RINGLING MacARTHUR RESERVE
Water Resources Investigation
(Appendices)

COUNTY
OF
SARASOTA

Dames & Moore

Bennett & Bishop

Boyle Engineering

#9

APPENDIX E

SITE SURFACE WATER INVESTIGATION

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SITE SURFACE WATER INVESTIGATION

E.1 INTRODUCTION

The purpose of the Ringling-MacArthur Reserve (RMR) site surface water investigation was the development of a hydrologic data base and estimation of RMR surface water supply capabilities. The investigation centered on quantification of the various site hydrologic regime parameters. These parameters are rainfall, evapotranspiration, evaporation, soil infiltration capabilities, recharge and discharge from the Surficial Aquifer, and surface runoff. Discussion of the regional long-term waterbudget is provided in Appendix F.

The investigation was conducted with field data collection and office studies. Field data collection, described in detail in Section E.3.1, was conducted with a network of discharge and stage gages, evapotranspirometers, evaporation pans, and rainfall monitors. Data collection was initiated in March 1985. The final station in the network was installed in July 1985. This analysis incorporates data collected up through December 1985. Data collection is continuing and will continue at least through June of 1986.

Office analysis, initiated in the Fall of 1985, consisted of:

- estimation of the numerical range and probable averages of each of the hydrologic parameters,
- assessment of the relationship between the hydrologic parameters,
- development of predictive models for estimating the long-term site water balance, and
- estimation of the impact to the hydrologic components under various withdrawal scenarios.

The following sections include a general physical description of the site, description of the field and office analysis procedures, a quantification of the hydrologic site parameters, a description of the site water balance modeling procedure, and the results of an analysis of various withdrawal scenarios.

E.2 SITE PHYSICAL DESCRIPTION

The RMR is an approximately 51 square mile tract of land located in eastern Sarasota County, immediately east of the Myakka River, and southeast of Myakka River State Park (Figure E.2-1). Hydrologically the site is a typical southern Gulf Coast wetland/upland mosaic with overall basin slopes of less than 1 foot per mile. The site rises from a low elevation of approximately 5 feet mean sea level (msl) in the southwestern corner near the Myakka River to elevations approaching 35 feet msl in the north central and northeastern portions of the site. Land surfaces in the western one-third of the site slope west to southwest towards the Myakka River. From the central portion of the site to the eastern boundary the land surface is characterized by a relatively level plateau with elevations ranging from 30 to 35 feet msl. The eastern third of the plateau within the RMR boundaries is bisected by the southwestern trending Deer Prairie Slough.

The drainage area divides within and adjacent to the site are shown on Figure E.2-2. The site is characterized by four major drainage systems. Moving across the site from the northwest to the southeast the first system is a series of interconnected, small, poorly drained wetland sloughs draining west to northwest from the site towards Myakka River State Park and ultimately the Myakka River and Lower Myakka Lake. Except for a small basin near Highway 72 these sloughs originate on the RMR and provide drainage into Myakka River State Park. These features drain a total of approximately 9.5 square miles.

The second major drainage system is a series of creeks known as Blackburn Slough, Manace Wallace Slough, and Mell Williams Canal.

These three parallel tributaries drain an area of approximately 16.2 square miles to the southwest towards the Myakka River. The drainage area consists of parallel terraces which gradually rise from the Myakka River floodplain to the plateau feature which characterizes the central and eastern portions of the site.

The third major system is Deer Prairie Slough which drains the aforementioned plateau. The slough has two major tributaries, Windy Sawgrass and High Hammock Canal, in addition to the main channel as shown on Figure E.2-2. Major reaches of Windy Sawgrass and Deer Prairie Slough have been channelized. Deer Prairie Slough at the southern property boundary drains approximately 31 square miles. Approximately 9.5 square miles of this area is off-site drainage into Deer Prairie Slough and approximately 3 square miles is off-site drainage into High Hammock Canal. The 12.5 square miles of off-site area draining into the Deer Prairie Slough drainage system is the only significant off-site drainage into the RMR.

The fourth major drainage feature is Rustler Slough on the south-eastern corner of the site. This slough which drains approximately 3.6 square miles is the only area on the site which drains to the east into Cowpen Slough.

E.3 FIELD MONITORING NETWORK CHARACTERISTICS

The Dames & Moore field network DM-1 through DM-6, used to monitor the surface water characteristics of the RMR are shown on Figure E.3-1. The criteria used to locate these installations were as follows:

- Each site required accessibility during dry as well as wet seasons.
- Each site was located in an area of hydrologic interest and distinctiveness.
- Each site possessed the necessary exposure and physiographic characteristics for proper instrumentation.

E.3.1 Network Purpose

The measured hydrologic parameters include: rainfall, runoff, surficial ground water levels, soil moisture, evaporation, evapotranspiration, wind totals, water temperatures, and air temperatures. This data was used to quantify the site water balance and to develop a mass balance model for prediction of RMR surface water supply capabilities.

E.3.2 Network Components

The following is an outline of the equipment and parameters measured at each Dames & Moore surface water monitoring station.

E.3.2.1 DM-1

1. Leupold and Stevens Type A Model 71 Water Level Recorder
   - Deer Prairie Slough water levels at north powerline crossing
2. Six Leupold and Stevens Staff Gages - DM-1 (A-F)
   - Water levels along Deer Prairie Slough
3. All-Weather Rain Gage
   - Periodic (bi-weekly to weekly) rainfall totals

E.3.2.2 DM-2

1. MICROSCOUT Microprocessor
2. Sierra-Misco Tipping Bucket Rain Gage
   - Instantaneous rainfall measurements
3. All-Weather Rain Gage
   - Periodic (bi-weekly to weekly) rainfall totals
4. Belfort Portable Liquid Level Recorder
   - Measure water levels across concrete flume

5. Two Druck Pressure Transducers

   - Measure water levels across concrete flume
   - Measure surficial ground water levels in Well BH-2

6. Concrete V-Notch Flume

   - Flow control for discharge measurement

7. Surficial Well BH-2

   - Measure surficial ground water levels

8. Two Evapotranspirometers

   - Measure wetland evapotranspiration
   - Measure pine flatwood evapotranspiration

9. Soil Moisture Blocks

   - Measure soil moisture in the unsaturated soil zone

E.3.2.3 DM-3

1. MICROSCOUT Microprocessor

2. Sierra-Misco Tipping Bucket Rain Gage

   - Instantaneous rainfall measurements

3. All-Weather Rain Gage

   - Periodic (bi-weekly to weekly) rainfall totals

4. Belfort Portable Liquid Level Recorder

   - Measure water levels across Plasti-Fab trapezoidal flume

5. Druck Pressure Transducer

   - Measure water levels across Plasti-Fab trapezoidal flume

6. Shape Pressure Transducer

   - Measure surficial ground water levels in Well BH-1

7. Plasti-Fab Trapezoidal Flume

° Flow control for discharge measurement

8. Surficial Well BH-1

   ° Measure surficial ground water levels

9. Evapotranspirometer

   ° Measure dry prairie evapotranspiration

10. Soil Moisture Blocks

    ° Measure soil moisture in the unsaturated soil zone

11. Totalizing Anemometer

    ° Measure average wind speed

12. Humidity and Temperature Sensor

    ° Measure relative humidity

    ° Measure air temperature

E.3.2.4 DM-4

1. Belfort Continuously Recording Rain Gage

   ° Instantaneous rainfall measurements

2. All-Weather Rain Gage

   ° Periodic (bi-weekly to weekly) rainfall totals

3. ISCO 1870 Flow Meter

   ° Measure water levels across Plasti-Fab trapezoidal flume

4. Plasti-Fab Trapezoidal Flume

   ° Flow control for discharge measurement

E.3.2.5 DM-5

1. National Weather Service Class A Evaporation Pan

   ° Measure evaporation rates from a free water surface

2. Totalizing Anemometer

   - Measure average wind speed

3. All-Weather Rain Gage

   - Periodic (bi-weekly to weekly) rainfall totals

4. Submersible Maximum/Minimum Thermometer

   - Measure maximum, minimum, and present evaporation pan water temperatures

E.3.2.6 DM-6

1. Floating Fiberglass Lake Evaporation Pan

   - Measure evapotranspiration rates in Lower Myakka Lake

2. All-Weather Rain Gage

   - Periodic (bi-weekly or weekly) rainfall totals

E.3.3 Network Maintenance

All of the surface water monitoring stations were visited and maintained at a minimum of once per week. Maintenance procedures are documented in the RMR Surface Water Monitoring Manual (Dames & Moore, 1985). The manual stipulated data collection procedures, data transfer control, manifest systems, and maintenance procedures.

E.3.4 Network Performance

Data retrieved from the instruments at each Dames & Moore surface water monitoring station was critical to the successful quantification and understanding of the surface hydrology for the RMR. Consequently, a continuous record of data for each of the parameters studied is needed. In most cases a continuous record was retrieved from the monitoring stations. Occassional lightning strikes and other environmental features damaged some electronic equipment. Other less frequent problems affected the mechanical instruments. However, in nearly every case there were two levels of instrumentation measuring each parameter. This was invaluable in maintaining a continuous data record. Estimation procedures based on actual field data were later used to fill in gaps in the data record.

E.4 RAINFALL QUANTIFICATION

Rainfall is the input parameter for determining the available site water crop. It represents the input into the water budget relationship whether it be for the entire site, a small stream basin, or an evapotranspirometer. The following two sub-sections contain a description of the monitoring network and a discussion of the data analysis.

E.4.1 Rain Gage Network

One or more types of rain gages were located at each of Dames & Moore's surface water monitoring stations shown on Figure E.3-1. The gages themselves were postioned for proper exposure at each site. Most gages are placed on wooden stands 10 to 12 feet high. A list of the types of rain gages used at each monitoring station is included in Section E.3 of this appendix.

Three types of rain gages were utilized: two Sierra Misco tipping bucket rain gages, one Belfort continuously recording rain gage, and six All-Weather rain gages. The Sierra Misco rain gages work on the tipping bucket principle and each bucket tip is recorded electronically on a MICROSCOUT microprocessor. Bucket tips are summed together for the entire sampling period of 1 hour, thus recording the time and amount of precipitation. Data is later retrieved from the microprocessor.

The Belfort rain gage is a weighting gage and converts weight into inches of rainfall, in addition to denoting the time of the rainfall event. This information is recorded on a paper chart which is later interpreted. The All-Weather rain gages measure rainfall totals of one or more events over the inspection interval. The timing of the events is not known, only the total rainfall. These All-Weather rain gages were the primary gages at DM-1, DM-5, and DM-6. At the remaining stations, they were utilized to provide a check on the Sierra-Misco or Belfort rain gages.

The rainfall totals found for the All-Weather and Belfort rain gages for the same time period were in agreement. However, the rainfall totals for the All-Weather and Sierra-Misco rainfall gages were inconsistent. As a result, rainfall data reported for DM-2 and DM-3 are taken from the All-Weather rainfall gages, and represent the totals of one or more rainfall events. Corrective action for improving the accuracy of the tipping bucket gages has to date been insufficient to allow dependence on the rainfall data. Prior to issuance of this report a major factory, reprogramming of the equipment was conducted. The results of that effort have not been assessed to date.

E.4.2 Rainfall Data

Rainfall data collected from the RMR appears in two forms. It appears as monthly totals in Table E.4-1 for each station. Secondly, it appears in the form of monthly hyetographs for each station (DM-1 - DM-5) on Figures E.4-1 through E.4-28 and Figure E.5-4. The hyetographs for stations DM-1, DM-2, DM-3, and DM-5 are periodic totals of multiple rainfall events and not individual events. The totals are shown when they were determined. Hyetographs for DM-4 are actual hourly totals.

The data shows a typical distribution of rainfall corresponding to the wet and dry seasons of southwest Florida. Significant rainfall amounts occurred during the months of June to September. Although much of this rainfall occurred as isolated thunderstorms over different portions of the 51-square mile site, the monthly rainfall totals for individual stations are in general agreement.

An assessment of long-term rainfall patterns is provided in the mass balance discussion in Section E.7 and Appendix F.

E.5 EVAPOTRANSPIRATION QUANTIFICATION

E.5.1 Introduction

Evapotranspiration (ET) is the most critical parameter other than rainfall governing the water supply potential of the RMR. Unfortunately, unlike rainfall, there is no site specific long-term ET data and the parameter is difficult to quantify (Shih, 1981). Due to the problems inherent in the assessment of ET, a discussion of the background literature on ET is provided as a preface to the description of Dames & Moore's quantification efforts.

E.5.1.1 Terminology

As stated by Tanner (1968), the term evaporation applies primarily to the transport of water vapor from the source of vaporization to the atmosphere. However, within the scope of data acquisition and water management, the physical implications associated with the term can differ.

The term evaporation possesses some ambiguity due to its lack of source specificity. This ambiguity has lead to some confusion in terminology. For the purpose of this report, the following definitions shall be observed. The term evaporation itself will be limited to the vaporization of water from an open water surface directly exposed to the atmosphere, while transpiration is used to designate the vaporization of water from plant surfaces. Where the vaporization and exchange of water from a surface composed of vegetative cover and bare soil is described, the collective term evapotranspiration shall be used.

Two basic forms of evapotranspiration are cited in the literature. The first term, "potential evapotranspiration" is most commonly defined as, "the amount of water transpired per unit of time for optimum water supply by a short, green plant stand which has uniform height and which completely covers the ground" (Penman, 1956). The study of potential evapotranspiration is an attempt to single out the climatic factors influencing evapotranspiration. By providing a well-watered actively growing plant surface, physiographic and biological affects remain less complex. The scientific community is undecided on a reference plant surface and any values for potential evapotranspiration must reference the growing surface. In addition, according to the National Handbook of Recommended Methods for Water Data Acquisition (1982) "Potential evapotranspiration is defined as the rate of water loss from a wet soil or well-watered, actively growing vegetation, or as the rate of evaporation from a water surface." Consequently, potential evapotranspiration is a meteorological quantity only applicable for the studied conditions.

On the other hand, "actual evapotranspiration," will depend on weather, soil, and plant factors. Evapotranspiration, therefore, cannot be determined from weather elements only (De Bruin, 1981). This parameter of actual evapotranspiration combines meteorological variables with physiographic variables such as soil conditions, plant water requirements, vegetation spatial coverage, soil characteristics, and water availability. Thus, it is a more accurate evaluation of consumptive use or evapotranspiration. Unfortunately, effective determination of this value requires tremendous effort to understand and quantify the complex interaction of the various components of the evapotranspiration process.

E.5.1.2 Evapotranspiration Study Purpose

A knowledge of evapotranspiration is essential in planning and implementing water management systems. In the case of the RMR, evapotranspiration has a dual impact. First, evapotranspiration removes water that may be incorporated into the water crop. Secondly, it represents the water required to maintain the soil-water balance. Poor management of this soil-water balance could result in detrimental alterations in the landscape. As a result, the quantification and understanding of water-shed consumptive use is important to the evaluation of an available water crop.

E.5.2 Literature Review of Available Methods

The literature revealed four categories of methods for quantifying evapotranspiration. These are:

- Water balance methods
- Energy balance
- Mass Transfer
- Prediction Methods

Within each category are varying techniques, each with its limitations, advantages, and disadvantages. A description of the four categories is provided in Sections E.5.2.1 - E.5.2.4.

E.5.2.1 Water Balance Methods

Evapotranspirometers/Lysimeters

If properly constructed, located, and operated, evapotranspirometers can provide the most accurate information on actual or potential evapotranspiration and are the only means to calibrate other methods of measuring or estimating evapotranspiration (Gangopadhyaya and others, 1966; Harrold, 1966; Tanner, 1967; Blad and Rosenberg, 1975).

According to the National Handbook of Recommended Methods for Water Data Acquisition (1982), an evapotranspirometer is an instrument consisting of a block of soil, usually planted with some vegetation, and enclosed in a container which isolates it hydrologically from its surroundings. If there is provision for drainage of the soil water, one speaks of lysimeters (literally "leach meters"). In addition, the distinction should be made as to whether the evapotranspirometer contains a disturbed or undisturbed soil profile.

Weighable Lysimeters

These instruments possess the basic characteristics of all lysimeters, however as the term implies, the entire apparatus is weighed.

By continuously weighing the soil column within the lysimeter, an accurate evaluation of the water balance can be determined. Water fluctuations will be recorded as changes in weight from which losses due to evapotranspiration may be determined. Weighable lysimeters can be used to follow daily, hourly, or even more frequent changes in evapotranspiration. However, as attractive as these instruments may appear, they are very expensive and difficult to install and maintain.

Nonweighable Lysimeters

These instruments make use of lysimeter principles but do not use changes in weight to determine moisture changes. Moisture fluctuations are determined by monitoring the water table in the lysimeter and soil moisture changes in the unsaturated zone. Due to moisture hysteresis effects and monitoring limitations they are not suited for short-term measurements.

According to the National Handbook of Recommended Methods for Water Data Acquisition (1982), instruments without drainage outlets should be used with a degree of caution. This is particularly important when the water table is maintained at a constant level. This scenario results in increased soil water salinity (Williamson, 1963; van Hylckama, 1966; Robinson, 1970). In addition, installations without drainage suffer from atmospheric affects resulting in water-level fluctuations which obscure the true losses due to evapotranspiration (Stevenson and van Schaik, 1967; van Hylckama, 1968). Apart from these shortcomings, nonweighable lysimeters are easier to install and maintain than weighable lysimeters, and combined with their low cost, they are an attractive alternative.

Large-Area Water Budget Method

Evapotranspiration estimates can be made for a large-area using the hydrologic-budget method, also referred to as the water budget method. The hydrologic components may include precipitation, surface runoff, ground water movement, surface and subsurface storage. The remainder of this algebraic relationship for a given period is an estimate of evapotranspiration. Estimating evapotranspiration from natural watersheds involves measuring the inflow as precipitation and the outflow as stream flow for a number of years in order to determine the average annual inflow and outflow rates. The difference between these two rates is an estimate of evapotranspiration (Hewlett and others, 1969).

To use the large-area water budget method for this project would require years of site specific historical data on the site or a project study period of several years. Since neither of these requirements can be met, this method is not a plausible alternative. In addition, although such parameters as precipitation and stream flow may be easily determined, values for ground water flows and surface storage are much more difficult to obtain. The permeable surficial sands and limestone aquifers underlying the RMR transcend surficial drainage divides and have the potetial for transmit large quantities of water underground. These waters may reappear downstream, but the areas or points of recharge of all the underground water cannot be determined, nor can all the underground flow be accounted for with certainty. These unknown hydrologic characteristics prohibit the use of the large-area water budget relationship.

<u>Soil-Moisture Depletion Method</u>

According to the National Handbook of Recommended Methods for Water Data Acquisition (1982), probably the oldest and most commonly used method of determining evapotranspiration is measurement of the change in soil-moisture content at representative sites over a period of a few days to several weeks. Soil-moisture depletion approximates evapotranspiration under ideal conditions, where rainfall is estimated from gage data, the water table is considerably below the root zone, and there is no significant drainage from the root zone. Attempts to measure the flux of soil water upward or downward below the root zone at representative sites using flux meters have not been successful. Similarly, attempts to estimate the upward or downward movement of soil water by measuring the hydraulic gradient and calculating the hydraulic conductivity have not been very successful. Also, soil moisture may move upward or downward due to the thermal and salt gradients which can introduce significant error.

The inherent physical problems associated with this method, the shallow water tables and the periodic interaquifer flux (Appendix C) made this an unacceptable approach for estimating evapotranspiration on the RMR.

E.5.2.2 Energy Balance Method

The energy balance method of determining evapotranspiration is based on the conservation of heat energy. This method requires complex instrumentation and is often plagued by maintenance problems while voluminous records of data must be generated and processed. Only a few studies have been conducted continuously for more than several days. The requirements and limitations of this method were unsuitable for the RMR project.

E.5.2.3 Mass Transfer Methods

Mass transfer methods utilize the vertical vapor flux to determine consumptive use. Like the energy balance method these methods require complex instrumentation and other needs that are not commensurate with the level of accuracy required for this study.

E.5.2.4 Prediction Methods

Empirical Equations

Numerous empirical equations have been developed for estimating potential evapotranspiration. These equations utilize air temperature, solar radiation, humidity, and wind speed. Also included in these empirical equations is a crop coefficient commonly referred to as K.

This coefficient is experimentally determined, thereby calibrating the empirical relation. Empirical equations often make use of available climatological data, but the experimentally determined crop coefficient and the site-specific environment used to develop the relation severely limit the use of the equation in other areas.

Pan Evaporation

Pan evaporation data can provide very reliable estimates of potential evapotranspiration. However, pan evaporation appears to be more sensitive to wind conditions than well-watered short grass, especially when radiation levels are lower (National Handbook of Recommended Methods for Water Data Acquisition, 1982). Pan evaporation values have coefficients applied to them in order to predict future losses due to actual evapotranspiration. These coefficients are a means of calibrating pan evaporation data to model losses due to seasonal evapotranspiration. These coefficients must be derived from actual evapotranspiration information in order for their application to be meaningful.

E.5.3 RMR Evapotranspirometer Approach

The evapotranspirometer was chosen for monitoring and determining evapotranspiration based on the previous discussion of available methods. The criteria for this decision were as follows:

- credibility from the scientific community
- practical construction and installation requirements
- instrumentation
- maintenance
- cost

The ultimate goal of this investigation is development of empirical evapotranspiration relationships for RMR individual macroscale vegetative communities based on seasonal evapotranspirometer and National Weather Service Pan Evaporation data.

E.5.3.1 Evapotranspirometer Site Selection Strategy

Evapotranspirometers were located in the three most distinct and characteristic vegetative habitats found on the RMR:

- Pine Flatwood
- Wetland
- Dry Prairie

The purpose of locating an installation in each of these habitats was to incorporate into the analyses the affects of varying exposures, vegetation, and soil properties.

E.5.3.2 Evapotranspirometer Construction

The three evapotranspirometers are identical in construction. Each cylindrical tank is 6 feet in diameter and 7 feet tall. The tanks are sealed at the base and were formed from fiberglass by a local fiberglass fabricator.

E.5.3.3 Evapotranspirometer Installation

The pine flatwood and dry prairie installations were installed using identical procedures (see Figures E.5-1 and E.5-2). First, a convenient and representative location was chosen and verified with the project environmental consultants. The soil was removed by small backhoe in 1- to 3-foot layers closely corresponding to visible changes (e.g., color, texture) in the soil profile. The soils were stock piled separately, and then covered by filter fabric to prevent drying and any degredation of biota to the degree possible. Prior to the removal of each layer, the in situ permeability, wet and dry densities, and water content were determined (see Table E.5-1). The base of each evapotranspirometer is located 5 to 6 feet below the natural ground surface.

Once the tank was in place, a 4-inch PVC well was installed inside the tank to monitor future water table fluctuations within the tank. An additional monitoring well was installed outside of the tank to determine natural ground water fluctuations. Next a 1-foot layer of gravel was spread around the base of each well and covered with filter fabric. This gravel layer should prevent clogging and enhance seepage from the soil column to the well so that water level fluctuation response times may be shortened. In addition, because the soil layer adjoining the gravel layer contained significant amounts of clay with low permeability, gravel columns were placed into this layer to provide better hydraulic flow between the gravel layer and the remaining upper soil horizons.

As each soil horizon was replaced to its original depth, it was again tested for permeability, wet and dry densities, and water content. These values were compared with earlier in situ measurements (see Table E.5-1). Soil moisture blocks were also installed within each soil horizon in order to monitor moisture fluctuations in the unsaturated zone. After the soil column was completed, vegetation native to the surrounding habitat was transplanted both inside and outside of the installation in order to homogenize the environment adjacent to the evapotranspirometer.

The procedure used to install the wetland evapotranspirometer differed slightly from the previous installations (see Figure E.5-3). Due to the poor foundation conditions encountered within the wetland, operation of the backhoe was limited and hand installation was required. Installation difficulties combined with problems in testing the in situ soil column led to a decision not to continue testing individual layers of the soil column. It was also found during the excavation, that the soil column was homogeneous to a depth of 6 feet and extensive testing was not necessary. In addition, due to the poorly drained nature of the wetland soils, moisture blocks and an external monitoring well were omitted from the installation.

E.5.3.4 Evapotranspirometer Principles

The principle of evapotranspirometer operation is as follows: a representative soil column with actively growing vegetation is isolated from the surrounding soil. All fluxes of water entering or leaving the system are measured except for the evapotranspiration flux. Inputs to the system are precipitation and water added by the field technician to maintain certain water table levels. Outputs are evapotranspiration and water removed by the field technician (Buell and Ballard, 1972). The change in water storage is determined by measuring the changes in the soil column water table level and multiplying this change by the specific yield of the system. The specific yield of the system ( ) is defined as the amount of water that must be added or removed to cause the water table in the evapotranspirometer to change 1 foot.

Changes in water storage occur in the soil both above and below the water table (Buell and Ballard, 1972). Fluctuations below the water table can be monitored by a well within the soil column. Moisture storage measurement in the unsaturated zone is more problematic. Soil moisture blocks were installed to varying depths within the soil column to measure soil moisture. In the study done by Buell and Ballard (1972) water storage in the unsaturated zone was not measured and assumed to be zero. According to the Buell study, the long-term mean of the water storage in the unsaturated zone is a term that is small compared to the total evapotranspiration of the evapotranspirometer. Although ignoring this storage may reduce the accuracy of weekly evapotranspiration estimates, the error in estimating monthly evapotranspiration will be small. The water budget equation for the evapotranspirometer becomes:

$$\text{Input} - \text{Output} = \Delta\ \text{Storage} \qquad \text{Eq. E.5-1}$$

$$(\text{Precipitation} + \text{Additions}) - (\text{Evapotranspiration} + \text{Removals}) = \rho(\Delta\ \text{Water Table}) \qquad \text{Eq. E.5-2}$$

$$\text{Evapotranspiration} = \text{Precipitation} + \text{Additions} - \text{Removals} - \rho(\Delta\ \text{Water Table}) \qquad \text{Eq. E.5-3}$$

The advantage of the evapotranspirometer approach is that hydrologic parameters such as runoff, ground water seepage, and ground water interflow are eliminated from the water budget of the installation. By isolating the soil column from the surrounding soil, ground water fluxes have been eliminated. In addition, the evapotranspirometers used on the RMR extend approximately 1 foot above the natural ground surface and capture any water that would normally run off this surface. Consequently, the evapotranspirometer design prevents runoff and ground water fluxes that would normally be difficult to measure.

In general, within this isolated environment of the evapotranspirometer all water fluxes can be determined except for the evapotranspiration flux. This flux is computed by solving the algebraic expression relating all of the fluxes.

E.5.3.5 Evapotranspirometer Monitoring

Monitoring of the evapotranspirometers began in July 1985, and has been ongoing at weekly and occasionally daily intervals along with other surface water investigation activities. Data necessary to the calculation of evapotranspiration from these devices includes inches of rainfall, and depth to the evapotranspirometer water table. The rainfall represents an input while the water table level, when compared to the previously recorded level, represents the change in soil water storage.

Water table levels within the pine flatwoods and dry prairie installations were allowed to fluctuate undisturbed unless ponded water was present. When water ponded on the surface, it was bailed from a perforated sump bucket. This outflow was measured, recorded, and entered into the calculation of evapotranspiration. This procedure was followed except when the natural water table surrounding a monitor exceeded the ground surface. In such conditions the water table in the evapotranspirometer was allowed to break the surface in an attempt to mimic natural conditions. As the natural water table subsided, water was bailed from the evapotranspirometer. Above ground surface water levels were periodically experienced at the pine flatwod monitor but not at the dry prairie installation.

In contrast to the two monitors discussed above, the wetland evapotranspirometer is located within an area of ponded water. As a result, the monitor water level was kept to a depth nearly equivalent to natural wetland pool depth, provided this depth did not exceed the volume limitations of the evapotranspirometer. Water was removed or added as needed and recorded for use in calculating evapotranspiration.

E.5.3.6 Evapotranspirometer Installation Deficiencies

Several problems and concerns have resulted from this evapotranspirometer approach and consequent evapotranspiration investigation.

Drainage

Literature cited earlier in this section discouraged the use of evapotranspirometers without drainage. Drainage from the tank was considered during the inception of the evapotranspirometer design. However, it was decided not to provide drainage for several reasons. First, the shallow water table found on the site, combined with low topographic relief reduces the natural subsurface drainage. This led to the conclusion that drainage from the evapotranspirometer may not be significant enough to warrant measurement. Secondly, if drainage from the tank was permitted, it would be necessary to determine the quantities leaving the system in order to limit any confounding effects this variable may induce into the water budget computation. During the design of the installations, it was decided that to provide accurate drainage monitoring complicated the design and required an effort not commensurate with the intended use of the evapotranspirometers.

The problems of increased salinity and barometeric effects are currently being investigated, and mitigative measures devised. Possible measures include estimating barometric affects on water level fluctuations and making allowances for such effects in the evapotranspiration calculation. In addition, the tank design provides a method for flushing or purging the tank of excess salinity. Once the affects are better evaluated and understood, procedures can be implemented to mitigate their influence.

Disturbed Soil Column

Each evapotranspirometer was backfilled with a representative but disturbed soil column. This disturbed soil column is suspected of adversely affecting the evapotranspiration process within the evapotranspirometer. The other alternative would have been to excavate around a monolithic soil column and isolate it hydraulically from the surrounding soils. The level of effort for such an installation was not commensurate with the possible benefits that may be derived. Table E.5-1 displays the in situ and replacement soil characteristics. In most cases, the replacement parameters approximate the in situ values, and it is expected that the disturbed soil column will have little affect on evapotranspiration modeling.

Soil Moisture Storage in the Unsaturated Zone

Soil moisture blocks were installed to evaluate the soil moisture in the unsaturated zone above the water table within the evapotranspirometer soil column. The data derived from these devices was to be utilized in the calculation of the global water budget for the evapotranspirometer. Unfortunately, the actual data derived from the soil moisture blocks has been inconclusive.

Reasons for the ineffectiveness of the soil moisture blocks is two-fold. First, during the course of the field study the gypsum media blocks surrounding the electronic sensors gradually dissolved. The slightly acidic pH found in the surficial ground water is likely responsible for this dissolution of the gypsum. This phenomenon is the probable explanation for the variability of the measured resistances. In addition, lightning strikes and the associated voltage surge induced into the ground water may have damaged these devices. Although the soil moisture fluctuations in the unsaturated zone have not been determined, the analytical methods used to calculate evapotranspiration reduced the importance of this parameter. (See Section E.5.6 for this analytical procedure.)

Vegetative Influences

According to H. Riekerk (1982), vegetation cover and its stage of development are major determinants of water fluxes in the hydrologic cycle. This statement addresses two more areas of uncertainty: vegetation density surrounding the evapotranspirometer and vegetation development. Both issues were identified early in the fall of 1985 as major concerns.

Impacts to the vegetation in the immediate area of the evapotranspirometers were the result of construction activities, and even these areas are recovering. Further, plant species that were planted within each of the evapotranspirometers, except the wetland installation, did not grow with the same vigor as the surrounding vegetation. In addition, a fire at DM-2 further degraded the conditions of transplanted vegetation. These situations undoubtedly affected initial evapotranspiration estimates from the installations. As conditions improve, however, so will plant vigor.

Recommendations

Such growing pains are commonly experienced with these installations (Fritschen, Personal Communication, 1972) and the following are recommendations to the deficiencies described. Inaccuracies induced by the drainage conditions will be mitigated by periodic purging of the tanks and investigation of the impact of barometric pressure on study results. Soil tensiometers will be installed in the unsaturated zone and monitored weekly. Additional vegetation will be transplanted into and around the monitors.

E.5.4 National Weather Service Class A Pan Evaporation

Measurement of evaporation from pans is considered one of the easiest and most accurate ways of estimating evaporation from a "free water surface." The National Weather Service Class A Pan has been the standard in this country for many years (National Handbook of Recommended Methods for Water Data Acquisition, 1982).

E.5.4.1 Class A Pan Principles

Evaporation estimates obtained from the Class A pan represent evaporation occurring from a thin free water surface having no heat storage. In order to make these estimates meaningful to an actual lake or reservoir, they must be adjusted by a pan coefficient. Pan coefficients for the Class A pan have been found to average about 0.7 on an annual basis, but will generally vary from 0.8 to 0.6. In some climates or during certain seasons, much larger variations can be expected (National Handbook of Recommended Methods for Water Data Acquisition, 1982).

E.5.4.2 Class A Pan Monitoring

The Class A pan installed on the RMR was monitored on a weekly and sometimes daily schedule. Data obtained from the installation includes total rainfall, water surface levels, maximum, minimum, and present water temperatures, and wind totals. A summary of the results from this data is shown in Figure E.5-4.

E.5.4.3 Class A Pan Data Application

The data collected from the Class A pan has a number of intended applications. The most obvious application involves the estimation of evaporation losses from a reservoir. Secondly, comparing data from the three evapotranspirometers with the pan evaporation data helped develop pan coefficients for evapotranspiration. These values were used in the water balance for the entire RMR.

E.5.5 Lower Myakka Lake Pan Evapotranspiration

E.5.5.1 Lower Myakka Lake Pan Principles

Although the National Weather Service Class A Evaporation Pan provides an approach to estimating losses from a lake or reservoir, it does not model the possible consumptive use of floating vegetation. Floating aquatic weeds such as hyacinths and duckweeds are common in natural water bodies such as Lower Myakka Lake. To quantify this variable, an evapotranspirometer filled with lake water and native vegetation was designed to float on the lake. In addition, by locating the evapotranspirometer on the lake, climatic and physiographic variables effecting evapotranspiration may be more accurately modeled. Data from this Dames & Moore surface water monitoring station, DM-6, should better represent the evapotranspiration experienced by a lake or reservoir.

A water budget relationship, similar to other evaporation/evapotranspiration installations, was used to determine evapotranspiration losses (see Section E.5-6).

E.5.5.2 Lower Myakka Lake Pan Monitoring

The Lower Myakka Lake Evapotranspiration Pan at DM-6 was monitored on a weekly basis. Rainfall amounts were determined from an All-Weather rain gage and represented the input to the system. Periodically water level and vegetation coverage had to be manually adjusted. These inputs and outputs were recorded and factored into the water budget relationship.

The floating nature of the pan made water level determination difficult. In order to solve the problem of an uneven and unstable water surface, four staff gages were installed with 90 degree spacings on the perimeter of the pan. The average of these staff gage values represented the water surface level, and fluctuations in this level represented the change in storage.

Periodic high lake levels also created monitoring difficulties. Problems were three-fold. First, high lake levels limited station access. Second, periodic high lake levels inundated the rain gage and confounded subsequent measurements. Third, high lake levels corresponded with high pan levels during increased rainfall periods. Although the pan floated on the lake, it was found that high water levels in the pan resulted in water spillage. As a result, a water budget was periodically unobtainable. Data obtained during these periods was omitted from study.

E.5.5.3 Lower Myakka Lake Pan Data

A summary of the average monthly evapotranspiration rates calculated for the Lower Myakka Lake Evapotranspiration Pan are shown in Table E.5-2. These rates are compared with the monthly average evaporation rates obtained from the National Weather Service Class A Evaporation Pan. The table shows that Class A pan evaporation is slightly greater than lake evapotranspiration in all months but August and September. However, during these 2 months the station was difficult to maintain and the data obtained during these periods is questionable. In general, the average monthly lake evapotranspiration rates followed a similar trend as the Class A pan evaporation rates. The absolute values approximated 70 to 81 percent of Class A pan evaporation.

E.5.6 Evapotranspiration Calculation

Different calculation procedures were utilized for the upland and wetland/lake evapotranspirometers. The procedures are discussed separately in the following subsections.

Upland Evapotrometers

A unique method was used to calculate the evapotranspiration losses from the dry prairie and pine flatwoods evapotranspirometers. This method was developed to compensate for the lack of information regarding moisture changes in the unsaturated zone.

The method involved scrutinizing the field data and selecting a period of record when the evapotranspirometer ground water began at an initial level, fluctuated, and returned to the initial level. These periods range from a few days to 30 days. By using this approach, any error in estimating the change in soil water storage would be minimized (see Figure E.5-5). Although moisture losses would still continue from the unsaturated zone, the magnitude of these undetermined losses would be small compared to the total evapotranspiration losses experienced by the system.

As described earlier, the governing equation for dry prairie and pine flatwood lysimeters is as follows:

$$\text{Input} - \text{Output} = \Delta\ \text{Storage} \qquad \text{Eq. E.5-1}$$

$$(\text{Precipitation} + \text{Additions}) - (\text{Evapotranspiration} + \text{Removals}) = {}_{\rho}(\text{Water Level Change}) \qquad \text{Eq. E.5-2}$$

Consequently, by calculating the water budget for a period when initial and final evapotranspirometer ground water levels were nearly equivalent, the term (water level change) is nearly zero and has very little influence on the water budget calculation. The governing equation thus reduces to:

$$(\text{Precipitation} + \text{Additions}) - (\text{Evapotranspiration} + \text{Removals}) \cong 0 \qquad \text{Eq. E.5-3}$$

$$\text{Evapotranspiration} \cong \text{Precipitation} + \text{Additions} - \text{Removals} \qquad \text{Eq. E.5-4}$$

The parameters in this final expression can be easily determined.

Wetland/Lake Evapotranspirometers

The governing expressions for the wetland and Lower Myakka Lake pan evapotranspirometers make use of the same water budget principles with some minor refinements. The refinements involve calculating the change in storage for the system. Unlike the dry prairie and pine flatwood evapotranspirometers, changes in storage for these installations involved a phreatic surface without any overburden soils. An attempt was made to calculate a water budget for the wetland evapotranspirometer when water levels fell below the soil surface, but unlike the dry prairie and pine flatwood evapotranspirometers, the unlike the dry prairie and pine flatwood evapotranspirometers, the poorly drained wetland soils prevented representative water level fluctuations. Consequently, the data presented for the wetland evapotranspirometer are for periods when the water surface was above the soil column. Changes in storage for the wetland and Lower Myakka Lake evapotranspirometers were calculated based on the difference between successive water levels.

Because the free water surface of the wetland and Lower Myakka Lake evapotranspirometers was void of any overburden soils, there was no need to multiply water level changes by a specific yield. The resulting equation reduces to:

Evapotranspiration = Precipitation - (water level (i) - water level (i-1)) Eq. E.5-5

The term "water level (i)" represents the unadjusted level immediately observed when monitoring the station. The term "water level (i-1)" is the final adjusted level from the preceding site visit. Water levels had to be adjusted periodically to prevent overflowing or desiccation. This water budget calculation is identical to the procedures used to determine National Weather Service Class A Pan Evaporation.

E.5.7 Evapotranspiration Results

The results contained in this section represent the work done by the Dames & Moore study of the RMR and those obtained by similar evapotranspiration studies in the central Florida region.

E.5.7.1 Wetland Evapotranspiration

There is disagreement within the scientific community as to the influence of wetland vegetation on evapotranspiration from wetlands. Linacre (1976) discussed this problem fully and concluded that wetland vegetation reduces evaporation in comparison with that from lakes when the surrounding country is dry; when it is wet, however, the evaporation from wetlands is approximately similar to that from lakes.

On the other hand, several authors have reported that the actual evapotranspiration from wetlands exceeds that from open water. Brown (1981) found that a National Weather Service Class A Pan coefficient of 0.95 had to be applied to standard pan data in order to approximate consumptive use requirements of a Florida floodplain forest. In addition, the high pan ratio for the floodplain forest was comparable to pan ratios for a variety of Florida marshes where transpiration was equal to or greater than pan evaporation during the summer months.

The data derived from the Dames & Moore study reflects the discoveries of Brown (1981). Wetland evapotranspiration data and corresponding Class A pan data are summarized in Table E.5-4. In nearly all of the corresponding months studied, wetland evapotranspiration exceeds typical lake evaporation rates

E.5.7.2 Dry Prairie Evapotranspiration

Data derived from the dry prairie evapotranspirometer at Dames & Moore surface water monitoring station DM-3 is summarized in Table E.5-3 and Figure E.5-6. Table E.5-3 compares monthly dry prairie evapotranspiration rates with monthly Class A pan evaporation rates for the site. The comparison shows that dry prairie evapotranspiration rates approximate 50 to 70 percent of pan evaporation. However, Figure E.5-6 reveals a stronger relationship between Class A pan evaporation coefficients and the depth to the soil column water table. The graph shows that dry prairie evapotranspiration may be approximated by a linear function of water table depth in terms of percent of pan evaporation.

Measured data was only available to a depth of 2.5 feet. An estimate of the of ET is unavailable for greater depths. However, several assumptions are possible. First, capillary rise may be as much as 6.5 feet (Cohman, 1972) in the silty material characterizing the B Horizon (2 to 5 feet in depth) of the RMR. Second, the supply of water to the root zone will remain roughly constant for a water table depth from the root zone to the depth of capillary rise. Third, the zone of "dense" root concentration extends on an average from 3 to 5 feet below the ground surface.

Based on the above assumptions, ET may be approximated as being roughly constant for a depth of water table below ground surface of from three to 11.5 feet. This approximation allowed the employment of a constant of 25 percent of pan evaporation for water table depths below 3 feet. The linear expression for the applied pan coefficient is:

$$y = 76.0 - 18.72\ X \qquad \text{Eq. E.5-6}$$

where:

y = evapotranspiration as percent of pan evaporation
X = water table depth.

These results are consistent with literature values that indicate bare soil evapotranspiration exceeds surface water evaporation (Thornthwaite, 1939).

As has been noted vegetation cover in the dry prairie evapotranspirometer may not be representative of macro-scale site conditions. During initial operation, a significant portion of the evapotranspiration was likely from bare soil. This data must, therefore, be used only as a guide.

E.5.7.3 Pine Flatwood Evapotranspiration

Data obtained from the pine flatwood evapotranspirometer at DM-2 is inconclusive. After completing the water budget calculations and plotting the data following the procedures used for the dry prairie evapotranspirometer, no conclusive trends emerged as were seen at other installations.

Two sources of difficulty were identified that may have confounded the data. First, the water table exceeded the surface periodicaly in conjunction natural ponding conditions. Ponded water produces simultaneously two mechanisms. Free surface water losses occur simultaneous with subsurface evapotranspireation. Account of both above and below ground change in storage is an intractable problem. As a result, the water-budget for the evapotranspirometer is inaccurate.

Secondly, an intense fire burned through the DM-2 station and the area immediately surrounding the pine flatwood evapotranspirometer. Although the fire did not damage the installation and only singed the vegetation within the evapotranspirometer; it is likely that the influence of the heat from the fire and the subsequent damage to the canopy adjacent to the pine flatwood evapotranspirometer affected the data.

Data from this evapotranspirometer is still being collected and the analysis continues. The affects from the fire have diminished and water will no longer be allowed to pond on the surface. These actions should make the results from this installation more conclusive.

E.5.7.4 Lower Myakka Lake Evapotranspiration

As stated in Section E.5.5.3, the average monthly lake evapotranspiration rates approximate 70 to 81 percent of Class A pan evaporation rates. These results are neither confirmed or disputed by the available literature. According to DeBusk and Ryther (1982), the actual extent to which water hyacinths increase water loss is not known, and is largely determined by the interaction of meteorological parameters (e.g., maximum temperature and wind velocity) with the physical structure of the plant canopy. In addition, Brown (1981) states that there appeared to be no difference between the evaporation from the duckweed surface and the open water surface. Results from the water losses experienced by the Lower Myakka Lake evapotranspirometer generally show greater evapotranspiration rates than that of an open water surface. The magnitude of the effects of floating aquatic weed is yet to be conclusively quantified.

E.6 RUNOFF QUANTIFICATION

E.6.1 Flume Network

Four stream locations were chosen in order to quantify the runoff characteristics of the RMR. Flumes were installed at Dames & Moore surface water monitoring stations DM-2, DM-3, and DM-4 to provide a flow control for measuring stream discharge. In addition, staff gages were installed along Deer Prairie Slough.

E.6.2 Discharge Data

In general, runoff from each of the four gaged streams did not occur until August. This was attributed to the drought conditions and depressed ground water levels throughout the site. Stream flow ended at DM-3 and DM-4 in October while water was still present at DM-1 and DM-2, but flows were not discernible.

E.6.2.1 Deer Prairie Slough Discharge/Water Level Data

A continuous water level recorder was installed at the north power line crossing of Deer Prairie Slough in the northeastern part of the site. In addition, six staff gages were installed to monitor the slough water levels at weekly intervals. The goal of these measurements was to determine the discharge parameters for the slough. However, due to the drought conditions experienced during this study, limited measurable flows occurred at the north or south power line crossing. As a result, discharge relationships could not be established.

Rainfall data and Deer Prairie Slough water surface elevation data are shown on Figures E.4-1 through E.4-5 and E.6-1. As drought conditions diminish, more useful data should be obtained from these installations.

E.6.2.2 DM-2 Discharge Data

A concrete flume was installed in Windy Sawgrass Trail to measure the runoff from a 2.28 square mile basin. The data from this station could not be used to quantify the runoff from this area. This resulted from the extremely low flows that emerged from this basin. Two sources were cited that could have affected the stream flow in this area. First, after the flume was installed, an earthen weir was placed upstream of the flume at the request of the Florida Department of Environmental Regulation in order to restore the natural hydroperiod of the upstream wetland system. This weir undoubtedly restrained much of the flow that was estimated to cross the flume. Secondly, a control section further downstream, beyond the stream length surveyed, may have resulted in back water affects at the flume. These back water affects would limit the upstream flows at the flume. Finally, these affects may have combined to limit the flows across the flume, causing the stream to behave more like a pool.

Although discharge data could not be determined from this station, water levels in the stream were continuously measured (see Figure E.4-6 through E.4-12). In addition, ground water level fluctuations were determined in a nearby surficial well, BH-2, and plotted adjacent to stream water levels (see Figure E.4-6 through E.4-12). This plot reveals that water did not appear in the stream until the ground water table encroached upon the elevation of the stream invert. Further analysis of the stream and ground water levels show little response between stream fluctuations and ground water fluctuations except when the ground water surface falls well beneath the elevation of the stream invert.

E.6.2.3 DM-3 Discharge Data

A trapezoidal flume was installed on Dames & Moore Slough which drains a 1.58 square mile basin. Unlike DM-2, continuous discharge data was produced by this station. The data is presented in Figures E.4-13 to E.4-19 as runoff hydrographs. Also plotted adjacent to the runoff hydrograph is the ground water fluctuations in the surficial well, BH-1, located at the surface water monitoring station. Like DM-2, stream flows did not occur until ground water levels encroached upon the elevation of the stream invert. In addition, stream flows ceased when the ground water levels dropped well beneath the stream invert.

The total monthly runoff for the basin during the months of August, September, and October were 0.39, 2.23, and 0.03 mgd, respectively. These also correspond to depths of 0.030, 0.200, and 0.003 inches of water spread over the entire basin. This is contrasted by the 7.95, 9.77, and 1.26 inches of rainfall that fell over this basin during the study.

E.6.2.4 DM-4 Discharge Data

Continuous discharge data was also obtained at the trapezoidal flume located on Blackburn Slough which drains a 3.88 square mile basin. This data appears as runoff hydrographs on Figures E.4-20 through E.4-28. Surficial well data from the ROMP19 west well, plotted adjacent to the hydrographs, shows ground water fluctuations north of the flume site. Although this well is some distance from the flume, it was the nearest well with continuous data. Furthermore, close examination of the rainfall events measured at DM-4 and the ground water fluctuations at the ROMP19 west well indicate a positive correlation between rainfall event and water table rise.

The discharge and ground water elevation data from the DM-4 figures also show a strong correlation between the occurrence of runoff and ground water depth. Like the other two stations previously cited, runoff did not occur until the ground water elevation reached the invert of the stream.

The total monthly runoff for the basin during the months of August, September, and October were 0.77, 1.54, and 0.02 mgd, respectively. These also correspond to depths of 0.029, 0.057, and 0.001

inches of water spread over the entire basin. This is contrasted by the 8.02, 5.86, and 2.70 inches of rainfall that fell over this basin during the study.

E.6.3 Runoff Quantification Results

The results derived from quantifying the runoff from three typical basins within the RMR are summarized below:

- Runoff does not commence until the local ground water level approachs the stream invert elevation.
- Runoff ceases as the local ground water level recedes below the stream invert.
- Limited amounts of rainfall actually appear as runoff.

These results imply that much of the rainfall that falls onto the site initially replenishes subsurface storage deficits and fills surface depressions such as wetlands before appearing as runoff.

E.6.4 Rainfall/Runoff Relationship

Rainfall and runoff data obtained from DM-3 and DM-4 were used to develop hydrologic parameters to aid in understanding the rainfall and runoff relationships for the site. Once these relationships were determined, they were used to extrapolate data for those periods when instrument failure occurred.

E.6.4.1 Soil Conservation Service Runoff Curve Number, CN

The Soil Conservation Service (SCS) Runoff Curve Number, CN combines the influences of soil characteristics with land use and treatment to estimate runoff quantities during varying hydrologic conditions. The soil classifications most frequently found on the site are the Adamsville fine sand, Delray fine sand, Immokalee fine sand, and Pompano fine sand. These SCS soil classifications all fall under the A/D soil groups. The A/D designation represents the runoff behavior for drained/undrained conditions.

The range of CN values assigned to the A/D soil groups is 45/89 for pasture and wooded land treatments. A value of 45 represents low runoff rates during drained conditions while 89 represents high runoff rates during undrained conditions. Drained conditions exist when the ground water table is significantly below the ground surface. Undrained conditions exist when the ground water table is at or slightly below the surface.

The CN values calculated from the runoff events monitored by the Dames & Moore surface water monitoring stations ranged from 65 to 86. These values lie within the undrained range of the CN values for the A/D soil groups. This is explained by the fact that runoff did not occur until the ground water table reached the ground surface or undrained conditions existed.

E.6.4.2 Dimensionless Unit Hydrograph

Another hydrologic concept used in the RMR surface water hydrology approach was the dimensionless unit hydrograph. The unit hydrograph is a typical basin hydrograph for 1 inch of direct runoff from a storm of specified duration. The dimensionless unit hydrograph is computed from the unit hydrograph by dividing the ordinates of the unit hydrograph by the peak flow and the time to the peak flow. The dimensionless form eliminates the effect of basin size and much of the effect of basin shape (Linsley, Kohler, and Paulhus, 1958).

The dimensionless unit hydrograph used for the RMR was averaged from four runoff events that occurred at DM-4 and is shown in Figure E.6-2. In order to fill in the occasional gaps in the runoff hydrograph data, the ordinates of the dimensionless unit hydrograph were multiplied by the total runoff and a time to peak flow of 7 hours. The total runoff was determined by applying a value of CN to the total rainfall. This synthetic runoff hydrograph was then retrofitted into the actual data record.

E.7 MASS BALANCE ANALYSIS

E.7.1 General

The mass balance analysis consisted of the development of a mass balance model and the utilization of that model for the assessment of modifications to the ground water regime with respect to time under various withdrawal scenarios. The steps involved in the analysis consisted of the following:

- Model Conceptulization (Section E.7.2)
- Data Base Development (Section E.7.3)
- Model Calibration/Verification (Section E.7.4)
- Analysis of Site Water Balance (Section E.7.5)
- Analysis of Various Withdrawal Scenarios (Section E.7.6).

E.7.2 Model Conceptualization

The controlling equations for the mass balance analysis are:

Q(i) = f[y(i), eta, r(i) a(i)]	Equation 1
eta(i) = f[et(i), et(i-1)]	Equation 2
et(i) = f[pc(i), y(i)]	Equation 3
y(i) = f[s, y(i-1), r(i),eta]	Equation 4
a(i) = f[y(i), z(i)]	Equation 5

where:

Q(i) = runoff in inches for month i
y(i) = water table depth below the surface in feet at end of month i
et(i) = evapotranspiration rate (inches/month) at end of month i
eta(i) = average evapotranspiration (inches/month) for month (i)
s = storativity (feet of available storage)/(depth in feet)
pc(i) = pan evaporation in inches
r(i) = rainfall for month i in inches
a(i) = water table aquifer leakage in inches for month i
z(i) = Secondary Aquifer potentiometric head for month i

The independent variables in the above equations are rainfall, pan evaporation, and storativity. The equations as presented cannot be solved as there are six variables and five relationships. Equation 5 cannot be practically solved analytically given the complexities of the aquifer water fluxes. The solution was accomplished by taking the z(i) variable as a constant independent parameter derived from the various aquifer pumping and non-pumping scenarios.

The data bases available for calibration and verification of a mass balance model and subsequent analysis are:

1) 40 years of rainfall data at Myakka River State Park (1944-1985),
2) 4 years of continuous water level measurements at SWFWMD Wells ROMP19ES and ROMP19WS,
3) evapotranspiration data at the Dames & Moore evapotranspirometer,
4) pan evaporation data at Station DM5,
5) 4 months of continuous RMR site runoff data at DM3 and DM4,
6) regional runoff data (1936-1985) on the Myakka River at the USGS Highway 72 gage, and
7) storativity data from Dames & Moore pump tests and the evapotranspirometer.

E.7.3 Data Base Development

Each of the data bases listed above are discussed separately in the following subsections.

Rainfall at Myakka River State Park

The monthly totals of rainfall at Myakka River State Park are provided in Table E.7-1. A comparison of 5 months of available synoptic rainfall data at the park and at gaging stations on the RMR is provided in Table E.7-2. At a time step of one month the synoptic data indicates relative uniformity of rainfall between the park and stations within the RMR. This is to be expected although the convective thunderstorms occurring during the higher runoff months produce very localized cells of intense rainfall activity. It is not unusual to record very intense rainfalls in one location while simultaneously recording little or no rainfall at nearby gaging stations. The pattern, however, is dampened as the time step is increased, especially up to a period of 1 month. Months in which rainfall is either unusually high or unusually low between adjacent rainfall gaging stations do occur. The data in Table E.7-2 indicates just such an occurrence for the month of September in which rainfall totals exceeded 9.5 inches at the state park but were less than 6 inches at DM4.

The Myakka River State Park rainfall data is considered an appropriate data base for a long-term indication of probable monthly rainfall totals over the RMR. It is recognized however, that monthly totals at the State Park may periodically be unrepresentative of the rainfall that occurred over the site.

Water Level Measurements - SWFWMD Surficial Wells

Water levels at Surficial, Secondary, and Floridan wells on the RMR have been monitored continuously for approximately 4 years. The wells, installed by SWFWMD as documented in the Geraghty and Miller report (1981), are located as shown on the site vicinity map (Figure E.7-1). The plot of the approximate end of month water level for each month in the surficial wells and the secondary and Floridan wells at both the western and eastern sites is provided in Figures E.7-2 and E.7-3.

As shown in the figures and discussed in Appendix B, the Surficial Aquifer or water table aquifer are closely correlated to the potentiometric head in the underlying Secondary Aquifer. At both locations head differentials are generally less than 0.5 feet and the direction of the vertical gradient changes over time. The vertical gradient tends to be towards the Secondary Aquifer during the wet season with gradient reversal during the dry season. As discussed in Appendix B the low head differential and the relatively tight confining layer between the aquifers leads to the conclusion that there is limited continuous recharge of the Secondary Aquifer from the surficial.

On a long-term basis it is probable that there is little net water flux between the aquifers. Exceptions to this may occur during extended drought conditions in which either: 1) the potentiometric surface of the Secondary Aquifer is depressed significantly due to pumping or decreased up-gradient recharge, or 2) the surficial aquifer is depressed by ET. Under such conditions, sufficient hydraulic gradients may develop to produce significant aquifer water flux.

Figure E.7-4 provides a comparison of the depth of the water table at Wells ES and WS. There is significant variation of the depth on a monthly basis between the two sites. The general trends of the water table across the site however are comparable. The depth differences are likely due to the short term rainfall variations associated with localized intense thunderstorms and the water table response lag to a rainfall event.

For utilization in calibration of the mass balance model the two sets of data were averaged to obtain an approximate site-wide water table depth below ground surface during the period of continuous monitoring. The results of the averaging are shown on Figure E.7-5.

The absence of significant flux between the surficial and Secondary Aquifers is advantageous with respect to development of a predictive mass balance model. The governing equations (Equations 1 through 5) are sufficient only if an additional variable other than rainfall, pan evaporation, and storativity is independent. Unfortunately there is no long-term record for any of the other variables which could be used as input to the model. However, for normal conditions, the apparent lack of water flux between aquifers allows eliminating the recharge variable "a" and the Secondary Aquifer variable $z(i)$.

For well field operational scenarios involving Secondary Aquifer pumping the modeling is still complicated. This complexity was avoided for the conceptual phase by conservatively assuming that all of the water removed from the Secondary Aquifer would be recharged from the Surficial Aquifer. Further, for simplification the area of recharge was assumed to correspond to the area of 3.0 feet of drawdown in the Secondary Aquifer.

Evapotranspiration

Details of the evapotranspiration (ET) analysis and conclusions are provided in Section E.5. Based on assessment of the habitat maps approximately 30 percent of the RMR is characterized by wetlands. ET from this portion of the site is assumed to be best approximated by the data obtained from the wetland evapotranspirometer at Station DM2.

The remaining 70 percent of the site is predominantly dry prairie and pine flatwoods. ET rates associated with these habitat types are represented best by the dry prairie and pine flatwood evapotranspirometers at DM2 and DM3, respectively. As discussed in Section E.5, ET rates estimated with the two upland monitors must be used with some degree of caution. The two boxes do provide a site-specific approximation of the hydrometeorological conditions associated with varying habitats. Although vegetation conditions within the boxes could not be stabilized within the study time-frame, the data is responsive to cover meteorological conditions, and macroscale vapor gradients that would be found within similar habitats across the site.

The dry prairie evapotranspirometer at DM3 provided the most dependable data (see Figure E.5-6). Measurements indicated a strong correlation between evapotranspiration and water table depth. The dry prairie ET depth relationship was utilized as being a reasonable approximation of an average ET relationship for the RMR. On the microscale, the ET/depth relationship developed as shown on Figure E.5-6 will vary significantly. However, a microscale investigation of ET would be cost prohibitive and unlikely to improve the model predictive accuracy to a degree commensurate with data collection and analysis cost.

The ET relationship used in the model consists of a composite relationship between the wetland and upland relationships. It is assumed that below an average depth of 1 foot below the ground surface average ET over the RMR may be predicted as shown by the relationship on Figure E.5-6. When the water table is within 1 foot of the surface it is assumed that much higher ET rates assocated with wetlands is a factor. Above a 1 foot depth therefore, ET was weighted between the wetland rate and the rate calculated from the Figure E.5-6 relationship.

Pan Evaporation

The pan evaporation data collected at DM5 is shown in Figure E.5-4. Pan evaporation data is assumed to: 1) represent a composite indicator of the driving mechanisms associated with soil/vegetation vapor transfer to the atmosphere, and 2) represent the least variable on a monthly basis of the hydrometeorological parameters. Absence of long-term humidity and wind data in the vicinity of the RMR inhibits verification of monthly pan evaportion variation from year to year. The proximity of the ocean and the low degree of variation of the long-term temperature data indicates however that humidity is likely to be relatively consistent for a given month from year to year. Although the consistency of the wind in a given month from year to year is less certain, it is one of the least important variables driving pan evaporation.

The assumption of relative consistency in relative humidity allowed the use of the ten site-specific monthly pan evaporation values as an approximation of the long-term monthly pan evaporation. Due to the availability of this data, ET from the previous step was put in terms of percent of pan evaporation. Evapotranspiration for each month of the period of record was therefore calculated as a function of ground water depth and pan evaporation (see Figure E.5-6).

RMR Runoff

Runoff data was available for 3 months at DM3 and DM4. The short period of record renders this data base of limited use for calibration and verification of the mass balance model. It was utilized, as discussed in the verification section to assess the likely water flux between the aquifers.

Regional Runoff on Myakka River

Runoff data for the Myakka River is available for the period 1936 through 1985. A detailed discussion of the runoff data and its variability is provided in Appendix F. This data set was utilized in the analysis for verification purposes. Myakka River runoff was converted to inches of runoff per month for the period of synoptic data with Myakka River State Park rainfall (1944-1984) record. Myakka River basin unit runoff is assumed to be at least representative of the unit RMR runoff. Runoff for the Myakka River is most likely comparable to runoff from the site on time steps of 3 or 4 months. On a monthly basis divergence between site runoff and Myakka River runoff could periodically be quite high due to the variation in time of concentration between a 229 square mile basin and the smaller basins on the RMR. In addition, it is expected that runoff from the RMR basins will tend to have higher peaks per unit area than peaks on the Myakka River. This is associated with the larger volume of channel storage available in the Myakka River both from the Myakka River floodplain and Upper and Lower Myakka Lakes.

Storativity

Soil storativity was taken as an average of values obtained from both the pump test (Appendix B and C) and measurements from the evapotranspirometers. Pump test data indicated values of between 0.01 and 0.10. The evapotranspirometers indicated the range could be from 0.03 to 0.12. An average for the site of 0.055 was selected.

E.7.3 Calibration/Verification

Calibration

The model was calibrated against the average of measured water depth data from SWFWMD Wells ROMP19ES and ROMP19WS. The results of the calibration are shown in Figure E.7-5. The predicted water levels are in reasonable agreement with the average of the water levels measured at the two ROMP wells. In particular, as demonstrated with correlation coefficients (Table E.7-3), the trends of water level variation are in close agreement except for the winter of 1982. The predicted and measured water table depths are within 1 foot for 70 percent of the 52 months of measured water level data. The remaining 30 percent of the months were within 1.5 feet except for 2 months when the difference exceeded 2 feet. Although large variances do occur, they are not perpetuated from year to year. As shown for 1982 and 1985, large variations are factored out during each wet season and reasonable correlation is recovered.

It is likely that the primary forcing function behind the variation in predicted and measured water depth is the variability of recharge/discharge between the Surficial and Secondary Aquifers. This variation is best seen in the drought period of 1984-1985 in which the measured water levels are considerably lower than the predicted water levels. It is possible that this difference represents water loss from the Surficial Aquifer into the Secondary Aquifer during the later part of the 1984 wet season (see Figure E.7-3). The general close agreement of the model with measured data (except during extreme drought) indicates that the movement of water for this conceptual phase can be predicted with fair accuracy without inclusion of recharge/discharge between the aquifers.

Verification

Following calibration, the model predictive capabilitiy was verified by comparison of predicted RMR site runoff and measured Myakka

River basin unit runoff. The results of the comparison are shown in Figure E.7-6.

The comparison between predicted and measured values is generally good except for periodic wet season months in which predicted values significantly exceed the measured values. The exceptionally high predicted runoff, relative to measured, is most pronounced in the years following 1964 in which there were eight monthly occurrences. The variation between the predicted and measured data is lessened considerably when the comparison is on a quarterly basis. This is explained by the fact that peak monthly rainfall in the Myakka Basin may be measured as runoff over a period of perhaps several months where as monthly rainfall peaks on the RMR will be reflected as runoff more rapidly.

Overall agreement between predicted and measured is best for the first 20 years of the period of record (1945-1965). Following 1965 several months of intense rainfall produce yearly predicted runoff totals that are in excess by up to 20 percent of measured values. For the overall period of record the predicted runoff is 15.6 inches. This represents a difference of approximately 8 percent over the measured Myakka River runoff of 14.4 inches. The predicted runoff and evapotranspiration is shown on Table E.7-4.

The difference between predicted and measured values may be a function of one or more of the following factors:

1. Aquifer recharge which is not accounted for in the model.
2. Actual higher unit runoff within the RMR than in the basin as a whole due to differences in Myakka system storage.
3. Erroneous estimation of ET.
4. Unrepresentative rainfall from Myakka River State Park.
5. Errors in storativity.

E.7.4 Site Water Budget Model Analysis

Two assessments were made with the Site Water Budget Model (SWBM). One was an assessment of representative water table modifications under different withdrawal criteria. The second was an assessment of available freshwater runoff on a monthly basis for the period of record for the various drainage systems of the RMR.

The results generated from both assessments must be used with caution. The model predictions are limited in that: 1) the model itself utilizes a relatively simple water budget approach that does not attempt to define the micro-scale variability across the RMR, 2) the model may overestimate the volume of runoff for very high rainfall months, especially following a drought period, 3) the model may underestimate the drop in the water table for drought periods, 4) the model assumes a constant recharge rate from the Surficial Aquifer to the Secondary.

Despite the above model limitations it does provide an indication of the long-term water budget that is more than sufficient for conceptual planning purposes.

The assessments of water table variation under different aquifer pumping scenarios and monthly runoff for the period of record are discussed separately in the following subsections.

Water Table Variation Assessment

Water table variations were estimated for two pumping scenarios:

1. Secondary Aquifer pumping in Areas I, II, and III at the maximum rate that may possibly be sustained without violating SWFWMD drawdown criteria in the Secondary Aquifer.

2. Secondary Aquifer pumping in Area I as a supplement for surface reservoir storage supply (see Section F.5).

For both scenarios simplifying assumptions were made to allow investigation of the likely water table variations. The basic assumption was that leakance from the Surficial to the Secondary Aqufier may occur at an average rate of 0.02 inches per day over the area of the well field. This assumption was based on the assessment of aquifer characteristics presented in Appendices B and C.

The actual leakance will obviously vary significantly depending on actual potentiometric head differentials both spatially and temporally. As with the other assumptions involved in the modeling effort the selected values are believed to provide a reasonable approximation of what may occur under actual pumping conditions.

The results of the water table variation analysis is shown in Figure E.7-7. As is to be expected the maximum water table depressions are associated with the higher rate withdrawals. The maximum pumping scenario results in hydroperiod reductions of 1 to 2 months in almost every year. For the 20-years of historical record examined (1964-1985) the water table did return to the ground surface every year except for a two year period.

Utilization of the Secondary Aquifer for supplemental supply to surface storage resulted in signficantly less water table depressions. Water tables were depressed below normal during most dry seasons but returned to normal levels within 1 month of the onset of the rainy season.

Runoff Assessment

A summary of the runoff assessment is provided in Tables E.7-5 and E.7-6 for two drainage systems, Deer Prairie Slough and the series of parallel sloughs draining the southwest portion of the site (Figure E.2-2). It is assumed that the withdrawal of surface water will be constrained by the same criteria applied to withdrawals from the Myakka River (5 percent). It is expected that the volume of withdrawals may be increased depending on regulatory attitudes.

Regardless of the percent withdrawal that will ultimately be allowed by SWFWMD, both Deer Prairie and the Southwest Sloughs experience near zero flow for an average of 8 months out of every year. This results in an average required storage of approximately 750 acre-feet for every 1 mgd of supply. Further, extreme low flows periodically occur for periods of up to 18 months. This would result in a storage requirement of up to 1750 acre-feet for every 1 mgd of supply.

With an average zero flow period of 8 months, an average withdrawal of 3 mgd for each of the four wet season months is required for each 1 mgd supply.

Specific implications of the surface water assessment are presented in Section 4.3 of the main report.

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TABLE E.4-1

MONTHLY RAINFALL TOTALS AT DAMES & MOORE STATIONS
INCHES OF RAINFALL

	STATION					
				BELFORT/AW		
Month	DM-1[a]	DM-2[a]	DM-3[a]	DM-4[b]	DM-5[a]	DM-6[a]
January	ND[c]	ND	ND	ND/ND	ND	ND
February	ND	ND	ND	ND/ND	ND	ND
March	ND	ND	ND	ND/ND	ND	ND
April	ND	ND	ND	3.04/ND	ND	ND
May	ND	ND	ND	1.23/ND	2.40	ND
June	ND	6.48	6.92	4.22/3.72	8.51	ND
July	ND	6.67	7.71	7.76/9.16	9.20	ND
August	7.86	7.80	7.95	8.02/6.97	9.76	7.35
September	6.90	6.64	9.77	5.86/7.45	7.43	10.63
October	2.28	1.75	1.26	2.70/2.43	2.11	ND
November	2.77	2.90	3.22	2.06/2.95	3.28	2.90
December	0.57	0.49	0.48	0.48/0.59	0.46	0.26
January	1.42	1.50	1.45	1.72/1.56	1.52	1.33

[a] All-Weather (AW) Rain Gage Measurements: Periodic totals of one or more rainfall events.

[b] Monthly totals for each instrument (Belfort/All-Weather), may not be compared directly due to non-coincident sampling intervals.

[c] ND = No data available.

TABLE E.5-1

EVAPOTRANSPIROMETER SOIL PROPERTIES
COMPARISON BEFORE AND AFTER INSTALLATION

Station	Soil[c] Horizon	Measurements: In Situ/Replacement			
		Permeability (in/hr)	Wet Density (lb/ft^3)	Water Content (%)	Dry Density (lb/ft^3)
DM-2	1	2.1/2.3	113.8/106.8	7.5/ 7.9	105.9/ 99.0
	2	3.1/4.8	116.8/106.1	13.0/ 8.8	103.4/ 97.5
	3	2.7/1.9	129.2/114.7	16.6/13.3	110.8/101.2
	4	a	132.3/126.4	14.2/14.6	115.8/110.3
	5	b	b	b	b
DM-3	1	5.4/6.7	99.2/ 91.1	5.5/ 7.7	94.0/ 84.6
	2	5.2/4.9	104.0/103.6	4.3/ 5.7	99.7/ 98.0
	3	3.1/0.8	111.8/112.6	7.2/10.7	104.3/101.7
	4	1.7/[a]	125.8/123.0	15.2/12.0	109.2/109.8
	5	1.7/[a]	131.7/[b]	17.6/[b]	112.0/[b]

[a] Beyond measurement range.
[b] Not determined.
[c] Refer to Figure E.5-1 for key to soil horizons.

TABLE E.5-2

LOWER MYAKKA LAKE EVAPOTRANSPIRATION PAN
DATA COMPARISON

	National Weather Service Class A Pan (in/day)[a]	Lower Myakka Lake Pan (in/day)[a]	Lower Myakka Lake Pan %[b]
August	0.19	0.26	136
September	0.21	0.41	195
October	0.16	0.13	81
November	0.13	0.10	77
December	0.10	0.08	80
January	0.10	0.07	70
February	0.13	0.10	77

[a] Evaporation and evapotranspiration rates in inches per day.
[b] Lake evapotranspiration shown as a percent of pan evaporation.

TABLE E.5-3

DRY PRAIRIE EVAPOTRANSPIROMETER
DATA COMPARISON

	National Weather Service Class A Pan (in/day)[a]	Dry Prairie Evapotranspirometer (in/day)[a]	Dry Prairie Evapotranspirometer %[b]
August	0.19	0.12	63
September	0.21	0.11	52
October	0.16	0.06	37
November	0.13	0.09	69
December	0.10	0.06	60

[a] Evaporation and evapotranspiration rates in inches per day.
[b] Dry Prairie evapotranspiration shown as a percent of pan evaporation.

TABLE E.5-4

WETLAND EVAPOTRANSPIROMETER
DATA COMPARISON

	National Weather Service Class A Pan (inches)[a]	Wetland Evapotranspirometer (inches)[a]	Wetland Evapotranspirometer %[b]
August	5.95	6.72	113
September	6.53	7.96	122
October	4.60	4.87	106
November	3.67	3.02	82

[a] Evaporation and evapotranspiration monthly totals in inches.
[b] Wetland evapotranspiration shown as a percent of pan evaporation.

TABLE E.7-1

RAINFALL DATA
MYAKKA RIVER STATE PARK

DATA TO BE PROVIDED

TABLE E.7-2

MONTHLY RAINFALL TOTALS AT DAMES & MOORE STATIONS

	AW DM-1	AW DM-2	AW DM-3	Belfort/AW DM-4	AW DM-5	Myakka River
February						
March						
April				3.04/--		
May				1.23/--	2.40	0.23
June		6.48	6.92	4.22/--	8.51	5.58
July		6.67	7.71	7.76/--	9.20	7.46
August	7.86	7.80	7.95	*8.02/6.97	9.76	9.10
September	6.90	6.64	9.77	5.86/7.45	7.43	9.59
October	2.28	1.75	1.26	**2.70/2.43	2.11	1.91
November	2.77	2.90	3.22	2.06/2.95	3.28	
December	0.57	0.49	0.48	0.48/0.59	0.46	
January	1.42	1.50	1.45	1.72/1.56	1.52	

*Significant (1.10) rainfall at end of month (Hurricane Elena)
**0.47 inches at end of month.

TABLE E.7-3

ASSESSMENT OF VARIANCE BETWEEN MEASURED AND PREDICTED WATER LEVELS

	1981	1982	1983	1984
Predicted Depth Mean	0.82	1.13	0.88	2.14
Measured Depth Mean	1.58	0.99	0.95	3.47
Standard Deviation Predicted Depth	0.96	1.30	1.00	1.77
Standard Deviation Measured Depth	0.92	1.00	0.83	1.53
Coefficients of Variation-Predicted Depth	117.89	115.35	114.20	82.53
Coefficients of Variation-Measured Depth	57.94	101.05	87.64	44.08
Covariance	0.36	1.18	0.63	2.24
Correlation Coefficient	0.41	0.90	0.75	0.83
Sum of Predicted Depth	9.82	13.50	10.53	25.72
Sum of Measured Depth	19.00	11.89	11.42	41.60

TABLE E.7-4

PREDICTED MONTHLY EVAPOTRANSPIRATION/RUNOFF
SURFACE WATER BALANCE MODEL (SWBM)

Year	Rainfall (inches)	Evapotranspiration (inches)	Runoff (inches)
1944	39.40	36.07	2.95
1945	50.05	35.44	14.49
1946	43.72	39.49	5.05
1947	62.22	40.82	21.30
1948	46.97	39.24	8.31
1949	47.20	32.62	14.36
1950	41.40	36.14	4.89
1951	56.43	39.89	15.68
1952	44.28	36.47	7.81
1953	58.23	45.77	12.46
1954	60.87	50.70	10.47
1955	48.96	38.05	12.34
1956	39.12	31.28	6.11
1957	72.66	47.94	24.72
1958	60.91	44.05	16.86
1959	84.12	53.26	30.86
1960	64.35	43.93	21.29
1961	39.60	38.01	2.60
1962	63.75	37.06	26.71
1963	49.35	36.69	13.21
1964	54.63	41.59	11.91
1965	64.52	40.22	23.06
1966	47.58	40.83	6.94
1967	57.30	43.90	14.73
1968	52.77	33.92	17.94
1969	61.26	42.72	17.94
1970	56.46	47.21	11.43
1971	56.46	32.23	22.57
1972	49.61	38.68	10.93
1973	67.26	44.91	24.57
1974	52.38	35.04	17.87
1975	53.23	35.97	14.52
1976	45.83	37.78	9.56
1977	51.37	38.40	13.49
1978	55.63	41.35	14.28
1979	67.76	38.51	28.75

Year	Rainfall (inches)	Evapotransporation (inches)	Runoff (inches)
1980	57.40	41.13	16.43
1981	66.70	40.68	25.55
1982	81.07	46.74	33.83
1983	69.45	43.06	26.42
1984	61.65	41.83	19.60
1985	43.26	33.30	5.01

TABLE E.7-5

DEER PRAIRIE SLOUGH SUPPLY
IN MGD FOR INDICATED MONTH AT 5 PERCENT OF FLOW

Year	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.
1944	--	--	--	--	--	--	--	0.99	0.42	0.17	1.07	--
1945	--	--	--	--	--	--	--	0.78	7.21	5.04	--	--
1946	--	--	--	--	--	--	--	2.16	0.50	1.07	0.81	--
1947	--	--	--	--	3.03	--	--	4.77	5.79	2.98	2.57	--
1948	1.55	--	--	--	--	0.02	--	--	2.18	0.31	3.41	--
1949	--	--	--	--	--	--	--	--	0.10	10.30	2.50	--
1950	--	--	--	--	--	--	--	0.03	1.92	1.86	0.59	--
1951	--	--	--	--	--	0.20	--	--	6.14	1.10	1.23	5.42
1952	--	--	--	--	0.69	--	--	--	--	--	1.53	4.80
1953	0.20	--	--	0.19	--	--	--	3.69	--	1.39	3.87	1.86
1954	1.76	--	--	--	--	0.98	--	1.67	--	0.47	4.53	--
1955	0.33	--	--	--	--	--	--	--	8.82	1.83	0.10	--
1956	--	--	--	--	--	--	--	--	--	0.48	3.88	1.14
1957	--	--	--	0.89	2.17	0.12	--	4.08	--	6.44	6.45	2.08
1958	--	--	2.73	--	4.01	--	--	--	1.70	4.71	--	2.00
1959	1.40	2.32	--	--	4.60	--	4.94	1.72	0.32	6.77	2.33	3.35
1960	--	--	--	0.29	0.65	--	--	--	8.65	1.91	7.55	--
1961	--	--	--	1.13	--	--	--	--	0.15	1.06	--	--
1962	--	--	--	--	--	--	--	2.61	--	6.26	15.14	--
1963	--	--	--	1.87	--	--	--	--	--	8.41	1.59	--
1964	0.17	--	--	0.96	1.04	--	--	--	1.40	3.03	4.12	--
1965	--	--	--	--	--	--	--	4.12	7.13	8.45	--	1.02
1966	--	--	1.08	--	--	--	--	1.54	1.31	2.31	--	--
1967	--	--	--	--	--	--	--	1.99	2.27	3.44	5.55	--
1968	--	--	--	--	--	--	--	9.08	6.15	--	0.90	--
1969	0.09	--	--	--	5.38	--	--	3.63	--	2.90	3.11	1.02

TABLE E.7-5 (Continued)

Year	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.
1970	--	0.42	--	--	2.11	--	--	--	1.24	6.50	--	--
1971	--	--	--	--	--	--	--	--	4.29	12.11	2.56	1.33
1972	--	--	--	0.38	--	--	--	7.06	--	1.99	--	0.38
1973	1.65	0.10	4.05	--	--	--	--	--	11.75	2.34	2.20	--
1974	--	--	--	--	--	--	--	12.28	3.79	--	--	--
1975	--	--	--	--	--	--	--	1.00	10.19	--	--	1.87
1976	--	--	--	--	--	--	--	4.81	0.18	0.03	3.57	--
1977	--	--	--	--	--	--	--	0.49	3.37	3.56	4.71	--
1978	--	2.52	1.04	1.57	--	--	--	3.40	--	1.74	2.57	--
1979	--	--	3.22	--	--	--	--	--	1.46	7.29	13.88	--
1980	--	--	0.35	--	--	--	--	--	0.62	9.76	4.05	--
1981	0.28	--	--	1.22	--	--	--	6.38	1.59	13.23	0.28	--
1982	1.26	--	--	--	3.11	--	--	8.27	6.15	5.93	5.69	--
1983	--	--	--	5.21	3.66	--	--	2.50	4.94	2.04	5.42	--
1984	1.49	4.01	--	--	1.50	--	--	--	8.68	1.94	--	--
1985	--	--	--	--	--	--	--	--	1.77	2.59	0.15	--

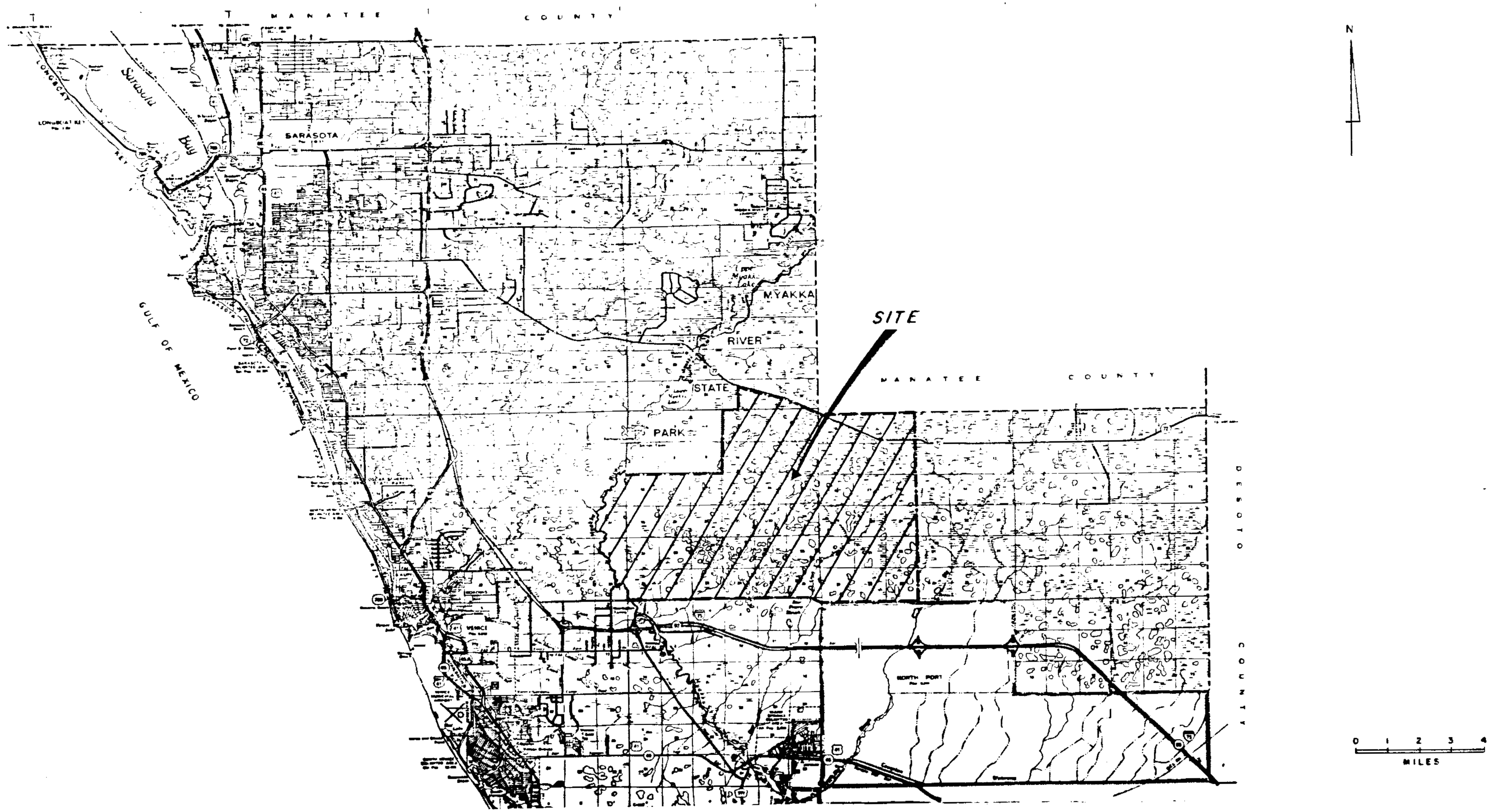
TABLE E.7-6

SOUTHWEST DRAINAGE SYSTEMS SUPPLY
IN MGD FOR INDICATED MONTH AT 5 PERCENT OF FLOW

Year	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.
1944	--	--	--	--	--	--	--	0.52	0.22	0.09	0.56	--
1945	--	--	--	--	--	--	--	0.41	3.77	2.63	--	--
1946	--	--	--	--	--	--	--	1.13	0.26	0.56	0.42	--
1947	--	--	--	--	1.58	--	--	2.50	3.03	1.56	1.35	--
1948	0.81	--	--	--	--	0.01	--	--	1.14	0.16	1.78	--
1949	--	--	--	--	--	--	--	--	0.05	5.39	1.31	--
1950	--	--	--	--	--	--	--	0.02	1.00	0.97	0.31	--
1951	--	--	--	--	--	0.11	--	--	3.21	0.58	0.64	2.84
1952	--	--	--	--	0.36	--	--	--	--	--	0.80	2.51
1953	0.10	--	--	0.10	--	--	--	1.93	--	0.73	2.02	0.97
1954	0.92	--	--	--	--	0.51	--	0.87	--	0.25	2.37	--
1955	0.17	--	--	--	--	--	--	--	4.61	0.96	0.05	--
1956	--	--	--	--	--	--	--	--	--	0.25	2.03	0.59
1957	--	--	--	0.46	1.14	0.06	--	2.13	--	3.37	3.37	1.09
1958	--	--	1.43	--	2.10	--	--	--	0.89	2.46	--	1.05
1959	0.73	1.21	--	--	2.40	--	2.58	0.90	0.17	3.54	1.22	1.75
1960	--	--	--	0.15	0.34	--	--	--	4.52	1.00	3.95	--
1961	--	--	--	0.59	--	--	--	--	0.08	0.55	--	--
1962	--	--	--	--	--	--	--	1.37	--	3.27	7.92	--
1963	--	--	--	0.98	--	--	--	--	--	4.39	0.83	--
1964	0.09	--	--	0.50	0.54	--	--	--	0.73	1.59	2.15	--
1965	--	--	--	--	--	--	--	2.16	3.73	4.42	--	0.53
1966	--	--	0.56	--	--	--	--	0.80	0.68	1.21	--	--
1967	--	--	--	--	--	--	--	1.04	1.19	1.80	2.90	--
1968	--	--	--	--	--	--	--	4.75	3.22	--	0.47	--
1969	0.04	--	--	--	2.81	--	--	1.90	--	1.52	1.62	0.53

TABLE E.7-6 (Continued)

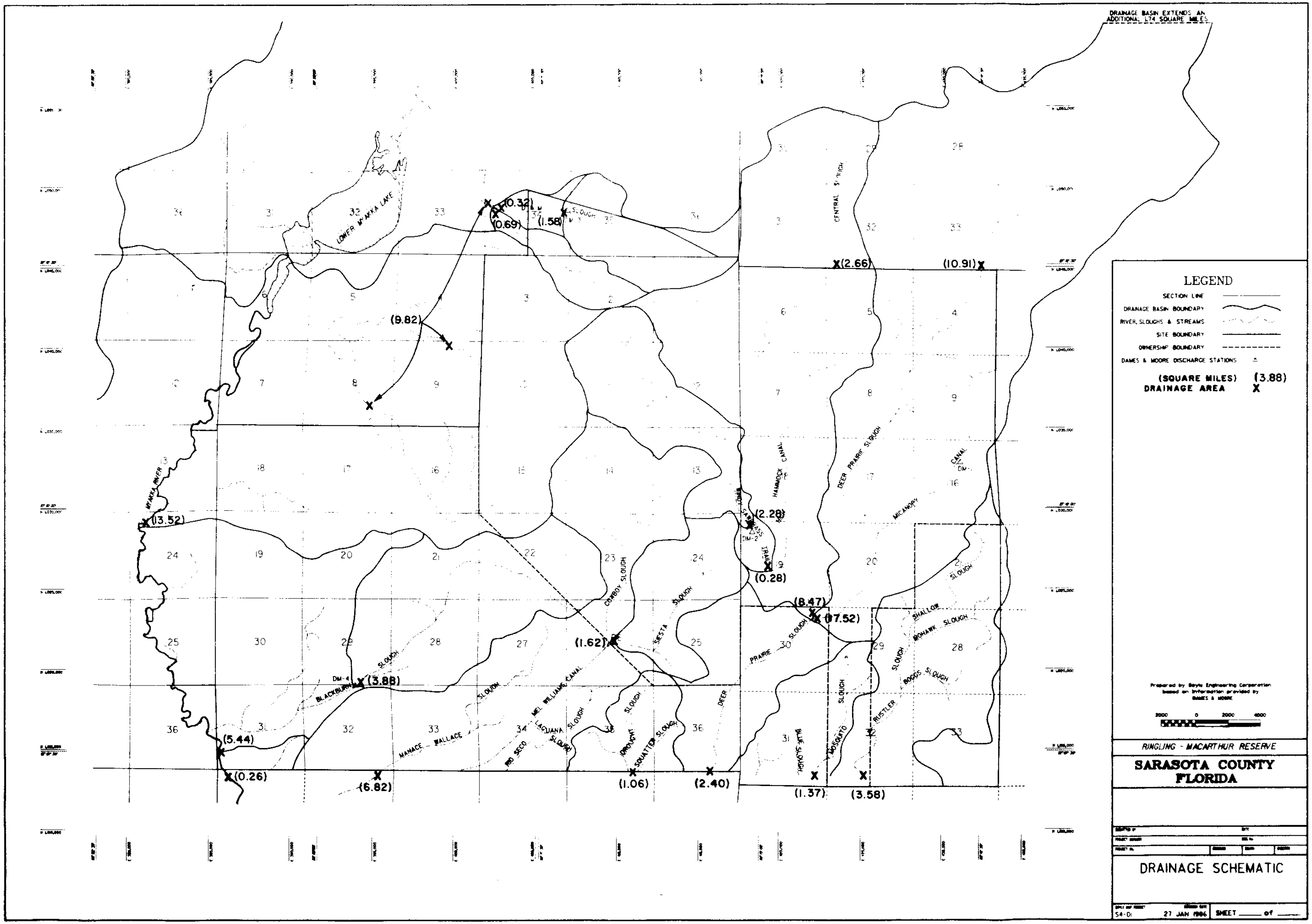
Year	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.
1970	--	0.22	--	--	1.10	--	--	--	0.65	3.40	--	--
1971	--	--	--	--	--	--	--	--	2.24	6.33	1.34	0.70
1972	--	--	--	0.20	--	--	--	3.69	--	1.04	--	0.20
1973	0.86	0.05	2.12	--	--	--	--	--	6.14	1.22	1.15	--
1974	--	--	--	--	--	--	--	6.42	1.98	--	--	--
1975	--	--	--	--	--	--	--	0.52	5.33	--	--	0.98
1976	--	--	--	--	--	--	--	2.52	0.09	0.02	1.87	--
1977	--	--	--	--	--	--	--	0.26	1.76	1.86	2.46	--
1978	--	1.32	0.54	0.82	--	--	--	1.78	--	0.91	1.35	--
1979	--	--	1.68	--	--	--	--	--	0.76	3.81	7.25	--
1980	--	--	0.18	--	--	--	--	--	0.32	5.10	2.12	--
1981	0.14	--	--	0.64	--	--	--	3.33	0.83	6.92	0.15	--
1982	0.66	--	--	--	1.63	--	--	4.32	3.22	3.10	2.98	--
1983	--	--	--	2.72	1.91	--	--	1.30	2.58	1.07	2.83	--
1984	0.78	2.09	--	--	0.78	--	--	--	4.54	1.01	--	--
1985	--	--	--	--	--	--	--	--	0.92	1.36	0.08	--



REFERENCE :
GENERAL HIGHWAY MAP, SARASOTA COUNTY, 1975

Figure E.2-1. Vicinity Map.

Dames & Moore


DRAINAGE BASIN EXTENDS AN ADDITIONAL 1.74 SQUARE MILES
LEGEND
SECTION LINE
DRAINAGE BASIN BOUNDARY
RIVER, SLOUGHS & STREAMS
SITE BOUNDARY
OWNERSHIP BOUNDARY
DAMES & MOORE DISCHARGE STATIONS
(SQUARE MILES) (3.88)
DRAINAGE AREA X
Prepared by Boyle Engineering Corporation based on information provided by DAMES & MOORE
RINGLING - MACARTHUR RESERVE
SARASOTA COUNTY
FLORIDA
DRAINAGE SCHEMATIC
27 JAN 1986
SHEET of
LOWER MYAKKA LAKE
MYAKKA RIVER
CENTRAL SLOUGH
HAMMOCK CANAL
DEER PRAIRIE SLOUGH
MICANOPY CANAL
COWBOY SLOUGH
SIESTA SLOUGH
PRAIRIE SLOUGH
SHALLOW SLOUGH
MOHAWK SLOUGH
BOGGS SLOUGH
RUSTLER
MOSQUITO
BLUE SLOUGH
DEER
SQUATTER SLOUGH
LAGUANA SLOUGH
MEL WILLIAMS CANAL
RIO SECO
MANACE - WALLACE SLOUGH
BLACKBURN SLOUGH
DM-2
DM-4
(0.32)
(0.69)
(1.58)
(2.66)
(10.91)
(9.82)
(13.52)
(2.28)
(0.28)
(8.47)
(17.52)
(1.62)
(3.88)
(5.44)
(0.26)
(6.82)
(1.06)
(2.40)
(1.37)
(3.58)
FIGURE E.2-2

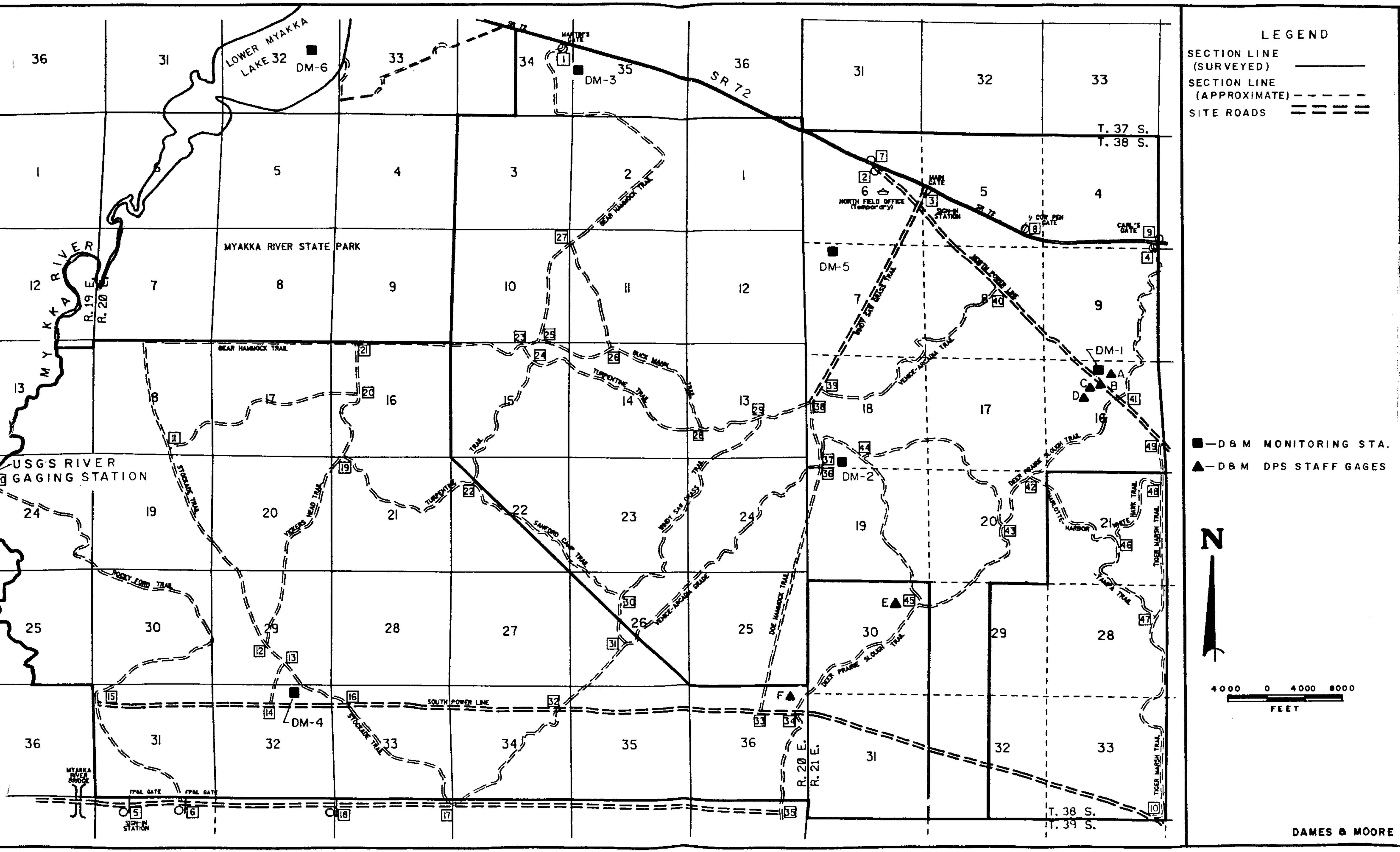


Figure E.3-1. Location Map of Water Monitoring Stations

DAMES & MOORE
DAYS
RAINFALL (INCHES)
SEE NOTE FIGURE E.4-20
RAINFALL HYETOGRAPH
GROUNDWATER DEPTH (DATA FROM WELL ROMP 19 (WEST) NOT AVAILABLE)
STREAM DISCHARGE (NO FLOW)
RUNOFF HYDROGRAPH
STATION: DM 4 NOV. '85
DEPTH TO GROUNDWATER (FEET)
DISCHARGE (CFS)
DAYS

FIGURE E.4-27

DAMES & MOORE
DAYS
RAINFALL (INCHES)
0.00
0.20
0.40
0.60
0.80
1.00
1.20
1.40
1.60
1.80
2.00
SEE NOTE FIGURE E.4-20
RAINFALL HYETOGRAPH
RUNOFF HYDROGRAPH
– – – GROUNDWATER DEPTH (DATA FROM WELL ROMP 19 (WEST) NOT AVAILABLE)
—— STREAM DISCHARGE (NO FLOW)
STATION: DM 4 DEC. '85
DEPTH TO GROUNDWATER (FEET)
DISCHARGE (CFS)
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31
DAYS

FIGURE E.4-28


DAMES & MOORE
DAYS
RAINFALL (INCHES)
SEE NOTE FIGURE E.4-6
2.43"
RAINFALL HYETOGRAPH
RUNOFF HYDROGRAPH
STREAM
STATION: DM1 AUG '85
WATER SURFACE ELEVATION (FEET-MSL)
25.64 FEET = STREAM INVERT ELEVATION
DAYS

FIGURE E.4-1

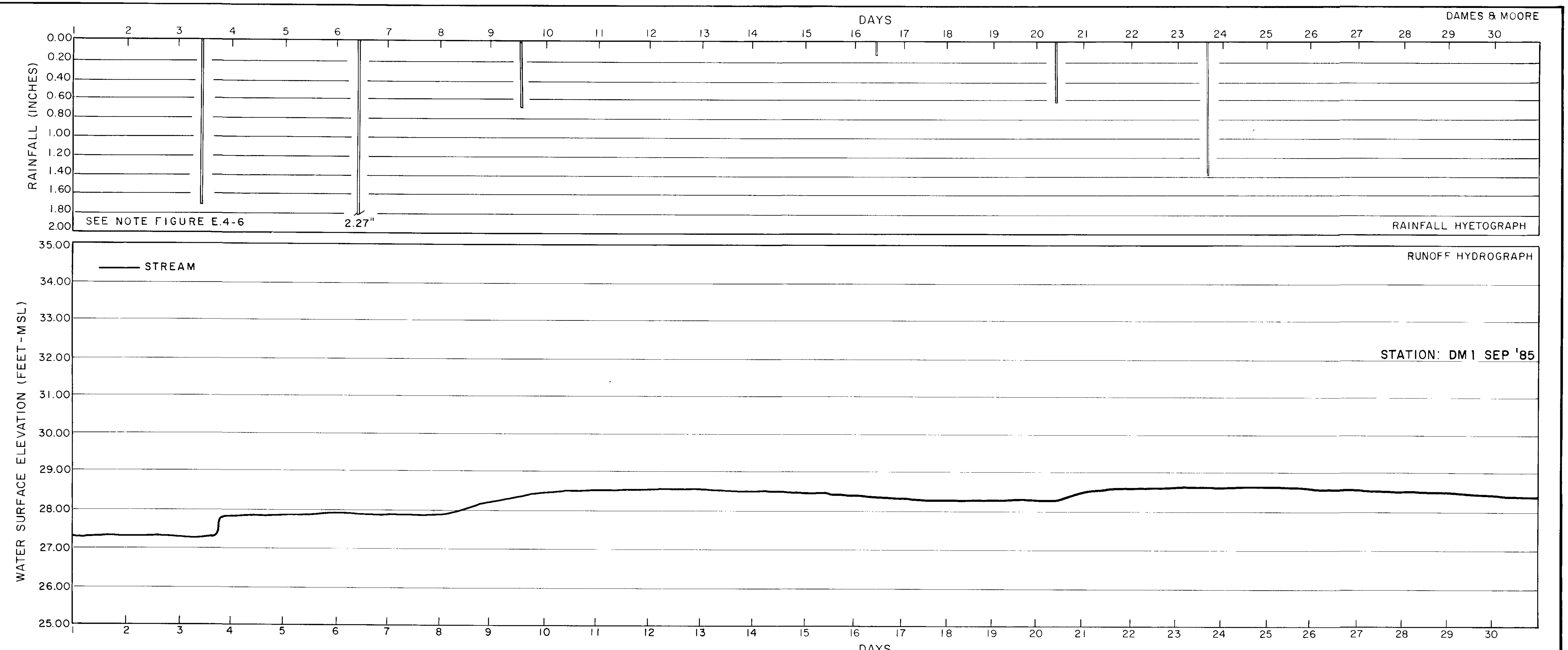

DAMES & MOORE
DAYS
RAINFALL (INCHES)
0.00
0.20
0.40
0.60
0.80
1.00
1.20
1.40
1.60
1.80
2.00
SEE NOTE FIGURE E.4-6
2.27"
RAINFALL HYETOGRAPH
RUNOFF HYDROGRAPH
STREAM
STATION: DM1 SEP '85
WATER SURFACE ELEVATION (FEET-MSL)
35.00
34.00
33.00
32.00
31.00
30.00
29.00
28.00
27.00
26.00
25.00
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30
DAYS

FIGURE E.4-2

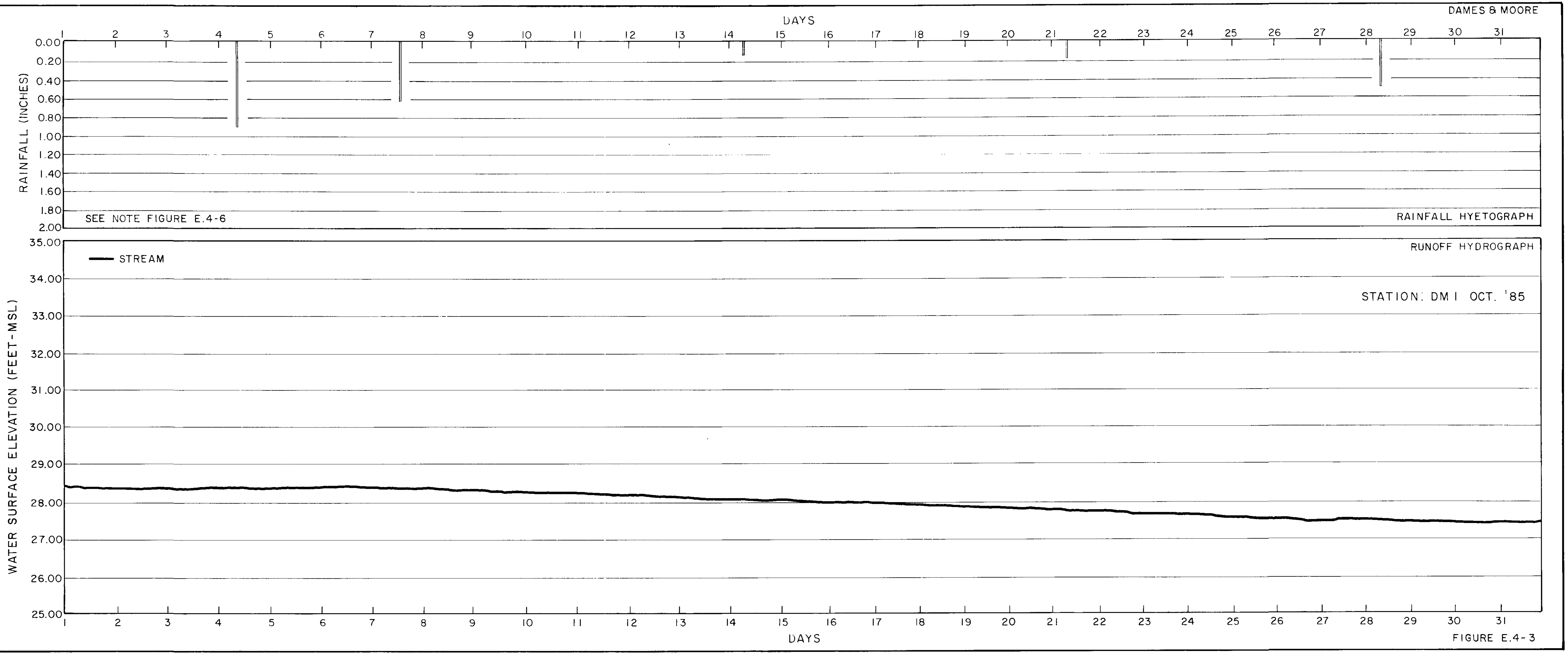

DAMES & MOORE
DAYS
RAINFALL (INCHES)
SEE NOTE FIGURE E.4-6
RAINFALL HYETOGRAPH
RUNOFF HYDROGRAPH
STREAM
STATION: DM I OCT. '85
WATER SURFACE ELEVATION (FEET-MSL)
DAYS

FIGURE E.4-3

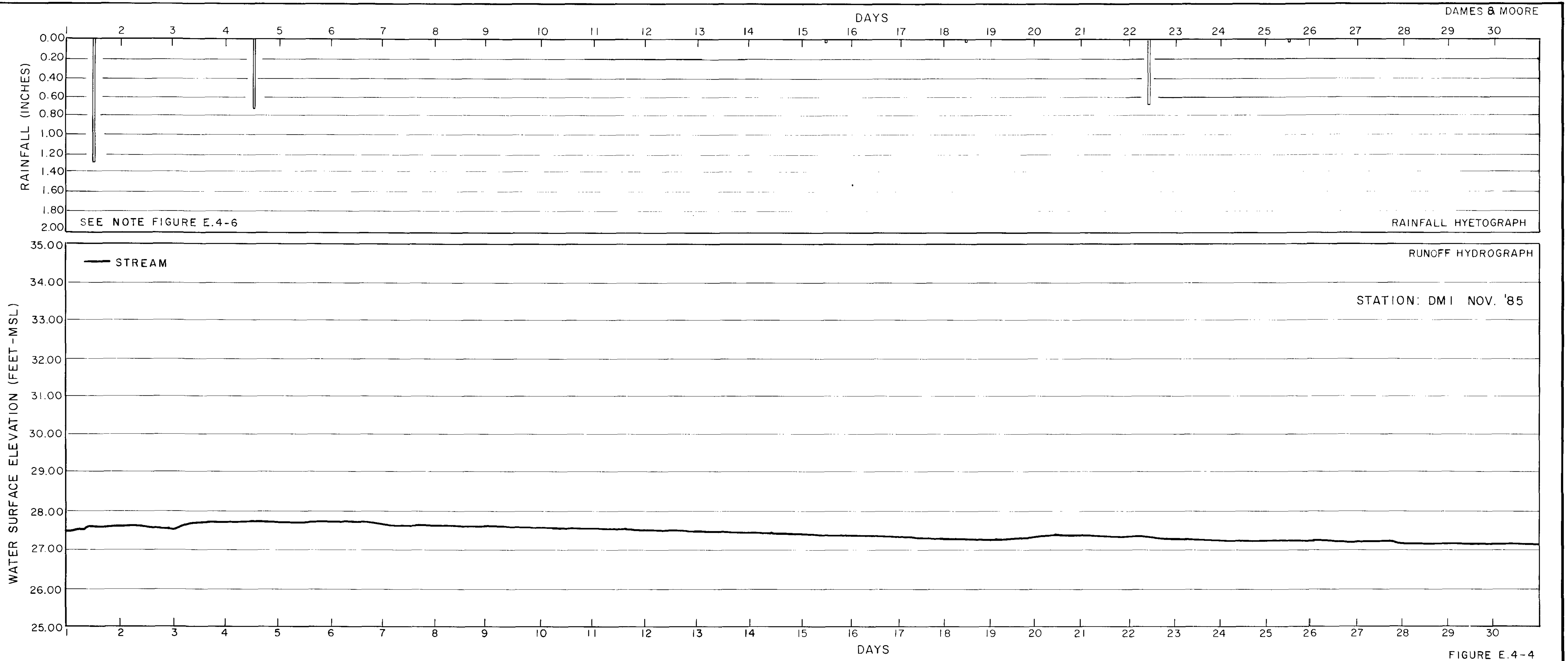

DAMES & MOORE
DAYS
RAINFALL (INCHES)
SEE NOTE FIGURE E.4-6
RAINFALL HYETOGRAPH
STREAM
RUNOFF HYDROGRAPH
STATION: DMI NOV. '85
WATER SURFACE ELEVATION (FEET-MSL)
DAYS

FIGURE E.4-4

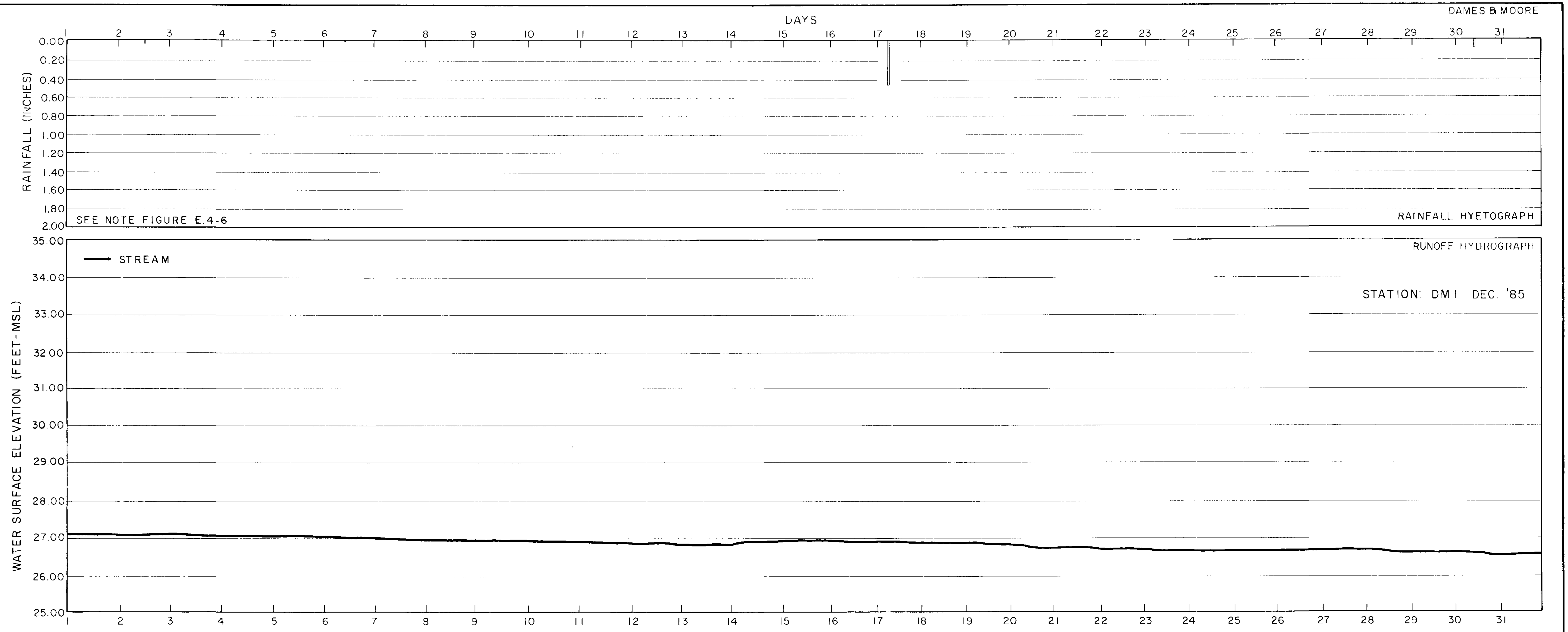
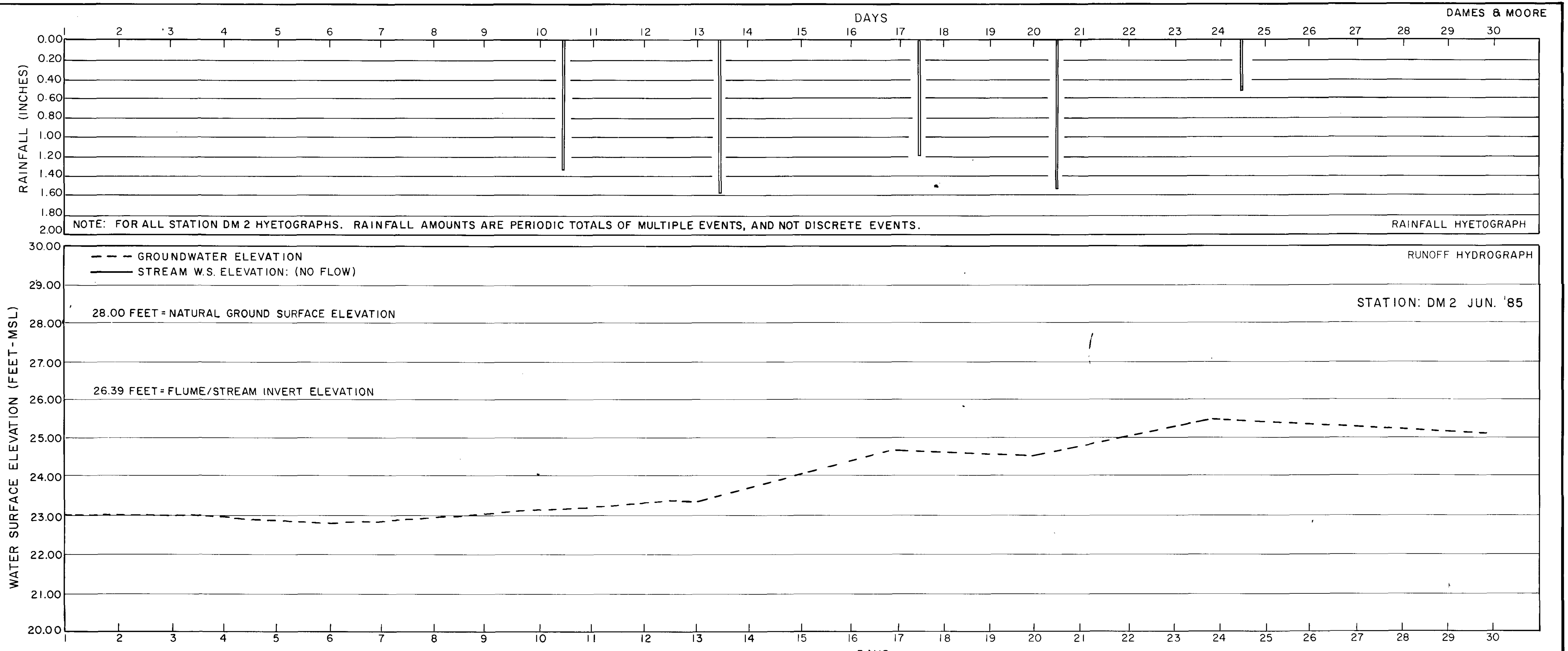

DAMES & MOORE
DAYS
RAINFALL (INCHES)
SEE NOTE FIGURE E.4-6
RAINFALL HYETOGRAPH
RUNOFF HYDROGRAPH
STREAM
STATION: DM1 DEC. '85
WATER SURFACE ELEVATION (FEET-MSL)
DAYS

FIGURE E.4-5


DAMES & MOORE
DAYS
RAINFALL (INCHES)
NOTE: FOR ALL STATION DM 2 HYETOGRAPHS. RAINFALL AMOUNTS ARE PERIODIC TOTALS OF MULTIPLE EVENTS, AND NOT DISCRETE EVENTS.
RAINFALL HYETOGRAPH
GROUNDWATER ELEVATION
STREAM W.S. ELEVATION: (NO FLOW)
RUNOFF HYDROGRAPH
STATION: DM 2 JUN. '85
28.00 FEET = NATURAL GROUND SURFACE ELEVATION
26.39 FEET = FLUME/STREAM INVERT ELEVATION
WATER SURFACE ELEVATION (FEET-MSL)
DAYS
FIGURE E.4-6

DAMES & MOORE
DAYS
RAINFALL (INCHES)
0.00
0.20
0.40
0.60
0.80
1.00
1.20
1.40
1.60
1.80
2.00
SEE NOTE FIGURE E.4-6
2.79"
4.30"
RAINFALL HYETOGRAPH
GROUNDWATER ELEVATION
STREAM W.S. ELEVATION
RUNOFF HYDROGRAPH
STATION: DM 2 JUL. '85
NO FLOW
WATER SURFACE ELEVATION (FEET-MSL)
30.00
29.00
28.00
27.00
26.00
25.00
24.00
23.00
22.00
21.00
20.00
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31
DAYS

FIGURE E.4-7

DAMES & MOORE
DAYS
RAINFALL (INCHES)
SEE NOTE FIGURE E.4-6
RAINFALL HYETOGRAPH
GROUNDWATER ELEVATION
STREAM W.S. ELEVATION
RUNOFF HYDROGRAPH
STATION: DM 2 AUG. '85
WATER SURFACE ELEVATION (FEET-MSL)
DAYS

FIGURE E.4-8

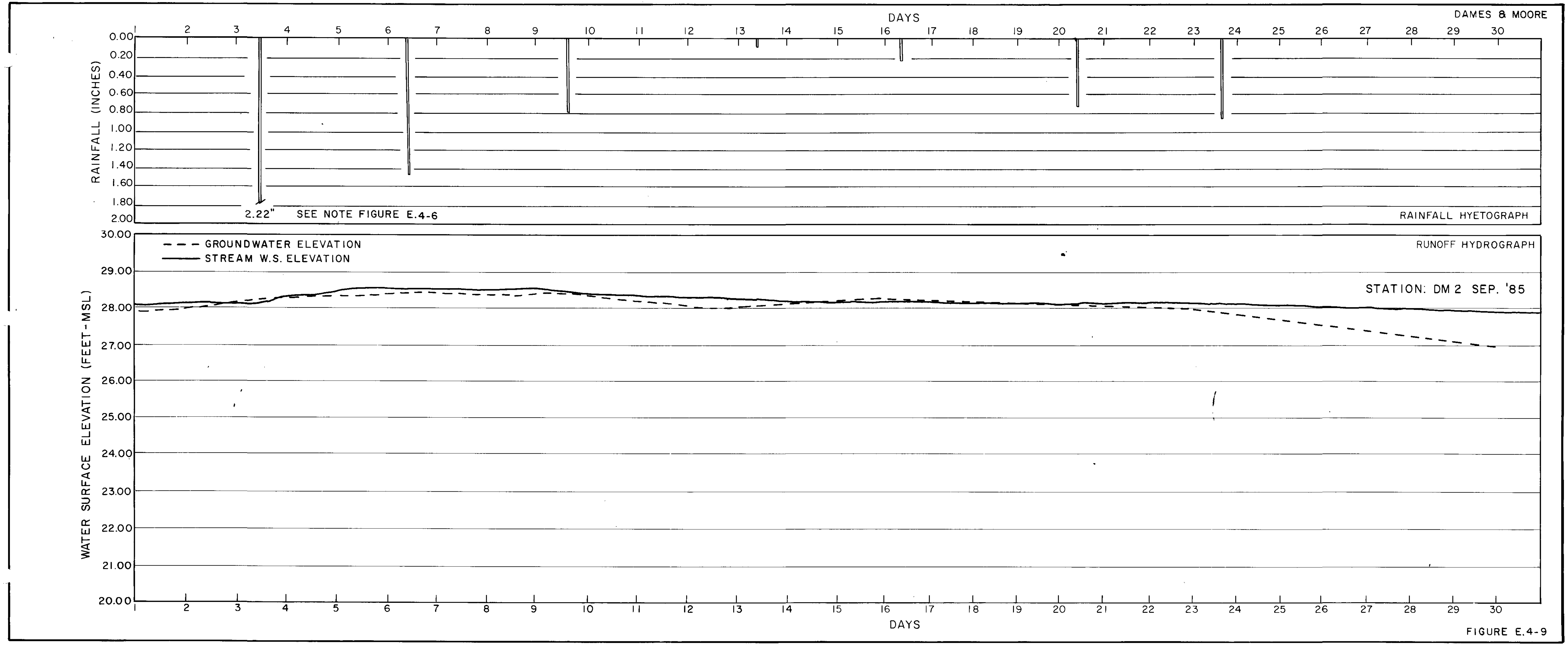
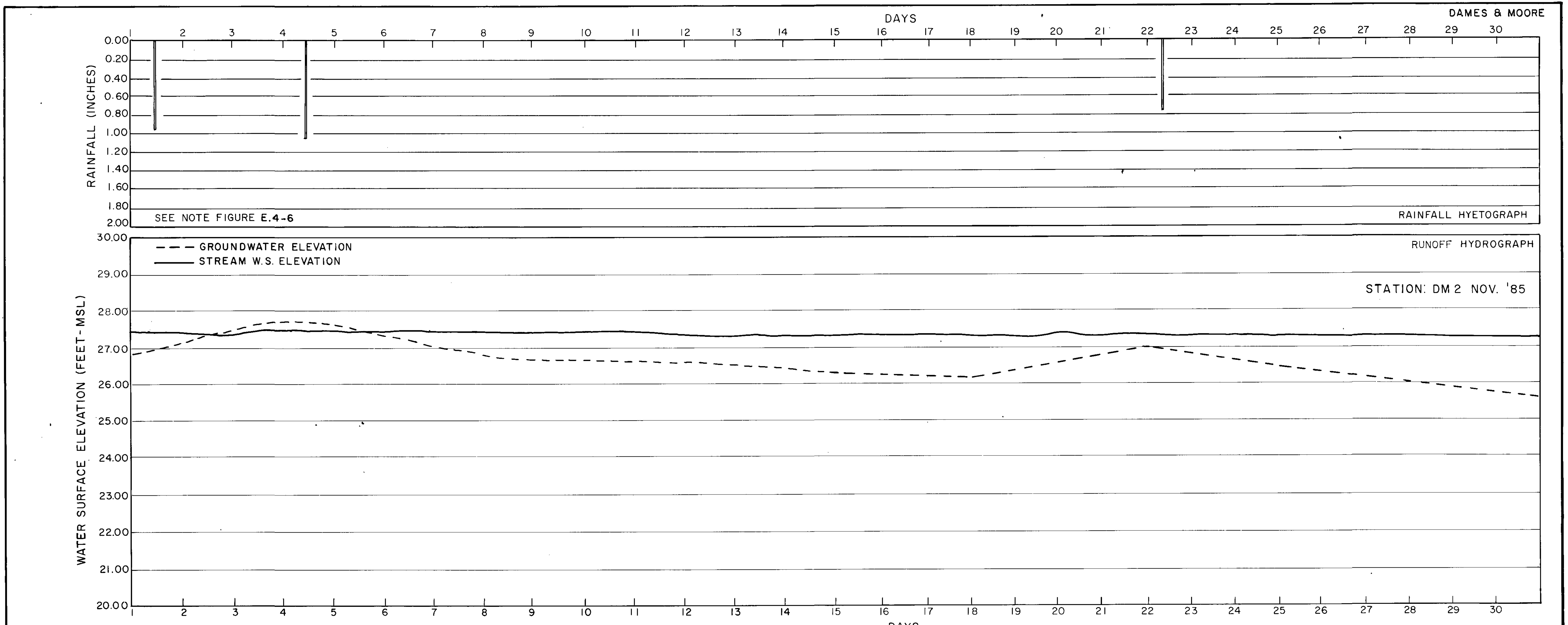

DAMES & MOORE
DAYS
RAINFALL (INCHES)
0.00
0.20
0.40
0.60
0.80
1.00
1.20
1.40
1.60
1.80
2.00
2.22"
SEE NOTE FIGURE E.4-6
RAINFALL HYETOGRAPH
GROUNDWATER ELEVATION
STREAM W.S. ELEVATION
RUNOFF HYDROGRAPH
STATION: DM 2 SEP. '85
WATER SURFACE ELEVATION (FEET-MSL)
30.00
29.00
28.00
27.00
26.00
25.00
24.00
23.00
22.00
21.00
20.00
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30
DAYS

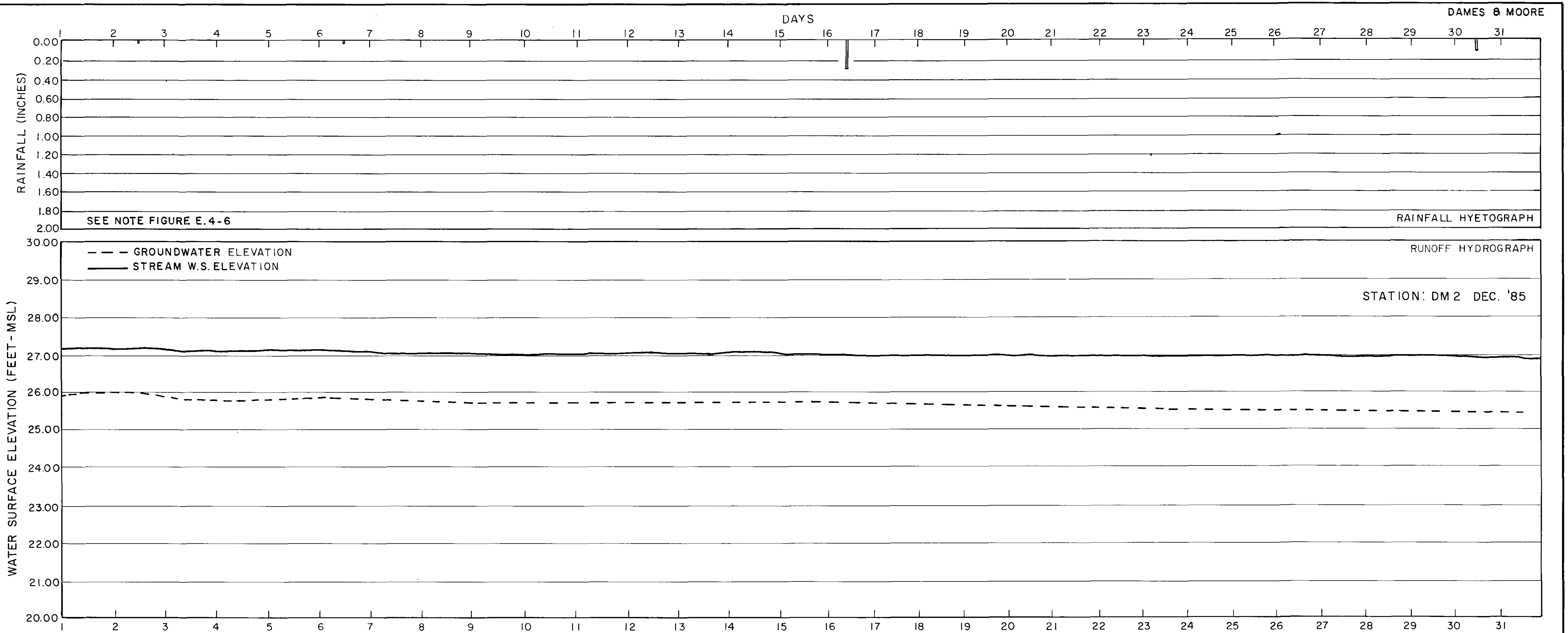
FIGURE E.4-9


DAMES & MOORE
DAYS
RAINFALL (INCHES)
0.00
0.20
0.40
0.60
0.80
1.00
1.20
1.40
1.60
1.80
2.00
SEE NOTE FIGURE E.4-6
RAINFALL HYETOGRAPH
GROUNDWATER ELEVATION
STREAM W.S. ELEVATION
RUNOFF HYDROGRAPH
STATION: DM 2 OCT. '85
WATER SURFACE ELEVATION (FEET-MSL)
30.00
29.00
28.00
27.00
26.00
25.00
24.00
23.00
22.00
21.00
20.00
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31
DAYS
FIGURE E.4-10



DAMES & MOORE
DAYS
RAINFALL (INCHES)
SEE NOTE FIGURE E.4-6
RAINFALL HYETOGRAPH
GROUNDWATER ELEVATION
STREAM W.S. ELEVATION
RUNOFF HYDROGRAPH
STATION: DM 2 NOV. '85
WATER SURFACE ELEVATION (FEET-MSL)
DAYS

FIGURE E.4-11



DAMES & MOORE
DAYS
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31
RAINFALL (INCHES)
0.00 0.20 0.40 0.60 0.80 1.00 1.20 1.40 1.60 1.80 2.00
SEE NOTE FIGURE E.4-6
RAINFALL HYETOGRAPH
GROUNDWATER ELEVATION
STREAM W.S. ELEVATION
RUNOFF HYDROGRAPH
STATION: DM2 DEC. '85
WATER SURFACE ELEVATION (FEET-MSL)
30.00 29.00 28.00 27.00 26.00 25.00 24.00 23.00 22.00 21.00 20.00
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31
DAYS

FIGURE E.4-12

DAMES & MOORE
DAYS
RAINFALL (INCHES)
NOTE: FOR ALL STATION DM 3 HYETOGRAPHS. RAINFALL AMOUNTS ARE PERIODIC TOTALS OF MULTIPLE EVENTS, AND NOT DISCRETE EVENTS.
2.22"
3.85"
RAINFALL HYETOGRAPH
GROUNDWATER DEPTH
STREAM DISCHARGE (NO FLOW)
RUNOFF HYDROGRAPH
STATION: DM 3 JUN. '85
STREAM INVERT IS 3.5 FEET BELOW NATURAL GROUND SURFACE AT WELL BH-1
DEPTH TO GROUNDWATER (FEET)
DISCHARGE (CFS)
DAYS

FIGURE E.4-13


DAMES & MOORE
DAYS
RAINFALL (INCHES)
SEE NOTE FIGURE E.4-13
4.05"
RAINFALL HYETOGRAPH
GROUNDWATER DEPTH
STREAM DISCHARGE (NO FLOW)
RUNOFF HYDROGRAPH
STATION: DM 3 JUL. '85
DEPTH TO GROUNDWATER (FEET)
DISCHARGE (CFS)
DAYS
FIGURE E.4-14


DAMES & MOORE
DAYS
RAINFALL (INCHES)
SEE NOTE FIGURE E.4-13
2.77"
RAINFALL HYETOGRAPH
GROUNDWATER DEPTH
STREAM DISCHARGE
RUNOFF HYDROGRAPH
STATION: DM 3 AUG '85
SCS RUNOFF CURVE NUMBER (C_N)= 74.6
NO FLOW
DEPTH TO GROUNDWATER (FEET)
DISCHARGE (CFS)
DAYS

FIGURE E.4-15

DAMES & MOORE
DAYS
RAINFALL (INCHES)
4.20"
SEE NOTE FIGURE E.4-13
RAINFALL HYETOGRAPH
RUNOFF HYDROGRAPH
STATION: DM 3 SEP. '85
(2) >10 CFS
(2) >10 CFS
DEPTH TO GROUNDWATER (FEET)
DISCHARGE (CFS)
GROUNDWATER DEPTH
STREAM DISCHARGE
(2) STREAM STAGE HEIGHT EXCEEDS RECORDER RANGE
SCS RUNOFF CURVE NUMBER (C_N) = 73.5
C_N=70.4
DAYS

FIGURE E.4-16

DAMES & MOORE
DAYS
RAINFALL (INCHES)
SEE NOTE FIGURE E.4-13
RAINFALL HYETOGRAPH
GROUNDWATER DEPTH
STREAM DISCHARGE
RUNOFF HYDROGRAPH
STATION: DM 3 OCT '85
DEPTH TO GROUNDWATER (FEET)
DISCHARGE (CFS)
NO FLOW
DAYS

FIGURE E.4-17

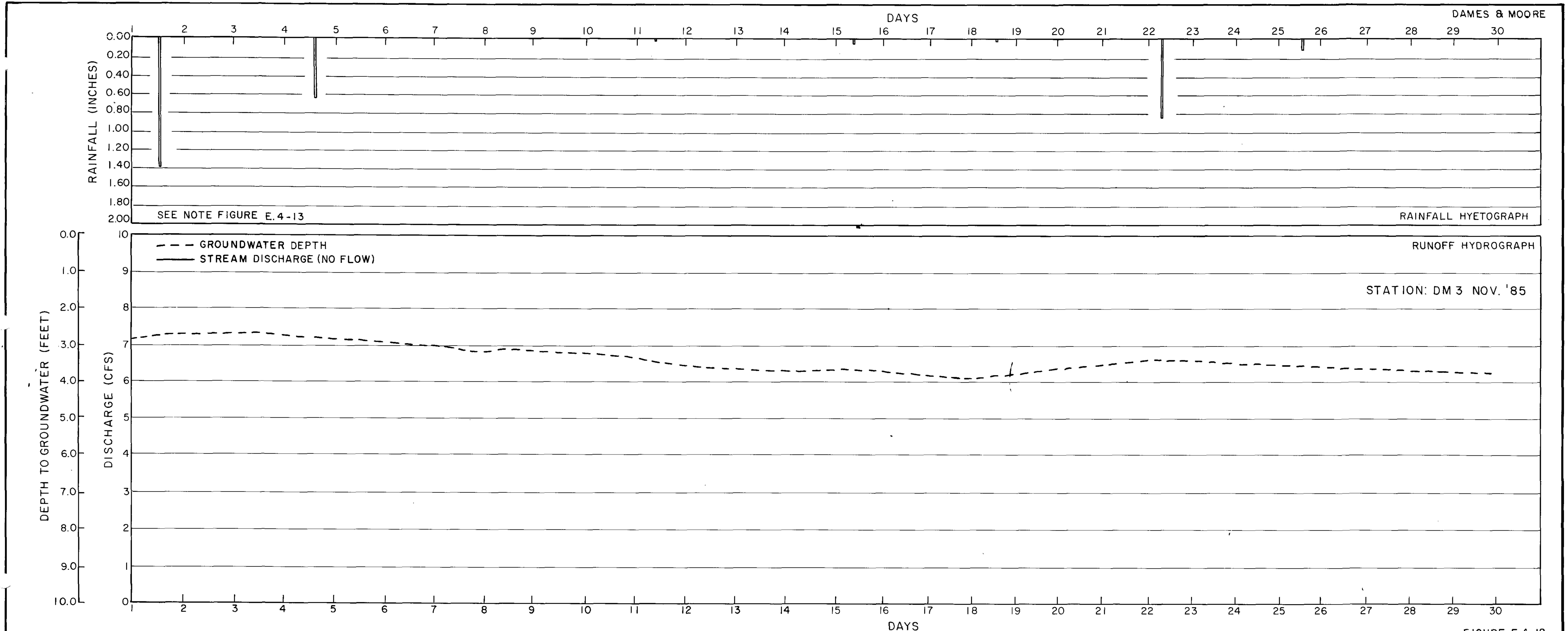

DAMES & MOORE
DAYS
RAINFALL (INCHES)
SEE NOTE FIGURE E.4-13
RAINFALL HYETOGRAPH
GROUNDWATER DEPTH
STREAM DISCHARGE (NO FLOW)
RUNOFF HYDROGRAPH
STATION: DM 3 NOV. '85
DEPTH TO GROUNDWATER (FEET)
DISCHARGE (CFS)
DAYS

FIGURE E.4-18


DAMES & MOORE
DAYS
RAINFALL (INCHES)
SEE NOTE FIGURE E.4-13
RAINFALL HYETOGRAPH
RUNOFF HYDROGRAPH
GROUNDWATER DEPTH
STREAM DISCHARGE (NO FLOW)
STATION: DM 3 DEC. '85
DEPTH TO GROUNDWATER (FEET)
DISCHARGE (CFS)
DAYS
FIGURE E.4-19


DAMES & MOORE
DAYS
RAINFALL (INCHES)
NOTE: RAINFALL DATA REPRESENTS DISCRETE EVENTS MEASURED AT ONE HOUR INCREMENTS
RAINFALL HYETOGRAPH
GROUNDWATER DEPTH
STREAM DISCHARGE (NO FLOW)
RUNOFF HYDROGRAPH
STATION: DM 4 APR. '85
STREAM INVERT IS 2.6 FEET BELOW NATURAL GROUND SURFACE AT WELL ROMP 19 (WEST).
DEPTH TO GROUNDWATER (FEET)
DISCHARGE (CFS)
DAYS

FIGURE E.4-20


DAMES & MOORE
DAYS
RAINFALL (INCHES)
SEE NOTE FIGURE E.4-20
RAINFALL HYETOGRAPH
GROUNDWATER DEPTH
STREAM DISCHARGE (NO FLOW)
RUNOFF HYDROGRAPH
STATION: DM 4 MAY '85
DEPTH TO GROUNDWATER (FEET)
DISCHARGE (CFS)
DAYS
FIGURE E.4-21

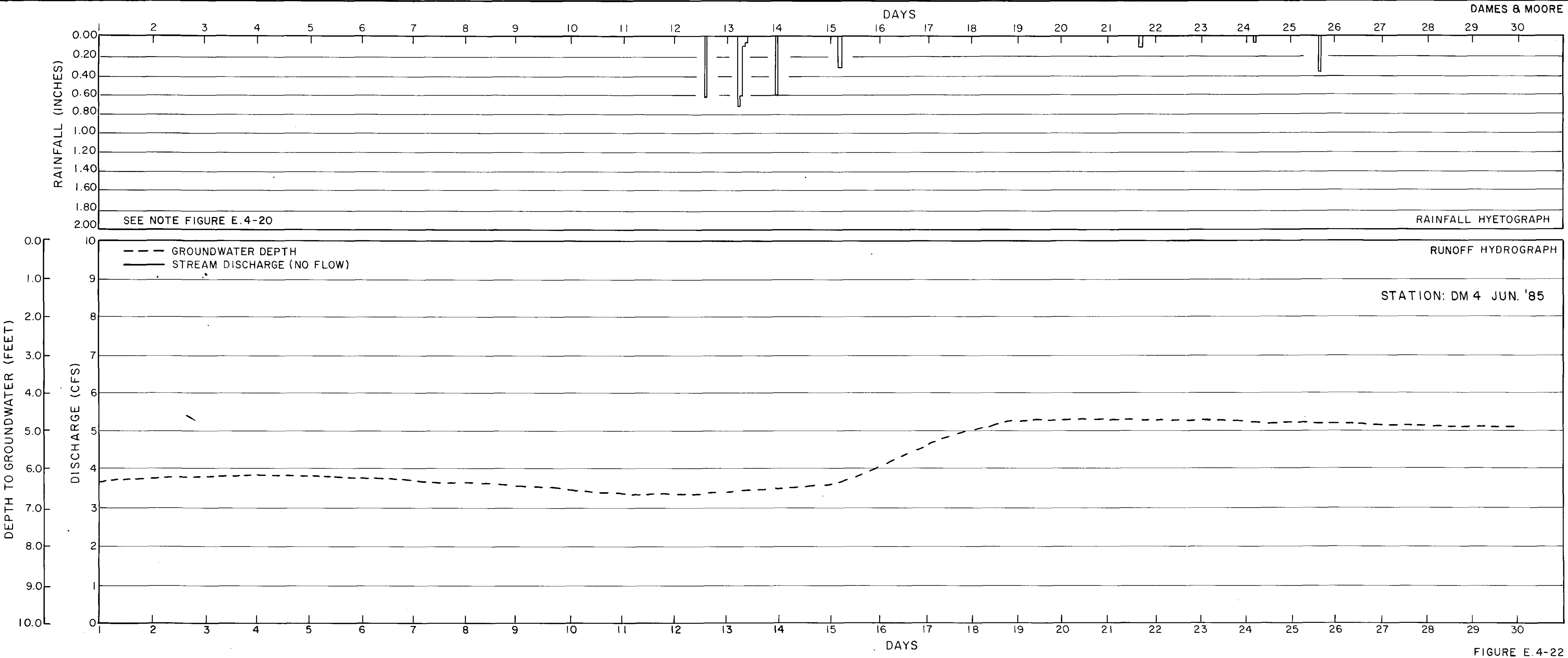
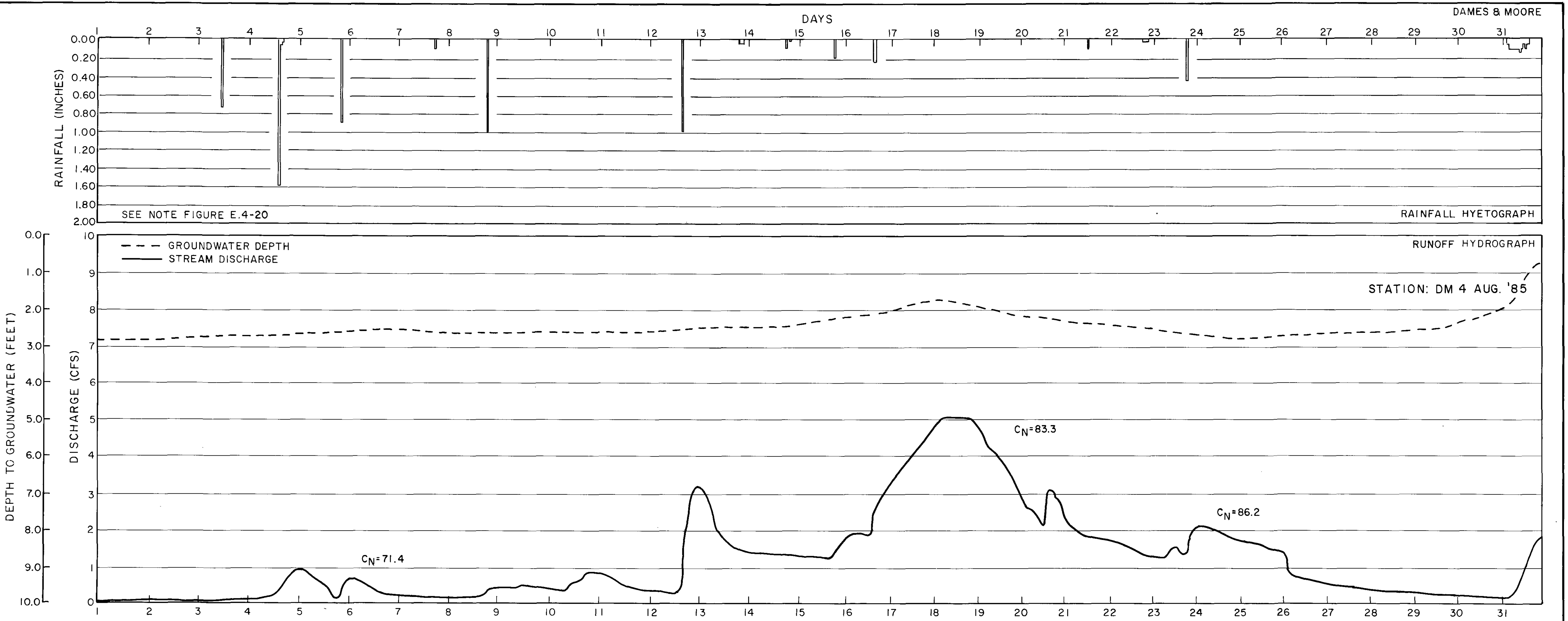

DAMES & MOORE
DAYS
RAINFALL (INCHES)
0.00
0.20
0.40
0.60
0.80
1.00
1.20
1.40
1.60
1.80
2.00
SEE NOTE FIGURE E.4-20
RAINFALL HYETOGRAPH
GROUNDWATER DEPTH
STREAM DISCHARGE (NO FLOW)
RUNOFF HYDROGRAPH
STATION: DM 4 JUN. '85
DEPTH TO GROUNDWATER (FEET)
DISCHARGE (CFS)
DAYS

FIGURE E.4-22


DAMES & MOORE
DAYS
RAINFALL (INCHES)
(1) RAINFALL AMOUNTS ARE PERIODIC TOTALS OF MULTIPLE EVENTS. BELFORT RAIN GAGE NOT RECORDING DURING THIS WEEK.
SEE NOTE FIGURE E.4-20
RAINFALL HYETOGRAPH
RUNOFF HYDROGRAPH
GROUNDWATER DEPTH
STREAM DISCHARGE (NO FLOW)
STATION: DM 4 JUL. '85
DEPTH TO GROUNDWATER (FEET)
DISCHARGE (CFS)
DAYS
FIGURE E.4-23



DAMES & MOORE
DAYS
RAINFALL (INCHES)
0.00
0.20
0.40
0.60
0.80
1.00
1.20
1.40
1.60
1.80
2.00
SEE NOTE FIGURE E.4-20
RAINFALL HYETOGRAPH
GROUNDWATER DEPTH
STREAM DISCHARGE
RUNOFF HYDROGRAPH
STATION: DM 4 AUG. '85
DEPTH TO GROUNDWATER (FEET)
DISCHARGE (CFS)
$C_N = 71.4$
$C_N = 83.3$
$C_N = 86.2$
DAYS

FIGURE E.4-24

DAMES & MOORE
DAYS
RAINFALL (INCHES)
SEE NOTE FIGURE E.4-20
RAINFALL HYETOGRAPH
RUNOFF HYDROGRAPH
GROUNDWATER DEPTH
STREAM DISCHARGE
STATION: DM 4 SEP. '85
C_N=73.0
C_N=82.0
C_N=65.0
DEPTH TO GROUNDWATER (FEET)
DISCHARGE (CFS)
DAYS

FIGURE E.4-25


DAMES & MOORE
DAYS
RAINFALL (INCHES)
SEE NOTE FIGURE E.4-20
RAINFALL HYETOGRAPH
GROUNDWATER DEPTH (DATA FROM WELL ROMP 19 (WEST) NOT AVAILABLE)
STREAM DISCHARGE
RUNOFF HYDROGRAPH
STATION: DM 4 OCT. '85
DEPTH TO GROUNDWATER (FEET)
DISCHARGE (CFS)
NO FLOW
DAYS
FIGURE E.4-26

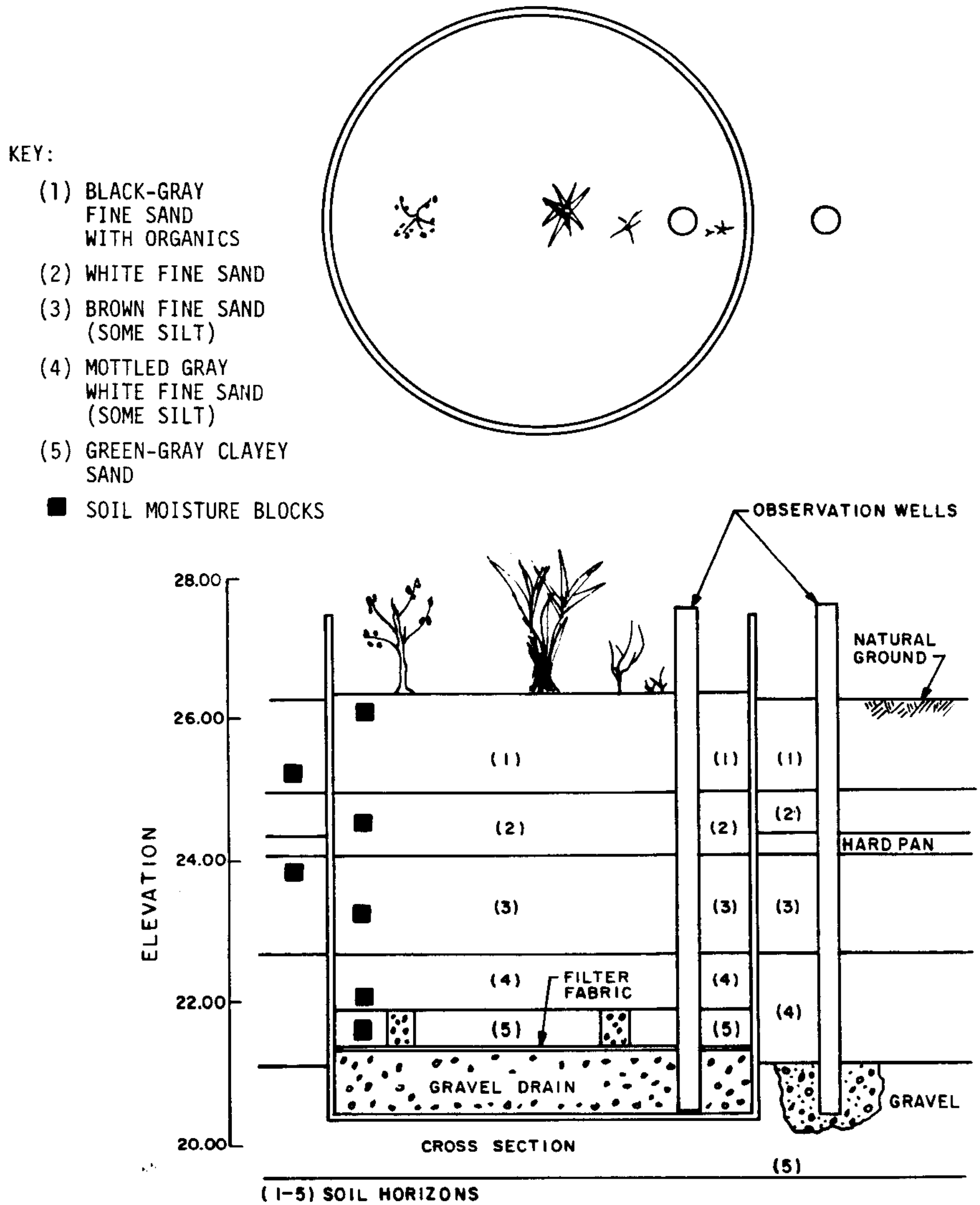


Figure E.5-1. DM 3 Dry Prairie Evapotranspirometer Schematic.

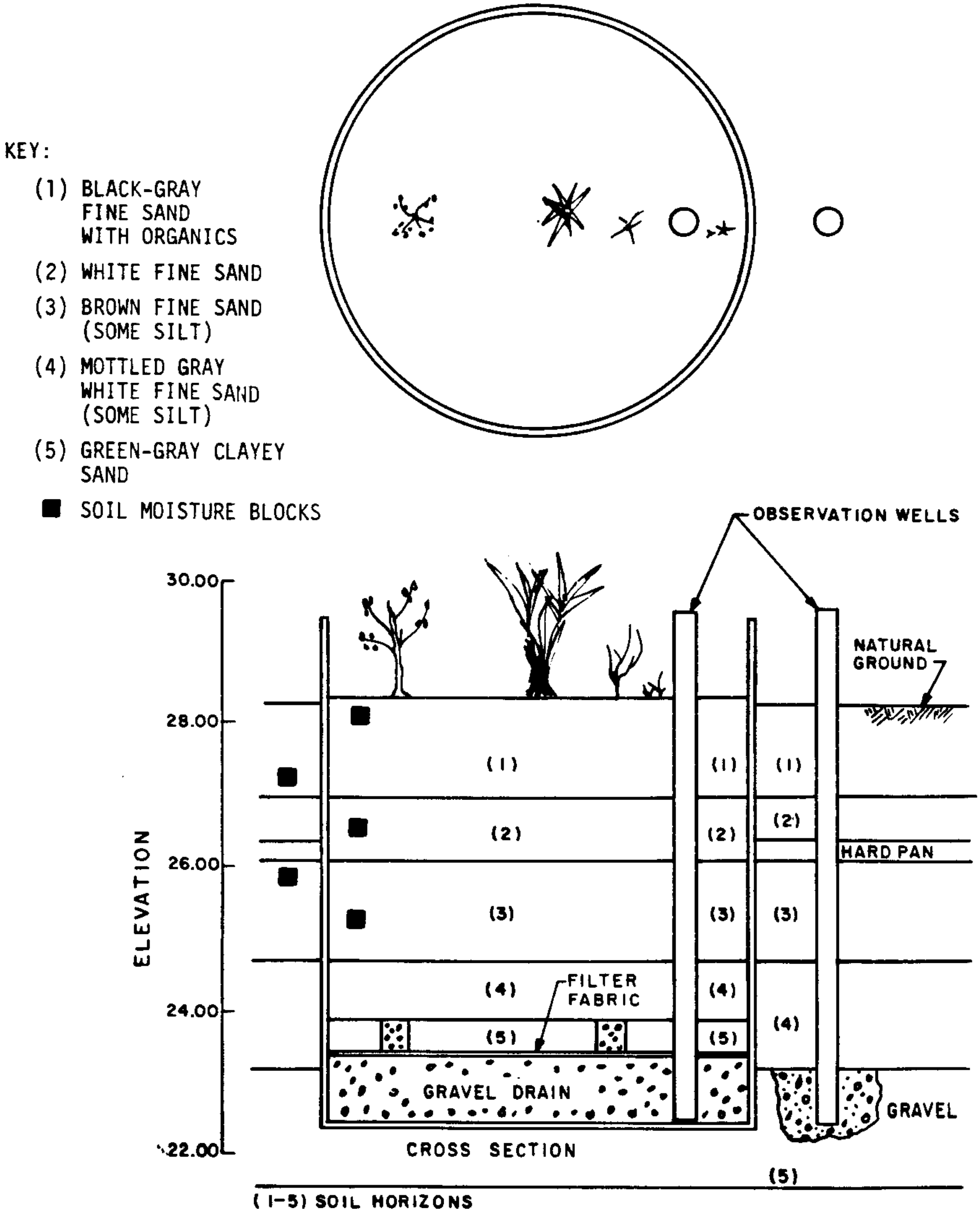


Figure E.5-2. Pine Flatwood Evapotranspirometer Installation Schematic.

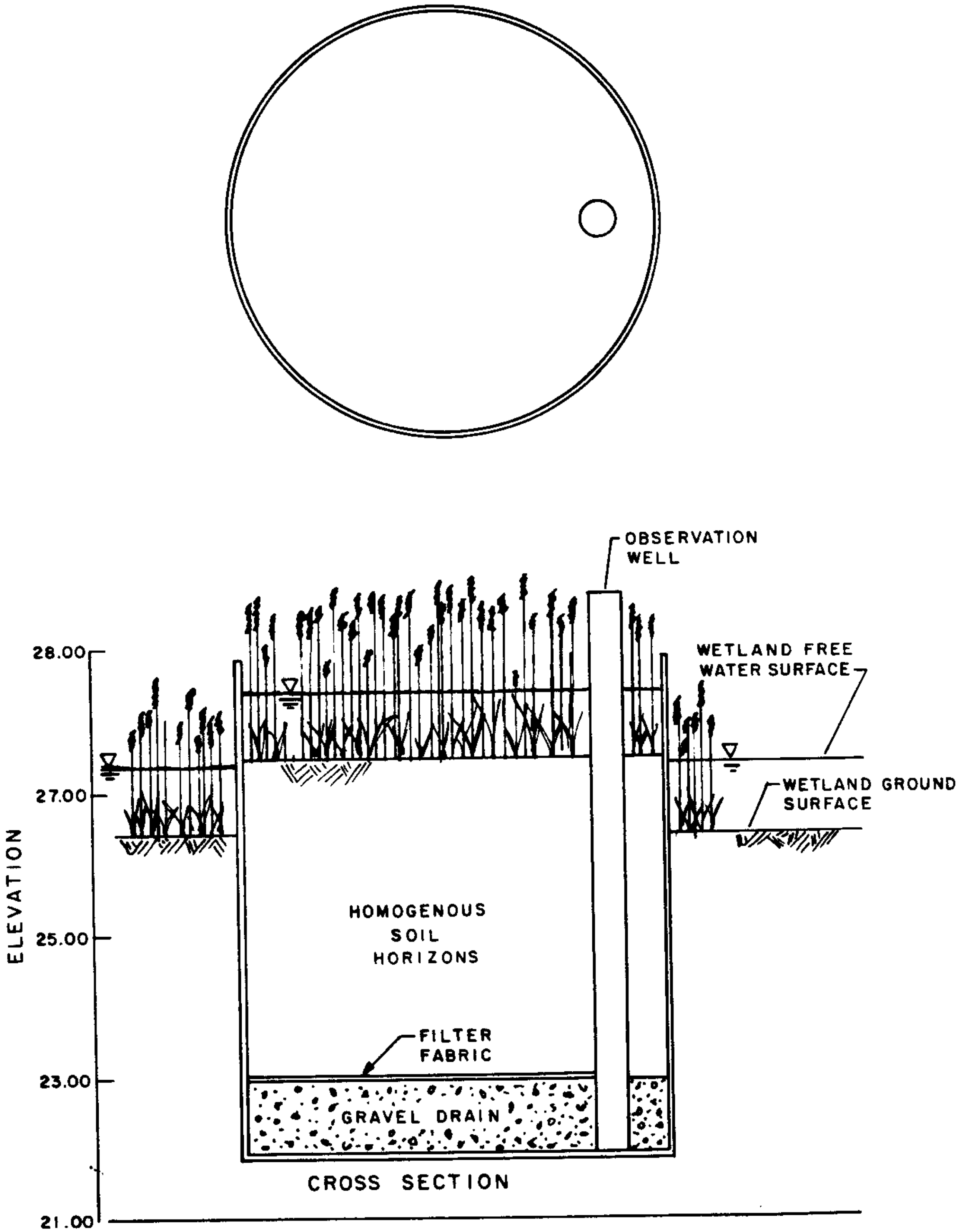
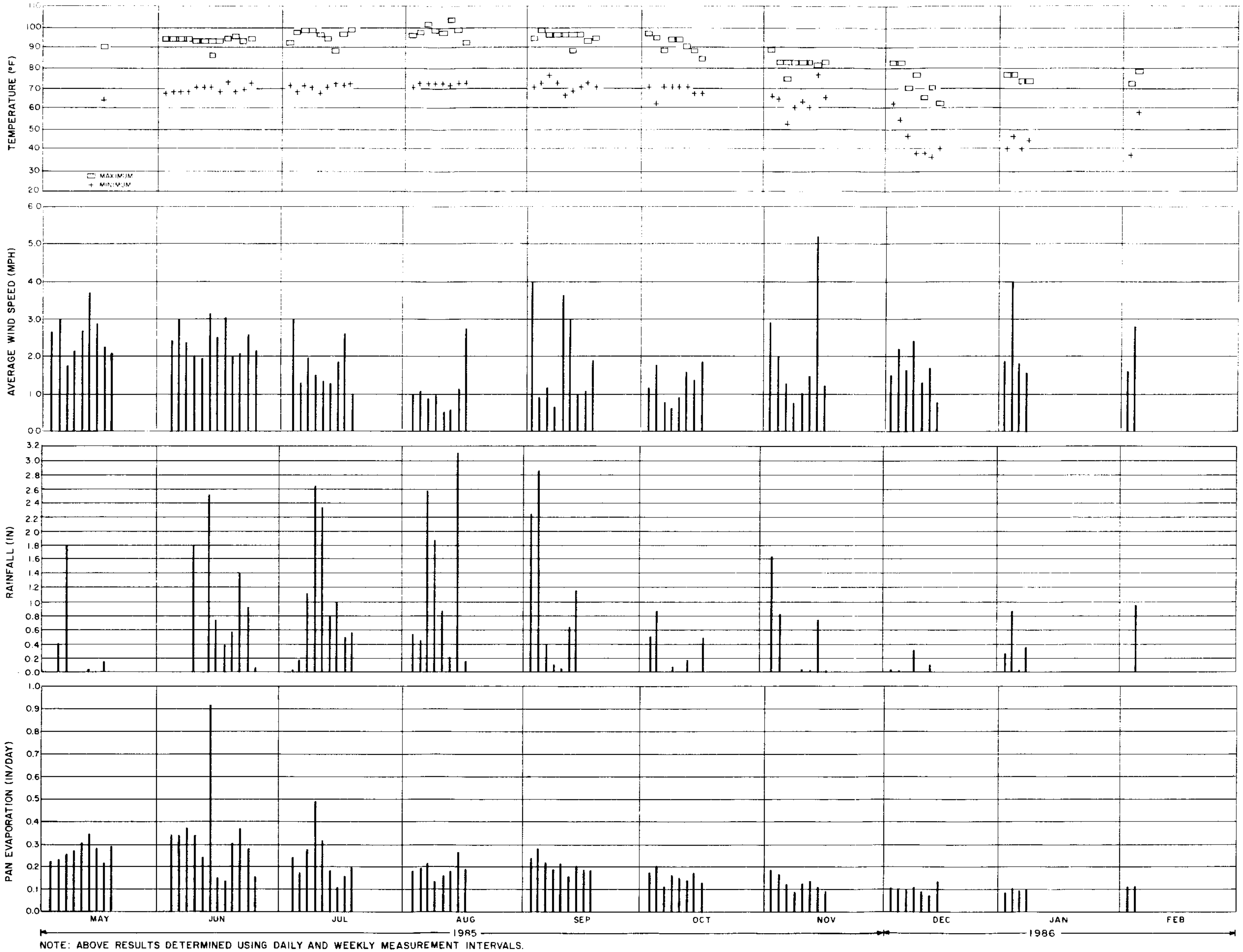
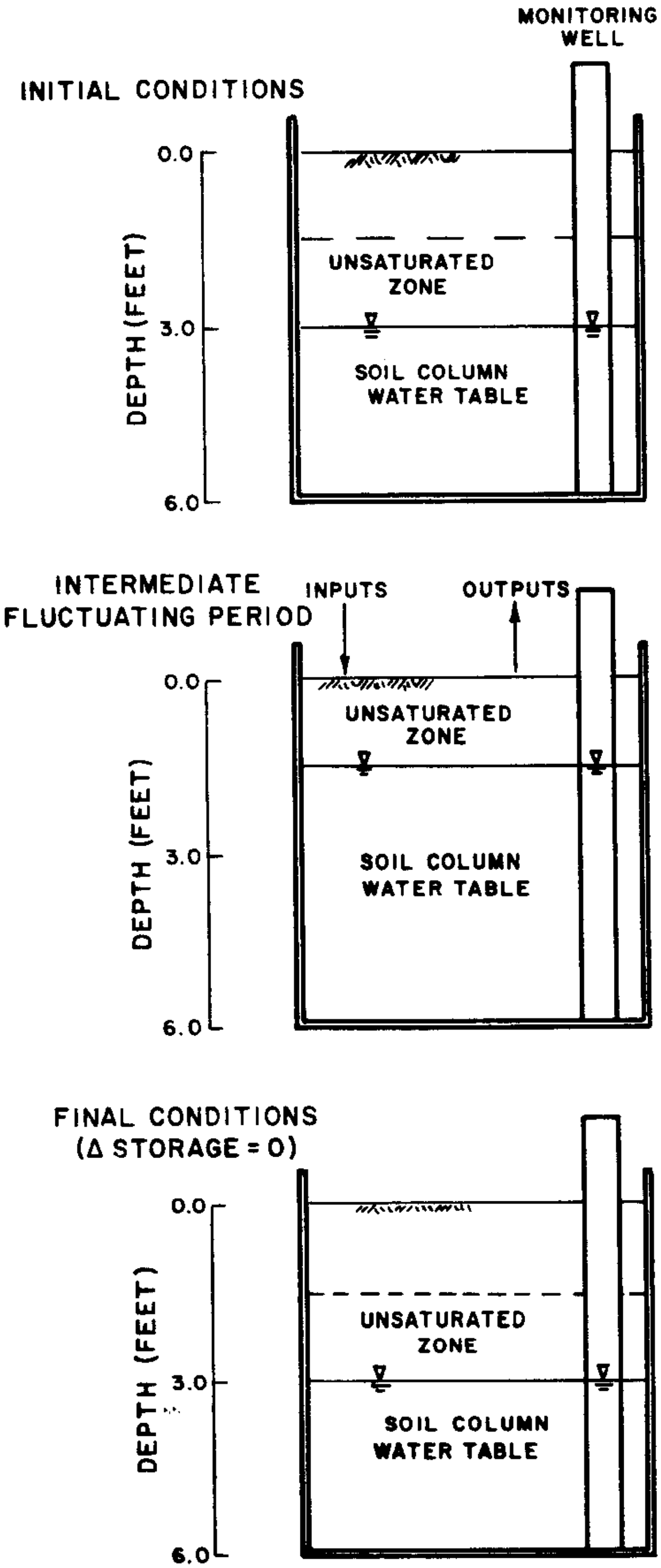


Figure E.5-3. DM 2 Wetland Evapotranspirometer Installation Schematic.



DAMES & MOORE

Figure E.5-4. DM5 National Weather Service Class A Evaporation Pan Results.



NOTE: 1. It is assumed that the unsaturated zone tracks with the soil column water table.

2. Because the final conditions are equivalent to initial conditions the change in storage is assumed to be zero.

Figure E.5-5. Estimation Procedure: Evapotranspiration Calculation.

Note: No measured data is available below a depth of 2.5 feet. See Section E.5.7.2 for assumption.

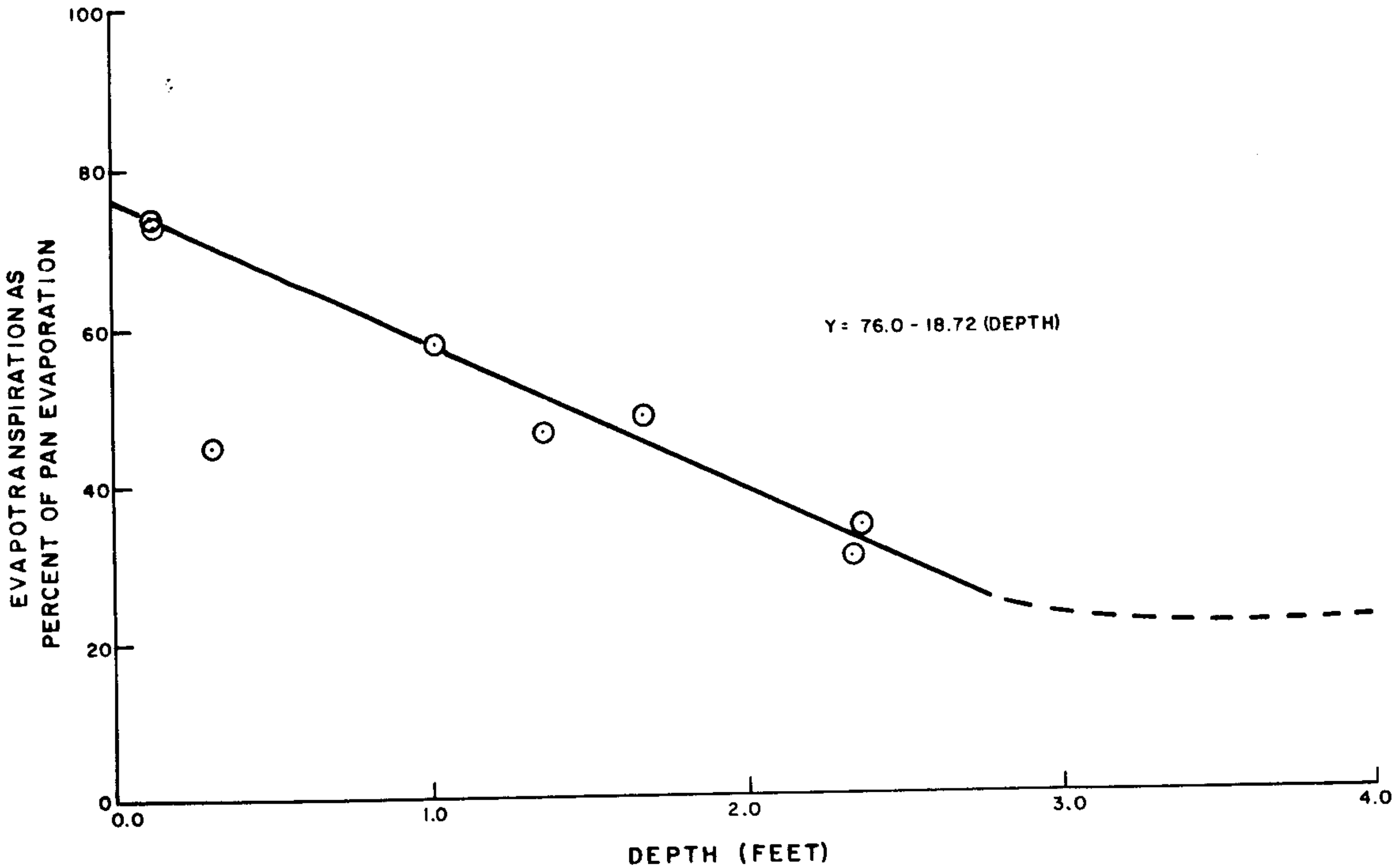


Figure E.5-6. Evapotranspiration Versus Depth Relationship Dry-Prairie.

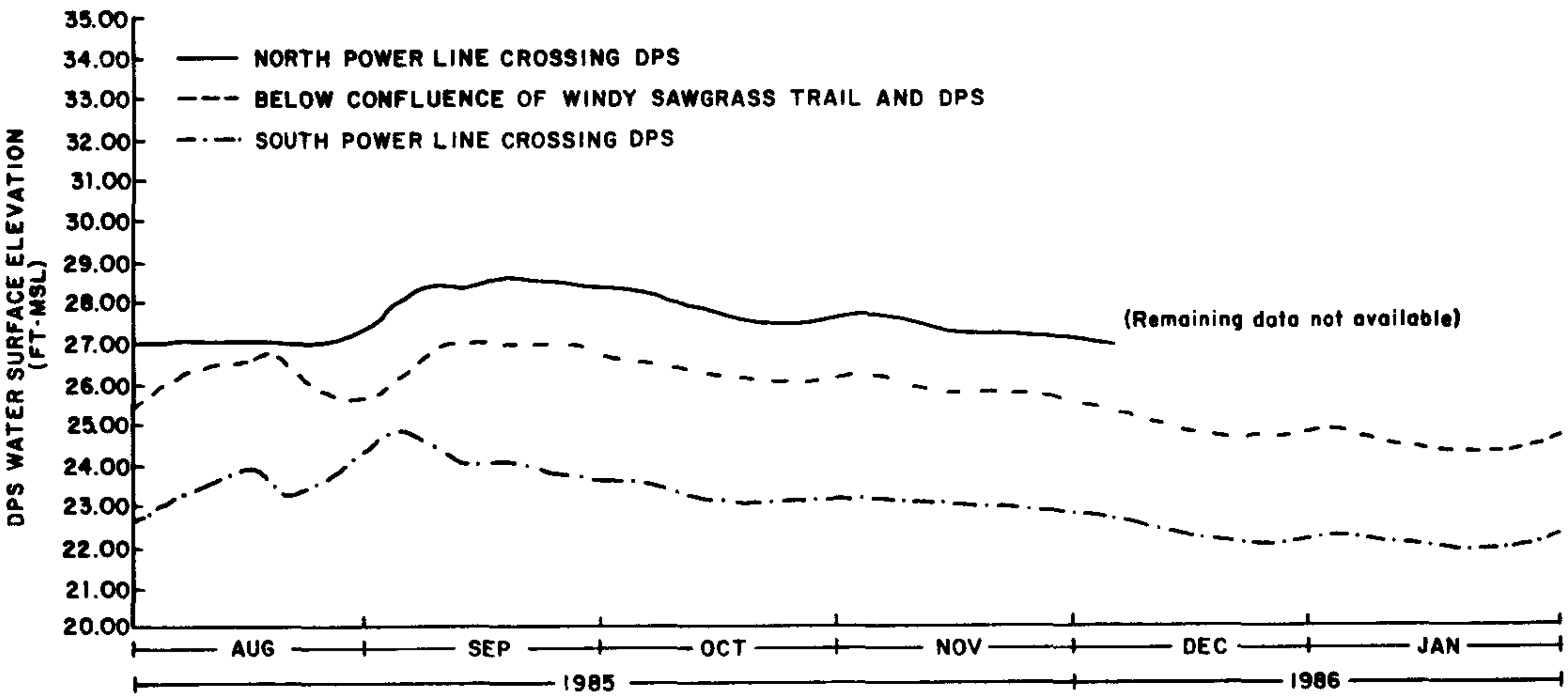
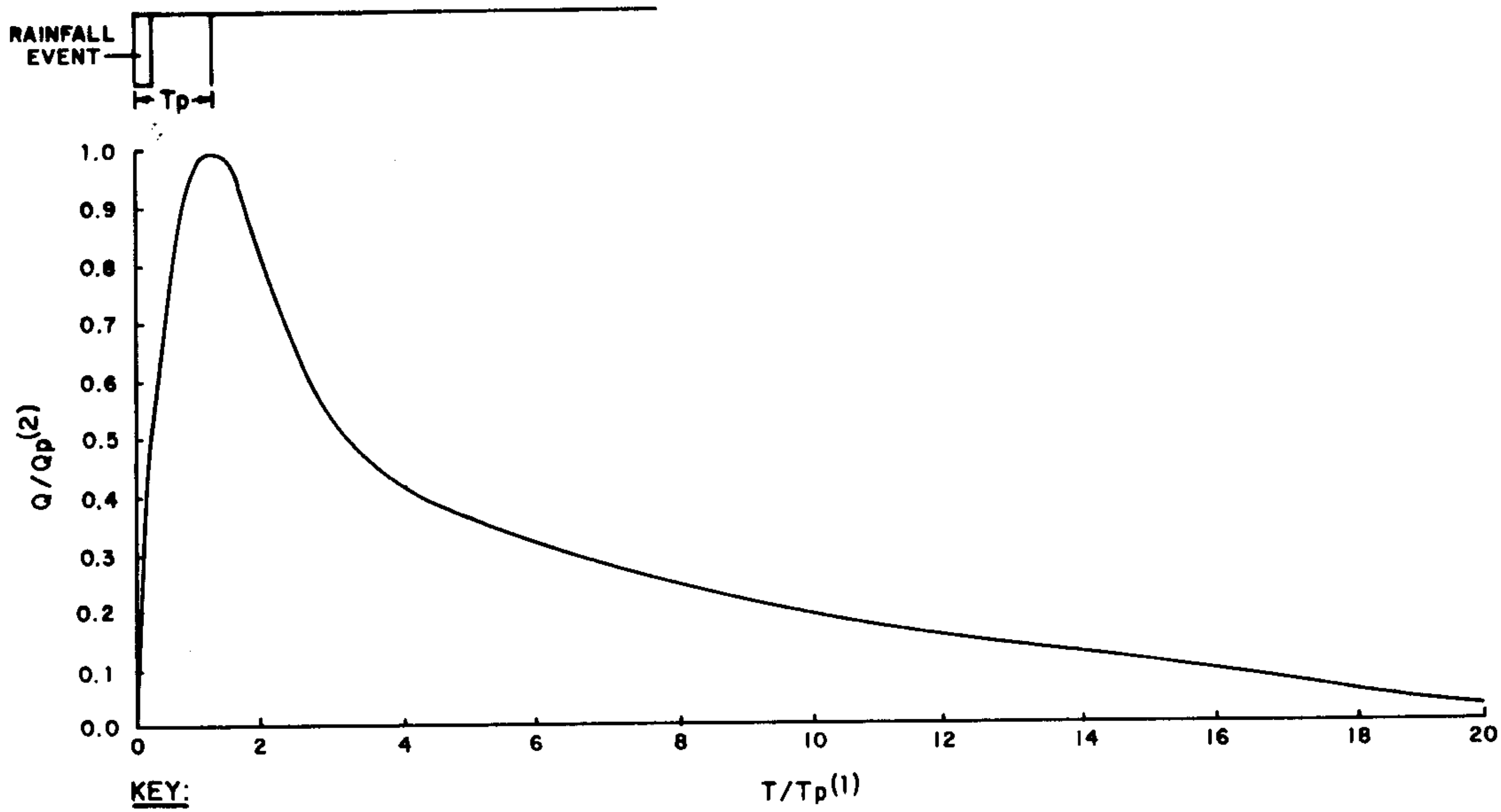


Figure E.6-1. Deer Prairie Slough Water Surface Elevation Data Summary.



KEY:

(1) T = TIME
Tp=TIME TO PEAK DISCHARGE

(2) Q = DISCHARGE
Qp=PEAK DISCHARGE

Figure E.6-2. RMR Dimensionless Unit Hydrograph.

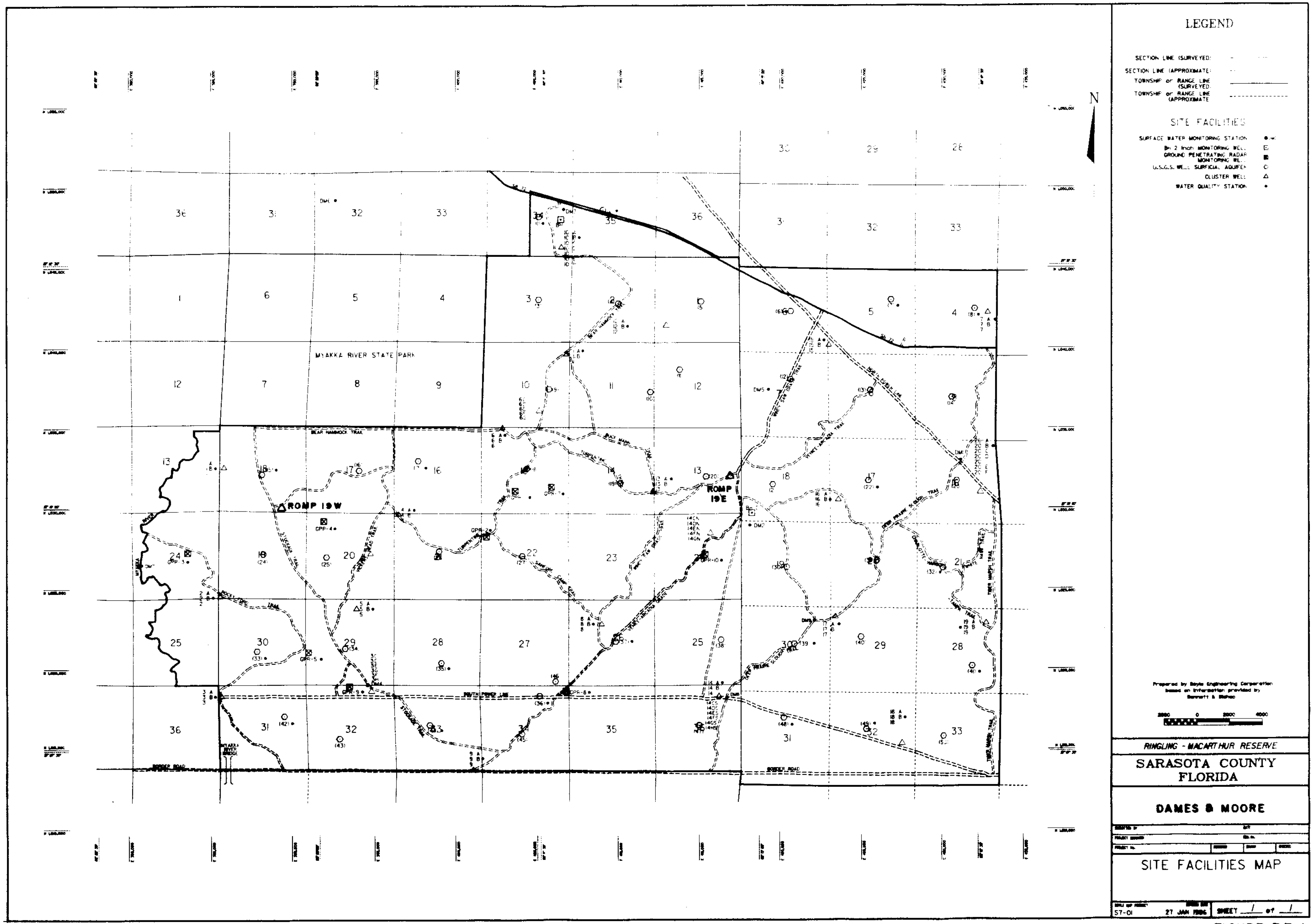
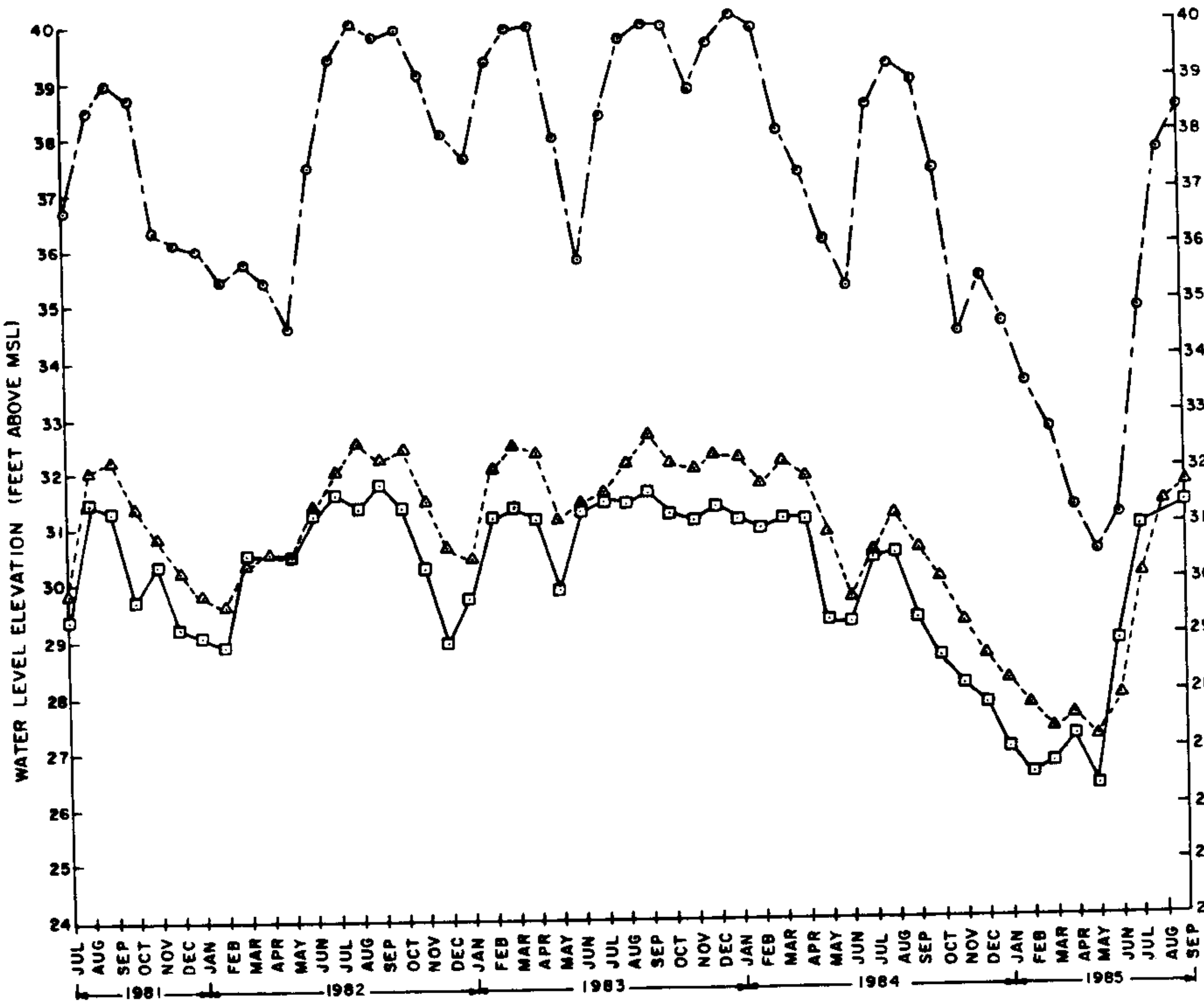

MYAKKA RIVER STATE PARK
ROMP 19W
ROMP 19E
LEGEND
SECTION LINE (SURVEYED)
SECTION LINE (APPROXIMATE)
TOWNSHIP or RANGE LINE (SURVEYED)
TOWNSHIP or RANGE LINE (APPROXIMATE)
SITE FACILITIES
SURFACE WATER MONITORING STATION
2 Inch MONITORING WELL
GROUND PENETRATING RADAR MONITORING WELL
U.S.G.S. WELL SURFICIAL AQUIFER
CLUSTER WELL
WATER QUALITY STATION
N
2000 0 2000 4000
RINGLING - MACARTHUR RESERVE
SARASOTA COUNTY
FLORIDA
DAMES & MOORE
SITE FACILITIES MAP
57-01
27 JAN
SHEET 1 of 1

FIGURE E.7-1



Key:
- ⊡ Surficial Aquifer Water Level Measurement
- △ Secondary Aquifer Water Level Measurement
- ⊙ Floridan Aquifer Water Level Measurement

Figure E.7-2. ROMP 19E Groundwater Hydrograph.

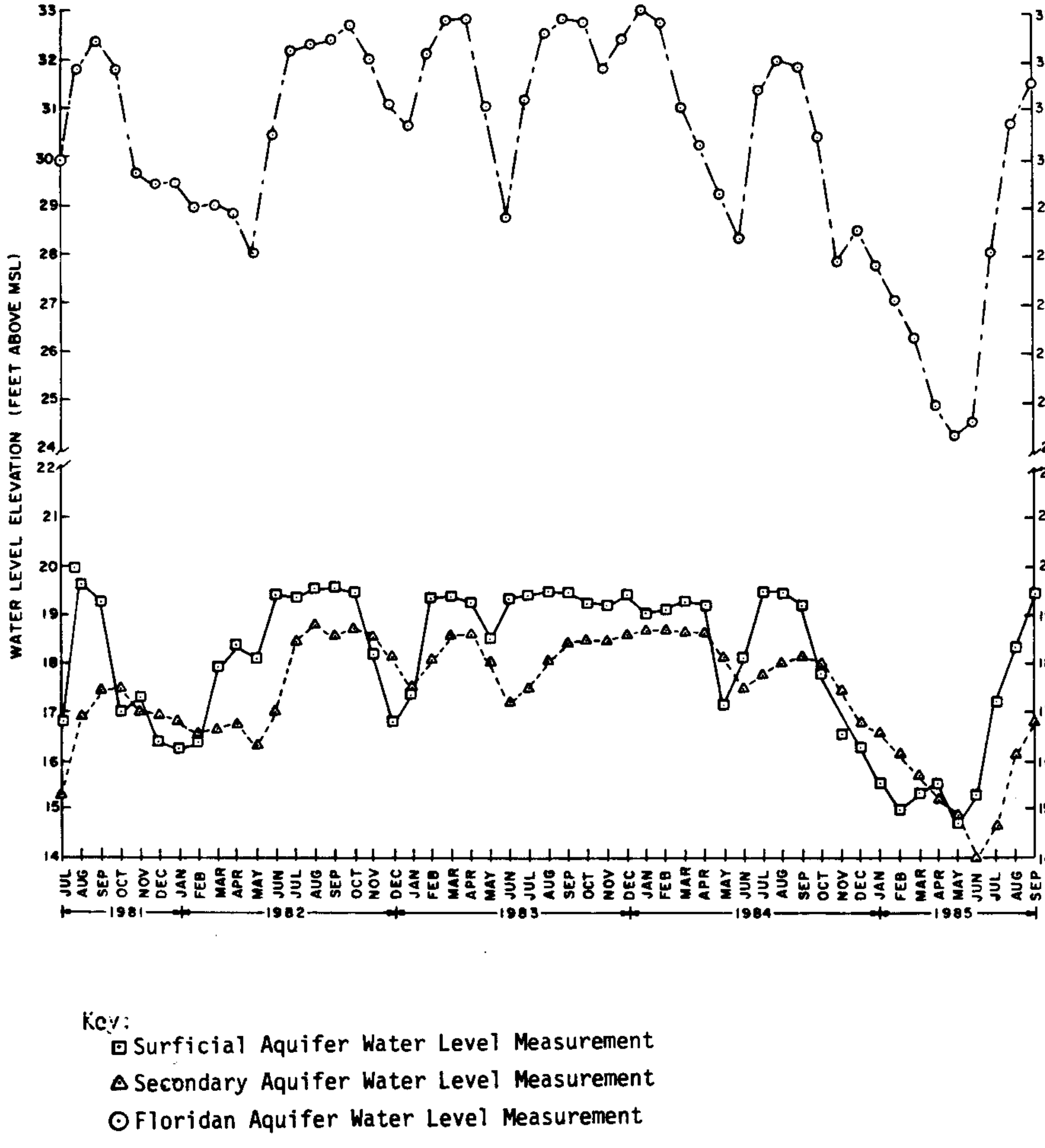


Figure E.7-3. ROMP 19W Groundwater Hydrograph.

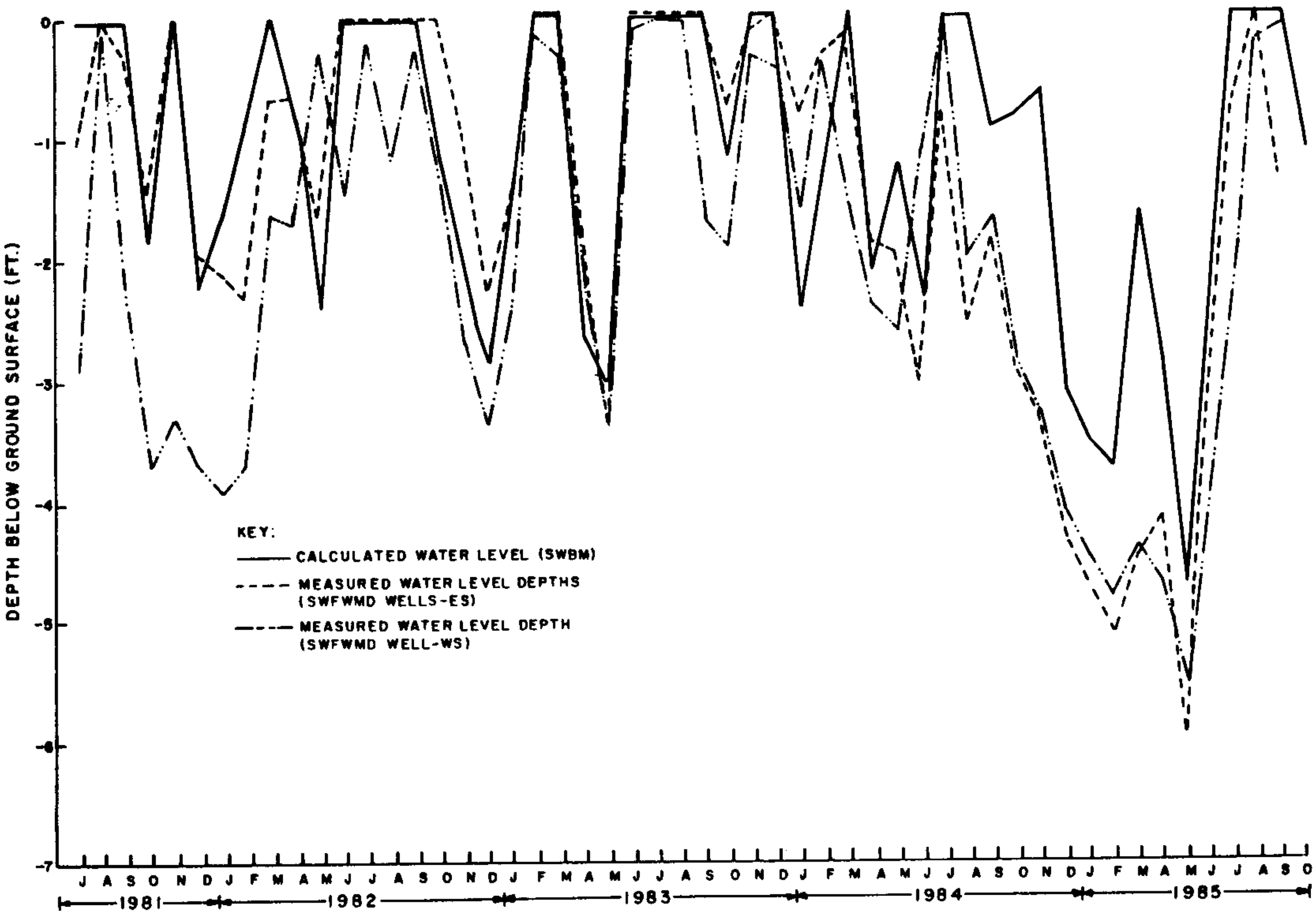


Figure E.7-4. Comparison of Well ES and WS Water Levels with Calculated Levels.

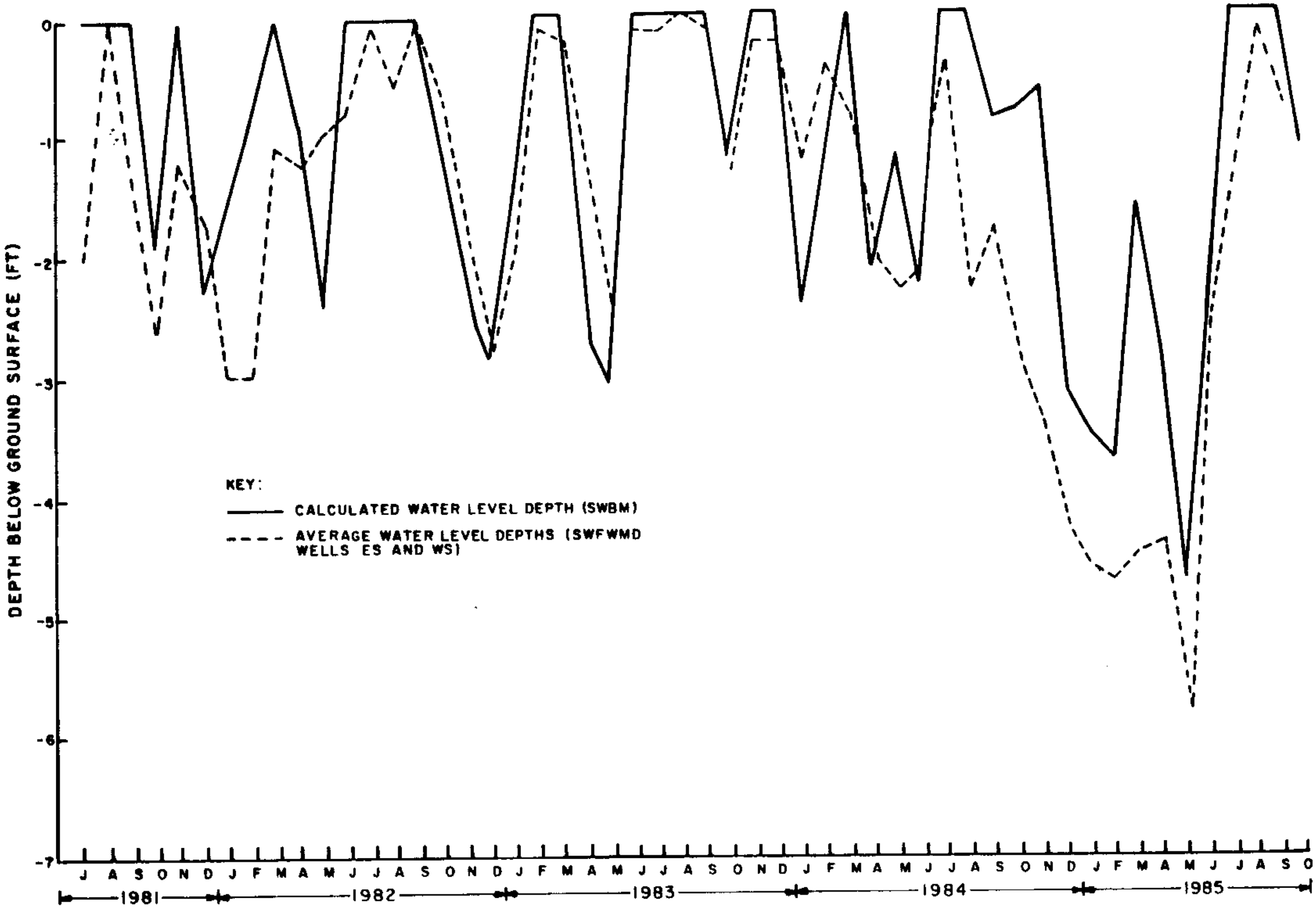


Figure E.7-5. Site Water Budget Model Calibration (SWBM).

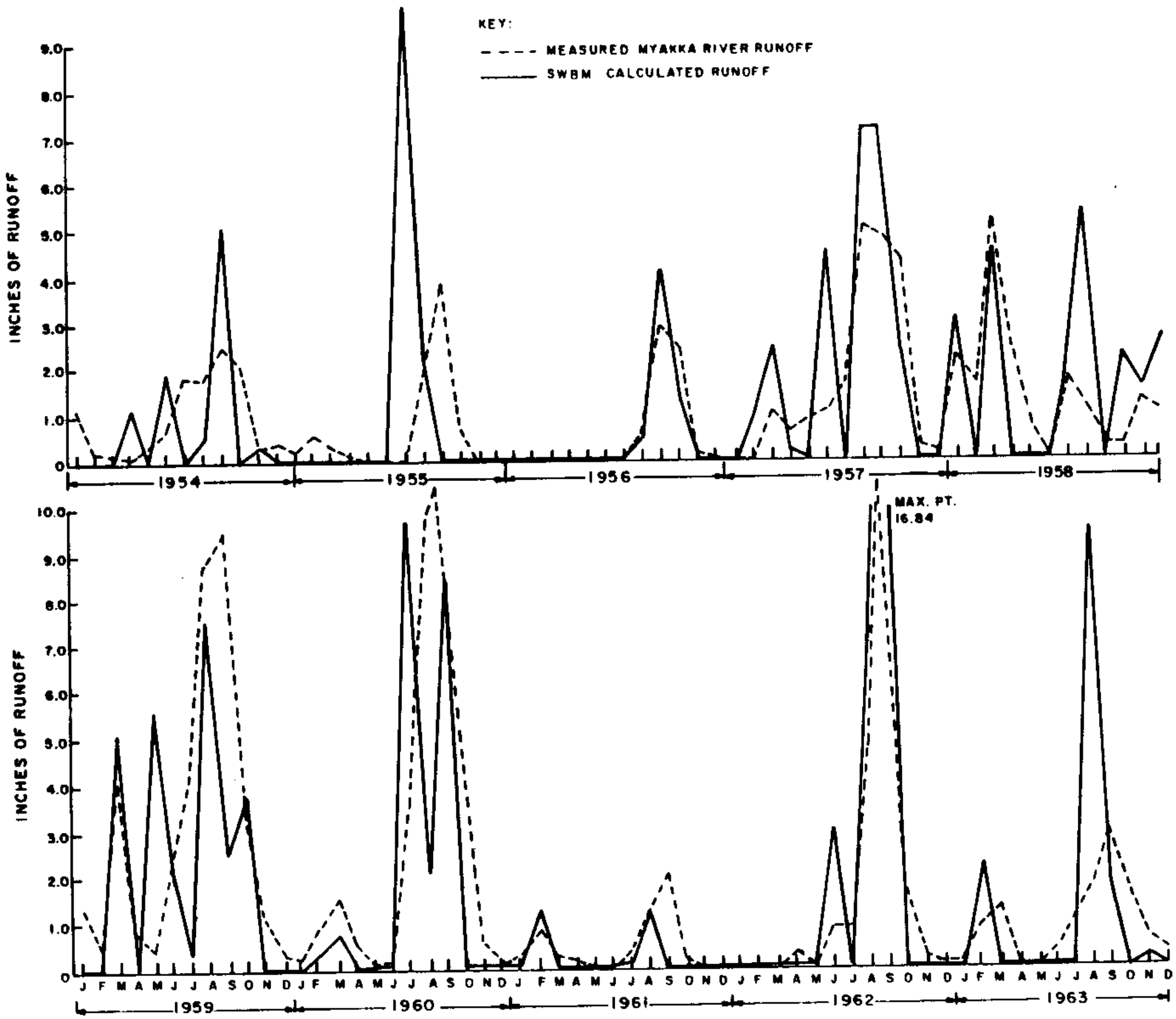


Figure E.7-6 (Continued) Site Water Budget Model Verification.

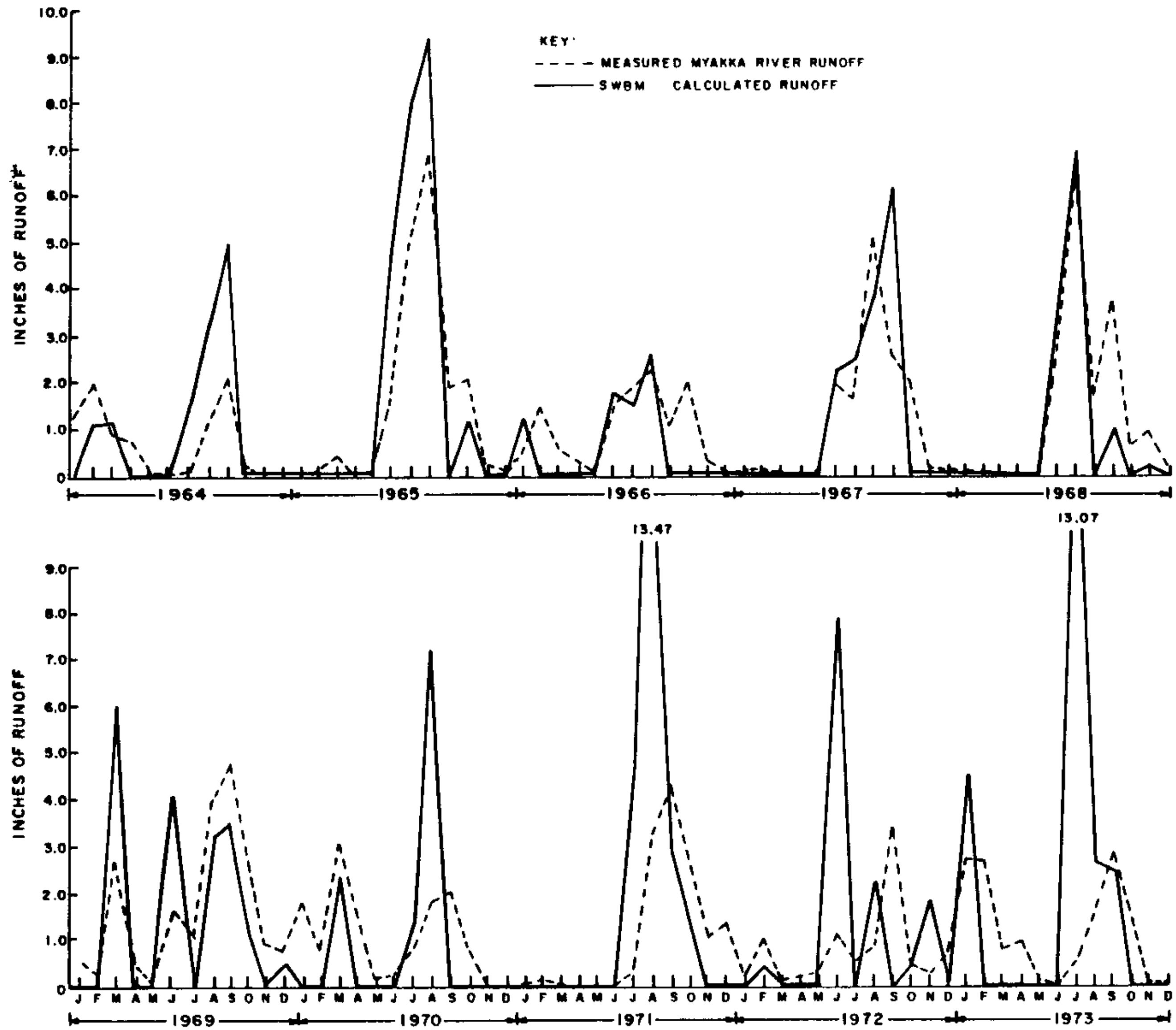


Figure E.7-6 (Continued). Site Water Budget Model Verification.

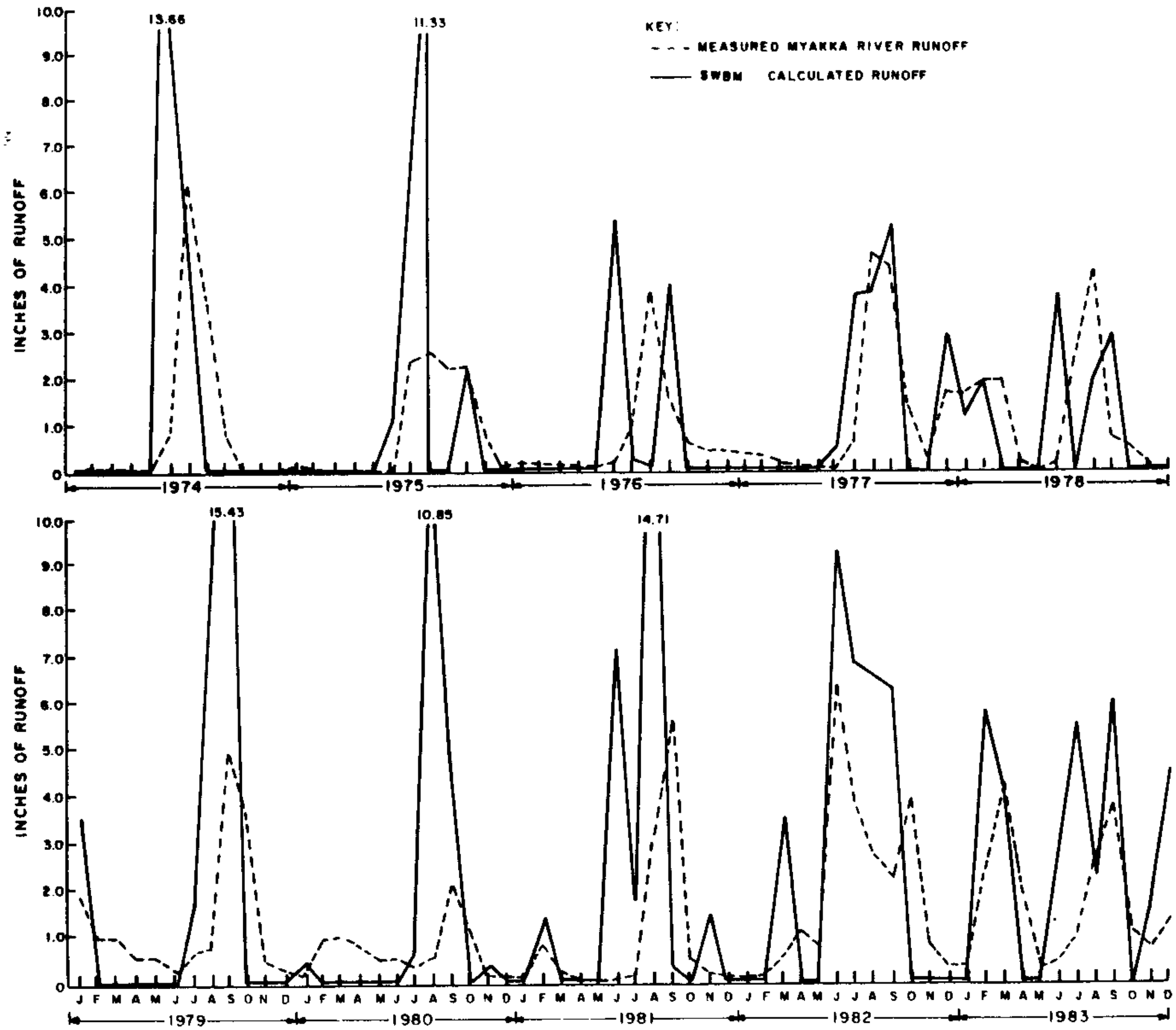


Figure E.7-6 (Continued). Site Water Budget Model Verification.

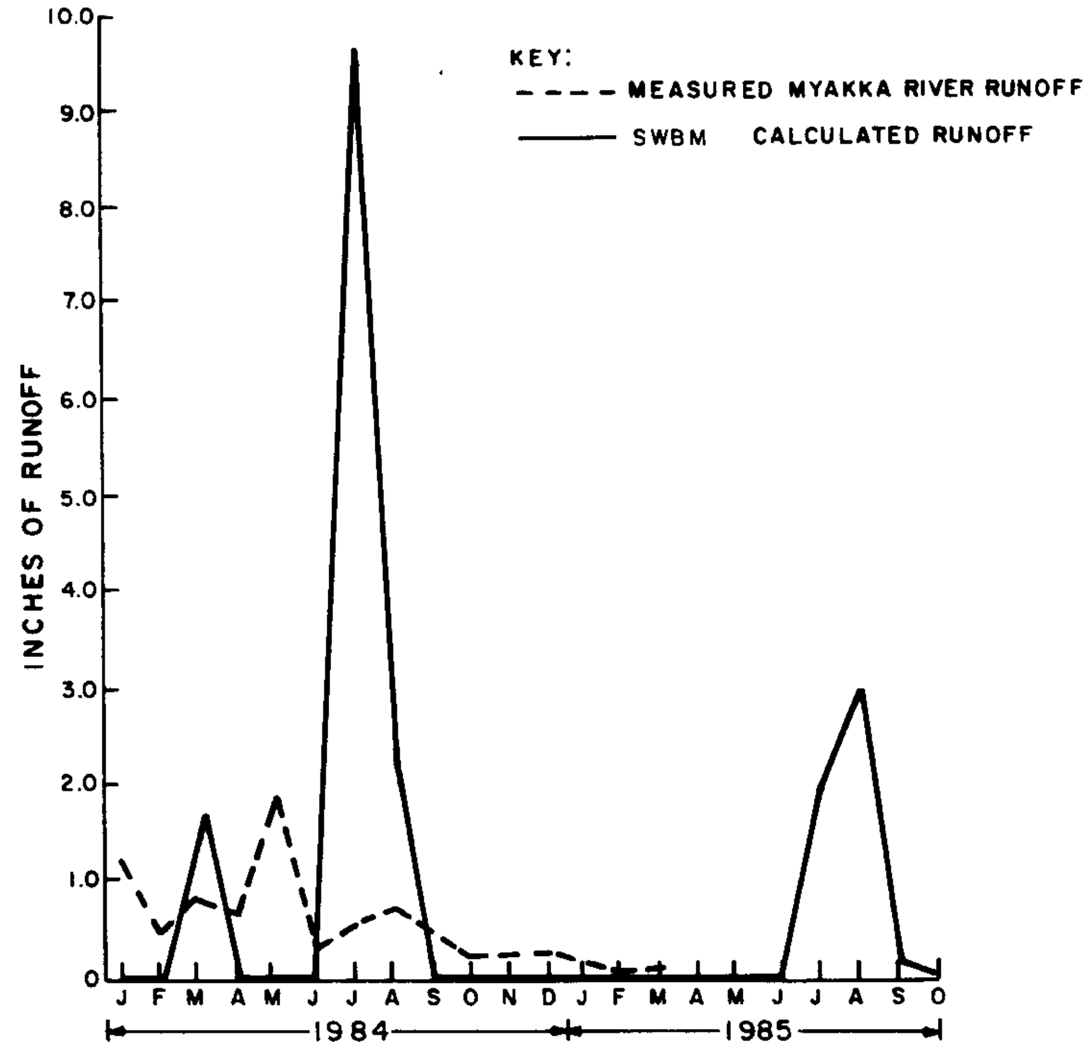


Figure E.7-6 (Continued). Site Water Budget Model Verification.

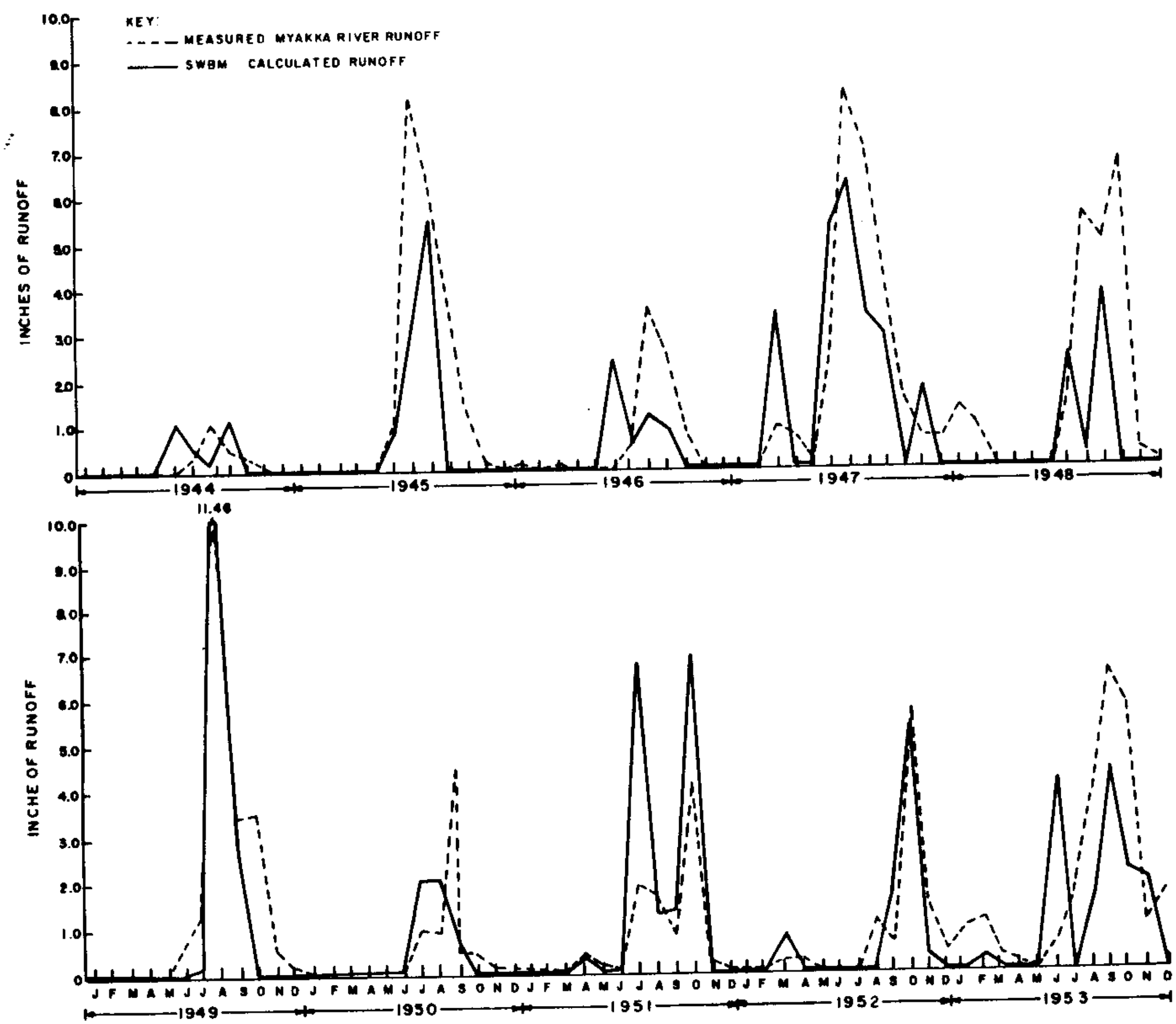


Figure E.7-6. Site Water Budget Model Verification.

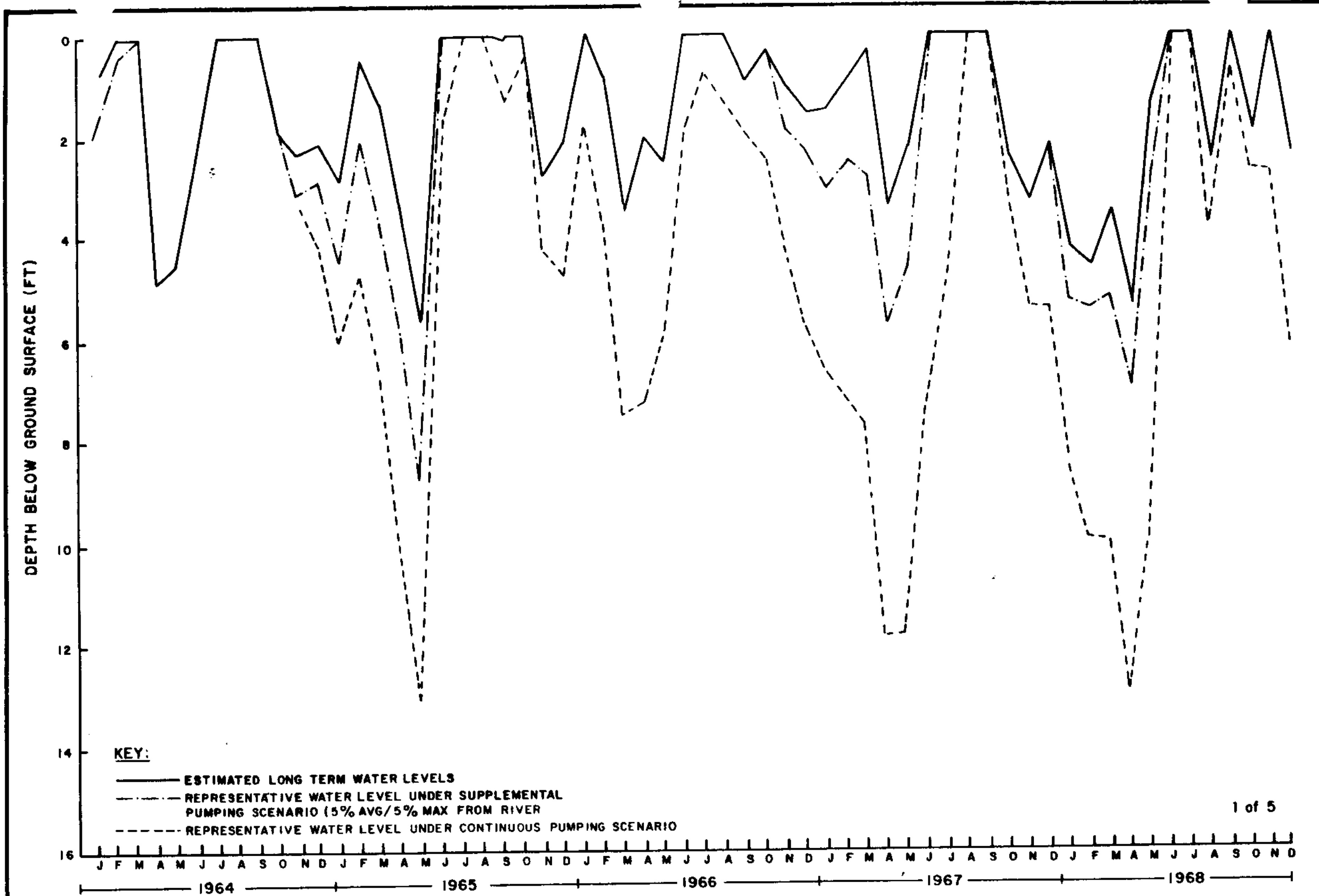


Figure E.7-7. Representative Water Level Changes Under Different Pumping Scenarios.

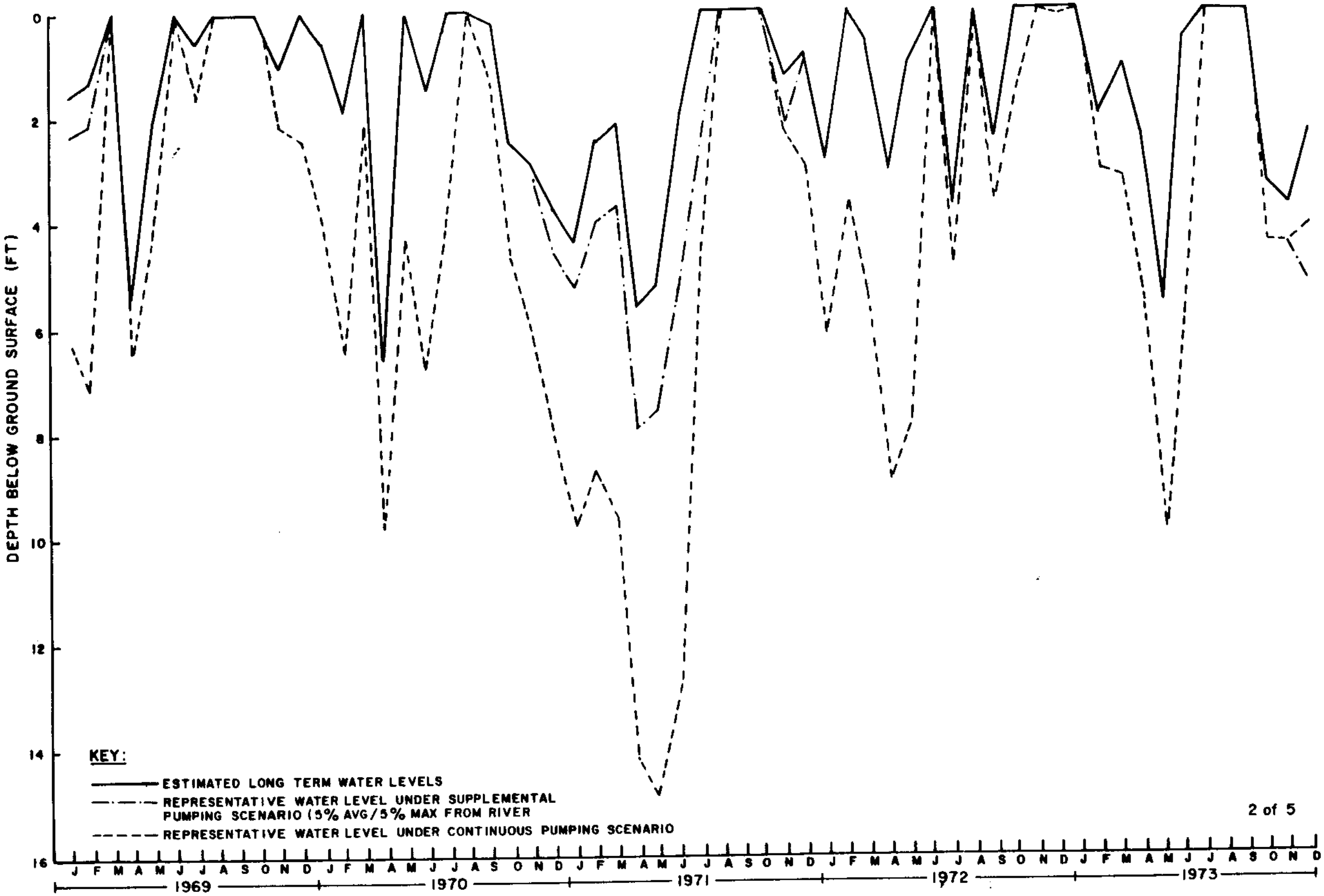


Figure E.7-7 (Continued). Representative Water Level Changes Under Different Pumping Scenarios.

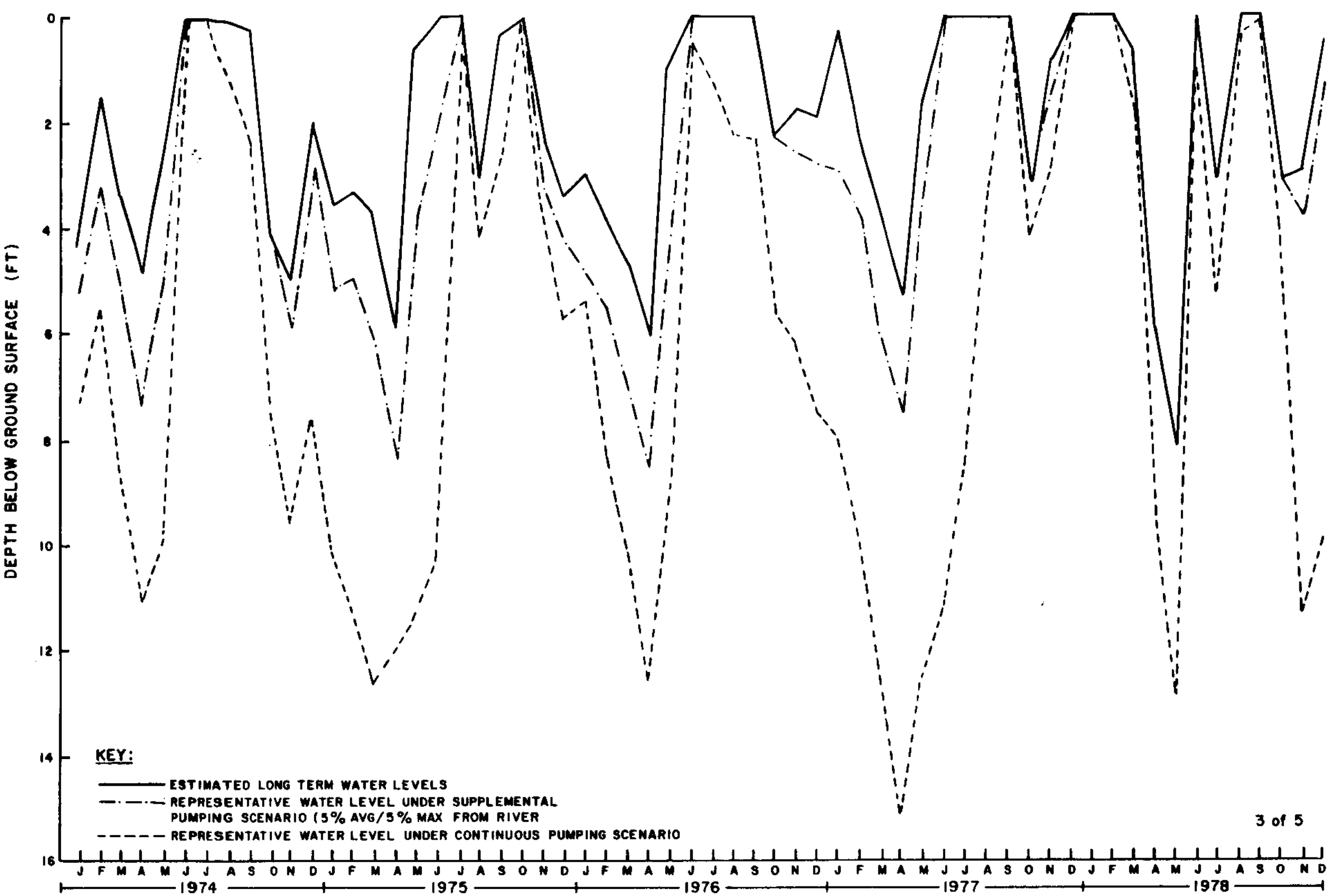


Figure E.7-7 (Continued). Representative Water Level Changes Under Different Pumping Scenarios.

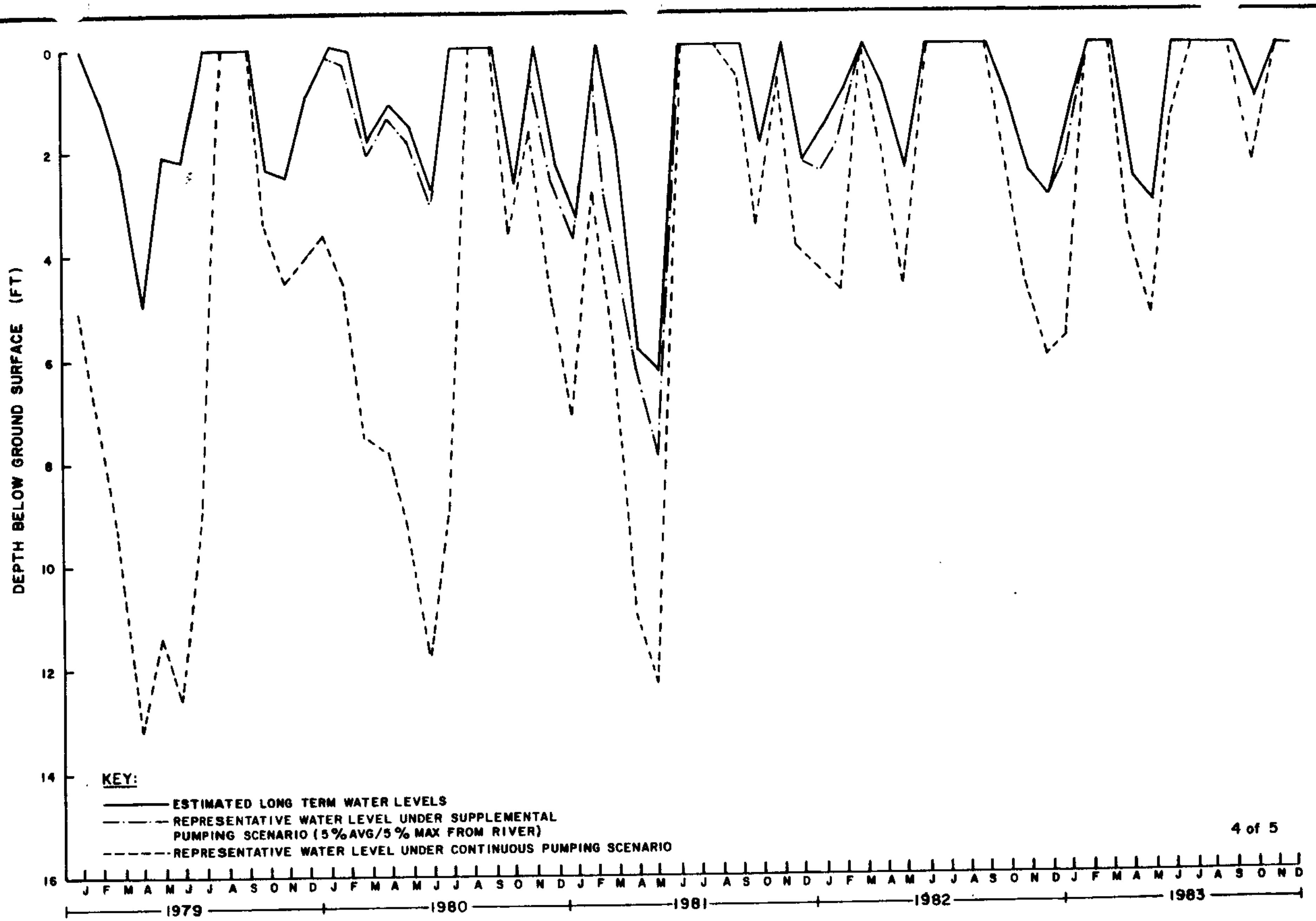


Figure E.7-7 (Continued). Representative Water Level Changes Under Different Pumping Scenarios.

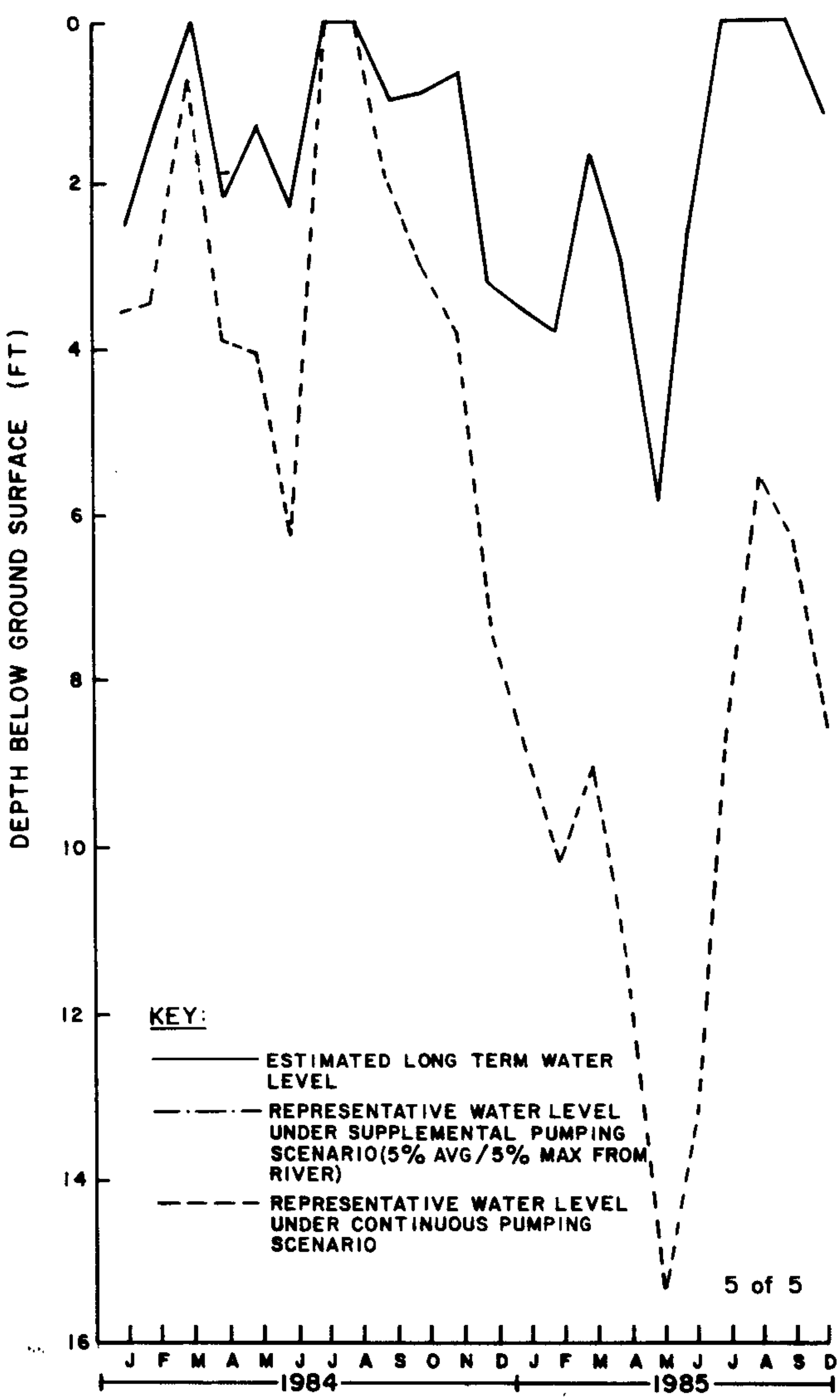


Figure E.7-7 (Continued). Representative Water Level Changes Under Different Pumping Scenarios.

APPENDIX F

MYAKKA RIVER SUPPLY ANALYSIS

APPENDIX F

MYAKKA RIVER SUPPLY ANALYSIS

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APPENDIX F

MYAKKA RIVER SUPPLY ANALYSIS

F.1 INTRODUCTION

The purpose of the Myakka River supply analysis was the estimation of the water supply that could be utilized from the Myakka River under various regulatory and environmental constraints and the associated storage required for drought season supply. The analysis consisted of:

1) regional assessment of the long-term river cycles and flow regime changes;

2) synthetic flow simulations for development of a relationship between dependable supply, drought storage, and withdrawal criteria;

3) assessment of the historical record under various combinations of demand, storage and withdrawal criteria; and

4) assessment of supplemental supply alternatives for reduction of required reservoir storage.

The following sections include a general description of the river regime, a description of the synthetic and historical flow demand/storage simulations, and a description of river diversion/wellfield operation alternatives for reduction of well-field and river environmental impact and construction costs.

F.2 RIVER REGIME DESCRIPTION

F.2.1 General

The Myakka River basin is located in the Gulf Coastal Lowlands physiographic province. The land surface is typified by numerous wetlands interspersed with pine palmetto flatwoods, range lands, and agricultural lands. The river rises in the Polk uplands and drains a portion of the Desoto plain (Florida State University, 1984). Surface elevations range from approximately 150 feet mean sea level (msl) to sea level at Myakka Bay. Except for a limited portion of the basin headwaters, the land surface is quite flat.

The river itself is characterized by a wide floodplain of up to and over 1 mile in width with two major in-stream surface water bodies, Upper and Lower Myakka Lake. The open water bodies within the river basin itself exceed 2.5 square miles.

Continuous discharge data (1937 - 1985) is available for the Myakka River at the Highway 72 bridge downstream of Upper Myakka Lake. The records are good for normal flows. Accuracy decreases as flow increases due to the wide floodplain and the existence of periodic cross basin flow transfers between the Myakka River and Vanderipe Slough. Rainfall data is available for the basin at Myakka River State Park near the Highway 72 gaging station.

For the period of synoptic rainfall/runoff data collection (1944 through 1985) the rainfall has averaged 56.27 inches and the runoff 14.4 inches. Rainfall has ranged from a low of 39.4 inches to a high of 81.07 inches and runoff from a low of 2.73 inches to a high of 35.44 inches. A graphic display of the rainfall and runoff data is provided in Figures F.2-1 and F.2-2, respectively.

F.2.2 Flow Variation With Respect to Time

General

The Myakka River discharge data indicates a gradual decrease in average Myakka River flow over time, particularly since the early 1960's. An assessment was conducted of the relationship between the Myakka River runoff and drainage basin rainfall. In addition, the regional nature of flow variations was assessed through a comparison of Myakka River and Peace River discharges. The summarized results of the two data set comparisons follow:

Myakka River/Rainfall Comparison

A comparison of Myakka River Basin runoff and rainfall as a function of percentage of averages for each data set is provided in Figure F.2-3. The trend of the long-term rainfall data does not correlate with the trend of the Myakka River average discharge. A summary of the statistical trend of the correlation assessment of the two data sets are shown on Table F.2-1. The correlation between rainfall and runoff weakens significantly towards the latter part of the record. There is a positive but weak correlation coefficient for the period of record of 0.52. The middle two 10 year periods of record (1955 to 1964; 1965 to 1974) show a relatively strong positive correlation with coefficients of 0.86 and 0.84, respectively. By contrast, the first 10 years of the synoptic period of record (1945 to 1954) and the latter 10 years of synoptic record (1975 to 1984) are more weakly positively correlated with coefficients of 0.55 and 0.27, respectively. Both of these time periods experienced severe droughts (1945, 1984) with the latter time period drought being less severe than that which occurred in 1945.

Myakka River/Peace River Comparison

The results of the Myakka River/Peace River data comparison is shown in Figure F.2-4. The data indicates that the decrease in average discharge is common to the physiographic region drained by both the Peace and Myakka Rivers. Both rivers demonstrate higher than average discharges for the 20 year period prior to 1965. The average discharge for the 10 year period from 1945 to 1955 was 10 percent and 28 percent higher than the long-term (1944 - 1985) period record for the Myakka and Peace Rivers, respectively. For the 1955 through 1965 period, the discharges on both rivers were approximately 20 percent higher than the long-term average. By comparison, following 1965 both rivers showed a decrease in average discharges. Decreases below the long-term averages were 16 percent and 20 percent for the Peace and Myakka rivers, respectively.

Runoff Variation Assessment

The comparison of Myakka River runoff data with the other regional data sets (rainfall and Peace River flow) does not provide an indication of the parameters associated with the variation in the Myakka River discharges. The comparison does indicate, however, that decreases in average river flows are unlikely to be a function of rainfall alone. Both Figure F.2-3 and and Table F.2-1 suggest a marked weakening in the correlation between rainfall and runoff in the latter 10 years of record.

The decreasing correlation could be partially related to severe droughts and associated water table depressions. Depressed water tables could decrease rainfall/runoff correlation by absorbing the rainfall from several high rainfall months following each dry season. This would produce high rainfall measurements without concurrent high runoff.

Although drought conditions affect the correlation between rainfall and runoff, other factors may be involved. Even though the droughts within the 1945-1954 period are more severe than those in 1975-1984, the correlation coefficient is significantly higher. This indicates that, if another factor is involved, it may be unique to the latter portion of the period or record.

The other factor may be water removal from the hydrologic system by pumping of the subsurface aquifers. As discussed in Appendix C , pumping of the Secondary Aquifer may periodically depress the water table aquifer in some areas. The result of such depression could be periodic, pumping induced runoff decreases while the water table aquifer is being recharged.

The impact of depressed water tables on runoff should be most evident during the early part of the wet season (June and July). Conversely, the impact should vanish during the late wet season (August and September) when the water table inevitably reaches the surface. If aquifer pumping is a factor in the change in Myakka River flows, a marked weakening of the correlation between June and July rainfall/runoff should have occurred over the past 10 to 20 years. Further, the correlation in September data should have remained fairly constant. As shown on Tables F.2-2 and F.2-3, however, the June and September monthly correlations are inconclusive.

Even when the comparisons are increased to three month periods (e.g., May, June, July rainfall with July runoff and July, August, September rainfall with September runoff) the results (Tables F.2-4 and F.2-5) are inconclusive. The poor correlation of monthly data is likely the result of the large lag time built into the rainfall/runoff relationship for a basin such as the Myakka. The lag results in a delayed runoff expression of rainfall. This comparison indicates that the hypothesis that decreases in Myakka River flow over time are associated with decreases in the basin water tables will require additional consideration.

F.3 SYNTHETIC STREAM FLOW SUPPLY/STORAGE ANALYSIS

F.3.1 Methodology

Utilization of the Myakka River as a supply source implies the provision of storage for accommodation of supply during the dry season and periodic extended droughts. The assessment of Myakka River supply therefore requires estimation of the drought or dry season storage required for the desired dependable yield. The historical record indicates that near zero flow has occurred in the Myakka River for periods of up to 6 months. Even in a normal year the river will experience near zero flow for approximately 2 months. The impact of Myakka River low flows on a supply system can be brought into perspective by the observation that a 1 mgd supply over a 6-month period is equivalent to almost 600 acre-feet of water. This volume of water in turn is equivalent to a lake 20 feet deep over 30 acres.

The general procedure used in estimating the storage was:

1. Compile a set of statistically valid series of synthetic stream flow data (Section F.3.2).

2. Subject synthesized data to withdrawal criteria (Section F.3.3).

3. Develop a yield versus storage/allowable percent withdrawal relationship under given regulatory and physical withdrawal constraints (Section F.3.4).

Each of the three steps are discussed separately in Sections F.3.2 through F.3.4

F.3.2 Synthetic Stream Flow Time Series

The synthetic stream flow analysis was utilized due to the recognition that the historical record provides a pattern of flow which is unlikely to occur during the period in which the proposed system would be operative. Furthermore, the recorded values of high flow, low flow, and other characteristics of the record are unlikely to be consistent during the design period for the facilities. This is particularly true with respect to floods or droughts. Finally, the historical record alone does not provide a sufficient measure of the risk. For this particular application the historical record in actuality is sufficient for estimating a 95 percent dependability for yield under various storage and withdrawal scenarios. However, the utilization of synthetic data allows a reasonable extension of the record which provides a higher level of confidence in the operational parameters associated with river withdrawals.

The synthetic stream flow analysis consisted of generating a sequence of monthly values of flows which are statistically comparable in terms of mean, standard deviation, and skewness to the historical monthly flow values. The generation of the flows assumes that flows are the result of a random process, a process whose results change with time in a way that involves probability (Moran, 1959). This approach assumes that the statement "The probability or chance that the flow in a given stream next year will be less than X units is P1" is taken as an accurate approximation. The approach does not assume or imply that exact flows can be predicted. Instead the generation process is based on probabilistic characteristics.

In addition, the technique recognizes an important tendency towards persistence in extreme flows; low flows tend to be followed by other low flows and high flows tend to be followed by other high flows. This characteristic allows the generation of flows that retain the statistical characteristics of observed flows. It is emphasized that the statistical method does not provide a causal model for actual flows.

The synthetic flows for the Myakka River were generated as a sequence of numbers of values produced by a random process in the succession of monthly time intervals. In general, the ith member, X_i, of a time series is estimated as the sum of two parts:

$$X_i = D_i + E_i \qquad \text{Eq. F.3-1}$$

In this basic model, D_i is the deterministic component determined by an exact functional rule from the statistical parameters and previous values of the process. The random component is E_i. This is a random number drawn or sampled from the set of random numbers with a certain probability distributional pattern.

For this analysis the deterministic portion is based on the normal distribution function and the mean standard deviation lag one serial correlation coefficient and coefficient of skewness of the historical flow sequence. The justification for utilization of the normal function is the central limit theory. This theory states that a variable is distributed approximately normally if 1) it is the sum of identically distributed random variables and 2) the variables are derived from any distribution with a finite mean and variance. Since the flow of a stream in a given time period is an accumulation of different additive factors, the flows may be considered as sums of random variables; thereby being normally distributed. Finally, emperical studies

(Fiering and Jackson, 1971) have shown that the mean and standard deviation are much more important than other statistics in producing good results in basin simulation studies. As the historical mean and standard deviation are sufficient to approximately reproduce historical record the normal distribution is adequate.

Fortunately, the mean and standard deviation can be estimated tolerably well from moderate sized samples. Other statistics, such as higher order moments (the skewness coefficient) and correlation coefficients for large lags are subject to more pronounced sampling errors; however, these less stable statistics are not crucial for evaluation of alternative storage schemes especially on the one month time step.

The deterministic model selected for generating the synthetic flows was the lag one or Markovian flow model as presented in Fiering (1967). The generating equation is:

$$Q_{i+1} = \overline{Q}_{j+1} + b_j(Q_i - \overline{Q}_j) + t_i S_{j+1} \sqrt{(1-r_j^2)} \quad \text{Eq. F.3-2}$$

where:

Q_i and Q_{i+1} = discharges during the i^{th} and $(i+1)^{th}$ month, respectively, computed from the start of the synthesized sequence.

$\overline{Q}_j$ and $\overline{Q}_{j+1}$ = mean monthly discharges during the j^{th} and $(j+1)^{th}$ month, respectively, within a repetitive annual cycle of 12 months.

b_j = regression coefficient for estimating flow in the $(j+1)^{th}$ from the j^{th} month.

t_j = random normal deviate with zero mean and unit variance.

S_{j+1} = standard deviation of flows in the $(j+1)^{th}$ month.

r_j = correlation coefficient between the flows of the j^{th} and $(j+1)^{th}$ month.

It should be noted that the model assumes that the entire influence of the past on the current flow is reflected in the previous flow value. The equation characterizes a circular random walk, a model in which the discharge in the i+1 month is comprised of a component linearly related to that in the i^{th} month and a random additive component. The variation in sign and magnitude of the random additive component makes for a continuous, unbounded, and serially correlated sequence of data for simulation studies.

In the computations, the index j ran cyclically from 1 to 12, and the index i ran sequentially from 1 to 240 (240 values or 20 years of 12-month flows). By the same procedure, 100 traces of flow were computed. Each trace was defined as a 20-year record of monthly flows. The dependable yield under various withdrawal constraints was calculated by superimposing the constraints on each 20-year synthetic trace and estimating the dependable yield versus the various conditions. The dependable yield for each trace was taken as that yield for which there was a single failure to provide sufficient storage during the 20-year period. This resulted in what amounted to 100 failures in 2,000 years of synthetic data. The dependable yield was determined by averaging the 100 yields estimated from the 100 synthetic traces. Consequently, this resulted in 50 failures for the 2,000 years of synthetic flow. This was taken to approximate a supply dependability in anyone year of 97.5 percent.

F.3.3 Withdrawal Constraints

The dependable yield is a function of demand variation, the constraints of river flow variability, regulatory minimum flow rates in the river, maximum allowable percentage of river flow withdrawal, pump size, storage capacity, and loss/gain from net rainfall. It is assumed that only the maximum allowable percentage of river flow withdrawal, pump size and storage capacity can be varied. For simplification of the analysis this conceptual stage, pump capacity was assumed to be a non-limiting parameter. The other constraints are taken as given constants.

Each of the parameters listed above are discussed in the following sub-sections.

Flow Variation

Flow variation is accounted for with the synthetic flow time series simulations discussed in Section F.3.2.

Demand Variation

Demand variation by month is shown on Table F.2-6. The data was obtained as a representative variation from Sarasota County Water System Study Phase Supply Report (Joint Venture, 1985).

Regulatory Minimum Flow Rate

Minimum flow rates in the Myakka River are mandated by standards promulgated by the Southwest Florida Water Management District (SWFWMD). The criteria is as follows:

> Unless otherwise deemed appropriate by the Board, the minimum rates of flow at a given point on a stream or other watercourse shall be established by the Board for each month, January through December. Minimum rates of flow shall be established as follows: For each month, the five (5) lowest monthly mean discharges for the preceding twenty (20) years shall be averaged. Minimum rates of flow shall be established as seventy percent (70%) of these values for the four (4) wettest months and ninety percent (90%) of these values for the remaining eight (8) months. The determination shall be based on available data, or in the absence of such data, it shall be established by reasonable calculations approved by the Board (SWFWMD Standards, Section 40D-8.041, Paragraph (2)).

A summary of the assessment of minimum flows is provided in Table F.2-7.

Maximum Allowable Percent Withdrawal/Pump Capacity

The following percent withdrawals from the Myakka River were examined: 5, 7.5, 10, 15, and 25 percent. It should be noted that the regulatory (SWFWMD) standards mandate a maximum withdrawal of 5 percent. Exceptions to the 5 percent standard at any time are only acceptable if it can be demonstrated that periodic short-term exceedances can be environmentally tolerated. Pump capacities between 100 and 400 cfs were examined. For the purposes of this conceptual study, the higher pump withdrawals were utilized in the final assessment to avoid the pump size being the controlling constraint.

Storage Capacity

The following storage capacities in acre-feet were examined in the simulation: 5,000, 10,000, 12,000, 15,000, 18,000, 20,000, and 25,000.

Net Rainfall

Net rainfall is the sum of rainfall and lake evaporation. For the purposes of this study, reservoir seepage was neglected. Monthly rainfall was estimated as the average for each month for the period of record. Evaporation was estimated as 70 percent of each monthly pan evaporation measurement from the Station DM-5 Class A evaporation pan on the RMR. The net rainfall values are summarized in Table F.2-8.

Computations of yield were conducted for one hundred 20-year traces. For each trace a trial value of demand was initially assumed. The trial value of demand was compared to the available water from the Myakka River based on the withdrawal criteria combination. If the demand was in excess of the available water the difference was obtained from storage; conversely when there was a surplus, the surplus was utilized to recharge the storage deficit associated with previous pumpage. In addition, the net rainfall onto the reservoir as a function of evaporation and rainfall gain was incorporated into the analysis. The procedure was applied for each month of the trace and the reservoir storage monitored. If the storage stage dropped below the minimum volume of supply, the trial value of demand was adjusted downward and the procedure repeated. When a successful run to the entire trace was accomplished within a preset tolerance, the trial value of demand was taken as a yield for that trace.

F.3.4 YIELD VS. STORAGE/MAXIMUM PERCENT RIVER WITHDRAWAL

The relationship developed from the above simulation is provided on Figure F.3-1. The volume of storage associated with a given yield is highly dependent on the maximum percent of allowable withdrawal. For example, for a dependable yield of 8 mgd the required storage ranges from over 18,000 acre-feet for 5 percent maximum withdrawal to less than 8,000 acre-feet for 25 percent maximum withdrawal. For both cases the average volume of water withdrawn from the river is the same at approximately 5 percent. The principal difference is that at the outset of the wet season, following an extended drought, withdrawals are higher than average until the reservoir is filled. Higher maximum withdrawal percentages allow more rapid refilling of the reservoir following severe droughts. The more rapidly the reservoir can be refilled, the less often carry-over storage occurs from one year to the next. Lower percent withdrawals result in numerous years in which the reservoir is not refilled during the wet season. With the required storage supply being cumulative from year to year, very high storage requirements result.

The relationship shown in Figure F.3-1 was used as a guide in selecting appropriate storage requirements to be used in the assessment of system operation under historical conditions (Section F.4).

F.4 MASS CURVE ANALYSIS

F.4.1 Methodology

The mass curve analysis utilized a basic procedure first introduced by Rippl (1883). The method evaluates the cumulative deficiency between the outflow and inflow and selects the maximum cumulative value as the required storage. The procedure is comparable to that utilized for the yield versus storage assessment calculated with the synthetic stream flow data. The principal difference is, that instead of using a sythetically generated long-term period of record, the historical period of record from 1937 to 1985 was employed. In addition the mass curve analysis with the historical data utilized actual monthly rainfall data obtained at Myakka River State Park in the calculation of net precipitation.

The controlling equations for the mass curve analysis are:

$$s(i) = \text{minimum } [(s(i-1) + \Delta s), \text{vol}] \qquad \text{Eq. F.4-1}$$

$$\Delta s = \text{minimum } [(np - \text{draft} + \text{draw}), (\text{vol} - s(i-1))] \qquad \text{Eq. F.4-2}$$

$$\text{Draw} = \text{minimum } [\text{pump}, (Q - \text{FMIN}), (mp \times Q), (\text{vol} - s(i-1)] \qquad \text{Eq. F.4-3}$$

where:

s(i) = storage in reservoir in month i
Δs = change in storage from month i-1 to month i
volume = maximum reservoir volume
np = net precipitation
draft = monthly demand
draw = river water withdrawal
pump = pump capacity for transfer of water from the river to the reservoir
FMIN = minimum flow requirement in the Myakka River
mp = maximum percent withdrawal from the Myakka
Q = Myakka River flow

The mass curve analysis was conducted on a monthly basis. Input parameters were Myakka River flow, net precipitation as estimated from historical rainfall and average evaporation, the required draft by month, the starting reservoir volume, the pump capacity, the minimum flow requirement, and the maximum percent withdrawal. Utilizing Equations F.4-1 through F.4-3, a continuous simulation was conducted of the reservoir storage for the period of record under different rates of supply demand. The simulation provided a mass balance of the reservoir storage over the period of record and an estimate of the maximum storage required for each year of the record under each supply demand simulated. The withdrawal constraints analyzed are discussed in Section F.4.2 and a representative simulation is presented in Section F.4-3.

F.4.2 Withdrawal Constraints

Mass curve analysis was conducted utilizing various maximum percent withdrawals and starting reservoir sizes. The representative supply demand utilized was 8 mgd. This demand was taken as the likely average freshwater supply requirement for either Phase I or II of the phases of water supply development on the RMR. Starting reservoir sizes were selected based on the reservoir sizes estimated from the synthetic stream flow supply storage analysis conducted as described in Section F.4.3. Two maximum percent withdrawals were assessed, 5 percent and 15 percent. All other withdrawal constraints were as discussed in F.3.3.

F.4.3 Representative Simulation

The results of a representative mass curve analysis simulation are shown on Table F.4-1. The table shows the maximum storage required for each year of record based on an average demand of 8 mgd as distributed based on the distribution in Table F.2-6 assuming the SWFWMD regulatory constraint of 5 percent withdrawal rate. Table F.4-1 indicates that a demand of 8 MGD would have resulted in a storage requirement in excess of 18,500 acre-feet twice during the 50-year historical period of data. This relates to a dependability of approximately 95 percent for a reservoir of 18,500 acre-feet.

The large volume of storage is the result of carry over storage between years. An example of carry over storage is provided for the years 1943 and 1945 as shown on Table F.4-2. At the end of 1944 the reservoir was 10,000 acre-feet below full storage. This 10,000 acre-feet was then added to an additional 7,500 acre-feet of storage utilized in 1945. If supplemental supply had been available to refill the reservoir and eliminate the carry-over, required reservoir storage would have been sharply reduced.

Because storage requirements are extreme for a sole dependence on the Myakka River for a given supply, supplemental supply is a critical concern. Alternatives for the utilization of supplemental supply are discussed in the next section.

F.5 SUPPLEMENTAL SUPPLY ALTERNATIVES

F.5.1 Assessment Methodology

The purpose of the supplemental supply assessment was to develop a set of alternatives for reducing carry over storage and thereby reduce the size of surface storage facilities. The approach consists of the following tasks:

- Supply/withdrawal criteria
  - Selection of a target yield to be supplied by the Myakka River and supplemental supply.
  - Selection of representative withdrawal criteria in terms of maximum allowable percent of river flow. All other withdrawal criteria such as minimum flow are assumed as specified in Section F.3.3.
- Simulation of monthly supply requirements from storage for the period of historical flow data record using the mass curve analysis described in Section F.4.
- Calculation of within year supply required from storage following each month of the year for all 49 years of record (water year; November - October).
- Selection of a function that describes the frequency distribution of the 49 within year storage supply volumes associated with each month of the dry season. (Note: Supply volume associated with a given month is that volume of supply that will be taken from storage from the end of the month until the end of the dry season.)
- Selection, based on the frequency distributions, of the storage supply associated with each month that has a 2 percent chance (50-year return interval) of being required in any one year.

- Estimation of the available supplemental supply storage based on physical constraints of aquifer yield and water quality from the Secondary and Floridan Aquifers.
- Estimation of the volume of supplemental supply that can be provided during the dry season between the end of each dry season month and the end of the dry season. This provides an estimate of the storage that must be supplied from a surface reservoir after each month of the dry season. These volume estimates associated with each month provide a "rule curve." The "rule curve" is the guide for determining how much water must be in the reservoir at the end of each month to maintain a 98 percent level of confidence that demand will be met throughout the dry season.
- Simulation of the long-term operation of the reservoir using historical data and the "rule curve" developed in the previous step.

Each of the above steps are discussed individually in the following section.

F.5.2 Supplemental Storage Analysis

Selection of Yield

The supplemental storage analysis required the selection of a representative target yield. Environmental and permitting uncertainties prevent accurate prediction of the allowable quantities of yield from the Myakka River. In order to provide representative numbers for use by the County and the environmental consultants in the assessment of impacts a probable yield had to be estimated. For this assessment a yield of 8 mgd from the Myakka River, which represents 5 percent of the long-term flow average, was utilized. The 8 mgd yield will be refined as the state of knowledge of environmental impacts is improved. However, for this conceptual engineering phase the 8 mgd is taken as an appropriate representative yield from the river as it falls within the SWFWMD general permitting guidelines.

Selection of Withdrawal Criteria

The withdrawal criteria consists of those items discussed earlier in Section F.3. The criteria consists of such things as average and maximum percent withdrawal from the river, pump capacity, minimum river flow criteria, etc. Minimum flow was taken as the SWFWMD criteria as discussed in Section F.3.3. Pump capacity was assumed to be a non-controlling parameter. The average percent withdrawal of 8 mgd was utilized as the only demand to be assessed. The monthly variation in demand was taken as the distribution presented by Smally and Wellford (1985) (Table F.2-6).

The only withdrawal criteria varied in the analysis was the maximum percent withdrawal allowed on a periodic basis. This maximum withdrawal percent is of primary importance when the reservoir is low and requires recharging. As discussed in Section F.3 the higher the maximum allowable withdrawal the less storage required due to the greater capabilities for 1) providing supply from the river rather than storage during lower flow conditions. and 2) preventing carry-over storage depletions.

Two maximum instantaneous withdrawal values were assessed 5 and 15 percent. The 5 percent was selected as a reflection of the SWFWMD general withdrawal criteria of 5 percent of river flow. The 15 percent was selected to provide a comparison of system operational modifications and facility sizes associated with higher withdrawal rates.

Calculation of Within Year Storage Supply Requirements

Within year storage supply is that amount of total yearly demand that must be supplied from storage. The objective of the assessment was the estimation of how much of the supply following each month of the dry season (October - June) must be provided from storage for each year of the period of record. This assessment was conducted for the two maximum allowable withdrawal rates mentioned earlier, 5 and 15 percent.

Within Year Supply Frequency Distribution

The supply required from storage following each month for each year of record was fitted on a monthly basis to various frequency distributions. The best fit for the data for both withdrawal criteria and all months analyzed was estimated to be the log normal distribution. The results of the frequency plots for each month are provided in Figures F.5-1 through F.5-16. As can be seen from the figures, the amount of within year supply required from storage following each month decreases as the dry season progresses. This is the result of the diminishing dry season period before the return of significant rainfall in the summer.

Although the log normal distribution provides a good data fit some months have a significant outlier in the 2 percent chance of occurrence range. This is particularly true for the storage required for the 5 percent average/5 percent maximum diversion scenario (Figures F.5-1 through F.5-8). The outlier in the 2 percent occurrence range (50 year return interval) falls well outside of the range of the predicted value from the frequency distribution. These outliers are supply associated with the 1944-1945 drought in which the average river flow was 46 cfs or less than 20 percent of the long-term average flow of 243 cfs. As a contrast to the 1944-1945 drought the 1984-1985 drought (October-September 1985) averaged approximately 95 cfs (estimate). The frequency distribution indicates that the 1945 drought was much more severe than a one in 50-year event and likely has a recurrence interval closer to one in 200 years or larger.

Required Storage Following Each Month

A 98 percent supply dependability (2 percent chance of failure in any one year) was selected as the appropriate design level for this conceptual analysis. The within-year storage supply required following each month is summarized in Table F.5-1. As can be seen in the table the storage associated with each month is significantly greater for the 5 percent average/5 percent (5/5) maximum withdrawal criteria than for the 5 percent average/15 percent (5/15) maximum. As indicated on the table the maximum within year storage required for the 5/5 alternative is 10,500 acre-feet while the 5/15 alternative storage is 7,800 acre-feet.

Available Supplemental Supply

The data discussed in the previous subsection indicates the total amount of storage required to meet the within-year demand with a 98 percent dependability. This supplemental storage can be provided either with surface reservoir(s) or alternative sources within the RMR.

The practical supplemental sources are the Secondary Aquifer and the Floridan Aquifer. As discussed in Appendix E surface runoff sources on the site are not appropriate for supplemental supply. Surface runoff cannot provide supply when it is needed during the low flow conditions on the Myakka River.

The volume of supplemental supply that can be obtained from the Secondary Aquifer is constrained by the yield characteristics of the aquifer. As discussed in Appendix C the most practical area in terms of aquifer yield and economic constraints is the Area I well field in the northwest corner of the site near Myakka River State Park. In order to minimize cost of access roads and collection systems, a limited number of wells pumped at relatively high rates is preferred. The Area 1 well field is estimated as being capable of providing 3 to 4 mgd with approximately six wells. The well field, however, will likely only be capable of providing this supply for 30 to 60 days before SWFWMD drawdown criteria is violated. As a conservative assumption the Area I well field was utilized as being able to provide a 3 mgd supply for 30-day periods. It was further assumed that the aquifer would recover in the same time period as the pumping cycle. Therefore, the aquifer would yield an average of 1.5 mgd during a pumping cycle of one month on/one month off.

As opposed to the Secondary Aquifer, utilization of the Floridan Aquifer is constrained by the water quality. Several assumptions were required for estimating the potential supply of Floridan water for blending with freshwater supplies. The assumptions were as follows:

- The Myakka River freshwater supply will undergo conventional lime softening (L/S) treatment incapable of removing sulfates.
- Myakka River supply will be used in conjunction with Floridan water treated with reverse osmosis (R/O). The two supplies will be used in equal parts. Average supply from each source, at least through Phase II of the water system development, will be 8 mgd for a total of 16 mgd (24 mgd maximum).
- Normal operating sulfate concentrations for the output stream from the combined R/O and L/S treatment plant will be 20 milligrams per liter.
- Under emergency conditions the sulfate concentrations can be increased to the regulatory limit of 250 milligrams per liter.

A flow chart of the blending of Surficial, Secondary, and Floridan water is shown in Figure F.5-17. Based on the assumptions as to water quality from each of the treatment streams, the Floridan Aquifer can provide up to 3.7 mgd for blending purposes during emergency low flow conditions. This supply is considered a last resort supplement and would only be brought on line once it was obvious that the surface reservoir and Secondary Aquifers could not provide a sufficient level of assurance that the demand will be met through the dry season.

The estimated supply of 1.5 mgd from the Secondary Aquifer and 3.7 mgd from the Floridan Aquifer was converted into volumes of available supply that could be pumped from each aquifer during the dry season following each month of the dry season. This volume of water then was subtracted from the total volume of storage supply as determined from the frequency distributions. The remaining storage supply above that which can be provided by the aquifers is that volume of water which must be provided by a surface reservoir. The volumes thus determined provide a "rule curve" which indicates the volume of water which must be in the reservoir at the end of each month in order to maintain a 98 percent dependability that the reservoir will not be exhausted prior to the end of the dry season. A summary of the rule curves is shown in Figures F.5-18 and F.5-19.

F.5.3 Simulations of Reservoir Operation with Supplemental Storage

5 Percent Average/5 Percent Maximum Withdrawal Criteria

The effect of utilizing supplemental supply on storage can be observed from Tables F.4-1 and F.5-2. The principle effect is that the required storage for the 5 percent average/5 percent maximum (5/5) withdrawal criteria is reduced from 18,500 acre-feet to 10,000 acre-feet. The 10,000 acre-foot reservoir which includes an 8,000 acre-feet, 20-foot deep supply pool, and a 2,000 acre-feet, 5-foot deep permanent pool would have provided sufficient storage for the entire 49 years of record. The supply pool of 8,000 acre-foot would have been sufficient for all but one year of the record. The deficiency would have occurred in the 1943-1944 drought in which the permanent pool would have been utilized extensively. The supply pool would have been completly exhausted in one other year, the 1955 drought. Both the 1945 drought and the 1955 drought were more severe than the drought experienced in 1984/1985.

The supplemental storage produced an average withdrawal over the period of record from the river of less than 5 percent (Table F.5-3). The average withdrawal by month is summarized in Table F.5-4. The withdrawals would have averaged 5 percent only for the months of May and June when the reservoir would typically be at its lowest point in every year. From July through October the percent withdrawal for the period of record is reduced down to the range of 3 to 4 percent. This is due to the simulation of many years of a full reservoir during the summer and less water being required from the river for reservoir recharge.

The frequency of pumping of the Secondary and Floridan Aquifers is shown in Table F.5-5. As can be seen in the table the Secondary Aquifer pumps are utilized on the average of once every other year for a period of 3 to 4 months. The Floridan Aquifer is utilized approximately the same except during the later portion of the dry season when it is somewhat less often utilized.

5 Percent Average/15 Percent Maximum Withdrawal Criteria

The effect of utilizing supplemental supply and increasing the maximum allowable withdrawal can be observed from Tables F.5-2 and F.5-6. The principle effect is that the required storage for the 5 percent average/15 percent maximum (5/15) withdrawal criteria is reduced to less than 6,500 acre-feet. The 6,250 acre-foot reservoir which includes a 5,000 acre-feet, 20-foot deep supply pool, and a 1,250 acre-feet, 5-foot deep permanent pool would have provided sufficient storage for the entire 49 years of record. The supply pool of 5,000 acre-foot would have been sufficient for all but three years of the record. The deficiency would have occurred in the 1943-1944 drought in which the permanent pool would have been utilized extensively. Both the 1945 drought and the 1955 drought were more severe than the drought experienced in 1984-1985.

The supplemental storage produced an average withdrawal over the period of record from the river of less than 5 percent (Table F.5-7). The average withdrawal by month is summarized in Table F.5-8. The withdrawals would have averaged up to 13 percent for the months of April through June when the reservoir would typically be at its lowest point in every year. From July through October the percent withdrawal for the period of record is reduced down to the range of 1.5 to 5 percent. This is due to the probability in many years of the reservoir being full during the summer and less water being required from the river for recharge reservoir. As with the 5/5 withdrawal criteria, the maximum allowable diversion capacity from the Myakka River was 300 cfs.

The frequency of pumping of the Secondary and Floridan Aquifers is shown in Table F.5-9. As can be seen in the table the Secondary Aquifer pumps are utilized on the average of once every five years for a period of 1 to 2 months. The Floridan Aquifer is utilized approximately the same except during the later portion of the dry season when it is utilized somewhat less often.

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TABLE F.2-1

MYAKKA RIVER RUNOFF/RAINFALL STATISTICAL COMPARISON
(YEARLY)

	Time Period				
	45-84	45-54	55-64	65-74	75-84
Rainfall Mean (inches)	56.49	49.96	58.88	55.57	61.56
Runoff Mean (cfs)	243.50	267.60	285.50	230.50	190.40
Standard Deviation Rainfall (inches)	11.46	8.44	14.48	7.98	11.86
Standard Deviation Runoff (cfs)	135.17	163.08	179.53	96.85	70.97
Coefficients of Variation-Rainfall	20.28	16.90	24.60	14.36	19.27
Coefficients of Variation-Runoff	55.51	60.94	62.88	42.02	37.28
Covariance	812.50	762.24	2,224.48	652.04	231.54
Correlation Coefficient	0.52	0.55	0.86	0.84	0.27

Notes: Yearly total runoff taken from USGS Highway 72 gage.
Yearly total rainfall from Myakka River State Park gage.

TABLE F.2-2

MYAKKA RIVER RUNOFF/RAINFALL STATISTICAL COMPARISON
(JUNE)

	Time Period				
	45-86	45-54	55-64	65-74	75-84
Rainfall Mean (inches)	8.52	8.66	6.58	9.10	9.55
Runoff Mean (cfs)	166.97	119.12	118.04	281.39	152.28
Standard Deviation Rainfall (inches)	4.22	3.17	2.72	4.13	5.76
Standard Deviation Runoff (cfs)	245.34	184.55	144.83	206.56	357.81
Coefficients of Variation-Rainfall	49.49	36.67	41.29	45.41	60.31
Coefficients of Variation-Runoff	146.94	154.92	122.70	73.41	234.96
Covariance	-140.21	-363.07	-143.09	-139.81	-118.01
Correlation Coefficient	-0.14	-0.62	-0.36	-0.16	-0.06

Notes: June runoff taken from USGS Highway 72 gage.
June rainfall taken from Myakka River State Park gage.

TABLE F.2-3

MYAKKA RIVER RUNOFF/RAINFALL STATISTICAL COMPARISON
(SEPTEMBER)

	Time Period				
	45-86	45-54	55-64	65-74	75-84
Rainfall Mean (inches)	8.56	7.52	9.87	7.16	9.52
Runoff Mean (cfs)	735.28	692.83	1109.77	586.6	539.10
Standard Deviation Rainfall (inches)	4.05	1.50	5.70	2.9	4.51
Standard Deviation Runoff (cfs)	580.00	720.08	789.48	244.93	351.90
Coefficients of Variation-Rainfall	47.27	19.96	57.82	40.56	47.39
Coefficients of Variation-Runoff	78.88	103.93	71.14	41.75	65.28
Covariance	-296.92	342.42	-1471.63	-307.29	-296.58
Correlation Coefficient	-0.11	0.32	-0.33	-0.43	-0.19

Notes: September runoff taken from USGS Highway 72 gage.
September rainfall taken from Myakka River State Park gage.

TABLE F.2-4

LATE SPRING RAINFALL/RUNOFF STATISTICAL COMPARISON

	Time Period				
	45-86	45-54	55-64	65-74	75-84
Rainfall Mean (inches)	21.00	19.19	18.36	22.35	24.10
Runoff Mean (cfs)	384.32	530.58	263.96	484.16	258.60
Standard Deviation Rainfall (inches)	6.35	5.10	5.21	6.26	7.65
Standard Deviation Runoff (cfs)	426.79	586.47	285.68	485.67	243.65
Coefficients of Variation-Rainfall	30.25	26.57	28.39	27.99	31.73
Coefficients of Variation-Runoff	111.05	110.53	108.23	100.31	94.22
Covariance	141.63	86.07	158.72	-190.28	782.52
Correlation Coefficient	0.05	0.03	0.11	-0.06	0.42

TABLE F.2-5

SUMMER RAINFALL/RUNOFF STATISTICAL COMPARISON

	Time Period				
	45-86	45-54	55-64	65-74	75-84
Rainfall Mean (inches)	26.44	23.78	26.57	26.29	29.11
Runoff Mean (cfs)	750.50	845.60	1021.37	563.10	571.92
Standard Deviation Rainfall (inches)	7.17	5.70	7.14	8.01	7.69
Standard Deviation Runoff (cfs)	586.94	678.72	810.21	274.31	366.70
Coefficients of Variation-Rainfall	27.10	23.97	26.86	30.48	26.40
Coefficients of Variation-Runoff	78.21	80.26	79.33	48.71	64.12
Covariance	-1510.47	-1296.18	-3241.79	-670.76	-597.76
Correlation Coefficient	-0.36	-0.33	-0.56	-0.31	-0.21

TABLE F.2-6

DEMAND VARIATION BY MONTH
(Joint Venture, 1985)

Month	Percent of Average
January	96
February	91
March	113
April	112
May	114
June	101
July	96
August	92
September	89
October	98
November	97
December	100

TABLE F.4-1 (Continued)

Year	Maximum Reservoir Storage (acre-feet)
1976	16,651.61
1977	12,244.51
1978	13,980.48
1979	12,179.95
1980	16,145.24
1981	14,137.98
1982	4,034.34
1983	3,717.26
1984	12,828.91

TABLE F.4-2

EXAMPLE OF CARRY-OVER STORAGE

Year	Month	Flow (cfs)	Draw (cfs)	Total Storage in Reservoir (acre-feet)
1943	November	21.20	1.06	19,630.33
1943	December	3.11	0.00	18,556.11
1944	January	3.10	0.00	17,579.36
1944	February	1.47	0.00	16,503.90
1944	March	0.36	0.00	15,670.84
1944	April	0.01	0.00	14,352.97
1944	May	0.00	0.00	13,406.21
1944	June	0.00	0.00	12,870.53
1944	July	67.90	3.40	12,550.60
1944	August	221.00	11.05	12,622.42
1944	September	146.00	0.00	12,174.20
1944	October	75.60	3.78	11,687.25
1944	November	20.30	1.01	10,719.67
1944	December	2.97	0.00	9,667.94
1945	January	11.00	0.55	8,762.65
1945	February	3.38	0.00	7,703.43
1945	March	0.07	0.00	6,612.98
1945	April	0.00	0.00	5,360.08
1945	May	0.00	0.00	3,941.01
1945	June	207.00	10.35	4,559.69
1945	July	1,618.00	80.90	9,795.93
1945	August	1,288.00	64.40	13,718.05
1945	September	680.00	34.00	15,089.17
1945	October	240.00	12.00	15,117.55

TABLE F.5-1

TOTAL REMAINING WITHIN YEAR REQUIRED STORAGE SUPPLY
DEMAND = 8 MGD

Month	Withdrawal Criteria: 5% Average/5% Maximum (acre-feet)	Withdrawal Criteria: 5% Average/15% Maximum (acre-feet)
October	10,500	7,800
November	9,500	7,000
December	8,000	6,500
January	7,200	5,300
February	6,000	4,200
March	4,800	3,500
April	3,900	3,000
May	3,200	2,100

TABLE F.5-2

YEARLY RESERVOIR STORAGE
5 PERCENT AVERAGE/5 PERCENT MAXIMUM
WITHDRAWAL CRITERIA WITH SUPPLEMENTAL SUPPLY

Year	Maximum Reservoir Storage (acre-feet)
1936	2925.40
1937	4845.36
1938	4809.33
1939	5103.28
1940	5266.61
1941	4383.77
1942	5930.17
1943	6794.20
1944	9335.72
1945	5240.38
1946	4942.70
1947	4130.44
1948	5059.64
1949	5266.59
1950	5754.46
1951	6850.10
1952	4778.80
1953	2935.92
1954	5473.28
1955	8010.77
1956	6935.31
1957	2143.22
1958	1417.09
1959	3256.98
1960	5119.78
1961	7159.82
1962	4340.48
1963	4025.52
1964	5862.15
1965	3448.95
1966	5121.60
1967	5004.48
1968	2387.50
1969	830.38
1970	5800.52
1971	3755.07
1972	2710.63
1973	5336.04
1974	5273.17
1975	5386.36

TABLE F.5-2

Year	Maximum Reservoir Storage (acre-feet)
1976	5870.35
1977	2312.37
1978	3499.09
1979	4129.06
1980	5846.88
1981	3570.90
1982	1164.21
1983	4278.64
1984	6994.32

49 years average of yearly reservoir supply storage required = 2931.76 in acre-feet

TABLE F.5-3

YEARLY SUMMARY OF RIVER WITHDRAWALS
WITHDRAWAL CRITERIA 5 PERCENT AVERAGE/5 PERCENT MAXIMUM
(WITH SUPPLEMENTAL SUPPLY)

YEAR	FLOW	ALLOWABLE WITHDRAWAL	ACTUAL WITHDRAWAL	% OF FLOW
1937 AVGS.	240.70	12.05	11.59	4.81
1938 AVGS.	276.80	13.77	11.97	4.33
1939 AVGS.	398.67	19.92	10.20	2.56
1940 AVGS.	171.94	8.57	8.57	4.99
1941 AVGS.	125.30	6.24	6.24	4.98
1942 AVGS.	207.43	10.00	10.00	4.82
1943 AVGS.	354.38	17.71	12.33	3.48
1944 AVGS.	44.78	1.61	1.61	3.57
1945 AVGS.	339.23	16.93	16.57	4.89
1946 AVGS.	131.33	6.52	6.52	4.97
1947 AVGS.	553.15	27.62	11.80	2.13
1948 AVGS.	372.73	18.63	12.39	3.32
1949 AVGS.	332.48	16.58	10.76	3.24
1950 AVGS.	126.42	6.31	6.31	4.99
1951 AVGS.	158.09	7.21	7.21	4.56
1952 AVGS.	140.40	6.43	6.43	4.58
1953 AVGS.	389.73	19.49	14.44	3.70
1954 AVGS.	230.97	11.55	11.13	4.82
1955 AVGS.	135.73	6.77	6.77	4.98
1956 AVGS.	103.68	4.62	4.62	4.45
1957 AVGS.	328.73	16.39	14.32	4.36
1958 AVGS.	261.98	12.67	8.35	3.19
1959 AVGS.	628.06	31.40	11.97	1.91
1960 AVGS.	542.70	27.14	11.58	2.13
1961 AVGS.	106.59	5.17	5.17	4.85
1962 AVGS.	327.45	16.36	13.59	4.15
1963 AVGS.	170.25	8.51	8.51	5.00
1964 AVGS.	164.34	7.88	7.88	4.79
1965 AVGS.	302.56	15.09	11.77	3.89
1966 AVGS.	200.09	10.00	10.00	5.00
1967 AVGS.	231.79	11.56	10.96	4.73
1968 AVGS.	263.51	13.15	9.97	3.79
1969 AVGS.	328.10	16.40	11.84	3.61
1970 AVGS.	283.20	14.16	11.44	4.04
1971 AVGS.	179.09	8.91	8.91	4.97
1972 AVGS.	179.49	8.97	8.97	5.00
1973 AVGS.	251.69	12.58	12.44	4.94
1974 AVGS.	176.80	9.11	8.92	4.53
1975 AVGS.	161.54	8.02	8.02	4.97
1976 AVGS.	139.91	7.00	7.00	5.00
1977 AVGS.	213.29	10.66	10.66	5.00
1978 AVGS.	263.77	12.63	11.33	4.30
1979 AVGS.	257.08	12.85	11.59	4.51
1980 AVGS.	143.03	6.71	6.71	4.69
1981 AVGS.	182.08	9.03	9.03	4.96
1982 AVGS.	370.44	18.52	10.77	2.91
1983 AVGS.	326.53	16.33	10.88	3.33
1984 AVGS.	127.35	5.78	5.78	4.54
1985 AVGS.	127.35	5.78	5.78	4.54

See Table F.5-5 for summary of supplemental storage operation.

TABLE F.5-4

AVERAGE PERCENT WITHDRAWAL BY MONTH
5 PERCENT AVERAGE/5 PERCENT MAXIMUM WITHDRAWAL CRITERIA

Month	Average Flow (cfs)	Average Draw (cfs)	Average Percent of Flow
November	4.20	83.28	4.98
December	3.36	68.88	4.80
January	5.28	107.52	4.95
February	5.76	120.72	4.80
March	5.64	154.56	3.68
April	3.84	82.80	4.68
May	1.44	27.60	5.00
June	8.76	175.92	5.00
July	19.80	429.12	4.60
August	24.00	637.80	3.77
September	21.12	691.68	3.05
October	12.24	381.60	3.22

49 Year Average Draw	9.62
49 Year Average Flow	246.80
49 Year Average Percent	3.90

TABLE F.5-5

SUPPLEMENTAL PUMPING FROM SECONDARY/FLORIDAN AQUIFERS
WITHDRAWAL CRITERIA = 5 PERCENT AVERAGE/5 PERCENT MAXIMUM

Year	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct
1937	--	--	--	1	--	--	--	--	--	--	--	--
1938	--	--	--	--	--	--	--	--	--	--	--	--
1939	--	--	1	--	1	--	--	1,2	--	--	--	--
1940	--	--	1	--	--	--	--	--	--	--	--	--
1941	1,2	2	1,2	--	1	--	--	--	--	--	--	--
1942	1,2	2	1,2	2	1	--	--	--	--	--	--	--
1943	1,2	2	1,2	2	1,2	2	1,2	2	--	--	--	--
1944	--	--	1	--	1	--	1	2	--	--	--	--
1945	1,2	2	1,2	2	1,2	2	1,2	2	--	--	--	--
1946	--	--	1	--	1	--	--	1,2	--	--	--	--
1947	1,2	2	1,2	2	1,2	--	1	2	--	--	--	--
1948	--	--	--	--	--	--	--	--	--	--	--	--
1949	--	--	1	--	1	--	--	1,2	--	--	--	--
1950	--	--	1	--	--	--	1	2	--	--	--	--
1951	1,2	2	1,2	2	1,2	2	1,2	2	--	--	--	--
1952	1,2	2	1,2	2	1,2	2	1,2	2	--	--	--	--
1953	1,2	2	1,2	2	1	--	--	--	--	--	--	--
1954	--	--	--	--	--	--	--	--	--	--	--	--
1955	--	1	--	1	--	--	--	--	--	--	--	--
1956	1,2	2	1,2	2	1,2	2	1,2	2	--	--	--	--
1957	1,2	2	1,2	2	1,2	2	1,2	2	--	--	--	--
1958	--	--	1	--	--	--	--	--	--	--	--	--
1959	1,2	2	1	--	--	--	--	--	--	--	--	--
1960	--	--	--	--	--	--	--	--	--	--	--	--
1961	--	--	1	--	--	--	--	--	--	--	--	--
1962	1,2	2	1,2	2	1,2	2	1,2	2	--	--	--	--
1963	--	--	1	--	--	--	--	--	--	--	--	--
1964	1,2	2	1,2	--	--	--	--	--	--	--	--	--
1965	1,2	2	1,2	2	1,2	2	1,2	2	--	--	--	--
1966	--	--	1	--	--	--	--	--	--	--	--	--
1967	1,2	--	1,2	--	1	--	--	1,2	--	--	--	--
1968	--	--	1	--	1	--	--	1,2	--	--	--	--
1969	--	--	1	--	--	--	--	--	--	--	--	--
1970	--	--	--	--	--	--	--	--	--	--	--	--
1971	--	1	--	1	--	1	--	1	--	--	--	--
1972	1,2	--	--	--	--	--	--	--	--	--	--	--
1973	1,2	2	1,2	--	--	--	--	--	--	--	--	--
1974	--	1	--	1	--	1	--	1,2	--	--	--	--
1975	1,2	2	1,2	--	1	--	1	2	--	--	--	--
1976	1,2	2	1,2	--	1	--	1	--	--	--	--	--
1977	1,2	2	1,2	2	1,2	--	1,2	2	--	--	--	--
1978	1,2	2	--	--	--	--	--	--	--	--	--	--
1979	1,2	--	1	--	--	--	--	--	--	--	--	--
1980	--	-	1	--	--	--	--	--	--	--	--	--
1981	1,2	2	1,2	2	1,2	--	1,2	2	--	--	--	--
1982	1,2	2	1,2	--	1	--	---	--	--	--	--	--

TABLE F.5-5 (Continued)

Year	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct
1983	--	--	1	--	--	--	--	--	--	--	--	--
1984	--	--	--	--	--	--	--	--	--	--	--	--
1985	1,2	2	1,2	2	1	--	--	--	--	--	--	--
Number of Months of Pumping												
Secondary	25	3	37	4	24	2	15	7	0	0	0	0
Floridan	25	22	21	14	11	8	10	20	0	0	0	0

Note: 1 = Secondary Aquifer pumping 3.0 mgd.
2 = Floridan Aquifer pumping 3.7 mgd.

TABLE F.5-6

RESERVOIR STORAGE
5 PERCENT AVERAGE/15 PERCENT MAXIMUM WITHDRAWAL CRITERIA
(WITH SUPPLEMENTAL SUPPLY)

Year	Maximum Reservoir Storage (acre-feet)
1936	1860.82
1937	3397.23
1938	4586.30
1939	2393.24
1940	2257.14
1941	1221.02
1942	4759.40
1943	4729.93
1944	6181.73
1945	4801.09
1946	2546.04
1947	3179.69
1948	4245.37
1949	4776.06
1950	4485.22
1951	5468.95
1952	1805.72
1953	1618.84
1954	3151.18
1955	6017.28
1956	2331.18
1957	1300.68
1958	0.00
1959	1061.03
1960	2646.74
1961	4337.37
1962	1748.93
1963	2191.74
1964	4380.27
1965	1217.66
1966	4364.96
1967	4129.49
1968	966.69
1969	501.97
1970	4686.05
1971	1504.95
1972	1452.68
1973	4531.50
1974	4931.79
1975	4339.28

TABLE F.5-6 (Continued)

Year	Maximum Reservoir Storage (acre-feet)
1976	3055.56
1977	1940.40
1978	1506.68
1979	740.47
1980	4096.07
1981	1995.72
1982	457.41
1983	1852.19
1984	1914.66

49 years average of yearly storage required = 2931.76 in acre-feet

TABLE F.5-7

YEARLY SUMMARY OF RIVER WITHDRAWALS
WITHDRAWAL CRITERIA 5 PERCENT AVERAGE/15 PERCENT MAXIMUM
(WITH SUPPLEMENTAL SUPPLY)

YEAR	FLOW	ALLOWABLE WITHDRAWAL	ACTUAL WITHDRAWAL	% OF FLOW
1937 AVGS	240.90	35.14	12.09	5.02
1938 AVGS	276.80	41.31	12.09	4.37
1939 AVGS	398.87	59.77	11.70	2.93
1940 AVGS	171.94	25.72	12.09	7.03
1941 AVGS.	125.30	18.15	12.09	9.65
1942 AVGS.	207.43	28.85	10.61	5.12
1943 AVGS.	354.38	53.14	10.76	3.04
1944 AVGS.	44.98	4.76	4.76	10.58
1945 AVGS.	339.23	50.80	12.82	3.78
1946 AVGS.	131.33	19.56	11.21	8.54
1947 AVGS.	553.15	77.02	11.52	2.08
1948 AVGS.	372.73	55.90	12.35	3.31
1949 AVGS.	332.48	49.35	11.05	3.32
1950 AVGS.	126.42	18.93	11.65	9.21
1951 AVGS.	158.09	21.64	12.10	7.65
1952 AVGS.	140.40	19.28	12.43	8.85
1953 AVGS.	389.73	58.46	12.02	3.08
1954 AVGS.	230.97	34.65	11.94	5.17
1955 AVGS.	135.73	20.30	12.29	9.06
1956 AVGS.	103.68	13.80	10.51	10.13
1957 AVGS.	328.73	49.17	11.21	3.41
1958 AVGS.	261.98	38.00	10.12	3.86
1959 AVGS.	628.06	94.21	13.09	2.08
1960 AVGS.	542.70	78.32	11.84	2.18
1961 AVGS.	106.59	15.52	11.50	10.78
1962 AVGS.	327.45	46.30	10.40	3.18
1963 AVGS.	170.25	25.54	12.28	7.21
1964 AVGS.	164.34	23.64	11.07	6.74
1965 AVGS.	302.56	45.27	10.82	3.58
1966 AVGS.	200.09	30.01	12.33	6.16
1967 AVGS.	231.79	34.67	11.21	4.84
1968 AVGS.	263.51	39.41	10.95	4.16
1969 AVGS.	328.10	49.21	11.93	3.64
1970 AVGS.	283.20	42.48	12.06	4.26
1971 AVGS.	179.09	26.72	10.00	5.58
1972 AVGS.	179.49	26.92	12.27	6.84
1973 AVGS.	251.69	37.75	11.76	4.67
1974 AVGS.	196.80	27.23	8.59	4.36
1975 AVGS.	161.54	24.07	10.79	6.68
1976 AVGS.	139.91	20.99	12.38	8.85
1977 AVGS.	213.29	31.99	12.22	5.73
1978 AVGS.	263.77	37.90	11.58	4.39
1979 AVGS.	257.08	37.71	12.26	4.77
1980 AVGS.	143.03	20.14	12.05	8.42
1981 AVGS.	182.08	27.10	11.77	6.47
1982 AVGS.	370.44	55.57	11.35	3.07
1983 AVGS.	326.53	48.98	11.69	3.58
1984 AVGS.	127.35	16.21	9.33	7.32
1985 AVGS.	127.35	16.21	11.53	9.05

See Table F.5-9 for summary of supplemental storage operation.

TABLE F.5-8

AVERAGE PERCENT WITHDRAWAL BY MONTH
5 PERCENT AVERAGE/15 PERCENT MAXIMUM WITHDRAWAL CRITERIA

Month	Average Flow (cfs)	Average Draw (cfs)	Percent of Flow
November	8.16	83.28	9.78
December	5.88	68.88	8.56
January	7.20	107.52	6.65
February	8.76	120.72	7.25
March	8.16	154.56	5.31
April	8.16	82.80	9.78
May	3.84	27.60	13.84
June	16.56	175.92	9.41
July	23.52	429.12	5.47
August	22.44	637.80	3.52
September	11.40	691.68	1.64
October	12.84	381.60	3.86

49 Year Average Draw	11.40
49 Year Average Flow	243.80
49 Year Average Percent	4.62

TABLE F.5-9

SUPPLEMENTAL PUMPING FROM SECONDARY/FLORIDAN AQUIFERS
WITHDRAWAL CRITERIA = 5 PERCENT AVERAGE/15 PERCENT MAXIMUM

Year	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct
1937	--	--	--	--	--	--	--	--	--	--	--	--
1938	--	--	--	--	--	--	--	--	--	--	--	--
1939	--	--	--	--	--	1	--	1,2	--	--	--	--
1940	--	--	--	--	--	--	--	--	--	--	--	--
1941	--	--	--	--	--	--	--	--	--	--	--	--
1942	--	--	--	--	--	--	--	--	--	--	--	--
1943	--	1	2	1	--	1	2	1,2	--	--	--	--
1944	--	--	--	1	--	1	2	1,2	--	--	--	--
1945	1,2	2	1,2	2	1,2	2	1,2	2	--	--	--	--
1946	--	--	--	1	--	--	1,2	--	--	--	--	--
1947	--	--	--	1	--	--	--	--	--	--	--	--
1948	--	--	--	--	--	--	--	--	--	--	--	--
1949	--	--	--	--	--	--	1,2	2	--	--	--	--
1950	--	--	--	--	--	--	1	2	--	--	--	--
1951	--	--	--	--	--	--	--	--	--	--	--	--
1952	--	--	--	--	--	--	--	--	--	--	--	--
1953	--	--	--	--	--	--	--	--	--	--	--	--
1954	--	--	--	--	--	--	--	--	--	--	--	--
1955	--	--	--	--	--	--	--	--	--	--	--	--
1956	--	--	--	1	--	1	2	1,2	--	--	--	--
1957	--	--	--	1	--	--	--	--	--	--	--	--
1958	--	--	--	--	--	--	--	--	--	--	--	--
1959	--	--	--	--	--	--	--	--	--	--	--	--
1960	--	--	--	--	--	--	--	--	--	--	--	--
1961	--	--	--	--	--	--	--	--	--	--	--	--
1962	--	--	1,2	--	1	2	--	1,2	--	--	--	--
1963	--	--	--	--	--	--	--	--	--	--	--	--
1964	--	--	--	--	--	--	--	--	--	--	--	--
1965	--	--	1,2	--	--	--	--	1,2	--	--	--	--
1966	--	--	--	--	--	--	--	--	--	--	--	--
1967	--	--	--	--	--	--	--	1,2	--	--	--	--
1968	--	--	--	1	--	--	--	1,2	--	--	--	--
1969	--	--	--	--	--	--	--	--	--	--	--	--
1970	--	--	--	--	--	--	--	--	--	--	--	--
1971	--	--	--	1	--	1	2	1,2	--	--	--	--
1972	--	--	--	--	--	--	--	--	--	--	--	--
1973	--	--	--	--	--	--	--	--	--	--	--	--
1974	--	--	--	1	--	1	--	1,2	--	--	--	--
1975	--	1,2	--	1	--	1,2	2	1,2	--	--	--	--
1976	--	--	--	--	--	--	--	--	--	--	--	--
1977	--	--	--	--	--	--	--	--	--	--	--	--
1978	--	--	--	--	--	--	--	--	--	--	--	--
1979	--	--	--	--	--	--	--	--	--	--	--	--
1980	--	--	--	--	--	--	--	--	--	--	--	--
1981	--	--	--	--	--	--	--	--	--	--	--	--
1982	--	--	--	--	--	--	--	--	--	--	--	--

Year	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct
1983	--	--	--	--	--	--	--	--	--	--	--	--
1984	--	--	--	--	--	--	--	--	--	--	--	--
1985	1,2	--	--	--	--	--	--	--	--	--	--	--
Number of Months of Pumping												
Secondary	2	2	2	10	2	7	4	11	0	0	0	0
Floridan	2	2	4	2	1	3	8	14	0	0	0	0

Note: 1 = Secondary Aquifer pumping 3.0 mgd.
2 = Floridan Aquifer pumping 3.7 mgd.

TABLE F.2-7

MYAKKA RIVER MINIMUM FLOW CRITERIA CALCULATION
MONTHLY MEAN STREAMFLOW DATA (cfs)
(USGS Station at Highway 72)

Water Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep
1963	333.00	49.70	17.20	20.00	205.00	252.00	3.90	0.02[a]	79.20	191.00	350.00	583.00
1964	292.00	125.00	66.80	271.00	425.00	181.00	154.00	13.40	2.65[a]	26.60[a]	250.00[a]	402.00
1965	54.60[a]	3.24[a]	5.77[a]	12.40[a]	25.00	79.40	2.79[a]	0.09[a]	305.00	1027.00	1370.00	391.00[a]
1966	409.00	47.50	21.50	85.20	306.00	110.00	58.60	2.28	321.00	369.00	467.00	209.00[a]
1967	404.00	69.20	16.50	7.22[a]	22.10	14.70[a]	0.70[a]	0.00[a]	398.00	319.00	996.00	525.00
1968	413.00	23.10	18.70	12.10[a]	5.74[a]	11.60[a]	0.82[a]	1.01	608.00	1258.00	341.00	759.00
1969	123.00	181.00	40.20	123.00	50.50	543.00	84.60	4.89	343.00	225.00	795.00	983.00
1970	564.00	193.00	160.00	367.00	161.00	600.00	272.00	30.40	522.00	165.00	358.00	420.00
1971	150.00	11.40[a]	1.88[a]	3.63[a]	21.40[a]	5.21[a]	0.51[a]	0.00[a]	0.00[a]	61.10[a]	634.00[a]	892.00
1972	518.00	220.00	275.00	37.10	218.00	29.50	43.00	48.10	235.00	96.80[a]	170.00	702.00
1973	80.40[a]	48.60	136.00	550.00	603.00	158.00	203.00	19.30	3.69[a]	99.70	324.00[a]	584.00
1974	291.00	14.70[a]	7.21[a]	8.17[a]	16.50[a]	15.60[a]	3.84	1.53	171.00	1221.00	729.00	166.00[a]
1975	7.09[a]	0.66[a]	8.53[a]	24.90	9.04[a]	3.30[a]	0.15[a]	0.00[a]	0.00[a]	472.00	512.00	455.00
1976	453.00	181.00	13.40[a]	28.40	22.10	21.90	4.74	6.03	28.40	184.00	763.00	311.00[a]
1977	115.00	88.20	80.70	75.40	64.50	37.40	9.22	4.08	5.95	122.00	914.00	902.00
1978	256.00	52.60	333.00	326.00	416.00	379.00	22.70	2.50	31.50	503.00	857.00	133.00[a]
1979	109.00[a]	18.40[a]	17.20	370.00	200.00	187.00	109.00	104.00	49.30	127.00	153.00[a]	1018.00
1980	732.00	86.10	43.80	32.60	176.00	192.00	168.00	92.50	98.20	63.20[a]	105.00[a]	434.00
1981	225.00	29.20	22.00	15.10	169.00	37.10	11.60	1.01	5.17[a]	16.80[a]	628.00	1140.00
1982	110.00[a]	41.80	16.50	16.30	18.70[a]	104.00	230.00	152.00	1277.00	775.00	560.00	458.00
Average Lowest Flow	72.0	9.67	7.36	8.70	14.3	10.1	0.99	0.02	2.30	52.9	200.4	242.0
Percent Multiplier	0.70	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.70	0.70	0.70
Minimum Flow	50.50	8.70	6.62	7.83	12.87	9.09	0.89	0.02	2.07	37.03	140.28	169.40

[a] Five lowest flows.

TABLE F.2-8

MONTHLY NET RAINFALL

Month	Average Net Rainfall (inches/month)
January	-0.50
February	-0.08
March	-0.04
April	-1.75
May	-1.41
June	1.46
July	2.09
August	2.64
September	2.21
October	-0.02
November	-0.27
December	-0.55

TABLE F.4-1

YEARLY STORAGE REQUIREMENTS
5 PERCENT AVERAGE/5 PERCENT MAXIMUM
WITHDRAWALS

Year	Maximum Reservoir Storage (acre-feet)
1936	3,548.16
1937	5,192.45
1938	5,952.86
1939	5,437.43
1940	7,098.79
1941	7,727.17
1942	12,294.42
1943	10,812.75
1944	18,558.99
1945	14,410.89
1946	17,339.77
1947	4,927.34
1948	7,385.85
1949	7,296.42
1950	11,932.19
1951	16,173.17
1952	16,214.52
1953	8,665.93
1954	11,870.37
1955	18,887.35
1956	19,782.23
1957	11,817.50
1958	8,514.53
1959	3,660.80
1960	6,273.89
1961	12,655.92
1962	6.909.47
1963	7,490.82
1964	13,172.32
1965	7,187.10
1966	10,369.10
1967	10,780.57
1968	5,355.38
1969	860.19
1970	8,618.04
1971	5,380.59
1972	5,122.74
1973	8,652.76
1974	10,635.63
1975	12,324.86

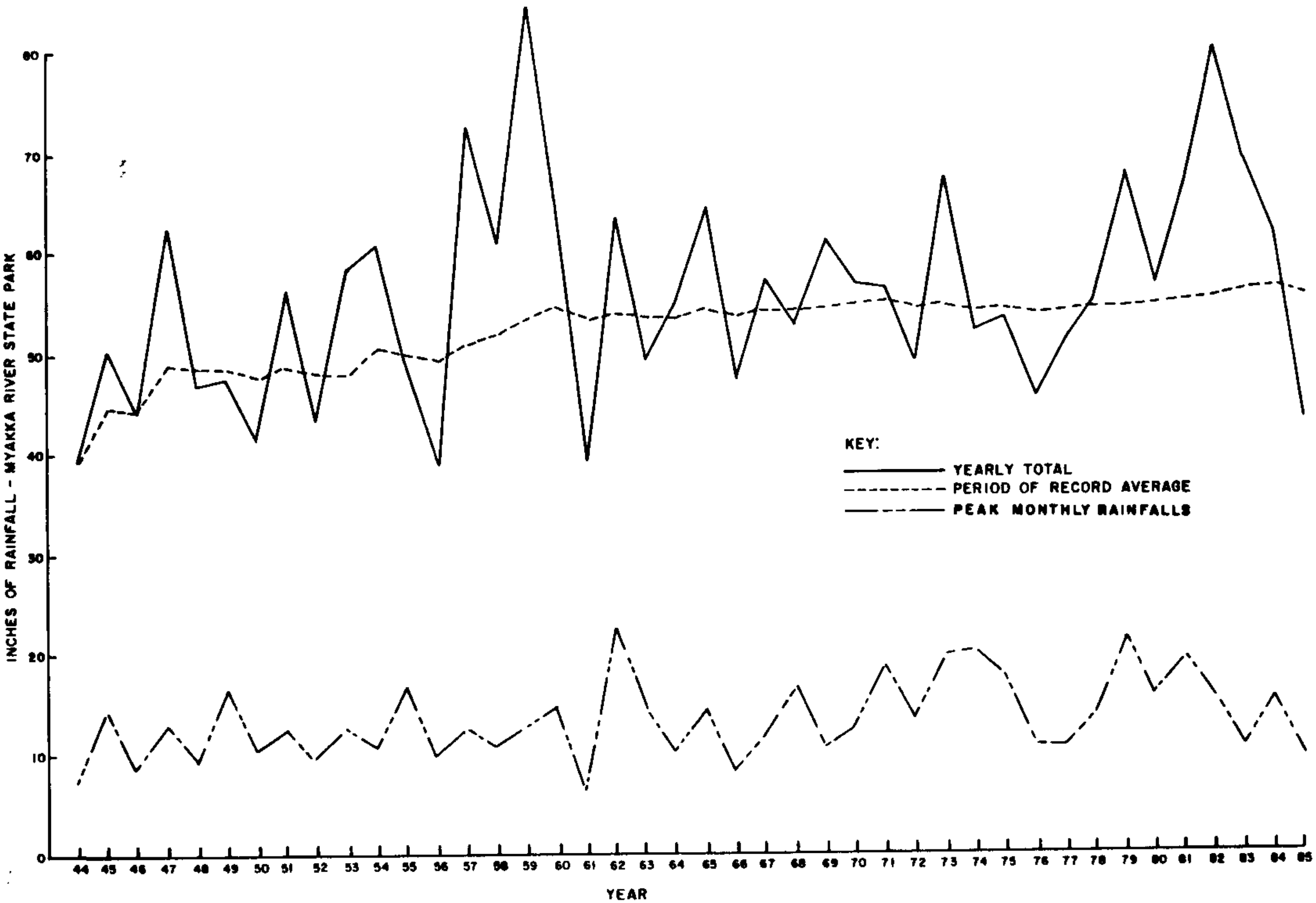

Figure F.2-1. Myakka State Park Rainfall Data Summary.

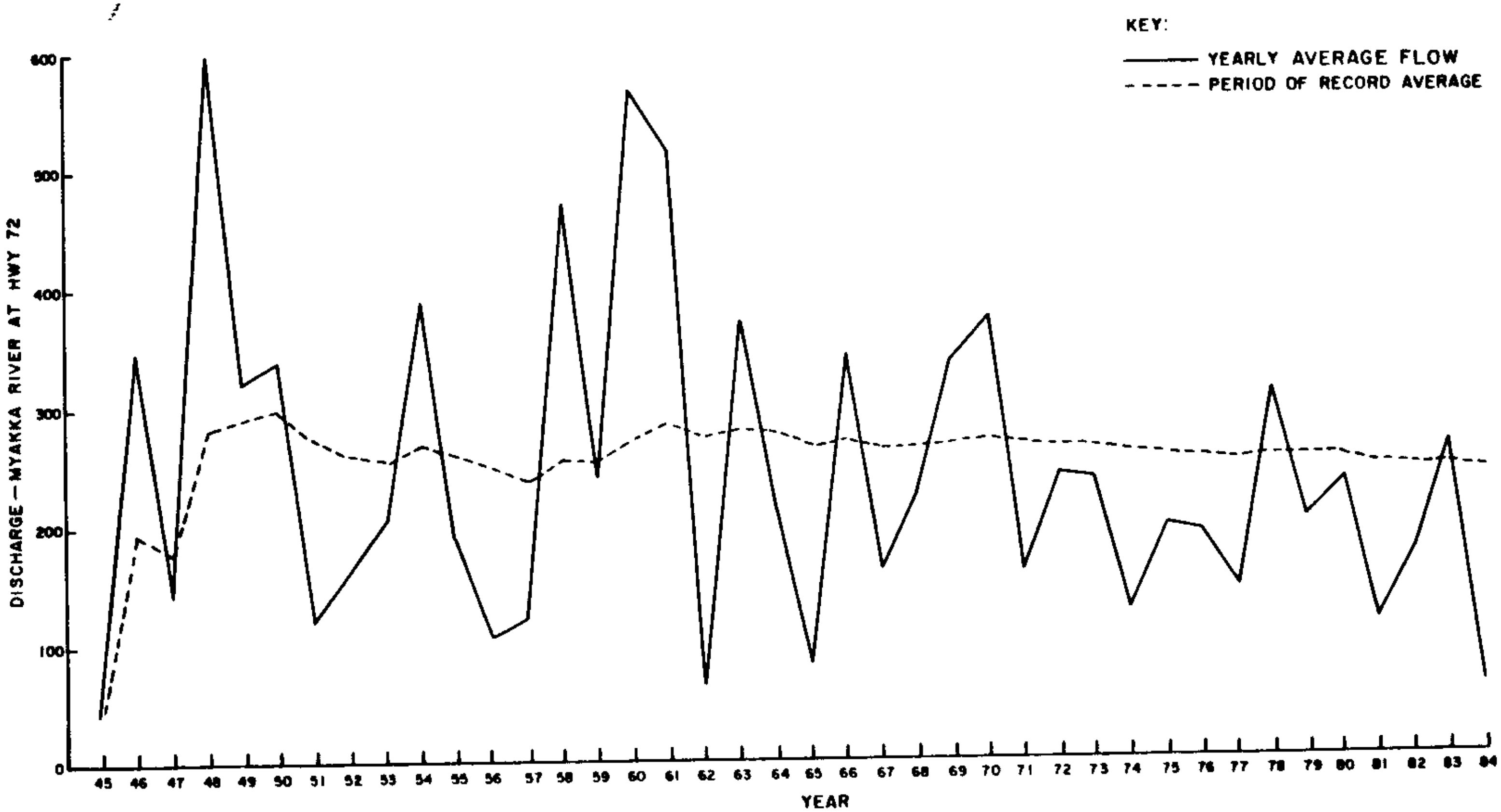

Figure F.2-2. Myakka River Discharge Data Summary.

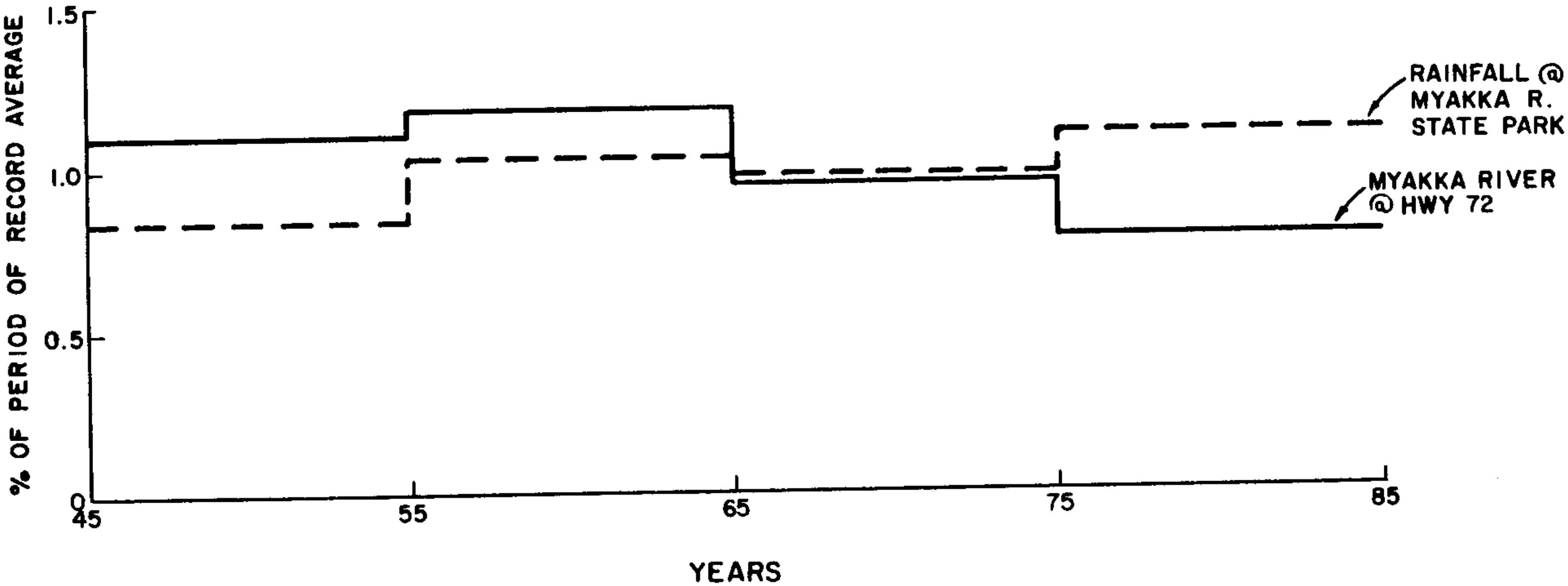

Figure F.2-3. Rainfall/Runoff Variation from Period of Record Average.

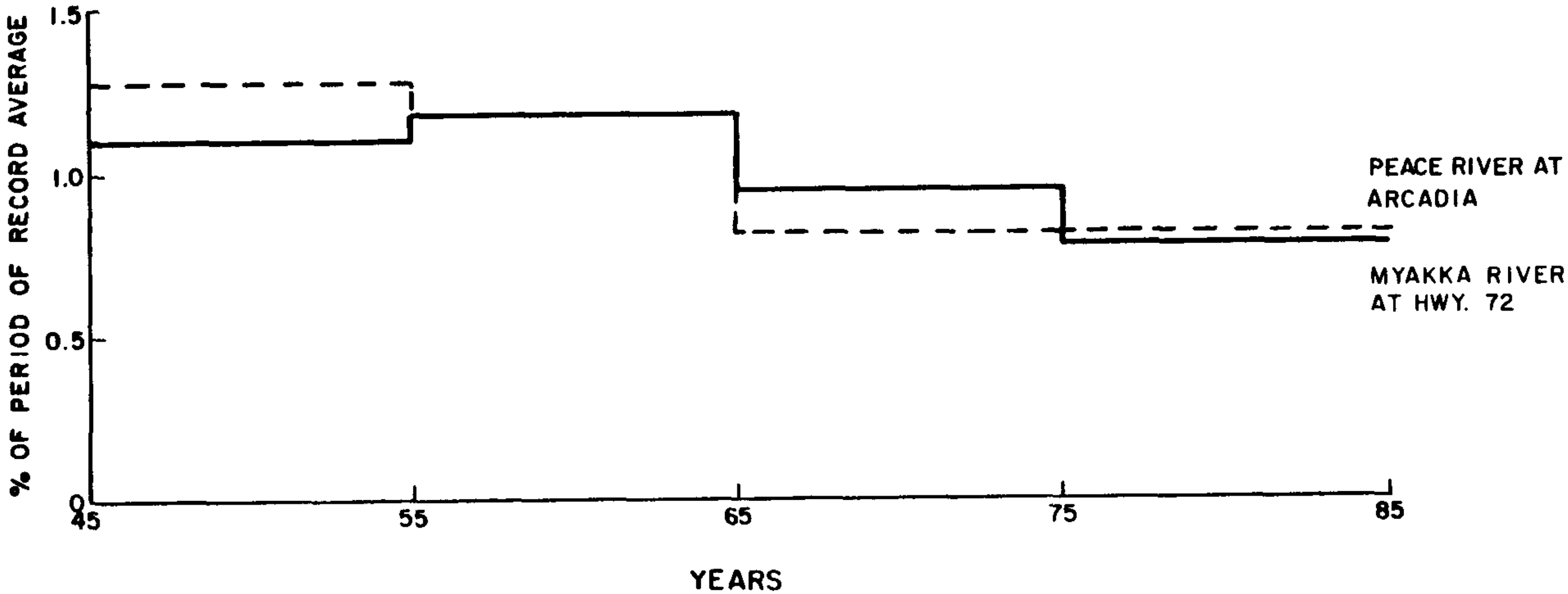

Figure F.2-4. Myakka River/Peace River Discharge Comparison.

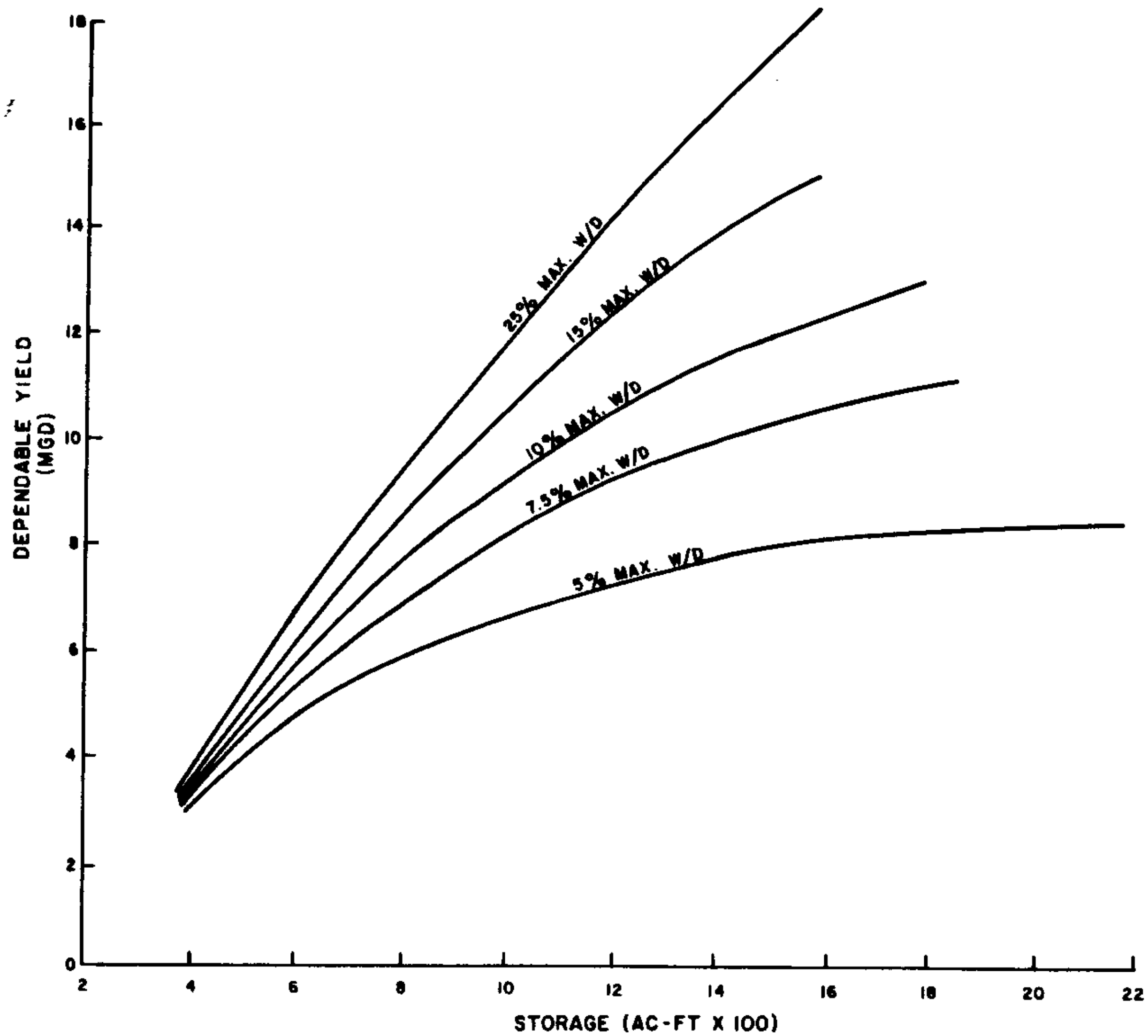

Figure F.3-1. Yield Versus Storage and Maximum % River Withdrawal.

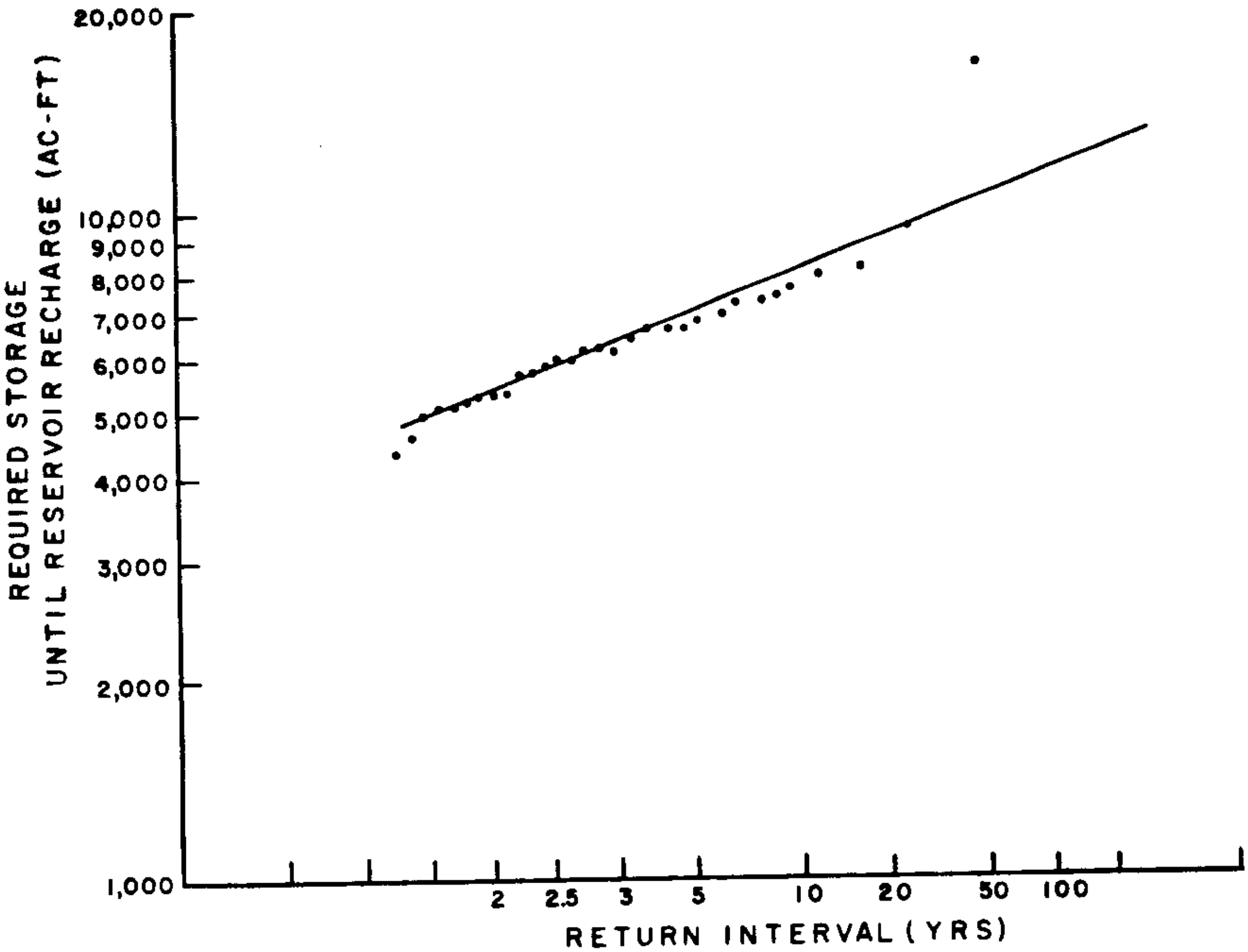

Figure F.5-1. Within Year Storage Supply: Oct.-Oct. 5% Avg./5% Maximum Withdrawal Criteria.

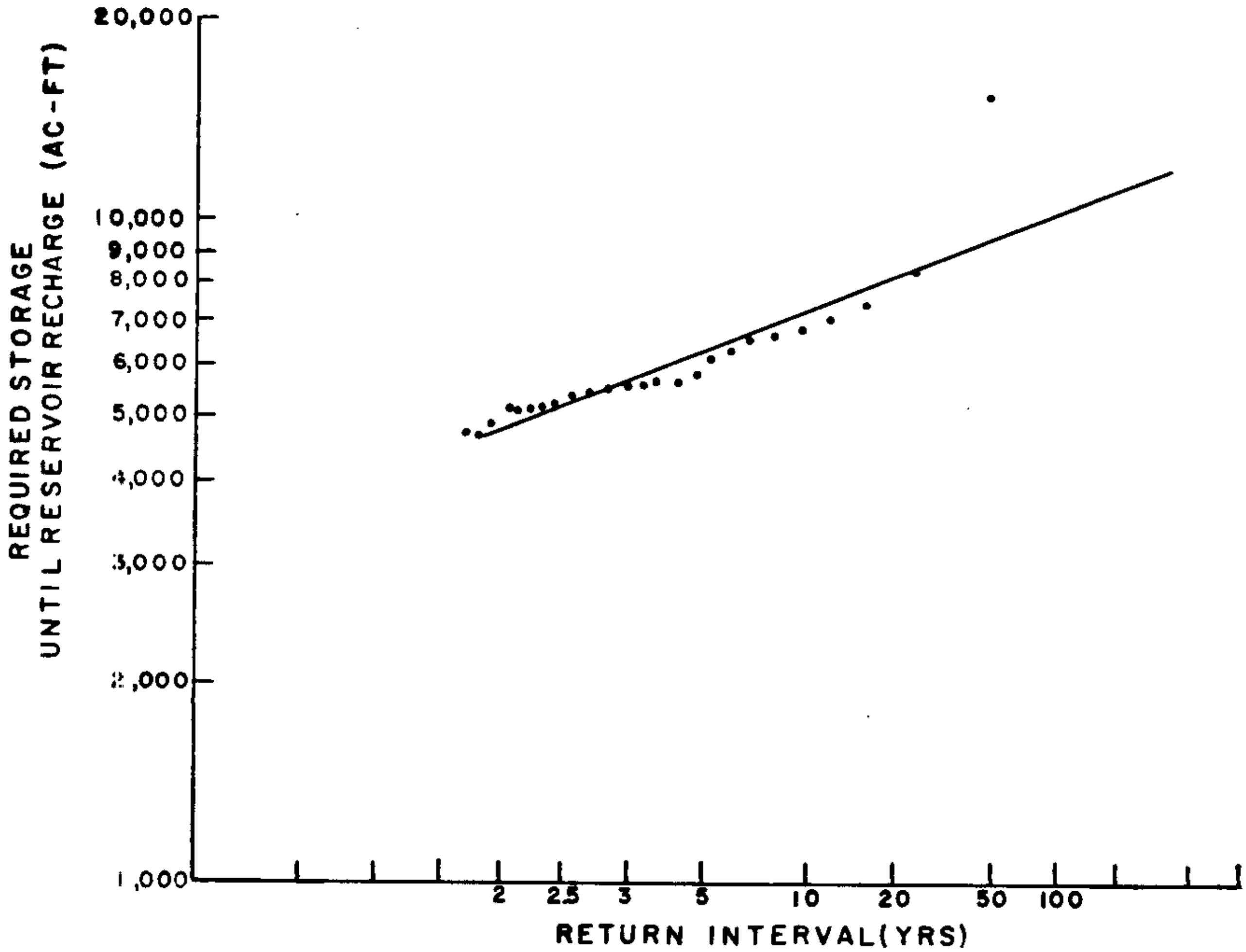

Figure F.5-2. Within Year Storage Supply: Nov.-Oct. 5% Avg./5% Maximum Withdrawal Criteria.

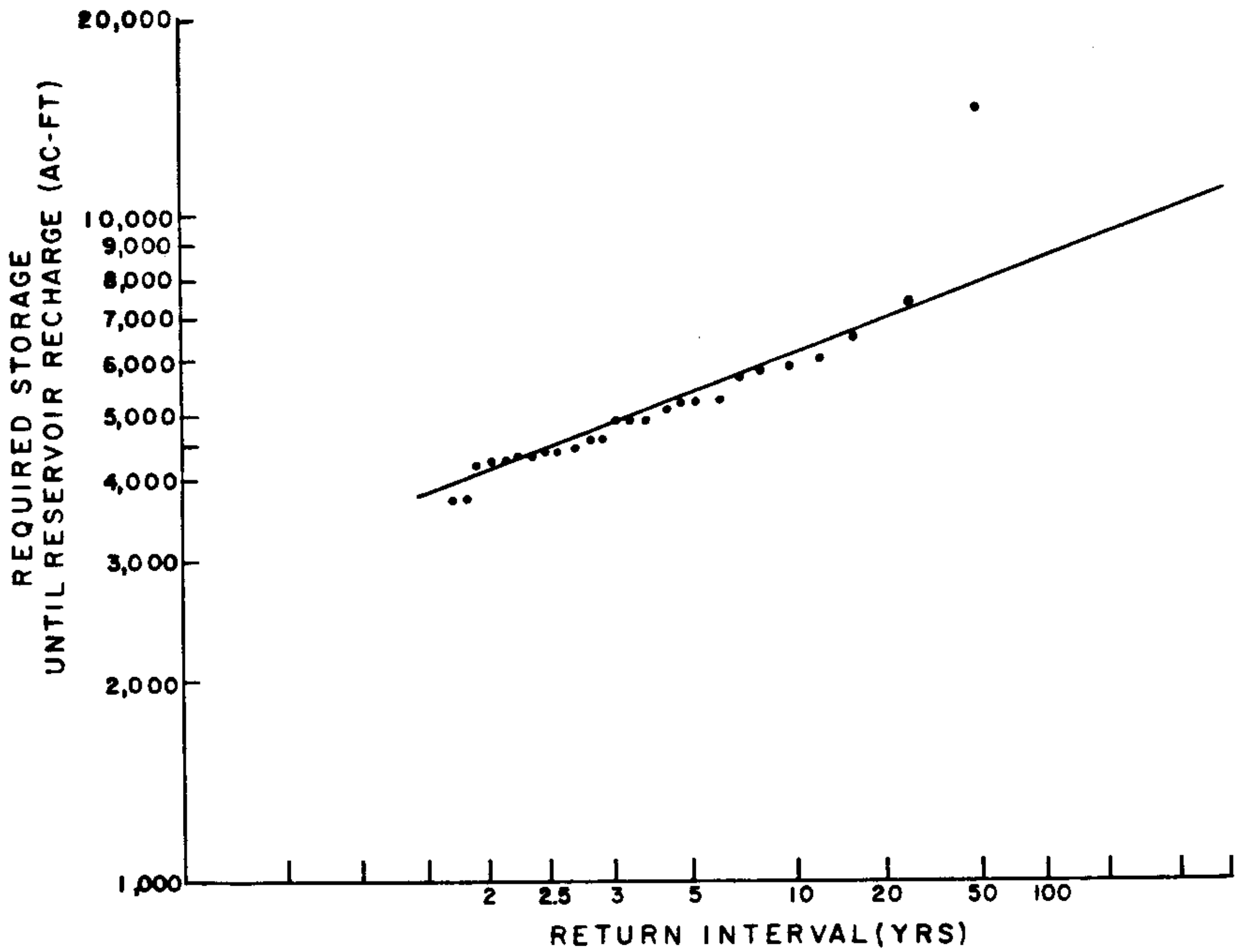

Figure F.5-3. Within Year Storage Supply: Dec.-Oct. 5% Avg./5% Maximum Withdrawal Criteria.

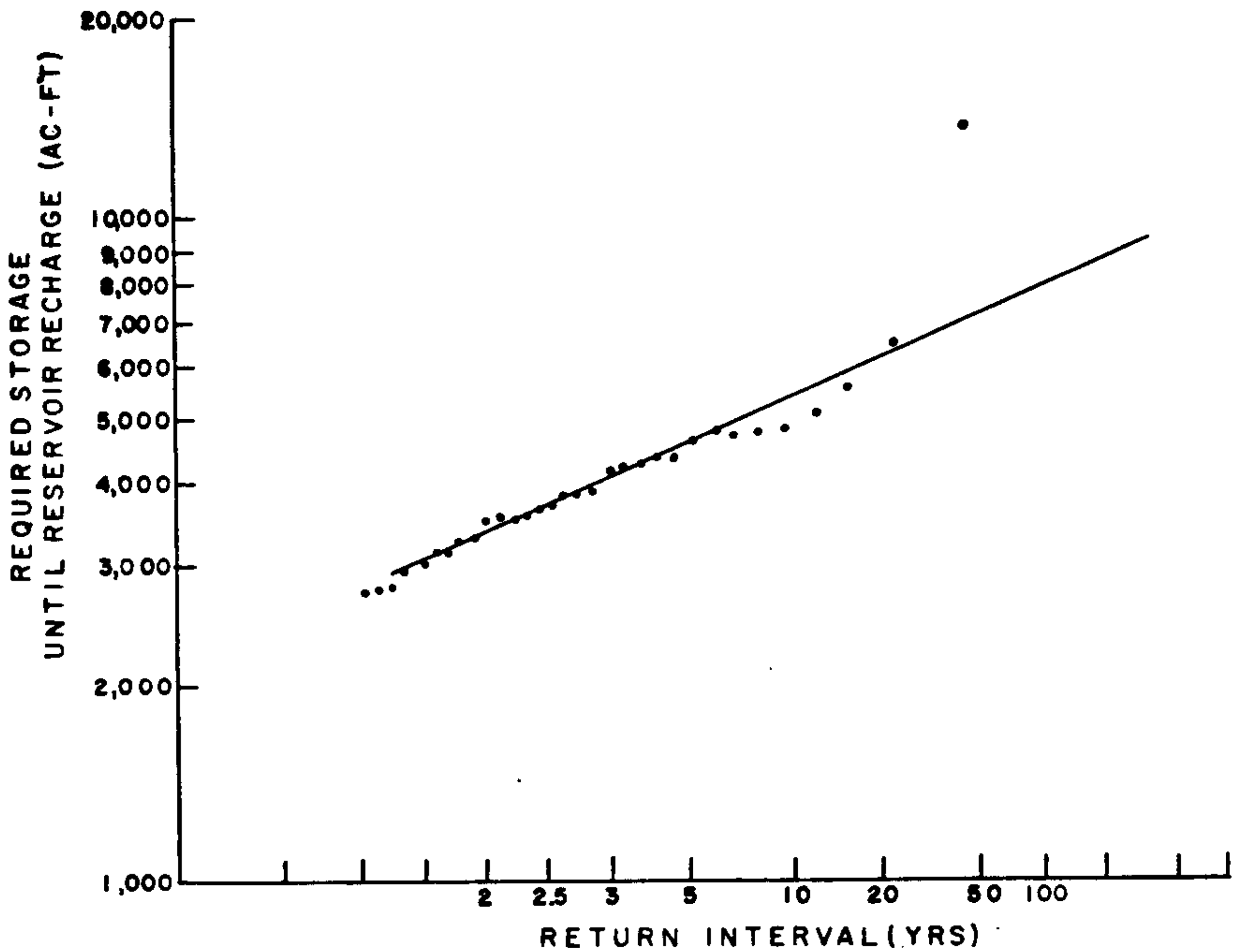

Figure F.5-4. Within Year Storage Supply: Jan.-Oct. 5% Avg./5% Maximum Withdrawal Criteria.

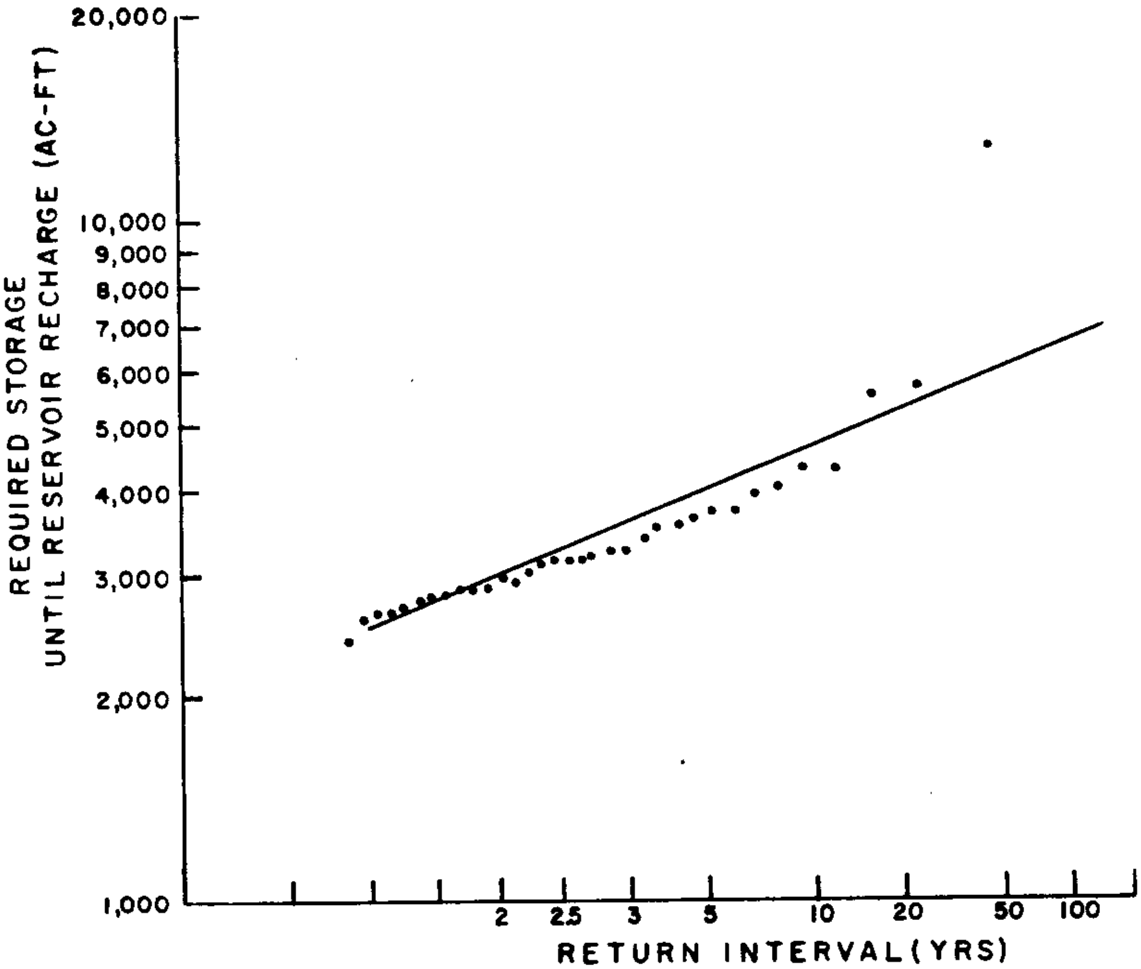

Figure F.5-5. Within Year Storage Supply: Feb.-Oct. 5% Avg./5% Maximum Withdrawal Criteria.

DATA TO BE PROVIDIED

Figure F.5-6. Within Year Storage Supply: Mar.-Oct. 5% Avg./5% Maximum Withdrawal Criteria.

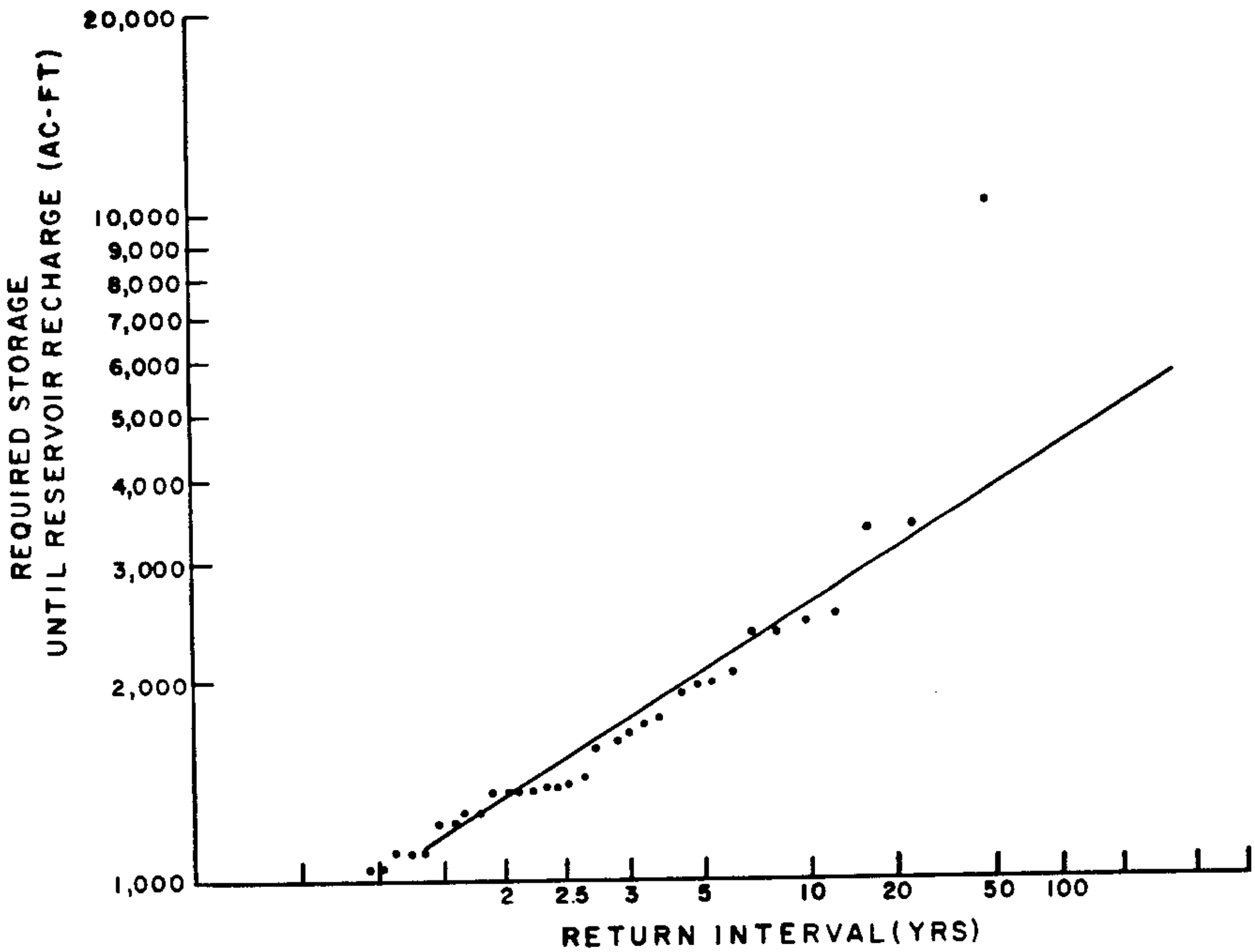

Figure F.5-7. Within Year Storage Supply: Apr.-Oct. 5% Avg./5% Maximum Withdrawal Criteria.

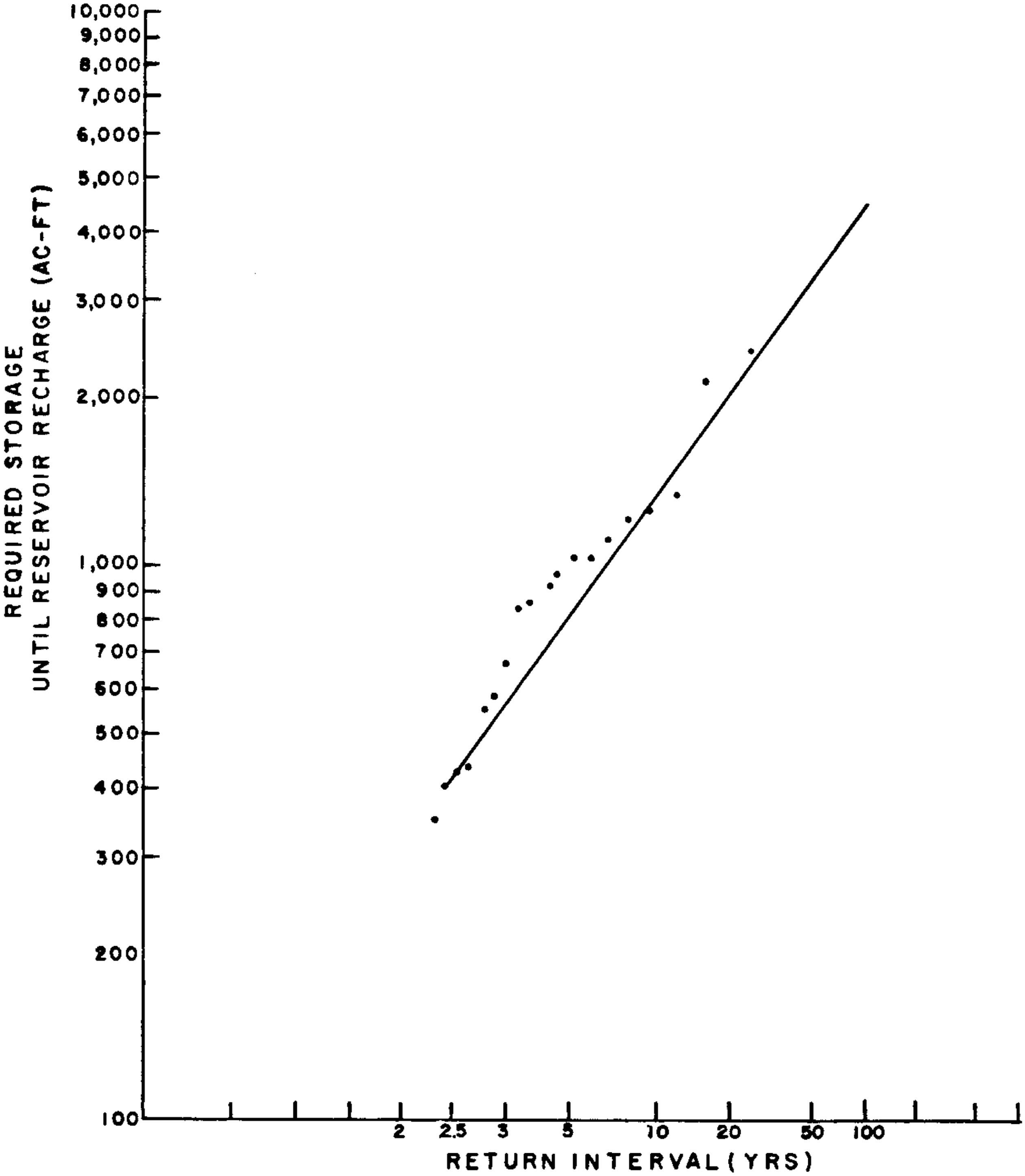

Figure F.5-8. Within Year Storage Supply: May-Oct. 5% Avg./5% Maximum Withdrawal Criteria.

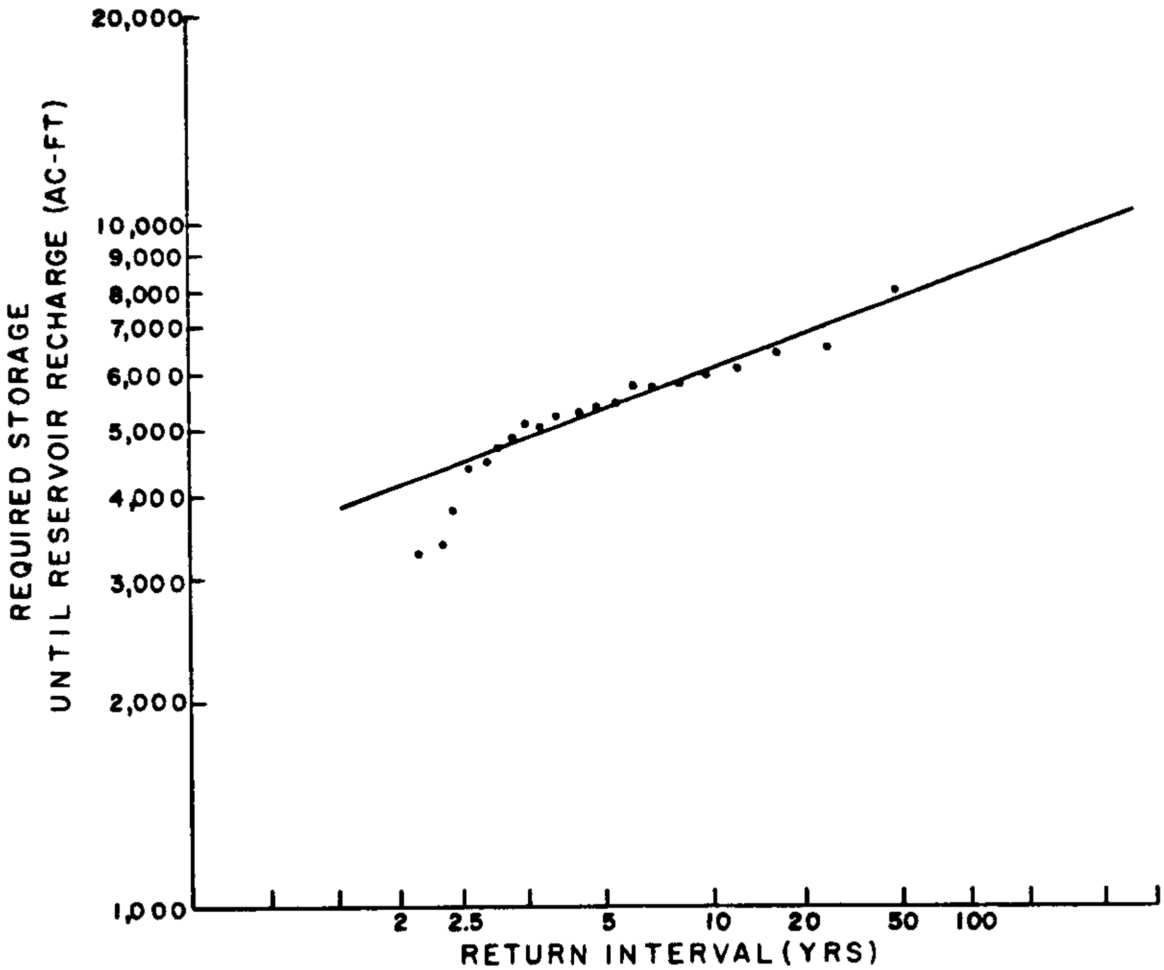

Figure F.5-9. Within Year Storage Supply: Oct.-Oct. 5% Avg./15% Maximum Withdrawal Criteria.

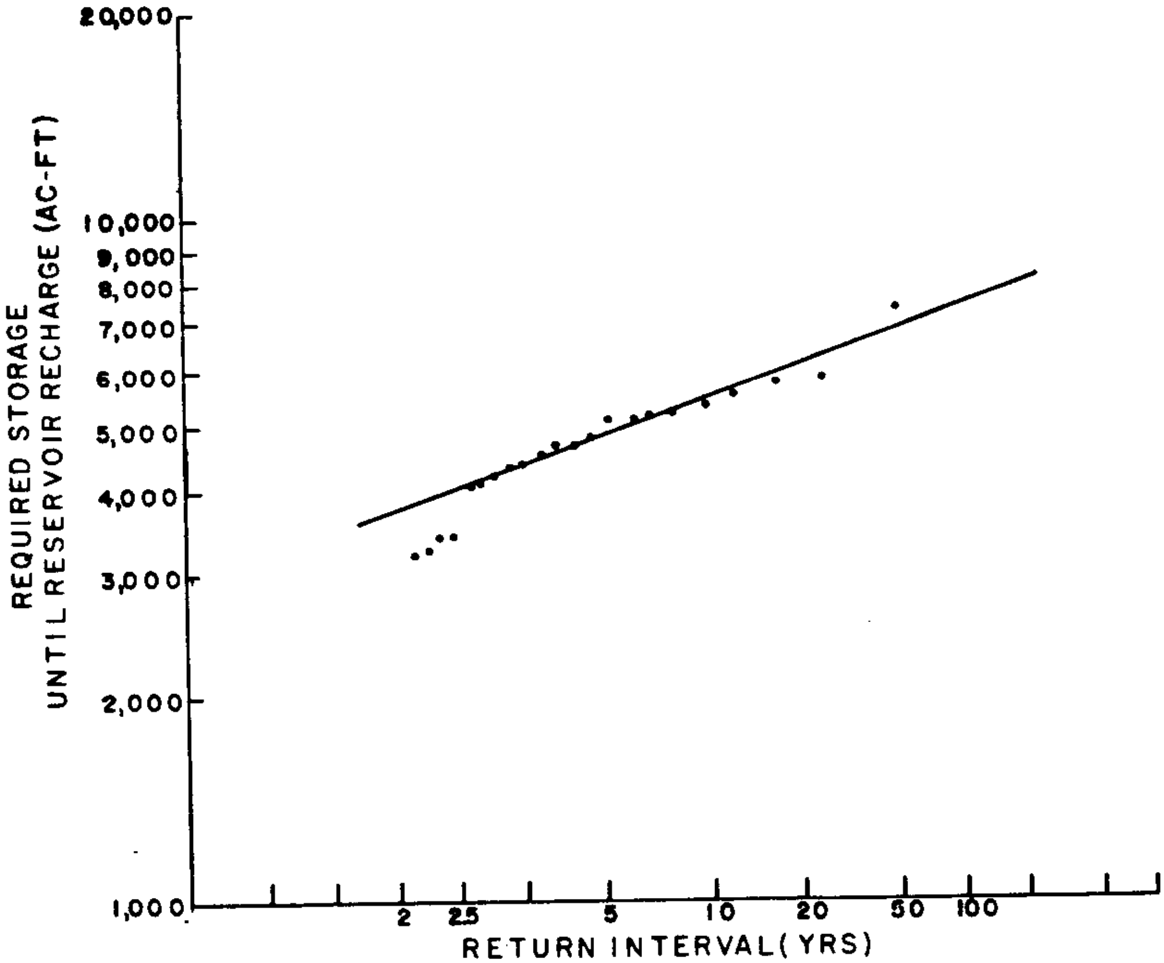

Figure F.5-10. Within Year Storage Supply: Nov.-Oct. 5% Avg./15% Maximum Withdrawal Criteria.

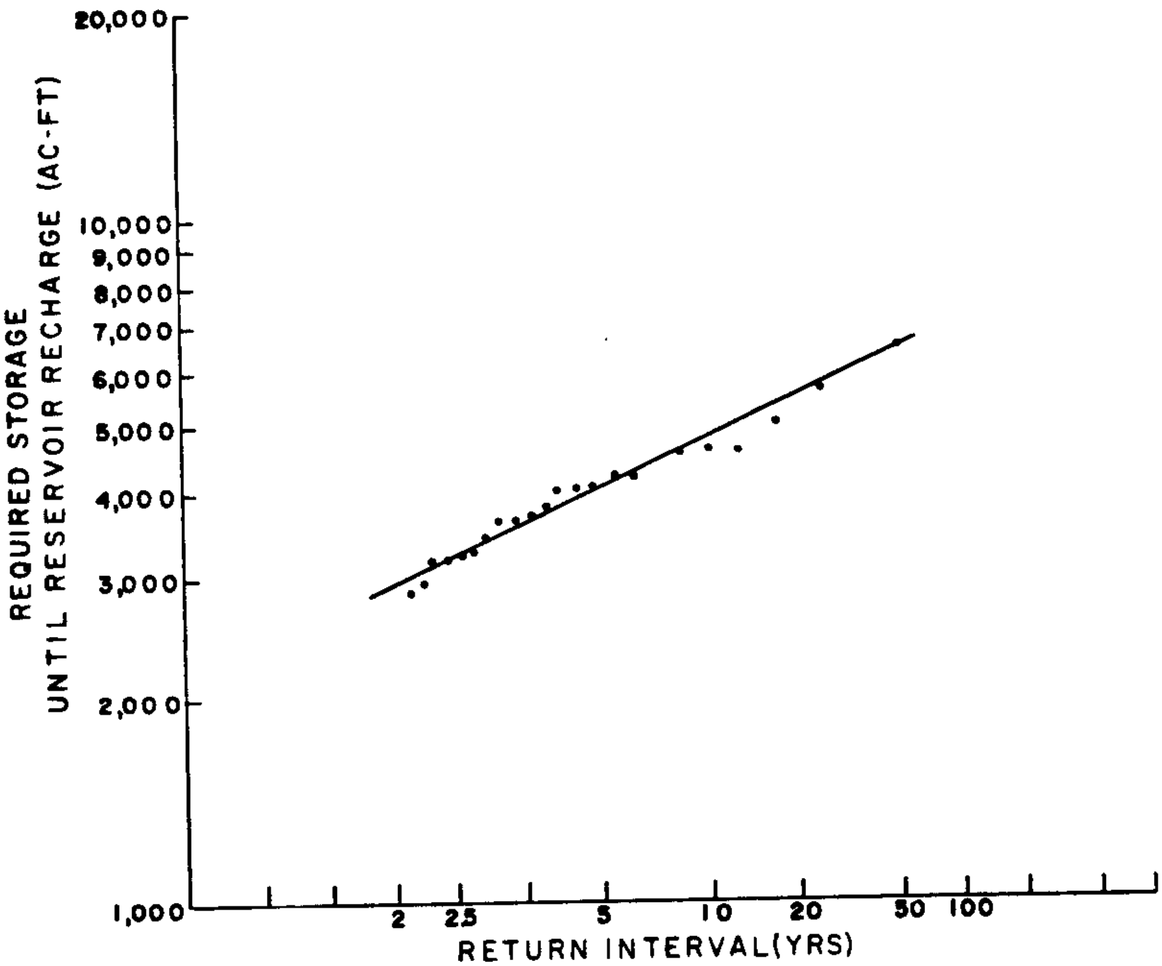

Figure F.5-11. Within Year Storage Supply: Dec.-Oct. 5% Avg./15% Maximum Withdrawal Criteria.

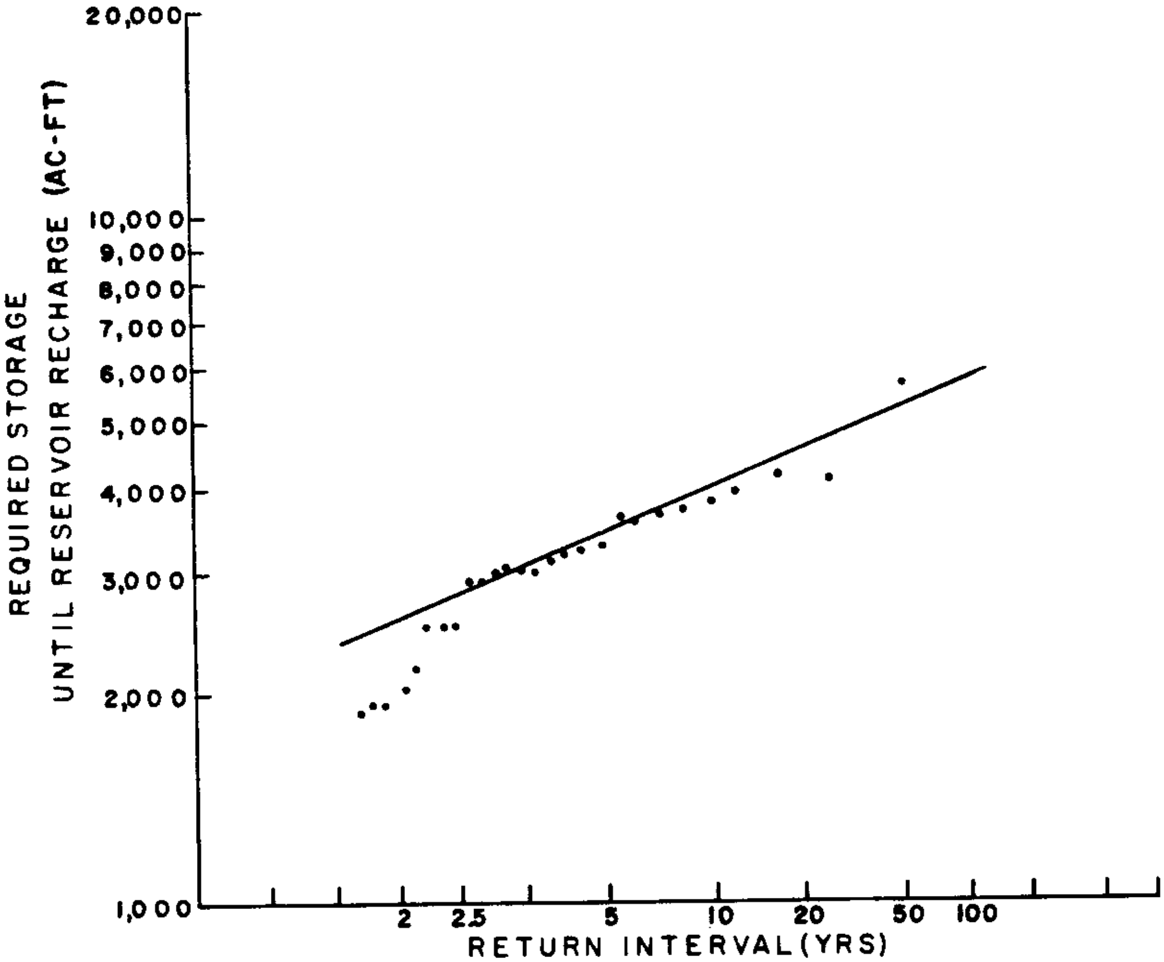

Figure F.5-12. Within Year Storage Supply: Jan.-Oct. 5% Avg./15% Maximum Withdrawal Criteria.

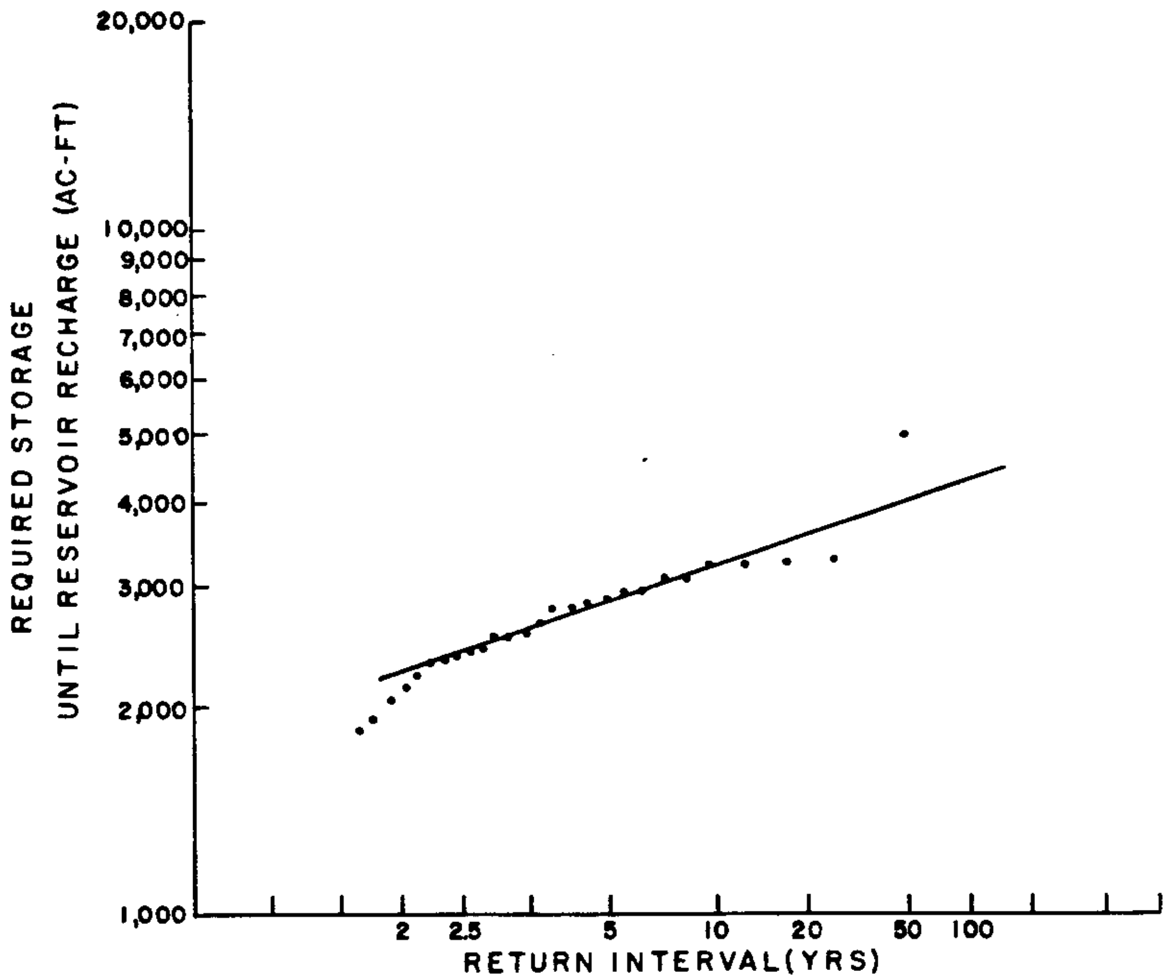

Figure F.5-13. Within Year Storage Supply: Feb.-Oct. 5% Avg./15% Maximum Withdrawal Criteria.

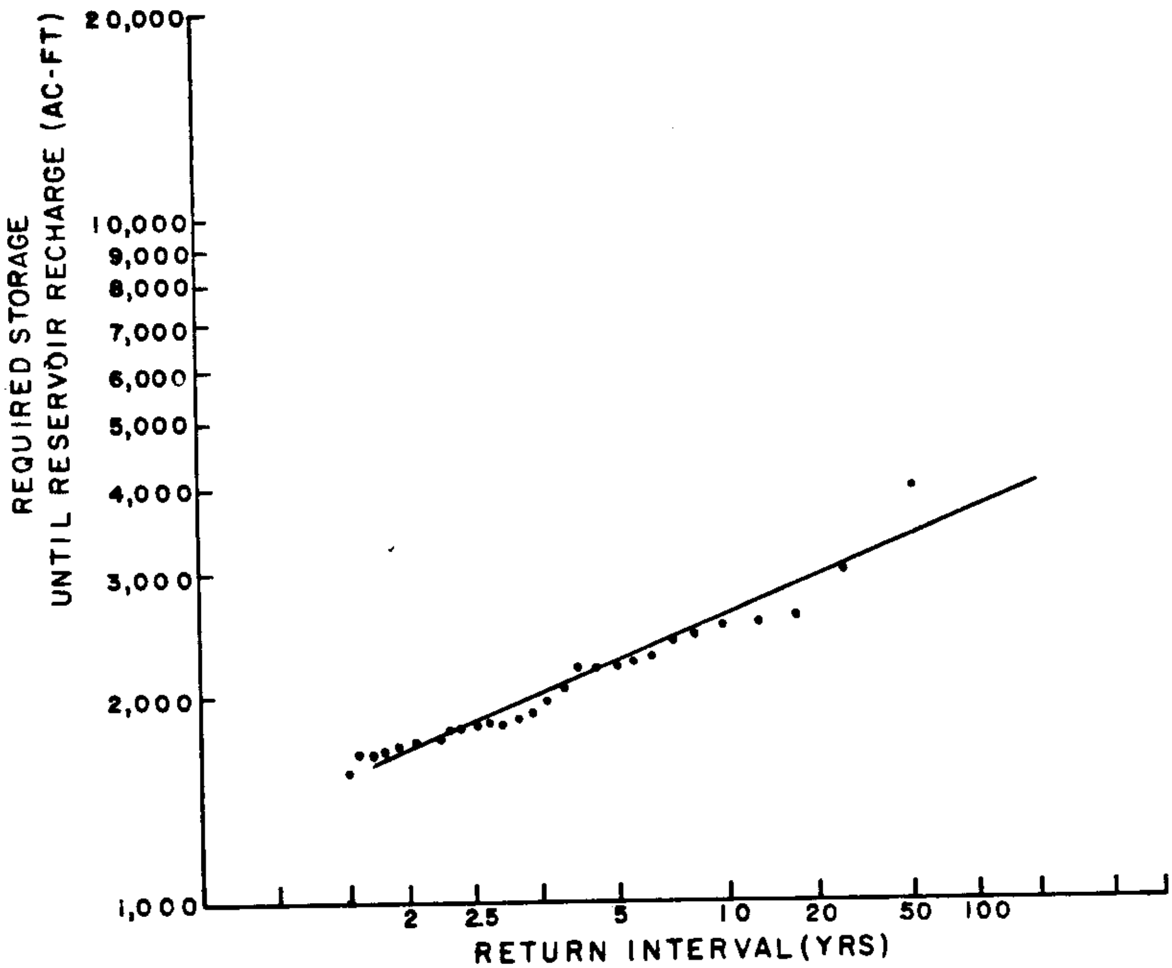

Figure F.5-14. Within Year Storage Supply: Mar.-Oct. 5% Avg./15% Maximum Withdrawal Criteria.

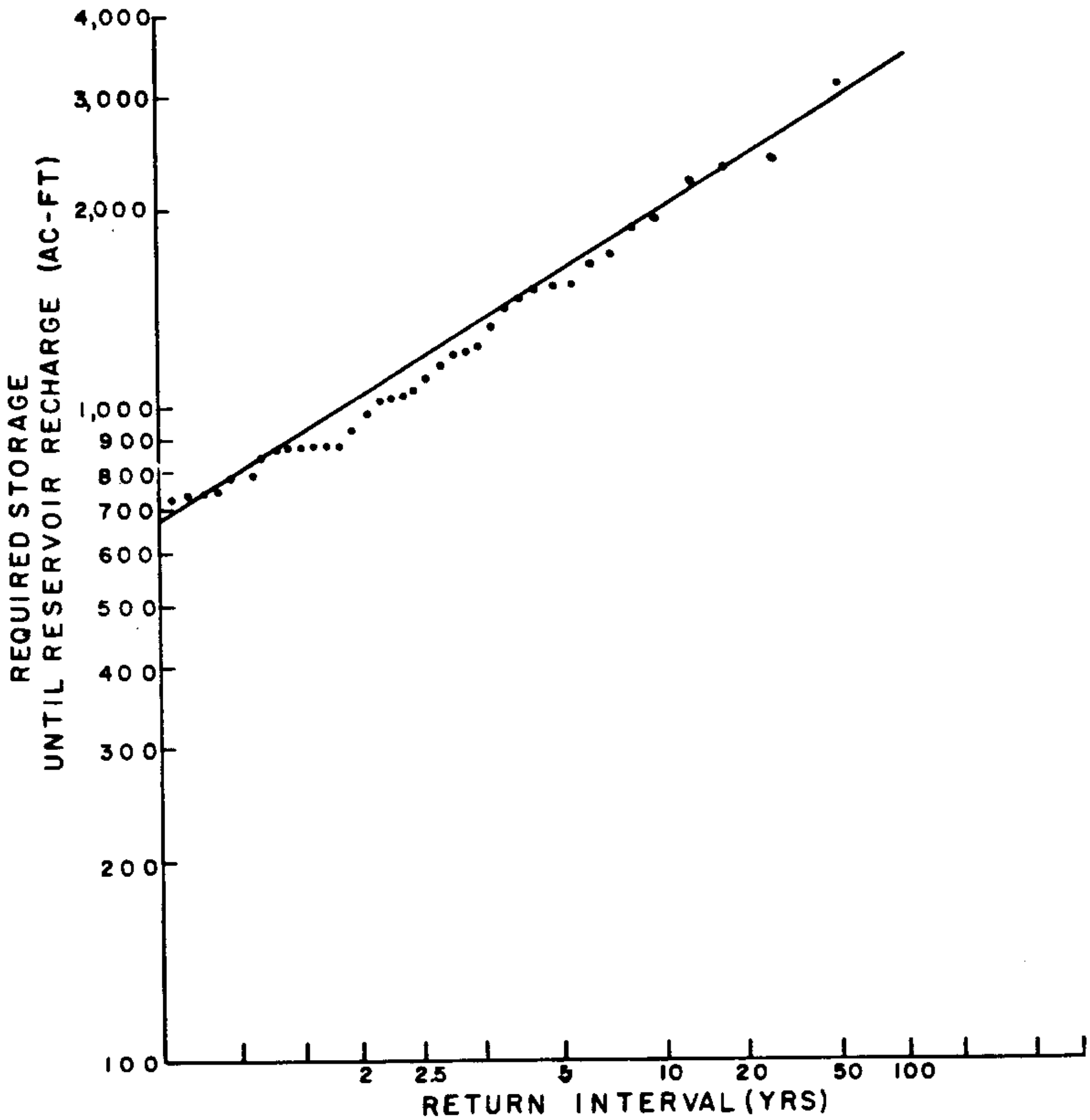

Figure F.5-15. Within Year Storage Supply: Apr.-Oct. 5% Avg./15% Maximum Withdrawal Criteria.

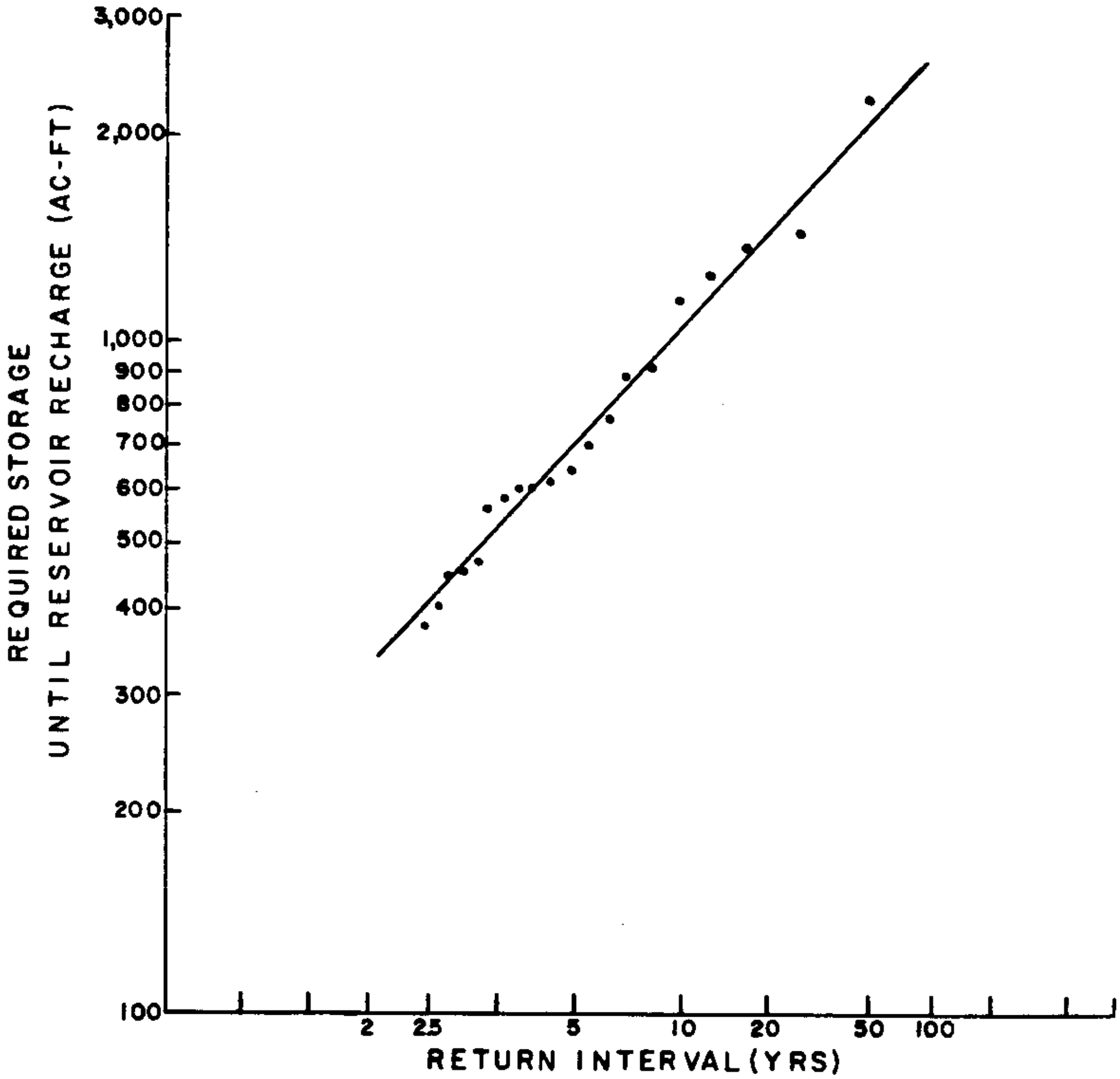

Figure F.5-16. Within Year Storage Supply: May-Oct. 5% Avg./15% Maximum Withdrawal Criteria.

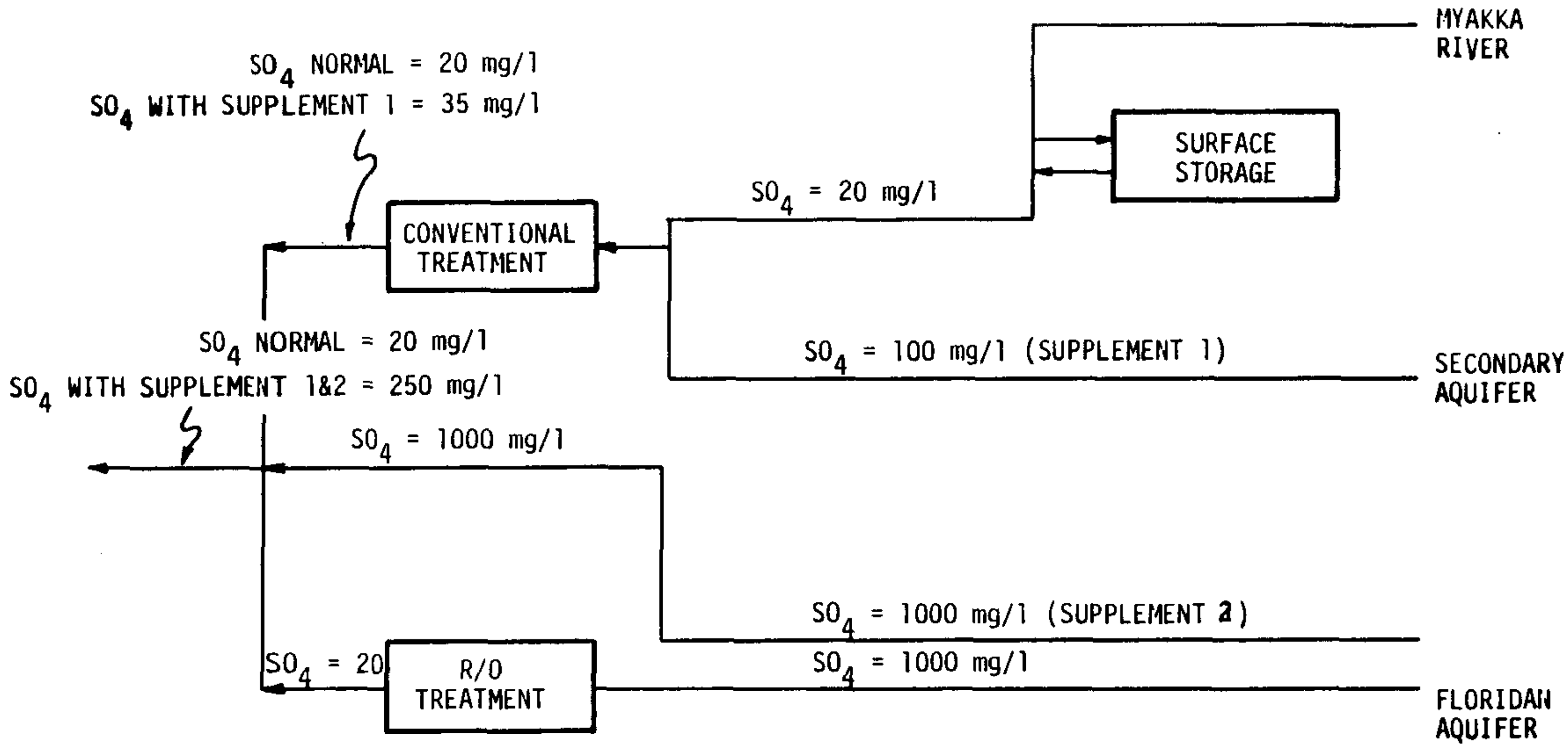

Figure F.5-17. Supply Schematic.

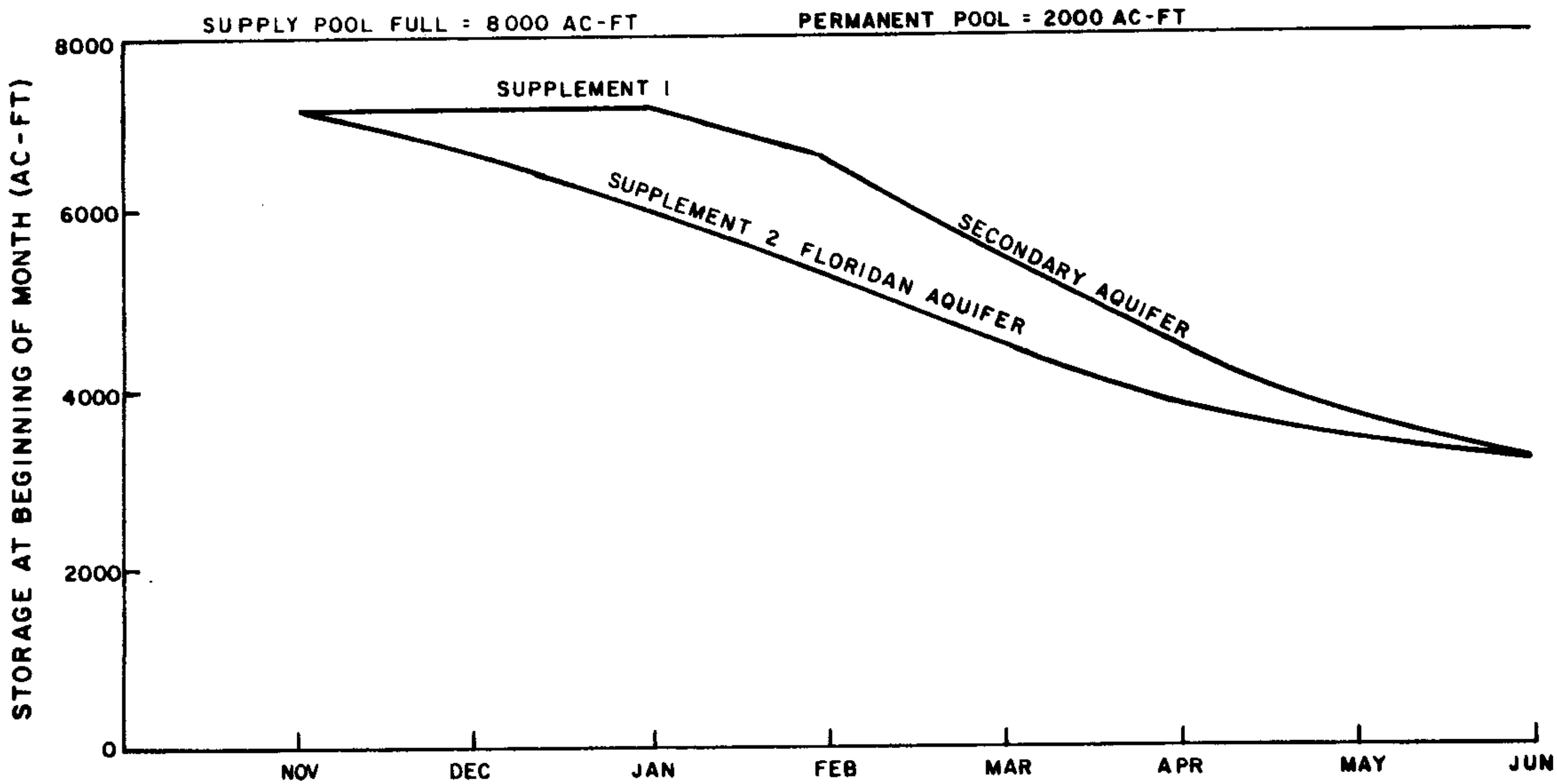

Figure F.5-18. Reservior Rule Curve 5% Avg./5% Max. Withdrawal Criteria.

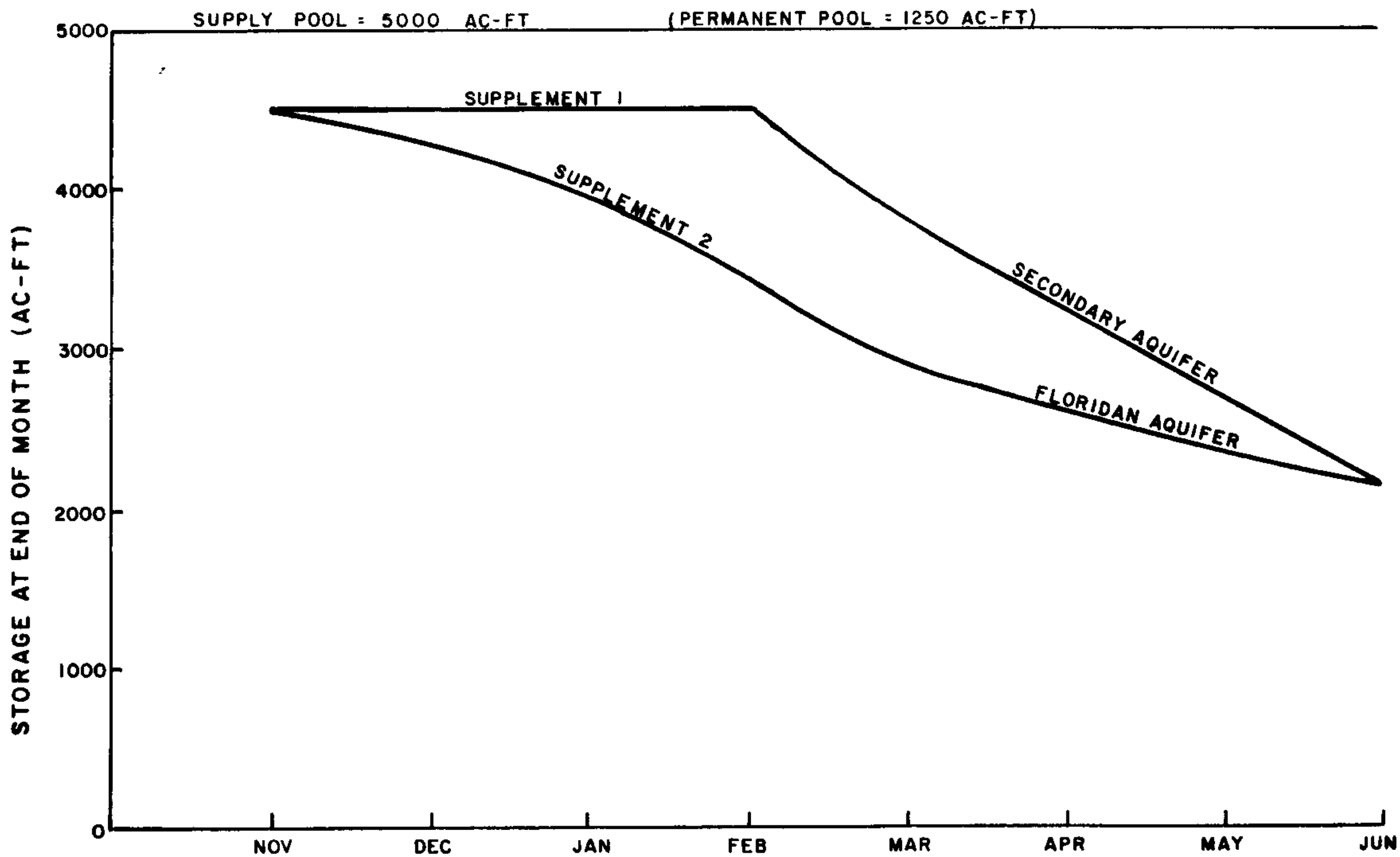

Figure F.5-19. Reservoir Rule Curve 5% Avg./15% Max. Withdrawal Criteria.

APPENDIX G

CONCEPTUAL DESIGN REPORT

by
BOYLE ENGINEERING CORPORATION

Boyle Engineering Corporation

320 East South Street
Orlando, Florida 32801

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CONCEPTUAL DESIGN REPORT
RIVER DIVERSION/TRANSMISSION SYSTEM
SARASOTA COUNTY
RINGLING MACARTHUR RESERVE
WATER SUPPLY PROJECT

OR-D28-103-30

PREPARED FOR DAMES AND MOORE

FEBRUARY 1986

BOYLE ENGINEERING CORPORATION

Principal in charge:	H.W. Haeseker, PE
Project Manager:	H. William Persons, PE
Project Engineers:	David L. Farabee, PE John T. Shugert, PE Steven M. Lockington

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RIVER DIVERSION/TRANSMISSION SYSTEM

I. INTRODUCTION

A. RIVER DIVERSION SYSTEM

The River Diversion System consists of facilities necessary to transport water from the Myakka River east to a proposed surface water reservoir on the RMR tract. The River Diversion System includes a canal approximately 0.75 miles in length, a River Pump Station, and discharge piping to the reservoir. This report is intended to identify an appropriate design concept and to provide conceptual cost estimates which may be used in conjunction with cost estimates for other project components to determine the appropriate design capacity for the river diversion system components prior to preliminary design.

B. TRANSMISSION SYSTEM

The Transmission System for this project consists of facilities necessary to transport untreated water from the RMR tract, east of the Myakka River, to a proposed water treatment plant to be located just west of Cow Pen Slough in

Section 21, Township 38 South, Range 19 East, Sarasota County. The raw water source for this project, as discussed elsewhere in this report, will consist of a combination of pumped groundwater, collected surface water and diverted Myakka River water.

The proposed Transmission System will include a pumping station (with pumps, forebay tank and motor control center) and about 4 miles of pipeline(s) with river crossing and other appurtenant facilities.

II. RIVER DIVERSION SYSTEM

A. DESIGN CRITERIA AND GENERAL CONCEPTS

1. Design Criteria

The allowable withdrawal rate from the Myakka River is currently unknown. It is assumed that the diversion rate will initially be limited to 5 percent of the instantaneous flow in the river. Because of the possibility that the allowable diversion rate may be increased, the conceptual design presented in this report has been based on a maximum allowable diversion rate of 25%.

Three different pump station sizes were investigated. The maximum pumping rates were 150, 300, and 500 cfs (96.9, 193.9, and 323.1 MGD).

It is recognized that, due to the extreme seasonal flow variations in the Myakka River, the use of the river pump station will be intermittent, with the largest pumps used infrequently. For this reason, it was assumed that the pumps would be diesel engine driven. This should result in substantial savings by avoiding the demand charge for electrical service to large electric motors. During preliminary design a comparison of diesel engine driven versus electrical driven pumps should be made.

To meet the wide range of flows anticipated without having numerous pumps, the pumps would need to operate at variable speeds. This can be accomplished by varying the engine speed.

Diesel engines can generally operate at speeds ranging from 1200 to 2400 rpm. This makes it possible to utilize a combination of several engine-driven pumps to obtain a wide range of pumping rates to match the allowable diversion rates under differing river flow conditions.

2. Operational Concept

Development of the conceptual design included consideration of the operational conditions anticipated. The following paragraphs describe some of these conditions and their accommodation in the conceptual design.

Raw water will be diverted from the Myakka River through a diversion channel extending from the river to a pump station located adjacent to the reservoir. The pump station will pump water from the channel into the storage reservoir.

In order to optimize water withdrawals from the river without exceeding regulatory constraints, it is expected that close control over the withdrawal rate will be desirable. Normally, this can be done by adjusting the speed of a carefully selected assortment of pumps. However, at very low flow rates, the use of continually-operated engine-driven pumps becomes impractical. Diversion at these low flow rates would be better accomplished through the use of small electric motor-driven pumps or by intermittent pumping coupled with a low-flow control structure between the river and the diversion channel. This intermittent pumping arrangement would involve storage in the diversion channel or in an off-channel basin.

For this reason, storage of the water in the pump station sump, diversion channel, or off channel should be investigated during preliminary design. Storage would be sized to limit pump cycling at low diversion rates and to utilize pumps within an efficient operating range. This option should be compared to the alternative of providing several small pumps to pump these flows. For flows less than some minimum rate, the cost of pumping is expected to exceed the benefit of pumping; this minimum cost-effective diversion rate should be estimated during preliminary design.

B. RIVER DIVERSION CHANNEL

1. Location

The diversion channel was preliminarily located in the northern part of the site as shown on Exhibit 1. This location was selected because it entails higher stream channel elevations along the river. The higher stream channel elevation is desirable to minimize the effects of high tides in the Gulf of Mexico and possible migration of salt water upstream, especially during low or no flow. The river channel bottom in the area selected is near elevation 2.0 feet MSL. For comparison, the elevation of the stream bed just downstream of Rocky Ford (approximately one mile south of the selected area) is 0.3

feet MSL. (These stream bed elevations are based on survey information supplied by Bennett and Bishop.)

2. Conceptual Hydraulic Analysis and Channel Sizing

The channel was sized for the condition of a 25 percent diversion rate. This is a conservative assumption because, for a given diversion flow rate, the water surface elevation in the Myakka River is lower for a 25 percent diversion than for a 5 percent diversion. Table 1 compares the streamflow required for 150, 300, and 500 cfs diversion rates at 5 and 25 percent withdrawal.

TABLE 1

STREAMFLOW DATA

Diversion Rate (cfs)	Streamflow Rate 5% Withdrawal (cfs)	Streamflow Rate 25% Withdrawal (cfs)
150	3,000	600
300	6,000	1,200
500	10,000	2,000

A river section near the diversion point was analyzed to determine the approximate water surface elevation in the Myakka River for these 25 percent streamflow rates. Manning's equation was used to estimate the stream depths. The effects of backwater conditions from downstream channel restrictions were neglected. The diversion channel was then sized to limit headloss in the channel to 1 foot. The anticipated channel length is approximately 3,200 feet. It was assumed that the diversion channel invert will be level at an elevation of 2.0 feet MSL. This arrangement was developed to limit excavation for the channel and keep velocities low. This arrangement also results in more favorable pump station geometry than would be possible if a deeper channel were used.

The diversion channel was assumed to be trapezoidal in shape with side slopes of 4 horizontal to 1 vertical. Table 2 lists the preliminary channel characteristics.

TABLE 2

DIVERSION CHANNEL CHARACTERISTICS

Diversion Rate (cfs)	Water Surface Elevation at Myakka River (ft)	Water Surface Elevation at Pump Station (ft)	Bottom Width (ft)	Velocity at Pump Station (fps)
150	5.8	4.8	9	1.7
300	7.6	6.6	10	2.0
500	9.1	8.1	12	2.2

The bottom of the channel appears to be in very poor rock. It has been assumed that excavated material below 5.0 feet MSL will be rock which may present construction difficulties. This assumption should be checked in more detail by soil borings during preliminary design.

Specific channel locations and channel cross sections should also be investigated in more detail during preliminary design.

3. Diversion Channel Costs

Unit costs for construction of the diversion channel were obtained by review of recent bid tabulations for similar large excavation projects. These unit costs were used to estimate the total probable construction costs which are intended to include only clearing, excavation, and grassing of the channel. They do not reflect transition of the channel dimensions to match into the pump station; those costs are included in the pump station costs. It is assumed that the excavated material can be either disposed of locally or used for construction of the reservoir embankment. The estimated probable construction costs for diversion channels sized for a 25% minimum diversion rate are shown in Table 3.

TABLE 3

ESTIMATED DIVERSION CHANNEL COSTS

Diversion Flow Rate (cfs)	Channel Bottom Width (ft)	Estimated Probable Construction Cost
150	9	$374,000
300	10	$385,000
500	12	$410,000

NOTE: Based on ENR = 4200

C. RIVER PUMP STATION

1. General

The River Pump Station will lift water from the river diversion channel to a surface water storage reservoir. Pumping is required to overcome the static difference in water levels between the channel and the reservoir and to overcome friction losses in the piping.

2. Site Facilities

Tentative site location of the pump station is shown on Exhibit 1. It is anticipated that the pump station will be adjacent to the reservoir's west embankment.

The location was selected to meet the requirements discussed previously for the river diversion channel and to locate the pump station outside the 100 year flood plain. The limits of the 100 year flood plain are published by the Federal Emergency Management Agency (FEMA). Based on these location constraints, the conceptual design conclusion is that the pump station should be located in Section 18, Township 38 South, Range 20 East.

The configuration of the pump station facility is shown schematically in Exhibit 2. The actual number of pumping units will depend on the maximum pumping capacity of the station. The pumps would discharge into a common manifold which would connect to a short pipe discharging to the reservoir. A typical pump station section is shown in Exhibit 3.

A pumping unit to provide redundancy for the largest pumping unit is included. It is recognized that the small pumping units will operate most of the time during both high and normal flow conditions. For this reason, two small pumps should be provided to reduce wear on a single unit.

The river diversion channel will include a transition section to match into the pump station sump. Maximum channel divergence should be 5 degrees to provide a smooth transition of flow, minimize headloss, and reduce velocities entering the sump.

The fuel storage tanks shown on the pump station schematic will provide fuel to the pump engine drives. The fuel tanks may be below ground with a small electric driven pump to deliver fuel to the engine drive units or above ground to allow for gravity flow to the engines. For buried tanks, groundwater monitoring wells will be required. For above ground tanks, an enclosing berm will be required. It is assumed that a minimum of two tanks will be required.

3. <u>Pumping Units</u>

The pumping units contemplated for this installation would be variable speed vertical type mixed flow pumps with engine drivers. Mixed flow pumps provide high operating efficiency for the relatively high flows and low heads associated with this facility.

System head curves were developed to reflect the potentially varying conditions to evaluate pumping requirements. The system curve was developed for a single pipe entering the reservoir beneath the low water level. The sizing of the pipe will be discussed in a subsequent section. System head curves were based on headloss calculations using the Darcy-Weisbach formula with a headloss coefficient of 0.013. The suction elevation for the pumps was assumed to vary between elevation 2 to elevation 10 and the reservoir variation was assumed to vary from elevation 15 to 25 feet. The static conditions are reflected in the system curves in terms of a minimum of 9.5 feet and maximum of 23.0 feet. From the system curve it is possible to evaluate the performance of individual and multiple pumping units.

For selection of pumping units, it was assumed that the reservoir was full and each pumping unit was operating at full speed. The pumping units were sized based on the principle that each pumping unit would have twice the pumping capacity of the next smaller unit at the design point. By slowing down or speeding up the engine drives, the units can be varied to meet other conditions on the system curve. All tables will reflect the design point conditions. Pump curves have been adjusted to reflect losses in the suction and discharge piping, valves and fittings. During preliminary design, after selection of a pump station capacity and reservoir operating elevations, the pump selection should be refined by identifying the reasonable operating range for each pump and each combination of pumps. This refinement may result in a reduction of the total number of pumps required.

For purposes of conceptual design, it has been assumed that a building will be provided to house the pumping units, engine drives, controls and instrumentation.

a. 150 CFS Pump Station:

For the 150 cfs pumping station, it is anticipated that there would be 6 main pumping units and that lower flows would be handled by additional smaller pumping units.

The typical pump operating conditions for the pumping units are shown in Table 4. These points are adjusted to reflect suction and discharge losses. The characteristics of the pumping units are shown in Exhibit 4 based on performance curves published by a recognized pump manufacturer.

TABLE 4

150 CFS PUMP STATION
TYPICAL PUMP OPERATING CHARACTERISTICS

Pumping Unit	Design Flow (cfs)	Cumulative Flow (cfs)	Design TDH (ft)	Efficiency (%)	H.P.
1	9.38	9.38	28.3	75	40
2	9.38	18.75	28.3	75	40
3	18.75	37.50	30.3	85	76
4	37.50	75.00	28.1	80	149
5	37.50	112.50	28.1	80	149
6	37.50	150.00	28.1	80	149

b. 300 CFS Pump Station:

For the 300 cfs pump station it is anticipated that there would be 7 main pumps.

The typical operating conditions for the pumping units are shown in Table 5. Pump characteristics are shown in Exhibit 5 based on performance curves published by a recognized manufacturer.

TABLE 5

300 CFS PUMP STATION
TYPICAL PUMP OPERATING CHARACTERISTICS

Pumping Unit	Design Flow (cfs)	Cumulative Flow (cfs)	Design TDH (ft)	Efficiency (%)	H.P.
1	9.38	9.38	28.3	75	40
2	9.38	18.75	28.3	75	40
3	18.75	37.50	30.3	85	76
4	37.50	75.00	28.1	80	149
5	75.00	150.00	26.4	88	255
6	75	225	26.4	88	255
7	75	300	26.4	88	255

c. 500 CFS Pump Station:

It is anticipated that there would be 8 main pumping units for the 500 cfs pump station because of the larger range of flow. Table 6 lists the typical operating characteristics for the 500 cfs pump station units. The characteristics of the pumping units are shown in Exhibit 6 based on actual performance curves published by a recognized pump manufacturer.

TABLE 6

500 CFS PUMP STATION
TYPICAL PUMP OPERATING CHARACTERISTICS

Pumping Unit	Design Flow (cfs)	Cumulative Flow (cfs)	Design TDH (ft)	Efficiency (%)	H.P.
1	7.81	7.81	25.6	81	28
2	7.81	15.63	25.6	81	28
3	15.63	31.25	26.9	78	61
4	31.25	62.50	25.4	86	105
5	62.50	125	28.8	84	243
6	125	250	29.0	89	462
7	125	375	29.0	89	462
8	125	500	29.0	89	462

4. Pump Station Sump

Sizing of the sump for the pump units was based on the guidelines of the Hydraulic Standards Institute, pump manufacturer guidelines, and reasonable clearance requirements for pump driver units. The sump sizing allows for divider walls between the pumps. Sump floor elevations were based on the pump submergence and floor clearances required for each pump size. The pump station operating floor was assumed to be at elevation 18.5 ft MSL which is estimated to be 2 feet above the 100 year flood elevation. Table 7 lists the basic sump characteristics for each pump station configuration described previously.

TABLE 7

TYPICAL PUMP STATION SUMP CHARACTERISTICS

Pump Station Capacity	Width (ft)	Length (ft)	Sump Floor Elevation (ft, MSL)
150	36	75	-5.0
300	36	92	-5.5
500	36	104	-4.5

A trash rack should be provided over the entrance of the sump to prevent large debris from entering the sump. The pump station section shown previously in Exhibit 3 shows some of the features of the sump.

5. Channel Divergence

To allow for smooth transition from the diversion channel to the pump station a slowly diverging section should be used. The maximum amount of divergence should be 5 degrees.

A plan view of the pump station in Exhibit 7 depicts some of the basic features of the divergence and its connection to the pump station.

Due to the close juxtaposition of the pumping station and reservoir and the presence of piping between them, access through this area may be restricted. For this reason, it is likely that a road crossing the diversion channel will be desirable immediately upstream of the divergence area. Such a road crossing could utilize several large concrete culverts to carry the diversion flow, providing additional protection from floating debris. This possibility should be explored in more detail during preliminary design after the diversion pump station size has been selected.

6. Pump Station Building

The pumps and engines should be housed in a building to alleviate environmental concerns over engine noise and to provide for convenient engine maintenance. The building should be concrete block and steel construction with a ceiling height of approximately 30 feet. Since the diversion channel and discharge piping will limit access by an outside crane, a bridge mounted crane should be provided in the building for removal of engines, pumps, and valves. The building plan dimensions are shown in

Table 8. This building can be flanked by berms on the north and south to reduce noise.

TABLE 8

PUMP STATION BUILDING PLAN DIMENSIONS

Pump Station Capacity	Width (ft)	Length (ft)
150	30	75
300	30	92
500	30	104

7. Fuel Storage Tank

The actual size of the fuel storage tanks should be considered in more detail in preliminary design. This will entail an analysis of streamflows, possible time before tanker trucks can access the site after major flooding events, and owner/operator requirements. For conceptual design, it has been assumed the tanks will be 20,000, 30,000 and 40,000 gallons for for the 150, 300 and 500 cfs pump stations respectively.

An all-weather road will be required to allow tanker trucks to deliver fuel. The location and size of this road has not been included in the conceptual design. The road should be integrated with access to other portions of the site and other site facilities beyond the scope of this conceptual design.

8. <u>Instrumentation, Controls, and Switchgear</u>

The sequencing and operation of the pumps can be either manual or automatic based on the requirements of the Owner and Southwest Florida Water Management District (SWFWMD).

Instrumentation of streamflows, sump water levels, flows pumped to the reservoir, and reservoir levels will be required to set pump speeds and decide which pumps will operate. The requirements of SWFWMD will also dictate what other information will be required and may greatly influence whether the station can be operated manually or will require more sophisticated automatic operation.

For this conceptual design, basic manual engine drive controls have been considered. Remote automatic control of these pumps should be integrated with other project instrumentation during preliminary and final design.

9. <u>Pump Station Estimated Construction Costs</u>

The probable construction costs for the pump station facilities were estimated using bid tabulations from similar facilities. The costs include the following:

* Site work, including piping and valves.
* Basic controls for engine drives.
* Channel divergence.
* Building and sump.

* Fuel tanks.

* Basic pumping units.

The estimated probable construction costs are outlined in Table 9.

TABLE 9

ESTIMATED PUMP STATION CONSTRUCTION COSTS

Pump Station Capacity	Item	Probable Construction Cost ($)
150	Pump, Drivers & Piping	874,000
	Sump & Channel Divergence	407,000
	Building	159,000
	Fuel Tank	40,000
	TOTAL	1,480,000
300	Pump, Drivers & Piping	1,392,000
	Sump & Channel Divergence	556,000
	Building	169,000
	Fuel Tank	57,000
	TOTAL	2,174,000
500	Pump, Drivers & Piping	1,900,000
	Sump & Channel Divergence	627,000
	Building	176,000
	Fuel Tank	70,000
	TOTAL	2,773,000

NOTE: Based on ENR = 4200

D. RESERVOIR INFLUENT PIPE

1. Preliminary Hydraulic Analysis and Pipe Sizing

Sizing of the reservoir influent pipe must take into account several factors. These factors include velocities and associated headlosses, estimated amount of time the flows will be pumped (which will relate to energy costs), and standard pipe sizes.

The maximum flow rates for the pipeline are the same as the pump station flows. Peak rates may occur only seasonally, but the amount of time these flows may occur can have a significant effect on the operating costs. This effect (and the stream flow analysis necessary to estimate it) should be investigated during preliminary design.

For conceptual design purposes, the size of the pipeline was estimated based on reasonable velocity and headloss constraints. The pipe was assumed to be 250 feet long. The pipe sizes selected for conceptual design are as follows in Table 10.

TABLE 10

RESERVOIR INFLUENT PIPE SIZES

Design Max Flow (cfs)	Pipe Diameter (in)
150	54
300	72
500	84

A life cycle cost analysis should be performed during preliminary design to select an economical pipe size for the design conditions. This analysis should also include a consideration of the cost and desirability of the use of multiple influent pipes.

The outlet of the influent pipe in the reservoir would be a gradually diverging fabricated section designed to reduce exit velocity and subsequent headloss, and to dissipate energy, thereby preventing scouring in the reservoir. The design of this structure was not considered during conceptual design.

2. Pipe Materials

For pipes in the size ranges anticipated, the most appropriate material options are ductile iron and steel. The 54-inch diameter is the upper limit of the ductile iron pipe range. The use of multiple pipes would provide the option of using ductile iron pipe and could provide improved reliability.

3. Reservoir Pipeline Estimated Construction Costs

The costs for the reservoir pipeline were estimated from recent bid tabulations for similar sizes and types of pipe. The costs include pipe material, transportation, excavation and backfill, seepage collars, reservoir inlet structure, cathodic protection and contractor overhead and profit. The probable construction costs are shown in Table 11.

TABLE 11

RESERVOIR PIPELINE
ESTIMATED CONSTRUCTION COST

Design Max Flow (cfs)	Pipe Size (in)	Probable Construction Cost($)
150	54	314,000
300	72	350,000
500	84	373,000

NOTE: Based on ENR = 4200

E. SUMMARY

Based on the data available at this time and the extent of evaluation currently possible, it is not appropriate to select one size of facilities over another. The conceptual design and costs provided here should be analyzed in comparison with other project components to arrive at a design concept for the project as a whole.

The total estimated construction costs for the facilities mentioned previously are shown in Table 12.

TABLE 12

ESTIMATED CONSTRUCTION COST FOR RIVER DIVERSION SYSTEM

Pump Station Capacity	Probable Construction Cost ($)
150	2,168,000
300	2,909,000
500	3,556,000

NOTE: Based on ENR = 4200

III. TRANSMISSION SYSTEM

A. DESIGN CRITERIA AND GENERAL CONCEPTS

1. Design Flow Rate

The preliminary sizing of the transmission facilities has been based on the following flow projections provided by Joint Venture:

TABLE 13

FLOW PROJECTIONS

Year	Maximum Daily Flow	Average Flow
1988	12 MGD	8.6 MGD
1995	24 MGD	17.1 MGD
2000	36 MGD	25.7 MGD
2010	52 MGD	37.1 MGD

The above flow rates reflect the projected demands of the proposed water treatment plant and are irrespective of the individual or combined quantities of the three supply sources. Variations between source supply and demand will be provided by major storage facilities which are discussed elsewhere in Appendix __.

2. Operational Concept

Raw water, from the three sources, which are the Myakka River, Secondary Aquifer and Surface runoff, will be collected and stored in a reservoir to regulate variations between supply and demand. The reservoir will be located east of the Myakka River and will supply a positive suction for the Transmission Pump Station, also to be located east of the River. The required flows will then be pumped through the Transmission Pipeline to a storage tank at the proposed treatment plant. Exhibit 2 schematically illustrates this concept.

The main storage reservoir will regulate the variations between the seasonal supply (Myakka River, surface runoff and Secondary aquifer) and the demand by the treatment plant. The water surface elevation for this reservoir is anticipated to range between 15 and 25 feet above mean sea level (MSL). These reservoir facilities are discussed in Appendix __.

A ground storage tank will serve as a terminal control reservoir for the well collection system and as a forebay for the Transmission Pump Station. This tank has been preliminarily sized at 1.5 million gallons. This forebay tank would provide the following operational functions:

* Start/stop control of well field pumps.
* Allow interim and/or periodic separation of groundwater source from other raw water sources to control water quality.
* Act as forebay for the Transmission Pump Station and provide regulated positive suction head for pumps.
* Act as an intermediate flow regulating structure between river intake pumps and transmission pumps during emergency bypass of the main reservoir.

The water surface elevation at the headwater of the treatment plant will be determined by a raw water regulating tank to be incorporated into the facilities at that site. Based on preliminary data, this reservoir was assumed to have an operating water level range between elevations 34 and 62 feet with a typical operating level assumed at elevation 55 feet (all MSL). The Transmission Pump Station would be primarily controlled by the changes in the level at this raw water tank.

B. TRANSMISSION PIPELINE

1. Routing

The Transmission Pipeline is the most costly element of the Transmission System. These construction costs are influenced by pipeline length, diameter, material (strength) and existing field conditions along the pipeline route.

Two alternative pipeline routes have been considered and are shown on Exhibit 8. These alternatives were selected for study as a result of field investigations and a review of aerial photographs, topographic maps and ownership plats. Major factors contributing to these alternative alignments are as follows:

* Location of proposed treatment plant.
* Land ownership configurations and location of property lines.
* Location of proposed Transmission Pump Station.

The most viable alternative will probably depend on the ultimate siting of the Transmission Pump Station which in turn will be influenced by the positioning of the main storage reservoir. Due to the open terrain along each of the alignments, construction of the pipeline should be relatively straightforward utilizing standard methods of dewatering and open trench installation. Special considerations will be required to cross the Myakka River, however.

2. Myakka River Crossing

A crossing of the Myakka River is required for either alternative pipeline alignment. Two methods of crossing this river were considered; bridging overhead (suspended or supported) and burial beneath the river bottom.

The primary advantage of an overhead crossing is avoiding the need to dewater the river. In some instances, this could result in reduced construction costs. If a vehicle access bridge or similar type of overhead facility were necessitated for other reasons, it would probably be more cost effective to incorporate the pipeline crossing in the design of the bridge. However, this project does not contemplate any such facility. Disadvantages of a suspended crossing would include exposure to potential vandalism, adverse impacts aesthetically and risk of damage during high stage flooding of the river.

A buried crossing would not be subject to the disadvantages of an overhead crossing. However, higher construction cost could be a significant disadvantage of a buried crossing.

Crossing of the Myakka River for this project, at either of two locations is proposed to be by the burial method. It is anticipated that the low flows which occur during the drier season of the year could be diverted around a portion of the crossing and thereby permit complete dewatering of a segment of the pipeline trench. The two alternative proposed crossing sites lend themselves to this approach as a result of existing topographic features.

As a basis for conceptual design, construction of the buried crossing was assumed to include restrained joints and concrete encasement around the pipe. During preliminary design, geotechnical data should be gathered to permit a comparison of this technique with other possibilities, such as jack-and-bore crossing or the use of ball-and-socket pipe under water.

3. Preliminary Hydraulic Analysis and Pipeline Sizing

Sizing of the Transmission Pipeline must take into account several factors. Primary factors include the velocities and head losses (due to friction) resulting from the quantity of flows which are to be transmitted throughout the useful life of the facilities. These factors have a direct impact on the selection of efficient pumping equipment and must be weighed against the initial capital cost of the facilities. Selection of pipe sizes must of course take into consideration the standard nominal sizes which are available with locally used and produced pipe products. An additional factor that needs to be taken into consideration is the potential for hydraulic surges which result from changes in velocity caused by pump sequencing and/or valve closing.

The peak quantities of water to be transmitted are the projected maximum daily flow rates shown in Table 1. While these peak rates may occur only during the seasonal high demand periods, the facilities must be sized to handle them with adequate concern of the foregoing factors. Consideration of annual operating costs should of course take into account the projected average flows as well as the peak flows.

Based on applications similar to this project with comparable flow rates, experience has shown that pipeline velocities in the range of 5 to 6 feet per second during peak flows result in the most cost effective pipe sizes while meeting the above considerations. Based on this, a single pipeline to handle the 52 MGD flow projected for the year 2010 is selected at a standard nominal size of 48-inch. During the preliminary design stage of this project, a more detailed life-cycle cost analysis of alternative pipe sizes should be conducted to compare 42, 48 and 54-inch pipelines.

As an alternative to installing a single pipeline to handle flows which will not occur until future years, a smaller pipeline could be installed initially. Then a second pipeline could be installed in the future when needed. Advantages of two pipelines would include the following:

* Reduce initial capital costs.
* Permits evaluation of the size of the second pipe in later years to reflect updated flow projections.
* System reliability will be increased after installation of the second pipeline because of the ability to keep one pipe in service if the other is down for repairs or maintenance.
* Segregate different water quality supplies or sources.

For this alternative, two 36-inch diameter pipelines were selected. Each pipe would have one-half the capacity of a 48-inch pipe while keeping velocities and headlosses about the same. Unless factors in the future were to dictate otherwise, both pipelines should probably be constructed within the same rights-of-way. In addition, it is recommended that both pipes be initially installed under the Myakka River to facilitate future construction of the second pipeline without disruption of the river crossing.

Based on the two alternative pipeline configurations (single 48-inch pipe and dual 36-inch pipes), hydraulic schematics were prepared to show the hydraulic grade lines, during pumping at various flow conditions. These are shown on Exhibits 9 (Single 48-inch pipe), 10 (Initial 36-inch pipe), and 11 (Dual 36-inch pipes). Head loss due to pipeline friction was computed using the Hazen-Williams formula and a roughness coefficient of 140. The typical static head condition is 33 feet.

The total dynamic head (TDH) at the transmission pump station ranges from approximately 40 feet to 88 feet based on typical operating levels in the two tanks. The potential ranges in pumping head shown on Exhibits 9, 10, and 11 will be even greater when extreme variations in water surface levels occur in the two reservoirs. This necessitates careful analysis and proper pump selection to design an efficient system. This will be discussed further in the section on the Transmission Pump Station.

4. Pipeline Materials

For the pipeline size and pressure requirements of this project, two pipe products are widely used and locally available, ductile iron pipe and prestressed concrete pipe. The final design of the pipeline for this project could be based on either or both of these products. Preliminary design should include a review of the corrosivity of soils along the route to aid in the selection of appropriate pipe materials.

5. Pipeline Unit Construction Costs

Preliminary "base level" unit construction costs have been prepared for 30-inch through 54-inch diameter pipes. Costs are intended to include pipe material, taxes, transportation, normal installation, an allowance for typical pipeline appurtenances and contractor's overhead and profit. Pipe material costs have been based on ductile iron pipe with cement mortar lining. It has been tentatively assumed that cathodic protection for corrosion protection will not be required. Costs for prestressed concrete pipe may be competitive, particularly in the larger sizes.

Base level installation costs assume medium dense soils with 1 to 1 trench side slopes, 4 feet of cover over the pipe, no major utility interference and routine clearing and grubbing requirements. Preliminary base level unit construction cost estimates are shown in Table 14.

Conceptual costs have been developed based on an Engineering News Record (ENR) cost index of 4200 which is reflective of 1985 construction costs. Appropriate adjustments may be necessary if the ENR Cost Index varies.

TABLE 14

PIPELINE UNIT CONSTRUCTION COST ESTIMATES
(ENR = 4200)

Pipe Diameter (Inches)	Probable Construction Cost ($/LF)
30	60
36	80
42	110
48	140
54	180

NOTE: Base level unit construction costs are based on Class 50 ductile iron pipe with cement mortar lining and include taxes, transportation, normal installation, typical appurtenances and contractor's overhead and profit.

6. Estimated Costs of Alternative Routes and Pipeline Sizes

As discussed previously, a single 48-inch pipeline, or a phased construction of two 36-inch pipelines, has been preliminarily selected for this project. Two alternative routes have also been selected as shown on Exhibit 8. Estimated costs have been applied to each of these alternatives, based on ENR cost index of 4200, to produce total estimated costs for the Transmission Pipeline.

Each alternative includes the estimated cost of crossing the Myakka River. These costs are based on a buried, concrete encased pipe installed under the river bottom with a minimum cover of 4 to 5 feet. Also included are the estimated costs to acquire rights-of-way for the pipeline(s). The widths of permanent easements to be acquired were estimated at 30-feet for a single pipe and 50-feet for a double pipe arrangement. Costs of easements are based on an assumed land value of $5,000 per acre.

The estimated constructed and right-of-way costs for the various alternatives are shown in Table 15.

TABLE 15

COMPARISON OF ALTERNTIVE PIPELINE COST ESTIMATES

Quantity	Unit	Description	Unit Cost $	Probable Construction Cost $
Route "A" with Single 48-inch Pipe				
20,000	L.F.	48-inch Pipe	$140.00	$2,800,000
460	L.F.	River Crossing	300.00	138,000
12,000	L.F.	30 ft. Wide R/W	3.50	42,000
			Total	$2,980,000
Route "A" with Double 36-inch Pipe				
20,000	L.F.	36-inch Pipe (Initial)	$ 80.00	$1,600,000
460	L.F.	River Crossing	350.00	161,000
12,000	L.F.	50 ft. Wide R/W	5.75	69,000
			Subtotal	$1,830,000
20,000	L.F.	36-inch Pipe (Future)	$ 80.00	1,600,000
			Total	$3,430,000
Route"B" with Single 48-inch Pipe				
22,000	L.F.	48-inch Pipe	$140.00	$3,080,000
440	L.F.	River Crossing	300.00	132,000
16,000	L.F.	30 ft. Wide R/W	3.50	56,000
			Total	$3,268,000
Route "B" with Double 36-inch Pipe				
22,000	L.F.	36-inch Pipe (Initial)	$ 80.00	$1,760,000
440	L.F.	River Crossing	350.00	154,000
16,000	L.F.	50 ft. Wide R/W	5.75	92,000
			Subtotal	$2,006,000
22,000	L.F.	36-inch Pipe (Future)	80.00	1,760,000
			Total	$3,766,000

NOTE: Based on ENR = 4200

7. Selection of Transmission Pipeline Alignment

The final selection of Route "A" versus Route "B" will, in all probability, be determined by the final siting of the main storage reservoir and the location of the outlet works. If the final location of the outlet works cannot be located near the north or south lines of Section 19 (T38S-R19E) then additional pipeline length will have to be added to the quantities for each of the alternatives. If the outlet works end up near the middle of Section 19, consideration should be given to a third, and possible shorter alignment extending through the center portions of the sectional ownerships between the Myakka River and the treatment plant.

Based on the most likely positioning of the main storage reservoir and outlet works, Route "B" is preliminarily selected. Also, the dual 36-inch pipe is recommended on the basis of the advantages discussed previously.

8. Life Cycle Cost Analyses

As presented earlier, preliminary evaluations indicate a 48-inch pipe or two 36-inch pipes to be the most economical sizes for the Transmission Pipeline. It is recommended, however, that a life cycle cost analysis be made during preliminary design of this project when other factors have been more clearly defined, to confirm these initial determinations.

9. Pipeline Surges

Pipeline surge analyses were not made for the Transmission Pipeline at this conceptual stage. Because of the low operating pressures, flat topography, relatively short length of pipeline and other hydraulic considerations, it is not anticipated that pipeline surges will be of major consequence. Special surge control devices may not be required. However, during the preliminary design and after the final operating conditions are known a surge analysis to confirm these initial assumptions should be conducted.

C. TRANSMISSION PUMP STATION

1. General

The Transmission Pump Station will boost water from a reservoir and/or well field supply system east of the Myakka River, through a pipeline to a receiving tank at the proposed treatment plant which is located about 2 miles west of the Myakka River. Pumping is required to overcome a static difference in water level of approximately 10 and 50 feet and also to overcome the friction losses generated within the pipeline and pump station facilities.

2. Site Facilities

Siting of the Transmission Pump Station will depend on the location of the main storage reservoir, the reservoir outlet works, the transmission pipeline route, and the well piping arrangement. It is anticipated that the pump station will be adjacent to the reservoir enbankment about one mile east of the Myakka River. This setback from the river is outside the 100 year flood plain limits as published by the Federal Emergency Management Agency (FEMA). The north-south position of the pump station is anticipated to be in, or near, Section 19. For purposes of this conceptual design, it has been assumed that the pump station will be located along the north or south line of Section 19. This is the basis for the pipeline alignments and quantities as previously presented.

The configuration of the pump station facilities is shown schematically on Exhibit 2. Initially, two equally sized pumping units are proposed, one of which would be a stand-by unit. One full duty pump would deliver about 17 MGD which would meet the projected maximum day requirement for a few years. Additional pump unit of the same size would be added as needed to meet the increasing demands.

A check valved by-pass between the suction and the discharge of the pumps is also shown. This is to minimize the possibility of water column separation in the event of a sudden shut-down of the pumps such as would occur with a power failure.

The 1.5 million gallon tank would serve primarily as a receiving vessel and pump station forebay for a groundwater supply system. As such, both reservoirs could float together hydraulically, thus blending the stored water with the groundwater. On the other hand, if it were deemed desirable to pump and treat only one of the source waters, either could be isolated without significantly changing the suction conditions for the pumps.

Another element of this pump station facility is a metering facility to measure the total flow rate and total flow of raw water being pumped to the treatment plant.

Also shown on Exhibit 2 is a by-pass pipeline from the River Intake Pump Station. This by-pass could serve two functions. It would permit isolation of the main reservoir, if needed, and pump into the suction manifold and forebay tank. Of course, this would necessitate fairly close matching of the outputs of each pump station.

A second possible function would permit the use of the main reservoir outlet pipe as a common inlet-outlet if the reservoir inlet facilities at the River Diversion Pump Station needed to be taken out of service.

3. Pumping Units

The pumping facilities contemplated for this installation would consist of constant speed vertical type mixed flow pumps with electric drivers. They would either be installed within a building to provide weather protection and sound control or be weather protected for outdoor use. All electrical equipment and controls would be installed in an air-conditioned building. As mentioned previously, one redundant stand-by pumping unit would be provided.

To evaluate the pumping requirements, system head curves were developed to reflect the potentially varying conditions. For this purpose, Route "B" as shown on Exhibit 8 was selected since it has the greatest pipe length and would result in a system curve with the greatest amount of variation. For this selected route, curves were developed reflecting both a single 48-inch pipe and double 36-inch pipes. For the latter, a system curve was developed for the initial single pipe to reflect initial pump performance. All of these system curves were based on headloss calculations utilizing the Hazen-Williams formula and a roughness coefficient of 140. The static conditions which affect the pumping requirements are also reflected in these system head curves in terms of the extremes (maximum and minimum) and an assumed average or typical condition.

From these system curves, it is possible to evaluate the performance of individual and multiple pumping units with respect to efficiencies and horsepower requirements. Exhibits 12 and 13 show the characteristics of actual pumping units based on performance curves published by a recognized pump manufacturer. Based on the pumping units represented on these curves, a total of four 250-horsepower pumps would be required to meet the future maximum flow of 52 MGD. Intermediate levels of pumping capability can be directly taken from these curves. In the case of the dual pipe system (Exhibit 13), potential pumping capacities are shown for the initial pipe as well as the dual arrangement. A summary of the resulting pumping conditions for the two alternative pipe configurations under typical operating conditions is shown in Table 16.

TABLE 16

TYPICAL PUMP OPERATING CONDITIONS

No. Pumps	Flow-MGD	TDH-Ft	Eff.-%	HP/Pump
Single 48-inch Pipe				
1	19	46	83	185
2	34	60	86	208
3	44	72	85	218
4	52	83	81	234
Single 36-inch Pipe				
1	17	63	86	218
2	26	88	80	250
Two 36-inch Pipes				
1	19	48	83	193
2	34	63	86	218
3	45	77	84	241
4	52	88	80	250

Since the sole source of water for the proposed treatment plant is supplied by the Transmission Pump Station, it is critical that its operation not be interrupted for any extended period of time beyond a few hours. With the potential for losing electrical power during and following major storms, it would be prudent to provide some sort of back-up power source on site. This could be accomplished by either providing a diesel fueled engine-generator set to generate sufficient electricity to drive some of the electric motors, or by providing diesel engine drives with automatic, quick coupling devices to drive the pumps directly when the motors fail. It would probably be sufficient to provide such capability for about one half of the number of pumping units required to meet the maximum demands. A more detailed analysis of this back-up feature should be made during preliminary or final design.

4. Instrumentation, Controls and Switchgear

The sequencing of pumps at this facility will be controlled by the water level in the tank at the proposed treatment plant. At any given time, if the flow through the plant exceeds the amount being pumped into the tank, the level will drop. At a predetermined drop in level, a signal transmitted to the pump station would start up the next pump. This sequence would reverse when the flow into the tank exceeds the amount being treated. Other controls would be provided to protect such things as loss of suction, downstream reservoir overflow, high (or low) discharge pressure, etc.

Level-initiated control will be provided in the forebay tank for such things as controlling the well pumps and/or river intake pumps and protecting the pump station.

Automatic start-up and transfer to diesel generator equipment could be provided in the event of power failure.

As mentioned previously, all instrumentation, controls and switchgear equipment would be housed in a building for protection against weather and vandalism.

5. Pump Station Construction Costs

Preliminary estimates of construction costs were determined for the proposed Transmission Pump Station as a "base" facility cost plus or minus other selected features. The "base" pump station facility would provide for the total maximum pumping capability of 52 MGD and would include the following:

* Site work including piping, valving, paving, drainage, etc.
* Five 250 HP vertical pumping units installed outdoors.
* On-site electrical service facilities, switchgear and motor controls.
* Instrumentation and controls.
* Motor Control Center Building.
* Standby Generator
* Reservoir Outlet Structure

The preliminary cost of this "base" pump station facility was determined from an empirical formula developed previously for other pump station projects with the same type of facilities. Adjustments were made to reflect an ENR cost index of 4200. As determined by this method, the estimated preliminary construction cost for this base facility is $1,730,000.

Other features which could be added (or deducted) from the base facility were separately estimated and are shown below.

*	Building to enclose pumps and motors (Additive)	$120,000
*	250 HP pump unit with switchgear (Deductive)	$ 80,000
*	500 KW stand-by engine-generator (Additive)	$160,000
*	1.5 million gallon forebay tank (Additive)	$420,000

The estimated cost of a complete facility, based on selected elements can be obtained by combining the above amounts with the base facility cost.

D. SUMMARY

1. Selected Preliminary Facilities

Based on the data that is available at this time and the extent of evaluation currently warranted, a recommended Transmission System has been selected. This selection is for purposes of arriving at preliminary costs to be incorporated into the budget for the overall project.

The selected pipeline facilities would consist of a dual 36-inch pipeline along Route "B". Initially, only the first "barrel" of this pipeline would be installed, except at the Myakka River crossing. Here, it is recommended that two 36-inch pipes be buried and encased in concrete under the river bottom.

The selected pump station facility would consist of the "base" facility, less three of the 250 hp pumping units (and associated switchgear) which would be installed in the future. The pumps would be installed outdoors.

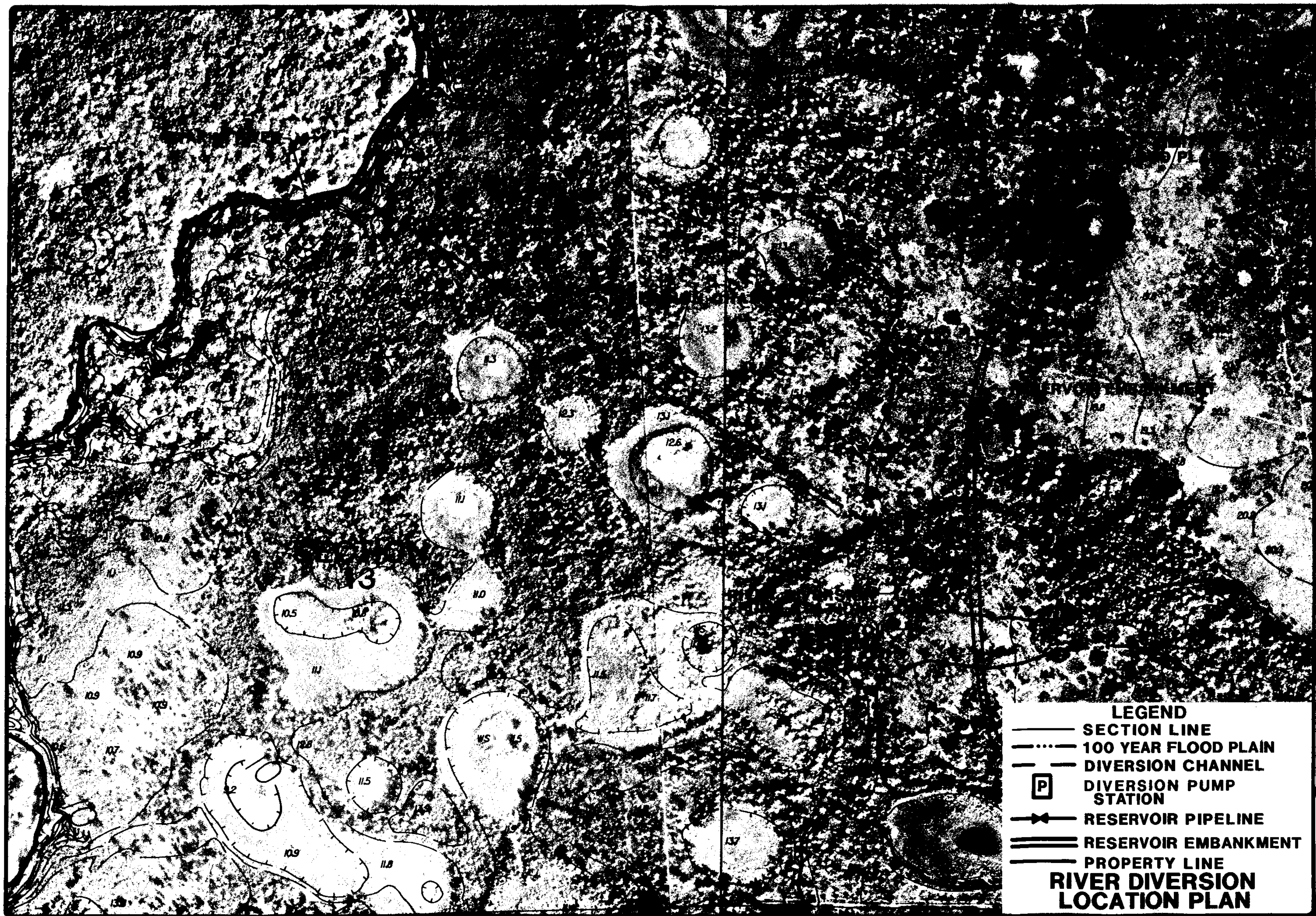
The 500 KW stand-by engine generator and the 1.5 million gallon forebay tank would be included with the recommended facility.

The estimated construction cost of the transmission facilities described in this report is shown in Table 17.

TABLE 17

ESTIMATED CONSTRUCTION COST FOR
TRANSMISSION SYSTEM

Facility	Probable Construction Cost
22,000 L.F. 36-inch Pipeline	$1,760,000
440 L.F. Dual 36-inch Pipe River Crossing	154,000
16,000 L.F. 50 ft. Wide R/W	92,000
Transmission Pump Station (with initial 2 pumps)	1,290,000
500 KW Stand-by Engine Generator	160,000
1.5 Million Gallon Forebay Tank	420,000
Total Preliminary Estimated Construction Cost of Transmission Facilities	$3,876,000

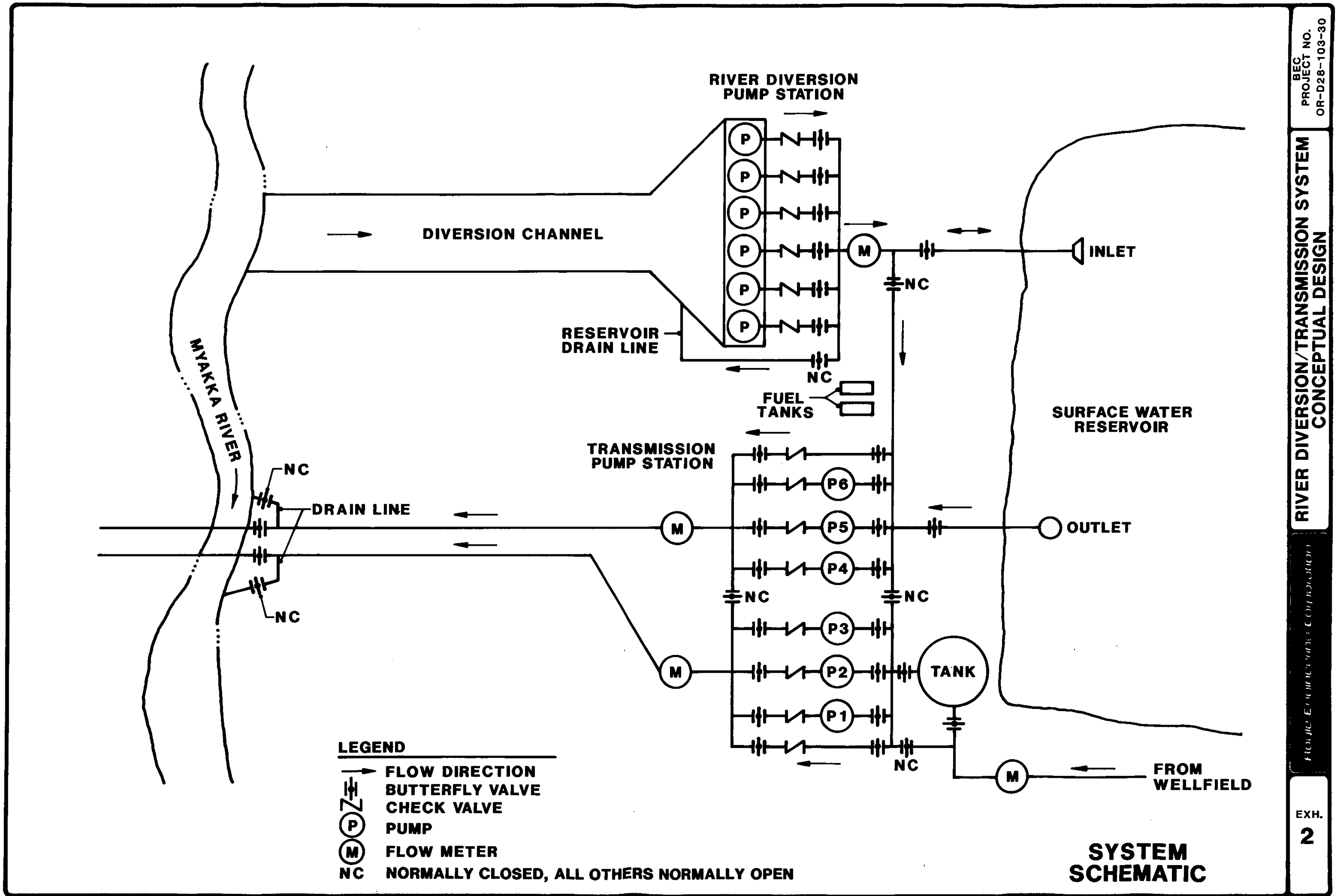


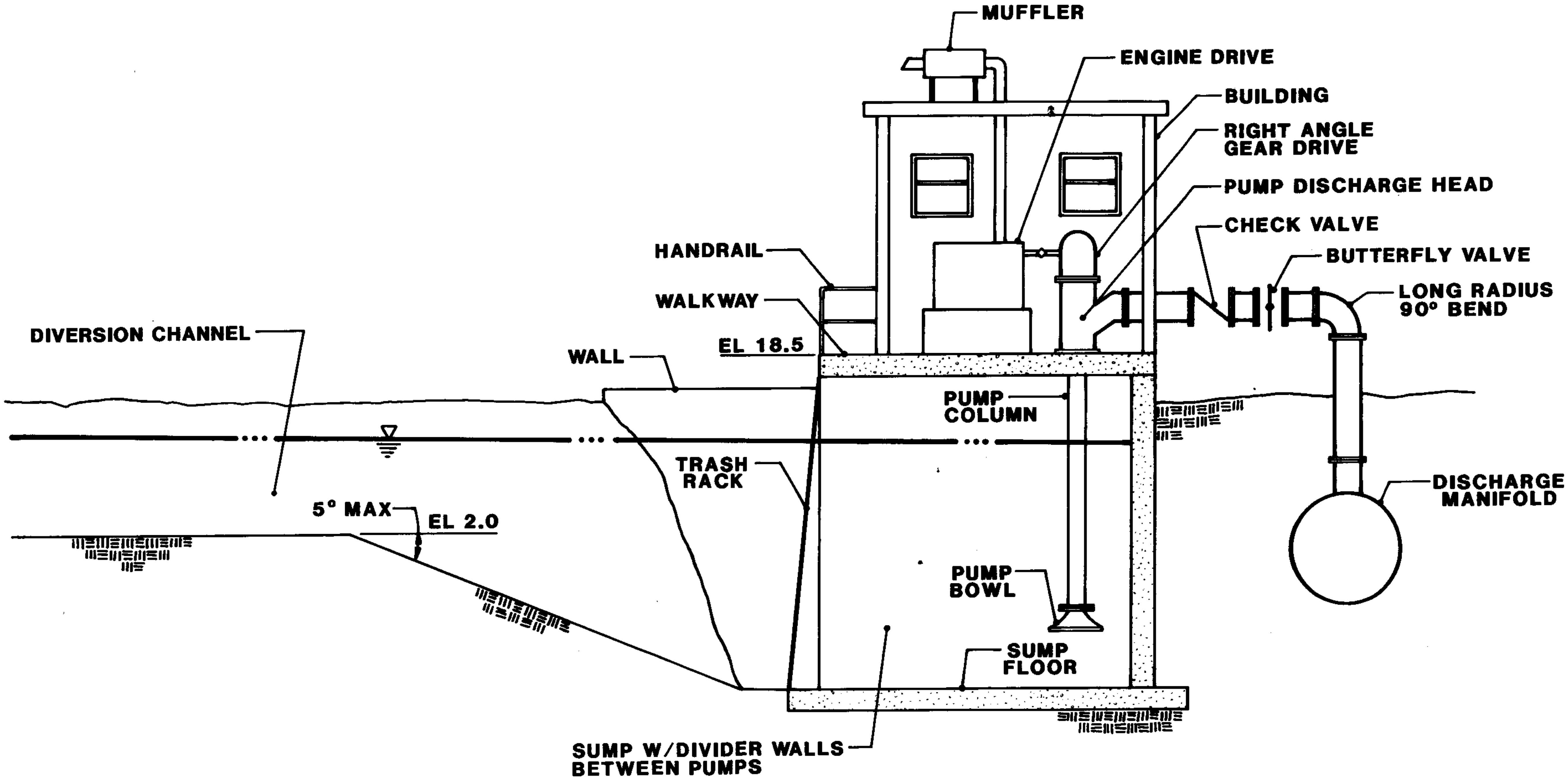
BEC PROJECT NO. OR-D28-103-30

RIVER DIVERSION/TRANSMISSION SYSTEM CONCEPTUAL DESIGN

Boyle Engineering Corporation

EXH. 1


RIVER DIVERSION
PUMP STATION
DIVERSION CHANNEL
INLET
NC
RESERVOIR
DRAIN LINE
NC
FUEL
TANKS
SURFACE WATER
RESERVOIR
MYAKKA RIVER
TRANSMISSION
PUMP STATION
NC
DRAIN LINE
P6
P5
OUTLET
P4
NC
NC
NC
P3
P2
TANK
P1
NC
FROM
WELLFIELD
LEGEND
FLOW DIRECTION
BUTTERFLY VALVE
CHECK VALVE
PUMP
FLOW METER
NC NORMALLY CLOSED, ALL OTHERS NORMALLY OPEN
SYSTEM
SCHEMATIC
BEC
PROJECT NO.
OR-D28-103-30
RIVER DIVERSION/TRANSMISSION SYSTEM
CONCEPTUAL DESIGN
EXH.
2



PUMP STATION SECTION

BEC PROJECT NO. OR-D28-103-30

RIVER DIVERSION/TRANSMISSION SYSTEM CONCEPTUAL DESIGN

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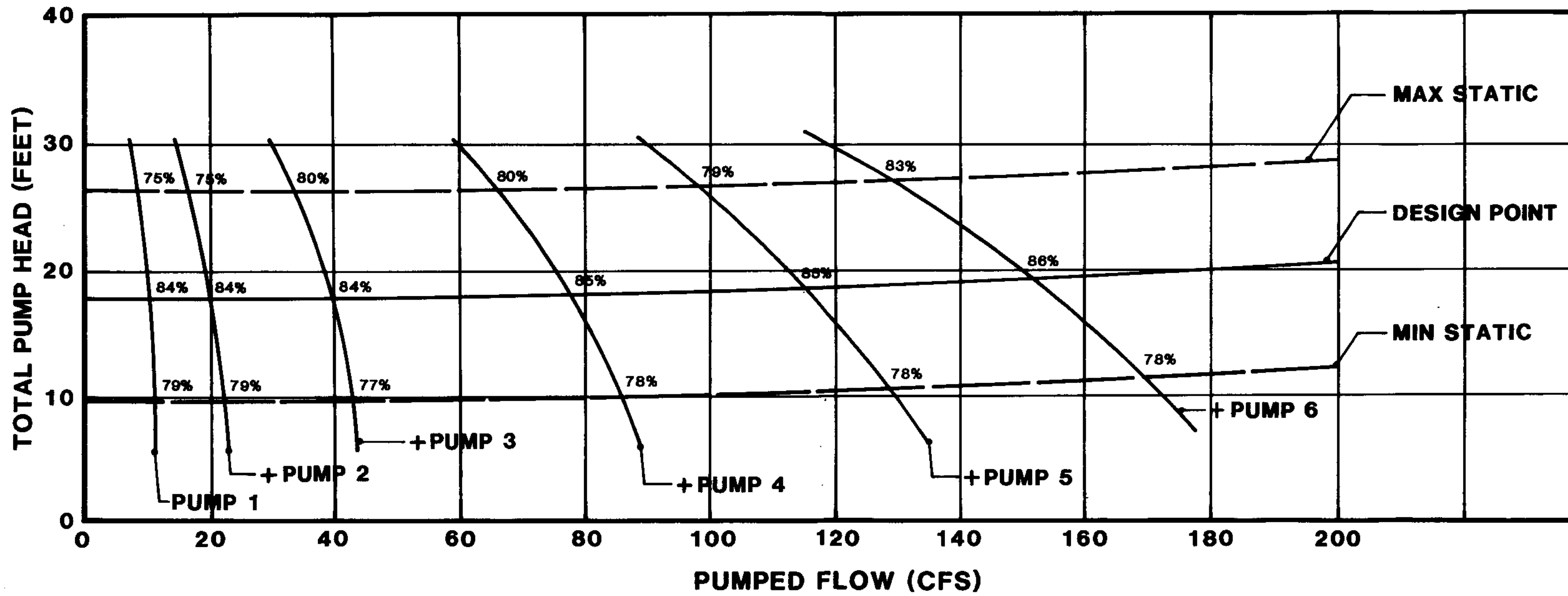
EXH. 3



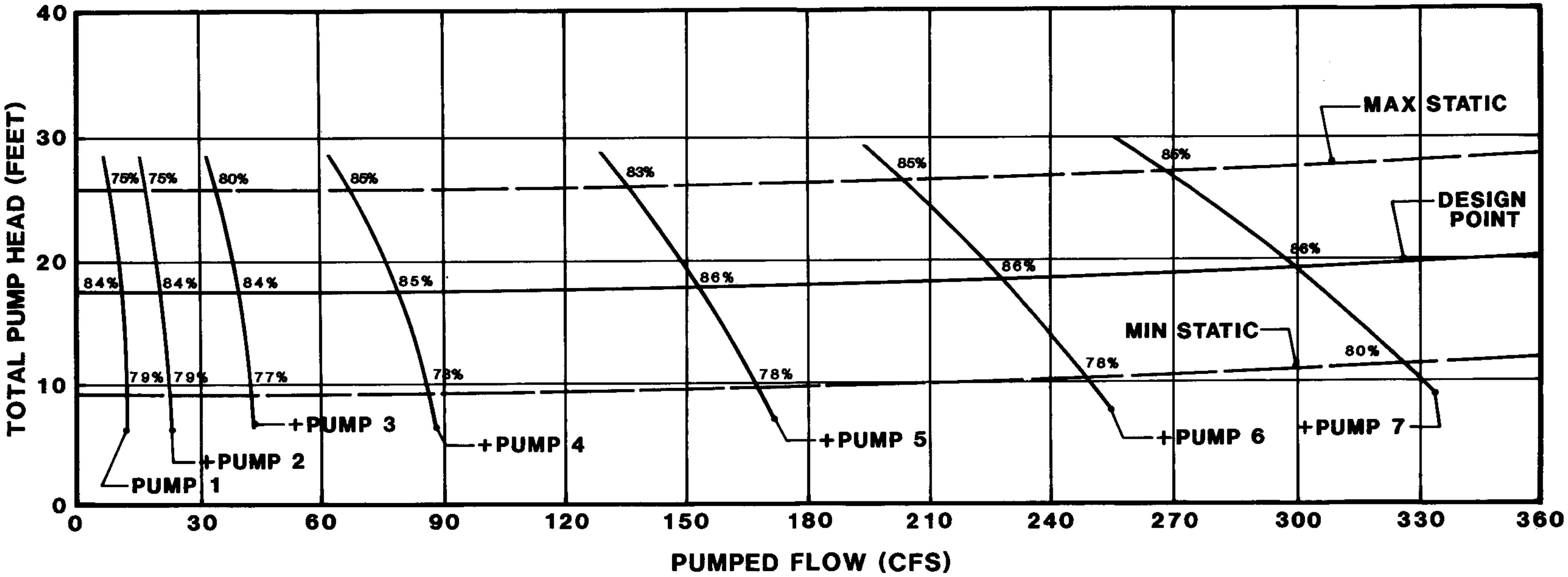
BEC PROJECT NO. OR-D28-103-30

RIVER DIVERSION/TRANSMISSION SYSTEM
CONCEPTUAL DESIGN

Boyle Engineering Corporation

EXH. 4

SYSTEM HEAD AND PUMP CURVES 150 CFS STATION

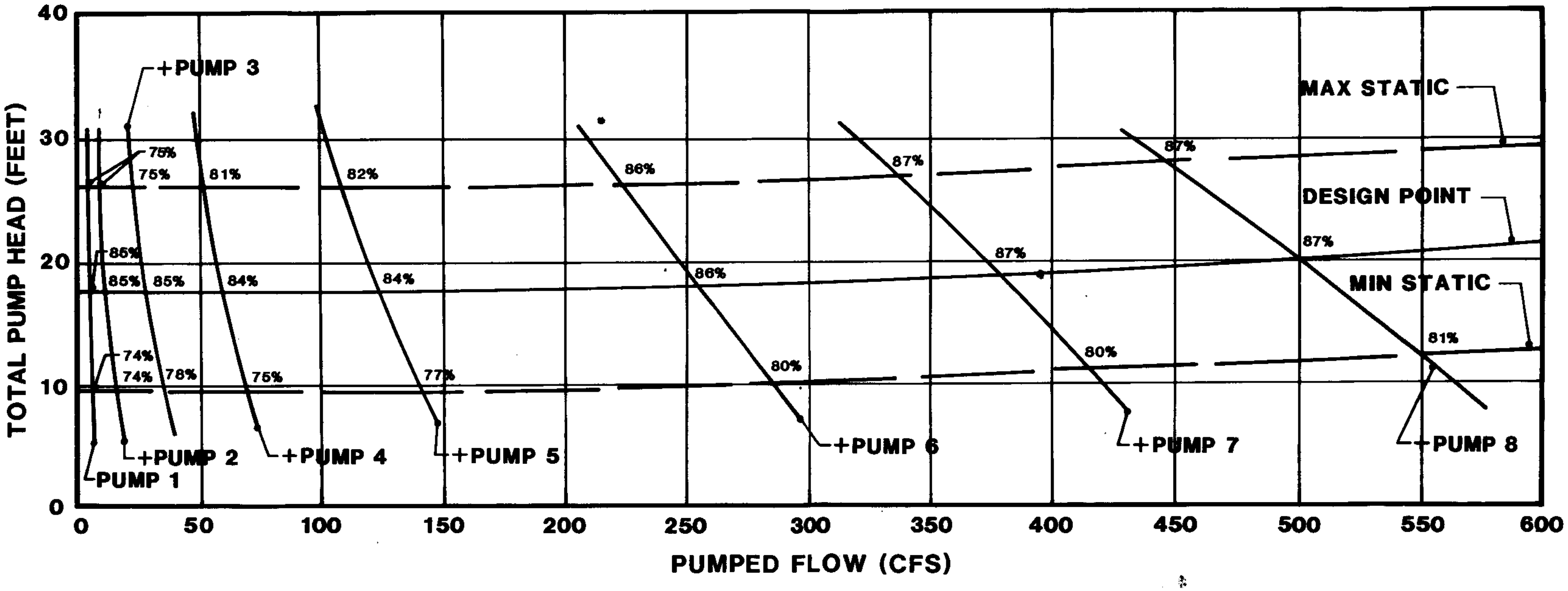


BEC PROJECT NO. OR-D28-103-30

RIVER DIVERSION/TRANSMISSION SYSTEM
CONCEPTUAL DESIGN

EXH. 5

SYSTEM HEAD AND PUMP CURVES 300 CFS STATION



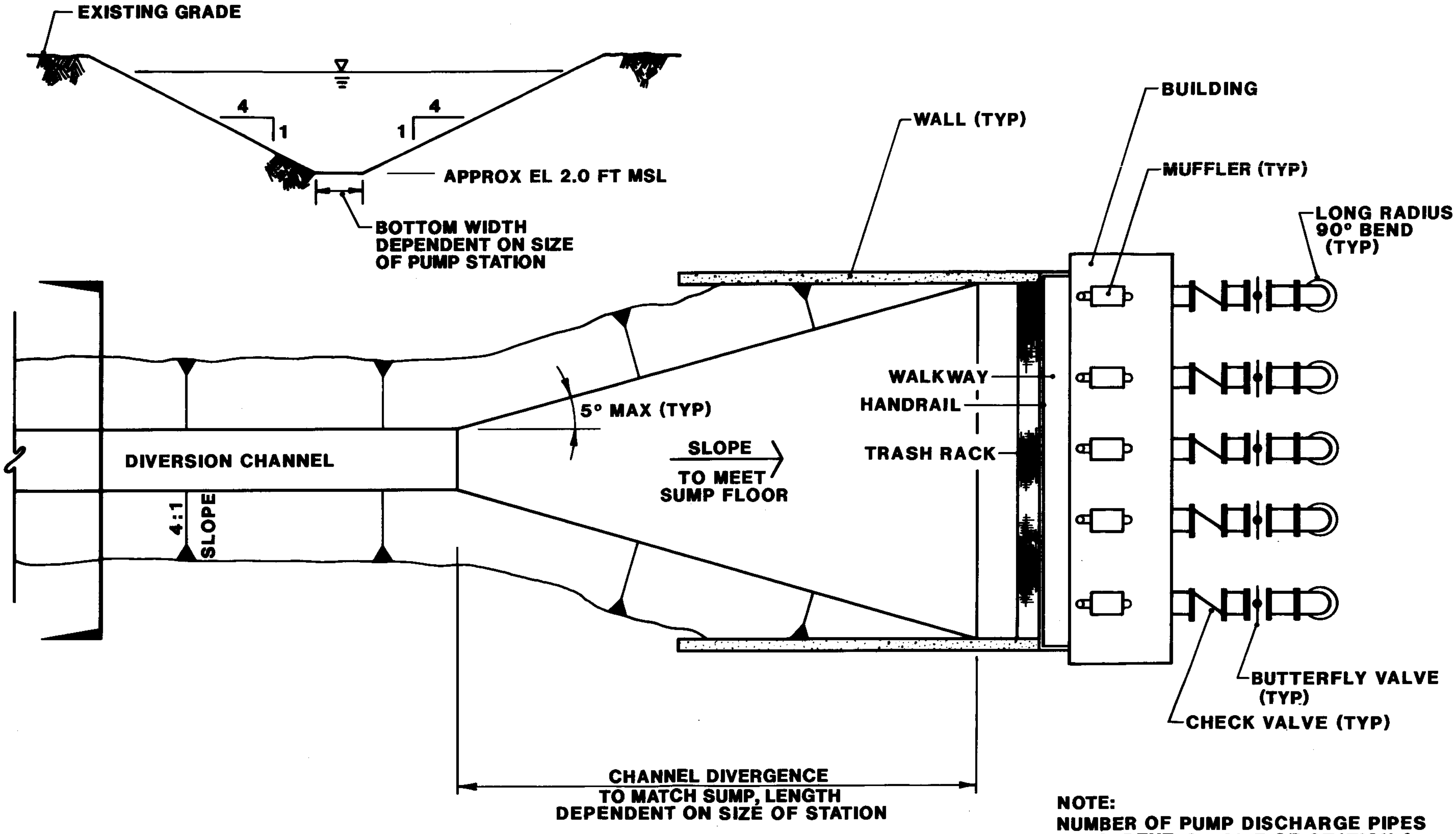
BEC PROJECT NO. OR-D28-103-30

RIVER DIVERSION/TRANSMISSION SYSTEM
CONCEPTUAL DESIGN

Boyle Engineering Corporation

EXH. 6

SYSTEM HEAD AND PUMP CURVES 500 CFS STATION



NOTE:
NUMBER OF PUMP DISCHARGE PIPES DEPENDENT ON SIZE OF STATION & SIZE OF PUMPS

TYPICAL CHANNEL SECTION AND PUMP STATION PLAN

BEC PROJECT NO. OR-D28-103-30

RIVER DIVERSION/TRANSMISSION SYSTEM CONCEPTUAL DESIGN

EXH. 7

LEGEND

- SECTION LINE
- 100 YEAR FLOOD PLAIN
- DIVERSION CHANNEL
- P TRANSMISSION PUMP STATION
- RESERVOIR PIPELINE
- RESERVOIR EMBANKMENT
- PROPERTY LINE

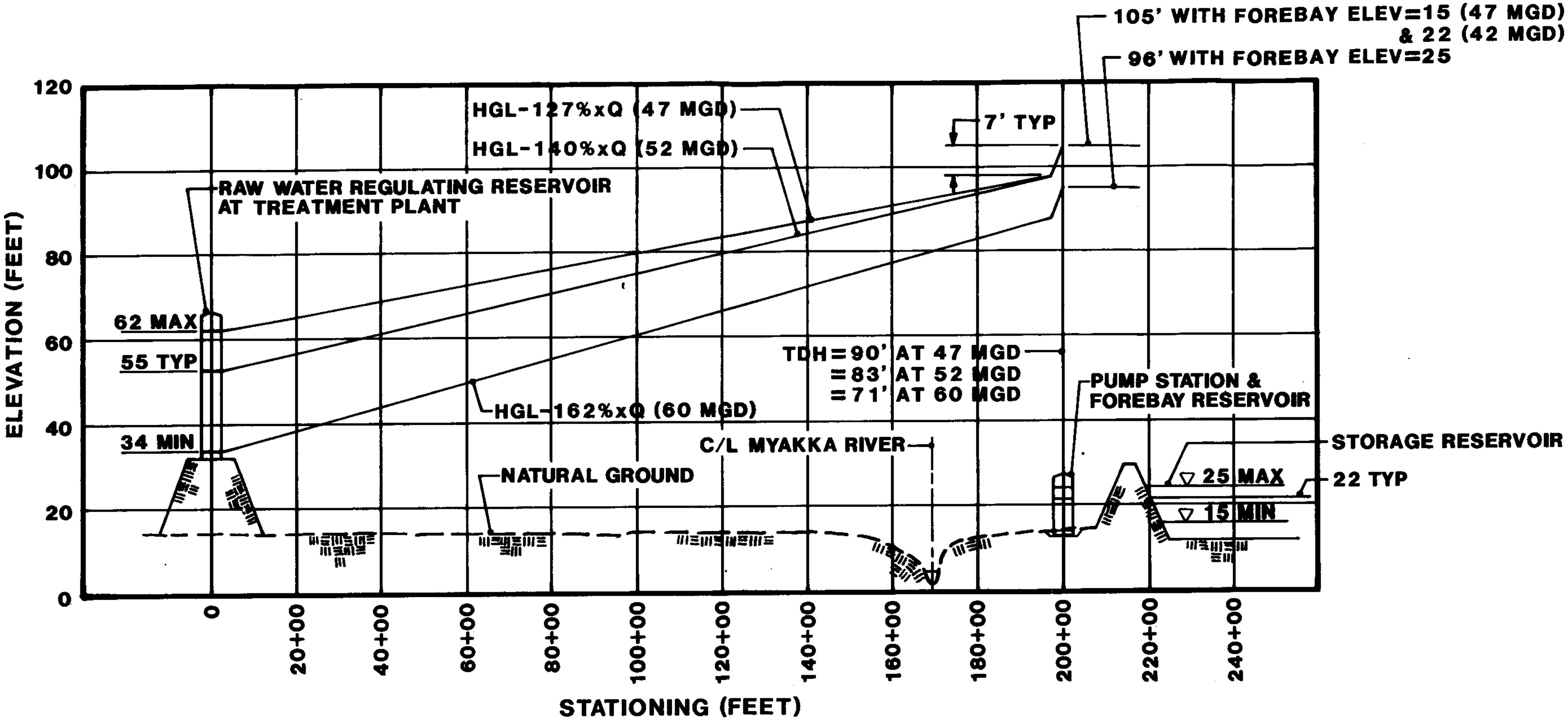
TRANSMISSION PIPELINE ROUTES

BEC PROJECT NO. OR-D28-103-30

RIVER DIVERSION/TRANSMISSION SYSTEM CONCEPTUAL DESIGN

Boyle Engineering Corporation

EXH. 8



NOTES:

Q=AVERAGE ANNUAL FLOW RATE IN MILLION GALLONS PER DAY. (Q ULTIMATE=37.5 MGD)
PROFILE BASED ON PROPOSED TRANSMISSION PIPELINE ALIGNMENT-ROUTE B

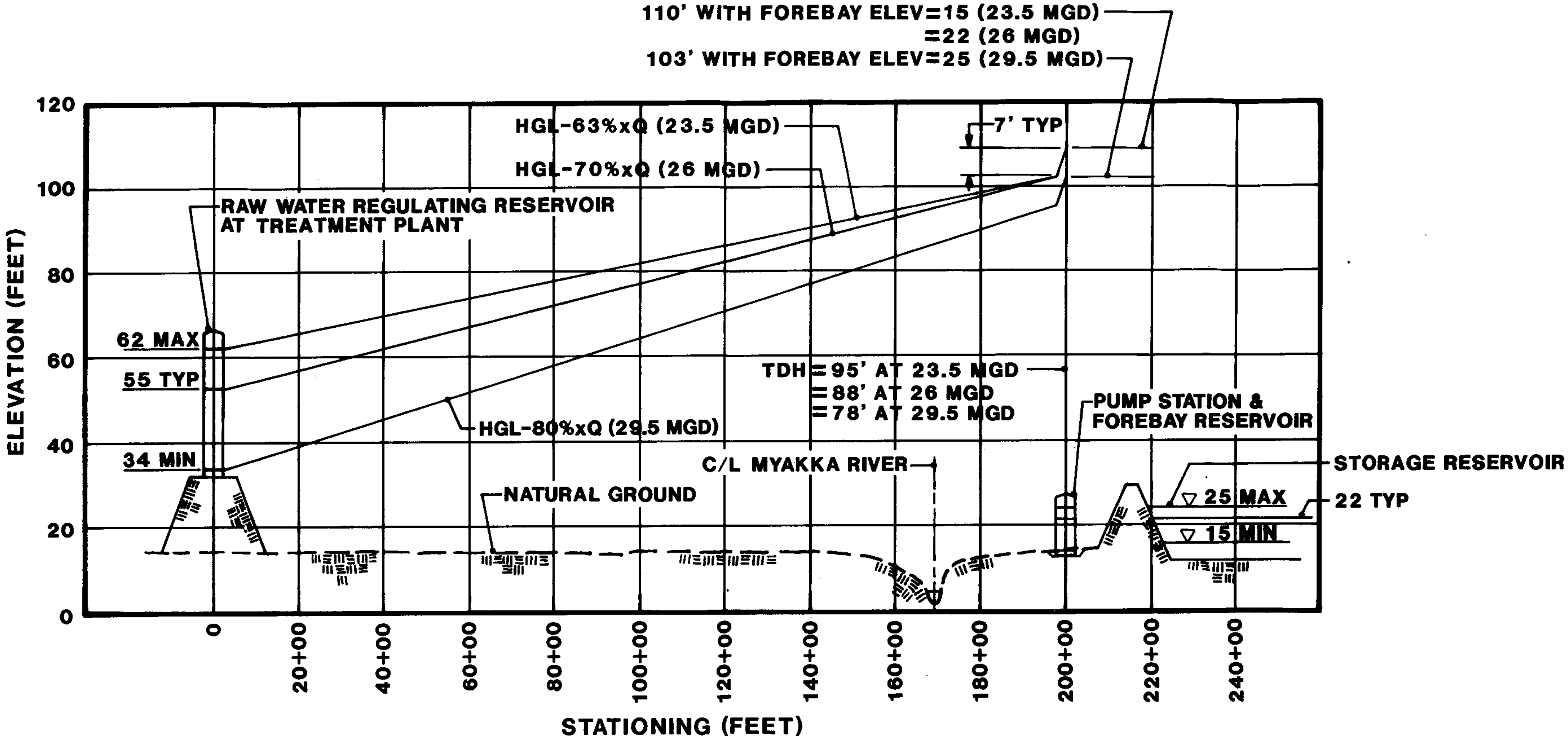
HYDRAULIC SCHEMATIC TRANSMISSION SYSTEM SINGLE 48-INCH PIPE

BEC PROJECT NO. OR-D28-103-30

RIVER DIVERSION/TRANSMISSION SYSTEM CONCEPTUAL DESIGN

Boyle Engineering Corporation

EXH. 9



NOTES:

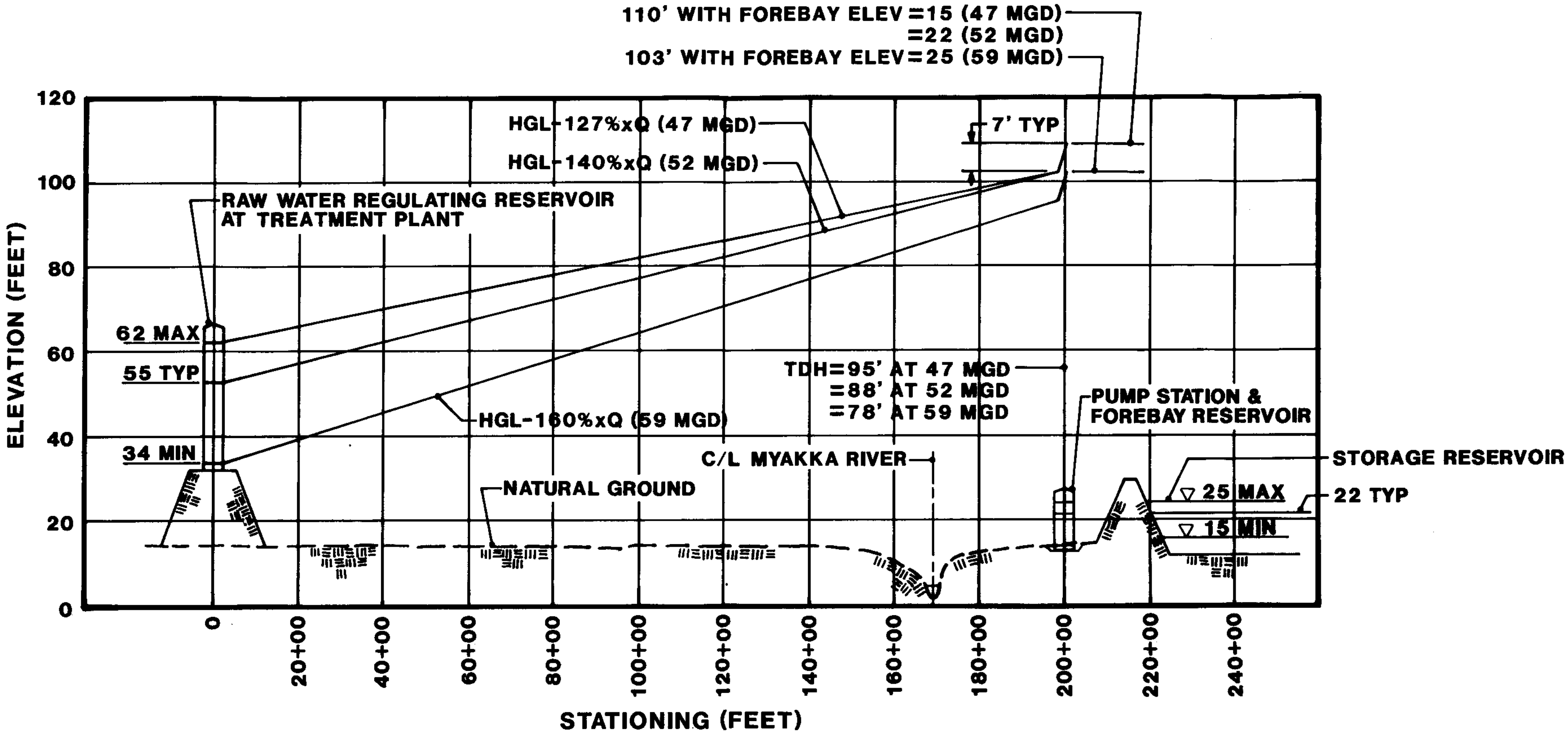
Q=AVERAGE ANNUAL FLOW RATE IN MILLION GALLONS PER DAY. (Q ULTIMATE=37.5 MGD)
PROFILE BASED ON PROPOSED TRANSMISSION PIPELINE ALIGNMENT-ROUTE B

BEC PROJECT NO. OR-D28-103-30

RIVER DIVERSION/TRANSMISSION SYSTEM
CONCEPTUAL DESIGN

HYDRAULIC SCHEMATIC TRANSMISSION SYSTEM SINGLE 36-INCH PIPE

EXH. 10



NOTES:

Q=AVERAGE ANNUAL FLOW RATE IN MILLION GALLONS PER DAY. (Q ULTIMATE=37.5 MGD)
PROFILE BASED ON PROPOSED TRANSMISSION PIPELINE ALIGNMENT-ROUTE B

HYDRAULIC SCHEMATIC TRANSMISSION SYSTEM DOUBLE 36-INCH PIPES

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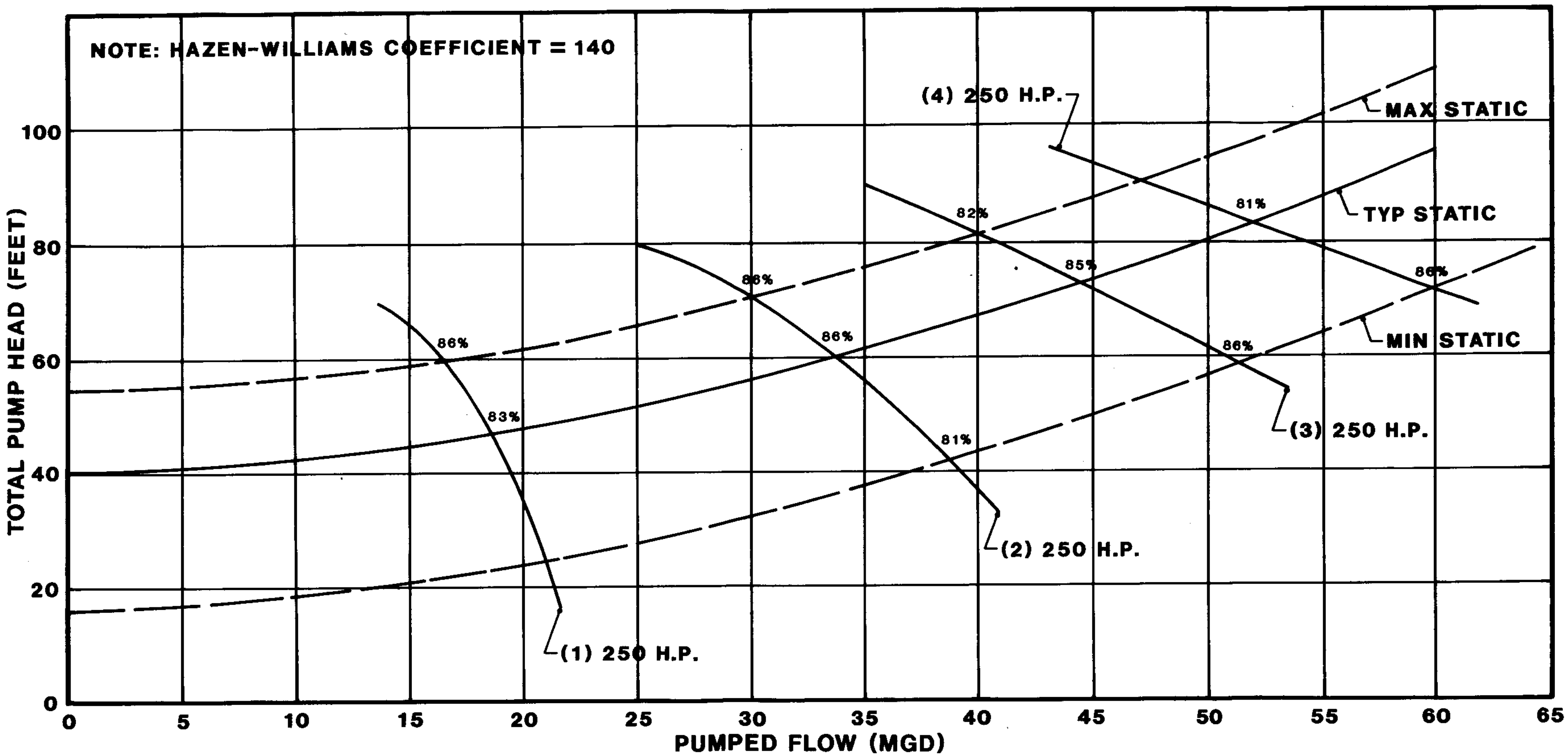
RIVER DIVERSION/TRANSMISSION SYSTEM CONCEPTUAL DESIGN

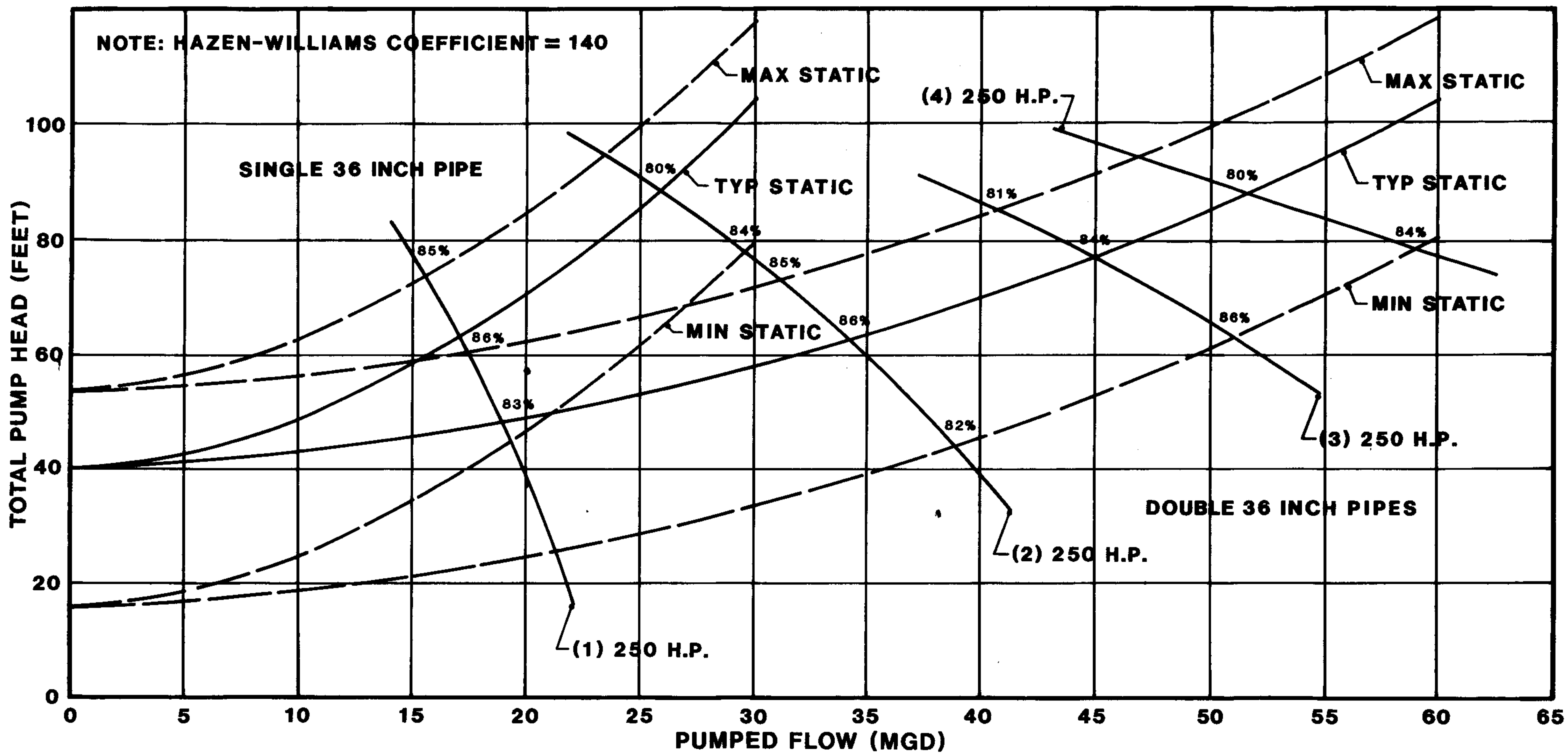
EXH. 11

48 - INCH PIPELINE



TRANSMISSION SYSTEM HEAD AND PUMP CURVES

36 - INCH PIPELINES



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RIVER DIVERSION/TRANSMISSION SYSTEM CONCEPTUAL DESIGN

EXH. 13

TRANSMISSION SYSTEM HEAD AND PUMP CURVES