, the motors that drive Boosters #1 and #2B draw excessive current when operated under full flow conditions. In order to prevent electrical overloading, the booster pumps are throttled by gate valves to the extent that their combined outflow is 168 gpm less than Well #1 and #2 production.

USF200002355

Booster Pump #2A is connected to the same suction header as Booster Pumps #1 and #2B, but delivers to the lower pressure Zone "C". It is used only rarely when boosters #1 and #2B are not operating and discharging directly to Zone "B". Boosters #3, #4, #5, and #6 are capable of supplying the lower pressure zone. In addition, Booster #2A is also gate-valve-throttled because its electric motor draws excessive current. It was indicated on p. 18 of PART II that Booster Pump 2A should be valved off from the system.

Test-Well #5, which is presently drilled, cased and capped, and a 1000 gpm well to be drilled in the northeast well field would be drilled and cased. Both wells would be equipped to discharge directly into the water transmission line to the Zone "C" water distribution system along with the existing pumpage from boosters at Wells No. 3, 4, and 6. The total pump delivery would then be 5310 gpm. This would furnish the 4.613 mgd demand in 14.5 hours. If the 1500 gpm well pump is temporarily out of service, the remaining pumps delivery of 3815 gpm would provide the 4.613 mgd in 20.2 hours. Both of these conditions would be adequate.

b. Ib(2) Optional Solution 2

Another alternative solution would consist of adding Test Well #5 pumping equipment, another new completely equipped 1000 gpm well, deactivating existing Well No. 5, replacing booster pump No. 1, reconstructing the starting and control sensing equipment for booster pump No. 1 and booster pumps Nos. 2A and 2B, and of lowering the Well Nos. 1 and 2 well pump bowls. The equipment at well Nos. 1 and 2 would discharge about 1100 gpm at an overall efficiency of about 50% for the well and booster pumps.

USF200002356

The renovated equipment together with the other pumping equipment as discussed above would furnish 5210 gpm. The 4.613 mgd would be delivered in 14.8 hours with all pumps operating and in 20.8 hours with the 1500 gpm pump out of service. These conditions would be adequate.

This alternative solution is not thought to be as desirable as Optional Solution 1 because the resulting pumping efficiency at Wells Nos. 1 and 2 is low and the equipment is quite old. Replacement of this pumping equipment is desirable. However, for comparison purposes, these changes will be estimated in the tables.

c. Ib(3) Optional Solution 3

Another alternative solution would consist of adding Test Well #5 pumping equipment, another new completely equipped 1000 gpm well, de-activating existing Well No. 5, replacing pumps in Well Nos. 1 and 2, replacing the pump in Well No. 3 with 700 gpm capacity pump connecting all 3 well pumps to the water transmission main to the 1.5 million gallon reservoir in Zone "B". This would also require a new 12" main from Well No. 3 to the 16" water transmission main.

Because of the proximity of Wells Nos. 1, 2, and 3 to the City of Sierra Vista it would be desirable to utilize other wells in preference to these. However, the City is growing to the east and new City wells for water supply to serve its additional population might logically be located east because of both the change in population center and the underground water availability. The most recently constructed wells in the City have been at the City Park and east of State Road 92. There appears to be little influence between wells in the City and those on the Post. However, because of the close proximity of Well Nos. 1 and 2, their combined pumpage should be held at about 1200 gpm.

USF200002357

If the Well No. 3 pump was connected directly to the water transmission line leading to the present 1.5 million gallon tank, rather than through the booster pumps into Zone "B", the present triple pumpage from boosting at the well-site and at the Wherry Booster Station could be prevented. However, the pumping head would be considerably greater and friction head loss in the transmission main to the 1.5 million gallon reservoir would be increased about 100 feet. This added friction loss would also be applied to the pumps in Wells Nos. 1 and 2. The capital costs for changing the pumps in the 3 wells and constructing a 3000 foot 12" water transmission line to the existing water main at the Well No. 1 site would be quite high.

This solution would provide total pumping capacity of 5430 gpm and with the Test Well #5 pump temporarily out of service, 3930 gpm. The 4.613 mgd requirement would be satisfied in 14.2 hours with all pumps operating and in 19.6 hours with the highest capacity pump not operating. This would be a satisfactory condition.

Since both the booster and well pumps at Well No. 3 are still in good condition, the added expense for changing the pumping equipment at all three wells could not be justified. The change is therefore not practical at the present time.

Several years in the future, possibly year 2000, additional people might be located in Zone "B" near the water transmission main to the 1.5 million gallon tank which floats on the Zone "B" distribution system. Distribution lines to serve them would be connected to the water transmission main which connects to the 1.5 million gallon reservoir. It would be desirable to have the 1900 gpm from the 3 well pumps serve the population directly from the line to the 1.5 million

USF200002358

gallon reservoir in addition to helping maintain the optimum water level in the tank. If the Well No. 3 pump and its booster pump were old enough to warrant replacement, the change-over as suggested above would be desirable. The drain-off from the water transmission main into the Zone "B" distribution system would shorten the effective length of the water transmission line. This would result in a lowering of the friction losses and lowering the pumping heads.

d. Ib(4) Optional Solution 4

The average daily potable water requirements during a maximum use month, as discussed in PART II of this report, is 4.613 mgd. During a desired 16 hour pumping day, the total output of the six well boosters is 3.22 mgd. Therefore, a daily deficit of 1.393 million gallons may occur during the three high demand months of May, June and July. The cumulative deficit for this 90 day period of high use would be about 125 million gallons. If a reservoir containing 125 million gallons existed at a proper location it could make up the deficit so that the existing pumps would need to pump only 16 hours per day at this present rate of production.

It would be possible to construct a reservoir with capacity of 125 million gallons at a site approximately two miles south of the intersection of Garden Canyon Road and the road to the Main Entrance of Fort Huachuca. The water surface elevation would be set at 5140, 30 feet higher than the 1.5 million gallon reservoir. Its supply would be from the Garden Canyon Springs, Well Boosters #1 and #2B via the piping system that presently connects these sources to the 1.5 million gallon reservoir, and surplus from Zone "B" via the Wherry booster station. See Plate I, PART II, for location.

USF200002359

Reviewing the information concerning spring water production on page 9 of PART II, it is estimated that Garden Canyon Springs would contribute about 10 million gallons toward the reservoir's capacity during a 12 month period. The well pumps would have to provide the remaining 115 million gallons during the 9 month period when the wells are meeting the demand in less than 16 hours per day of average pumping time.

Referring to Chart #1, PART II, p. 8a, it has been determined that the total well production for 1972, excluding production for May, June and July is 630 million gallons or 2.33 mgd. This daily production could be supplied by the well boosters in 11.6 hours. This would allow well boosters #1 and #2B to supply a daily average of 274,560 gallons to the 125 million gallon reservoir during the remaining 4.4 hours of pumping schedule. Over a 9 month period this will accumulate to 75 million gallons.

The additional 40 million gallons necessary to completely fill the reservoir would be supplied from Wells Nos. 3, 4, 5 and 6 through the Wherry booster station discharging to the 1.5 million gallon tank. A booster station for lifting the water from 12" water transmission line at the intersection with the Garden Canyon 12" water line to the 125 million gallon reservoir would be required. Also, an elaborate system of control valves and sensing equipment for maintaining optimum water elevations in the 125 million gallon reservoir, the 1.5 million gallon tank and the reservoirs in Zone "C" would be required.

The 125 million gallon reservoir would be used to offset the severe withdrawal rate from the 1.5 million gallon tank during the three high use months. As mentioned in PART II, approximately 72% of the population depends on the 1.5 million gallon tank either directly or indirectly through the Old Post Boosters. It has been recorded that, during the

USF200002360

irrigation season, about 1 million gallons have been withdrawn from this tank in two hours, even though well boosters #1 and #2B and the Wherry boosters were supplying it continuously. The 125 million gallon reservoir must be capable of supplying 2000 gpm for 8 hours to the 1.5 million gallon reservoir. This could be accomplished with 4 booster pumps with capacity of 500 gpm each. Their pumping capacity could furnish the one million gallons that boosters #1 and 2B and Wherry Booster station are not able to pump during maximum use periods.

All of the other changes indicated in Alternative I, previously discussed, would also need to be made. These changes include the addition of booster pumps from Zone "C" to Zone "B", a new 1.5 million gallon reservoir, replacement of pumps in Well Nos. 1 and 2, and reconditioning and replacing equipment and materials.

This concept of a 125 million gallon reservoir with booster pumps to serve Zone "B" has been considered as an alternative suggestion to the concept of additional wells and transmission mains. The operating costs for pumping time would be only slightly more than for other options. However, the cost of constructing the covered reservoir, booster station, and control equipment is excessive. Also, the complicated control system would be quite difficult to keep operating properly. This option should not be seriously considered but will be included in the cost estimates for comparison purposes.

USF200002361

5. Alternative II - 25,000 Population in Year 1980

a. Required Demand and Production

Data summarized in PART II of this report shows that the present population uses an average of 330 gpcd in May which is the maximum use month. In accordance with requirements of TM-5-813-1 the water quantities to serve the future additional population of 13,000 to reach the 25,000 total is as follows: By using the 1.15 capacity factor for the total population times 150 gpcd would require 172.5 gpcd for the 13,000 additional persons or an additional water supply of 2,240,000 gpd. The fire demand for this population is 50% of 2,240,000 + 2 fires requiring 1000 gpm each for 4 hours or 1,600,000 gpd. The domestic requirement is, therefore, the higher demand. However, because the climate is arid and warm and because of the historical water use record described in PART II, a water use of 300 gpcd is considered a more realistic quantity. This would be an additional supply of 13000 x 300 = 3,900,000 gpd. This added to the 4.613 mgd actual usage quantity for the current population makes a total maximum water supply requirement of 8,513,000 gpd.

If pumps are provided to deliver the 3,900,000 gallons of water in 16 hours, their combined capacity will need to be 4070 gpm. This could be supplied with 4 new wells and pumps producing the 1000 gpm rated capacity of each well in the northeasterly area of the reservation.

The proposed sources of the 4070 gpm of additional water supply would be four wells. Information on the locations and capacities of the wells are based on the most recent hydrogeological information made available through the Sacramento District Office.

USF200002362

Test Well #5 construction is assumed to be completed and placed in service to adequately supply 1974 needs (see PART II of this report). The four wells designated on PLATE "B", as A, B, C, and E would be drilled and cased and equipped with pumping, treatment and water transmission facilities.

b. Water Treatment

The chemical quality of the water from the test wells as reported in the most recent hydrogeologic report is quite good. The only treatment required would be chlorination for disinfection and possibly fluoridation.

c. Water Transmission Lines

The 4 additional wells would be connected to the water transmission line from Test-Well #5 as indicated on Plate "B" by (II). The water transmission lines as shown on the map are sized so the friction loss from Well E to the upper area of Zone B of the distribution system at total design flows is about 59 feet or 25 psi. This is considered a reasonable friction loss for use of optimum size pumps and motors and for equitably balancing operating and capitalization costs.

d. Storage

(1) Zone "A"

Since, for this 1980 Alternative, no additional population is anticipated in Zone "A", no additional gallons of storage will be necessary. However, as described on page 32 of PART II, the two existing reservoirs are 62 and 34 years old respectively in 1973. By the year 1980 they will be 69 and 41 years old and will have served a normal useful life. Therefore, they would be replaced with a 500,000 gallon reservoir and the old reservoirs would be torn down.

USF200002363

(2) Zone "B"

About 1500 of the additional 13,000 population are expected to be located in Zone "B" as shown on Plates "A" and "B". PART II of this report indicates that population in Zone "B" is 7980 x capacity factor of 1.23 or 9800. The sum of 7980 and 1500 equals the total of 9480. This is considered to be a stable population and so no capacity factor will be added. The water supply requirements in Zone "B" will therefore be considered to be for 9800 as described in PART II page 8, of this report. The storage requirements will be the same existing 1.5 and the new 1.5 million gallon tank as described on page 33 of PART II of this report.

(3) Zone "C"

About 11,500 of the additional 13,000 population are expected to be located in Zone "C" as shown on Plates "A" and "B". PART II of this report indicated that population in Zone "C" is 3060 x capacity factor of 1.23 or 3760. The sum of 3060 and 11,500 equals a total of 14,560. This is considered a stable population, therefore no capacity factor will be added. Since the present domestic water use in Zone "C", from PART II page 8, of 564,000 gpd is quite close to actual use, the additional requirements for future population should be added to it. The 564,000 gpd and the 300 gpcd for the 11,500 people make a total domestic requirement of 4,014,000 gpd.

The storage tank and reservoir whose water is floating on the water distribution system in this zone also supplies the water source for booster pumps to supplement the well pump supply to Zone "B". However, the well water supply and pumping facilities that were indicated as needed by Alternative Ia for year 1977 would provide for the higher zones, so that storage in Zone "C" would not be affected by the needs of the higher zones. The storage requirement to comply with TM-5-813-4 is that required for 50% of the domestic demand in

USF200002364

Zone "C" + 1000 gpm for 2 fires of 4 hours duration. This would be 4,014,000 x 50% + 1000 x 2 x 4 x 60 = 2,487,000 gallons. This is less than the existing 3,500,000 gallons and so no added storage facility is needed.

e. Booster Pumps

In PART II two additional booster pumps with capacity of 535 gpm (513,000 gallons in 16 hours) pumping against about 230 feet of static head and about 30 feet of friction head or TDH of 260 feet are designated to pump from Zone "C" into Zone "B". Because of the additional population of 1500 requiring 300 gpcd or 450,000 gpd another pump of the same capacity would be added. This would then be a total of 3 more booster pumps from Zone "C" to Zone "B" than now exist.

f. Controls

The central control panel is located in the mechanical room of the main Administration Building. The additional booster pumps will require a new control panel at headquarters and another at the pump house. The receiving-transmitting equipment would be constructed adjacent to and integrated into the existing control panel. In order to assure that pumps in all wells are operated occasionally, the controls should alternate the operation of pumps in Test Well No. 5, new Wells Nos. A, B, C, and E, with existing booster pumps at Wells Nos. 3, 4, 5, and 6. Control signal transmission lines would be extended to the new wells on the electric power poles to the new well sites. New booster pump controls at Burns and Hatfield Street should alternate their pump operations with the Wherry Booster Station pumps.

g. Power

In order to serve the new well pumps and controls, extension of electric power lines and equipment and control signal lines would be required to each of the new well sites.

USF200002365

h. Wastewater Treatment Plant Effluent

It is anticipated that the Post medical officials will approve more extensive use of treated wastewater for irrigation by year 1980. A reasonable assumption would be that the area around the administration building would be added to the areas presently being irrigated with wastewater effluent. The wastewater effluent pump at plant #2 (shown on PLATE A) that discharges into the 8 inch asbestos-cement transmission line to Chaffee field tank (about 8000 gallon capacity) has capacity of 500 gpm at 225' TDH. Elevation differential is about 175' and friction loss about 50'. The wastewater effluent pump is controlled from an electric probe in the 8000 gallon tank. Since the wastewater effluent pump cannot keep up with the booster from the tank, wastewater effluent pumping capacity would be increased. If the pumping capacity is increased by 250 gpm, 250,000 gallons could be delivered in 6 hours which is a reasonable length of time for daily irrigating so as to have it completed in one work shift. If more than the 250,000 gpd is to be used, the irrigation time could be extended to accommodate the requirement.

The added friction in the transmission line at 750 gpm would be 60 feet. This added friction would lower the delivery capacity of the existing pump to almost zero discharge. The existing wastewater effluent pump at wastewater plant 2 should therefore be moved to the 8000 gallon tank for a standby and two new pumps installed at the wastewater plants. Their capacities would be about 500 gpm at 285' TDH and 250 gpm at 285' TDH. They would be about 50 Hp 7 stage 10" and 30 Hp 12 stage 8" size respectively.

When the new pumps are installed an additional control probe and control equipment would be required for the automatic operation of the pumps.

USF200002366

6. Alternative IIIa - 50,000 Population in Year 2000

a. Required Demand and Production

As recommended in TM-5-813-1 the additional water required for serving an additional future population of 25,000 to reach the 50,000 total is 150 gpd x 1.0 capacity factor. However, the historical usage during recent years in the maximum use month is 330 gpcd with a maximum use daily of 400 gpcd. Because of the arid and warm climate the 150 gpcd water requirement is considered low. Also because of the emphasis being placed, long term, on wastewater reuse, the present 330 gpcd is considered to be too high for the future. The population base will be higher and extensive water reuse should be in operation by the year 2000. A 200 gpcd is therefore estimated to be a reasonable water use for the added 25,000 people in the year 2000. The additional water required is computed as 25,000 population x 200 gpcd = 5,000,000 gpd. This figure, added to the 8.513 mgd required for the original 25,000 population, makes a maximum water supply requirement of 13.513 mgd.

Since wells and casings may be expected normally to have a useful life of 50 years, Wells Nos. 1, 2, 3, 4 and 5 would be abandoned. New Well Nos. 1, 3, and 4 would be redrilled, cased, perforated and developed in the same area. Pumping equipment also would need to be replaced because its normal useful life is estimated at 25 to 35 years. New well pumps would discharge directly into the mains thereby eliminating the booster equipment. Existing Well Site No. 5 would not be redrilled because the site has not been productive. Well Site No. 2 would not be redrilled because of its proximity to Well No. 1 and the City of Sierra Vista. Well No. 6 will have been operating for 32 years. Well Nos. 1, 3, and 6 would pump to the transmission main to Zone "B". This would require a new water transmission line from Well No. 6 to the intersection with the discharge line from Well No. 3.

USF200002367

The 13.513 mgd water requirement would be supplied by the redrilled wells, the new wells assumed to have been drilled under the 1980 construction program and 6 new wells on the east range with an estimated capacity of 1000 gpm each. Information on the location and capacities of the new wells are based on the most recent hydrogeological information made available through the Sacramento District Office. The six wells designated on Plate "B" as D, F, G, H, J, and K would be drilled, cased and equipped with pumping, treatment, and water transmission facilities. (See TABLE II).

b. Water Treatment

The chemical quality of the water from the test wells as reported in the most recent hydrogeological report is quite good. The only treatment required is therefore chlorination for disinfection and possibly fluoridation.

c. Transmission Lines

The existing cast iron, asbestos cement, and steel lined and coated water transmission lines can normally be expected to have a useful life of 75 years. They should not require replacement by year 2000. However, the normal useful life of steel lines might be 25 years. The existing 12" and 16" steel lines may need replacement.

The 6 new east-range wells should be connected to a water transmission line from Well K constructed at the location indicated on Plate "B". The water transmission lines as shown on the map are sized so that the friction loss from Well J and K to the upper area of Zone B of the distribution system at total design flows is about 62 feet or 27 psi. This is considered a reasonable friction loss for use of optimum size pumps and motors and for equitably balancing operating and capitalization costs.

USF200002368

d. Storage

(1) Zone "A"

Since, for this Alternative, no additional population is anticipated in Zone "A" additional storage facilities will not be necessary. A new 500,000 gallon reservoir would have been built in the year 1980 construction program. This reservoir would be in good condition and so no other storage facilities will be required in Zone "A".

(2) Zone "B"

About 2000 of the additional 25,000 population are expected to be located in Zone "B" as shown on Plate "A". PART II of this report estimated population in Zone "B" as 7980 with a capacity factor multiplier making a total of 9800 design population. The stable population in Zone "B" with 25,000 total population in 1980 is 9240. Adding 2000 people in year 2000 would make a total of 11,240. This is considered to be a stable population and so no capacity factor will be added. The added water demand as described for the year 1980 population is 1500 x 300 gpcd = 450,000 gallons and for the year 2000 population 2000 x 200 gpcd = 400,000 gallons. Adding these to the 2,550,000 gpd from PART II page 8 makes a total demand of 3,400,000 gpd for Zone "B". This amount, plus the 779,000 gpd required for pumping into Zone "A", makes a total of 4,179,000 gpd. Fire demand storage in accordance with TM 5-813-4 would be 50% of 4,179,000 gallons + 2 fires each using 1000 gpm for 4 hours = 2,569,500 gpd.

Under the 1980 construction program a new 1,500,000 gallon reservoir in addition to the existing 1,500,000 gallon reservoir would have been constructed to float on the distribution system. Since the two reservoirs provide for 3,000,000 gallons of storage floating on the distribution system, no

USF200002369

additional storage is indicated. The existing reservoir should still be serviceable if it receives good maintenance.

(3) Zone "C"

About 23,000 of the 25,000 additional population are expected to be located in Zone "C" as shown on Plates "A", and "B". Alternative II, discussed earlier, estimates the 1980 population in Zone "C" at 14,560. This makes a total of 37,560 in year 2000. This population is considered stable and so no capacity factor will be added. Using a water requirement 4,014,000 gpd for the 1980 population and 200 gpcd x 23,000 = 4,600,000 gpd makes a total water requirement in this zone of 8,614,000 gpd.

The 3 million gallon reservoir and 500,000 gallon storage tank floating on the distribution system in this zone supplies the water source for booster pumps to supplement the well pump supply to Zone "B". However, storage here is not considered a requirement for the higher zones because adequate storage, supply, and pumping for them is to be provided. The storage requirement in accordance with TM-5-813-4 is therefore that required for 50% of the domestic demand + 1000 gpm for 2 fires of 4 hour duration. This would be 8,614,000 gallons x 50% + 1000 x 2 x 4 x 60 = 4.79 million gallons.

Since the existing reservoir and elevated tank will be about 55 years old and their normal useful life might be expected to be 50 years, they would be replaced with a 5,000,000 gallon reservoir that would satisfy the total storage requirement.

e. Booster Pumps

In PART II two additional booster pumps with capacity of 535 gpm (513,000 gallons in 16 hours) pumping against the static head and about 30 feet of friction head (about 260) were to be installed to pump from Zone "C" into Zone "B".

USF200002370

An additional pump with the same characteristics would have been installed under ALTERNATIVE II for construction in year 1980. The 2000 added population in this phase of development at 200 gpcd would require 400,000 gpd in Zone "B". Therefore another pump with the same characteristics (535 gpm at 260' TDH) would be added. This would then be a total of 8 booster pumps that lift water from Zone "C" storage into Zone "B". Since the Wherry booster pumps will be over 40 years old, all 4 pumps, interconnecting steel pipe, valves, and meters would be replaced.

f. Controls

The central control panel is located in the mechanical room of the main adminsitration building. All additional pumping equipment should be controlled from the same central panel. This will require an additional control panel adjacent to and integrated into the existing central equipment. However, the existing panel will be over 35 years old, which is its normal useful life, and so it would be completely replaced with one new panel. In order to assure that pumps in all wells are operated occasionally, the controls should alternate the operation of pumps in Test-Well #5, Wells "A", "B", "C", "D", "E", "F", "G", "H", "J" and "K" with existing booster pumps at Wells Nos 3, 4, and 6.

g. Power

In order to serve the new well pump and controls, electric power lines and equipment and control signal lines would be extended to each of the new well sites. With normal maintenance, power lines may last for 75 years and so no power line changes would be required.

USF200002371

7. Alternative IIIb - 50,000 Population in Year 2030

a. General

It is expected that in year 2030 the population at Fort Huachuca will remain at 50,000 people. Due to the expected technological advances in water distribution, use, and reuse, it is assumed that the daily per capita consumption will remain the same as in the year 2000 described for Alternative IIIa. In addition, it is assumed that the demands for all types of irrigation will be automatically controlled by computer. Therefore the actual daily demand for well production will be about 50,000 x 200 gpcd or 10 million gallons.

The water sources in year 2030 may be the same as in year 2000 or may come from other sources such as the Charleston Dam Reservoir. This would be a hydrogeological discussion which is not covered in this report.

Water wells, pumping equipment, transmission, storage and pumping controls may have been outmoded or their practical life exhausted. Some of these are discussed in relation to their normal life.

Assuming that the same type of water sources and equipment are used as in 1980, several changes would need to be made by or before the year 2030.

The useful life of drilled and cased wells is normally under 50 years. Therefore, the wells existing in year 1980 or earlier would require replacement. These wells would consist of the 5 wells discussed in Alternative II earlier. The wells drilled and equipped in year 2000 would probably be satisfactory.

USF200002372

The normal life of pumping equipment may be expected to be 25 to 35 years. Therefore, all pumping equipment installed by year 1980 will have needed replacement by about year 2015.

The normal life of concrete lined and coated steel, cast iron or asbestos-cement transmission mains may be expected to be a maximum of 75 years. Therefore transmission mains except where disturbed or damaged by outside factors would not require replacement.

The normal useful life of concrete water storage structures may be expected to be 50 years. Therefore, tanks or reservoirs constructed by or before year 1980 would have required replacement by or before year 2030.

Controls for all types of equipment including pump motors and water elevation sensing devices may very well be outmoded by newly developed electronic equipment and operations might be controlled by computer. There is also the possibility that even power transmission may be by sound waves, laser beams, or other methods now thought to be exotic. Therefore, there is a possibility that even controls and power transmission equipment installed in the year 2000 might be replaced by year 2030.

Another situation that will undoubtedly be developed will be reuse of 100% of wastewater not only for irrigation but for general use for potable purposes.

Because of the uncertainties in long term predictions regarding development of equipment and methods of operation, no estimates for this year 2030 Alternative will be made.

USF200002373

Gallons per minute = GPM

TABLE II

REQUIRED Hp FOR PROPOSED WELL PUMPS

Well No.	Pumping Rate GPM	Elev.Head at Well Site Ft.	Lift in Well Ft.	Transmission pipe Friction	Pump Column & Discharge Piping Friction Loss, Ft.	Total Head Ft.	Pump Size	Pump Stages	Motor Hp.
When all are Discharging to Zone "C" (Working Water Level 4887')									
Test-Well #5	1500	461	385	38	6	890	14"	13	400
A	1000	547	330	59	8	944	12"	16	300
B	1000	407	425	38	10	880	12"	15	300
C	1000	527	300	47	7	881	12"	15	300
E	1000	622	250	59	6	937	12"	16	300
D	1000	557	335	32	8	932	12"	16	300
F	1000	575	300	39	7	921	12"	16	300
G	1000	622	250	58	6	936	12"	16	300
H	1000	647	250	63	6	966	12"	16	300
J	1000	662	200	62	5	929	12"	16	300
K	1000	697	200	59	5	961	12"	16	300
PROPOSED REPLACED WELL PUMPS WHEN DISCHARGING INTO ZONE "B" (W.W. LEVEL 5110')									
1	600	472	620	75-175*	10	1167-1267*	10"	27-30	250-250
2	600	472	620	75-175*	10	1167-1267*	10"	27-30	250-250
3	650	502	550	200'*	10	1272*	10"	30	250
6	600	464	650	200*	10	1328*	10"	31	250

* 3 pumps operating

USF2000002374

BUDGET COST ESTIMATE

ALTERNATIVE I

Item No.	Description	Costs 1974
a.	Pressure Zone Changes	$13,600
c.	Two Additional Booster Pumps and new Building at Burns & Hatfield Streets including connecting piping, power, and controls	36,550
d.	Booster Pump Replacement with Deep Well Pumps discharging directly into transmission main	75,150
e.	New 1.5 million gallon Storage Tank and connecting piping	252,700
g.	Pump Controls additions	5,700
h.	Redevelop Well No. 5 and lower pump bowls	15,400
i.	Plug Abandoned Water Lines	1,000
k.	Check Valve Changes	6,000
	Total	$406,100
	Say	410,000

USF200002375

BUDGET COST ESTIMATE

ALTERNATIVE Ia(1)

MINIMUM REQUIREMENTS

Item No.	Description	Costs 1974	Costs 1977 Added 7%/yr = 22.5%	Costs 1977 Total
1.	All items under Alternative I except rehabilitating Well No. 5 (assumed not to have been done in 1974)	$390,700	$87,907	$478,607
2.	Check and recondition electric switch gear at Well Nos. 3, 4, 5 & 6	3,000	675	3,675
3.	Complete pumping equipment and transmission main for Test Well #5 including; pump; water treatment equipment; meter; valves & piping; pump house and fence; transmission main (18"); electric power service (using existing line to airport); and control equipment and signal wires.	414,950	93,364	508,314
	Total	$808,650		$990,596
	Say	$810,000		$990,000

USF200002376

BUDGET COST ESTIMATE

ALTERNATIVE Ia(2)

INCLUDING SIZING FOR FUTURE REQUIREMENTS

Item No.	Description	Costs 1974	Costs 1977 Added 7%/yr = 22.5%	Costs 1977 Total
1.	All items under Alternative Ia(1)	$808,650	$181,946	$990,596
2.	Extra for larger size transmission lines to serve 4 future well pumps and larger and longer power transmission lines and equipment for the 4 future well pumps	113,850	25,616	139,466
	Total	$922,500		$1,130,062
	Say	$920,000		$1,130,000

USF200002377

BUDGET COST ESTIMATE

ALTERNATIVE Ib(1)

OPTIONAL SOLUTION 1

Item No.	Description	Costs 1974	Costs 1977 Added 7%/yr = 22.5%	Costs 1977 Total
1.	All items under ALTERNATIVE Ia(2)	$ 922,500	$207,562	$1,130,062
2.	Drill and case and completely equip a new 1000 gpm capacity well "B" and connect to the water transmission main, power source, including control equipment	198,700	44,708	243,408
	Total	$1,121,200		$1,373,470
	Say	$1,120,000		$1,370,000

USF200002378

BUDGET COST ESTIMATE

ALTERNATIVE Ib(2)

OPTIONAL SOLUTION 2

Item No.	Description	Costs 1974	Costs 1977 Added 7%/yr = 22.5%	Costs 1977 Total
1.	All items under ALTERNATIVE I except item d (Deep Well Pumps) and Item "h" which is the reconditioning of Well No. 5	$315,550	$ 70.998	$ 386,548
2.	Complete pumping equipment and transmission main for Test Well #5 as described in ALTERNATIVE 1a(2) including sizing for future estimated needs	113,850 (1a(2))	25,616	139,466
		414,950 (1a(1))	93,364	508,314
3.	Construct and completely equip a new 1000 gpm capacity Well "B" and connect to the water transmission main, power source, and control equipment	198,700	44,708	243,408
4.	Replace motor on booster pump No. 1, overhaul motor starters and replace transformers for booster pump Nos. 1 and 2B; and overhaul switch gear for booster pump Nos. 1 and 2 and all pumps at Well Nos. 3, 4, and 6. Also, lower bowls on Well pump Nos. 1 & 2	25,100	5,648	30,748
	Total	$1,068,150		$1,308,485
	Say	$1,070,000		$1,310,000

USF200002379

BUDGET COST ESTIMATE

ALTERNATIVE Ib(3)

OPTIONAL SOLUTION 3

Item No.	Description	Costs 1974	Costs 1977 Added 7%/yr = 22.5%	Costs 1977 Total
1.	All changes under Alternative Ib(2) except rehabilitating booster pumps #1 and #2 equipment	1,043,050	234,687	1,277,737
2.	Add new well pumps in Well Nos. 1, 2 and 3 and completely equip; new 12" transmission lines from existing No. 3 well pump discharge line to the transmission line to Zone "B"	175,100	39,398	214,498
	Total	$1,218,150		$1,492,235
	Say	$1,220,000		$ 1,490,000

USF200002380

BUDGET COST ESTIMATE

ALTERNATIVE Ib(4)

OPTIONAL SOLUTION 4

Item No.	Description	Costs 1974	Costs 1977 Added 7%/yr = 22.5%	Costs 1977 Total
1.	All changes under Alternative I	$ 406,100	$91,373	$497,473
2.	New 125,000,000 gallon reservoir; new 4 pump booster station in new building; additional control panels and equipment to tie the operation into all existing controls	15,050,000	3,386,250	18,436,250
	Total	$15,456,100		$18,933,723
	Say	$ 15,500,000		$ 18,900,000

USF200002381

BUDGET COST ESTIMATE

ALTERNATIVE II

Item No.	Description	Costs 1974	Costs 1980 Added 7%/yr = 60.6%	Costs 1980 Total
1.	All items under Alternative Ia(2)	(Assumed constructed in 1977 $1,130,062		
2.	Construct and completely equip and house new 1000 gpm capacity wells "B", "C", "A", and "E" and connect them to the water transmission main, power source, and control equipment	$1,122,200	$680,053	$1,802,253
3.	Booster pump (50 Hp connected to existing header of Burns and Hatfield Street Booster Station	6,900	4,181	11,081
4.	Additional central control panel integrated into existing control panel	39,900	24,180	64,080
5.	Water feeder mains inter-connecting existing 24", 18" and 12" transmission mains	71,500	43,330	114,830
6.	A new 500,000 gallon reservoir and removal of 2 existing reservoirs	125,000	75,750	200,750
7.	Irrigation pumps and 6" irrigation main	39,600	23,998	63,598
	Total	$1,405,100		$2,256,592
	Say	$1,410,000		$ 2,260,000

USF200002382

BUDGET COST ESTIMATE

ALTERNATIVE IIIa

Item No.	Description	Costs 1974	Costs 2000 Added 4%/yr = 177.0%	Costs 2000 Total
1.	All items under Alternative II	(Assumed constructed in 1980 $2,256,592)		
2.	Construct and completely equip and house new 1000 gpm capacity wells "D", "F", "G", "H", "J" and "K" and connect them to new water transmission main, power source, and control equipment	2,697,600	4,774,752	7,472,352
3.	New 5,000,000 gallon reservoir connected to piping from existing 3,000,000 reservoir. Remove old reservoir and elevated tank.	841,500	1,489,455	2,330,955
4.	Booster pump connected to existing header of Burns and Hatfield street Booster station	6,900	12,213	19,113
5.	Complete new central panel to control all pump operations, sense and record reservoir levels, and to meter and record flows.	61,000	107,970	168,970
6.	Feeder main interconnecting existing 18" and 12" transmission mains	239,750	424,357	664,107
7.	Replace 12" steel mains	209,300	370,461	579,761
8.	Replace 4 pump Wherry Booster station; replace 3 pump Burns and Hatfield Street station and add new pump, valves, controls etc.	59,300	104,961	164,261
9.	Drill and completely equip new wells at sites of Well Nos. 1,3, and 4; install new pumping equipment in Well No. 6 and connect to 12" main to Well #3 transmission main	608,600	1,077,222	1,685,822
	Total	$4,723,950		$13,085,341
	Say	$4,720,000		$13,100,000

USF200002383

PART II

TABLE OF CONTENTS

		Page No.
I.	INTRODUCTION	1
	1. Authorization	1
	2. Purpose	1
	3. Background	2
II.	PRESENT SITUATION	5
	1. Water Usage	5
	a. Domestic & Fire	5
	b. Irrigation	7
	c. Summary	8
	CHART #1	8a
	CHART #2	8b
	2. Water Sources	9
	a. Springs	9
	b. Deep Wells	9
	TABLE I	10
	c. Wastewater Effluent	11
	3. Booster Pumps	13
	a. General	13
	b. Wells #1 and #2 Booster	14
	TABLE I-A	15
	TABLE II	17
	TABLE II-A	19
	TABLE II-B	20
	c. Boosters at Wells #3,#4,#5 and #6	21
	d. Wherry Boosters	21
	TABLE IV	22
	e. Old Post Boosters	24

Registered Professional Engineer (Civil) CERTIFICATE EXPIRES DEC. 31, 19 HENRY L. ROEHRIG ARIZONA, U.S.A.

USF200002384

Page No.

4. Water Distribution System 25
   a. General 25
   b. Transmission Lines 25

5. Water Pressure Zone Analysis 26
   a. Zone "A" (Old Post Area) 26
   b. Zone "B" 28
   c. Zone "C" 28
   d. Discussion of Another Pressure Zone 29
   e. Summary 30

6. Water Storage 31
   a. Zone A 31
   b. Zone B 32
   c. Zone C 33
      TABLE V 35
      TABLE V (continued) 36
   d. Discussion of Storage Facilities 37

7. Miscellaney 38

III. CONCLUSIONS AND RECOMMENDATIONS 41

1. Conclusions 41
   a. Pressure Zone Changes 41
   b. Production 41
   c. Wells #1 & #2 42
   d. Well #5 42
   e. Test Well #5 42
   f. Well Pump Meters 43
   g. Zone "B" Storage 43
   h. Spring Production 44
   i. Old Post Reservoir 44
   j. Booster Pump #1, #2A and #2B 44

Page No.

k.	Operational Controls	45
l.	Abandoned Lines	45
m.	Booster Replacement	45
n.	Wastewater Reuse	45
o.	Wherry Booster Addition	46
p.	Automatic Controls	46
q.	Storage Level Adjustments	47
r.	Irrigation Scheduling	47
2.	Recommendations	48
a.	Pressure Zone Changes	48
b.	Well #5 Pumping Equipment & Transmission Main	48
c.	Additional Booster Pumps	49
d.	Booster Pump Replacement	49
e.	Additional Storage	49
f.	Power Supplies & motors	50
g.	Pump Controls	50
h.	Redevelop Well #5	50
i.	Plug Abandoned Lines	51
j.	Reset Well #1 and #2 Bowls	51
k.	Valving Changes	51
l.	Extend non-potable irrigation system	51
m.	Additional Well Development	52
n.	Booster Pumping Station Replacement	52
	APPENDIX	
	REFERENCES	53
	PLATE NO. 1	
	PLATE NO. 2	
	TABULATION SHEETS	54 - 63
	CURVES	1 - 5
	SKETCHES	1 - 16

USF200002386

INVESTIGATIONS AND RECOMMENDATIONS
FOR UPGRADING THE WATER SYSTEM AT
FORT HUACHUCA, ARIZONA

I. INTRODUCTION

1. Authorization

The investigation reported herein was authorized through the District Engineer Sacramento by the Department of the Army, South Pacific Division, Corps of Engineers, Sacramento, California, through Job No. Fort Huachuca - 72-O&M-A, Directive No. SPD-3, dated 27 June 1972.

2. Purpose

Since its reestablishment on 1 February 1954, Fort Huachuca has been serving in several capacities. It is now under the control of the U.S. Army Strategic Communications Command.

In contrast to its 73 year history previous to 1954, development of permanent facilities and increased personnel has been accomplished in a rapid but orderly manner. Sources knowledgeable of the Fort's future agree that its expansion has probably reached its expected peak and that stable population conditions will prevail. It is therefore appropriate that the existing water supply and distribution system be re-evaluated and that recommendations be made to modify the current situation to one that will fully satisfy the needs of the present which in turn will satisfy the future.

USF200002387

3. Background

From its establishment in March 1877 to 1939 the water supply came from springs in Huachuca and Garden Canyons, which are located several hundred feet above and southerly of the building complex. A 200,000 gallon ground storage masonry basin, built in 1911, and another 250,000 gallon concrete ground reservoir constructed in 1939, served as the springs' collection point. This system is known as the Old Post Reservoir (See Sketch #1).

In 1939 and 1940 Wells #1 and #2 (See Tabulation Sheets 1 and 2) were drilled at the Fort's Main Gate. Their supply was delivered via a 10" tar lined and coated steel pipe nearly 20,000 feet long to the Old Post Reservoir via Booster Pumps #1 and #2B. This was a supply main with no connections. (See Sketch #2). A 500,000 gallon elevated steel tank was installed in 1941 between the wells and the Old Post Reservoir near 4th Street and Railroad Avenue. It received its supply from the surge tank at Well #2 via a 12" steel line connected to Booster Pump 2-A. (See Sketch #3).

Increased activity in 1942 due to World War II, required the construction of Wells #3, #4, and #5 (See Tabulation Sheets 3-5) and three 50,000 gallon surge tanks from which Booster Pumps #3, #4, and #5 take suction. Also a three million gallon concrete ground storage reservoir together with distribution and supply mains were constructed as well as another 500,000 gallon elevated steel tank. The second elevated tank was disconnected in 1946 by the War Assets Administration (See Sketch #4). These 5 wells and the springs constituted the water supply facilities up to 1954. At that time the springs were discontinued as a source.

USF200002388

By 1956 the Wherry, MCA, and Capehart Housing projects were constructed for military personnel assigned to the Electronics Proving Ground and their families. At this time the Wherry Booster Station, consisting of three booster pumps drawing from the 3 million gallon reservoir, was constructed to pump into a direct pressure distribution system serving the portions of these projects which were located too high in elevation to receive adequate elevation pressure from the reservoir. In 1958 a 1-1/2 million gallon prestressed concrete tank with high water elevation 223 feet above that in the 3 million gallon reservoir was completed. The Wherry Booster Station was then pumping both into the distribution mains and into the new reservoir which floats on the distribution system. (See Sketch #5).

Well #6, (see Tabulation Sheet #6) which was drilled in 1958, was tested and put into service. At approximately the same time, a fourth booster pump was installed at Wherry Station.

In 1962 three booster pumps were installed below the 1-1/2 million gallon tank to take suction from that tank to supply the Old Post Reservoir. The supply main from Well Pump Booster #1 and #2B was then to discharge overhead into the 1-1/2 million gallon reservoir. However, a junction box was installed so that this supply main could serve either the 1-1/2 million gallon and/or the Old Post Reservoirs.

In 1969, as a result of a 1966 study (WATER RESOURCES OF FORT HUACHUCA MILITARY RESERVATION, SOUTHEASTERN ARIZONA: GEOLOGICAL SURVEY WATER SUPPLY PAPER 1819 D), a 12" concrete steel cylinder pipe was installed from Garden Canyon Springs to the 1-1/2 million gallon prestressed tank and an 8" line was constructed between Huachuca Canyon Springs and Old Post Reservoir. The 30 year old 10" line from Wells 1 and 2 was

USF200002389

replaced by a 12" concrete steel cylinder pipe that is connected into the 12" Garden Canyon Line (see Sketch #6). The old 10" line is only usable from the booster station taking suction from the 1-1/2 million gallon tank to the Old Post Reservoir.

As noted on page 18 of TEST WELL DRILLING AND STUDY OF HYDROGEOLOGIC CONDITIONS AT FORT HUACHUCA by the Corps of Engineers, April 1972, a full production well, labeled Test Well No. 5 (see Tabulation Sheet #7), has been tested at 1650 gpm with 59' of drawdown. This capped well is located approximately 4 miles north by northeast of the Fort's central distribution center (see Sketch #16).

As a supplement to the total water supply requirements of the Fort, a project was completed in September 1972 to store 2.5 million gallons of secondary treated chlorinated wastewater plant effluent. This source is used in supplying the separate piping systems for irrigation of the 18 hole golf course and Chaffee parade field. This recent innovation is expected to be sufficient to supply all present irrigation requirements for those areas that are now or may later be approved by the medical departments for irrigation with treated wastewater plant effluent excepting such areas as the Post cemetery which is too remote to make construction of piping to it practical.

The above is a general summary of events and significant developments of the water supply facilities at Fort Huachuca. Plate #1 illustrates the present overall supply and distribution facilities.

USF200002390

II. PRESENT SITUATION

1. Water Usage

a. Domestic and Fire

The population consists of two classifications: 24 hour occupancy and 8 hour occupancy. This report will separate the population into those occupying each of 3 primary water pressure zones (See Plate I at end of this Section). The highest zone, Zone A, extends from the Post cemetery at elevation 5200 feet to Von Herman Circle at elevation 4920 feet. Zone B is from Henry Circle at elevation 5030 feet to Faison Circle at elevation 4810 feet. As the lowest, Zone C extends from the lower edge of Zone B to the northern boundary of the Fort at elevation 4400 feet. The present (1972) populations of these zones are as follows:

Zone	24-hour Occupancy	8-hour Occupancy	Equivalent 24-hour occupancy
A	1475	1400	1940
B	7100	2650	7980
C	2620	1325	3060

The present equivalent 24 hour occupancy population is 12,980. Using a capacity factor of 1.23, the design population is 15,970. Tabulated population reports for this period were supplied by the Post Housing Office. As indicated in the section "Purpose", this present equivalent population figure is expected to remain reasonably constant.

USF200002391

The following calculations and evaluations of demand of the potable water system are based on the criteria set forth in the U.S. Army Corps of Engineers Manual, Series TM 5-813.

ZONE A

Equivalent Population x Capacity Factor = Design Population

1940 x 1.23 = 23.85

Required Daily Domestic Demand:

2385 x 150 gpcpd = 358,000 gallons

Required Fire Demand:

1/2 x 358,000 + 1000 gpm x 4 hrs = 419,000 gallons

Since Fire Demand is greater than Domestic Demand for Zone A, it will be used in further analysis.

ZONE B

7980 x 1.23 = 9,800 Design Population

Required Daily Domestic Demand:

9,800 x 150 gpcpd = 1,470,000 gallons

Required Fire Demand:

1/2 x 1,470,000 + 1000 gpm x 2 fires x 4 hrs = 1,215,000 gallons

ZONE C

3060 x 1.23 = 3760 Design Population

Required Daily Domestic Demand:

3760 x 150 gpcpd = 564,000 gallons

Required Fire Demand:

1/2 x 564,000 + 1000 gpm x 4 hrs = 522,000 gallons

The total water required in all 3 zones for required daily domestic and fire demand is 419,000 + 1,470,000 + 564,000 = 2,453,000

USF200002392

b. Irrigation

Using irrigation maps furnished by Buildings & Grounds section, it is estimated that there are 90 acres of lawns, trees and shrubs in public areas not irrigated by residents. This does not include the golf course and Chaffee Field which are irrigated with wastewater effluent. A contractor is hired to operate the manually controlled irrigation systems in these public areas. Irrigation of similar plant life around the housing projects is done by the residents of houses. These areas are estimated at 150 acres. Monthly water production records indicate that actual usage increases during the irrigation period by approximately 2.0 M.G.D. (See Chart 2 at the end of this Section). These estimates of acreages were derived from general site maps.

It is generally accepted practice to estimate that 9000 gallons per acre per day are necessary to maintain good lawn growth. Therefore, irrigation demand during the two peak months of May and June would be 240 acres x 9000 gpapd = 2,160,000 gallons per day. This estimate is fairly close to the observed increases in the recorded water production for the two maximum usage months. Average irrigation demand estimates according to Zones, is as follows:

Zone A	40 acres x 9000 gpdpac =	360,000	gpd
Zone B	120 acres x 9000 gpdpac =	1,080,000	gpd
Zone C	80 acres x 9000 gpdpac =	720,000	gpd
		2,160,000	

There is a standing Fort order that there shall be no irrigation on Wednesday. This allows reservoirs to be partly replenished prior to Thursday. The contractor that irrigates public areas not adjacent to residences usually waters at night when other irrigation is low. Seeds for grass and plants for landscaping are furnished by the Post, therefore some control is exercised over the planting of things that would use a large amount of water.

USF200002393

C. Summary

Total consumption during the peak period of May and June is as follows:

ZONE A

419,000 gallons Fire Demand
360,000 gallons Irrigation

779,000 Total Daily Usage

ZONE B

1,470,000 gallons Domestic Demand
1,080,000 gallons Irrigation

2,550,000 Total Daily Usage

ZONE C

564,000 gallons Domestic Demand
720,000 gallons Irrigation

1,284,000 Total Daily Usage

Total Daily Usage = 4,613,000 gallons on an irrigation day.

This figure represents the amount necessary to satisfy the criteria set forth in the TM 5-813 series manuals, and is also the actual maximum use. It should be noted that the maximum daily water production figure was set in June 1972, at 5.34 M.G.D. (See Chart #2). Subtracting the 250,000 gallons that was used for golf course watering, which will be replaced by wastewater plant effluent in the future, will make the future peak daily production about the same as the daily design figure. Therefore, during the months of extensive irrigation, the demand occasionally approaches or equals the design daily usage.

USF200002394

CHART #1

MONTHLY WELL PRODUCTION (1,000 GALLONS)

FORT HUACHUCA, ARIZONA

(PREPARED FROM POST REPORTS)

	- year -						
	1966	1967	1968	1969	1970	1971	1972
January	42,887	62,824	55,562	59,517	64,151	NA	59,994
February	40,443	70,192	63,317	57,893	71,839	50,798	58,885
March	77,902	110,266	77,481	80,824	82,113	92,909	84,957
April	82,837	89,402	80,781	97,623	94,846	83,860	98,282
May	95,971	114,644	120,722	116,505	123,237	117,417	129,137
June	105,577	119,959	124,496	140,426	125,683*	131,222	111,038
July	90,718	79,668	94,065	95,415	99,602*	87,340	106,908
August	67,469	77,962	79,975	86,713	77,655*	72,513	84,054
September	64,492	71,673	95,573	96,506	79,327	88,986	66,246
October	73,448	73,038	105,465	104,089	67,073	68,930	65,407
November	65,375	70,350	75,537	72,269	60,775	70,669	52,866
December	55,515	51,837	59,028	60,666	48,066	56,371	-
TOTAL	862,624	991,815	1,032,002	1,068,446	994,367		
AVERAGE							
Monthly	71,885	88,651	86,000	89,037			
Daily	2,360	2,720	2,820	2,920			

* Huachuca and Garden Canyon Spring collection systems contributing to water system.

USF200002395

CHART #2

MAXIMUM DAY WATER PRODUCTION BY MONTHS

(Million Gallons)

FORT HUACHUCA, ARIZONA

(PREPARED FROM POST REPORTS)

Month	1966	1967	1968	1969	1970	1971	1972
	- year -						
January	2.01	3.00	2.69	2.22	3.81	NA	4.85
February	1.89	3.37	3.55	3.21	3.45	3.29	3.63
March	3.87	5.16	3.72	4.19	3.82	5.19	4.25
April	4.34	4.64	4.15	4.37	4.00	3.93	4.15
May	4.20	4.80	5.00	4.96	4.90	5.14	5.09
June	4.83	4.71	4.96	5.02	NA	5.07	5.34
July	4.46	3.54	4.49	4.94	5.30	4.95	5.05
August	2.69	4.05	3.19	3.92	4.65	3.57	4.13
September	2.93	3.25	4.34	4.30	4.16	5.22	3.34
October	3.39	3.02	4.56	4.48	3.41	3.30	3.54
November	2.62	2.93	3.42	3.56	3.34	2.74	2.29
December	2.56	2.25	2.56	2.76	2.97	2.60	

USF200002396

2. Water Sources

a. Springs

Huachuca Canyon Springs are closer to the Post and higher than the Garden Canyon Springs but they develop less flow. A serious attempt by the U.S. Geological Survey to monitor total spring flow in each canyon from 1960 to 1963, indicated that during the driest period in sixty years, Huachuca Canyon produced about a minimum of 20 gpm and Garden Canyon yielded a minimum of 83 gpm. During that same three year period a combined maximum of over 2300 gpm was recorded. However, in the two-and-a-half year period since 1969 when the new water line and collection facilities were developed, Huachuca and Garden Canyon Springs have been metered. The greatest daily production was recorded as 1,190,000 gallons on September 14, 1971. The minimum was 9,000 gallons on August 23, 1972.

The springs contribute a noteworthy amount of good quality water to the Post water system as well as providing replenishment water to the two lakes at the golf course and the officers club. The pipelines are being provided with automatic discharge valves so that when the first water following a rainstorm is turbid, it will be discharged to the ground surface thereby preventing contamination of the Post water system with the muddy material. The water flow rates from the springs vary too much to be seriously considered as a basic water source during the dry season when maximum use occurs. However, this water supply provides a valuable resting period for the wells during the spring when snow melts feed the springs and during the rainy season.

USF200002397

b. Deep Wells

The basic source of potable water for the Post is from the underground aquifer located about 500 feet below the ground surface. Pumps from 6 wells drilled into this aquifer discharge into adjacent surge tanks. Booster pumps, taking suction from these tanks, deliver water on demand to the various storage reservoirs which float on the 3 pressure zones in the distribution system.

As outlined in Part I, paragraph 3 of this report, the six wells were put into production at various times as water demands increased during the last 30 years. (See Tabulation Sheets #1-6). While all of them were considered good producers initially, significant changes in their capacity have been noted. Listed below in TABLE I is a comparison of each from tests performed by Post personnel showing these changes.

TABLE I

	PRODUCTION PER FOOT OF DRAWDOWN ON INDICATED DATES	1971 PRODUCTION PER FOOT OF DRAWDOWN
Well #1	16 gpm/ft @ 550 gpm (1963)	15.4 gpm/ft @ 593 gpm
Well #2	21 gpm/ft @ 620 gpm (1963)	17.1 gpm/ft @ 615 gpm
Well #3	25 gpm/ft @ 500 gpm (1963)	17.7 gpm/ft @ 760 gpm
Well #4	32 gpm/ft @ 450 gpm (1963)	40.7 gpm/ft @ 611 gpm
Well #5	10 gpm/ft @ 650 gpm (1963)	6.2 gpm/ft @*575 gpm
Well #6	25 gpm/ft @ 708 gpm (1958)	37.5 gpm/ft @ 601 gpm

* This quantity is presently only 450 gpm

At the present, water supply capacity from the wells for a 16 hour day, as recommended by Army Training Manual TM 5-813-2, would be about 3,500,000 gpd. However, these wells have been pumped as much as is necessary to meet the daily demand. While the quality of the water has not changed noticeably in the last 30 years, the level of the water table has

USF200002398

dropped at the rate of 2-1/2' to 3' per year since 1958. Similar drops have occured in wells in the nearby city of Sierra Vista.

The Army Corps of Engineers Study that was published in January 1972 included the drilling and developing of a well for production purposes. Data on this Test Well #5, listed in the study's summary, is as follows:

The well is located in the Northeast 1/4 of Section 16, Twp 21S R20E of G & SRB & M.

Well Depth - 807'
Casing Diameter - 16"

Test Rates:

Drawdown		Specific Capacity
375 gpm	15 feet	25 gal/ft.
710 gpm	26 feet	27.3 gal/ft.
1150 gpm	40 feet	28.7 gal/ft.
1500 gpm	52 feet	28.9 gal/ft.
1650 gpm	59 feet	27.9 gal/ft.

Site Surface elevation - 4426
Static Water elevation - 4144

This data indicates that optimum usage of the well would be at 1500 gpm or 1.44 mgd for a 16 hour pumping schedule. At present the well is capped awaiting allocation of money and engineering plans for construction of pumping equipment and a transmission main for connecting it to the existing system. Approximately 15,000 feet of main will be required.

c. Wastewater Effluent

Although it is not an all-purpose source, stored effluent from the 2 wastewater plants at which secondary treatment and chlorination are provided, is being used to irrigate the 18 hole golf course and Chaffee Parade Grounds which collectively demand as much as a million gallons per day during the early summer season.

USF200002399

A 1.8 million gallon concrete lined lagoon was completed about 1 September 1972, adjacent to Plant #2, to supplement the existing 0.7 million gallon earthen lagoon at Plant #1. A valved 4" potable water line is available to add to the non-potable supply for irrigation purposes if ever needed. The lowest recorded daily flow as averaged from the two successively lowest flow days during the high water use months of April, May, June, July, and August in 1972 was 1,213,000 gallons. The two lagoons, with a capacity of 2.5 million gallons will provide storage for two days of minimum flow from the two wastewater plants. This storage will accommodate a demand of as much as 2 million gallons on any one day and a subsequent demand of 1,213,000 gallons on the following days during the high irrigation period of the year.

Further relief of the potable water shortage could be provided if some of the other areas now being irrigated with potable water could be irrigated with wastewater. This change would depend on approval of the medical department and addition of pumping and piping for handling the wastewater. The areas around the residential areas and school playgrounds may not be approved for irrigation with wastewater plant effluent however, even though the residential irrigators are the major users of potable water for this purpose.

The phosphate content of the wastewater was measured to determine if it would be acceptable for replenishment of the evaporation from the lakes in lieu of using potable water. The wastewater as tested and analyzed with our equipment and confirmed by the Post laboratory technician contains about 8 mg/l of phosphates. This quantity is quite beneficial for plant life both on land and in water. However, profuse plant growth in the lakes is quite objectionable. Therefore, the

USF200002400

wastewater should be used and its use extended to the maximum possible for irrigation of land plants and the low phosphate spring or well water should be used for replenishing the water evaporating from the lakes.

In addition to the extensive construction and maintenance costs, another factor which makes use of non-potable water undesirable in inhabitated areas is the danger of possible cross connections between the potable and non-potable systems by unknowing maintenance and construction personnel.

3. Booster Pumps

a. General

At present there are 14 existing potable water booster pumps which are identified as follows:

Facility	Location	Rated Capacity	THD
Booster #1	adjacent to Deep Well #1 & #2	450 gpm @	700 feet
Booster #2A	adjacent to Deep Well #1 & #2	700 gpm @	270 feet
Booster #2B	adjacent to Deep Well #1 & #2	700 gpm @	715 feet
Booster #3	adjacent to Deep Well #3	900 gpm @	320 feet
Booster #4	adjacent to Deep Well #4	900 gpm @	320 feet
Booster #5	adjacent to Deep Well #5	900 gpm @	320 feet
Booster #6	adjacent to Deep Well #6	700 gpm @	290 feet
Booster #6A	at Wherry Booster Station	*560 gpm @	252 feet
Booster #6B	at Wherry Booster Station	*590 gpm @	252 feet
Booster #6C	at Wherry Booster Station	*520 gpm @	252 feet
Booster #6D	at Wherry Booster Station	*680 gpm @	252 feet
Booster #7A	at 1-1/2 M Reservoir	500 gpm @	240 feet
Booster #7B	at 1-1/2 M Reservoir	500 gpm @	240 feet
Booster #7C	at 1-1/2 M Reservoir	500 gpm @	240 feet

*Actual Field Test Observations

In addition to these 14 potable water booster pumps, there are 4 boosters for the non-potable irrigation systems.

USF200002401

b. Wells #1 and #2 Boosters (See Tabulation Sheet #8, Curves #1, #2 and #3, and Sketch #7 in APPENDIX): Boosters #1, #2A, and #2B can take suction from either of two 50,000 gallon concrete surge tanks located at Wells #1 and #2. Sketch #7 depicts the piping connections. As shown, each well can pump into either surge tank so that either one can be drained for repair work.

Boosters #1 and #2B discharge into a 16" steel main which connects into the 12" steel cylinder concrete pipe that supplies the 1-1/2 million gallon reservoir. The curves for these booster pumps (curves #1 and #3, in the APPENDIX) show design heads of 450 gpm at 700 feet TDH and 700 gpm at 715 feet TDH. The elevation head is about 470 feet. If there is 161 feet of friction head or a total head of 631 feet, the curves indicate delivery capacities of 630 gpm and 910 gpm respectively. (The 161' is from using "C" value of 100 and calculating losses in 17000' of 12" and 1600' of 16" pipe). Maximum pumpage requirements are normally required when the Huachuca springs are producing very little. Its flow from where it enters the transmission main near Huachuca Springs Road was therefore disregarded. The delivery from the booster pumps if operated at maximum rated capacity should therefore be 1540 gpm.

An attempt was made to determine the actual unrestricted flows from these 30 year old boosters, but it was found that they were valve throttled so that full flow was not being developed. The ampmeter on the booster pump motors registered excessive current usage when these restricting gate valves were momentarily opened fully. It is reported by Post operating personnel that these three booster pumps cannot be operated simultaneously without throwing the circuit breaker. However, Post personnel report that impellers have been examined within the last five years and that they show no wear and should still deliver their design capacity. As a result of the throttling, the combined flow of Booster #1 and #2B is less than the combined production of Wells #1 and #2. 1040 gpm against 1208 gpm. (See Table IA, page 15 and TABLE I, page 10).

USF200002402

TABLE IA

WELL BOOSTER DELIVERY

Observations by: Nocho, Roahrig, Tencza
1-10-73 on charts in control panel and
meter dial on 2A

BOOSTER #1	(Throttled) 300 gpm	32.5 Amp draw (Motor is very noisy)
BOOSTER #2B	(Throttled) 830 gpm	47 Amp draw (Motor relatively quiet)
BOOSTER #1 & 2B	(Throttled) 1040 gpm	48 Amp & 32.5 Amp draw
BOOSTER #2A	(Throttled per existing operation to pull 32.5 amps while only it and #3 were operating) 900 gpm	
BOOSTER #3	(Throttled per existing operation while only it and #2A were operating) 780 gpm (at central recorder)	
BOOSTER #2A	(Throttled per existing operation while all well site boosters were operating) 630 gpm (at meter)	
BOOSTER #3	(Throttled per existing operation while all well site boosters were operating) 580 gpm (at central recorder)	
BOOSTER #4	(Throttled per existing operation while all well site boosters were operating) 450 gpm (at central recorder)	
BOOSTER #5	(Throttled per existing operation while all well site boosters were operating) 440 gpm (at central recorder)	
BOOSTER #6	(Throttled per existing operation while all well site boosters were operating) 580 gpm (at central recorder)	

USF200002403

Booster Pump #2A takes suction from the same surge tank header and discharges to the lower zone distribution system on which the 3 million gallon reservoir and the 0.5 million gallon elevated tank float. Since the other two boosters (#1 and #2B) should be able to pump more than Well #1 and #2 can deliver, and since these booster pumps deliver water direct to the 1-1/2 million gallon reservoir without power being lost from boosting again at the Wherry Booster Station, there should be no need for Booster Pump #2A to be activated under normal conditions.

Curve #2 indicates that the shut-off head on pump #2A is 295'. Therefore when the other booster pumps discharging into the distribution system create a 45' friction loss, this pump should go entirely static. If the booster pumps from Wells #3, #4, #5, and #6 (See Curve #4-5 and Tabulation Sheet #9) were operated together at unthrottled design flow, the friction loss at the discharge point of Pump #2A into the system would be about 74 feet. (See Table II). However, the actual delivery of #2A with the above described boosters operating at the same time in a throttled condition was 630 gpm (See TABLE IIB, page 19a). This indicates that the pump curve information may be erroneous.

USF200002404

TABLE II

FRICTION LOSSES IN WATER TRANSMISSION SYSTEM TO 3M·G· RESERVOIR & 0.5M.G. ELEVATED TANK WHEN BOOSTER PUMPS #3,4,5, & 6 ARE DISCHARGING AT THEIR RATED CAPACITY AGAINST STATIC PRESSURE AND AN ESTIMATED 74' FRICTION LOSS

* "C" - 100

(See Sketch #13 in APPENDIX)

Northerly

Pump #4 -	1550' - 12"	@	770 gpm	@	2.6'/1000'	=	4.0'
	2800' - 16"	@	820 gpm	@	0.7'/1000'	=	2.0'
	3100' - 12"	@	1530 gpm	@	9.1'/1000'	=	28.2'
	2200' - 12"	@	1470 gpm	@	8.4'/1000'	=	18.5'
	1200' - 12"	@	1270 gpm	@	6.3'/1000'	=	7.6'
	550' - 12"	@	970 gpm	@	2.9'/1000'	=	1.5'
	750' - 12"	@	1220 gpm	@	5.9'/1000'	=	4.4'
	2400' - 14"	@	1220 gpm	@	2.8'/1000'	=	6.7'

Total Friction Loss = 72.9'

Southerly

Pump #6 -	50' - 12"	@	650 gpm	@	1.9'/1000'	=	0.1'
	900' - 6"	@	600 gpm	@	0.4'/1000'	=	0.4'
	2200' - 12"	@	1370 gpm	@	7.4'/1000'	=	16.3'
	4300' - 12"	@	1370 gpm	@	7.4'/1000'	=	31.8'
	1150' - 12"	@	900 gpm	@	3.4'/1000'	=	3.9'
	1750' - 12"	@	650 gpm	@	1.9'/1000'	=	3.3'
	1150' - 12"	@	915 gpm	@	3.5'/1000'	=	4.0'
	1050' - 12"	@	965 gpm	@	3.8'/1000'	=	4.0'
	4500' - 12"	@	715 gpm	@	2.3'/1000'	=	10.4'

Total Friction Loss = 74.2'

Balanced friction loss is about 74'

In TABLE II above the booster pump rated discharges are taken from heads shown on pump curves #4 and #5. Comparisons with discharges from matching well pumps in TABLE I page 10 follow:

	Booster Pumps				Well Pumps
Well No.	Static Head	Friction Head	Total Head	Discharge GPM	Discharge GPM
#3	269'	74'	343'	770	760
#4	269'	74'	343'	770	611
#5	278'	74'	352'	710	575**(450)
#6	241'	74'	315'	650	601

**Estimated present quantity

*A "C" value of 100 was used for these calculations because that is standard practice for older pipes some of which are ferrous such as these and it was not known what the friction coefficient should be.

When all booster pumps in the well site areas are operating at their rated capacity against both the static and 74' friction head, they will pump approximately (from pump curves):

Booster #3	=	725 gpm	
Booster #4	=	755 gpm	
Booster #5	=	705 gpm	
Booster #6	=	680 gpm	
Booster #2A	=	0	(goes static 45' fr. loss)
		2865 gpm	

MAXIMUM RATED BOOSTER PUMP CAPACITY INCLUDING BOOSTER PUMPS #1 AND #2B IS 2865 + 1540 = 4405 gpm.

However, the #3, 4, 5, and 6 boosters are throttled by adjusting the outflow gate valves of each. Operating personnel explained that this practice limits the booster capacities to match the well pump capacities when the friction loss is low with only one booster pump operating. As a result, the friction loss corresponding to this reduced flow is less than 45' so that Booster Pump 2-A is able to contribute flow to the system. This condition was observed in the field. (See Tables I-A, II-A, and II-B, and Sketch #14).

Since Booster Pump #2A cannot theoretically be used during maximum water requirements, it should normally be valved off from the system and used only by manual operation when water use requirements are low and operation personnel may want to rest other pumps for servicing or for keeping #2A in operating condition for service during times of low water use.

USF200002406

TABLE II-A

FRICTION LOSSES IN WATER TRANSMISSION SYSTEM TO 3M.G. RESERVOIR & 0.5M.G. ELEVATED TANK WHEN BOOSTER PUMPS #2A & #3 ARE OPERATING AT OBSERVED DISCHARGES WITH USUAL THROTTLED CONDITION

* "C" = 120

(See Sketch #13A in APPENDIX)

Northerly

Pump #3 -	1700' - 12"	@	780 gpm	@	1.9'/1000'	=	3.2'
	5200' - 16"	@	880 gpm	@	0.6'/1000'	=	3.1'
	3100' - 12"	@	880 gpm	@	2.3'/1000'	=	7.1'
	2200' - 12"	@	830 gpm	@	2.1'/1000'	=	4.6'
	1200' - 12"	@	730 gpm	@	1.7'/1000'	=	2.0'
	550' - 12"	@	600 gpm	@	1.1'/1000'	=	0.6'
	750' - 12"	@	750 gpm	@	1.7'/1000'	=	1.3'
	2400' - 14"	@	750 gpm	@	0.8'/1000'	=	1.9'
							23.8'

Southerly

Pump #2A -	1300' - 12"	@	900 gpm	@	2.4'/1000'	=	3.1'
	4300' - 12"	@	800 gpm	@	2.0'/1000'	=	8.6'
	1150' - 12"	@	600 gpm	@	1.1'/1000'	=	1.3'
	1750' - 12"	@	450 gpm	@	0.7'/1000'	=	1.2'
	1150' - 12"	@	610 gpm	@	1.2'/1000'	=	1.4'
	1050' - 12"	@	610 gpm	@	1.2'/1000'	=	1.3'
	4500' - 12"	@	460 gpm	@	0.7'/1000'	=	3.2'
							20.1'

Balanced friction loss is about 22'.

* A "C" value of 120 was used for these calculations because it was observed from pump curves under actual operating conditions that the pressure against the pumps from friction losses did not indicate that a higher friction was actually being developed.

USF200002407

TABLE II-B

FRICTION LOSSES IN WATER TRANSMISSION SYSTEM TO 3 M.G. RESERVOIR & 0.5M.G. ELEVATED TANK WHEN BOOSTER PUMPS #2A, #3, #4, #5, AND #6 ARE OPERATING AT OBSERVED DISCHARGES WITH USUAL THROTTLED CONDITION

* "C" = 120
(See Sketch #13B in APPENDIX)

Northerly

Pump #6 -	50'	- 12"	@	580	gpm @	1.1'/1000	=	0.1'
	1500'	- 16"	@	480	gpm @	0.2'/1000	=	0.1'
	2800'	- 16"	@	930	gpm @	0.6'/1000	=	1.7'
	3100'	- 12"	@	1370	gpm @	5.2'/1000	=	16.1'
	2200'	- 12"	@	1300	gpm @	4.9'/1000	=	10.8'
	1200'	- 12"	@	1230	gpm @	4.3'/1000	=	5.2'
	550'	- 12"	@	1080	gpm @	3.5'/1000	=	1.9'
	750'	- 12"	@	1350	gpm @	5.1'/1000	=	3.8'
	2400'	- 14"	@	1350	gpm @	2.4'/1000	=	5.8'
								45.5'

Southerly

Pump #6 -	50'	- 12"	@	580	gpm @	1.1'/1000	=	0.1'
	900'	- 16"	@	100	gpm @	0.1'/1000	=	0.1'
	2200'	- 16"	@	680	gpm @	0.4'/1000	=	0.9'
	4300'	- 12"	@	1310	gpm @	4.9'/1000	=	21.0'
	1150'	- 12"	@	900	gpm @	2.4'/1000	=	2.8'
	1750'	- 12"	@	650	gpm @	1.4'/1000	=	2.5'
	1150'	- 12"	@	1060	gpm @	3.3'/1000	=	3.8'
	1050'	- 12"	@	1110	gpm @	3.6'/1000	=	3.8'
	4500'	- 12"	@	840	gpm @	2.1'/1000	=	9.5'
								44.5'

Balanced friction Loss = 45'.

At flow from 2 pumps (TABLE II-A) friction loss was 22' or a difference of 23'. The flow drop from Booster #3 was 200 gpm less than when only pumps #2A and #3 were operating. The Booster #3 curve indicated that balanced friction loss should have been 30'. This would indicate that the "C" factor should have been lower but this one curve is not entirely conclusive evidence for a decisive assumption.

* A "C" value of 120 was used for these calculations because it was observed from pump curves that the pressure against the pumps from friction losses did not indicate that a higher friction was actually being developed.

USF200002408

c. Boosters at Wells #3, #4, #5 and #6 (See Tabulation Sheet #9, Curves #4 and #5, and Sketches #8 and #9 in APPENDIX).

Booster pumps at each of these well sites have as much or more capacity for discharge against the maximum head developed in the transmission mains than the well pumps can furnish to the respective 50,000 gallon tanks from which the boosters take suction. Their total capacities are shown in TABLE II, page 17, as 2865 gpm against maximum operating pressures.

However, due to the throttling conditions mentioned above, the combined flow as noted in TABLE IA, page 15 is only 2050 gpm. According to TABLE I, page 10, and Tabulation Sheets 3-6, total Well Production from Wells #3, 4, 5, and 6 should exceed 2420 gpm. This indicates that the total booster outflow was being throttled excessively in the data noted in TABLE IA, page 15.

d. Wherry Boosters

Booster Pumps 6A, 6B, 6C, and 6D (See Sketch #11 and Tabulation Sheet #10) each have a maximum positive suction head of 37' from the 3 million gallon reservoir. A maximum static head of 260 feet is reached at the pump levels when the 1-1/2 million gallon reservoir is full. Actual operating conditions are noted in TABLE IV, Page 22.

USF200002409

TABLE IV

FRICTION LOSS EXAMINATION

FROM

WHERRY BOOSTER STATION

Observations on 11/20/72:
by: Nocho - Roahrig - Tencza

Pumps On	Discharge GPM	Pumps On	Discharge GPM
#6A	560	#6A	560
6B	590	6A-6B	1120
6C	420	6A-6B-6C	1520
6D	680	6A-6B-6C-6D	1960
TOTALS	2350		* 1960

* This total confirmed by observation again on 11/27/72

Observations on 11/27/72:

Pumps On	Time	Pump Discharge Pressure	10" Station Outlet	Hydrant, 900' S. on Rucker
None	10:25 AM			107 psi
#6A	10:32 AM	109 psi	109 psi	113 psi
#6A, 6B	10:36 AM	112 psi	111 psi	115 psi
#6A, 6B & 6C	10:30 AM	114 psi	112 psi	112 psi
#6A, 6B, 6C & 6D	10:44 AM	116 psi	115 psi	110 to 115 psi

USF200002410

Elevation differential from the hydrant on Rucker to approximately 5' below the tank overflow = 5105 - 4860 = 245' = 106 psi. The above readings on the gauges which were furnished by the Post Fire Department and read as accurately as practicable, indicates about 8 psi maximum friction loss in the piping system during maximum pump discharge at 10:30 A.M. to 11:00 A.M. on a Monday morning which is considered to be an average water use period. (Two of the four pumps were operating just prior to starting the tests.)

The four Wherry boosters which have a total capacity of 1960 gpm are not capable of maintaining the water level at the top of the 1-1/2 million gallon reservoir during maximum demand. Installation of a new booster or repair of booster #1 would relieve the four Wherry boosters pumping requirement by the difference between the 1200 gpm well production and the 830 gpm capacity of booster #3 when pumping alone or 370 gpm. These calculated differences in flows are based on 24 hour operation.

Maximum water requirements in Zones "A" and "B" are approximately 72% of the total requirement. The calculated total is 4.93 mgd but actual use from TABLE V, page 34 is 5.25 mgd. The usage in the two zones is therefore actually measured as 3.78 mgd or 2620 gpm. If 1200 gpm of this requirement is satisfied by boosters #1 and #2B after booster #1 is replaced, the Wherry boosters would need a capacity of 1420 gpm if all boosters are operated 24 hours per day. However, if one of the #1 or #2B boosters is not operated so that 600 gpm is delivered direct to the 1-1/2 M reservoir,

USF200002411

the Wherry boosters would need to pump 1420 + 600 = 2020 gpm for 24 hours. If operated 16 hours per day as recommended by the manual, Wherry area boosters would need a combined capacity of 3030 gpm which is 1070 gpm in excess of the existing combined capacity.

When additional boosters are installed in the Wherry area they should, however, be located to take suction from the 18 inch main and discharge to the 14 inch main on Craig Street because the piping in the present booster building is already loaded to capacity. The new boosters should be of similar size to the existing ones. There should be two new boosters added with each having a capacity of about 535 gpm against the static head plus about 30' of friction head.

e. Old Post Boosters (See Tabulation Sheet No. 10 and Sketch #11 in APPENDIX). (No curves are available): Booster pumps #7A, 7B, and 7C are manifolded in a weatherproof building about 100' to the north of the 1-1/2 million gallon reservoir. Installed in 1962, the boosters have a positive suction head of from 20 feet to 37 feet. Each is rated for 500 gpm at 240' TDH. The usage in Zone "A" to which these pumps discharge is reported by Post water system personnel to be low enough so that not more than two pumps ever operate at the same time. The maximum daily demand for water in Zone "A" served by these boosters from calculations shown under "II. PRESENT SITUATION, 1.c" page 5, is 818,250 gallons per day. This can be satisfied by two of the existing pumps operating about 14 hours per day.

A summary of the relationships between the boosters, their supply mains, and their terminal storage facilities are depicted in Sketch #13.

USF200002412

4. Water Distribution System

a. General

Although there is a relatively large percentage of steel pipe in the distribution system, maintenance personnel report that there are no unusual problems due to leaks or corrosion. While Plate #1 depicts the water transmission lines and the main feeder lines, the effectiveness of the system is discussed below for each pressure zone.

b. Transmission Lines

The transmission line from Well #1 and #2 booster station to the 1-1/2 million gallon reservoir is 12" steel cylinder concrete pipe except for 1600' of 16" steel pipe at the lower end. This steel pipe was installed in 1969 and so is assumed to be in good condition. The line from Garden Canyon Springs, a 12" steel cylinder concrete transmission line, is also assumed to be in good condition, since normal estimated life of this type pipe is 50 years.

The pipe lines from Wells #3, #4, #5, and #6 are cast iron of adequate size out to the 16" cast iron collector main located in Brainard Road. This 16" line is connected to a 12" cast iron line at Carter Street and to a 12" cast iron line at Well #3 outlet on Brainard. A 12" cast iron main runs from the intersection of Brainard Road and Carter Street to the populated area. It serves the smaller lines along the way to Cushing Street where it connects to a 14" A-C transmission line which is connected to the bottom of the three million gallon reservoir which floats on the line.

A 12" steel main runs from this Well #3 outlet on Brainard southerly thence westerly to a 0.5 million gallon elevated tank floating on the system and thence northerly to form a loop with the above described Carter Street 12" main.

USF200002413

5. Water Pressure Zone Analysis

a. Zone "A" (Old Post Area)

Water is supplied in limited quantities to Zone A from Huachuca Canyon Springs via an 8" A-C line installed from a system of filter beds in the Canyon to the older portion of the Old Post Reservoir. The major supply is pumped via a 30 year old 10" steel pipe to both portions of the Old Post Reservoir by Boosters #7A, #7B, and #7C which take suction from the 1-1/2 million gallon reservoir. All distribution mains are 8" and 6" size of either cast iron or asbestos-cement mater.

There are 4 pressure reducing valves in the system to prevent excessive pressures in the lower areas of the system. This system distributes approximately 17% of the total daily potable water demand of the Fort. It is estimated that about 42% of the water used during peak demand periods in summer is used for irrigation of the landscaping around the residences, in the cemetery, and on the parade field along Grierson Avenue.

The water system maintenance personnel report that the booster plant for the Old Post Reservoirs is quite adequate but that during the dry and hot season, the 1-1/2 million gallon reservoir is almost empited several times each summer. This lowering of water level, however, is from use in the lower pressure zones rather than excessive use in Zone "A".

USF200002414

Maintenance personnel report that there are complaints of low pressure at residences near the 1-1/2 million gallon reservoir when its water level is lower than normal (about 10' from top). Examinations of relative elevations in the area indicate that ground elevations of some residences around Mills Circle are as close as 50 feet below the low water levels in the tank. Since the houses are built above ground level, some of their upper stories could have only 30 feet or 13 psi of pressure. This pressure in inadequate for automatic washing machines, flushometer valve toilets, etc. These residences should therefore be served from the higher pressure zone. When these residences are served from Zone "A", at least one fire hydrant should be kept on the lower pressure zone for emergency use from the lower system.

The Apache Flats area and both Motor Pool areas (see Sketch 15) are at elevations at least 90 feet below the lower water level in the 1-1/2 million gallon reservoir (39 psi) and so these areas could be adequately served from the 1-1/2 million gallon reservoir instead of the Old Post reservoirs. It is desirable to make these changes in pressure zones to save boosting the water when it is not necessary. Also, the larger storage facility will provide more water for emergency use in the areas.

The above discussed pressure zone changes can be made by installing a total of about 2000 feet of 6" pipe at six locations and 100 feet of 8" pipe at one location. The locations are shown on Sketch Number 15.

USF200002415

b. Zone "B"

Zone B, comprising that area from elevation 5020 to elevation 4810 is served from the 1-1/2 million gallon reservoir floating on the distribution system. This zone obtains its supply from Garden Canyon Springs; Booster Pumps #1 and #2B via a 16" and 12" steel cylinder concrete pipe; and from the Wherry Booster Station which draws from the 3 million gallon reservoir and discharges into the distribution system. The 14" A-C main into which the booster pumps discharge and the other branch lines in the system are adequate for serving the zone and are mostly A-C pipe reported to be in good condition. There are 8 pressure reducing valves in the system for preventing excessive pressures in the lower areas.

This distribution system satisfies about 55% of the total daily demand. It is estimated that approximately 40% of Zone "B" total demand during dry summertime is being used for irrigation of landscaping.

c. Zone "C"

Zone C extends from the lower end of Zone B to the Northern and Eastern limits of the Fort's boundary. The principal supply is from Wells #3, #4, #5, and #6 discharging thru 16" and 12" looped mains. These mains are connected to the 500,000 gallon elevated steel tank and the 3 million gallon reservoir which float on the system and thereby maintain the required pressure in the distribution system. An altitude valve on the elevated tank prevents its overflow since it fills up before the 3 million gallon reservoir because of friction loss in the longer lines to the 3 million gallon reservoir.

USF200002416

Generally speaking, Zone C distribution system serves several warehouses, family housing, a laundry, the golf course, and two wastewater treatment plants. Of the remaining portion of daily demand to be accounted for, about 49% would be for domestic needs, and 51% for irrigation.

d. Discussion of another Pressure Zone

A fourth pressure zone could be established to provide a reduction in power usage due to less boosting and less variation in pressures within the zones. This extra zone would require its own storage facility. In fact, good practice is to have pressure zones with not over 100 feet of pressure head variation in each zone. This criteria would require 5 zones in the presently populated areas on the Fort. There are now 5 pseudo-pressure zones being operated by means of pressure reducing valves located in two of them to maintain practical pressures in the lower areas of the zones where pressures would otherwise be excessive. The present method of control has advantages and disadvantages, some of which are as follows:

(1) Advantages

(a) One storage facility provides a reserve for emergency that is larger than would be available at any particular point if the same storage quantity was split into additional reservoirs at different elevations.

(b) One storage facility is less costly per gallon to construct and maintain than two facilities.

(c) A lesser number of booster pumps are needed to provide adequate standby capacity for breakdowns.

USF200002417

(d) Less transmission piping is required between the booster pumps and the storage facility. As an example, a storage facility might be desirable at elevation 4960 to serve a zone from elevations 4760 to 4860. This would require the storage reservoir, at least 8000 feet of 12" pipeline and appurtenances, as well as several other pipe changes and transmission main replacement along Smith Avenue and revision of and addition of booster pumps, piping, pumphouse, and controls.

(2) Disadvantages

(a) Approximately 15% of the potable water used is lifted about 100 feet higher than would be necessary if there were 5 fully independent zones.

(b) There are occasional problems encountered in maintaining the pressure reducing valves at the desired settings.

e. Summary

In view of the above discussion, it is concluded that retention of, and additions to, the existing storage facilities at the present working elevation are the preferred method of upgrading the storage and booster pumping facilities for all of the populated area. However, in order to satisfy pressure requirements in the area immediately below the 1-1/2 million gallon reservoir, this area should be placed in the higher Zone "A". Offsetting this would be an area in Apache Flats that could be removed from Zone "A" and placed in Zone "B".

Storage additions are discussed under the heading "WATER STORAGE REQUIREMENTS" of this report.

USF200002418

6. Water Storage

a. Zone "A"

This zone serves the upper area of the Fort which has about 17% of the total potable water demand. Storage consists of the following facilities:

(1) 200,000 gallon ground masonry reservoir

Top Elevation	=	5271.5
Bottom Elevation	=	5256.4

(2) 250,000 gallon ground concrete reservoir

Top Elevation	=	5264.5
Bottom Elevation	=	5255.0

As shown in Sketch #16 these reservoirs are interconnected in such a way that water can be transferred from #1 to #2, but not from #2 to the southerly compartment of #1. These facilities are known as the Old Post Reservoir; they serve Zone A. It can be seen in Sketch #14 that, at present the south 100,000 gallons compartment in #1 is used for fire reserve only. Huachuca Springs water flows into the south compartment continuously. In addition, an open port in the wall separating the two compartments allows overflow of water from the south compartment into the north one. This entire reservoir is 7' higher than #2. However, the bottom of the north compartment of #1 and the #2 reservoir are interconnected so that the water surfaces are level. This results in the upper part of the north compartment of #1, with storage volume of about 70,000 gallons, being nonusable. The only practical way that this storage volume could be utilized in conjunction with reservoir #2 would be to construct the sides 7' higher on #2 reservoir. Another method would be to put an altitude valve on #2 reservoir and

USF200002419

valve off and use the full capacity of #1 reservoir as the primary storage. However, this would cause water in the #2 reservoir to stagnate. Although these two facilities are 62 and 34 years old, they appear to be in good enough condition to serve for several more years. However, it is not thought to be feasible to construct additional structural material on top of #2 to revise the water level in it.

Water storage requirements for this zone for domestic purposes is 358,000 gallons per day. Water storage requirements for fire purposes for this zone in accodrance with TM 5-813-4 (EM 1110-345-223) are 358,000 gallons x 50% + 1000 gpm x 4 hours x 60 minutes = 419,000 gallons as shown in "II. PRESENT SITUATION, 1.c.". Total water usage including irrigation is 779,000 gallons per day. This is the oldest area of the Post and Post administrators do not expect any expansion of building or personnel in this zone. These reservoirs have been operated for several years with a 100,000 gallon compartment valved off for emergency fire use. This then always allows for utilizing 250,000 gallons + 30,000 gallons (or a total of 280,000 gallons) for normal storage and the additional 100,000 gallons for emergency use by manual operation of a gate valve.

Since the 280,000 gallons of storage is not adequate for that recommended by TM-5-813-4 criteria and since the reservoirs are quite old, the facility should have additional new storage to conform with the 419,000 gallon estimated requirement.

b. Zone "B"

Storage for this zone consists of a 1-1/2 million gallon prestressed concrete ground reservoir.

Top Elevation	=	5110
Bottom Elevation	=	5080

USF200002420

Completed in May 1958, this tank is in good condition needing only normal maintenance. It floats on the Zone "B" distribution system which serves the central populated area of the Post which has about 55% of the total potable water demand. This reservoir also acts as the suction source for booster pumps #7A, #7B, and #7C to the Old Post reservoirs.

The storage requirement for this area in accordance with TM-5-813-4 (EM 1110-345-223) is 1,470,000 gallons x 50% + 1000 gpm x 2 fires x 4 hours x 60 minutes = 1,215,000 gallons as shown in "II. PRESENT SITUATION, 1.c". The total calculated water use including irrigation is 2,550,000 gallons per day. These water usage figures do not include the 779,000 gallons lifted into Zone "A" and so the total storage requirement must be based on the 3,329,000 gallons daily and the maximum hourly usage. On several occasions during the summer irrigation season 1,000,000 gallons have been drawn from the reservoir in a 2 hour period with all booster pumps operating as discussed under II, 3.b., page 14. The storage at this location needs to be increased to accommodate the domestic and irrigation requirements of Zone "B" of 2,550,000 gallons and the 779,000 gallons pumped to Zone "A" from Zone "B". In view of the actual experience discussed above, even when some administrative restrictions have been imposed on irrigation, it is quite essential that an additional storage facility be provided. If the added storage has 1,500,000 gallons capacity, it would obviate requirements for additional storage in Zone "A".

c. Zone "C"

Located in this Zone is a 3 million gallon concrete ground storage reservoir that floats on the Zone "C" distribution system which serves about 28% of the potable water demands.

Top Elevation = 4887
Bottom Elevation = 4870

USF200002421

Although this unit is 30 years old, it has been well maintained. It has been re-roofed fairly recently so that no major maintenance is required at present.

A 500,000 gallon elevated steel tank also floats on Zone "C" and operates at the same elevation as the 3 million gallon reservoir.

Top Elevation = 4887
Bottom Elevation = 4870

This tank fills up sooner than the 3 million gallon reservoir which is fed thru longer transmission lines. An altitude valve has therefore been installed on it to prevent overflow.

This tank and reservoir not only furnish storage for Zone "C" but are the storage and suction source for the Wherry booster station which furnishes water to Zones "B" and "A". The daily water requirements for this Zone "C", as described in II.1., page 8, are 1,284,000 gallons. The storage requirements in accordance with TM 5-813-4 (EM 110-343-223) are 564,000 gpd x 50% + 1000 gpm x 4 hours x 60 minutes = 522,000 gallons.

On one recorded occasion during the summer irrigation season (June 1971 - See Table V on pages 35 and 36) the water storage during a 5 day period was depleted to as low as 1,650,000 gallons before operating booster pumps could replace the deficit. The fastest recorded depletion during a 2 hour period has been 400,000 gallons. Since the lowest storage volume of record (1,650,000 gallons) still exceeded the 1,284,000 gallon storage requirement, the storage is considered adequate.

USF200002422

INVESTIGATION AND RECOMMENDATIONS FOR UPGRADING THE WATER SYSTEM AT FORT HUACHUCA, ARIZONA

PART II

for

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
650 Capitol Mall
Sacramento, California 95814

February 1973

prepared by

BLANTON & CO.
ARCHITECTS AND ENGINEERS
Tucson, Arizona

Appendix 3-II

USF200002423

TABLE NO. V

RESERVOIR WATER DEPTHS & DAILY
STORAGE SHORTAGES DURING A CRITICAL PERIOD

JUNE 1971

RESERVOIR DEPTHS WHEN FULL

Old Post	9.0 feet
1-1/2 Million	29.0 feet
3 Million and 1/2 Million Tank	16.0 feet

Following depths are at 12:01 A.M. of date shown.
Water production is to midnight of date shown.
All pumping facilities were operating at maximum existing operating production capacity at all times.

Date	Old Post	Shortage	1-1/2 M	Shortage	3 M & 0.5 M	Shortage
10th	8.5'	10,000	27.5'	75,000	12.5'	250,000
11th	9.0'	0	11.0'	900,000	12.5'	250,000
12th	9.0'	0	11.0'	900,000	11.0'	350,000
13th	8.7'	5,000	19.0'	500,000	13.5'	175,000
14th	8.2'	20,000	10.0'	950,000	14.5'	105,000
15th	6.0'	90,000	10.0'	950,000	13.0'	210,000
16th	7.0'	60,000	2.0'	1,300,000	13.5'	175,000
17th	7.5'	45,000	26.0'	150,000	10.5'	385,000
18th	8.5'	10,000	7.0'	1,100,000	12.5'	250,000
19th	8.5'	10,000	6.0'	1,150,000	10.5'	385,000
20th	8.7'	NA	NA	NA	NA	NA

USF200002424

Continuation - TABLE NO. V

SUMMARY OF MAXIMUM WATER USE

June 1971	Total Reservoir Shortage	Total Water Production	TOTAL DAILY POTABLE WATER USE (Sum of Production & Reservoir Depletion)
Thursday 10	335,000	4,286,000	5,101,000
Friday 11	1,150,000	4,850,000	4,950,800
Saturday 12	1,250,000	4,946,600	4,376,600
Sunday 13	680,000	4,185,000	4,580,000
Monday 14	1,075,000	4,628,700	4,803,700
Tuesday 15	1,250,000	4,919,000	5,204,000
Wednesday 16	1,535,000	5,070,900	4,115,900
	(Irrigation Holiday)		
Thursday 17	580,000	4,464,700	5,244,700
Friday 18	1,360,000	5,037,800	5,222,800
Saturday 19	1,545,000	5,014,500	
20	CHARTS NOT OPERATING		

USF200002425

d. Discussion of Storage Facilities

It was evident on the recording charts that the water level in the 1-1/2 million gallon reservoir was regularly lowered 10 feet to 13 feet in a 1-1/2 hour period beginning at 6:00 P.M. during this critical period. This is the time, from 6:00 P.M. to 7:30 P.M., when residents in all zones are allowed to turn on their lawn sprinklers. This rate of withdrawal of 650,000 gallons in 1-1/2 hours is 4400 gpm greater than all four Wherry boosters and Booster #2B can supply.

The problem of drastic withdrawal in the 1-1/2 million gallon tank during the 6-7:30 PM domestic irrigation period may be resolved in two ways. An additional 1.5 million gallon storage facility can be connected to the present tank, or the irrigation schedule could be modified so that only a small part of the total domestic irrigating is done at any one time. The additional 1.5 million gallons of storage would allow for present hourly irrigation schedules to be followed and still have 1.5 million gallons of storage for emergencies.

USF200002426

7. Miscellany

A 500,000 gallon elevated steel tank, stripped of vital appurtenances in 1946, has been kept painted but is not considered available for use and no plans are known for putting it back into service.

Several 10,000 gallon tanks are used by specialized industries such as cold storage and the laundry. Their purpose is strictly for special local requirements and are not considered as a part of the system storage. There are also six 50,000 gallon surge tanks used at the six wells. However, only storage units described for zone storage, with a total capacity of 5,000,000 gallons, will be considered as Fort Huachuca's potable water storage.

Generally speaking, valving and metering of the lines and pumps is adequate. Swing check valves continue to be used to protect the booster pumps. These create serious water hammer and should at least be spring loaded checks. There are at least two instances where gate valves are used to shut off an abandoned line that was part of the existing system or connects to a lower pressure zone. It would be better to cut the abandoned lines and block the ends rather than depend on valves that may leak past the seat or that may have gates rusted out. Only those valves entirely necessary for emergency connection between pressure zones should be left in place.

There has been a considerable amount of effort expended and facilities constructed to develop and utilize the water from springs in Garden and Huachuca Canyons. These water sources have not contributed materially to the peak demands because these demands occur when spring flows are very low.

USF200002427

The spring flows, however, are effective in relieving the pumping of water from the aquifers supplying the wells when there are large flows from the springs during rainy periods.

The total water production of the springs could be increased by development of a program allowing more rain and snow water to reach the spring-charging areas. Officials of the Post Forestry Department and the Engineer's Office suggested several methods for assisting in this endeavor. Fencing could be constructed so that debris and pollution from people using recreation areas would not get onto the spring-charging areas. There could also be selective spot clearing of phreatophites and strip cutting of timber so that snow lodged in tree crowns would be minimized and so there would be more open areas for runoff to occur. Also, clean-up of deadfall would provide more water absorption in the spring collection area.

Some additional collection of water from the springs could also be made. At Saw Mill Spring, located above the Boy Scout camp in Huachuca Canyon, a maximum of possibly 200,000 gallons per day during deluge rain periods could be collected. However, construction of a damming and filtering system and about 3 miles of 6" water line would be required. It is not considered to be practical to

USF200002428

develop this spring for the limited quantity of water that could be collected. Another spring is evident along the roadway up Garden Canyon. This is adjacent to the transmission line from the upper collection points and so could be developed to collect possibly 200,000 gallons per day during deluge rain periods. Although the cost of developing this area would probably be considerably less than developing the Saw Mill Spring, again it is not considered practical as a source during peak demand periods and so if developed would be only for purposes of saving the mining of water from the aquifers at the well site.

While meters are installed at each booster pumping unit, not all of their recorders are operative. It would be useful to repair all meters and make regular 24 hour readings of each. It appears that some servicing needs to be done on the automatic control panel also. Post operations personnel reported that the meter installed on the wastewater effluent line leading to the Golf Course irrigation system was not operating properly. This should be repaired and daily reading of it should be reinstated.

USF200002429

III. CONCLUSIONS AND RECOMMENDATIONS

1. Conclusions

a. Pressure Zone Changes.

Better utilization of available pressure may be accomplished by changing the distribution system serving both upper and lower motor pool and Apache Flats areas from Zone A to Zone B. A pressure reducing valve currently prevents damage to the fittings in this area due to excessive pressure. This valve should be deactivated or removed. Similarly, a certain portion of Mills Circle residents would benefit by changing this water system from Zone B to Zone A. Pressures as low as 13 psi are experienced throughout the summer months in this area.

b. Production.

The peak daily water pumped from the 50,000 gallon supply reservoirs at the wells was 5,340,000 gallons on June 30, 1972. The maximum capacity of the six wells is 5.24 mgd on a 24 hour basis and evidently an additional 100,000 gallons was pumped from storage in the 50,000 gallon reservoirs. The maximum well capacity on a 16 hour pumping basis is 3.49 mgd. The minimum wastewater treatment plant effluent is 1,250,000 gallons per day. About 1.0 mgd of this water is being used for irrigation on the golf course and Chaffee Field. The maximum water used on any one day was 5.25 on Thursday, June 17, 1971 (See TABLE V, page 34). It is estimated that prior to September 1972 0.25 mgd was being used during summer days for supplementing the treated wastewater lagoon supply for golf course irrigation. This will no longer be necessary and so the 24 hour pumping of all wells could supply the daily water requirements. The deficit on the basis of 16 hour pumping of wells is 5.0 mgd - 3.49 mgd = 1.51 mgd. Additional sources of water therefore need to be introduced as soon as practicable.

USF200002430

c. Wells #1 and #2

It can be noted from Tabulation Sheets 1 and 2 that the bowl settings on Wells #1 and #2 have reached their limits. Both will require bowl lowering in the near future. It would be desirable to install new pumps and motors for these wells that would have sufficient head characteristics to discharge directly into the transmission main. This would eliminate the low efficiency of operation now prevailing with the double pumping system.

d. Well #5

A project for increasing the specific capacity of Well #5 would be worthwhile. Since its capacity has been good in the past, but is now about 60% of original capacity, it would be expected that a well driller could economically acidize and surge and redevelop it to its original capacity.

e. Test Well #5

Test Well #5, located in the east Range, has been proven to be a high production well. This well could be equipped and connected to the water system as an additional source of supply. The well has been tested to 1650 gpm and has maximum specific capacity or yield at 1500 gpm and so this should be a practical pumping rate for it. In order to maximize efficiency of pumping facilities and controlling operation the deep well pump should discharge directly into the transmission main. The pump should be capable of discharging against both elevation head and friction of 60'.

An 18" transmission main should be constructed from the well to the 18" main now serving Libby Field. If this main is constructed of asbestos-cement, or cement lined pipe with smooth bore, a "C" factor for friction of 120 can be used.

USF200002431

Under these conditions an 18" pipe from the well to the 3 million gallon reservoir (22,000') would develop a friction loss of 0.86 x 22.0 = 19'. If additional flow of 1280 gpm is introduced later making a total of 2780 gpm, friction loss would be 2.7 x 22.0 = 60'. If the main was enlarged to 24" the friction loss would be 2.7 x 7.0 + 0.65 x 15.0 = 28'. Since it is estimated that the larger pipe would cost 50% more than the 18", the 18" pipe should be used. Consideration was given to looping the new main with the existing 12" main on Carter Street. However the friction loss in the existing looped system from the presently connected wells was not thought to be excessive under present pumping conditions. Therefore, expenditure of funds for the line over to the new main is not thought to be justifiable.

f. Well Pump Meters

The arrangement of existing supply lines and booster pumps at the well sites is adequate. However, there are no meters on well pump discharge piping to determine flows.

g. Zone "B" Storage

It has been found that the storage capacity of the prestressed 1.5 million gallon reservoir is not sufficient to provide storage for emergency fire protection during maximum hourly use. In fact, frequently during peak demand periods 1,000,000 gallons above pumping replacement capacities were used during a 2 hour period. Although two additional booster pumps are recommended, they will only provide 65,000 gallons per hour. As discussed in "6. Water Storage, b. Zone B", a new 1.5 million gallon reservoir should be constructed at the same site and at the same high water and working level elevation as the 1.5 million gallon reservoir.

USF200002432

h. Spring Production

Although additional water sources can be developed from springs, their production is practically nil during peak demand periods and so it is not considered practical to develop more springs except for the purpose of lessening overdraw from the aquifers at the well sites.

i. Old Post Reservoir

The Old Post Reservoir has about 70,000 gallons of storage capacity that is not being used and another 100,000 gallons that is valved off for emergency use only. This 100,000 gallons is available only by opening a gate valve manually. However, the situation has operated satisfactorily since 1939. Since no changes in water use in the zone are anticipated, no changes in equipment or operations will be recommended in this report.

j. Booster Pumps #1, #2A, and #2B

There are no pressure control valves on the discharge side of Booster Pumps #1 and #2B. However, there is no unusual water hammer in the piping when pumps are turned on and off. The amp meters indicate that the motors are overloaded when flow is not throttled. Pump curves indicate that there should be no overload. Also, the motor on pump #1 is very noisy indicating some malfunction. Post operating personnel say that they cannot turn on Booster #1, #2A and #2B without tripping the circuit breakers. This situation should be investigated, and appropriate modifications or repairs implemented. Since the well pumps require modifying and lowering and since booster pumping and electrical switch gear are quite old and not operating efficiently, it would be desirable to eliminate the booster system and install new well pumps that discharge directly into the transmission main.

USF200002433

k. Operational Controls

On one occasion it was observed that the surge tank drain at Well #1 was open and continuously draining water from it without the operator being aware of it. A tell-tale device in the operating room could prevent this happening.

l. Abandoned Lines

It was observed that the gate valve on the old potable water inlet to the wastewater lagoon was leaking. It would be advisable for this line, the old line to the 1-1/2 million gallon reservoir, and any other lines that have been abandoned to be cut off and plugged.

m. Booster Replacement

Under existing operating conditions Booster Pump #1 is delivering only about 300 gpm against 500' TDH instead of its design capacity of 450 gpm against 710' TDH. Its motor is very noisy and sounds like it may be defective. Since this pump and motor are 34 years old they should be replaced instead of repaired.

n. Wastewater Reuse

The treated wastewater from the two plants has been used successfully for irrigating the golf course and Chaffee Field at a maximum rate of about 1.0 mgd. The minimum average daily flow of any 2 day period from the two treatment plants is about 1.25 million gallons. There is therefore 250,000 gallons per day of treated wastewater available for irrigating areas now being irrigated with potable water. These additional areas would have to be approved by the Medical Department. Some of these areas might be those

USF200002434

around Greeley Hall and Barnes Field as well as other less definable areas. Prior to September 1972, an estimated 250,000 gallons per day maximum of potable water was being used for golf course irrigation. This potable water will no longer be needed for irrigation. Although the Post cemetery is one of the larger irrigated areas, its location is too far from the available wastewater source to be considered for use of wastewater. If the Medical Department should permit use of wastewater plant effluent for irrigation of the hospital ground, there is a possibility that the abandoned 10" water transmission main could be used as a portion of the transmission system for the effluent if a surge tank is used for settling particles that may spall off from its interior. The 8 mg/l of phosphates in the wastewater make it unsuitable for use in the lakes because it creates profuse plant growth. The plant growth is desirable on land and so the wastewater should be used to the utmost for land irrigation.

o. Wherry Booster Additions

The Wherry Booster Station has not been able to keep up with the peak demands in Zone "B". They also create objectionable water hammer when starting and stopping and so should have spring loaded check valves on their discharge lines. The existing piping system in the booster station and building are not adaptable to expansion. It is therefore necessary to construct another booster station. This preferably should take suction from the 18" pipe and the 12" pipe now discharging overhead into the 3 million gallon reservoir and should also be tapped underground and connected to the suction header.

p. Automatic Controls

The pump control system is not yet being utilized continuously and so there is some question as to whether the pumping facilities are being operated at maximum efficiency.

USF200002435

Although, according to the curve data sheets, the motors were quite adequately sized to operate the booster pumps under all head conditions, some of the pumps are being throttled to prevent what is thought to be an overload on the motors at lower head conditions. The 12" discharge valve for booster pump #2B was found to be about 85% closed. Other booster pumps at the well sites are also being throttled with gate valves to prevent exhausting the storage in suction reservoirs faster than the well pumps can fill them. A more efficient method of controlling the discharge would be to install a control valve in conjunction with an orifice plate for sensing head differentials to control the flow to a constant quantity under varying head conditions.

q. Storage Level Adjustments

The well pump storage tank water levels and the booster pump shut-off at low reservoir levels are controlled by Mercoid switches with sensing line connected to the booster pump suction lines rather than to the reservoirs. This makes them difficult to adjust properly and so the operating differential is only about 4 feet in the 10 foot deep reservoirs rather than a desirable 7 feet.

r. Irrigation Scheduling

As a temporary means of operating the present water system until construction of recommended facilities can be completed, irrigation might be staggered by administrative direction so as not to concentrate it during any short period of time and to have opposite sides of streets or alternate blocks of houses to irrigate on only a 2 day alternating basis. Commercial irrigation might similarly be alternated.

USF200002436

2. Recommendations

The previous sections of this report have described the current status of Fort Huachuca's water supply system. This section lists recommendations for the major factors involved in adding to and changing the existing system and its operation to provide adequate water service to the Post facilities. Some other minor factors that could probably be accomplished by Post personnel were discussed under "1. Conclusions" above, and are not repeated below. Either planning for or actually processing the following recommendations should be started as soon as practicable: (Location of recommendations a thru k are shown on PLATE NO. 2).

a. Pressure Zone Changes

The boundary between Zones "A" and "B" should be changed by connecting water mains and by blocking the ends of others at seven locations as shown on Sketch No. 15.

b. Well #5 Pumping Equipment & Transmission Main

The existing test well #5 located in Section 16 should be equipped with well pumping equipment capable of delivering about 1500 gpm of water directly into the transmission main thru adequate water pressure control equipment against a head of 520 feet at the inlet to the main. The pumping equipment should be controlled from the central control panel and have its starting and stopping sequenced with pumps at the other wells. The panel should have motor running lights, on-off-automatic switch, and an indicator-totalizer-recorder showing meter performance.

An 18" transmission main should then be constructed from the well pump to tie into the 18" main at Libby Field. (See Sketch 16).

USF200002437

c. Additional Booster Pumps

A new booster station consisting of two pumps with capacity of 535 gpm each and necessary housing should be constructed to supplement the Wherry booster station. The pumps should take suction from the 18" main near Hatfield and Burns Streets and discharge into the 14" main at a discharge pressure at the entrance into the 14" main of 230 feet or 100 psi.

The pumps should be controlled from the central control station and have starting and stopping sequences alternated with pumps #6A, 6B, 6C, and 6D. The 12" main that discharges overhead into the 3 million gallon reservoir should also be tapped underground and connected to the booster station suction header with a 12" pipe.

d. Booster Pump Replacement

Booster Pump #1 and motor should be replaced. However, as described in "1. Conclusions, paragraphs c. and j."above, the present double pumping station should be eliminated and replaced with well pumps discharging directly into the transmission main.

e. Additional Storage

A 1.5 million gallon storage reservoir should be constructed at the same site and top water level working elevation as the existing 1-1/2 million gallon reservoir in Zone "B". It should be interconnected with the existing reservoir supply piping.

USF200002438

f. Power Supplies & Motors

The motors on all pumps should be checked out and should be replaced if overloaded by the pumps at maximum load conditions. The electrical power equipment at the site of Wells #1 and #2 should be reconditioned or replaced so that all pumps in the area can be operating simultaneously.

g. Pump Controls

Pump controls should be arranged or supplemented for activating and alternating the starting sequence of booster pump numbers 3, 4, 5, 6, and the proposed new test well #5 pump from water elevations in the 3 million gallon reservoir.

Similarly the controls for the four Wherry booster pumps and the proposed two new booster pumps nearby should be supplemented to activate the new pumps and to alternate their starting sequence with the existing pumps. Activation should be sensed from water elevations in the 1-1/2 million gallon reservoir but none of these pumps should start until pumps at the site of Wells #1 and #2 are unable to keep the reservoir full.

The automatic controls for pumps at the site of Wells #1 and #2 should continue to be activated from sensors in the 1-1/2 million gallon reservoir to maintain the water level as near the overflow as practicable.

h. Redevelop Well #5

Well #5 should be acidized and redeveloped by a well driller to obtain as nearly as practicable its original capacity.

USF200002439

i. Plug Abandoned Lines

The valves on the steel lines at the wastewater treatment lagoon and near the 1-1/2 million gallon reservoir should be removed and the lines plugged to prevent leakage. Any other abandoned lines not found during the investigation for this report should also be plugged.

j. Reset Well #1 and #2 Pump Bowls

Bowl settings on Well #1 and Well #2 pumps are fairly close to the danger point and should be lowered before they start sucking air. In lieu of lowering present pumps, it would increase operating efficiency by an estimated 100% if new pumps were installed in the wells and discharged directly into the distribution main.

k. Valving Changes

Install spring loaded check valves on all booster pump discharge lines on which patented slam valves are not currently installed. Wherry booster pumps particularly need these valves.

l. Extend non-potable irrigation system

The wastewater irrigation lines in the Chaffee Field area should be extended to any other locations such as areas around Greely Hall, Barnes Field House, and barracks complexes which the Medical Department will approve for the use of treated wastewater in Zone "B". Wastewater pumping facilities should then be expanded to transport and pressurize the approximately 0.25 mgd of surplus wastewater now available to the proposed new irrigation lines.

USF200002440

m. Additional Well Development

Drill, equip, and connect to the existing water system a new water well at a location directed by the Corps of Engineers from information either already developed or to be developed. This would be used for supplementing the existing water sources as additional supply may be needed in the future due to increases in average per capita uses or possible population increases.

n. Booster Pumping Station Replacement

As equipment at the #3, #4, #5, and #6 well site booster pumping stations deteriorates it would be desirable to eliminate the low efficiency of double pumping by installing pumps in the wells to discharge direct to the transmission mains.

USF200002441

REFERENCES

INVESTIGATION OF EXISTING AND PROPOSED WATER SYSTEM AT FORT HUACHUCA, ARIZONA, Blanton & Cole, 1958

WATER RESOURCES OF FORT HUACHUCA MILITARY RESERVATION, SOUTHEASTERN ARIZONA, Geological Survey Water-Supply Paper 1819-D, 1966

FINAL REPORT - TEST WELL DRILLING AND STUDY OF HYDROGEOLOGIC CONDITIONS FORT HUACHUCA, Corps of Engineers, April 1972.

BASIS FOR DESIGN - WATER DEVELOPMENT CANYONS (SECONDARY SEWAGE TREATMENT) AT FORT HUACHUCA, ARIZONA, Yost & Gardner, April 1969.

HOUSING POPULATION REPORT (as of 26 June 1972), Fort Huachuca

GENERAL WATER MAP OF FORT HUACHUCA w/Garden & Huachuca Canyons Springs development plans, Fort Huachuca Engineer's Office.

GENERAL INSTRUCTIONS FOR MILITARY DESIGN, 1 July 1966, w/Supplements

DOD MANUAL 4270 1-M, 1 March 1968, w/Supplements

WATER SUPPLY SYSTEM FOR UNITED STATES ARMY ELECTRONIC PROVING GROUNDS AT FORT HUACHUCA, ARIZONA, Blanton & Cole, September 24, 1962

DESIGN ANALYSIS - WATER DEVELOPMENT CANYONS SECONDARY SEWAGE TREATMENT S.T.P. No. 2 AT FT. HUACHUCA, ARIZONA, Yost & Gardner, Directive No. 3, 6 February 1970

WATER SUPPLY STUDY, PHASE I FORT HUACHUCA, ARIZONA, John Carollo Engineers, January 1971

USF200002442

TABULATION SHEET NO. 1

DEEP WELL AND PUMP NO. 1 - Floor Elev. = 4641.00

DRILLED:	1939
DRILLER:	W.E. Lane, Warren, Arizona
DIAMETER:	14 inches
CASED DEPTH:	703 ft. (Elev. 3938)
STATIC WATER LEVEL:	489 ft. (Elev. 4152) 1958
	519 ft. (Elev. 4122) 1971
RUNNING WATER LEVEL:	520 ft. @ 500 gpm (Elev. 4121) 1958
	*558 ft. @ 594 gpm (Elev. 4083) 1971
TOP OF BOWL SETTING:	560 ft. (Elev. 4081)
PUMP:	Submersible
PRIME MOVER:	Information Not available

* New pump after first test

USF200002443

TABULATION SHEET NO. 2

DEEP WELL AND PUMP NO. 2 - Floor Elev. = 4641.2

DRILLED:	1940
DRILLER:	W. E. Lane, Warren, Arizona
DIAMETER:	14 inches
CASED DEPTH:	710 ft. (Elev. 3931)
STATIC WATER LEVEL:	490 ft. (Elev. 4153) 1958
	516 ft. (Elev. 4125) 1971
RUNNING WATER LEVEL:	520 ft. @ 750 gpm (Elev. 4124) 1958
	552 ft. @ 615 gpm (Elev. 4089) 1971
TOP OF BOWL SETTING:	560 ft. (Elev. 4081)
PUMP:	Wintroath Pumps, Inc. Alhambra, California Serial #2803 Elev. C/L discharge = 4642.5
PRIME MOVER:	U.S. Asbestos Protected Motor Serial No. 1000340 Type: CVF NRR H.P: 125, 3 Ph Power: 2300 volts, 60 excess

USF200002444

TABULATION SHEET NO. 3

DEEP WELL AND PUMP NO. 3 - Floor Elev. = 4619.3

DRILLED:	1942
DRILLER:	Unknown
DIAMETER:	18 - 16 inches
CASED DEPTH:	802 ft. (Elev. 3817)
STATIC WATER LEVEL:	471 ft. (Elev. 4148) 1958
	503 ft. (Elev. 4116) 1971
RUNNING WATER LEVEL:	504 ft. @ 700 gpm (Elev. 4115) 1958
	*546 ft. @ 760 gpm (Elev. 4073) 1971
TOP OF BOWL SETTING:	650 ft. (Elev. 3969)
PUMP:	Worthington Size: 12 M903 8 stage Serial #VTP 23671
PRIME MOVER:	U.S. Motor Hollowshaft H.P: 200, 3 phase Power: 2300 volts, 60 cycles, 46 amps Speed: 1770 RPM

* New pump after first test

USF200002445

TABULATION SHEET NO. 4

DEEP WELL AND PUMP NO. 4 - Floor Elev. = 4619.3

DRILLED:	1942
DRILLER:	Unknown
DIAMETER:	18 inches
CASED DEPTH:	807 ft. (Elev. 3812)
STATIC WATER LEVEL:	462 ft. (Elev. 4157) 1942
	490 ft. (Elev. 4129) 1971
RUNNING WATER LEVEL:	480 ft. @ 700 gpm (Elev. 4139) 1942
	505 ft. @ 610 gpm (Elev. 4115) 1971
TOP OF BOWL SETTING:	555 ft. (Elev. 4064)
PUMP:	Pomona Serial #B-737 Elev. C/L of Discharge = 4620.1
PRIME MOVER:	General Electric Motor Model 12 F 3306 Type: KF H.P: 125 2300 volts, 3 phase, 29.2 amps

USF200002446

TABULATION SHEET NO. 5

DEEP WELL AND PUMP NO. 5 - Floor Elev. = 4610

DRILLED:	1942
DRILLER:	Unknown
DIAMETER:	18 - 16 inches
CASED DEPTH:	800 ft.
STATIC WATER LEVEL:	454 ft. (Elev. 4156) 1958
	489 ft. (Elev. 4121) 1971
RUNNING WATER LEVEL:	504 ft. (Elev. 4106) @ 700 gpm 1958
	583 ft. (Elev. 4027) @ 576 gpm 1971 (Throttled to 450 gpm)
TOP OF BOWL SETTING:	650 ft. (Elev. 3960)
PUMP:	K C 2721 - A - Worthington 12 M 90 E = 8 stage Serial #VTP 23673
PRIME MOVER:	U.S. Electrical Motors Model: Hollowshaft J1422422 Type: H V Speed: 1770 RPM Power: 2300 volts, 3 phase, 60 cycles H.P: 200

USF200002447

TABULATION SHEET NO. 6

DEEP WELL AND PUMP NO. 6 - Ground Elev. = 4642.7

DRILLED:	Feb. 15, 1958
DRILLER:	J.R. Sharp, Bisbee, Arizona
DIAMETER:	16 - 18 inches
DEPTH:	0'to 803'(16" Ø); 803'to 1230'(8"Ø)
CASED DEPTH:	803 ft.
DIAMETER:	16" I.D. 5/16" Wall, Welded Plate Pipe with 1-1/4" x 12" Drive Shoe, Perforated from 500' to 760'.
STATIC WATER LEVEL:	492 ft. (Elev. 4151) 1958 516 ft. (Elev. 4127) 1971
RUNNING WATER LEVEL:	Not available in 1958 532 ft. @ 600 gpm (Elev. 4111) 1971
TOP OF BOWL SETTING:	570 ft. (Elev. 4073)
PUMP:	Johnson Submersible Serial #JR 5981 Type: 10 DS - 14 stages H.P: 150 Model #S 1S50B3,10D Power: 440 volts, 3 phase, 60 cycles Capacity: 750 gpm @ 555' TDH

USF200002448

TABULATION SHEET NO. 7

DEEP WELL AND PUMP NO. Test Well #5 -Ground Eelv.= about 4426

DRILLED:	1971
DRILLER:	Beylik Drilling, Inc. La Habra, California
DIAMETER:	16 inches
CASING DEPTH:	800'
STATIC WATER ELEV:	4144±
RUNNING WATER ELEV:	4118± ft. @ 710 gpm (Test Pump) 4085± ft. @ 1650 gpm (Test Pump)
TOP OF BOWL SETTING:	415 ft. (Elev. 4011) (Test Pump)
PUMP:	(Temporary) 8" Peerless Turbine
PRIME MOVER:	(Temporary) GMC Diesel Engine

This Test Well has been capped, pending authorization for further development.

USF200002449

TABULATION SHEET NO. 8

BOOSTER PUMP NO. 1:

United Iron Works*
Oakland, California
Size: 4" M.S. - C. 4 stage
Job #4135
Date: 5-31-39
Rated @ 450 gpm @ 710' head
Elev. C/L Pump = 4637
(For additional information see Curve #4)

Prime Mover:
Ideal Electric & Manufacturing Co.
Model #228053
Type A
150 HP
2300 Volts

BOOSTER PUMP NO. 2A:

United Iron Works*
Oakland, California
Size: 6" - M.S.-C. 2 stage
Job #8830
Date: 1-23-41
Rated @ 700 gpm @ 270' head
Elev. C/L Pump = 4638.5
(For additional information see Curve #5)

BOOSTER PUMP NO. 2B:

United Iron Works*
Oakland, California
Size: 4" M.S.-C. 4 stage
Job #8829
Date: 1-10-41
Rated @ 700 gpm @ 710' Head
Elev. C/L Pump = 4638.5
(For Additional information see Curve #6)

Prime Mover:
U.S. Electric
J 1422438
100 H.P.
2300 V.

* NOW: United Centrifugal Pumps
580 Second Street
Oakland 7, Calif.

USF200002450

TABULATION SHEET NO. 9

BOOSTER PUMPS NO. 3, 4, and 5:

Floway Pumps, Inc.
P.O. Box 104, Fresno, California
Item 1 D4EA 18-68-C-0272
Size: 12" - Type D OMH, 5 stages
900 gpm; 320' TDH;
Serial #68-3161

Prime Mover:
U.S. Electric Hollowshaft
Serial #J 1422438
Type: R V, Design B, Code F
H.P: 100
Power: 2300 volts, 23.5 amps, 3 phase, 60 cycles
Speed: 1770 RPM

BOOSTER PUMP NO. 6:

Johnston Vertical Pump
Serial #JW - 3338

Prime Mover:
General Electric Motor
Model L #5 K 62266 x C88A
H.P. 75, 440 volts
Type: K, Code F
Speed: 3550 RPM

USF200002451

TABULATION SHEET NO. 10

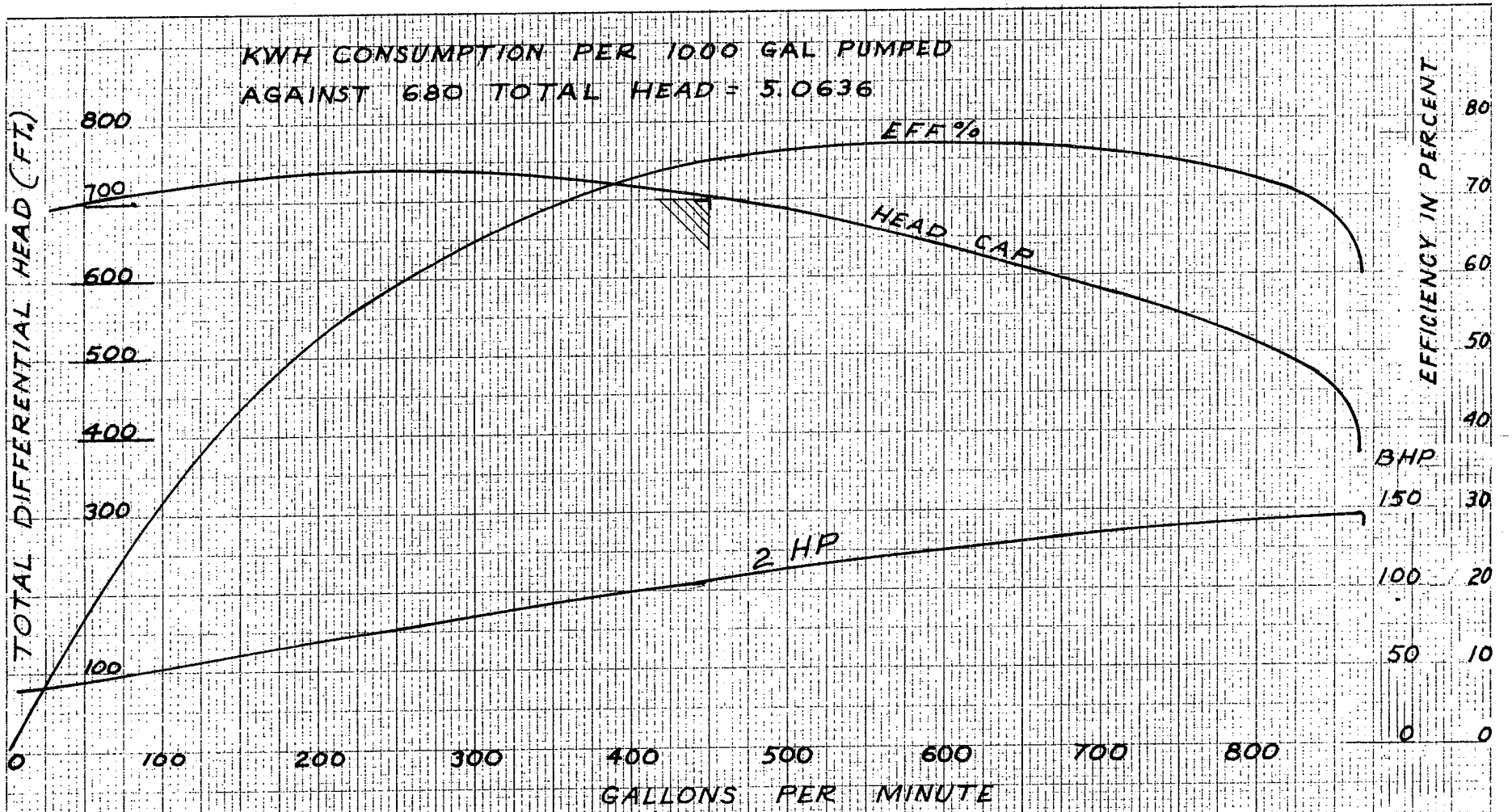
BOOSTER PUMPS NO. 6A, 6B, 6C, and 6D: (are identical)

PUMP:	Fairbanks-Morse 4" 5814 K56070 5814 K66690 5814 K67516 K67509
PRIME MOVER:	Fairbanks-Morse 50 H.P. 1770 RPM 440 volts, 59 amps, 3 phase S.F. 1.15 Type: K 2 K
SEE TABLE:	24 hour duty

BOOSTER PUMPS NO. 7A, 7B, and 7C: (are identical)

PUMP:	Worthington Serial #440411 500 gpm, 240' TDH 3600 RPM, 8-1/2" ø impeller
PRIME MOVER:	Marathon Electric Model # XE 364 USTDR5AA Type: TDR-BE H.P: 50 Power: 480 volts, 53 amps, 60 cycles Serial #X 52273

USF200002452



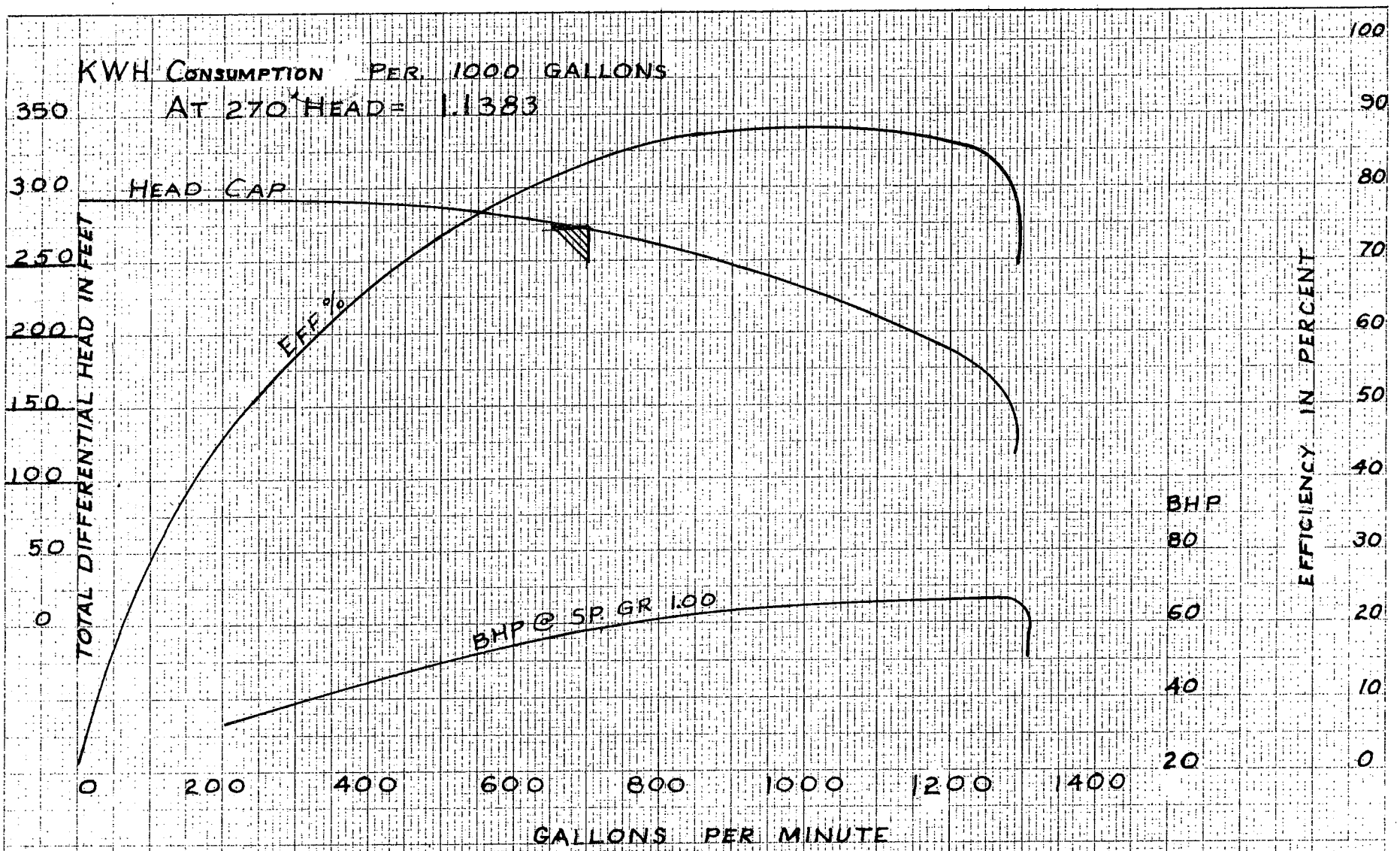
PUMP SERIAL Nº 4135 IN 1939

IMPELLER DIA 13 5/16"	4" MS-C 4 STG PUMP		
14 3/4" MAX DIA	IMPELLER MAT'L BRZ	IMPELLER 851 HI	1770 RPM

BOOSTER PUMP Nº 1

CURVE #1

USF200002453



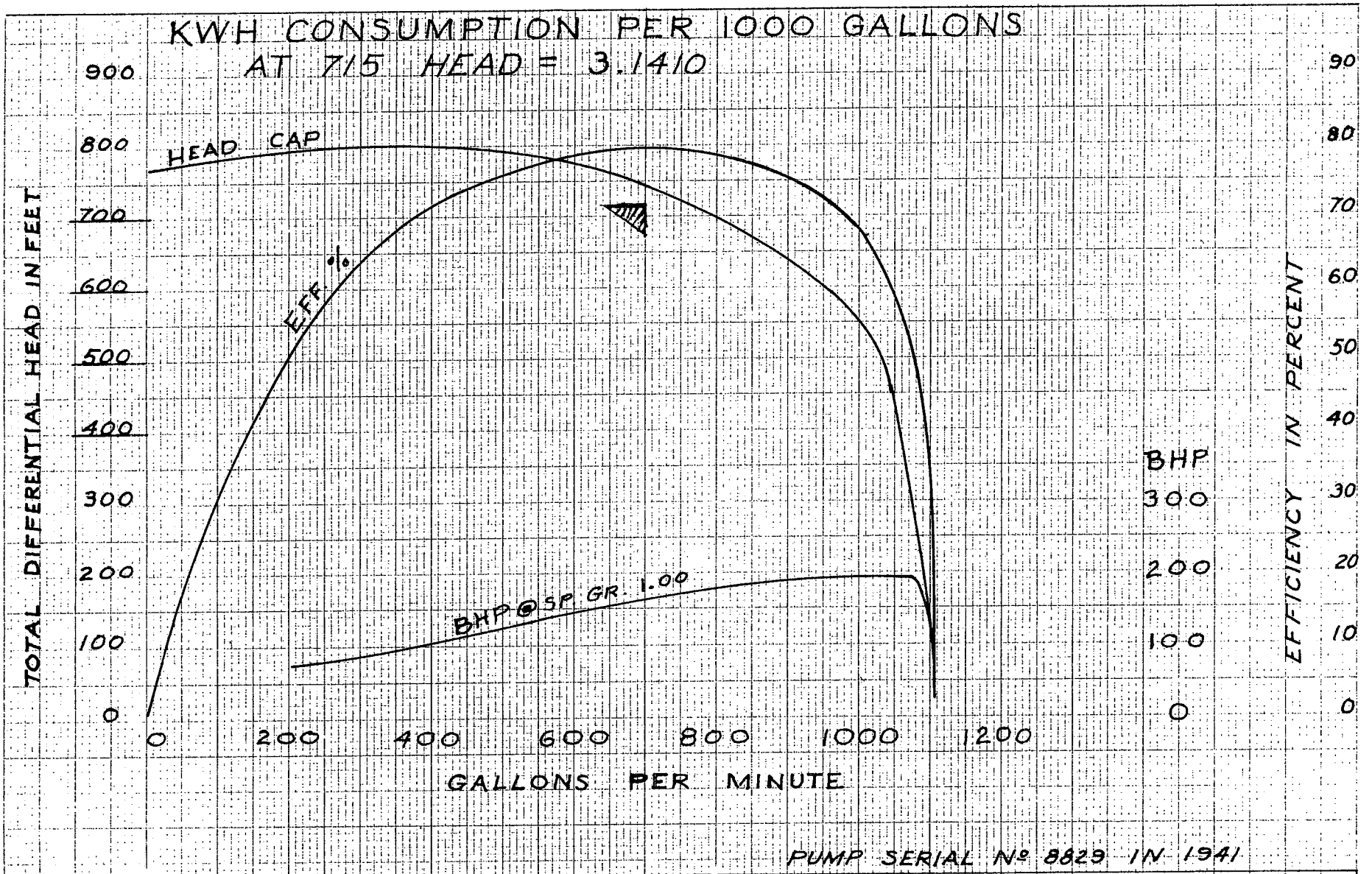
PUMP SERIAL Nº 8830 IN 1941

IMPELLER $11\frac{7}{8}''$		6"- MS-C 2-STAGE PUMP		
$11\frac{7}{8}''$ MAX DIA.	IMPELLER MAT'L. BRZ		IMPELLER 429-H	1780 RPM

BOOSTER PUMP Nº 2A

CURVE #2

USF2000024454



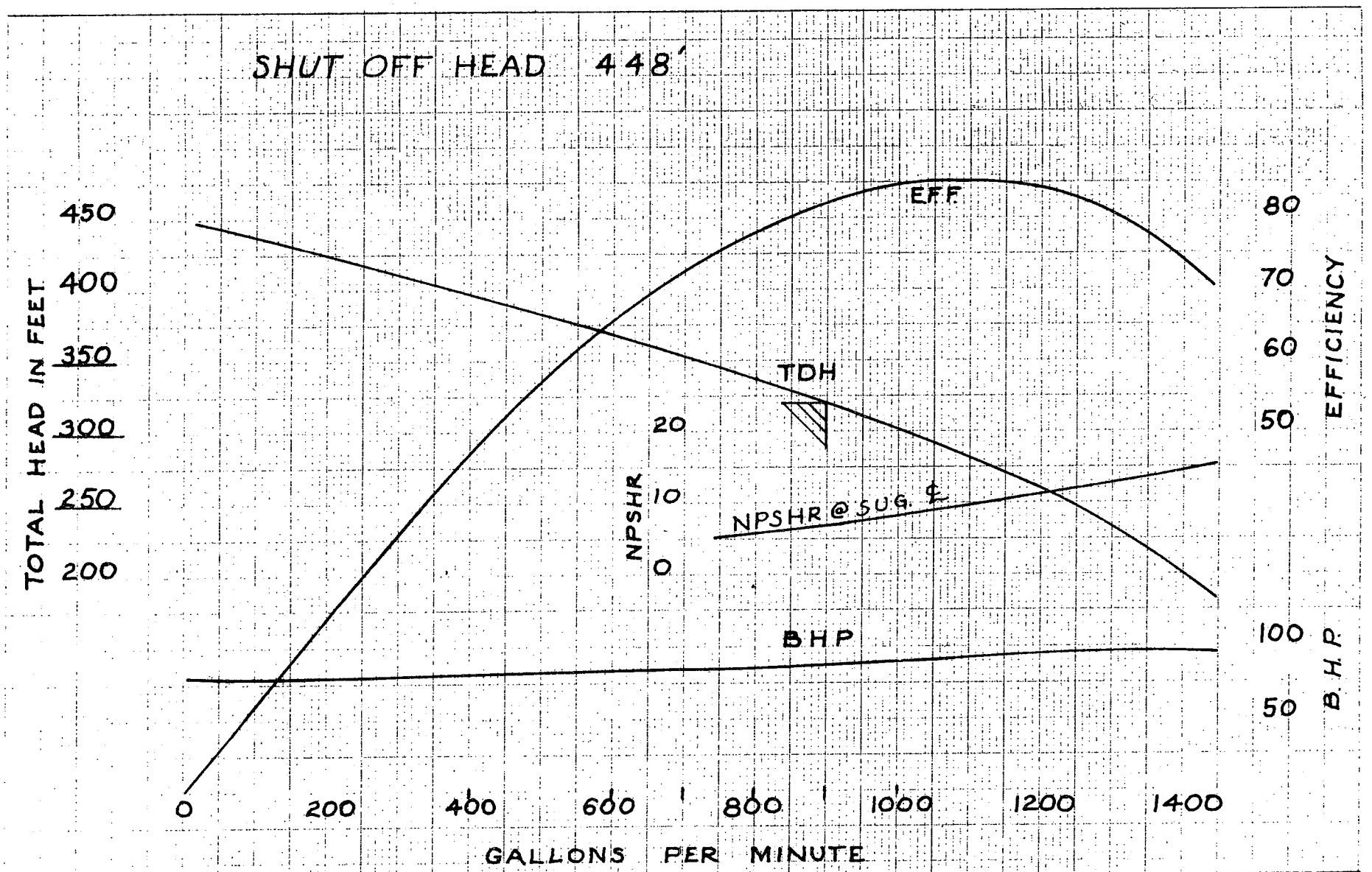
PUMP SERIAL Nº 8829 IN 1941

IMPELLER 13 11/16"	4" MS-C4-STAGE PUMP		
14 3/4" MAX DIA.	IMPELLER MAT'L. BRZ	851-H IMPELLER 851-H2	1770 R.P.M.

BOOSTER PUMP Nº 2-B

CURVE #3

USF2000024555



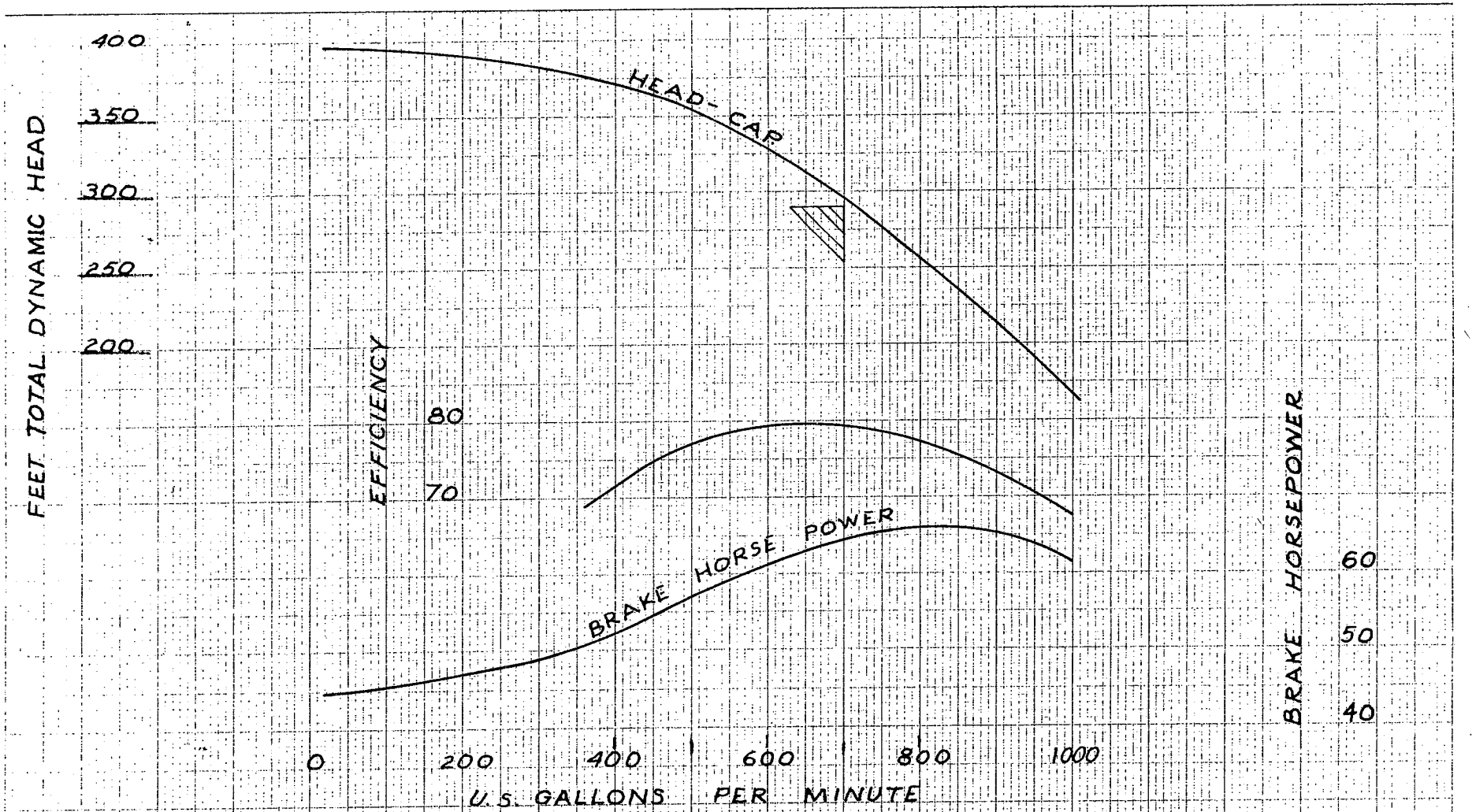
BOOSTER PUMPS Nº 3,4,&5

CURVE #4

	5 STAGE TURBINE PUMP		
		SIZE 12 DOMH	1770 RPM

3-11-67

USF2000002456



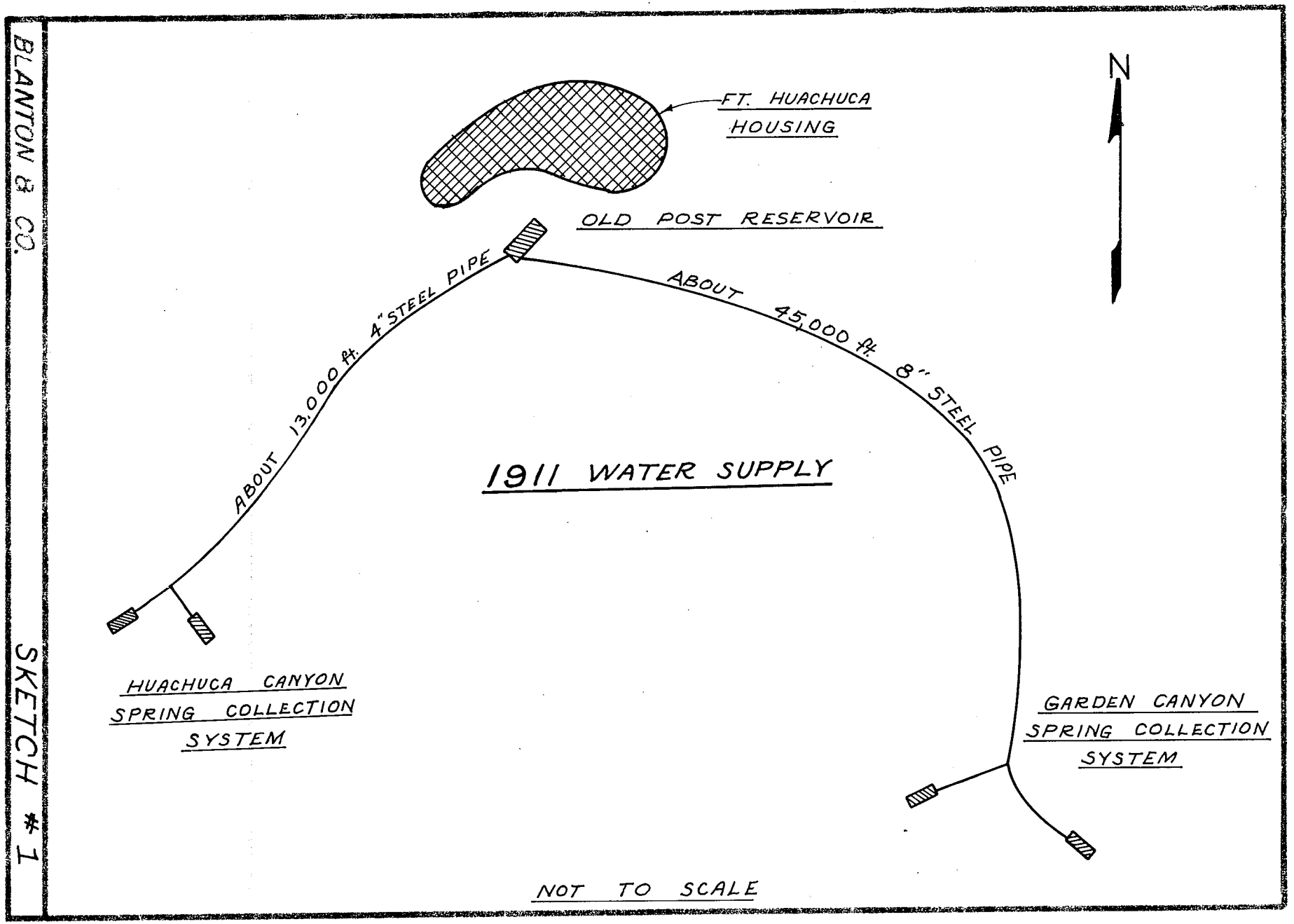
BOOSTER PUMP Nº 6

CURVE #5

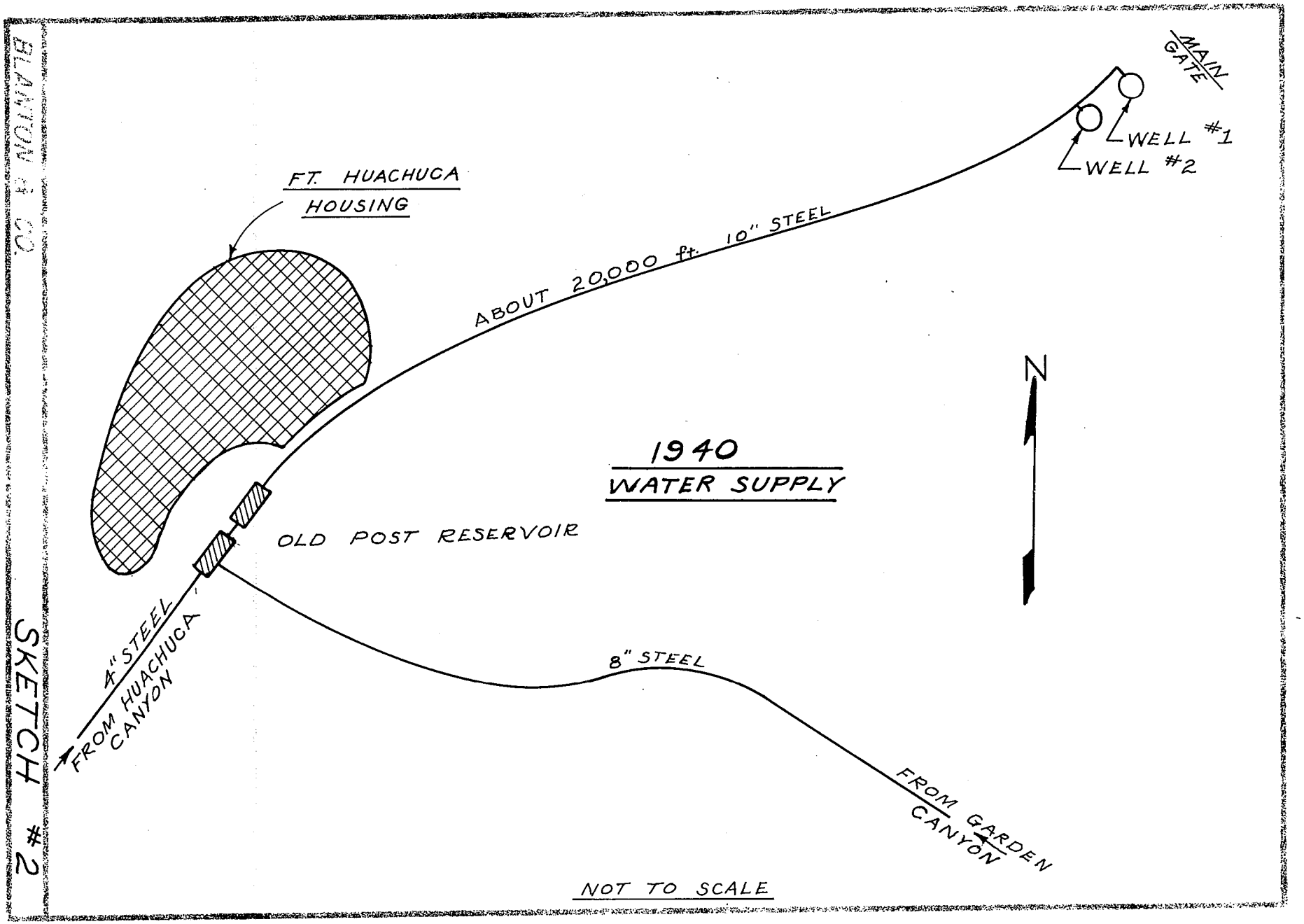
IMPELLER DIA. 7 1/8"	10AO TURBINE PUMP		
STAGES 2	IMPELLER BRONZE	BOWL CAST IRON	3500 R.P.M.

3-II-68

USF200002457

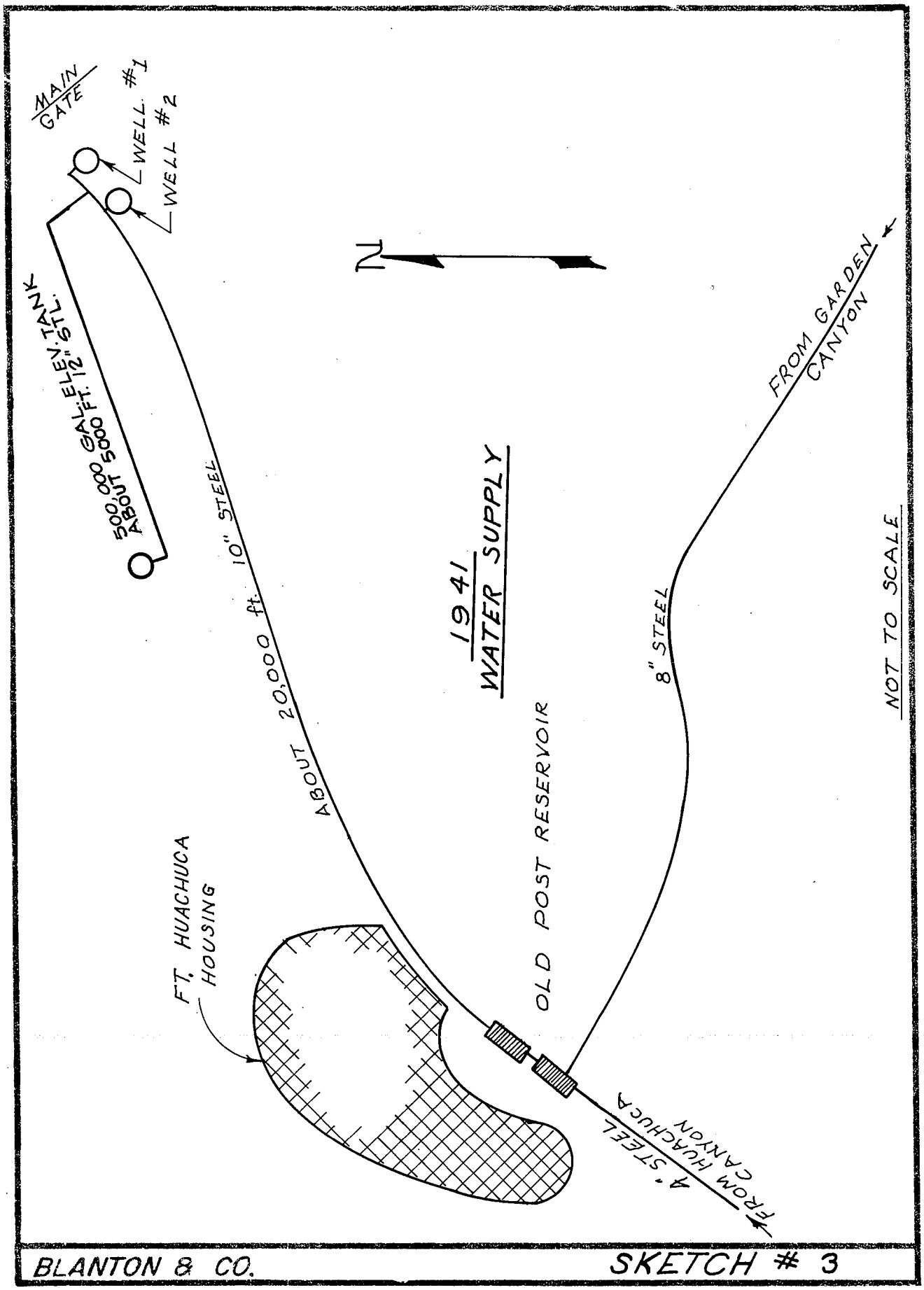


USF200002458



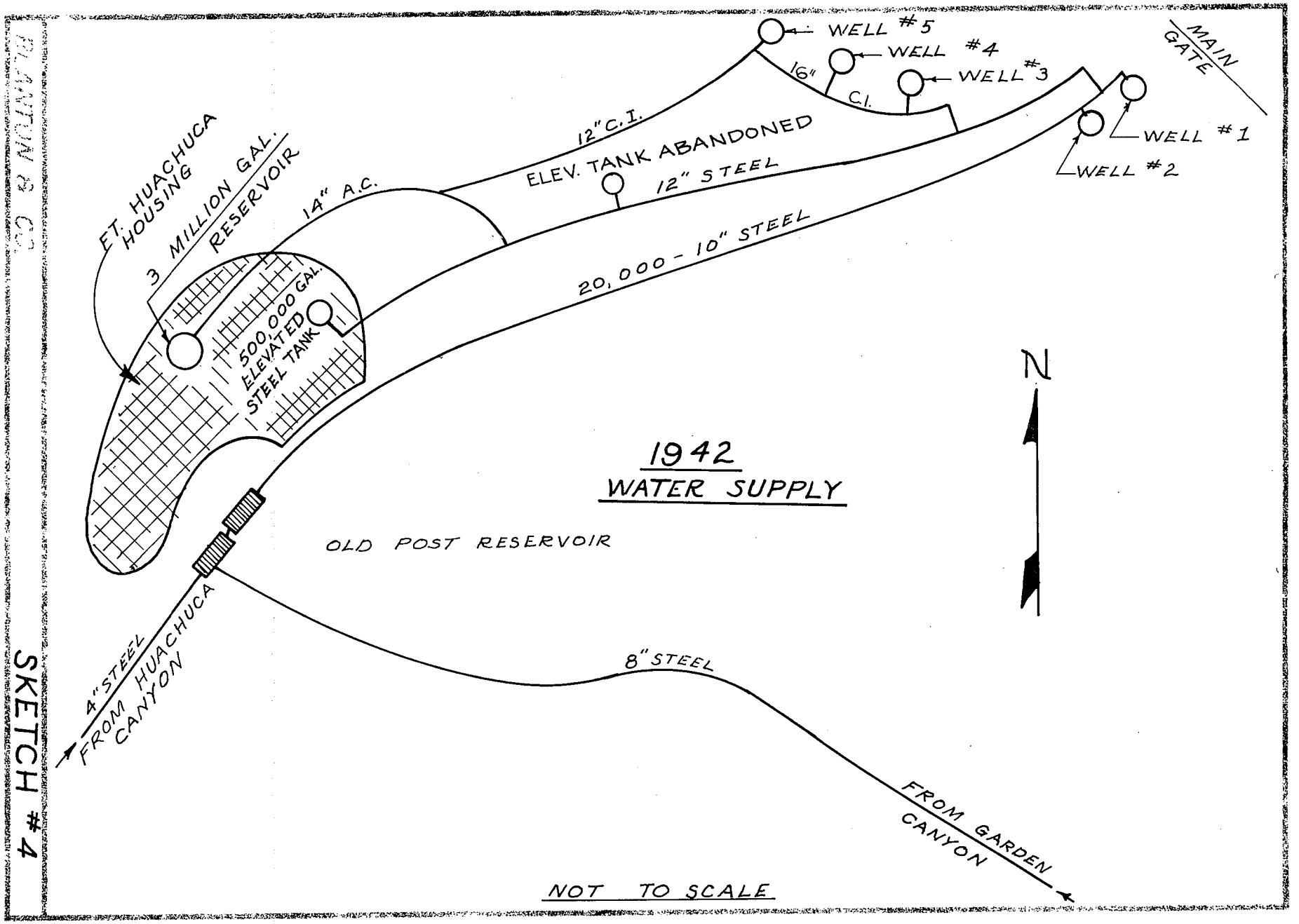
3-II-70

USF200002459



3-II-71

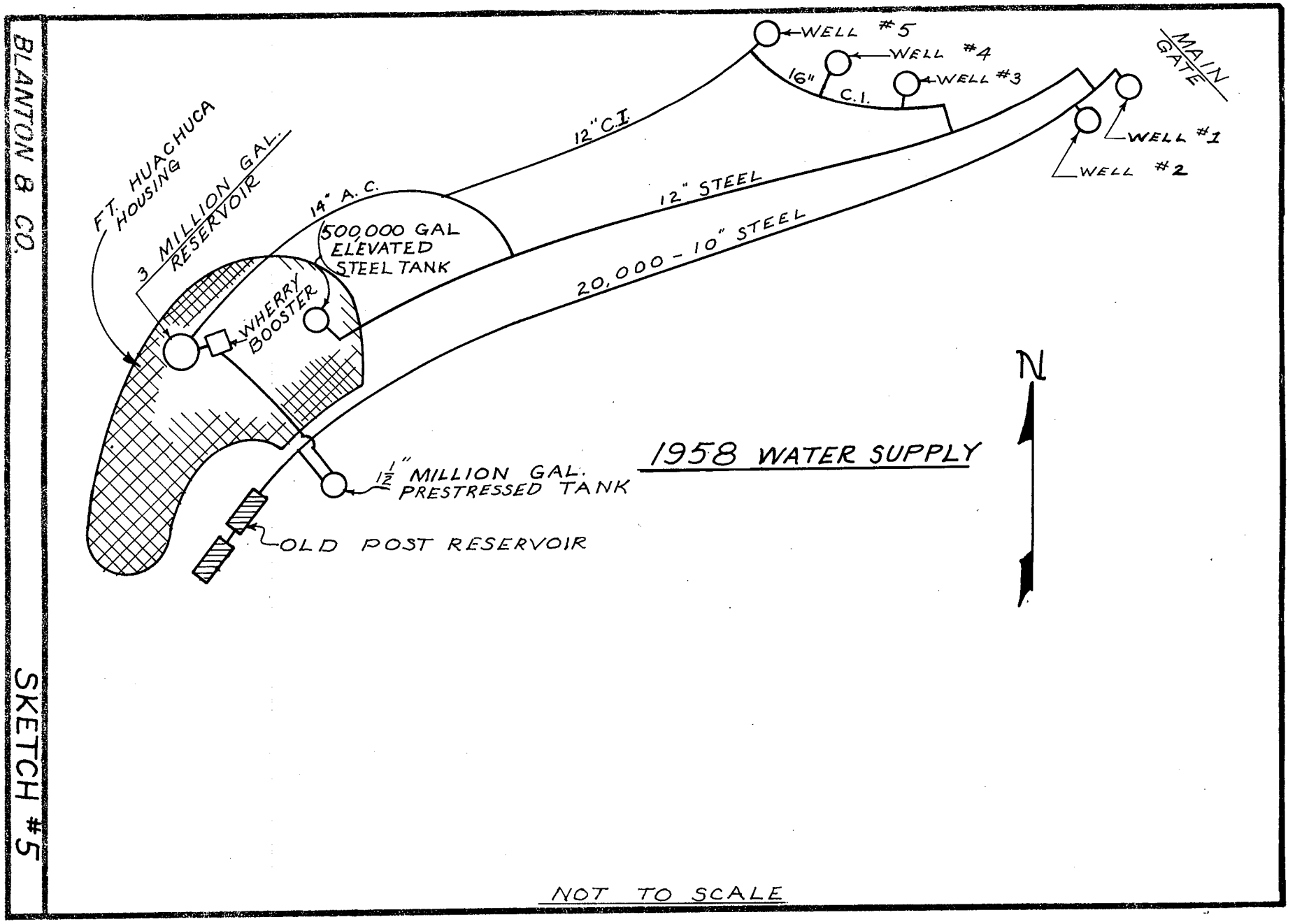
USF200002460



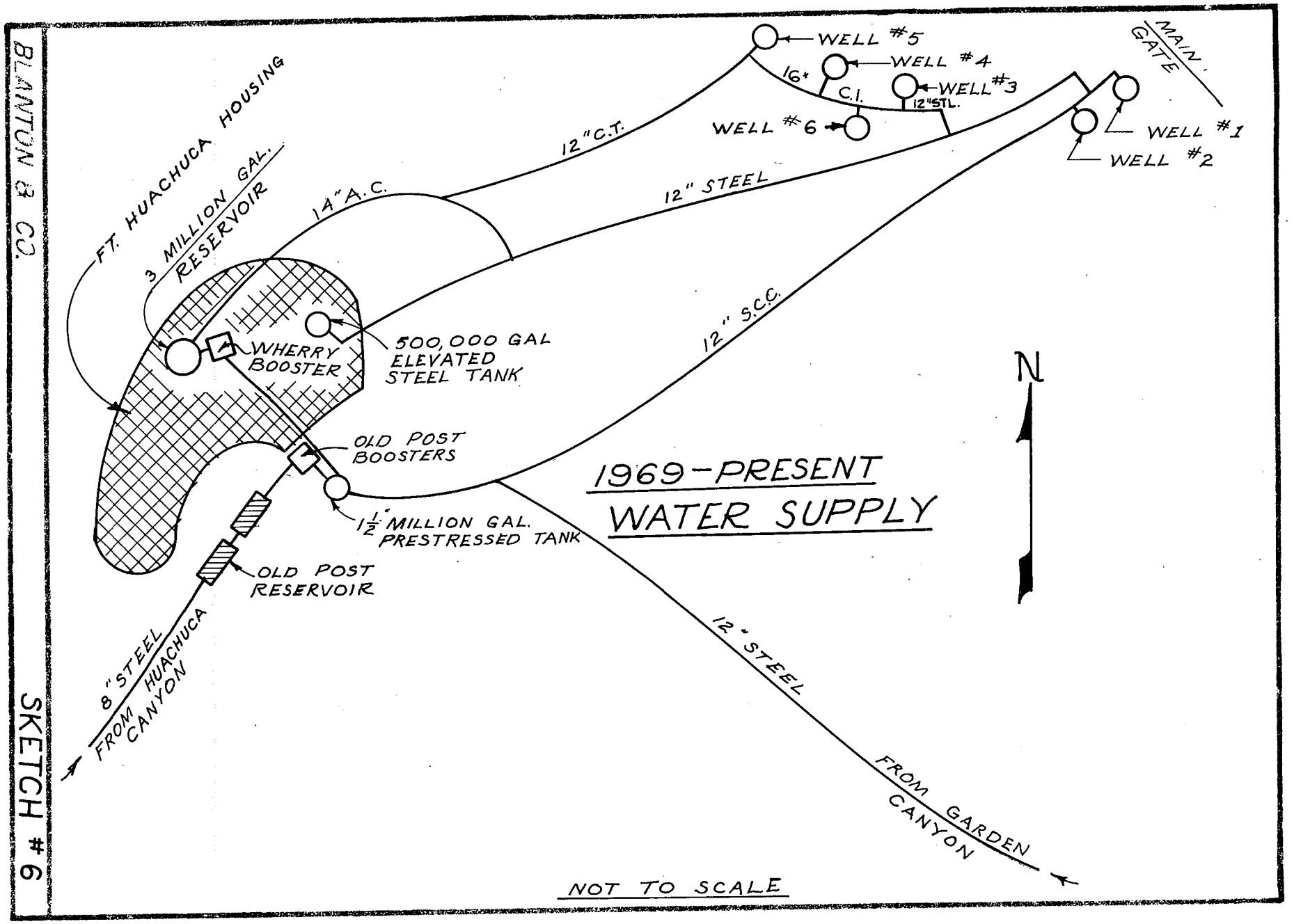
SKETCH #4

PLANTUN & CO.

USF200002461

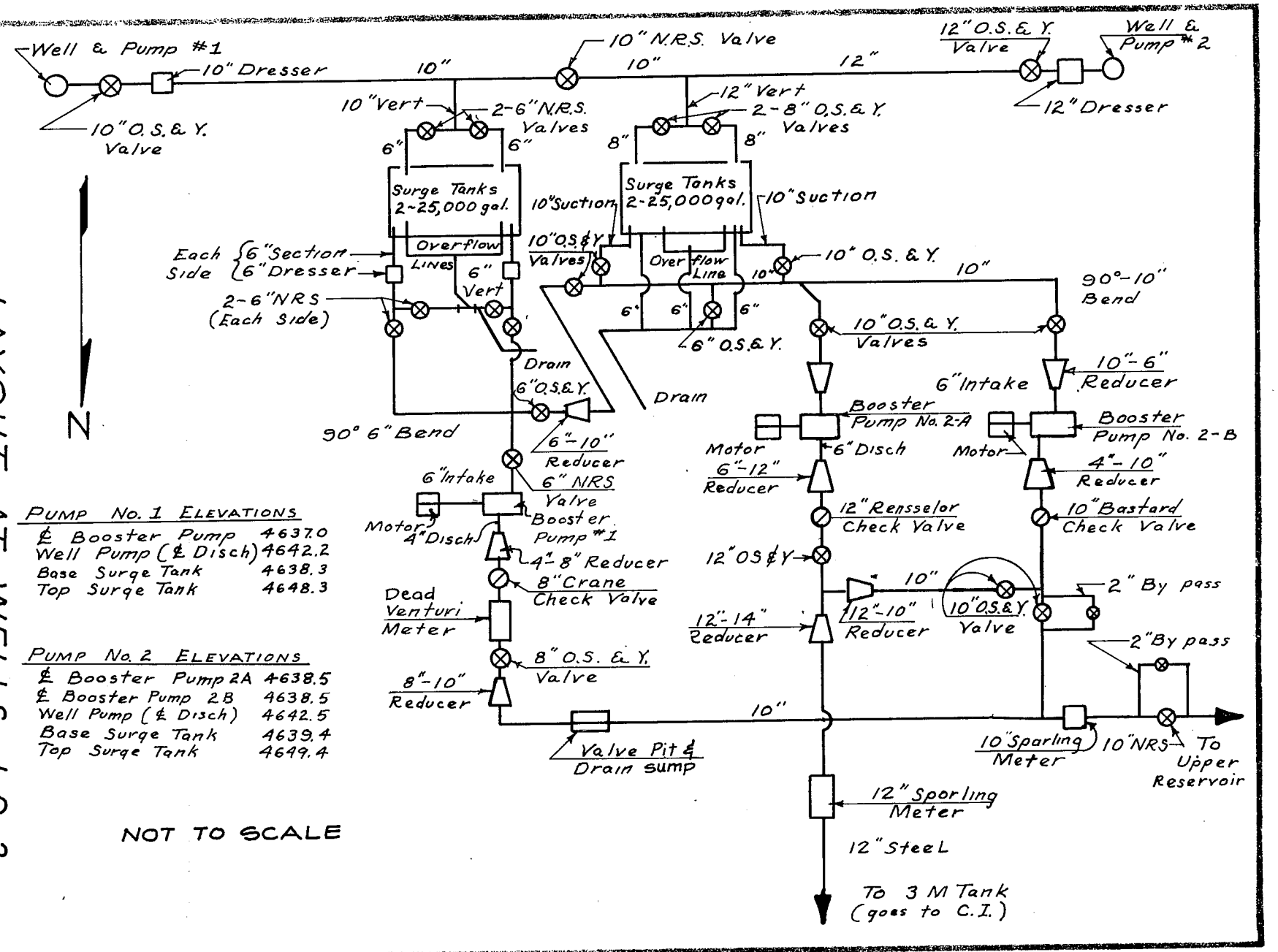


USF200002462



3-II-74

USF200002463



PUMP No. 1 ELEVATIONS

℄ Booster Pump	4637.0
Well Pump (℄ Disch)	4642.2
Base Surge Tank	4638.3
Top Surge Tank	4648.3

PUMP No. 2 ELEVATIONS

℄ Booster Pump 2A	4638.5
℄ Booster Pump 2B	4638.5
Well Pump (℄ Disch)	4642.5
Base Surge Tank	4639.4
Top Surge Tank	4649.4

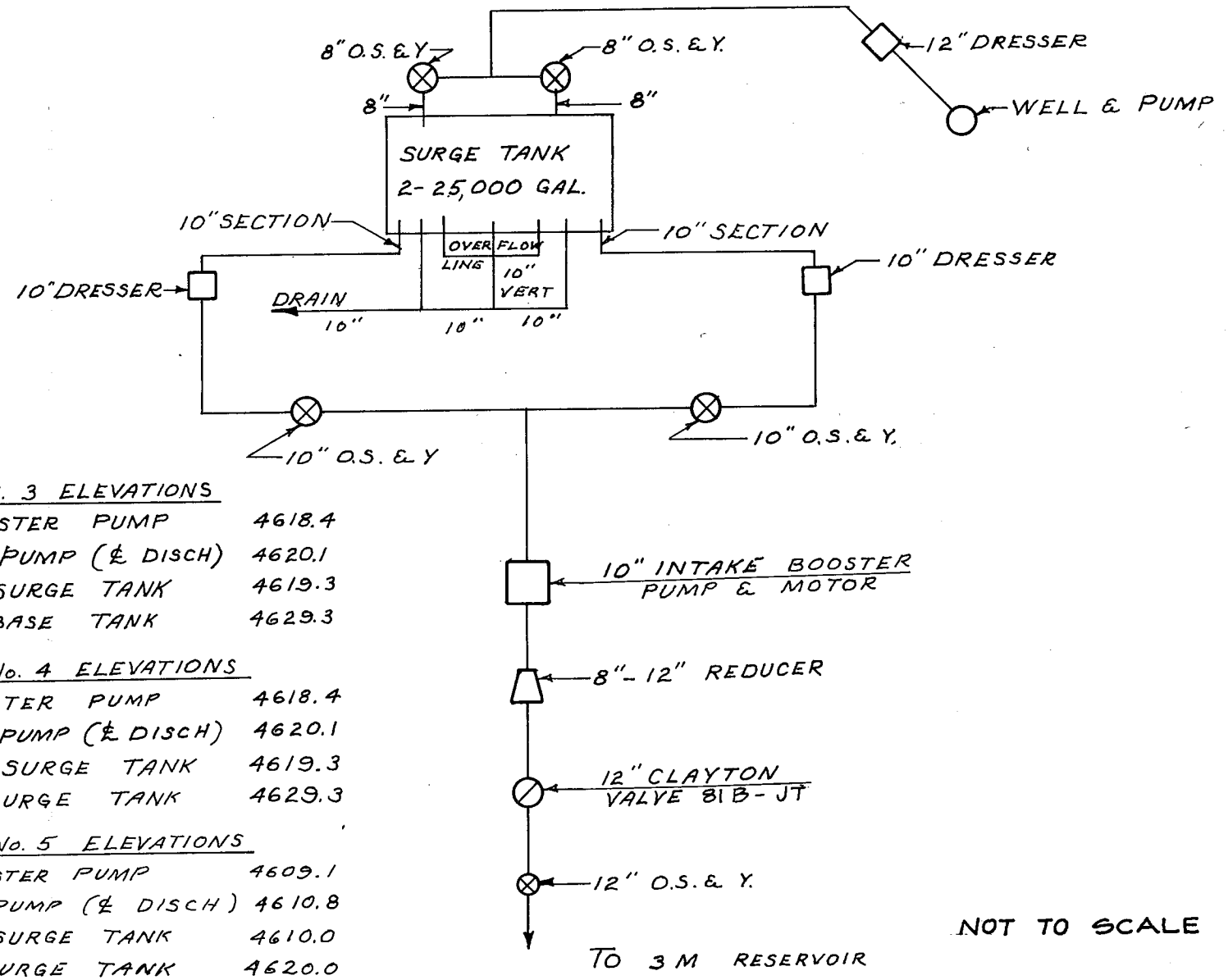
LAYOUT AT WELLS 1 & 2

SKETCH #7

3-II-75

USF200002464

TYPICAL LAYOUT WELLS 3, 4 & 5



NOT TO SCALE

PUMP No. 3 ELEVATIONS

℄ BOOSTER PUMP	4618.4
WELL PUMP (℄ DISCH)	4620.1
BASE SURGE TANK	4619.3
TOP BASE TANK	4629.3

PUMP No. 4 ELEVATIONS

℄ BOOSTER PUMP	4618.4
WELL PUMP (℄ DISCH)	4620.1
BASE SURGE TANK	4619.3
TOP SURGE TANK	4629.3

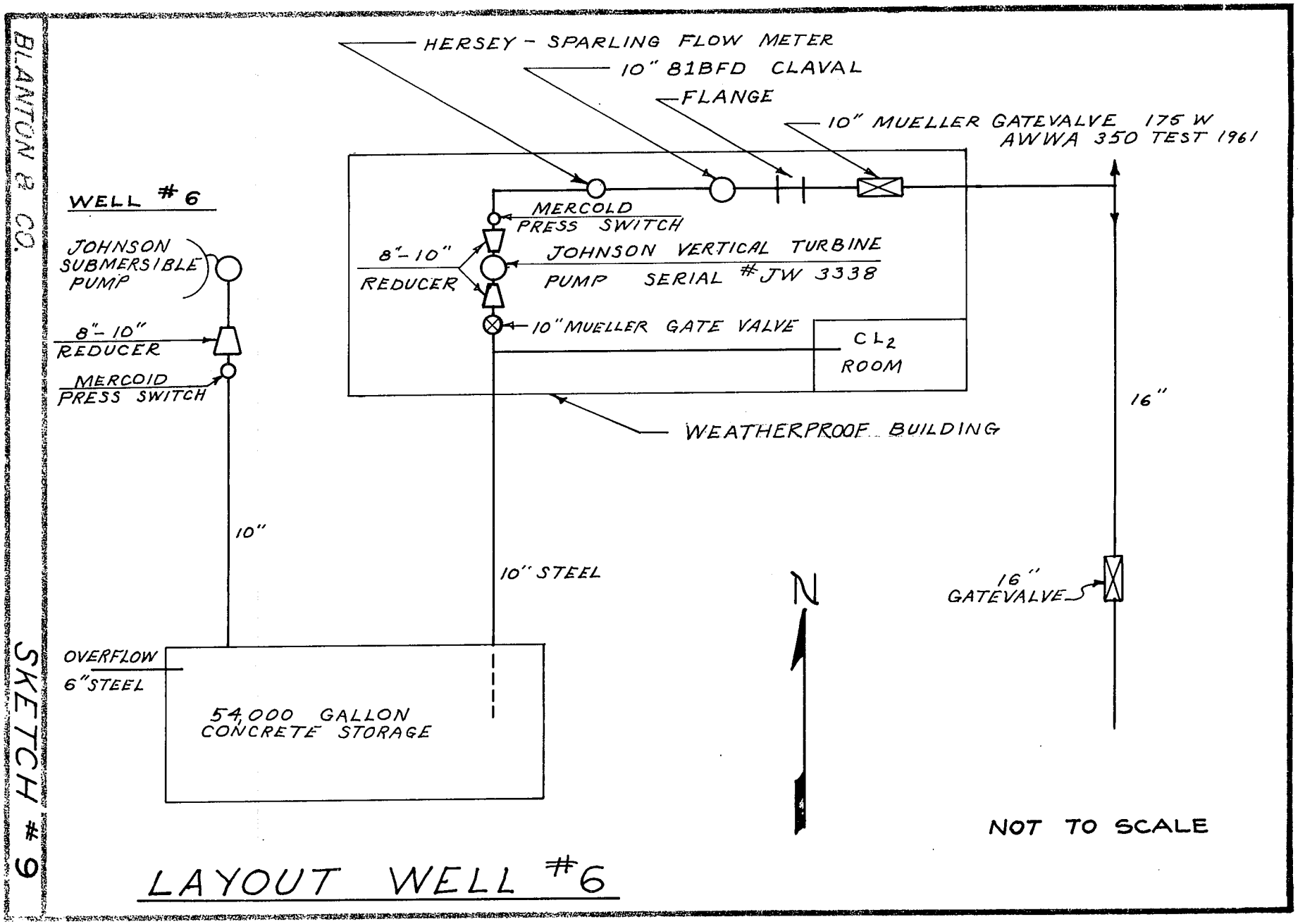
PUMP No. 5 ELEVATIONS

℄ BOOSTER PUMP	4609.1
WELL PUMP (℄ DISCH)	4610.8
BASE SURGE TANK	4610.0
TOP SURGE TANK	4620.0

SKETCH #8

9-11-76

USF200002465



BLANTON & CO.

SKETCH #9

3-11-77

USF2000002466

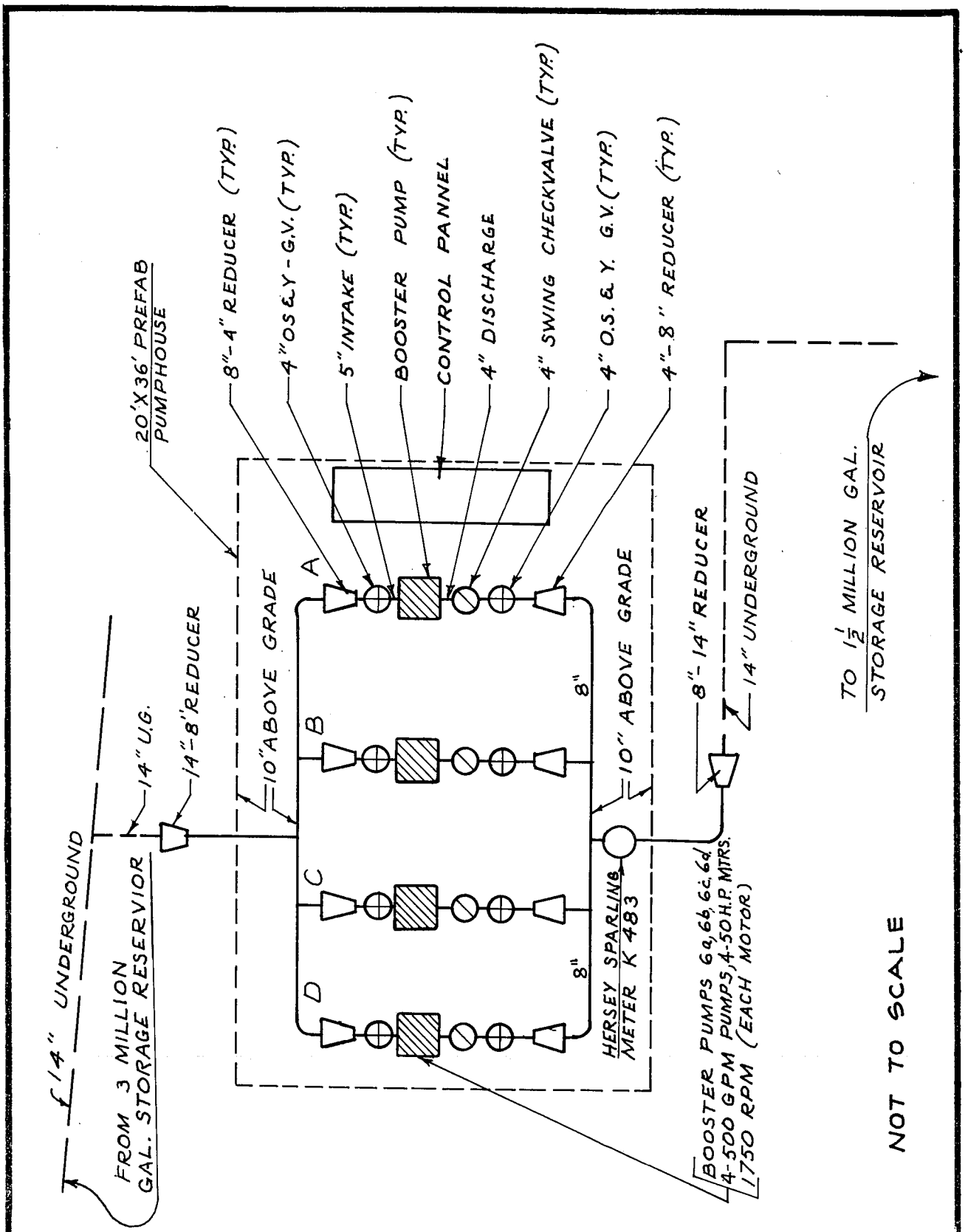
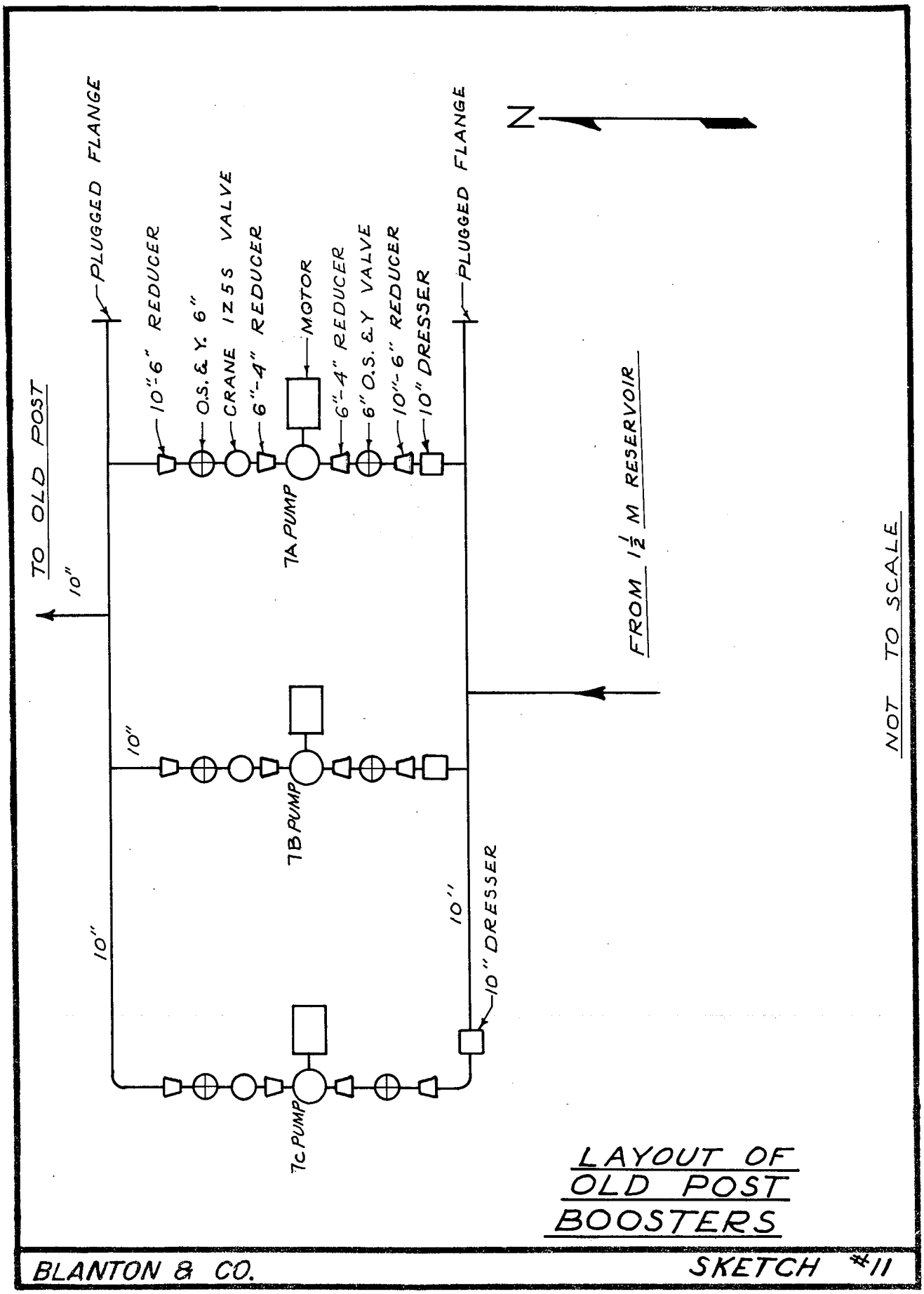


DIAGRAM OF BOOSTER PUMPS
NO. 6A, 6B, 6C, 6D. WHERRY BOOSTER STA.

BLANTON & CO. SKETCH #10

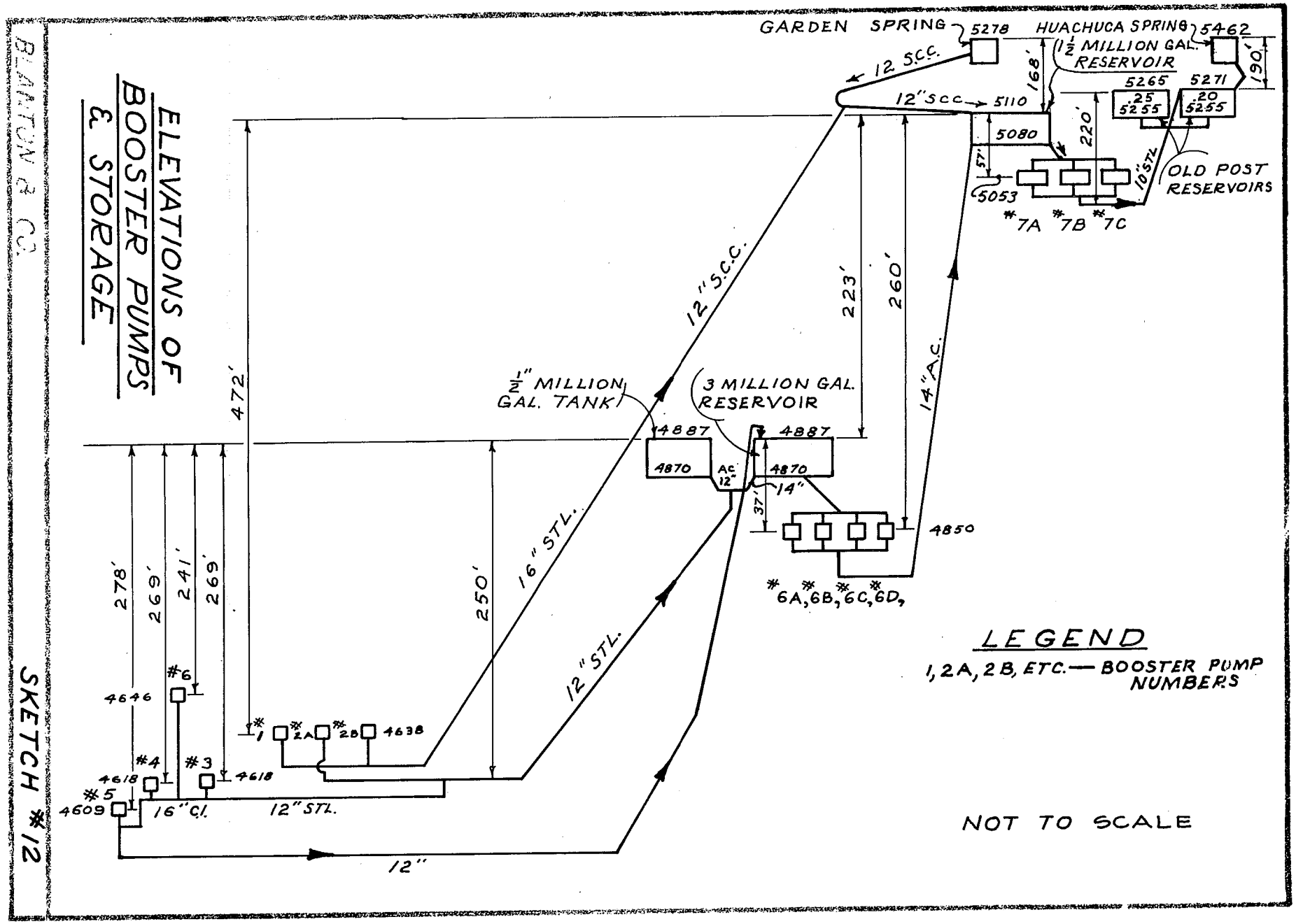
3-II-78

USF200002467

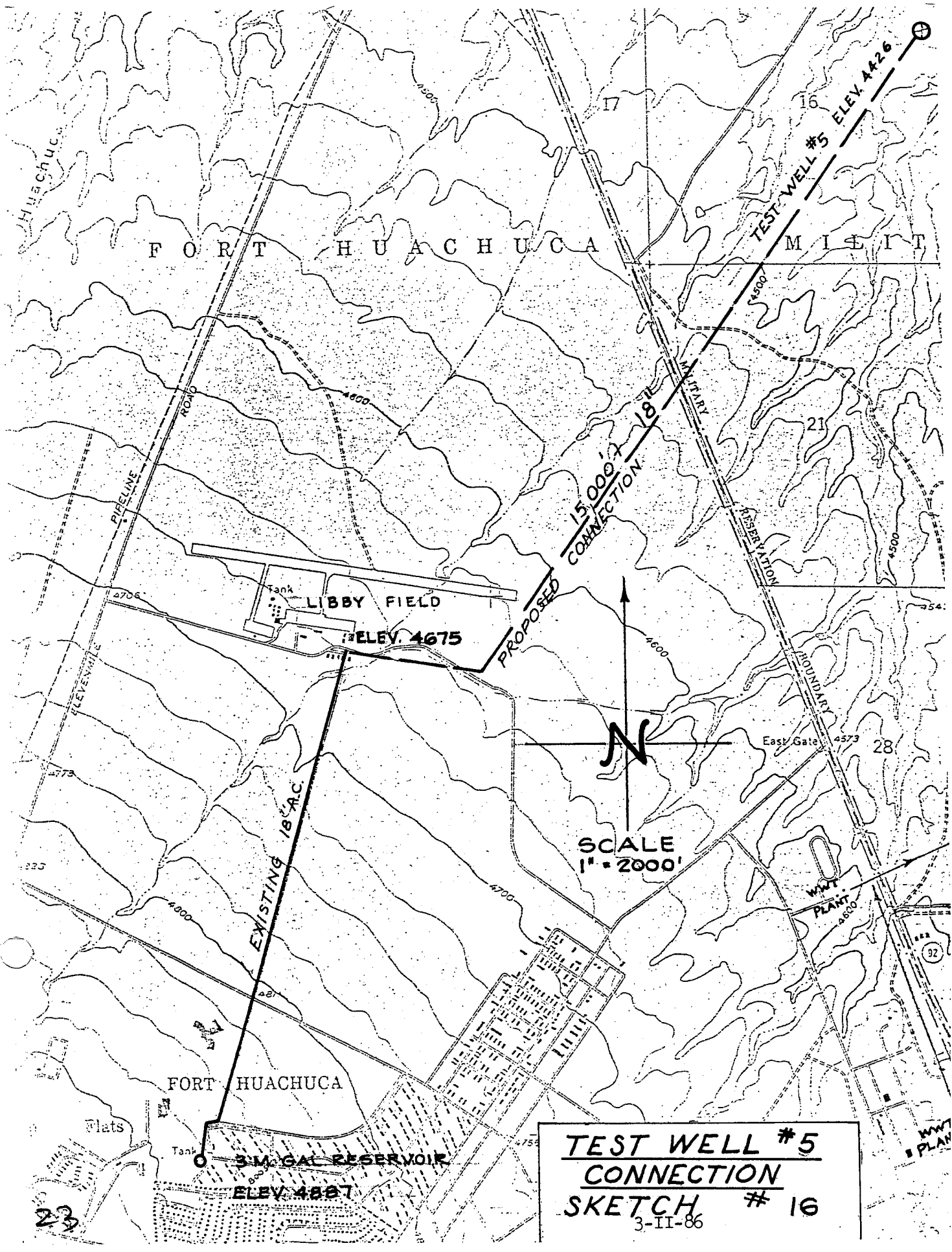


3-II-79

USF200002468



USF200002469


FORT HUACHUCA MILIT
TEST WELL #5 ELEV. 4426
15,000' 18" PROPOSED CONNECTION
MILITARY RESERVATION BOUNDARY
LIBBY FIELD
ELEV. 4675
Tank
PIPELINE ROAD
EXISTING 18" A.C.
SCALE 1" = 2000'
East Gate
WWT PLANT
FORT HUACHUCA
Flats
3 M. GAL. RESERVOIR
ELEV. 4887
TEST WELL #5
CONNECTION
SKETCH # 16
3-II-86

23

APPENDIX 4

TEST WELL DRILLING AND STUDY OF HYDROGEOLOGIC CONDITIONS

USF200002471

FORT HUACHUCA, ARIZONA
SUPPLEMENTAL REPORT

TEST WELL DRILLING
and
STUDY OF HYDROGEOLOGIC CONDITIONS

JANUARY 1974

US ARMY ENGINEER DISTRICT, SACRAMENTO
CORPS OF ENGINEERS

APPENDIX 4

USF200002472

FORT HUACHUCA, ARIZONA
SUPPLEMENTAL REPORT

TEST WELL DRILLING
and
STUDY OF HYDROGEOLOGIC CONDITIONS

JANUARY 1974

TABLE OF CONTENTS

	Page
PART I	
Authorization	1
Purpose and Scope	
References	
Geographic Setting	2
Place Names and Locations	
Climate and Rainfall	3
Geologic Setting	
Geology of the Test Well Area	4
Ground Water Conditions	5
Results of Test Well Drilling	9
Location and Design of New Well Field	10
Evaluation of Present Well Field	13
Conclusions	14
Recommendations	15
PART II	
Introduction	17
Field Work	
Test Well Construction	
Development	18
Test Pumping	19

INCLOSURES

1. Map, 1973 Ground Water Investigations
2. Sketch of Well Construction, (2 sheets)
3. Summary Sheet
4. Graphic Section
5. Geologic Logs (16 sheets)
6. Geophysical Logs, (12 sheets)
7. Water Quality Analyses, (6 sheets)
8. Drawdown - Rebound Curves, (5 sheets)

USF200002473

e. Fox, Robert C., 1972, Water Resources of a Portion of San Pedro River Valley, Arizona.

f. Heindl, L.A., 1952, Upper San Pedro Basin, Cochise County, in Ground Water in the Gila River Basin and Adjacent Areas, Arizona - a Summary, by L.C. Halpenny and others: US Geological Survey Open File Report, p. 69-86.

g. John Carollo Engineers, 1971, Water Supply Study, Phase I Fort Huachuca, Arizona; prepared for Sacramento District Corps of Engineers.

h. Post Engineer, Fort Huachuca, 1970, Analysis of Water Resources, Fort Huachuca, Arizona.

i. US Army Engineer District, Los Angeles Corps of Engineers, 1969, Master Plan Analysis of Existing Facilities, US Army Garrison, Fort Huachuca, Arizona.

j. US Army Engineer District, Sacramento Corps of Engineers, 1970, Summary of Ground Water Supply Conditions, Fort Huachuca, Arizona.

k. _____ 1971, Water Supply Investigation, Interim Report.

l. _____ 1972, Final Report on Test Well Drilling and Study of Hydrogeologic Conditions, Fort Huachuca, Cochise County, Arizona.

Geographic Setting. Fort Huachuca is located in Cochise County, southeastern Arizona in the Basin and Range Lowland province about 15 miles north of the Mexican Border and 75 miles southeast of Tucson. The south and southwest boundaries of the reservation follow the crest of the Huachuca Mountains, a northwest trending mountain range. The west boundary extends from the northern end of the range north to Babocomari River. To the north and northeast, the boundary extends along the Babocomari almost to the San Pedro River near Charleston. The main post area of Fort Huachuca is situated on the northeast flank of the Huachuca Mountains between elevations 4,700 feet and 4,900 feet. The ground surface slopes gently to the northeast and is cut by numerous drainage ravines. Immediately east of the main gate is the town of Sierra Vista. Private homes and ranches are scattered to the east and southeast. The main post area and the East Range are shown on the topographic map, Inclosure 1.

Place Names and Locations. The entire reservation is referred to as Fort Huachuca, and its boundaries as reservation boundaries (Inclosure 1). The town of Sierra Vista is east of the main post area adjacent to the reservation boundary and is rapidly extending eastward. The Post Well Field is between the post and Sierra Vista. The northern part of the reservation is divided by State Highway 90 running northwest from Sierra Vista. The area east of the highway is called the East Range. Garden Canyon is the largest canyon on the east side of the Huachuca Mountains; its mouth is about 5 miles south of the post living quarters area. Pre-existing wells are referred to by a numbering system used by the US Geological Survey. For example, the well about 3 miles northeast of the post, with water elevation 4,154

USF200002474

PART I

Authorization. The overall Upper San Pedro River Basin ground water study was authorized by TWX dated 20 June 1972 from the Department of the Army to STRATCOM, subject: "Water Resources, Fort Huachuca." SPD letter dated 15 November 1972, subject: "Water Supply Study, Fort Huachuca, Arizona," assigns technical responsibility to the Sacramento District in all engineering-geological aspects of the study. Authorization and funding for field exploration was contained in Directive No. SPD-1 dated 21 August 1972 and SPD-4 dated 29 March 1973. Foundation and Materials Branch was requested to develop an exploration program, prepare plans and specifications and supervise and administer contract work by Sacramento District Service Requests 511.1R 73.2 dated 8 September 1972 and 511.1R 73.3 dated 16 April 1973.

Purpose and Scope. The present study, which is a continuation of a study that began in 1970, is intended to make an overall assessment of the water resources at Fort Huachuca and the long range impact of water resources on the growth and development of the Post and surrounding areas. It is a cooperative project with contributions being made by the Tucson office of the US Geological Survey and the Arizona Water Commission. The Los Angeles District has responsibility for coordination with other agencies and for putting the compiled data into a single report. Inasmuch as the Sacramento District's responsibility is limited to engineering and geological aspects, this report will be restricted to a discussion of geologic field work, test well drilling and results, updating previous reports on geohydrologic conditions and making recommendations on areas that are considered to be the most promising for future ground water development. Information on recently completed field work including construction and testing of new test wells is contained in Part II of this report.

References. The following published and unpublished reports contain information pertinent to the present San Pedro Basin ground water study.

a. Blanton and Cole, 1958, Investigation of Existing and Proposed Water System at Fort Huachuca, Arizona; prepared for Los Angeles District Corps of Engineers.

b. Blanton and Company, 1973, Investigation and Recommendations for Upgrading the Water System at Fort Huachuca, Arizona; prepared for Sacramento District Corps of Engineers.

c. Brown, S.G., Davidson, E.S., Kister, L.R., and Thomasen, B.W., 1966, Water Resources of Fort Huachuca Military Reservation, Southeastern Arizona: US Geological Survey Water Supply Paper 1819-D, 57 p.

d. Davidson, E.S., and White, N.C., 1963, San Pedro River Valley in Annual Report on Ground Water in Arizona, Spring 1962 to Spring 1963, by N.D. White, R.S. Stulik, E.K. Morse, and others: Arizona State Land Department, Water Resources Report 15, p. 68-76.

USF200002475

(Inclosure 1) is numbered (21-20) 22bbb, which indicates that it is in T21S, R20E, Section 22. The code "bbb" places it in the northwest corner of the section (see p. 5 of reference c). This well is known as the "spatial resolution" well. Post production wells are known as Post Wells 1 through 6. The 9 test wells that have been drilled and tested to evaluate the area near Libby Field and the East Range are referred to as Test Wells 1 through 9. Using the USGS system East Range test wells are designated as follows:

Test Well 3	(21-20) 16 aac
Test Well 4	(21-20) 15 acd
Test Well 5	(21-20) 16 adb
Test Well 6	(21-20) 35 ccb
Test Well 7	(21-20) 11 bcd
Test Well 8	(21-20) 13 cbb
Test Well 9	(21-21) 17 bcc

Climate and Rainfall. The climate of the area is mild, sunny, and dry. Mean yearly temperature is 61°F. Average annual precipitation is about 15 inches at Post Headquarters and as much as 25 inches in the mountain range. Summer thundershowers fall mainly on the plains but often are accompanied by very little rainfall in the mountains. Conversely, the steady winter rains fall more heavily in the mountains than on the flat areas. Thus, in the 1964 Administrative Report of the US Geological Survey, Figure 5 shows heavy rainfall in July and August 1961 at Fort Huachuca, but practically no runoff in Garden Canyon. Figure 10 of USGS Water Supply Paper 1819-D shows that most of Garden Canyon runoff in 1959-63 occurred in the winter months.

Evaporation is a major factor during most of the year. According to the USGS, most of the summer rainfall and most of the canyon runoff is taken by evapo-transpiration before it can percolate into the alluvium and contribute to ground water recharge.

Geologic Setting.

The Huachuca Mountains on the southwestern boundary of the fort are a faulted complex of granite, carbonate rocks, conglomerate, and claystone ranging in age from Precambrian to Tertiary. The San Pedro Basin, northeast of the mountains, is filled with alluvial sediments referred to as basin fill. The basin fill is 360 to 800 feet thick. Bedding dips gently away from the mountain front toward the San Pedro River. The basin fill overlies variably cemented gravelly sediments referred to as the Pantano Formation, which is faulted and tilted to the southwest toward the mountain front. A detailed description of the geology is presented in USGS Water Supply Paper 1819-D.

A geologic description of the basin sediments follows:

Stream alluvium and terrace deposits:	Light-reddish-brown to light brown sand, gravel, and clay, locally derived, Thickness 0-100 feet.

USF200002476

Upper Unit of Basin Fill:	Reddish-brown gravel, sand, and silt; some red and green clay and limy silt. Thickness 163 to 331 feet in test well area.
Lower Unit of Basin Fill:	Light-gray to brown gravel and sandstone, strong to weakly cemented, lenticular; locally derived cobbles and pebbles of quartz, granite, limestone, quartzite, and rhyolitio tuff. The gravels are poorly sorted and contain a mixture of silt, sand, cobbles, and boulders. The sandstones contain interstitial clay. Thickness 100 to 420 feet in test well area.
Pantano Formation:	Dark reddish-brown to brownish-gray gravel, sand, and silt, variably cemented; boulders as much as 4 feet in diameter; fragments composed of purple to red thyolitic tuff, gray to pink felsitic tuff, purple to dark-green andesite, red and maroon shale and siltstone, light yellow granite, dark gray limestone, and light gray to white quartzite. Thickness is 663 feet in Test Well 8, 774 feet drilled in Test Well 2 near Libby Field.

Geology of the Test Well Area. Ground surface in the test well area is covered with terrace deposits or stream alluvium. The stream alluvium, mostly occupying the floors of dry washes that are typical of the desert, was probably derived chiefly from the terrace deposits and perhaps the upper part of the basin fill. It consists of sand or sand and gravel in various proportions with a few cobbles in some areas. In the East Range where Test Wells 3 through 9 are located, the terrace deposit ranges in thickness from 35 to 70 feet. It consists mostly of sand with varying, though generally low, amounts of reddish-brown clay. The upper few feet contain a higher percentage of clay, generally in the range of 20 to 40 percent, and in a few locations the near surface material is sandy clay. A few small gravel particles and thin streaks of caliche are present near ground surface.

The upper and lower units of basin fill, which underlie the terrace deposits, were described in detail in the previous report (Ref. 1). In the test wells just drilled, the upper unit consists of clay, fine to coarse sand and 1/4- to 1/2-inch gravel in various combinations. Some layers exhibit a high degree of sorting, some show little sorting in the size ranges between clay and small gravel. A degree of sorting is shown throughout by a distinct lack of larger gravel sizes and cobbles. The most common soil types found are sand and clayey sand followed by sandy clay and sand, or clay-sand combinations containing small gravels. Brown, reddish brown, and tan are the most common colors although light gray, greenish gray, and maroon clays are found in the eastern part of the area. The light gray clay

USF200002477

is limey. Cementation is generally weak or nonexistent and the material is easily drilled. In the seven test wells drilled in the East Range area, the thickness of the upper unit ranges from 163 feet in Test Well 7 to 331 feet in Test Well 9.

The lower unit of basin fill is similar to the upper unit as far as soil types are concerned. Sand and clayey sand are the most common types except in Test Well 9, which has more clay and sandy clay. The lower unit has scattered thin zones containing cobbles and a few layers or lenses containing small gravels. Overall color is generally a lighter shade of brown than the upper unit or it is brownish gray. At some locations the lower part of the lower unit is darker in color because of the increasing number of dark brown and maroon sand grains. It appears to be reworked Pantano Formation. Cementation and hardness are highly variable, although there is generally a gradual increase in both with depth. Overall, the layer is slightly more consolidated and cemented than the upper unit. The drilled thickness of the lower unit in the East Range area ranges from 139 feet in Test Well 3, to 420 feet in Test Well 7.

The Pantano Formation, which underlies the basin fill, is a slightly cemented to moderately well cemented, dark brown to reddish brown conglomerate. It was encountered in all four test wells and was penetrated completely by Test Well 8. In Well 8 it extends to a depth of 1,295 feet and is 663 feet thick. In Test Wells 3, 4, and 5 the Pantano Formation is coarse grained, consisting chiefly of sandy gravel and gravelly sand, with a small amount of interstitial clay. In the four new test wells, the formation is either much finer grained or else the cuttings were ground up to a large extent by drilling. In Test Wells 6, 7, 8, and 9, cuttings from the Pantano are mostly fine to medium grained, angular sand containing a small amount of brown clay. Geophysical logs indicates that the formation has alternating thin layers of harder (more cemented) and softer rock with a slight increase in cementation and decrease in porosity with increasing depth.

Underlying the Pantano Formation is a hard, well cemented volcanic conglomerate which is thought to be the Bronco Volcanics. The cuttings recovered from Test Well 8 were obviously ground up by drilling. They consisted of angular to subround, fine to coarse sand, mostly dark green, dark gray, brown and maroon volcanics, with a few small grains of calcite which were probably fracture fillings in larger fragments. The geophysical logs, as well as the drilling time log, indicate that the formation is considerably harder, more cemented and tighter than the Pantano Formation. In Test Well 8, it extends from 1,295 feet to 1,501 feet, the bottom of the well.

Ground Water Conditions. The test drilling just completed was intended, in at least one location, to define the vertical limits of the aquifer; that is, to locate the bottom of the Pantano Formation and explore the formation directly beneath it. The Pantano was fully penetrated by Test Well 8; and a hard, well cemented volcanic conglomerate with low porosity and permeability was encountered at 1,295 feet. The volcanic conglomerate is not considered to be a water producer except in fracture zones, and the aquifer can therefore

USF200002478

be defined as the saturated part of the upper and lower units of basin fill and the Pantano Formation.

The geohydrologic characteristics of the upper and lower units of basin fill are highly variable within short distances and are predictable only in a general way. It would be expected that the deposits would be coarser grained near their source, which is thought to be the Huachuca Mountains and the mountains to the southwest and west, and would gradually become finer grained with increasing distance from the mountains. This is only partly true, however. Boulders and cobbles apparently decrease with distance from the mountains, and gravel sizes may also decrease, but sand and silty-clay deposits do not follow a predictable pattern. In the East Range area, stream currents have apparently played a major role in determining aquifer characteristics by the sorting and deposition of the finer-grained materials. The only pattern suggested by test drilling indicates a more permeable band of deposits extending from Test Wells 3 and 5 through Test Well 7 to Test Well 6 and a less permeable area to the south and southeast where Test Wells 8 and 9 are located. There is no information concerning the area north and west of Test Wells 3, 5, 6, and 7. Test Wells 4 and 9 have the lowest specific capacity and highest proportion of fines of any of the East Range test wells. Yet Well 4 is only 1 mile southwest of Test Wells 3 and 5 and about 1 mile northwest of the spatial resolution well, which had a moderate specific capacity (16 gpm/foot). Test Well 9 is the easternmost well.

The highly variable character of the basin fill deposits is best illustrated by Test Well 8 and the associated observation well, which was drilled 100 feet to the west. The drill cuttings from the two wells had very little similarity from ground surface to 468 feet, the depth of the observation well. About 20 percent of the formation penetrated by Test Well 8 was clayey to the point of having very low permeability, but about 45 percent of the observation well had similar conditions. The observation well was extended beyond the 350-foot depth originally planned to insure that a sufficient thickness of permeable material had been penetrated to make an adequate hydraulic connection with Test Well 8. Drill cuttings and geophysical logs from Well 8 suggested that it would be a good producer, yet the specific capacity was relatively low. This suggests the liklihood of a barrier around the well that restricts inflow from the surrounding aquifer. That is consistent with the low permeability of the basin fill at the observation well.

Another indication of differing aquifer characteristics is the range of values for specific capacity of the different wells (specific capacity is a measure of well productivity. It is obtained by dividing the pumping rate in gallon per minute by the drawdown in feet and is expressed as gpm per foot of drawdown). Average figures for all the wells in the East Range area for which pumping data are available are tabulated below. For comparison, the specific capacities of the Post wells are also included.

USF200002479

Well Designation	Specific Capacity
Test Well 3	22 gpm/ft.
Test Well 4	7 gpm/ft.
Test Well 5	29 gpm/ft.
Test Well 6	21 gpm/ft.
Test Well 7	23 gpm/ft.
Test Well 8	10 gpm/ft.
Test Well 9	8.6 gpm/ft.
Spatial Resolution Well	16 gpm/ft.
Post Well 1	15.4 gpm/ft. at 593 gpm
Post Well 2	17.1 gpm/ft. at 615 gpm
Post Well 3	17.7 gpm/ft. at 760 gpm
Post Well 4	40.7 gpm/ft. at 611 gpm
Post Well 5	6.2 gpm/ft. at 450 gpm
Post Well 6	37.5 gpm/ft. at 601 gpm

Information on Post Wells from Blanton and Company report, reference b. Values are for 1971 production.

Previous reports have suggested that the upper unit of basin fill is a better water producer than the lower unit. The recent test well drilling indicates that this is not necessarily true and that permeability is related more to clay content than consolidation and cementation. It also suggests that either unit can be more permeable than the other depending on local conditions of sorting and deposition. For example, Test Well 9 has 208.5 feet of upper basin fill that is saturated but has the lowest specific capacity; Well 7 has the least saturated upper basin fill, 1.8 feet, and the highest specific capacity. Test Well 7 apparently draws mostly from the lower unit of basin fill. Test Well 6, which is second to Well 7 as a good producer, has only 81 feet of saturated upper basin fill and apparently produces mainly from its 320 feet of lower basin fill.

In spite of the highly variable nature of the basin fill deposits, they are considered to be the primary water producers in the East Range area. The Pantano Formation was thought to be barren except in fractured zones prior to the first test drilling. It now appears that it produces mainly through pore space between grains. Geophysical logs as well as drill cuttings suggest that it can produce small to moderate amounts of water, although it is not nearly as permeable as the basin fill units. This conclusion is also supported by pumping tests in Test Wells 1 and 2, which produce entirely from the Pantano.

The Pantano Formation is considered to be the basal unit of the aquifer. The Bronco Volcanics, which underlie the Pantano (at least in the vicinity of Test Well 8), is a hard, well cemented formation. Well cuttings and geophysical logs indicate that it is essentially impermeable except in fracture zones.

USF200002480

The four new test wells were drilled between the 4,100- and 4,150-foot ground water contours where the water table gradient is relatively flat (Inclosure 1). Water levels at the new test wells are consistent with previous measurements, and the most recent water level measurements in the East Range area indicate that the ground water surface has remained essentially stable since the previous report was issued in April 1972. Water levels for those wells in the East Range area that can be measured, are tabulated as follows:

Well Designation	Water Depth September 1973	Elevation	Water Depth February 1972	Elevation
Test Well 3	277.45	4,142.6	277.1	4,142.9
Test Well 4	275.54	4,149.5	275.05	4,150.0
Test Well 5	282.0	4,144.0	281.8	4,144.2
Spatial Resolution Well	326.02	4,154	326.3	4.153.7
Test Well 6	149.19	4,137.8		
Test Well 7	213.20	4,133.5		
Test Well 8	243.55	4,145.8		
Test Well 9	156.50	4,142.7		

Although the water level has been stable in the East Range and other areas that are not pumped appreciably, the most recent water level measurements, which were made September to November 1973, indicate that the steady decline of the water table reported previously is continuing in the areas that are heavily pumped. The best information available indicates that the decline since the last water level measurements were made ranges from 1 to 9 feet in the Post Well Field-Sierra Vista area. However, some of the previous readings went back as far as 1968. Still, the maximum rate of decline is as much as 1.8 feet per year, which is similar or slightly higher than that reported previously. Although distinct cones of depression have not formed in newly developed areas, the declining water table is following the residential and commercial development to the east and southeast and is expected to continue with further expansion of Sierra Vista. Little reliable information is available for the Huachuca City area, but it is apparent that the water table is declining, perhaps on the order of 1 foot or less per year. The cone of depression around Huachuca City does not appear to be expanding rapidly and is still separate from the cone at Sierra Vista.

A new ground water map has been prepared as part of the present investigation (Inclosure 1). The main differences between it and previous maps are slight modifications in the 4100 and 4150 contours to reflect recently determined water levels in Test Wells 6 through 9, a change in the configuration of the cone of depression surrounding the Post well field and the elimination of the ground water ridge or nose, east of Sierra Vista. The information obtained from the new test wells is reasonably consistent with previous data and the slight revisions in the ground water contours in the East Range requires no explanation. The change at the Post well field is a result of new readings that suggest previous readings from two wells were in error, probably

USF200002481

due to malfunctioning air line measuring devices. The new readings, which are believed to be reliable, indicate that the lowest part of the cone is elevation 4,117 at Well 2. This eliminates the 4,100-foot contour around Well 3 as previously shown and shifts the 4,150 contour to the southwest and west. The 4,150 contour is also expanding to the east as a result of increased pumpage in that direction at residential and commercial developments. The prominent nose that appeared on previous maps about 6 miles east of the Post's main gate has disappeared as a significant feature because of new water level measurements in that area. It appears now that old measurements made in shallow stock wells represented a perched water level. Readings from recently drilled deep wells reflect entirely different water levels and water table configuration.

Although the revised ground water map represents a minimal number of new water level readings, it represents the best information available. Other water level measurements would be highly desirable, especially in some key wells; but unfortunately they are impossible to obtain. Many privately-owned wells are not designed for taking water level readings and can only be measured when the pump head has been pulled for repairs or other reasons. However, the map depicts the conditions that would be expected; namely, that the water level is continuing to decline in heavily pumped areas and that the cone of depression is gradually extending into the recently developed areas where pumpage has been increasing. In areas that are isolated and have not been developed extensively for water production such as the East Range, the water level has remained essentially stable since February 1972. It is somewhat encouraging that the cones of depression surrounding the post well field and the Huachuca City areas have not expanded more than they have.

Results of Test Well Drilling. The recent test well drilling confirms much of the information obtained from the previous investigations; namely, that the water table is very flat in the East Range, that the primary water producers are the upper and lower units of basin fill, that the Pantano Formation has low to moderate permeability, that the basin fill deposits are highly variable laterally as far as soil types and permeabilities are concerned, that the vertical boundaries between geologic units are fairly regular and predictable and that the aquifer in the East Range is sufficiently permeable to make feasible a new well field there. New information resulting from the 1973 drilling include a determination of aquifer thickness at one location (Test Well 8) and an extension of detailed knowledge of geologic conditions into a large area of the East Range. This permits a realistic well field design based on aquifer permeability. The drilling and test pumping have also made a major contribution to the digital model analysis that is being done by the Arizona Water Commission. The digital model, when verification has been completed, will be used to analyze well locations and interactions to determine the best pattern to be used in a future well field as well as the broader consideration of how the water supply is going to react basin-wide to various projected population figures.

USF200002482

Location and Design of New Well Field. As the April 1972 report (ref. 1) pointed out, there are four distinct areas of Fort Huachuca that might be considered as potential locations of new wells: (1) North of the post proper and west of Highway 90 (Libby Field area and westward), where ground water is in the Pantano Formation, (2) the Post Well Field-Sierra Vista area, where the basin fill units are thick and produce most of the ground water being used, (3) the strip of alluvial apron extending southeast of the Post Well Field to the mouth of Garden Canyon, and (4) the East Range area. Area 1 was explored by Test Wells 1 and 2 in the previous test well drilling. The water table there is only a few feet higher in elevation than it is in the East Range test wells and the aquifer is the lower yielding Pantano Formation. Wells located in this area would be difficult and expensive to construct and would be low to moderate producers. A larger number of wells would, therefore, be needed for a given quantity of water and pumping costs would not be appreciably lower than from other areas. Initial costs of extra wells would partly or perhaps wholly offset the cost of pipeline to bring water in from other areas. The area is not considered feasible for the location of a well field. Area 2, the present Post Well Field, is overdrawn at the present and is too vulnerable to developments on private land in and around Sierra Vista. Area 3 is small and it too is vulnerable to private developments, such as Tenneco to the east and southeast.

The East Range area is the only one worth serious consideration. Its only real disadvantage is the distance from there to the use area in the main post. It has several advantages, which are discussed in the following paragraphs.

The East Range is an extensive area, comprising about 44 square miles. The city of Sierra Vista is developing along its southwestern boundary and Huachuca City is just outside its northwest corner. However, most of it is isolated from developments at the present and in the foreseeable future. Its size provides a great deal of isolation from private developments that might compete with the post for water and also permits sufficient spread between production wells to minimize interference of wells with each other. Of all the areas that might be considered, the East Range is least likely to be affected by city or private wells.

The testing that has been done in the East Range provides reasonable assurance that the aquifer in selected areas is permeable enough for construction of good production wells. The saturated basin fill ranges in thickness from 153 to 443 feet in the test wells; and where it consists of the coarser-grained sand or gravel facies, wells that will produce 1,000 gpm or more can probably be constructed. The upper part of the Pantano Formation will also contribute smaller but significant amounts of water.

According to the US Geological Survey (Ref. c, p. D20, plate 1), ground water in the East Range moves down gradient in a general northeast direction and is forced to the surface by a ground water barrier formed by consolidated volcanic and sedimentary rocks along the San Pedro River

USF200002483

and to a lesser extent by consolidated rocks in the valley of the Babocomari River. Ground water reaching the San Pedro River maintains a short reach of perennial flow at the Charleston gaging station. Thus, these two areas act as a natural drains for excess ground water in this part of the basin. A well field as contemplated in the East Range area may intercept and make use of a large part of this overflow, which is now being lost to evapotranspiration.

One of the major objectives of the present study was to determine, through use of the digital model, the reaction of the ground water basin to different water production levels needed to support several alternate population figures and time periods. These alternates are listed in Table 1. They include post population figures ranging from 12,000 (present population) to a maximum of 50,000 and a time period from the present to 2030. Growth of Sierra Vista and surrounding communities is being estimated by the Arizona Water Commission and is taken into account in the analysis.

In order to analyze the effects of water withdrawal, it is necessary to know the location and production levels of future wells. A well field has therefore been designed for the East Range area that can meet water requirements for a post population of up to 50,000. Pertinent data on water requirements and wells required are shown in Table 1; locations of proposed wells are shown on Plate 1. The locations of the proposed wells have been reviewed by the consulting firm, Harshbarger and Associates, to insure that optimum well locations are being used. The well field location and design will be analyzed by the digital model when verification is completed but the results are too late for discussion in this report.

The first step in designing a well field is to determine what water consumption will be for the different projected population figures. The methods used to determine water requirements are summarized in the following paragraphs.

Although some water is obtained from the collection systems in Garden and Huachuca Canyons, the flow rates vary considerably and these sources are not considered to be reliable. Peak requirements occur during the time of greatest irrigation demand and least production from Garden and Huachuca Canyons; therefore, these sources cannot be considered as contributors to the peak demand; and it is assumed that the entire water supply will be produced from wells. Irrigation water for the golf course and Chaffee Parade Ground presently is supplied by effluent from the post sewage plants after first undergoing secondary treatment and chlorination. Therefore, these two areas need not be included in determining irrigation requirements.

Much of the information on well production and water consumption was obtained from the report by Blanton and Company, reference b. According to the report, the capacity of the well field as it is presently operating is 3.49 million gallons per day (mgd) if the pumps are operated only 16 hours a day in accordance with the requirements of TM 5-813-2.

USF200002484

TABLE I

Summary of Present and Future Water Requirements

	Post Population	Peak Water Requirements	Deficit *	Actual Avg Consumption Year Round	Proposed Source to Make Up Deficit
Alt I	12,000	4.613 mgd	1.123 mgd	2.676 mgd	No additional source. Pump existing wells as needed.
Alt Ia	12,000	4.613 mgd	1.123 mgd	2.676 mgd	Test Well No. 5. Pumping 16 hrs/day at 1,500 gpm= 1.44 mgd. Total Available = 4.93 mgd.
Alt II	25,000	7.5 mgd	4.01 mgd	5.2 mgd	Test Well No. 5 plus new wells, A, B, C. Total Available - 7.8 mgd.
Alt III (1980)	Same	as	Alt	II	Test Well No. 5 plus 10 new wells, A through K. Total available 14.53 mgd.
(2000)	50,000	15.0 mgd	11.51 mgd	10.40 mgd	
(2030)	Same	as	Alt	III (2000)	

*Deficit is difference between present well field capacity (3.49 mgd) and peak requirement.

USF2000002485

The total water required to meet domestic and fire demand for the present population (equivalent population of 12,980) is 2.453 mgd. The daily requirement for irrigation (excluding the golf course and Chaffee Field) is 2.160 mgd during the May-June season of heavy irrigation. Those figures were used to obtain a maximum per capita per day water consumption value, which was in turn used to calculate maximum water requirements for other (projected) population figures. It was assumed that irrigation needs for both public areas and residences would grow in the same ratio as at the present and in direct proportion to the increasing population. This maximum or peak water requirement was used to determine the number of wells needed. Actual average daily water consumption was calculated from well production records kept by Fort Huachuca personnel. This actual water consumption value is used to determine the reaction of the ground water basin to water withdrawal.

To determine the number of additional wells required to meet future water needs, it was assumed that each of the new wells would be capable of producing 1,000 gpm. Pumping 16 hours a day, each new well could contribute 960,000 gallons per day to the peak water requirement. In practice, however, each new well (including Test Well 5) would only be required to produce about 665,000 gpd to meet average water needs on a year 'round basis.

Of the 8 deeper wells on the East Range, five are considered to be good producers (Test Wells 3, 5, 6, 7, and Spatial Resolution Well), and three are considered to be poor or mediocre producers (Test Wells 4, 8, and 9). The low-producing wells are roughly in an east-west line from 1 to 2.5 miles north of the reservation boundary (Inclosure 1). The good producers are generally to the north and west. Completely definitive coverage of the East Range by test wells is precluded by its size and related economic and time factors. However, it is believed that adequate information is available to define a less permeable area in the southern part of the East Range and a more permeable area to the north and west. The proposed well field is laid out according to this concept. New wells are spotted near the higher producing test wells, where definitive information is available, and near the Spatial Resolution Well, which is a moderate producer. The other wells are spread out over the area in a more-or-less random pattern except that spacing is at least one mile. At that spacing, mutual interference between wells will occur if adjacent wells are pumping at the same time, but the interference will be minimal.

The proposed wells have letter designations from A through K, which suggests construction priority. If the area is developed as a well field, it is assumed that Test Well 5, which is a full size (16 inch) well, will be put on line first. Logically, the sequence of development would start in this area, which will require less pipeline, and work to the northeast.

Evaluation of Present Well Field. Although the present well field is capable of producing 3.49 mgd, it should be noted that Post Well 5 appears to be deteriorating and may be in the beginning stages of failure. Since 1958 the specific capacity of the well has decreased from 14 gpm to 6.2 gpm.

USF200002486

This may reflect a deterioration of the well casing after 31 years or else the water level has declined so far that the main producing zone is now the less permeable lower part of the basin fill and/or the Pantano Formation. In either case, cleaning and renovating the well would not be worthwhile; in fact, the casing has probably corroded to the extent that acidizing or other method of cleaning may cause the casing to fail. It should be assumed that the useful life of Well 5 will soon end. Wells 1 and 2 are also questionable. According to Blanton and Company (Reference b, p.51), the pump bowls are so close to the dynamic water level that they are in danger of sucking air. The Blanton report recommends lowering the pump bowls, or, preferably, installing new pumps. Wells 1 and 2 are only about 750 feet apart, they were drilled in 1939 and 1940 and they have only about 150 feet of saturated basin fill. When a new well field in the East Range is put into production, it seems desirable to shut down one or both. This would relieve some of the pressure on the present well field and would save the expense of renovating pumps in wells whose value is questionable.

Conclusions. Since the ground water study was begun in 1970 a large amount of data has been collected and analyzed. The major contribution has been the results of test well drilling and pumping but also includes information that was available from the US Geological Survey and from the post engineer's office as well as from water companies in Sierra Vista and Huachuca City and private individuals throughout the area. The data have been updated as much as possible during the drilling and testing operations of 1971-72 and 1973. The following conclusions are based on an assessment of all data collected since the study was begun by the Sacramento District in 1970.

a. With the exception of a few small areas where perched water is present, all of the saturated materials in this part of the San Pedro Basin are part of a single aquifer, that is, they are hydraulically connected.

b. All deep wells in the area including the post wells, the Sierra Vista and Huachuca City semi-public wells and private wells draw their water from this aquifer.

c. The water level in the areas where wells are concentrated and are heavily pumped has declined for a number of years and is continuing to decline at a rate that is controlled by pumpage.

d. Cones of depression developed around the Sierra Vista-Post Well Field and Huachuca City areas are expanding slowly, and at Sierra Vista the cone is beginning to follow the new residential and commercial development to the east.

e. Recharge to the ground water basin is less than the water being withdrawn.

f. The major water producing zone of the aquifer consists of the coarser grained layers of the upper and lower basin fill units.

USF200002487

g. Permeability in the basin fill units is more dependent on sorting and content of fines than on depth or cementation.

h. There are wide lateral variations in permeability within short distances in the basin fill units and these variations can be predicted only in a general way.

i. The upper part of the Pantano Formation is slightly permeable and can produce smaller but significant amounts of water, which is probably produced through interconnected pores rather than through fractures as assumed prior to test well drilling.

j. The only area within the reservation boundaries that is worth considering for large scale ground water development is the East Range

k. The Post Well Field needs to be modified for the following reasons:

(1) The wells are overpumped during periods of high water usage.

(2) The wells are spaced too close to each other and to nearby Sierra Vista City wells.

(3) Three wells (Wells 1, 2, and 5) are deteriorating because of water level decline and/or other reasons and have a limited life expectancy.

Recommendations. Inasmuch as this report is being completed prior to verification of the Arizona Water Commission's digital model, it should be understood that some modification of recommendations might be necessary to reflect conclusions reached in the computer analysis of data. Modifications, if any, would probably be minor, however, and would likely involve only changes in the configuration of the proposed well field. Questions concerning water resources required to support future population growth can be answered better when the computer analysis is completed. Recommendations contained here are intentionally restricted to considerations dealing directly with immediate post water requirements and in a general way to future ground water development.

a. The need for the East Range well field is obvious, and development should begin there as soon as design and construction funds can be obtained.

b. Development should start by putting Test Well 5 into production to relieve pressure on the Post Well Field. This is necessary for the present post population.

c. As soon as possible Post Wells 2 and 5 should be phased out and replaced by constructing and putting Well A into production.

d. In general, the priority suggested by designations of proposed Wells A through K should be followed as additional water is required unless economics of distribution system or land usage suggests a modification of priorities.

USF200002488

e. Because there is considerable lateral variation in aquifer conditions, construction of a production well at any location must be preceeded by drilling a pilot hole and logging it geologically and geophysically as the initial part of the construction contract.

f. The basic design for future production wells, as suggested by test well drilling, should be as follows:

Depth: 800 feet
Well type: Nongravel packed
Drilling Method: Conventional or reverse rotary
Grout surface seal: Approximately 75 lineal feet
Casing: diameter - 16-inch I.D.
perforations - horizontal louver, $\pm$ 48 square inches of opening per foot
Development: By swabbing or bailing and pumping

USF200002489

PART II

Introduction. In order to continue the investigation of the ground water supply conditions at Fort Huachuca, Arizona, another exploratory drilling program was authorized during the summer of 1972. Specifications were prepared for the drilling of four additional small test wells with a smaller observation well to be drilled 100 feet away from a designated test well. The contract for the test drilling was awarded to Keith L. Rising of Lamar, Colorado. A description of the procedures followed to drill, develop and test pump the wells is contained in this section.

Field Work. For the most part the field work consisted of technical supervision of the drilling, developing and test pumping of the wells. The test wells are all located in the East Range area of Fort Huachuca (Inclosure 1).

Notice to proceed was given on 9 May 1973 and the contractor began to move in his equipment. The contractor was requested to drill the observation well and Test Well 8 first in order to furnish information to the Arizona Water Commission at the earliest time for use in their digital model. However, due to the busy construction season in the area, the contractor was unable to hire sufficient men to do the developing and pumping concurrently with the drilling operation. He then decided to drill and case all the holes first and then go back for the developing and pumping with his own crew. This plan was followed to the completion of the contract.

Cuttings were sampled continuously during the drilling and geologic logs (Inclosure 5, sheets 1 through 16) were made on the basis of the drill samples. The US Geological Survey ran sieve analyses of the samples. Geophysical logs (Inclosure 6, Wells 6, 7, 8, 9), run after the completion of drilling of each well, correlate with the geologic logs, and the combined information was used to set up the perforation schedule for each well. The geophysical logs also define the major geologic units in the wells.

A water quality analysis of each well sample was made by the US Geological Survey and the results are in Inclosure 7, sheets 1 through 6. The Preventive Medicine Unit of the base hospital ran a bacteria analysis of each sample.

Test Well Construction. A generalized picture of the test wells can be seen graphically in Inclosure 2. Tabulated data for each well along with a graphic section through the wells are shown in Inclosures 3 & 4. The drill rig was a Joy, Model 300, that had been modified to be equivalent to a Model 350. The wells were drilled by the standard rotary method. Drilling fluid pumped from one end of a small sump by means of a National C-150 (6-3/4" x 12") mud pump passes down the inside of the drill stem and out of the bit. It lubricates the bit, stabilizes the walls of the hole and carries the cuttings up the hole to be discharged over a shaker screen. Various sized, toothed, tri-cone roller bits were used to drill the holes.

USF200002490

The small observation well was drilled first using a 9-7/8-inch bit to a depth of 51 feet. A 7-7/8-inch bit was then used to advance the hole to bottom at 468 feet. The adjacent Test Well, Number 8 was advanced to 1,305 feet with a 13-3/4-inch bit. Due to the hardness of the materials encountered at this depth it was decided to reduce the bit size to 7-7/8 inch to complete the hole to 1,501 feet. This small heavy-duty tri-cone bit had chisel shaped compacts instead of teeth.

The other three Test Wells, Numbers 6, 7, and 9, were drilled to an approximate depth of 1,000 feet using 13-3/4-inch bits. The upper part of each hole was drilled with a slightly larger bit to facilitate the placement of a neat cement grout surface seal.

Upon completion of the drilling of each hole, geophysical logs were run by Welex Corporation of Santa Fe Springs, California. The logs consisted of induction electric, spontaneous potential, compensated acoustic velocity, and compensated density. These logs were used to determine formation and unit contacts, porosities and to aid in setting up the perforation schedules for the casing. The geophysical logs are included as Inclosure 6, Wells 6 through 9.

The casing for the observation well is standard 4-inch pipe with threaded collars. Perforations are 1/8 x 2-1/4-inch milled slots having four vertical rows and two horizontal rows per lineal foot to give 2-1/4 square inches of opening per lineal foot. The perforation schedule is shown in Inclosure 3. Casing for the Test Wells conforms to ASTM Spec. A36-70 and API Std. 5LX grade X52. It is 10 inches I.D. with standard 1/4-inch wall thickness. The perforations are 1/8" x 2-7/8" full flow horizontal louvre-type slots. Eight vertical rows with 12 openings per row provide 33-1/2 square inches of opening per lineal foot of perforated casing. The perforation schedules for the Test Wells are shown in Inclosure 3. The casing is collared and came in 30-foot lengths or slightly longer. The casing sections are joined by welding and the bottom sections are sealed with field-welded steel plates.

In Well 8, the 10-inch casing extends to 1,305 feet. Below this depth the 7-7/8-inch hole is open to the bottom at 1,501 feet. In Well 7, the casing extends to 810 feet, and the hole is open to the bottom at 1,002 feet. In both cases the materials encountered near the bottom were considered to be cemented enough to stand open without casing.

After casing was set in each hole, the grout surface seal was emplaced. A packer was set at the prescribed depth and neat cement grout was pumped through a 1-1/4-inch grout pipe by means of a Worthington (4-1/2" x 6") pump. Surface seal depths are shown in Inclosure 3. The extra depth of seal in Well 7 was intended to provide adequate support for the casing, which was hung in the hole.

Development. Initial development of the wells was started with the drill rig. Cuttings were pumped from the mud pits and fresh water was added to thin the

USF200002491

mud. With the bit on bottom this thinned fluid was circulated in the hole to flush out all the fine cuttings and some of the remaining mud cake. After this initial development, the drill rig was moved and a bailing rig was set in place. Bailing approximately 23 hours with an 8-inch I.D. bailer 23 feet long was required for each hole. A small three-inch bailer was used in the observation well and in the smaller diameter section of Well 8.

The development stage of the contract was completed by installing a turbine pump in each well and pumping and surging each for approximately 20 hours to clean out the remaining mud and pull sand in to open up the formation. The wells were pumped at a low initial rate and were surged whenever the water cleared. As development progressed, the pumping rate was increased until the wells were producing at least 400 gallons per minute. The pump used for developing and testing had 7-1/2-inch bowls with 19 stages and was powered by a Ford 428 V8 industrial engine. Water levels were measured by means of an electric probe. Water flows were measured with a three- or four-inch orifice in a six-inch pipe and a calibrated piezometer tube.

Test Pumping. Wells 6, 7, and 9 were tested at five pumping rates varying from 175 gpm to 450 gpm. Water levels were measured at timed intervals during drawdown until the pumping level stabilized. Then the pump was shut off and water levels were measured to determine the rate of rebound. When the water level stabilized at the highest pumping rate the well was pumped for 24 hours to determine transmissivities and to see if there would be any short term boundary effects on the aquifer. Well 8 was tested at four rates varying from 175 gpm to 400 gpm. Water level measurements were taken in Well 8 and the observation well simultaneously during the various pumping tests. The final pump test was run continuously for five days. Transmissivity and the storage coefficient were calculated from the results of this test. Boundary effects were also checked. Specific capacities of the wells at various pumping rates are in Inclosure 3; the drawdown rebound curves are in Inclosure 8, sheets 1 through 5.

Following the pump tests the wells were sterilized with granular calcium hypochlorite (HTH) and let stand for 24 hours. The chlorine was then pumped to waste and water samples were taken for standard water quality analyses, which were run by the US Geological Survey. The results are in Inclosure 7, sheets 1 through 6. Bacterial analyses were done by the Preventive Medicine Unit of Fort Huachuca; the results were negative.

Upon completion of the contract the Test Wells were capped and locked.

USF200002492

Fort Huachuca, Arizona
Geologic Log
Test Well No. 6

0.0 Terrace deposits
Gravelly sandy clay, reddish brown, moderately plastic, dry, sand and gravel are angular to rounded, maximum size 2-1/2 inches, sand is fine to medium, mostly fine.

1.6 Caliche with sand and gravel, reddish gray.

4.6 Sand, brown, fine grained, well sorted, clean, abundant mica.

15.0 Sand is slightly lighter brown color, angular to rounded.

30.0 Sand is light brown to gray, few medium size grains.

44.0 Sand is fine to medium grained, trace of tan clay.

62.0 Upper Unit of Basin Fill
Sand, medium brown, fine to medium grained, angular to rounded, clay content varies from 0 to 10 percent.

67.0 Sandy clay, tan, moderately plastic, about 30 percent fine to medium grained, angular to rounded sand.

70.0 Clayey sand, tan, fine to medium grained, some caliche, clay content variable, generally 30 to 40 percent, moderately plastic.

101.0 Sandy clay, tan, moderately plastic, sand content varies from 15 to 30 percent, fine to medium grained.

103.0 Clayey sand, fine to coarse, 30 to 40 percent tan moderately plastic clay.

105.0 Sandy clay as at 101.0 feet.

138.0 Sand, fine to medium grained, angular to rounded, well graded, numerous maroon colored grains, trace of tan clay.

147.0 Sandy clay, tan, moderately plastic, 20 to 35 percent fine to medium sand.

158.0 Clayey sand, fine to coarse, angular to rounded; clay is tan, moderately plastic, clay content variable from 10 to 40 percent.

USF200002493

Test Well No. 6 (cont'd)

189.0 Sandy clay, tan, moderately plastic, 10 to 30 percent fine to coarse sand.

195.0 Sand, fine to medium, angular to rounded, mostly dark brown and maroon, trace of tan clay.

208.0 Sandy clay-clayey sand; about equal amounts of moderately plastic, tan clay and fine to coarse grained, angular to subrounded sand.

230.0 Lower Unit of Basin Fill
Clayey sand, fine to medium with few coarse grains, more volcanics than above, angular to subround, about 40 percent tan clay.

250.0 Sandy clay, similar to 230.0 feet with a predominance of tan clay, 30 to 40 percent fine to medium sand.

258.0 Sand, fine to medium grained, scattered coarse grains, angular to subround, trace of tan clay

262.0 Clayey sand, fine to coarse, angular to subround, 10 to 30 percent tan clay.

268.0 Sand, fine to coarse, trace of tan clay.

272.0 Clayey sand as at 262.0 feet.

282.0 Sand, fine to medium grained, scattered coarse grains, angular to rounded, 5 to 15 percent brown, moderately plastic clay, scattered streaks of clay.

335.0 Clayey sand, fine to medium grained, clay content varies from 15 to 30 percent.

345.0 Clayey sand as at 335.0 feet with scattered small gravel fragments.

264.0 Clay content decreases to less than 5 percent.

378.0 Clayey sand-sandy clay, about equal amounts of brown clay and fine to medium grained sand.

390.0 Clayey sand, fine to medium with few coarse grains, 10 to 15 percent moderately plastic brown clay. Thin clay streak at ± 401.

USF200002494

Test Well No. 6 (cont'd)

410.0 Sand, mostly fine to medium grained, mostly angular, clay content varies from 0 to 10 percent; most of sand grains are dark brown and maroon, looks like reworked Pantano Formation.

490.0 Sand as at 410.0 feet with scattered small gravel fragments.

550.0 Pantano Formation
Sand, fine to medium with few coarse grains, angular to subround, most grains are brown, maroon, green, mostly volcanic rocks, cuttings have been ground up by drilling, slightly to moderately cemented becoming harder with depth. Little change in cuttings from 550.0 to bottom of well; drilling time increases slightly.

1004.0 Bottom of well

USF200002495

Fort Huachuca, Arizona
Geologic Log
Test Well No. 7

0.0 Terrace deposits
Clayey sand, reddish brown, sand is mostly fine to medium, some coarse, angular to subround, variable amount of lean clay.

3.5 Clayey Sand as above with scattered small gravel, 10 to 20 percent reddish brown clay.

20.0 Sand, mostly fine to medium grained, few coarse grains, angular to subround, trace to 5 percent reddish brown clay.

52.0 Upper Unit of Basin Fill
Sand, mostly medium to coarse, angular to subround, trace of brown clay.

93.0 Clayey sand, fine to coarse angular to subround sand with 20 to 30 percent brown, moderately plastic clay.

105.0 Sand is fine to medium grained, 10 to 20 percent brown clay.

130.0 Clayey sand as above, 20 to 30 percent brown clay.

150.0 Clayey sand as above, 30 to 40 percent clay.

158.0 Sand, fine to medium, angular to subround, less than 5 percent clay.

164.0 Sandy clay, brown, moderately plastic, 30 to 40 percent fine to medium grained sand.

181.0 Clayey sand, fine to medium grained. Few coarse grains, 30 to 40 percent brown, moderately plastic clay.

198.0 Sandy clay-clayey sand, cuttings are mixture of fine to medium sand and brown clay in varying proportions, probably layers of sand and sandy clay.

215.0 Lower Unit of Basin Fill
Sandy clay, brown, moderately plastic, 10 to 20 percent fine to medium sand.

USF200002496

Test Well No. 7 (Cont'd)

245.0 Thin layer of fine to medium sand, may be slightly clayey.

248.0 Sandy clay as at 220.0 feet.

263.0 Sand, fine to coarse, angular to subround, mostly medium to coarse, trace of brown clay.

279.0 Clayey sandy gravel, gravel is fine, subangular to rounded, sand is fine to coarse, 10 to 20 percent brown, moderately plastic clay.

284.0 Sand, fine to medium, mostly fine, angular to subrounded, trace of brown clay.

289.0 Sandy clay, brown, moderately plastic

294.0 Sand, fine to medium with few coarse grains angular to subround, variable amount of brown clay

315.0 Clayey sand, same as at 294.0 feet with 10 to 15 percent brown clay.

330.0 Sand, fine to medium, angular to subround, few coarse grains, trace to about 5 percent brown clay, may be thin layers of clay or clayey sand.

431.0 Thin streaks of clayey sand, sand is fine to medium coarse.

436.0 Sand as at 330.0 feet.

450.0 Clayey sand, fine to medium, angular to subangular few coarse grains, trace to 10 percent brown clay.

463.0 ± 15 percent light brown clay.

485.0 Clayey sand as at 450.0 feet.

510.0 Slight increase in number of dark brown and maroon sand grains, continues to increase with depth, decrease in quartz grains.

551.0 Sand, fine to medium, few coarse grains, angular to subrounded, clean, darker grains continue to increase (looks like reworked Pantano Formation).

USF200002497

Fort Huachuca, Arizona
Geologic Log
Test Well No. 8

0.0 Terrace deposits
Sandy clay, reddish brown, moderately plastic, small amount fine to coarse sand.

1.0 Clayey, gravelly sand, fine to coarse with small gravel, angular, 30 to 40 percent reddish brown lean clay.

3.5 Sandy clay, reddish brown lean clay as above, 15 to 20 percent, mostly fine, sand.

5.0 Clayey sand, mostly fine to medium grained, few coarse grains and scattered gravel, angular to subround, 10 to 20 percent reddish brown lean dry clay, few streaks of caliche.

13.0 Sand, fine to coarse grained, angular to subround; thin streaks of hard sandy clay 16 to 20 feet.

35.0 Upper Unit of Basin Fill
Sand, mostly fine to very fine, slightly cemented.

40.0 Clayey sand, fine to medium sand with 5 to 15 percent reddish brown clay.

44.0 Clayey sand, fine to medium coarse, well graded.

46.0 Sand, fine to coarse, angular to subround, trace of clay.

57.0 Gravelly sand, fine to coarse sand with about 40 percent 1/4 to 3/8-inch gravel, angular to subangular, few rounded, gravel content decreasing from 59.0 feet.

60.0 Sand, fine grained, contains scattered gravel, Coarser grained with more gravel at 70.0 feet.

USF200002498

Test Well No. 8 (Cont'd)

72.0 Gravelly sand, fine to coarse angular sand with 30 to 40 percent angular to subround gravel, trace of clay.

85.0 Sand, fine to coarse, mostly angular scattered small gravel; thin streaks of sandy clay 94.0 to 97.0 feet.

97.0 Clayey sand, sand as at 85.0 feet with 5 to 10 percent brick red to brown clay. Clay content decreases below 105 feet.

107.0 Sand, fine to medium grained, finer at 116.0 feet, trace of clay. Clay content 5 to 10 percent at 118 feet

120.0 Sand as at 107 feet with thin layers of clayey sand, clay content 5 to 15 percent.

140.0 Clayey sand-sandy clay, about equal amounts of fine to medium sand and lean brown clay; more sandy from 149 feet. Grades to clean sand at 157 feet

157.0 Sand, fine to medium grained, angular to subround.

160.0 Sandy clay, brown, moderately plastic, variable amount of fine sand.

166.0 Clay, reddish brown, lean, small amount of very fine sand.

174.0 Sand, fine to coarse with a few small gravels, cuttings are angular.

182.0 Sandy clay-clayey sand, varying proportions of fine to coarse sand and brown lean clay, few small gravels.

198.0 Sand, fine to coarse with few small gravels, angular, clean.

207.0 Gravelly clayey sand, fine to coarse sand with about 25 percent gravel up to 1/2 inch size, mostly angular to subangular, 5 to 15 percent clay.

213.0 Sandy clay, light brown, lean, fine to medium sand, scattered coarse sand and small gravel.

USF200002499

Test Well No. 8 (Cont'd)

230.0 Sand, fine to coarse, well graded, mostly angular.

237.0 Clayey sand grading to sandy clay, fine to coarse sand with increasing amount of light brown clay.

252.0 Sand, fine to medium grained, angular.

254.0 Clayey sand; sand as at 252 feet with 15 to 30 percent light brown clay.

258.0 Sand, fine grained, well sorted, clean.

268.0 Clayey sand, fine to medium grained angular sand with variable amounts of lean brown clay, clay content 10 to 40 percent.

300.0 Lower Unit of Basin Fill
Sandy clay, light brown, lean, 10 to 40 percent fine to medium sand.

305.0 Clayey sand, fine to medium grained sand with highly variable amounts of light brown lean clay, some thin layers of clean sand.

360.0 Sand, fine to medium grained, angular, trace to 10 percent brown clay.

405.0 Sand, mostly fine grained, slightly coarser below 422.0 feet, angular to subround.

445.0 Mostly medium grained sand, trace of clay.

497.0 Sand is fine grained, trace to 10 percent brown clay, abundant quartz and feldspar grains give overall medium gray color, slightly cemented.

570.0 Sand, fine to medium grained, angular to subround slight increase in darker colored grains, slightly to moderately cemented, cementing increases with depth.

600.00 Darker colored grains more abundant with increasing depth, probably reworked Pantano.

USF200002500

Test Well No. 8 (Cont'd)

632.0 Pantano Formation
Sand, fine to medium grained, cuttings are mostly fine and angular, probably being ground up, mostly dark colors with a small amount of quartz, overall brown color, moderately cemented, hardness (cementation) continues to increase with depth.

685.0 Cuttings are mostly medium grained, angular, ±10 percent fine.

693.0 Fine to medium coarse, cementation seems to vary as reflected by drilling rate.

780.0 Mostly fine grained but probably being ground up by bit.

880.0 Cuttings are fine to coarse angular sand (less grinding with new bit). Grain size seems to vary with bit condition and hardness (cementation)

1000.0 Overall color is grayish brown, cuttings are fine to medium.

1080.0 Cuttings are fine to coarse, angular to subangular

1095.0 Gravelly Sand, fine to coarse sand and small gravel, angular to subangular, some gravel particles are fragments of soft shale (Sandy siltstone).

1104.0 Sand, fine to coarse, angular to subround, scattered gravel size fragments of light reddish brown sandy siltstone.

1220.0 Fine to medium, mostly fine, no gravel sizes.

1295.0 Volcanic Conglomerate--(Bronco Volcanics) Drilled with 7-7/8 inch bit. Hole not cased from 1304. Cuttings are fine to coarse sand, mostly medium to coarse, angular to subround, grains are almost all dark colors-dark green, dark gray, brown, maroon, few small grains of calcite, very hard, well cemented.

1501.0 Bottom of Well.

USF200002501

Fort Huachuca, Arizona
Geologic Log
Observation Well - Site 8

0.0 Terrace deposits
Sandy clay, light reddish brown, lean clay, dry, contains a large percent of fine sand.

1.0 Clayey sand, reddish brown, very fine and medium grained, angular sand (skip graded), 40 to 60 percent reddish clay.

3.0 Clayey sand, reddish brown, fine to coarse grained, but mostly medium to coarse, angular, about 10 to 15 percent lean clay.

12.0 Sand, reddish brown, mostly fine to medium grained, angular, small amount of reddish clay.

15.0 Sand is fine to coarse, angular.

24.0 Upper Unit of Basin Fill
Sand, medium to coarse, slightly cemented.

30.0 Sand is medium grained, angular, cemented.

40.0 Sand is fine to medium grained, few rounded grains, small amount of clay.

43.0 Clayey sand, very fine to fine grained, 25 to 35 percent reddish brown clay.

46.0 Sand, very fine to fine grained, appears to be clean, partly cemented slightly.

75.0 Clayey sand, medium to coarse sand and scattered gravel up to 1/2-inch size, 10 to 15 percent clay.

84.0 Silty gravelly sand, medium to coarse sand with few small gravels and 15 to 20 percent silt or lean clay.

96.0 Sand alternating with sandy clay; mostly medium grained sand with varying percent lean clay or silt, mottled brick red and brown.

104.0 Sand, fine to medium grained, angular to subround, 5 to 10 percent reddish brown silt. Sand gets finer with depth.

USF200002502

Observation Well - Site 8 (cont'd)

115.0 Sand is very fine to fine grained, appears to be clean.

148.0 Sandy clay, brown, firm, small amount of fine to medium sand.

151.0 Sand, fine to coarse but mostly medium, few gravels up to 1/2-inch size.

160.0 Sandy gravel, subround to angular gravel up to 3/4-inch size, about 20 percent medium to coarse sand, clayey near bottom.

175.0 Sandy clay, brown, firm, small amount of fine to medium sand.

178.0 Sandy gravel, subround to angular gravel up to 3/4-inch size, about 20 percent medium to coarse sand.

187.0 Gravelly sandy clay, brown, lean clay with about 25 percent sand and gravel up to 3/4-inch size.

192.0 Clayey sandy gravel, angular to subround gravel up to 3/4-inch size with about 15 percent lean brown clay.

204.0 Sandy gravel, coarse sand and gravel up to 3/4-inch size, angular to subround.

210.0 Gravelly clay, brown, lean, sandy clay with 30 to 40 percent angular to subround gravel up to 1/2-inch size.

225.0 Clayey gravel - gravelly clay, clay and gravel as at 210.0 feet in variable proportions.

243.0 Clayey gravel as at 225.0 feet, 10 to 20 percent lean borrow clay, cobbles 243.0 to 247.0 feet.

253.0 Sandy clay, brown, firm, 10 to 15 percent fine to coarse sand, few small gravels. Gravel content increases at 255 feet, up to 40 percent.

255.0 Gravelly sandy clay, brown, firm, gravel content highly variable, sand mostly fine or coarse (gravel and coarse sand in a sandy clay matrix).

USF200002503

Observation Well - Site 8 (cont'd)

285.0 <u>Lower Unit of Basin Fill</u>
Sandy clay, brown to light gray, lean, low percent fine sand, scattered gravel and coarse sand grains.

318.0 Sandy clay as above but sand content highly variable — ± 5 to 30 percent

345.0 Sandy clay-clayey sand, about equal amounts of lean brown clay and fine to medium sand, few coarse grains.

360.0 Clayey sand, mostly medium grained, small amount is fine grained, 10 to 15 percent lean brown clay.

380.0 Sand, fine to medium grained, streaks of brown clay, overall 5 to 10 percent clay.

400.0 Clayey sand, fine and coarse sand (skip graded) with 15 to 25 percent brown clay. Clay content increase to almost 50 percent at ± 410 feet.

413.0 Sandy clay, grayish brown, 30 to 40 percent grayish brown clay.

446.0 Sand, fine to medium grained, few coarse grains, cemented.

468.0 Bottom of observation well

USF200002504

Fort Huachuca, Arizona
Geologic Log
Test Well No. 9

0.0 Terrace deposits
Gravelly clayey sand, reddish brown; sand is fine to coarse but mostly coarser sizes, angular to subround; gravel is mostly fine, angular to rounded, ± 25 to 40 percent reddish brown clay.

5.0 Clayey sand, reddish brown, sand is fine to medium grained, angular to subround, 30 t0 40 percent reddish brown clay.

16.5 Clayey sand as above but clay content is 5 to 10 percent.

24.5 Clay content estimated 30 percent.

26.5 Clay content estimated 5 to 10 percent

33.5 Sand, mostly medium grained, high percent of quartz grains, trace of clay.

34.5 Sand is clean, mostly quartz, angular to rounded.

39.0 Upper Unit of Basin Fill
Sand, fine to medium, angular to subround, overall brown color, trace of reddish brown clay.

56.5 Sand is moderately cemented, fine grained.

66.0 Sand contains 5 to 10 percent reddish brown clay.

74.0 Sand, fine to coarse, angular to rounded, gradation varies (well graded to poorly graded), clean.

85.0 Sand, mostly medium and coarse, contains a small amount of clay at 85.0 feet, increase to 10 to 15 percent at 90.0 feet.

90.0 Clayey Sand--sandy clay, probably in alternating layers, light brown and light gray sandy clay alternating with medium to coarse sand, gray clay is limey.

USF200002505

Test Well No. 9 (Cont'd)

107.0 Clay, light brown, soft, moderately plastic, small amount of fine to medium sand as thin streaks.

110.0 Color alternates from light greenish gray to light brown.

150.0 Color is mixed light gray (almost white) and light brown, no more than 5 percent fine sand.

170.0 Clayey sand, fine to medium, angular to subround, low percent brown clay at 170.0 feet, increase to ± 25 percent at 179.0 feet.

188.0 Sand, fine to medium grained, angular to subround less than 5 percent brown clay.

194.0 Clayey sand, sand as at 188.0 feet with about 25 percent brown clay at 194.0 feet; clay content varies from about 10 to 30 percent.

255.0 Sandy clay, brown, soft, moderately plastic contains about 30 to 35 percent fine to coarse sand.

265.0 Clay, brown, stiff, highly plastic, very small amount of fine sand.

308.0 Color change to dull maroon.

317.0 Thin streak is sandy, color change to brown.

340.0 Clay contains 5 to 10 percent fine sand and scattered small angular gravels.

370.0 <u>Lower Unit of Basin Fill</u>
Clayey sand, brown, sand fine to coarse, mostly medium to coarse, scattered angular pea gravel, ± 30 percent lean clay.

380.0 Thin streak of clean sand.

382.0 Clay content varies from 5 to 20 percent.

398.0 Thin streaks of clean sand from 398.0 to 400.0 feet.

400.0 Clayey sand, fine to coarse, angular to subround, clay content varies from about 10 to 30 percent.

USF200002506

Test Well No. 9 (Cont'd)

478.0 Sandy clay, brown, moderately plastic, 5 to 10 percent fine to medium grained sand.

500.0 Clay, brown, moderately plastic, contains less than 5 percent fine to medium sand.

598.0 Clayey sand, fine to coarse, mostly medium to coarse, angular to subround, about 5 percent brown clay, some evidence of cementation.

600.0 Pantano Formation
Sand, fine to coarse, angular to subround, mostly darker colors, some gray granitic rock and white quartz, slightly cemented, hardness and cementation increase with depth; contains a small amount of interstitial clay.

620.0 Sand, fine to coarse, angular to subround, less than 5 percent brown clay.

740.0 Gravelly sand, fine to coarse, angular to subangular sand and 1/4- to 3/8-inch gravel, few flat pieces of soft brown shale (claystone), 30 to 50 percent gravel, trace of brown clay.

780.0 Gravel content is no more than 15 percent

800.0 Sand, fine to coarse, mostly angular, scattered small gravels, trace of brown clay, slightly cemented.

840.0 Sand is mostly medium to coarse, scattered small gravels.

920.0 Sand is fine to medium, few coarse grains, angular to subangular, trace of brown clay, slightly to moderately cemented.

935.0 Sand is fine to coarse, angular to subangular, trace of brown clay.

964.0 Sand contains numerous fragments of brown shale.

1005.0 Bottom of hole.

USF200002507

INCLOSURE 6 is omitted because of bulk. It consists of geophysical logs for Test Wells 6, 7, 8 and 9. Logs for each well consist of:

1. Induction Electric Log
2. Compensated Acoustic Velocity Log
3. Compensated Density Log

These logs are available in the file copies of the report in the Los Angeles and Sacramento Districts.

USF200002508

UNITED STATES DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
WRD CENTRAL QW LAB, SALT LAKE CITY, UTAH

WATER QUALITY ANALYSIS

DATE: 740114

STATION I.D.313841110164701

(D-20-20 35CCB FT. HUACHUCA TW NO. 6)

STATE CODE LOCATION	COUNTY CODE	PROJECT CODE	AGENCY CODE	MAILING ADDRESS
04	003	02306		TUCSON, ARIZ.

BEGIN DATE	END DATE	TIME	TYPE	NO. OF DETR	SAMPLE I.D.	RECORD NO.	SOURCE
730917			2		320012	16607	GROUND WATER

FT. HUCHUCA TEST WELL NO. 6

	MG/L
BICARBONATE	207
CALCIUM DISS	40
CHLORIDE DISS	5.5
FLUORIDE DISS	0.2
LEAD DISS	
MANGANESE DISS	0.01
POTASSIUM DISS	1.6
SODIUM DISS	18
CADMIUM DISS	
SELENIUM DISS	0.004
PHOS ORTHO DISS AS P	0.09
$NO_2 + NO_3$ AS N DISS	0.79
HARDNESS TOTAL	150
HARDNESS NONCARB	0.0
RESIDUE DISS CALC SUM	220

	MG/L
BORON DISS	0.04
CARBONATE	0.0
CHROMIUM HEXAVALENT	0.0
COPPER DISS	
IRON DISS	0.0
MAGNESIUM DISS	12
SILICA DISS	31
SULFATE DISS	5.6
PHOSPHATE DISS ORTHO	0.28
MERCURY DISS	0.0
ALK,TOT (AS $CACO_3$)	170
PH	7.9
SODIUM PERCENT	21
RES DISS TON/AFT	0.3
SAR	0.6
CONDUCTIVITY	357

MAJOR IONS

CATIONS (MG/L)	
CALCIUM DISS	40
MAGNESIUM DISS	12
POTASIUM DISS	1.6
SODIUM DISS	18

ANIONS (MG/L)	
BICARBONATE	207
CARBONATE	0.0
CHLORIDE DISS	5.5
FLUORIDE DISS	0.2
SULFATE DISS	5.6
$NO_2 + NO_3$ AS N	0 79

APPENDIX 4 INCLOSURE 7 SHEET 1

UNITED STATES DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
WRD CENTRAL QW LAB, SALT LAKE CITY, UTAH

WATER QUALITY ANALYSIS DATE: 731009

STATION I.D.
3137161101716 01

(D-21-20 11BCD FT. HUACHUCA TW NO. 7)

STATE CODE	COUNTY	PROJECT	AGENCY	MAILING ADDRESS
LOCATION 04	CODE 003	CODE 02306	CODE	TUCSON, ARIZ.

BEGIN DATE	END DATE	TIME	TYPE	NO. OF DETR	SAMPLE I.D.	RECORD NO.	SOURCE
730828			2	35			GROUND WATER

FT. HUCHUCA TEST WELL NO. 7

	MG/L		MG/L
BICARBONATE	170	BORON DISS	0.040
CALCIUM DISS	36	CARBONATE	0
CHLORIDE DISS	3.5	CHROMIUM HEXAVALENT	0
FLUORIDE DISS	0.1	COPPER DISS	0.001
LEAD DISS	.0	IRON DISS	0
MANGANESE DISS	.010	MAGNESIUM DISS	6.6
POTASSIUM DISS	1.2	SILICA DISS	29
SODIUM DISS	16	SULFATE DISS	6.1
CADMIUM DISS	0	PHOSPHATE DISS ORTHO	0.09
SELENIUM DISS	.005	MERCURY DISS	0.0
PHOS ORTHO DISS AS P	0.03	ALK,TOT (AS $CACO_3$)	139
$NO_2 + NO_3$ AS N DISS	0.56	PH	7.8
HARDNESS TOTAL	120	SODIUM PERCENT	23
HARDNESS NONCARB	0.	RES DISS TON/AFT	0.25
RESIDUE DISS CALC SUM	185	SAR	0.6
		CONDUCTIVITY	281

MAJOR IONS

CATIONS (MG/L)		ANIONS (MG/L)	
CALCIUM DISS	36	BICARBONATE	170
MAGNESIUM DISS	6.6	CARBONATE	0
POTASIUM DISS	1.2	CHLORIDE DISS	3.5
SODIUM DISS	16	FLUORIDE DISS	0.1
		SULFATE DISS	6.1
		$NO_2 + NO_3$ AS N	0.56

APPENDIX 4 INCLOSURE 7 SHEET 2

UNITED STATES DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
WRD CENTRAL QW LAB, SALT LAKE CITY, UTAH

WATER QUALITY ANALYSIS

DATE: 740114

STATION I.D. 313613110161901

(D-21-20 13CBB FT. HUACHUCA TW NO. 8)

STATE CODE LOCATION	COUNTY CODE	PROJECT CODE	AGENCY CODE	MAILING ADDRESS
04	003	02306		TUCSON, ARIZ.

BEGIN DATE	END DATE	TIME	TYPE	NO. OF DETR	SAMPLE I.D.	RECORD NO.	SOURCE
730903		1430	2		320009	16598	GROUND WATER

FT. HUCHUCA TEST WELL NO. 8

	MG/L
BICARBONATE	199
CALCIUM DISS	35
CHLORIDE DISS	3.1
FLUORIDE DISS	0.2
LEAD DISS	0.005
MANGANESE DISS	0.08
POTASSIUM DISS	1.3
SODIUM DISS	26
CADMIUM DISS	0.0
SELENIUM DISS	0.003
PHOS ORTHO DISS AS P	0.1
NO_2 + NO_3 AS N DISS	0.03
HARDNESS TOTAL	120
HARDNESS NONCARB	0.0
RESIDUE DISS CALC SUM	213

	MG/L
BORON DISS	0.04
CARBONATE	0.0
CHROMIUM HEXAVALENT	0.0
COPPER DISS	0.001
IRON DISS	0.0
MAGNESIUM DISS	7.4
SILICA DISS	33
SULFATE DISS	8
PHOSPHATE DISS ORTHO	0.31
MERCURY DISS	0.0
ALK, TOT (AS $CACO_3$)	163
PH	8.0
SODIUM PERCENT	32
RES DISS TON/AFT	0.27
SAR	1.0
CONDUCTIVITY	321

MAJOR IONS

CATIONS (MG/L)	
CALCIUM DISS	35
MAGNESIUM DISS	7.4
POTASIUM DISS	1.3
SODIUM DISS	26

ANIONS (MG/L)	
BICARBONATE	199
CARBONATE	0.0
CHLORIDE DISS	3.1
FLUORIDE DISS	0.2
SULFATE DISS	8.0
NO_2 + NO_3 AS N	0.03

APPENDIX 4 INCLOSURE 7 SHEET 3

UNITED STATES DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
WRD CENTRAL QW LAB, SALT LAKE CITY, UTAH

WATER QUALITY ANALYSIS DATE: 740114

STATION I.D.313613110161901

(D-21-20 13CBB FT. HUACHUCA TW NO. 8)

STATE CODE	COUNTY	PROJECT	AGENCY	MAILING ADDRESS
LOCATION 04	CODE 003	CODE 02306	CODE	TUCSON, ARIZ.

BEGIN DATE	END DATE	TIME	TYPE	NO. OF DETR	SAMPLE I.D.	RECORD NO.	SOURCE
730903		0700	2		320008	16595	GROUND WATER

FT. HUCHUCA TEST WELL NO. 8

	MG/L		MG/L
BICARBONATE	200	BORON DISS	0.02
CALCIUM DISS	37	CARBONATE	0.0
CHLORIDE DISS	3.2	CHROMIUM HEXAVALENT	0.0
FLUORIDE DISS	0.2	COPPER DISS	0.001
LEAD DISS	0.002	IRON DISS	0.02
MANGANESE DISS	0.08	MAGNESIUM DISS	7.6
POTASSIUM DISS	1.2	SILICA DISS	33
SODIUM DISS	25	SULFATE DISS	7.7
CADMIUM DISS	0.001	PHOSPHATE DISS ORTHO	0.31
SELENIUM DISS	0.006	MERCURY DISS	0.0
PHOS ORTHO DISS AS P	0.1	ALK,TOT (AS $CACO_3$)	164
NO_2 + NO_3 AS N DISS	0.06	PH	7.8
HARDNESS TOTAL	120	SODIUM PERCENT	30
HARDNESS NONCARB	0.0	RES DISS TON/AFT	0.27
RESIDUE DISS CALC SUM	214	SAR	1.0
		CONDUCTIVITY	323

MAJOR IONS

CATIONS (MG/L)		ANIONS (MG/L)	
CALCIUM DISS	37	BICARBONATE	200
MAGNESIUM DISS	7.6	CARBONATE	0.0
POTASIUM DISS	1.2	CHLORIDE DISS	3.2
SODIUM DISS	25	FLUORIDE DISS	0.2
		SULFATE DISS	7.7
		NO_2 + NO_3 AS N	0.06

USF200002512

UNITED STATES DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
WRD CENTRAL QW LAB, SALT LAKE CITY, UTAH

WATER QUALITY ANALYSIS DATE: 740114

STATION I.D. 313613110161901

(D-21-20 13CBB FT. HUACHUCA TW NO. 8)

STATE CODE LOCATION	COUNTY CODE	PROJECT CODE	AGENCY CODE	MAILING ADDRESS
04	003	02306		TUCSON, ARIZ.

BEGIN DATE	END DATE	TIME	TYPE	NO. OF DETR	SAMPLE I.D.	RECORD NO.	SOURCE
730908		0800	2		320010	16601	GROUND WATER

FT. HUCHUCA TEST WELL NO. 8

	MG/L		MG/L
BICARBONATE	199	BORON DISS	0.03
CALCIUM DISS	36	CARBONATE	0.0
CHLORIDE DISS	3.5	CHROMIUM HEXAVALENT	0.0
FLUORIDE DISS	0.2	COPPER DISS	0.03
LEAD DISS	0.003	IRON DISS	0.0
MANGANESE DISS	0.04	MAGNESIUM DISS	7.6
POTASSIUM DISS	1.2	SILICA DISS	33
SODIUM DISS	26	SULFATE DISS	8.3
CADMIUM DISS	0.0	PHOSPHATE DISS ORTHO	0.31
SELENIUM DISS	0.003	MERCURY DISS	0.0
PHOS ORTHO DISS AS P	0.1	ALK, TOT (AS $CACO_3$)	163
NO_2 + NO_3 AS N DISS	0.06	PH	8.2
HARDNESS TOTAL	120	SODIUM PERCENT	32
HARDNESS NONCARB	0.0	RES DISS TON/AFT	0.27
RESIDUE DISS CALC SUM	215	SAR	1.0
		CONDUCTIVITY	330

MAJOR IONS

CATIONS (MG/L)		ANIONS (MG/L)	
CALCIUM DISS	36	BICARBONATE	199
MAGNESIUM DISS	7.6	CARBONATE	0.0
POTASIUM DISS	1.2	CHLORIDE DISS	3.5
SODIUM DISS	26	FLUORIDE DISS	0.2
		SULFATE DISS	8.3
		NO_2 + NO_3 AS N	0.06

APPENDIX 4 INCLOSURE 7 SHEET 5

UNITED STATES DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
WRD CENTRAL QW LAB, SALT LAKE CITY, UTAH

WATER QUALITY ANALYSIS DATE: 731009

STATION I.D.
313622110142001

(D-21-21 17 BCC FT. HUACHUCA TW NO. 9)

STATE CODE	COUNTY	PROJECT	AGENCY	MAILING ADDRESS
LOCATION 04	CODE 003	CODE 02306	CODE	TUCSON, ARIZ.

BEGIN DATE	END DATE	TIME	TYPE	NO. OF DETR	SAMPLE I.D.	RECORD NO.	SOURCE
730816			2	29			GROUND WATER

FT. HUCHUCA TEST WELL NO. 9

	MG/L		MG/L
BICARBONATE	190	BORON DISS	0.030
CALCIUM DISS	35	CARBONATE	0
CHLORIDE DISS	3.4	CHROMIUM HEXAVALENT	
FLUORIDE DISS	0.9	COPPER DISS	
LEAD DISS		IRON DISS	0
MANGANESE DISS	0.010	MAGNESIUM DISS	8.3
POTASSIUM DISS	1.0	SILICA DISS	32
SODIUM DISS	21	SULFATE DISS	5.7
CADMIUM DISS		PHOSPHATE DISS ORTHO	0.09
SELENIUM DISS		MERCURY DISS	
PHOS ORTHO DISS AS P	0.03	ALK,TOT (AS $CaCO_3$)	156
$NO_2 + NO_3$ AS N DISS	0.22	PH	7.9
HARDNESS TOTAL	120	SODIUM PERCENT	27
HARDNESS NONCARB	0	RES DISS TON/AFT	0.27
RESIDUE DISS CALC SUM	202	SAR	0.8
		CONDUCTIVITY	312

MAJOR IONS

CATIONS (MG/L)		ANIONS (MG/L)	
CALCIUM DISS	35	BICARBONATE	190
MAGNESIUM DISS	8.3	CARBONATE	0
POTASIUM DISS	1.0	CHLORIDE DISS	3.4
SODIUM DISS	21	FLUORIDE DISS	0.9
		SULFATE DISS	5.7
		$NO_2 + NO_3$ AS N	0.22

USF200002514