White Pine Power Project

GROUNDWATER INVESTIGATION
PHASE 3

TECHNICAL REPORT

EXHIBIT #9
FOR:
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☐ PROTESTANT
☑ APPLICANT
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Groundwater Investigation

Phase 3

Technical Report

for the

WHITE PINE POWER PROJECT

Prepared for

Los Angeles Department of Water and Power

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SUMMARY

The White Pine Power Project (WPPP) is a proposed 1500-megawatt coal-fueled, steam-electric generating facility to be located in White Pine County, Nevada. The project is anticipated to consist of two 750-megawatt units, with Unit 1 scheduled for commercial operation in mid-1989.

A water supply of about 25,000 acre-feet per year, (afy) is required by the WPPP for cooling purposes. This water supply is to be provided by developing existing groundwater resources. In order to obtain information on groundwater resources in White Pine County and to develop design criteria for the water supply system, the WPPP, through the Los Angeles Department of Water and Power (LADWP), contracted with Leeds, Hill and Jewett, Inc. (LEEDSHILL) to undertake a three-phased groundwater investigation and related studies.

In the Phase 1 investigation, LEEDSHILL obtained and analyzed available data and other groundwater related information. In the Phase 2 investigations, Phase 1 findings were verified and refined through geophysical investigations and field testing of existing wells. Specific water supply plans were developed for each of the eight power plant sites selected in the Site Recommendation Report dated May 1, 1981.

In this Phase 3 investigation aquifer characteristics in the vicinity of proposed well fields were determined by drilling, constructing and testing production wells in Spring and Steptoe Valleys. In addition, vertical electrical resistivity surveys were conducted in each valley to determine relative subsurface characteristics at potential well field locations.

Two-dimensional finite element groundwater simulation models were developed for each valley. These models were operated to simulate potential piezometric surface drawdowns for various basin management conditions.

Well Drilling and Testing

The well drilling program demonstrated the feasibility of obtaining 1960 gallons per minute (gpm) from a well in Steptoe Valley and 1300 gpm from a well in Spring Valley. Well tests indicate that leaky confining aquifer conditions occur in Steptoe Valley where pump test data indicate that transmissivity varies from 94,000 to 160,000 gallons per day per foot (gpd/ft), storativity varies between 0.00017 to 0.00025, and leakage varies from 2800 to 8600 feet.

Both confined and unconfined aquifers were encountered in Spring Valley where pump test data indicate that appropriate transmissivity and storativity values for the unconfined aquifer are 38,000 gpd/ft and 0.069 respectively. For the confined aquifer corresponding values are 14,700 gpd/ft and 0.0002 respectively.

Geophysical Survey

Geophysical surveys of potential well field locations in Spring Valley and Steptoe Valley indicated generally favorable aquifer conditions along the sides (alluvial fan areas) of the valleys with relatively unfavorable areas in the central portion (playa areas) of these elongated valleys. In addition, the surveys also revealed certain unfavorable areas, both at depth and in areal extent, along the alluvial fans.

Groundwater Modeling

The groundwater models were calibrated using the aquifer characteristics determined during the well tests, and constant-head boundary conditions to reproduce observed static water levels in the two valleys for average annual recharge and consumptive use conditions. These models were then operated under various WPPP well field pumpage scenarios and assumed future agricultural requirements. These scenarios included the following Cases:

Case I - Current agriculture plus 20,000 afy WPPP pumping

Case II - Current agriculture plus 25,000 afy WPPP pumping

Case III - Current agriculture plus 15,000 afy for future agricultural water use and 25,000 WPPP pumpage.

Case IIB - Same as Case II, but with transmissivity values in fan area decreased by 20% for a sensitivity analyses

Additionally, a sensitivity analyses using an impermeable boundary condition, with no recharge was conducted for the scenario of current agriculture plus 25,000 afy WPPP pumpage.

The magnitudes and extent of the calculated piezometric surface average drawdowns in both Steptoe Valley and Spring Valley at the end of 36 years of WPPP pumping for Case II are presented on Plate IX and Plate X of this report. The results of the impermeable boundary sensitivity analyses are presented on Plate XI and Plate XII.

It should be recognized that Plate IX and Plate X represent piezometric surface drawdowns, and Plate XI and XII represent water table drawdowns. Both of these drawdown estimates are valid only under the stated modeling and aquifer characteristic assumptions, and actual values may deviate from predicted values due to the proximity of recharge or discharge areas, uncertainties in the estimate of aquifer characteristics, and other localized conditions.

It is recommended that the WPPP wells be constructed to draw water from the lower, semi-confined or confined aquifers in

Steptoe Valley and Spring Valley respectively. Under this assumption, the calculated piezometric surface drawdowns represent piezometric surface changes (pressure changes) and are not necessarily declines in water table levels. However, as pumpage continues over the years, less water will be removed from the confined and semi-confined aquifers, and more water will be developed from one or more of the following sources: reduced phyreatophyte evapotranspiration, induced recharge, or leakage of water from the unconfined aquifer through the partially confining layer. As leakage occurs through the partially confining layer, water levels in the overlying unconfined aquifer may lower. This will in turn reduce consumptive use by plants which use water from the groundwater table. These lowered water levels will also allow water from streams currently running off to and evaporating from the playa areas (rejected recharge) to be captured and recharge the groundwater basin. These effects cannot be precisely quantified at this time. In this regard, it can be conservatively assumed that the piezometric surface changes as illustrated on Plate IX and Plate X, may be considered as maximum water table drawdowns.

Recognizing those uncertainties, the project has initiated a groundwater monitoring program to measure, record and report actual groundwater levels and changes related to project activities in Spring Valley and Steptoe Valley.

Well Field Design

The following is a tabulation of recommended well field and individual well design criteria.

1. Wells should be located at least one mile from known existing wells or private land.

2. In Steptoe Valley, wells should be located on the east side of the valley to minimize the possibility of interference with existing thermal springs, which are generally located on the west side of the valley.

3. In Spring Valley, well fields should not be located near the Shoshone Ponds area, to minimize the possibility of interference with piezometric levels in that area.

4. Well fields should be located in the higher transmissivity areas. These areas are generally located on the alluvial fans, between the mountain fronts and the finer grained playa deposits.

5. Well fields should generally have a single-row configuration which parallels the elongated north-south direction of the valley orientation.

6. A well field should consist of a minimum of two wells.

7. Well spacings should be approximately two miles in Steptoe Valley and one mile in Spring Valley.

8. Well fields should be located to receive recharge from as large a drainage area of high elevation mountains as possible.

9. Well fields should be located in areas found to have relatively favorable aquifer characteristics in the geophysical survey.

10. In addition to monitoring existing wells, springs and streams, monitoring wells should be installed at various locations to verify the areal extent and drawdown of the pumping cone, caused by initiation of project pumping. These wells should be installed at least one year before initiation of well field pumping to establish baseline data.

The following parameters should be considered for use in the design of individual wells.

1. Wells should be designed to discharge at an average maximum rate of 1250 gpm in Steptoe Valley and 750 gpm in

Spring Valley. Areas having favorable conditions may produce higher discharges, therefore the foregoing rates represent average conditions anticipated to be encountered.

2. Although individual well depths will vary due to site specific geologic conditions, and test wells were not constructed at all proposed well field locations, an evaluation of the geophysical surveys indicates generally favorable aquifer conditions at the proposed well field sites. Generally in Spring Valley, well depths are estimated to be less than 600 feet deep along the west side of the valley, and somewhat deeper along the east side. In Steptoe Valley, wells are estimated to generally be less than 700 feet deep. These cased well depths also generally consider the estimated long-term individual well drawdowns, and mutual well interference effects caused by WPPP pumpage.

3. Geophysical logs should be correlated with geologic logs developed by a geologist for at least one borehole in each well field area.

4. The suite of geophysical logs should include:

(1) Natural gamma (if the pressence of large quantities of volcanic rock are not detected in cutting samples)

(2) Self-potential

(3) 0.25 and 2.5 normal resistivity

(4) 0.25 and 2.5 lateral resistivity and

(5) Caliper

5. Specifications requiring drillers to collect samples at each ten foot interval and formation changes, for use in developing geologic well logs, correlating with geophysical logs and determining appropriate slot openings and gravel pack specifications, should be written into the job specifications.

6. Design of the appropriate screen slot size and gravel pack should be based upon conditions encountered at each of the specific borehole locations. Sieve analysis of selected samples obtained from potential aquifer zones should be the basis for design of the gravel pack and corresponding screen slot size. The screened interval should be based on available geologic and geophysical logs. It is anticipated that the minimum screen length will be on the order of 100 to 200 feet.

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1.0 INTRODUCTION

1.1 GENERAL

1.1.1 Background

The White Pine Power Project (WPPP) is a proposed 1500-megawatt coal-fueled, steam-electric generating facility to be located in White Pine County, Nevada. The will project consist of two 750-megawatt units, requiring a water supply of about 25,000 acre-feet per year (afy). Unit 1 is scheduled for commercial operation in mid-1989.

In order to obtain information on groundwater conditions in White Pine County and develop design criteria for the water supply system, the WPPP, through the Los Angeles Department of Water and Power (LADWP), entered into an agreement with Leeds, Hill and Jewett, Inc. (LEEDSHILL) to undertake certain groundwater related studies. These studies were divided into Phase 1, Phase 2 and Phase 3.

The Phase 1 investigation was a reconnaissance level report to describe surface and groundwater resources in White Pine County which might provide the required water supply and to establish a priority ranking of valleys for further investigation.

The report, completed in April 1981, concluded that the perennial yields were not being exceeded in valleys in White Pine County and that the required water supply could be provided by well fields in one or more valleys.

The Phase 2 investigation was conducted to verify Phase 1 findings through geophysical investigations and pump tests of existing wells, and to develop specific water supply plans for each of eight alternative power plant sites. The specific water supply plans reported in the Phase 2 report dated August 1981 consisted of well fields located in seven valleys in White Pine County.

After completion of the Phase 2 report, the Department concluded, based on siting studies, that alternative plant sites located in northern Butte Valley, southern Spring Valley and northern Steptoe Valley should be given further more detailed consideration. This Phase 3 investigation is a report of the water supply programs for these alternative sites.

1.1.2 Authority

The "Agreement for Groundwater Resource Consulting Services", for White Pine Power Project Development work provides the authority of this work. Under this Agreement, LADWP serves as agent of the Owners and is designated Development Manager for the Project.

This Agreement provides that LEEDSHILL and its subcontractors, Environmental Dynamics Inc. (EDI) and Harding-Lawson Associates (HLA), are to undertake specific groundwater related studies.

Attachment I of this Agreement is the Scope of Services which generally describes the services LEEDSHILL is to perform. Attachment I divides LEEDSHILL's work into three successive phases. The first phase of the work was completed in April 1981, the second phase was completed in August 1981 and this report is part of the phase three work.

1.1.3 Scope

The scope of the Phase 3 work includes the following tasks:

a. Drill, case, develop and pump test wells in Spring Valley and Steptoe Valley.

b. Conduct field electrical resistivity surveys in Spring Valley and Steptoe Valley.

c. Develop, calibrate, verify and operate groundwater models for Spring Valley and Steptoe Valley.

d. Assist with public hearings and meetings.

e. Prepare progress reports and Phase 3 reports.

1.2 DESCRIPTION OF STUDY AREA

The study area consists of two valleys in White Pine County -- Spring Valley and Steptoe Valley. Spring Valley is located in the eastern part of White Pine County and has a total area of about 1700 square miles. Steptoe Valley is located west of Spring Valley and has a drainage area of about 1975 square miles. Both valleys are elongated in the north-south direction essentially from the south boundary to the north boundary of the county with Steptoe Valley extending into Elko County to the north.

Well fields entirely located in Spring Valley would supply water to the powerplant sited in Spring Valley. Steptoe Valley well fields would supply water to a powerplant located in either northern Steptoe Valley, or Butte Valley. If the powerplant was located in Butte Valley, the water supply from the Steptoe Valley wells would be supplemented by well fields located in Butte Valley. However, field investigation of groundwater conditions in Butte Valley was not part of this Phase 3 work.

1.3 ORGANIZATION OF REPORT

This Phase 3 report is divided into two volumes. Volume I includes the text of the report plus Plates, Tables, and Figures. Volume II contains the appendices to the report.

Volume I chapters are as follows:

Chapter 1.0 - Introduction
Chapter 2.0 - Well Drilling and Construction Program
Chapter 3.0 - Aquifer Testing Program
Chapter 4.0 - Supplemental Monitoring Programs
Chapter 5.0 - Electrical Resistivity Investigations
Chapter 6.0 - Groundwater Modeling
Chapter 7.0 - Water Supply System

Appendices included in Volume II include:

Appendix A - List of Abbreviations
Appendix B - Geologic Logs
Appendix C - HLA Geophysical Investigation
Appendix D - Mathematical Basis for Groundwater Modeling Study
Appendix E - Supplemental Monitoring Program Data

2.0 WELL DRILLING AND CONSTRUCTION PROGRAM

2.1 GENERAL

The well drilling program consisted of five water level monitoring wells and two water production wells, constructed in accordance with specifications and drawings developed by LEEDS-HILL.[24] Two test drilling sites were investigated simultaneously during the July through September 1982 period. One drilling site (Site No. 1) is located in northern Steptoe Valley, approximately 45 miles north of Ely, Nevada. The other drilling site (Site No. 2) is located in Spring Valley, approximately 35 miles east from Ely. These general drilling site locations are shown on Plates 1 and 2 respectively.

2.1.1 Steptoe Valley

The Steptoe Valley drilling site is located along the eastern edge of northern Steptoe Valley approximately four miles north of Schellbourne Station, two miles south of Cherry Creek Road, and one mile west of Highway 93. At this site, a total of four wells were constructed, three water level monitoring wells, and one production well. Figure 2-1 shows the relative location of these wells.

As originally planned, the Steptoe Valley site would have only three wells constructed, two water level monitoring wells and one production well. However, in lieu of hauling water to the site, the drilling contractor opted to install a relatively shallow, temporary well to provide construction water. This water supply well (WSW) provided the Project with an additional shallow, water level monitoring well at no additional cost. The WSW was the first well drilled at the site, with construction beginning on July 21. The well was drilled using air/foam techniques to a depth of approximately 125 feet, and is located 130 feet southwest of production well 1A.

The WSW was constructed by installing 85 feet of eight-inch diameter steel casing and 40 feet of factory slotted casing. Construction details for this well are illustrated on Figure 2-2. The WSW was not gravel packed or developed prior to its use, however it was estimated that between 50 to 100 gpm could be produced using a six-inch submersible pump. Static water level in this well was approximately 90 feet below ground surface. At the conclusion of all pumping and water level recovery tests, the casing was removed and the borehole backfilled to within 50 feet of the surface with native material. The upper 50 feet of the borehole was then sealed with a cement grout mixture as required by the Nevada State Engineer's Office.

Pilot borehole 1A, which would become the production well, was drilled next using conventional mud rotary techniques. The 12-inch diameter borehole was drilled to a total depth of 995 feet to determine lithologic conditions with depth, and to accurately locate potential production aquifer zones. Representative formation cuttings were collected at ten foot intervals, or at every formation change, which ever occurred first. Upon completion of borehole drilling, the following eight geophysical borehole logs were run to confirm lithologic conditions with depth: (1) natural gamma; (2) self-potential; (3) .25 normal resistivity; (4) 2.5 normal resistivity; (5) .25 lateral resistivity; (6) 2.5 lateral resistivity; (7) caliper; and (8) temperature (see Appendix C for more discussion of geophysical borehole logging).

After completing the necessary geophysical logging in pilot borehole 1A, the drilling equipment was relocated to pilot borehole 1B. Pilot hole 1B was located approximately 924 feet west of borehole 1A (see Figure 2-1), and constructed to monitor water level changes during all drawdown and recovery tests. Borehole 1B was drilled using conventional mud rotary techniques to a depth of 460 feet with an 11-inch diameter bit. Representative cuttings were collected at ten foot intervals and formation changes. A suite of four geophysical logs consisting of natural gamma, self-potential, 2.5 lateral resistivity, and caliper were conducted at the completion of borehole drilling.

After interpretation and analysis of all lithologic and geophysical information, construction details for observation well 1B were established. The well was constructed to a depth of 455 feet using 50 feet of 12-inch diameter steel casing, 355 feet of blank and 100 feet of factory slotted four-inch diameter PVC casing. The factory slotted sections of casing were located at depths of 155 to 170, 225 to 235, and 375 to 450 feet, as measured from the ground surface. The well was gravel packed using locally obtained "pea gravel", and then developed to a "clear water" condition by air surging techniques for six hours. Figure 2-3 summarizes the construction details for this well.

Pilot borehole 1C was located 250 feet south of borehole 1A, and constructed to monitor water level fluctuations. This borehole contains 50 feet of 12-inch diameter steel conductor casing, and 460 feet of six-inch diameter PVC casing to monitor water level fluctuations. Well 1C contains 360 feet of blank casing and 100 feet of factory slotted casing located from 355 to 455 feet. This observation well was developed by air surging for a period of six hours, and contains a locally obtained "pea gravel" pack. Figure 2-4 summarizes the construction details for this well.

Upon completion of observation well 1C, the drilling crew mobilized back to the location of borehole 1A and began construction of production well 1A. Well 1A was constructed by installing 50 feet of 24-inch diameter steel conductor casing, and then enlarging the 12-inch diameter pilot bore to a diameter of 22-inches using reverse rotary drilling techniques. Design and construction details for this production well were established from interpretation and analysis of all lithologic and geophysical information.[20,40]

Grain size distribution curves of samples collected during drilling of borehole 1A are presented on Figures 2-5, 2-6, 2-7, and 2-8. These four samples were collected from 350 to 360 feet, 387 to 397 feet, 440 to 447 feet, and 460 to 470 feet, all representative of the screened production aquifer zone. Table 2-1 presents the results of these analysis. The largest suitable gravel pack material commercially available was a #4-8 Colorado Silica Sand. A screen slot size of 0.100 inches was selected to retain approximately 95 percent of this material.

The production well was constructed to a depth of 489 feet and screened with 100 slot, Johnson continuous wire-wound screen in the following intervals: 354 to 374 feet; 384 to 404 feet; and 419 to 479 feet. Gravel pack for this well consists of #4-8 Colorado Silca Sand located between a depth of 200 to 489 feet, and locally obtained "pea gravel" from the ground surface to 200 feet. Figure 2-9 illustrates the final construction details for the production well 1A. Production well 1A was then developed for a period of 12 hours by a combination of high velocity jetting and air surging.

Prior to initiating the pump testing, this well was further developed for a period of eight hours by pumping, until a "sand free, clear water" condition was obtained.

2.1.2 Spring Valley

The Spring Valley drilling site is located along the western edge of Spring Valley, approximately seven miles north of the intersection of Highways 50 and 293, and approximately one mile east of Highway 293. A total of three wells were constructed in Spring Valley, two water level monitoring wells, and one production well. The drilling contractor opted to haul water from a nearby surface source, and no water supply well was constructed. Figure 2-10 llustrates the relative location of these wells.

Pilot hole 2A, which would become the production well, was the first borehole drilled at the Spring Valley test site. This 12-inch diameter borehole was drilled using conventional mud rotary techniques, to a total depth of 1000 feet to determine lithologic conditions with depth, and to accurately locate potential production zones. Representative formation cuttings were collected at ten-foot intervals, or at every formation change, whichever occurred first. Upon completion of borehole drilling, a suite of eight geophysical borehole logs were run (see Appendix C for more discussion of borehole logging).

After completing the necessary geophysical logging, the drilling equipment was moved to the location of pilot borehole 2B. Pilot hole 2B is located approximately 913 feet east of borehole 2A, and constructed to monitor water level changes in the lower aquifer zone. Representative cutting samples and geophysical borehole logs taken at both 2A and 2B confirm the existance of an impermeable clay, gravelley clay layer located approximately 200 to 250 feet below the ground surface. This confining bed of impermeable material effectively separates two aquifer zones occurring above and below this confining material (see Section 2.2.4 for more discussion). Well 2B was constructed with a 60-foot subsurface grout seal to permit monitoring water levels only in the lower aquifer, below the confining layer. This well is cased to a total depth of 470 feet with 4-inch diameter PVC casing, and the following perforated intervals: 290 to 390 feet and 450 to 465 feet. This well has a local pea gravel pack from 249 to 470 feet, a sand layer from 245 to 249 feet, a subsurface grout seal from 185 to 245 feet and a local pea gravel pack from the ground surface to the top of the grout seal. Well 2B was developed by air surging techniques for a period of six hours. Figure 2-11 summarizes the construction details for this well.

Pilot borehole 2C located 303 feet north of borehole 2A was the next drilling location. This borehole contains 50 feet of 12-inch diameter steel conductor casing, and a total of 178 feet of six-inch diameter PVC, of which 120 feet is factory slotted casing located between depths of 58 to 178 feet. With the confining layer between depths of approximately 200 to 250 feet, well 2C was constructed to a depth of 178 feet, to monitor water levels only from the upper, unconfined aquifer. This well was packed with local pea gravel from the ground surface to a depth of 178 feet and was developed by air surging to a "clear water condition" for 11 hours. Figure 2-12 summarizes the construction details for this well.

Upon completion of observation well 2C, the drilling crew moved back to the location of borehole 2A and began construction of production well 2A. Well 2A was constructed by installing 50 feet of 24-inch diameter steel conductor casing, and then enlarging the 12-inch diameter pilot hole to a diameter of 22-inches using reverse rotary drilling methods. Design and construction details for this production well were established from interpretation and analysis of all lithologic and geophysical information.

Grain size distribution curves of samples collected during drilling of borehole 2A are presented in Figures 2-13, 2-14, 2-15, and 2-16. These four samples, collected from depths of 270 to 280 feet, 310 to 320 feet, 370 to 380 feet, and 560 to 570 feet, are representative of the screened production aquifer zones. Table 2-2 presents the results of these analysis. The aquifer material at this site is slightly finer than encountered at Steptoe, so that a slightly smaller #8-12 Colorado Silica Sand gravel pack was selected with a screen slot opening of 0.080 inches.

The well was constructed to a depth of 580 feet and is screened with #80 slot, Johnson continuous wire-wound screen in the following intervals: 170 to 200 feet, 245 to 275 feet, 290 to 325 feet, 370 to 400 feet, 455 to 470 feet, 485 to 510 feet, and 540 to 570 feet. The gravel pack for this well consists of #8-12 Colorado Silica Sand. Figure 2-17 illustrates the final construction details for this production well. Production well 2A was developed for 40 hours by a combination of high velocity jetting and air surging. Prior to beginning the pump tests, this well was further developed for a period of 12 hours by pumping, until a "sand free, clear water" condition was obtained.

2.2 GEOLOGIC CONDITIONS

2.2.1 Geologic Setting

The test drilling sites are located in the Basin and Range physiographic province, a region characterized by alluviated valleys separated by north-south trending mountain ranges. Structurally, the region is composed of a complex series of uplifted mountains, termed horsts, and downdropped valleys, termed grabens.

In the Paleozoic era, eastern Nevada was covered by a shallow sea in which were deposited sediments consisting primarily of limestone and dolomite, and to a lesser degree, sandstone, siltstone and shale overlying Precambrian quartzites. During middle Cenozoic time, Nevada was subjected to tectonism resulting in normal faulting, thrust faulting and folding of the rock. Tertiary volcanism produced lava flows, tuffs and intrusive bodies presently exposed in the Schell Creek Range and elsewhere. These volcanic bodies are especially prevalent in the mountains east of the Steptoe Valley test site.

Unconsolidated gravel, sand, silt and clay eroded from the mountains were deposited in the valleys by alluvial and lacustrine processes during Quaternary time. The valley fill has been interpreted by Drewes, as occurring in the following sequence.[6] Older alluvial fan units consisting of angular to subangular poorly sorted gravel and cobbles in a sandy, silty, limey matrix were deposited in early to middle Pleistocene time. These older alluvial deposits were then overlain by late Pleistocene clays, sandy clays and gravelly clays deposited in a lacustrine environment. These lacustrine deposits were then overlain by younger alluvial and fan gravel units of recent age.

2.2.2 Method of Investigation

To gain a better understanding of the geologic and hydrologic characteristics of the valley fill material, cuttings developed during drilling operations were closely monitored at both the Spring Valley and Steptoe Valley test sites. Drilling and testing of five observation wells and two production wells were performed during the period July to October, 1982 by the Thompson Drilling Company of Las Vegas, Nevada. Two rotary drill rigs, a Challenger Model 124 and a Challenger Model 280 using tricone drill bits, and fresh water drilling fluid with bentonite and polymers were employed. Reaming of the pilot borings for the two production wells was accomplished using the Model 280 drill rig, operating in the reverse rotary mode with fresh water.

Drilling was performed under the supervision of a LEEDS-HILL or LADWP geologist or engineer. Drill bit penetration rates were obtained with a stopwatch by timing drilling intervals of approximately five feet and recorded on field drilling logs. Samples of the drill cuttings were collected every two and one-half to five feet and stored as composite samples of approximately ten foot intervals. Cutting samples at Steptoe Valley were obtained by placing a sieve directly in the wash return. At Spring Valley, a "shale shaker" was used to separate the cuttings from the drilling fluid.

Upon completion of drilling but prior to installation of the casing, geophysical surveys of two borings at each of the sites were performed by Harding-Lawson & Associates of Novato, California. These surveys consisted of natural gamma, self potential, normal and lateral resistivity, temperature and caliper logs. A detailed explanation of the borehole geophysical program and results are presented in Appendix C.

2.2.3 Hydrogeology of the Steptoe Valley Exploration Site

The site at Steptoe Valley is located on a gently sloping alluvial fan along the eastern side of the valley floor, approximately 45 miles north of Ely as shown on Plate 1. An ephemeral stream, Duck Creek, is located about two miles to the west, and the Schell Creek Mountains are two and one-half miles to the east.

A total of four boreholes were drilled at the Steptoe Valley location. Initially, a 125 foot deep boring, WSW, was drilled and cased for use as a drilling water supply. This well passed mainly through silty sand and gravel layers, overlaying a water producing gravel zone at approximately 115 feet. A representative field drilling log is presented in Appendix B.

Borehole 1A was designed as the production well and was drilled to a total depth of 995 feet. Sediments in the first 110 feet of this boring consisted of moderately to very silty sands and gravels. From 110 feet to approximately 265 feet a zone of sand and gravel layers interbedded with silts and minor amounts of clay was encountered. Within this zone, fairly clean sand and gravel strata were logged in the following intervals: 112 to 122 feet, 160 to 195 feet, 202 to 240 feet and 260 to 265 feet. Material between 265 and 360 feet was primarily very silty sand. A fairly clean gravel zone was encountered at 360 feet, which continued down to a depth of approximately 675 feet. Penetration rates and observations of particle angularity indicate that this zone includes cobbles and boulders. Below a depth of 675 feet, the material became finer grained and was logged as silty sands and gravels. A representative field drilling log is presented in Appendix B. Geophysical logs of borehole 1A confirm the location of the sand and gravel layers at depths of 110 to 265 feet, and 360 to 675 feet.

Borehole 1B, located 924 feet west of boring 1A, designed as an observation well, was drilled to a total depth of 460 feet. Silty to very silty sands and gravels were encountered from the ground surface to approximately 400 feet, with intervals of clean sand or gravel layers occurring at 160 to 170 feet and 230 to 245 feet. Below 400 feet are clean coarse sand and gravel deposits which correlate to those encountered in borehole 1A below 360 feet. Sediments in boring 1B are generally siltier than those encountered in 1A. This can be attributed to the closer proximity of 1B to the center of the valley where the finer sediments would tend to be deposited. Geophysical logs confirmed the presence of the sand and gravel zones logged during drilling and suggest the existence of an additional gravel layer from 105 to 115 feet (see Appendix B for boring log). Such a layer would be consistent with the aquifer encountered in the WSW at a depth of 115 feet.

Borehole 1C was drilled as a 455 foot deep observation well, located two hundred and fifty feet south of borehole 1A. Silty sands and gravels were encountered in the first 320 feet which were similar to those found at comparable depths in boring 1A. Clean sand and gravel layers were identified from 115 to 120 feet, 160 to 178 feet, 208 to 215 feet and 225 to 245 feet. The material below 320 feet was primarily composed of clean sands and gravels (see Appendix B for boring log).

A generalized geologic cross-section of the valley fill sediments at the Steptoe Valley drilling site is presented on Figure 2-18. Lithologically, the well cuttings sampled in Steptoe Valley are primarily of volcanic texture and composition. The origin of this material appears to be the Tertiary volcanic outcrops of the Schell Creek Range exposed east of the drilling site. Sediments in the upper 350 feet range from layers of silty sand and gravel in borehole 1B to layers of clean sand and gravel in borehole 1A. One possible interpretation is the siltier material in borehole 1B represents remnants of an old Pleistocene lakebed situated to the northwest of the test site, while the coarser sediments at the site of borehole 1A may have been deposited on an alluvial fan or in a meandering braided stream. Resistivity surveys conducted along an east-west profile across Cherry Creek Road indicate an increase in the silt and clay content across the valley. Additionally, other resistivity measurements conducted near the center of northern Steptoe Valley produced similar results.

It is believed that below a depth of 350 feet, the materials are composed of older alluvial sediments of early to middle Pleistocene age. These sediments were deposited during different climatic and topographic conditions, producing a relatively coarse material containing few fines. From a groundwater production standpoint, the sand and gravel aquifer located in the 350 to 675 foot interval has excellent potential as a water supply source. Overlain by very silty material, this aquifer appears to exist in a leaky artesian state. The clean sand and gravel layers interfingered in the 100 to 265 foot zone are also viewed as potential groundwater sources for smaller water supply requirements.

2.2.4 Hydrogeology of the Spring Valley Exploration Site

The Spring Valley drilling site is situated on a gently sloping beach terrace deposit along the western edge of the valley floor. The Schell Creek Mountains are located three and one-half miles to the west, and a local, northerly flowing ephemeral stream is located three miles to the east.

The three borings drilled at the Spring Valley site contained considerably more clay than those drilled in Steptoe Valley. Borehole 2A was drilled as a production well to a total depth of 1000 feet. The material encountered in the first 200 feet consisted of relatively clean sands and gravels. Underlying this material was approximately 50 feet of relatively impermeable lacustrine clays, clayey sands and clayey gravels. Relatively clean sands and gravels were again encountered from 260 to 335 feet. The remainder of the materials logged below 335 feet were clayey and silty sands and gravels, interfingered with layers of clean sand and gravel. These zones of relatively clean sand and gravel were identified from 450 to 470 feet, 540 to 560 feet, 640 to 670 feet, 700 to 720 feet and 880 to 900 feet.

Correlation between the geophysical logs and the lithologic log conducted in borehole 2A is generally good. In the first 250 feet the contacts between the sand and gravel units and the clay layer are clearly depicted on the self-potential and resistivity logs as well as the geologic log. However, with increased depth the correlation was not as good. It is believed that this was caused by an increased up-hole travel time associated with transporting the cutting samples through the viscous drilling fluids necessary to hold the borehole open. If the boring log is adjusted to reflect this increased up-hole travel time, the correlation between the electric log and the boring log is fairly good. It should also be noted that the drill rig at Spring Valley achieved faster penetration rates but had a slower sampling method than that at Steptoe Valley. A representative boring log is presented in Appendix B.

Borehole 2B was drilled as an observation well to a total depth of 490 feet at a location 913 feet east of borehole 2A. This borehole encountered relatively clean sands and gravels for the first 210 feet and then passed through lacustrine clays, clayey sands and clayey gravels from 210 to 280 feet. Relatively clean sand and gravel layers were encountered between 280 and 350 feet, below which the coarser sediments become intermixed with finer grained material. A very clayey material was encountered at 470 feet. Identification of the sand and gravel zones versus the clayey layers from the electric logs are in agreement with those on the boring logs (see Appendix B for boring log).

The third borehole, 2C, was drilled as an observation well to a total depth of 210 feet. It was completed in the sand and gravel layers located above the confining clay layer. The borehole passed through 195 feet of sand and gravel and then graded into 15 feet of very clayey gravel (see Appendix B for boring log).

A geologic cross-section of the valley fill sediments at the Spring Valley site is presented on Figure 2-19. The upper strata of this section are composed of 200 feet of younger alluvial sands and gravels underlain by approximately 50 feet of Pleistocene lakebed deposits. Material below this consists of older alluvial sands and gravels interfingered with old lacustrine deposits. The source for the sediments comprising the valley fill material at the Spring Valley site is believed to be primarily from the mountains located to the west. The cutting samples were composed principally of quartzite, limestone and granitic rock fragments. The most probable source for this type of alluvium is the Precambrian and Cambrian quartzites, the Paleozoic sedimentary units and the Triassic volcanics of the Schell Creek Range, although similar type rocks are found in the Snake Range which lies to the east.

Comparison of water levels recorded in wells completed above and below the clay layer which occurs at a depth of 200 feet indicates that this is a confining layer, creating a multi-aquifer system. Well 2B, completed and sealed in the lower sands and gravels, has a piezometric groundwater elevation of 5792.7 feet. This is approximately two feet higher than the water level in 2C, which was completed in the unconfined aquifer located above the clay layer. The water surface elevation in 2C is 5790.6 feet.

3.0 AQUIFER TESTING PROGRAM

3.1 DESCRIPTION OF AQUIFER TESTS

Upon completion of well construction and development, several different aquifer pumping tests were conducted at the Steptoe and Spring Valley sites. The tests were conducted by pumping water from the production well, while monitoring water levels in nearby observation wells. These aquifer tests were utilized to determine important aquifer characteristics such as transmissivity and storativity coefficients. A description of the testing procedures is included in the following sections.

3.1.1 Steptoe Valley

During the week of August 23, 1982, equipment such as the pump, pump column, bowls, prime water mover, discharge line, orifice plate and energy dissipating structure were installed at the Steptoe site. The prime water mover consisted of a 280 hp Detroit Diesel engine (model 6V-92) connected by a drive line to a right angle gear head and vertical turbine pump assembly. The bottom of the six stage, 12-inch diameter bowl assembly was set at a depth of 335 feet below ground surface. Water discharge measurements were obtained by reading head differentials as measured by a calibrated, submerged orifice plate constructed by Badger Meter, Inc.

Pumped water was conveyed through a 10-inch diameter aluminum irrigation line equipped with sprinklerheads, to a low swale approximately 2000 feet north of the production well, where most of the water was discharged through an energy dissipating structure. At various times during the pumping tests, sprinklers located on the aluminum irrigation line were opened and allowed to discharge.

As previously discussed (in Section 2.1.1), production well 1A was developed by high velocity jetting and actual pumping, prior to conducting the aquifer pumping tests. During this eight hour pumping development time, and subsequently during the pumping tests, sand content analyses were performed.

Sand content was measured using a centrifugal sand sampler, also known as a Rossum Sand Tester. Water enters the body of the tester tangentially. The small radius of the tester, and high water entrance velocity create a large centrifugal acceleration, which throws the sand to the side of the testing device. The sand falls down the side of the device and is collected in the centrifuge tube, while sand-free water flows out through a hole in the top of the tester. The flow is maintained at a constant value by means of a control valve. At suitable time intervals, the volume of sand collected is recorded. From these data, the average sand content can be computed, since flow through the tester is known. As illustrated on Figure 3-1, the sand content versus time plots indicate that most of the sand production occurs when the well is first started. And after a period of 20 minutes of pumping, the sand content is negligible. During the actual aquifer pumping tests, sand content tests were conducted which confirmed the negligible sand content in the water as also illustrated on Figure 3-1.

3.1.1.1 Variable Rate Pump Test No. 1

After developing production well 1A, water levels in 1A and observation wells WSW, 1B and 1C were allowed to return to pre-development static levels, prior to initiating the first variable rate pump test. This test was conducted from 0800 (8:00 a.m.) to approximately 1800 (6:00 p.m.) on August 27. During this ten hour period, the discharge rates were increased from 545 gpm to 885 gpm to 1170 gpm and finally 1520 gpm, each rate change occurring after approximately 150 minutes of constant pumping. Figure 3-2 illustrates the relationship between drawdown and time in production well 1A. From an analysis of this data, the long term constant rate pump test discharge was established at approximately 1430 gpm (also see Section 3.2.1.1).

3.1.1.2 Constant Rate Pump Test No. 1

Water levels in all four wells constructed at the Steptoe site were allowed to recover for a period of approximately 39 hours, before proceeding with the constant rate pumping test. This constant rate pump test began at 0900 (9:00 a.m.) on August 29, and continued until its unscheduled termination at approximately 1320 (1:20 p.m.) on September 3. During this five day period the pump was down for only two very brief periods of time. These interruptions in continuous pumping averaged only seven minutes each, and were necessary to check oil levels in the diesel engine. Water levels were monitored during these pumping interruptions, however over the long term, these interruptions did not appear to cause significant variations in water levels. Over the duration of this constant rate test, pumping rates did vary slightly (±5 gpm), but approximated 1430 gpm.

Water levels were monitored at each of the three observation wells (WSW, 1B, and 1C), and the production well (1A) with the use of calibrated, electric depth to water sounders. Measurements were taken at one minute intervals from startup to twenty minutes into the test; every five minutes until 100 minutes, every 30 minutes until 400 minutes, and then every one hour for the first 24-hour period. After that, water levels were monitored every four hours. Figures 3-3, 3-4, and 3-6 graphically illustrate the drawdown vs. time plots for wells 1A, 1B, 1C, and WSW respectively.

As originally planned, this constant rate pump test was to continue until water levels in the monitoring wells had stabilized (estimated to take 20 to 30 days). On September 3 at 1320 (1:20 p.m.), during a routine shut down for an oil level check, without any advanced warning, the gravel pack dropped approximately 17 feet and the 12-inch diameter steel casing dropped approximately four feet in production well 1A. This sudden movement occurred over a period of several minutes, and effectively terminated the constant rate test before its scheduled conclusion. It is hypothesized that the cuttings which filled the original pilot bore from 485 to 995 feet either bridged and then broke loose, or consolidated. In either case, this created a "void" which allowed the gravel packing and well casing to move.

With the termination of the constant rate pump test, the following remedial action was taken. First, the drive line and pump head were removed from the well. An excavation was dug around the 24-inch diameter steel conductor casing (which did not move), and the cement grout surface seal was chipped away exposing the 24-inch steel casing. This casing was then cut off approximately six feet below the ground surface and removed to permit access to the 12-inch well casing. A four-foot length of 12-inch casing was then added to bring the 12-inch casing level with the ground surface. The conductor casing was then replaced and the excavation backfilled. A one-half-inch thick steel plate was then welded to the top of both the 12- and 24-inch casings to secure them. The drive line and pump head were then reset, and the well run to insure its proper operation.

3.1.1.3 Variable Rate Pump Test No. 2

Water levels were allowed to recover for a period of nearly four full days. Within this time, all well levels returned to their pre-pumping static levels. The orifice plate which was originally installed in the discharge line had a maximum discharge rating of approximately 1500 gpm. During the remedial work to the production well, a new orifice plate with a maximum rating of approximately 2000 gpm was ordered and installed to allow measurement of higher discharges needed to further stress the aquifer.

On September 7, variable rate pump test No. 2 was conducted at production well 1A. During this nine hour testing period, discharge rates were increased from 1170 gpm to 1700 gpm to 1960 gpm and finally to nearly ±2100 gpm, with each rate being pumped for approximately 150 minutes. Figure 3-7 shows a plot of drawdown vs. time as observed in production well 1A. From an analysis of this data, the long-term constant rate pump test discharge was established at approximately 1960 gpm (see Section 3.2.1.1 for analysis details).

3.1.1.4 Constant Rate Pump Test No. 2

After the completion of variable rate test No. 2, water levels were allowed to recover for a period of approximately 36 hours. Constant rate pump test No. 2 began at 1020 (10:20 a.m.) on September 9, and continued for 20 days until its termination on September 29. During this 20 day period, the prime water mover was down once every 48 hours for a period of five to 10 minutes for a necessary oil level check in the diesel engine. Monitoring of water levels during these brief interruptions indicated no significant long-term effects. Pumping rates did vary slightly (±10 gpm), however a discharge rate of approximately 1960 gpm was typically held. Water levels were monitored at each of the three observation wells (WSW, 1B, and 1C) and the production well (1A) with calibrated, electric water level sounders. Measurements were taken at the same frequency as previously discussed for constant rate test No. 1. Figures 3-8, 3-9, 3-10, and 3-11 graphically illustrated the drawdown vs. time plots for wells 1A, 1B, 1C, and WSW respectively.

3.1.1.5 Recovery Test No. 2

Collection of water level recovery data from wells 1A, 1B, 1C, and WSW began immediately upon termination of the constant rate pump test No. 2 on September 29. Water level recovery measurements were collected at the same frequency as drawdown measurements for a period of approximately 72 hours, after which time all wells had returned to within 0.8 feet of pre-pumping static levels. Figures 3-12, 3-13, 3-14, and 3-15 show plots of recovery vs. time for wells 1A, 1B, 1C, and WSW respectively.

3.1.1.6 Constant Rate Pump Test No. 3

During constant rate pump test No. 2, the question of potential water recirculation was raised due to the close proximity of discharging sprinkler heads, located on the 10-inch diameter discharge line. It was queried whether the sprinklers could be artifically inducing recharge to the aquifer being tested. To respond to this concern, constant rate pump test No. 3 was conducted with all sprinkler heads shut off. The constant discharge selected was 1420 gpm, which closely approximates the constant discharge rate of 1430 gpm used during pump test No. 1 (in which the sprinkler heads were discharging).

Pump test No. 3 began on October 2 at 0900 (9:00 a.m.) and continued for approximately 50 hours with no pumping interruptions. Figures 3-16, 3-17, 3-18 and 3-19 present drawdown vs. time plots for wells 1A, 1B, 1C and WSW respectively. At the conclusion to this pumping test, water levels at all four wells were monitored for several hours to establish baseline recovery data and possible seasonal fluctuations.

3.1.2 Spring Valley

Necessary equipment to conduct the various pumping tests was installed during the week of September 5. The prime water mover consisted of a trailer mounted 238 hp Detroit Diesel engine (model 6V-71), connected by a drive line to a right angle gear head and vertical turbine pump assembly. The eight-inch diameter pump column was connected to a four stage 12-inch diameter bowl assembly set at a depth of 285 feet. Water discharge was measured by reading the upstream/downstream head differential of a calibrated, submerged orifice plate. This orifice was capable of reading discharges to approximately 1500 gpm.

Pumped water was conveyed through 10-inch diameter aluminum irrigation line which terminated in a swale located approximately 2000 feet north of the production well. In order to avoid excessive erosion at the discharge point, an energy dissipating structure was constructed.

As previously discussed, production well 2A was developed by high velocity jetting and pumping prior to conducting the aquifer pumping tests. During this 13 hour pumping development time, a Rossum sand content analysis was performed. Figure 3-20 shows the sand content versus time plot for production well 2A. It can be seen that well 2A produced more sand than well 1A, indicative of the finer grained materials present at the Spring Valley site. This caused a significantly longer pumping development time, than that required in Steptoe valley. However once properly developed, sand content tests conducted during the constant rate pumping tests (see Figure 3-20), showed negligible sand content.

3.1.2.1 Variable Rate Pump Test

Prior to conducting the variable rate pump test, water levels in 2A, 2B, and 2C were allowed to recover for a period of almost five days after development of 2A. The variable rate test began at 0900 (9:00 a.m.) on September 13, and concluded at approximately 1900 (7:00 p.m.) that same day. During this ten hour period, discharge rates were increased from 470 gpm to 840 gpm to 1250 gpm and finally to 1490 gpm, each rate change occurring after approximately 150 minutes of continuous pumping. Figure 3-21 shows the graphic relationship between drawdown and time for this variable rate pumping test. From an analysis of this data, the long term constant rate pump test discharge of approximately 1300 gpm was selected (see Section 3.2.2.1 for details of this analysis).

3.1.2.2 Constant Rate Pump Test

Water levels in all three wells constructed at the Spring Valley site were allowed to recover for approximately 38 hours before proceeding with the constant rate pumping test. This pump test began at 0900 (9:00 a.m.) on September 15, and continued for a period of 23 days until its scheduled termination on October 8 at 0900 (9:00 a.m.). During this 23 day period, the prime water mover was down once every 48 hours for a period of five to ten minutes for oil level checks. Monitoring of water levels during these brief interruptions indicated no significant long-term effects. Pumping rates did vary slightly (±10 gpm), however the pumping rate approximated 1300 gpm. Water levels were monitored at each of the observation wells (2B and 2C) and at the production well (2A) with calibrated, electric water level indicators. These draw-down vs. time plots are presented on Figures 3-22, 3-23, and 3-24 for wells 2A, 2B, and 2C respectively.

3.1.2.3 Recovery Test

Collection of water level recovery data from wells 2A, 2B and 2C began immediately following termination of the constant rate test on October 8. After a period of approximately 127 hours all wells had returned to within 1.2 feet of their pre-pumping static levels. The data are presented on Figures 3-25, 3-26, and 3-27.

3.1.2.4 Flowmeter Test

As discussed in Section 2.1.2, production well 2A was constructed to produce water from aquifer zones located both above and below the confining clay layer. The relative contribution from each zone was measured during a flowmeter (or spinner) test, which was conducted by Westech Geophysical on October 15.

The flowmeter is an instrument which measures fluid velocities (flow) in the well casing as the well is pumped. By lowering this instrument into the well casing, and measuring the fluid velocities at various depths, a fluid velocity profile of the well can be obtained. By inspection of this velocity profile, the relative magnitude and location of contributing zones can be determined. Additional information can be obtained during the uphole run by retrieving the probe at a known constant rate. The uphole movement of the instrument causes the probe to record a baseline fluid velocity. As contributing aquifer zones are passed, the water velocity from these zones causes a change in the baseline fluid velocity being recorded by the probe. Inspection of this velocity profile can yield the relative magnitudes and locations of contributing zones.

The flowmeter test at Spring Valley was conducted at three flow rates; 450 gpm, 725 gpm, and 935 gpm, with the pump bowls in production well 2A set at a depth of approximately 150 feet. Velocity profiles were conducted from a depth of approximately 160 to 525 feet. The data are presented on Plates III and IV.

3.2 ANALYSIS OF AQUIFER TESTS

3.2.1 Steptoe Valley

Of the various aquifer pumping tests conducted at the Steptoe Valley test site (discussed in Section 3.1.1), the most useful data were produced by variable and constant rate pump test No. 2. Variable rate test No. 2 was used to determine well efficiency, specific capacity curves and establishing the discharge rate to be used during constant rate pump test No. 2. Constant rate test No. 2 was conducted to adequately stress the aquifer for a significant period of time to produce reliable drawdowns, and subsequent recovery data. These data were analyzed to determine the various aquifer coefficients.

3.2.1.1 Variable Rate Pump Test

The variable rate pump test (step drawdown test) is conducted for two purposes: (1) to aid in the design of proper pumping equipment, and (2) to evaluate the performance and efficiency of the well relative to formation and well losses.

The observed drawdown s_w in a pumping well is composed of the formation loss (BQ), and the well head loss (CQ^2). The equations are as follows:

$$S_w = BQ + CQ^2 \qquad \text{and} \qquad BQ = \frac{Q\,W(u)}{4\pi T}$$

where:

B = constant related to formation loss

C = constant related to well loss

Q = discharge from pumping well

W(u) = well function

T = transmissivity

The term BQ relates to the Theis nonequilibrium equation and is directly related to the formation loss as water flows to the well. The factor CQ^2 represents a head loss component related to the flow into the well, similar to any other hydraulic loss in pipeline flow.

To evaluate constants B and C, a step-drawdown test was conducted as previously discussed. Figures 3-2 and 3-7 present the variable rate pump test drawdown versus time plots for tests 1 and 2 respectively. Data collected during these variable rate tests are tabulated on Tables 3-1 and 3-2, along with their respective specific capacity values. From an analysis of the specific capacity data, the constant rate pumping rates of approximately 1430 gpm and 1960 gpm were selected.

The well efficiency calculations are presented on Figures 3-28 and 3-29 for variable rate pump tests 1 and 2 respectively. Well efficiency plots are presented on Figures 3-30 and 3-31. It can be seen that production well 1A was more efficient during variable rate test 2 than during test 1. This is probably due to the increased development which occurred during constant rate pump test No. 1. Production well 1A currently has an efficiency of approximately 87 percent at a discharge of 1960 gpm.

3.2.1.2 Drawdown and Recovery Data

As previously discussed, production well 1A was pumped at a discharge rate of approximately 1960 gpm for a period of essentially 20 continuous days. During this period, water levels were monitored in the pumping well, both deeper observation wells, and the shallow water supply well (WSW), as well as additional monitoring locations (see Chapter 4, SUPPLEMENTAL MONITORING PROGRAMS). Inspection of the actual time versus drawdown data previously shown on Figures 3-8, 3-9, 3-10, 3-11 indicates the following:

(1) The drawdown cone resulting from the pumping of production well 1A appears to be relatively steep at the well itself, but flattens out rapidly as one moves away from the pumping well. This is indicative of an aquifer which has a relatively high transmissivity coefficient.

(2) From a knowledge of the site geologic conditions, and the fact that essentially all of the pumpage is occurring from the lower aquifer, the time versus drawdown response of the shallow WSW indicates that some vertical leakage does take place through the semi-confining layer.

Based upon these assumptions, drawdown data obtained from the production well and both deeper observation wells, were initially analyzed using a method detailed by M.S. Hantush in his paper "Analysis of Data From Pumping Tests in Leaky Aquifers". [13] However, due to difficulties inherent in accurately measuring a pumping well (i.e. oil layer, water splash, etc.), data obtained from the monitoring wells are more reliable indications of aquifer response and were used in the following analysis. Drawdown in a leaky aquifer with full penetration can be described by the following equation:

$$s = \frac{\Omega}{4\pi T} W(u, \frac{r}{B})$$

$$u = \frac{r^2 S}{4Tt} \quad \text{and} \quad B = \sqrt{T/(K'/b')}$$

where:

- s = drawdown
- Q = discharge in the pumping well
- S = storage coefficient
- T = transmissivity coefficient
- B = leakage factor
- $W(u,\frac{r}{B})$ = well function for leaky systems
- r = distance between the pumping well and the observation point
- t = time after pumping started

K' and b' are the hydraulic conductivity and thickness of the semi-confining bed through which the leakage occurs.

Data from observation wells 1B and 1C are plotted and presented on Figures 3-32 and 3-33 respectively, with curve characteristics such as the slope (m_i), maximum drawdown (s_m) and inflection point (s_i) determined. Utilizing this information and the above equations, transmissivity, storativity and leakage factors can be calculated. These calculated values are presented in Table 3-3.

It should be noted that these values were calculated assuming full aquifer penetration. This assumption is valid at radial distances larger than 0.5 to 2 times the saturated thickness of the aquifer, where partial penetration effects are negligible. In this case the aquifer is approximately 330 feet thick, therefore at radial distances greater than 660 feet from the production well, partial penetration effects can be neglected, and the assumption of full aquifer penetration is valid.

Observation well 1B is located approximately 924 feet west of production well 1A. This distance is sufficient for the effects of partial penetration to be neglected. Assuming that this is correct, a hypothetical time versus drawdown curve was generated. This generated curve is shown on Figure 3-34, along with the actually measured field data. It can clearly be seen that after approximately 15 minutes of pumping, the actual field drawdown data can be reproduced and matched using the generated data.

Observation well 1C is located approximately 250 feet south of production well 1A. At this radial distance, partial penetration effects cannot be neglected so an analysis which considers these effects must be utilized. Observation well 1C was analyzed for the case of partial penetration in a leaky aquifer using another method derived by M.S. Hantush.[14] Using these equations it was possible to determine the transmissivity, storage coefficient and leakage factor. These values are tabulated on Table 3-3. Utilizing these calculated coefficients it was possible to generate a hypothetical time versus drawdown curve. This generated curve, which reproduces the actual field drawdown data, is presented on Figure 3-35.

At the conclusion of the 20 day pumping test, water levels were monitored for several days as previously discussed. Recovery data from the production well and both deeper observation wells were analyzed using methods detailed by M.S. Hantush.(14) These data and curve characteristics are shown on Figures 3-36, 3-37, and 3-38. Aquifer coefficients were determined from the recovery data and are tabulated on Table 3-3.

Table 3-3 shows transmissivities in the 94,000 to 160,000 gpd/ft range, storativities in the 0.00017 to 0.00025 range and leakage factors of 2800 to 8930 feet. It can be seen that aquifer characteristics determined from an individual well utilizing either drawdown or recovery data produced very consistent results. However a well to well comparison of these aquifer characteristics indicated that the aquifer may be somewhat anisotropic in character.

During constant rate test No. 2, there was some concern over the close proximity of the production well to the discharging sprinkler heads, located on the 10-inch diameter discharge line. It was queried whether the sprinklers could be artifically inducing recharge to the aquifer being tested. In response to these concerns, at the conclusion of water level recovery monitoring from constant rate test No. 2, constant rate test No. 3 was conducted with all discharging sprinkler heads closed. As discussed in Section 3.1.1.6, the constant discharge for this test was approximately 1420 gpm, and drawdowns were measured in all observation wells. Figures 3-39 and 3-40 present a comparison of the drawdown data from observation wells 1B and 1C during constant rate pump tests No. 1 and No. 3 respectively. These figures illustrate the reproducibility and match of the drawdown data, regardless of the status of the discharging sprinklerheads.

Additionally, infiltration rates were estimated from pygmy current meter stream gagings, conducted at three locations along the drainage ditch created below the point of discharge.(5) These data are presented on Table 3-4. Due to difficulties inherent in pgymy meter measurements, current meter and orifice plate discharge measurements do not always correspond. In spite of these differences, the pygmy meter measurements can be used as an estimate of infiltration. From a review of this infiltration data, and an analysis of the comparative drawdown data from constant rate tests 1 and 2, it does not appear that the discharging sprinkleheads influence the various pumping tests conducted at the Steptoe Valley site.

During the pump test analysis, a concern regarding the construction of observation well 1-B was expressed. As illustrated in Figure 2-3, well 1-B is perforated from 155 feet to 170 feet, 225 feet to 235 feet, and 375 feet to 450 feet. Both the production well 1-A and observation well 1-C were only perforated below 350 feet. Because of the upper perforations in well 1-B, it was questioned what effect, if any, this would have on the pump test results.

To address this concern, well calculations assuming that well 1-B was perforated only below 350 feet were conducted. These analyses showed that water contributions from the upper zones were minor, as expected since water was being pumped only from the lower aquifer. Well calculations also indicated that the construction differences do not effect the pump test results.

3.2.2 Spring Valley

Three types of aquifer pumping tests were conducted at the Spring Valley testing site. As described in section 3.1.2, these tests consisted of (1) a variable rate pump test, (2) a constant rate pump test, and (3) a flowmeter test. The variable rate test was used to determine well efficiency, specific capacity curves and to establish the constant discharge rate for the long-term pump test. The constant rate pump test was conducted to adequately stress the aquifer for a significant period of time to produce reliable drawdowns, and subsequent recovery data. The flowmeter test was performed to determine the relative water contributions from the different aquifers encountered. All data were used to determine the various aquifer coefficients.

3.2.2.1 Variable Rate Pump Test

As discussed in section 3.2.1.1, in a new well, the variable rate pump test is conducted to aid in the design of proper pumping equipment, and to evaluate the performance and efficiency of the well.[40] Figure 3-21 presents the variable rate pump test drawdown versus time plots, with the data tabulated on Table 3-5. From an analysis of the specific capacity data, the constant pumping rate of approximately 1300 gpm was selected. The well efficiency calculations are presented on Figures 3-41 and 3-42.

3.2.2.2 Drawdown and Recovery Data

As previously discussed, production well 2A was pumped at a discharge rate of approximately 1300 gpm for a period of essentially 23 continuous days. During this period, water levels were monitored in the pumping well, and both observation wells, as well as additional monitoring locations (see Chapter 4, SUPPLEMENTAL MONITORING PROGRAMS). Water levels in the upper, unconfined aquifer were monitored by observation well 2C, while the piezometric water level in the confined aquifer system was reflected by measurements obtained in observation well 2B. Again, due to difficulties inherent in accurately measuring a pumping well, data obtained from the two monitoring wells were used to calculate the various aquifer coefficients.

Drawdown data obtained from observation well 2C, which fully penetrates the upper, unconfined aquifer were analyzed using a Cooper-Jacob analysis.[21,40] The following equations were used:

$$T = \frac{2.30Q}{4\pi\Delta s} \quad \text{and} \quad S = \frac{2.25Tt_o}{r^2}$$

where:

Δs = change in drawdown per log cycle of time

Q = discharge in the pumping well

T = transmissivity

S = storage coefficient

r = distance between the pumping well and the observation point

t_o = projected time at which zero drawdown occurs

Figure 3-43 presents the drawdown data and the calculated unconfined aquifer coefficients. Transmissivity is estimated to be 38,000 gpd/ft, and the storativity is estimated at 0.069. These values are also tabulated on Table 3-6.

Observation well 2C constructed totally in the unconfined aquifer has a static water surface elevation of approximately 5790.6 feet. However, observation well 2B, constructed in aquifers located beneath the confining clay layer reflects a piezometric level of approximately 5792.7 feet, illustrating a pressure differential of over two feet. As expected, production well 2A which taps both the unconfined and confined aquifers, shows water levels that are neither water table nor piezometric, but a composite of both. The water surface elevation in 2A is approximately 5791.9 feet.

Drawdown data obtained from well 2B were plotted on log-log paper and are presented on Figure 3-44, along with the best fit Theis Curve. Utilizing this curve, and the coordinates of the match point, values of T and S for a confined aquifer with full penetration and no boundary conditions can be determined from the following equations:

$$s = \left(\frac{Q}{4\pi T}\right) W(u) \quad \text{and} \quad u = \frac{r^2 S}{4Tt}$$

where:

- s = drawdown
- W(u) = well function
- Q = discharge from pumping well
- T = transmissivity
- S = storage coefficient
- r = distance from pumping well
- t = time

The calculated transmissivity value of 20,000 gpd/ft and storativity value of 0.00012 are presented on Table 3-6. Inspection of Figure 3-44 indicates that near the end of the pump test, the theoretical drawdown in observation well 2B should be larger than that which was actually observed. In other words, it appears that a recharge boundary condition was encountered.

In an attempt to better define the existance and location of this recharge boundary, the next analysis conducted was for a confined aquifer intercepting a recharge boundary. The equation for drawdown in a confined aquifer intercepting a recharge boundary is given by:

$$s = \frac{Q}{4\pi T} W_R(u,\beta)$$

where β = the distance between the observation well and the image well (r_i), divided by the distance between the observation well and the real well (r_r). An iterative procedure was then used to determine the possible location of this recharge boundary. Once the location of this boundary had been determined, various aquifer coefficients were tested to generate the best possible correlation to the actually observed field data. Values of W_R (u, β) were obtained from "Analysis and Evaluation of Pumping Test Data", by G.P. Kruseman et. al.[21] Figure 3-45 shows a plot of the best fit type curve W_R (u, β) versus u, superimposed on the actually observed field data. It can clearly be seen that the field data can be reproduced and matched using the generated data. Transmissivity was calculated to be 14,700 gpd/ft and a storativity of 0.00019 was calculated with this method. These calculated aquifer coefficients are shown on Table 3-6.

Calculations indicate that the intercepted recharge boundary is approximately 8600 feet away from the well. Plate VI shows the potential recharge sources which include: (1) the Bastian Creek area west of the test site; (2) a possible source located east of the test site, toward the center of the valley; or (3) possibly a buried recharge fault.

Although the actual field data could be simulated through use of a confined aquifer analysis with a recharge boundary, it was necessary to determine whether this solution was unique. An analysis considering a leaky confined aquifer system was performed. This analysis is identical to that which was performed at Steptoe and the equations presented in section 3.2.1.2 Utilizing an iterative approach, Figure 3-46 graphically presents a comparison of the observed field drawdown data, and the hypothetically generated drawdown data computed from the aquifer characteristics presented in Table 3-6. The correlation between the observed and generated drawdown data is fairly good until large values of time, where the presence of a barrier becomes evident. At this point the two curves begin to differ significantly, with the actually observed drawdowns being greater than the anticipated, hypothetically calculated drawdowns. This deviation can be explained by the possible interception of an impermeable boundary, which would cause additional drawdowns.

Pursuing this reasoning, the data were also analyzed for the case of a leaky confined aquifer with an impermeable boundary condition. Again using an iterative solution, the boundary was located at a distance of aproximately 21,000 feet from the well, and the best fit hypothetical drawdown curve was generated. This curve is presented on Figure 3-47, along with the actually observed drawdown data. A comparison of the two curves appears to show fairly good correlation. However, it is important to note that with the logrithmic vertical scale, slight curve differences, especially with increasing time, can represent discrepancies of several feet. So in actuality, the two drawdown curves are off by approximately one foot or more for time periods greater than 10,000 minutes. Thus it may be concluded that the confined aquifer intercepting a recharge boundary scenario presented earlier in this section is the most probable interpretation. Figures 3-48 and 3-49 show the recovery data and curve characteristics from production well 2A and observation well 2B respectively. Aquifer coefficients determined from the recovery data are presented in Table 3-6. It should be noted that all aquifer coefficients shown on Table 3-6 are similar for each case examined.

As discussed in section 3.2.1.2, infiltration rates were estimated from pygmy current meter stream gagings, conducted at three locations along the drainage ditch created below the point of discharge. These data are presented on Table 3-7. It should be noted that the measurements indicate a gain of water between Stations No. 1 and No. 2. Infiltration losses presented on Table 3-7 were calculated from data obtained from Stations No. 1 and No. 3. These infiltration rates are slightly lower than those calculated for Steptoe Valley.

3.2.2.3 Flowmeter Test Analysis

As discussed in Section 3.1.2.4, a flowmeter (or spinner test) was conducted in production well 2A to determine the relative water contributions from aquifers both above and below the confining clay layer present at this site. Plate III graphically presents the results of the flowmeter test conducted at flow rates of 450 gpm, 725 pgm and 935 gpm.

Construction details for production well 2A were presented on Figure 2-17. These details indicate that flow velocities measured at a depth of 160 feet to 170 feet are a measure of the total discharge produced by the entire well (both upper and lower aquifers). Flow velocities measured below a depth of 210 feet indicate water contributions from only the lower confined aquifer. Using this ratio of measured velocities, proportional flow contributions can be determined from both upper and lower aquifer zones for each of the three measured discharge rates. Figure 3-50 graphically presents the results of this analysis.

The constant rate pump test at production well 2A was actually conducted at a discharge of approximately 1300 gpm, slightly higher than the 935 gpm maximum discharge of the flowmeter test. However, extrapolation of the obtained data provides an approximation of the relative water contributions from both upper and lower aquifer zones. It is estimated that at a discharge rate of 1300 gpm, 17 percent (220 gpm) is produced from the upper aquifer, and 83 percent (1080 gpm) is contributed from the lower aquifer. A tabulation of these results is presented on Table 3-8.

4.0 SUPPLEMENTAL MONITORING PROGRAM

4.1 GENERAL OVERVIEW OF PROGRAM

A baseline data acquisition program was implemented in Steptoe Valley and Spring Valley to obtain a better understanding of the present regional hydrogeologic conditions, and to monitor the response of nearby existing wells and springs to prolonged pumpage at the test sites. Water level measurements and/or rates of discharge from a total of 17 wells and five springs were collected before, during, and after pumping tests were completed at both the Spring Valley and Steptoe Valley sites. Nine of the wells are located on private property and eight are located on Bureau of Land Management (BLM) lands, as are the five springs. Readings were taken by either LEEDSHILL or LADWP personnel.

4.1.1 Steptoe Valley Monitoring Program

The Steptoe Valley Program consisted of monitoring one BLM and six privately owned wells. Six of these stations designated as FP, WM, ET, SW, HB and DC, are located near fairly level, open range land near the center of the valley. These wells are typically shallow stock watering wells and are located within a three mile radius west of the exploration site as illustrated on Plate V. Total depth of these wells range from approximately 6 to

35 feet, tapping shallow groundwater located in the upper silt and gravel sediments. These sediments were probably deposited by Duck Creek as it meandered through this floodplain area. The nearest deep monitoring well was located on an alluvial fan 12 miles north of the test site. This well, designated HW, was sounded at a depth of 223 feet. A summary of these monitoring stations are presented on Table 4-1.

Water levels in these wells were monitored using either a calibrated electronic water level indicator, or a steel tape measure. Readings were taken approximately daily prior to start-up of the pump tests, once to twice a day during the tests, and daily to every other day during subsequent recovery periods.

Figures 4-1 and 4-2 illustrate plots of water level changes with time from wells DC and ET respectively. In addition to water level data, these graphs also present a histogram of precipitation (from the Ely Airport) from July through October 1982. Figures 4-1 and 4-2 show a slight water level decrease from July to late August, when levels stabilize. This decline in water levels is believed to be due to seasonal fluctuations and not related to WPPP pump testing. During the continuous pumping tests which were conducted from late August until October 4, none of the monitoring wells showed any effects. On September 26, a major storm passed through the Ely vicinity producing a total 2.52 inches of rainfall in a 24 hour period, the wettest 24-hour period since 1969. After September 26, water levels steadily rose (see Appendix E for water level plots from all monitored wells).

Temperature and conductivity readings recorded at the exploration site compared with measurements collected at wells WM and HB indicate significantly different groundwater aquifers. Samples collected from pumping well 1A indicate that water in the lower aquifer has a temperature of approximately 65°F and a specific conductivity of approximately 400 micromhos/cm. These values differ from the monitored stock watering wells which have a temperature of approximately 50°F and a conductivity of 2000 micromhos/cm. It should be noted that surface water samples collected from Duck Creek had a temperature of 40°F and a conductivity of 2500 micromhos/cm. Complete water quality analyses for production well 1A, observation well 1C and supplemental monitoring well DC are presented in Tables 4-3 and 4-4.

4.1.2 Spring Valley Monitoring Program

The Spring Valley program consisted of monitoring five springs and five wells located on BLM property, as well as five private wells. The five springs are located within a two-mile radius east of the exploration site, toward the center of the valley. Monitoring at Springs 1, 2, and 3 was accomplished by installing and measuring shallow two-inch diameter PVC observation wells. These observation wells were located adjacent to the ponds created by the springs. Staff gages were also installed in Springs 2 and 3. Discharge rates were measured at Springs 4 and 5 with a stopwatch and calibrated bucket.

Monitoring station EL, formerly a deep irrigation well, is located on a ranch approximately two miles north of the site. This well was sounded to a depth of approximately 625 feet. Other shallow stock watering wells and deeper irrigation wells were also monitored to evaluate seasonal fluctuations and response to WPPP pumping tests. These additional monitoring locations are shown on Plate VI. Several artesian wells were also monitored near the Shoshone ponds area, approximately 14 miles southeast of the testing site. All wells were monitored with either an electronic water level indicator or a steel tape. A summary of the monitoring stations is presented in Table 4-2.

Figures 4-3 and 4-4 present water level changes with time as observed in Spring 2 and well EL respectively. Data collected from Spring 2 indicates a slight rise in water levels from late July through mid-October. However, well EL generally shows a decrease in water level beginning in late July, stabilizing in mid-September, and then increasing to mid-October. These water level various appear to be due to seasonal fluctuations. In either case, neither of these two wells, or any of the other monitoring wells showed any response to pump tests conducted at the WPPP test site.

Groundwater temperatures and conductivity measurements obtained from Springs 4, 5 and production well 2A were relatively similar. The average temperature at Springs 4 and 5 was 54° versus 60° at well 2A. Water samples from both springs and wells showed conductivity values ranging from 275 to 300 micromhos/cm. The wells in the Shoshone ponds area had temperatures which ranged between 65° and 70°F, and conductivity values ranging from 120 to 160 micromhos/ cm. Complete water quality analyses from production well 2A, observation well 2B (deeper aquifer), and Springs 4 and 5 are presented on Tables 4-5 and 4-6.

Plots of water level changes with time for all supplemental monitoring wells are presented in Appendix E of this report.

5.0 ELECTRICAL RESISTIVITY INVESTIGATIONS

5.1 GENERAL

During August 1982, electrical resistivity investigations were performed by Harding-Lawson Associates (HLA) in both Steptoe Valley and Spring Valley of White Pine County, Nevada. Forty-three vertical electrical soundings using the Schlumberger array were conducted; 22 located in Steptoe Valley and 21 in Spring Valley. These vertical electrical sounding (VES) locations for Steptoe Valley and Spring Valley are shown on Plate 1 of Appendix C.[15] The purposes of these soundings were to assist in (1) estimating depths to ground water; (2) determining locations and depths of potential production aquifer formations; (3) determining locations and depths of unsuitable production aquifer material; and (4) developing geologic cross sections through the survey areas. The depth of investigation was of the order of 1000 feet (for further details see HLA Report presented in Appendix C).

The electrical resistivity of a geologic material is determined by several factors. The most important of these are (1) the resistivity of the pore fluid; (2) the porosity; (3) the degree of saturation; (4) the amount of clay present in the formation; (5) the resistivity of the mineral grains; and (6) the size, shape, and interconnectivity of the pores. In this analysis, it was assumed that all pore spaces were fully saturated below the water table, therefore areas having high formation resistivities related to partial pore water saturation were considered to be above the water table. In addition, it was assumed that the mineral grains were non-conducting (i.e., that little or none of the sediments are metallic or graphitic) and that (with the exception of a few deep, high-resistivity areas that may represent bedrock) the materials present in both valleys down to at least 1500 feet beneath the surface consist of unconsolidated gravels, sands, silts, and clays.

In order to relate formation resistivity and lithology, it was necessary to obtain all available "ground truth". In addition to the actual field resistivity values obtained, other "ground truth" data include published results of resistivity surveys in areas of known lithology and VES data from this survey taken in areas of surface clay deposits. Considering all of these data, the approximate correlations listed in Table 5-1 were established.

Several points should be noted with respect to Table 5-1. First, the formation resistivity ranges given are only approximate, and in reality there is considerable overlap between ranges. For this reason, the boundaries between formations of different resistivities shown on Plates 8 through 13 in Appendix C serve only as rough indicators of transitional zones between regions of generally higher or lower resistivity, and boundaries between regions having resistivities near the limits of a given range (e.g., between regions of 19 and 21 ohm-m or 240 and 260 ohm-m) are somewhat arbitrary. Secondly, no numerical values are given for clay content and porosity, as these would have to be established by actual sampling and measurement for each formation. Thirdly, without knowledge of formation clay content and type, it is not possible to choose a single combination of clay content and porosity from the multiple possibilities within each formation resistivity range. Such a choice can be made only by considering the sampling and well testing results from the formation, along with other available geological information.

The results of the geophysical well logging and surface electrical resistivity (VES) studies, combined with the geologic logs and the well production test data, indicate that formations which lie below the water table, and have resistivities between about 20 ohm-meters (ohm-m) and 250 ohm-m consist of combinations of fresh water saturated gravels, sands and silts that probably have low clay content and good potential for fresh water production. Formations having resistivities below about 20 ohm-m probably contain significant amounts of clay and may, in some cases, have saline pore water. Resistivity values greater than about 250 ohm-m may represent partially saturated material above the water table or saturated sediments or rock of very low porosity.

Both the correlation of geophysical well logs between boreholes and the general nature of the VES data indicated lateral and vertical variation of sediment properties within many formations. That is, a layer characterized by a single VES geologic description may be composed of a number of thinner layers of widely varying materials, and these thinner layers may pinch out or significantly change composition over a lateral distance of a few hundred feet. Such variability can be expected in valleys located in this Basin and Range depositional environment.

Quantitative modeling studies of the VES data indicate that it would be difficult to trace most relatively thin layers (less than about 50 feet thick) between VES stations, if these layers lie below a depth of about 200 feet. Therefore, the absence of a relatively thin high-resistivity aquifer layer or low-resistivity clay layer on a given VES section does not necessarily mean that the layer does not exist at that station.

Most of the sediments in both Spring Valley and Steptoe Valley have resistivities between 20 ohm-m and 250 ohm-m, which indicates good water production potential. Most of the areas having resistivities greater than 250 ohm-m are found at relatively shallow depths in Spring Valley and may represent unsaturated materials above the water table. Major areas of low resistivity (below 20 ohm-m) are seen mainly in the deeper, central portions of the two valleys. These areas probably represent clay-rich formations of poor water production potential (see Appendix C for HLA's detailed report.)

5.2 STEPTOE VALLEY

As previously mentioned, 22 vertical electric soundings were conducted in Steptoe Valley. The majority of these soundings were located along the eastern side of northern Steptoe Valley, beginning north of McGill, and continuing north near the vicinity of the White Pine - Elko County Line. In addition, several VES soundings were conducted in an east-west direction across the valley.

Cross-sections developed from interpretation and analysis of the VES data for Steptoe Valley are shown on Plates 11, 12, and 13 of the HLA report presented in Appendix C.

Resistivity values in this valley appear to be generally lower than those at similar depths in Spring Valley. This may be caused by one of the following: higher clay content, higher porosity values, a somewhat different predominant clay type, or differences in pore water salinity between the two valleys.

The VES profile for VES-ST1, ST2, and ST3 are shown on Plate 11 of Appendix C. This profile indicates the presence of a relatively clay-rich 14 ohm-m material beginning at a depth of a few hundred feet, and continuing for several hundred feet at both VES-ST1 and VES-ST3. This 14 ohm-m clay-rich material does not occur at VES-ST2. Instead VES-ST2 indicates the presence of material generally in the 29 to 194 ohm-m range, overlain by three shallow high-resistivity lenses. These lenses may represent partial saturation of these formations above the water table. However below these high resisitivity lenses, the formation appears to be composed of potential production aquifer material to a depth of over 800 feet. VES-ST3 also shows the presence of this potential production aquifer to a depth of approximately 400 feet.

Plate 11 also presents a profile between VES-ST4 and VES-ST13. Data from these soundings indicate that all of the shallow and much of the deep material is of greater than 20 ohm-m resistivity, indicating a promising potential for water production. Two large, deep wedges of low-resistivity clay-rich material come within 100 feet of the surface at VES-ST5 and VES-ST12. The material in these wedges probably has relatively poor potential for water production and should be avoided.

The north-south section between VES-ST14, VES-ST15, and VES-ST20 (Plate 12, Appendix C) is of nearly uniform resistivity, with the great bulk of the material ranging between 20 ohm-m and 60 ohm-m. Data from VES-ST15 and VES-ST20 indicate formation resistivity values which are near the low end of the expected good aquifer formation range.

The material shown on the cross section between VES-ST16 and VES-ST18 (Plate 12) is of generally higher resistivity than the material seen in the north central portion of the valley. The exception are a few shallow, high resistivity lenses located at VES-ST16 and VES-ST18, which may represent unsaturated material above the water table. The remainder of the material in this area appears to have good water production potential throughout the entire depth of investigation.

The east-west cross section in the Cherry Creek area (Plate 13) shows that underlaying a relatively shallow resistive material is a thick section of low resistivity clays having poor water production potential. This clay-rich material becomes more clayey with depth, and extends through the entire depth of investigation indicating the possible presence of old lakebed deposits.

5.3 SPRING VALLEY

A total of 21 vertical electric soundings designated VES-1 through VES-20 were conducted in Spring Valley. The interpretive cross-sections are shown on Plates 8, 9, and 10 of Appendix C. Plate 8 shows a north-south profile along the east side of the valley; Plate 9 shows a north-south section along the west side of the valley; and Plate 10 shows a series of east-west profiles across the valley.

As illustrated on Plate 8, the VES profile for the eastern side of Spring Valley shows resistivities that fall between 20 ohm-m and 250 ohm-m, probably representing good potential water production zones. A near-surface area of high resistivity begins south of VES-7, and extends northward through VES's 16, 17, 19 and 20. These high resistivities may represent partially saturated material above the water table. With the potentially deep water table, production wells should be located slightly west of these VES locations to minimize drilling through partially saturated materials.

The resistivity distribution on the west side of Spring Valley (Plate 9) appears to be more complex that that encountered on the east side. The bulk of the material on the west side of the valley has resistivity values which fall between 20 ohm-m and 250 ohm-m, representing good water production potential. However, a significant region of near-surface high resistivity, possibly representing partially saturated material above the water table, extends between VES-14 through VES-5. Similar high resistivity material also is seen at VES-3 and VES-1. The extensive depth of this material at VES-1 suggests the presence of low-porosity consolidated rock, rather than partial saturation, at depth in this area and should be avoided.

Significant areas of low and very low resistivity material also are seen on Plate 9. The upper contact layer of this material, which probably contains a high percentage of clay, begins at a depth of approximately 650 feet at VES-14 and rises to approximately 350 feet at VES-8. A deeper zone of this clay-rich material extends from VES-8 to VES-4, with the very low resistivity readings at the bottom of VES-5 and VES-5A, probably representing saline pore water.

The east-west sections across Spring Valley are shown on Plate 10. These soundings indicate that the material in the central portions of the valley has a resistivity of approximately 20 ohm-m. This value is on the low end of the expected good aquifer formation range, and probably marginal at best. These profiles also indicate the presence of very clay-rich or saline formations at depth.

6.0 GROUNDWATER MODELING

6.1 GENERAL

Two-dimensional, finite element mathematical models were utilized by Environmental Dynamics Inc. (EDI) to simulate both pre-project and post-project groundwater conditions in Spring Valley and Steptoe Valley. These models were used to estimate drawdowns and impacts of the proposed WPPP well field pumpage on existing groundwater levels at the end of a 36-year pumping period.

The modeling procedure involved the following steps:

1. Subdivide the valley into elements which represent areas of constant transmissivity (T) and storativity (S).

2. Quantify the hydrologic water balance components in the model in order to determine the areal distribution of groundwater recharge and discharge.

3. Calibrate and verify the model using historic steady state groundwater data and current water usage in the valleys.

4. Superimpose WPPP pumpage on this calibrated flow system and perform simulation runs using specified input scenarios to determine project impacts on groundwater levels over time and space for the 36-year expected economic life of the project.

5. Perform various sensitivity analysis.

6.2 ANNUAL RECHARGE

Annual recharge to the valleys was estimated using the Maxey-Eakin methodology. Basically this approach consists of (1) estimating the relationship between average annual precipitation and elevation, and (2) applying established annual infiltration or recharge percentages over various precipitation zones, to establish the total recharge to the groundwater basin.

Detailed review of precipitation data collected at stations in or near White Pine County indicated that eight stations had a sufficient period of record to be statistically reliable. Characteristics of these stations are presented in Table 6-1. All of these stations are in Steptoe Valley except Lehman Caves National Monument, which is in Snake Valley just east of Spring Valley.

A least squares regression equation was developed for the seven precipitation stations located in Steptoe Valley. This equation related mean annual precipitation to elevation for water years 1965 through 1980. As shown on Figure 6-1, the least squares equation compares well with relationships previously developed by the U.S. Geological Survey (USGS) and the Nevada Department of Conservation and Natural Resources (NDCNR). Precipitation was estimated at various elevations and applied over the topography of Steptoe Valley. Recharge was then estimated in each elevation zone by applying the recharge percentages developed by the NDCNR as shown in Table 6-2.[9] This resulted in an estimated annual recharge of approximately 81,600 afy in Steptoe Valley.

The calculation in Spring Valley was similar, with the addition of the Lehman Caves precipitation station. Figure 6-1 illustrates the favorable comparison with the USGS/NDCNR equation. Precipitation estimates using this equation were then applied throughout Spring Valley, and applying the NDCNR recharge percentages resulted in an estimated annual recharge of approximately 69,500 afy as shown on Table 6-3.

6.2.1 Estimates of Recharge Based on Available Streamflow Records

There are four gaged watersheds in or near the Spring Valley - Steptoe Valley Area. Duck Creek and Steptoe Creek are located in Steptoe Valley, Cleve Creek is in Spring Valley and Little Currant Creek is located in Railroad Valley, which is situated in the southwestern part of White Pine County. Table 6-4 summarizes the data available from each of these stations.

Measured average annual runoff from each gaged watershed were compared with the estimated recharge computed using the USGS/NDCNR methodology, and with alternative percentages of recharged precipitation estimated by EDI. The results of these comparisons are shown in Tables 6-5 through 6-8 and summarized in the following tabulation.

Stream	Estimated Recharge USGS/NDCNR (acre-feet)	Measured Runoff (acre-feet)	Period of Record
Cleve Creek	5377	6958	1961-67,1977-80
Duck Creek	11,700	8910	1909-15,1957-65, 66,67,74,76
Little Currant Creek	1602	2600	1965-80
Steptoe Creek	2035	5146	1967-80

Except for Duck Creek, the recharge estimated using the USGS/NDCNR methodology is less than the measured average runoff, implying that the USGS/NDCNR recharge estimates are somewhat low. Duck Creek is not a representative watershed because only the low flows are measured, and larger flows above the weir capacity are diverted and unmeasured. As a result, the measured runoff on Duck Creek is less than what actually occurs.

Thus, it may be concluded that the annual recharge estimates of 81,600 afy for Steptoe Valley and 69,500 afy for Spring Valley may be conservative. Nevertheless these annual values were used in the groundwater simulation model.

6.2.2 Distribution of Recharge

The mountainous areas surrounding Spring Valley and Steptoe Valley are the principal sources of groundwater recharge. These areas are not evenly distributed either by drainage area or by elevation, therefore recharge can be expected to reflect these spatial and elevation variations.

The Steptoe and Spring Reconnaissance Reports by NDCNR estimate the following areal distribution of recharge:[8, 34]

Mountain Range	Percentage of Recharge	
	Steptoe	Spring
Schell Creek	67	81
Cherry Creek	19	-
Egan Creek	14	-
Snake	-	19

Each mountain range was further subdivided into subareas, with each subarea in each valley receiving recharge from the adjacent mountains based upon the elevation and drainage area within that watershed.

The distribution of these recharge areas and the percent for each area are shown on Figure 6-2 for Spring Valley and 6-3 for Steptoe Valley.

6.3 ANNUAL DISCHARGE

Annual discharge quantities from both Spring Valley and Steptoe Valley were calculated using evapotranspiration (ET) data, irrigated acreage maps, and vegetative cover maps provided by the U.S. Soil Conservation Service (SCS).[50,51] Monthly ET data for alfalfa and improved pasture were compared with data obtained during ET studies conducted on wet meadow areas, with very similar results. These average annual consumptive use factors are approximately 1.5 afy for irrigated acreage, and 1.0 afy for wet meadow areas. Agricultural and wet meadow areas, as determined by vegetative cover maps, were planimetered in both valleys to determine total valley irrigated acreage and wet meadow areas. Table 6-9 shows these factors applied over the appropriate acreages in each of the valleys. Calculated discharges from Spring and Steptoe Valleys, respectively are approximately 69,400 afy and 84,500 afy. These values are comparable with discharge estimates developed by the USGS/NDCNR and were used during calibration of the groundwater model. It may be noted that these estimates of consumptive use may be conservative. If actual consumptive use rates are higher, annual valley discharges must also be larger and valley perennial yields, such as in Steptoe Valley, must also increase since USGS/NDCNR perennial yield estimates are limited by computed discharge quantities.

6.4 RECHARGE AND PERENNIAL YIELD

In Water Resources Reconnaissance Series Report No. 33 on Spring Valley, the USGS/NDCNR present estimates of runoff, groundwater recharge and groundwater discharge. Runoff was estimated at 90,000 afy, but the annual groundwater recharge was estimated to be 75,000 acre-feet of the latter quantity. About 65,000 acre-feet was obtained from rainfall on the mountainous areas, and 10,000 acre-feet from precipitation on the alluvial aprons.(34) Estimated annual groundwater discharge was 74,000 acre-feet, of which 70,000 acre-feet was estimated to be consumed by evapotranspiration and 4,000 acre-feet was estimated to flow from the valley into neighboring Hamlin Valley to the south. These discharge estimates do not include evaporation losses from the playas. It has been estimated that more than 30,000 acre-feet of runoff wastes to the two playas located in the center of Spring Valley. The report concludes that the estimated minimum annual yield of Spring Valley is 70,000 acre-feet, but that if a substantial part of the runoff which was estimated to be wasting to the playas could be salvaged through efficient groundwater development and management, the perennial yield might be on the order of 100,000 acre-feet.

In the Water Resources Reconnaissance Series Report No. 42 on Steptoe Valley, the USGS/NDCNR estimated runoff at 78,000 afy, and groundwater recharge at 85,000 afy.(8) It was noted in this report that the estimated recharge is about seven percent of total precipitation, as compared with five percent in most Nevada valleys. On this basis the USGS authors suggested that the estimate of recharge may be high. Estimated annual discharge was approximately 70,000 acre-feet and this value was also given as the estimated perennial yield.

Groundwater discharge is now estimated by LHJ/EDI to be approximately 84,500 afy as shown on Table 6-9 based on current maps of vegetative cover and irrigated land.

6.5 SIMULATION MODELS

The computer simulation models numerically solve the underlying groundwater flow equations used in the two-dimensional mathematical model. The mathematical basis for this model is presented in Appendix D. Data requirements of the model include specifying:

1. initial water surface and boundary conditions at each node,

2. the thickness, transmissivity and storativity parameters at each node,

3. the location and magnitude of recharge and withdrawal, and

4. the location (x,y) of each nodal point in the finite element network.

The model assumes that within any element the head is a linear function of the x and y coordinates, i.e., linear quadrilateral elements.

Finite element grids were developed for Spring Valley and Steptoe Valley based in part on distribution of sand and gravel maps presented on Plate II of LEEDSHILL's Phase 2 Report.[23] The Steptoe Valley model elements are generally four square miles in area in the interior of the valley, and approximately two square miles in the boundary areas. In Spring Valley, to accommodate the one-mile well spacing, elements are one to two square miles in area in the region of the proposed well fields, four square miles in the remainder of the interior, and two square miles in the boundary areas. These elemental areas are considered to be appropriate in view of the availability of data and the large areas to be modeled. Maps showing the node numbers and element shapes in each valley are presented as Plates VII and VIII for Steptoe and Spring Valleys respectively.

6.5.1 Model Calibration

The application of a simulation model to a particular aquifer system is ideally a three-step process involving calibration, verification and prediction. The aquifer parameters should be calibrated using one period of the historical time period and then verified against another period of the historical record. However, due to a lack of long-term basic data in both Spring and Steptoe Valleys, the calibration and verification processes were combined.

The calibration process involved the trial-and-error adjustment of both the areal distribution and magnitude of aquifer parameters and boundary conditions. Initial estimates of these aquifer parameters were input to the simulation model which then calculated an estimated water surface elevation. These computed water surface elevations were then compared to water levels previously measured. If the match was not acceptable, the process was repeated using different aquifer parameters or assumptions until an acceptable simulation was attained. In general the models were considered to be calibrated when the mean difference between computed groundwater elevation and observed elevations was less than six feet or one percent of the aquifer thickness and maximum deviation at any node was less than 15 percent of the wetted thickness.

2. Constant flux across the boundaries, and

3. Constant head boundaries.

The impermeable boundary condition was simulated by placing a series of injection wells around the perimeter of the valley. These injection wells were used to simulate the recharge into the aquifer from the surrounding mountainous areas. The injection well flow rates and their spatial distribution were determined from previous water balance/recharge calculations. Using this approach, it was assumed that inflow to the valley (injection wells) and output from the valley (evapotranspiration and agricultural demand via pumping wells) would be identical.

The results of this particular calibration process were not satisfactory in either valley. The computed water levels were significantly different than historically measured levels. Trial runs using varying transmissivity values and zones of distribution indicated no substantial improvement. Another problem associated with this representation was a water level oscillation effect observed over time. Water levels fluctuated year to year in a "sea-saw" manner. Due to the large number of recharge wells, a large amount of time would be required to achieve an acceptable balance and match, because of the many possible recharge well combinations.

These difficulties were probably caused because the groundwater recharge in the valleys does not occur as a point or line source, but is a distributed flux along the boundaries. Furthermore, the representation of a point sink on or near an impermeable boundary in a finite-element model is physically untenable (mathematically - a singularity on a no flow boundary). Since the transmissivity, storativity, and recharge rates were reasonably known, it was decided to abandon this boundary condition, and, instead represent the boundary conditions of each valley as constant fluxes (the Neumann conditions).

With the constant flux approach, flux rates and their spatial variations were calculated from previous estimates of recharge and its distribution. Inflow or recharge was also simulated using infiltration through the perimeter boundary elements. The results of repetitive simulations indicated that the predicted head levels were extremely sensitive to minor variations in the boundary fluxes and to the cross-sectional area of the saturated thickness of each perimeter element. Any error in measurement resulted in the violation of the mass balance constraints and over time the valley either gained or lost water. After performing 25 simulations using each set of recharge conditions, it was decided that although this approach to calibrate the model was plausible, it would take several months to complete, and was therefore terminated.

The third boundary approach was to represent each valley's boundary condition as a constant head (the Dirichlet conditions). With this approach, the hydraulic or piezometric heads on the boundary were estimated and established from groundwater recharge calculations. A limitation which must be recognized when using this approach is the potential to increase flow across the boundaries of the model.

With the constant head boundary condition, the models were calibrated utilizing various transmissivity zones. In Steptoe Valley, two transmissivity zones were used. The fan area of the valley was modeled using a value of 94,000 gpd/ft, and the center of the valley was generally modeled using a transmissivity of 25,000 gpd/ft. Spring Valley was simulated using five different transmissivity areas. The fan area was modeling using values of 19,450 gpd/ft; 39,000 gpd/ft; and 58,350 gpd/ft. The playa in northern Spring Valley was modeled using a value of 9725 gpd/ft and the remaining playa areas were simulated at 5000 gpd/ft.

Model calibration involved matching computed water levels with existing static groundwater levels. Figures 6-4 through 6-6 show the comparison between estimated static groundwater levels and groundwater levels computed by the model at cross-sections A, B and C in Spring Valley for nodes 70-74, 127-131 and 176-179 respectively (see Plate VIII for cross-section locations). Computed water levels generally correspond to measured static levels. It may be noted that at nodes 129 and 130, computed levels are above the ground surface; however since artesian conditions are encountered at many locations in the valley, these estimates may represent actual field conditions.

Figures 6-7 through 6-9 show similar comparisons for valley cross-sections A, B, and C in Steptoe Valley, located near Cherry Creek Station (nodes 266-269), nodes 287-291, and at the county line (nodes 333-338). (See Plate VII for cross-section locations.)

These figures indicate that the model is capable of closely approximating estimated static water levels, and can serve as a basis for estimating potential future drawdowns or piezometric surface changes resulting from WPPP well field pumpage.

6.5.2 Modeling Results

At the completion of the calibration phase, various scenarios were run to determine the effects of WPPP well pumpage on groundwater levels. Several alternative assumptions concerning future well withdrawals and agricultural usage in the valleys were investigated.

In Steptoe Valley the three following scenarios were run:

Scenario	Average Annual WPPP Well Field Pumping (afy)	Agricultural Pumpage (afy)
Case I	20,000	Current
Case II	25,000	Current
Case III	25,000	Current plus 15,000 afy, in northern Steptoe

In Spring Valley the following three scenarios were run:

Scenario	Average Annual WPPP Well Field Pumping (afy)	Agricultural Pumpage (afy)
Case I	20,000	Current
Case II	25,000	Current
Case III	25,000	Current plus 15,000 afy, in an area south of Highway 50, next to existing agricultural users

Average groundwater drawdowns were computed for each of the scenarios over a period of 36-years, the projected economic life of the project. Computed piezometric surface drawdowns for Case II, current agricultural pumpage plus WPPP pumpage of 25,000 afy are shown on Plates IX and X for Steptoe Valley and Spring Valley, respectively. Potential well field locations as well as saline meadow areas are also shown on these Plates. These computations were based on the aquifer characteristics shown in Table 6-10 and the well field pumpage rates given in Table 6-11.

In Steptoe Valley, it can be seen on Plate IX that elemental areas closest to the actual well field locations will experience piezometric surface drawdowns of approximately 15 feet to 20 feet. Plate X which illustrates Case II conditions for Spring Valley, shows piezometric surface drawdowns to be both larger in areal extent and deeper than those calculated for Steptoe Valley. These larger piezometric surface drawdowns are due to the significantly lower transmissivity values used in the Spring Valley model to simulate groundwater flow. The largest calculated average piezometric surface changes in Spring Valley are estimated to be 30 feet to 40 feet, throughout the elemental areas adjacent to the well field centers.

Calculations using the Case III scenario, current agricultural requirements plus 25,000 afy for the WPPP plus an assumed additional 15,000 afy for future agricultural growth in northern Steptoe Valley were also conducted. This analysis indicates that the additional 15,000 afy future agricultural growth could cause the drawdown cone to increase in areal extent in northern Steptoe Valley over and above the anticipated Case II piezometric surface declines.

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The Case III scenario in Spring Valley, involved current agricultural requirements plus 25,000 afy required for the WPPP plus 15,000 afy for future agricultural growth in southern Spring Valley. It was assumed that this 15,000 afy future agricultural growth would occur south of Highway 50, generally along the eastern side of the valley adjacent to existing agricultural areas. The analysis indicated that this additional agricultural growth could cause piezometric surface drawdowns in southern Spring Valley to both increase in areal extent and magnitude. This is attributable to a concentrated pumping area assumed in the modeling to be located in an area of low transmissivity.

A case by case comparison of the average estimated piezometric surface drawdowns for selected nodes in wet meadow and other areas are presented in Tables 6-12 and 6-13 for the various scenario conditions in Steptoe Valley and Spring Valley respectively.

It should be emphasized that it was assumed that production wells would draw water only from the lower, semi-confined or confined aquifer systems. Therefore the drawdowns, as shown on Plates IX and X, are changes in piezometric surface (pressure changes), and not necessarily reductions in water table levels. It should also be noted that the areal extent of these confining layers throughout the entire valley are not accurately known.

As pumpage continues over the years, less water will be removed from the confined and semi-confined aquifers and more water will be developed from one or more of the following sources: reduced phyreatophyte evapotranspiration, induced recharge, or leakage of water from the unconfined aquifer through the partially confining layer. As leakage occurs through the partially confining layer, water levels in the overlying unconfined aquifer may lower. This will in turn reduce consumptive use by plants which use water from the groundwater table. These lowered water levels will also allow water from streams currently running off to and evaporating from the playa areas (rejected recharge) to be captured and re-charge the groundwater basin. These effects cannot be precisely quantified at this time. In this regard it can be conservatively assumed that the piezometric surface changes as illustrated on Plates IX and X, may be considered as maximum water table drawdowns.

There are several assumptions which were used during the groundwater modeling which are very conservative and offset the somewhat non-conservative constant head boundary condition. One of these assumptions is the annual use of 25,000 afy by the project. The model assumes that all wells are continuously pumped 24 hours per day, 365 days a year, for a period of 36-years. This water quantity of 25,000 afy is significantly greater than that actually anticipated for plant use. Another conservative assumption is the aquifer characteristics used in the groundwater model. The values used were those developed during the pump test which reflect the semi-confined or confined aquifer systems. The storativity factor of 0.0002 would be far too low if unconfined aquifer conditions materialized. Such conditions would significantly reduce the areal extent of the computed drawdown cones. Another conservative assumption is that salvage water is not considered in the model. With lowered groundwater levels, plants which rely on groundwater will use less water. Also with lowered groundwater levels some stream runoff which currently wastes to and evaporates from playas may become available for groundwater recharge.

It should be recognized that these estimated piezometric surface drawdowns are valid only under the stated modeling and aquifer characteristic assumptions, and actual values may deviate from predicted values due to the proximity of recharge or discharge areas, uncertainties in the estimate of aquifer characteristics, and other localized conditions. The aquifer characteristics used in this simulation are listed for each valley on Table 6-10.

Recognizing those uncertainties, the project has initiated a groundwater monitoring program to measure, record and report actual groundwater levels and changes related to project activities in Spring Valley and Steptoe Valley.

6.5.3 Sensitivity Analyses

Several analyses were conducted to determine the sensitivity of the calculated drawdowns to variations in the boundary conditions and coefficients used in the simulation model.

One assumption used in the simulation model was that of a constant head recharge boundary. Under some circumstances, employing this boundary assumption could result in underestimates of the magnitude of well field drawdowns. To test the sensitivity of this assumed boundary condition, the constant head boundary was replaced with a totally impermeable boundary condition. The WPPP pumpage of 25,000 afy was then superimposed upon this system for the 36-year project life. Under this hypothetical and highly unrealistic condition of zero recharge, all project pumpage would be obtained from storage. This condition was investigated to develop an ultra-conservative case.

After pumping for a period of years in either valley, pressure reductions would occur in the semi-confined or confined aquifers which would tend to induce leakage from the overlying unconfined aquifers. Under these conditions, the extremely small storage coefficient of 0.0002 appropriate for confined conditions, would be inappropriate for an unconfined aquifer condition. The unconfined aquifers have storage coefficients (specific yields) of approximately 10 percent to 15 percent in Spring Valley and 10 percent to 20 percent in Steptoe Valley. Because the time period within which this transition occurs is uncertain, specific yields of 12 percent for Steptoe Valley and 10 percent for Spring Valley were used for all years in this impermeable boundary, no recharge, scenario.

Plates XI and XII show the calculated drawdowns under this hypothetical and highly unrealistic scenario caused by WPPP pumpage for Steptoe and Spring Valleys respectively. For Steptoe Valley, the average drawdowns are on the order of 40 to 45 feet in the vicinity of the well fields. However, because of the zero recharge assumption, the areal extent of the drawdown cone is computed to be larger than for the constant head boundary case. In Spring Valley, because of the smaller transmissivity values and the closer center to center spacing between wells, the average drawdowns are computed to be greater. Such averages are computed to be over 120 feet in the vicinity of the well field located along the western side of the valley. As in Steptoe Valley, the zero recharge assumption results in a computed drawdown cone which is larger in areal extent than that calculated for the constant head boundary case.

It must be emphasized that the assumptions required to compute the drawdowns illustrated on Plate XI and Plate XII are hypothetical, improbable and highly unrealistic of existing conditions. These contours do not depict anticipated drawdowns.

In another sensitivity test, the simulation model was operated under the Case II assumptions with the transmissivity values reduced by 20 percent. Tables 6-12 and 6-13 present a comparison of the average estimated piezometric surface drawdowns for selected nodes in Steptoe Valley and Spring Valley respectively. The general drawdown patterns are similar to the Case II condition in each valley, but the drawdown depths and areal extents are somewhat larger.

6.5.4 Individual Well Drawdowns

It should be emphasized that the contours presented on Plates IX through XII are contours of average piezometric surface changes over a given area, and not changes in the depth to the groundwater table. Individual drawdowns from WPPP wells were calculated and are tabulated on Tables 6-14 and 6-15 for Steptoe and Spring Valleys respectively. In Steptoe Valley 14 wells, each pumping at 1250 gpm comprise the WPPP well field. Assuming an aquifer transmissivity (T) of 94,000 gpd/ft, storativity of 0.0002 and a time period of 10 years of continuous pumping, individual well drawdowns, including mutual well interference from only WPPP wells, could be on the order of 125 feet to 155 feet. These drawdowns are calculated at the wells themselves, and are not average drawdowns in the nodal or elemental areas where the wells are located. The 10-year time period of continuous pumping was considered to be a reasonable period during which time the confined storage coefficient of 0.0002 would be valid. Plate XIII presents the proposed well field locations and well numbers.

Figure 6-10 illustrates a typical cone of depression created by a WPPP well. Drawdown at the well itself is approximately 155 feet, however as one moves away from the actual well site, drawdown impacts are significantly reduced. At a distance of one mile, average piezometric surface drawdowns are approximately 20 feet under the Case II conditions.

In Spring Valley, the WPPP well field will be composed of 22 wells, each pumping at 750 gpm. Using aquifer transmissivities (T) of 58,350 gpd/ft and 39,000 gpd/ft, storativity (S) of 0.0002 and a time period of 10 years, drawdowns on the order of 120 feet to 240 feet were calculated at individual wells. These drawdowns are larger than those anticipated in Steptoe Valley because of the smaller transmissivity values and the closer well spacings used in Spring Valley, which can cause greater mutual well interference. Plate XIV presents the proposed well field locations and well numbers.

Figure 6-11 presents a typical WPPP well drawdown cone. After approximately 10-years, drawdown at the well itself may be on the order of 235 feet, but at a distance of one mile from the pumping well, the piezometric surface drawdown may be approximately 25 feet, under the Case II conditions.

6.6 SUBSIDENCE

Land surface subsidence can occur as a result of groundwater pumpage through either or both of two mechanisms, (1) by lowering the water table in an unconfined aquifer and/or, (2) through lowering the piezometric surface in an artesian aquifer system.[25]

In an unconfined aquifer, a lowering of existing water levels can remove a portion of the buoyant force exerted on the sediments by the water. Removal of this buoyant force will result in an increase in the effective unit weight of the dewatered material. If sufficiently large, this additional load on the underlying sediments can result in compaction and consolidation.

In a multiaquifer system composed of interlayered aquicludes and aquifers, total geostatic pressure, P_t, at any depth can be expressed as

$$P_t = P_h + P_i$$

where P_h is the hydrostatic pressure (pore pressure) and P_i is the intergranular pressure.[40] Water in an artesian aquifer exerts a hydraulic head against adjacent aquicludes, developing hydrostatic pressures within these interfacing zones. If water is pumped from an artesian layer, pore pressures, P_h within this zone can be reduced, resulting in increased intergranular pressure, P_i. Aquicludes composed of compressible clays may exhibit consolidation under these conditions.

Plate IX and Plate X depict areas which may experience piezometric surface changes of five to 20 feet in Steptoe Valley, and five to 40 feet in Spring Valley due to pumpage from WPPP wells. This change in piezometric surface may produce stresses within the geologic strata capable of consolidating these materials.

During the summer of 1982, Ertec Western Inc. (ERTEC), performed preliminary geotechnical studies for the WPPP. These studies involved the collection and analysis of undisturbed soil samples at the proposed power plant sites. Based upon these studies, ERTEC has conservatively estimated that subsidence at the proposed power plant sites could be on the order of several inches in Steptoe Valley and range from several inches to two feet in Spring Valley.[10] It should be noted that these calculations are general order-of-magnitude approximations only, and applicably only at the proposed power plant sites. In other valley areas affected by WPPP pumpage, additional investigation will be required to adequately determine the site specific soil characteristics necessary to access the subsidence potential.

7.0 WATER SUPPLY SYSTEM

7.1 GENERAL

In this section of the report preferred well field locations and recommended well field design parameters are discussed.

7.2 WELL FIELD LOCATIONS

In the Phase 2 report preliminary well field locational criteria were presented. These criteria have been modified and refined as a result of discussions with representatives from Dames and Moore, and results of the Phase 3 investigation, to more closely achieve the following objectives -

- minimize interference with existing pumpers
- minimize drawdowns in existing environmentally sensitive areas
- develop efficient wells which tap reliable groundwater supplies

These recommended criteria are listed as follows:

1. Wells should not be located within one mile of known existing wells or private land.

2. In Steptoe Valley, wells should be located on the east side of the valley to minimize the possibility of interference with existing thermal springs, which are generally located on the west side of the valley.

3. In Spring Valley, well fields should not be located near the Shoshone Ponds area, to minimize the possibility of interference with pressure levels in that area.

4. Well fields should be located in the higher transmissivity areas. These areas are generally located on the alluvial fans, between the mountain fronts and the finer grained playa deposits.

5. Well fields should generally have a single-row configuration which parallels the elongated north-south direction of the valley.

6. A well field should consist of a minimum of two wells.

7. Well spacings should be approximately two miles in Steptoe Valley and one mile in Spring Valley.

8. Well fields should be located to receive recharge from as large a drainage area of high elevation mountains as possible.

9. Well fields should be located in areas found to have relatively favorable aquifer characteristics in the geophysical survey.

10. In addition to monitoring existing wells, springs and streams, monitoring wells should be installed at various locations to verify the areal extent and drawdown of the pumping cone, caused by initiation of WPPP pumping. These wells should be installed at least one year before initiation of well field pumpage to establish baseline data.

Well field configurations which reflect the foregoing constraints are shown for Steptoe Valley on Plate XIII and for Spring Valley on Plate XIV.

The generally proposed two mile well spacing in Steptoe Valley and the one mile spacing in Spring Valley are a result of evaluating several concerns. These major concerns are: (1) minimizing WPPP well field interference with existing groundwater users; (2) generally maximizing induced recharge from areas where recharge is currently being rejected because of high groundwater levels, (3) generally minimizing the magnitude of the WPPP drawdown cone, by distributing the wells over a large area. A concentrated well field would create a much deeper cone of depression, affecting existing pumpers to a much greater degree; and (4) "permitted wells" must be at least one quarter of a mile away from another existing well.

7.3 WELL DESIGN PARAMETERS

The following parameters should be considered for use in the design of individual wells.

1. Wells should be designed to discharge at an average maximum rate of 1250 gpm in Steptoe Valley and 750 gpm in Spring Valley. Areas having favorable conditions may produce higher discharges, therefore the foregoing rates represent average conditions anticipated to be encountered.

2. Although individual well depths will vary due to site specific geologic conditions, and test wells were not constructed at all proposed well field locations, an evaluation of the geophysical surveys indicates generally favorable aquifer conditions at the proposed well field sites. Generally in Spring Valley, well depths are estimated to be less than 600 feet deep along the west side of the valley, and somewhat deeper along the east side. In Steptoe Valley, wells are estimated to generally be less than 700 feet deep. These cased well depths also generally consider the estimated long-term individual well drawdowns, and mutual well interference effects caused by WPPP pumpage.

3. Determination of an appropriate well diameter should be based on economic comparison of well losses and installation costs. However it is anticipated that a minimum casing and screen diameter of 16-inches will be quite competitive because this size is the standard size for irrigation wells in the area.

4. Geophysical logs should be correlated with geologic logs developed by a geologist for at least one borehole in each well field area.

5. The suite of geophysical logs should include:

   (1) Natural gamma (if the presence of large quantities of volcanic rock are not detected in cutting samples),

   (2) Self-potential,

   (3) .25 and 2.5 normal resistivity,

   (4) .25 and 2.5 lateral resistivity,

   (5) caliper, and

6. Specifications requiring drillers to collect samples at each ten foot interval, and formation change, for use in developing geologic well logs, correlating with geophysical logs and determining appropriate slot openings and gravel pack specifications, should be written into the job specifications.

7. Design of the appropriate screen slot size and gravel pack should be based upon conditions encountered at each of the specific borehole locations. Sieve analysis of selected samples obtained from potential aquifer zones should be the basis for design of the gravel pack and

corresponding screen slot size. The screened interval should be based on available geologic and geophysical logs. It is anticipated that the minimum screen length will be on the order of 100 to 200 feet.

8. Preliminary estimates indicate that individual pump motor hp requirements in Steptoe Valley may be slightly larger than those required for Spring Valley because of the larger average discharge rate required in Steptoe Valley . It is anticipated that Steptoe Valley wells may require approximately 125 to 150 hp motors, and Spring Valley wells may require approximately 75 to 100 hp motors. These hp requirements also account for increased pump lifts due to mutual well interference and water level declines.

7.4 WATER SUPPLY SYSTEM

The water supply system for the powerplant will include transmission pipelines, storage facilities and possibly booster pumps. The following is a list of generalized design criteria for such facilities.

1. Pipeline diameter should be based on an economic analysis in which the future operating costs, including power, are

compared with the present capital cost to purchase and install the pipeline.

2. The pipeline design should consider the tradeoffs between booster pumps and additional capacity in well pumps.

3. Pipeline design should consider potential surges during outages and/or startups.

4. Potential sand accumulation should be dealt with in design by providing manholes, providing additional reservoir storage and/or providing sand extractors.

5. Pipelines should be designed to withstand or avoid potential flood flows at stream crossings.

6. Reservoir capacity sizing should consider potential tradeoffs between off-peak and on-peak pumping, as well as emergency storage requirements, operational requirements, reserves for sedimentation, and plant water use.

7. Economic comparison of alternative storage reservoir configurations should consider future evaporation losses, embankment costs and lining costs in developing an appropriate configuration.

8. Reservoir embankments should be designed with sufficient crest widths for vehicle access with side slopes determined by the engineering properties of the local borrow materials.

9. Potential dam sites in stream channels may offer economic water storage facilities.

10. Wells should be operated by a computerized telemetry control system which is keyed to reservoir water levels and water system demand.

11. A separate supply system which bypasses the storage reservoir should be used to provide water for domestic use at the plant.

12. In planning for emergency operations, consideration should be given to separate pipelines and/or emergency well fields as well as standby storage.

13. Storage reservoirs should be located and operated so as to minimize pumping to the plant from the storage facility. This can be achieved, in part, through the use of an altitude valve on the inflow line which allows the flow to bypass the storage facility most of the time.

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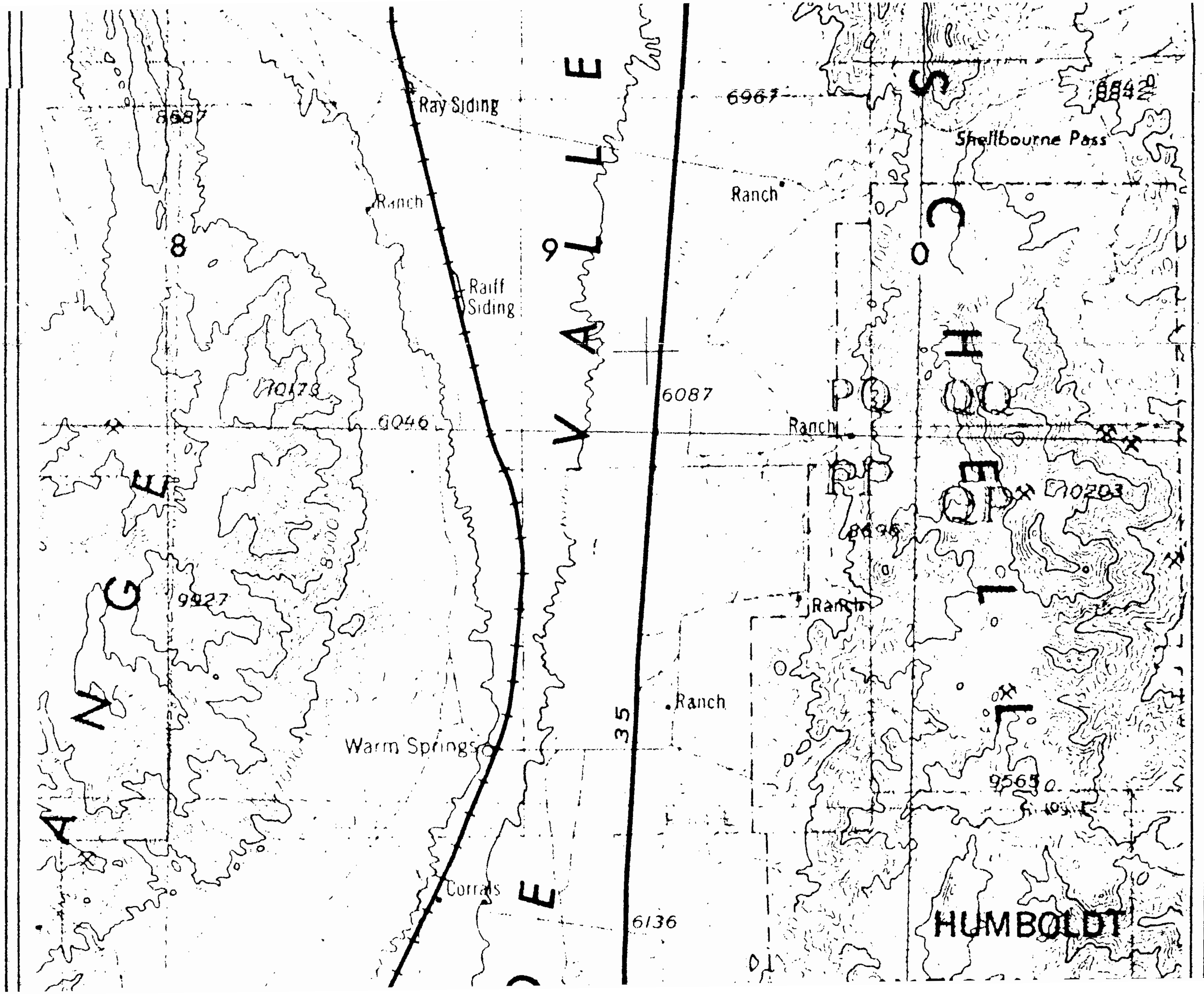
PLATES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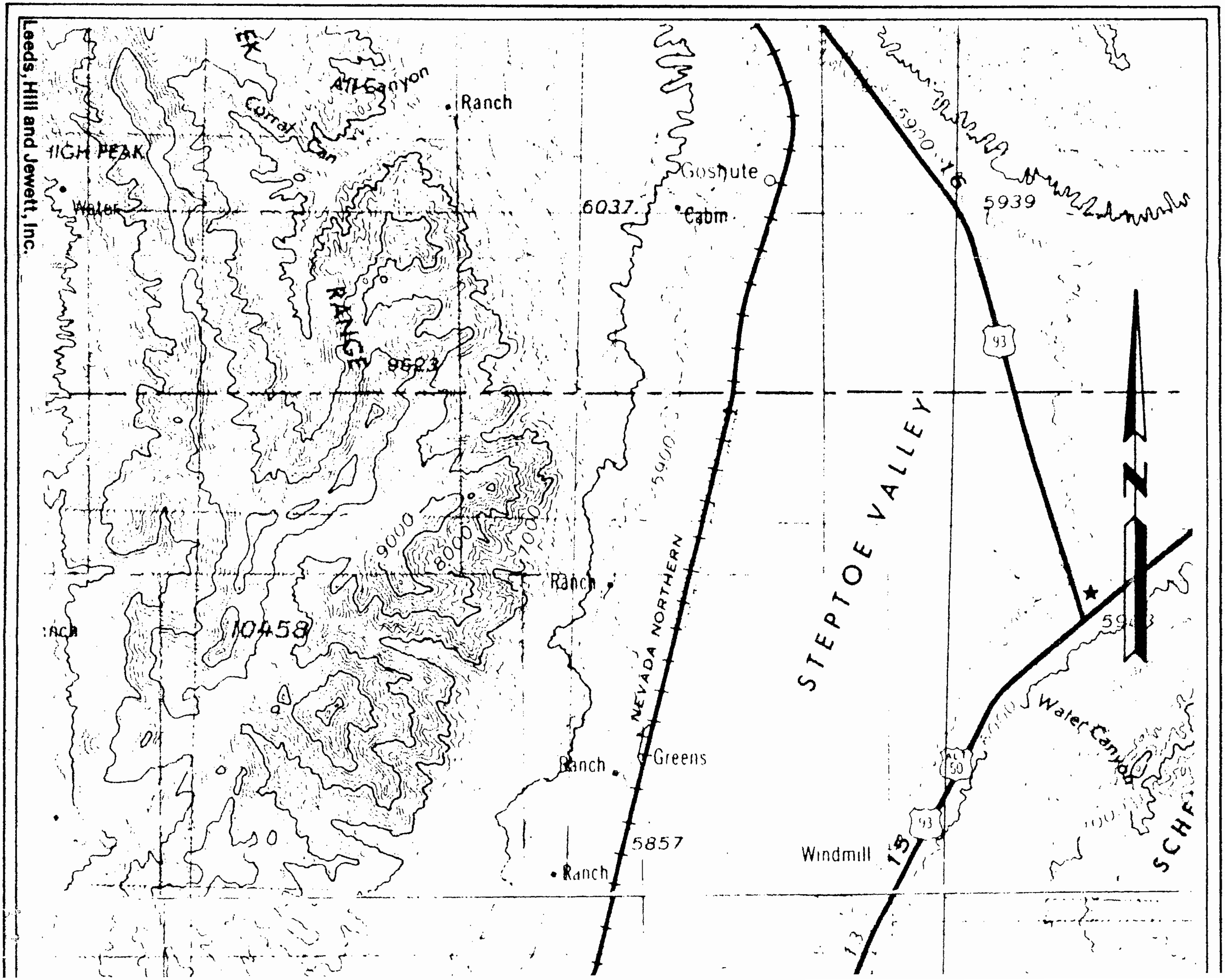
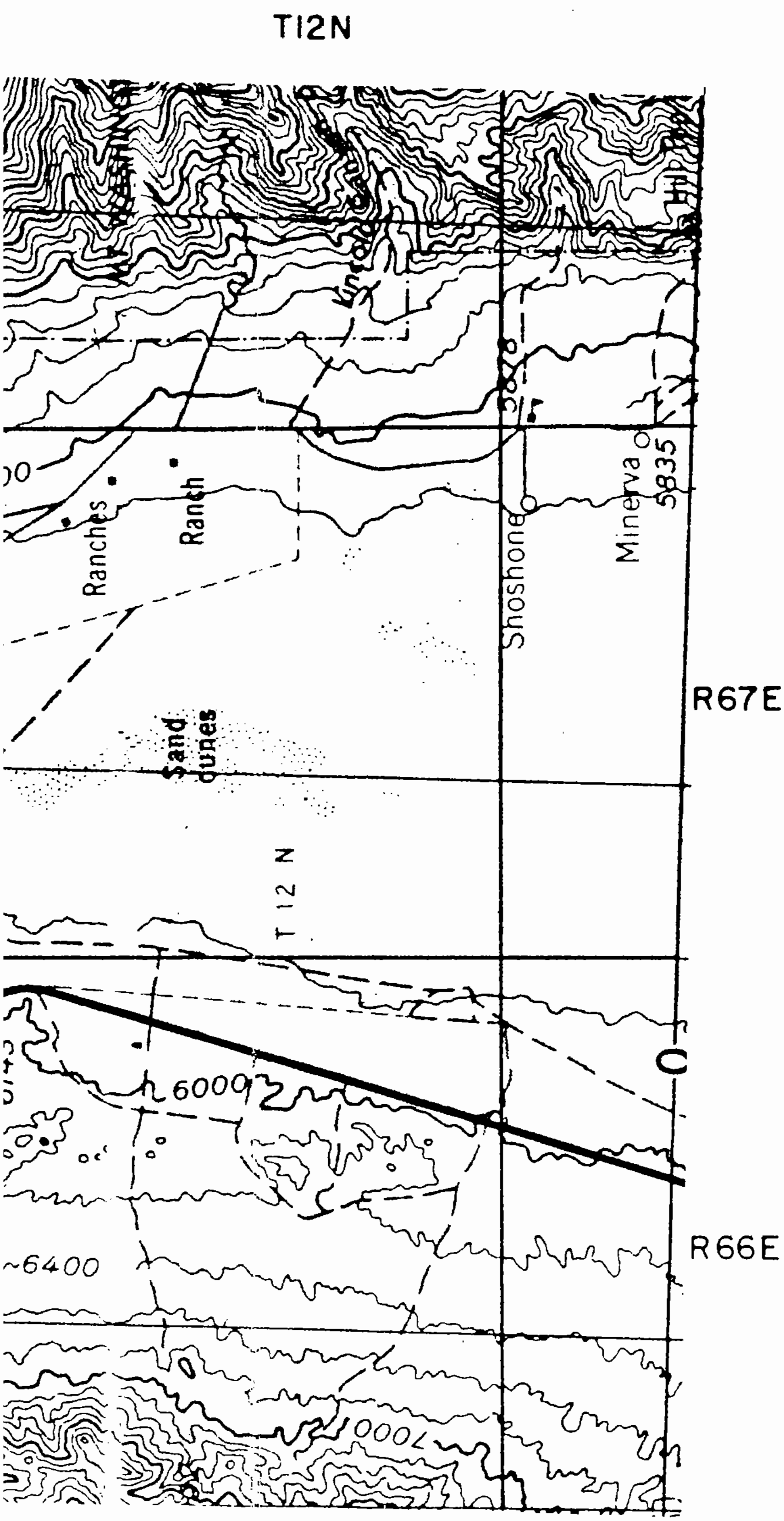
Warm Springs
9565
Corrals
6136
HUMBOLDT


Leeds, Hill and Jewett, Inc.
HIGH PEAK
Water
All Canyon
Corral Can
Ranch
RANGE
9623
6037
Goshute
Cabin
10458
9000
8000
7000
Ranch
NEVADA NORTHERN
5900
Greens
Ranch
5857
Ranch
STEPTOE VALLEY
93
16
5939
N
Water Canyon
50
93
Windmill
15
13



LEGEND

● Test Site

WHITE PINE POWER PROJECT

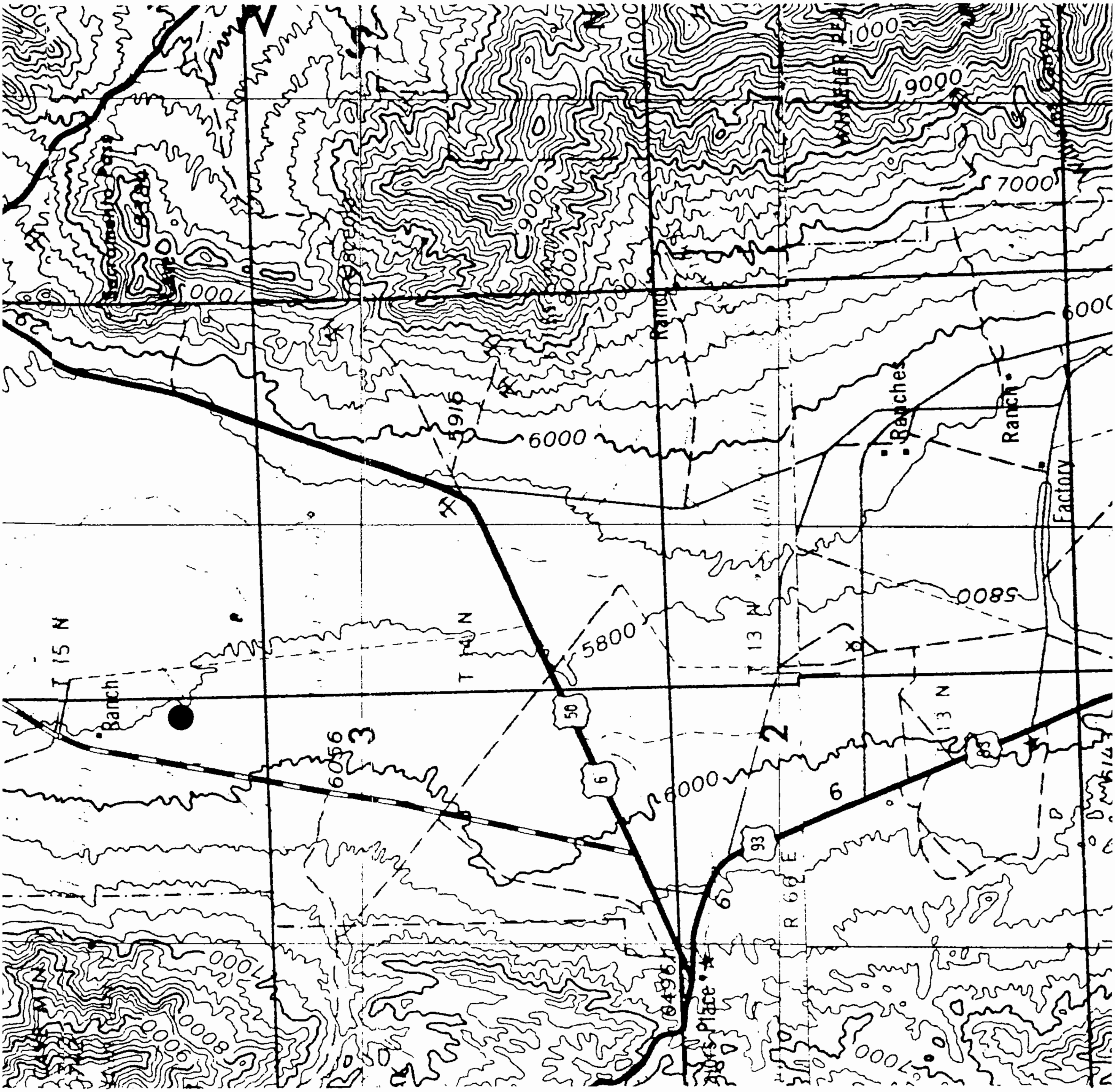
SPRING VALLEY

General Test Site Location Map

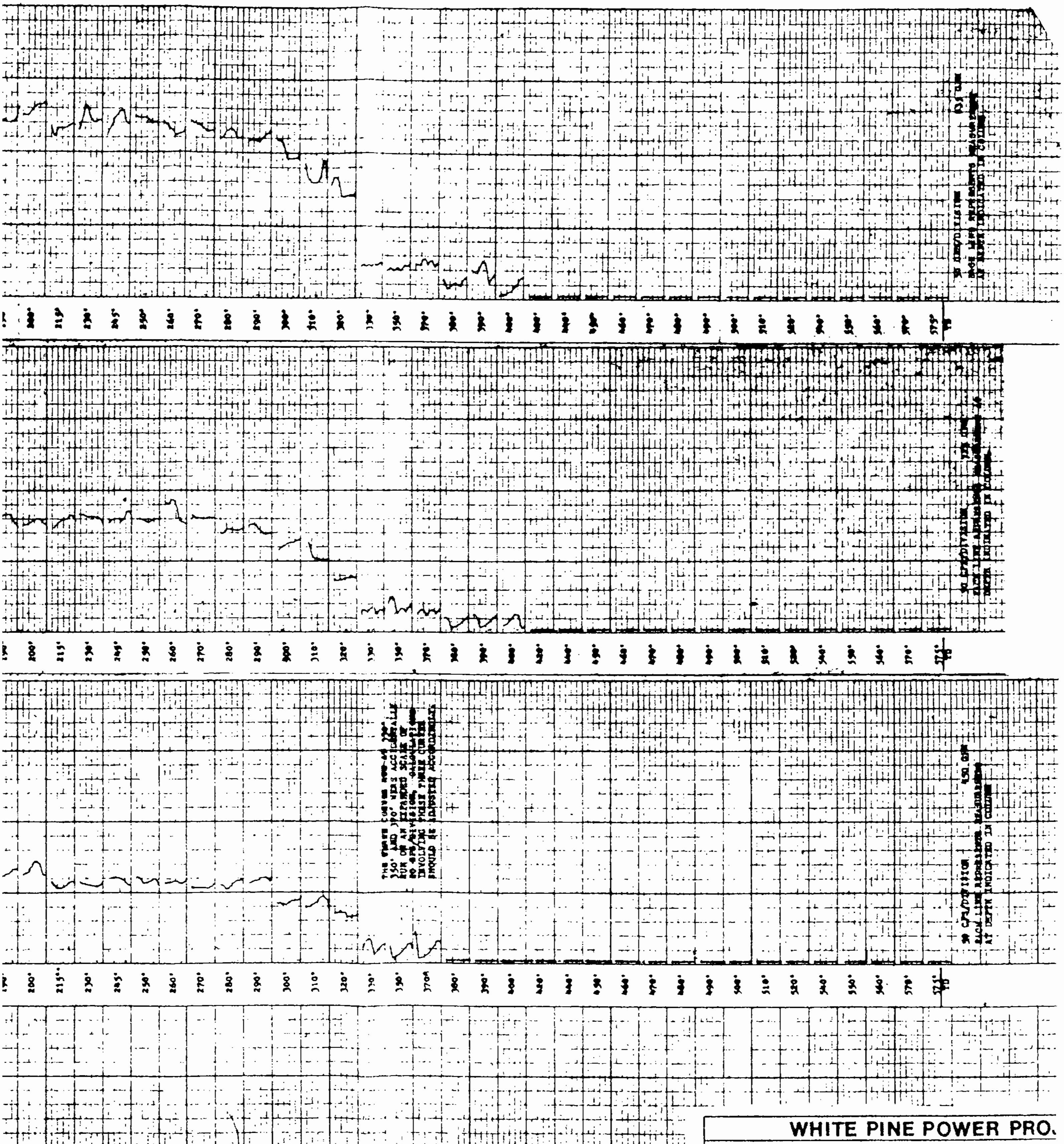
May 1983

Plate II

T14N



7000
9000
6000
5916
5800
6056
6000
Ranches
Ranch
Factory
Ranch
T 15 N
T 14 N
T 13 N
R 60 E
50
6
93
3
2



WHITE PINE POWER PRO

SPRING VALLEY

Flow Meter Test
at Various Discharge

December 1982

Plate III

Log Heading 1

Welenco (Water Well Survey), 2747 Sherwin Ave., #1, Ventura, CA 93003 · (805) 647-3281

COMPANY	...-LEEDS, HILL AND JEWETT, INC.
WELL	SPRING VALLEY #6A
FIELD	SPRING VALLEY
COUNTY	WHITE PINE
STATE	NEVADA
Location	SPRING VALLEY, HWY. 6 EAST OF HWY. 93
Permanent Datum	TOP OF CASING
Log Measured From	TOP OF CASING
Drilling Measured From	G.L.
Date	10-16-82
Type Fluid in Hole	WATER
Casing	12" @ 680'

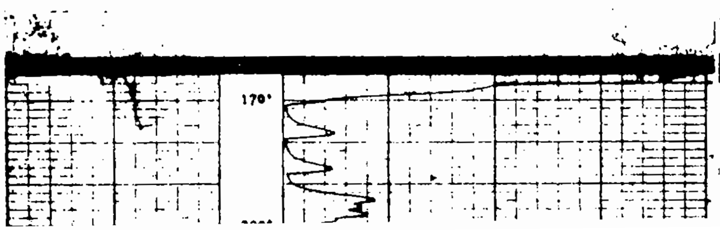
Type Log	Depth	Horizontal Scale
FLOWMETER		
(CONTINUOUS)	575'	50 CPS/DIVISION
(STEP-DOWN)	575'	50 CPS/DIVISION EXCEPT WHERE NOTED ON LOG

EQUIPMENT DATA

Run No.	Tool Type and No.	Pad Type	Tool Position	Other
ONE	FLOWMETER		FREE	

VERTICAL SCALE

S.P. MV	DEPTH (FT.)	LOG/SCALE	LOG/SCALE
		CONTINUOUS 50 CPS/DIVISION	VERTICAL SCALE 20 ft. per inch
		STEP-DOWN 50 CPS/DIVISION EXCEPT WHERE NOTED	450 GPM



Log Heading 2

Welenco (Water Well Survey), 2747 Sherwin Ave., #1, Ventura, CA 93003 · (805) 647-3281

COMPANY	...-LEEDS, HILL AND JEWETT, INC.
WELL	SPRING VALLEY #6A
FIELD	SPRING VALLEY
COUNTY	WHITE PINE
STATE	NEVADA
Location	SPRING VALLEY, HWY. 6 EAST OF HWY. 93
Permanent Datum	TOP OF CASING
Log Measured From	TOP OF CASING
Drilling Measured From	G.L.
Date	10-19-82
Casing	12" @ 680'

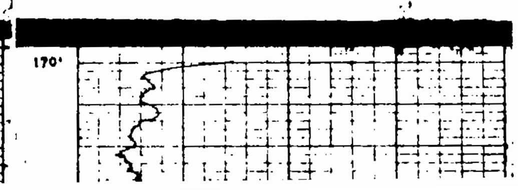
Type Log	Depth	Horizontal Scale
FLOWMETER		
(CONTINUOUS)	575'	50 CPS/DIVISION
(STEP-DOWN)	575'	50 CPS/DIVISION

EQUIPMENT DATA

Run No.	Tool Type and No.	Pad Type	Tool Position	Other
TWO	FLOWMETER		FREE	

SCALE

DEPTH (FT.)	LOG/SCALE	LOG/SCALE
	CONTINUOUS 50 CPS/DIVISION	VERTICAL SCALE 20 ft. per inch
	STEP-DOWN 50 CPS/DIVISION	725 GPM



Log Heading 3

Welenco (Water Well Survey), 2747 Sherwin Ave., #1, Ventura, CA 93003 · (805) 647-3281

COMPANY	...-LEEDS, HILL AND JEWETT, INC.
WELL	SPRING VALLEY #6A
FIELD	SPRING VALLEY
COUNTY	WHITE PINE
STATE	NEVADA
Location	SPRING VALLEY, HWY. 6 EAST OF HWY. 93
Permanent Datum	TOP OF CASING
Log Measured From	TOP OF CASING
Drilling Measured From	G.L.
Date	10-19-82
Casing	12" @ 680'

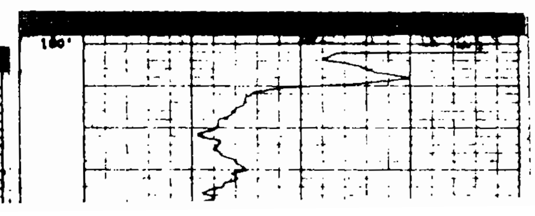
Type Log	Depth	Horizontal Scale
FLOWMETER		
(CONTINUOUS)	575'	50 CPS/DIVISION
(STEP-DOWN)	575'	50 CPS/DIVISION

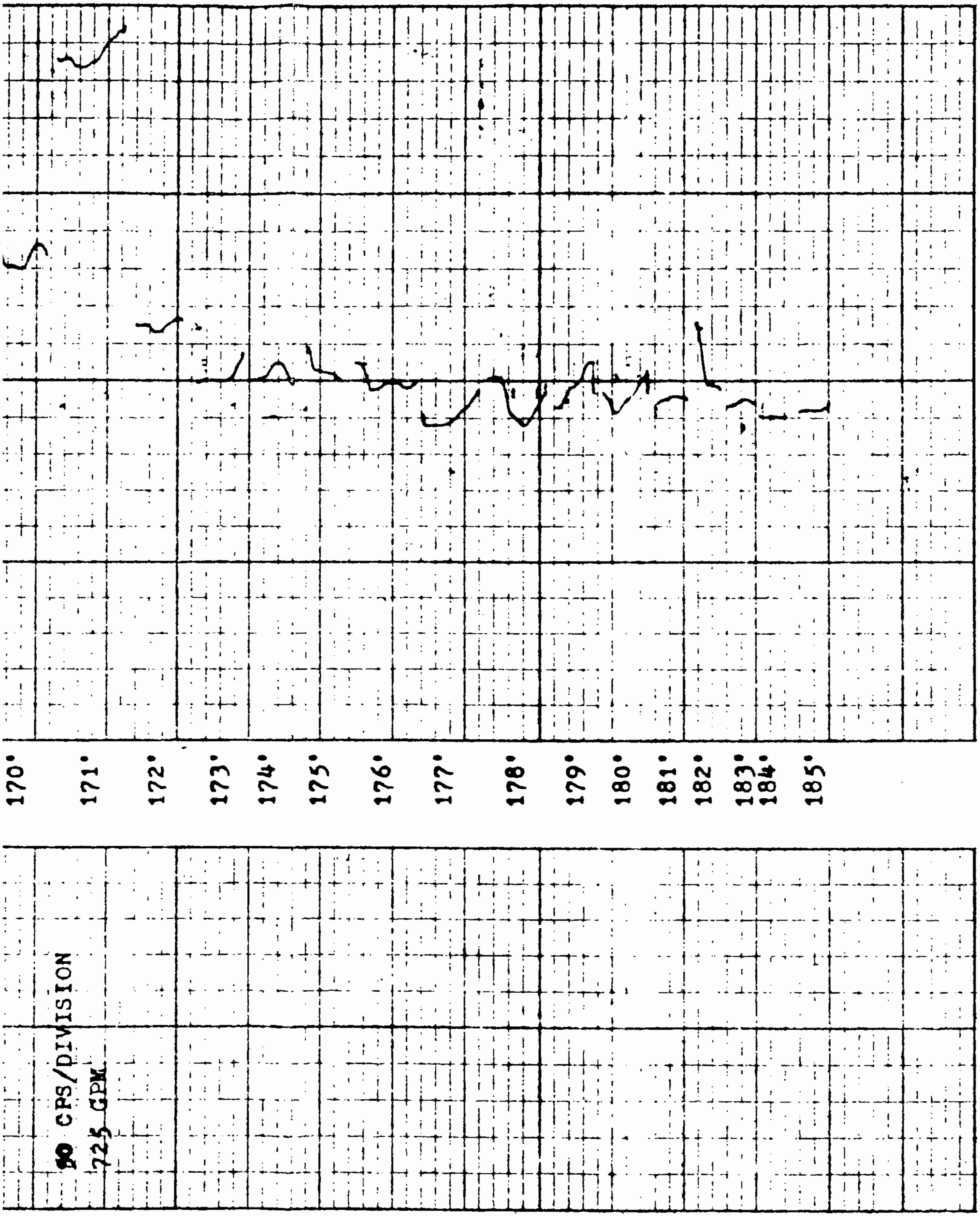
EQUIPMENT DATA

Run No.	Tool Type and No.	Pad Type	Tool Position	Other
THREE	FLOWMETER		FREE	

SCALE

DEPTH (FT.)	LOG/SCALE	LOG/SCALE
	CONTINUOUS 50 CPS/DIVISION	VERTICAL SCALE 20 ft. per inch
	STEP-DOWN 50 CPS/DIVISION	975 GPM





WHITE PINE POWER PROJECT

SPRING VALLEY

Flow Meter Test

Between 168 and 185 Feet

December 1982

Plate IV

water well surveys

(805) 647-3281

2747 Sherwin Ave., #1, Ventura, CA 93003

COMPANY LEEDSHILL

WELL SPRING VALLEY #2A

FIELD SPRING VALLEY

COUNTY WHITE PINE STATE NEVADA

Location SPRING VALLEY

Sec ____ Twp ____ Rge ____

Other Services

COMPANY ____ WELL ____ FIELD ____ County ____ State ____

Permanent Datum ____ Elev ____ Elev K B ____

Log Measured From 168' to 185' Ft Above Perm Datum D F ____

Drilling Measured From ____ G L ____

Date	10-15-82			
Run No				
Depth — Driller				
Depth — Logger				
Btm Log Inter	185'			
Top Log Inter	168'			
Casing — Driller	12" @	@	@	@
Casing — Logger				
Bit Size				
Type Fluid in Hole	WATER			
Dens I Visc	I	I	I	I
pH I Fluid Loss	I ml	I ml	I ml	I ml
Source of Sample				
R_m @ Meas Temp	@ °F	@ °F	Cm	@ °F
R_{mf} @ Meas Temp	@ °F	@ °F	@ °F	@ °F
R_{mc} @ Meas Temp	@ °F	@ °F	@ °F	@ °F
Source R_{mf} R_{mc}	I	I	I	I
R_m @ BHT	@ °F	@ °F	@ °F	@ °F
Time Since Circ.				
Max. Rec. Temp.	°F @	°F @	°F @	°F @
Equip. I Location	E-2 VENTURA			
Recorded By	BRIAN COSNER			
Witnessed By	ERIC WEINSTOCK			

Service Ticket No.

Remarks

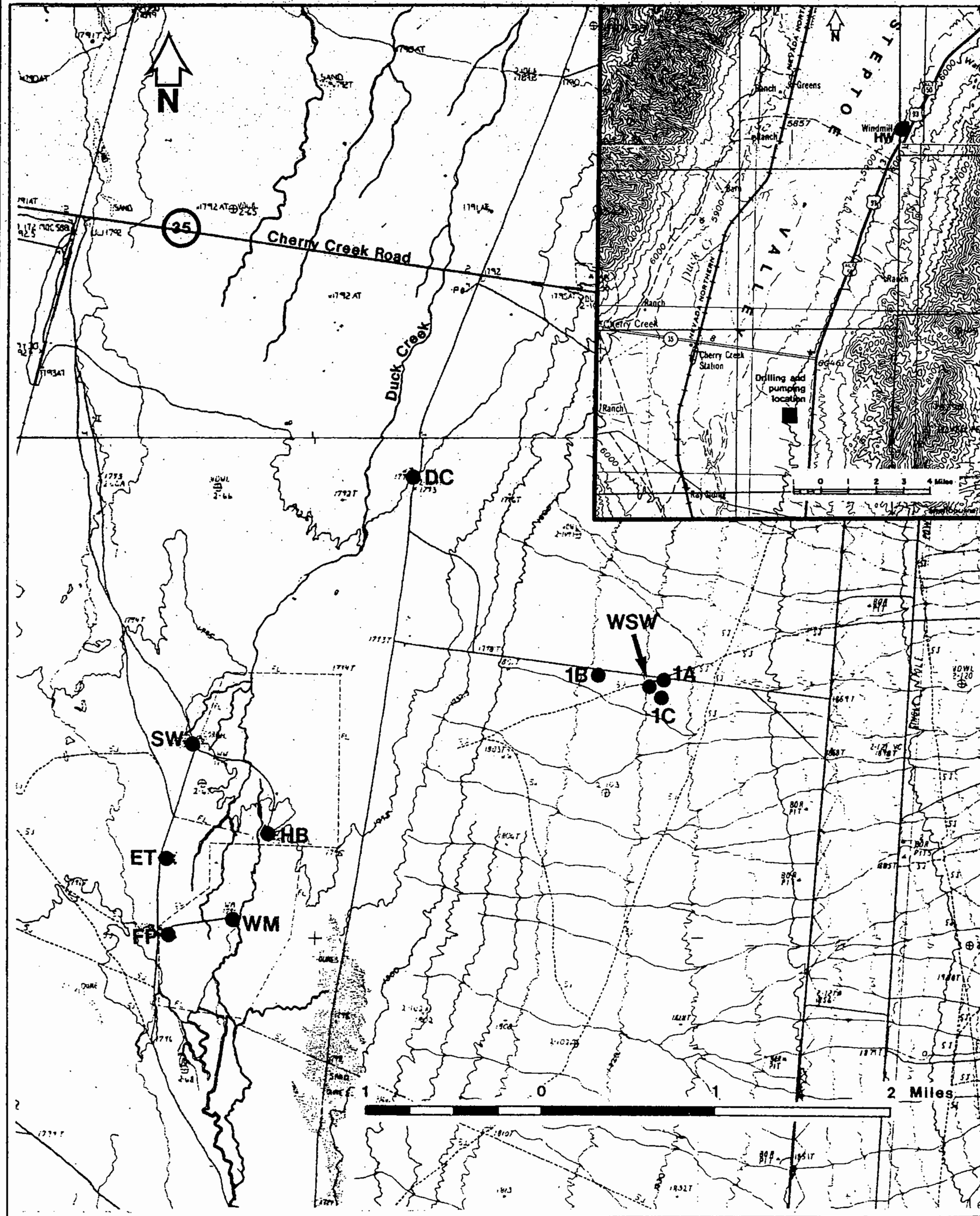
Change in Mud Type or Additional Samples

Date	Sample No.	Depth — Driller	Type Fluid in Hole	Dens	Visc	ph	Fluid Loss	Source of Sample
10-15-82			WATER				ml	
	—			—	—			

Type Log	Depth	Horizontal Scale
FLOWMETER		50 CPS/DIVISION

EQUIPMENT DATA

Leeds, Hill and Jewett, Inc.

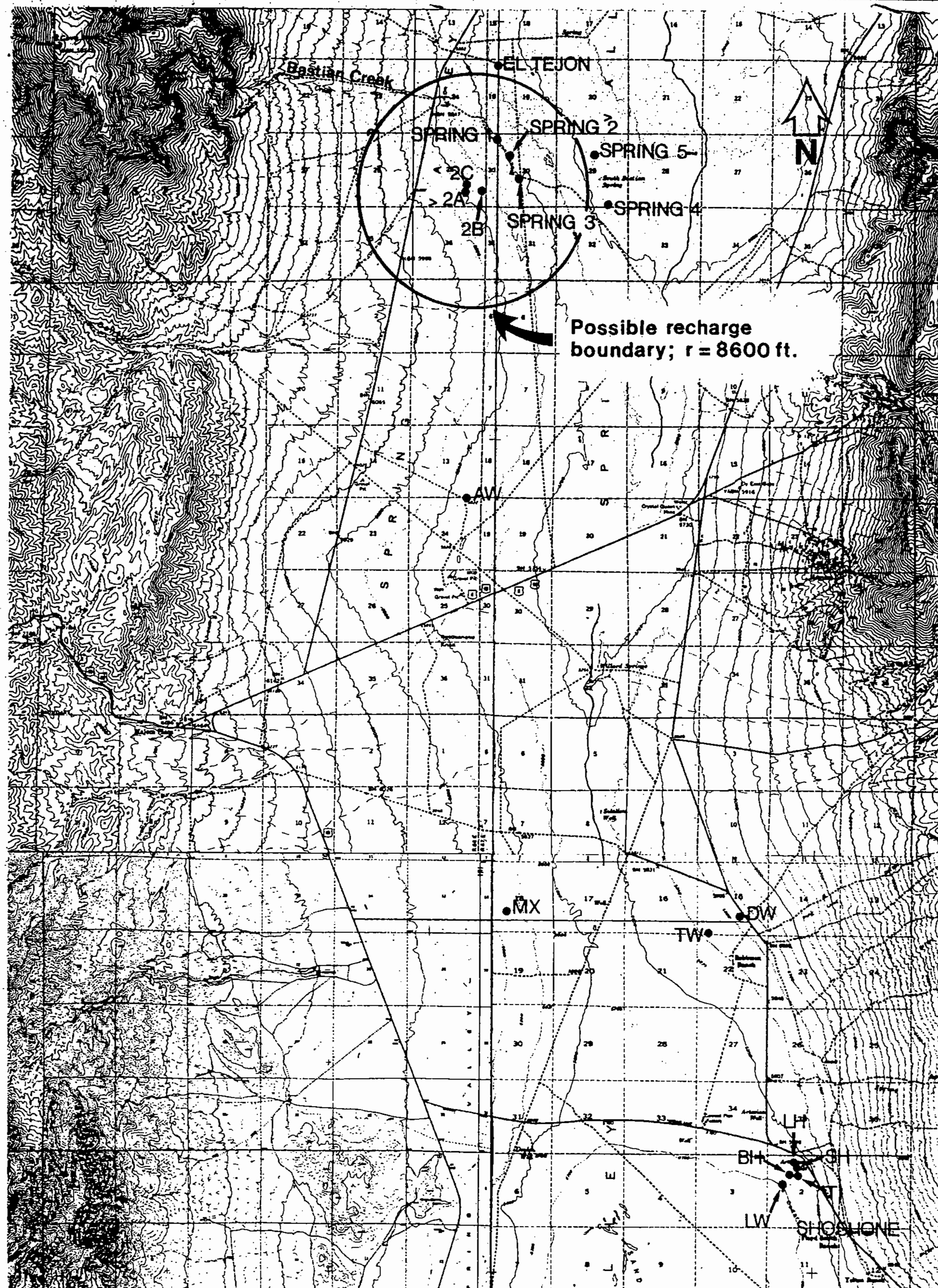


WHITE PINE POWER PROJECT

STEPTOE VALLEY
Monitoring Program
Location Map

Leeds, Hill and Jewett, Inc. December 1982

Plate V



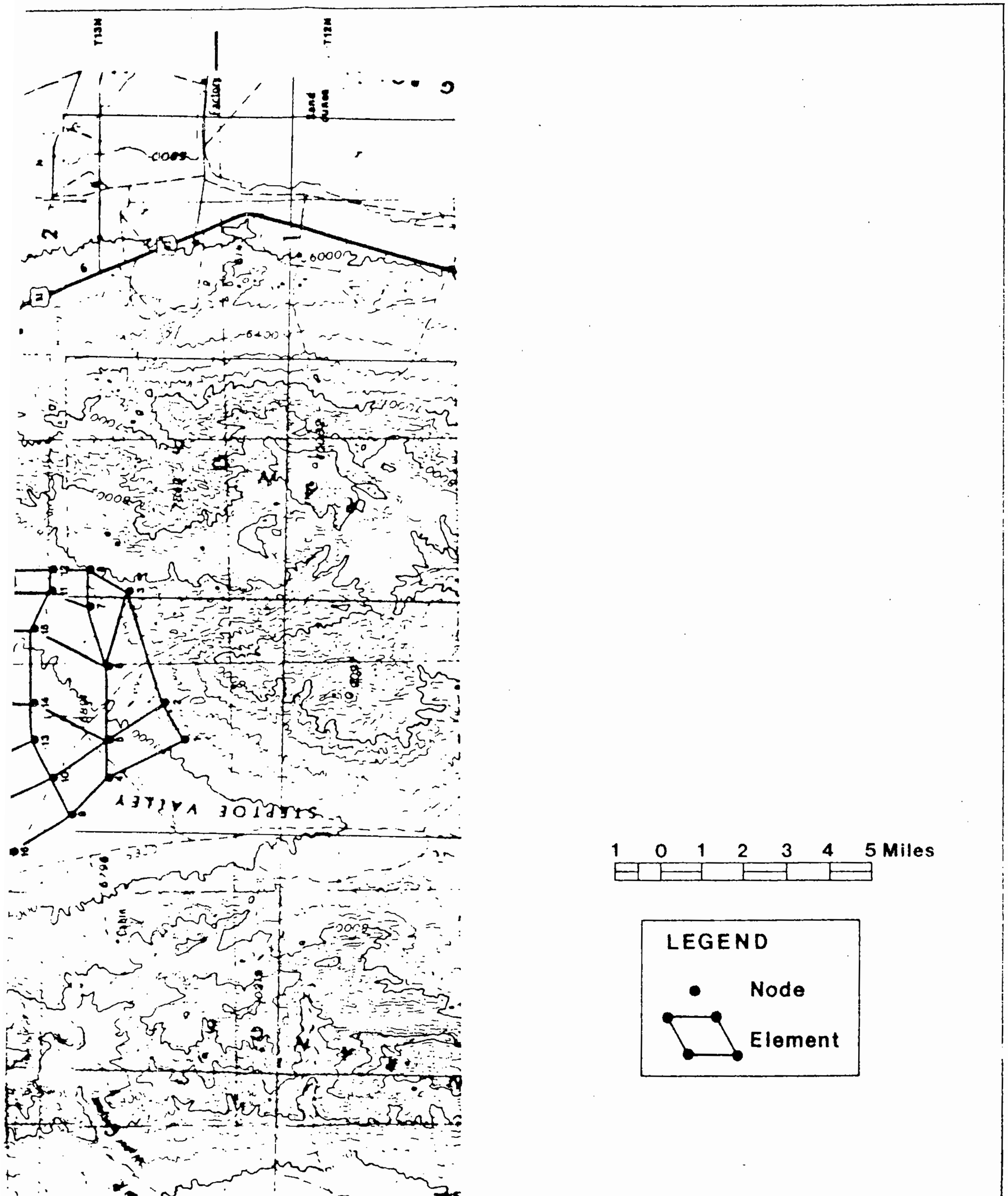
1 0 1 2 Miles

WHITE PINE POWER PROJECT

SPRING VALLEY
Monitoring Program
Location Map

Leeds, Hill and Jewett, Inc. December 1982

Plate VI



WHITE PINE POWER PROJECT

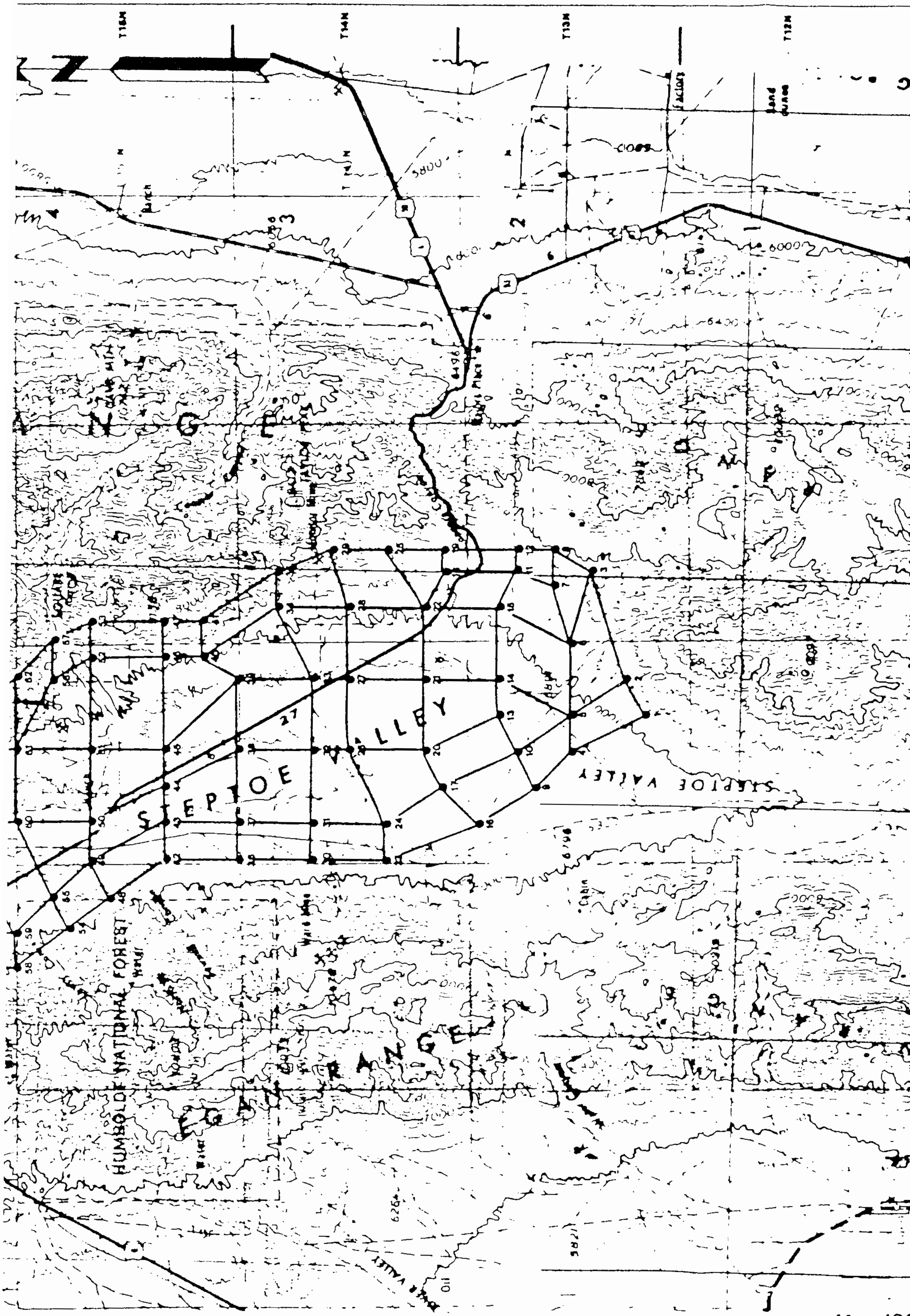
STEPTOE VALLEY

Groundwater Model

Nodes and Elements

May 1983

Plate VII



1 0

WHITE

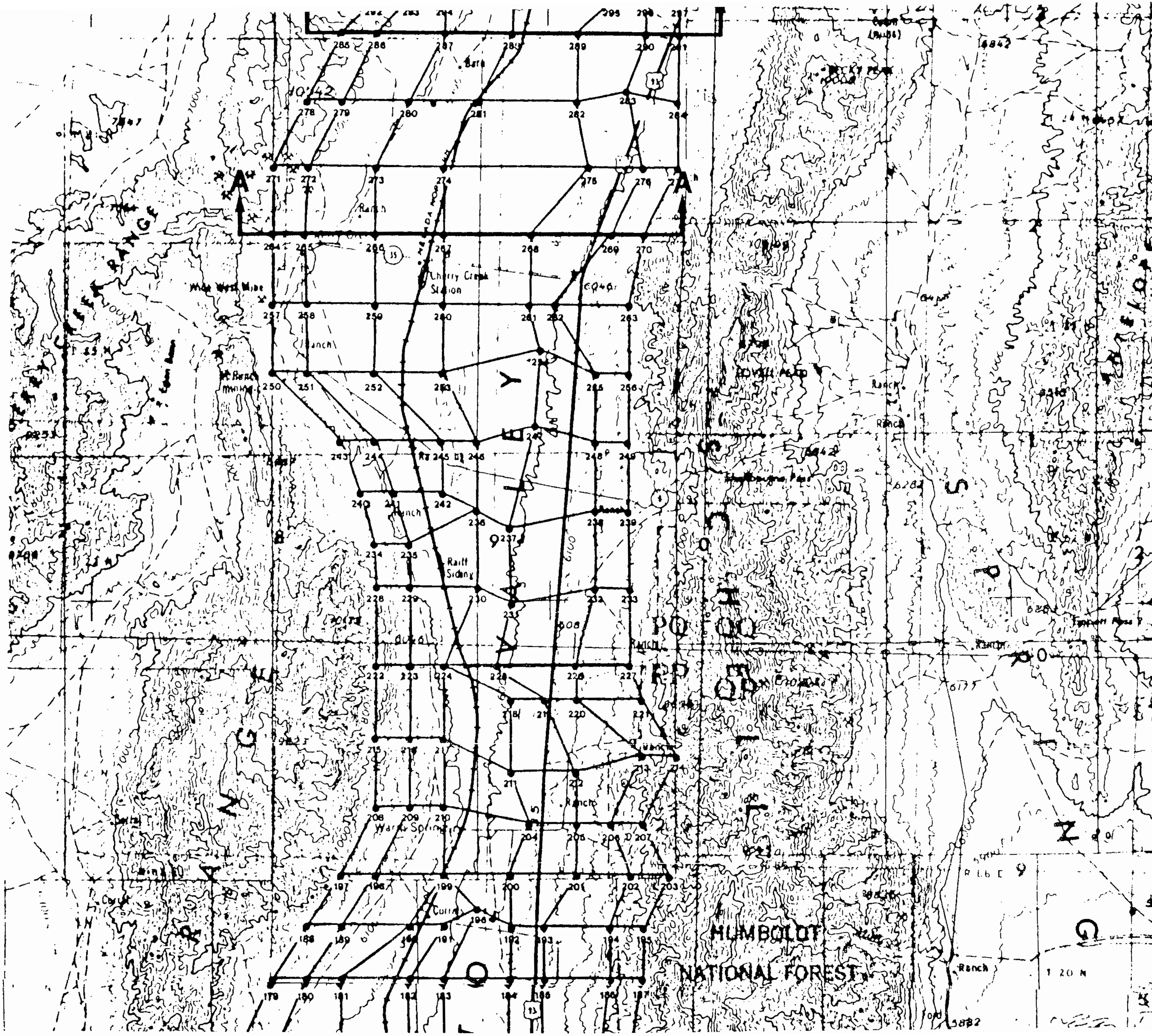
S

Gi

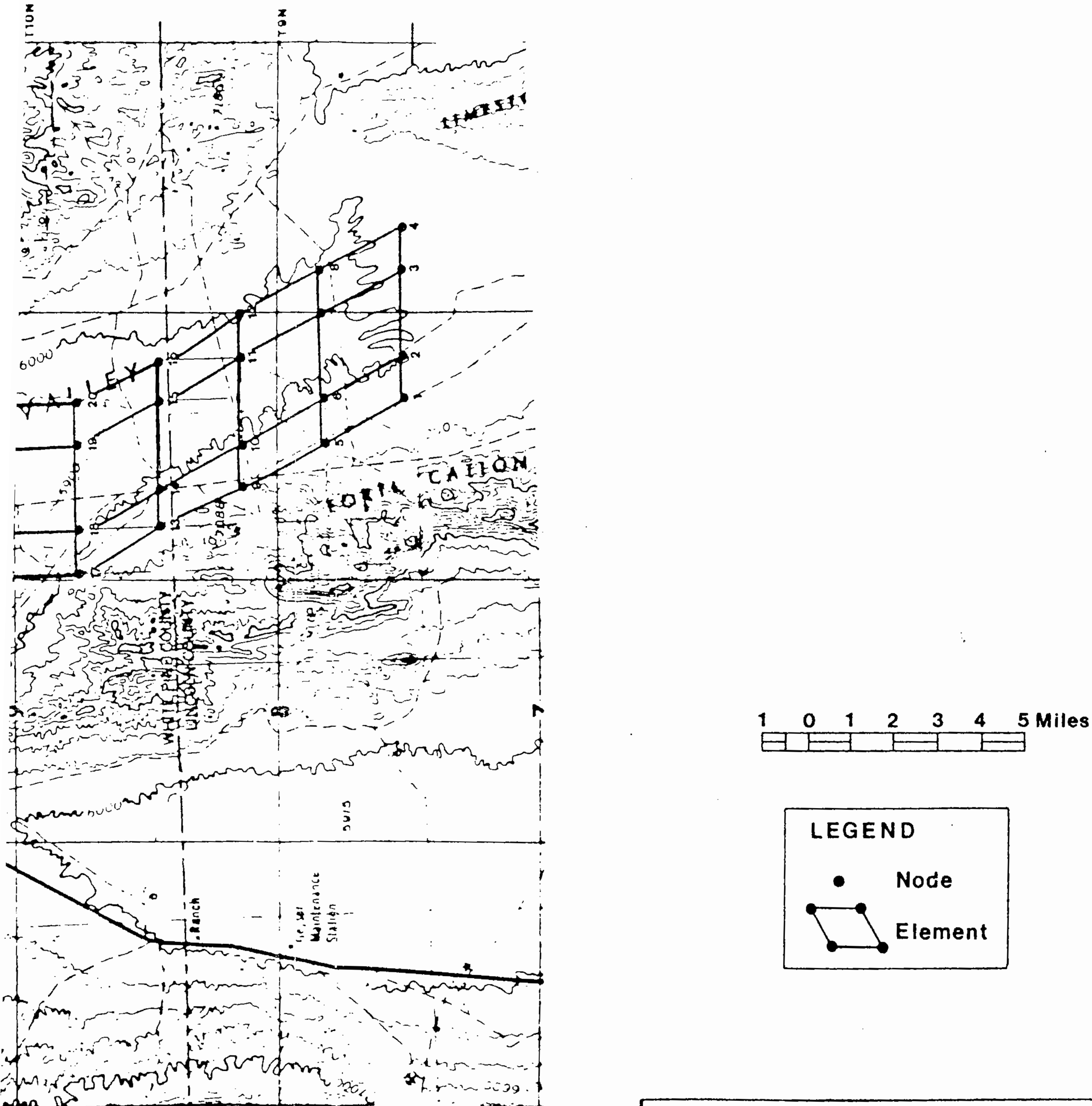
No

May 1983

Plate VII



HUMBOLDT
NATIONAL FOREST
Cherry Creek
Station
Raift
Siding
Warm Spring
T24N
T23N
T22N
T21N
T20N



WHITE PINE POWER PROJECT

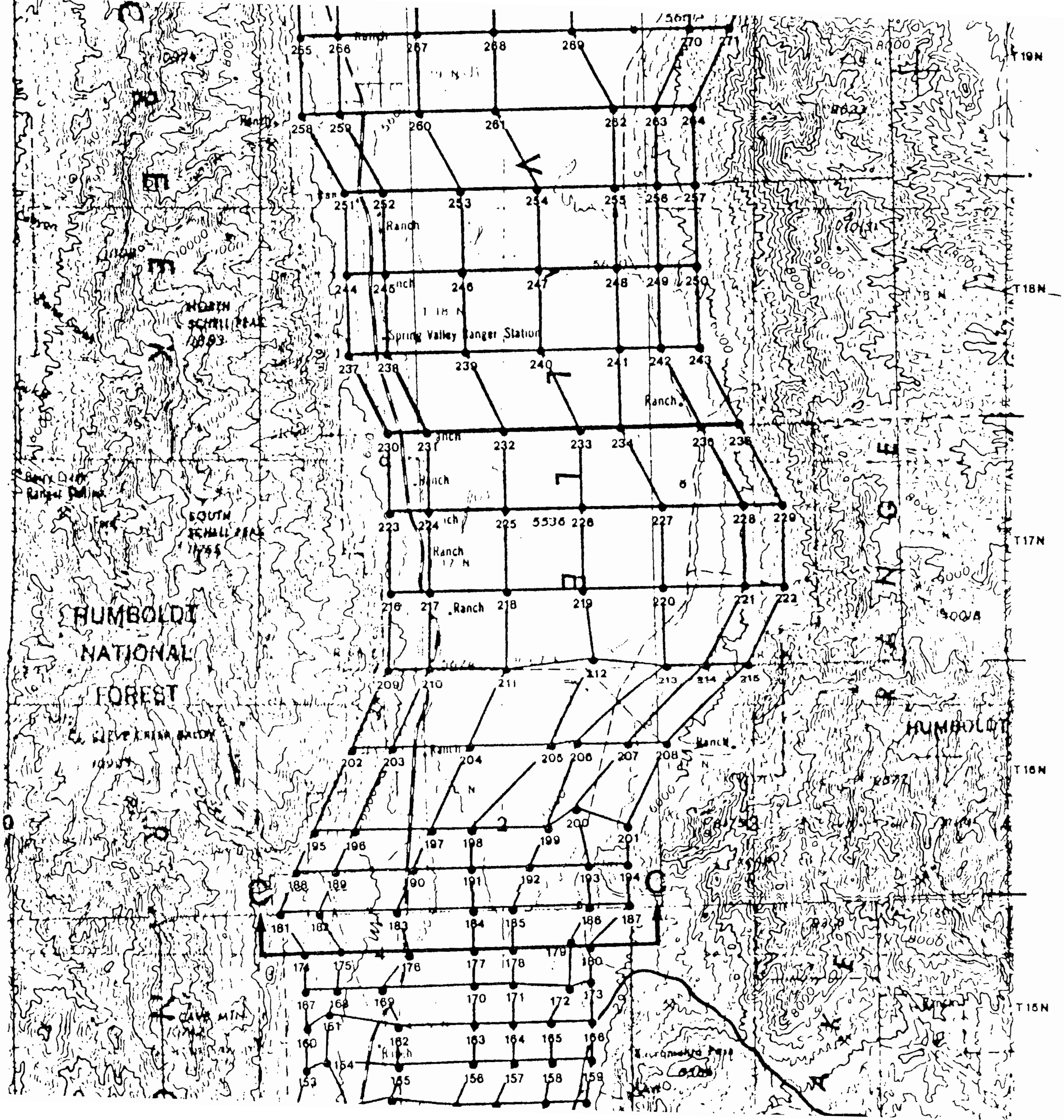
SPRING VALLEY

Groundwater Model

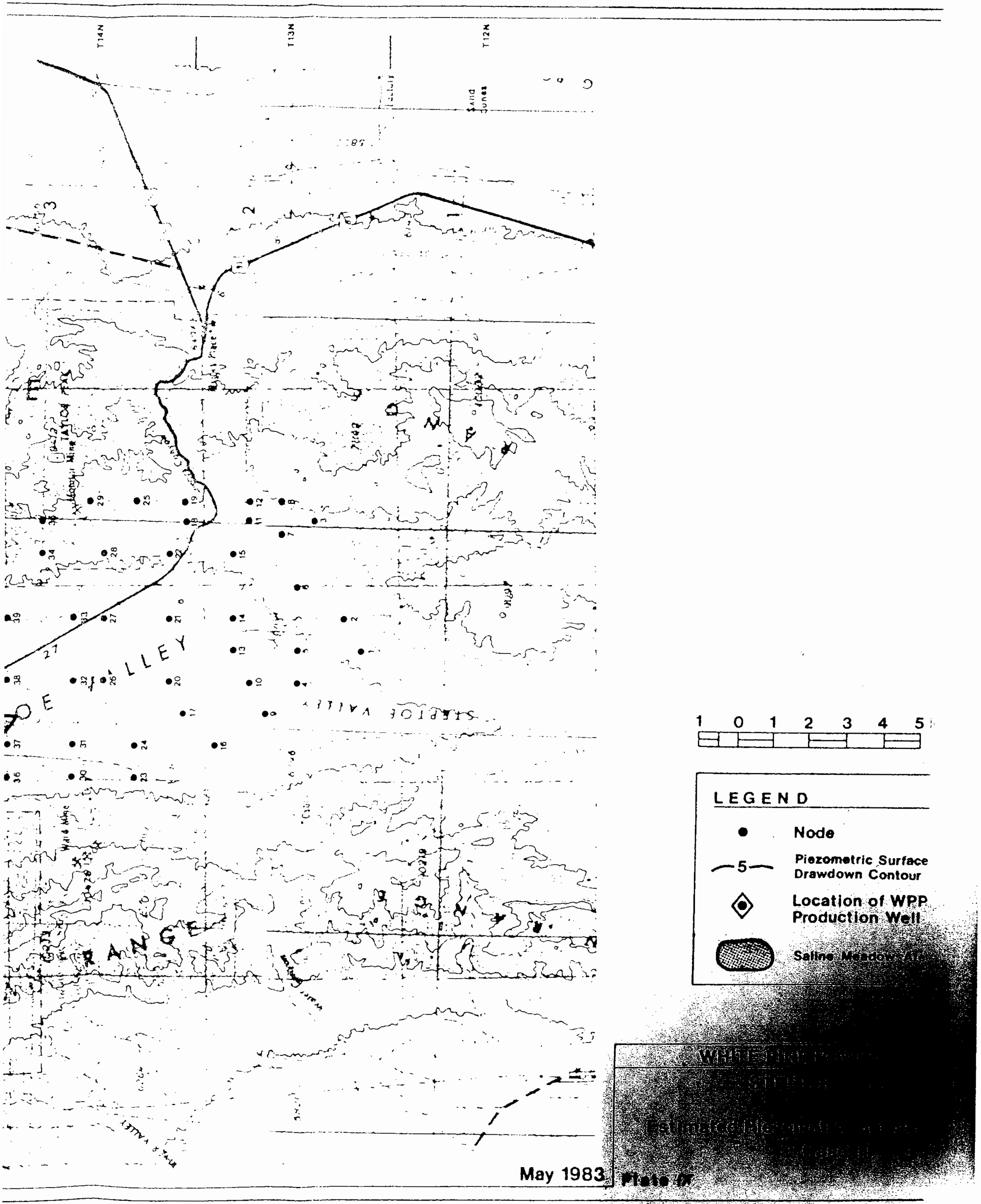
Nodes and Elements

May 1983

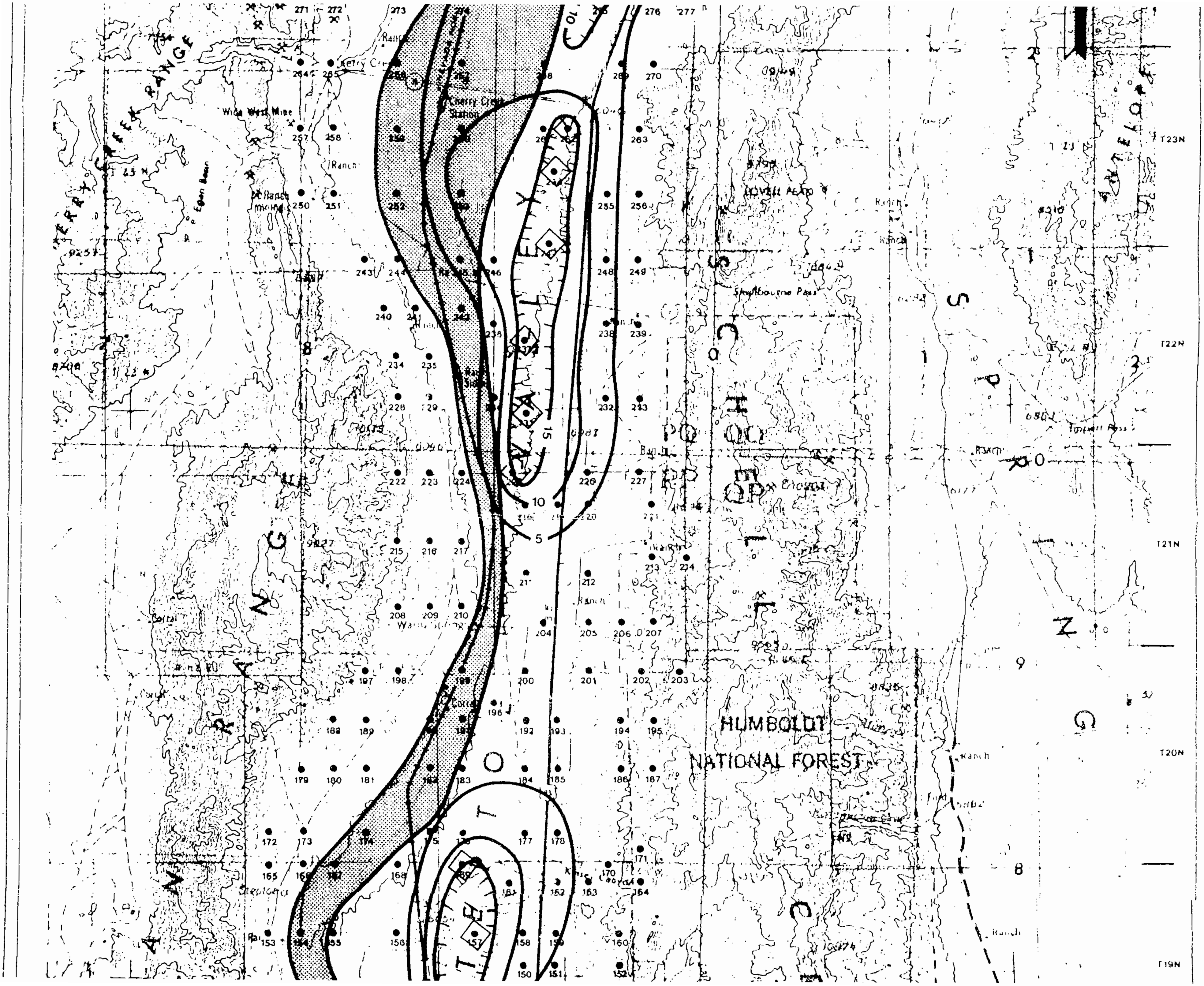
Plate VIII



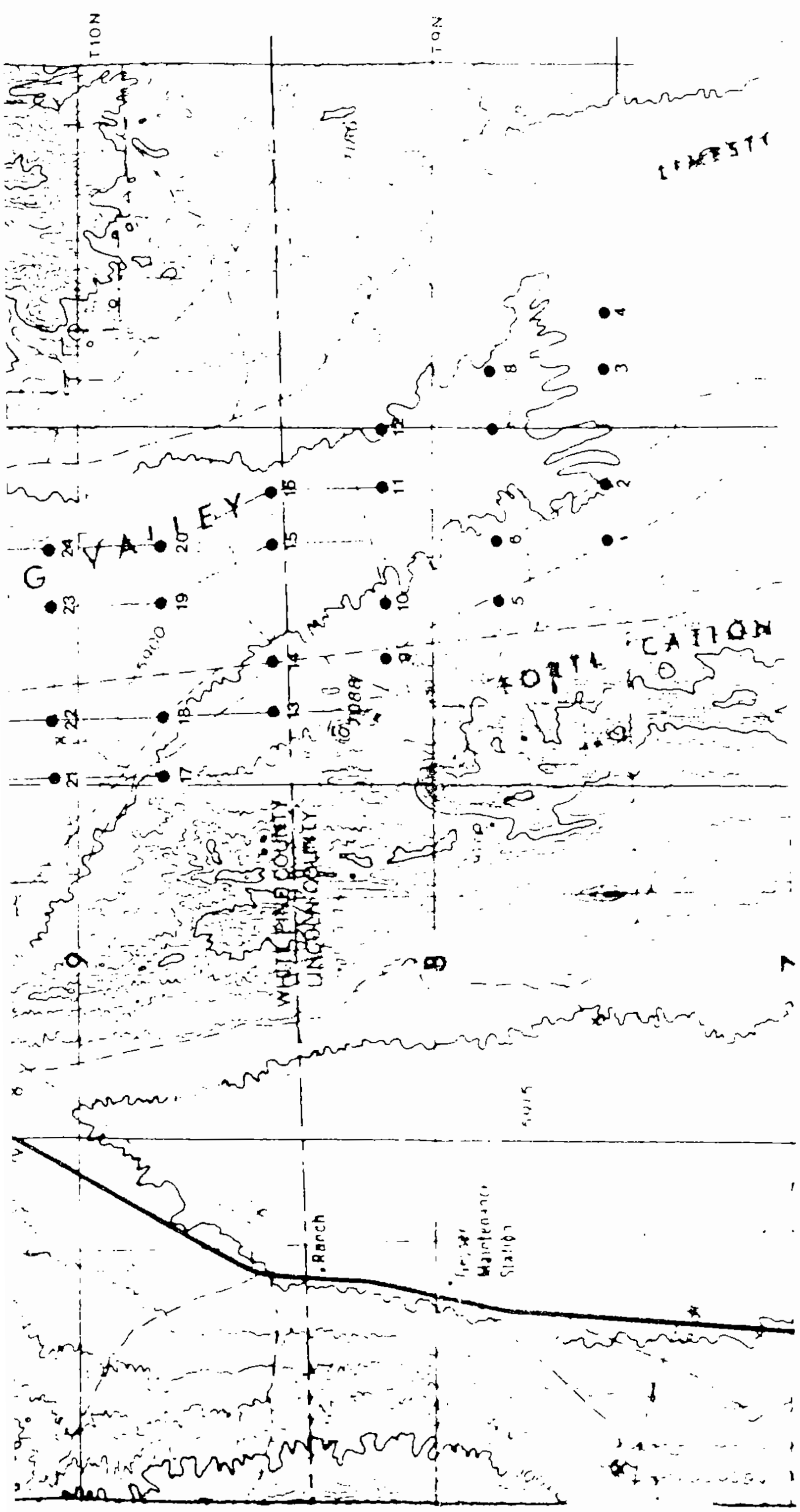
T19N
T18N
T17N
T16N
T15N
HUMBOLDT
NATIONAL
FOREST
HUMBOLDT
NORTH
SCHELL PEAK
SOUTH
SCHELL PEAK
Spring Valley Ranger Station
Ranch
5536



T14N
T13N
T12N
Sand Dunes
STEPTOE VALLEY
RANGE
1 0 1 2 3 4 5
LEGEND
Node
5
Piezometric Surface Drawdown Contour
Location of WPP Production Well
Saline Meadow
WHITE
May 1983
Plate



T23N
T22N
T21N
T20N
T19N
HUMBOLDT
NATIONAL FOREST
Cherry Creek Station
Wide West Mine
Schellbourne Pass
Tippett Pass
15
10
5



1 0 1 2 3 4

LEGEND

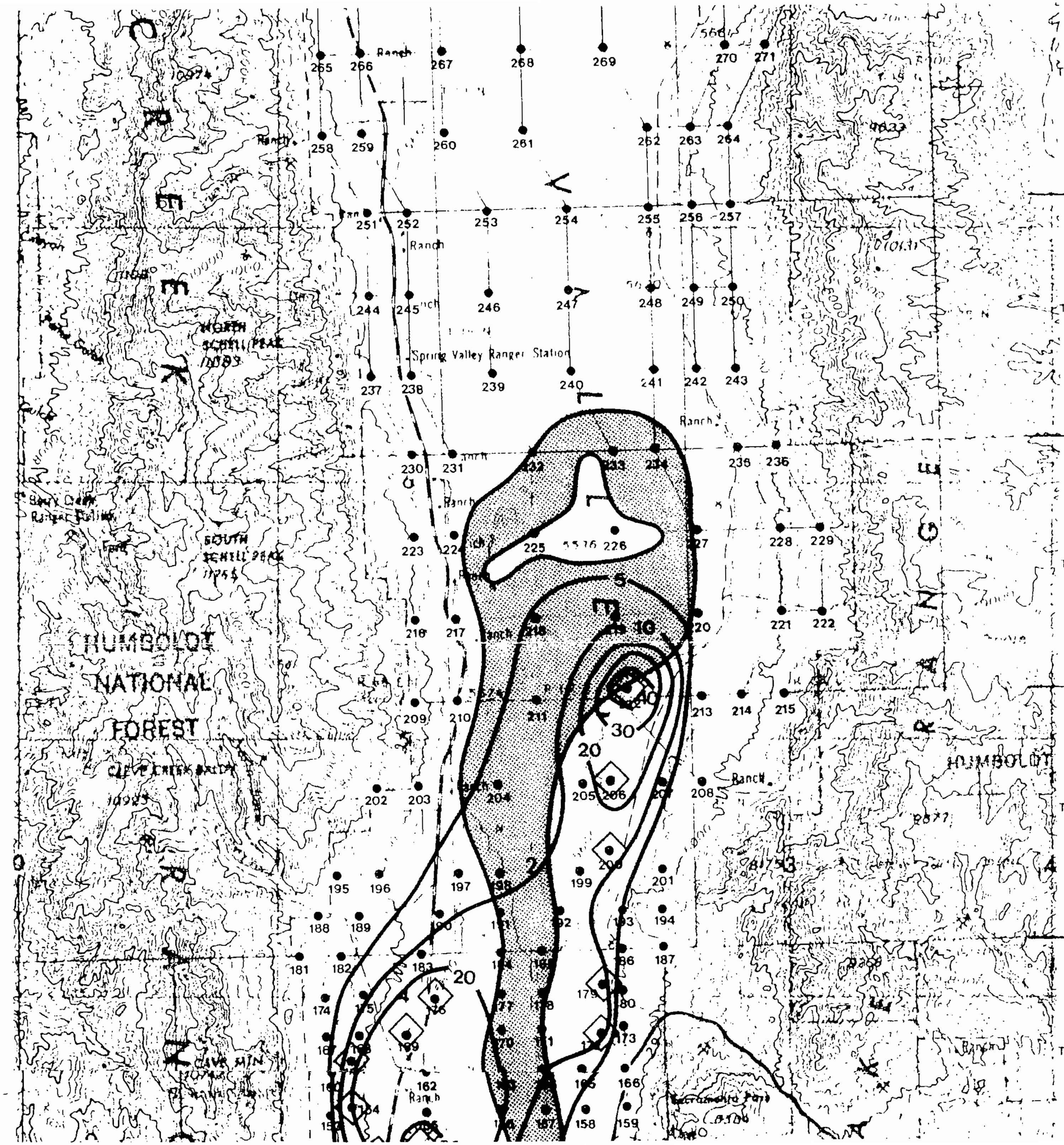
- ● Node
- —5— Piezometric Surfac Drawdown Contou
- ◈ Location of WP Production Wel
- Saline Meadow Ar

WHITE PINE POWER P

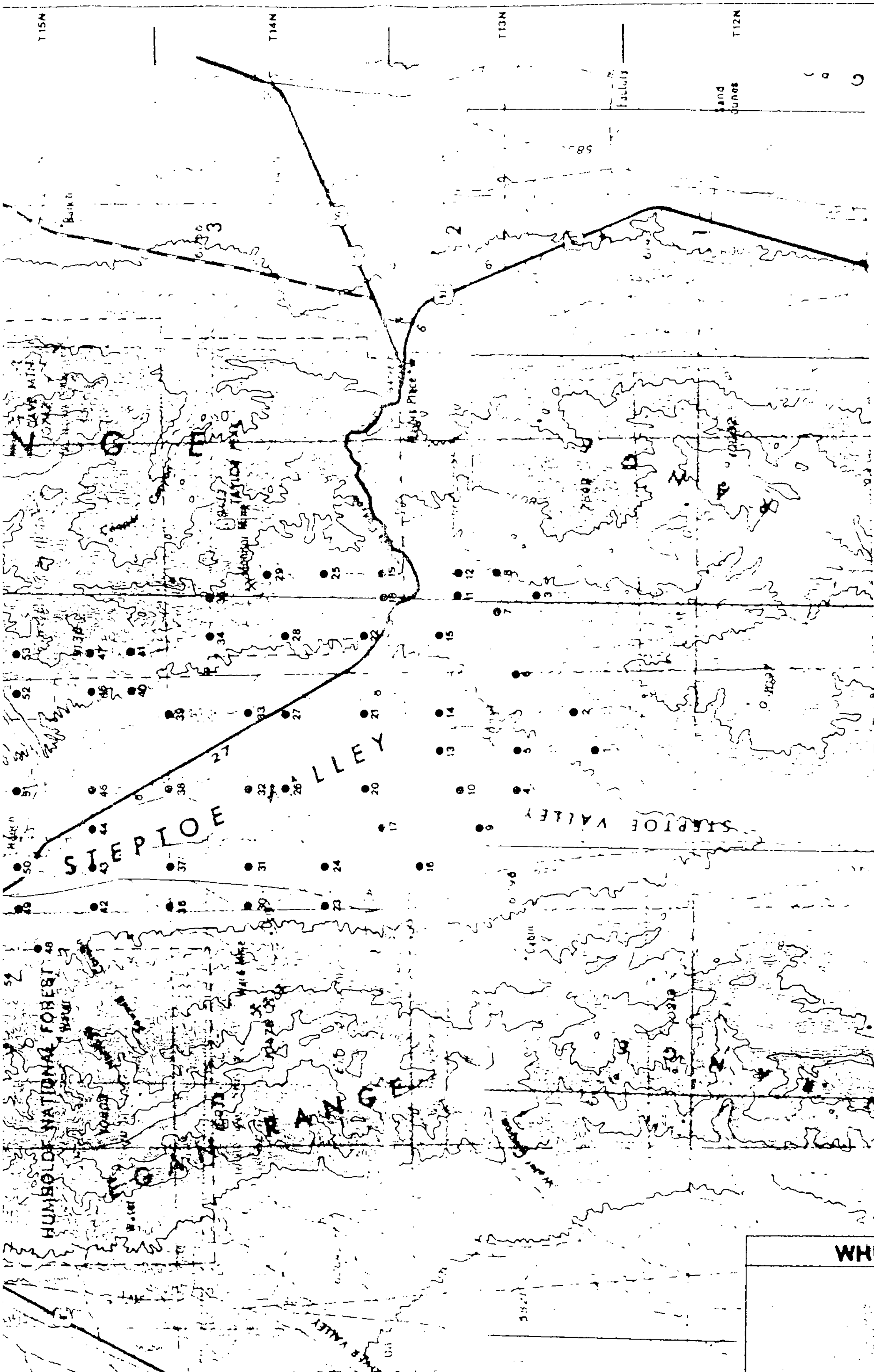
SPRING VALL

Estimated Piezometric Surf
Case II

May 1983 | Plate X



T19N
T18N
T17N
T16N
T15N
Spring Valley Ranger Station
Ranch
HUMBOLDT
NATIONAL
FOREST
HUMBOLDT
NORTH SCHELL PEAK
SOUTH SCHELL PEAK
5
10
20
30
40
20



NOTE:

These piezometri
drawdowns were c
under the hypoth
improbable and h
realistic assump
zero recharge. T
sumptions are no
sentative of exi
conditions, and
pict anticipated

1 0 1 2 3

LEGEND

- ● Node
- —5— Piezometr Drawdowr
- ◈ Locatior Producti

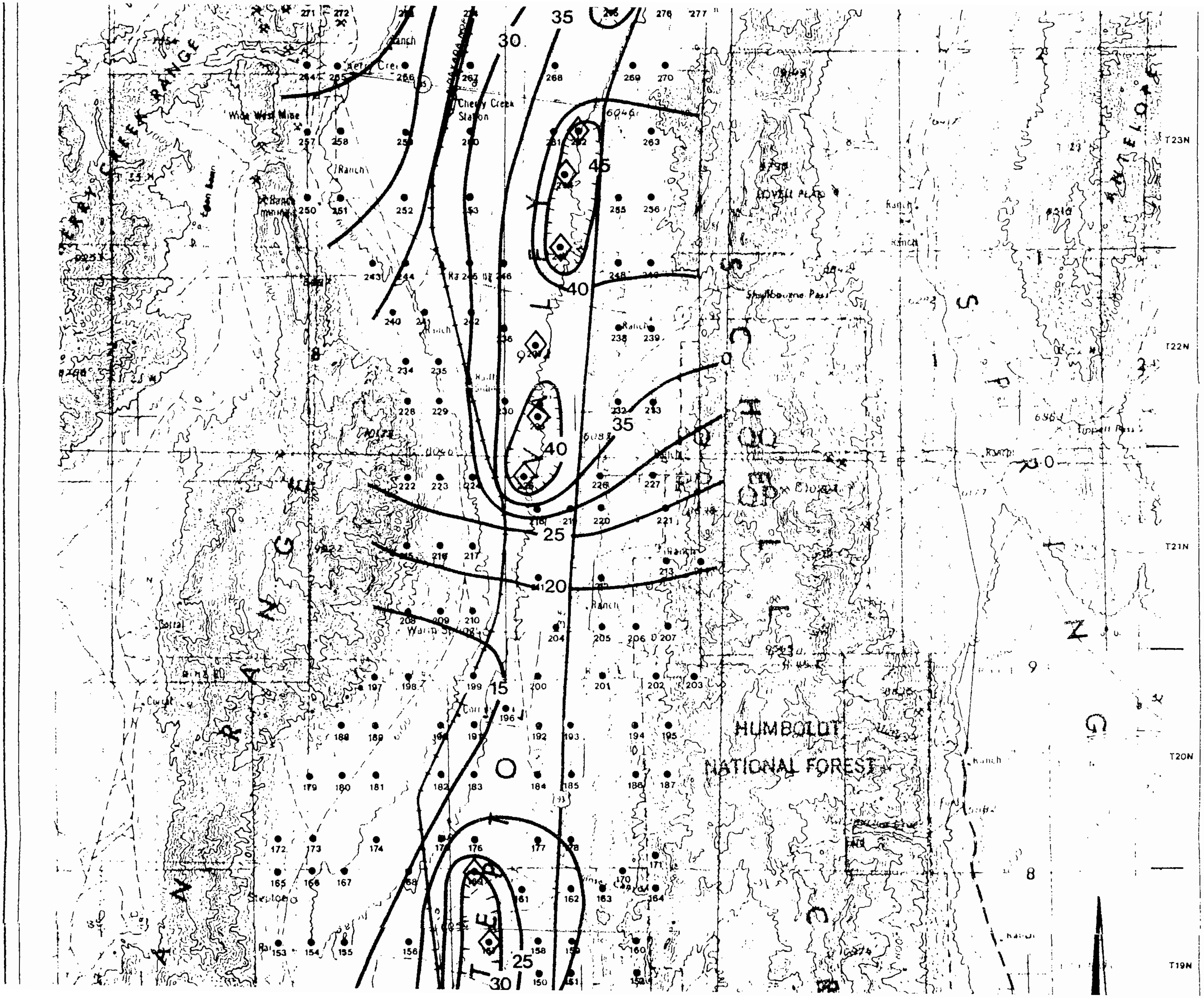
May 1983

WHITE PINE POWER PRO

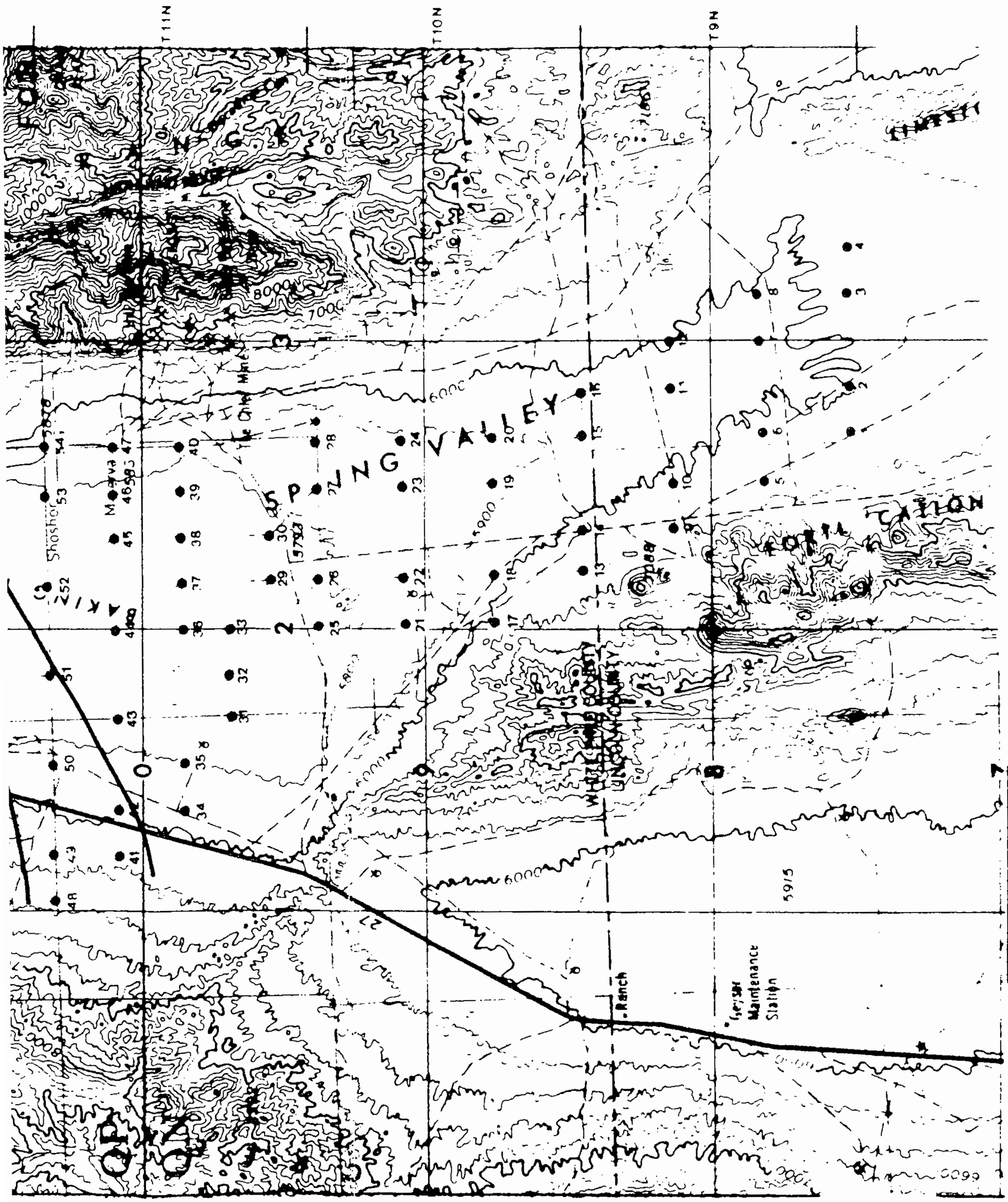
STEPTOE VALLEY

Sensitivity Analys Zero Recharge Ca

Plate XI



HUMBOLDT
NATIONAL FOREST
Cherry Creek Station
Wide West Mine
Steptoe
Warm Springs
T23N
T22N
T21N
T20N
T19N



NOTE:

These p drawdow under th improbal realist zero rec sumptio sentativ conditio pict an

LEGE

—5—

1 0 1 2 3 4 5 Miles

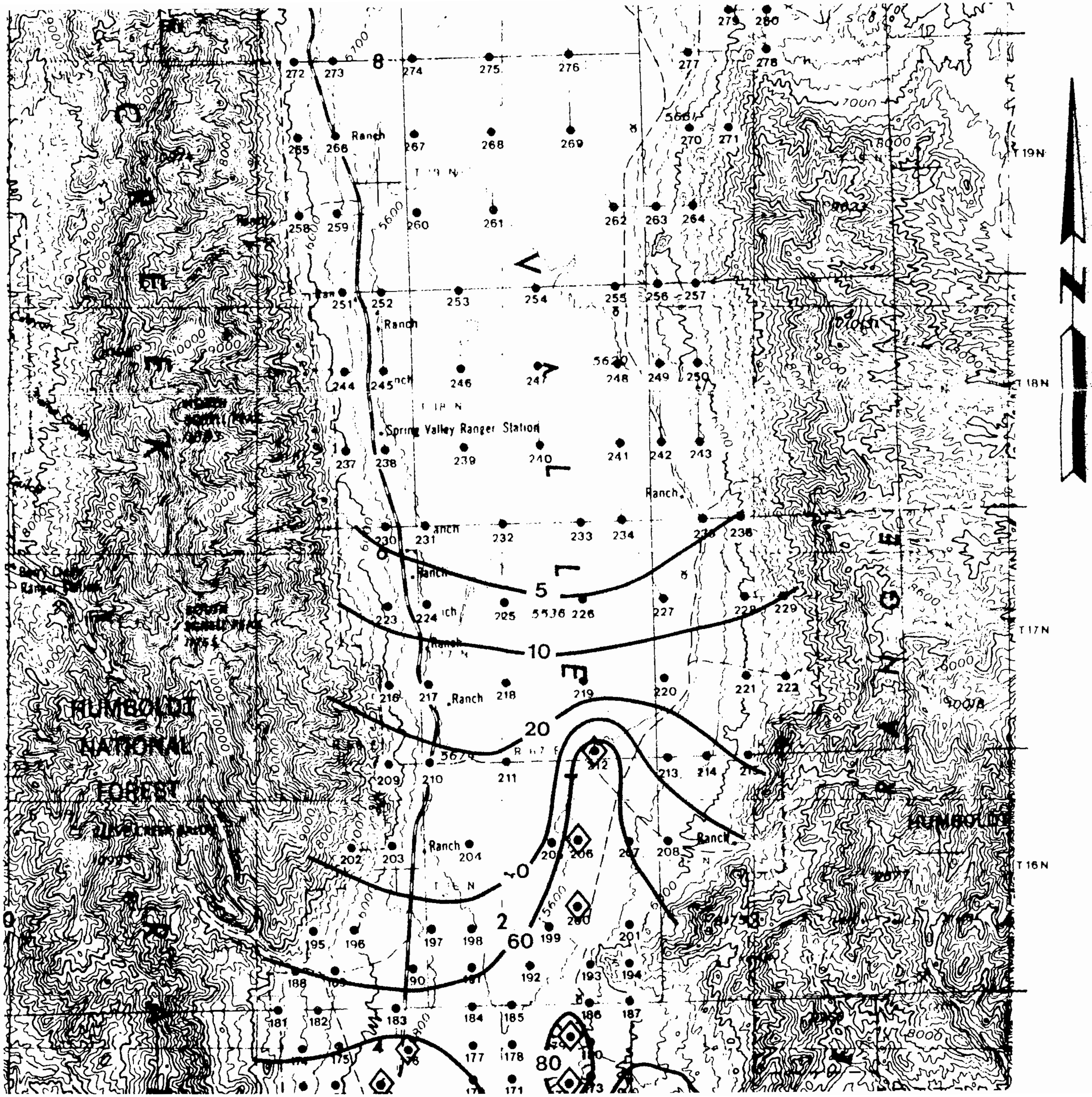
May 1983

WHITE PINE POW

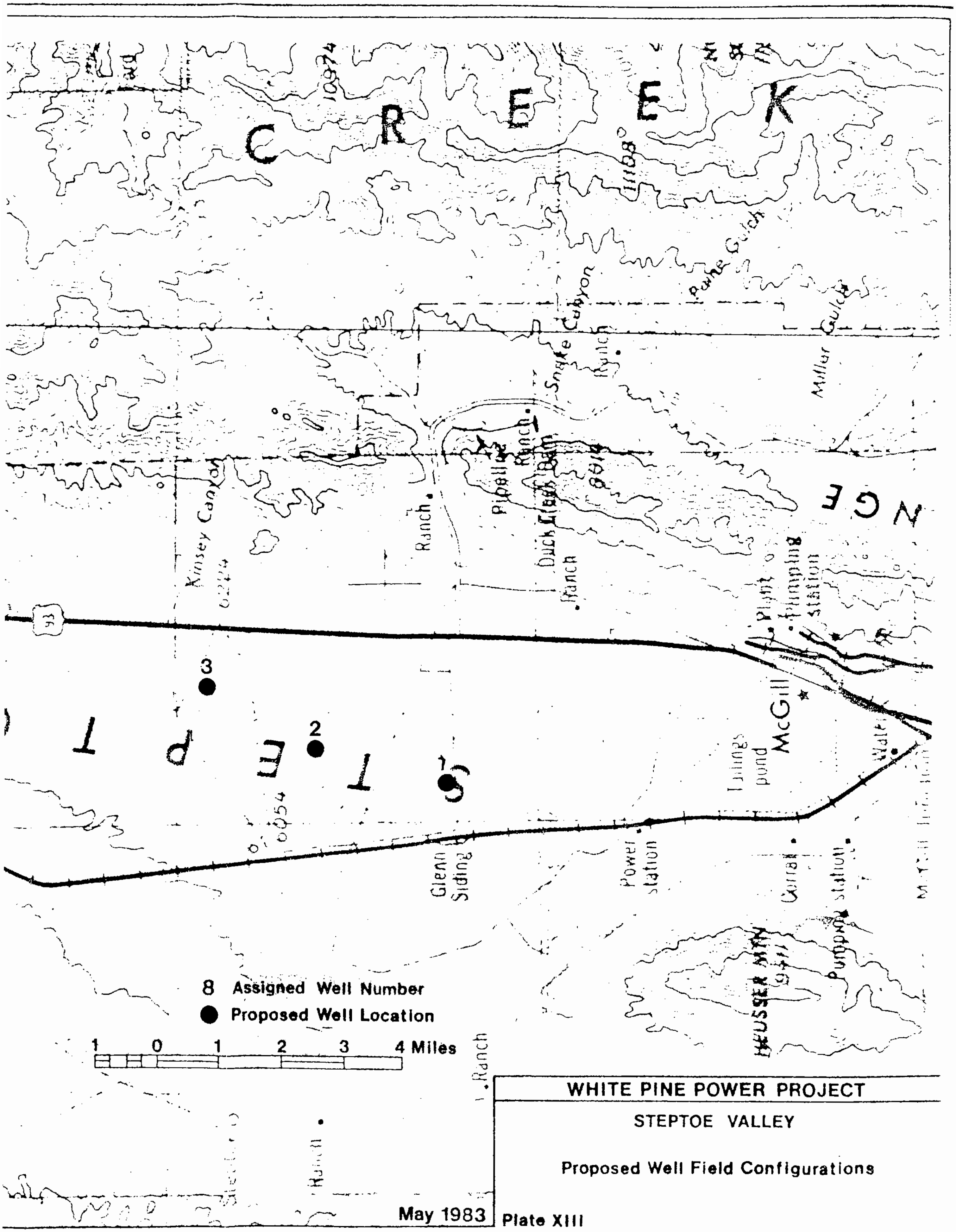
SPRING V

Sensitivity
Zero Recha

Plate XII



N
T 19N
T 18N
T 17N
T 16N
Spring Valley Ranger Station
Ranch
HUMBOLDT
NATIONAL
FOREST
5
10
20
60
80



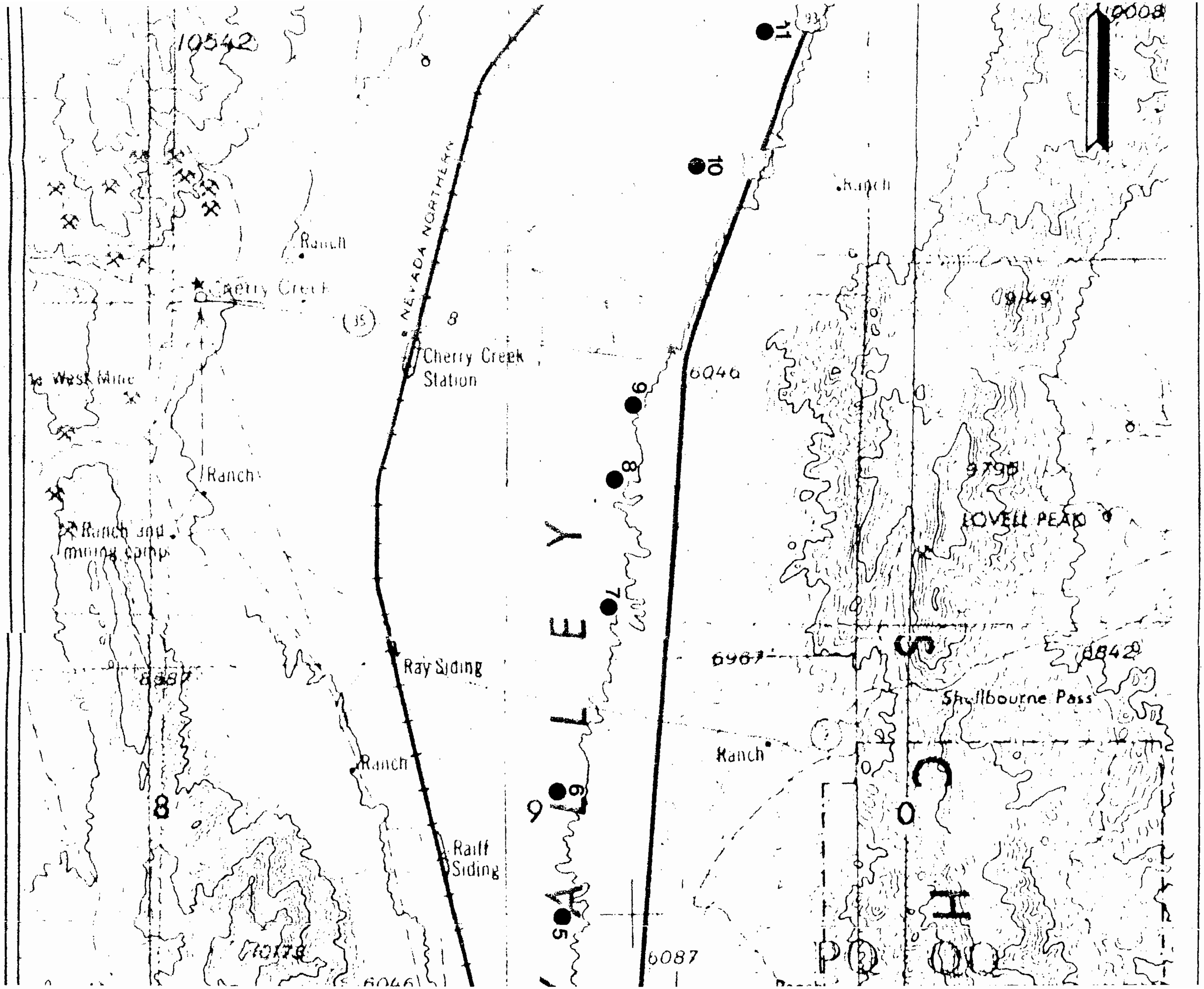
WHITE PINE POWER PROJECT

STEPTOE VALLEY

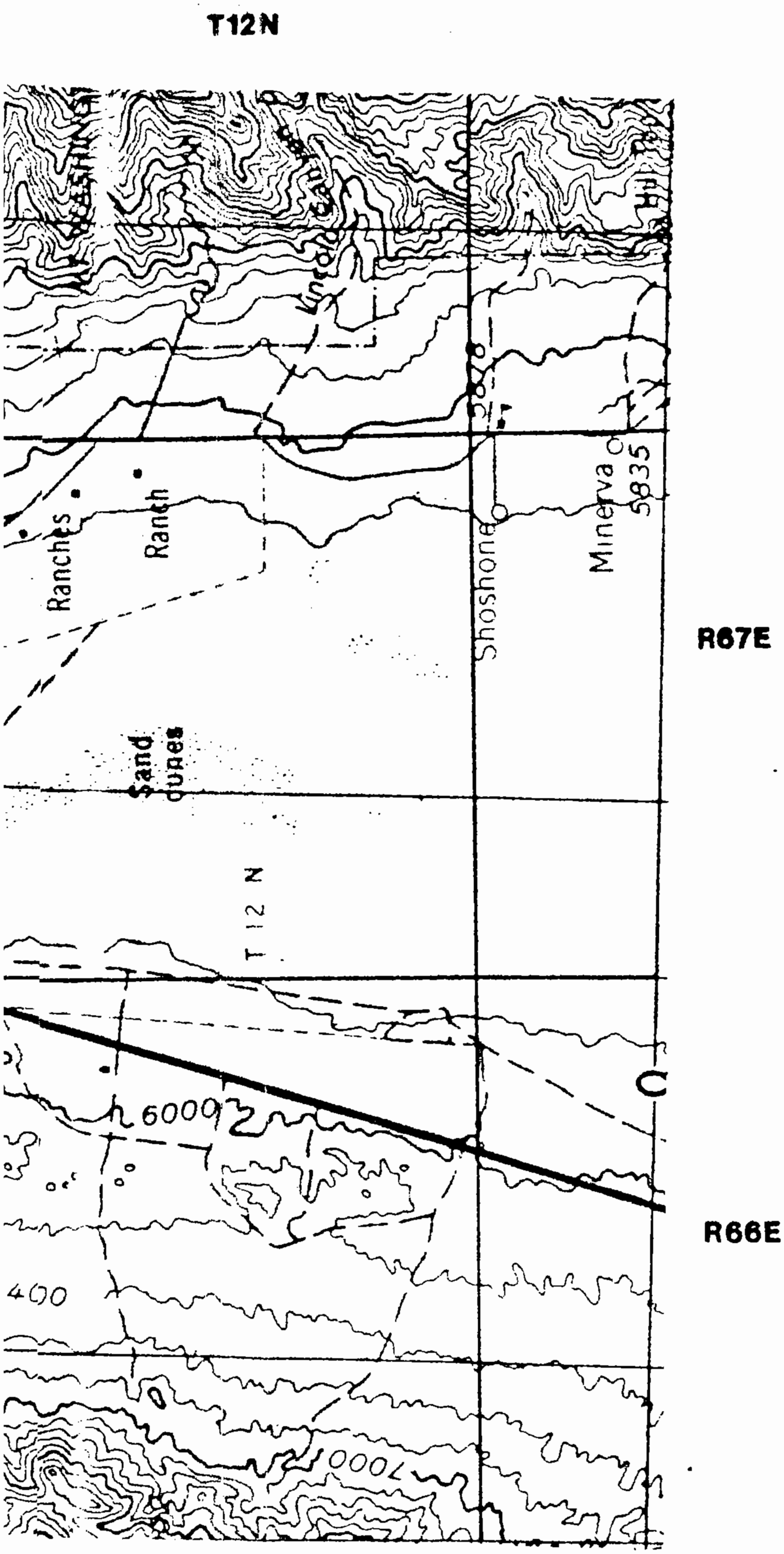
Proposed Well Field Configurations

May 1983

Plate XIII



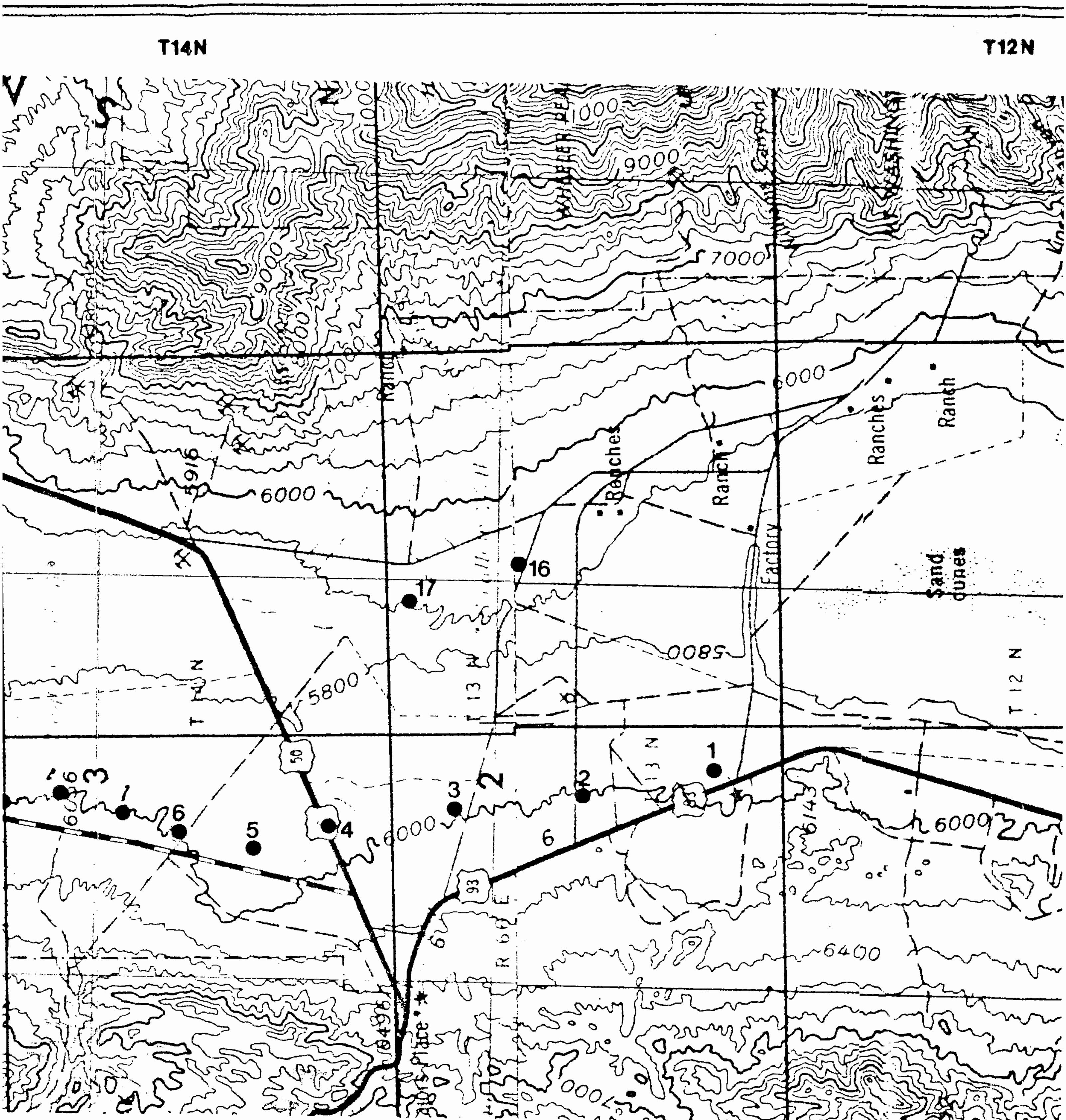
10542
Ranch
Cherry Creek
NEVADA NORTHERN
Cherry Creek Station
Wesk Mine
Ranch
Ranch and mining camp
Ray Siding
Ranch
Raiff Siding
10178
6046
11
10
9
8
7
6
5
V A L L E Y
6046
6087
6967
Ranch
Ranch
9949
9798
LOVELL PEAK
8842
Shellbourne Pass
S C H O
P O
10008



WHITE PINE POWER PROJECT

SPRING VALLEY

Proposed Well Field Configurations


T14N
T12N
7000
9000
6000
5800
Ranches
Ranch
Factory
Sand dunes
T 13 N
T 12 N
50
93
6400
6143
5916
1
2
3
4
5
6
16
17

TABLES

TABLE 2-1

STEPTOE VALLEY

GRAIN SIZE ANALYSIS FROM BOREHOLE 1A

Sampling Interval (feet)	Effective Particle Size D_{90} (inches)	Mean Particle Size D_{50} (inches)	Uniformity Coefficient D_{40}/D_{90}
350-360	0.016	0.054	4.13
387-397	0.030	0.066	2.57
440-447	0.038	0.077	2.66
460-470	0.008	0.047	6.88

TABLE 2-2

SPRING VALLEY

GRAIN SIZE ANALYSIS FROM BOREHOLE 2A

Sampling Interval (feet)	Effective Particle Size D_{90} (inches)	Mean Particle Size D_{50} (inches)	Uniformity Coefficient D_{40}/D_{90}
270-280	-	0.052	-
310-320	0.033	-	-
370-380	0.030	0.060	2.40
560-570	0.030	0.072	2.77

TABLE 3-1

STEP DRAWDOWN TEST NO. 1
STEPTOE VALLEY WELL 1A

Step	Discharge, Q (gpm)	Discharge, Q (cfs)	Drawdown, s Total (ft)	Drawdown/Discharge (ft/cfs)	Specific Capacity (gpm/ft)
1	545	1.21	18.7	15.5	29.1
2	885	1.97	32.9	16.7	26.9
3	1170	2.60	47.1	18.1	24.8
4	1520	3.38	65.5	19.4	23.2

TABLE 3-2

STEP DRAWDOWN TEST NO. 2
STEPTOE VALLEY WELL 1A

Step	Discharge, Q (gpm)	Discharge, Q (cfs)	Drawdown, s Total (ft)	Drawdown/Discharge (ft/cfs)	Specific Capacity (gpm/ft)
1	1170	2.60	47.2	18.2	24.8
2	1700	3.78	71.3	18.9	23.8
3	1960	4.36	84.3	19.3	23.3
4	±2100	±4.7	91.8	19.5	22.9

TABLE 3-3

COMPARISON OF AQUIFER COEFFICIENTS FROM STEPTOE VALLEY WELL TESTS

Method of Analysis	Aquifer Coefficients	Production Well 1A	Observation Well 1B	Observation Well 1C
Leaky confined aquifer with full penetration (drawdown test #2)	T(gpd/ft)	-	160,000	94,000
	S	-	0.00017	0.00025
	B (ft)	-	8930	2180
Leaky confined aquifer with partial penetration (drawdown test #2)	T(gpd/ft)	-	160,000	98,000
	S	-	0.00017	0.00025
	B (ft)	-	8930	2800
Leaky confined aquifer with full penetration (recovery test #2)	T(gpd/ft)	132,600	160,000	96,600
	S	-	0.00019	0.00017
	B (ft)	-	8840	3220

TABLE 3-4

ESTIMATED INFILTRATION RATES AT STEPTOE VALLEY *

Station No.	Date of Measurement	Flow (cfs)	Loss (cfs)	Distance Between Stations (feet)	Estimated Infiltration (cfs/mile)
1	Oct. 2, 1982	3.48			
			0.08	717	
2	Oct. 2, 1982	3.40			
			0.35	1065	
3	Oct. 2, 1982	3.05			
					1.27
1	Oct. 3, 1982	3.80			
			0.31	715	
2	Oct. 3, 1982	3.49			
			0.10	1065	
3	Oct. 3, 1982	3.39			
					1.22
1	Oct. 4, 1982	3.50			
			0.12	715	
2	Oct. 4, 1982	3.38			
			0.43	1065	
3	Oct. 4, 1982	2.95			
					1.60

* Measurements have been rounded off.

TABLE 3-5

STEP DRAWDOWN TEST
SPRING VALLEY WELL 2A

Step	Discharge, Q (gpm)	(cfs)	Drawdown, s Total (ft)	Drawdown/Discharge (ft/cfs)	Specific Capacity (gpm/ft)
1	470	1.04	21.58	20.8	21.8
2	840	1.87	44.86	24.0	18.7
3	1250	2.78	76.77	27.6	16.3
4	1490	3.31	100.19	30.3	14.9

TABLE 3-6

COMPARISON OF AQUIFER COEFFICIENTS FROM SPRING VALLEY WELL TESTS

Method of Analysis	Aquifer Coefficients	Production Well 2A	Observation Well 2B	Observation Well 2C
Unconfined aquifer with full penetration (drawdown)	T(gpd/ft)	-	-	38,000
	S	-	-	0.069
Confined aquifer with full penetration and no boundary (drawdown)	T(gpd/ft)	-	20,000	-
	S	-	0.00012	-
Confined aquifer with full penetration and recharge boundary (drawdown)	T(gpd/ft)	-	14,700	-
	S	-	0.00019	-
Leaky confined aquifer with full penetration and no boundary (drawdown)	T(gpd/ft)	-	14,400	-
	S	-	0.00017	-
Leaky confined aquifer with full penetration and impermeable boundary (drawdown)	T(gpd/ft)	-	14,100	-
	S	-	0.00019	-
Leaky confined aquifer with full penetration (recovery test)	T(gpd/ft)	24,700*	15,200	-

* This is a composite value of transmissivity since the well is actually perforated in both upper and lower aquifers.

TABLE 3-7

ESTIMATED INFILTRATION RATES AT SPRING VALLEY ***

Station No.	Date of Measurement	Flow (cfs)	Loss (cfs)	Distance Between Stations (feet)	Estimated Infiltration (cfs/mile)
1	Oct. 5, 1982	2.89			
			+0.26*	305	
2	Oct. 5, 1982	3.15			
			0.64	2380	
3	Oct. 5, 1982	2.51			
					0.73**
1	Oct. 6, 1982	2.92			
			+0.41*	305	
2	Oct. 6, 1982	3.33			
			0.97	2380	
3	Oct. 6, 1982	2.36			
					1.10**

* Measurements have been rounded off.
** Based on losses between Station No. 1 and 3.
*** Measurements have been rounded off

TABLE 3-8

EVALUATION OF FLOWMETER DATA

Discharge Rate (gpm)	Percent of Flow From Upper Aquifer	Percent of Flow From Lower Aquifer
450	41	59
725	33	67
935	27	73
1300 (pump test discharge rate)	17 *	83 *

* Values graphically derived from flowmeter test data presented on Plates III and IV and Figure 3-50.

TABLE 4-1

STEPTOE VALLEY MONITORING STATIONS

Station Code	Use	Casing Diameter (inches)	Total Sounded Depth (feet)	Ownership	Remarks
FP	Presently not in use	8	6.3	Private	
WM	Stock water supply	6	33.6	Private	SC = 2,000 micromhos/cm Temp. = 56°F
ET	Presently not in use	6	26.2	Private	
SW	Presently not in use	6	7.9	Private	
HB	Stock water supply	6	33.4	Private	SC = 2,000 micromhos/cm Temp. = 48°F
DC	Stock water supply	N/A	Est. approx. 35	Private	Windmill
HW	Presently not in use	N/A	222.5	BLM	

N/A - Not available

TABLE 4-2

SPRING VALLEY MONITORING STATIONS

Station Code	Use	Casing Diameter (inches)	Total Sounded Depth (feet)	Ownership	Remarks
Spring-1	Monitoring well	2	14.1	BLM	Installed for monitoring program
Spring-2A	Monitoring well	2	20.0	BLM	Installed for monitoring program
Spring-2B	Monitoring well	2	14.7	BLM	Installed for monitoring program
Spring-3A	Monitoring well	2	14.1	BLM	Installed for monitoring program
Spring-3B	Monitoring well	2	12.4	BLM	Installed for monitoring program
Spring 4	Stock water supply	6	48.9	BLM	Flowing artesian well: SC = 270 micromhos/cm Temp. = 53°F
Spring 5	Stock water supply	6	77.5	BLM	Flowing artesian well: SC = 280 micromhos/cm Temp. = 53°F
EL	Presently not in use	10-1/3	Approx. 625	Private	
AW	Stock water supply	N/A	N/A	BLM	SC = 460 micromhos/cm; Temp. = 53°F
MX	Presently not in use	2	131.5	BLM	Dry well
DW	Presently not in use	16	400*	Private	
TW	Irrigation	N/A	800*	Private	
Shoshone Ponds Area					
LH	Stock water supply	N/A	N/A	BLM	Flowing artesian: SC = 120 micromhos/cm Temp. = 69°F.
SH	Stock water supply	8	82.3	BLM	
BH	Stock water supply	2-1/2	N/A	BLM	Flowing artesian: SC = 160 micromhos/cm Temp. 74°F.
TI	Stock water supply	8-5/8	138	BLM	Flowing artesian: Temp. 61°F. SC = 120 micromhos/cm
LW	Stock water supply	12	N/A	BLM	Flowing artesian: SC = 150 micromhos/cm Temp. 65°F.

N/A - Not Available
* Depth estimated by owner

TABLE 4-3

WATER QUALITY ANALYSES
STEPTOE VALLEY WELLS

Sample Location / Date of Sample	Concentrations in mg/l		
	Production Well 1A 9/1/82	Production Well 1A 9/1/82	DC 8/24/82
Alkalinity	174	170	300
Bicarbonate	174	170	261
Carbonate	0	0	39
Conductance umhos/cm	262	420	588
Hardness	160	163	148
Suspended solids	1	1	1
Total dissolved Solids	259	256	480
Turbidity	0.8	0.7	0.3
Aluminum	<0.5	<0.5	<0.5
Calcium	50	49	34
Magnesium	10	10	15
Potassium	5.0	5.0	12
Sodium	17	17	104
Chloride	5.9	6.1	14.8
Nitrate	0.7	0.5	0.6
Nitrite	<0.001	<0.001	<0.001
Phosphate	0.04	0.08	0.07
Silica, colloidal	0.11	0.062	0.035
Silica, dissolved	59	72	87
Sulfate	17	17	48
pH, (at site)	7.10	7.27	7.20
Temperature, (at site) °C	17	16	12.5

Analysis performed by LADWP

TABLE 4-4

WATER QUALITY ANALYSIS
STEPTOE VALLEY OBSERVATION WELL 1C

Constituents	Concentrations in (mg/l)	
Phenolphththalein Alkalinity $CaCO_3$	18	
Total Alkalinity as $CaCO_3$	187	
CO_3	22	
HCO_3	184	
Hardness as $CaCO_3$	163	
Cl	6.8	
SO_4	13.0	
NO_3	0.4	
F	0.55	
Arsenic	0.02	
Ca	49	
Mg	10	
Na	20	
K	4.8	
Fe	>0.1)	All below
Mn	>0.01)	detection
Cu	>0.05)	limits
Zn	>0.1)	
Boron	>0.1	
Total Silica ("Molybdate - Reactive" Silica)	28	
pH	8.6	
SAR	0.96	

Analysis performed by Great Basin Laboratories, Reno, Nevada.

TABLE 4-5

WATER QUALITY ANALYSES
SPRING VALLEY WELLS

Sample Location / Date of Sample	Production Well 2A 9/15/82	Production Well 2A 9/30/82	Spring 4 8/24/82	Spring 5 8/24/82
Alkalinity	145	147	139	155
Bicarbonate	145	147	139	155
Carbonate	0	0	0	0
Conductance umhos/cm	280	340	241	248
Hardness	146	144	138	148
Suspended solids	1	1	1	1
Total dissolved Solids	156	160	160	173
Turbidity	0.7	0.7	0.4	0.4
Aluminum	<0.5	<0.5	<0.5	<0.5
Calcium	37	37	35	34
Magnesium	13	12	13	15
Potassium	1.4	1.4	1.8	1.4
Sodium	4.6	4.6	6.5	5.7
Chloride	2.4	2.5	2.4	3.7
Nitrate	0.7	0.7	0.6	0.5
Nitrite	<0.001	<0.001	<0.001	<0.001
Phosphate	0.11	0.11	0.02	0.03
Silica, colloidal	0.037	0.18	0.26	0.017
Silica, dissolved	9	10	20	26
Sulfate	7	5	5.4	5.4
pH, (at site)	7.83	7.75	7.00	6.85
Temperature, (at site) °C	14	14	12	12

Analysis performed by LADWP

TABLE 4-6

WATER QUALITY ANALYSIS
SPRING VALLEY OBSERVATION WELL 2B

Constituents	Concentrations in (mg/l)	
Total Alkalinity as $CaCO_3$	152	
HCO_3	185	
Hardness as $CaCO_3$	140	
Cl	3.9	
SO_4	4.7	
NO_3	1.2	
F	0.28	
Arsenic	0.01	Below detection limit
Ca	38	
Mg	11.0	
Na	8.2	
K	1.8	
Fe (total)	1.2	
Mn	>0.01	) Below detec-
Cu	>0.05	) tion limit
Zn	>0.1	)
Boron	>0.1	
Total Silica ("Molybdate - Reactive" Silica)	14	
pH	7.85	
SAR	0.4	

Analysis performed by Great Basin Laboratories, Reno, Nevada.

TABLE 5-1

CORRELATION BETWEEN FORMATION RESISTIVITY AND GEOLOGIC PROPERTIES

Formation Resistivity Classification	Formation Resistivity Range (ohm-m)	Formation Characteristics: Pore Water Salinity	Formation Characteristics: Clay Content	Formation Characteristics: Effective Porosity	Probable Fresh-Water Production Capability
Very low	Less than 3-5	Saline	Moderately high	Low	Very poor
		Fresh	High to very high	Low to very low	
Low	5 to 20	Probably fresh	Moderate	High	Poor
			High	Moderate to low	
Moderate	20 to 50	Fresh	Low	High to very high	Good
			Moderate	Moderate	
Moderately high	50 to 100	Fresh	Low	Moderately high	Good
			Moderate	Moderately low	
High	100 to 250	Fresh	Low	Moderate to low	Good
Very high	Greater than 250	Fresh	Very low	Low to very low (possibly consolidated rock or partial saturation above water table)	Poor

TABLE 6-1

PRECIPITATION STATIONS IN WHITE PINE COUNTY
USED FOR ANALYSIS

Station	Lat.	Long.	Elevation Above MSL (feet)	Period of Record
BLM	38°50'	114°47'	7700	1965-80
Connors Pass	39°02'	114°39'	7330	1964-68,1972-80
Ely WSO AP	39°17'	114°51'	6253	1889-91,1893-1902, 1908-10,1941-Present
Lehman Caves Nat'l. Mon.	39°00'	114°13'	6825	1939-43,1945-48, 1951-Present
McGill	39°24'	114'46'	6340	1909-18,1927-31 1933-Present
Robinson Summit	39°25'	115°05'	7630	1964-68,1972-80
Ruth	39°17'	114°59'	6832	1962-77
Shellbourne Pass	39°48'	114'39°	8150	1954-64

TABLE 6-2

ESTIMATED AVERAGE ANNUAL RECHARGE

STEPTOE VALLEY

Elevation of zone	Effective Area Represented by this zone	Average Annual Precipitation		Estimated Recharge	
(feet)	(acres)	(feet)	(acre-feet)	Percentage of (*) Precipitation	(acre-feet per year)
Above 9000	57,000	1.67	94,943	25	24,211
8000 to 9000	156,000	1.38	215,759	15	32,364
7000 to 8000	266,000	1.10	292,670	7	20,487
6000 to 7000	213,000	0.71	152,390	3	4,571
Below 6000	573,000	0.54	306,404	0	0
TOTALS	1,265,000		1,062,166		81,600**

* References 8 and 9
** Rounded value

TABLE 6-3

ESTIMATED AVERAGE ANNUAL RECHARGE

SPRING VALLEY

Elevation of zone (feet)	Effective Area Represented by this zone (acres)	Average Annual Precipitation (feet)		Estimated Recharge Percentage of (*) Precipitation	
		(feet)	(acre-feet)	Percentage of (*) Precipitation	(acre-feet per year)
Above 9000	59,100	1.65	97,540	25	24,385
8000 to 9000	107,300	1.38	148,530	15	22,280
7000 to 8000	183,500	1.12	205,199	7	14,364
6000 to 7000	393,000	0.71	281,171	3	8,435
Below 6000	342,000	0.57	196,372	0	0
TOTALS	1,084,900		928,812		69,500**

(*) References 9 and 34
(**) Rounded value

TABLE 6-4

MEASURED STREAM DISCHARGE DATA

Stream	Drainage Area (sq. mi.)	Period of Record (years)	Average Annual Discharge (acre-feet)
Cleve Creek	31.8	1961-67,77-80	6958
Duck Creek	78.6	1909-15,57-65, 66,67,74,76	8910
Little Currant Creek	12.9	1965-80	2600
Steptoe Creek	11.1	1967-80	5146

TABLE 6-5

AVERAGE ANNUAL PRECIPITATION
VERSUS
AVERAGE ANNUAL RUNOFF

CLEVE CREEK

Approximate Equivalent Altitude Zone (feet)	Area Represented by this Zone (acres)	Average Annual Precipitation		Estimated Runoff/Recharge			
				NDCNR		EDI Estimate	
		(feet)	(acre-feet)	Percentage of * Precipitation 1/	(acre-feet per year)	Percentage of * Precipitation	(acre-feet per year)
Above 9000	10,035	1.61	16,156	25	4039	33	5332
8000 to 9000	4762	1.37	6534	15	979	20	1305
7000 to 8000	4064	1.13	4592	7	321	10	459
Below 7000	1491	.85	1267	3	38	4	51
Total	20,352		28,549		5377		7147

Average Annual Discharge 1961-67/1977-80 = 6958 acre-feet

*/ Percent of Precipitation falling on each zone which becomes groundwater recharge.
1/ References 9 and 24

TABLE 6-6

AVERAGE ANNUAL PRECIPITATION
VERSUS
AVERAGE ANNUAL RUNOFF

DUCK CREEK

Approximate Equivalent Altitude Zone (feet)	Area Represented by this Zone (acres)	Average Annual Precipitation		Estimated Runoff/Recharge			
				NDCNR		EDI Estimate	
		(feet)	(acre-feet)	Percentage of * Precipitation 1/	(acre-feet per year)	Percentage of * Precipitation	(acre-feet per year)
Above 9000	16,752	1.62	27,138	25	6785	17	4613
8000 to 9000	17,181	1.4	24,053	15	3608	8	1924
7000 to 8000	15,830	1.18	18,679	7	1307	2	374
Below 7000	-		-		-		-
Total	49,763		69,870		11,700		6911

Average Annual Discharge 1909-15, 57-65, 66, 67, 74, 76 = 8910 acre-feet

* Percent of Precipitation falling on each zone which becomes groundwater recharge.
1/ References 8 and 9

TABLE 6-7

AVERAGE ANNUAL PRECIPITATION
VERSUS
AVERAGE ANNUAL RUNOFF

LITTLE CURRANT CREEK

Approximate Equivalent Altitude Zone (feet)	Area Represented by this Zone (acres)	Average Annual Precipitation		Estimated Runoff/Recharge			
				NDCNR		EDI Estimate	
		(feet)	(acre-feet)	Percentage of * Precipitation 1/	(acre-feet per year)	Percentage of * Precipitation	(acre-feet per year)
Above 9000	1997	1.65	3295	25	824	34	1120
8000 to 9000	2362	1.38	3260	15	489	27	880
7000 to 8000	3597	1.12	4029	7	282	15	604
Below 7000	301	.81	244	3	7	7	17
Total	8257		10,828		1602		2621

Average Annual Discharge 1967-80 = 2600 acre-feet

* Percent of Precipitation falling on each zone which becomes groundwater recharge.
1/ Reference 9

TABLE 6-8

AVERAGE ANNUAL PRECIPITATION
VERSUS
AVERAGE ANNUAL RUNOFF

STEPTOE CREEK

Approximate Equivalent Altitude Zone (feet)	Area Represented by this Zone (acres)	Average Annual Precipitation (feet)	Average Annual Precipitation (acre-feet)	Estimated Runoff/Recharge: NDCNR Percentage of * Precipitation 1/	NDCNR (acre-feet per year)	EDI Estimate Percentage of * Precipitation	EDI Estimate (acre-feet per year)
Above 9000	2957	1.69	4997	25	1249	65	3248
8000 to 9000	3436	1.41	4845	15	727	39	1890
7000 to 8000	736	1.14	839	7	59	8	151
Below 7000	-		-		-		-
Total	7129		10,681		2035		5289

Average Annual Discharge 1967-80 = 5146 acre-feet

* Percent of Precipitation falling on each zone which becomes groundwater recharge.
1/ References 8 and 9

TABLE 6-9

AVERAGE ANNUAL VALLEY DISCHARGES

SPRING VALLEY

Type of Discharge Region	Consumptive Use Factor (acre-ft/yr.)	Acreage (acres)	Consumptive Use (acre-ft/yr.)
1. Irrigated Lands	1.5	26,400	39,600
2. Wet Meadow (Not Irrig.)	1.0	14,400	14,400
3. Transitional Desert (Shrub-Greasewood)	.1	154,000	15,400
TOTAL			69,400

STEPTOE VALLEY

Type of Discharge Region	Consumptive Use Factor (acre-ft/yr.)	Acreage (acres)	Consumptive Use (acre-ft/yr.)
1. Irrigated Lands	1.5	28,900	43,350
2. Wet Meadow (Not Irrig.)	1.0	41,150	41,150
TOTAL			84,500

References 22, 50 and 51.

TABLE 6-10

BASIS FOR GROUNDWATER MODELS IN SPRING AND STEPTOE VALLEYS

	Steptoe Valley	Spring Valley
Fan Zone Transmissivity	94,000 gpd/ft.	Varies, either 58,300 gpd/ft., 39,000 gpd/ft., or 19,500 gpd/ft.
Fan Zone Storativity	0.0002	0.0002
Playa Zone Transmissivity	25,000 gpd/ft.	Varies, either 9,700 gpd/ft., or 5,000 gpd/ft.
Playa Zone Storativity	0.0001	0.0001
Annual Average Recharge	81,600 AF/yr.	69,500 AF/yr.
Consumptive Use:		
Irrigated Acreage	1.5 AF/Acre-yr.	1.5 AF/Acre-yr.
Wet Meadow	0.93 AF/Acre-yr.	1.0 AF/Acre-yr.
Transitional Zones	-	0.10 AF/Acre-yr.

TABLE 6-11

BASIS FOR ESTIMATING GROUNDWATER DRAWDOWNS FOR POTENTIAL WHITE PINE POWER PROJECT WELL FIELDS IN SPRING AND STEPTOE VALLEYS

	Steptoe Valley	Spring Valley
No. of WPPP Wells	14	22
Maximum Design Discharge Per Well	1250 gpm	750 gpm
Maximum Well Field Discharge	17,500 gpm 28,230 AF/yr.	16,500 gpm 26,615 AF/yr.
Average Annual Well Field Discharge	25,000 AF/yr.	25,000 AF/yr.
Average Annual Design Discharge Per Well	1110 gpm	705 gpm
Minimum Well Spacing	Two miles	One mile

TABLE 6-12

ESTIMATED PIEZOMETRIC SURFACE DRAWDOWNS AT VARIOUS NODAL LOCATIONS IN STEPTOE VALLEY

Node Nos.	Drawdowns in feet			
	Case I	Case II	Case IIB	Case III
In Wet Meadow Areas				
146	0	0	1	0
199	1	1	5	5
224	2	2	6	6
245	5	6	12	12
268	8	9	15	14
289	5	6	10	9
In Alluvial Fan Areas near Proposed Well Fields				
139	2	2	4	2
158	8	10	12	10
169	7	9	12	9
200	2	2	4	4
225	14	17	23	19
237	15	19	26	21
254	14	17	25	22
269	4	5	7	6
282	5	6	11	10
296	6	7	10	8
322	1	1	3	4

Case I - Current agriculture plus WPPP at 20,000 afy.

Case II - Current agriculture plus WPPP at 25,000 afy.

Case IIB - Same as Case II but with transmissivity value in fan area decreased by 20%.

Case III - Current agriculture plus 15,000 afy future agriculture in north Steptoe plus WPPP at 25,000 afy.

TABLE 6-13

ESTIMATED PIEZOMETRIC SURFACE DRAWDOWNS AT VARIOUS NODAL LOCATIONS IN SPRING VALLEY

Node Nos.	Drawdowns in feet			
	Case I	Case II	Case IIB	Case III
In Wet Meadow Areas				
67	0	0	2	19
150	5	6	12	6
156	17	21	38	21
164	7	10	18	10
170	15	19	34	19
177	13	17	29	17
185	12	15	24	15
198	5	7	15	7
204	5	6	13	6
218	4	5	13	5
In Alluvial Fan Areas near Proposed Well Fields				
79	11	13	20	36
101	21	27	36	29
116	3	4	6	5
121	20	25	36	27
141	22	27	41	27
169	21	26	41	26
183	10	13	20	13
190	6	8	15	8
199	8	10	15	10
207	7	9	12	9
213	2	3	6	3

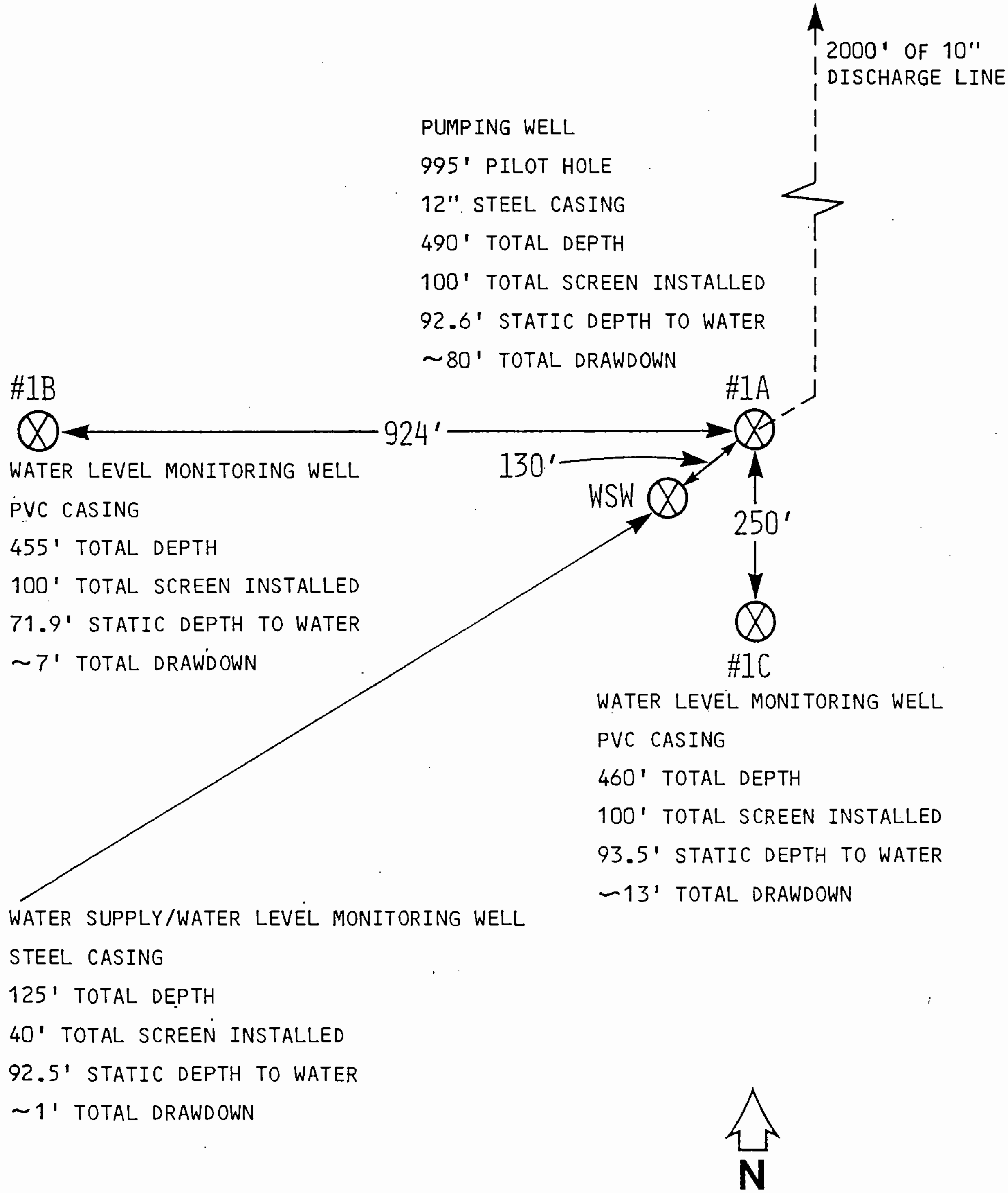
Case I - Current agriculture plus WPPP at 20,000 afy.

Case II - Current agriculture plus WPPP at 25,000 afy.

Case IIB - Same as Case II but with transmissivity value in fan area decreased by 20%.

Case III - Current agriculture plus 15,000 afy future agriculture in southern Spring plus WPPP at 25,000 afy.

FIGURES



WHITE PINE POWER PROJECT

STEPTOE VALLEY

General Site Description and Construction Details

Leeds, Hill and Jewett, Inc. December 1982

FIGURE 2·1

Location: Steptoe Valley

Use: Drilling Water Supply Well/Observation Well

Elevation of top of well casing: ± 5,982 feet

Datum: Field Approximation using 7-1/2 USGS Quad

Date constructed: July 21, 1982

Conductor casing: None

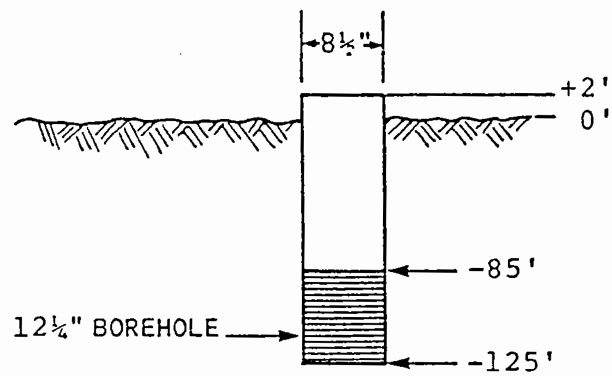
Thickness:

Well casing: Steel

Well screen: Factory Slotted

Slot size: 3/16"

Gravel Pack Material: None



Remarks: Temporary drilling water supply well, casing removed and boring back filled and grouted upon completion of field investigation.

VERTICAL SCALE
1 inch = approx. 100 feet

Leeds, Hill and Jewett, Inc. December 1982

WHITE PINE POWER PROJECT

CONSTRUCTION DETAILS
WELL NO. WSW-1

FIGURE 2·2

Location: Steptoe Valley

Use: Observation Well

Elevation of
top of well casing: 5,962.74 feet

Datum: Nov. 11, 1982 survey by Boundy & Forman, Inc., Ely, Nevada

Date constructed: Aug. 4, 1982

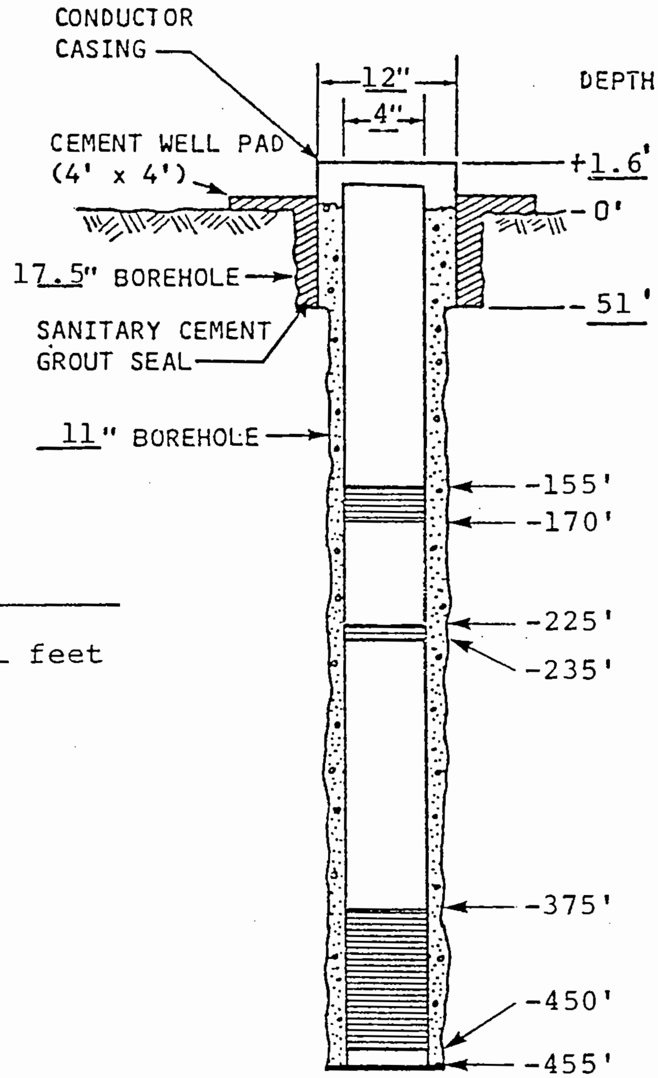
Conductor casing: Steel
Thickness: 5/16"

Well casing: Sch 80 PVC
Thickness: 1/3"

Well screen: Factory Slotted
Slot size: 0.100 inch

Gravel Pack
Material: Local Pea Gravel

0 feet to 455 feet



VERTICAL SCALE
1 inch = approx. 100 feet

WHITE PINE POWER PROJECT

CONSTRUCTION DETAILS
WELL NO. 1-B

Leeds, Hill and Jewett, Inc. December 1982

FIGURE 2-3

Location: Steptoe Valley

Use: Observation Well

Elevation of top of well casing: 5,984.70 feet

Datum: Nov. 11, 1982 survey by Boundy & Forman, Inc., Ely, Nevada

Date constructed: Aug. 11, 1982

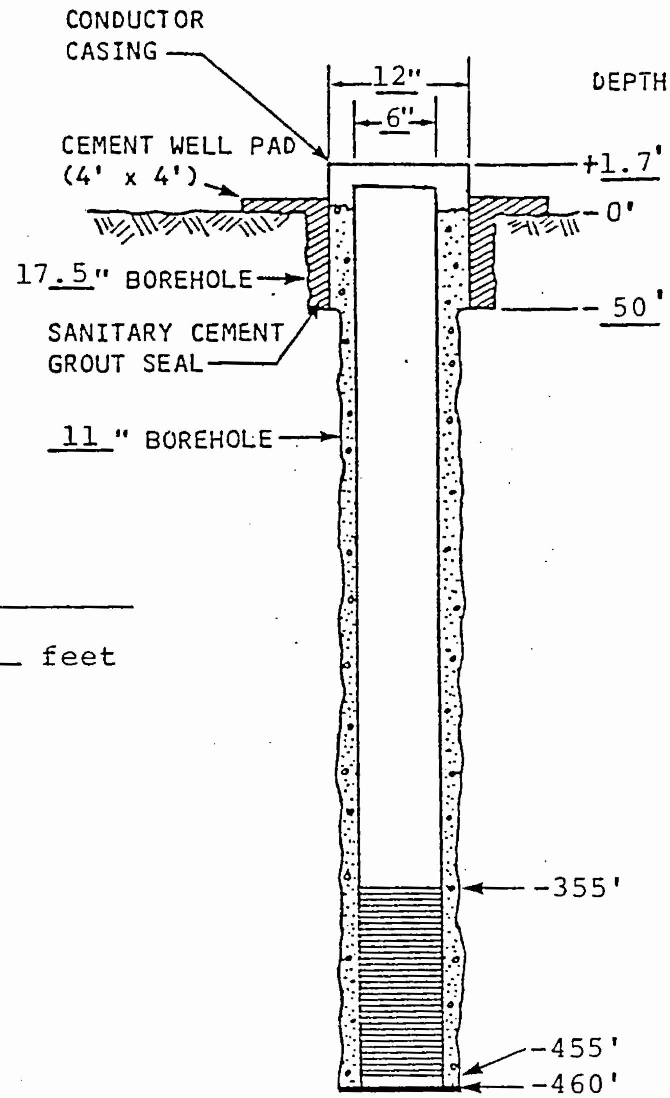
Conductor casing: Steel
Thickness: 5/16"

Well casing: Sch 80 PVC
Thickness: 1/3"

Well screen: Sch 80 PVC
Slot size: 0.100 inch

Gravel Pack Material: Local Pea Gravel

0 feet to 460 feet



VERTICAL SCALE
1 inch = approx. 100 feet

WHITE PINE POWER PROJECT

CONSTRUCTION DETAILS
WELL NO. 1-C

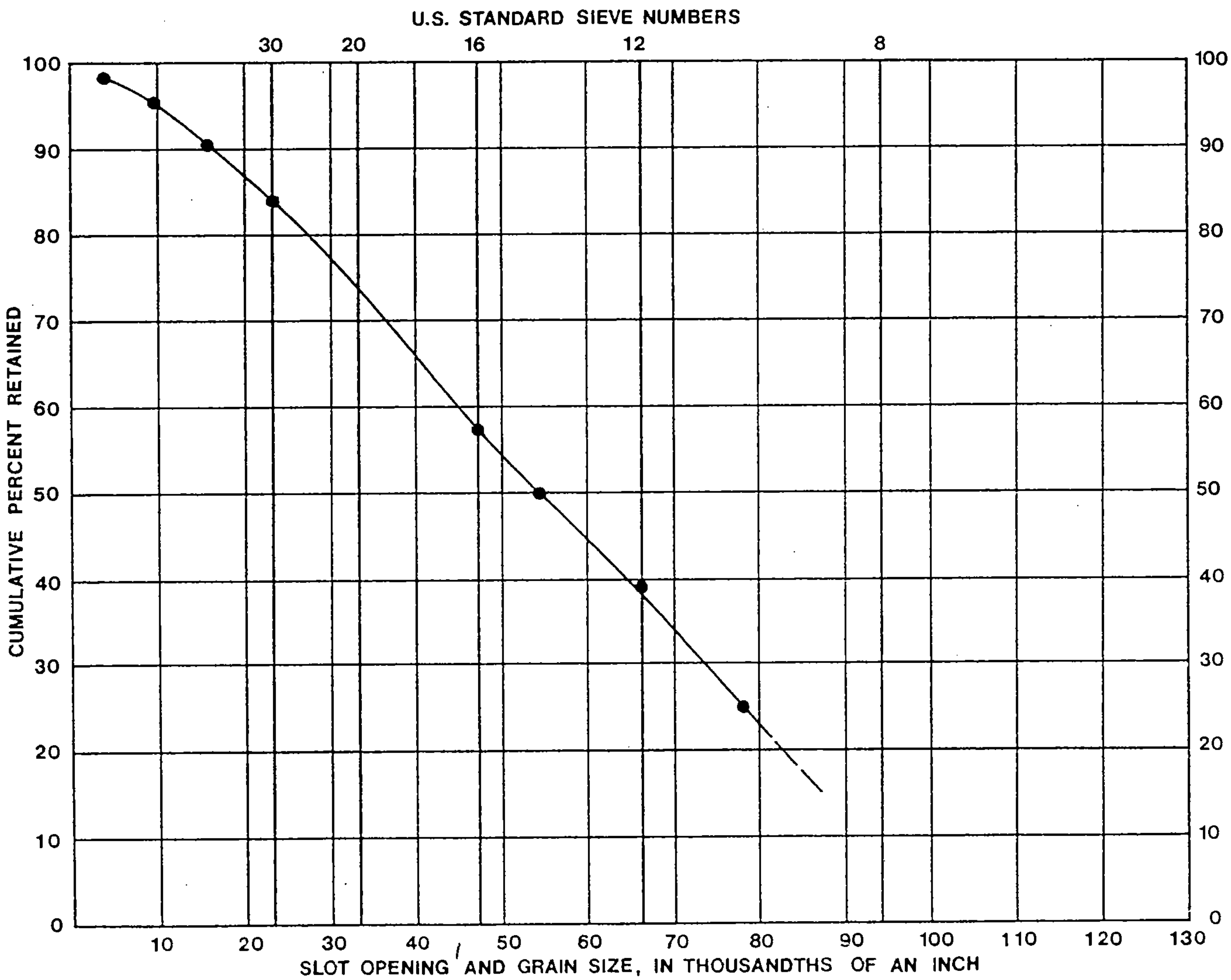
Leeds, Hill and Jewett, Inc. December 1982

FIGURE 2-4

Sample Location STEPTOE VALLEY, BOREHOLE 1A

Sampling Interval 350 to 360 feet

Date Sampled AUGUST 1982



WHITE PINE POWER PROJECT

STEPTOE VALLEY
Grain Size Analysis
Borehole 1A

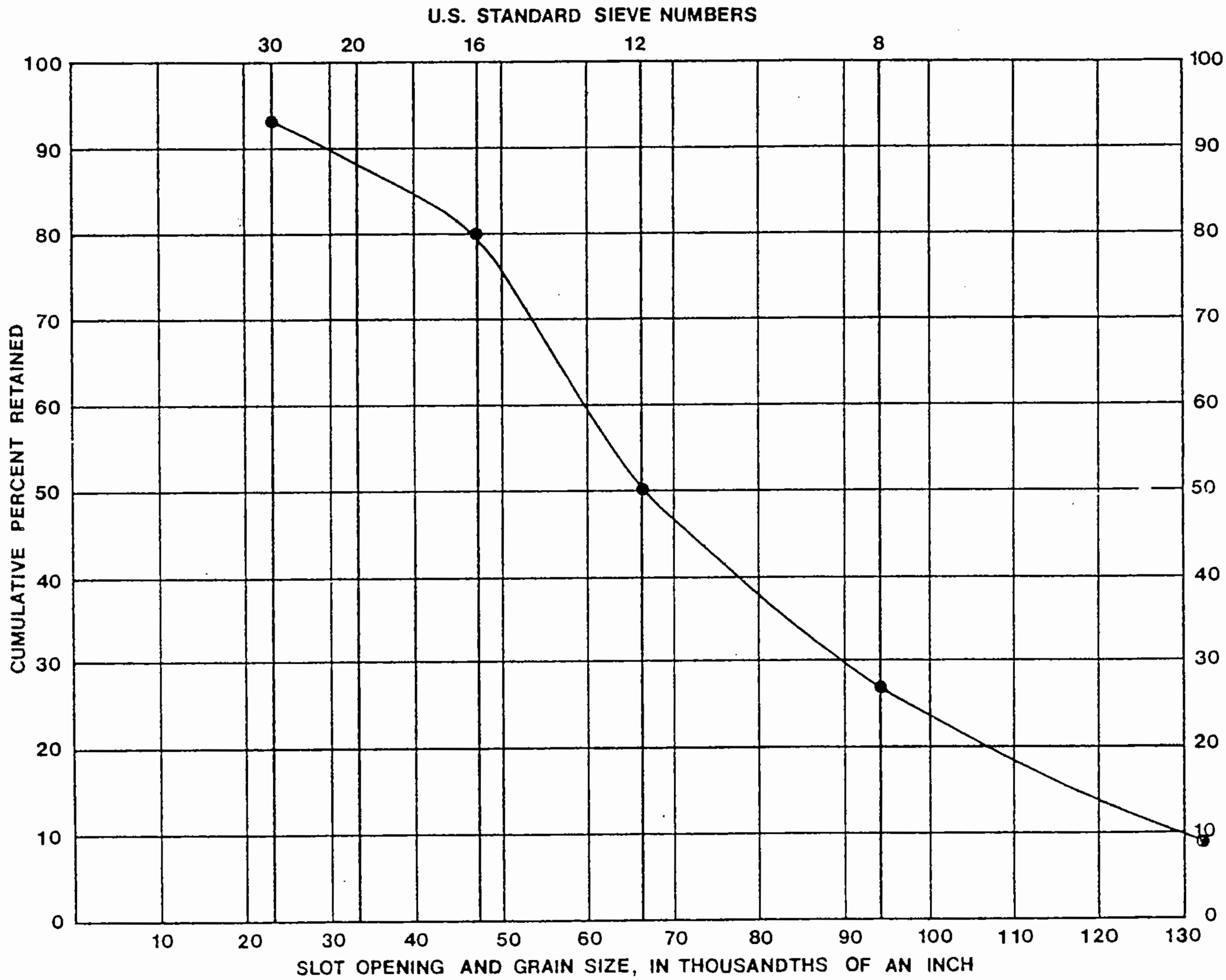
FIGURE 2·5

Leeds, Hill and Jewett, Inc. December 1982

Sample Location STEPTOE VALLEY, BOREHOLE 1A

Sampling Interval 387 to 397 feet

Date Sampled AUGUST 1982



WHITE PINE POWER PROJECT

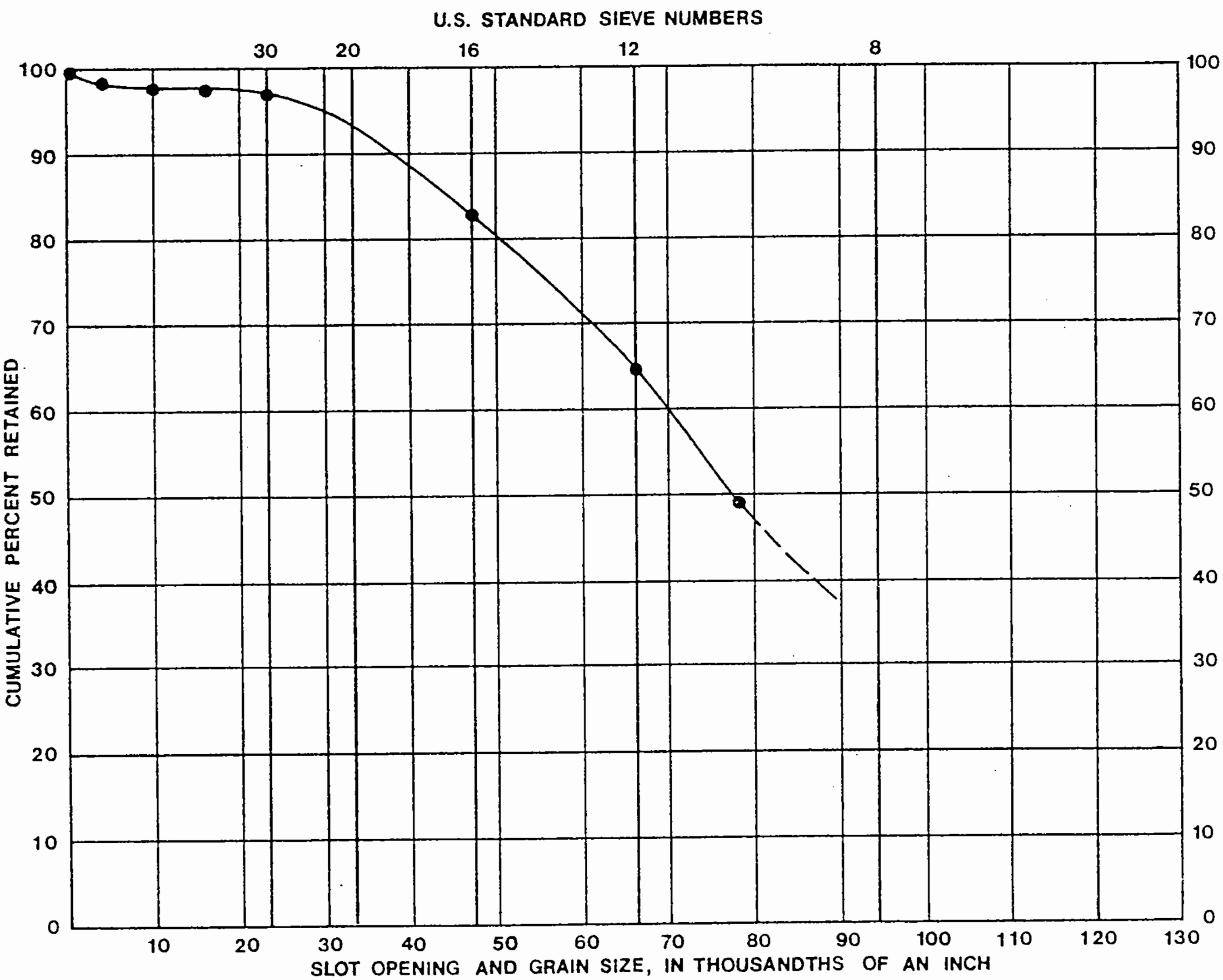
STEPTOE VALLEY
Grain Size Analysis
Borehole 1A

Leeds, Hill and Jewett, Inc. December 1982 FIGURE 2·6

Sample Location STEPTOE VALLEY, BOREHOLE 1A

Sampling Interval 440 to 447 feet

Date Sampled AUGUST 1982



WHITE PINE POWER PROJECT

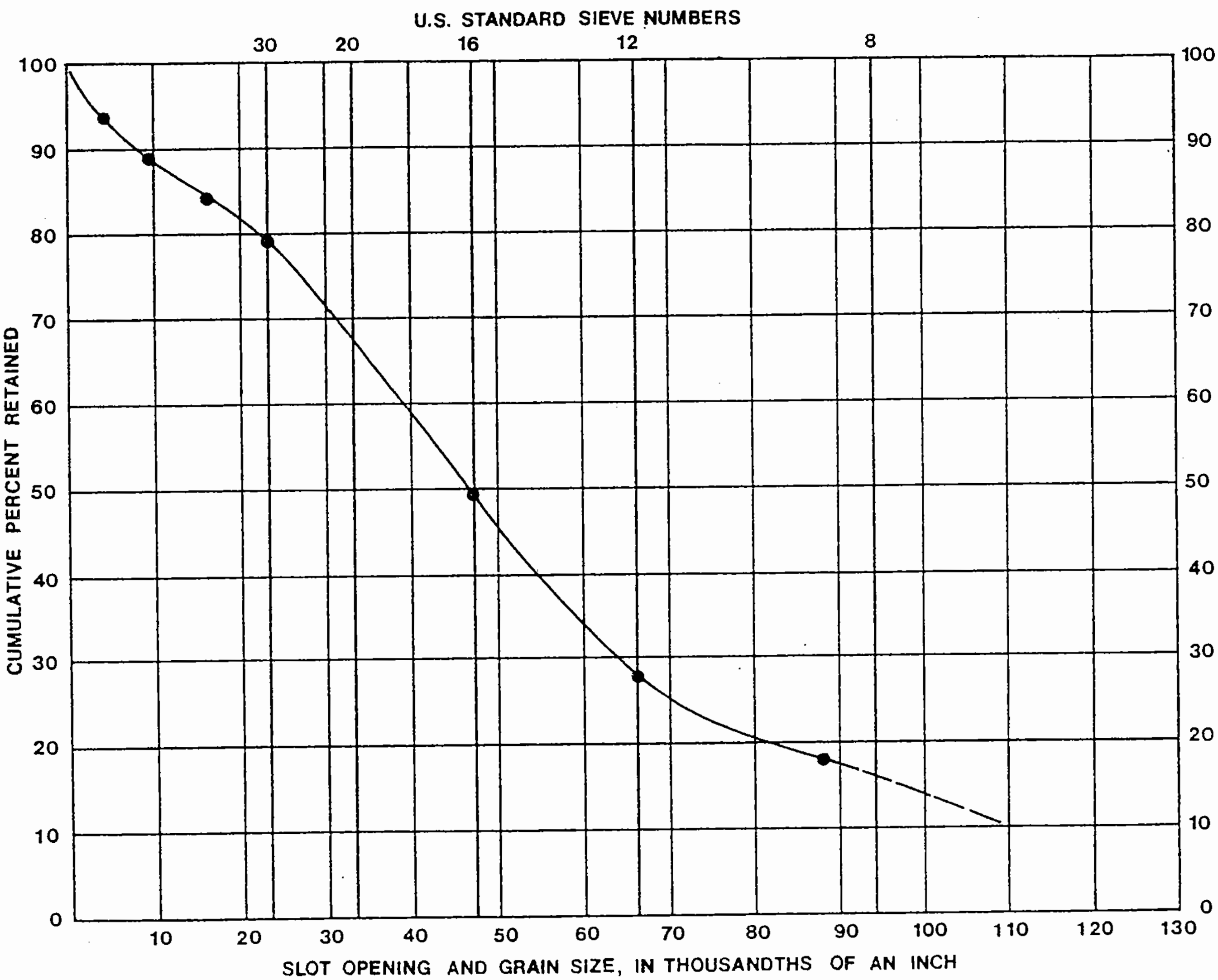
STEPTOE VALLEY
Grain Size Analysis
Borehole 1A

Leeds, Hill and Jewett, Inc. December 1982 FIGURE 2-7

Sample Location STEPTOE VALLEY, BOREHOLE 1A

Sampling Interval 460 to 470 feet

Date Sampled AUGUST 1982



WHITE PINE POWER PROJECT

STEPTOE VALLEY
Grain Size Analysis
Borehole 1A

Leeds, Hill and Jewett, Inc. December 1982 FIGURE 2 · 8

Location: Steptoe Valley

Use: Production Well

Elevation of top of well casing: 5,983.65 feet

Datum: Nov. 11, 1982 survey by Boundy & Forman, Inc., Ely, Nevada

Date constructed: Aug. 19, 1982

Conductor casing: Steel
Thickness: 3/8"

Well casing: Steel
Thickness: 5/16"

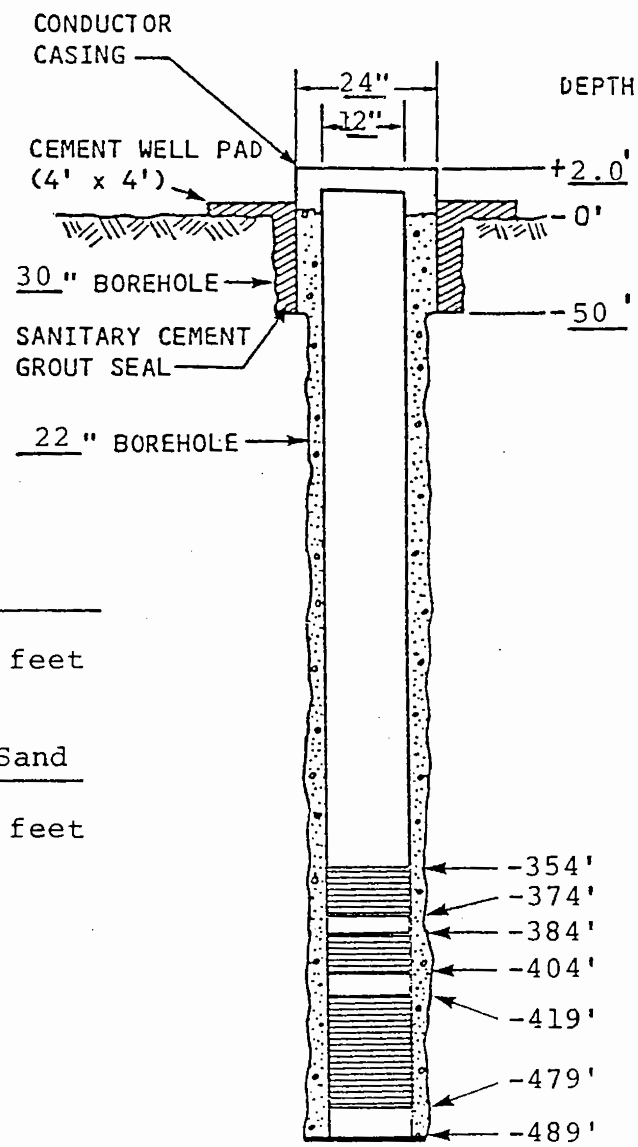
Well screen: Johnson Galvanized
Slot size: 0.100 inch

Gravel Pack
Material: Local Pea Gravel

0 feet to 200 feet

4-8 Colorado Silica Sand

200 feet to 485 feet



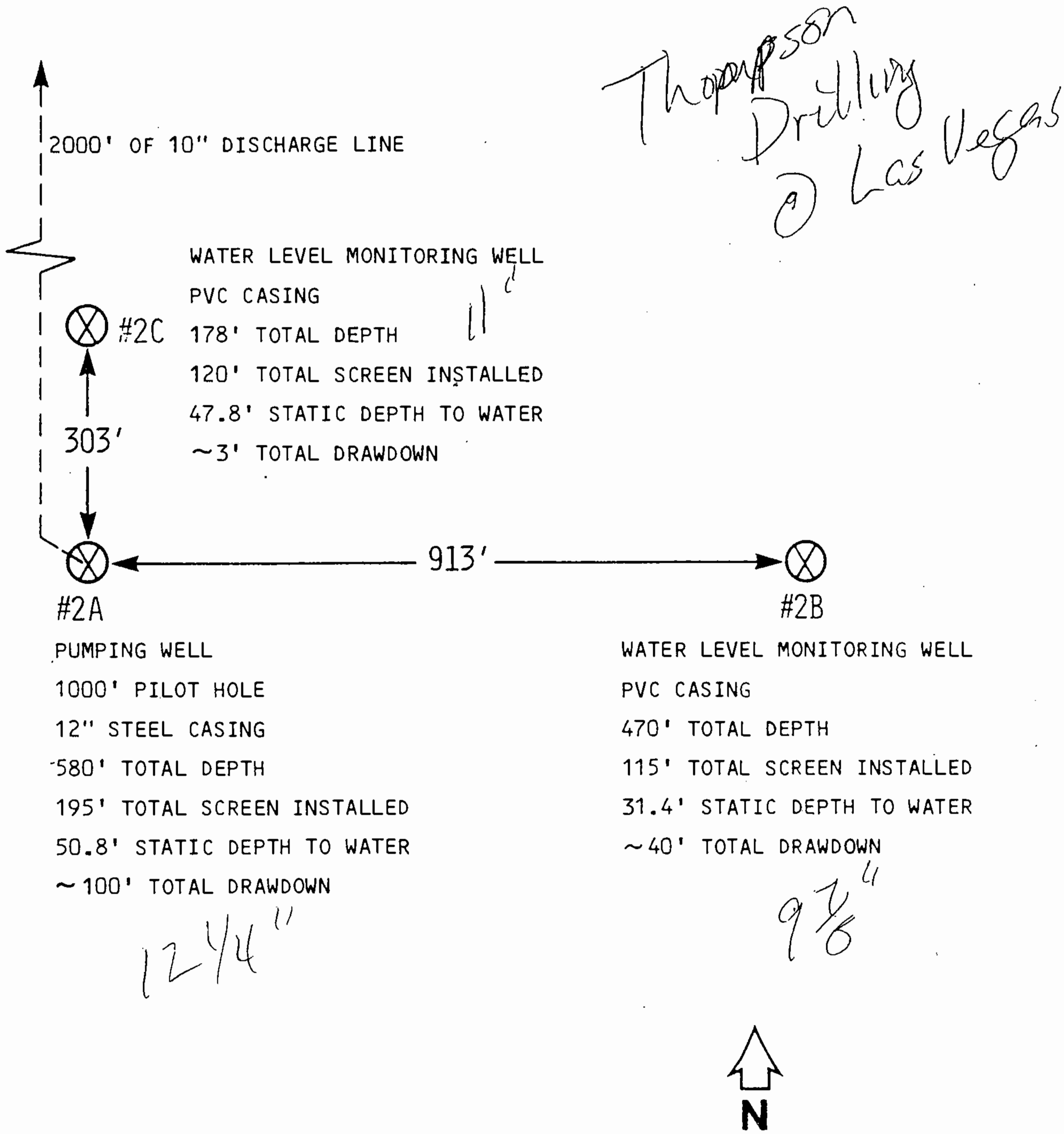
VERTICAL SCALE
1 inch = approx. 100 feet

Leeds, Hill and Jewett, Inc. December 1982

WHITE PINE POWER PROJECT

CONSTRUCTION DETAILS
WELL NO. 1-A

FIGURE 2·9



WHITE PINE POWER PROJECT

SPRING VALLEY

General Site Description and Construction Details

Leeds, Hill and Jewett, Inc. December 1982 FIGURE 2 · 10

Location: Spring Valley

Use: Observation Well

Elevation of top of well casing: 5,824.01 feet

Datum: Nov. 11, 1982 survey by Boundy & Forman, Inc., Ely, Nevada

Date constructed: Aug. 13, 1982

Conductor casing: Steel

Thickness: 5/16"

Well casing: Sch 80 PVC

Thickness: 1/3"

Well screen: Sch 80 PVC

Slot size: 0.100 inch

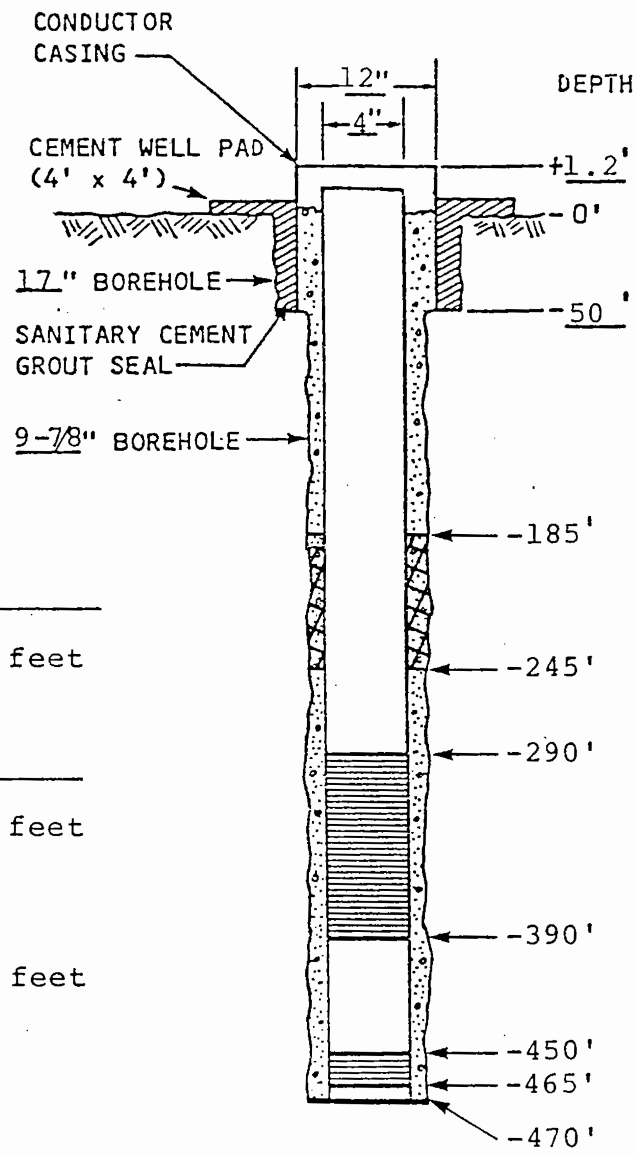
Gravel Pack Material: Local Pea Gravel

0 feet to 185 feet

Local Pea Gravel

245 feet to 470 feet

Subsurface Seals: 185 feet to 245 feet



VERTICAL SCALE
1 inch = approx. 100 feet

WHITE PINE POWER PROJECT

CONSTRUCTION DETAILS
WELL NO. 2-B

Leeds, Hill and Jewett, Inc. December 1982

FIGURE 2-11

Location: Spring Valley

Use: Observation Well

Elevation of
top of well casing: 5,838.39 feet

Datum: Nov. 11, 1982 survey by Boundy & Forman, Inc., Ely, Nevada

Date constructed: Aug. 22, 1982

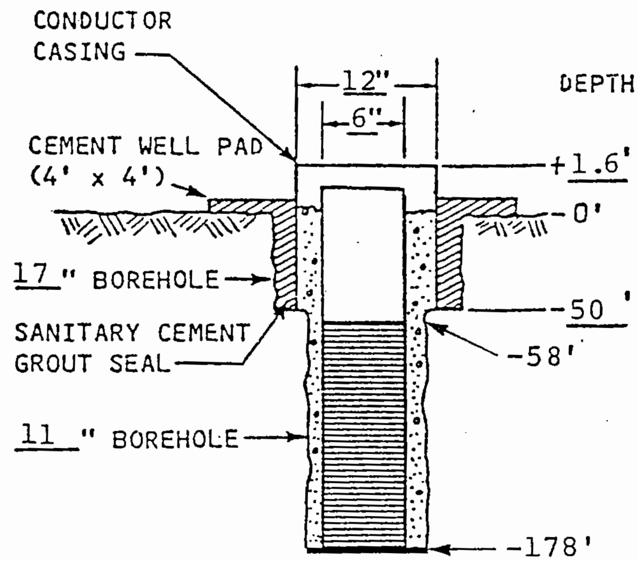
Conductor casing: Steel
Thickness: 5/16"

Well casing: Sch 80 PVC
Thickness: 1/3"

Well screen: Sch 80 PVC
Slot size: 0.100 inch

Gravel Pack
Material: Local Pea Gravel

0 feet to 178 feet



VERTICAL SCALE
1 inch = approx. 100 feet

WHITE PINE POWER PROJECT

CONSTRUCTION DETAILS
WELL NO. 2-C

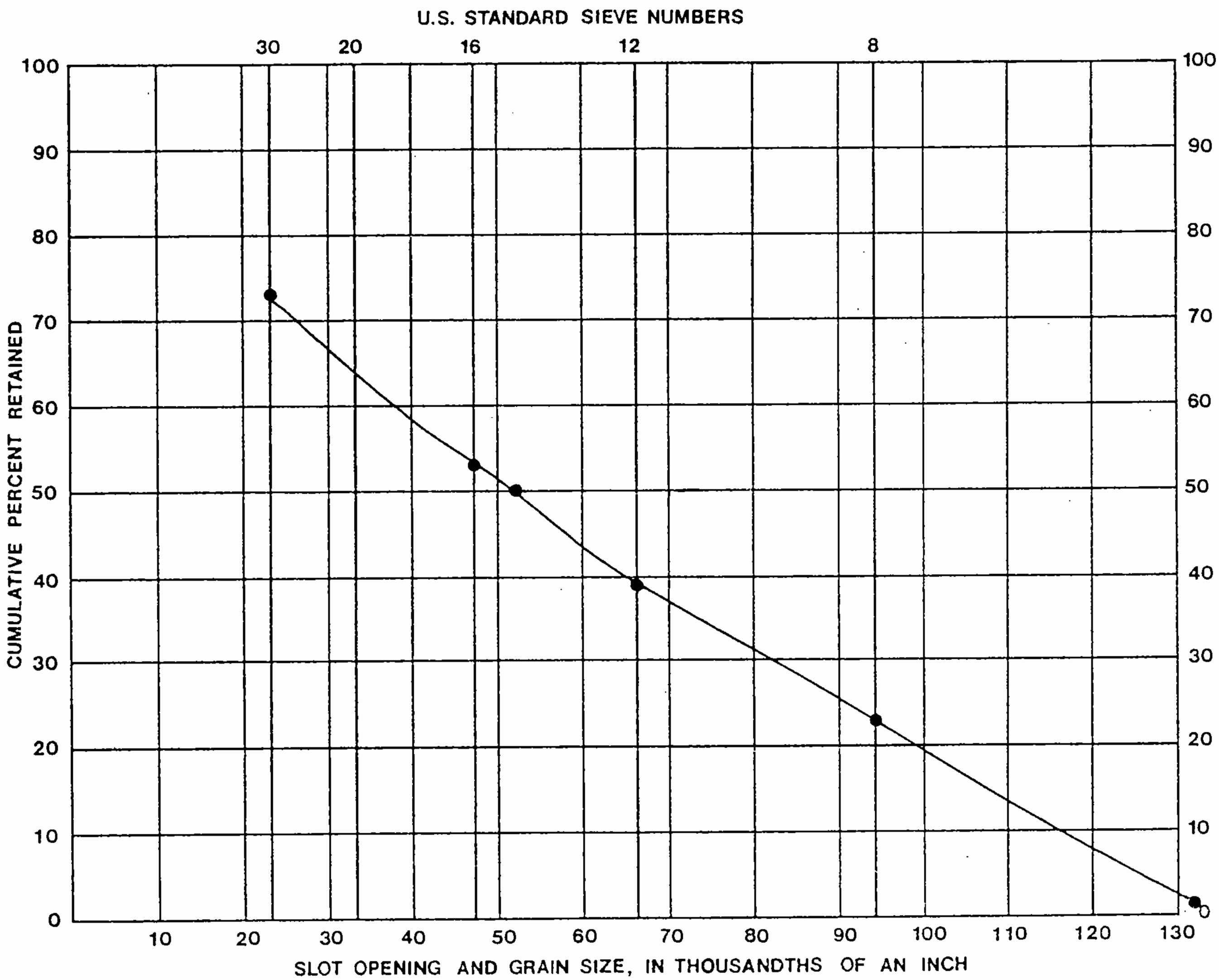
Leeds, Hill and Jewett, Inc. December 1982

FIGURE 2 · 12

Sample Location SPRING VALLEY, BOREHOLE 2A

Sampling Interval 270 to 280 feet

Date Sampled AUGUST 1982



WHITE PINE POWER PROJECT

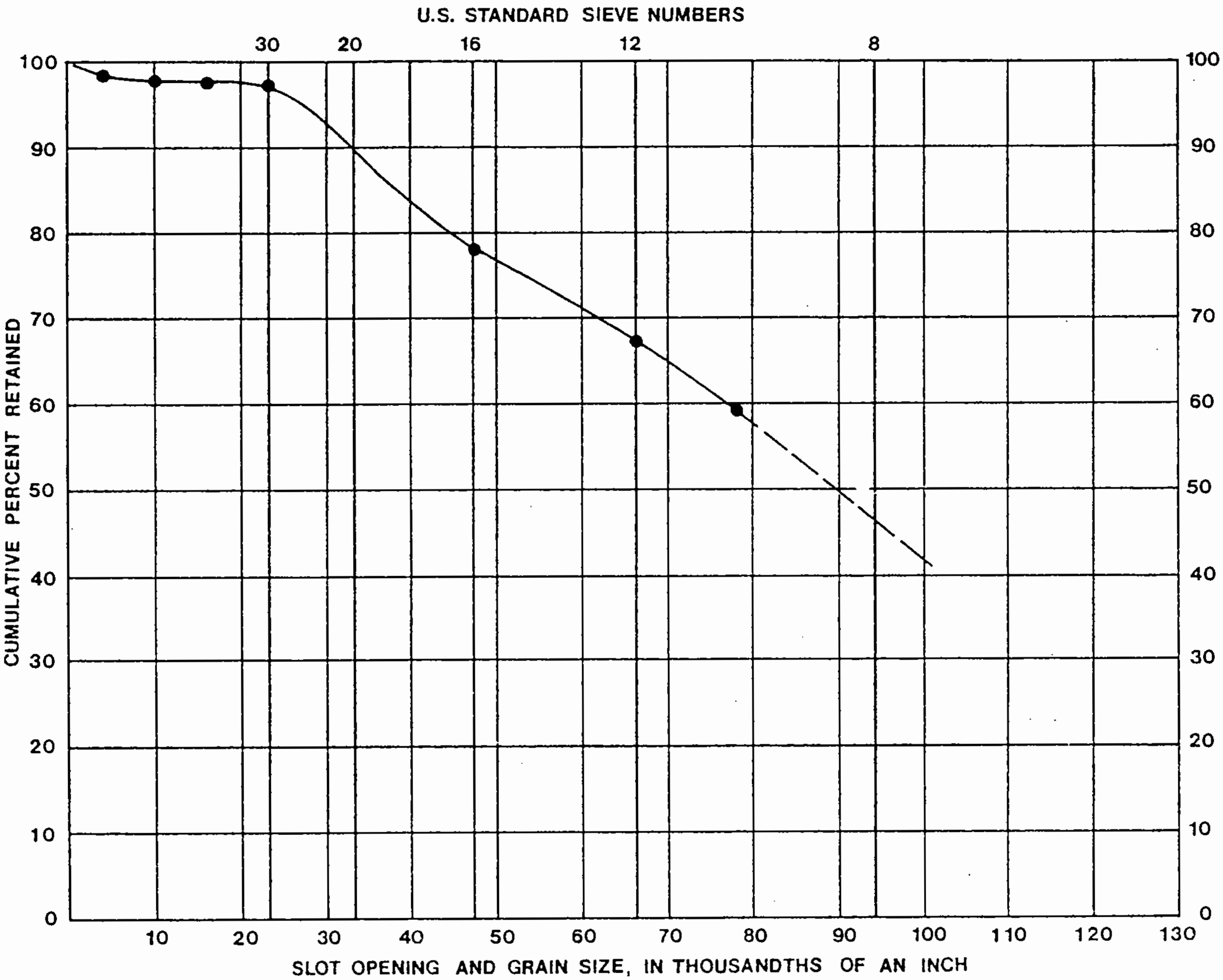
SPRING VALLEY
Grain Size Analysis
Borehole 2A

Leeds, Hill and Jewett, Inc. December 1982 FIGURE 2 · 13

Sample Location SPRING VALLEY, BOREHOLE 2A

Sampling Interval 310 to 320 feet

Date Sampled AUGUST 1982



WHITE PINE POWER PROJECT

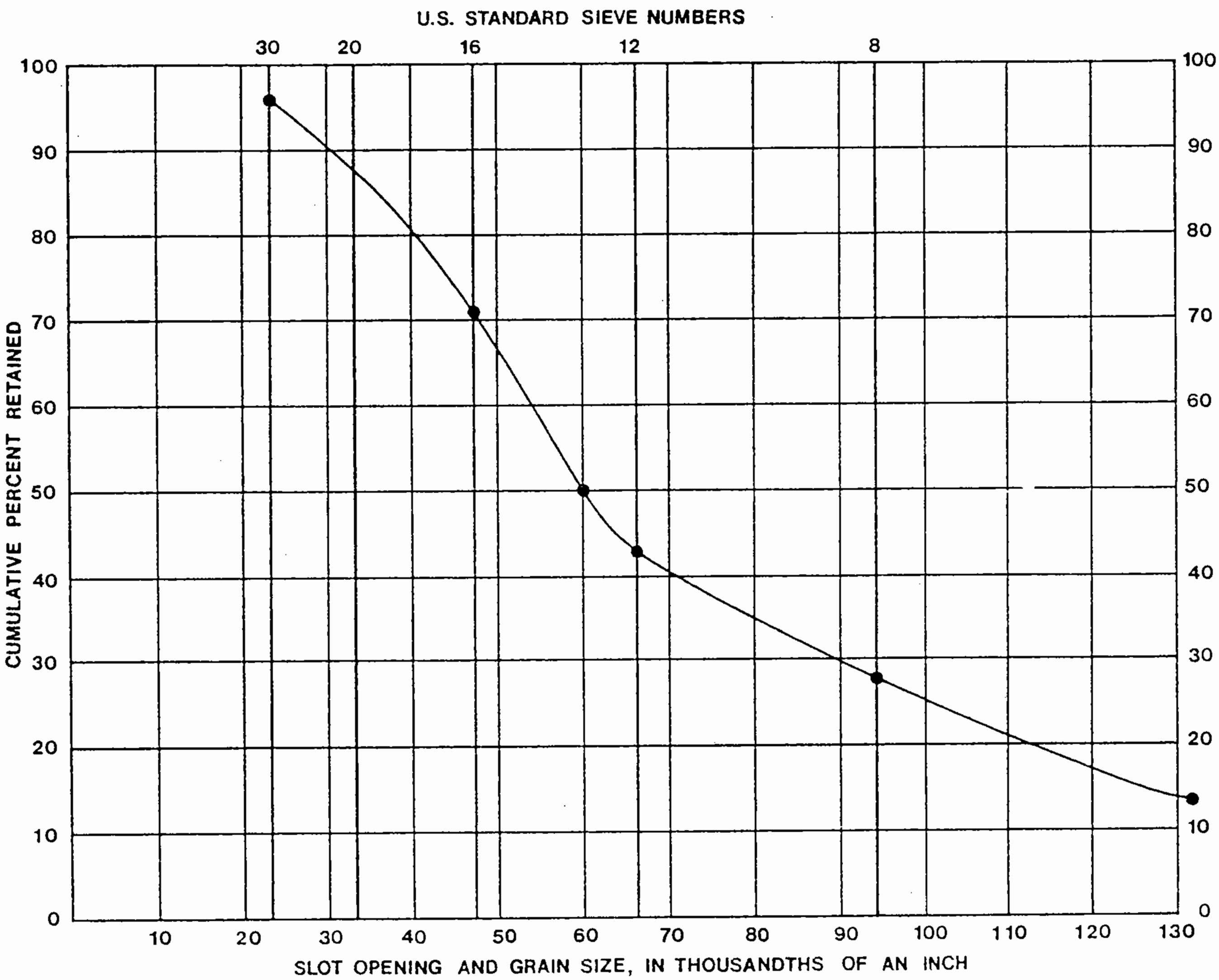
SPRING VALLEY
Grain Size Analysis
Borehole 2A

Leeds, Hill and Jewett, Inc. December 1982 FIGURE 2 · 14

Sample Location SPRING VALLEY, BOREHOLE 2A

Sampling Interval 370 to 380 feet

Date Sampled AUGUST 1982



WHITE PINE POWER PROJECT

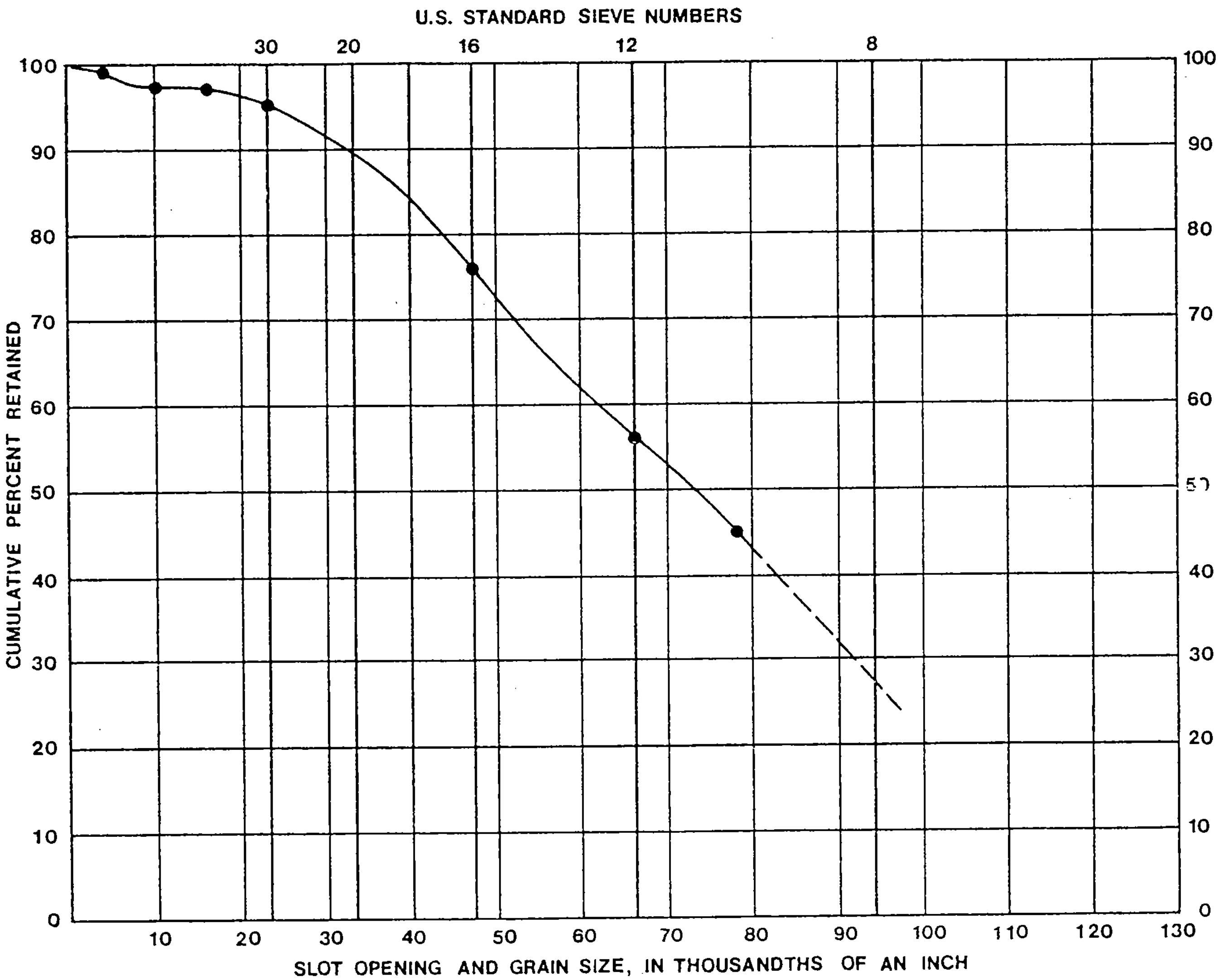
SPRING VALLEY
Grain Size Analysis
Borehole 2A

Leeds, Hill and Jewett, Inc. December 1982 FIGURE 2-15

Sample Location SPRING VALLEY, BOREHOLE 2A

Sampling Interval 560 to 570 feet

Date Sampled AUGUST 1982



WHITE PINE POWER PROJECT

SPRING VALLEY
Grain Size Analysis
Borehole 2A

Leeds, Hill and Jewett, Inc. December 1982 FIGURE 2 · 16

Location: Spring Valley

Use: Production Well

Elevation of
top of well casing: 5,842.66 feet

Datum: Nov. 11, 1982 survey by Boundy & Forman, Inc., Ely, Nevada

Date constructed: Sept. 1, 1982

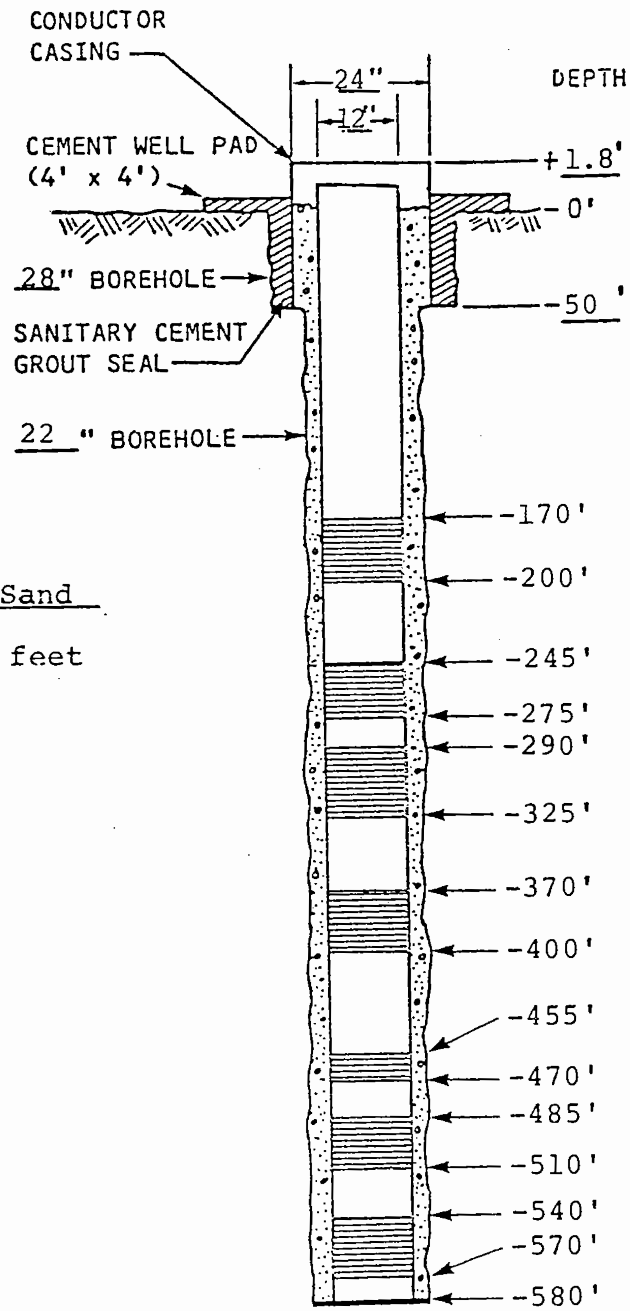
Conductor casing: Steel
Thickness: 3/8"

Well casing: Steel
Thickness: 5/16"

Well screen: Johnson Galvanized
Slot size: 0.080 inch

Gravel Pack
Material: 8-12 Colorado Silica Sand

0 feet to 580 feet



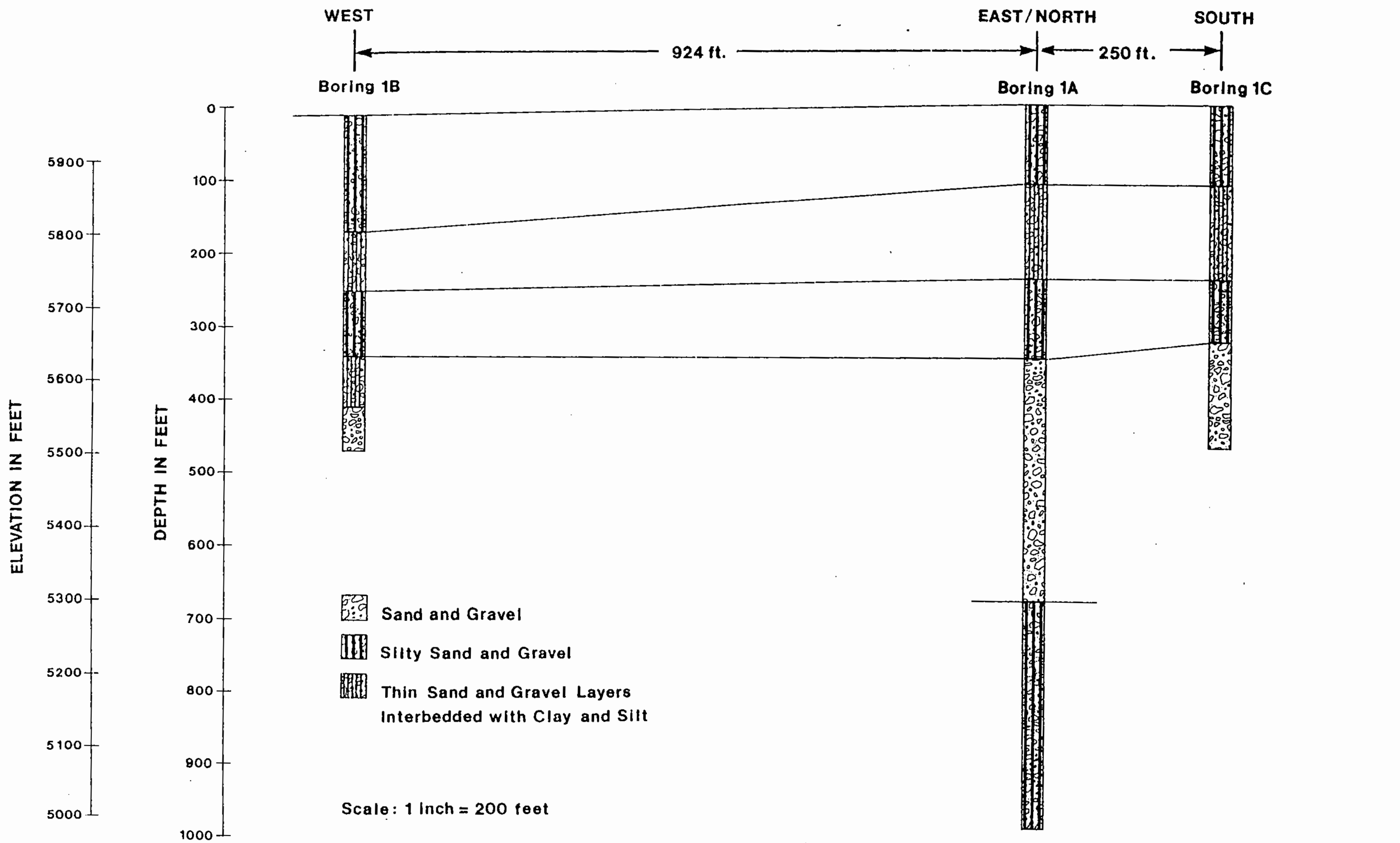
VERTICAL SCALE
1 inch = approx. 100 feet

Leeds, Hill and Jewett, Inc. December 1982

WHITE PINE POWER PROJECT

CONSTRUCTION DETAILS
WELL NO. 2-A

FIGURE 2·17

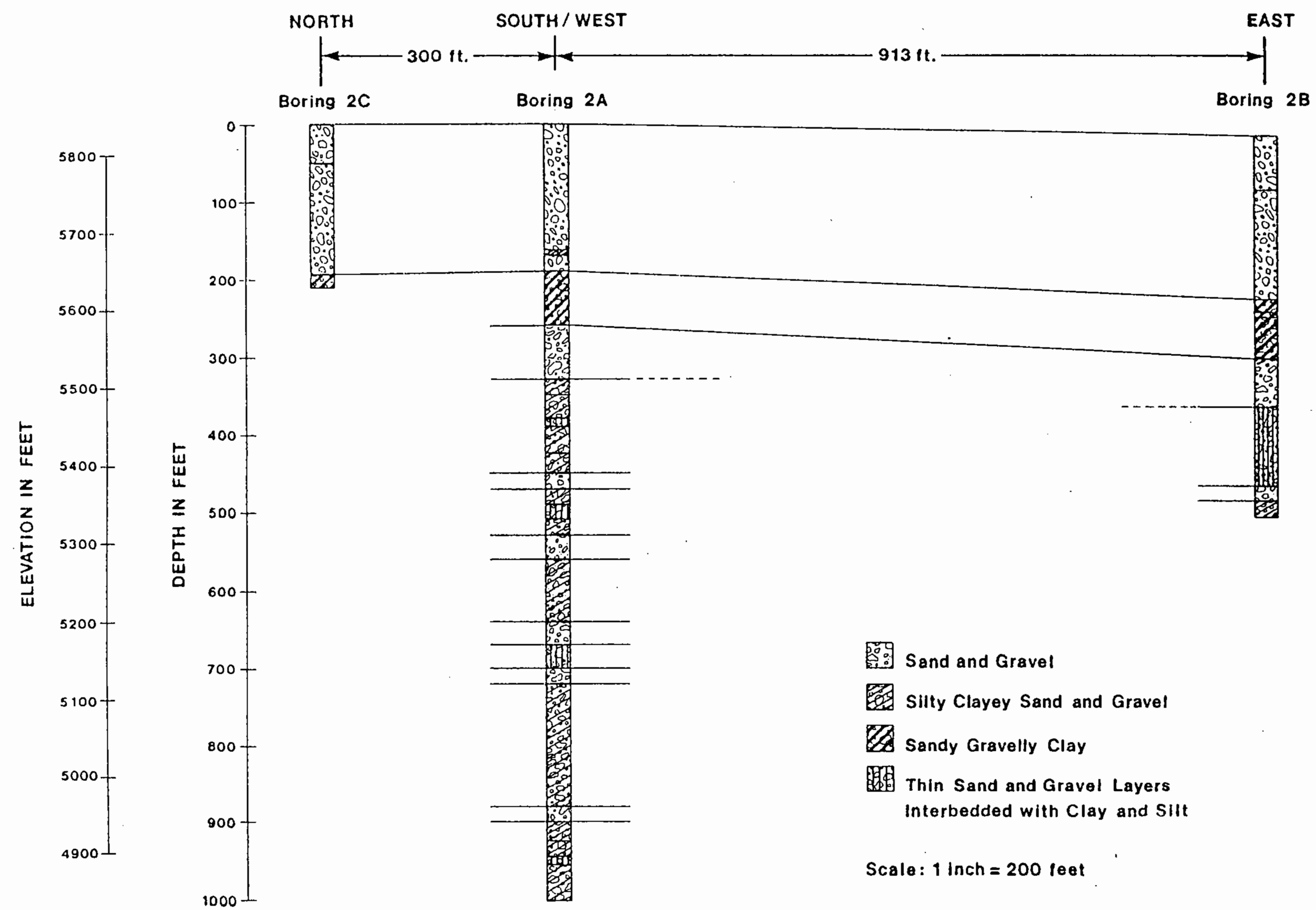


WHITE PINE POWER PROJECT

STEPTOE VALLEY

Geologic Cross-Section

Leeds, Hill and Jewett, Inc. December 1982 FIGURE 2-18



NOTE: Geologic contacts in Borehole 2A below approximately 300 feet have been adjusted for lag time between drill bit penetration and sample uphole travel time based on drilling penetration rates and observations recorded during drilling operations.

Leeds, Hill and Jewett, Inc.

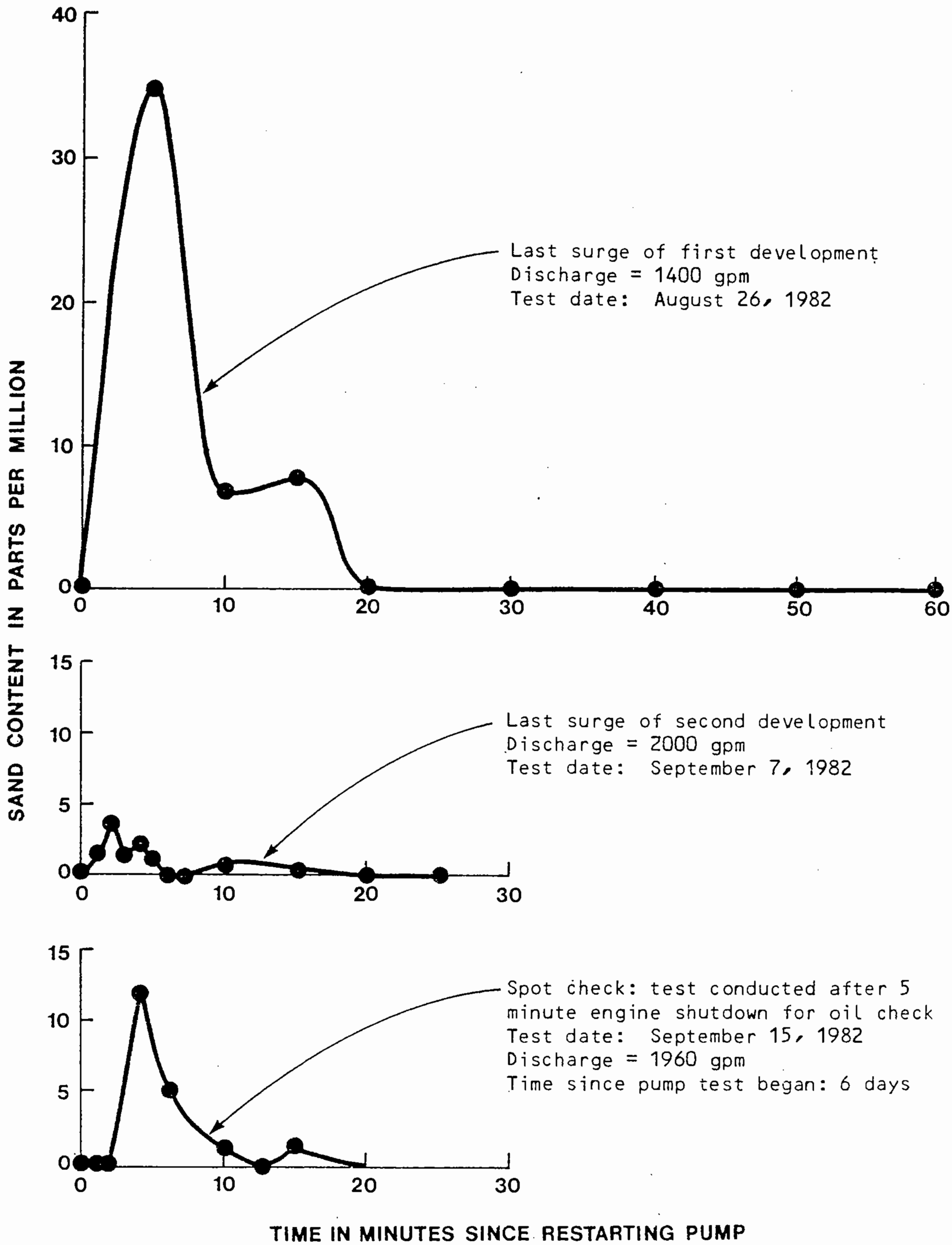
December 1982

WHITE PINE POWER PROJECT

SPRING VALLEY

Geologic Cross-Section

FIGURE 2·19

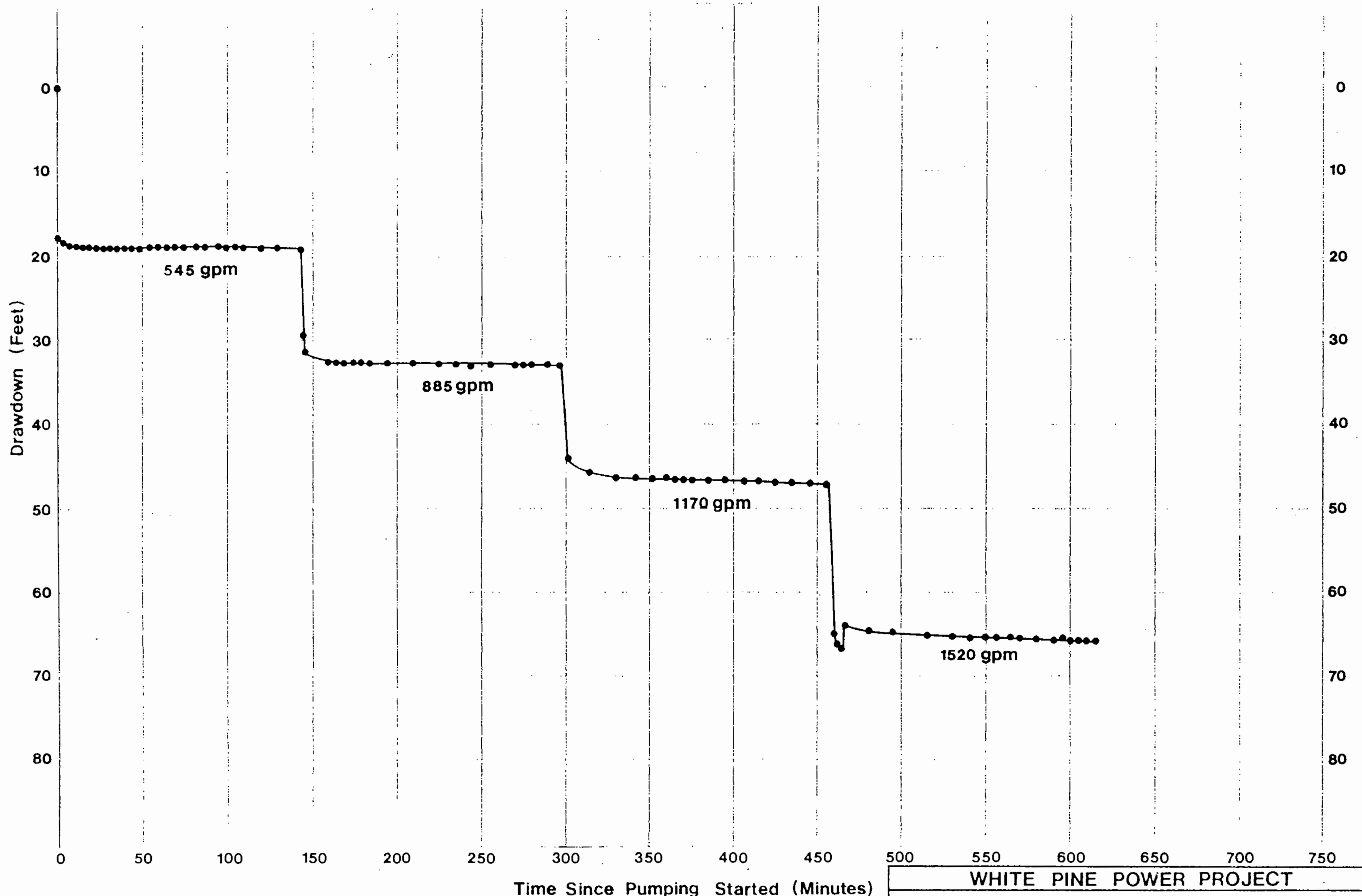


WHITE PINE POWER PROJECT

STEPTOE VALLEY

Sand Content Graphs

Leeds, Hill and Jewett, Inc. December 1982 FIGURE 3-1



Leeds, Hill and Jewett, Inc.

December 1982

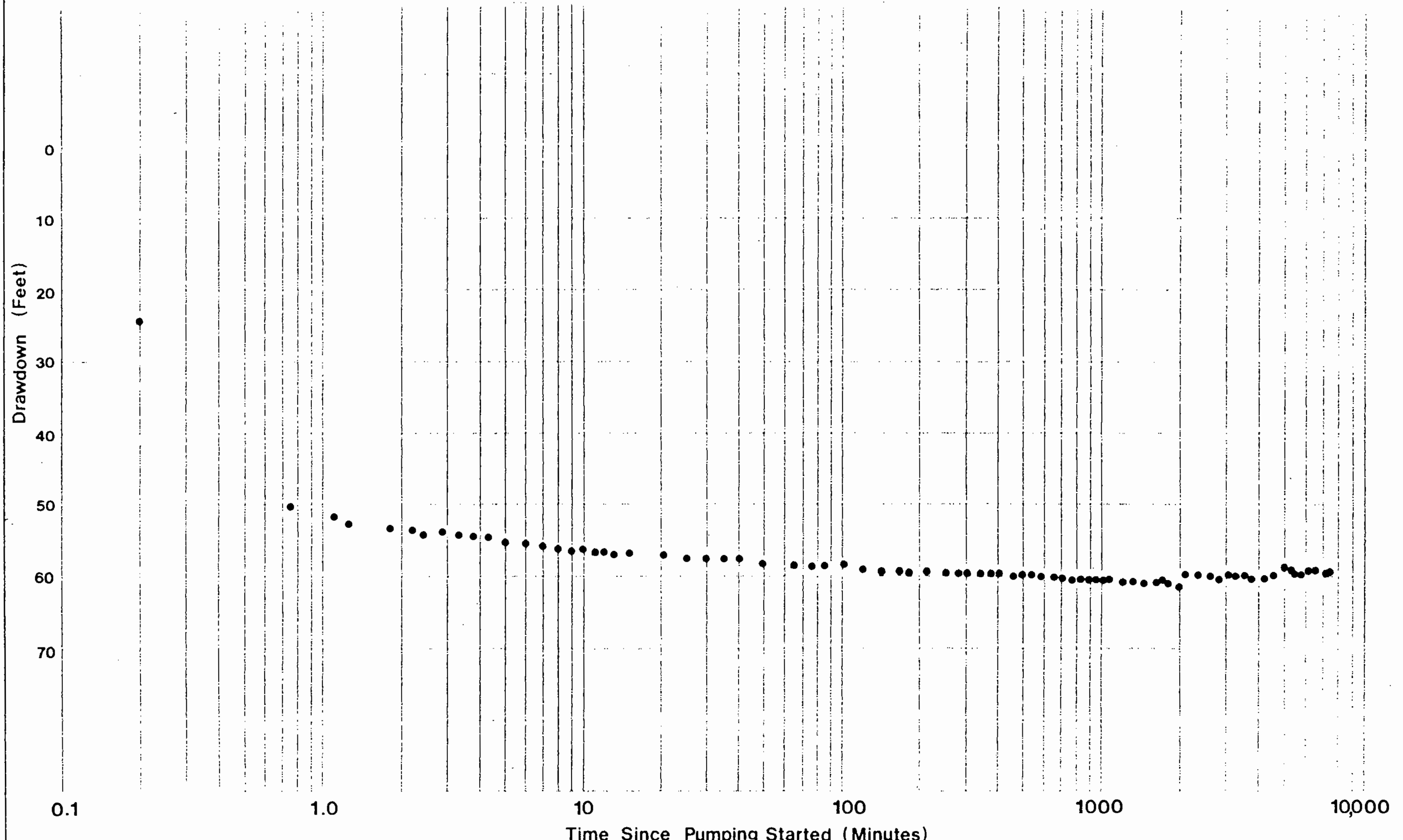
WHITE PINE POWER PROJECT

STEPTOE VALLEY

Variable Rate Pump Test 1

Production Well 1A

FIGURE 3·2



WHITE PINE POWER PROJECT

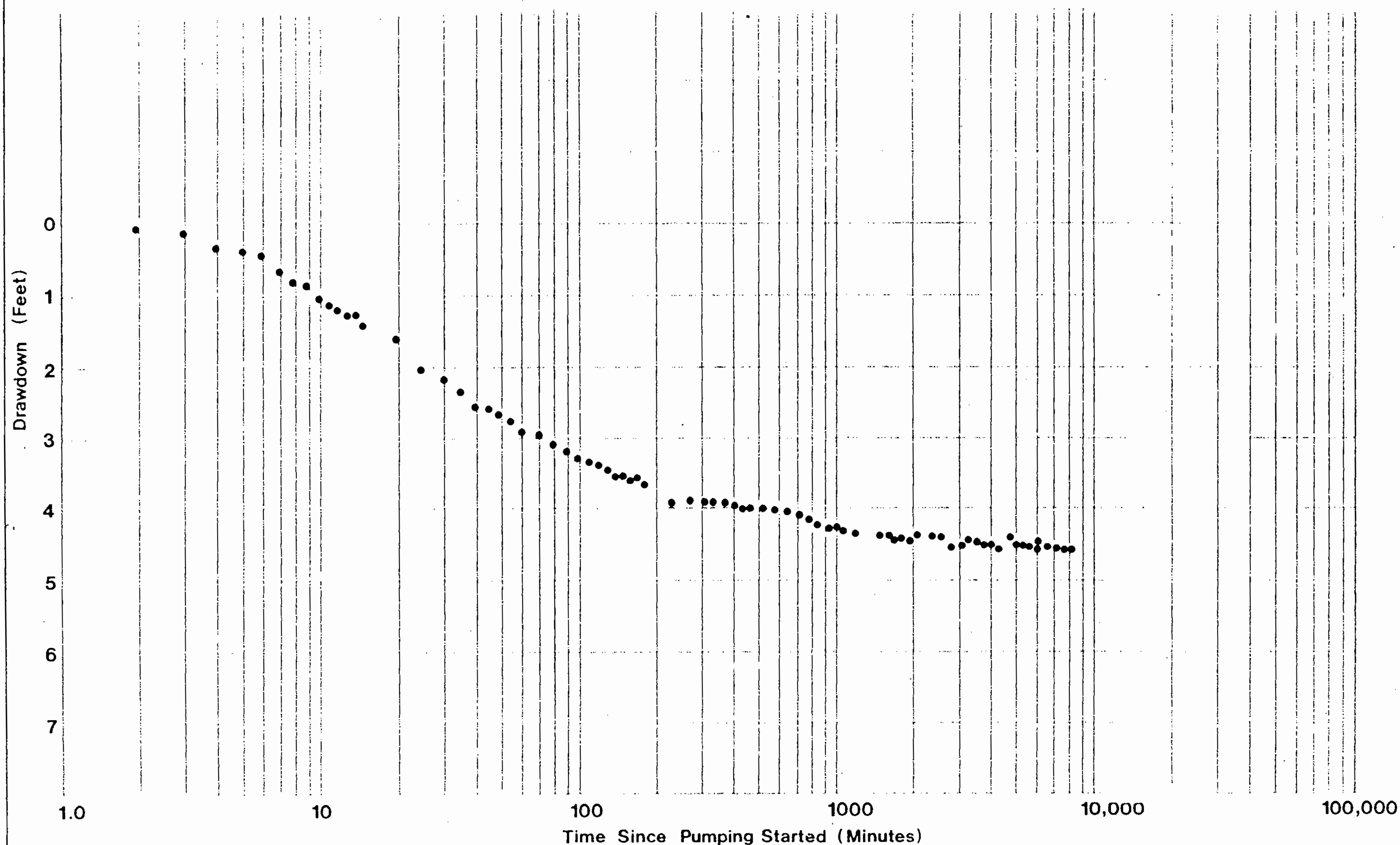
STEPTOE VALLEY

Time-Drawdown Curve from Production Well 1A Constant Rate Test 1

FIGURE 3 · 3

Leeds, Hill and Jewett, Inc.

December 1982



WHITE PINE POWER PROJECT

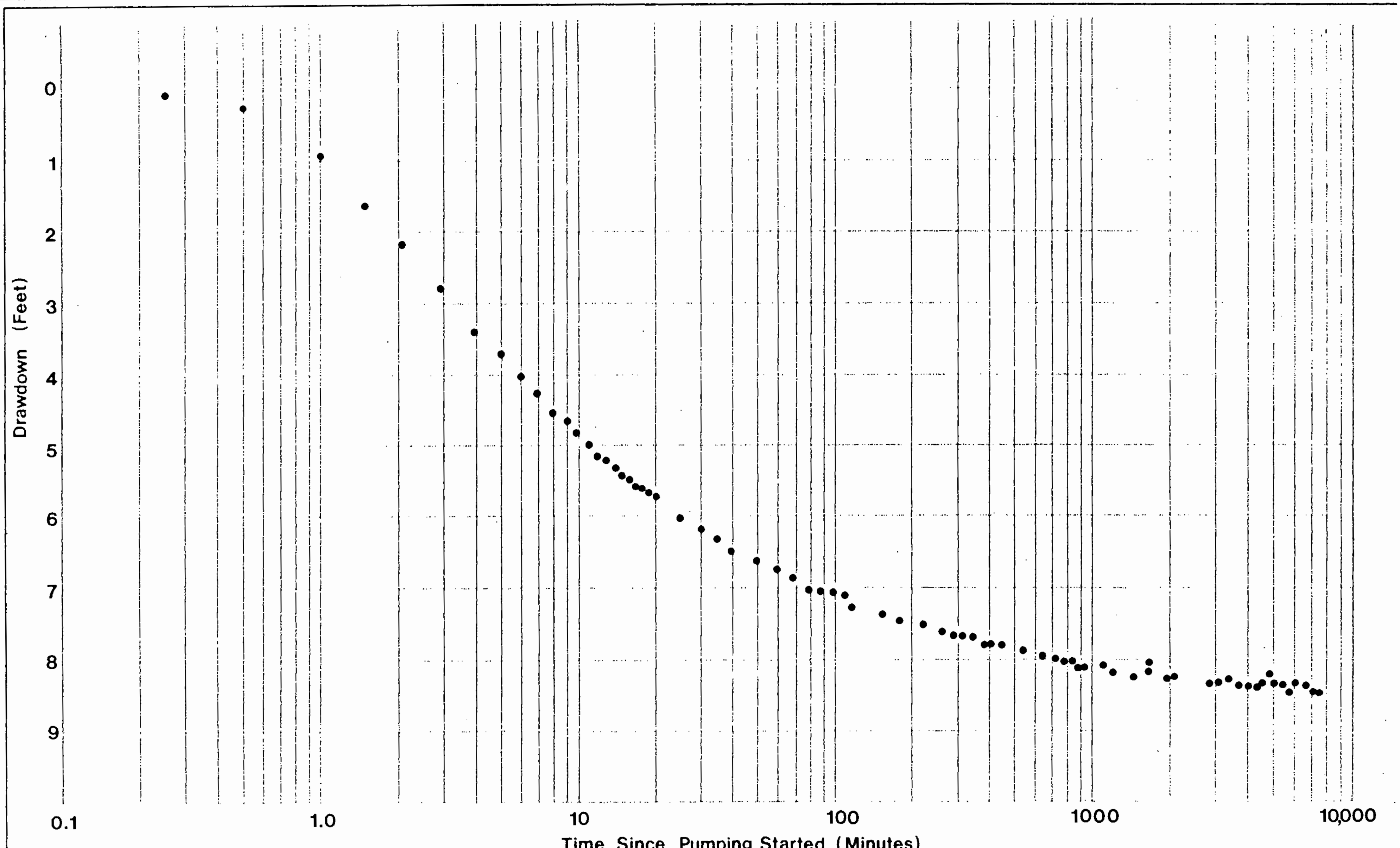
STEPTOE VALLEY

Time-Drawdown Curve from
Observation Well 1B
Constant Rate Test 1

Leeds, Hill and Jewett, Inc.

December 1982

FIGURE 3·4



WHITE PINE POWER PROJECT

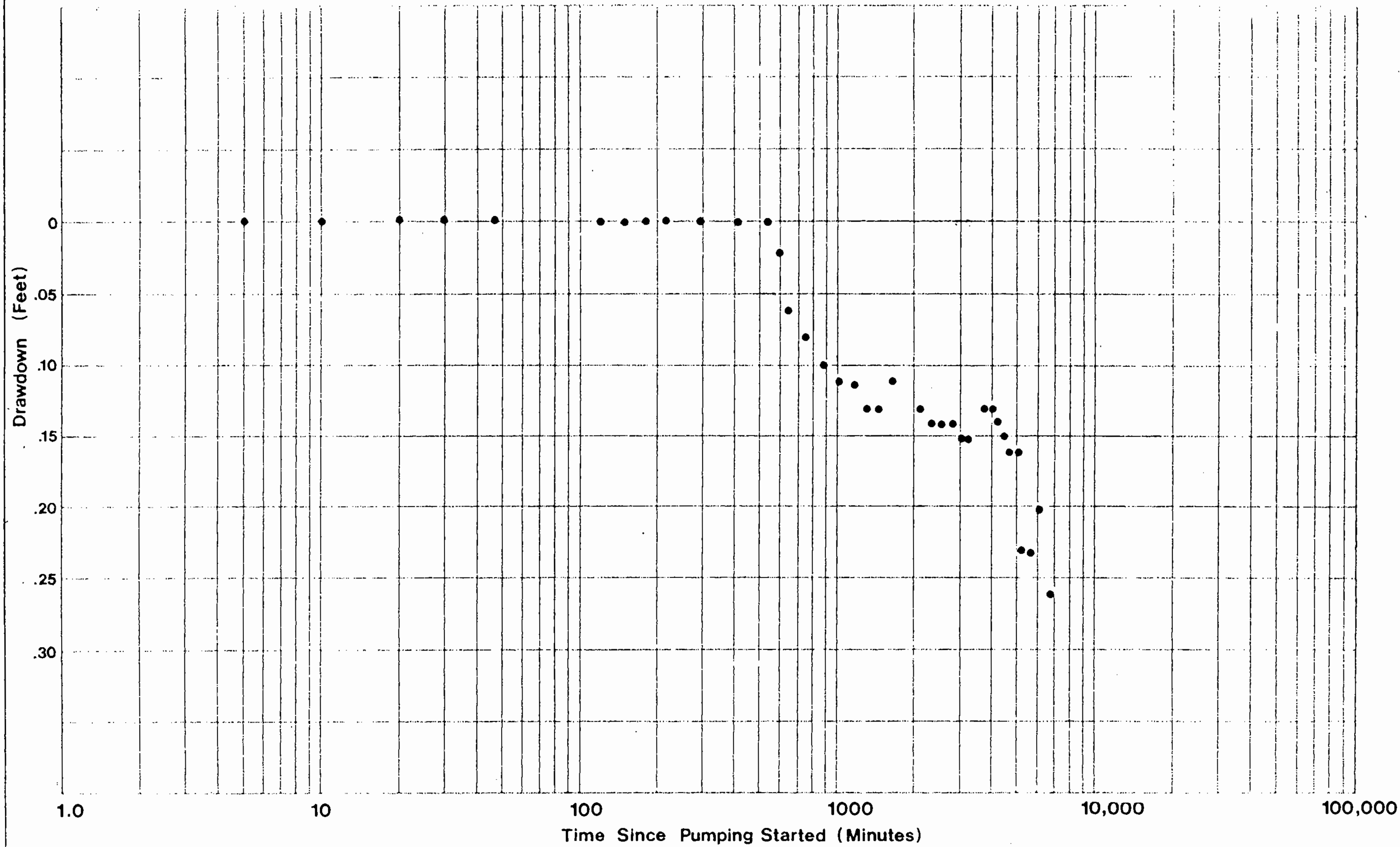
STEPTOE VALLEY

Time-Drawdown Curve from
Observation Well 1C
Constant Rate Test 1

Leeds, Hill and Jewett, Inc.

December 1982

FIGURE 3·5



Leeds, Hill and Jewett, Inc.

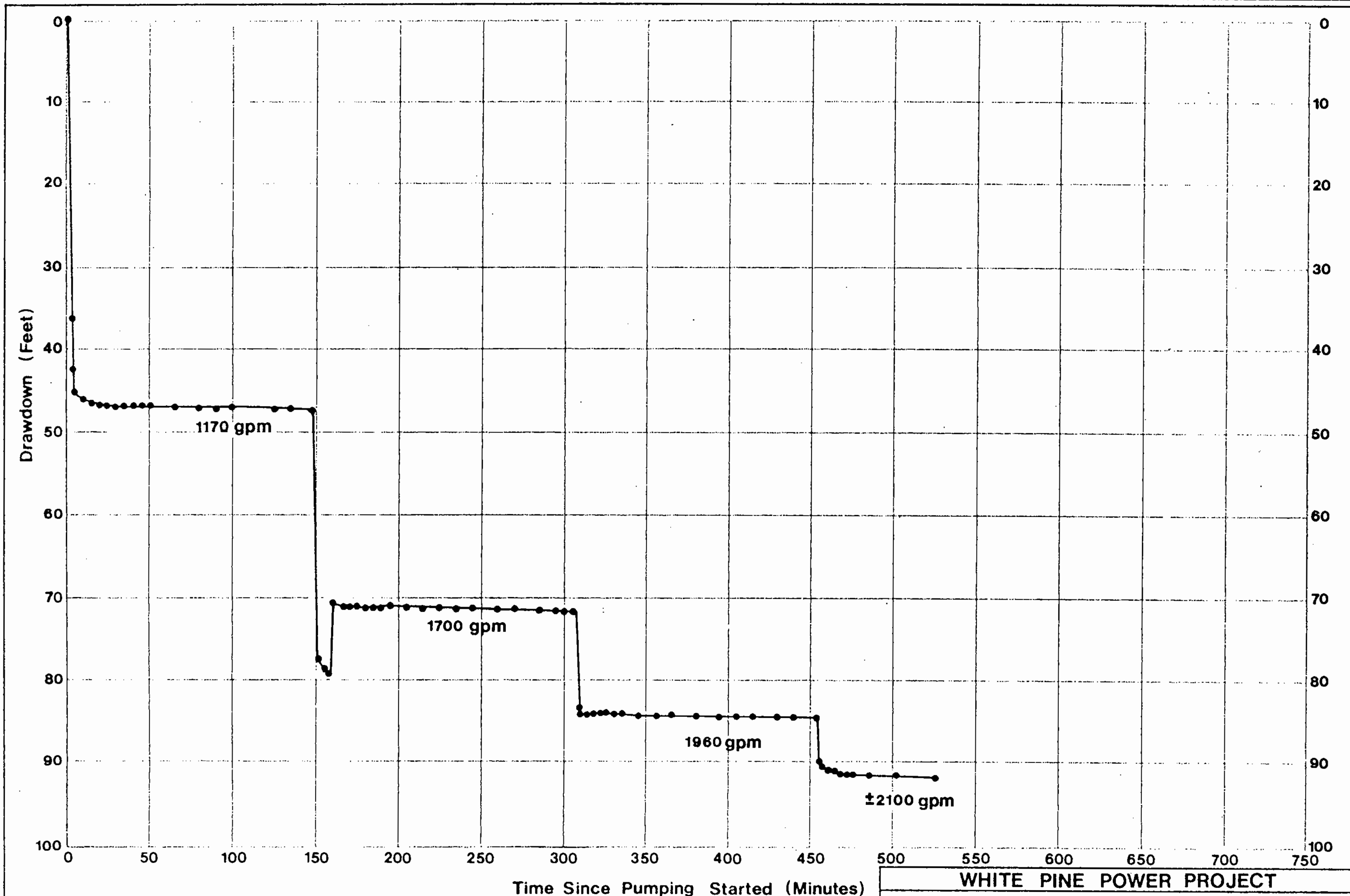
December 1982

WHITE PINE POWER PROJECT

STEPTOE VALLEY

Time-Drawdown Curve from Water Supply Well Constant Rate Test 1

FIGURE 3·6



WHITE PINE POWER PROJECT

STEPTOE VALLEY

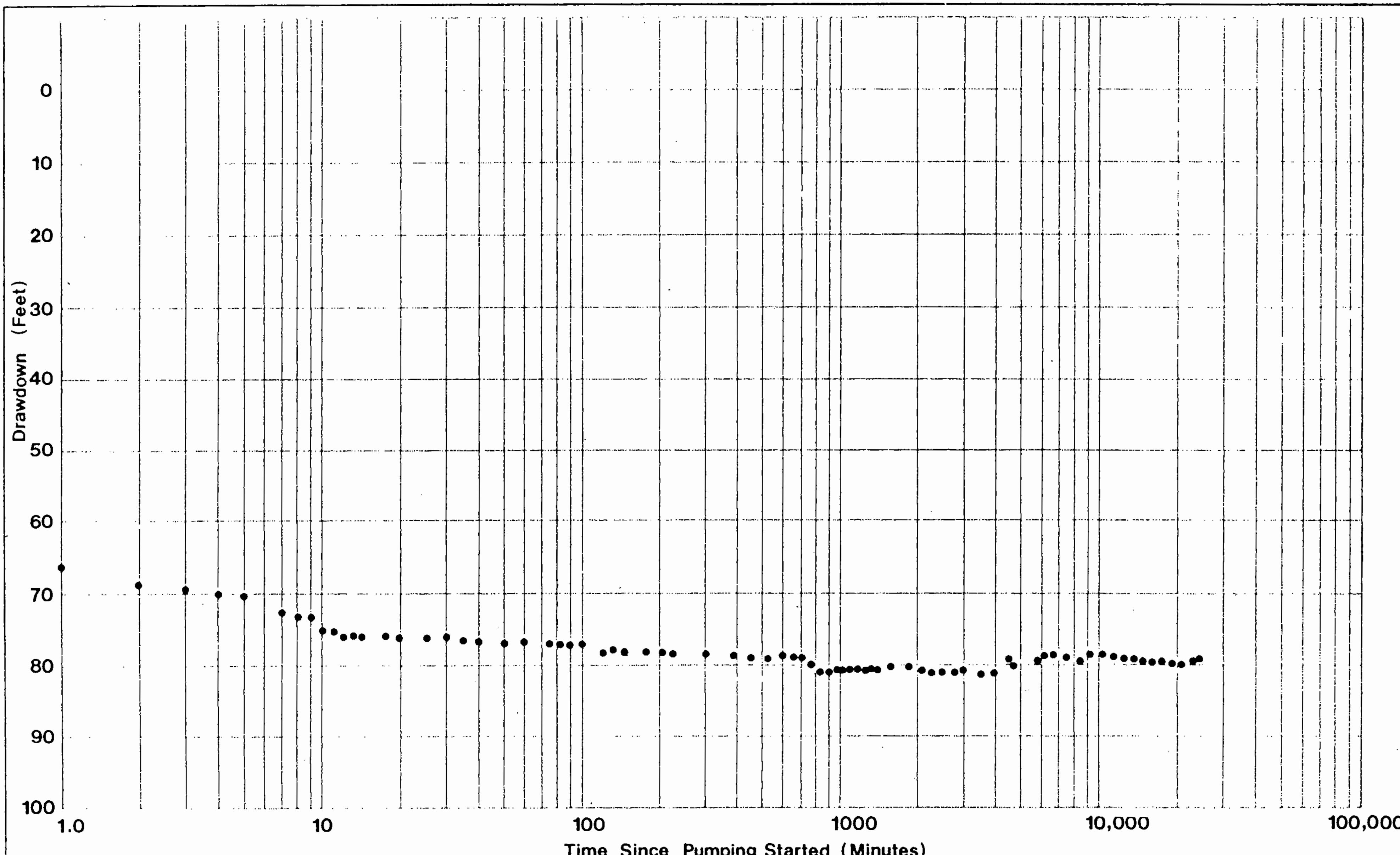
Variable Rate Pump Test 2

Production Well 1A

FIGURE 3·7

Leeds, Hill and Jewett, Inc.

December 1982



WHITE PINE POWER PROJECT

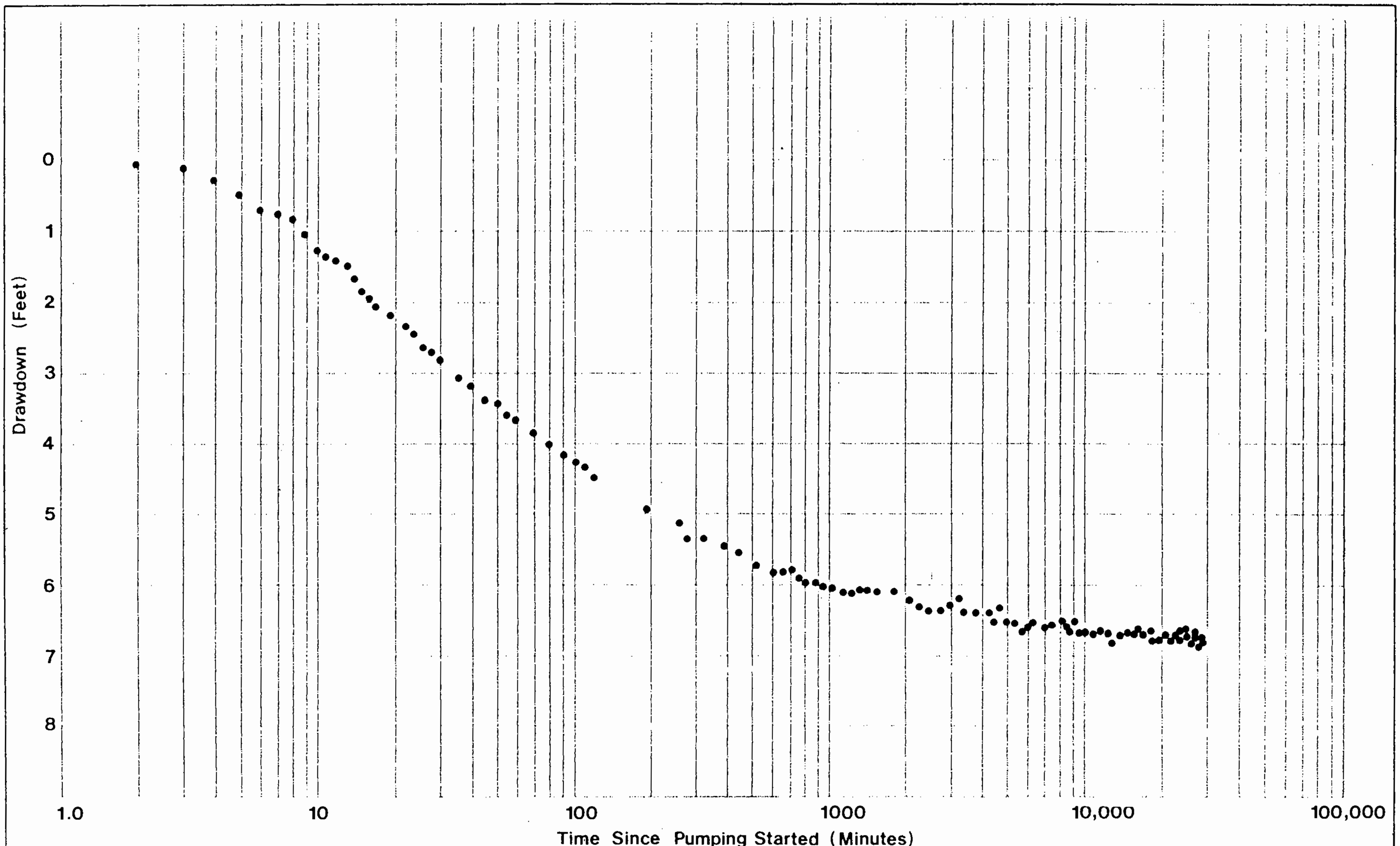
STEPTOE VALLEY

Time-Drawdown Curve from Production Well 1A Constant Rate Test 2

FIGURE 3.8

Leeds, Hill and Jewett, Inc.

December 1982



WHITE PINE POWER PROJECT

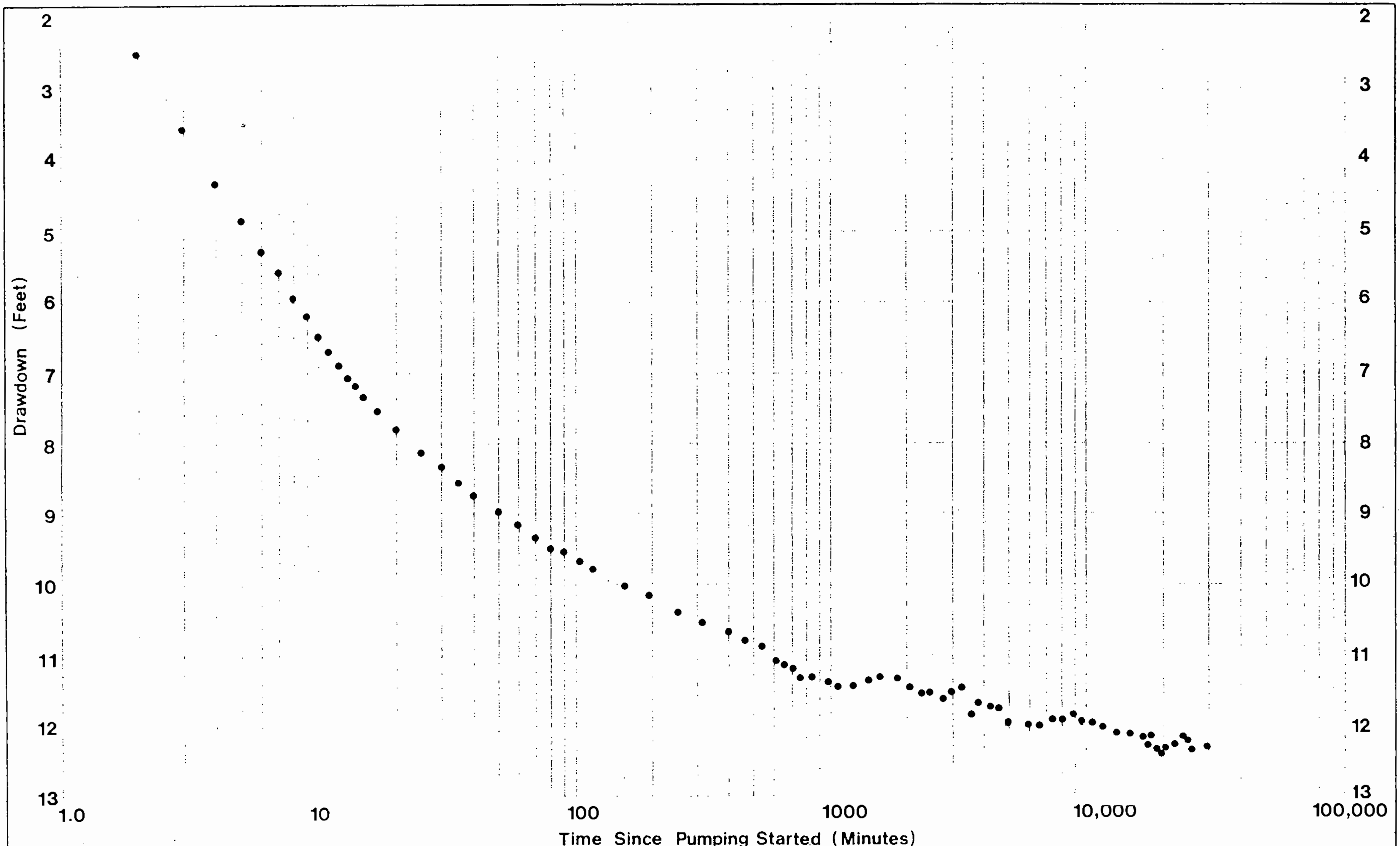
STEPTOE VALLEY

Time-Drawdown Curve from Observation Well 1B Constant Rate Test 2

Leeds, Hill and Jewett, Inc.

December 1982

FIGURE 3.9



WHITE PINE POWER PROJECT

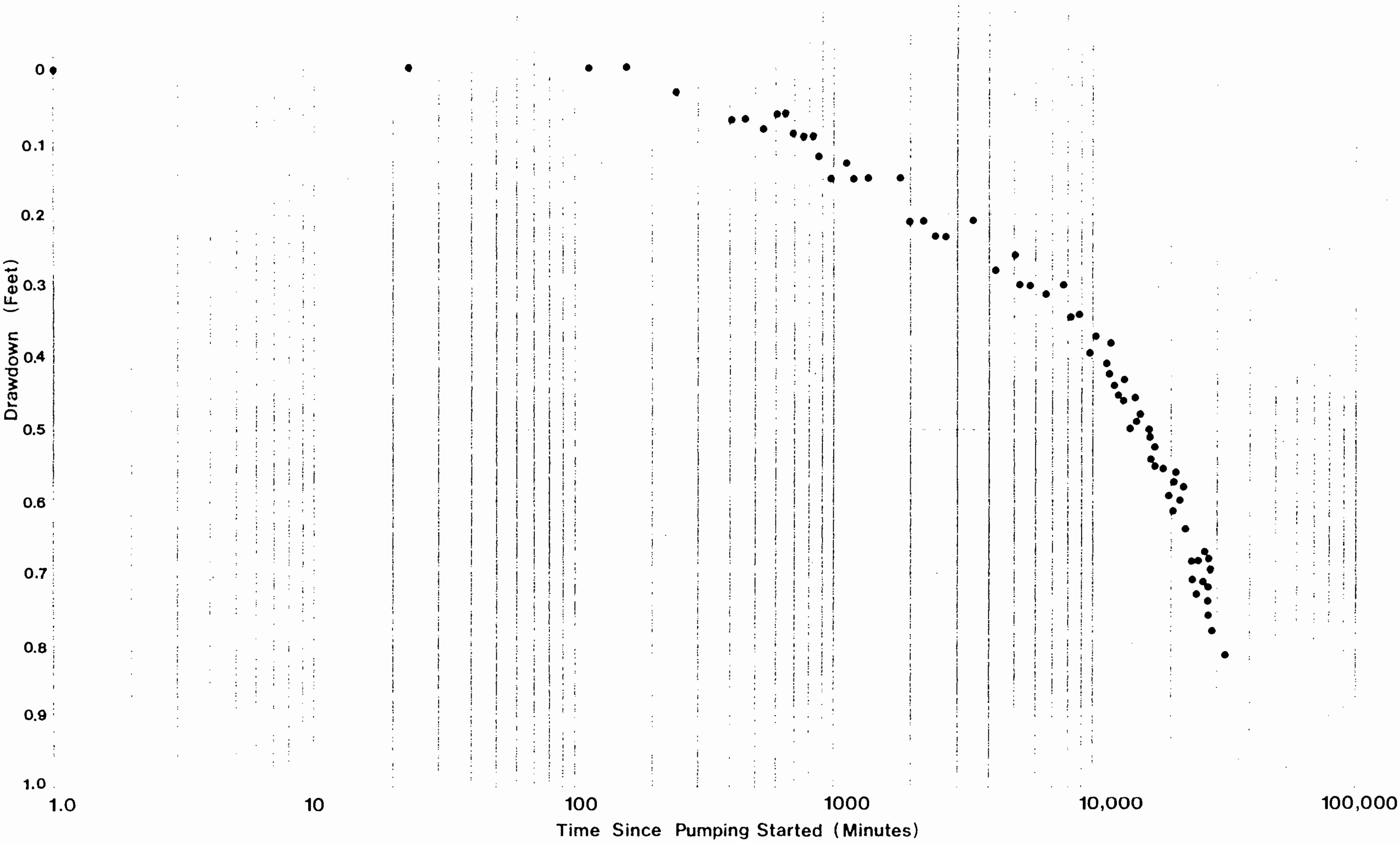
STEPTOE VALLEY

Time Drawdown Curve from Observation Well 1C Constant Rate Test 2

Leeds, Hill and Jewett, Inc.

December 1982

FIGURE 3·10



WHITE PINE POWER PROJECT

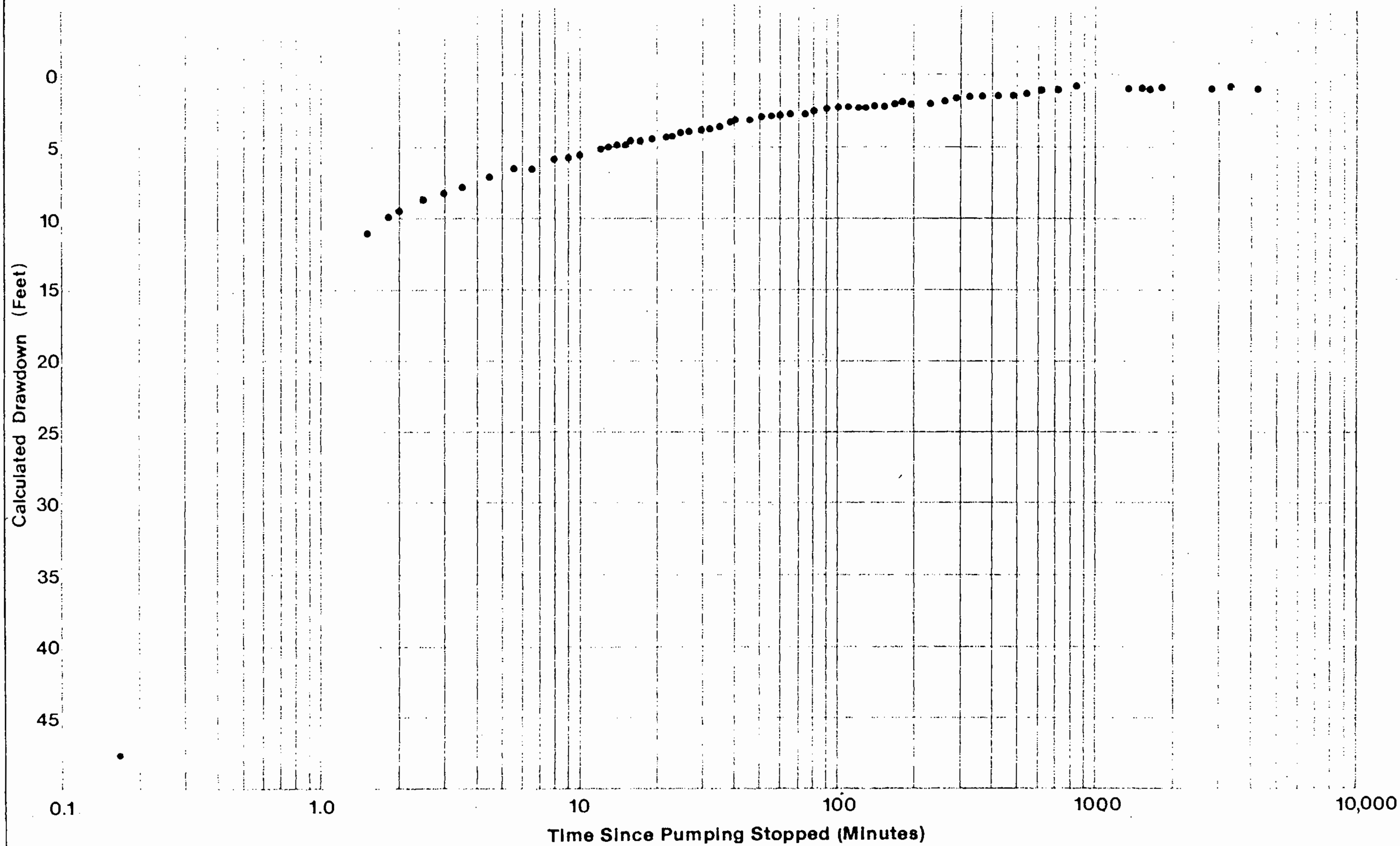
STEPTOE VALLEY

Time-Drawdown Curve from Water Supply Well Constant Rate Test 2

FIGURE 3·11

Leeds, Hill and Jewett, Inc.

December 1982



WHITE PINE POWER PROJECT

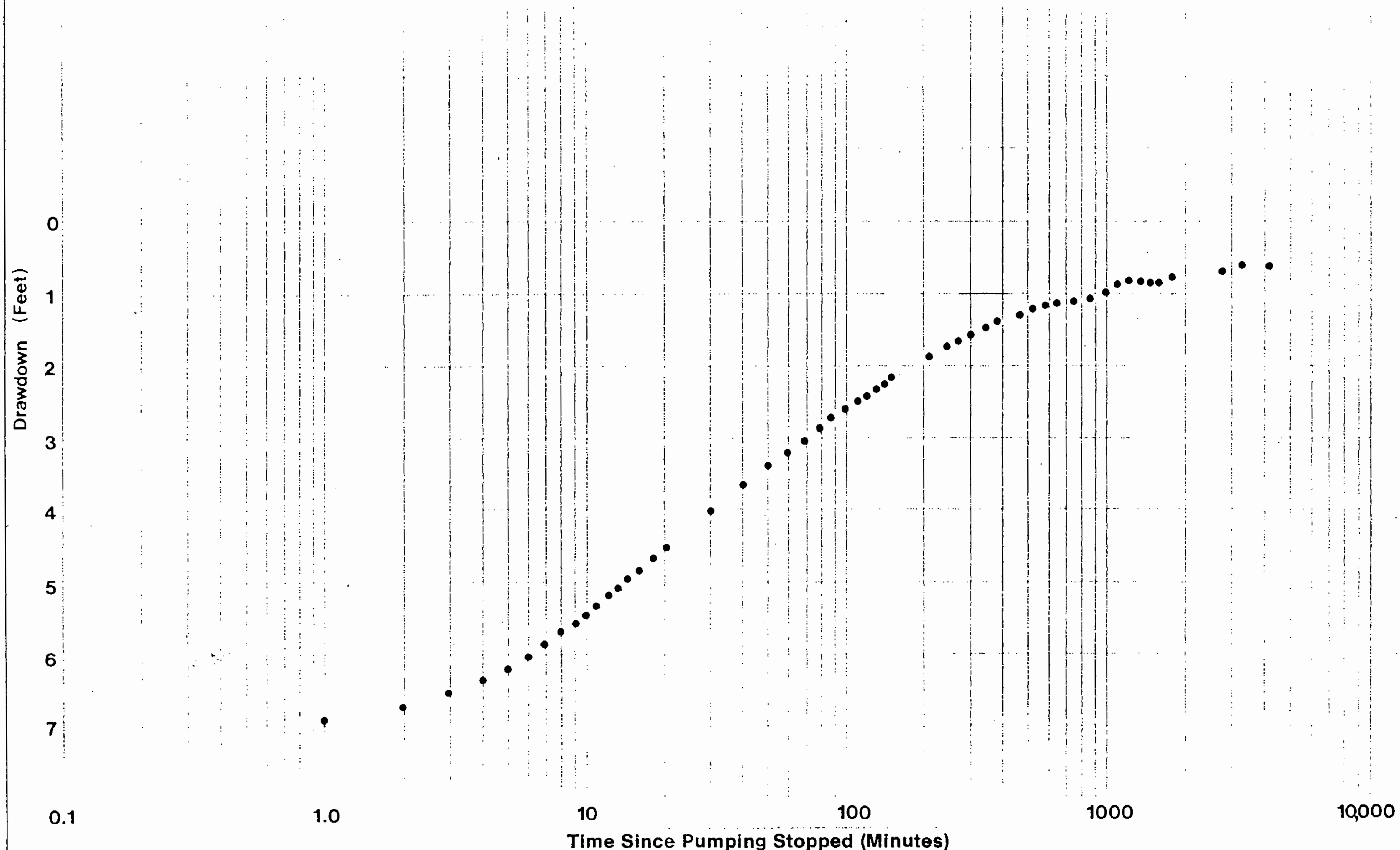
STEPTOE VALLEY

Time-Recovery Curve from Production Well 1A Recovery Test 2

Leeds, Hill and Jewett, Inc.

December 1982

FIGURE 3·12

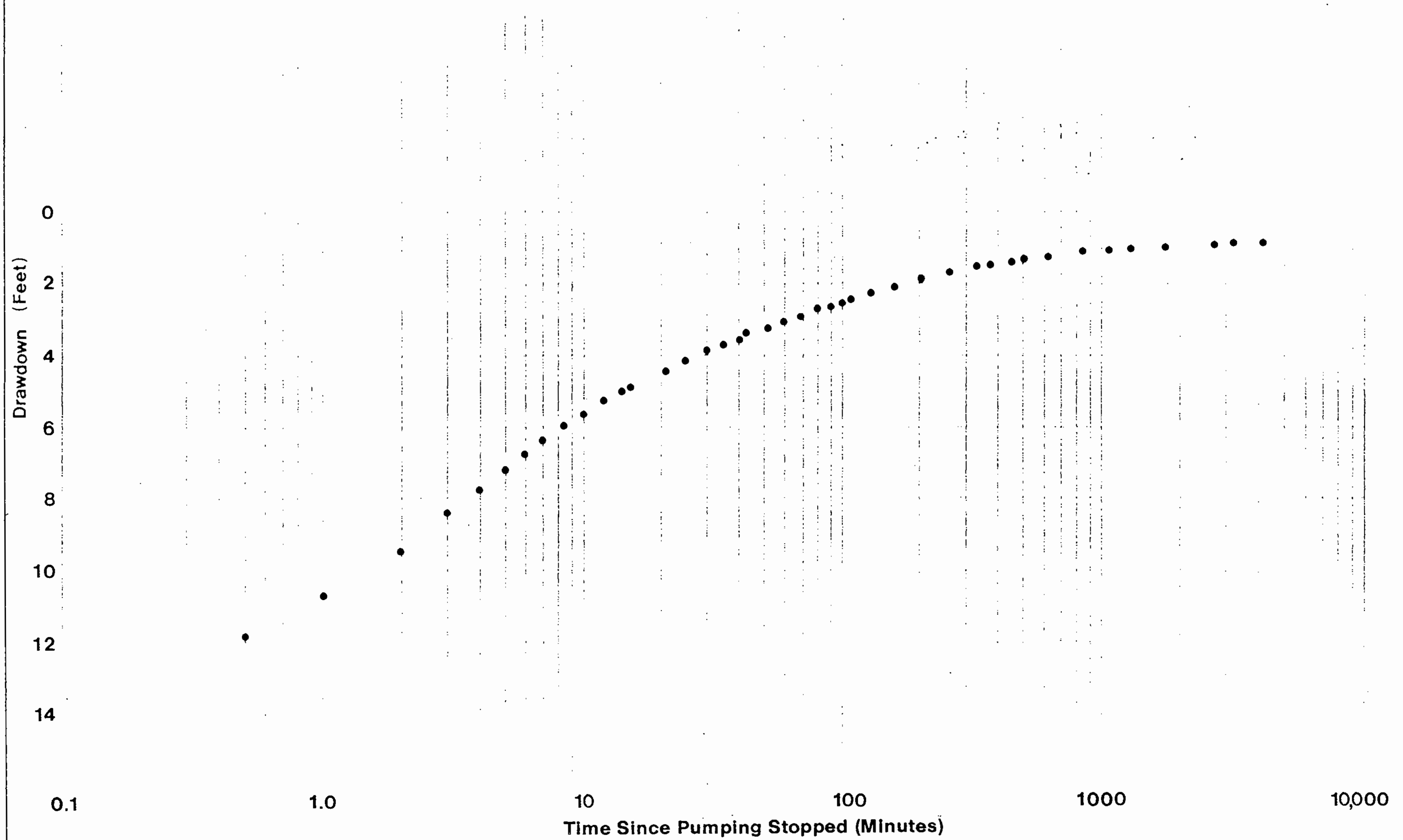


WHITE PINE POWER PROJECT
STEPTOE VALLEY
Time-Recovery Curve from Observation Well 1B Recovery Test 2

Leeds, Hill and Jewett, Inc.

December 1982

FIGURE 3·13



WHITE PINE POWER PROJECT

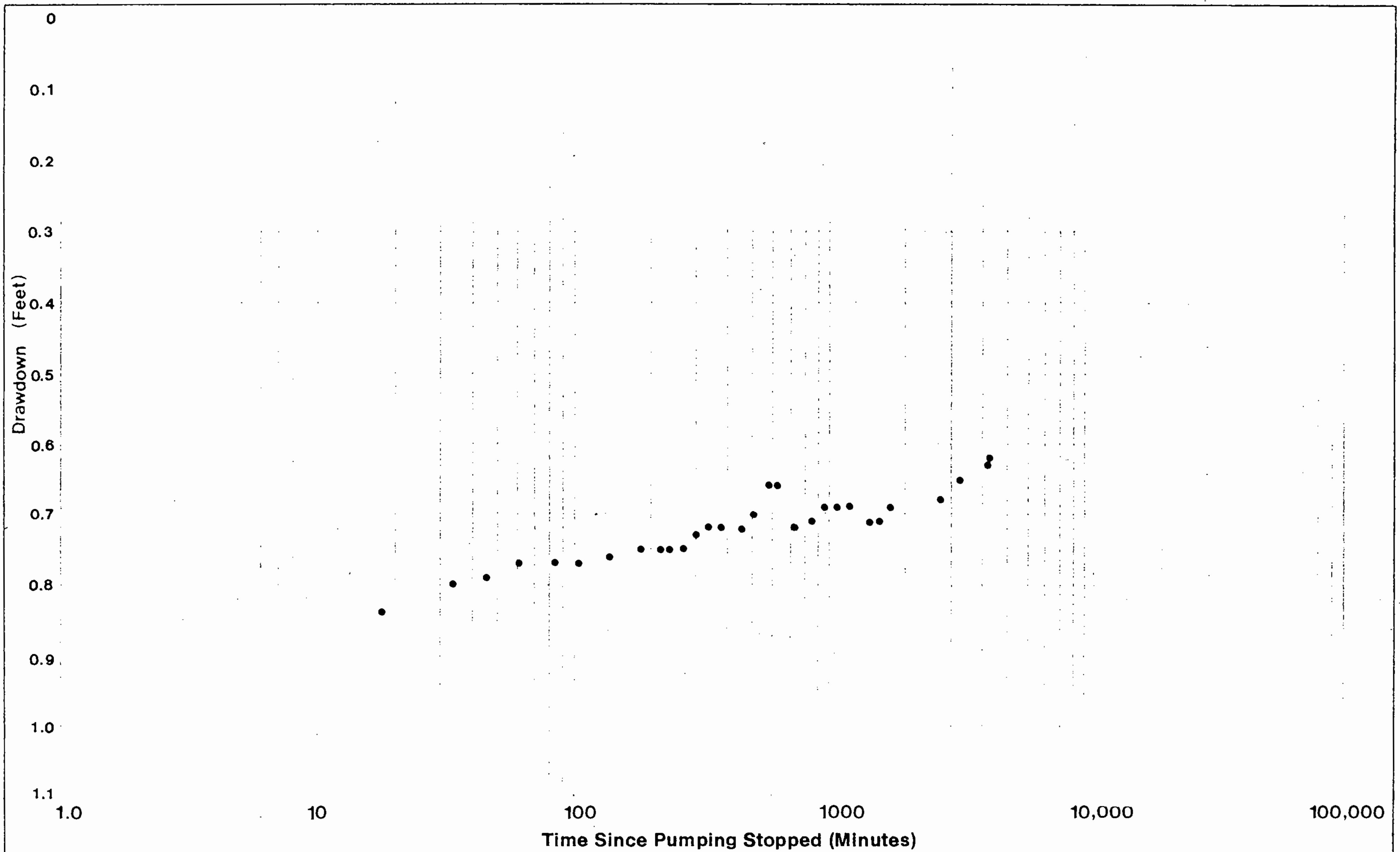
STEPTOE VALLEY

Time-Recovery Curve from Observation Well 1C Recovery Test 2

Leeds, Hill and Jewett, Inc.

December 1982

FIGURE 3 · 14



WHITE PINE POWER PROJECT

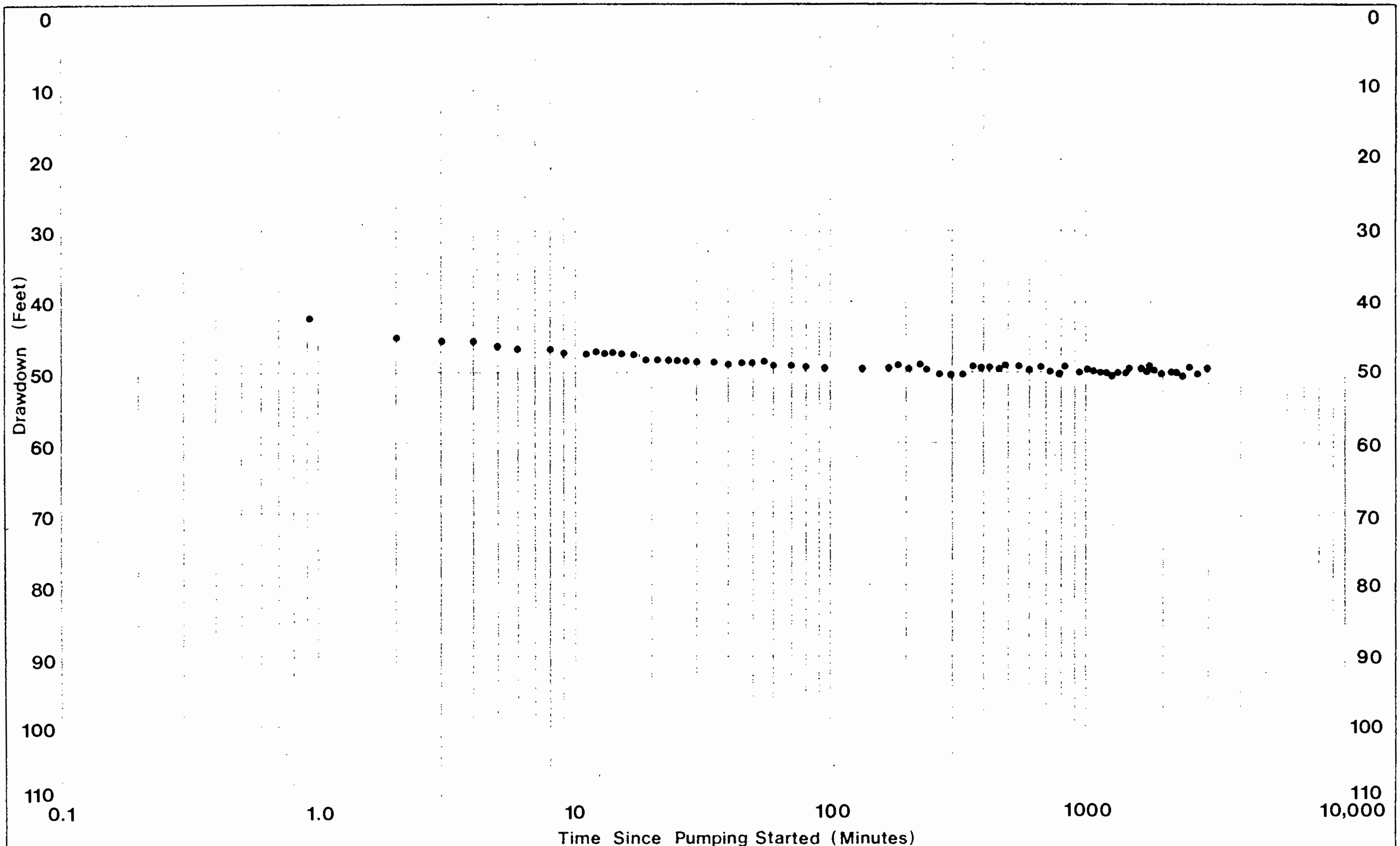
STEPTOE VALLEY

Time-Recovery Curve from Water Supply Well Recovery Test 2

Leeds, Hill and Jewett, Inc.

December 1982

FIGURE 3·15



Leeds, Hill and Jewett, Inc.

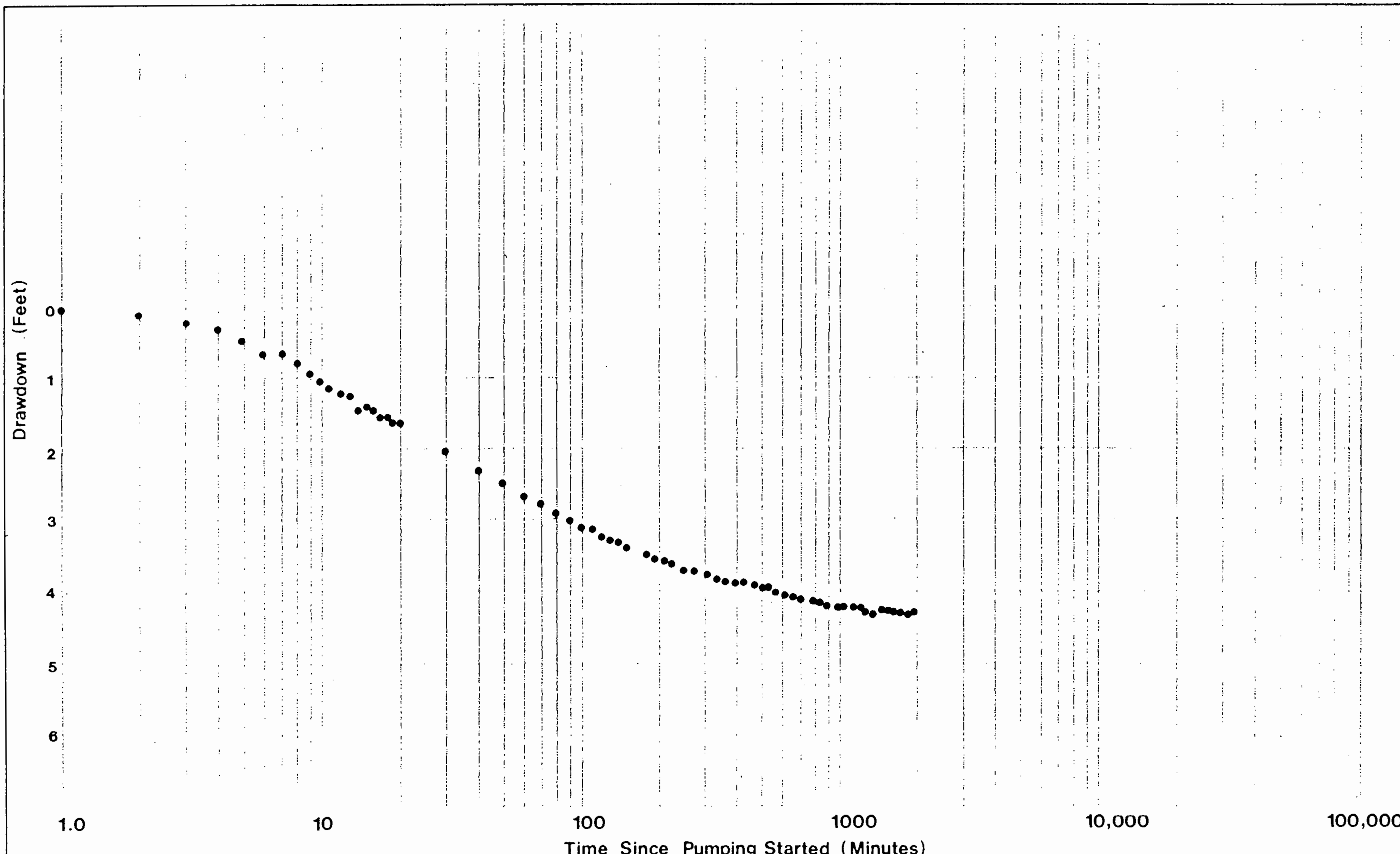
December 1982

WHITE PINE POWER PROJECT

STEPTOE VALLEY

Time-Drawdown Curve from
Pumping Well 1A
Constant Rate Test 3

FIGURE 3·16



WHITE PINE POWER PROJECT

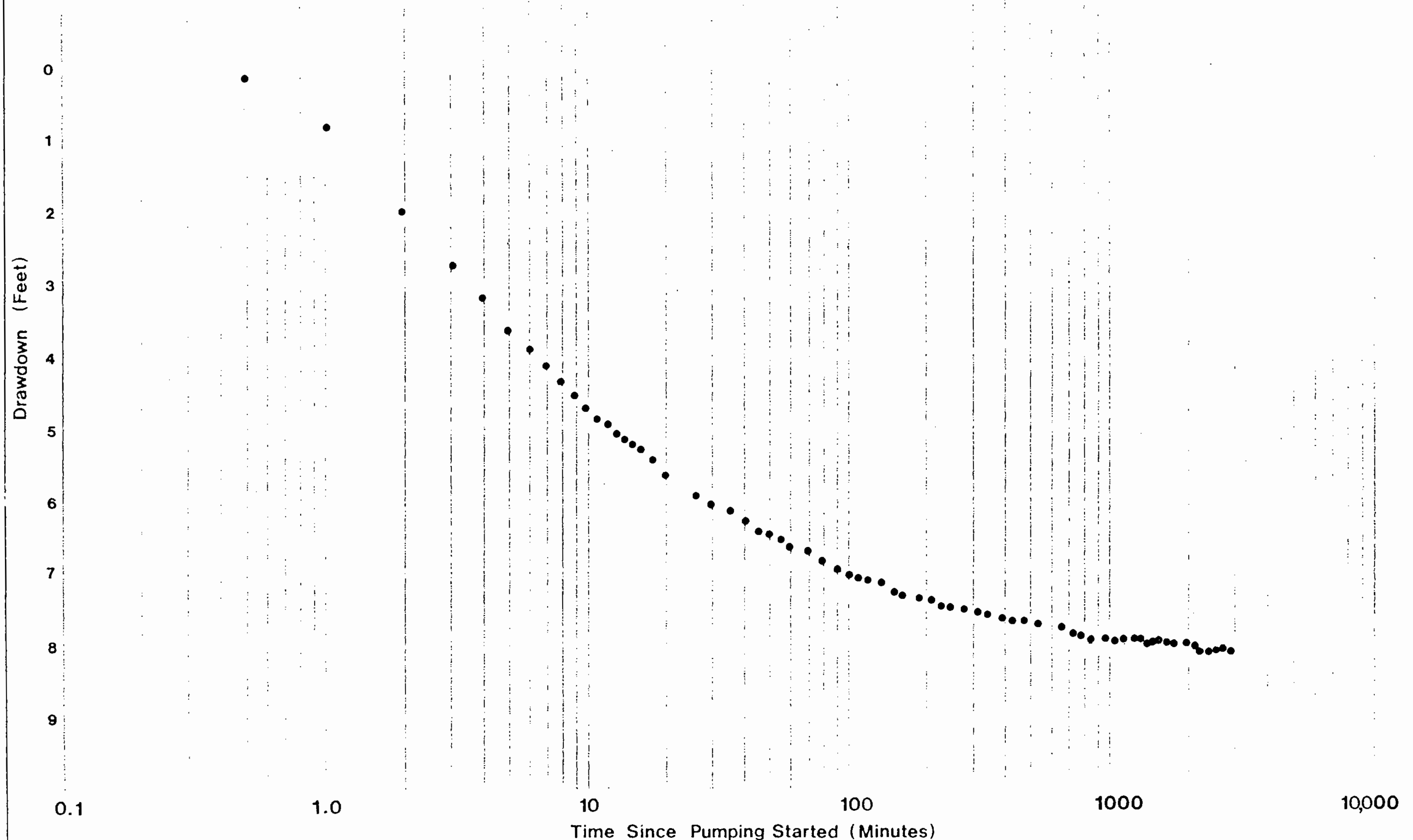
STEPTOE VALLEY

Time-Drawdown Curve from Observation Well 1B Constant Rate Test 3

Leeds, Hill and Jewett, Inc.

December 1982

FIGURE 3·17



WHITE PINE POWER PROJECT

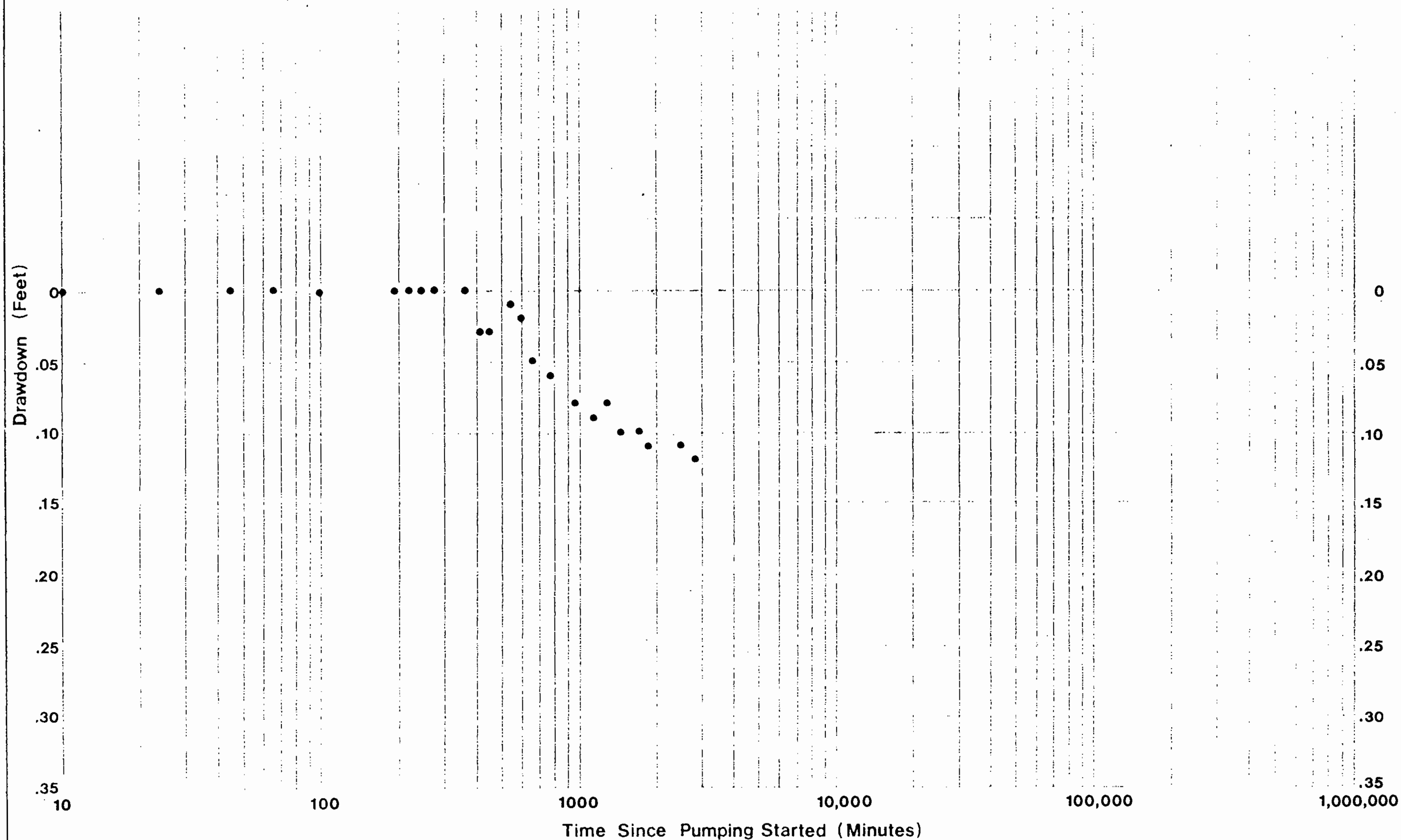
STEPTOE VALLEY

Time-Drawdown Curve from Observation Well 1C Constant Rate Test 3

FIGURE 3·18

Leeds, Hill and Jewett, Inc.

December 1982



WHITE PINE POWER PROJECT

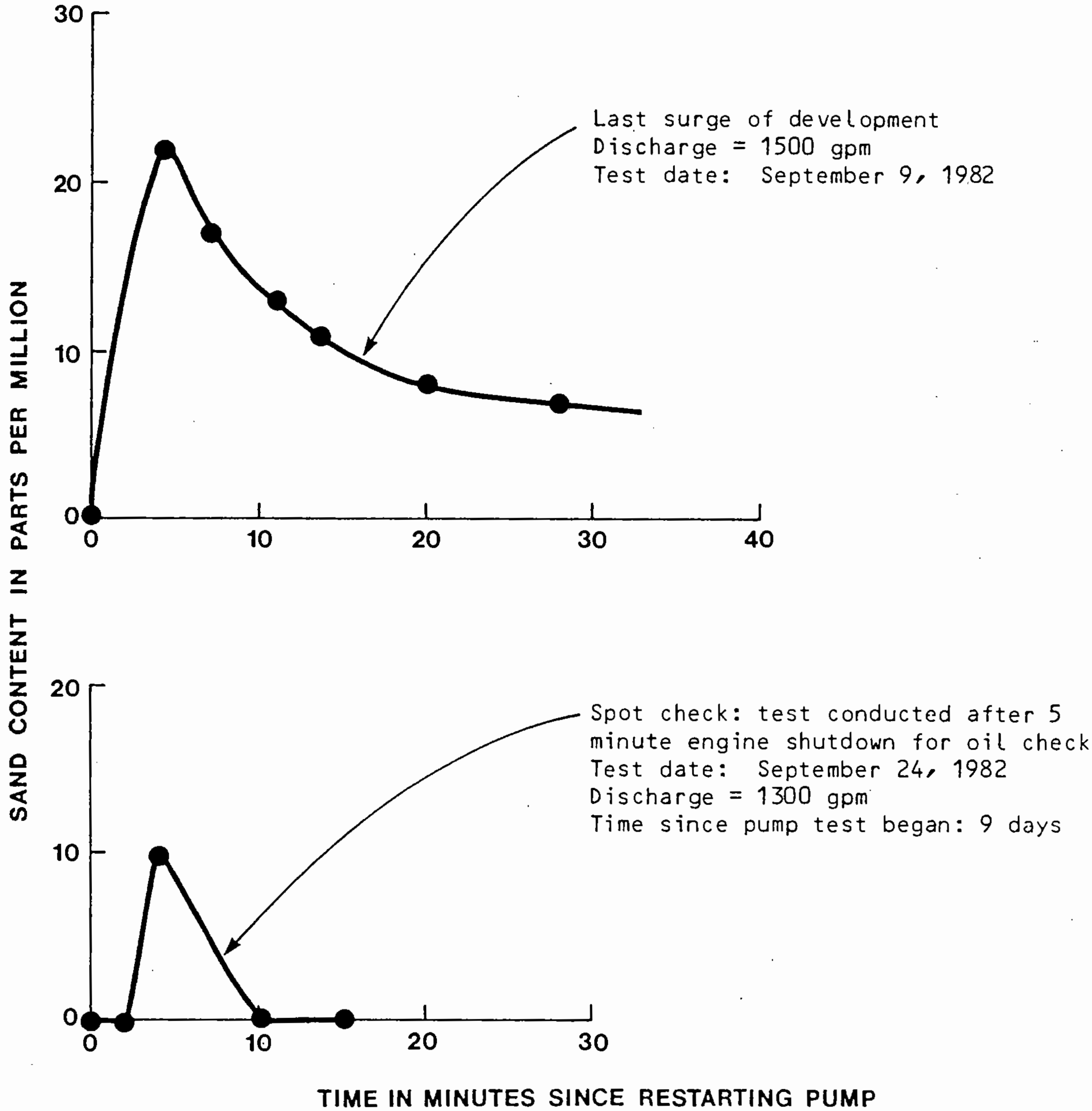
STEPTOE VALLEY

Time Drawdown Curve from Water Supply Well Constant Rate Test 3

FIGURE 3·19

Leeds, Hill and Jewett, Inc.

December 1982

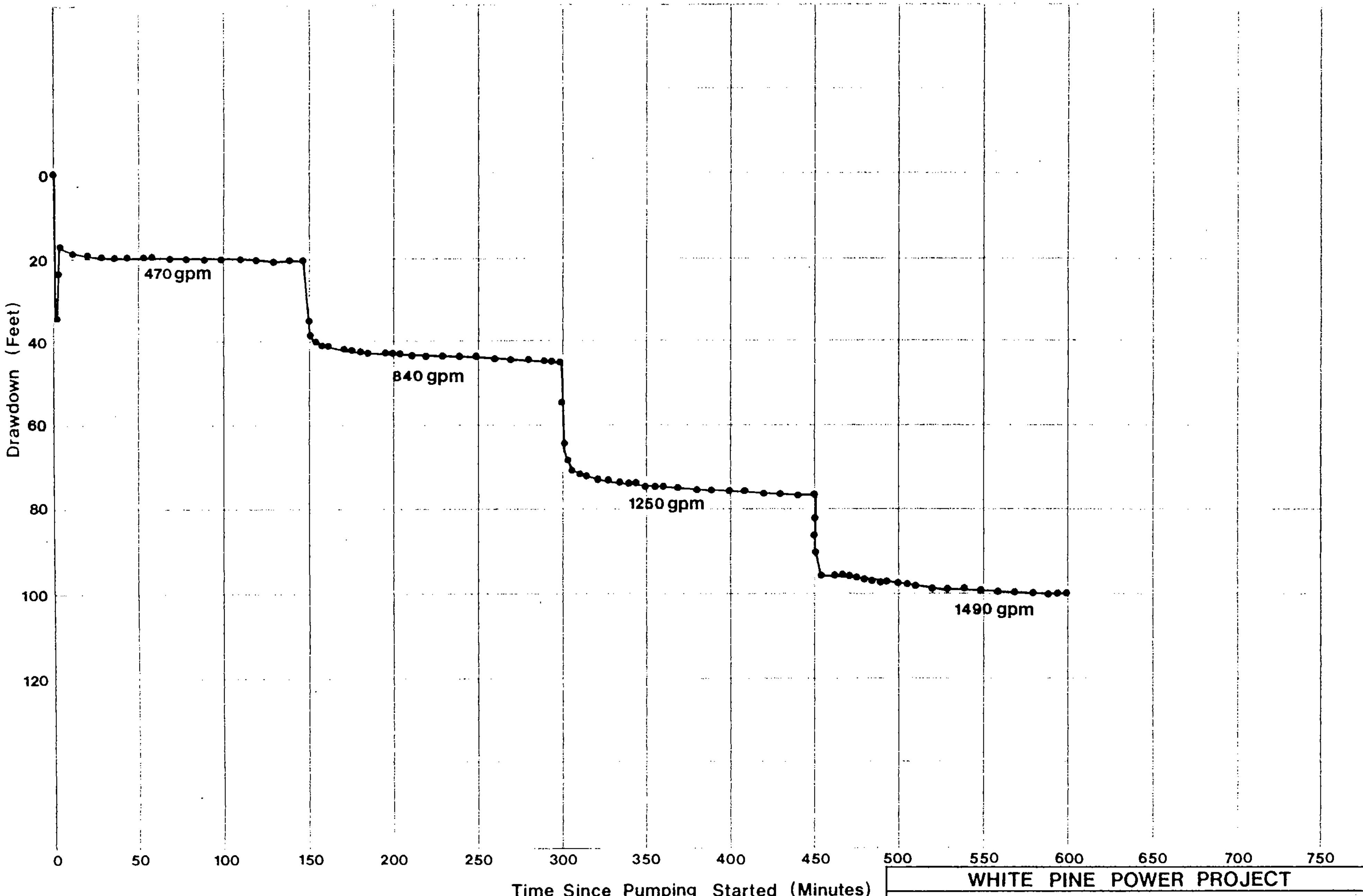


WHITE PINE POWER PROJECT

SPRING VALLEY

Sand Content Graphs

Leeds, Hill and Jewett, Inc. December 1982 FIGURE 3·20



Leeds, Hill and Jewett, Inc.

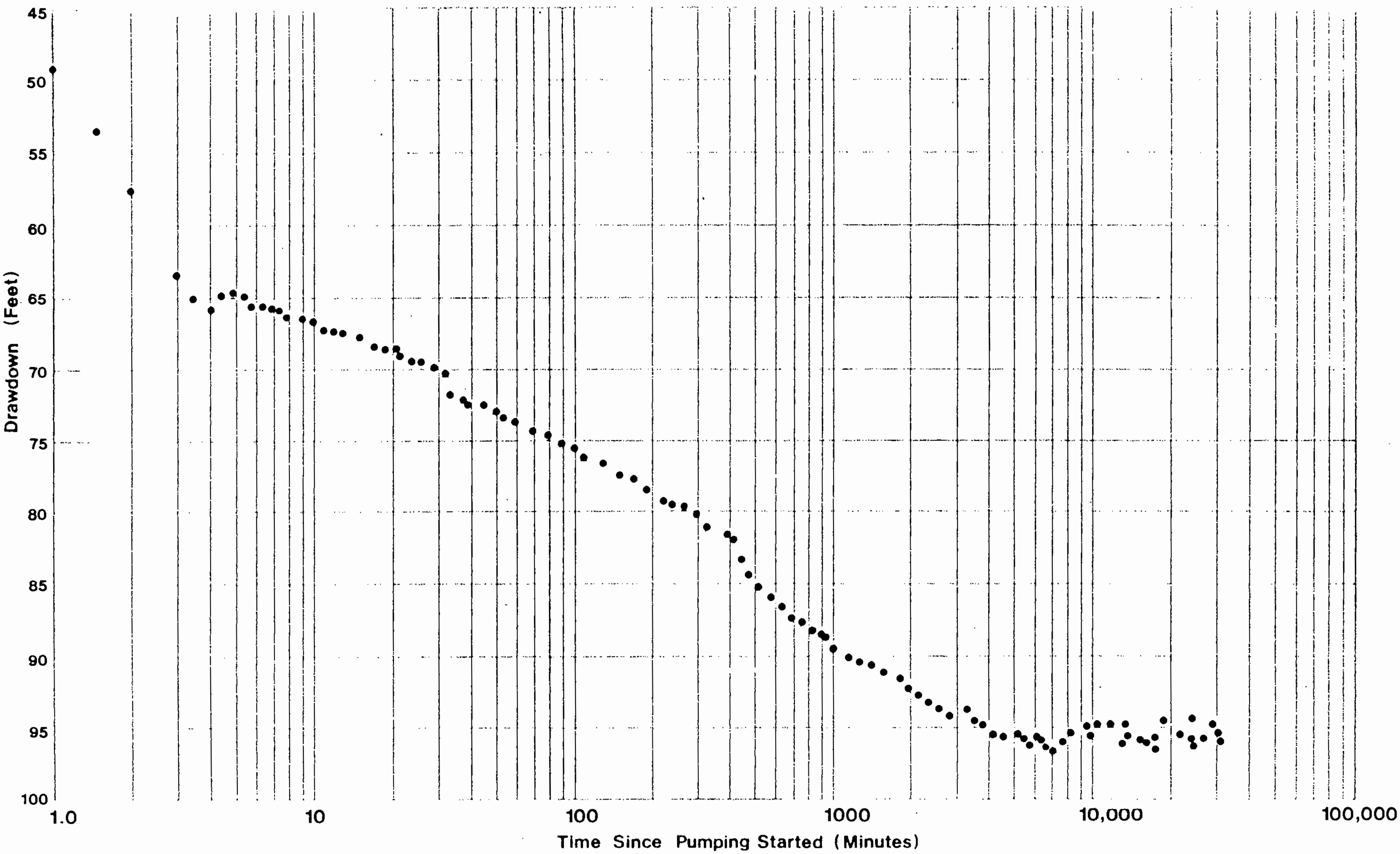
December 1982

WHITE PINE POWER PROJECT

SPRING VALLEY

Variable Rate Pump Test
Production Well 2A

FIGURE 3·21



Leeds, Hill and Jewett, Inc.

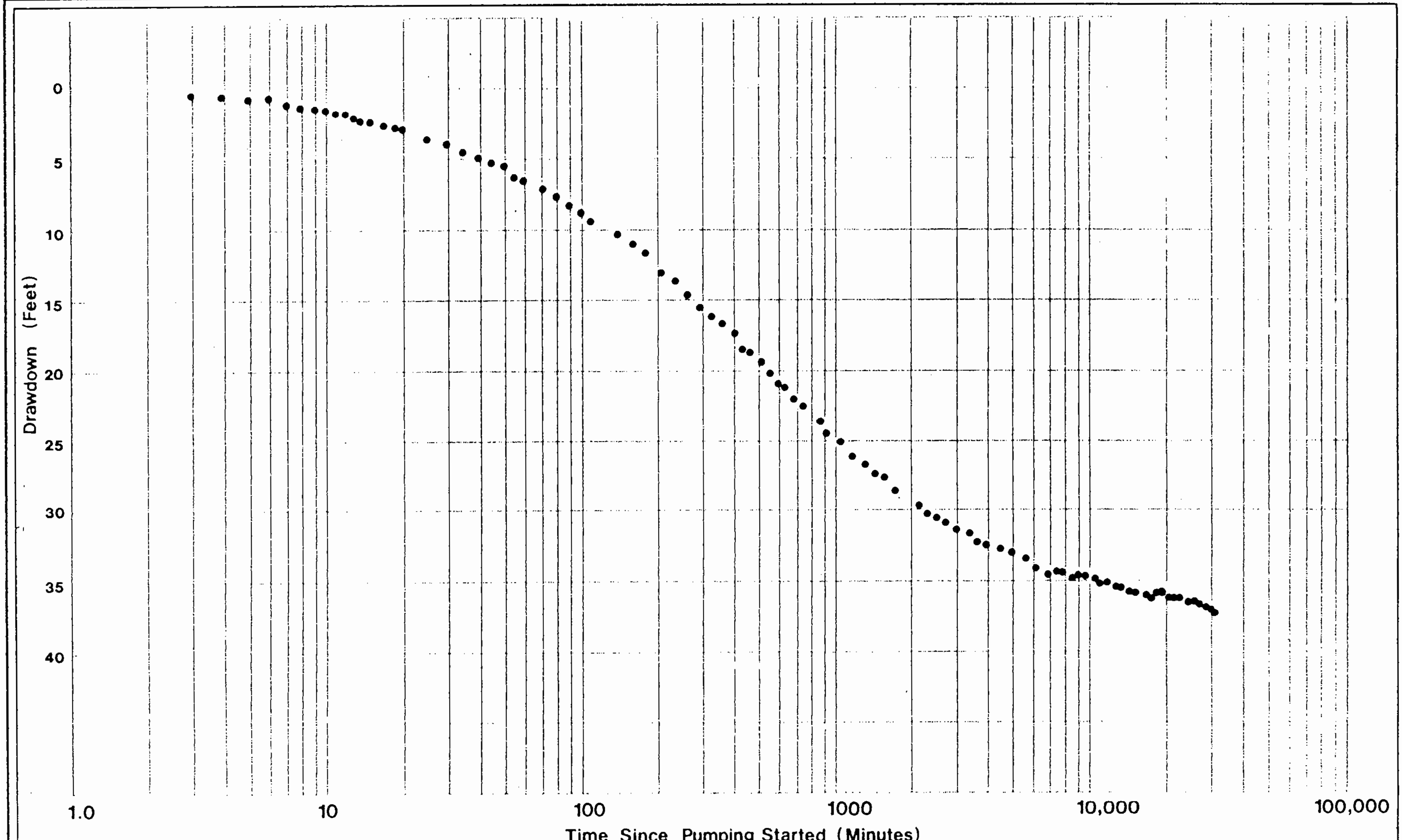
December 1982

WHITE PINE POWER PROJECT

SPRING VALLEY

Time-Drawdown Curve from
Production Well 2A
Constant Rate Pumping Test

FIGURE 3·22



WHITE PINE POWER PROJECT

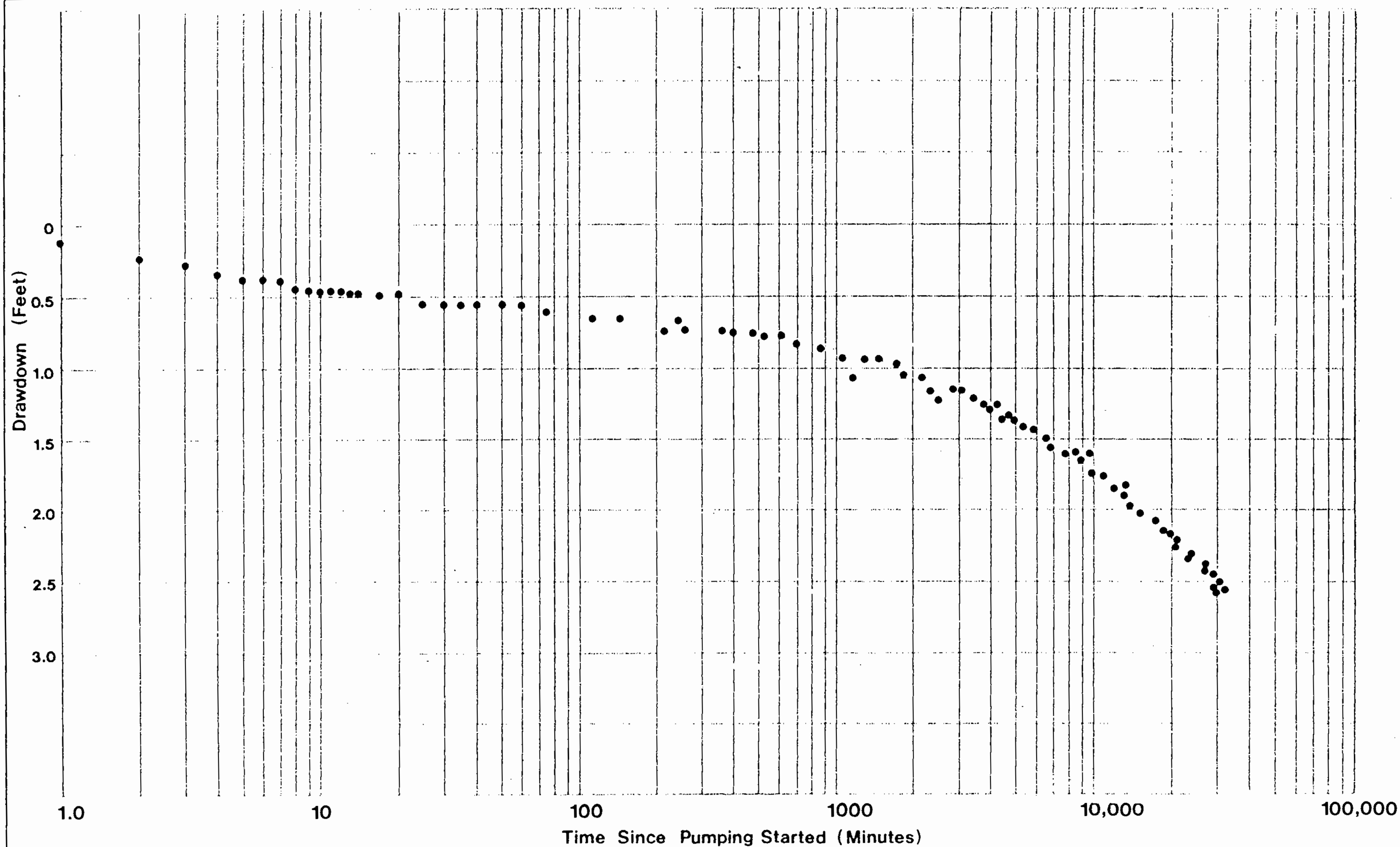
SPRING VALLEY

Time-Drawdown Curve from
Observation Well 2B
Constant Rate Pumping Test

Leeds, Hill and Jewett, Inc.

December 1982

FIGURE 3·23



WHITE PINE POWER PROJECT

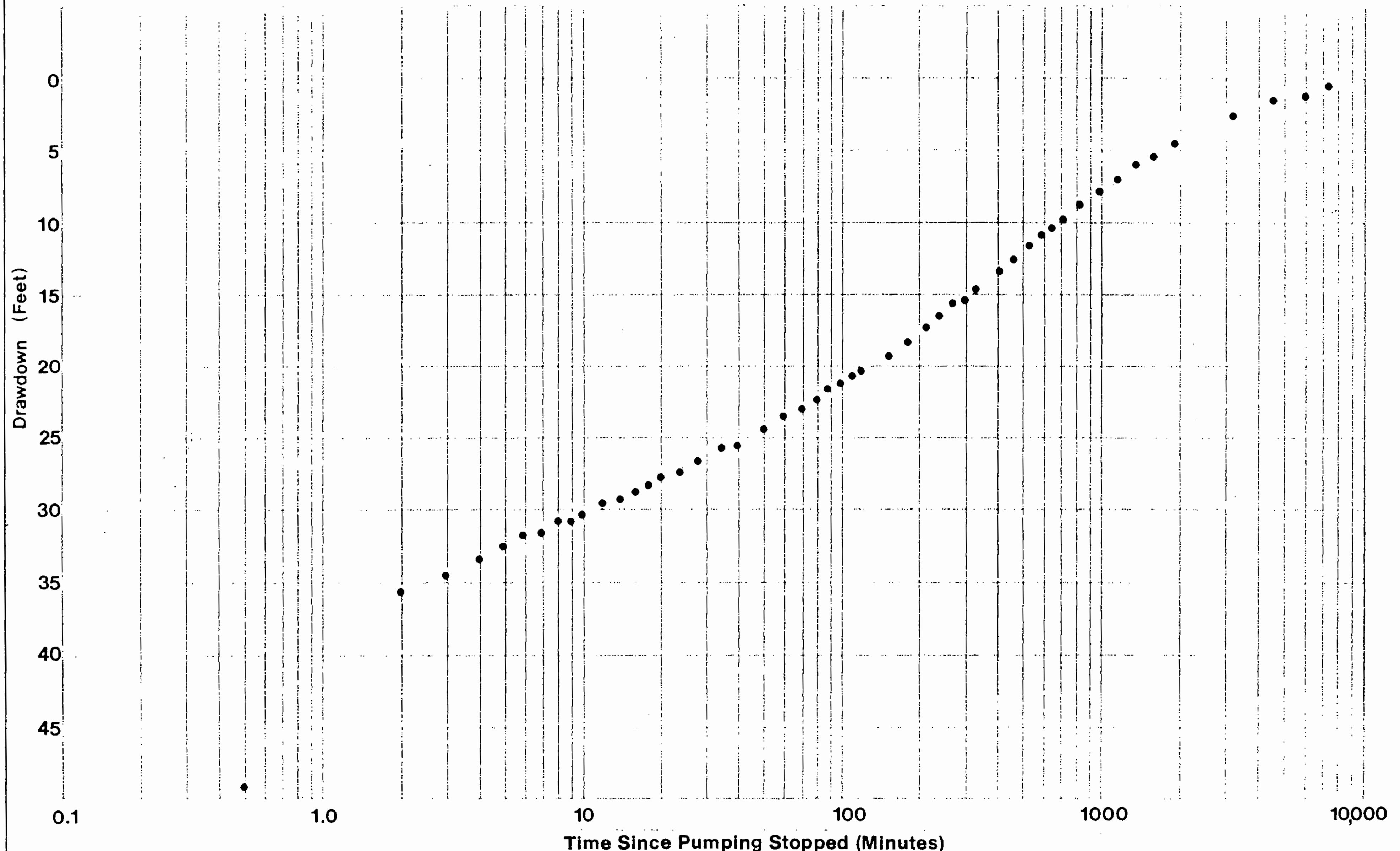
SPRING VALLEY

Time-Drawdown Curve from
Observation Well 2C
Constant Rate Pumping Test

Leeds, Hill and Jewett, Inc.

December 1982

FIGURE 3·24



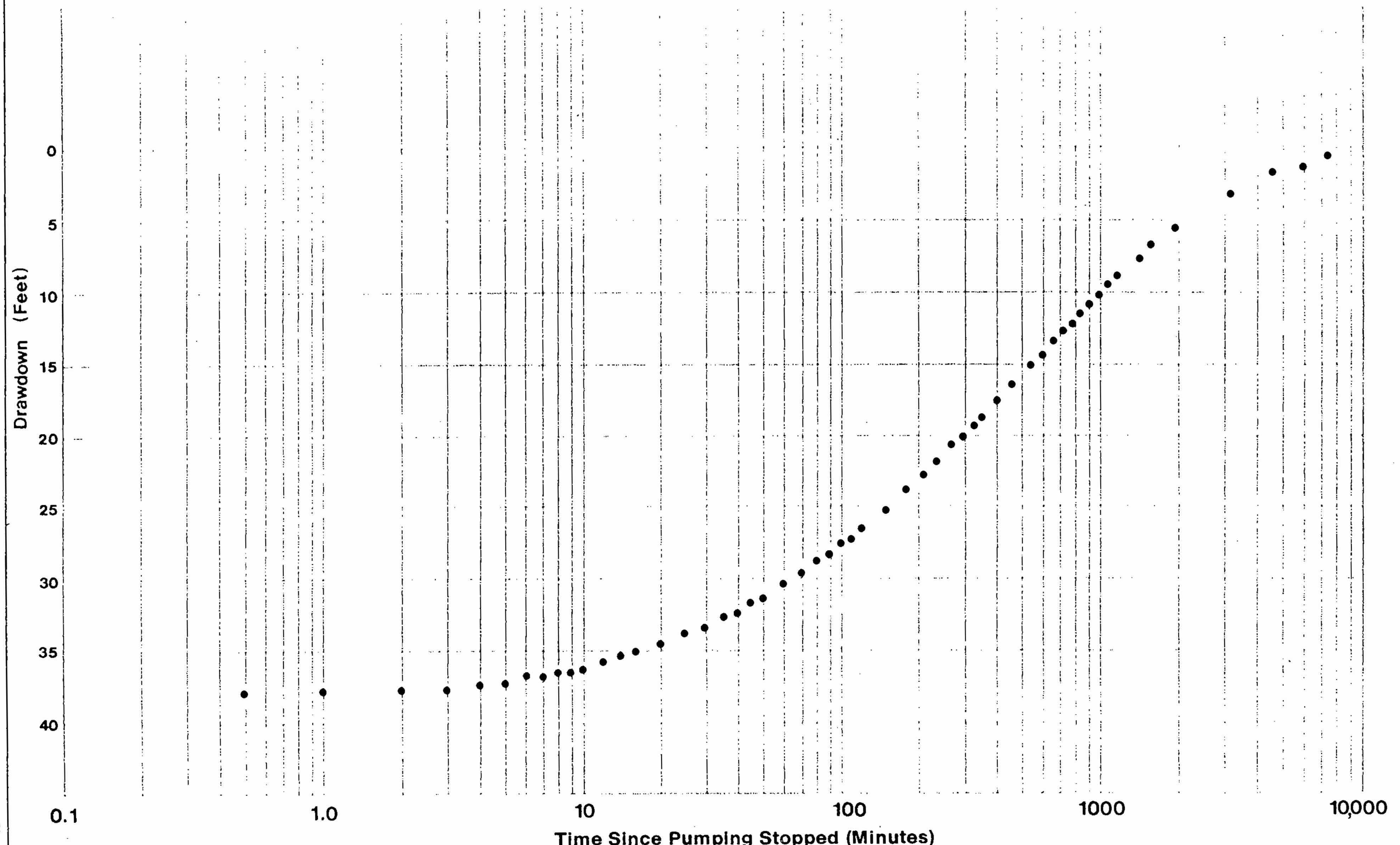
WHITE PINE POWER PROJECT

SPRING VALLEY
Time-Recovery Curve from
Production Well 2A
Recovery Test

FIGURE 3 · 25

Leeds, Hill and Jewett, Inc.

December 1982



Leeds, Hill and Jewett, Inc.

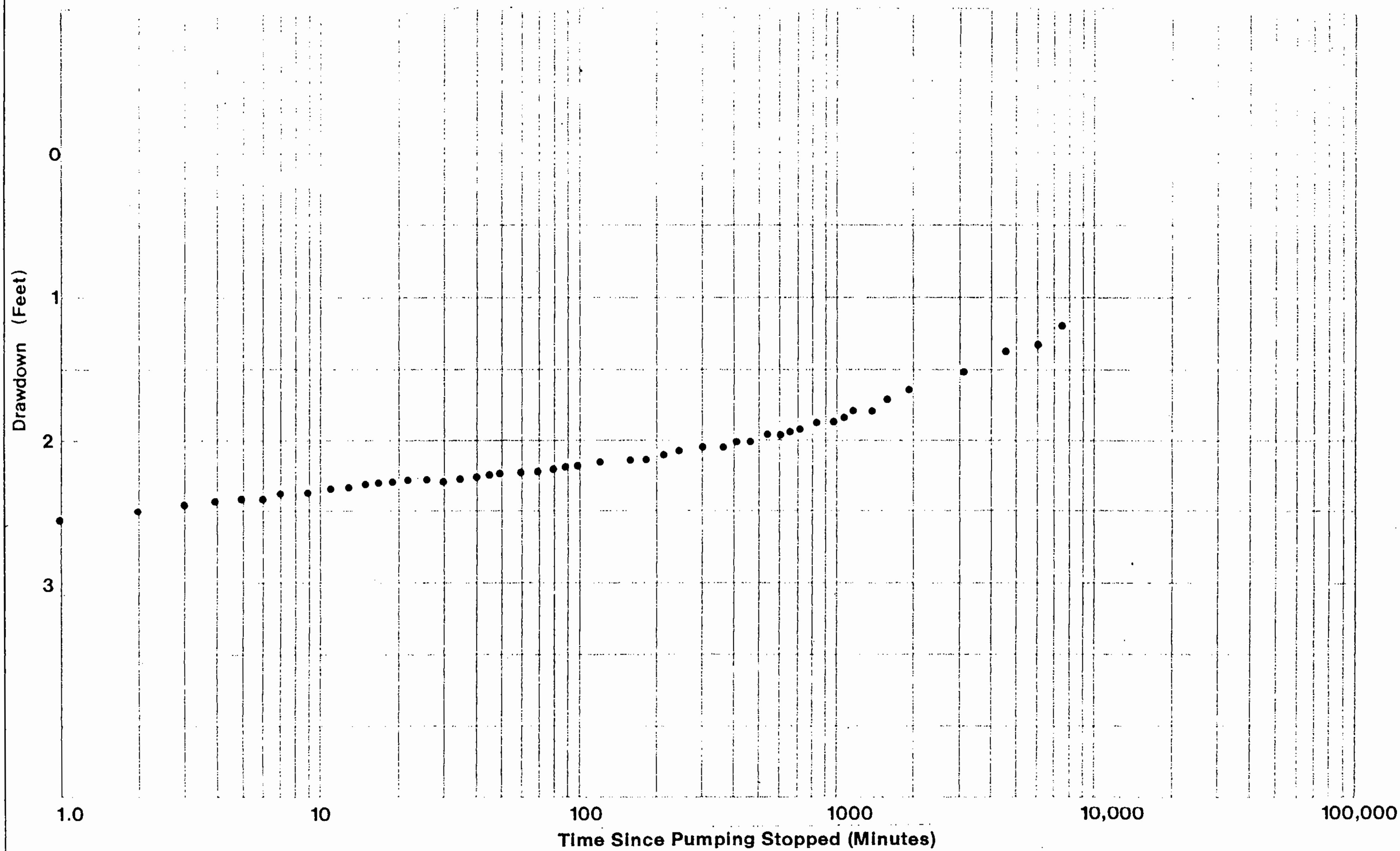
December 1982

WHITE PINE POWER PROJECT

SPRING VALLEY

Time-Recovery Curve from
Observation Well 2B
Recovery Test

FIGURE 3·26



WHITE PINE POWER PROJECT

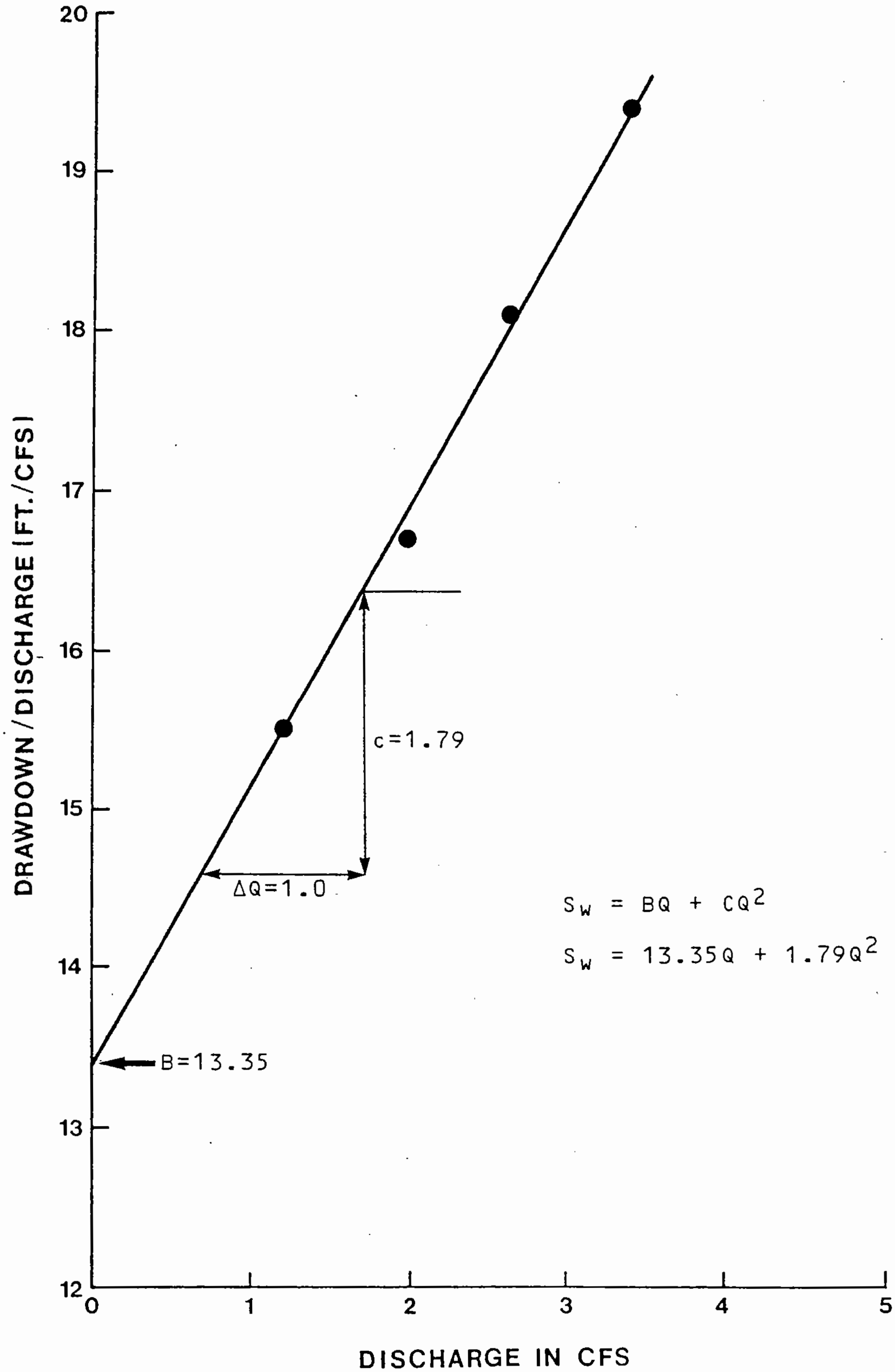
SPRING VALLEY

Time-Recovery Curve from Observation Well 2C Recovery Test

Leeds, Hill and Jewett, Inc.

December 1982

FIGURE 3-27

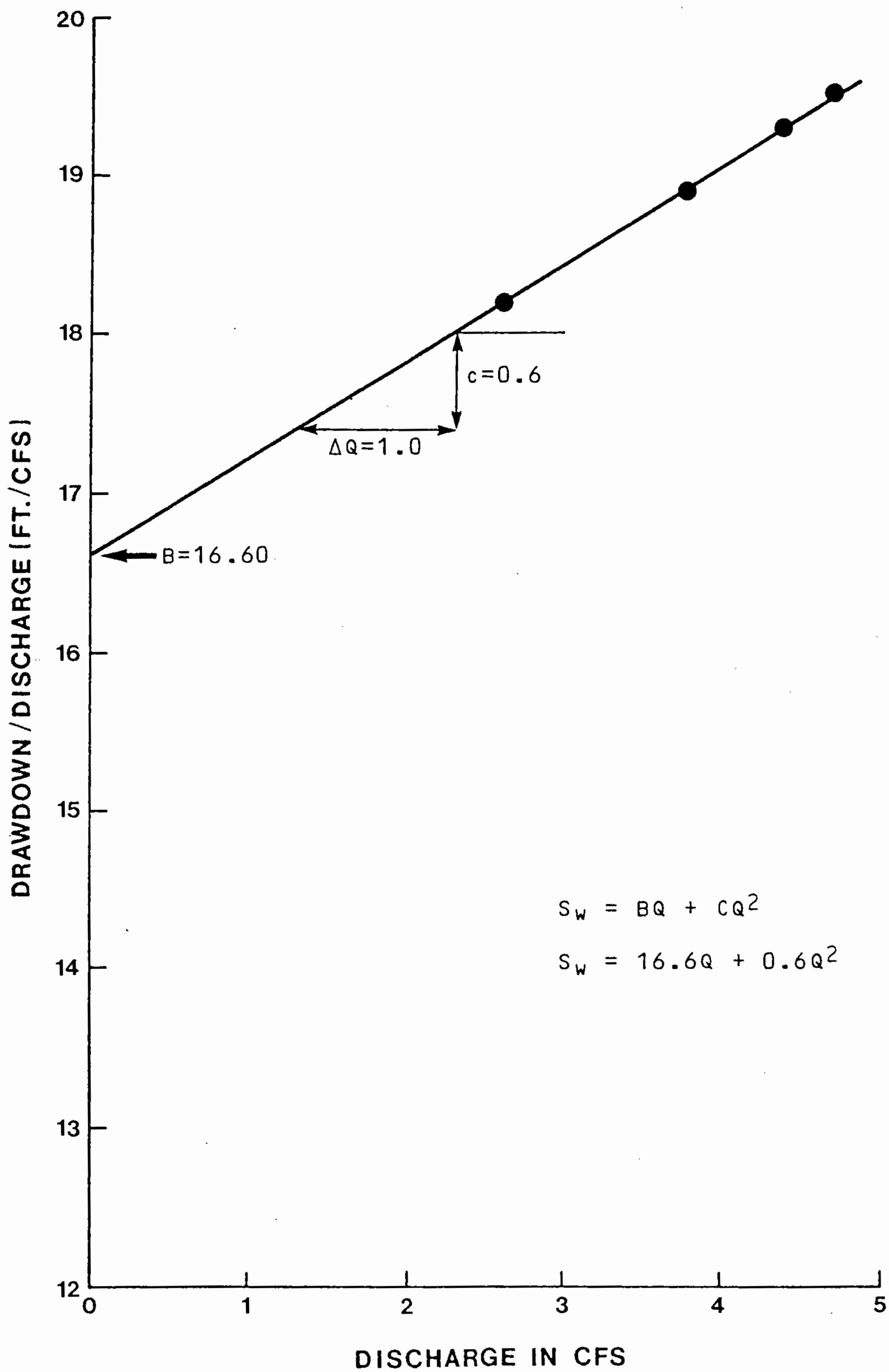


WHITE PINE POWER PROJECT

STEPTOE VALLEY

Variable Rate Pump Test 1
Determination of Formation and
Well Loss Characteristics

Leeds, Hill and Jewett, Inc. December 1982 FIGURE 3·28

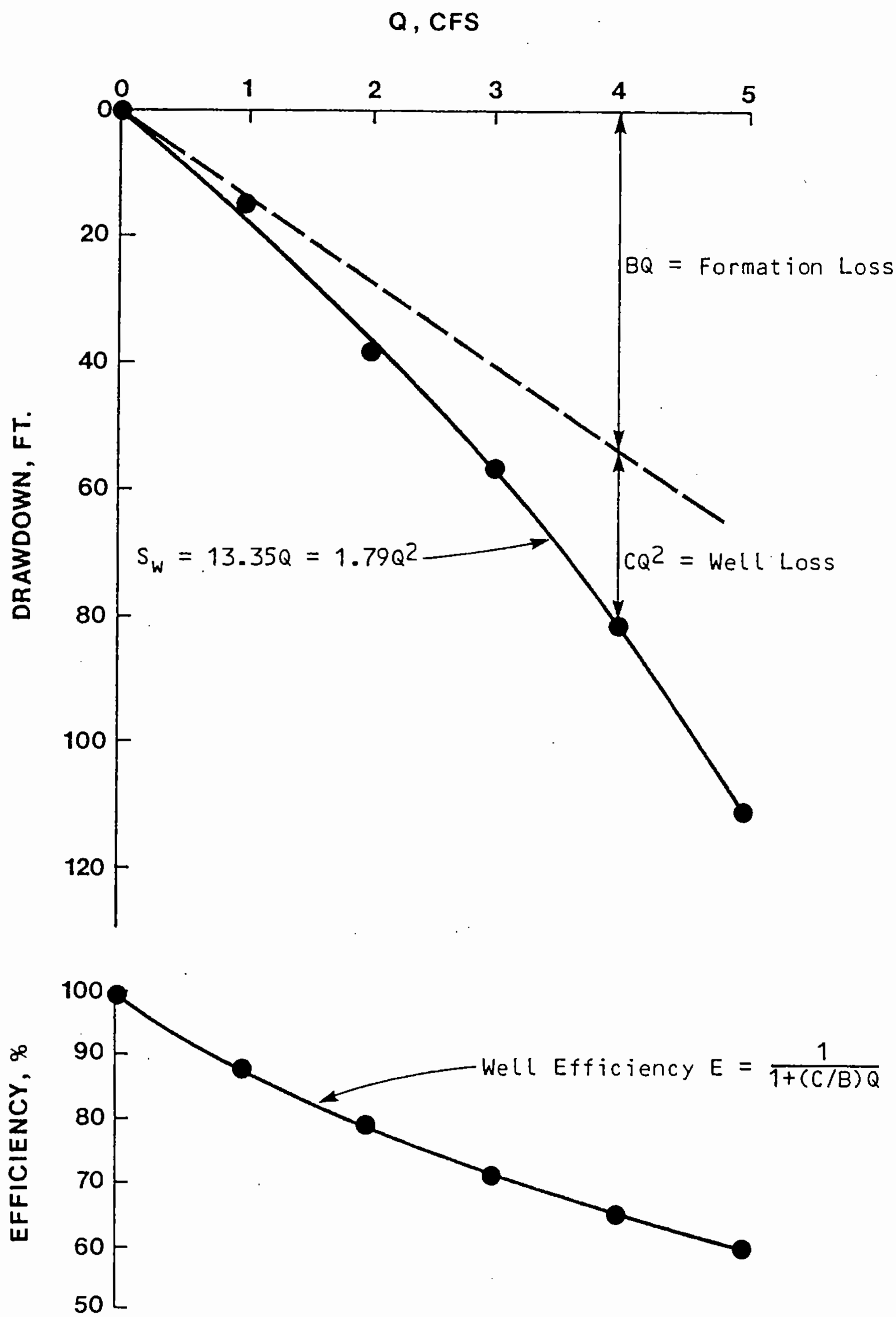


WHITE PINE POWER PROJECT

STEPTOE VALLEY

Variable Rate Pump Test 2
Determination of Formation and
Well Loss Characteristics

Leeds, Hill and Jewett, Inc. December 1982 FIGURE 3·29



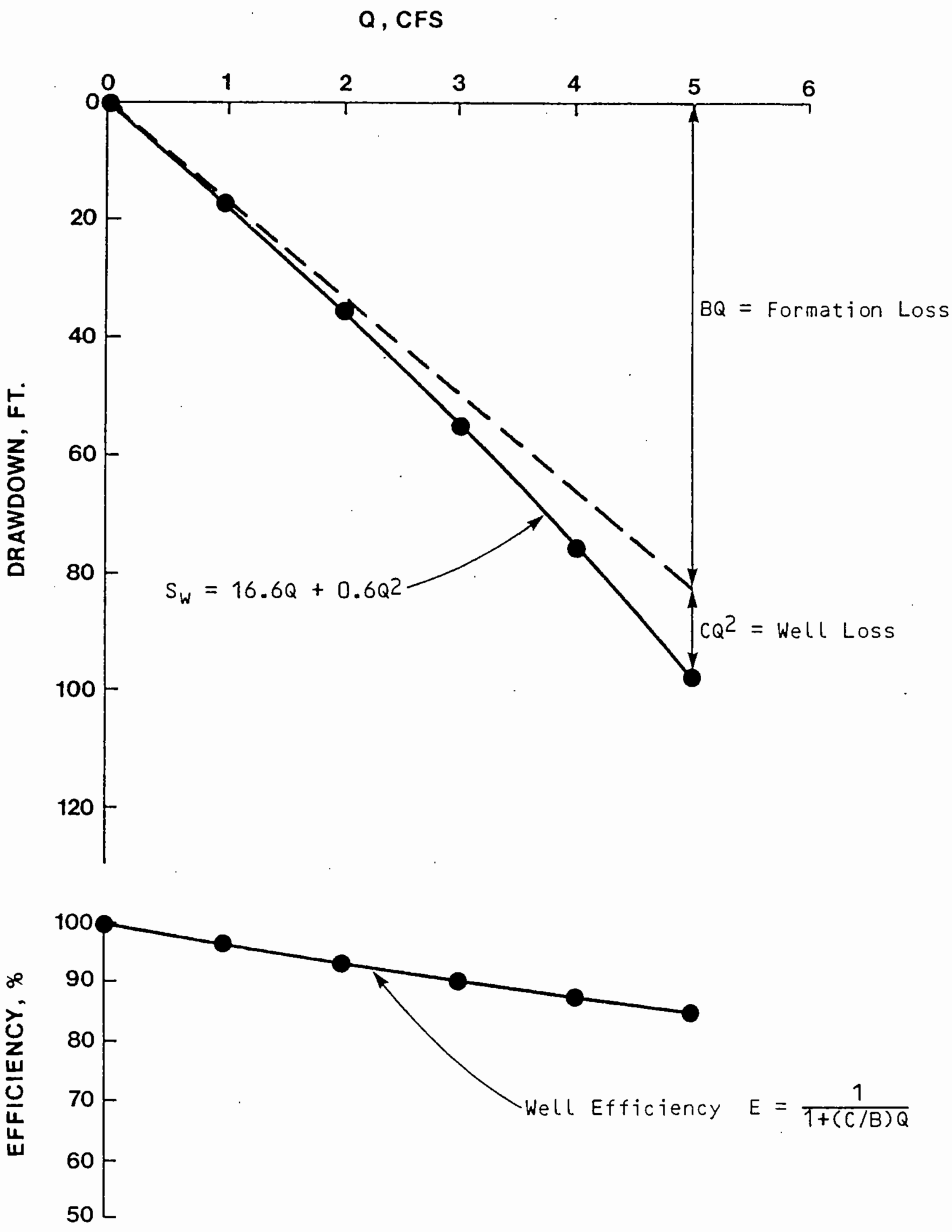
WHITE PINE POWER PROJECT

STEPTOE VALLEY

Variable Rate Pump Test 1

Efficiency of Production Well 1A

Leeds, Hill and Jewett, Inc. December 1982 FIGURE 3·30



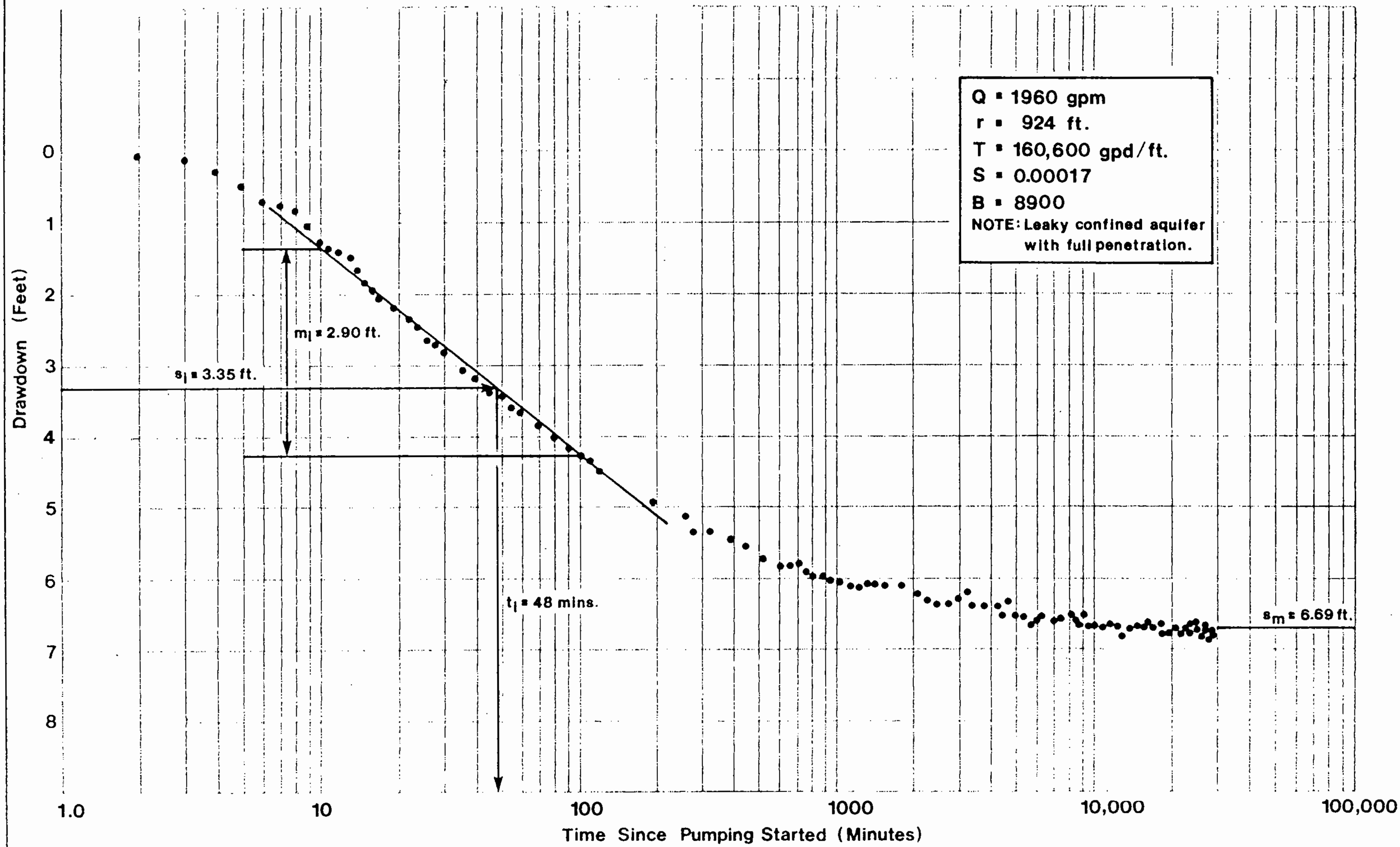
WHITE PINE POWER PROJECT

STEPTOE VALLEY

Variable Rate Pump Test 2

Efficiency of Production Well 1A

Leeds, Hill and Jewett, Inc. December 1982 FIGURE 3·31



Leeds, Hill and Jewett, Inc.

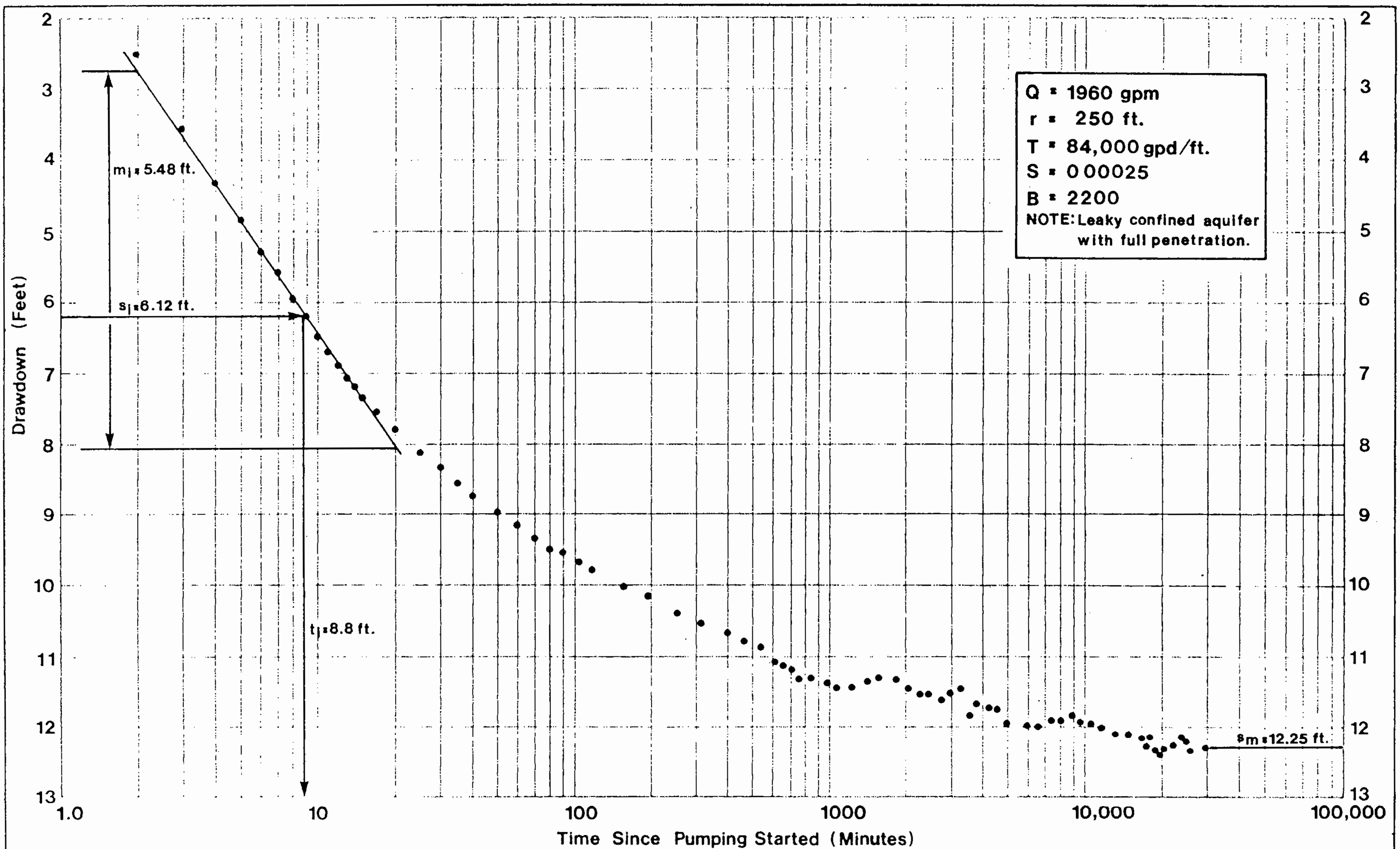
December 1982

WHITE PINE POWER PROJECT

STEPTOE VALLEY

Pump Test Analysis from Observation Well 1B Constant Rate Test 2

FIGURE 3·32



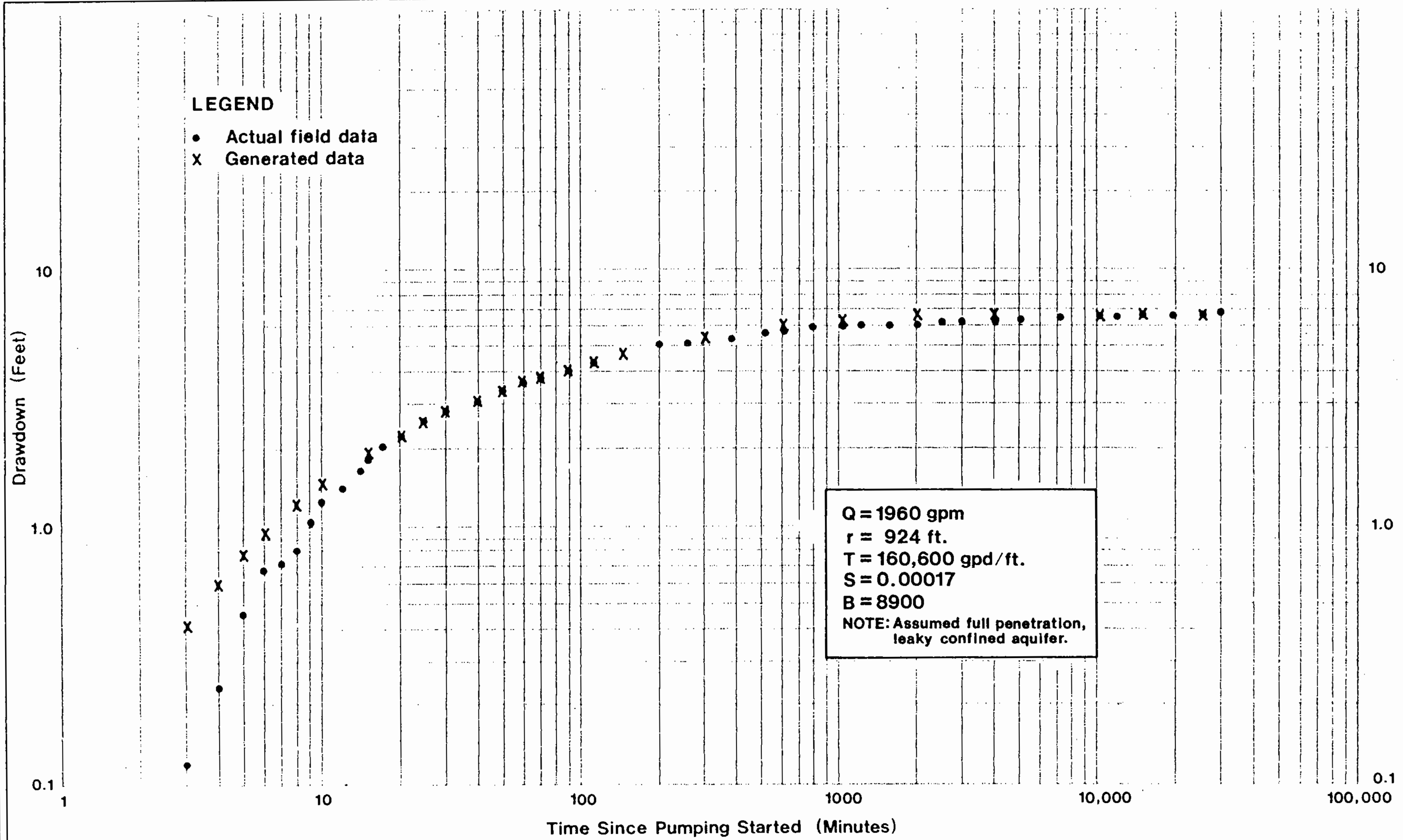
Leeds, Hill and Jewett, Inc.

December 1982

WHITE PINE POWER PROJECT

STEPTOE VALLEY
Pump Test Analysis from
Observation Well 1C
Constant Rate Test 2

FIGURE 3·33



WHITE PINE POWER PROJECT

STEPTOE VALLEY

Pump Test Analysis

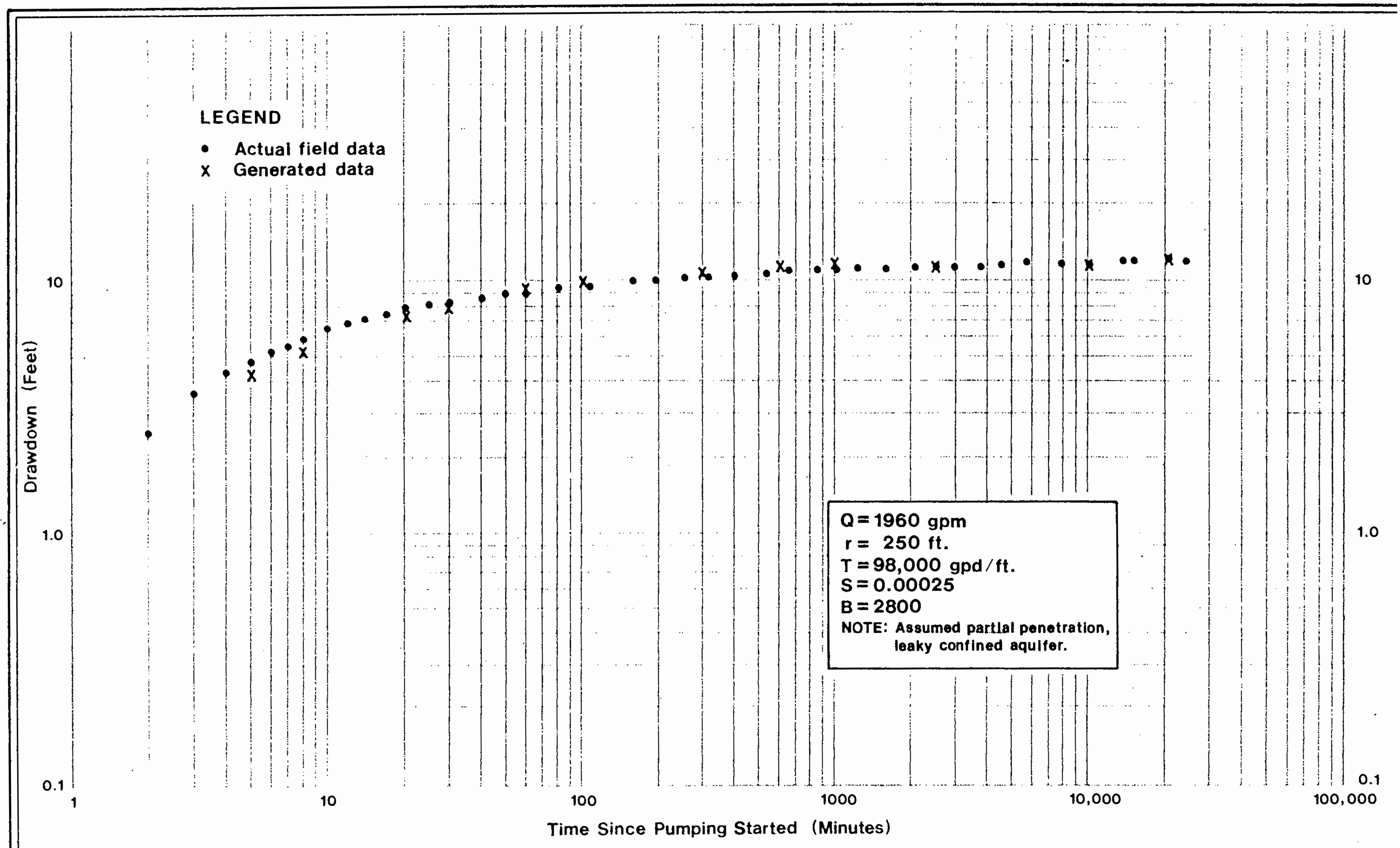
Observation Well 1B

Constant Rate Test 2

FIGURE 3·34

Leeds, Hill and Jewett, Inc.

December 1982



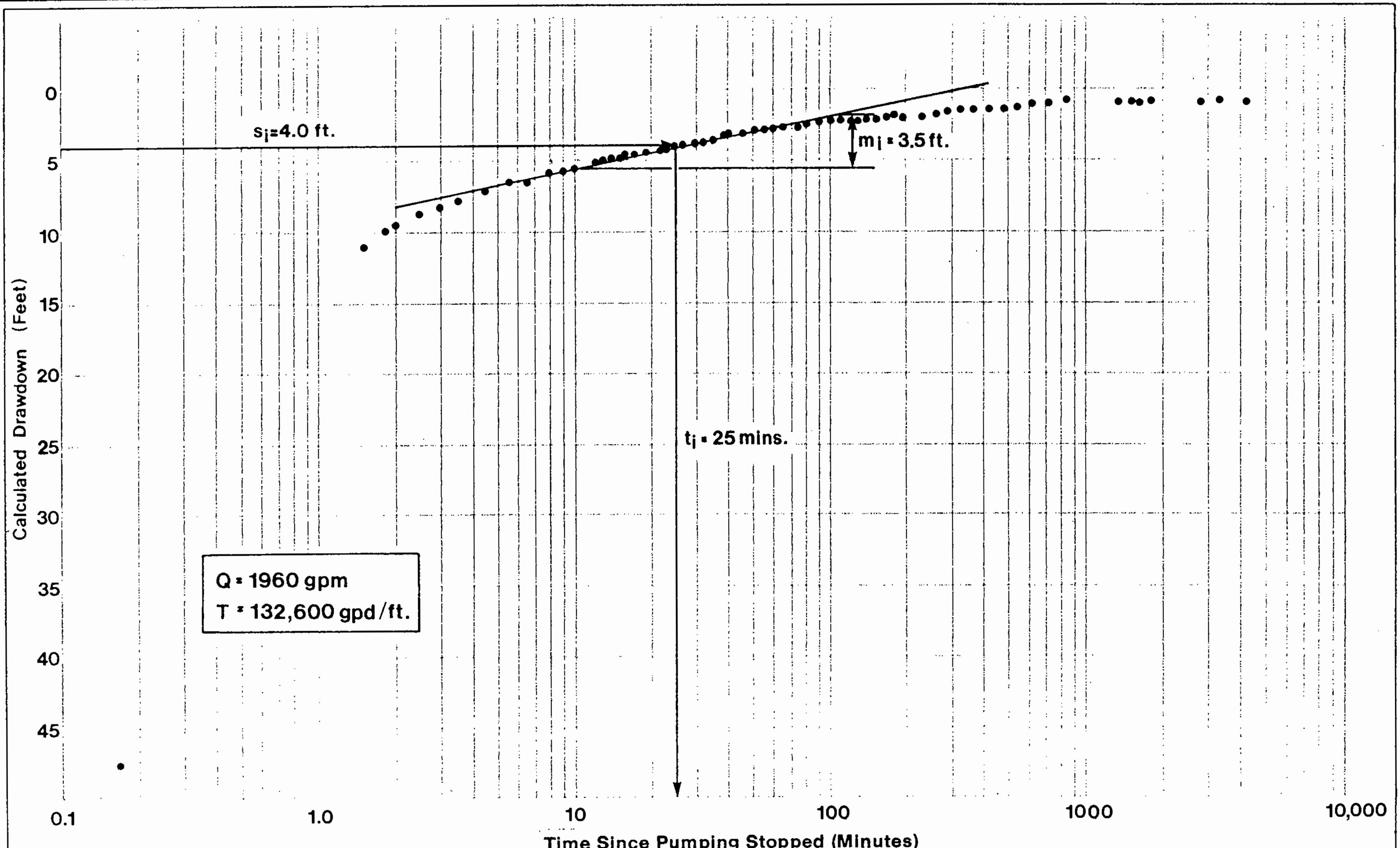
WHITE PINE POWER PROJECT

STEPTOE VALLEY
Pump Test Analysis
Observation Well 1C
Constant Rate Test 2

Leeds, Hill and Jewett, Inc.

December 1982

FIGURE 3-35



Leeds, Hill and Jewett, Inc.

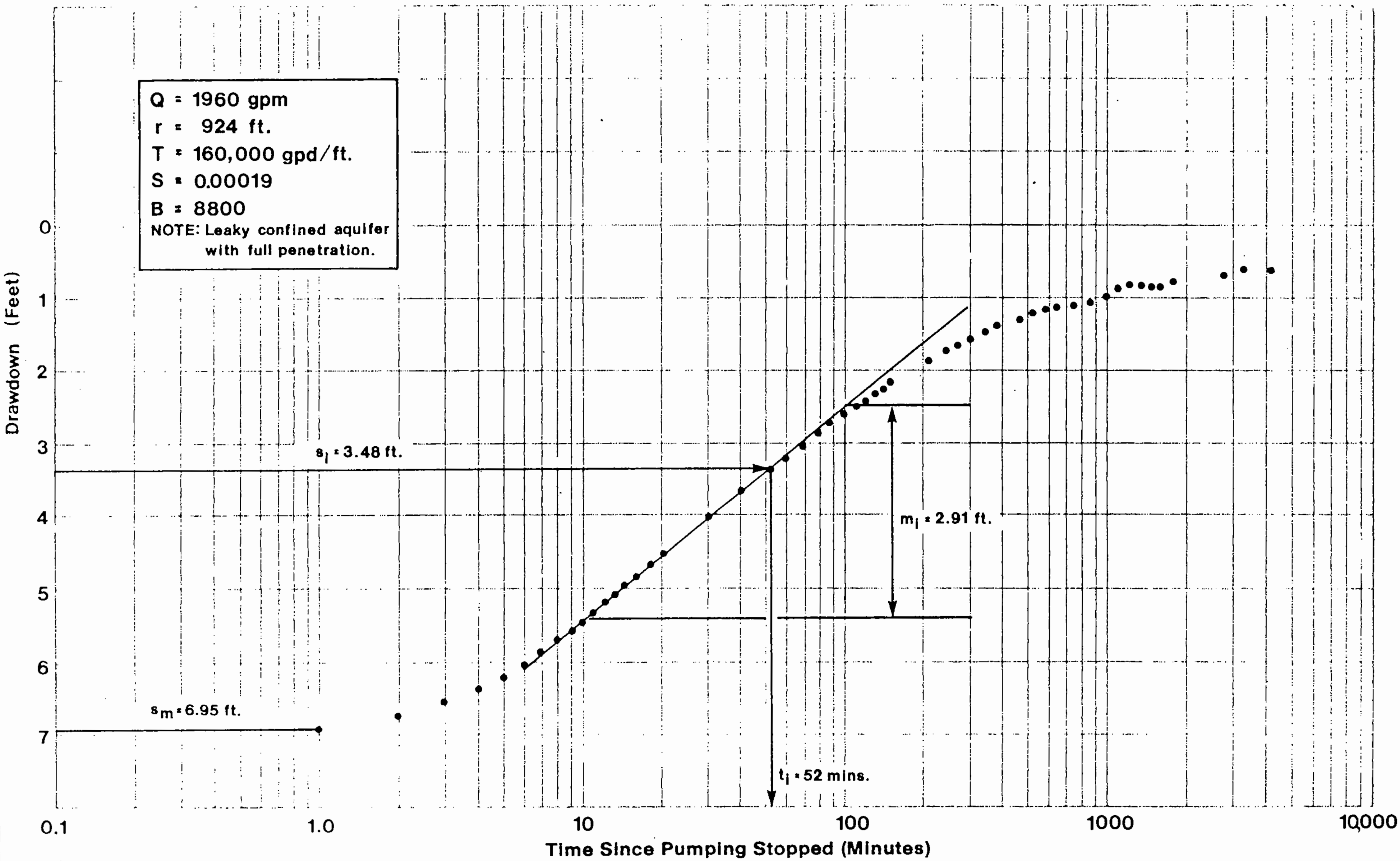
December 1982

WHITE PINE POWER PROJECT

STEPTOE VALLEY

Time-Recovery Curve from Production Well 1A Recovery Test

FIGURE 3·36



WHITE PINE POWER PROJECT

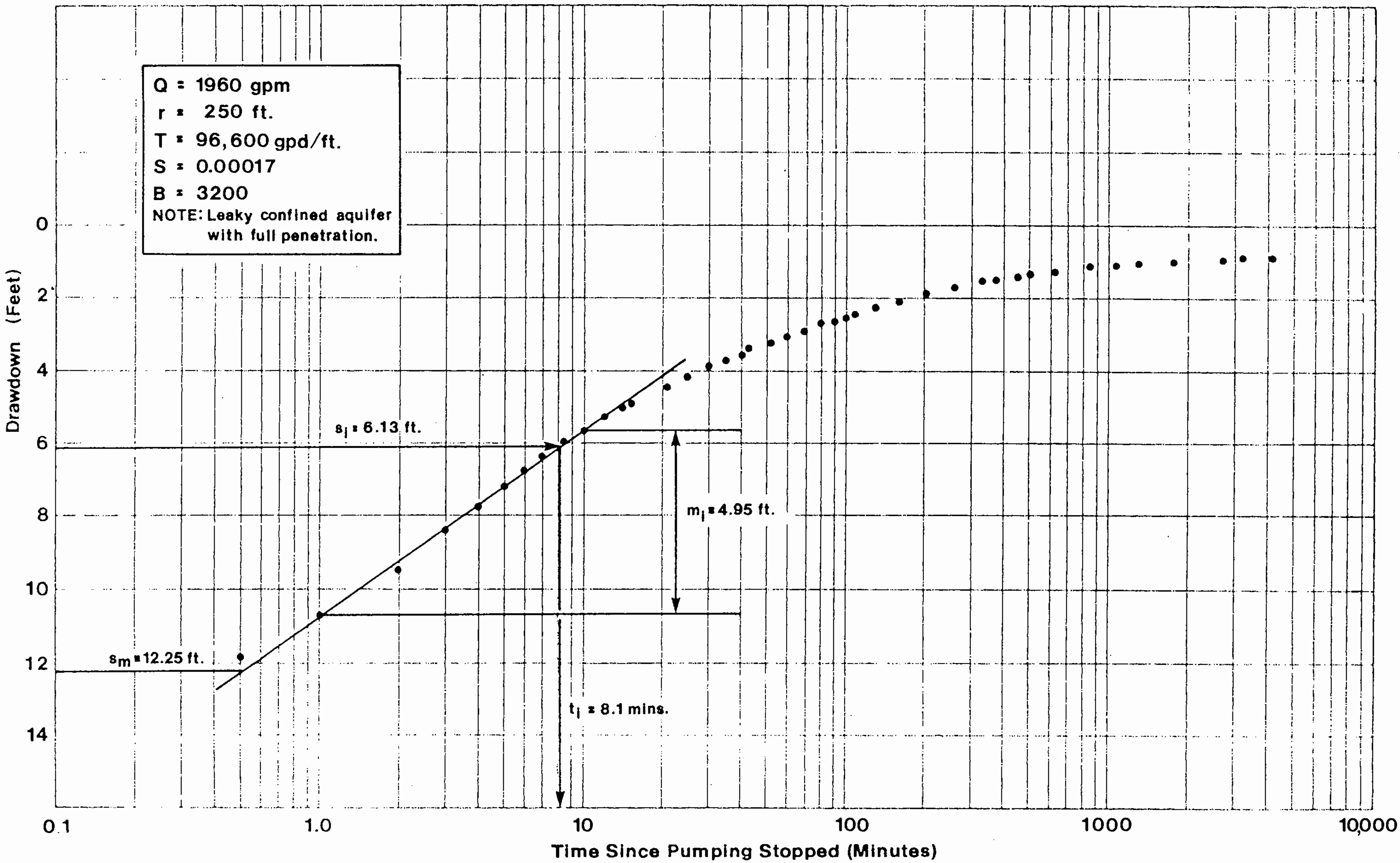
STEPTOE VALLEY

Time-Recovery Curve from Observation Well 1B Recovery Test 2

Leeds, Hill and Jewett, Inc.

December 1982

FIGURE 3·37



WHITE PINE POWER PROJECT

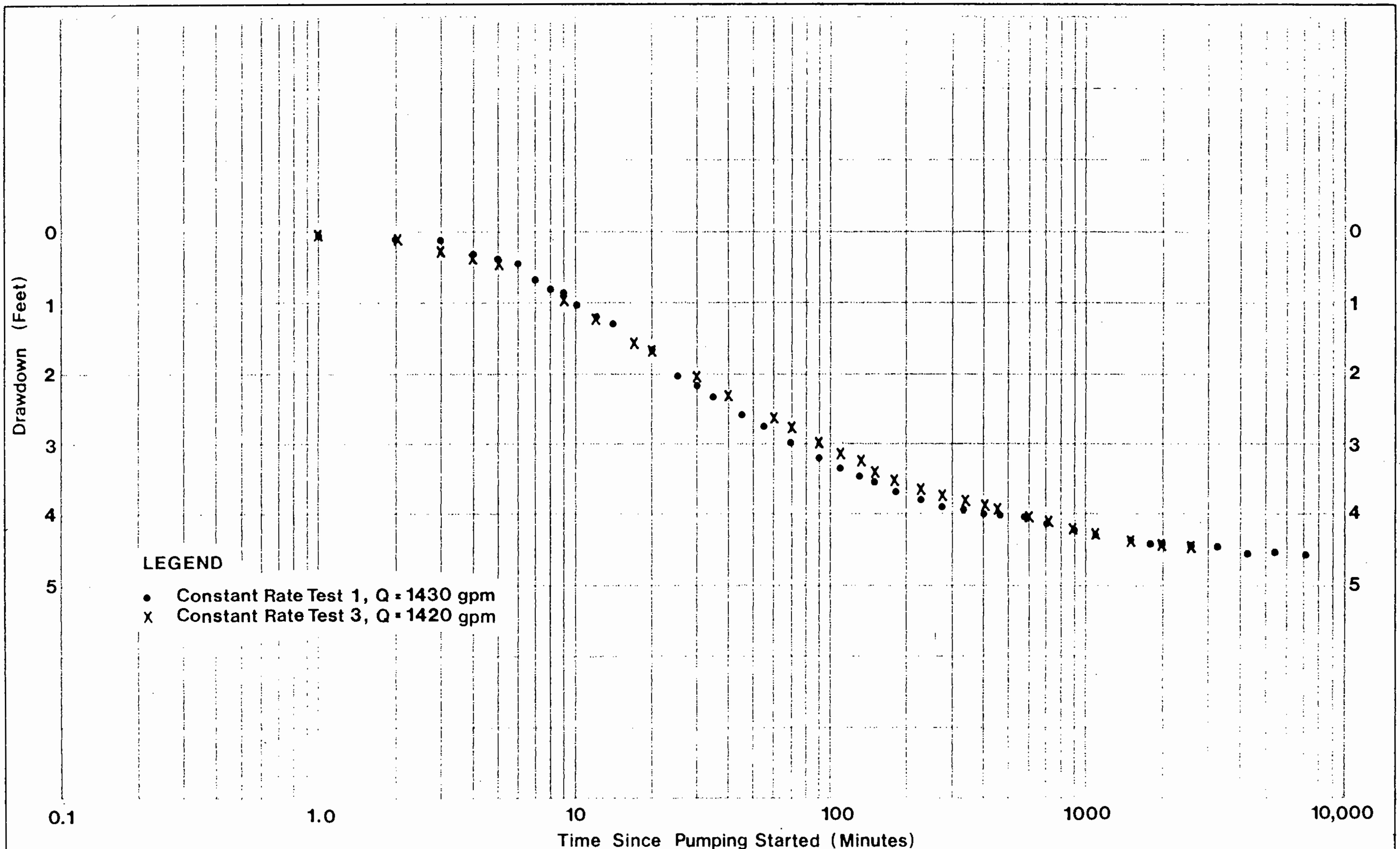
STEPTOE VALLEY

Time-Recovery Curve from Observation Well 1C Recovery from Test 2

Leeds, Hill and Jewett, Inc.

December 1982

FIGURE 3-38



Leeds, Hill and Jewett, Inc.

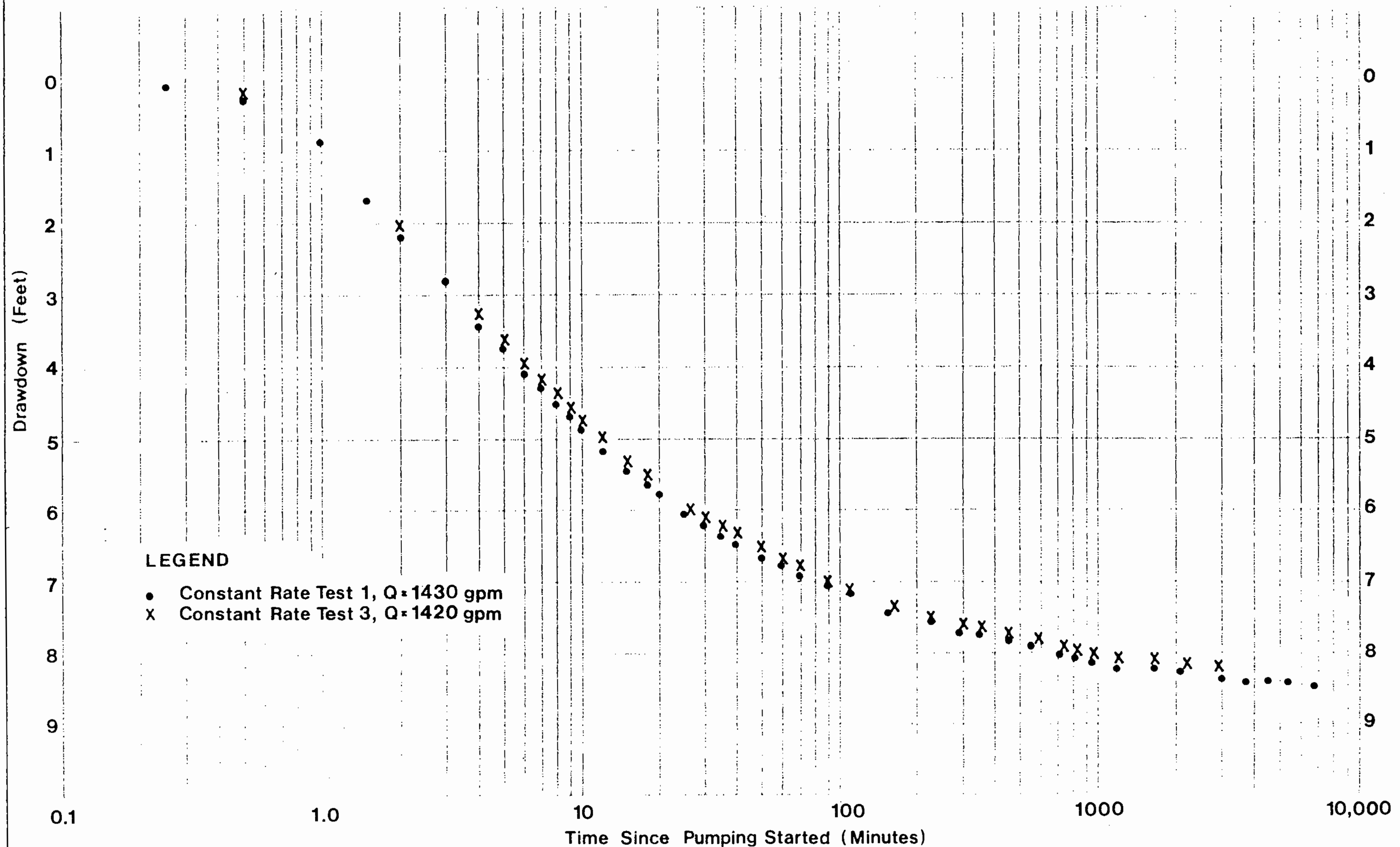
December 1982

WHITE PINE POWER PROJECT

STEPTOE VALLEY

Time-Drawdown Curve from Constant Rate Tests 1 and 3 Observation Well 1B

FIGURE 3·39



Leeds, Hill and Jewett, Inc.

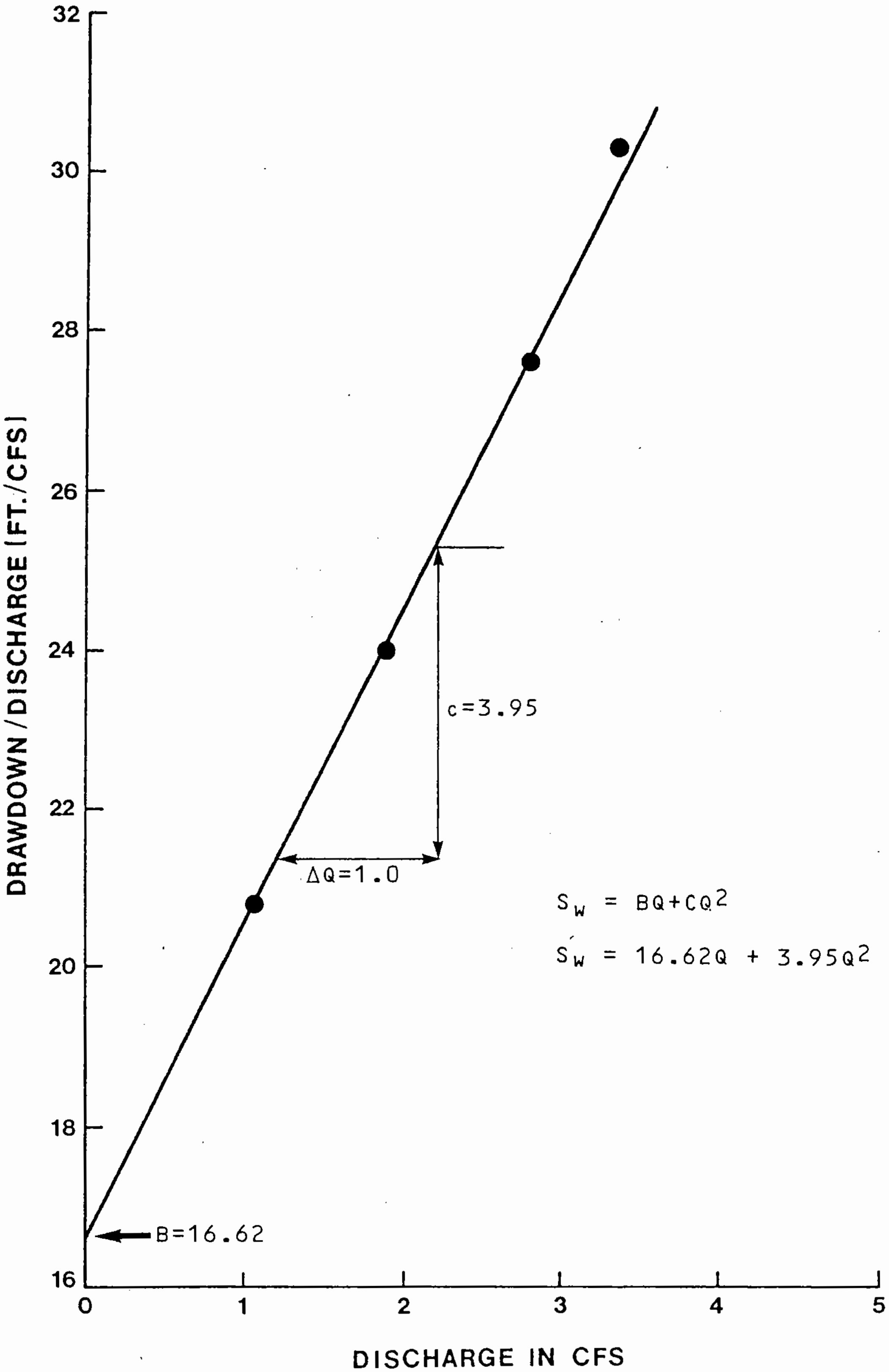
December 1982

WHITE PINE POWER PROJECT

STEPTOE VALLEY

Time-Drawdown Curve from Constant Rate Tests 1 and 3 Observation Well 1C

FIGURE 3·40

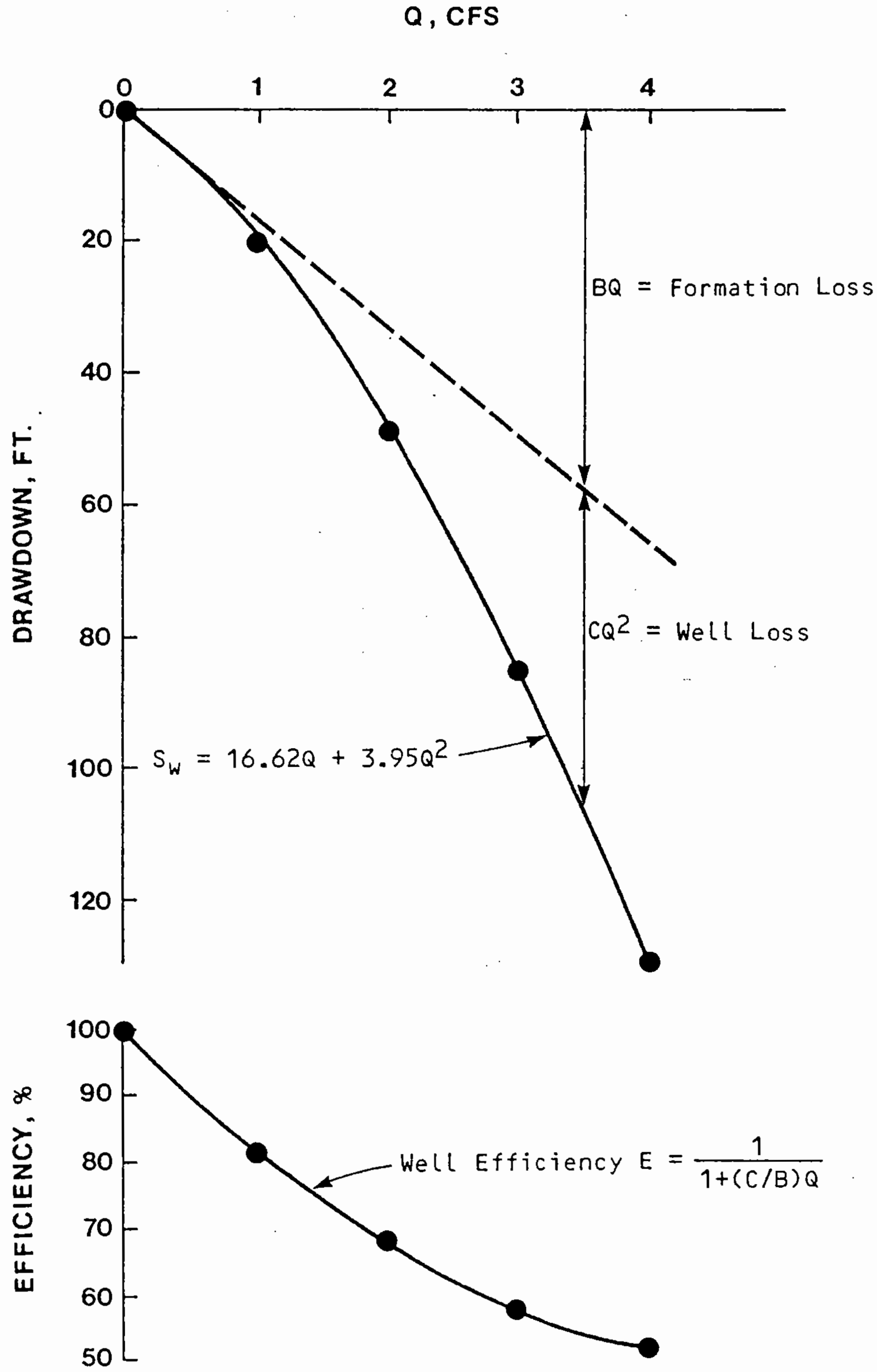


WHITE PINE POWER PROJECT

SPRING VALLEY

Variable Rate Pump Test
Determination of Formation and
Well Loss Characteristics

Leeds, Hill and Jewett, Inc. December 1982 FIGURE 3·41



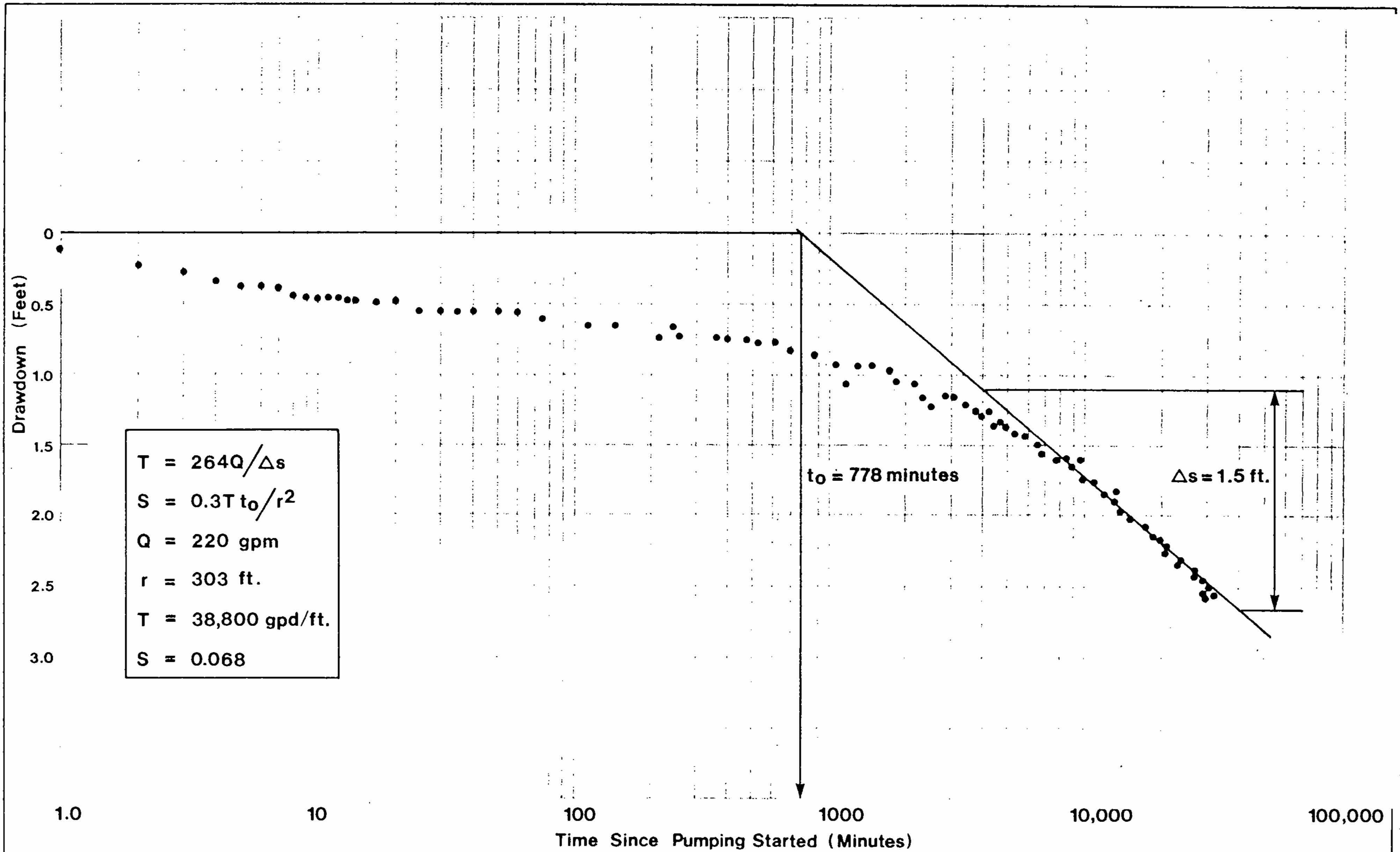
WHITE PINE POWER PROJECT

SPRING VALLEY

Variable Rate Pump Test

Efficiency of Production Well 2A

Leeds, Hill and Jewett, Inc. December 1982 FIGURE 3·42



WHITE PINE POWER PROJECT

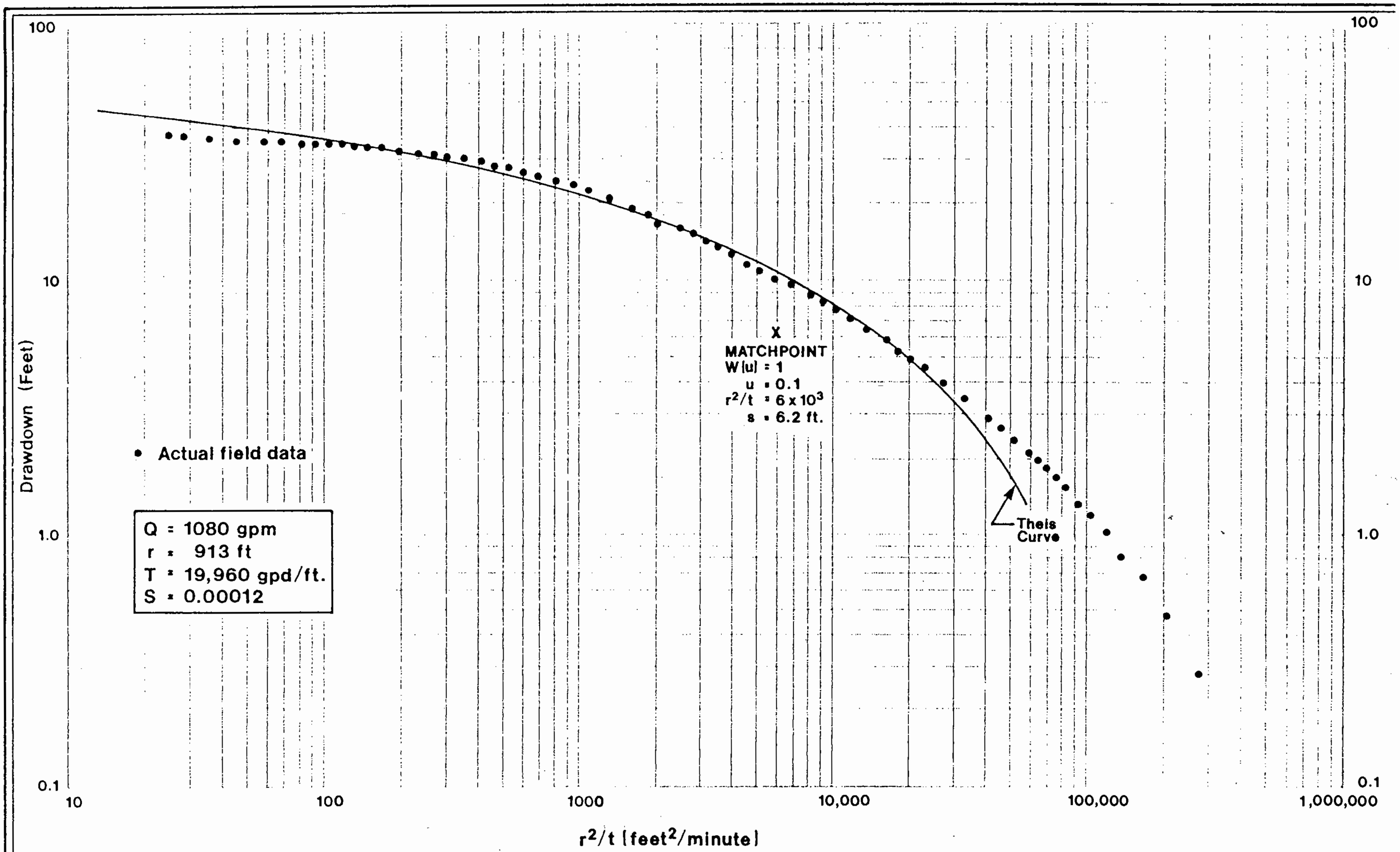
SPRING VALLEY

Pump Test Analysis from Observation Well 2C Constant Rate Pumping Test

Leeds, Hill and Jewett, Inc.

December 1982

FIGURE 3·43



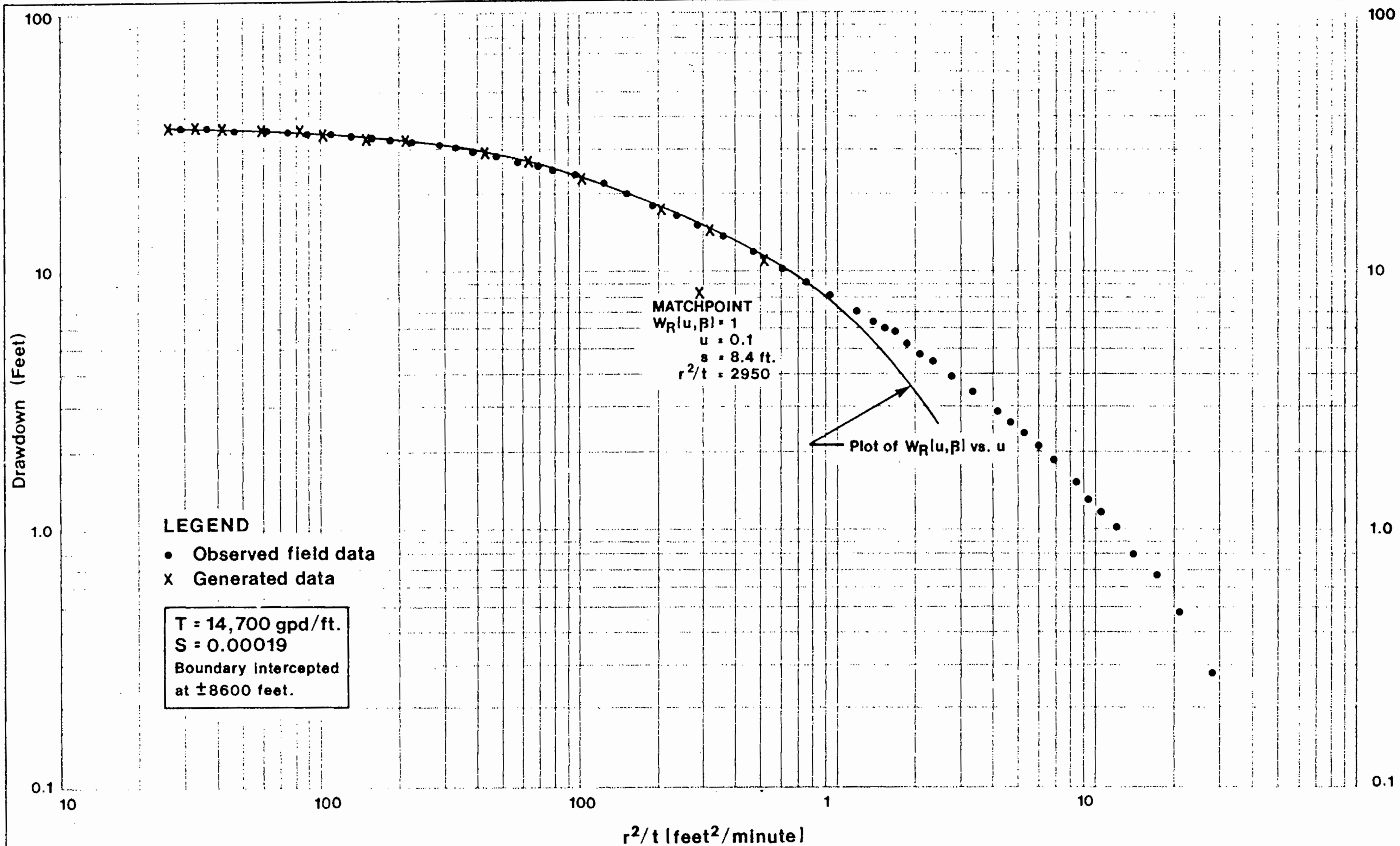
WHITE PINE POWER PROJECT

SPRING VALLEY
Log-Log Plot of Drawdown
Confined Aquifer Analysis
Observation Well 2B

Leeds, Hill and Jewett, Inc.

December 1982

FIGURE 3·44



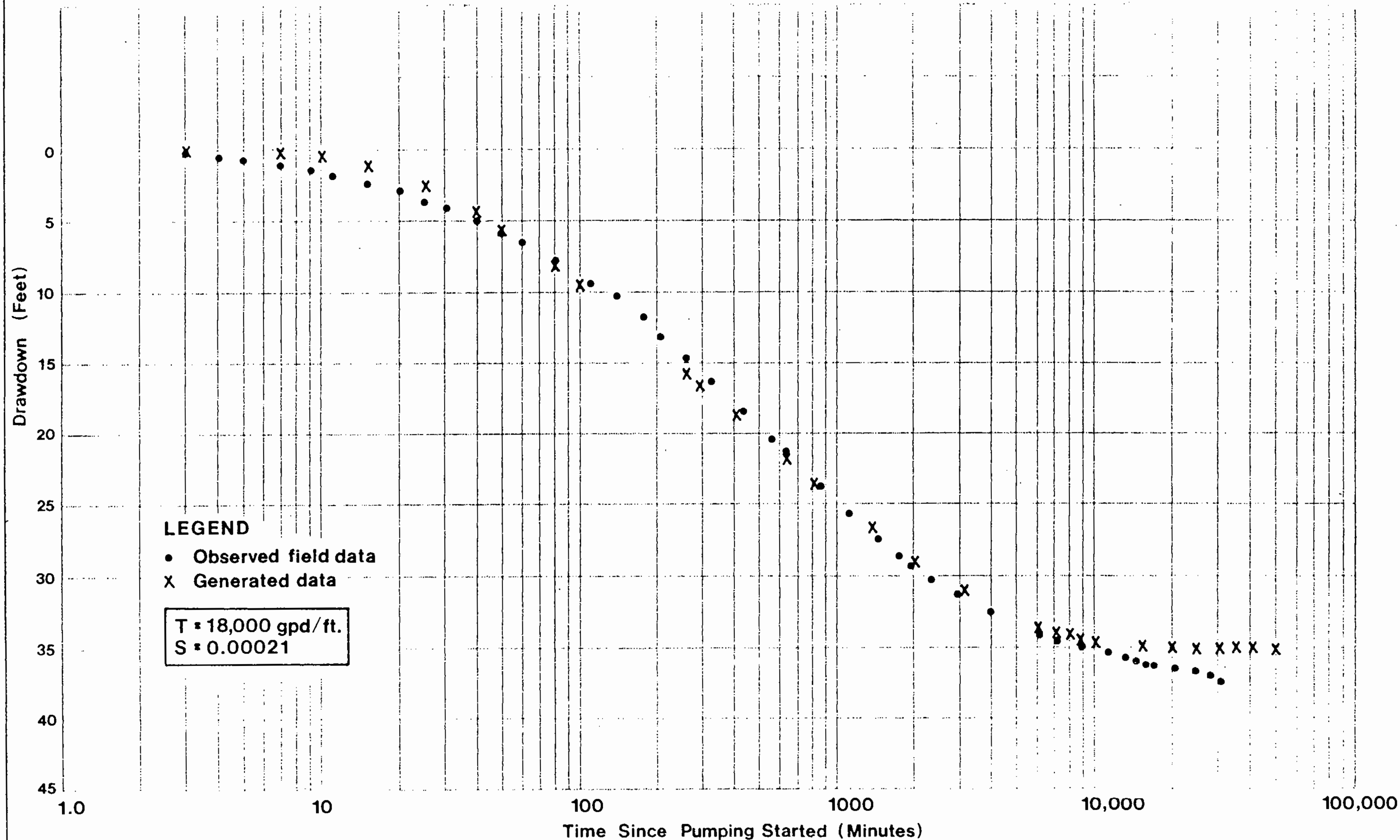
WHITE PINE POWER PROJECT

SPRING VALLEY
Leaky Confined Aquifer
Intercepting A Recharge Boundary
Observation Well 2B

Leeds, Hill and Jewett, Inc.

December 1982

FIGURE 3·45



WHITE PINE POWER PROJECT

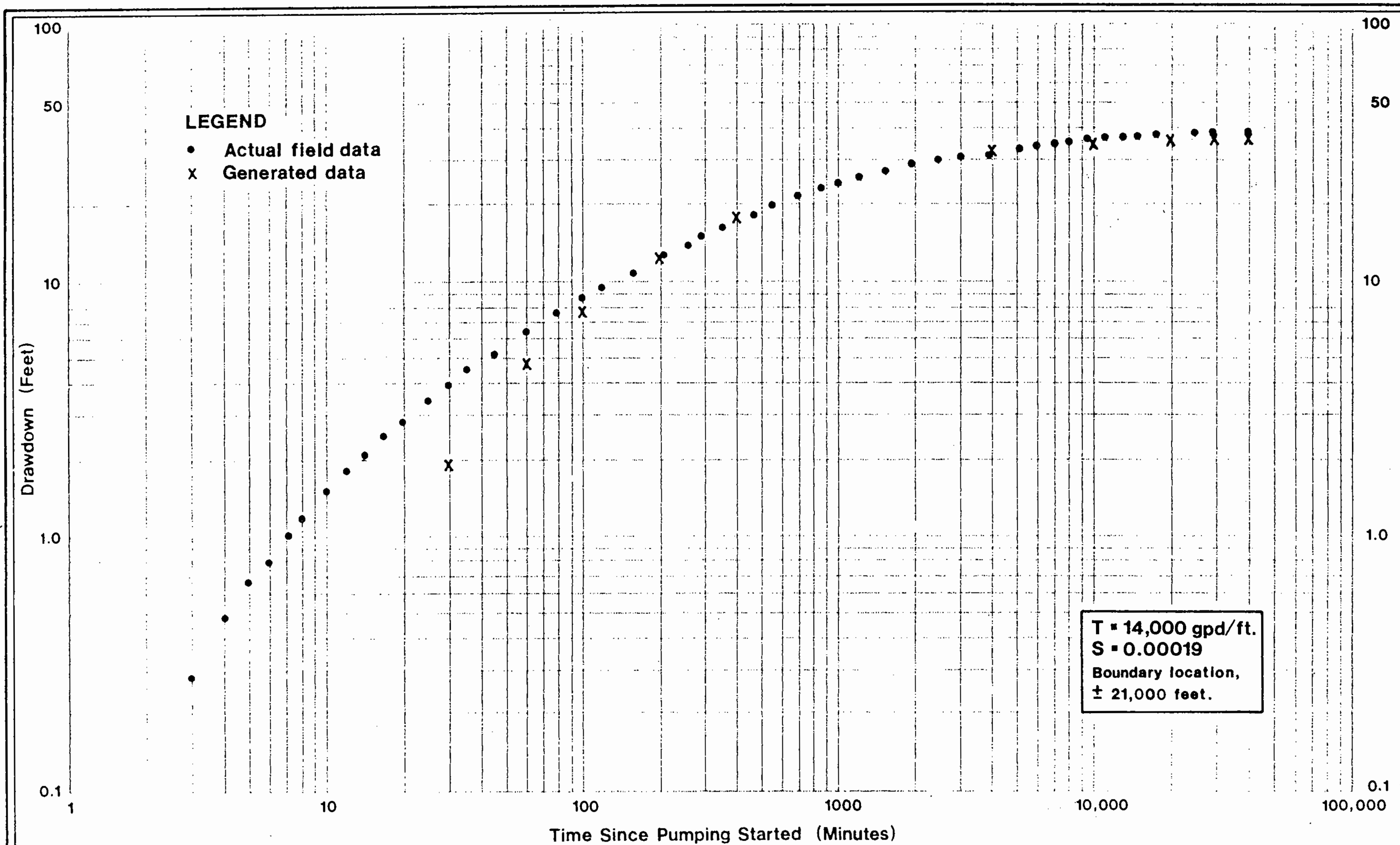
SPRING VALLEY

Leaky Confined Aquifer
No Boundary Conditions
Observation Well 2B

Leeds, Hill and Jewett, Inc.

December 1982

FIGURE 3·46



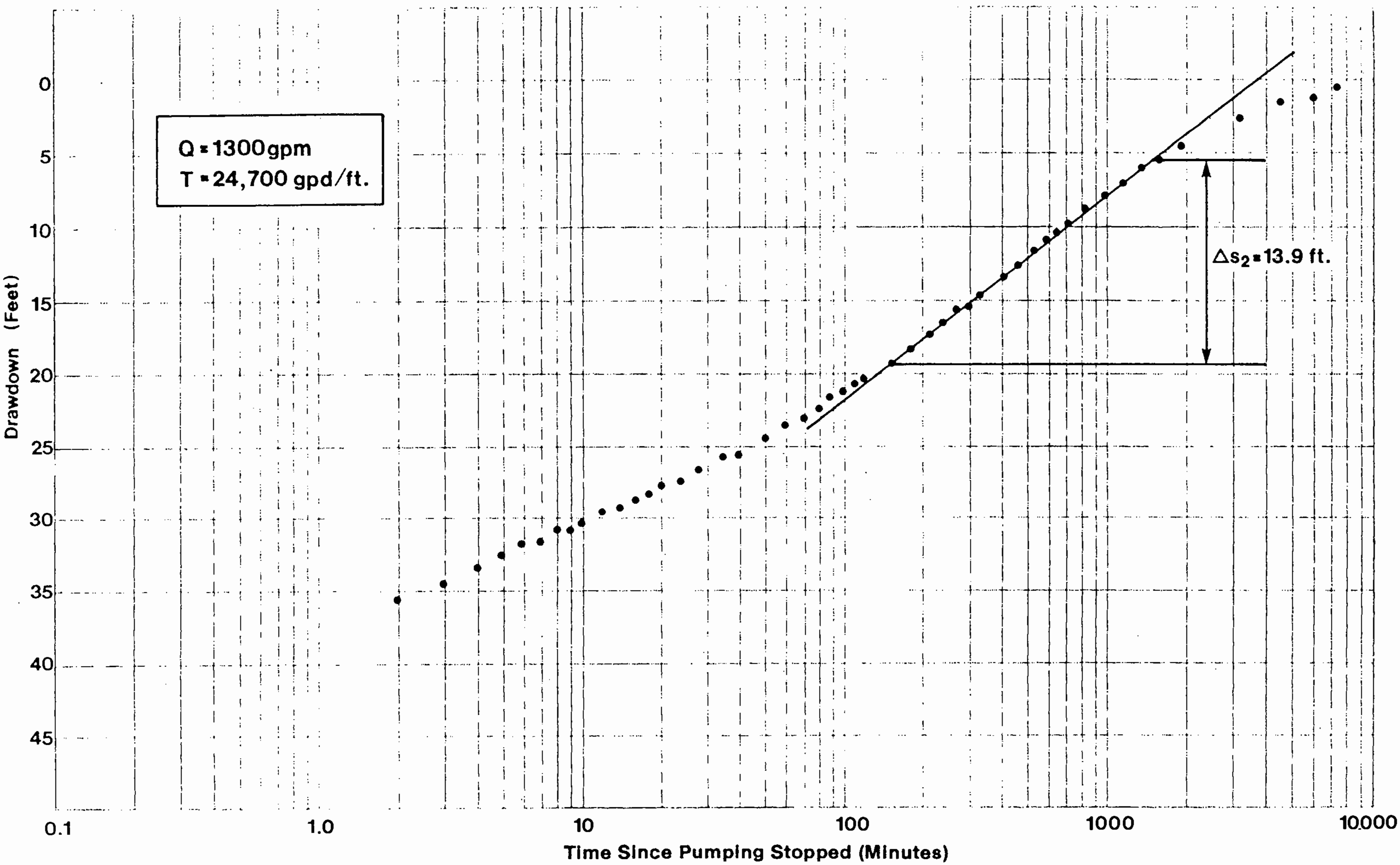
WHITE PINE POWER PROJECT

SPRING VALLEY
Leaky Confined Aquifer
Intercepting an Impermeable Boundary
Observation Well 2B

Leeds, Hill and Jewett, Inc.

December 1982

FIGURE 3·47



WHITE PINE POWER PROJECT

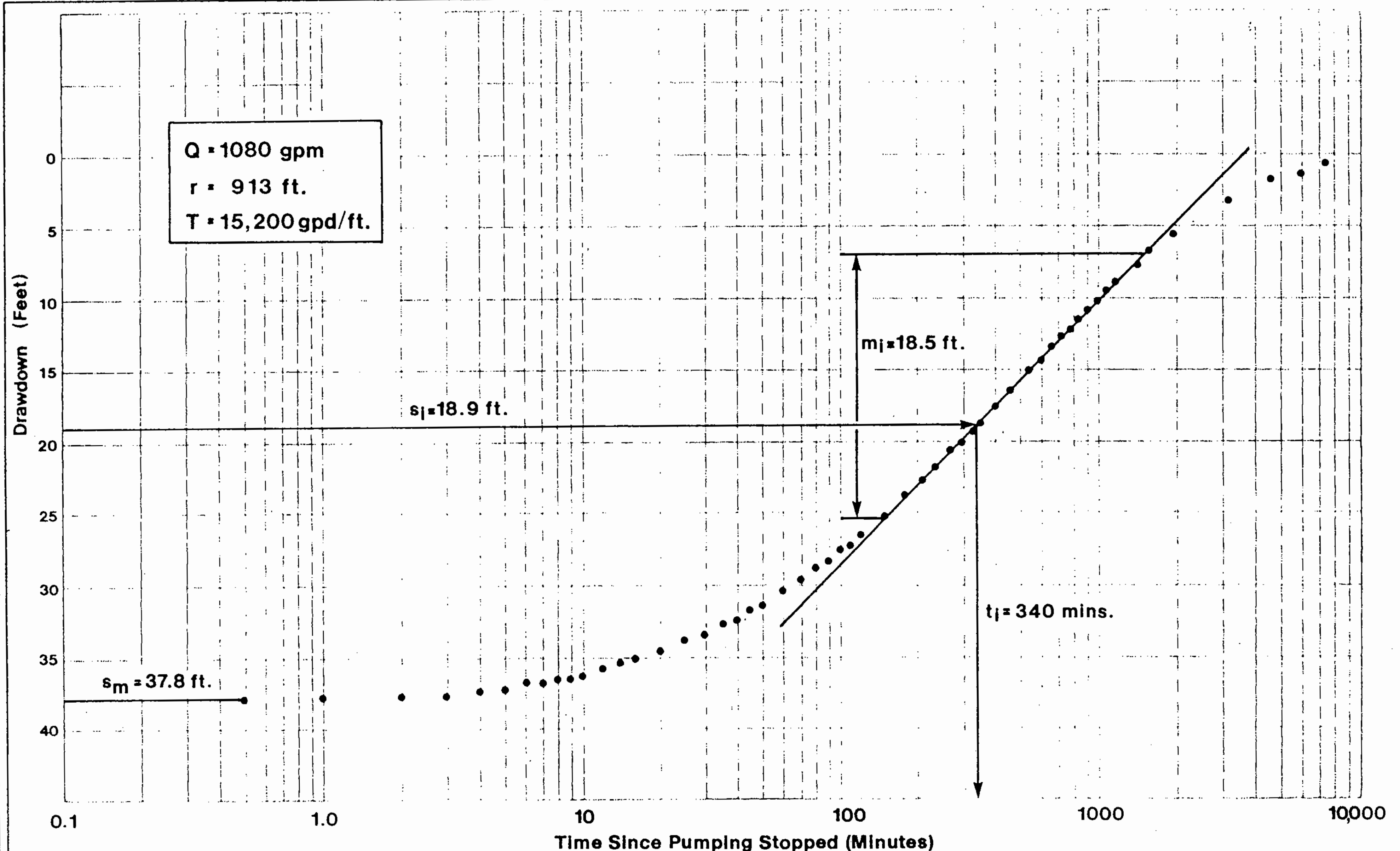
SPRING VALLEY

Time-Recovery Curve from Production Well 2A Recovery Test

FIGURE 3·48

Leeds, Hill and Jewett, Inc.

December 1982



WHITE PINE POWER PROJECT

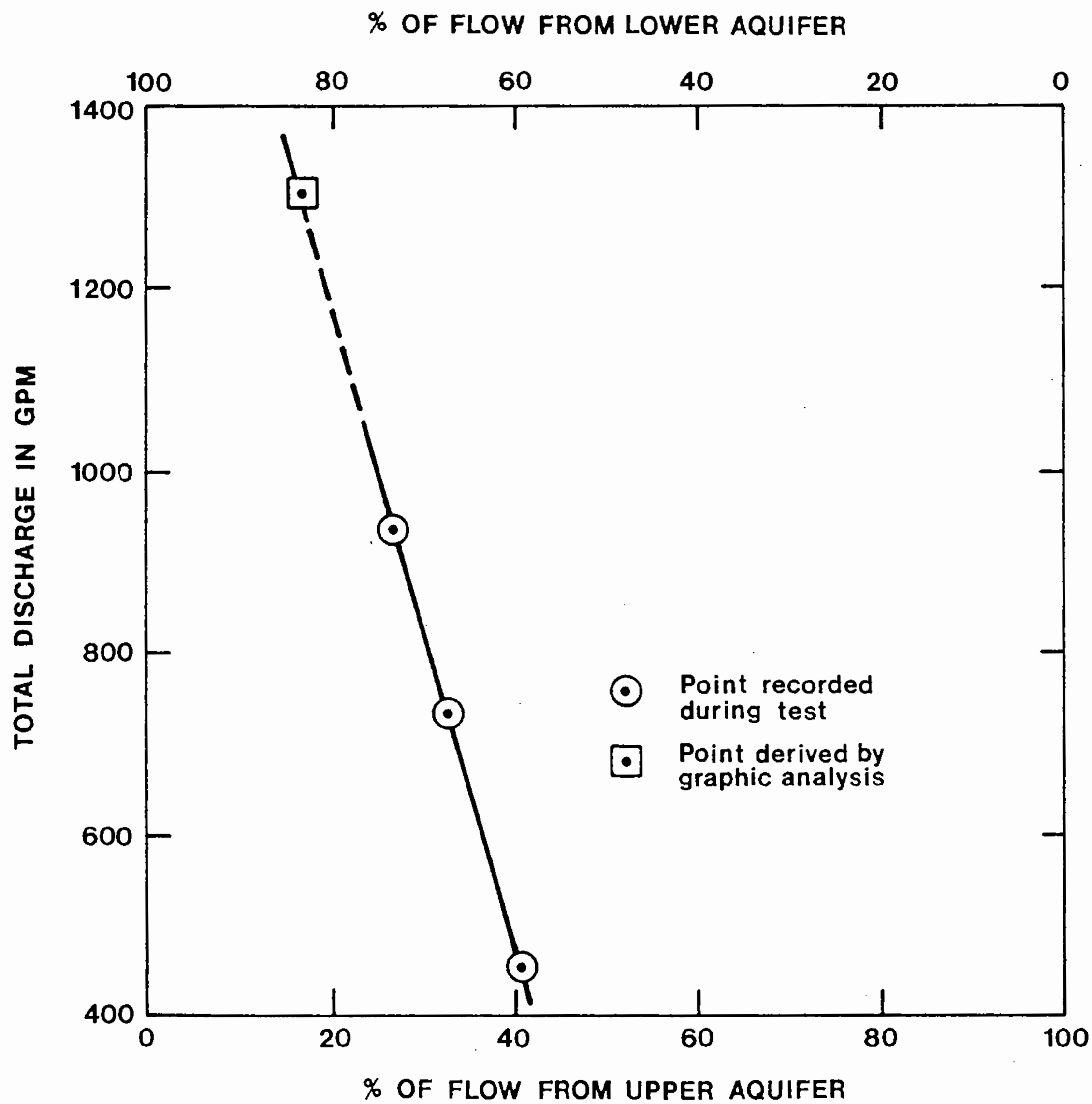
SPRING VALLEY

Time-Recovery Curve from Observation Well 2B Recovery Test

Leeds, Hill and Jewett, Inc.

December 1982

FIGURE 3·49

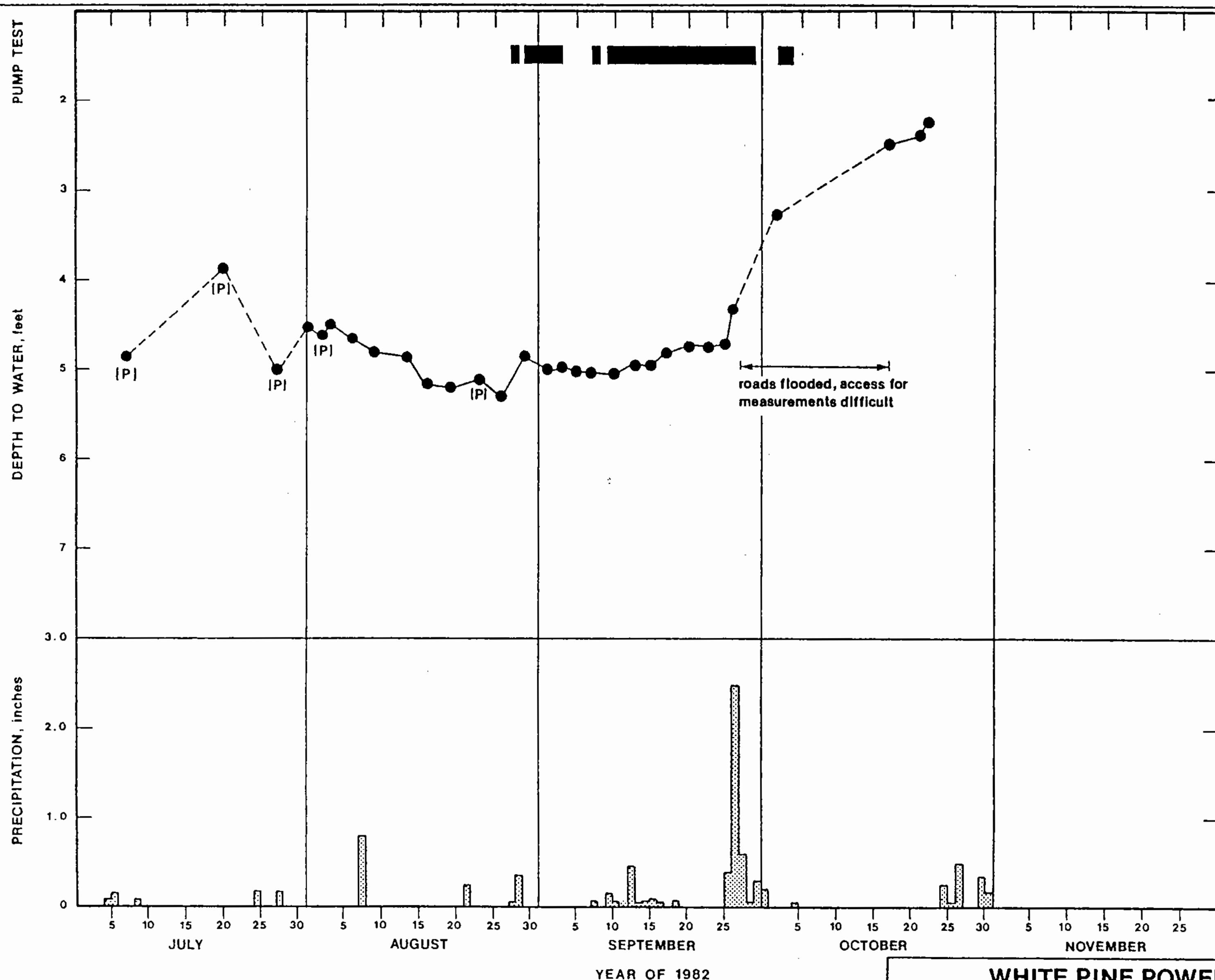


WHITE PINE POWER PROJECT

SPRING VALLEY

Analysis of Flowmeter Test

Leeds, Hill and Jewett, Inc. December 1982 FIGURE 3·50



NOTE: (P) indicates well was pumping at time of measurement.

Leeds, Hill and Jewett, Inc.

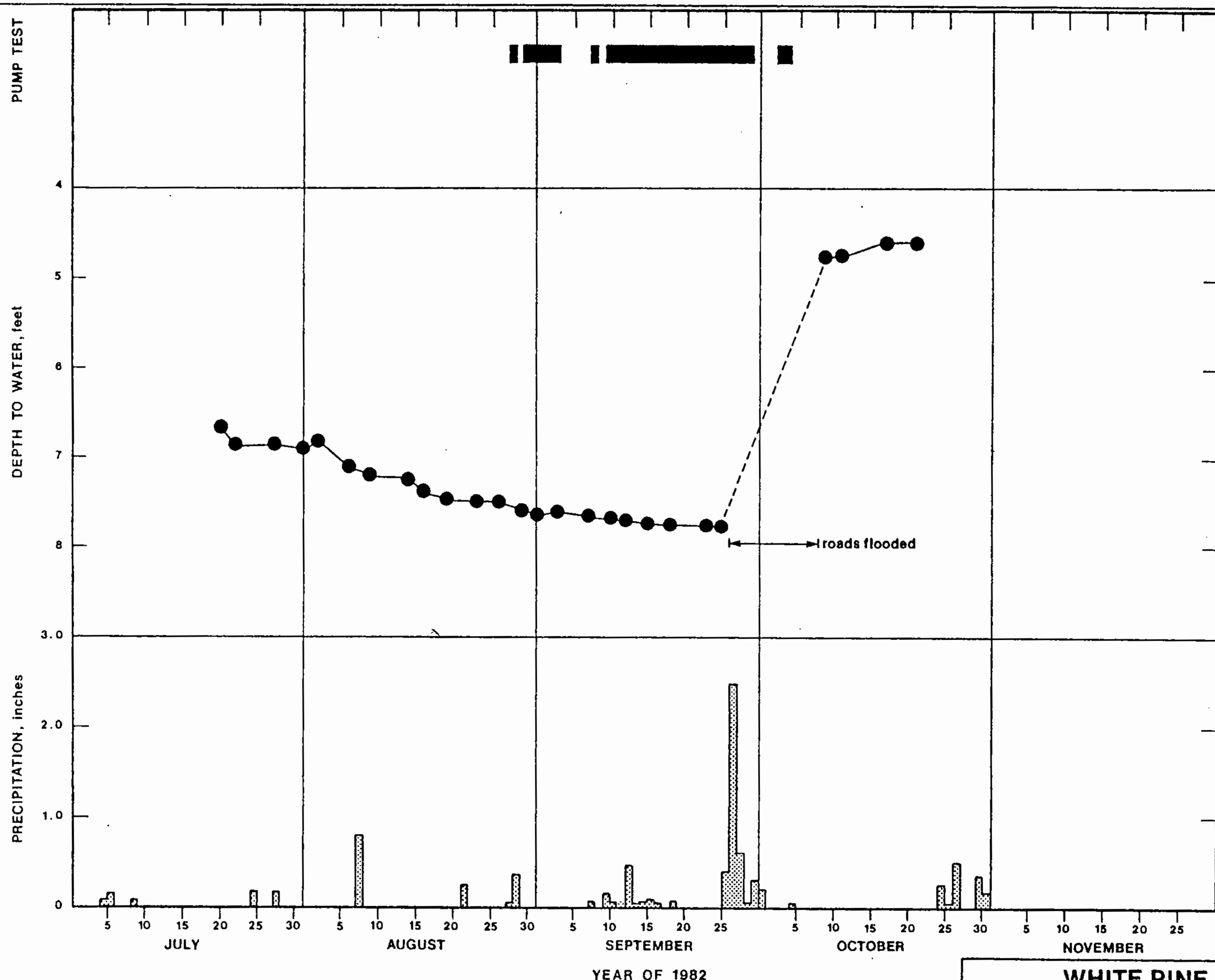
December 1982

WHITE PINE POWER PROJECT

STEPTOE VALLEY

Monitoring Station DC

FIGURE 4·1



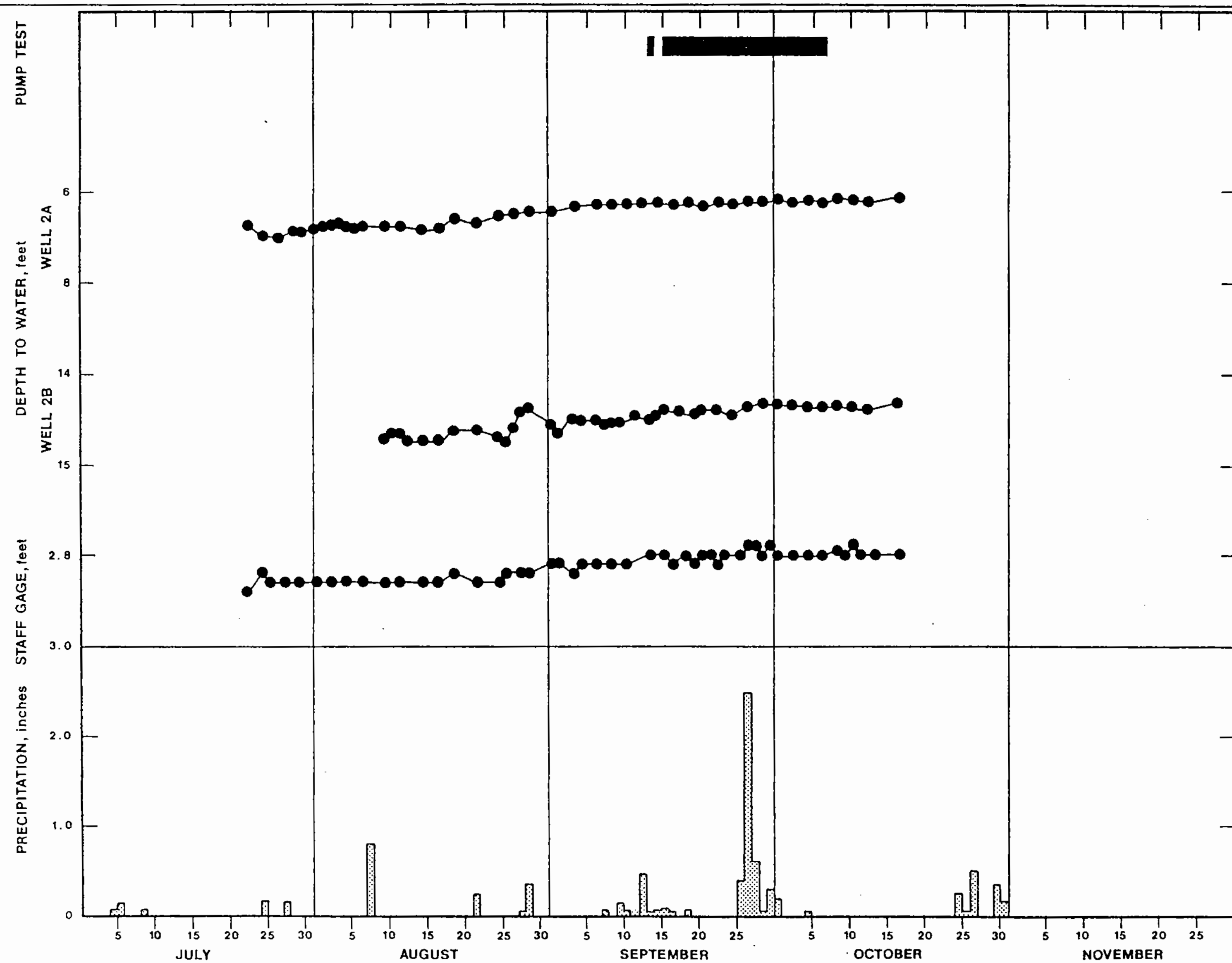
WHITE PINE POWER PROJECT

STEPTOE VALLEY

Monitoring Station ET

Leeds, Hill and Jewett, Inc.

December 1982 FIGURE 4·2



WHITE PINE POWER PROJECT

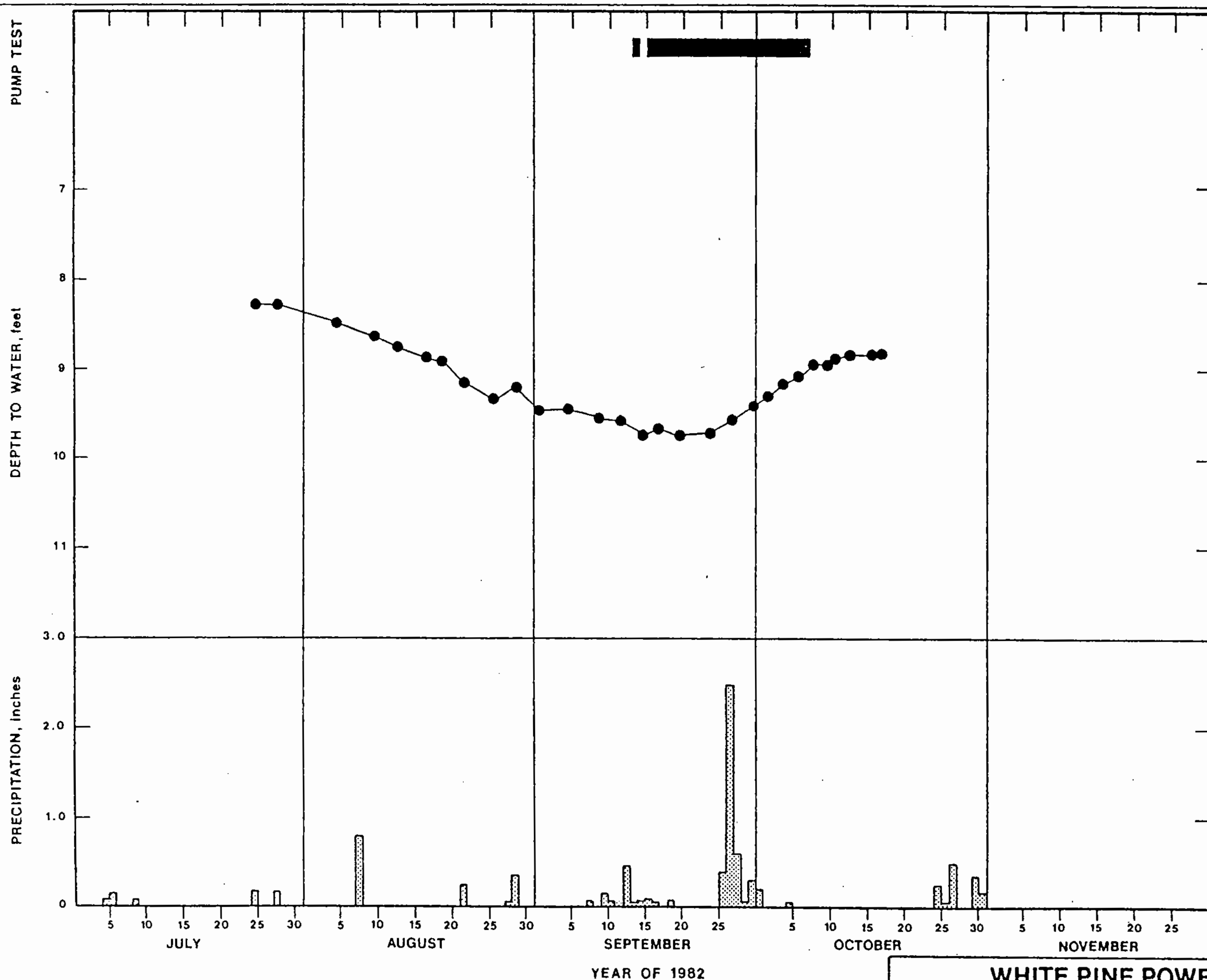
SPRING VALLEY

Monitoring Station
Spring No. 2

Leeds, Hill and Jewett, Inc.

December 1982

FIGURE 4·3



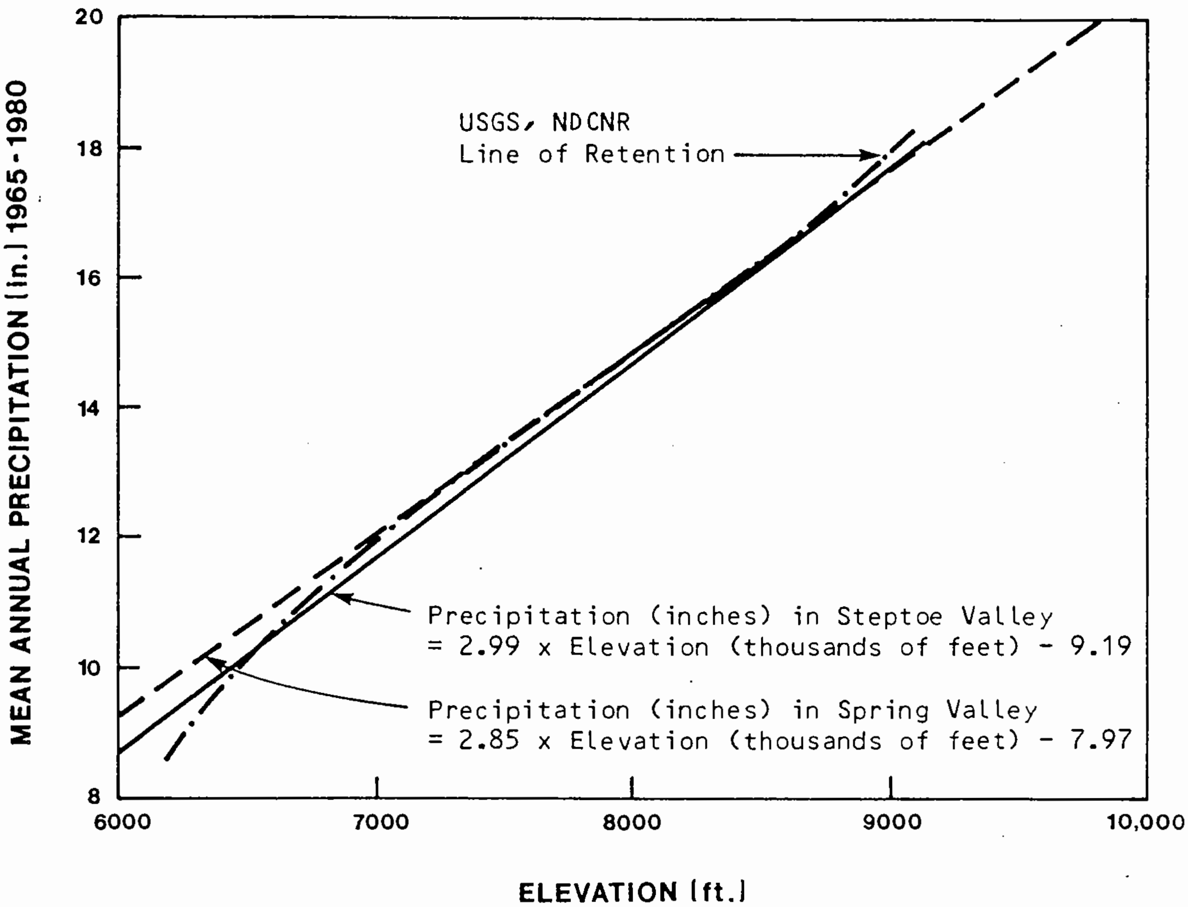
WHITE PINE POWER PROJECT

SPRING VALLEY

Monitoring Station EL

Leeds, Hill and Jewett, Inc.

December 1982 FIGURE 4·4



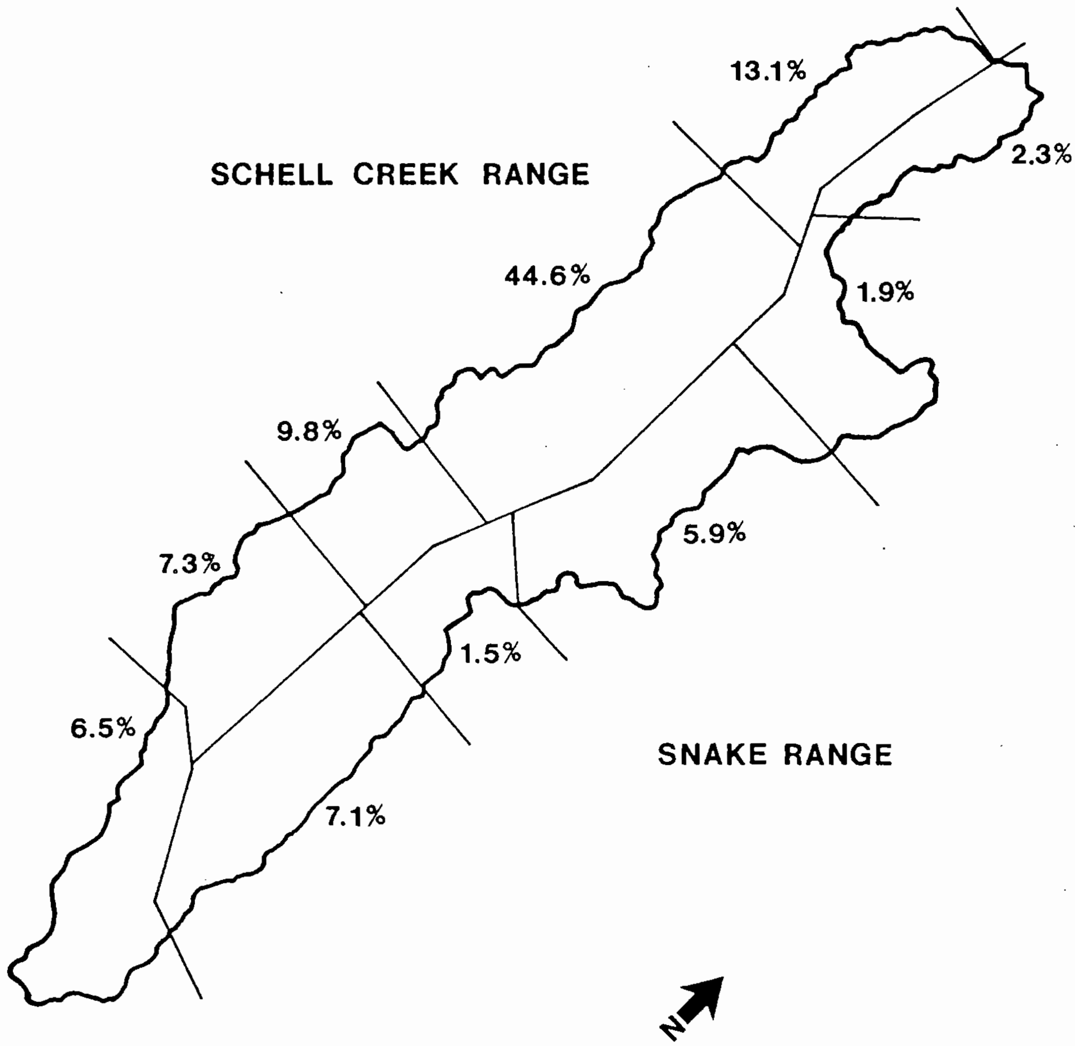
WHITE PINE POWER PROJECT

PRECIPITATION

vs.

ELEVATION

Leeds, Hill and Jewett, Inc. December 1982 FIGURE 6-1

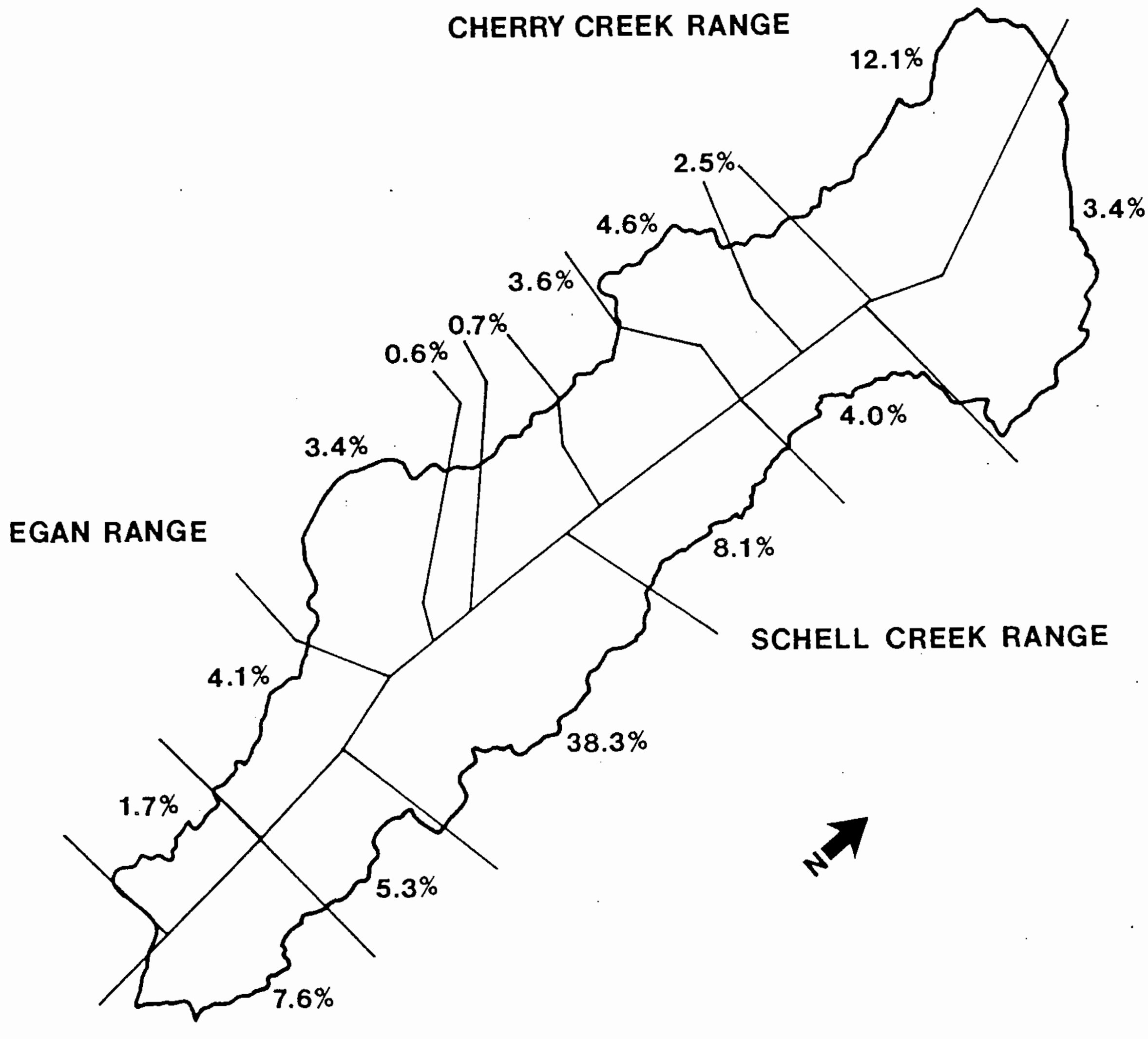


WHITE PINE POWER PROJECT

SPRING VALLEY

Schematic Distribution of Recharge

Leeds, Hill and Jewett, Inc. December 1982 FIGURE 6-2

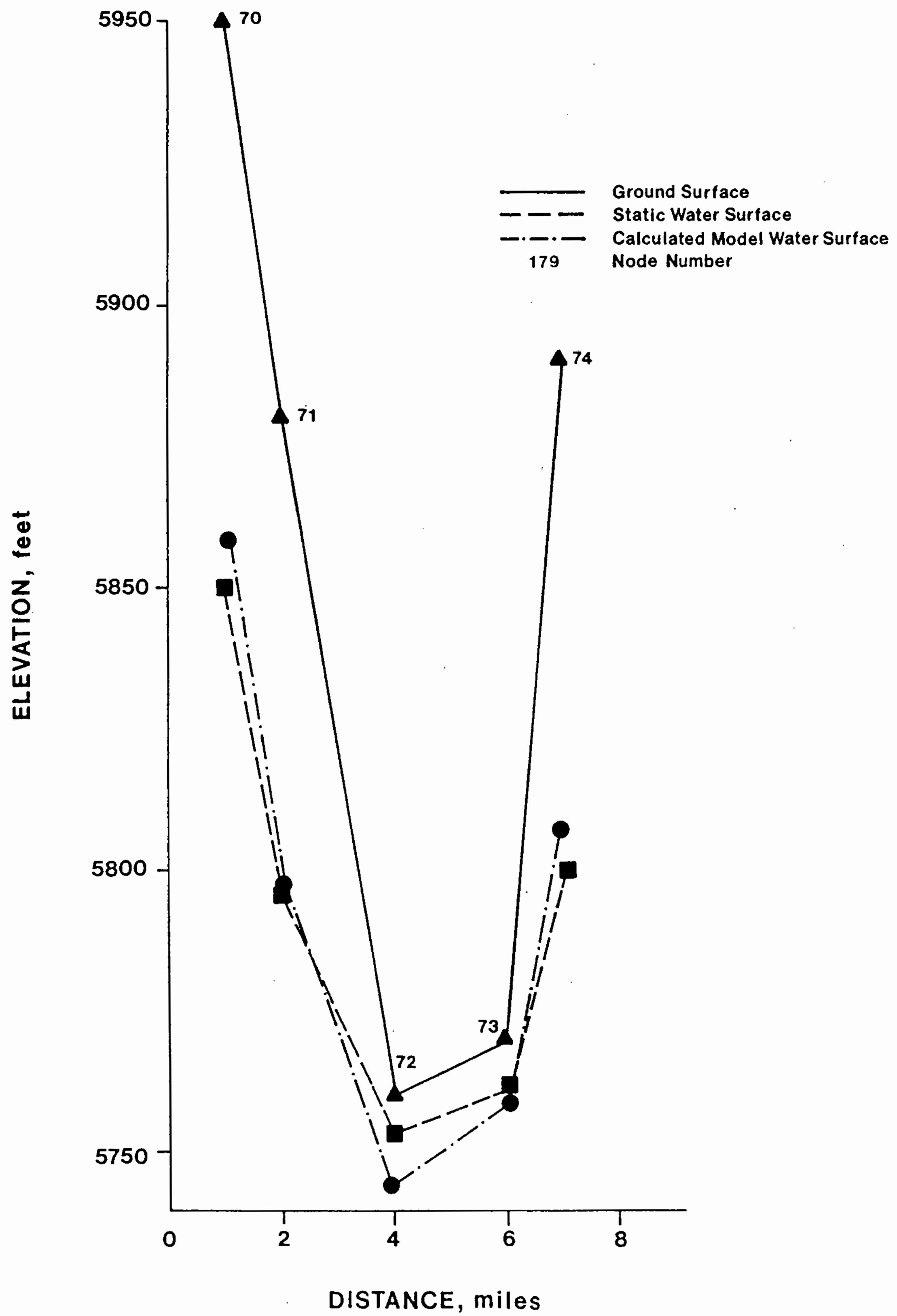


WHITE PINE POWER PROJECT

STEPTOE VALLEY

Schematic Distribution of Recharge

Leeds, Hill and Jewett, Inc. December 1982 FIGURE 6·3



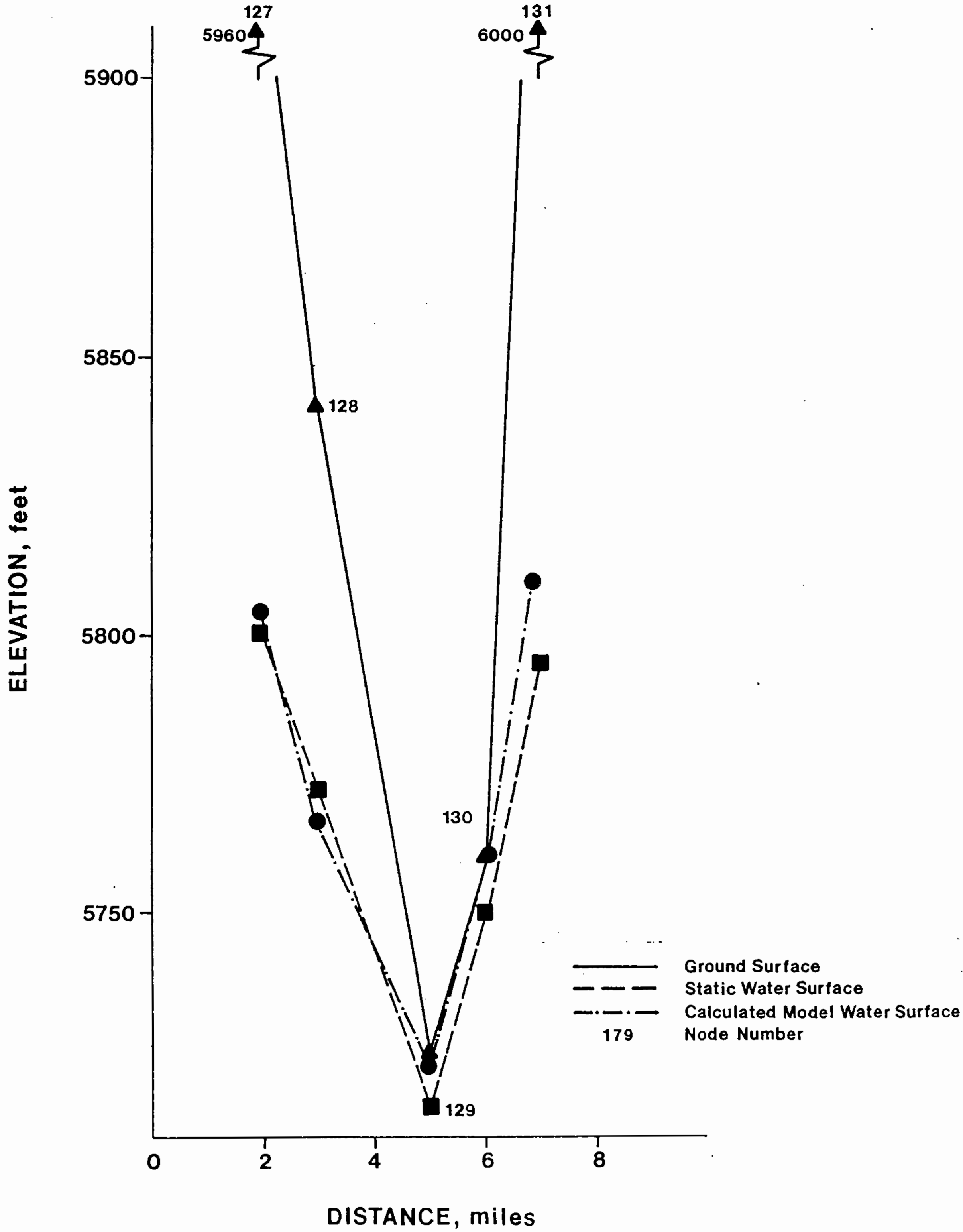
WHITE PINE POWER PROJECT

SPRING VALLEY

East-West Profile A

Nodes 70-74

Leeds, Hill and Jewett, Inc. December 1982 FIGURE 6-4



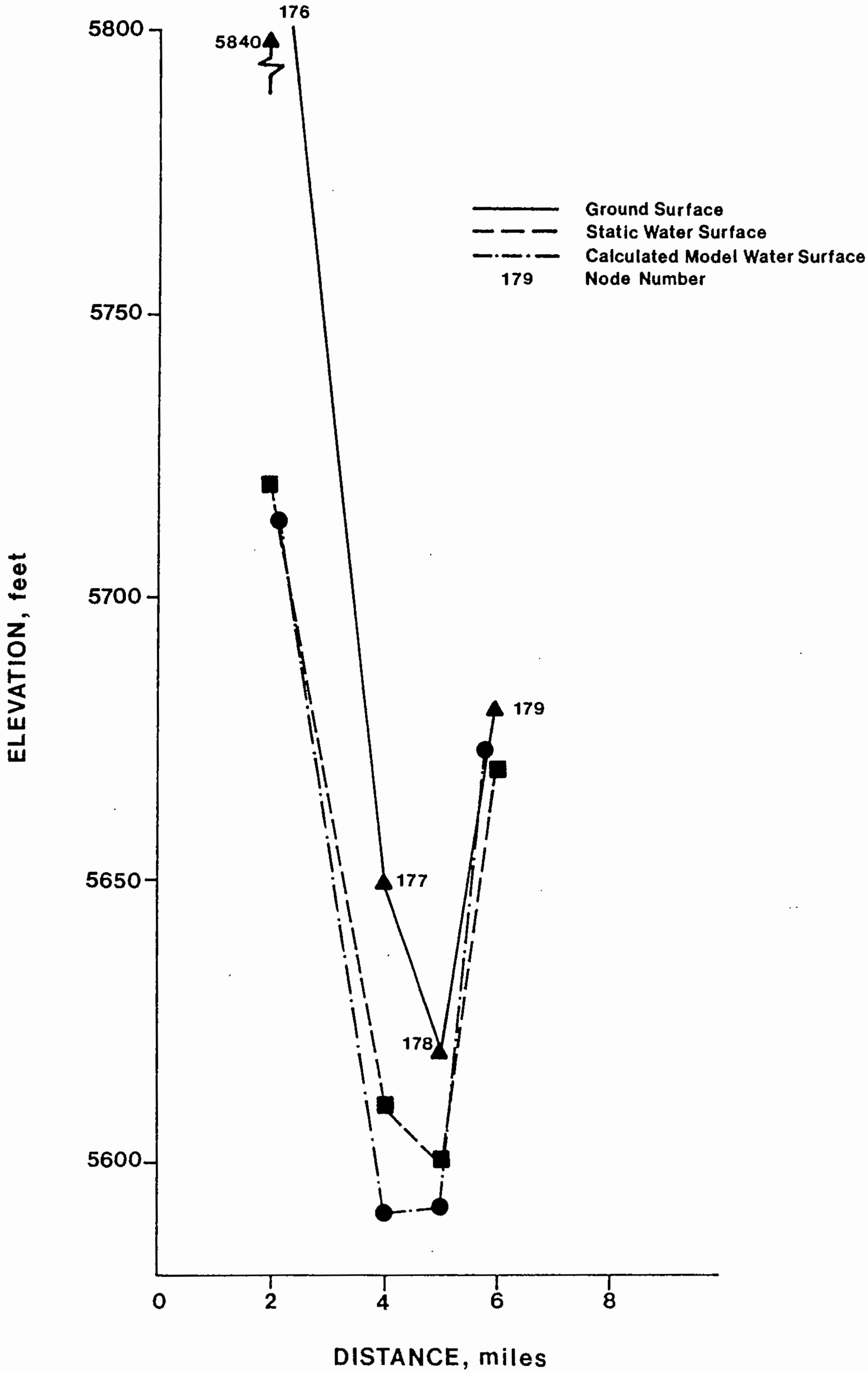
WHITE PINE POWER PROJECT

SPRING VALLEY

East-West Profile B

Nodes 127-131

Leeds, Hill and Jewett, Inc. December 1982 FIGURE 6·5



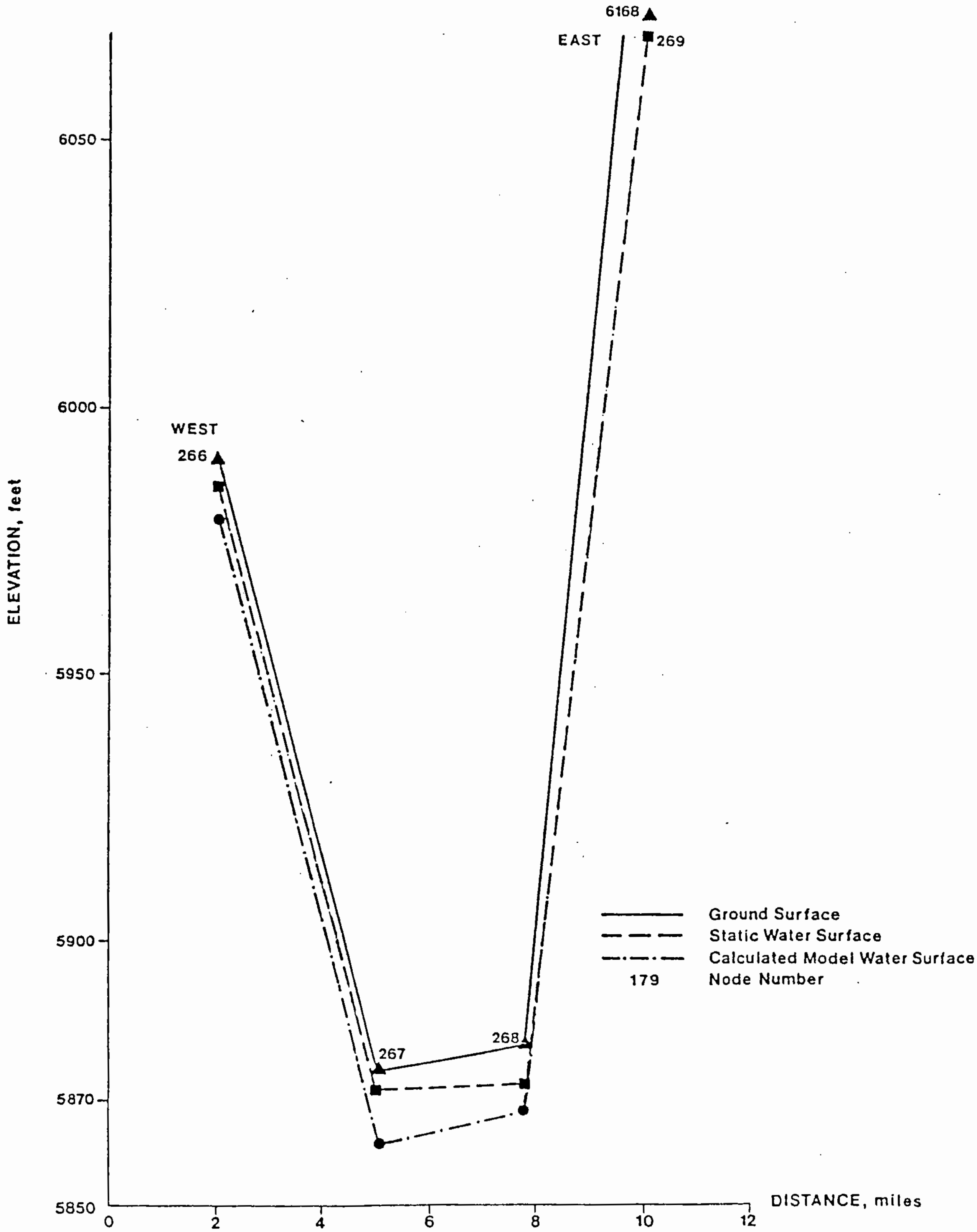
WHITE PINE POWER PROJECT

SPRING VALLEY

East-West Profile C

Nodes 176-179

Leeds, Hill and Jewett, Inc. December 1982 FIGURE 6-6



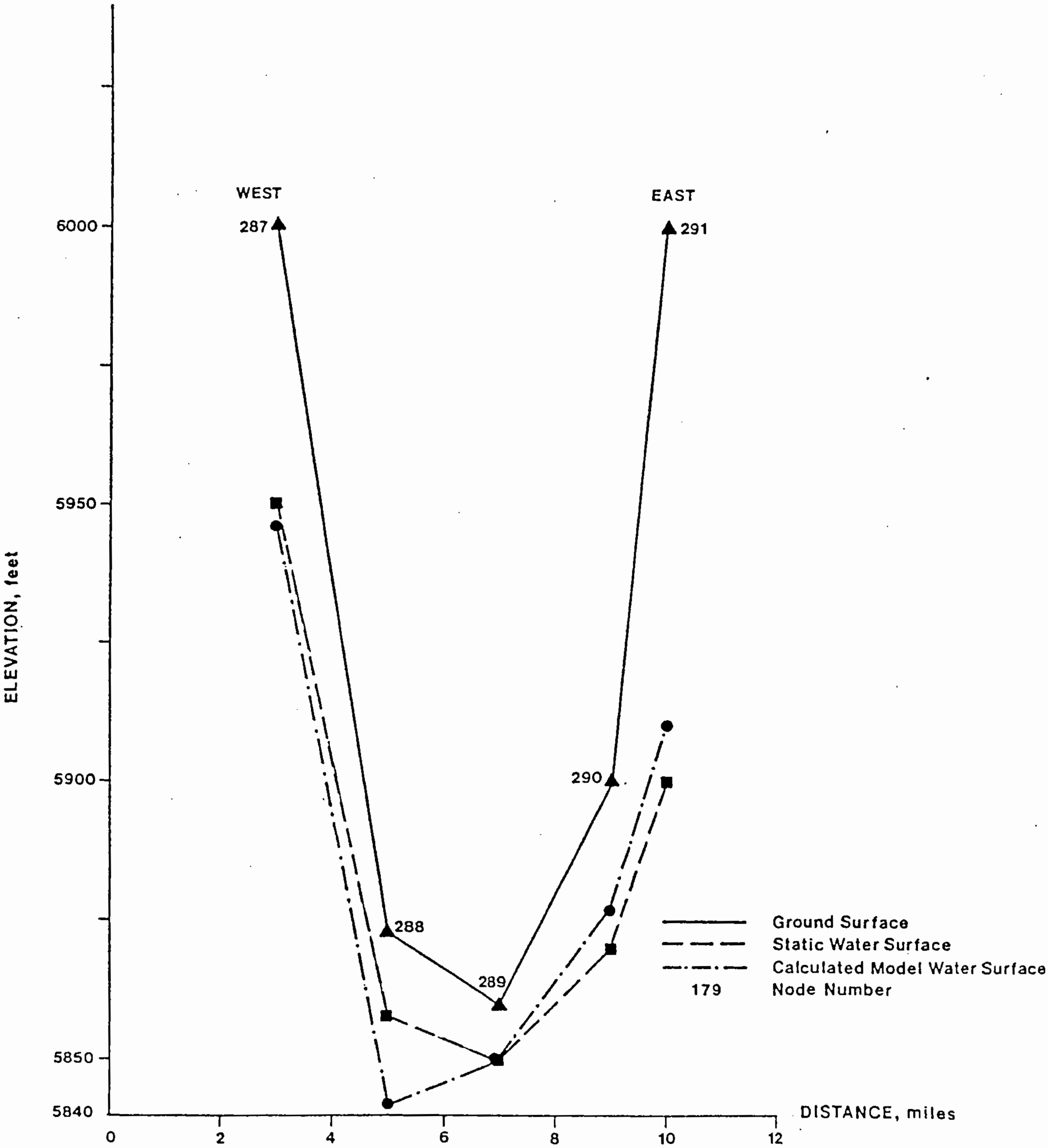
WHITE PINE POWER PROJECT

STEPTOE VALLEY

East-West Profile A

at Cherry Creek Station

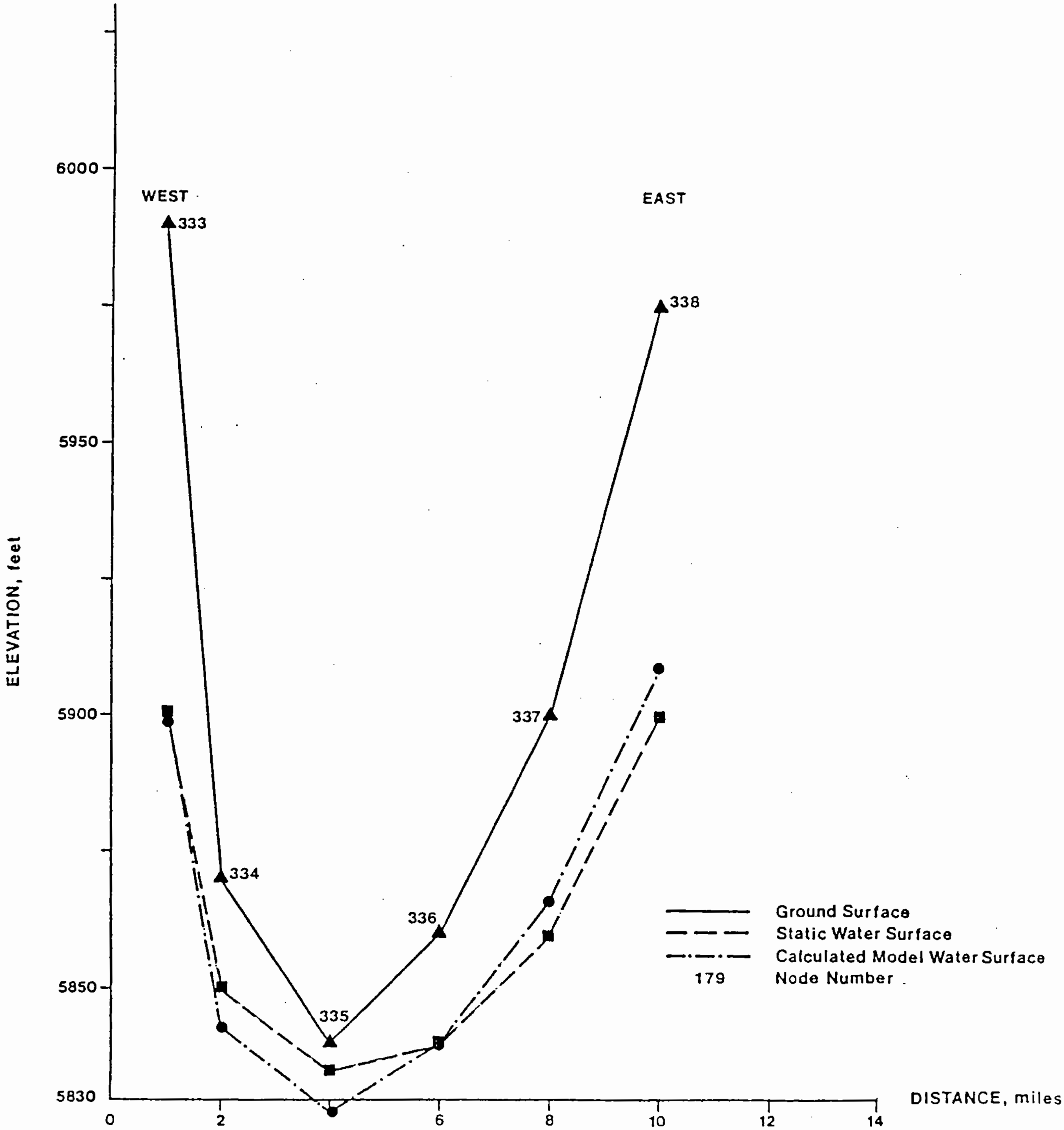
Leeds, Hill and Jewett, Inc. December 1982 FIGURE 6·7



WHITE PINE POWER PROJECT

STEPTOE VALLEY
East-West Profile B
between County Line
and Cherry Creek Station

Leeds, Hill and Jewett, Inc. December 1982 FIGURE 6·8



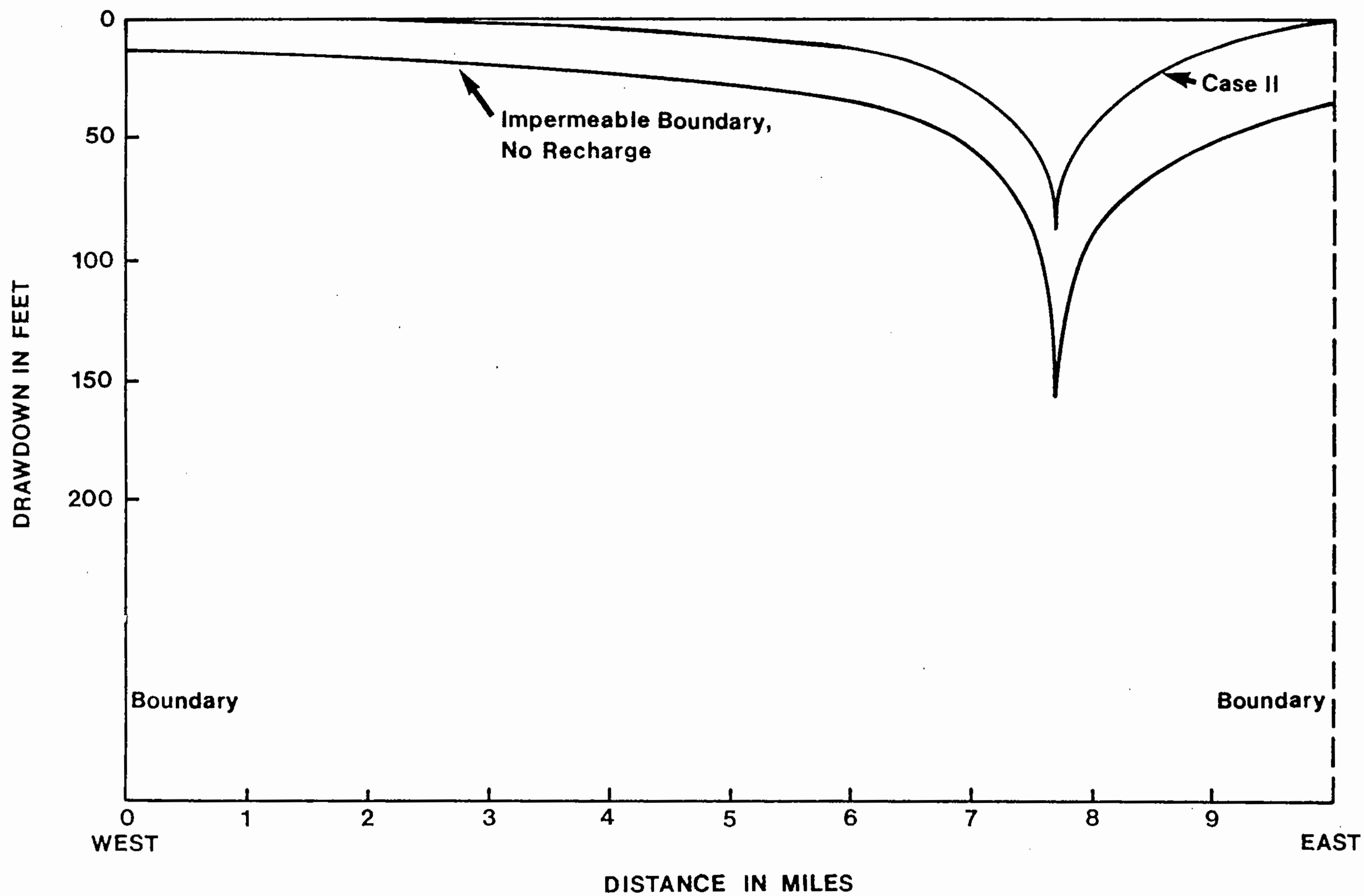
WHITE PINE POWER PROJECT

STEPTOE VALLEY

East-West Profile C

at County Line

Leeds, Hill and Jewett, Inc. December 1982 FIGURE 6·9



NOTE: East-West Cross-Section through Well No. 8

WHITE PINE POWER PROJECT

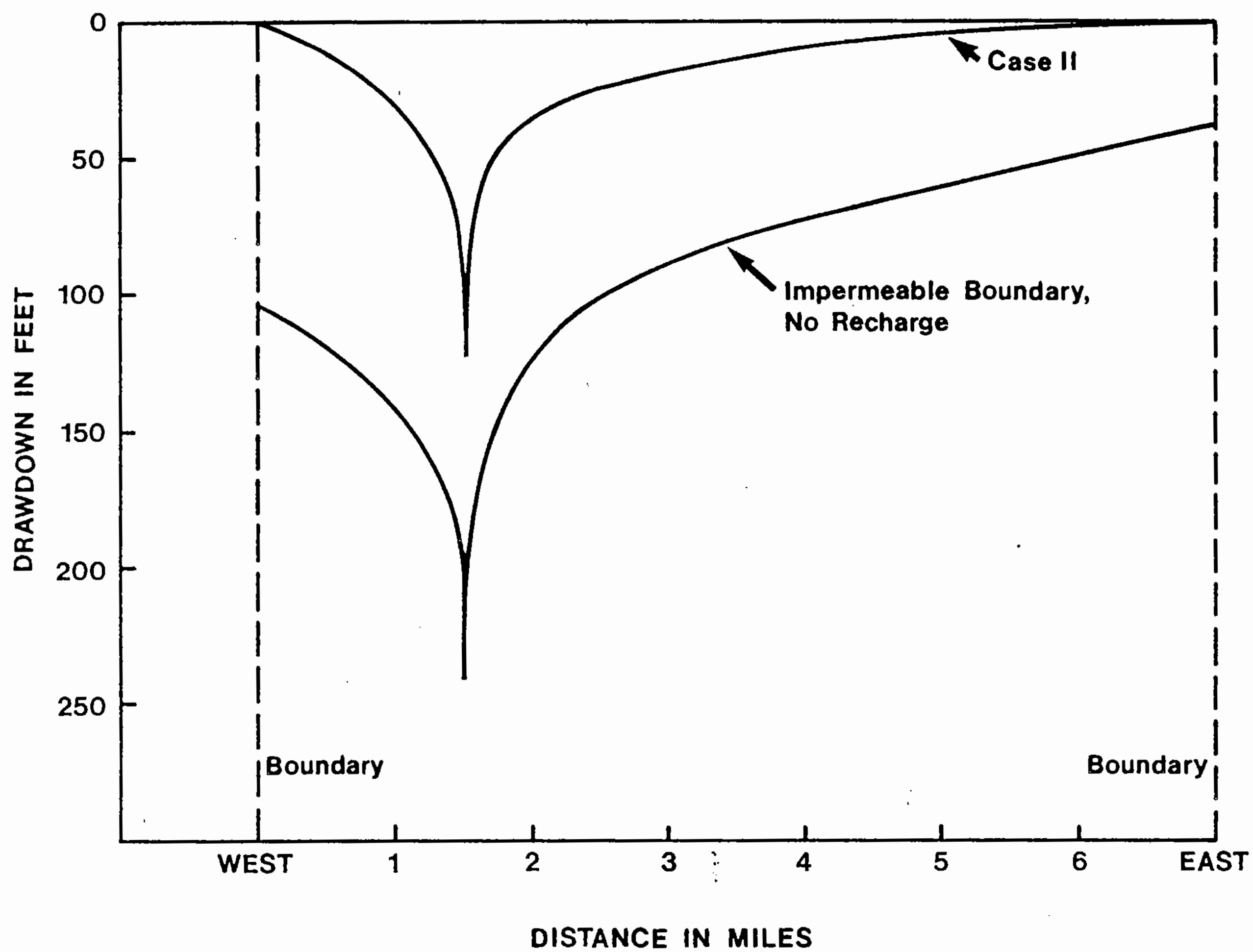
STEPTOE VALLEY

Sensitivity to Various Boundary Conditions

Leeds, Hill and Jewett, Inc.

May 1983

FIGURE 6·10



NOTE: East-West Cross-Section through Well No. 6

WHITE PINE POWER PROJECT

SPRING VALLEY

Sensitivity to Various Boundary Conditions

Leeds, Hill and Jewett, Inc.

May 1983 FIGURE 6·11