ECOLE NATIONALE DU GENIE DE L'EAU ET DE L'ENVIRONNEMENT DE STRASBOURG

ENGEES

SOUTH FLORIDA WATER MANAGEMENT DISTRICT

"Effects of groundwater withdrawals and surface water management systems on groundwater levels within the isolated wetlands in Jensen Beach area, Florida."

A dissertation submited in partial fulfilment of the degree of 'Ingénieur de l'Ecole Nationale du Génie de L'Eau et de l'Environnement de Strasbourg'

Sandrine DIAZ

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ABSTRACT

"Effects of groundwater withdrawals and surface water management systems on the groundwater levels within the isolated wetlands in Jensen Beach area, Florida."

The purpose of this study was first to determine the aquifer parameters of the Surficial Aquifer system in the Jensen Beach, Florida. The second task of the study was to input those parameters in a pre-existing MODFLOW groundwater model.

A review of pre-existing hydrogeologic studies of the Surficial Aquifer system revealed the presence of three formations : an upper sand zone ; a 3 to 6 meters thick semi-confined unit constituted of fine sand and clay; and finally the principal producing zone that consists of limestone and calcarinite. The public water supply wells are pumping from this producing zone. In the wetland area, a low permeability unit was identified at less than two meters from the ground surface. The continuity of this layer has not been established.

During this study, constant rate and step-drawdown pumping tests were conducted. The data from the tests were analysed using three different methods (Hantush 1955, Hantush Inflection point 1956 and Cooper-Jacob 1946). The results of the analysis shows that transmissivity of the producing zone ranges between 150 to 380 m^2/d and storativity ranges between 0.00028 and 0.009. The analysis showed the influence of lakes on localized groundwater levels within the wetlands. The analysis provided values of the vertical hydraulic conductivity of the semi-confined unit, ranging from 0.0027 and 0.054 m/d. The transmissivity of the upper sand zone was determined by conducting slug tests and ranges between 0.3 and 3 m/d.

A pre-existing groundwater model was modified to emulate the field observations during the constant rate pumping test. Calibration of the model in transient conditions indicated that the model was sensitive to modification of aquifer parameters. The values of the simulated aquifer parameters derived from the calibrated model were within the range of values obtained in the aquifer analysis. Based on the model results, it can be inferred that the aquifer analysis results are correct. In addition, the interaction between the lakes and the groundwater, as determined from the aquifer analysis was verified by the modeling effort.

This study has demonstrated the need to improve the existing model before using it to simulate impacts on wetlands that may result from groundwater withdrawals. Discretization of the model should be modified to account for the confined unit in the wetlands area and to improve simulation of the lakes.

This study provided guidelines for future investigations that will be conducted as part of the isolated wetland program conducted by the South Florida Water Management District.

RESUME

"Effets des prélèvements d'eau et des systèmes de gestion de l'eau de surface sur le niveau des nappes du système aquifère superficiel dans les zones de marais de Jensen Beach, Floride".

La présente étude a consisté dans un premier temps à déterminer les paramètres hydrodynamiques du système aquifère superficiel dans la zone de Jensen Beach. Ces paramètres ont ensuite été utilisés dans un modèle mathématique préexistant utilisant le logiciel MODFLOW.

La lithologie du système aquifère superficiel se divise en trois zones : une formation sableuse de surface, une formation de plus faible perméabilité constituée de sable fin et de lentilles d'argile et enfin la zone principale de production d'eau constituée de sable, de calcaire et d'alluvions. Au niveau des marais, à une profondeur inférieure à deux mètres, un horizon induré de faible perméabilité a également été identifié mais sa continuité n'a pas été prouvée.

Au cours de cette étude, deux essais de pompage ont été réalisés l'un à débit constant et l'autre à débit variable. Les résultats de ces tests ont été analysés par diverses méthodes (Hantush 1955, Point d'inflexion d'Hantush 1956, Cooper-Jacob 1946). Les résultats de l'analyse indiquent une valeur de la transmissivité de la zone de production d'eau variant entre 150 et 380 m^2/j et une valeur du coefficient d'emmagasinement de cette formation variant entre 0.00028 et 0.009. L'analyse des données des essais de pompage a également démontrer l'influence des lacs sur le niveau de l'eau des nappes au niveau des marais.L'analyse de l'essai de pompage a enfin permis de déterminer la valeur de la conductivité hydraulique verticale de la formation de faible perméabilité, cette valeur varie entre 0.0027 et 0.054 m/j. La transmissivité de la formation supérieure sableuse a été déterminée par la réalisation de tests de Bouwer et Rice (1976). Les résultats obtenus indiquent une conductivité hydraulique se situant entre 0.6 et 3 m/j.

Un modèle mathématique préexistant du site de l'étude a été modifié afin de simuler les conditions de terrain observées au cours de l'essai de pompage à débit constant. La calibration du modèle en condition d'écoulement transitoire a montré que le modèle était principalement sensible à la modification des paramètres hydrodynamiques et a confirmé l'influence des lacs sur le niveau des nappes. Le modèle calibré correspondait à des valeurs des paramètres hydrodynamiques incluses dans les intervalles précédemment cités ce qui a permis de conclure que les résultats de l'analyse hydrodynamique étaient cohérents.

Cette étude a également démontré que des améliorations devaient être apportées à la conceptualisation du modèle avant de pouvoir l'utiliser à la détermination des impacts sur les marais résultant des captages d'eau. Il est notamment recommandé de modifier la discrétisation du modèle afin de prendre en compte la présence de l'horizon induré au-dessous des zones marécageuses, et d'améliorer la représentation des lacs.

Par ailleurs, cette étude fournit un ensemble de directives pour les investigations similaires qui sont prévues dans le cadre du programme de recherche mené par le District sur l'amélioration de la protection des marais.

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INTRODUCTION

Two hundred years ago, south Florida was a vast flat, wet, and largely unexplored landscape, consisting of many low lying marshy areas, inhabited primarily by mosquitoes, alligators, snakes, other wildlife and native American "Indians". Progressively people from other parts of the United States and foreign immigrants started to move to south Florida and change the land, draining the swampy areas, by constructing a network of canals. These canals were eventually destined to control the circulation and distribution of surface water. The canals allowed control of flooding that resulted from the rainy and violent storm events occurring regularly in Florida during the wet season.

During the past two hundred years, urban development in south Florida has mostly occurred along the coast, keeping the central part practically intact. The development that has taken place poses a constant threat to preserving the environment. In recent years, it has become recognized that the marshy areas are a very fragile system, but also constitute an important natural asset.

The wetlands represent a very important water resource that must be protected. The South Florida Water Management District is responsible for regulating and controlling regional water distribution and pumping capacities of the cities and counties for public water supply. The District must make sure that the withdrawal of water does not generate impacts on the environment. Several studies have been undertaken to follow the changes that occur in plant communities as a result of pumping and surface water management systems and to avoid or decrease these impacts. The success of such studies depends on establishing a direct relationship between water withdrawals from the aquifer due to pumping and surface water management practices, and water levels relative to ground elevation that occur in adjacent wetlands.

The purpose of this study was to first analyze hydrologic conditions in the Jensen Beach wetlands area to determine aquifer parameters, and then use a pre-existing model to examine the response of the water table to pumping that occurs near the wetlands.

In order to conduct this study, hydrologic data were collected, and analyzed and the results of this analysis were used to calibrate the hydrogeologic model of the site. The modeling was based on utilization of a software package called MODFLOW.

1. DESCRIPTION OF THE STUDY SITE

In this part, the geology of the site, the vegetation, the climate and the evolution of the water treatment facilities and the population will be detailed. The study site is located in Jensen Beach, Martin County, a small city located on the Florida East-coast, situated at about 130 Kilometers North from West Palm Beach (Figure 1). More precisely, the study concerns an area of about 15 square kilometers, adjacent to the water treatment plant in the North Martin County.

1.1 Geological information

Three main hydrostratigraphic units are present within a thickness of 300 meters below the ground level. They are the :

- Surficial Aquifer system,
- The Hawthorn formation,
- Floridan Aquifer system.

The rocks found from the surface to a depth of 4,000 meters below the ground level in this area are sedimentary types, such as sandstone, limestone, clays and sands. Beyond this depth, metamorphic rocks occur. In this study, the focus was placed on the Surficial Aquifer system and on the Floridan Aquifer system because they are the two units that can be used for water supply. Indeed, the Hawthorn formation has a low hydraulic transmissivity and does not allow withdrawal of large quantities of water.

1.1.1 Geology of the Surficial Aquifer system

The Jensen Beach peninsula lies wholly within a physiographic feature known as the Atlantic Coastal Ridge (Lichtler, 1960). This feature parallels the present coastline and is approximately 6 kilometers in width in northern Martin County (Figure 2).

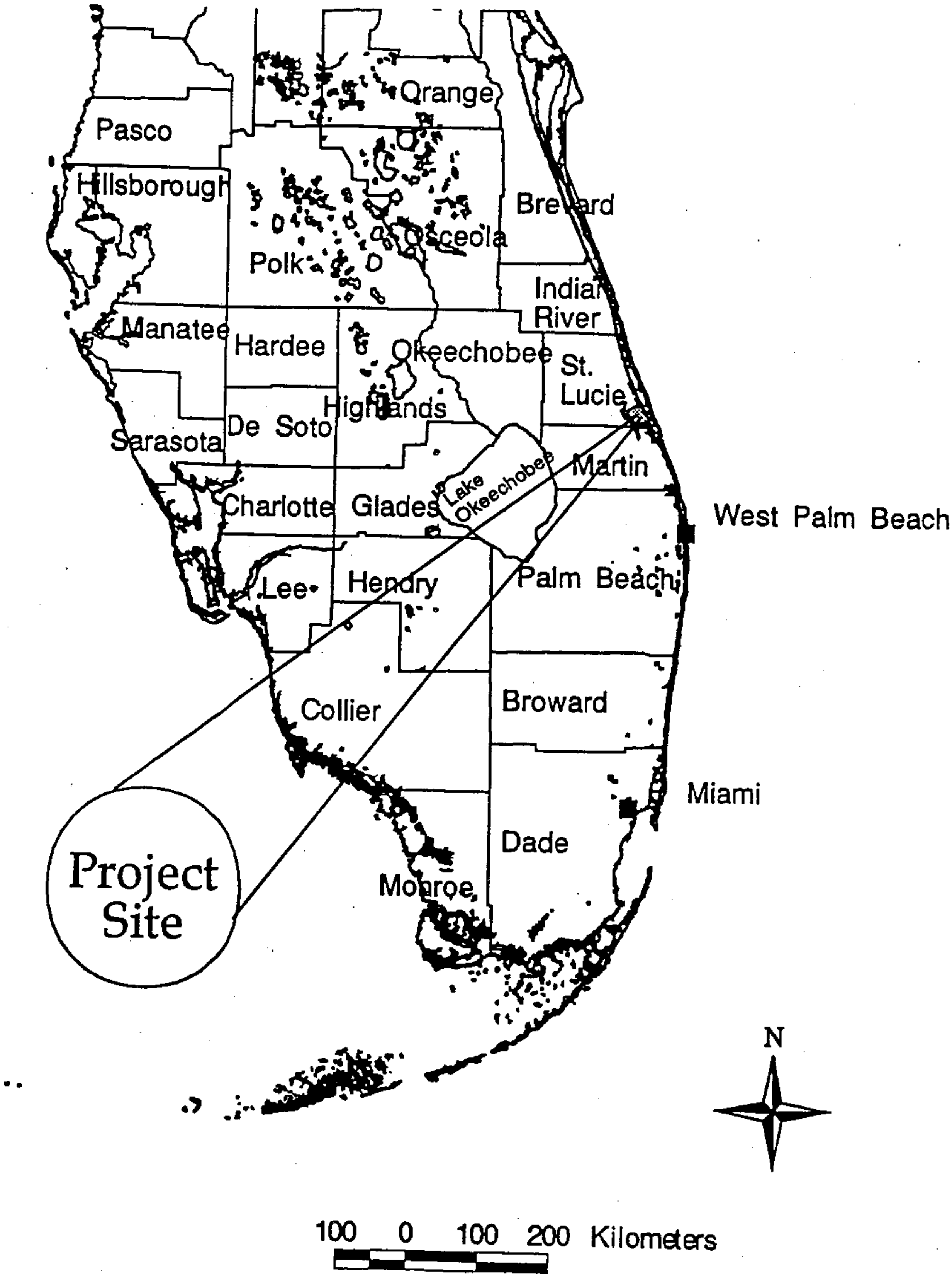
✎ *1.1.1.1 Geology*

The surficial aquifer in the study area consists of formations ranging in age from Upper Miocene to Pleistocene.

Three zones compose this aquifer system

- the upper sand zone.
- the principal producing zone.
- the base of the producing zone.

Figure 1. *Study site location.*



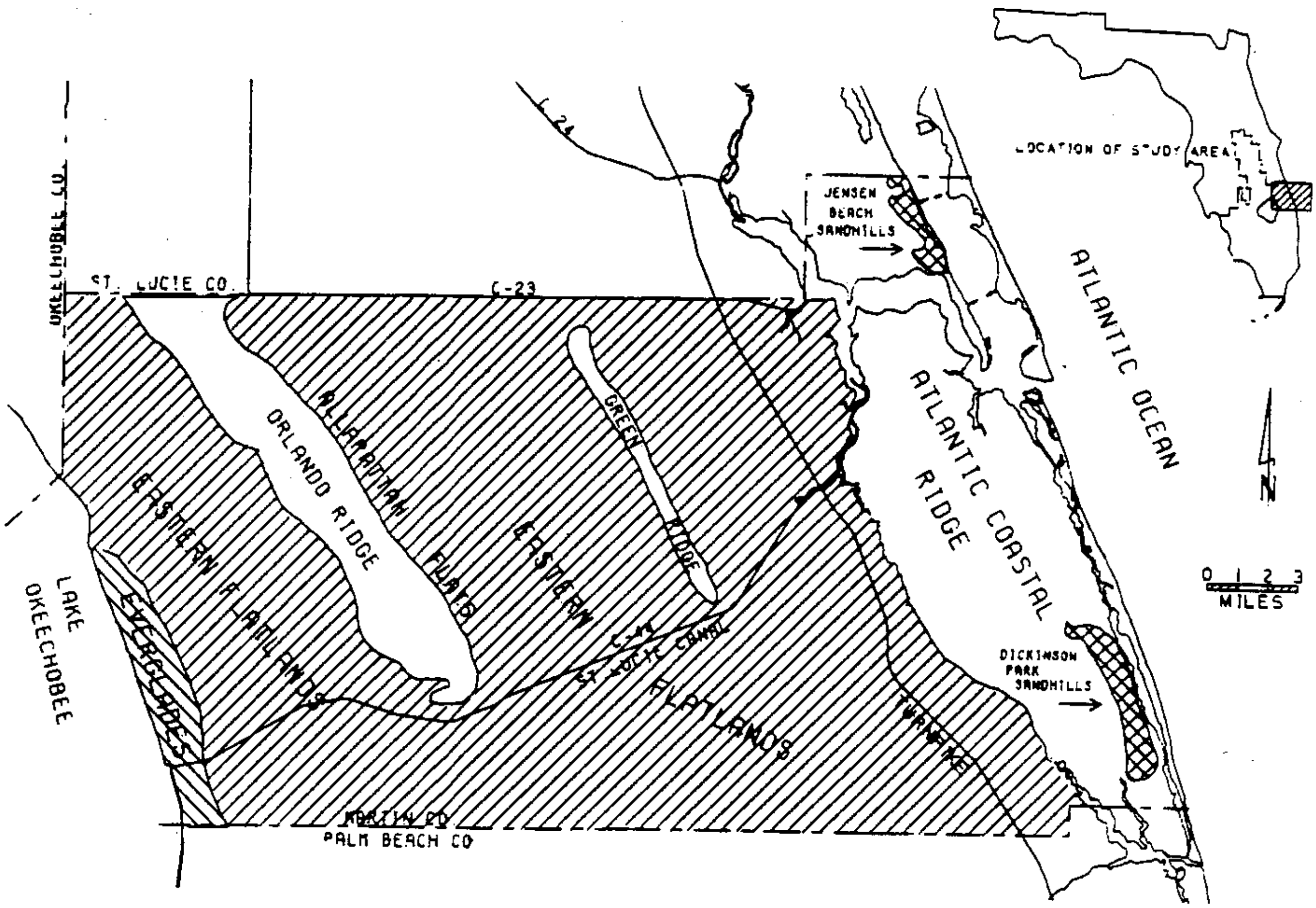


Figure 2. *Physiographic regions of Martin County (Lichtler, 1960).*

The sandy formation is shallow and, at some periods of the year, may not be saturated. It consists mainly of Pamlico sand of the Pleistocene Age. The principal producing zone consists of limestone and calcarenite interbedded with sand and shell proceeding from the Anastasia, Caloosahatchee and Thompson formations. The aquitard consists of shell, marl, limestone and clay of the Tamiami and Hawthorn formations. The elevation of the base of the surficial aquifer in the study area ranges from less than - 48 to in excess of - 55 meters National Geodetic Vertical Datum of 1929 (NGVD) (Figure 3).

The Caloosahatchee marl, of Pleistocene Age consists of "shelly, sandy limestone" and overlies the Tamiami formation in the Jensen Beach area. However, the continuity and thickness of the Caloosahatchee have not been established.

The Fort Thompson formation (Pleistocene) is composed of shell, marl and limestone as far east as the Atlantic Coastal Ridge where it merges with the Anastasia formation (Nealon et al., 1878). Lichtler (1960) indicates that the Anastasia and Fort Thompson formations are contemporaneous. In the study area, the Anastasia formation consists of sand, shell beds, and thin discontinuous layers of limestone (Enos et al., 1977).

1.1.1.2 Hydrogeology

The principal source of fresh water within the Jensen Beach peninsula is the Surficial Aquifer system. The general lithology of the Surficial Aquifer system can be subdivided into three zones :

- From the surface to about 12 to 18 meters below ground, the lithology consists of white gray and brown, predominantly fine to coarse-grained quartz sand, interspersed with shell beds.

- Below this surficial sand, is a 3 to 6 meters thick unit of tan and gray fine to very fine sand, with some traces of shells and a slight increase in clay.

- These elements overlie the principal producing zone that consists of limestone and calcarenite interbedded with sand and shells. This producing zone has a thickness of 40 to 46 meters (Figures 4 and 5).

Lichtler (1960) determined, by use of the Hantush-Jacob analysis method (1956), the aquifer parameters of the Surficial Aquifer system in the Stuart wellfield. This area is located south of the North Martin County Wellfield.

The results of his investigation indicate the transmissivity ranges from 200 to 335 m^2/d and storativity is about 0.025. This value corresponds to a confined unit. The storage coefficient ranges from 0.0233 to 0.00642 day $^{-1}$.

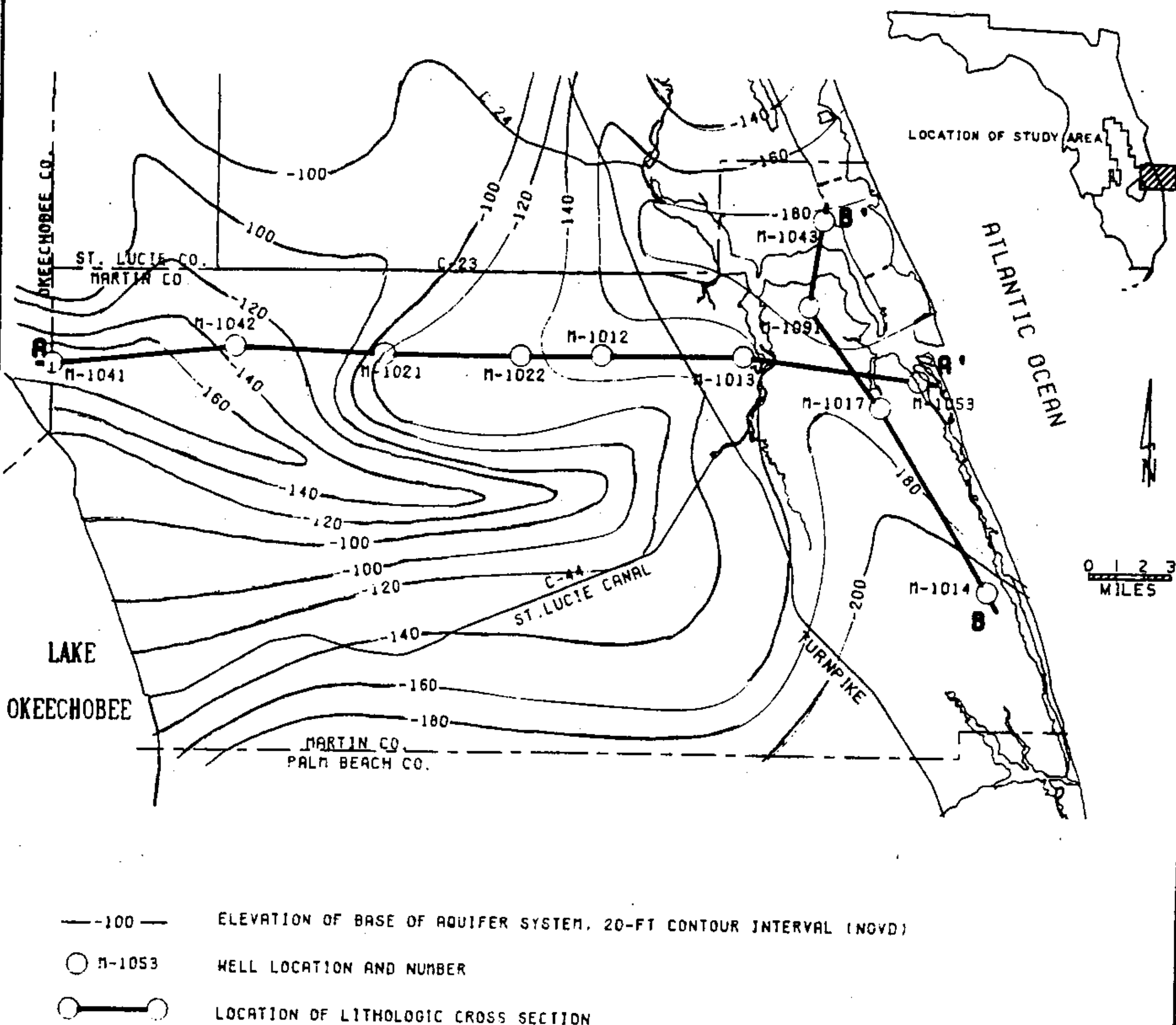


Figure 3. *Altitude of the base of the Surficial Aquifer System, Martin County.(Modified from Miller, 1980).*

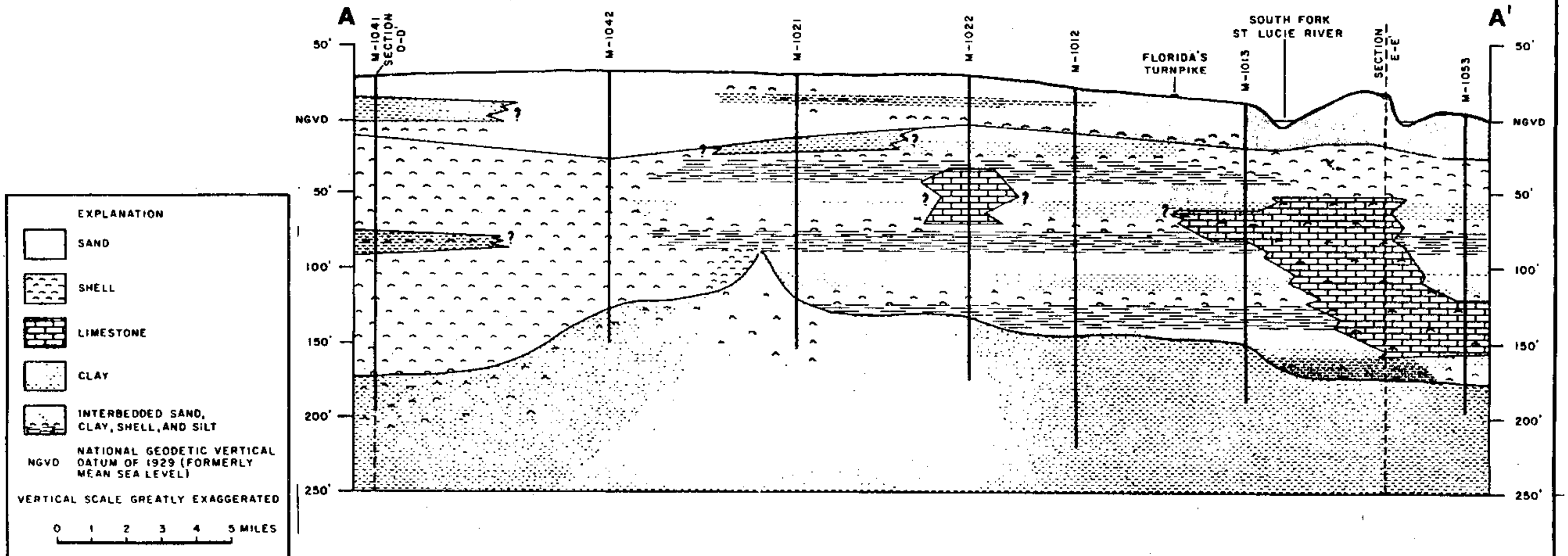


Figure 4. *East-West geologic cross section (After Miller, 1980).*

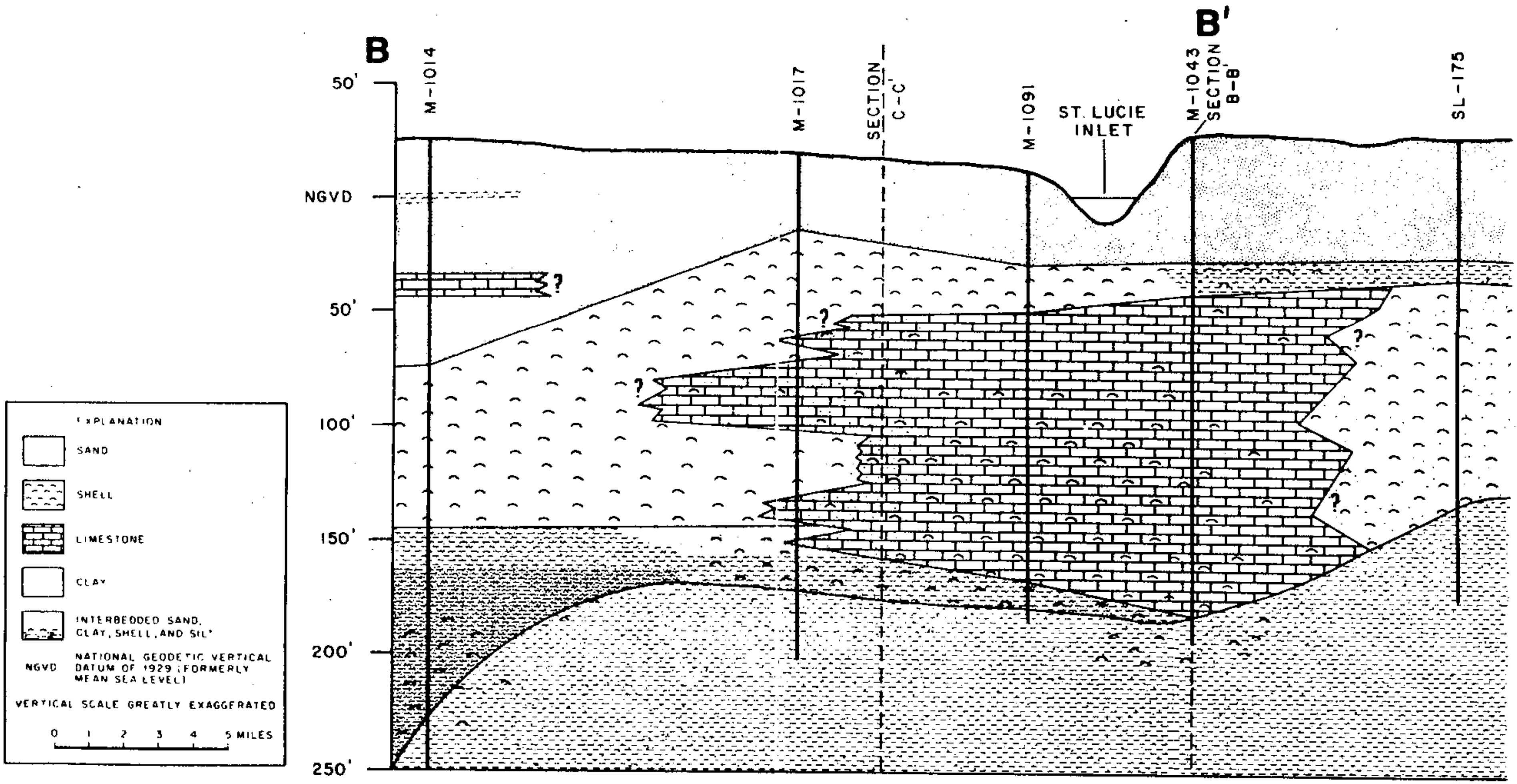


Figure 5. *North-South geologic cross section (After Miller, 1980).*

In 1987, a study of Martin County was conducted by the District and a special report has been written (Nealon et al., 1987). Many transmissivity values were determined and analyzed statistically. A summary of the results is presented in Figures 6 and 7. Inspection of the water table elevation in the study area indicates that regional flow in the shallow aquifer is from Northwest to the Southeast.

Peninsula groundwater levels are monitored from the Saltwater Intrusion Monitoring Program (SWIM) well network required by the Water Use Permit issued by the District to the cities for the wellfield. A number of the wells are installed in clustered configurations to evaluate vertical head variations in the shallow aquifer. Inspection of the SWIM well data (JMM, 1988) indicates that recharging conditions exist at inland locations. This is shown by the decline of heads with depth which in turn, indicates that groundwater is discharging in these areas. Aquifer recharge occurs mainly by rainfall and canal recharge. Indeed, most of the soils in the county consist of sand and are sufficiently permeable to absorb an important part of the water from the rain. A small quantity of water enters the system due to infiltration from the St. Lucie channel when the water table is low. However, the water level in the channel is generally lower than the water table of the Surficial Aquifer system and water is therefore discharged toward the channel.

Generally, the deep ground water is discharged upward to sinks lakes and streams, discharged into the ocean, lost by evaporation ; or withdrawn by pumping.

1.1.2 Geology of the Floridan Aquifer system

The Floridan Aquifer system is composed of thick sequences of interbedded limestone and dolomites. Miller (1982) indicates that this aquifer system is extensive throughout South Florida and ranges in thickness from 850 to 1036 meters in Martin County.

The top of the Floridan Aquifer system is encountered between 198 to 213 meters below sea level in Martin County, as shown in Figure 8. According to Shaw and Trost (1984), this aquifer system is highly permeable due to the fractured nature of the limestone units as well as the high degree of secondary porosity derived from dolomitization and dissolution. A map of the transmissivity values of the Floridan Aquifer system was done by the District in 1987 (Figure 9).

The Floridan Aquifer system is classified as a confined aquifer, because the water within this system is separated from the atmosphere by the thick, relatively impermeable Hawthorn confining beds and the sediments of the Surficial Aquifer system. The hydrostatic pressure of water in the Floridan Aquifer system is greater than atmospheric pressure. Therefore, it can also be referred to as an artesian aquifer.

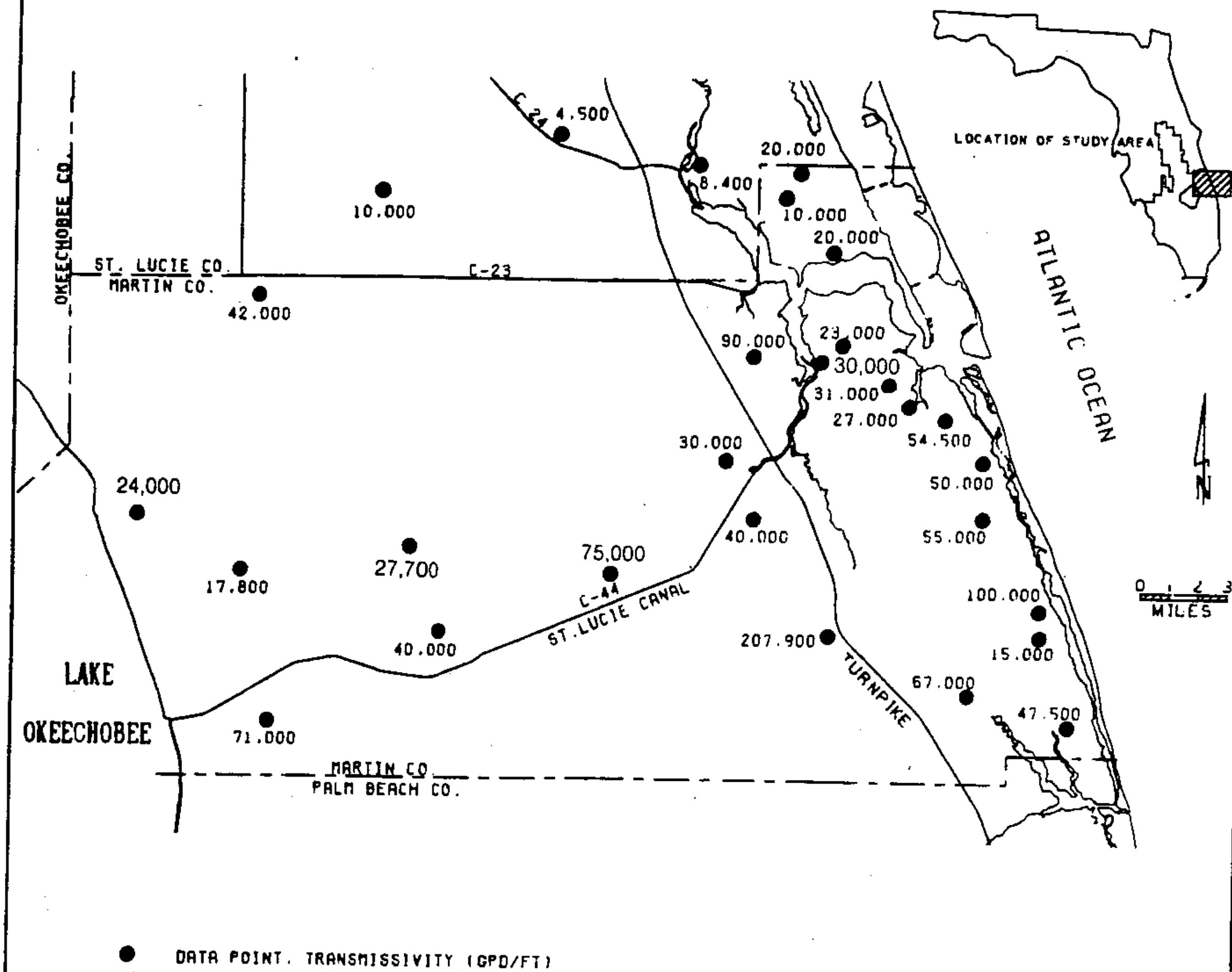


Figure 6. *Locations of the transmissivity values of the Surficial Aquifer System, Martin County.*

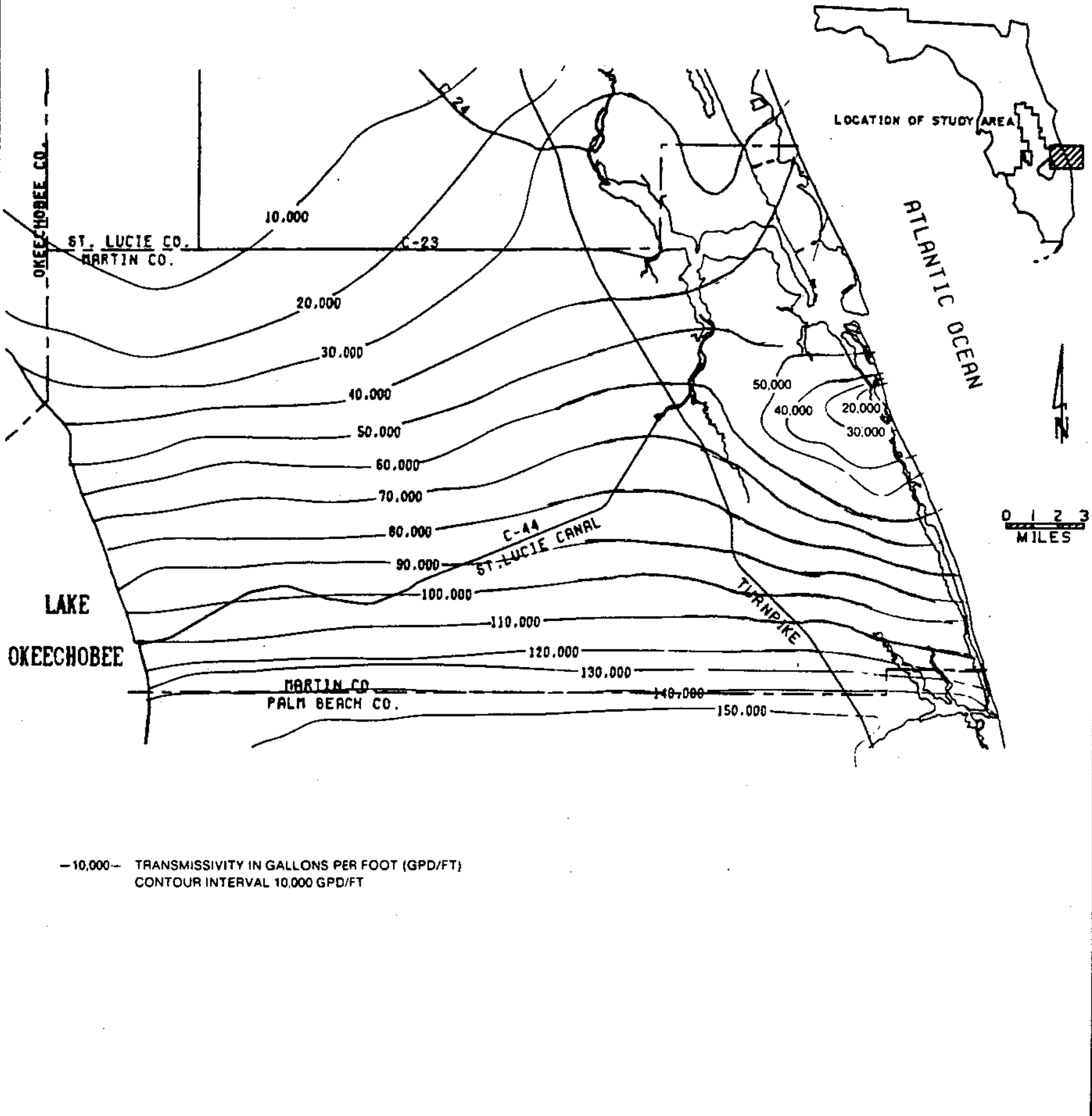


Figure 7. *Statistical trends of the transmissivity of the Surficial Aquifer System, Martin County.*

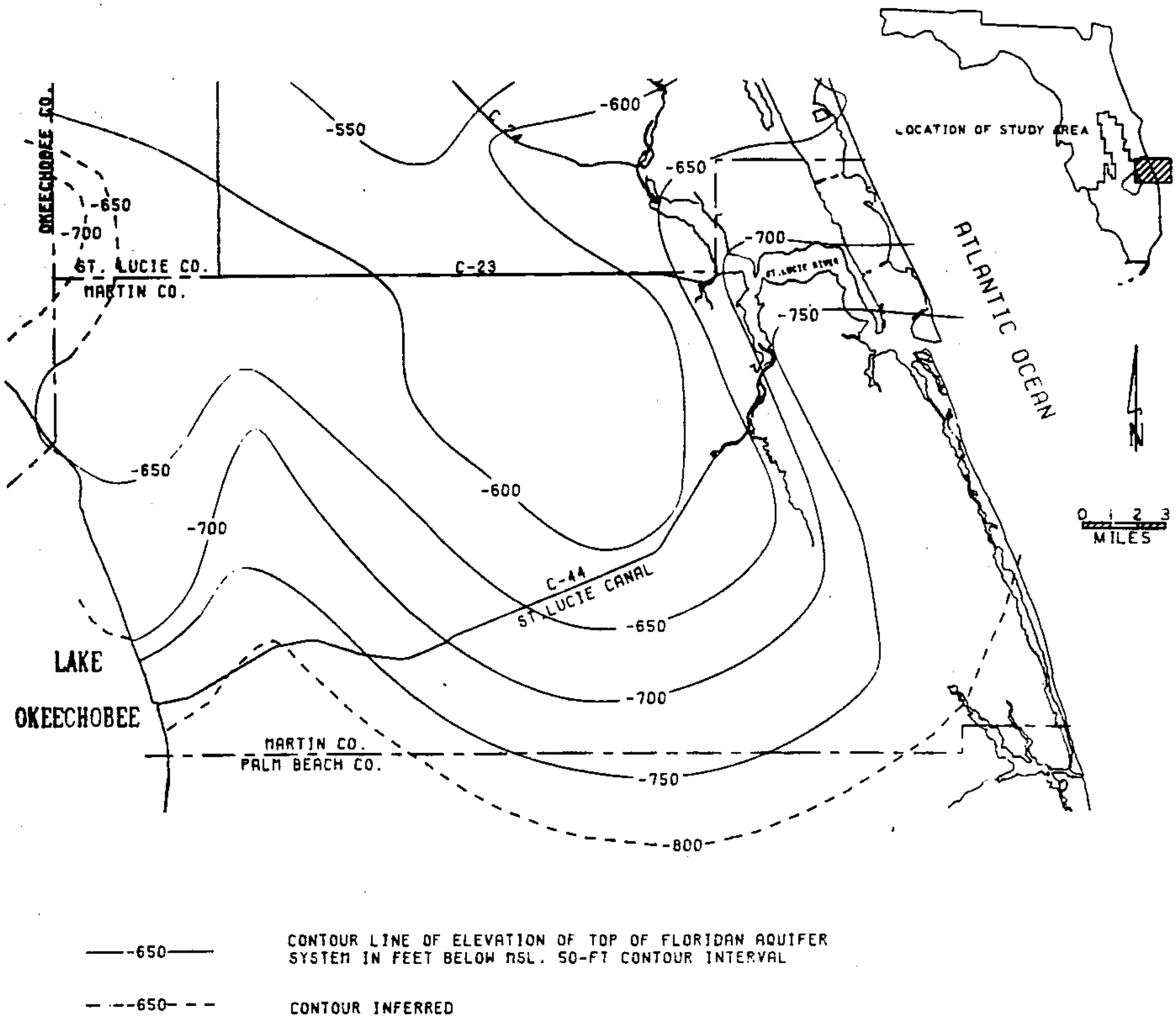


Figure 8. *Elevation of the top of the Floridan Aquifer System (Brown and Reele, 1979).*

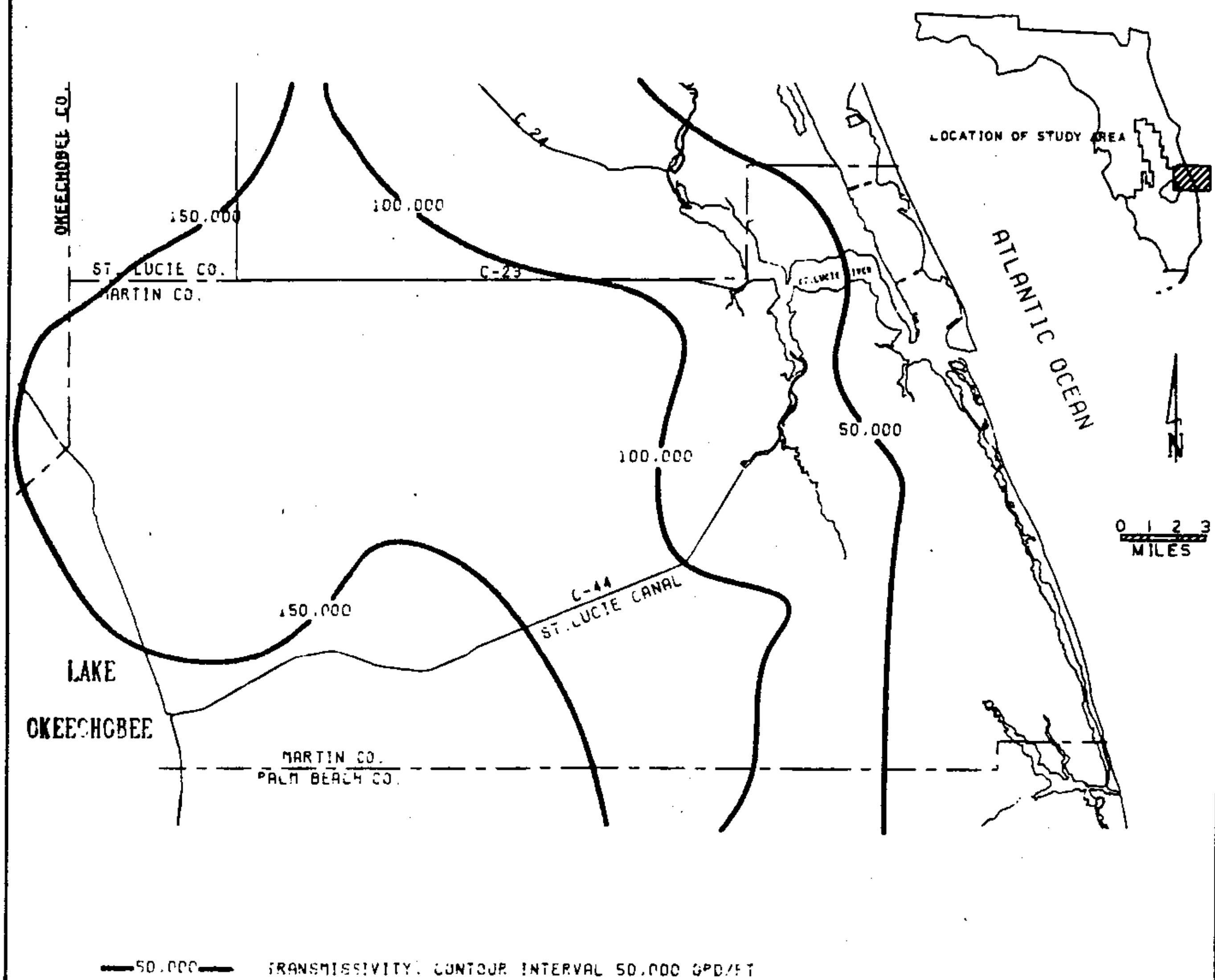


Figure 9. *Transmissivity of the upper Floridan Aquifer System, Martin County.*

The comparison between the potentiometric map presented by Lichlter in April 1957 and the one presented by the District in 1984 indicates a decline of 2 meters for the Northeast part of the county and a decline of 1.5 m for the Southwest part (Figures 10 and 11). This gradual lowering of the potentiometric surface is probably due to an increase in irrigation withdrawals from the aquifer system. Indeed, the water from the Floridan Aquifer system contains high concentration of chlorides and has not been heavily utilized for public distribution because its treatment requires the use of a reverse osmosis process.

The main use of water from the Floridan Aquifer system is therefore for irrigation. The farmers tend to discharge this water into ditches, where it mixes with surface water and ground water from the better quality Surficial Aquifer system. This practice allows farmers to supplement surface water supplies when canal stages are low, and it also minimizes the adverse effects of the saline Floridan Aquifer system waters through dilution. Currently, the potentiometric surface is even lower than it was in 1987. This is due to an increase of the withdrawals both for irrigation and for public supply.

1.2 Climate and Vegetation

1.2.1 Climate

The climate in Florida is subtropical and is characterized by warm, wet summers and mild, with somewhat drier, winters. Heaviest rainfall occurs during the four month period from June through September. Tropical storms and hurricanes may substantially augment wet season precipitation. The dry season includes the months from October through May with April and May as typically the driest months. In Martin County, the annual temperature is 24° Celsius and the annual rainfall is 1448 mm. About 60% of the rainfall occurs during the months of June through October. Evapotranspiration is similar to the rainfall by the fact that it is irregularly distributed during the year. From March to October, evapotranspiration is high and from November to February it is less important. For this study, meteorological data (daily rainfall and temperature) were obtained from the Stuart 1N weather station, situated at about 3 km south of the study area. The North Martin County water treatment also maintains a rainfall gauge. Rainfall data from both stations were used during the pumping tests and during the modeling efforts.

1.2.2 Vegetation

Jensen Beach peninsula is characterized by numerous wetland areas (Figure 12). They are generally small, shallow depressions that have a hydroperiod ranging from several weeks to permanently inundated. Hydroperiod is defined as the number of days a wetland is inundated per year. Hydroperiod of a wetland is one factor that determines the types of wetland plant and animal species present.

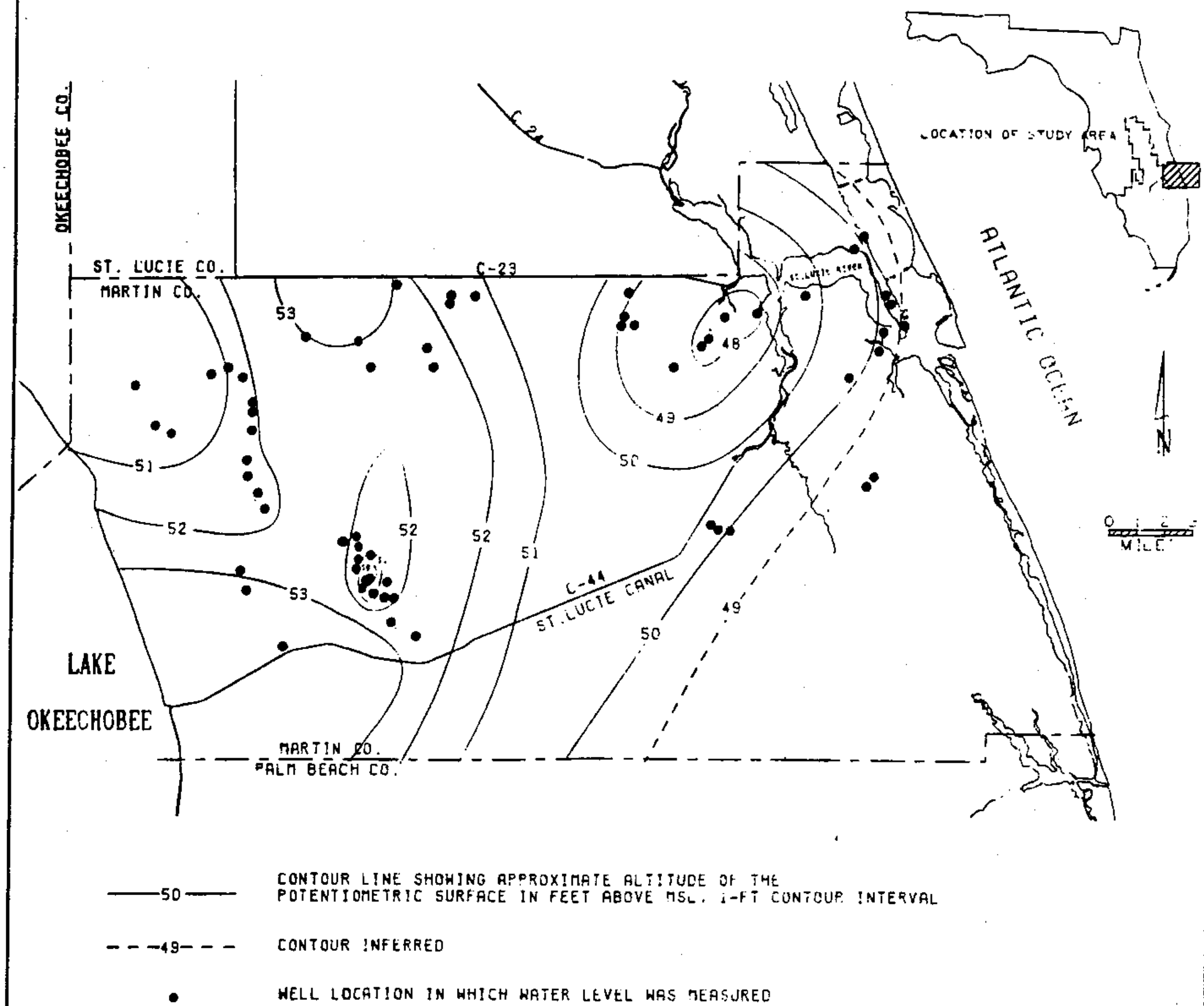


Figure 10. *Potentiometric surface of the Floridan Aquifer System, April 1957, Martin County (Lichtler 1960).*

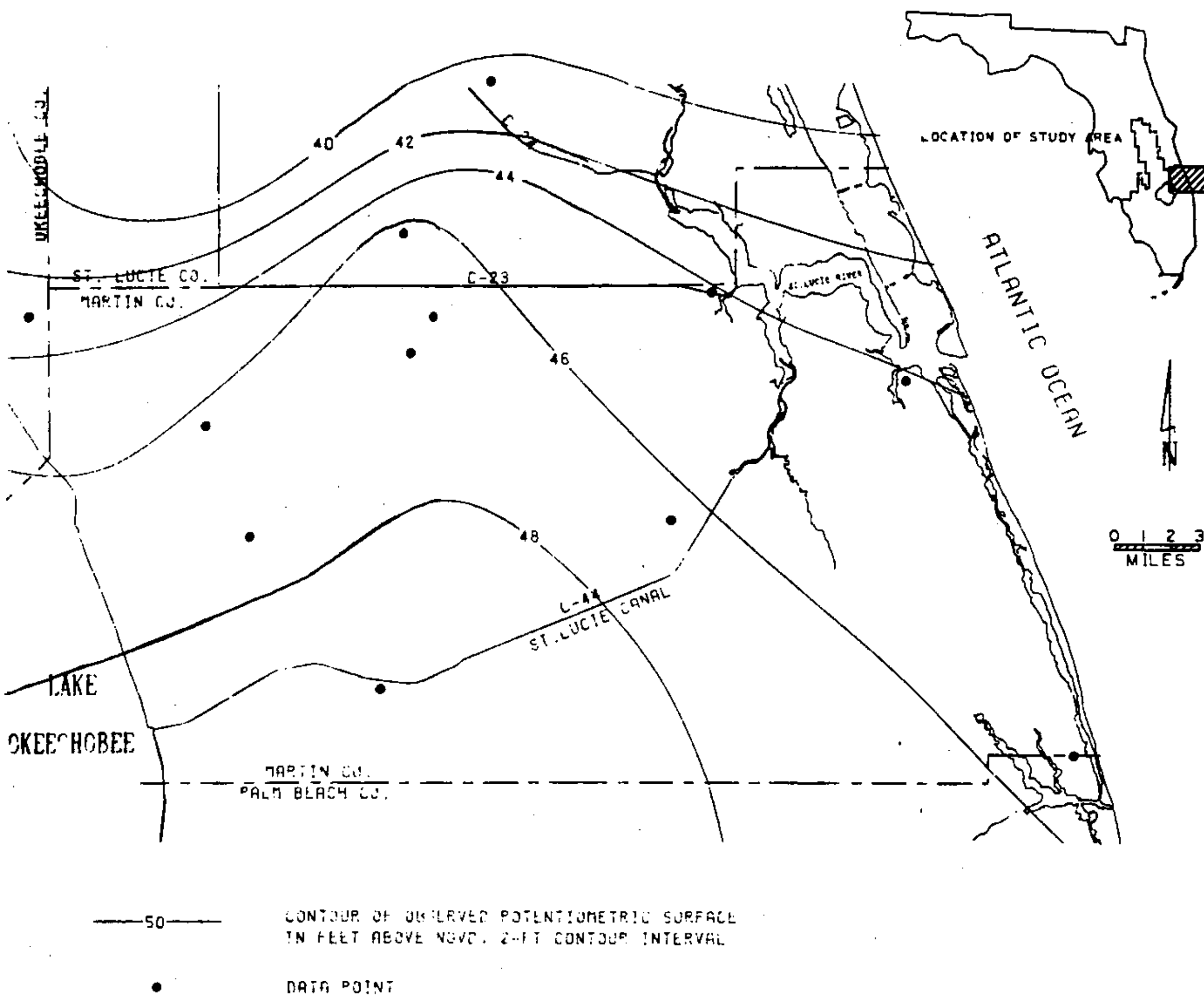
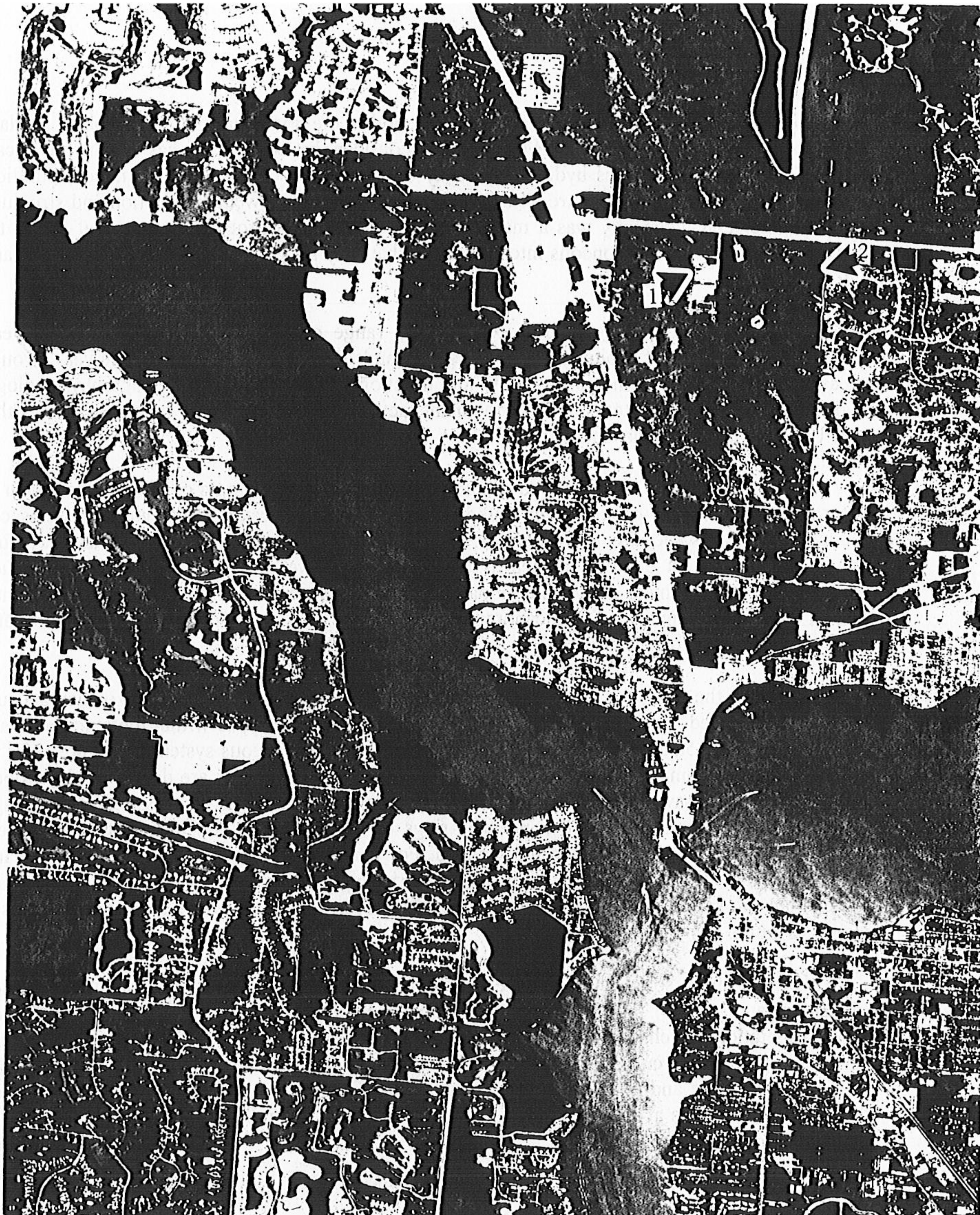


Figure 11. *Potentiometric surface of the Floridan Aquifer System, May 1984, Martin County.*



1. Water treatment plant

2. Water supply well PW-7

Figure 12. *Aerial Photo of Jensen Beach area.*

Duever et al. (1986) suggest that hydroperiod is the dominant factor controlling wetland structure. In contrast, O'Brien and Motts (1980) consider that water level has a more significant effect on wetlands than does hydroperiod. Day et al. (1988) consider that frequency, duration, depth and timing of inundation are all critical factors in analyzing wetland function and structure. They suggest that hydrology was a major influence on wetland processes above and below the ground, and warned of "erroneous interpretation" that may arise by only observing surface water flooding.

Natural wetlands (Figure 13) experience a range of water levels throughout the year, resulting in a gradient of habitats, from adjacent uplands that remain dry most of the year through areas that are exposed to increasing depth and duration of standing water. Topography, geology, hydrology and rainfall patterns are the dominant factors that determine the type of wetlands that may be found in a particular location.

Native plants and animals have adapted to the range of hydrologic conditions that occur in natural wetlands. Both groundwater withdrawals and surface water management systems can reduce water levels and alter hydroperiods, resulting in adverse impacts to such wetlands (Figure 14). One of the most obvious impacts is the reduction in wetland size. Other impacts include increased fire, replacement of wetland fauna and flora by upland species, invasion of exotic plants, loss of tree cover, thinning of tree canopy, loss of organic soils and reduced wetland habitat and wildlife values (S. Mortellaro, 1995).

Within the Jensen Beach area, various wetlands communities composed of herbaceous, shrubs and forested species can be found. The dominant wetland types within the study area are marshes (Figures 15a and b). Marshes refer to depressional herbaceous systems. Swamps refer to forested wetland dominated with trees such as cypress. Shrub wetlands are dominated by woody vegetation such as saw palmetto or wax myrtle (U.S. Army Corp of Engineers, 1988). Although some wetland systems may have concentric rings of varying vegetation, the center or deepest vegetated portion is used to name the wetland. For example, the center of a particular wetland may be dominated with cypress and have a concentric ring of herbaceous plants that is equal to or larger than the spacial coverage of the cypress. However, the entire wetland is referred to as a cypress swamp.

In this study some impacts have been observed such as the presence of exotic species (melaleuca and schinus). These plants are problems to wetland communities because they displace native vegetation and change the community structure. An historical study of the study area using aerial photography is currently under contract with SFWMD to determine the changes in the wetlands over time and provide possible cause and effect relationships. The list of the species found in the wetlands at this study area was prepared by S. Mortellaro, SFWMD on May 1996. This list is presented in Appendix 1.

Figure 13. *Typical Cross Section of Marsh/Woody Wetland*

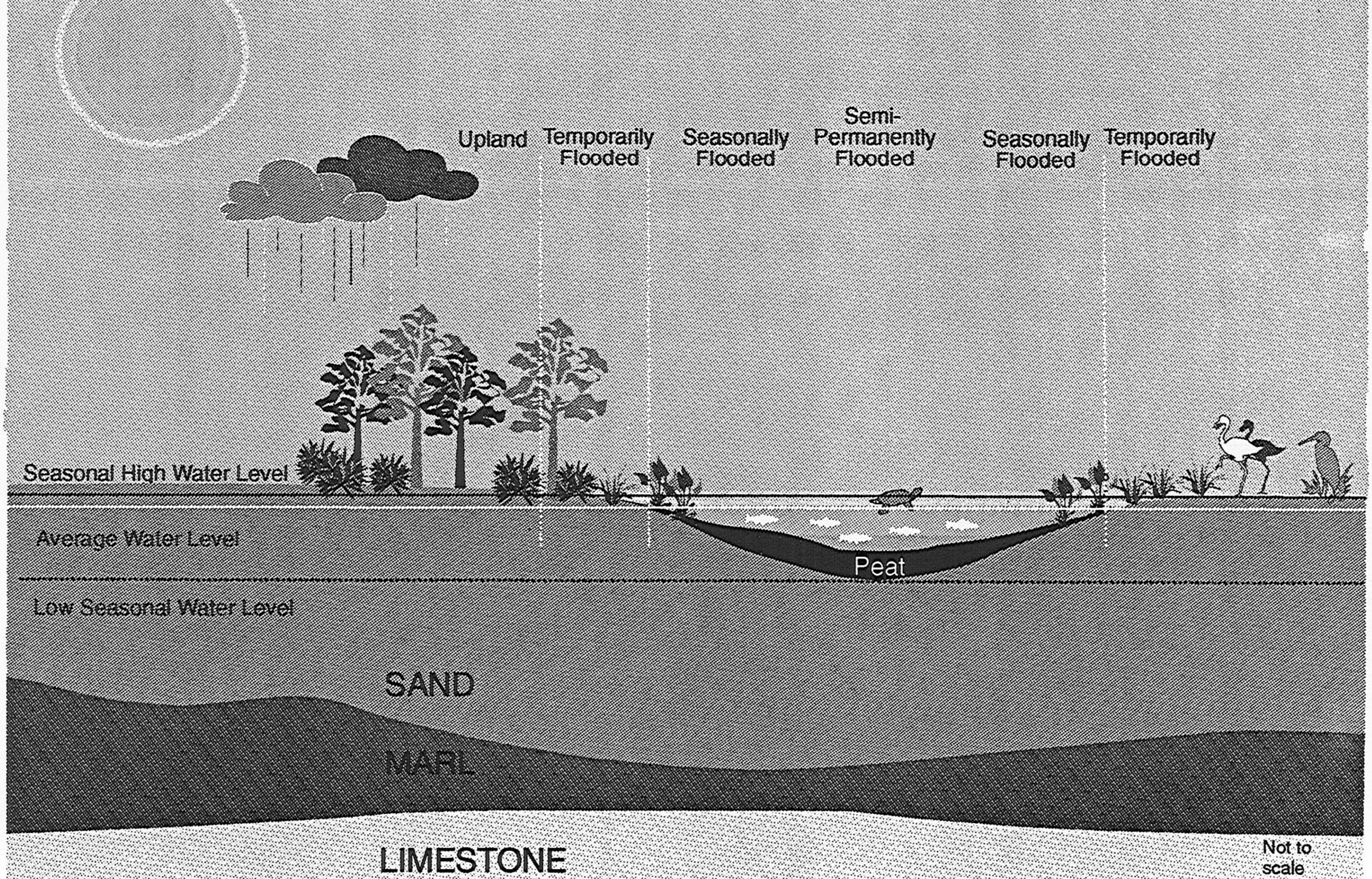


Figure 14. *Typical Cross Section of Marsh/Woody Wetland impacted by Groundwater Withdrawals*

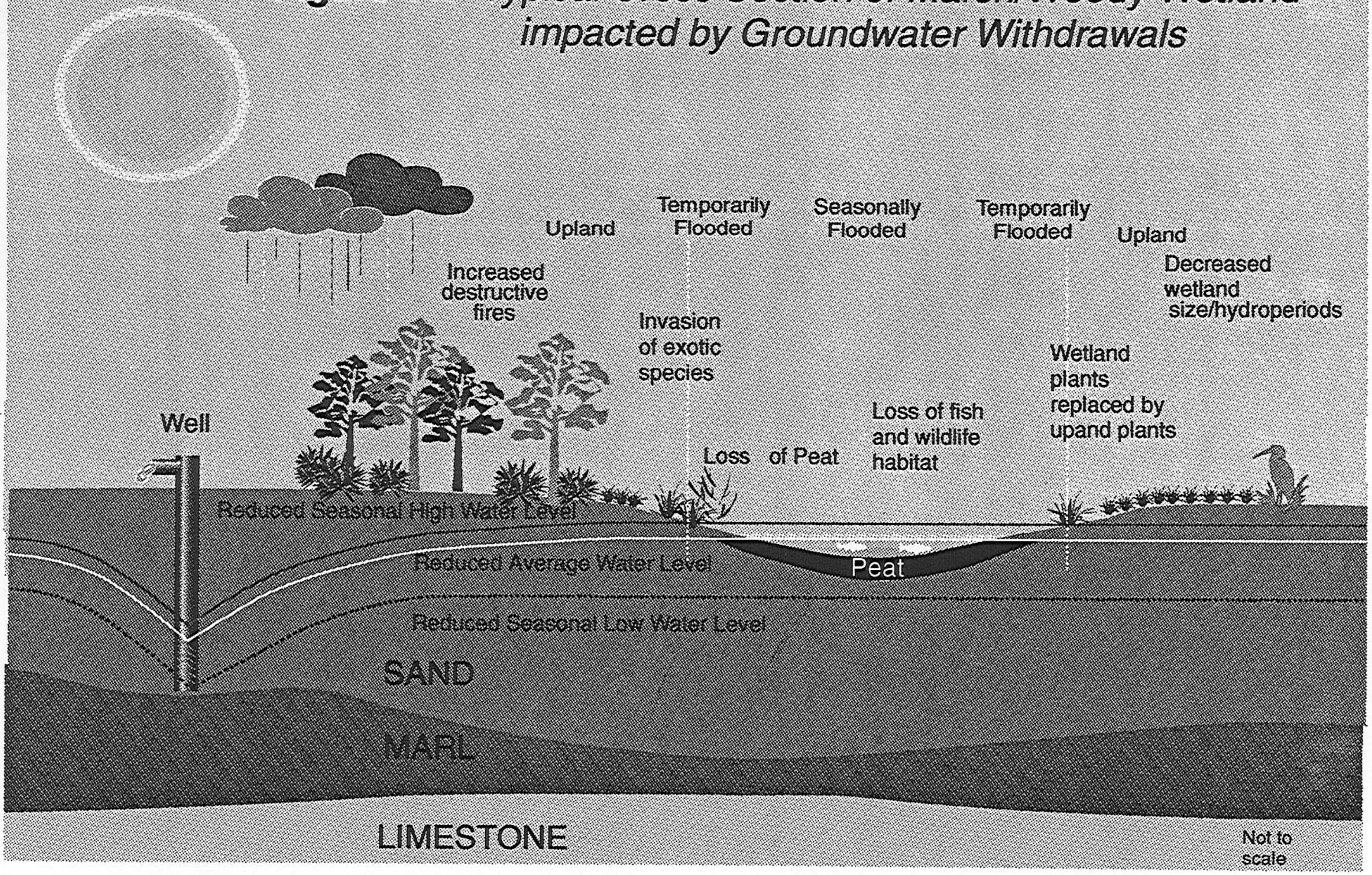


Figure 15a. *Marsh wetland on wet season (8-16-95) in Jensen Beach, standing water.*

Figure 15b. *Marsh wetland on dry season (3-23-96) in Jensen Beach, no standing water.*

1.3 Utilization and quality of water

1.3.1 Changes in water demand

1.3.1.1 Changes in the population

Martin County is composed of many small towns and cities. One of the largest and most important urban area is the city of Stuart. The county has two main wellfields, the North and the Port Salerno wellfields. The water pumped at the North wellfiled near the study site is treated directly by the water treatment plant and is used mainly to supply the Jensen Beach peninsula. Data concerning the population growth are presented in Table 1.

Year	Population of the North Martin County area
1984	8,290
1985	8,872
1986	9,375
1987	9,933
1988	11,650
1989	12,925
1990	13,522
1991	13,965
1992	14,422
1993	15,105
1994	16,168
1995	16,959

Table 1. *Population of the North Martin County area from 1984 to 1995 (SFWMD, 1996).*

The population has grown quickly. This can be explained by the fact that a lot of people who live in the northern areas of the United States, spend 6 months of winter (dry season) to benefit from the warm sunny days in South Florida and leave the area during the hot, humid rainy season.

In order to better understand better shifts in population within the study area, the District did projections over a nine year period.

These projections are presented in Table 2.

Year	Population projection of the North Martin County area
1996	17,866
1997	18,965
1998	20,064
1999	21,161
2000	22,262
2001	23,361
2002	24,462
2003	25,563
2004	26,660

Table 2. *Population projection of the North Martin County area over a nine year period (SFWMD, 1996).*

✎ *1.3.1.2 Water consumption evolution*

The growth of the water consumption is presented in Table 3.

Year	Total yearly consumption for the North zone (Mm^3)	Daily mean (m^3/d)	Daily maximum (m^3/d)
1984	1.567	4,314.9	6,699.5
1985	1.692	4,617.7	7,570.0
1986	2.225	5,564.0	8,251.3
1987	2.555	7,040.1	8,705.5
1988	2.758	7,532.1	9,507.9
1989	2.961	8,099.9	10,446.6
1990	3.084	8,440.5	12,149.8
1991	2.631	7,191.5	11,127.9
1992	3.054	8,3270	11,279.3

Table 3. *Water consumption in the North Martin county area from 1984 to 1992 (SFWMD, 1992).*

A direct correlation between population and pumpage from the North Martin County is evident. Unlike the population estimates, an annual projection of the future water consumption is not available. The only evaluation available was for the year 2010 and is based on an increase of the total consumption of 84% relative to the 1990 consumption (5.67 Mm^3 total during the year).

1.3.2 Evolution of the facilities

✎ *1.3.2.1 Rainfall*

An estimate of the annual rainfall for various drought frequencies up to a 1 to 100 year return frequency was done by the District in 1987 for Martin County.

The results shown in Table 4 were obtained.

Return frequency (years)	Annual rainfall in mm
1 in 10	1 041
1 in 20	991
1 in 50	940
1 in 100	914

Table 4. *Drought frequencies for Martin County area (SFWMD, 1987).*

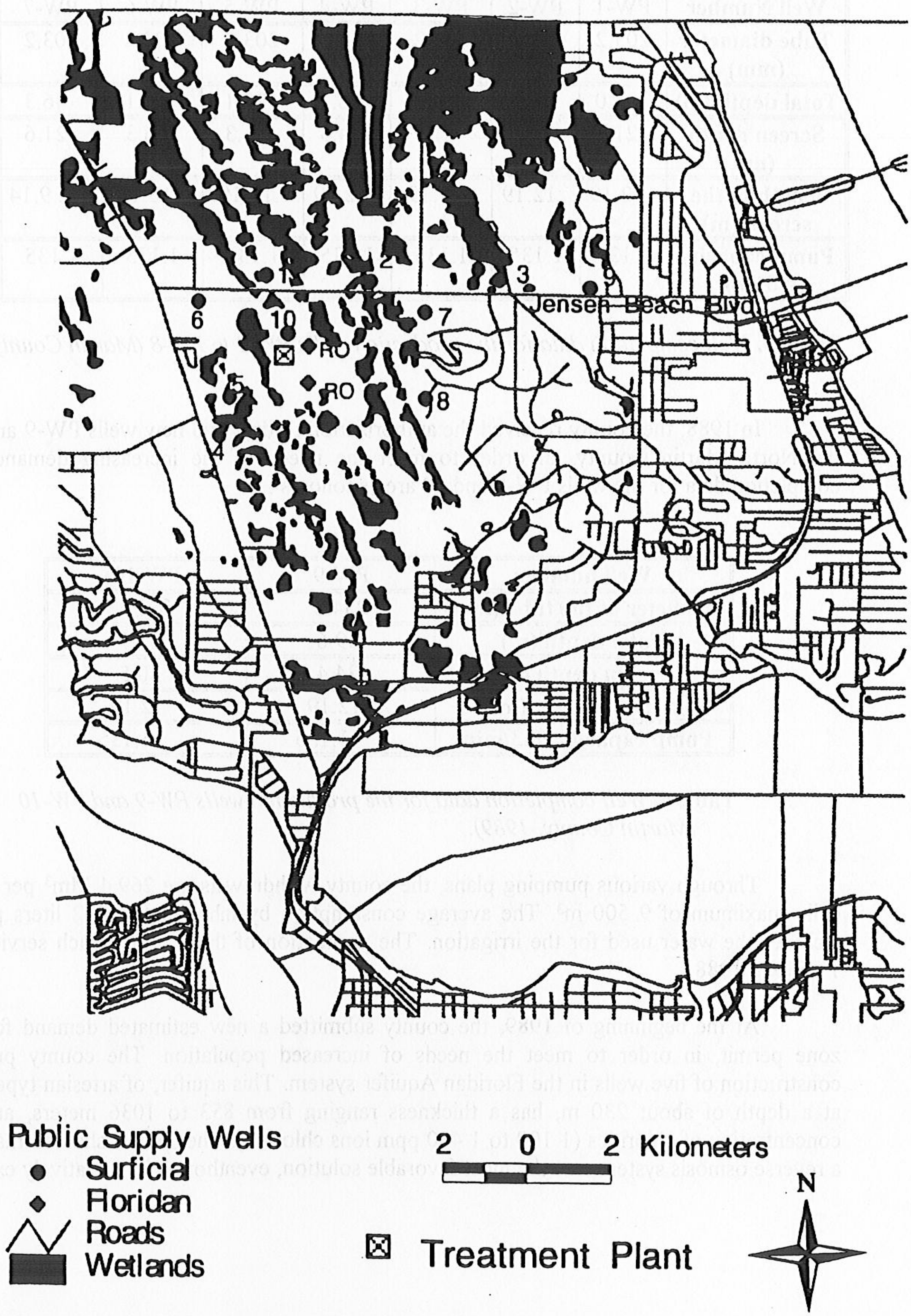
The average rainfall is 1448 mm per year. In 1995, the rainfall recorded at the Stuart weather station was 1699.5 mm.

✎ *1.3.2.2 Pumping permit history*

The District is responsible for the issuing of water supply pumping permits for South Florida. Criteria developed to protect the water resources and the environment are applied to all permits issued by the District and an evaluation is made to ensure that those withdrawals do not generate impacts on the environment. In 1982, the county received the authorization to drill eight production wells in the Jensen Beach area, PW-1 to 8 (Figure 16). The wells PW-1, 2, 4, 6, 7, 8 were constructed and put in service in 1982. Then in 1983, wells PW-3 and PW-5 were drilled and put on line.

The water treatment plant located in Northern Martin County was constructed in 1983. The treatment process consists of the classic stages of aeration, lime softening, filtration and chlorination. The capacity of the water treatment plant is 9 462 Mm^3/d. A storage reservoir with a capacity of 1 892 m^3 is also available on the site.

Figure 16. *Location of the Martin County public water supply wells.*



The well completion data are presented in Table 5.

Well Number	PW-1	PW-2	PW-3	PW-4	PW-5	PW-6	PW-7	PW-8
Tube diameter (mm)	203.2	203.2	203.2	203.2	203.2	203.2	203.2	203.2
Total depth (m)	35.0	35.0	35.0	38.1	38.1	38.1	46.3	38.1
Screen depth (m)	21.3	21.3	21.3	24.3	21.3	24.3	21.6	21.3
length of the screen (m)	12.19	12.19	12.19	12.19	12.19	12.19	3 & 9.14	6.1& 6.1
Pump capacity (m3/min)	1.135	1.135	1.135	1.135	1.135	1.135	1.135	1.135

Table 5. *Well completion data for the production wells PW-1 to PW-8 (Martin County, 1983).*

In 1988, the county received the authorization to drill two new wells PW-9 and PW-10 in the North Martin County, in order to meet the needs of the increasing demand. The well completion data for the wells PW-9 and 10 are as follows :

Well number	PW-9	PW-10
Diameter of the tube (mm)	254.0	254.0
Total depth (m)	39.6	42.7
Screen depth (m)	24.4	30.5
Screen length (m)	12.19	12.19
Pump capacity (m3/min)	1.135	1.135

Table 6. *Well completion data for the production wells PW-9 and PW-10 (Martin County, 1989).*

Through various pumping plans, the county withdrawals are 269.4 Mm^3 per year with a daily maximum of 9 500 m^3. The average consumption by inhabitant is 632 liters per day and includes the water used for the irrigation. The population of the Jensen Beach service area was 11 650 in 1988.

At the beginning of 1989, the county submitted a new estimated demand for the north zone permit, in order to meet the needs of increased population. The county proposed the construction of five wells in the Floridan Aquifer system. This aquifer, of artesian type, is situated at a depth of about 230 m, has a thickness ranging from 853 to 1036 meters, and has high concentration of chlorides (1 100 to 1 400 ppm ions chloride). The studies showed that the use of a reverse osmosis system was the most favorable solution, eventhough it is relatively expensive.

Numerous studies done by the District have demonstrated the impacts in wetlands that result from withdrawals in the surficial aquifer. The need for new water facilities therefore led the District to agree for the construction of two deep wells R0-1 and R0-2 (Figure 16). The District modified the pumping permit in 1991 to meet the increase of the demand for water and to decrease withdrawals from the surficial aquifer. The two wells have a pump capacity 1.135 m^3/min, and a total depth of 393 meters.

The new treatment unit and the two deep wells were put in service in August 1993. The county had authorization to withdrawing a maximum 6 434.5 m^3/d from the deep aquifer and must decrease the withdrawals from the surficial aquifer. The quantities of water pumped between January 1993 and June 1994 appear on Appendix 2. They indicate a reduction of the quantity of water pumped from the surficial aquifer.

During 1994, the county submitted a modification of the North zone permit, proposing the construction of six new pumping wells in the surficial aquifer. This was made to alleviate the pumping of the wells already present and also to supply the Port Salerno area which has some important water supply problems. For the moment, the permit has not been modified and the study is still in progress. The county has hired consultants to demonstrate that their proposition would meet the District criteria. The latest proposal from the county corresponds to daily withdrawals in the north zone of 18 925 m^3 of which 3 974 m^3 is sent after treatment toward the area of Salerno Harbor (Appendix 3).

The Port Salerno area hasexperienced water resources problems, including impacts on the environment, notably in wetlands areas. Now that the water needs for this zone have increased at least as quickly as in the north zone, it is necessary for the county to find new water resources. Currently, the county is authorized to pump the capacities as shown in table 7 :

Well name	Maximum number of pumping hours allowed by month	Maximum of water pumped by month (m^3)
PW-1	744	50,685
PW-2	96	6,540
PW-3	96	6,540
PW-4	744	50,685
PW-5	744	50,685
PW-6	744	50,685
PW-7	96	6,540
PW-8	744	50,685
PW-9	168	11,445
PW-10	744	50,685
Total	**4 920**	**335,175**

Table 7. *Maximum day pumpage that the county can currently withdraw from the North Martin County wellfield.*

The daily withdrawal authorized by the District corresponds therefore to 11,172 m^3.

1.3.3 Water quality data

✎ *1.3.3.1 Surficial Aquifer*

Water quality standards are set by the United States Environmental Protection Agency and each state is responsible for application of the standards. Selected analyses of water from the production wells are done every month by the staff of the water treatment plant. The values obtained from September 1995 to April 1996 were presented as representative of the water quality for the study area. The seasonal values are very similar and have been averaged to characterize water quality. The following results were obtained :

- Alkalinity

This parameter characterizes the capacity of water to neutralize an acid. Alkalinity has little influence on health, but plays an important role during treatment. If alkalinity is too weak, coagulation will be incomplete, especially if this process uses aluminum sulfate. The average value obtained is 237 mg/l of $CaCO_3$.

- Total Hardness

This parameter is a measure of the concentration of calcium and magnesium salts in water. The average value for the concentration of calcium is 235 mg/l. This value exceeds the recommended concentration of 25 mg/l and can be explained by the fact that the surficial aquifer is composed of a significant amount of redeposited calcium carbonate on the sand grains. In addition there are areas with interbedded limestone at depth. The average value obtained for magnesium is 10 mg/l. A concentration less than 50 mg/l is considered minimal. Total hardness is therefore 245 mg/l of $CaCO_3$. This value is relatively high and requires additional treatment. Water distributed to the public in United States should have a hardness of 75 to 150 mg/l of $CaCO_3$.

- pH

This value measures the acidity or basicity of the solution. The average value of raw water is 7.2 which corresponds to a neutral solution. The water distributed to the public must have a pH between 6.5 and 8.5.

- Color

Color is caused by mineral elements and organic particles that leach from soil or the vegetation. Color in the water is a problem for the consumer in the sense that they will have some reluctance to drink tinted water. Concerning health, color is not a dangerous feature. The average value obtained for the raw water is 27 CU. A value less than 15 CU passes generally unobserved. The obtained value corresponds therefore to a light coloration and requires treatment.

- Chlorides ions

Water containing elevated concentration of chlorides can be corrosive and, combined with sodium, has a salty taste. A concentration greater than 750 mg/l can damage plants and is harmful to cattle. The concentration recommended for drinking water is 250 mg/ l. The average value obtained for water in the surficial aquifer is 42 mg/l, which is well within the acceptable range.

• Iron

The iron found in water results from dissolution of iron that is naturally present in the soil. It has no harmful effect on health. The imposed limit is justified by the fact that high levels of iron in the water cause some aesthetic and taste problems. In addition, iron is a food for some bacteria such as Gallionella. A concentration greater than 0.3 mg/l must be treated. The average value for water from the surficial aquifer is 0.5 mg/l, which exceeds the standard.

• Turbidity

Turbidity is caused by the presence of suspended particles in the water. Turbidity must be treated because it presents some health hazards, gives water an unpleasant appearance, and causes problems in the course of the treatment. Suspended particles disturb chlorination because the bacteria adhere to the particles and escape the treatment process. Turbidity levels above 1 NTU require treatment. The average value obtained is 0.875 NTU.

Other parameters have been analyzed also, such as sulfate, sodium, potassium and arsenic. These analyses are not done regularly because the county considers the concentration of these ions to be fairly constant.

✎ *1.3.3.2 Floridan Aquifer system*

Water quality of the Floridan Aquifer System is poor because the concentration of chloride is very high. It requires desalinization which is done by reverse osmosis. The chloride concentration of the water from the Floridan wells R0-1 and R0-2 is generally around 1100 mg/l, well above the drinking water standard of 250 mg/l.

2. DETERMINATION OF THE AQUIFER PARAMETERS

The determination of the aquifer parameters is divided in two main phases :

- data collection, including two pumping tests,
- analyses of the pumping test results using various methods.

The entire wellfield area has already been impacted by drainage ditches and the existing surface water management system. The purpose of this study is to evaluate the impacts of groundwater withdrawals during the constant rate pumping test on wetlands.

2.1 Data collection

The pumping test was conducted in one production well PW-7. The idea is to extrapolate the results to the whole study area, based on the assumption that the aquifer is homogeneous.

2.1.1. Site description

The site where the pumping tests were conducted is located in the North Martin County wellfield. Production well PW-7 was put in service in 1982, and is situated about 180 meters south of Jensen Beach Boulevard and 700 meters northeast of the water treatment plant. A small wetland, approximately 8,000 square meters in area, is located approximately ten meters southwest of the well.

PW-7 is completed in the surficial aquifer. The screen is in two sections, one from 21.7 to 24.7 m and the other one from 35.7 to 44.8 m below land surface. A general description of the well is presented in the section 1.3.2.2 and the well completion report is presented in Appendix 4.

The Surficial Aquifer system is not homogeneous, as seen in the well completion report. The geological cross section prepared by Miller in 1980 (Figure 5) shows the presence of a low permeability layer, situated about 1.5 meters below ground level, which corresponds to what is termed the "hardpan". This same hardpan is present under the study area wetland and is formed by the following phenomenon : percolating rainwater, rich in mineral elements, reaches the water table of the Surficial Aquifer system. The minerals precipitate around sand grains and form a dense, less permeable lense called the hardpan. Thousand of years of percolating rainwater through the unsaturated zone of the Surficial Aquifer system has formed small scale layers within the aquifer that have a lower hydraulic conductivity. The presence of the hardpan helps to explain why the surface water elevation in the wetland is generally higher than the groundwater elevation around the wetland. However, the continuity of this horizon has not been established.

A second low permeability horizon is located about 10 meters below the ground surface. It consists of a 3 to 6 meters thick unit of tan and gray fine to very fine sand, with some traces of shells and clay.

Following the completion of well PW-7, the drilling company conducted a pumping test. Results of the specific capacity test indicated a value of 0.08442 m^3 per minute per meter of water table drawdown. The well is equipped with a control valve, an in-line flowmeter, and a well access portal. The pump may be operated manually, independent of the plant controls or automatically via an electronic signal. Another pumping test was conducted during the period October 5 through October 9, 1989 by the consulting engineering company James M. Montgomery, under contract with the District (J. M. M. 1989). At the time of this test, 15 observation wells were constructed around PW-7. At the time of this study, only 13 of these wells were still in place. Their characteristics are described in Table 8.

Well name	Cased depth (m)	Total depth (m)	Radial distance from PW-7	Elevation of the Top of casing (m, NGVD)
PZ 136 H	0.84	1.14	136	5.84
PZ 136 I	13.70	15.20	136	5.77
PZ 136 D	32.00	33.50	136	5.77
PZ 87 H	0.84	1.14	87	5.61
PZ 87 I	13.70	15.20	87	5.68
PZ 87 D	32.00	33.50	87	5.63
PZ 36 H	1.22	1.52	36	5.70
PZ 36 I	13.70	15.20	36	5.66
PZ 36 D	32.00	33.50	36	5.68
PZ 106 H	1.00	1.33	106	0.28
PZ 106 H'	1.93	2.23	106	1.22
PZ 206 H	0.23	0.53	206	5.11
PZ 206 H'	1.32	1.62	206	5.08

Table 8. *Well completion data for observation wells situated on the pumping site (James M. Montgomery, 1989).*

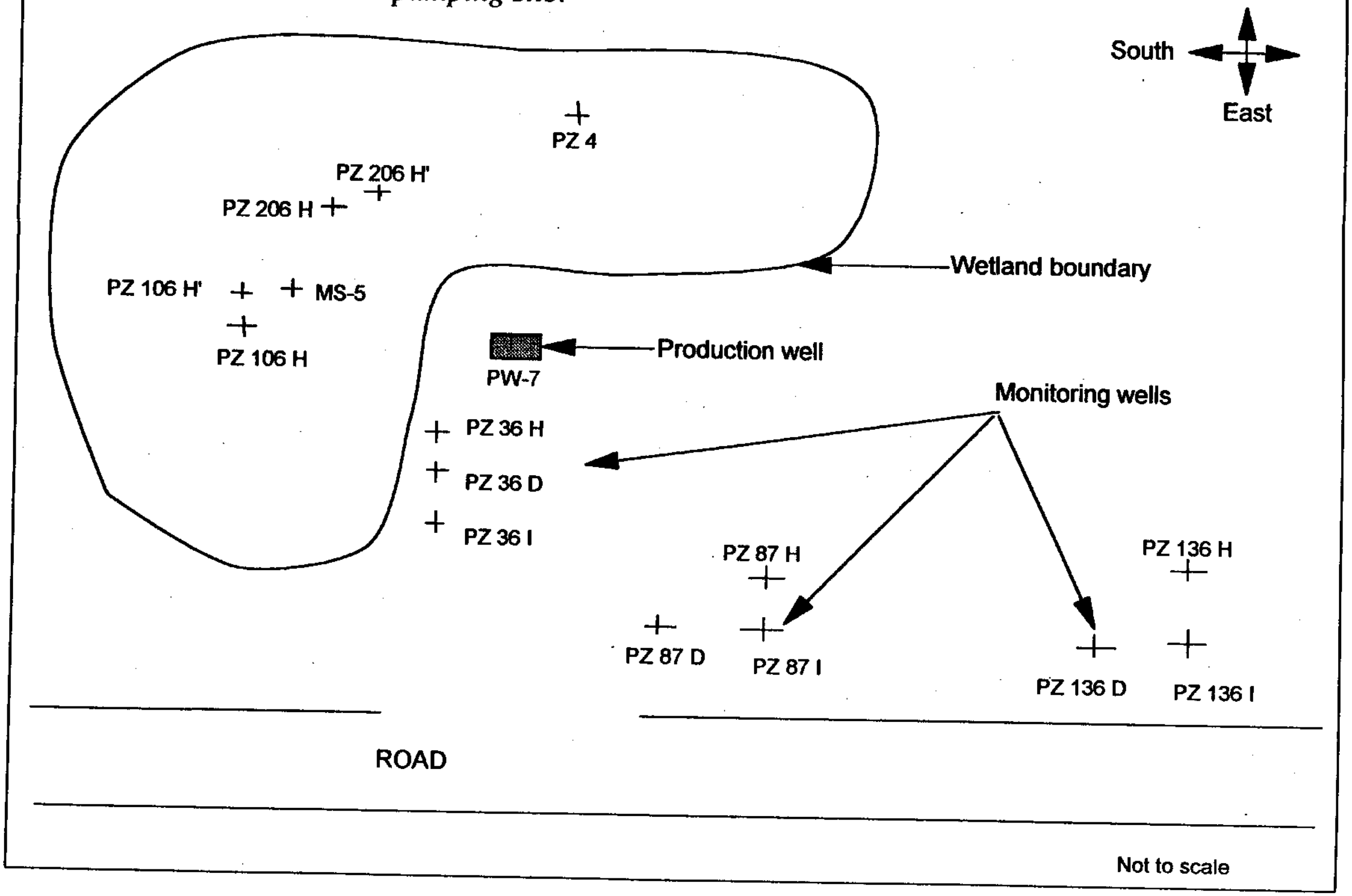
The observation well diameter is 50.8 mm. A site plan of the monitoring site is presented in the Figure 17. The wells were located in three main groups to assess the heterogeneity of the aquifer. Indeed, when an aquifer is composed of sandy deposits interbedded with low hydraulic conductivity horizons, it is recommended that an observation well should be situated in each of the sandy horizons. The surficial aquifer in this study area contains two horizons with low permeability. The following naming convention is used in this investigation :

- The letter D indicates that the wells are situated in the deep portion of the surficial aquifer system.

- The letter I indicates that the wells are situated in the intermediate portion of the surficial aquifer system.

- The H and H' letters correspond to the shallow wells. For wells situated in the wetlands, wells marked with the H are located above the hardpan, whereas those marked with the H' are located below the hardpan.

Figure 17. *Location of the monitoring wells around the pumping site.*



2.1.2 Data collection and objectives

The pumping test consists of determining :

- The pumping well efficiency.
- The aquifer parameters.

The pumping tests were conducted by pumping water from a well at a certain rate and then recording water level fluctuations in the pumping well and in several observation wells. Two types of pumping tests were conducted during this investigation :

- Constant rate
- Step drawdown.

The first test consisted of pumping at a constant rate for a period greater than 48 hours. The second test consisted of pumping at increasing rates for relatively short periods of time. The entire test lasted generally no more than one day.

The aquifer parameters that are determined through pumping tests are more accurate than those obtained by analyzing in situ soil samples. This can be explained by the fact that it is difficult to get undisturbed samples that are really representative of the natural state of the soil. When the pumping tests are conducted in situ in the aquifer, they better reflect natural conditions. Under some conditions the pumping tests also allow investigators to determine (Driscoll, 1986) :

- drawdowns that may be expected from long-term pumping at different discharge rates.
- existence of impervious boundaries.
- existence of recharge sources which may not be apparent otherwise.

During the pumping test the cone of depression expands at a rate that depends on :

- Time since start of pumping,
- Aquifer characteristics,
- Recharge.

The length of a pumping test therefore depends upon both the test purpose and the hydraulic properties of the aquifer. At the beginning of the test, the cone expanded quickly as aquifer storage in the immediate vicinity of the well is depleted. As pumping continued, horizontal expansion of the cone slows as larger and larger volumes of water become available (Lohman, 1979). Quantitatively this time and distance expansion may be stated as :

$$S = \alpha \log(\frac{t}{r^2})$$

where S is the drawdown at distance r and time t after pumping begins.

Steady-state conditions will not occur until the cone of depression has expanded to the point where recharge to the cone equals the discharge of the well. In some wells, equilibrium may be reached within a few hours whereas others may require days, weeks, or longer. Some may never reach steady state conditions. It may not be necessary to continue the test until steady-state conditions are reached, such as nonsteady-state methods are also available for analysis (Walton, 1988). The essential measurements taken during a pumping test are :

- time,
- depth to water level in the observation and pumping wells,
- discharge rate.

Measurements of the start time, stop time, and the pumping interval must be made with reasonable accuracy (0.1 min.). Any irregular events, such as pump failure and restart that occur during the test should be noted.

Water levels decline or recover most rapidly following a change in pumping rate. For this reason, frequent measurements were required during the first few minutes to tens of minutes from the beginning of each step in a step-drawdown test, and after start and stop of pumping in a constant rate test. The discharge rate had to be carefully regulated during the pumping test. This was achieved by placing flow-measuring devices in line. Discharge rate accuracy should be within 2%. A commonly used discharge-measuring device, the propeller meter, was placed in a straight section of the discharge pipe. The meter averages flow by counting propeller revolutions per time period. This rate is proportional to velocity. It was necessary to assure that the pumped water is discharged at a minimal distance of 100 m from the well with a decreasing gradient so that the water didn't come back to the pumping site via groundwater seepage. This was accomplished by pumping the discharge water directly to the water treatment plant.

2.1.3 Measurement techniques

✎ *2.1.3.1 Measuring water level*

Measurements of depth to water-level during a pumping test should be accurate within 1.5 cm in observation wells and 3 cm in the pumping well. There were various measuring devices and methods used in this study as follows :

- a manual method using a tape with markings in tenths and hundredths of feet. An electric wire was embedded in the tape and, when contacted with water in the wells, it completed an electrical circuit to activate a buzzer and a red light.

- continuous water-level recording devices consisting of transducers connected to a recorder.

The pressure transducers used during the study are submersible, models In-Situ PXD-260 (Appendix 5). The recorders are called dataloggers, models In-Situ Hermit 2000 (Appendix 6) and In-Situ Hermit 1000. The pressure transducers are connected to the dataloggers. All programming occurs through the dataloggers. Although the transducer can measure pressure, flow, water-level, only the water level was recorded during this study.

Each transducer has specific calibration parameters that are input into the datalogger prior to the start of recording data. These parameters are also input as coefficients for the quadratic equation that is used to convert the transducer output to meaningful units. The values for these parameters are found on a data tag attached to the cable reel. There are three parameters : linearity, scale factor and offset. On the data tag, there is also the maximum range of pressure that the transducer can tolerate without damage. The datalogger computes pressure readings as follows :

$$P = LX^2 + \frac{SX}{16} + O$$

With P: pressure in PSI (1 PSI = 0.703 m of water)
X: transducer value (0-16 milli Amperes)
L: linearity
S: scale factor in PSI full scale
O: offset in PSI

The transducer pressure is converted to a head value using the following formula :

$$H = P \times U \times SG$$

With H: head value
P: pressure in PSI (1 PSI = 0,703 m of water)
U: conversion units 2.30667 feet of water/ PSI
or 0.703 meters of water/ PSI
SG: specific gravity (fluid density/water density)

The warm-up delay of the datalogger is also an input parameters and has a value of 50 milliseconds (ms).

Two water level modes exist within the datalogger, top of casing (TOC) or Surface level. The surface mode is used to monitor surface water situations such as streams and lakes. The top of casing mode is used when monitoring groundwater, where readings referenced to the top of the well casing are required. During the course of this study, only the TOC mode was used. With this mode, decreasing water levels correspond to increasing top of casing readings.

2.1.3.2 Collecting water quality data

Some measurements of water quality were also made. They consisted of recording the temperature and the conductivity of the water, using the following probes In-Situ models CTS-200. These probes must be connected to a recording system. Each probe is connected to two channels of the datalogger. The temperature values range from 0 to 40° Celsius and conductivity range from 0 to 20 000 microSiemens/cm (Appendix 7). The probes also have some characteristic parameters that the datalogger uses to transform the electric signal transmitted by the probe, into a value of temperature or of conductivity. The two probes used during the course of this study had the parameters shown in Table 9.

Linearity	0.0
Scale factor	40 for temperature 20 000 for conductivity
Offset	0.0

Table 9. *Characteristic parameters of the probes In-Situ PXD-260.*

The warm-up delay for the probes is 100 ms for temperature and 15 000 ms for conductivity.

2.1.4 Measurements

The schedule of measurement was as follows :

- The equipment used to record water quality and level in the wells was installed on November 7 and 8, 1995.
- Prior to any pumping test, background water level and water quality data were collected. This information was used to evaluate the equipment and to monitor background water levels.
- A constant rate pumping test has been conducted during the period from January 29 through February 2, 1996.
- A step-drawdown test was conducted on February 8, 1996.

✧ 2.1.4.1 *Observation period preceding the pumping test*

Prior to the pumping test, it is crucial to understand hydrologic influences in the area that may affect its results, in order to differentiate the effects of pumping from the natural behavior of the aquifer. All the information that relates to fluctuations of water level must be noted, including :

- discharge rate,
- water level in all the observation wells around the pumping well,
- pumping schedule,
- influence of pumping near the pumping site,
- localized effects of the hydraulic variability, and
- daily variations of the atmospheric pressure.

The duration of the test is generally set by the supervisor of the pumping test. Several site visits were made before setting up the equipment to get familiar with the pumping site and, collect data such as the total depth of the wells and topographic measurements of the wells. These visits also allowed us to formulate the distribution of the transducers and the probes. Observation wells were pumped using a portable pump to clean the well screen. The wells have not been used since 1989 (last pumping test) and the sediments were obstructing the slots of the screen.

At the time the equipment was installed, water was present in all wells and there was water standing in the wetland near the pumping well PW-7. The equipment available in November included nine water level transducers, two water quality probes and two dataloggers (In-Situ Hermit 2 000 with 8 channels each). Connections were made as shown in Table 10 and 11.

- **Datalogger 1**

Channel number	Data	Well name	Elevation of the top of casing MSL (m)	Transducer or probe depth from the top of casing (m)
1	Water level	PZ 87 H	5.61	1.50
2	Water level	PZ 36 I	5.67	10.00
3	Water level	PZ 106 H'	5.19	3.05
4	Water level	PZ 206 H'	5.08	2.24
5	Water level	MS-5	4.33	1.50
6	Water level	PZ 87 D	5.63	10.00

Table 10. *Details of the connections to the datalogger 1 In-Situ Hermit 2000.*

• **Datalogger 2**

Channel number	Data	Well name	Elevation of the top of casing MSL (m)	Transducer or probe depth from the top of casing (m)
1	Level of water	PZ 136 I	5.77	10.8
2	Level of water	PZ 136 H	5.84	1.61
3	Level of water	PZ 87 I	5.68	6.46
4	Temperature	PZ 206 H'	5.08	2.24
5	Conductivity	PZ 206 H'	5.08	2.24
6	Temperature	PZ 106 H'	5.19	3.05
7	Conductivity	PZ 106 H'	5.19	3.05

Table 11. *Details of the connection to the datalogger 2 In-Situ Hermit 2000.*

The recording frequency was set to 15 minutes prior and after the pumping test. This allowed sufficient accuracy and still provided the ability to measure the effect of a brief rain event. This frequency also allowed ample time between downloading of data from the dataloggers. The duration of logging capabilities was about 50 days. In December 7, 1995, a new probe, model In-Situ Troll 4 000, was installed in well PZ 136D. This probe recorded the water level and the temperature (Appendix 8). It was not connected to a datalogger. The programming and data transfer were done by connecting a computer to the probe. The storage capacity was 280 Kb and like the transducers the recording frequency was set to 15 minutes. During the observation period, data were collected every 15 days using the following procedures :

• Once on the site, the first task was to tape all the wells by hand. This required approximately 15 minutes and was used to verify the accuracy of the values provided by the dataloggers and to serve as a calibration control for the electronic data.

• Then, it was necessary to stop the test in progress in one of the dataloggers, connect the portable computer to the datalogger, and run the data transfer program. The time of transfer depended on the quantity of data stored. For a two-week period of observation, with 6 transducers connected, and a sampling frequency of 15 minutes, the time of transfer was around 45 minutes.

• Once the transfer was accomplished, the second datalogger was stopped and the data transfer program was run.

• When all the information recorded by the dataloggers was transferred and verified, the memory was cleaned and a new test was programmed for each datalogger. The start time of the test had to be chosen, as a function of the record frequency, to have a whole number of cycles.

• Before the two new tests began, all the water levels in the wells were taped manually and used as referenced values.

• All the data was then classified, compiled and represented graphically.

✎ *2.1.4.2 Constant rate test*

A constant rate pumping test was conducted during the dry season, from January 29 to February 2, 1996. This test consisted of 100 hours and 45 minutes of pumping and a recovery period of about 48 hours. The deep wells recovered their initial levels quicker than the intermediate ones. Most of the equipment was already on site because it had been deployed during the observation period. Some modifications in the installation of the transducers and probes were made prior to the pumping test.

These modifications were as follows :

• On January 17,1996, three new transducers were installed, using the connections shown in table 12.

Datalogger number/ Input channel	Well name	Elevation of the top of casing MSL (m)	Transducer depth from the top of casing (m)
1/ 7	PZ 106 H	3	1.74
1/ 8	PZ 206 H	1	1.12
2/ 8	PZ 36 D	9	15.27

Table 12. *Detail of the connection of the three new transducers installed on 01/17/96.*

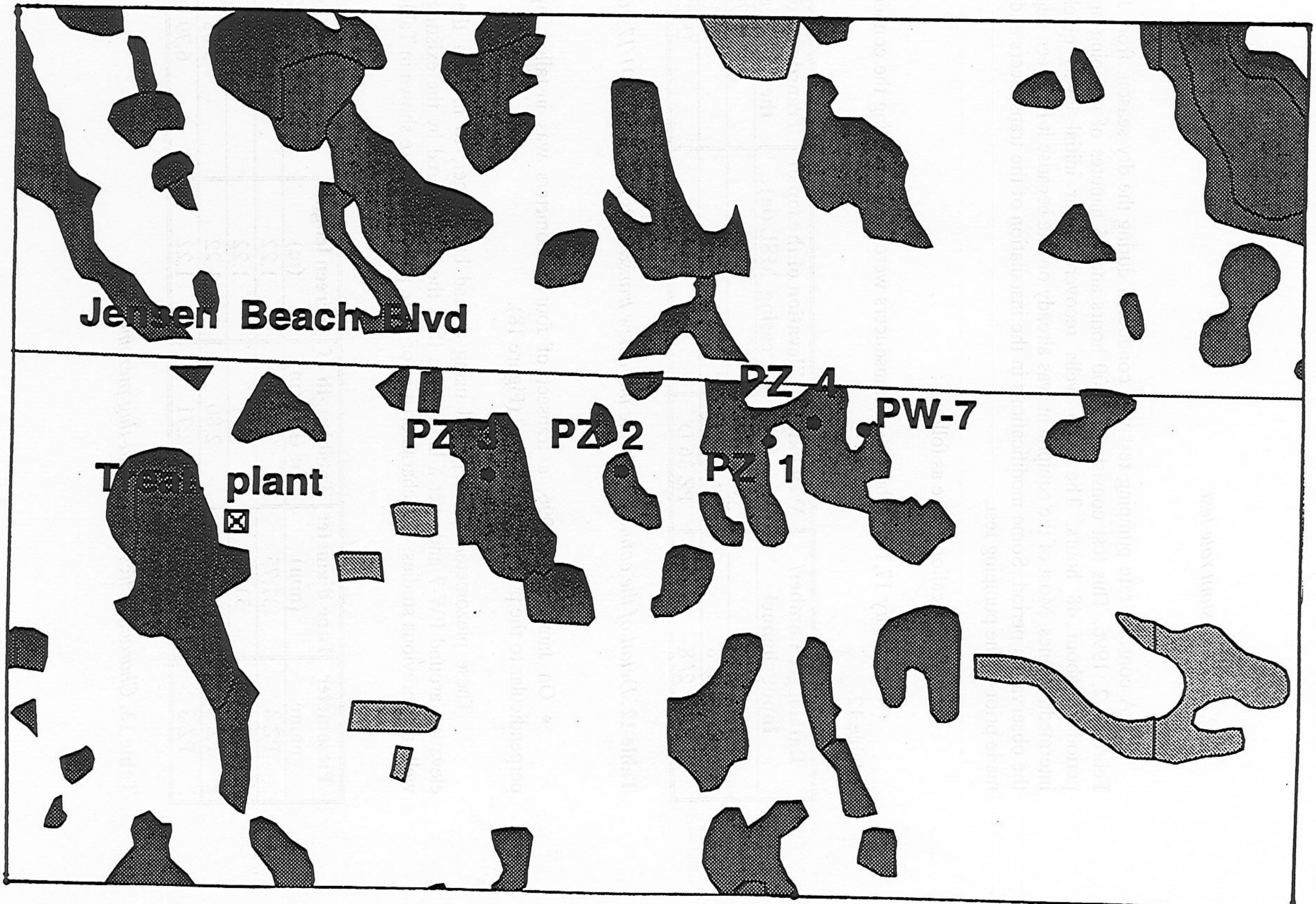
• On January 22 1996, a transect of four piezometers was installed, approximately perpendicular to the pumping well PW-7 (Figure 18).

These piezometers were placed using a sand hand-auger, to monitor the water table elevation around PW-7 and over a larger area than the one covered by the existing observation wells and previous studies. The characteristics of these piezometers are shown in Table 13.

Piezometer name	Tube diameter (mm)	Total length of the tube (m)	Screen length (m)	Elevation of the top of casing MSL (m)
PZ 4	31.75	2.80	1.22	6.18
PZ 1	31.75	2.71	1.22	5.20
PZ 2	31.75	2.80	1.22	5.85
PZ 3	31.75	2.71	1.22	6.79

Table 13. *Characteristics of the hand-dug piezometers.*

Figure 18. *Location of the piezometers installed in January 1996.*



Regional wells surrounding the study area are monitored by Martin County for water levels and chlorides. All regional wells were taped daily during the five days of the constant rate pumping test, to follow the water table fluctuations during the pumping test. Data from 23 regional wells located at 12 different sites were used in modeling the study area. The observation wells PZ 87 H and PZ 136 H were dry at the time of the test. The transducers placed in these wells were transferred to other wells. The average pumping rate was around 1.325 m^3/min during the test. The following data were collected :

- water level in all the observation wells around PW-7.
- water level in the pumping well.
- pumping rate of PW-7.
- water quality data in some observation wells.
- rainfall.
- atmospheric pressure.
- water level in the 23 regional observation wells.

Two piezometers were situated relatively far from the pumping well so they were not connected to the dataloggers near PW-7. Therefore, a datalogger In-Situ Hermit 1 000 with two channels was installed to monitor the water level in the piezometers PZ 1 and PZ 2. A datalogger In-Situ Hermit 2 000 with four channels was also installed at the time of the pumping test to accomodate the water quality probes and the sampling frequency of the water levels.

The connections were made as shown in Table 14-17.

- **Datalogger 1**, In-Situ Hermit 2000, with 8 channels :

Input number	Well name
1	PW-7
2	PZ 36 I
3	PZ 106 H'
4	PZ 206 H'
5	MS-5
6	PZ 87 D
7	PZ 106 H
8	PZ 4

Table 14. *Connection of the datalogger during 1 In-Situ Hermit 2000 during the pumping test*.

• **Datalogger 2**, In-Situ Hermit 2000, 8 channels :

Input number	Well name
1	PZ 136 I
2	PZ 136 D
3	PZ 87 I
4	PZ 36 D

Table 15. *Connection of the datalogger 2 In-Situ Hermit 2000 during the pumping test*.

• **Datalogger 3,** In-Situ Hermit 2 000 4 channels :

Input number	Well name	Parameter measured
1	PZ 36 D	Temperature
2	PZ 36 I	Conductivity
3	PZ 106 H'	Temperature
4	PZ 106 H'	Conductivity

Table 16. *Connection of the datalogger 3 In-Situ Hermit 2000 during the pumping*

• **Datalogger 4**, In-Situ Hermit 1000 2 channels :

Input number	Well name
1	PZ 1
2	PZ 2

Table 17. *Connection of the datalogger In-Situ Hermit 1000 during the pumping test.*

In all the shallow wells the transducers were placed in the bottom of the well. The frequency of measurement for the water quality probes was set to 15 minutes. It was not set at a higher frequency for two reasons :

• the parameters of quality do not vary as much as the water level.

• the warm-up delay for the probes is longer than for the transducers.

Sensor warm up times, prior to data acquisition, vary from sensor to sensor. Warm up times for water quality are 1500 ms as compared to 50 ms for water levels. Thus the limiting sampling frequency is controlled by the longest warm up period. The frequencies of the datalogger recording the water level are shown in Table 18.

Log-cycle	Elapsed time	Time intervals
1	0- 20 s	0.5 s
2	20- 60 s	1 s
3	1 - 10 min	12 s
4	10 - 100 min	2 min
5	100 - 1000 min	5 min
6	> 1000 min	15 min

Table 18. *Sampling frequency of the dataloggers recording water levels during the pumping test.*

The calculation of pump rate is based on the number of gallons of water pumped from the well. This number is displayed on the in-line flowmeter placed in the well discharge line. Rainfall data was collected at the water treatment plant as a daily total. Atmospheric pressure was recorded directly at the study site using a portable barometer with a sampling frequency of 15 minutes.

During the pumping test the water levels in the wells were recorded manually for two reasons :

- to control the accuracy of the data provided by the dataloggers.
- to plot the drawdown versus the time to determine if the equilibrium was reached.

The water quality parameters were recorded by the In-Situ probes connected to datalogger 3, in the wells PZ 36D and PZ 106 H.'

2.1.4.2 Step-drawdown pumping test

On February 8, 1996 a step-drawdown pumping test was conducted. The equipment used during that test was the same used during the constant rate test except the In-Situ 1000 datalogger, which was recording the water level in the two piezometers PZ 1 and PZ 2, was not available. The test can be broken in four phases: Three phases of pumping and a phase of recovery.

- The first phase lasted 220 minutes with a pumping rate of 0.314 m^3/min
- The second phase lasted 220 minutes with a pumping rate of 0.727 m^3/min
- The last phase lasted 229 minutes with a pumping rate of 1.106 m^3/min
- The recovery lasted around 150 minutes.

The two dataloggers recording the water level were programmed with the same log cycles as the ones used during the constant rate test. Just before each pumping rate modification, the dataloggers were stopped and reprogrammed. The pumping rate was modified for to the next step and the dataloggers were restarted. The sampling frequency was very high during the initial portion of next phase. This assured data collection during the early phases of the step. As well occurred during the constant rate test, the sampling frequency for water quality data was set to 15 minutes. Atmospheric pressure was also recorded using a portable barometer each 15 minutes.

2.2 Results of the observation period

The results obtained with the dataloggers during the observation period were expressed as curves. These curves were later analyzed. Prior to creating graphics, all data were corrected to National Geodetic Vertical Datum of 1929. All data were further verified against the hand recorded water levels. Elapsed time was converted to dates and hours.

2.2.1 Treatment of the data

The files obtained from the dataloggers have the following information :

- the start date and time, the parameters input at the time of the programming (linearity, scale factor offset and the warm-up time).
- the first column contains the elapsed time.
- the remaining columns correspond to data recorded by the transducers or the water quality probes.

The correction of the data was made using Lotus 1-2-3 (Version 4.0). The data was then plotted utilizing the Freelance (Version 2.0) graphing package. The purpose of the plotting was to see if there were any correlations and to make some initial assumptions concerning the type of the aquifer (unconfined or confined). The formulas used are the following :

- For water level, it is first necessary to have all the water levels at the beginning of the test (t=0). It is necessary therefore to apply the following formula to every column.

Water level referenced to NGVD = Uncorrected water level at t – Uncorrected water level at t = 0

It is necessary to use the manually taped water levels before the beginning of each test and to apply the following formula :

Level referenced to the sea at t = level at t= 0 referenced to the sea – level corrected at t

The following formulas was used to calculate the date of the recording from the elapsed time :

Date corresponding to the elapsed time t = start date + (start time in hours / 24)+ (elapsed time t in min / 24 / 60)

The hour was obtained by adding 15 minutes to every previous value on a cycle of 24 hours. For the water quality data, only the elapsed time was manipulated. The conductivity and the temperature are given respectively in microSiemens per cm and in degrees Celsius.

Once corrected, all the data were then compiled in one file and plotted using Freelance. The hydrographs represent fluctuations of water level, conductivity and temperature as a function of time.

2.2.2 Results

♥ *2.2.2.1 Fluctuations of water levels*

Graphs of all data obtained during the observation period are presented on Appendix 9. The following observations are drawn about the graphed data :

- When the pump was turned on, a drawdown of the water table occur.
- For the wells completed in the same zone, the closer they was to the pumping well, the greater the drawdowns that occured.
- For wells situated at a same distance from the pumping well, the drawdown was greater in the deep wells than in the intermediate wells.
- During the observation period, there was a steady decline of the water table.
- Wells installed in the wetland also experienced a drawdown when the pump was turned on.
- Drawdowns associated with pumping were from 4 to 2.5 m for the deep wells and from 1.4 to 0.7 m for the intermediate wells.
- Water levels in the wells denoted by "H" were not recorded because the wells were dry at the time of the test.

✎ 2.2.2.2 *Water quality*

Graphs of the fluctuations of the water quality parameters versus the time are presented in Appendix 10. The results are not consistent. This can be explained by several factors :

- The wells PZ 106 H' and PZ 206 H' were almost dry. To give reliable results the probes must be under at least 60 cm of water. During the observation period, this condition was not met.

- The model of In-Situ probes used for this study is designed to measure variable conductivity from 0 to 20 000 μS/cm. The precision of measurement for conductivity is 2% of the whole range, or 400 μS/ cm. The conductivity of water in the wells is around 300 μS/cm, which is right at the sensitivity of the probe.

In conclusion, it can be inferred that the In-Situ probes were of limited use and did not adequately meet the needs of the study. The probes required a long warm-up time, and were not accurate for the range of the sampled parameter. For this reason, after the pumping tests, the probes were returned to the factory.

2.2.3 Interpretation of the results

Water level fluctuations in the observation wells can be interpreted as follows :

- The drawdown was greater in the wells near the pumping well because of the shape of the cone of depression.

- A progressive lowering of the water table occurred during the dry season because rainfall was almost equal to zero and there was not sufficient recharge to compensate for withdrawals that occur due to pumping or natural process (by evapotranspiration of the plants).

- The drawdown in the deep wells and in the intermediate wells situated at a same distance from the pumping well reflect the fact that the deep aquifer is partially confined or leaky.

- If the aquifer was confined but not leaky the drawdowns in the intermediate wells would not be similar to water levels in the deep wells (Reed, 1980).

- It appears that the aquifer system consists of two principal layers separated by a leaky aquitard or by areas of discontinuous hardpan.

Figure 19 is a representation of the Surficial Aquifer system in the Jensen Beach wetland area, when there is no withdrawal of groundwater. Figure 20 represents the influence of groundwater withdrawals in the Jensen Beach wetland area.

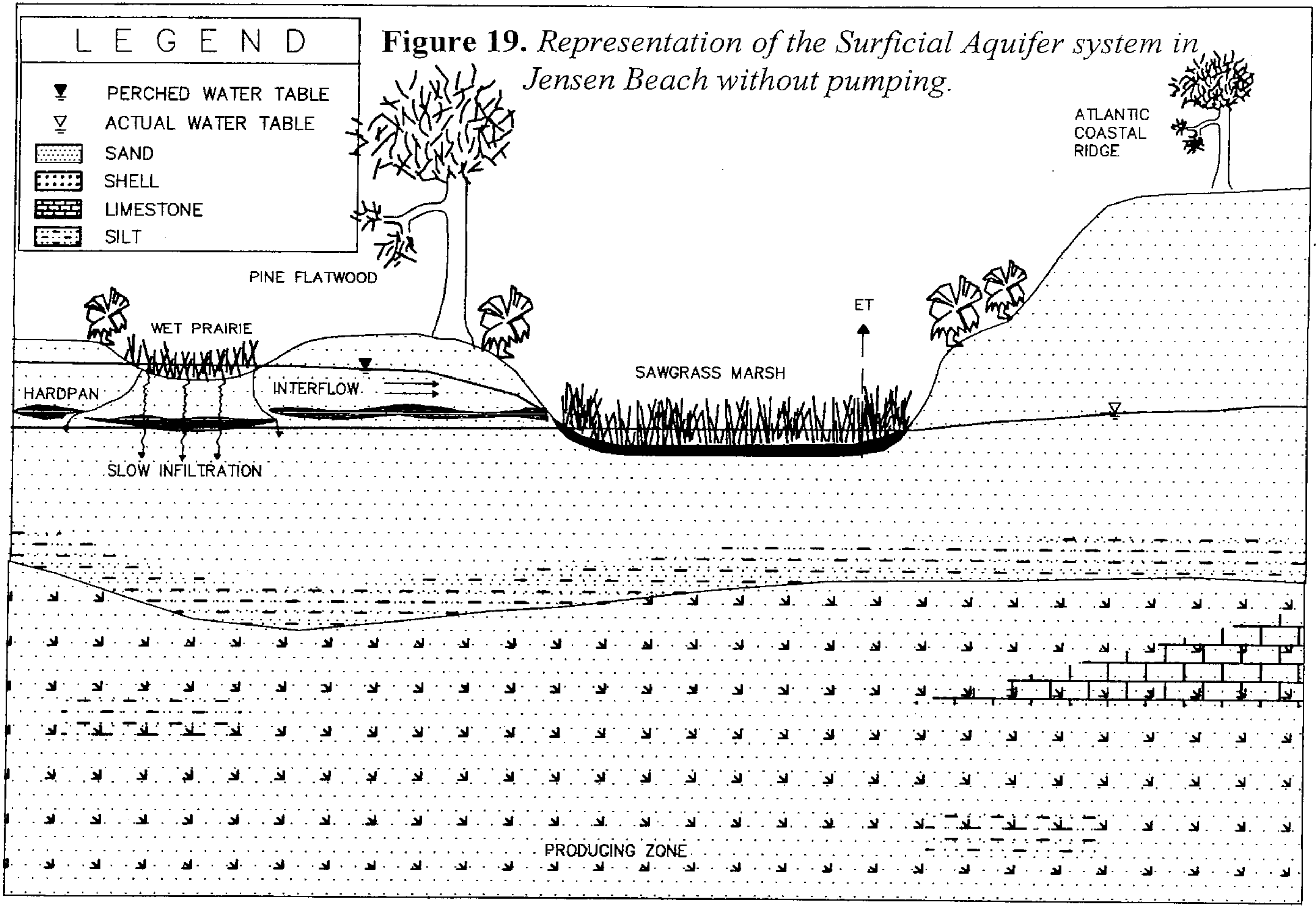


Figure 19. *Representation of the Surficial Aquifer system in Jensen Beach without pumping.*

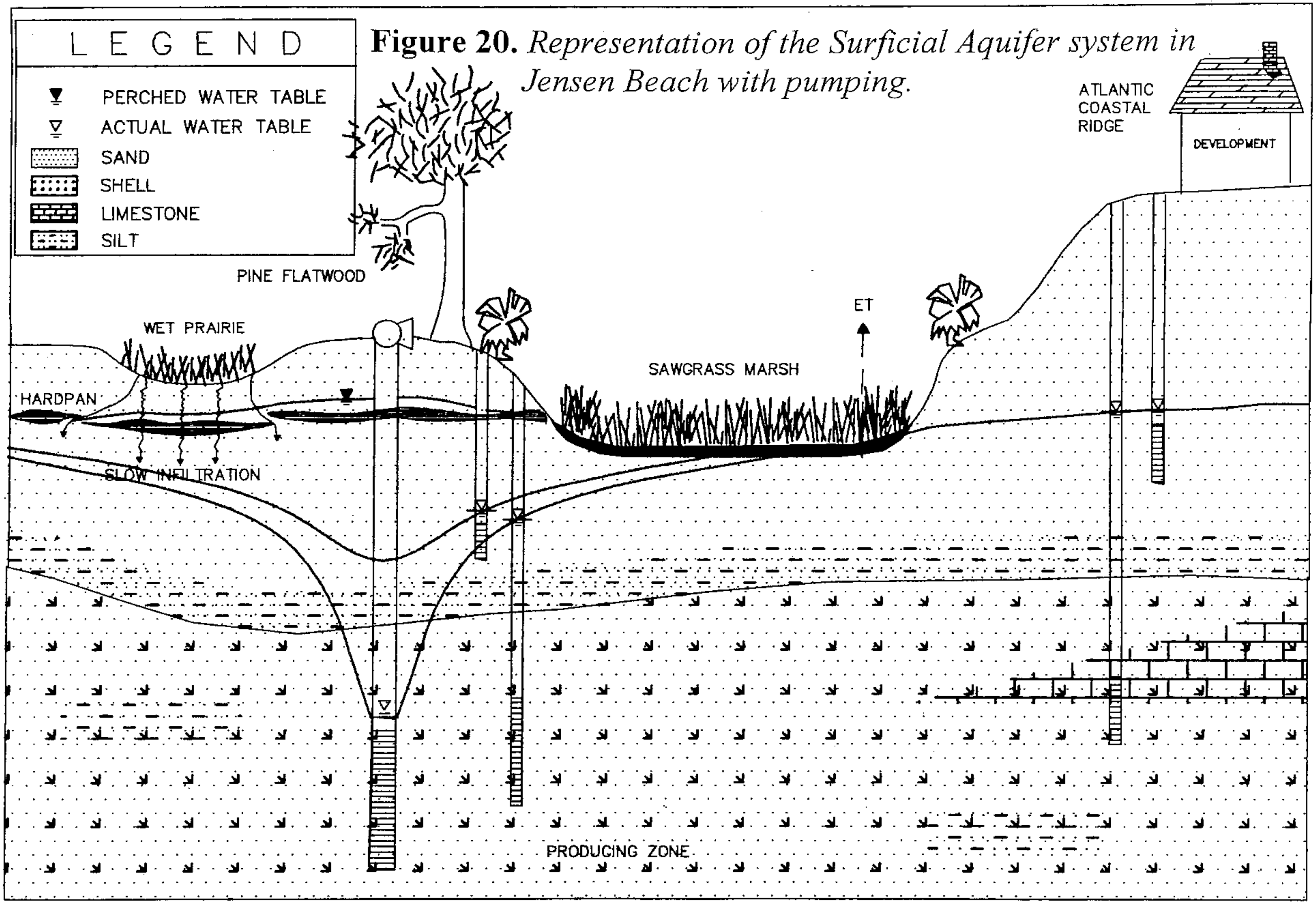


Figure 20. *Representation of the Surficial Aquifer system in Jensen Beach with pumping.*

2.3 Results of the constant rate pumping test

A large amount of data was collected during this test especially water level data recorded in the wells around PW-7. Water quality data obtained during the investigation did not meet the needs of the study and hence it was not used.

2.3.1 Water level data

The raw water level data was corrected and referenced as mentioned previously.

2.3.1.1 Correction of water levels

Fluctuations of atmospheric pressure influence water level in the shallow and deep aquifers. Water levels were atmospherically corrected utilizing a formula presented in Dawson and Istok, 1991.

The method consists of determining the barometric efficiency of the aquifer which is noted as B.E. B.E corresponds to the rate of change in head with changing atmospheric pressure and is expressed by the following formula :

$$BE = -\frac{dh}{(dp_a / \gamma_w)} \times 100\%$$

Where dh = change in head, L

dp_a / γ_w = change in atmospheric pressure expressed as a height of water, L.

pa = atmospheric pressure.

γ_w = unit weight of water

Barometric efficiency is a positive value, the negative sign in the previous formula represents the fact that an increase in barometric pressure results in a decrease of water level. Barometric efficiency is determined prior to the start of the pumping test. It is necessary to record water level fluctuations in all the wells when there is no stress on the aquifer. For each well, a curve is developed for the head versus the change in atmospheric pressure and a best fit line is projected on the graphs. The slope of the line is the B.E.

The water levels in two intermediate wells and two deep wells were used to obtain the curves. The fluctuations were recorded for seven hours and the results are presented in Appendix 11.

Using the barometric efficiency of the aquifer, the water level recorded during the pumping test can be corrected by using the following formula :

$$\Delta h = \frac{BE}{100\%} \times \Delta(p_a / \gamma_w)$$

Where h = change in head that resulted from a change in atmospheric pressure (m).
B.E. = barometric efficiency.
dp_a / γ_w = change in atmospheric pressure expressed as a height of water, L.

The maximal change in atmospheric pressure dp_a / γ_w is :

34.08- 33.96 = 0.12 foot = 3.6 cm

The results of the barometric efficiency are 86 % for the intermediate aquifer and 100% for the deep aquifer. All water levels were corrected by using the following formula :

Water level corrected at t = Water level at t + h at t

Atmospheric pressure was recorded with a frequency of 15 minutes, whereas water levels were recorded at different frequencies. Correction of the values is made by assuming that atmospheric pressure was constant over a 15-minute interval. This assumption was proven by the data collection. The data collected during the observation period was not corrected for atmospheric pressure effects because the difference between the corrected and uncorrected water levels has been modest (maximum deflection of 3.6 cm). The goal of the observation period was to get a general description of Surficial Aquifer system behavior prior to the evaluation period.

✎ *2.3.1.2 Graphic representation*

Water levels obtained during the pumping test were corrected for atmospheric pressure effect, the data were plotted with Freelance software. Correction of water level data allowed the investigator to correct for barometric pressure effects on water levels. The water levels recorded by hand were compared to those recorded by the dataloggers. The difference remained very small and did not exceed four centimeters. The graphs obtained are presented in Appendix 12.

During the pumping test, the pump was accidentally stopped by a technician working at the water treatment plant. The first hydrograph represents water levels obtained without eliminating the effect of the pump stoppage. The second hydrograph was obtained by removing the data recorded between the pump stoppage and the time at which the water table recovered its normal level.

The following observations can be seen in the graphs :

• For the wells situated at the same depth, the drawdown is greater in wells located nearest to the pumping well. This can be explained by the shape of the cone in the water table.

• At a same distance from the pumping well, drawdown levels in paired wells are not the same. This confirms the hypothesis that the entire aquifer does not behave as an unconfined unit.

• During the pumping period, some changes occur in the drawdown curve slope. These changes correspond to a slowing of the drawdown and indicate that the cone of depression has reached a horizon with a lower hydraulic conductivity. This horizon corresponds to the hardpan separating the intermediate aquifer.

• During the recovery period, a slowing of the water table recovery can be noted, indicating that the water level encountered the low permeability unit (hardpan) separating the shallow portions of the intermediate aquifer.

2.3.2 Analyses of the deep aquifer data

The purpose of this analysis is to determine aquifer characteristics. The first requirement is to identify the type of the aquifer and to provide the necessary hypotheses for the mathematical analysis.

↳ *2.3.2.1 Hypotheses of the analysis*

By looking at geological information and hydrographs, it is evident that the aquifer in which the deep wells are placed is different from the one in which the intermediate wells are located. For this reason, the analyses of these two aquifers were made separately.

Well M-1043 is the most geologically representative of the study area. This well indicated the presence of a lower permeability unit made-up of clay, sand shell and interbedded silt. The completion report of well PW-7 confirmed the presence of this low permeability horizon at the study area. The lower permeability horizon occured at 16 meters below ground level. The intermediate wells where therefore placed in the upper formation (depth < 15 m), whereas the deep wells reached the principal production zone of the Surficial Aquifer system.

For the analysis of the deep well data, the aquifer was classified as confined and leaky. The analysis methods are the following :

• Hantush-Jacob method (1955) using a software called AQTESOLV.

• Distance-drawdown method, Cooper-Jacob (1946).

• Hantush Inflection-point (1956).

The cross section representation of the aquifer is shown in Figure 21.

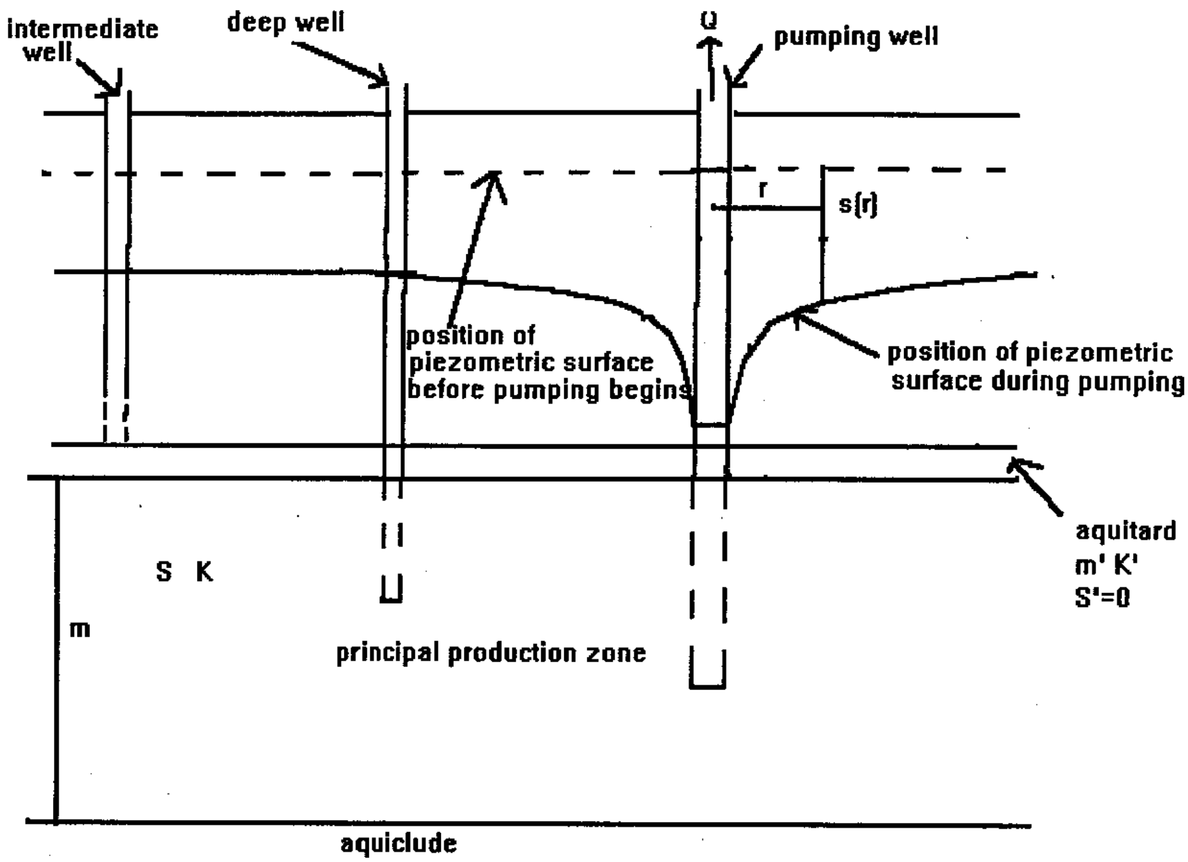


Figure 21 : *Theoretical cross section representation of the Surficial Aquifer system near PW-7.*

In this cross section, the terms are defined as follows :

K = aquifer hydraulic conductivity.
K' = aquitard vertical hydraulic conductivity.
m = aquifer thickness.
m' = aquitard thickness.
Q = constant pumping rate.
r = radial distance from the pumping well to a point on the cone of depression.
s = drawdown of piezometric surface during pumping.
S = aquifer storativity (dimensionless).
S' = aquitard storativity (dimensionless).

The mathematical analysis of a confined leaky aquifer implies the following hypotheses (Stallman, 1971) :

- The aquifer is bounded above by an aquitard and an unconfined aquifer ("the source bed") and bounded below by an aquiclude.
- All layers are horizontal and extend infinitely in the radial direction.
- The initial piezometric surface (before pumping begins) is horizontal and extend infinitely in the radial direction.
- The aquifer and aquitard are homogeneous and isotropic.
- Groundwater density and viscosity are constant.
- Groundwater flow can be described by Darcy's law.
- Groundwater flow in the aquitard is vertical. Groundwater flow in the aquifer is horizontal and directed radially toward the well. This assumption is valid when $m/B < 1$.
- The pumping and observation wells are screened over the entire aquifer thickness.
- The pumping rate is constant.
- Head losses through the well screen and the pump intake are negligible.
- The pumping well has an infinitesimal diameter.
- The aquifer is compressible and completely elastic. The aquitard is incompressible (i.e., no water is released from aquitard storage during pumping). This assumption is valid when $t > 0.036m'S'/K'$ (Hantush, 1960).

All of the assumptions were not met in the in situ conditions but, for this analysis, it will be assumed that they are. The hypothesis that was clearly not met is that the pumping and observation wells are screened over the entire thickness of the aquifer. Indeed, the principal production zone has a thickness of 46 meters whereas the screen of the pumping well has a length of 23 meters.

The Hantush-Jacob method has a variant that takes into account problems such as this. The two other methods do not correct for the effect of partial penetration of the well.

⇝ *2.3.2.2 Hantush-Jacob analysis method*

This method was completed using a software called AQTESOLV. The stages of the analysis are the following :

- provide the data electronically in the proper format.
- input the data required by the software.
- choose the type of solution.
- manual match the curves.
- visualize the results.

To put the data in the correct format, it was necessary to transform the water levels into drawdowns. The drawdown is equal to zero at the beginning of the test then the drawdown at the time t is obtained using the following formula :

S (t)= water level at t - water level at the initial instant

The input data required before running the program are listed below :

- The saturated zone thickness, the rate of the vertical hydraulic conductivity with the horizontal hydraulic conductivity (only for partially penetrating wells)
- For the pumping well, it is necessary to indicate the name of the well, its coordinates, if it is a fully or partial penetrating well, the top and the bottom of the pumping well screen and the pumping rate. For the pumping rate, it is possible to input variable pumping rate by taping or importing the file of pumping as a function of the elapsed time.
- For the observation wells, it is necessary to indicate the name, the coordinates and the type of the well (piezometer, well partially or entirely penetrating), the water table drawdown as a function of the time.
- It is then necessary to indicate the type of aquifer and the mathematical solution corresponding to the aquifer type.

The different aquifer types and usable solutions for analysis of pumping tests through this software are shown in Table 19.

Aquifer type	Mathematical solutions
Unconfined	• Theis (1935) type curve solution with corrected drawdown. • Cooper-Jacob (1946) straight-line solution with corrected drawdown. • Neuman (1974) Type A and Type B curve solution • QuickNeuman Type A and Type B curve solution. • Streltsova (1974) Type A curve solution.
Confined	• Theis (1935) curve solution. • Cooper-Jacob (1946) straight-line solution. • Papadopulos-Cooper (1967) large diameter solution. • Theis recovery straight-line solution.
Leaky	• Hantush-Jacob (1955) type curve solution with no storage in the aquitard. • Hantush-Jacob (1960) type curve solution with storage in the aquitard. • Moench (1985) large-diameter well type curve solution.

Table 19. *Aquifer types and corresponding mathematical solutions proposed by the AQTESOLV software.*

Once the aquifer type and the mathematical solution were chosen, it was possible to run an automatic solution by successive iterations and then to visualize the graphic results. The most current method is first to visualize the graphs representing the matching curve and the field data curve to see if the superposition of the curves is good and to try to improve it manually by modifying the value of the aquifer parameters. When the best superposition is obtained, the automatic solution can then be run. When the software reaches the maximum precision, a message appears on the screen and it is then possible to visualize the results of the statistical analysis and graph the new values for aquifer parameters.

For analysis of deep well the Hantush-Jacob solution for partially penetrating wells without storage in the aquitard was chosen. The Hantush-Jacob equation is obtained from the Theis equation (1935). Theis was the first to take into account the effects of pumping time on well yield. The derivation of his equation is based on the Darcy's law and on the principle of conservation of mass in a radial coordinate system.

The Theis equation used is the following :

$$\frac{\partial^2 s}{\partial r^2}+(\frac{1}{r})\frac{\partial s}{\partial r}-\frac{s}{B^2}=(\frac{S}{T})\frac{\partial s}{\partial t}$$

with s = drawdown, L.
r = radial distance from the pumping well, L.
S = aquifer storativity.
T = aquifer transmissivity, T= Km, L^2T^{-1}.
t = elapsed time, T.

The initial and boundary conditions are the following :

- before pumping begins drawdown is zero everywhere
$$s\,(r, t = 0) = 0$$

- at an infinite distance from the pumping well, the drawdown is zero
$$s\,(r = \infty\,, t) = 0$$

- groundwater flow to the pumping well (Q) is constant and uniform over the aquifer thickness (which is a result of the assumption of horizontal groundwater flow in the aquifer).

$$\lim_{r\to 0}\frac{r\partial s(r,t)}{\partial r}=-\frac{Q}{2\pi T}$$

In 1955, Hantush and Jacob simplified the equation based on the following assumptions :

- the water-bearing is uniform in character and the hydraulic conductivity is the same in all directions.

- the formation is uniform in thickness and infinite in areal extent.

- the formation receives no recharge from any source.

- the pumped well penetrates into and receives water from the full thickness of the water-bearing formation.

- the water removed from storage is discharged instantaneously when the head is lowered.

- the pumping well is 100-percent efficient.

- all water removed from the well comes from aquifer storage.

- laminar flow exists throughout the well and aquifer.

The Hantush-Jacob solution can be used when :

$$t > 30r^2(\frac{S}{T})(1-(\frac{10r}{B})^2)$$

and $$r / B < 0.1$$

Under these assumptions the Theis equation becomes :

$$s = \frac{1}{4\Pi}\frac{Q}{T}W(u,r/B)$$

With :

$$W(u,r/B) = \int_{u}^{+\infty} \exp(-y-\frac{r^2}{4B^2y})\frac{1}{y}dy$$

W(u, r / B) is read "well function of u". In the W(u) function u is equal to :

$$u = \frac{r^2S}{4Tt}$$

The values of the W(u, r / B) are obtained from tables.

✎ 2.3.2.3 *Hantush Inflection-Point method*

When the aquifer is confined and leaky, the plot of the drawdown versus the logarithm of the time has an inflection point (Roscoe, 1990). At the inflection point the following equations apply :

$$u_i = \frac{r^2S}{4Tt_i} = \frac{r}{2B} \qquad (1)$$

$$m_i = \frac{2.3Q}{4\pi T} e^{-r/B} \qquad (2)$$

$$s_i = 0.5 s_m = \frac{Q}{4\pi T} K_0 (r / B) \qquad (3)$$

$$2.3 \frac{s_i}{m_i} = \exp(r / B) K_0 (r / B) \qquad (4)$$

where i indicates that this is the value of a variable at the inflection point.
sm = maximum drawdown.
mi = slope of the drawdown curve at the inflection point (this can be approximated by the slope of the straight-line portion of the curve on which the inflection point lies).
Ko = zero order modified Bessel function of the second type.

These equations are the basis for the following method of analysis :

- Plot s versus log(t).
- Estimate the maximum drawdown, sm, and compute s_i = 0.5sm.
- Using the value of s_i, locate the inflection point and record the value of t_i.
- Fit a straight line to the drawdown data through the inflection point.
- Using the fitted line determine mi by measuring the change in drawdown occurring over one log cycle.
- Calculate s_i/m_i and use equation (4) and a table to determine exp(x)Ko(x) to compute B.
- Substitute inflection point values into equations (1) (2) and (3) to compute T, S and K'.

2.3.2.4 *Distance-Drawdown method*

From the Theis equation (1935), Cooper and Jacob (1946) have shown that when u is sufficiently small, the Theis equation can be modified to get the following simplified shape :

$$s = \frac{0.183Q}{T}\log\frac{2.25Tt}{r^2S}$$

When the value of u is less than 0.05, the results obtained from this simplified equation are in practice, identical to those obtained from the equation for W (u). When the value of u increases, t decreases and r increases. It is necessary therefore to have a sufficiently large t and a sufficiently small r. Therefore, when the pumping rate is constant, Q, T and S are constant. The drawdown varies with $\log(t/r^2)$ when u is less than 0.05. Two important relations can be deduced from this analysis.

- For a particular aquifer in all specific point (constant r), the terms s and t are the only variables. s varies as a function of log(Ct) where C is a constant representing all the constant terms.

- For a particular aquifer and a value of t, the terms s and r are the only variables. In this case, s varies as a function of $\log(C'/r^2)$ where C' is a constant term representing all the constant terms.

Values of the aquifer parameters can then be obtained. It is necessary to plot values of s on a semi log paper versus r obtained during the pumping test at a fixed time t. A minimum of three observation points situated at various distances from the pumping well must be used. A straight line is obtained and is used to compute the coefficients T and S using the simple following relations :

$$T = \frac{0.336Q}{\Delta s}$$

with: T = transmissivity coefficient in m^2/d.
Q = pumping rate in m^3/d.
s = slope of the line, expressed as drawdown (m), for one log cycle.

$$S = \frac{2.25Tt_0}{r_0^2}$$

with: S = storage coefficient
T = transmissivity in m2/d.
t_0 = elapsed time for which no drawdown is observed in days.
r_0 = zero drawdown intercept.

✎ *2.3.2.5 Determination of the well efficiency*

The well efficiency is defined as the ratio of actual to theoretical drawdown at the pumping well, for a given pumping rate. The method of defining the well efficiency consists of :

- Plotting the distance-drawdown curve on semi-log paper, for a minimum of three observation points and fitting a straight line.
- Determining the theoretical drawdown of the water table by projecting the fitted line to a distance equal to the production well radius.

The flow must be laminar. When the best line was fitted, the theoretical and observed drawdown appeared to be identical (Appendix 13). This corresponds to a well efficiency of about 100 %. This result is not correct (it is much too high), since the well has been used for the past 13 years. Such results can be explained by the fact that well PZ 36 D is situated too close to the pumping well and flow of water in the well is therefore probably turbulent. By using only the two wells PZ 87 D and PZ 136 D, a theoretical drawdown of 9.02 m (29.6 ft) was obtained. The well efficiency is therefore :

$$(29.6/ 37.6) \times 100 = 78.7\ \% \approx 79\ \%.$$

2.3.3 Analysis results

✎ *2.3.3.1 Hantush-Jacob method*

The curves, and the values of aquifer parameters obtained by using the AQTESOLV software, are presented in Appendix 14. The curves do not match during the first minutes of the test. This does not constitute a problem because the Hantush-Jacob solution is valid only when t is sufficiently large. To match the curves for the first minutes of the test it was necessary to increase the value of r/B. However, in this case, the curves no longer matched when the data correspond to an elapsed time of more than a few minutes. By elsewhere, the hypotheses of resolution require to have r/B inferior to 0.1. The curve's solution that matched best for longer time values (t greater than couple of minutes) was chosen.

The shape of the curves, characterized by a high slope at the beginning followed after 13 minutes of pumping by a marked decrease in slope, can be explained by the fact that the cone of depression has hit a recharge boundary. After closely reviewing all field notes and aerial photographs, the assumption was made that this recharge boundary corresponded to the lake situated at 240 meters from the pumping site.

To verify this assumption, the appropriate aquifer characteristics (thickness, transmissivity) and the pumping information were input in a program created by Dawson and Istok (1991). The program simulated drawdown in a confined and leaky aquifer as a function of time and allowed discretization of time and spatial distance from the center of the pumping well. A range of values for aquifer parameters was input, varying from 0.05 to 0.008 m/d for K' and ranging from 150 to 340 m^2/d for T.

In all the cases, the results showed that the cone of depression reached the position of the lake at approximately 13 min. One of the simulations is presented in Appendix 15. This observation pointed out the influence of lakes on this study area. During the dry season the lake drains the groundwater in the nearby wetland and decrease its hydroperiod.

The value of vertical hydraulic conductivity of the aquitard has been computed using the following formula :

$$K' = \frac{Tm'}{B^2}$$

Where m' = thickness of the aquitard (3.05 m) and using the result of r/B given by AQTESOLV.

The value of horizontal hydraulic conductivity for the producing zone was computed using the following formula :

$$K = \frac{T}{m}$$

Where m = producing zone thickness (80 feet = 24.4 m)

The results for aquifer parameters, after the units were converted, are shown in Table 20.

Well name	Storativity coefficient	Vertical hydraulic conductivity K' m/d	Hydraulic conductivity m/d	Transmissivity m²/d
PZ 36 D	0.0009072	0.0540	11.0	265
PZ 87 D	0.0006227	0.0120	14.0	340
PZ 136 D	0.0047900	0.0055	15.6	382

Table 20. *Results of the aquifer parameters computed from the constant rate pumping test using AQTESOLV.*

Verification of the assumptions requires that :

$$r/B < 0.1 \quad (1)$$

$$t > 30r^2\left(\frac{S}{T}\right)\left(1-\left(\frac{10r}{B}\right)^2\right) \quad (2)$$

For the three wells, r/B = 0.09 < 0.1. Therefore, the hypothesis (1) is not rejected.

Solving for t, the results of $30r^2(\frac{S}{T})(1-(\frac{10r}{B})^2)$ are as shown in Table 21.

Well name	Computed values (min)
PZ 36 D	3.40
PZ 87 D	10.5
PZ 136 D	17.7

Table 21. *Values of* $30r^2(\frac{S}{T})(1-(\frac{10r}{B})^2)$.

The superposition of the two curves was good when the elapsed time was greater than the values presented in Table 21. So, the assumptions for the Hantush-Jacob method were supported.

2.3.3.2 Hantush Inflection-Point method

The curves obtained and the details of the calculations are presented in Appendix 16. The hydraulic conductivity of the principal production zone of water was computed using the following formula :

$$K = \frac{T}{m}$$

Where m = producing zone thickness (80 feet = 24.4 m)

The results obtained by this method are shown in Table22.

Well name	Storage coefficient	Transmissivity T in m²/d	Vertical hydraulic conductivity K' in m/d	Hydraulic conductivity K in m/d
PZ 36 D	0.000558	178.6	0.020	7.3
PZ 87 D	0.0002864	242.0	0.012	10.0
PZ 136 D	0.0003449	280.0	0.008	11.4

Table 22. *Value of the aquifer parameters computed with the constant rate pumping test data and using the Hantush inflection-point method.*

The inflection point of the curve representing the data from the well PZ 36 D was not very apparent, probably because the flow of the water in this well was turbulent due to the proximity of the pumping well.

✎ 2.3.3.3 *Distance-Drawdown method*

This method was applied for the values of drawdown corresponding to the time t = 1435 minutes. The curves obtained are presented in Appendix 17. Based on these curves the value for s was estimated as 4.2 m (13.75 feet) and the value of r_0 was 140 m (460 feet).

By substituting these values in the formulas presented in part 2.3.2.4, the following results were obtained:

Storage coefficient	Transmissivity T in m³/d	Hydraulic conductivity K in m/d
0.0004	152	6.3

Table 23. *Value of the aquifer parameters computed using the date from the constant rate test and the distance-drawdown method.*

It is necessary to verify that u < 0.05. Table 24 gives the values of u calculated from the values of S and T, using the following formula :

$$u = \frac{r^2 S}{4Tt}$$

Well name	Computed values of u
PZ 36 D	0.003
PZ 87 D	0.019
PZ 136 D	0.047

Table 24. *Calculated values of u.*

2.3.4 Analysis method for the intermediate aquifer

For wells in the intermediate aquifer, examination of the geologic data and hydrographs indicated that these wells are situated in an unconfined aquifer, separated from the principal production zone by an aquitard or lower permeability area. Their behavior should therefore resemble that of an unconfined aquifer.

Analysis of the curves representing the water table drawdown versus the elapsed time of the pumping test on logarithmic paper, indicates a delayed yield. This phenomenon is due to the fact that, in some cases, in unconfined aquifers the release of water from storage does not occur instantaneously as the head declines. The drop in the water table usually occurs faster than the rate at which pore water is released. The remaining pore water drains slowly by gravity until it reaches the water table.

Indeed, by comparing the curves obtained to the classic Theis curves (1935) representing the behavior of a confined aquifer, it appeared that the drawdown observed in the wells was less important than the one predicted by the Theis curve indicating the occurrence of a delayed yield phenomenon. Based on these data, it was assumed the Boultons method could be used, since the aquifers met the basic assumptions of being unconfined, showing delayed yield and having non steady state flow to the well.

This method consists of graphing, for each intermediate well the water table drawdown versus the time on logarithm paper. Next, the observed-data curve of each well is superimposed on a family of "Boulton type curves" to find out which curve provides the best match. It was not possible to do this superposition with the field data, none of the family of Boulton curves provided a match. The reason why the curves did not match were explored.

In order to assure that the behavior of the aquifer corresponded to an unconfined delayed yield aquifer, the curves of the drawdown versus time were plotted on semi-log and log-log paper. The observed data curve was compared with the theoretical curves describing such behavior. The shape of this curves did not correspond to the shape of the theoretical curves (Appendix 18). By reviewing and comparing these data with data from other studies of the delayed yield phenomenon, it was concluded that, in some cases, several weeks of pumping may be required to differentiate delayed yield behavior from leaky behavior (Kruseman et al., 1991). Since delayed yield of the aquifer presents a problem for conventional solutions it was decide to determine the aquifer characteristics by conducting slug test on the appropriate wells.

2.4 Results of the step-drawdown pumping test

2.4.1 Generalities

The purpose of the step-drawdown pumping test was to confirm the value of the aquifer parameters obtained by analyzing the constant rate pumping test. Usually the step-drawdown test gives less accurate results than the constant rate test (Kawecki, 1995).

The method was the same as the one used for the constant rate test. Water levels were recorded at several observation points while the pump was running. The water levels were then corrected for atmospheric pressure effects. The data were then represented graphically using Freelance to verify that none of the data are incoherent (Appendix 19). The water levels recorded manually were compared to the ones recorded by the dataloggers.

2.4.2 Analysis

Only the deep wells were analysed using the step-drawdown pumping test. The same hypotheses were applied as were used for the constant rate test. The only usable method was Hantush-Jacob based on the AQTESOLV software. Use of this software was detailed in part 2.3.2.2. The curves obtained are presented on Appendix 20. Superposition of the curves was good during all the duration of the test and can be expected to produce coherent results as presented in Table 25.

Well name	Storage coefficient	Hydraulic conductivity in m/d	Vertical hydraulic conductivity K' of the aquitard in m/d
PZ 36 D	0.0004211	14.6	0.00569
PZ 87 D	0.00004408	12.4	0.0008
PZ 136 D	0.0005356	16.9	0.0027

Table 25. *Value of the aquifer parameters computed from the step-drawdown data and by utilization of AQTESOLV software.*

The formula used in order to compute K' is the following :

$$K' = \frac{Tb'}{B^2}$$

with T : Transmissivity of the main production zone.
b' : aquitard thickness.

It is necessary to verify the following hypotheses :

$$r / B < 0.1 \qquad (1)$$

$$t > 30r^2(\frac{S}{T})(1-(\frac{10r}{B})^2) \qquad (2)$$

Since superposition of the curves was good throughout the test, the second condition was not an issue. The first condition was also verified, except for PZ 136 D where r/B = 0.11 which is little bit greater than 0.1.

2.5 Slug test

The aquifer analysis indicated a delayed yield phenomenom. Results from the pumping tests were not applicable. Thus slug tests were conducted in the intermediate wells on June 12 1996.

2.5.1 Slug test completion method

The slug test solution developed by Bouwer and Rice (1976) permits the measurement of saturated hydraulic conductivity (K) of an aquifer with a single well. The well can be partially penetrating and partially screened, perforated or otherwise open. The method consists of quickly lowering or raising the water level in a well or borehole from equilibrium, and measuring its subsequent rate of rise or fall, respectively. While originally developed for unconfined aquifers, the method can also be used for confined or stratified aquifers if the top of the screen or perforated section is some distance below the upper confining layer. An evaluation of the Bouwer and Rice method was recently done by Hyder et al., 1995.

For this study, slug tests were conducted on all intermediate wells. The data were recorded by a datalogger (In-Situ Hermit 1000) connected to a transducer (In-Situ PXD-260). The method consisted of :

- placing the transducer in the well,
- measuring the water level,
- placing the slug in the well,
- allowing the water level to reach the equilibrium,
- quickly removing the slug and recording water levels.

The slug consisted of a pipe filled with sand to give it weight and sealed with caps on both ends. The diameter of the pipe was 32 mm (1.25 inches). Data were recorded using log-cycle frequency, the same frequencies used during the constant rate test. The data were recorded until the water in the well reached the same level as before removing the slug (equilibrium water level).

2.5.2 Analysis method

The rate of flow of ground water into the well when water in the well is a distance y below the static ground-water table, is calculated with the Thiem equation as

$$Q = 2\pi KL \frac{y}{\ln(R_e / R_w)} \qquad (1)$$

Where Q = volume rate of flow into the well $[L^3T^{-1}]$,
K = hydraulic conductivity around the well $[LT^{-1}]$,
L = length of screened section of the well [L],
y = vertical difference between water level inside the well and static water level outside the well [L],
R_e = effective radial distance over which y is dissipated [L],
R_w = radial distance of undisturbed portion of the aquifer from the centreline [L].

The following drawing represents the geometry and symbols for slug test :

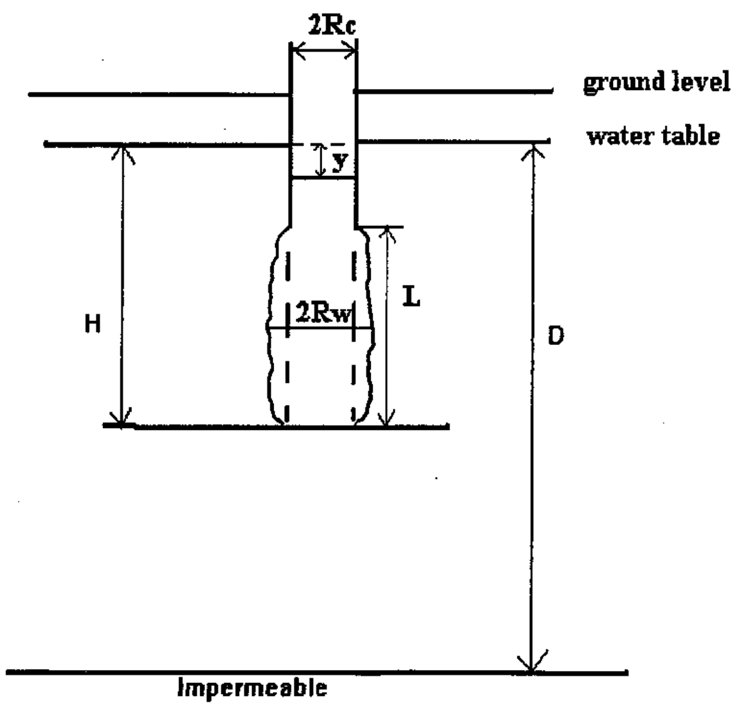


Figure 22. : *Geometry and symbols for slug test.*

The value of Rw is the radius of the screened section of the well plus the thickness of a sand or gravel pack and of the developed zone around the well. Thus, Rw is the radial distance from the center of the well to the normal K of the aquifer. Because the thickness of the developed zone is almost never known, the tendency is to ignore it and take only gravel or sand packs into account.

The previous equation and the following are based on the following assumptions :

- drawdown of the water table around the well is negligible.
- flow above the water table (in the capillary fringe) can be ignored.
- head losses that occur as water enters the well are negligible.
- the aquifer is homogeneous and isotropic.

The rate of rise, dy/dt, of the water level in the well after suddenly removing a slug of water can be related to the inflow Q by the equation :

$$\frac{dy}{dt} = -\frac{Q}{\pi R_c^2} \qquad (2)$$

Where R_c = radius of the casing of the well where the rise of the water level is measured.

Combining (1) and (2) yields

$$\frac{1}{y}dy = -\frac{2KL}{R_c^2 \ln(R_e / R_w)}dt \qquad (3)$$

which can be integrated to

$$\ln y = -\frac{2KLt}{R_c^2 \ln(R_e / R_w)} + \text{constant} \qquad (4)$$

Applying this equation between limits y_0 at $t = 0$ and y_t at t and solving for K yields

$$K = \frac{R_c^2 \ln(R_e / R_w)}{2L}\frac{1}{t}\ln(\frac{y_0}{y_t}) \qquad (5)$$

This equation enables K to be calculated from the rise of water level in the well after suddenly removing a slug of water from the well. Since K, R_c , R_w , R_e and L are constants, (1/t) $\ln(y_0/y_t)$ must also be constant. Thus field data should yield a straight line in a plot of ln y versus t.

2.5.3 Results

The raw data were entered using an Excel spreadsheet computer program. The Excel program was created by Paul Linton and calculates the hydraulic conductivity using the recovery data from a slug test. The results obtained are presented in Table 26.

Well Name	PZ 136I	PZ 87I	PZ 36I
Hydraulic conductivity (m/j)	0.9	3	0.6

Table 26. *Results of the hydraulic conductivity for the intermediate aquifer obtained with the slug tests.*

The graphs and the details of the calculation are presented in Appendix 21.

2.6 Comparison of the aquifer analysis results

A constant rate pumping test was conducted by J. M. Montgomery, Consulting Engineers, Inc. between October 5 and October 9 1989, on Production well 7 at Martin County Wellfield (J. M. Montgomery, 1989). This is the same site as this pumping site study. The results obtained by J. M. Montgomery were compared to the results obtained in this study. The analysis methods used by J. M. Montgomery, Inc. were the following :

- Hantush Inflection point (1956),
- Distance-drawdown method, Cooper and Jacob (1946),
- Walton (1962)

The two first methods are identical to the one used in this study. The Walton method is very similar to Hantush method used in this study.

A summary of the aquifer parameters results is presented in Tables 27, 28 and 29.

Well Name	Analysis method used	Transmissivity T obtained by J.M.M in m^2/d	Transmissivity T obtained in this study during the constant rate pumping test in m^2/d	Transmissivity T obtained in this study during the step drawdown test in m^2/d
PZ 136 D	Hantush Inflection point	273	280	–
PZ 136 D	Walton or Hantush	254	382	412
PZ 87 D	Hantush Inflection point	242	242	–
PZ 87 D	Walton or Hantush	184	340	302
PZ 36 D	Hantush Inflection point	124	179	–
PZ 36 D	Walton or Hantush	118	265	356
PZ 136 D PZ 87 D PZ 36 D	Jacob, distance-drawdown	205	152	–

Table 27. *Transmissivity values obtained by J.M.M. and in this study.*

Well Name	Analysis method used	Storage coefficient obtained by J.M.M	Storage coefficient obtained during the constant rate pumping test in this study	Storage coefficient obtained during the step drawdown pumping test in this study
PZ 136 D	Hantush Inflection point	0.0003	0.00034	–
PZ 136 D	Walton or Hantush	0.0004	0.004	0.00053
PZ 87 D	Hantush Inflection point	0.0003	0.00028	–
PZ 87 D	Walton or Hantush	0.0003	0.0006	0.000044
PZ 36 D	Hantush Inflection point	0.0002	0.0005	–
PZ 36 D	Walton or Hantush	0.0003	0.009	0.00042
PZ 136 D PZ 87 D PZ 36 D	Jacob, distance-drawdown	0.0004	0.0004	–

Table 28. *Storage coefficient values obtained by J.M.M. and in this study.*

Well Name	Analysis method used	Vertical hydraulic conductivity K' obtained by J.M.M in m/d	Vertical hydraulic conductivity K' obtained in this study during the constant rate pumping test in m/d	Vertical hydraulic conductivity K' obtained in this study during the step drawdown pumping test in m/d
PZ 136 D	Hantush Inflection point	0.011	0.008	0.008
PZ 136 D	Walton or Hantush	0.0174	0.0055	0.0055
PZ 87 D	Hantush Inflection point	0.0126	0.012	0.012
PZ 87 D	Walton or Hantush	0.031	0.012	0.012
PZ 36 D	Hantush Infection point	0.08	0.02	0.02
PZ 36 D	Walton or Hantush	0.122	0.054	0.054

Table 29. *Vertical Hydraulic conductivity K' obtained by J.M.M. and in this study.*

The values for vertical hydraulic conductivity were obtained using the leakance and a thickness of the aquitard of 3 meters. The results obtained in this study are relatively close to the values observed by J M M. Fluctuations of water levels in the intermediate wells were recorded by J M M. during the test but, no aquifer characteristics based on those water levels were included in the report.

2.7 Conclusion

The study has generated results for the following parameters :

- hydraulic conductivity of the principal producing zone of the Surficial Aquifer system,
- hydraulic conductivity of the upper sand zone,
- vertical hydraulic conductivity for the confined horizon separating these two zone.

The aquifer analysis results were applied in the second part of this study using a pre-existing mathematical model of the Jensen Beach peninsula.

The following recommendations can be given for the future investigation in the wetlands :

- Installation of the groundwater monitor wells should be installed at depths and radial locations recommended by previous USGS publications. Wells should not be placed in locations merely to accommodate the drilling of the wells. Utilizing this existing criteria ($r=1.5*b$) for groundwater monitoring well construction (for unconfined aquifers) none of the existing wells met this criteria. Data obtained from wells closer than the criteria should be closely evaluated or discarded.

- Water levels within the wetland and the surrounding aquifer need to be monitored (electronically) on a minimum of one hour to observe storm events and recharge from upland areas.

3. MODELING OF THE STUDY SITE

The purpose of modeling was to verify the aquifer analysis data obtained during the hydrologic study.

3.1 General descriptive of the model

A mathematical model of the Jensen Beach peninsula was developed by E.Hopkins in 1991 using the U.S. Geological Survey modular three-dimensional finite-difference ground water flow model code (McDonald and Harbaugh, 1988), commonly known as MODFLOW. In this study, a previously created groundwater flow model was used as a base case and modified it to emulate field conditions. MODFLOW is essentially a water budget program based on Darcy's law and the equation of continuity, which when applied to the aquifer can be written as :

Inflow – Outflow = Change in Storage

3.1.1 Method of the finite-difference

The three-dimensional movement of ground water of constant density through porous earth material can be described by the following equation :

$$\frac{\partial}{\partial x}(K_{xx}\frac{\partial h}{\partial x})+\frac{\partial}{\partial y}(K_{yy}\frac{\partial h}{\partial y})+\frac{\partial}{\partial z}(K_{zz}\frac{\partial h}{\partial z})-W=S_s\frac{\partial h}{\partial t} \qquad (1)$$

With K_{XX} = hydraulic conductivity along the x coordinate axes.
K_{yy} = hydraulic conductivity along the y coordinate axes.
K_{zz} = hydraulic conductivity along the z coordinate axes.
h = potentiometric head.
W = volumetric flux per unit volume.
S_S = specific storage of the porous material.
t = time.

No analytical solutions of this equation exist, except for very simple systems. Various numerical methods can be employed to obtain approximate solutions. One of those methods is the finite-difference method. This approach consists of replacing the continuous system described by equation (1) by a finite set of points. The process leads to systems of simultaneous linear algebraic difference equations ; their solution yields values of head at specific points and times. These values constitute an approximation to the time-varying head distribution that would be given by an analytical solution of the equation (1) (McDonald and Harbaugh, 1988).

3.1.2 Advantages of the software

MODFLOW presents many advantages such as :

- It is compatible with most computers with only minor modification,

- The modular structure of the code and its documentation allow easy modification and the addition of new modules for special applications,

- MODFLOW allows great flexibility of data file structure and management ; this facilitates the employment of, and interaction with, other software for data manipulation,

- The cell-by-cell flow feature of the code can be used to :

  ✓ Evaluate in detail flow and head changes associated with various withdrawal scenarios.
  ✓ Generate boundary conditions for higher-resolution models within the regional flow model.

- It can be coupled with currently available non-density dependent solute transport models.

3.1.3 Hydrologic properties simulated by MODFLOW

The hydrologic properties or conditions which the model can represent include :

- Aquifer properties of hydraulic conductivity or transmissivity, storage capacity, and vertical conductance.

- Initial water level conditions.

- Recharge.

- Evapotranpiration (ET).

- Rivers and drains. Rivers can both drain and recharge the aquifer, depending on the relationship of the river stage to the adjacent aquifer heads ; drains do not recharge.

- Wells, as either discharge or recharge.

3.1.4 Organization of the program

The modular structure consists of a Main Program and a series of highly independent subroutines called "modules." The modules are grouped into "packages." Each package deals with a specific feature of the hydrologic system which is to be simulated (McDonald and Harbaugh, 1988). The division of the program into modules permits the user to examine specific hydrologic features of the model independently. This also facilitates development of additional capabilities because new packages can be added to the program without modifying the existing packages. (Anderson et. al., 1992). The list of the modules and their utilization are presented in Table 30.

Package Name	**Abbreviation**	**Package Description**
Basic	BAS	Handles those tasks that are part of the model as a whole, including specification of boundaries, determination of time-step length, establishment of initial conditions and printing of results.
Block-Centered Flow	BCF	Calculates terms of finite-difference equations which represent flow within porous medium ; specifically flow from cell to cell and flow into storage.
Well	WEL	Adds terms representing flow to wells to the finite-difference equations.
Recharge	RCH	Adds terms representing areally distributed recharge to the finite-difference equations.
River	RIV	Adds terms representing flow to rivers to the finite-difference equations.
Drain	DRN	Adds terms representing flow to drains to the finite-difference equations.
Evapotranspiration	EVT	Adds terms representing ET to the finite-difference equations.
General-Head Boundaries	GHB	Adds terms representing general-head boundaries to the finite-difference equations.

Table 30. *List of packages usable in MODFLOW.*

Three iterative solution schemes are available for solving the finite difference equations governing flow in porous media :

- slice-successive over relaxation (SSOR)
- strongly implicit procedure (SIP)
- preconditioned conjugate gradient (PCG).

The SIP method was used in this study as well as in study by Hopkins (1991) and Adams (1992) .

3.2 Set up of the model

The study area modeled in this study corresponds to the Jensen Beach peninsula.

3.2.1 Discretization of the model

The finite-difference method depends upon discretization of the region of flow into a finite number of blocks (cells), each cell having unique hydrogeologic properties. The hydraulic head for the entire cell is defined at the center or node. The grid used in this study was the one created by Hopkins (1991). The study area with the model grid used in this analysis is shown in Figures 23a and b.

✎ *3.2.2.1 Horizontal discretization*

Because the wetlands in the vicinity of the wellfield are small, a large grid cell size would make it impossible to view them as discrete features. It was necessary, therefore, to have a cell size that would be small enough to represent individual wetlands. On the other hand, as the cell size decreases. Data input needed for the model becomes more intensive, data collection and computational limitations become a constraint. For those reasons, a cell dimension of 240 by 240 feet was chosen for most of the model. Beginning in the eight row, the cell length expands to the Northwest, away from the main area of interest, by a factor of 1.5 times per row. The grid is 96 rows by 98 columns (Hopkins, 1991).

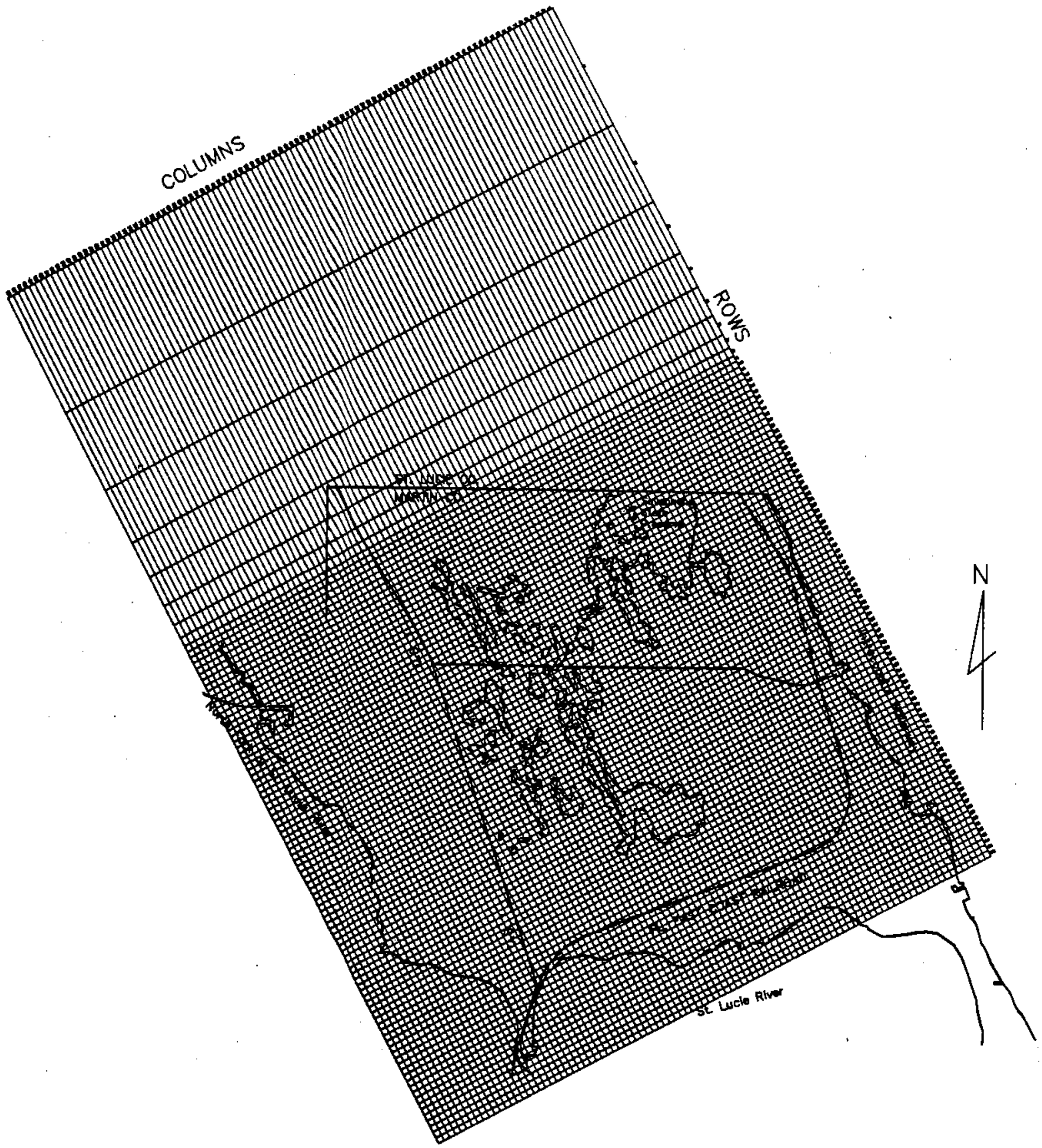
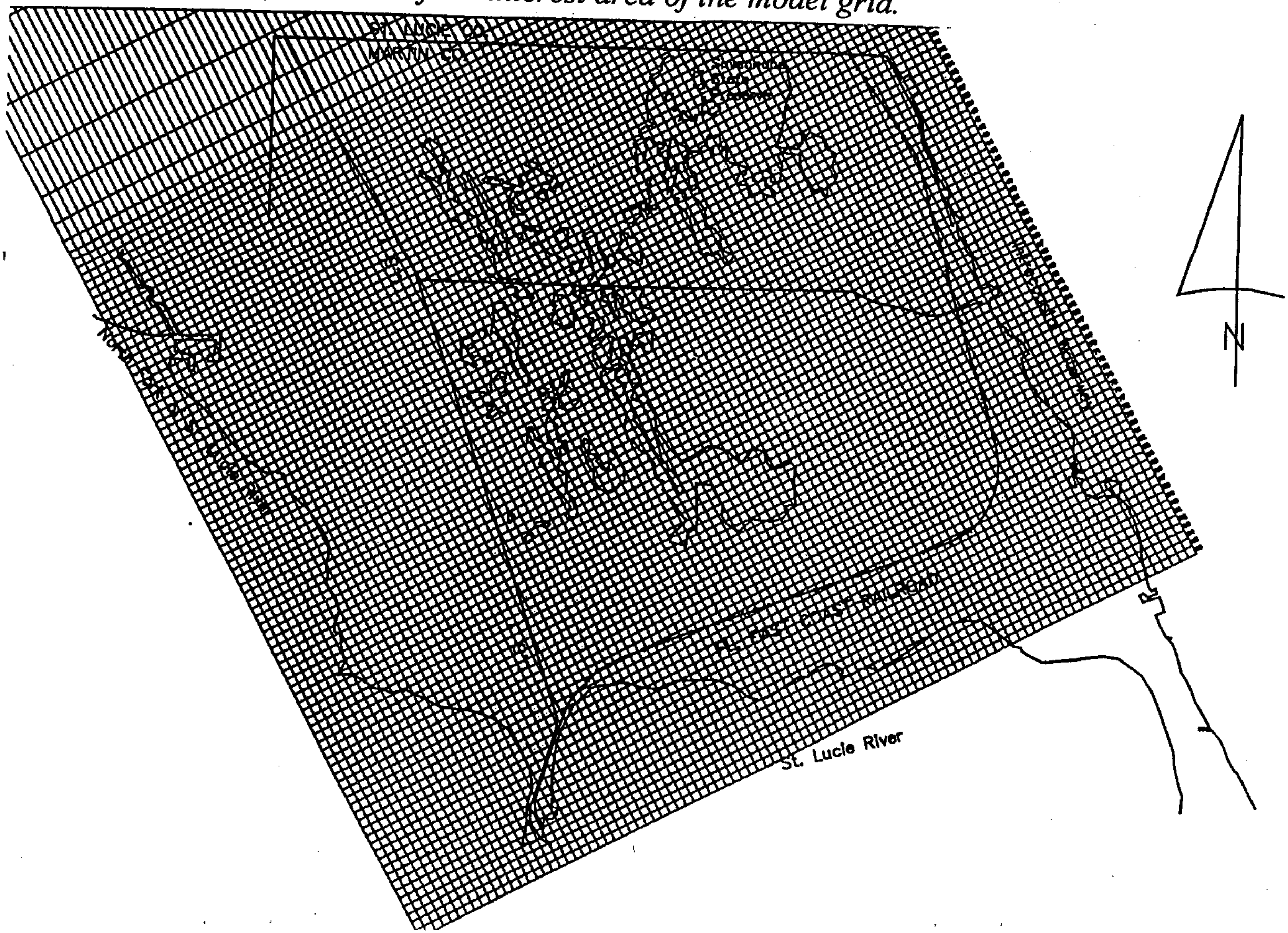


Figure 23a. *Model grid.*

Figure 23b. *Zoom of the interest area of the model grid.*



3.2.2.2 *Vertical discretization*

The model was discretized into two layers based on existing lithologic data and information obtained from the J. M. Montgomery reports (1988 and 1989) and Adams (1992) and Hopkins (1991) investigations.

The top layer represents the portion of the aquifer that is composed of medium to fine grained sand, which tends to grade finer with depth. This surficial sand layer ranges in thickness from 12 meters in the vicinity of the wellfield to more than 24 meters on the eastern edge of the study in the sandhills of the Atlantic Coastal Ridge. Interbedded lenses of sandy clay and silt are present at the base of this unit in some areas. Layer two consists of the main production zone of the aquifer. This zone has a thickness of 15 to 42.5 meters and was separated from the surficial sand layer on the basis of its higher transmissivity.

MODFLOW requires each layer of a model to be classified as either confined, unconfined, or fully or partially convertible between confined and unconfined. Both layer one and two are part of the unconfined, or water table aquifer, though flow between the two is sluggish due to the presence of fine material near the base of layer one. However MODFLOW does not allow the designation of more than one unconfined layer in a model. For this reason, layer one was defined as unconfined and layer two was designated as partially convertible between confined and unconfined. These designations determine the way in which water levels will be calculated within a layer.

- In an unconfined layer, transmissivity is continually recalculated as a product of hydraulic conductivity and the saturated thickness of the layer. Storage is determined from the specific yield.

- Under the confined/unconfined designation, it is assumed that the majority of the layer remains saturated throughout the simulation, so that it is not necessary to continually recalculate transmissivity. Hydraulic conductivity and the elevations of the top and bottom of the aquifer are input and MODFLOW calculates layer transmissivity.

After each iteration the model checks to determine whether the head in the layer is above or below the elevation of the top of the layer. If the head is higher than the top of the layer, the layer is assumed to be confined. If the head is less than the elevation of the top of the layer, the layer is assumed to be unconfined. This layer type requires the input of both a specific yield and a storage coefficient so that storage may alternate between confined and unconfined values (Hopkins, 1991).

3.2.3 Stress Period

The period of simulation is divided into a series of stress periods within which stress parameters are constant. Each stress period is then divided into a series of time steps. The model created by Hopkins (1991) had originally a stress period of one-month duration and a daily time steps.

For this study, a stress period of one-day duration was chosen, because calibration of the model was based on the data collected during the constant rate pumping test that lasted for a period of five days. A duration of six hours was chosen for each time step. Therefore, for each stress period, there were four time steps.

To determine the total duration of the simulation, it was necessary to take into account the fact that the model needed several iterations before converging correctly. It was assumed that 150 iterations were enough for the model to stabilize and converge correctly. For this reason, the total duration of the simulation was 40 days, from January 1st to February 9, 1996, which included a period of 28 days for the numerical code to stabilize.

3.2.3 Hydraulic properties

The pumping and slug tests provided results for the aquifer characteristics. Those results that were used as a verification during the calibration process of the model.

Little information is available concerning vertical hydraulic conductivity within the study area. Todd (1980) states that the anisotropy ratio for horizontal to vertical conductivity usually falls between 1 to 10 for sand, but may range upwards of 100 if clay is present. The study area is mainly composed of fine to medium sand with interbedded areas of clay and clay size particles.

For a MODFLOW simulation involving more than one layer, the modeler has to calculate a vertical transmission or leakage term, known as VCONT. This parameter is input for each nodal block in the grid except for blocks in the bottom layer, which is assumed to be underlain by impermeable material.

3.2.4 Recharge

Daily precipitation data for the period January 1 to February 9 1996, collected from the rainfall station at the Martin County Utility plant, were used to calculate aerial recharge. Not all of the rainfall that falls in an area becomes recharge to the aquifer. Some rainfall is intercepted by impervious surfaces (buildings, roads, etc.) or plant life, and some never reaches the ground. Of that portion of rainfall which reaches land surface, some will run off into ditches and canals and out to sea, part will be held at land surface in depressions until it evaporates, and another part will be held as storage in shallow soils.

J. Giddings, from the Lower East Planning Division at the SFWMD, created the recharge package used in this model. Two approaches were evaluated to determine recharge rates. The first approach was to use daily rates obtained from the nearby rainfall station. The second approach, which was used in this study, was to estimate the recharge rate reaching the water table. This approach was necessary because MODFLOW does not adequately simulate the unsaturated zone. In South East Florida a large percentage of the rainfall is intercepted by runoff or in the unsatured zone with only a fraction of the rainfall actually reaching the water table. The method used was developed by Jones et al., 1984.

3.2.5 Evapotranspiration

Loss of ground water due to evaporation, and transpiration from plants was represented in the model by the evapotranspiration (ET) package. MODFLOW requires the input of a maximum ET rate to each cell from which ET may occur. This rate is used when the water table in a cell equals the elevation of the land surface. No evaporation occurs when the water table declines below an assigned extinction depth. In between these two extremes the ET rate is assumed to be linear. For this model a modified ET package (Restrepo, 1989) was used which represented the decline in the rate of ET as a non-linear function, and allowed for the designation of a capillary fringe zone.

This modified code was used to more accurately reflect the natural evapotranspiration process. Even if the water table is relatively deep, evapotranspiration will not necessarily go to zero because upward transport can still occur. Water can be drawn upward by capillary action from the water table into a zone where the pores are saturated but the pressure is less than atmospheric. This zone is known as the capillary fringe zone. Within the capillary fringe moisture decreases gradually with height above the water table. The height to which the water will rise is a function of the grain size, shape, and lithology. Deep rooted plants may draw moisture from within the capillary fringe. The modified ET package includes this capillary fringe zone as a contributor of soil moisture. This model uses a capillary fringe thickness of one foot, which is within the range expected for fine to medium grain sand (Hopkins, 1991).

The ET rate was obtained in Hopkins, 1991 study and correspond to data from January and February 1988. Pan evaporation from Vero Beach, and were adjusted using stage data from a North Martin County wetland monitoring station (JMM, 1988). An extinction depth of five feet below land surface, based on root zone depths for indigenous vegetation and a one foot capillary fringe, was used throughout the modeled area. This falls within the reported range of root zone depths for South Florida (Restrepo, 1989).

3.2.6 Boundary conditions

The function of boundaries is to impose the effects of the external regional flow system on the modeled area. Several types of boundary conditions are available in MODFLOW including prescribed head and prescribed flux. Constant head boundaries where the head at the boundary remains constant for the model duration are one example of a prescribed head boundary. Prescribed flux boundaries are used when there is a flux that changes with time at the outer edges of the boundaries. No flow boundaries are a type of prescribed flux boundary where the flow across the boundary is not expected to occur. For this simulation, constant heads were used to simulate boundary conditions (Hopkins, 1991).

• For a constant head assumption, the hydraulic head within the cell is input by the user and does not vary with time. Water may flow into or out of the cell, depending on the head gradient, but head within the cell does not change. Constant head boundaries occur where part of the boundary surface of the aquifer coincides with a surface of essentially constant head (e.g. the Intracoastal Waterway).

The Intracoastal Waterway, St. Lucie Inlet, and North Fork of the St. Lucie River are represented as constant head boundaries in both layers of the model. Water levels in the reaches of the St. Lucie Inlet are strongly influenced by the tides.

Mean monthly high and low tide stages were available from SFWMD for :

- the St. Lucie Inlet at Stuart (1973-1976),
- the Intracoastal Waterway at State Road A1A bridge (9/82-3/84),
- the North Fork of the St. Lucie River at Sandpiper Bay (6/81-8/82),
- Britt Creek (5/82-8/82),
- Kellstadt Bridge (2/84-10/85).

Because the period of record at each station was short, and represented a variety of time intervals, it was decided to use an average yearly value for each section of the coastline considered. The values used were the following :

- 0.18 meter NGVD for the North Fork of the St. Lucie River,
- 0.27 meter NGVD for the Intracoastal Waterway,
- 0.1 meter NGVD for the St. Lucie Inlet.

A constant head boundary was placed at the coastline in layer one. All cells in this layer, from the coast to the edge of the grid, are designated as constant head. In layer two the aquifer was extended an additional two cell widths (144 meters) out from the coast to more closely simulate natural conditions, in which water in the deeper zone of the aquifer can flow under shallow bodies of water. The cells in layer two between the constant head boundary and the edge of the grid are inactive, or no flow cells. Figure 24 illustrates the arrangement of model boundaries. Where the modeled area is not bordered by sea water, water levels were measured and used to represent constant head boundaries in both layers. Water levels along the northern boundary were estimated based on actual water level data from monitor wells in the area.

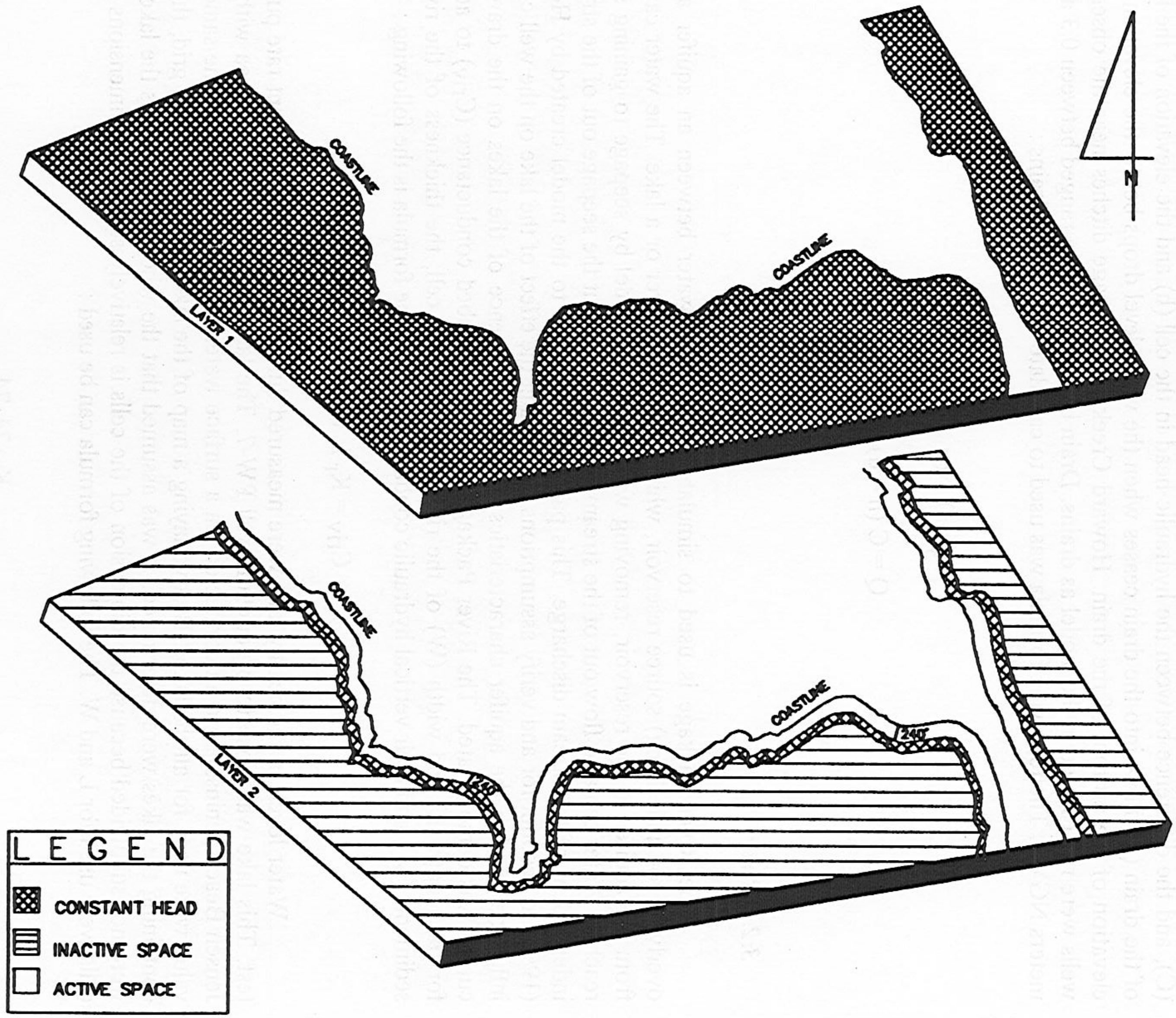


Figure 24. *Arrangement of model boundaries.*

3.2.7 Drains

The drain package simulates uni-directional flow from the aquifer to the drain. This flow occurs when simulated head in the aquifer rises above the bottom elevation of the drain. The rate of flow into the drain from any one cell (Q) is a function of the hydraulic conductance of the drain (C), and the difference between the hydraulic head in the cell (h) and the elevation of the bottom of the drain (d). Flow into the drain ceases when the water level drops below the elevation of the elevation of the bottom of the drain. Howard Creek and drainage ditches near the observation wells were represented in the model as drains. Drain bottom elevation ranged between 0.3 and 3.4 meters NGVD. The following formula was used to calculate Q from the drains :

$$Q = 0 \quad \text{for} \quad h \leq d$$

$$Q = C\,(h - d) \quad \text{for } h > d$$

3.2.8 River

The River Package is used to simulate the flow of water between an aquifer and an overlying (or underlying) source reservoir, which is usually a river or a lake. The water can flow from the aquifer to the reservoir, removing water from the model by seepage to gaining stream reaches. Water can also flow out of the stream into the aquifer but the seepage out of the stream is independent of the stream discharge. This package was added to the model created by Hopkins (1991) to account for and verify assumptions concerning the effect of the lake on the well cone of influence. During the aquifer characteristics study, the influence of the lakes on the drawdown curves was demonstrated. The River Package uses the streambed conductance (C_{riv}) to account for the length (L) and width (W) of the river channel in the cell, the thickness of the riverbed sediments (M), and their vertical hydraulic conductivity (K_r). The formula is the following :

$$C_{riv} = K_r\, L\, W / M$$

Water levels in the nearby lake were measured during and after the constant rate pumping test. This lake was close to production well PW-7. The lakes East of the study area within the Jensen Beach peninsula are all connected via a surface water management permit. The same head value were used for all the lakes. By overlaying a map of the lakes and the original grid, the cells containing the lakes were determined. It was assumed that the whole cell contains the lake, cells were not subdivided because the dimension of the cells is relatively small. The dimensions of the cells were used for L and W. The following formula can be used :

$$C_{riv} = \frac{K_r \cdot 74 \cdot 74}{M}$$

For the vertical hydraulic conductivity, a value of 0.3 m/d (one foot per day) was used and for the thickness of the riverbed sediments a value of 0.3 m was used. The value of C_{riv} used in the model was 5476 m^2/d. Flow between the river and the aquifer (Q_{riv}) is proportional to the streambed conductance (C_{riv}) and to the head difference existing between the lake reach (H_{riv}) and the aquifer directly below the lake (h). When the water table falls below the bottom of the streambed (R_{bot}), leakage stabilizes. The flow is calculated using the following formula :

$$Q_{riv} = C_{riv} (H_{riv} - h) \qquad \text{if } h > R_{bot}$$

$$Q_{riv} = C_{riv} (H_{riv} - R_{bot}) \qquad \text{if } h \leq R_{bot}$$

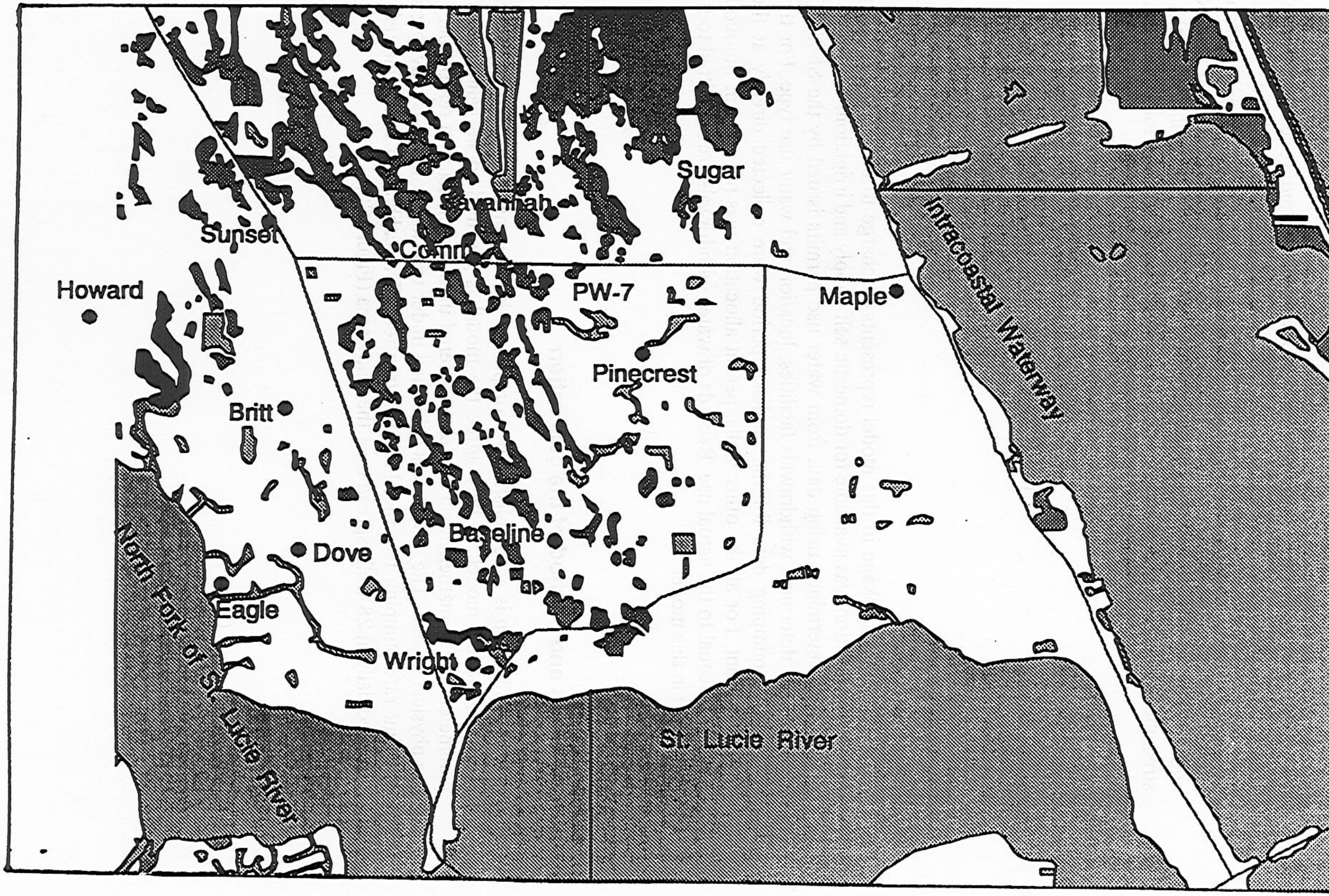
3.2.9 Well

The well package in this model represents discharge from public water supply only. Not enough information was available for domestic self-supply and irrigation. Water use figures for the model were determined using data from water use permits issued by the SFWMD. The permits were used to determine withdrawals facilities, location and water use type. For the public supply well, actual pumping data for the modeled period were collected directly at the North water treatment plant. For all the other wells, permit allocation was not used in the model, as it is not necessarily equal to the actual use. Records of water withdrawals are submitted to the District, and were used in this model.

3.3 Steps and method of the calibration

Calibration is accomplished by comparing the response of the actual physical system with the mathematical model. If they agree the model is assumed to be calibrated. If not, various parameters in the mathematical model are altered until the model is in reasonable agreement with the physical system. The model was calibrated under transient conditions only, because during the constant rate pumping test the flow was transient. The calibration was done using water levels recorded daily in 28 wells spread over the study area (Figure 25).

Figure 25. *Location of the observation wells used during the model calibration.*



The following table gives the characteristics of the monitoring wells :

Well Name	Elevation of the TOC NGVD (m)	Layer	Row	Column
Sugar A	19.04	2	37	74
Sugar B	19.61	1	37	74
Savannah	18.06	2	37	60
Sunset A	14.51	2	24	34
Sunset B	15.58	1	24	34
Commercial B	17.13	1	38	51
Commercial A	16.28	2	38	51
PZ 136 I	18.92	2	43	56
PZ 4	20.14	1	43	55
PZ 3	22.2	1	43	51
PZ 2	19.3	1	43	53
PZ 1	17.2	1	43	54
Maple A	17.11	2	61	88
Howard C	5.06	2	25	12
Howard A	5.05	2	25	12
Howard B	5.09	1	25	12
Pinecrest A	12.42	2	55	62
Pinecrest B	12.05	2	55	61
Britt A	12.99	2	43	26
Britt B	12.86	1	43	26
Baseline A	12.33	2	68	44
Baseline B	12.16	1	68	44
Dove B	6.99	1	56	20
Dove A	7.09	2	56	20
Eagle A	5.34	2	56	12
Eagle B	5.42	1	56	12
Wright C	5.95	2	75	31
Wright A	6.1	2	75	31

Table 31. *Characteristics of the observation wells used to calibrate the model.*

3.3.1 Sensitivity analysis

The calibration process started with a sensitivity analysis of individual parameters. The purpose of a sensitivity analysis was to test the effect of changes in parameter values by changing one parameter value at a time. The parameters that generate the most important changes are used to calibrate the model.

The sensitivity analysis was done by modifying the following parameters :

- VCONT,
- Transmissivity of layer 2,
- Specific yield of layer 1,
- Storativity of layer 2,
- Specific yield of layer 2,
- Recharge,
- ET,
- Pumping.

The sensitivity analysis began with a base run. One parameter was modified at a time the model was rerun and the water level variations in each observation well were noted. After each run of the model, the calculated and observed in the observation wells were plotted using a computer graphics program called Precision Visual Wave (PVWAVE). This program performed a statistical analysis of the data and calculated the average absolute error, the standard error and the extreme errors.

3.3.2 Calibration of the model

The intent was to calibrate the model so that agreement between observed water levels in monitoring wells and simulated water levels were within 0.3 meter (one foot) of each other. This correspond to have extreme errors less than 0.3 meter. This criteria is classically chosen by modeled at the District.

The calibration was done with the idea of simulating the pumping tests results. The modification consisted of :

- changing the value of the multiplier in the Block Centered Flow package (BCF). This affected the entire model.

- changing the value of the parameter in particular cells.

3.4 Calibration results

3.4.1 Results of the sensitivity analysis

The results of the sensitivity analysis are presented in Table 32 and 33. A first intent of calibration was done without the river package and the model was not sensitive to changes of specific yield of layer 1. Then, the river package was added and the model became sensitive to specific yield of layer 1. This proved that lakes have an influence on groundwater level and confirmed the remarks done about lakes in the aquifer analysis.

• Wells situated in layer one :

Parameter modified	Modification	Max. increase of the simulated water levels (m)	Max. decrease of the simulated water levels (m)
Transmissivity of layer 2	Multiplied by 10	0.06	0.76
Transmissivity of layer 2	Divided by 10	0.025	0.06
Storage coefficient of layer 2	Multiplied by 10	0.03	0.03
Storage coefficient of layer 2	Divided by 100	0.03	0.03
Specific yield of layer 2	Multiplied by 10	0.03	0.03
Specific yield of layer 2	Divided by 100	0.03	0.03
Specific yield of layer 1	Multiplied by 2	0.3	0.0
Specific yield of layer 1	Divided by 2	0.0	0.24
VCONT	Multiplied by 0.02	0.09	0.0
VCONT	Multiplied by 2	0.0	0.06
Evapotranspiration	Equal to zero	0.15	0.0
Recharge	Equal to zero	0.0	0.09
Pumping	Equal to zero	0.09	0.0

Table 32 : *Fluctuations of the water levels in the observation wells of layer 1 during the sensitivity analysis.*

• Wells situated in layer two :

Parameter modified	Modification	Max. increase of the simulated water levels (m)	Max. decrease of the simulated water levels (m)
Transmissivity of layer 2	Multiplied by 10	0.06	1.13
Transmissivity of layer 2	Divided by 10	0.21	0.12
Storage coefficient of layer 2	Multiplied by 10	0.03	0.03
Storage coefficient of layer 2	Divided by 100	0.03	0.03
Specific yield of layer 2	Multiplied by 10	0.03	0.03
Specific yield of layer 2	Divided by 100	0.03	0.03
Specific yield of layer 1	Multiplied by 2	0.46	0.0
Specific yield of layer 1	Divided by 2	0.0	0.36
VCONT	Multiplied by 0.02	0.12	4.85
VCONT	Multiplied by 2	0.07	0.03
Evapotranspiration	Equal to zero	0.15	0.0
Recharge	Equal to zero	0.0	0.09
Pumping	Equal to zero	0.18	0.0

Table 33. *Fluctuations of the water levels in the observation wells of layer 2 during the sensitivity analysis.*

The three most sensitive parameters are the transmissivitty of layer 2, the leakage term VCONT and the specific yield of layer 1.

3.4.2 Results of the calibration

Calibration of the model was done by modifying the transmissivity of layer 2, the VCONT and the specific yield of layer 1. The value of each parameter used in the calibrated model at each observation well site are presented in the following table :

The main changes were done on the VCONT. For simulations conducted by Hopkins (1991), a multiplier of 0.25 and a value in the cell ranging from 0.02 to 0.05 were used. This corresponds to a vertical hydraulic conductivity of the confined unit of 0.0015 to 0.039 m/d (using an estimated thickness of 3 meters for the semi-confining unit).

Table 34 presents values for the parameters used in this model in the area of the observation wells. The hydraulic conductivity K' of the semi-confining unit has been calculated based on the values of VCONT input to the model and using a thickness of three meters for the semi-confined unit for all the wells.

Well Name	Transmissivity of layer 2 m^2/d	Vertical hydraulic conductivity K' m/d	Specific yield of layer 1
Sugar A and B	349	0.0024	0.2
Savannah	311	0.0268	0.2
Sunset A and B	316	0.005	0.2
Comm. A and B	316	0.0006	0.05
PZ 136 I	302	0.0015	0.6
PZ 4	302	0.0015	0.6
PZ 3	295	0.0015	0.5
PZ 2	298	0.0015	0.7
PZ 1	304	0.0015	0.6
Maple A	544	0.002	0.2
Howard A, B and C	214	0.003	0.2
Pinecrest A and B	200	0.0006	0.2
Britt A and B	511	0.0012	0.1
Baseline A and B	390	0.002	0.2
Dove A and B	302	0.002	0.2
Eagle A and B	311	0.002	0.2
Wright A and C	423	0.002	0.2

Table 34. *Results of the aquifer parameters obtained during the calibration of the model.*

For the others parameters, the following values were used :

- A specific yield of layer 2 of 0.3, constant over all the model.
- A storage coefficient of layer 2 of 0.0004 was held constant over all the model.
- A hydraulic conductivity of layer 1 equal to 3 meters per day (10 ft/d).

Listed below are comments regarding the preliminary re-calibration of Hopkin's (1991) transient ground water model.

• Several wells were already calibrated before the first run and were relatively insensitive to the modification of parameters. Those wells are Howard A, B, C, Dove A, B, Eagle A, B and Wright A, C.

• Several wells were calibrated after modifying slightly the value of VCONT or the transmissivity of layer 2. Those wells are Baseline B and Britt B.

• All the other wells required a significant amount of time and number of runs to achieve calibration. Over the total of the 28, observation wells 85% met the imposed criteria of ± 0.3 meter (1 foot) for the calibration process.

The hydrographs representing the last model simulation are presented in Appendix 22.

3.4.3 Discussion

Values of the aquifer parameters used during the calibration were relatively close from the results of the pumping tests and the slug tests. This allowed us to verify that the aquifer tests results were correct. The specific yield in layer 1 in the pumping well PW-7 area and the piezometers PZ 1, 2, 3 and 4 are relatively high. This can be justified by the fact that those wells are in wetlands or relatively close to the wetland and the water table was near or at ground surface.

The difficulties encountered during the calibration of this model can be explained by several factors :

• **Change of the stress period duration**. Hopkins (1991), model was based on a monthly stress period and this model was done using a daily stress period.

• **Discretization of the model**. The model was created using two layers, but this discretization did not take into account the presence of the hardpan and zones of lower permeabilities. Even if the continuity of this horizon was established, it would have probably facilitated the calibration of the piezometers PZ 1, 2, 3 and 4. During the set-up of the piezometers, the hardpan was identified at a depth of 2 to 3 feet below the ground and with a thickness ranging from 1 to 2 feet.

• **Storage coefficient and the specific yield of layer 1.** These coefficients were introduced as constants over all the model. This assumption did not take into account the heterogeneity of the surficial sand.

• **Boundary conditions.** According to a study done by the U.S. Geological Survey in 1987 about the effects of boundary conditions (Franke et al.), it has been showed that the most critical aspect of describing ground-water systems for purposes of simulation is the specification of appropriate boundary conditions. The boundary package created by Hopkins (1991) utilized an average yearly value for each coastline section. This method may not be the best one to simulate the tidally influenced ground-water system.

• **The wetlands were not simulated**. Even though this area contains numerous wetlands, the model does not simulate the unsaturated portions of the Surficial Aquifer system. Thus water levels within the wetland can only be estimated or supported by actual field data.

• **Accuracy of the pumping data**. Domestic self-supplied wells in the study area have not been taken into account in the well package. This can be explained by the fact that some people drill wells and are not required to obtain permits under state law. Another problem encountered was the accuracy of the pumping data recorded by the water treatment facilities. The meter used to record the water pumped from PW-7 was not very reliable. During the constant rate pumping test, the meter placed in the straight section of the pump discharge pipe broke down and we had to "bump it" (as per instruction from the water treatment plant operators) with a wrench to make it work.

• **The accuracy of the survey**. The piezometers PZ 1, 2, 3 and 4 were surveyed during this study. As in typical surveying methods, the survey loop was shot on the outward bound from the benchmark. Upon returning from the farthest point and closing the loop, the data indicated a small discrepancy between from the initial reference point. The difference showed was of 0.1 meter (0.3 feet). Survey inaccuracy could be a problem encountered during the calibration process. This was a limitation of the availability of quality survey equipment during this study.

The previous remarks provides some possible explanations why the wells that did not meet the calibration criteria as described below.

• For Commercial B, it is suspected that the original surveyed elevations were not accurate. A difference of almost 0.1 m is shown between the elevation of the top of casing of Commercial A and Commercial B. In the field, the two wells are at the same level.

• For the two Sugar wells, it is suspected that the presence of the large wetland located in the Savannah State park has a buffering control on groundwater and surface water levels within the study area. At the time of the pumping test the wetland had water standing in it.

• For PZ 3, the difference between the simulated and observed water levels may be due to the fact that the vertical discretization did not take into account the hardpan or layers of lower permeability. PZ3 is situated in a wetland. Even though the wetland was dry at the time of the test, the heterogeneity of the aquifer may explain that difference.

3.4.4 Groundwater model conclusion

In conclusion the MODFLOW groundwater model needs to be improved before it is used to simulate impacts to wetlands. The model created by Hopkins (1991), has never been calibrated under steady state conditions. It did not affect this study because the flow of the ground water during the constant rate pumping test was transient. This factor must be taken into account for the further development and improvement of the model. The following recommendations are suggested to improve the weaknesses of the model as pointed out in the previous paragraph.

• improve the vertical and horizontal discretization of the model with more field work and data collection. Create a multiple layer model which emulates local lithology more closely. It will necessitate additional geological investigation within the study area.

• collect tidal stage information for the boundary conditions during the pumping test,

• include the wetlands in all future simulations,

• improve the river package that was created by monitoring the surface water levels in the surrounding lakes.

• work with the water utilities to improve meters problems and obtain information about all the wells that are pumping water adjacent to the site. Improve the accuracy of the pumping estimates for non-reporting users.

• Simulate the heterogeneity of the aquifer in layer 1 by using non-constant values for the aquifer parameters over the study area.

CONCLUSION AND PROSPECTIVES

Pumping and slug tests were conducted in order to establish characteristic aquifer parameters of the Surficial Aquifer system. This study has demonstrated the effects of a lake situated at 250 meters from the pumping well and on the groundwater levels under the nearby wetlands. The water levels in the lakes were lower than the groundwater in the nearby wetlands during the dry season so the groundwater is draining through the substrate to the lake.

Modeling efforts confirmed the local influence of the lakes on groundwater levels within the wetlands. It was also demonstrated that the pre-existing model did not simulate correctly the hydrodynamic conditions in the Surficial Aquifer system, especially in the wetland near PW-7. The groundwater model must be improved before doing future simulations in order to represent different pumping scenarios and to establish a pumping rate allowing the county to meet the existing District drawdown criteria : "One-foot of drawdown over a one month period with 90 days with no-recharge". This study has pointed out weaknesses of the model and given recommendations for the future evolution of the model.

The prospectives of this study are :

- for the short-term period, improve the model conceptualization by modifying the horizontal and vertical discretization and by improving the river package, the boundary package and the well package.

- for the long-term period, this study provided guidelines that can be used to direct future field investigations of the isolated wetland program conducted at the District.

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Appendix 15 Lake drawdown simulation

Appendix 16 Supporting information for the Hantush inflection-point method for the deep aquifer during the constant rate pumping test

Appendix 17 Plot of the distance-drawdown analysis of the deep aquifer for the constant rate pumping test

Appendix 18 Log-log and semi-log plots of the time-drawdown relationships for the intermediate aquifer during the constant rate pumping test

Appendix 19 Hydrographs of the monitor wells during the step-drawdown pumping test

Appendix 20 Hantush-Jacob plots for the deep aquifer during the step-drawdown pumping test

Appendix 21 Supporting information for the slug test conducted in the intermediate wells

Appendix 22 Model calibration hydrographs

APPENDIX 1

LIST OF THE PLANTS FOUND IN THE JENSEN BEACH WETLANDS

SCIENTIFIC NAME	COMMON NAME	Herb	Shrub	Tre
Amphicarpum muhlenbergianum	Little Blue Maidencane; Perennial Goobergrass	X		
Andropogon virginicus var. virginicus	Broomsedge	X		
Aristida beyrichiana	Wiregrass; Pineland Three-awn Grass	X		
Aristida spiciformis	Bottlebrush or Pinebarren Threeawn			
Bigelowia nudata subsp. australis	South Florida Rayless-goldenrod	X		
Blechnum serrulatum	Toothed Mid-sorus Fern			
Carex	Sedge			
Cyperus haspan	Sheathed Flatsedge	X		
Dichanthelium ensifolium var. unciphyllum	Panic Grass	X		
Drosera capillaris	Pink Sundew	X		
Eriocaulon decangulare	Ten-angle Pipewort; bog buttons	X		
Fuirena scirpoidea	Southern Umbrella-sedge	X		
Hypericum fasciculatum	St. John's-wort		X	
Hypoxis juncea	Fringed Yellow/Common Stargrass	X		
Ilex cassine	Dahoon Holly; Dahoon		X	X
Ilex glabra	Inkberry; Gallberry		X	X
Juncus sp.		X		
Lachnanthes caroliniana	Bloodroot; Carolina Redroot	X		
Lachnocaulon anceps	White-head Bog-buttons	X		
Lachnocaulon beyrichianum	Southern Bog-button	X		
Ludwigia	Primrose Willow		X	X
Lyonia ferruuginea	Rusty Lyonia		X	
Lyonia lucida	Fetterbush; Shinyleaf		X	
Melaleuca quinquenervia	Punk Tree; Cajeput; Melaleuca			X
Myrica cerifera	Wax Myrtle; Southern Bayberry		X	X
Myriophyllum pinnatum	mermaid weed	X		
Nymphaea odorata	White or Fragrant Waterlily; floating lily pad	X		
Nymphoides cordata	Little Floating-heart	X		
Panicum hemitomon	Maidencane	X		
Panicum sp.		X		
Pinus elliottii var. densa	South Florida Slash Pine			X
Pluchea rosea	Godfrey's Fleabane; Rosy Camphor-weed	X		
Polygala rugelii	Yellow Batchelor's Button; Yellow Milkwort	X		
Pontederia cordata	Pickerelweed	X		
Pteridium aquilinum	Bracken Fern	X		
Quercus pumila	Runner Oak		X	
Rynchospora sp.	Beak Rush	X		
Sabatia grandiflora	Large-flower Rose-gentian	X		
Schinus terebinthifolius	Brazilian Pepper; Brazilian Holly		X	X
Serenoa repens	Saw Palmetto		X	
Sphagnum	Sphagnum moss	X		
Spiranthes longilabris	Thanksgiving Ladies'-Tresses	X		
Typha sp.	Cattail	X		
Vaccinium myrsinites	Shiny Blueberry		X	
Vitis				
Woodwardia virginica	Virginia Chain Fern	X		
Xyris sp.	Small's Yellow-eyed Grass	X		
•	Orange Vine	X		
•	Hat Pins	X		

APPENDIX 2

WATER USE DATA FOR A 18 MONTH PERIOD
NORTH MARTIN COUNTY WATER SYSTEM

MONTH/YEAR	FLORIDAN WELLS RAW WATER PUMPAGE			SURFICIAL WELLS RAW WATER PUMPAGE			TOTAL WATER PUMPED (MGM)	TOTAL WATER TREATED (MGM)
	AVGERAGE DAY (MGD)	MAXIMUM DAY (MGD)	TOTAL PUMPAGE (MGM)	AVGERAGE DAY (MGD)	MAXIMUM DAY (MGD)	TOTAL PUMPAGE (MGM)		
JAN 1993				2.234	2.583	69.255	69.255	64.200
FEB 1993				2.321	2.799	64.998	64.998	59.292
MARCH 1993				2.549	2.871	76.225	76.225	68.744
APRIL 1993				2.382	2.770	73.839	73.839	70.548
MAY 1993				2.697	3.150	83.605	83.605	78.343
JUNE 1993				2.194	2.898	65.823	65.823	66.178
JULY 1993				2.369	2.900	73.451	73.451	73.857
AUG 1993 *	0.960	1.728	21.124	1.863	2.882	57.751	78.875	73.359
SEPT 1993	0.999	1.341	20.972	1.560	2.229	46.786	67.758	64.521
OCT 1993	0.825	1.285	12.375	1.515	2.278	46.971	59.346	61.677
NOV 1993	0.883	1.341	15.897	1.650	2.300	49.505	65.402	69.672
DEC 1993	0.865	1.322	17.305	1.714	2.444	53.144	70.449	70.640
JAN 1994	0.885	1.244	13.269	1.721	2.223	53.352	66.621	67.087
FEB 1994	1.482	2.007	20.750	1.930	2.489	34.746	55.496	58.500
MARCH 1994	0.917	1.367	9.168	2.051	2.510	63.569	72.737	70.701
APRIL 1994	0.156	1.649	4.678	2.101	2.800	63.020	67.698	69.217
MAY 1994	0.489	1.067	6.848	2.140	2.763	66.321	73.169	72.999
JUNE 1994	0.681	1.475	12.944	1.753	2.334	52.602	65.546	64.323
TOTAL	9.142	15.826	155.330	36.744	47.223	1094.963	1250.293	1223.858
AVERAGE	0.831	1.439	14.121	2.041	2.624	60.831	69.461	67.992

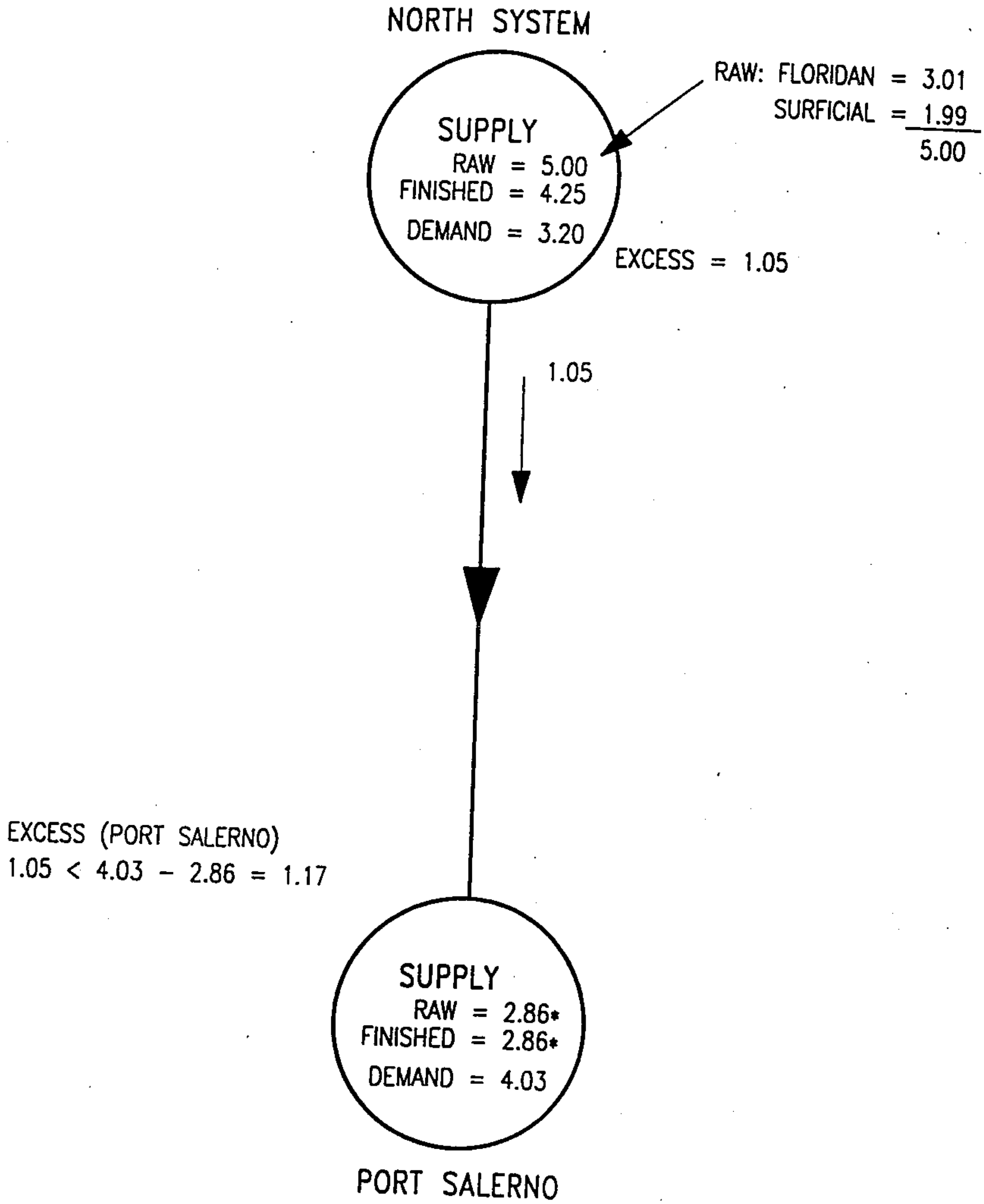
* REVERSE OSMOSIS PLANT START-UP

RATIO OF WATER PUMPED TO WATER TREATED WAS 1.03

MAXIMUM DAY PUMPAGE WAS 2.79 MGD AND OCCURRED IN MAY 1993

APPENDIX 3

PROJECTED WATER USE FOR MARTIN COUNTY UTILITIES YEAR 2004 AVERAGE DAILY FLOW (MGD)



* BASED ON 1997 CONSUMPTIVE USE

APPENDIX 4

WELL COMPLETION REPORT OF PW-7

DEPARTMENT OF ENVIRONMENTAL REGULATION

WELL COMPLETION REPORT

OWNER: MARTIN CO
Last Name / First Name / Initial

50 KINDER ST
Number / Street

STUART FL
City / State

305 283 6760
Area Code / Phone Number / Zip Code

WELL LOCATION:
___ ¼ ___ ¼ ___ ¼ of Section [ ]
[ ] (N-S) [ ] (E-W)
Township / Range
Latitude [ ] Deg. [ ] Min. [ ] Sec. N
Longitude [ ] Deg. [ ] Min. [ ] Sec. W
Locate in Section

COMMERCIAL STREET
Number / Street/Road

Lot No. / Subdivision

JENSON BEACH MARTIN
City / County

OWNER WELL NUMBER OR NAME: PW#7

DRILL METHOD: [X] Rotary [ ] Cable Tool [ ] Jet [ ] Auger
[ ] Other:

SURFACE CASING, CASING, AND LINER MATERIAL:

Steel Dia. (In.) Black	Galv.	S. Steel	Steel Wt. (lb/ft)	Dia. (In.) PVC	Other[†]	From (Ft.)	To (Ft.)	Schedule No.	Joints*
16	in					0	70		
8	in					0	70		

Describe Material:
*TC = Threaded and Coupled, TCW = Threaded, Coupled, and Welded, W = Welded, B = Bonded (PVC), O = Other:

GROUT: [ ] None [X] Neat Cement [ ] Other:
Type and Percent of Additives and Grout Volume or Number of 94 lb. Sacks

	From (Ft.)	To (Ft.)
	70	LS

FINISH: [ ] Open Hole [ ] Perforated or Slotted Casing [ ] Gravel Pack
[ ] Sandpoint or Screen Attached to Well Casing [ ] Sandpoint or Screen Telescoped with Packer Inside Casing (Packer Material:)

Sandpoint/Screen Material	Dia. (In.)	Slot Size (In.)	From (Ft.)	To (Ft.)
		.040		

[ ] Other Finish:

QUALITY TEST: [ ] None [X] Bacteria 5-83 Date [X] Chemical 5/23/83 Date
By: [ ] Health Dept. [ ] USGS [ ] Other
[ ] Clear [ ] Colored [ ] Sulphur [ ] Salty [ ] Iron [ ] Other
Conductance (Micromhos) [ ] Chloride [48] ppm
Hardness [264] ppm as calcium carbonate pH [7.9] Temp [ ] °F
Well Disinfected: [ ] No [X] Yes 5-83 (Date)

WELL TEST, by: [ ] Natural Flow [ ] G.P.M. [ ] Airlift
[ ] Bailer [ ] Permanent Pump [X] Test Pump [ ] None
Discharge Measured By: [ ] Bailer [X] Estimated [ ] Current Meter
[ ] Orifice [ ] Trajectory [ ] Venturi [ ] Volumetric [ ] Other
Measured Static Water Level [ ] + [ ] - [5.48] Ft
Measured Pumping Water Level [ ] + [ ] - [46] Ft
After [ ] Hours At [ ] G.P.M.

[ ] Deepening [ ] Plugging
[ ] Other:

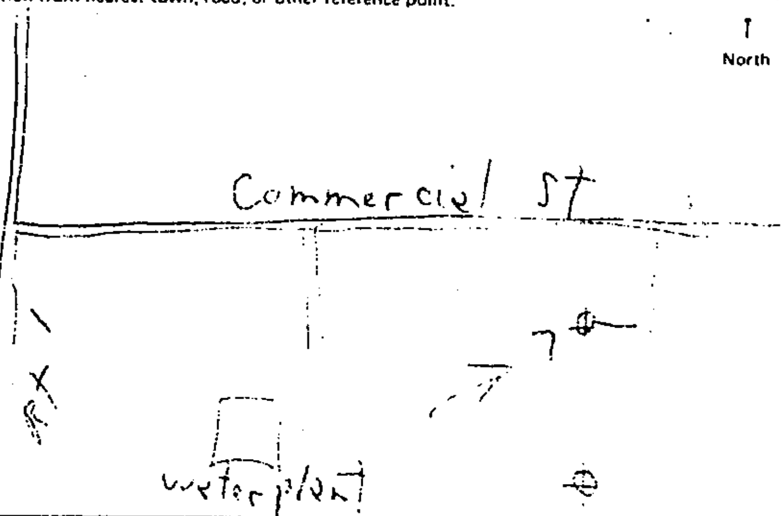
WW43 57659

WELL NUMBER
7

TYPE OF WELL: [X] Water Well [ ] Test Well [ ] Recharge [ ] Drainage
[ ] Waste Disposal [ ] Observation [ ] Other

USE: [ ] Domestic [ ] Irrigation [ ] Industrial [ ] Livestock [X] Public Supply
[ ] Other:

SKETCH LOCATION OF WELL in relation to local landmarks, giving distance and direction from nearest town, road, or other reference point.



GEOPHYSICAL LOGS: Type: By:

WELL LOG

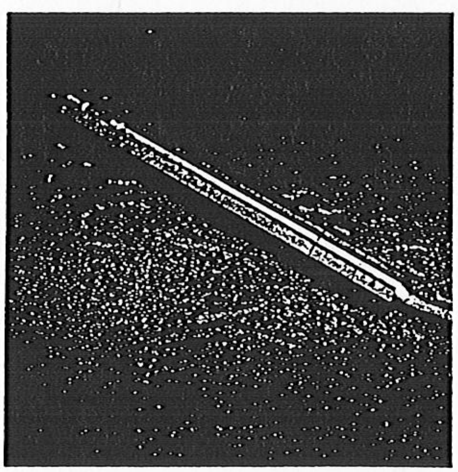
Bore Hole (In.)	Casing Size (In.)	Depth (Ft.) From	To	Examine cuttings at 20 ft. or smaller intervals and at changes. Give color, grain-size and type of material. Note any cavities. Indicate producing zones. Attach additional sheets if necessary.
		0	10	SAND, v. fine - Med., silty, tr. clay + organics, dk. br.
		10	30	SAND, v.f., silty (silica) lt. gray
		30	40	CLAY, silty, dk. br. w/organic tr. clay
		40	50	SAND, v.f. - f, silty, med. gray loose
		50	65	SAND, v.f. - Med., silty, w/shell fragments, gray
		65	83	Shell Fs., weathered calcarenite loose, interbedded w/clay, lt. gray, trace sand, v.f.
		83	112	SAND, v.f. - Med., silty, gray
		112	118	SAND, v.f. - coarse, w/shell, sand gray silt, trace.
		118	136	SAND, f - Med., + shell F., f., tr. silt, tr. weathered calcarenite, gr.
		136	138	Calcarenite, hard, gray
		138	150	SAND, v. fine - f., tr. shell + silt loose, gray. + tr. calcar. thinly bedded

APPENDIX 5

PXD-260 PRESSURE TRANSDUCER SPECIFICATIONS

PXD-260 PRESSURE TRANSDUCER SPECIFICATIONS

General	
Transduction principle:	Integrated silicon strain gauge bridge
Wetted Materials:	316 stainless steel, Viton*
Size:	1 in. (25.4 mm) diameter, 10.9 in. (277 mm) long
Weight:	1.5 lb. (0.68 kg)
Output:	4-20 mA (typical) over pressure range
Ranges	
Standard:	5 PSIG (12 ft. water, 34.5 kPa) 10 PSIG (23 ft. water, 68.9 kPa) 15 PSIG (35 ft. water, 103.4 kPa) 20 PSIG (46 ft. water, 137.9 kPa) 30 PSIG (69 ft. water, 206.8 kPa) 50 PSIG (115 ft. water, 344.7 kPa) 100 PSIG (231 ft. water, 689.5 kPa) 250 PSIG (578 ft. water, 1723.7 kPa)
Special:	Contact In-Situ Inc. for available ranges
Over pressure tolerance:	2x full range
Accuracy	
At reference temperature (15°C, 59°F):	±0.15% of range-linear coefficients ±0.05% of range-quadratic coefficients
Over other temperatures (quadratic coefficients):	10°C to 20°C (50°F to 68°F), ±0.08% of range 5°C to 25°C (41°F to 77°F), ±0.16% of range 0°C to 30°C (32°F to 86°F), ±0.30% of range
Cable	
Wetted materials:	Polyurethane, Teflon*
Size:	0.26" (6.7mm) OD nominal
Weight	
Polyurethane:	3.01 lb./100 ft. (1.35 kg/30 m)
Teflon:	3.50 lb./100 ft. (1.58 kg/30 m)
Maximum length:	4500 ft. (2027 m)
Reels:	ABS plastic, up to 350 ft. (107m) capacity (standard) Small steel, up to 550 ft. (168 m) capacity Large steel, up to 1500 ft. (450 m) capacity
Temperature Range	
Operating:	-14°C to 80°C (7°F to 176°F)
Storage:	-40°C to 125°C (-40°F to 257°F)



*Viton and Teflon are registered trademarks of E.I. DuPont de Nemours Co.
Due to continuing product development this information is subject to change without notice.

In-Situ Inc.

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APPENDIX 6

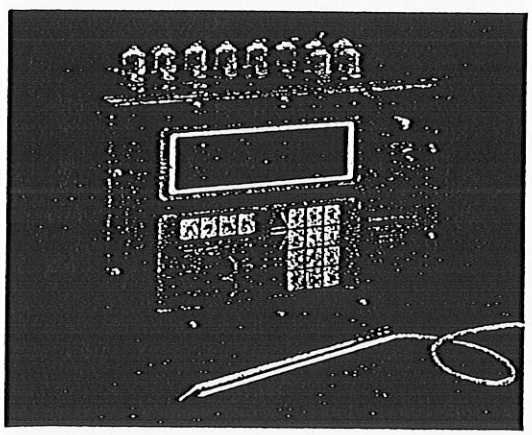
HERMIT SE2000 SPECIFICATIONS

FEATURES:

- Up to 16 channels
- Long-life lithium battery for dependable, low-cost power over a wide temperature range or lead acid option available
- Non-volatile memory, expandable to 512K
- Large LCD and simple keyboard for easy menu-driven programming
- Delayed start capability for synchronizing multi-well tests and for collecting data in remote areas accessible only part of the year
- Pre-programmed logarithmic sampling schedules or user-defined linear sampling schedules
- Data recorded in user-selectable English or SI units
- User-programmable reference level, eliminating the need to adjust the data later on
- Internal real-time clock
- RS232 interface for transferring, printing, or plotting data in the field or office
- Portable construction; rugged, weathertight, water-resistant case does not require special housing
- Programmable HI/LO alarm to electronically signal an alarm condition

RENTALS & MAINTENANCE AGREEMENTS

The HERMIT 2000 can be rented. Maintenance plans are also available. Contact In-Situ Inc. for details and availability.



In-Situ Inc.

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HERMIT SE2000 SPECIFICATIONS:

General

Dimensions:	10" x 16" x 11" (25.4 x 40.6 x 28 cm)
Weight with:	
Lithium Battery:	20 lb. (10 kg)
Lead Acid Battery:	23.6 lb. (10.7 kg)
Operating & Storage Temperature with:	
Lithium Battery:	-40°C to 70°C (-40°F to 158°F)
Lead Acid Battery:	-15°C to 40°C (5°C to 104°F)
Accuracy:	±0.06% of FS (at constant temp.) ±0.2% of FS (includes temp. effects)
Resolution:	±0.015% of FS
Stability:	±0.002% of FS/°C
LCD:	2.1" x 7.8" (5.3 x 19.8 cm); 2 lines of 20 characters each; display back-lighte

Battery

Type:	Lithium, Lead acid
Expected Life:	
Lithium:	Greater than 100,000 data points
Lead Acid:	Dependent on usage & temperature

External Power Input

Input Voltage:	+12 VDC to +18 VDC (+13.8 VDC nom.
Input Current:	50 mA typical, 500 mA peak

Alarm Contacts

Contact Voltage:	30 VDC or 30 VAC max
Contact Current:	1 ampere max

RS232C Interface

Output Voltage Swing:	±4 VDC min, ±5 VDC typical
Handshake Input Voltage:	±15 VDC max, ±3 VDC min
Baud Rate:	Selectable 300 to 9600 baud
Character Length:	Selectable 7 or 8 bits
Parity:	Selectable Odd, Even, or None
End-of-Line Sequence:	Selectable CR or CR/LF
Hardware Handshake:	CTS or XON/XOFF (DC1/DC3)

Data Sampling

Memory Type:	Non-volatile EEPROM
Memory Capacity:	64K standard, expandable to 512K
Data Point Capacity:	32,000 standard, expandable to 256,00
Sampling Options:	Linear, log, or user-defined
Linear Sampling Rates:	
Mode 1:	1 minute to 24 hours
Mode 2:	2 to 59 seconds

Logarithmic Sampling Rates:

Elapsed Time	Sample Interval	Points/Cycle
0-20 sec	0.5 sec	41
20-60 sec	1 sec	40
60-600 sec	12 sec	45
10-100 min	2 min	45
100-1000 min	20 min	45
1000-10,000 min	200 min	45
>10,000 min	1440 min	—

APPENDIX 7

CONDUCTIVITY PROBE MODEL CTS-200 SPECIFICATIONS

Conductivity Probe Model CTS-200

Specifications:

General:

Length:	20.7 inches
Diameter:	1.3 inches
Wetted Materials:	Acetal/Delrin, Viton, 316 Stainless Steel

Range:

Conductivity:	0 to 20,000 µS/cm *Check with your sales consultant for other available ranges.*
Temperature:	0 to 40°C
Pressure Rating:	100 psi @ 25°C

Accuracy:

Accuracy (Temperature 0 to 40°C):	±1°C
Linearity (FS Output at 25°C):	±2.0%

Power:

Power Supply Voltage:	9-24 VDC
Output:	Dual 4-20 mA loops
Warm-up:	Conductivity, 15 sec Temperature, 100 mSec
Storage Temperature:	-40° to 70°C
Operating Temperature:	0 to 50°C

Due to continuing product development this information is subject to change without notice.

In-Situ Inc.
The Solutions People

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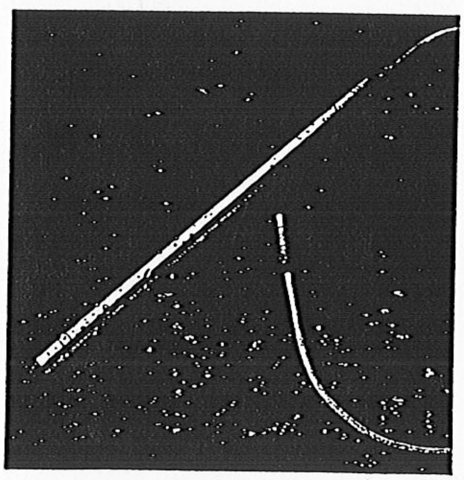
FAX: (307) 721-7598

3/95

APPENDIX 8

TROLL SP 4000 SPECIFICATIONS

- Integral water level sensor available in 15, 30, or 75 psi (100, 200, or 500 kPa); available either vented to atmosphere (i.e., "gauge-type") or non-vented (i.e., "absolute-type")
- Water level sensor can withstand up to 2X overpressuring without damage
- Available with RS485 or RS232 communications protocol; RS485 is an ideal choice where multiple TROLLs are installed in a well field and connected to a separately-supplied telemetry system, or to a computer via a separately-supplied RS485 ↔ RS232 converter box and cable. RS232 can hook directly to a PC without the use of a converter box, but is not telemetry-compatible.
- Full system accuracy ±.05% FS.
- Automatic temperature compensation of water level readings
- 14-bit a/d converter



- 316 stainless steel tube with end-mounted integral pressure sensor
- Probe measures 20 inches long; 1.5 inches outside diameter; well-top system can mount in 2-inch wells or larger

RENTALS & MAINTENANCE AGREEMENTS

The TROLL can be rented. Maintenance plans are also available. Contact In-Situ Inc. for details and availability.

TEST TYPES:

Linear: Similar to the linear test from previous In-Situ instruments. From one every three seconds up to one measurement every year.

Logarithmic: This test type incorporates true logarithmically-defined decaying-rate sampling at 40 measurements per logarithmic decade. The logarithmically-defined sampling begins at 6 seconds into the test, and the sampling rate continues to decrease until a) the user stops the test, or b) a user-defined condition is achieved, or c) a default condition is achieved, whereupon the test becomes linear. The first six seconds of the test are linearly-sampled at 5 measurements per second.

Event Sampling: This test type lets you design a test whereby small and essentially insignificant changes in transducer measurements are not stored into the data file, but larger and more significant changes measured by the transducer are stored. This acts to minimize the size of a data file but stores all meaningful data.

TROLL SP 4000 SPECIFICATIONS

General

Dimensions:	1.5" x 20" (3.8 x 50 cm)
Weight:	7 lbs. (14 kg)
Operating Temperature:	0 to 30°C (32 to 86°F)
Storage Temperature:	-40 to 70°C (-40 to 158°F)
Accuracy	
Pressure Sensor:	±0.03% of FS at 15°C ±0.05% (throughout entire operating temp. range)
Temp. Sensor:	±0.1°C
Resolution:	0.006% of FS

Data Sampling

Memory Type:	Flash EPROM
Data Capacity:	208 kb
Data Point Capacity:	up to 100,000

External Power Input

Input Voltage:	+12 to +18VDC
Input Current:	20 mA typical, 100 mA peak

Due to continuing product development this product is subject to change without notice.

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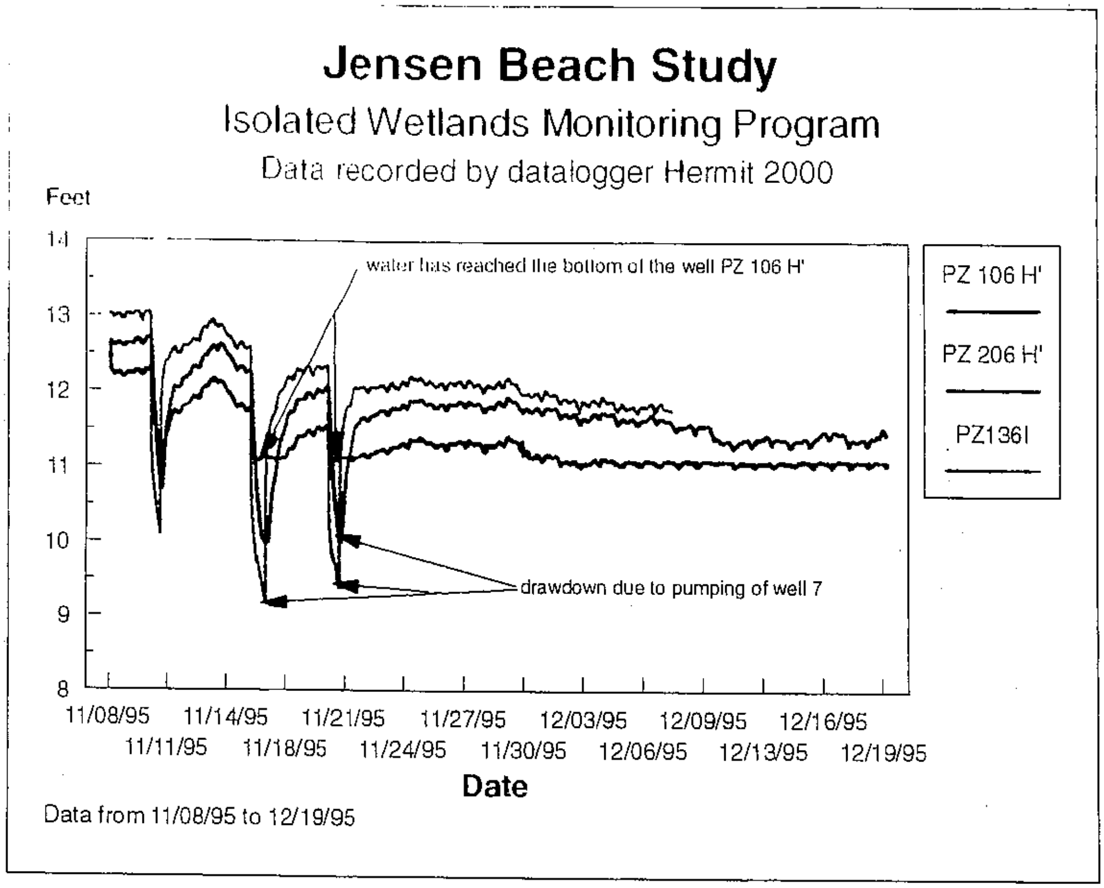
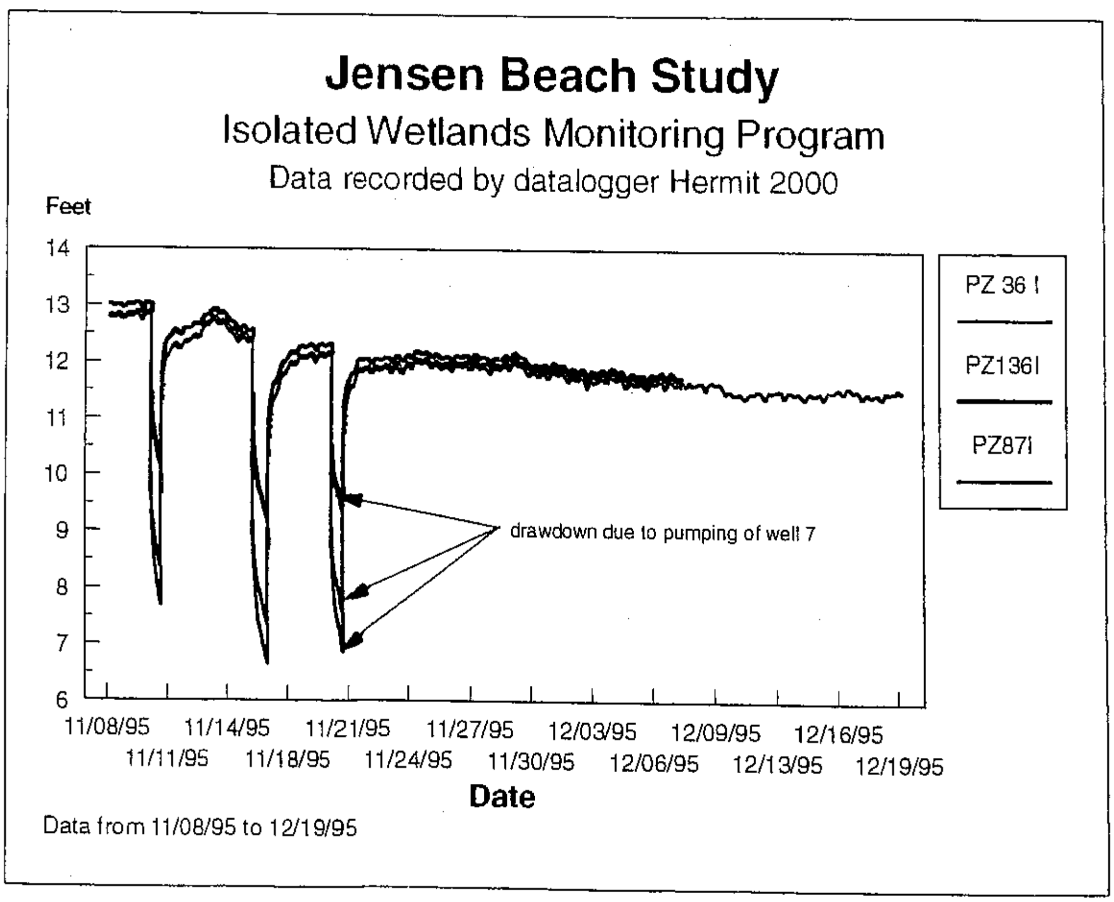
APPENDIX 9

HYDROGRAPHS OF THE MONITOR WELLS DURING THE OBSERVATION PERIOD

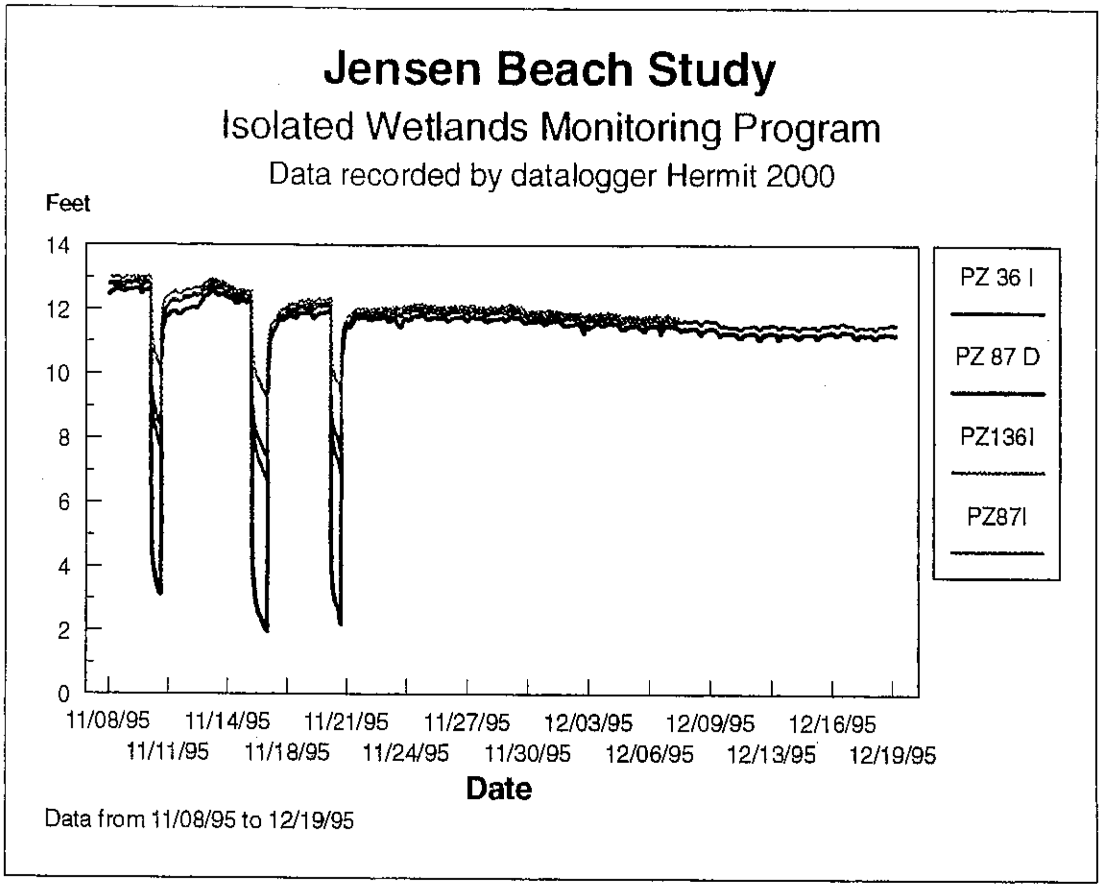
NOVEMBER 8, 1995 TO DECEMBER 19, 1995



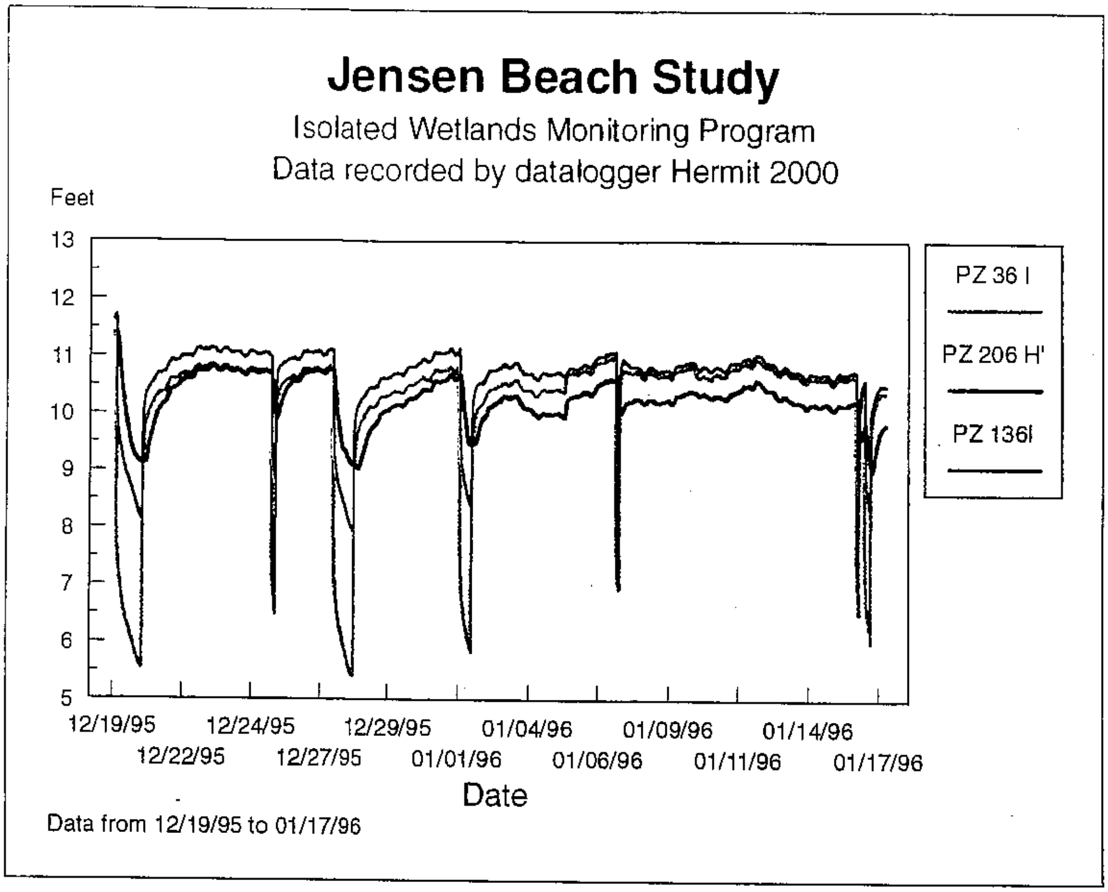
Jensen Beach Study
Isolated Wetlands Monitoring Program
Data recorded by datalogger Hermit 2000
Feet
14
13
12
11
10
9
8
water has reached the bottom of the well PZ 106 H'
drawdown due to pumping of well 7
PZ 106 H'
PZ 206 H'
PZ136I
11/08/95
11/11/95
11/14/95
11/18/95
11/21/95
11/24/95
11/27/95
11/30/95
12/03/95
12/06/95
12/09/95
12/13/95
12/16/95
12/19/95
Date
Data from 11/08/95 to 12/19/95



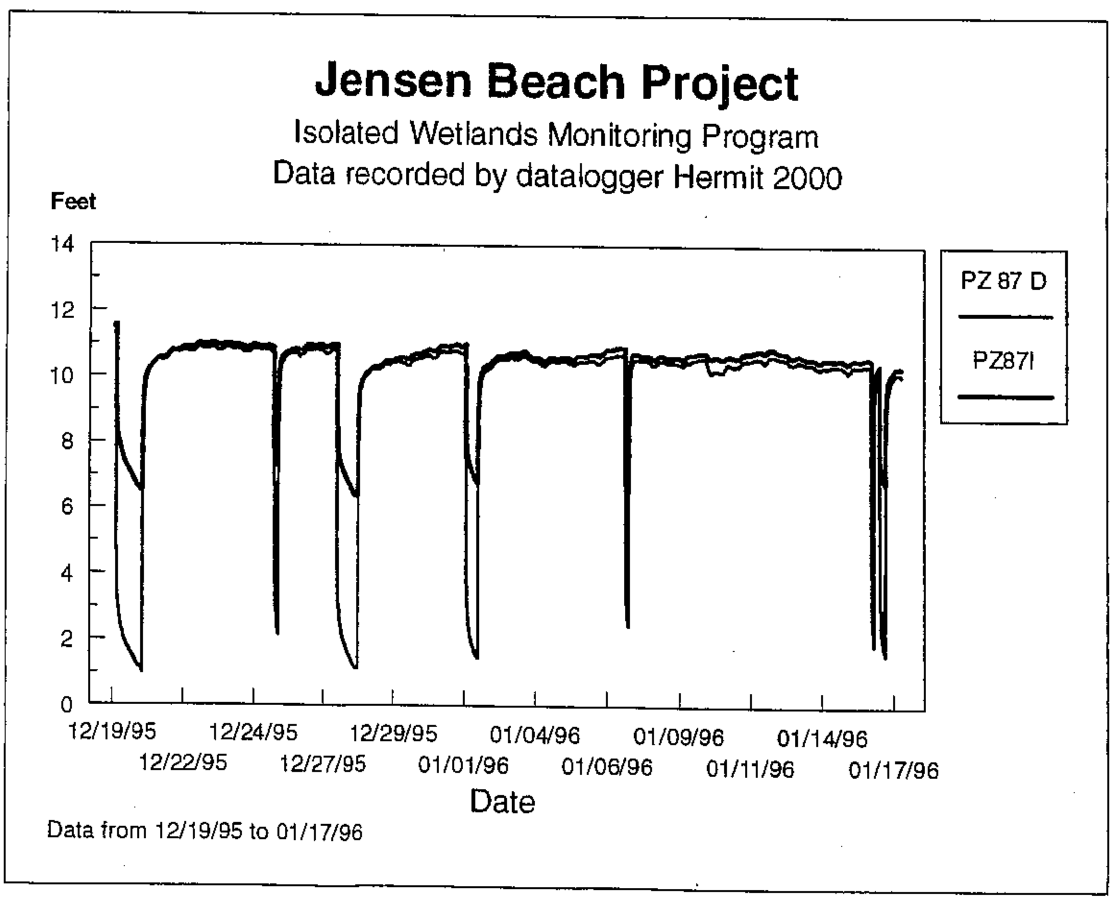
Jensen Beach Study
Isolated Wetlands Monitoring Program
Data recorded by datalogger Hermit 2000
Feet
14
13
12
11
10
9
8
7
6
drawdown due to pumping of well 7
PZ 36 I
PZ136I
PZ87I
11/08/95
11/11/95
11/14/95
11/18/95
11/21/95
11/24/95
11/27/95
11/30/95
12/03/95
12/06/95
12/09/95
12/13/95
12/16/95
12/19/95
Date
Data from 11/08/95 to 12/19/95


Jensen Beach Study
Isolated Wetlands Monitoring Program
Data recorded by datalogger Hermit 2000
Feet
14
12
10
8
6
4
2
0
11/08/95
11/11/95
11/14/95
11/18/95
11/21/95
11/24/95
11/27/95
11/30/95
12/03/95
12/06/95
12/09/95
12/13/95
12/16/95
12/19/95
Date
PZ 36 I
PZ 87 D
PZ136I
PZ87I
Data from 11/08/95 to 12/19/95

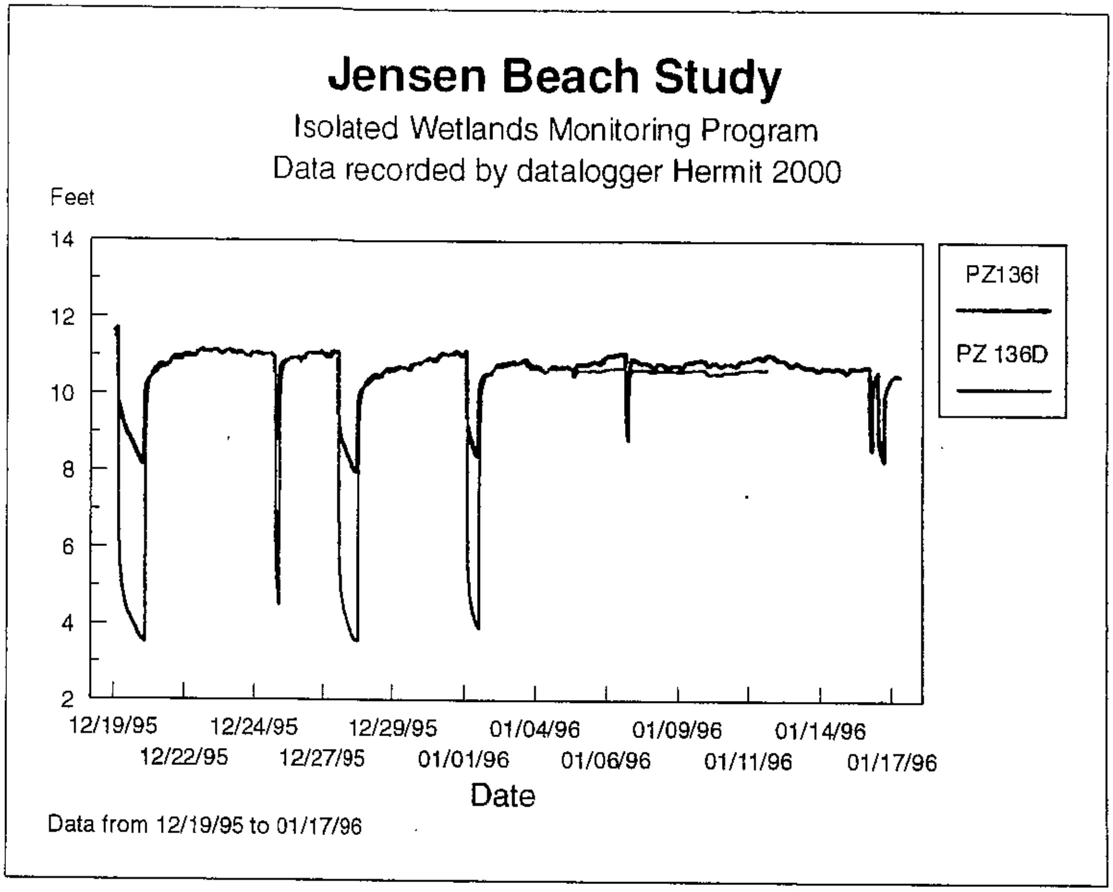
DECEMBER 19, 1995 TO JANUARY 17, 1996

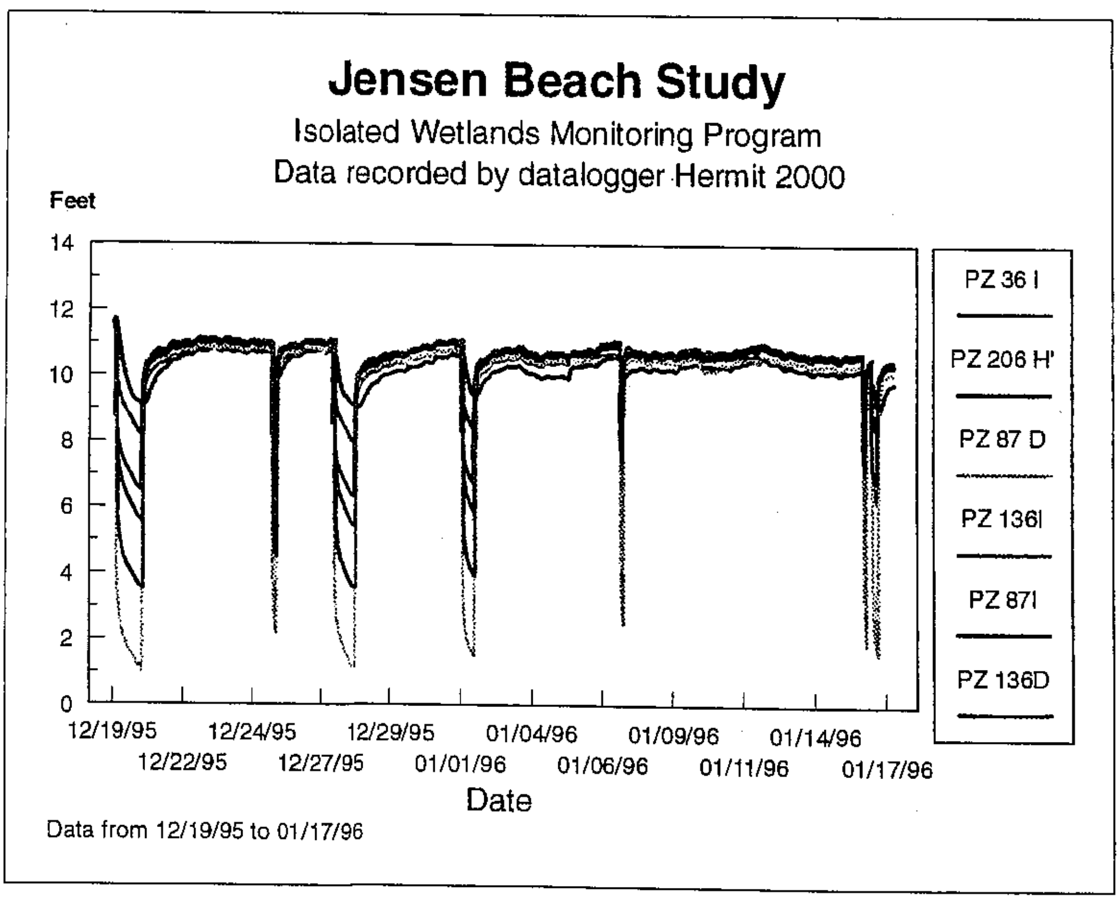


Jensen Beach Study
Isolated Wetlands Monitoring Program
Data recorded by datalogger Hermit 2000
Feet
13
12
11
10
9
8
7
6
5
PZ 36 I
PZ 206 H'
PZ 136I
12/19/95
12/22/95
12/24/95
12/27/95
12/29/95
01/01/96
01/04/96
01/06/96
01/09/96
01/11/96
01/14/96
01/17/96
Date
Data from 12/19/95 to 01/17/96

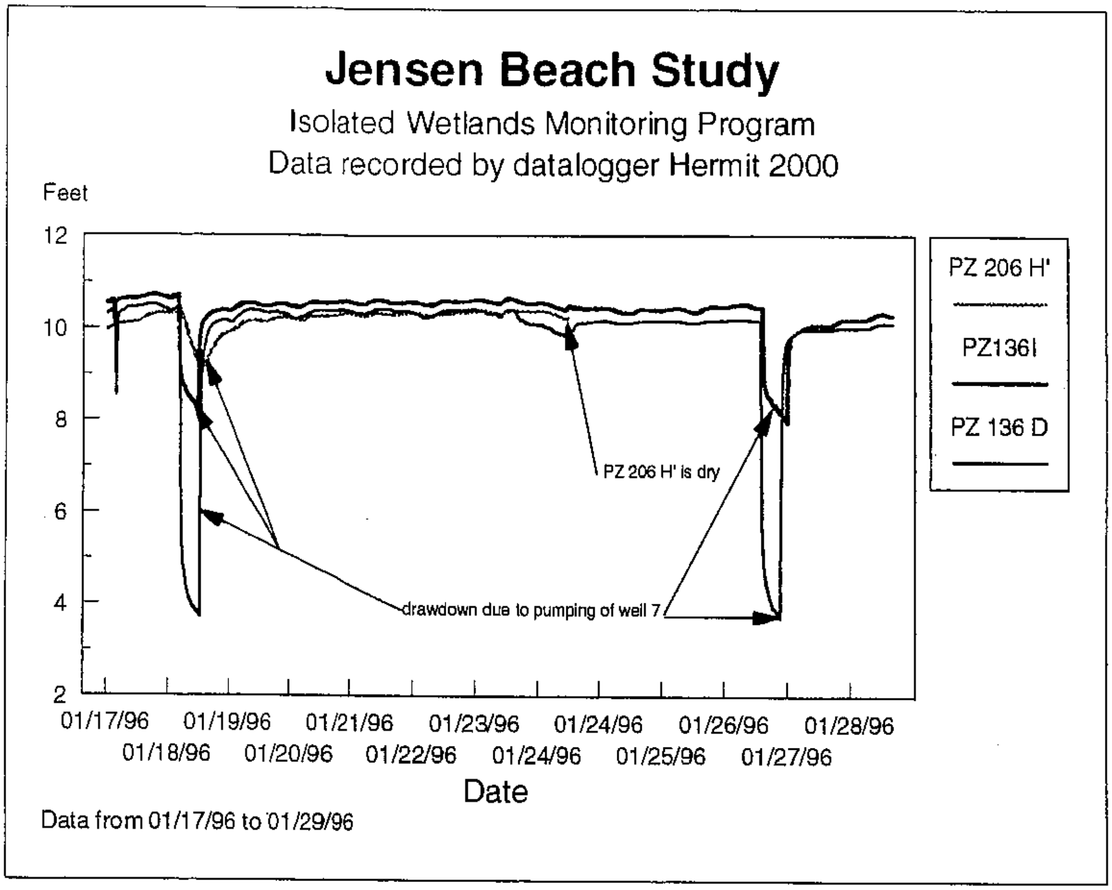


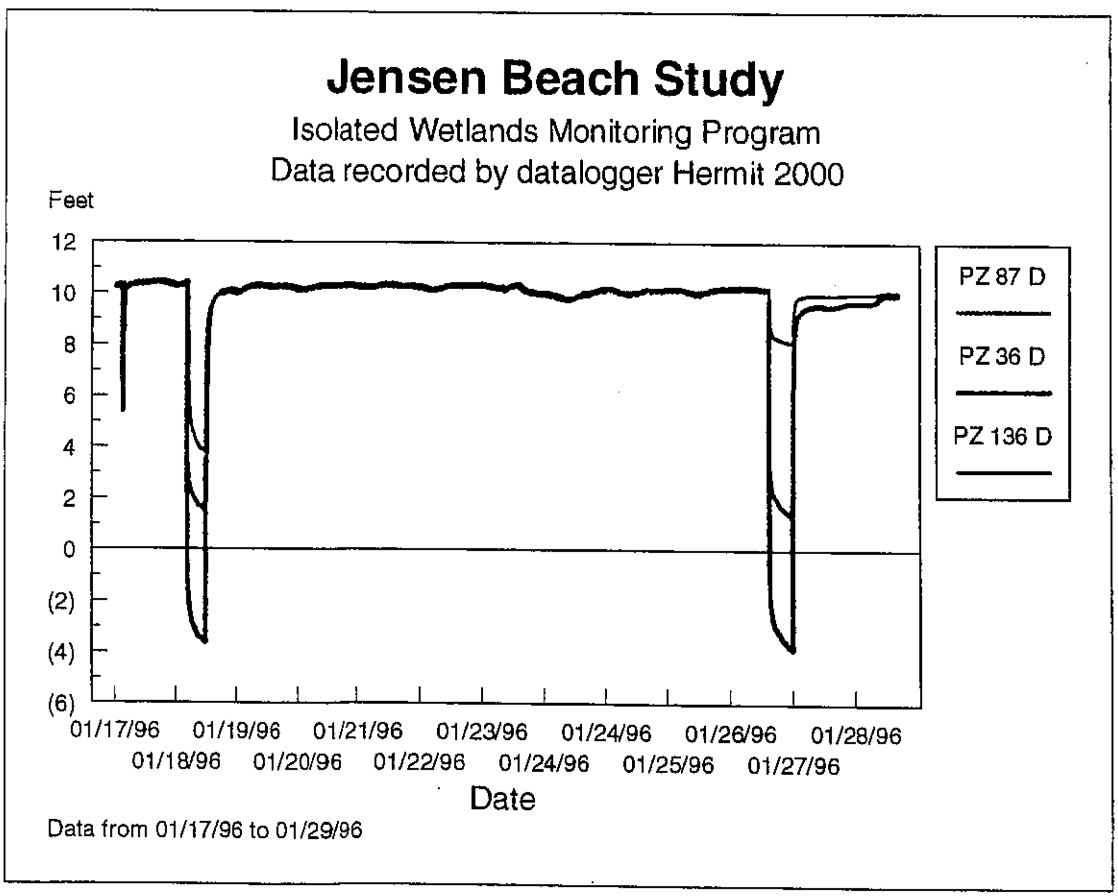
Jensen Beach Project
Isolated Wetlands Monitoring Program
Data recorded by datalogger Hermit 2000
Feet
14
12
10
8
6
4
2
0
PZ 87 D
PZ87I
12/19/95
12/22/95
12/24/95
12/27/95
12/29/95
01/01/96
01/04/96
01/06/96
01/09/96
01/11/96
01/14/96
01/17/96
Date
Data from 12/19/95 to 01/17/96

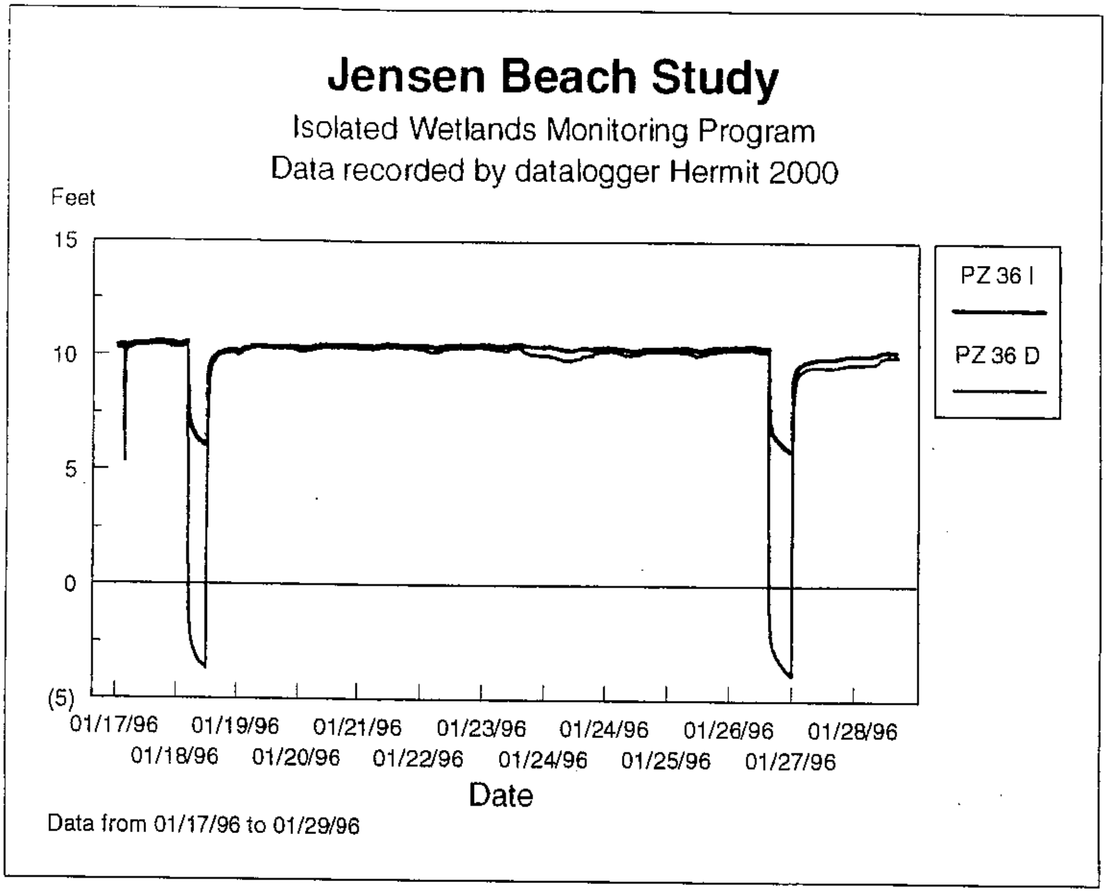

Jensen Beach Study
Isolated Wetlands Monitoring Program
Data recorded by datalogger Hermit 2000
Feet
14
12
10
8
6
4
2
12/19/95
12/22/95
12/24/95
12/27/95
12/29/95
01/01/96
01/04/96
01/06/96
01/09/96
01/11/96
01/14/96
01/17/96
Date
PZ136I
PZ 136D
Data from 12/19/95 to 01/17/96

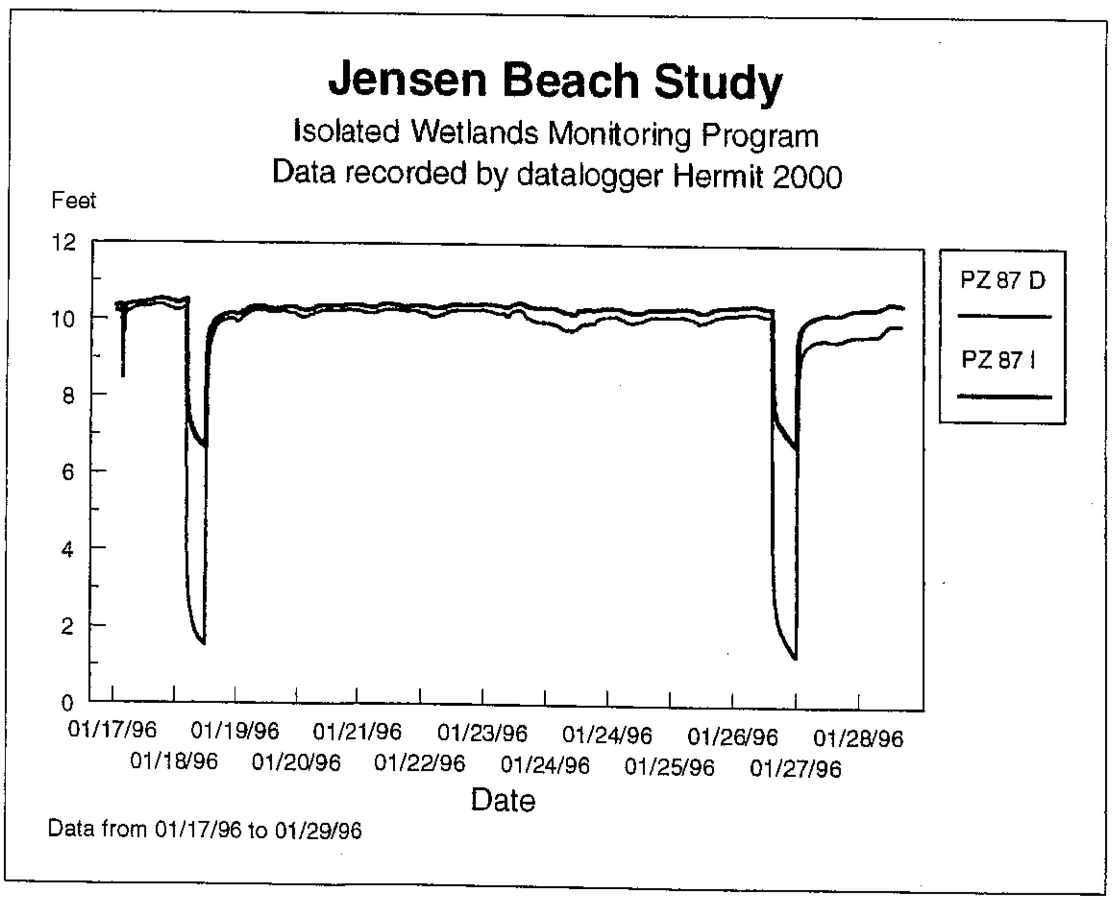

Jensen Beach Study
Isolated Wetlands Monitoring Program
Data recorded by datalogger Hermit 2000
Feet
14
12
10
8
6
4
2
0
12/19/95
12/22/95
12/24/95
12/27/95
12/29/95
01/01/96
01/04/96
01/06/96
01/09/96
01/11/96
01/14/96
01/17/96
Date
PZ 36 I
PZ 206 H'
PZ 87 D
PZ 136I
PZ 87I
PZ 136D
Data from 12/19/95 to 01/17/96

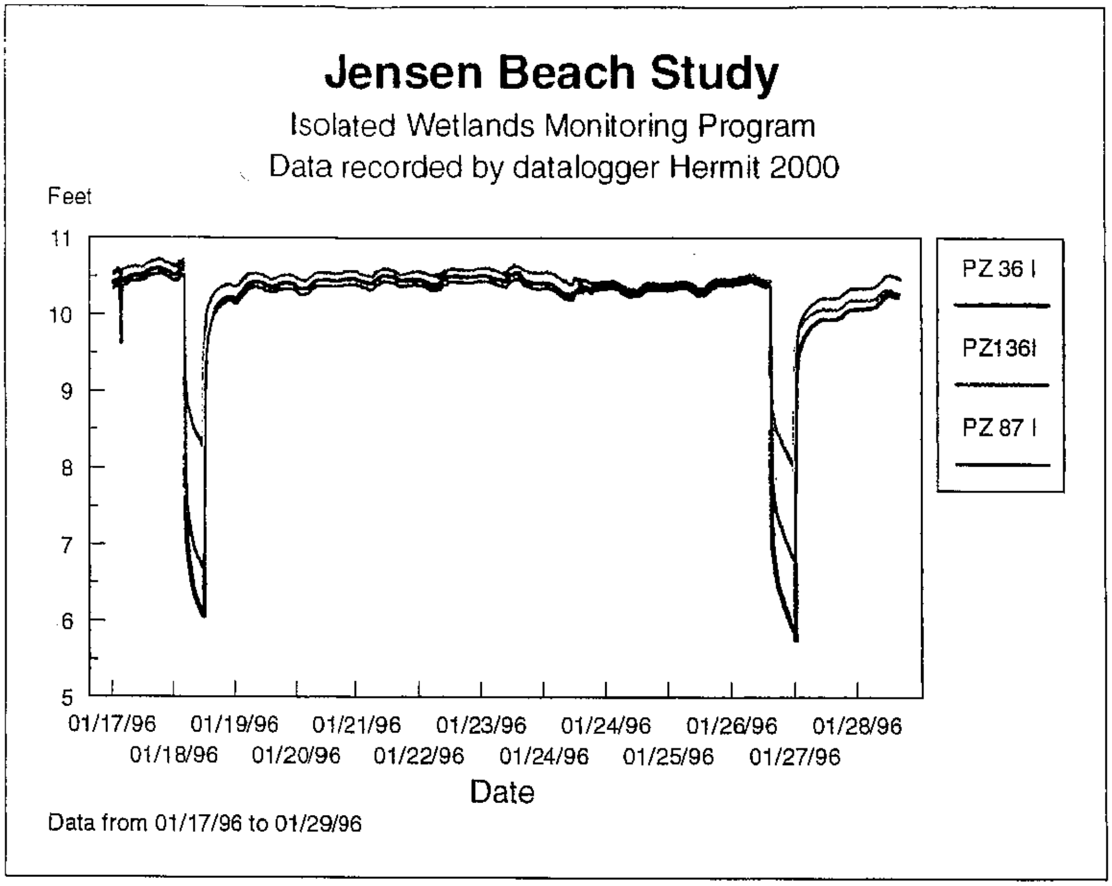
JANUARY 17 TO JANUARY 29, 1996

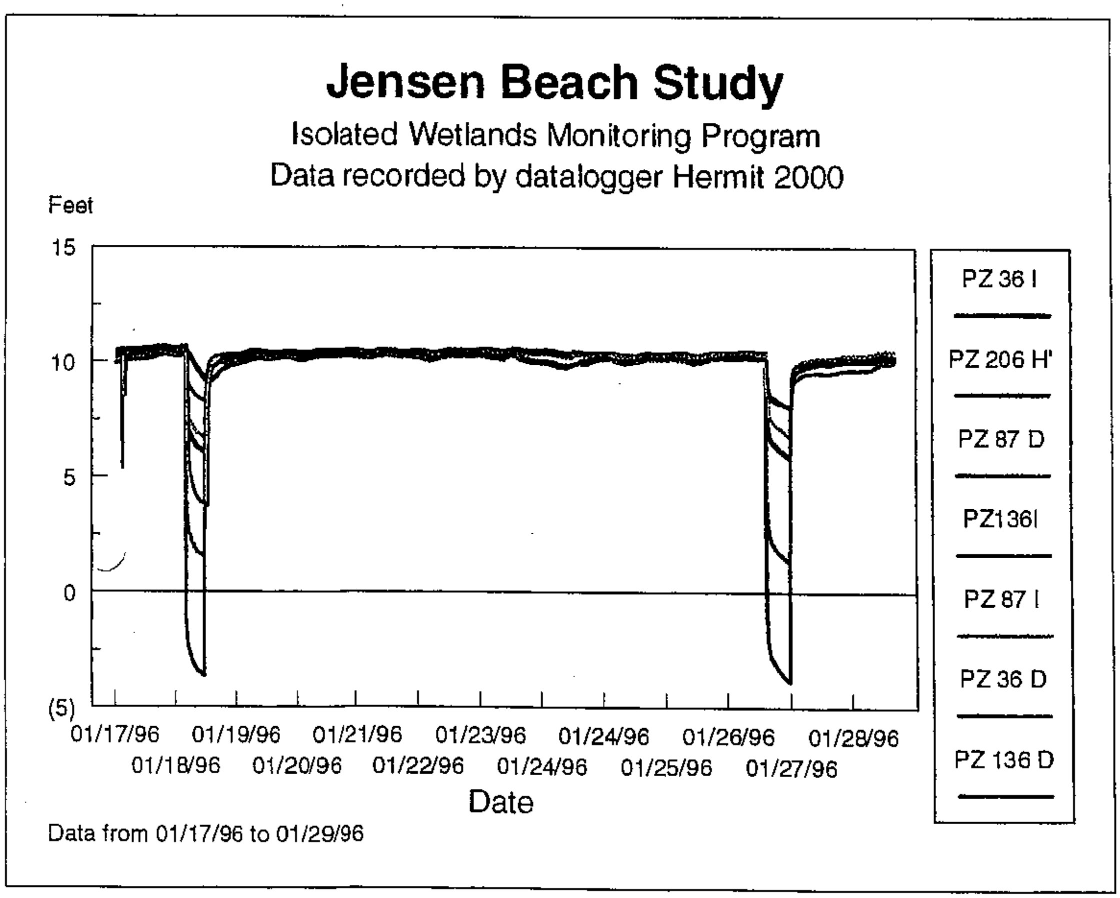

Jensen Beach Study
Isolated Wetlands Monitoring Program
Data recorded by datalogger Hermit 2000
Feet
12
10
8
6
4
2
PZ 206 H'
PZ136I
PZ 136 D
PZ 206 H' is dry
drawdown due to pumping of well 7
01/17/96
01/18/96
01/19/96
01/20/96
01/21/96
01/22/96
01/23/96
01/24/96
01/24/96
01/25/96
01/26/96
01/27/96
01/28/96
Date
Data from 01/17/96 to 01/29/96


Jensen Beach Study
Isolated Wetlands Monitoring Program
Data recorded by datalogger Hermit 2000
Feet
12
10
8
6
4
2
0
(2)
(4)
(6)
PZ 87 D
PZ 36 D
PZ 136 D
01/17/96
01/18/96
01/19/96
01/20/96
01/21/96
01/22/96
01/23/96
01/24/96
01/24/96
01/25/96
01/26/96
01/27/96
01/28/96
Date
Data from 01/17/96 to 01/29/96

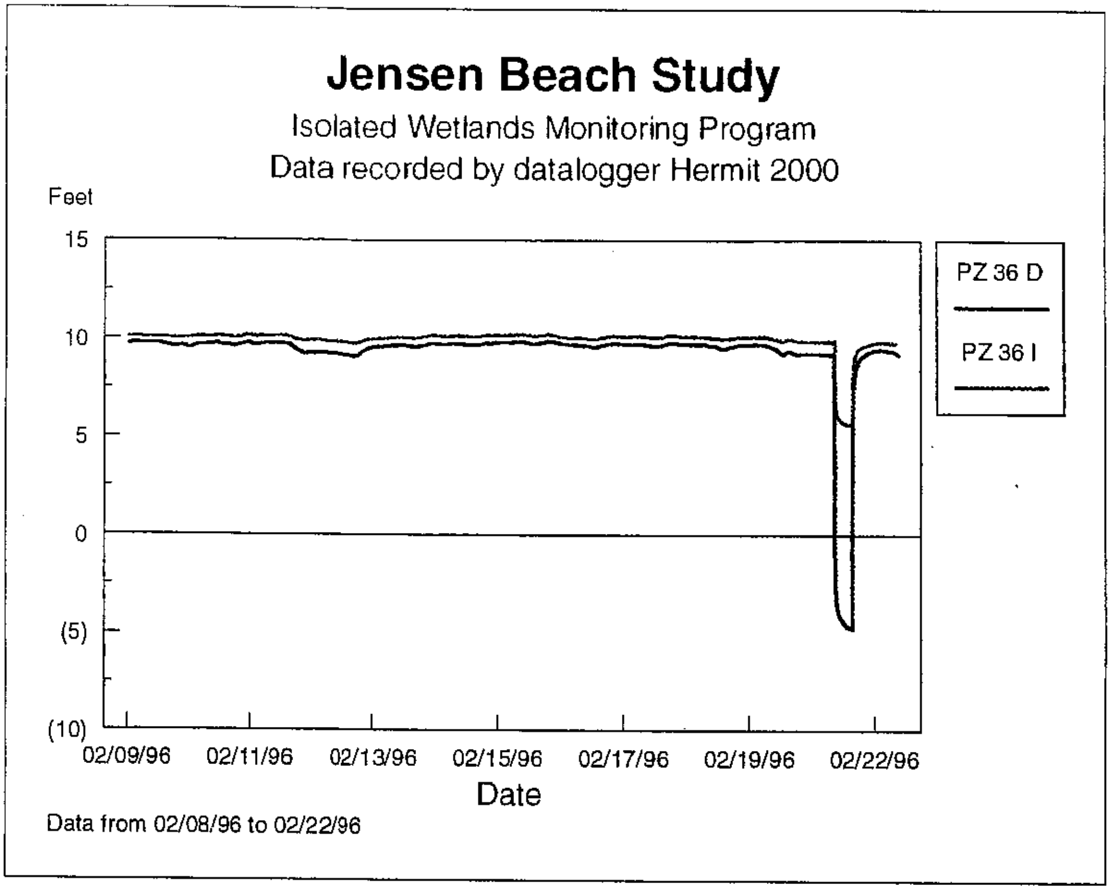

Jensen Beach Study
Isolated Wetlands Monitoring Program
Data recorded by datalogger Hermit 2000
Feet
15
10
5
0
(5)
PZ 36 I
PZ 36 D
01/17/96
01/18/96
01/19/96
01/20/96
01/21/96
01/22/96
01/23/96
01/24/96
01/24/96
01/25/96
01/26/96
01/27/96
01/28/96
Date
Data from 01/17/96 to 01/29/96

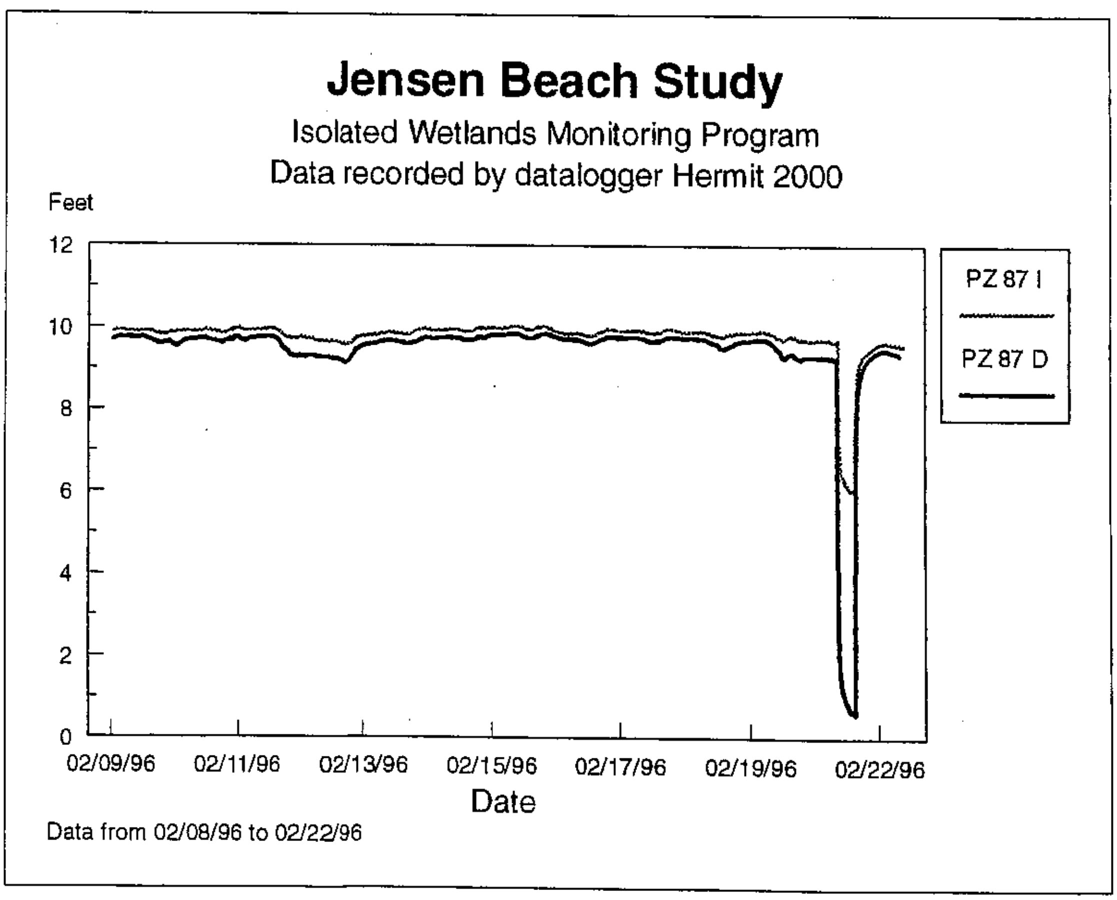

Jensen Beach Study
Isolated Wetlands Monitoring Program
Data recorded by datalogger Hermit 2000
Feet
12
10
8
6
4
2
0
PZ 87 D
PZ 87 I
01/17/96
01/18/96
01/19/96
01/20/96
01/21/96
01/22/96
01/23/96
01/24/96
01/24/96
01/25/96
01/26/96
01/27/96
01/28/96
Date
Data from 01/17/96 to 01/29/96

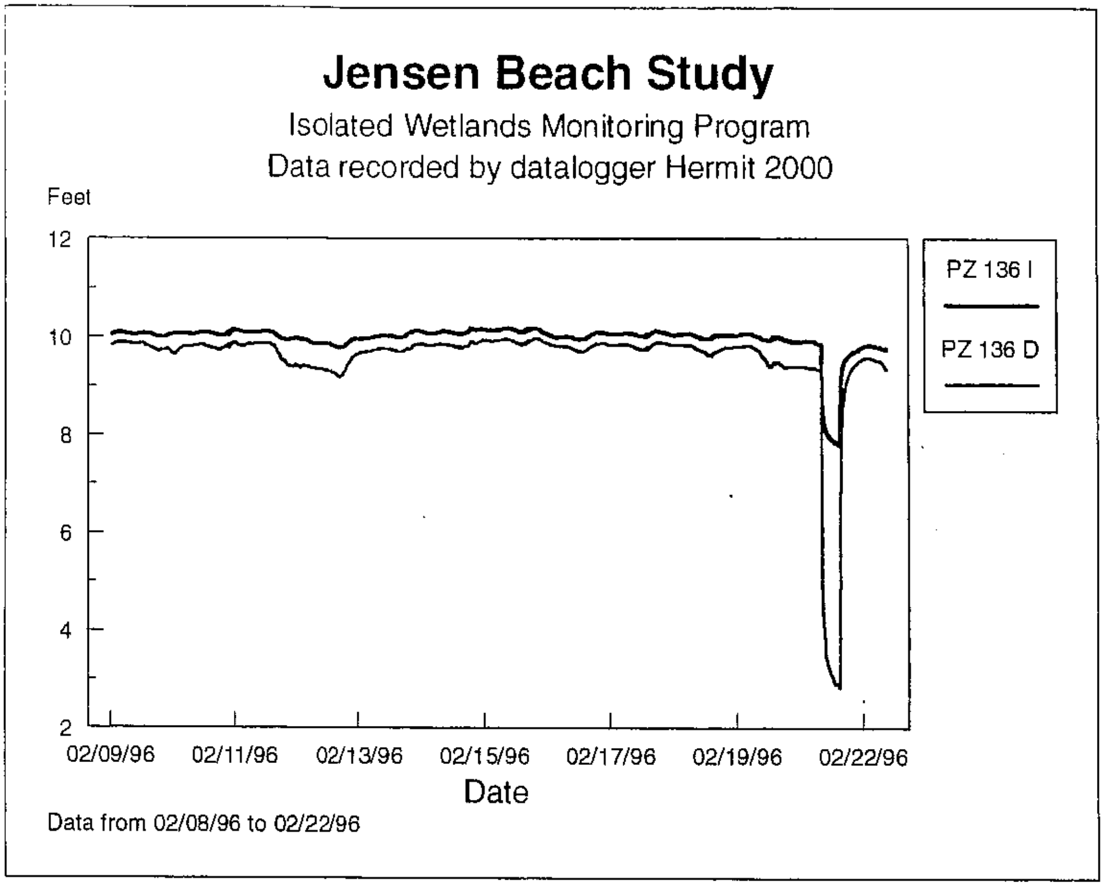

Jensen Beach Study
Isolated Wetlands Monitoring Program
Data recorded by datalogger Hermit 2000
Feet
11
10
9
8
7
6
5
01/17/96
01/18/96
01/19/96
01/20/96
01/21/96
01/22/96
01/23/96
01/24/96
01/24/96
01/25/96
01/26/96
01/27/96
01/28/96
Date
Data from 01/17/96 to 01/29/96
PZ 36 I
PZ136I
PZ 87 I

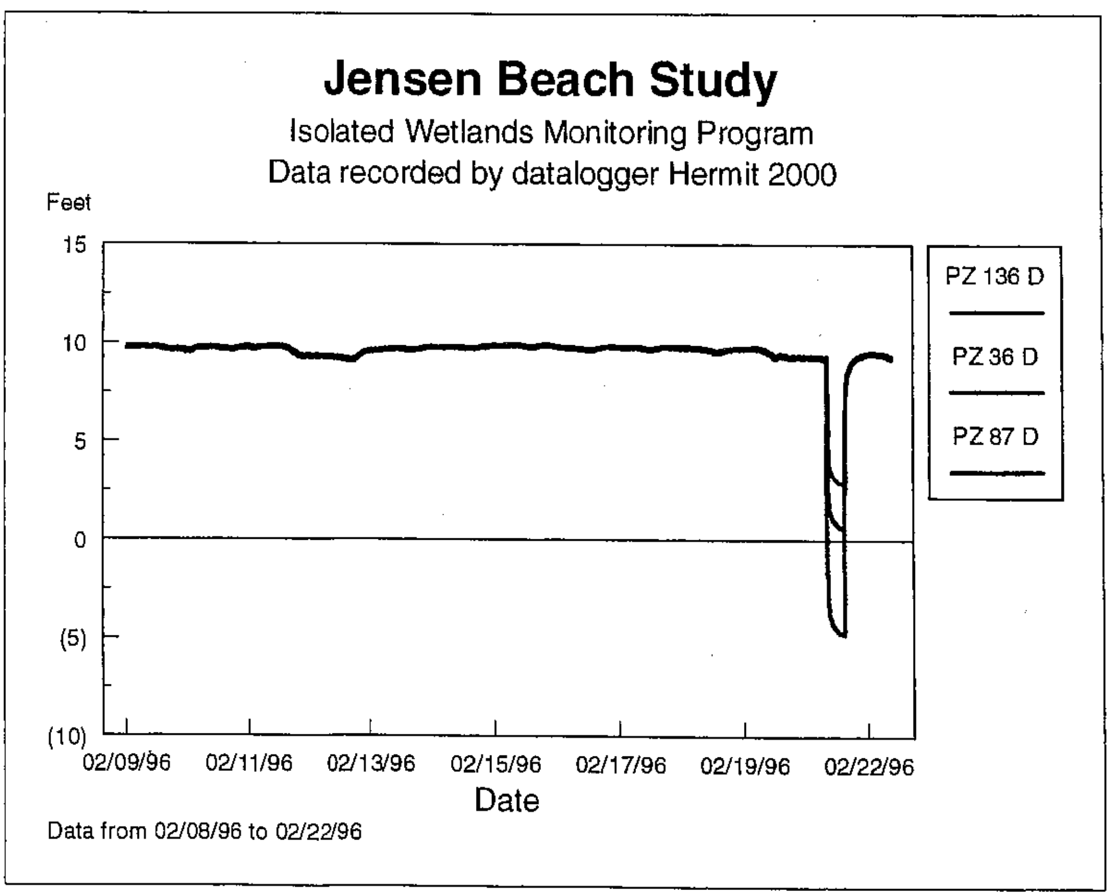

Jensen Beach Study
Isolated Wetlands Monitoring Program
Data recorded by datalogger Hermit 2000
Feet
15
10
5
0
(5)
01/17/96
01/18/96
01/19/96
01/20/96
01/21/96
01/22/96
01/23/96
01/24/96
01/24/96
01/25/96
01/26/96
01/27/96
01/28/96
Date
Data from 01/17/96 to 01/29/96
PZ 36 I
PZ 206 H'
PZ 87 D
PZ136I
PZ 87 I
PZ 36 D
PZ 136 D

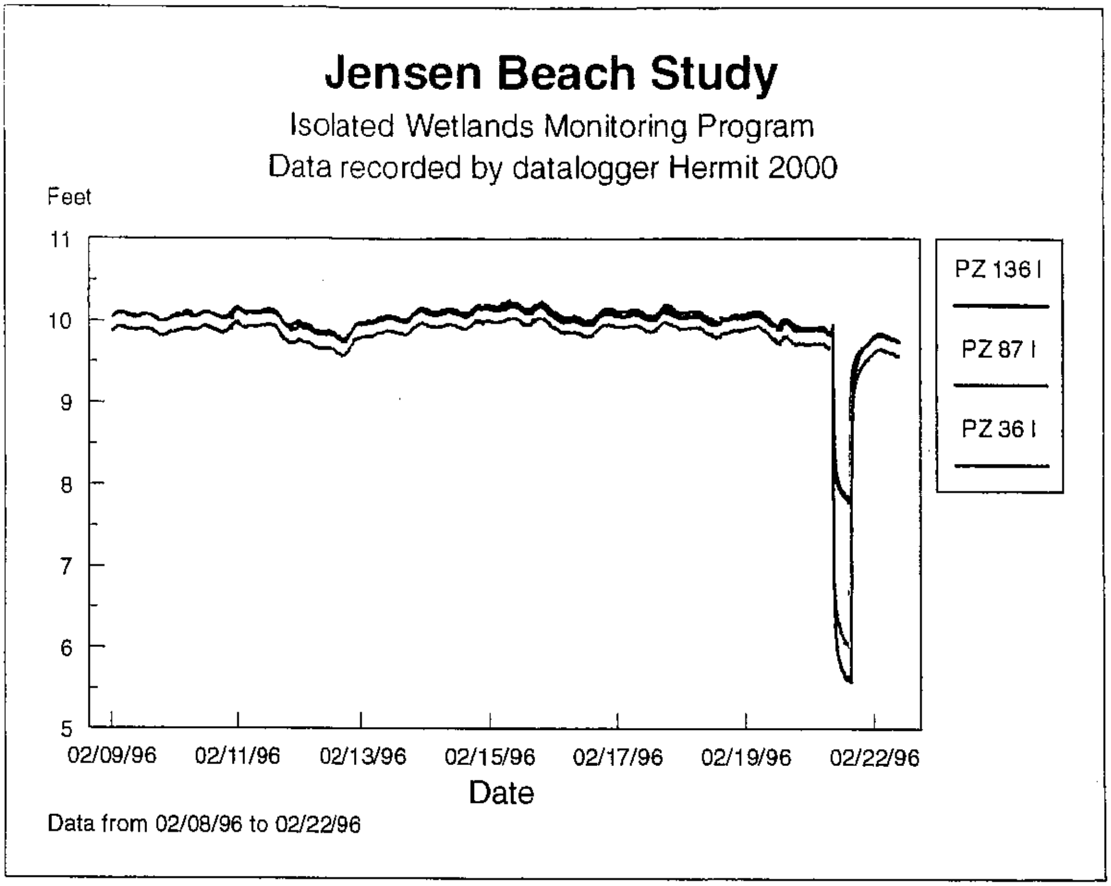
FEBRUARY 8 TO FEBRUARY 22, 1996

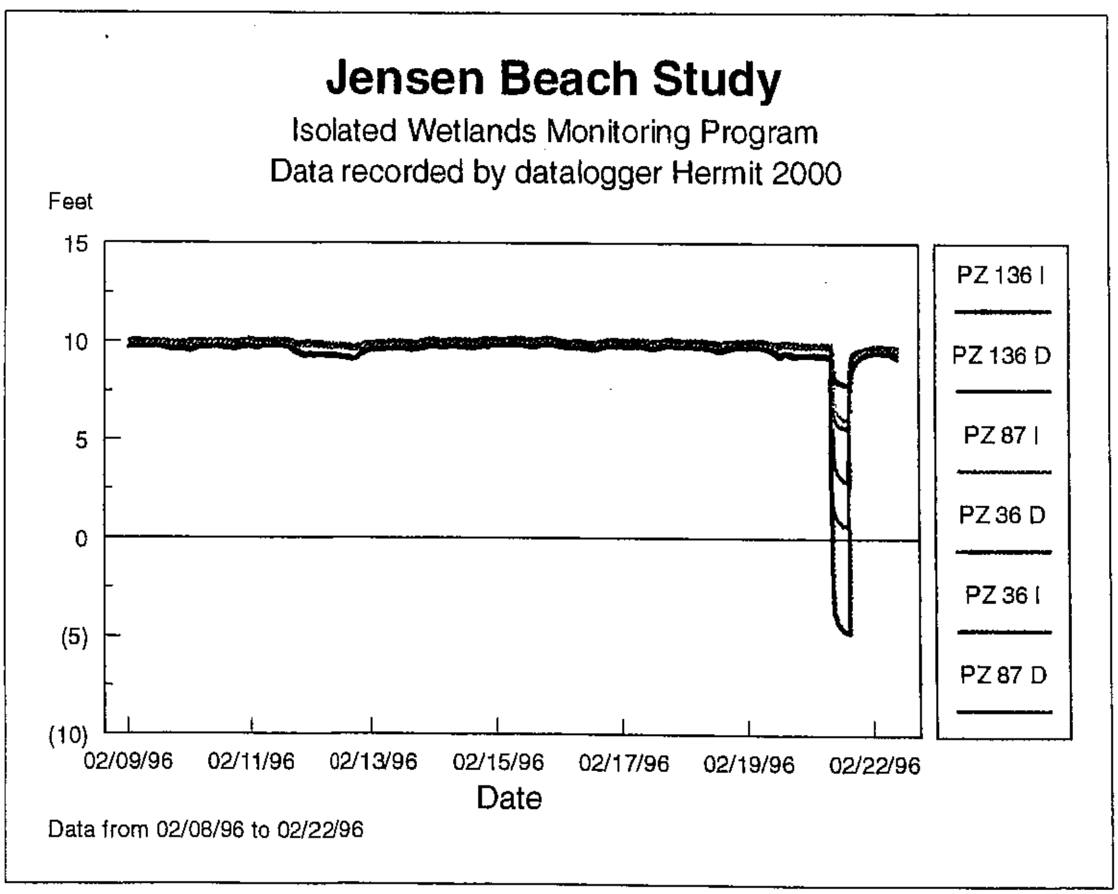

Jensen Beach Study
Isolated Wetlands Monitoring Program
Data recorded by datalogger Hermit 2000
Feet
15
10
5
0
(5)
(10)
02/09/96
02/11/96
02/13/96
02/15/96
02/17/96
02/19/96
02/22/96
Date
Data from 02/08/96 to 02/22/96
PZ 36 D
PZ 36 I


Jensen Beach Study
Isolated Wetlands Monitoring Program
Data recorded by datalogger Hermit 2000
Feet
12
10
8
6
4
2
0
02/09/96
02/11/96
02/13/96
02/15/96
02/17/96
02/19/96
02/22/96
Date
Data from 02/08/96 to 02/22/96
PZ 87 I
PZ 87 D

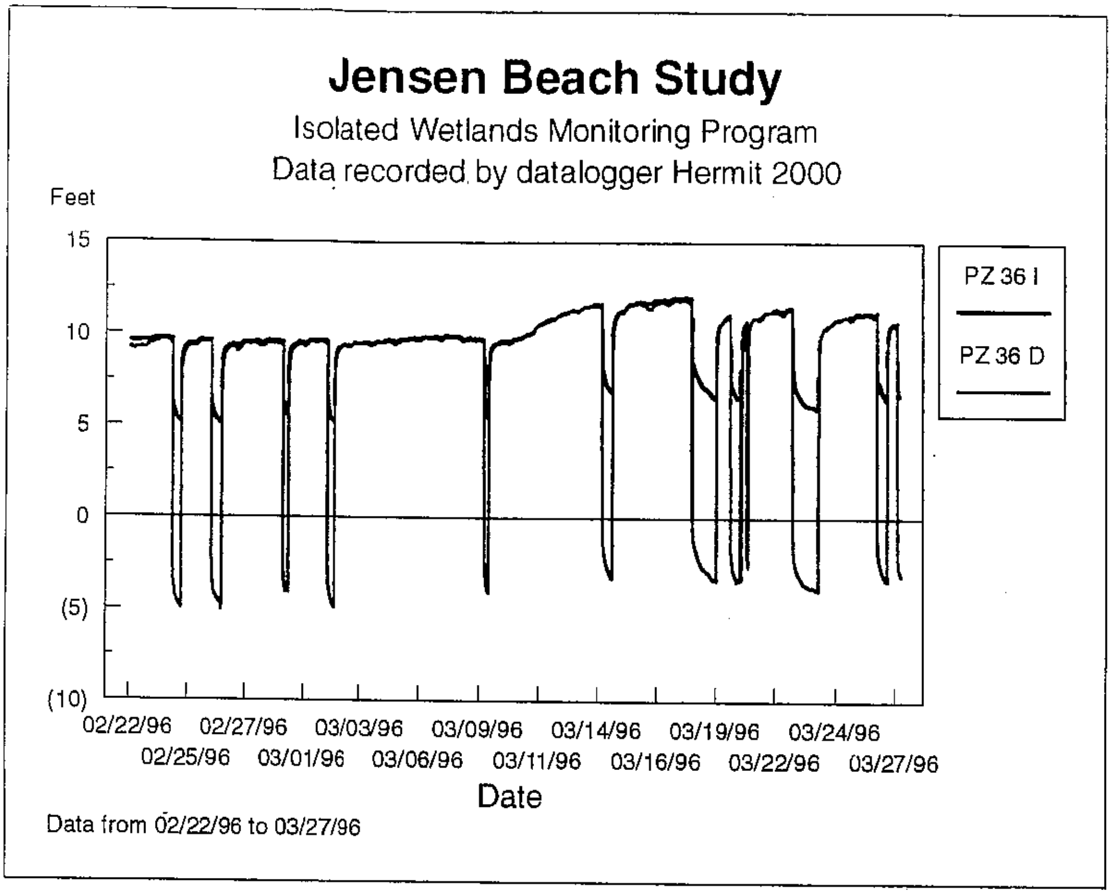

Jensen Beach Study
Isolated Wetlands Monitoring Program
Data recorded by datalogger Hermit 2000
Feet
12
10
8
6
4
2
02/09/96
02/11/96
02/13/96
02/15/96
02/17/96
02/19/96
02/22/96
Date
PZ 136 I
PZ 136 D
Data from 02/08/96 to 02/22/96


Jensen Beach Study
Isolated Wetlands Monitoring Program
Data recorded by datalogger Hermit 2000
Feet
15
10
5
0
(5)
(10)
02/09/96
02/11/96
02/13/96
02/15/96
02/17/96
02/19/96
02/22/96
Date
PZ 136 D
PZ 36 D
PZ 87 D
Data from 02/08/96 to 02/22/96

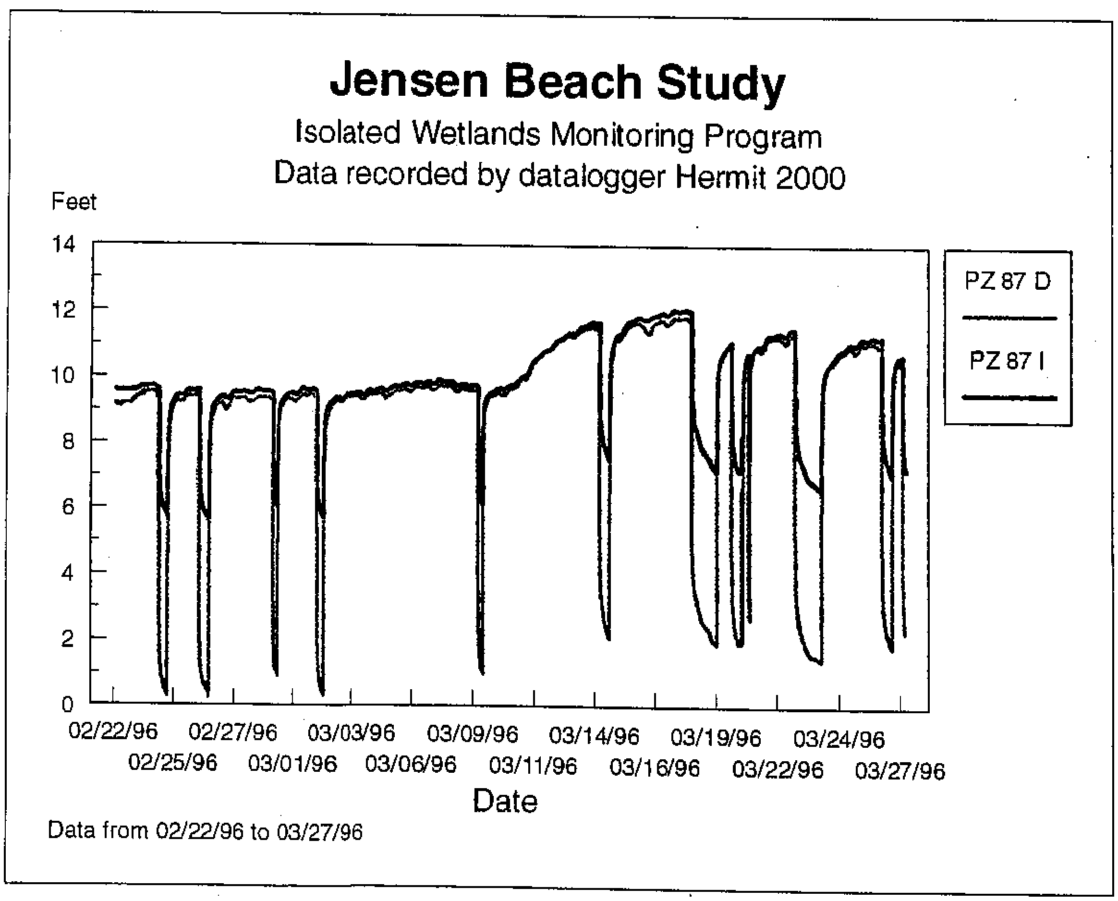

Jensen Beach Study
Isolated Wetlands Monitoring Program
Data recorded by datalogger Hermit 2000
Feet
11
10
9
8
7
6
5
02/09/96
02/11/96
02/13/96
02/15/96
02/17/96
02/19/96
02/22/96
Date
PZ 136 I
PZ 87 I
PZ 36 I
Data from 02/08/96 to 02/22/96


Jensen Beach Study
Isolated Wetlands Monitoring Program
Data recorded by datalogger Hermit 2000
Feet
15
10
5
0
(5)
(10)
02/09/96
02/11/96
02/13/96
02/15/96
02/17/96
02/19/96
02/22/96
Date
PZ 136 I
PZ 136 D
PZ 87 I
PZ 36 D
PZ 36 I
PZ 87 D
Data from 02/08/96 to 02/22/96

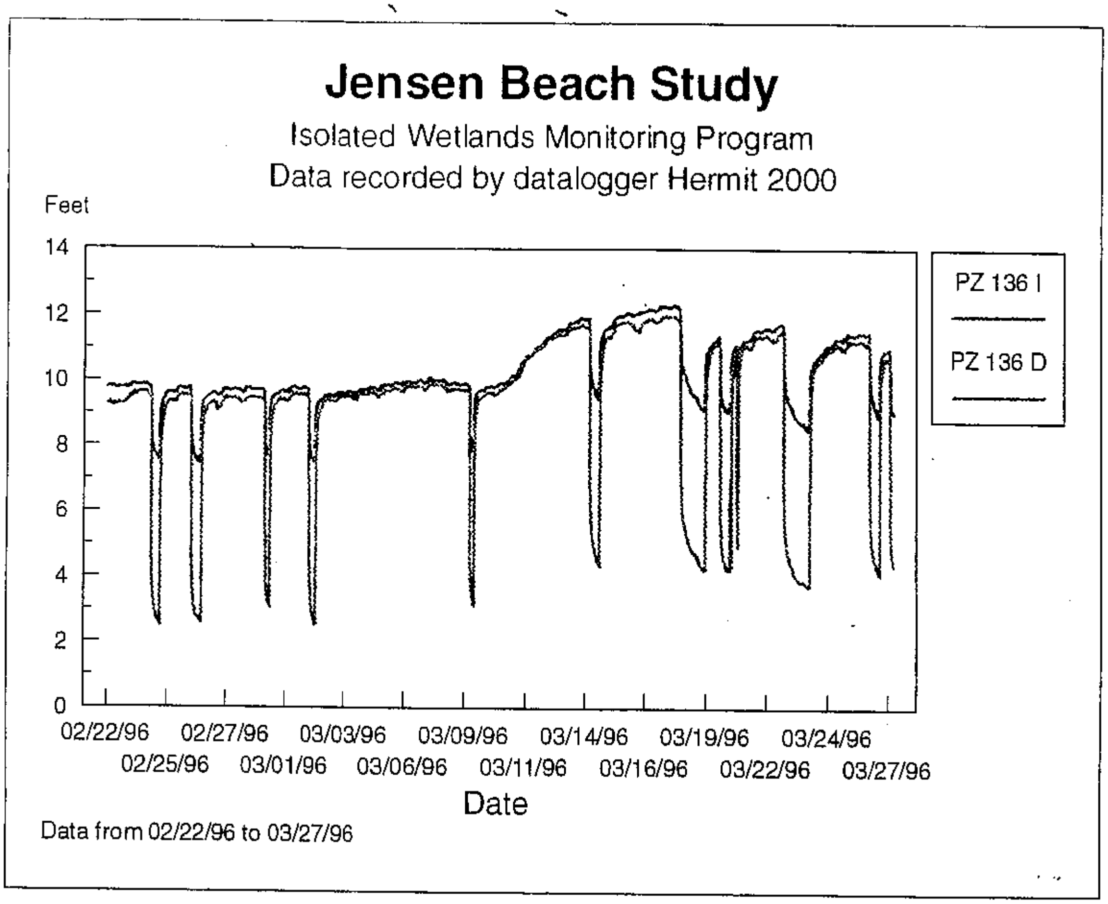
FEBRUARY 22 TO MARCH 27, 1996



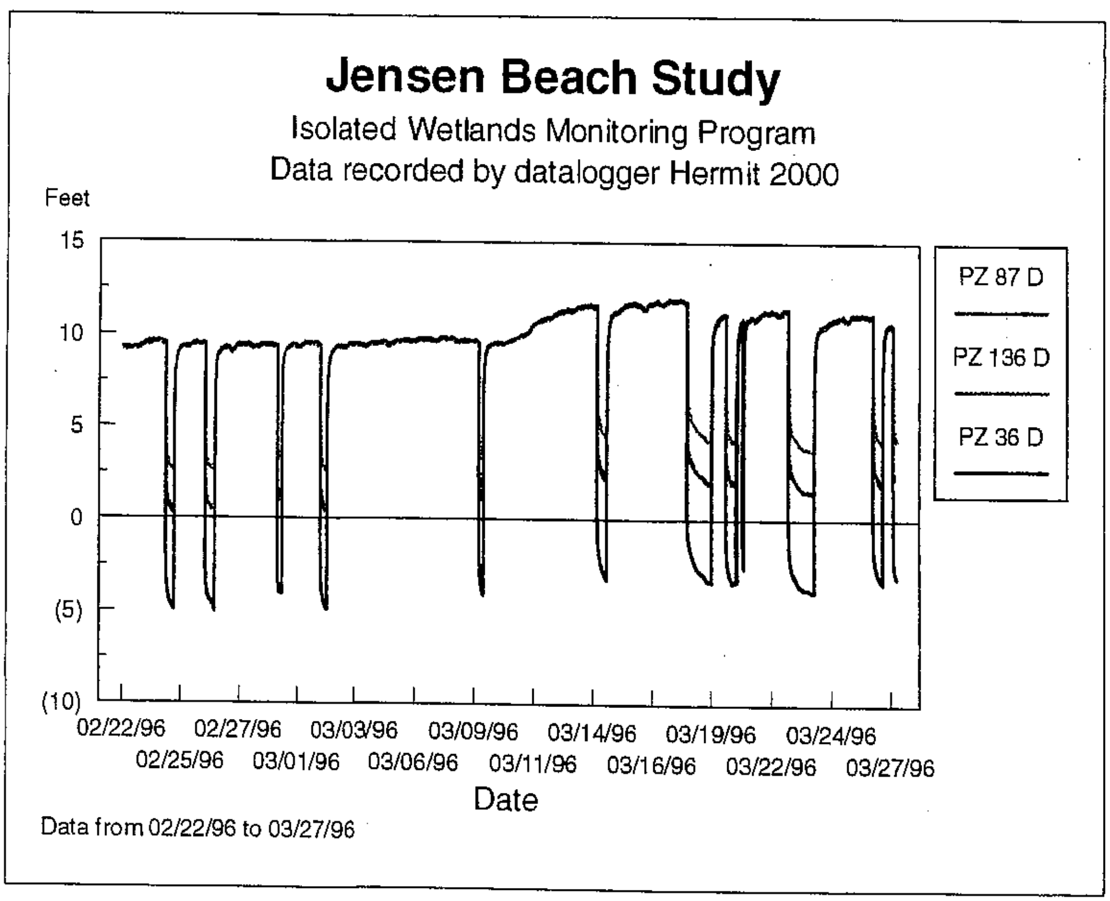
Jensen Beach Study
Isolated Wetlands Monitoring Program
Data recorded by datalogger Hermit 2000
Feet
15
10
5
0
(5)
(10)
02/22/96
02/25/96
02/27/96
03/01/96
03/03/96
03/06/96
03/09/96
03/11/96
03/14/96
03/16/96
03/19/96
03/22/96
03/24/96
03/27/96
Date
PZ 36 I
PZ 36 D
Data from 02/22/96 to 03/27/96



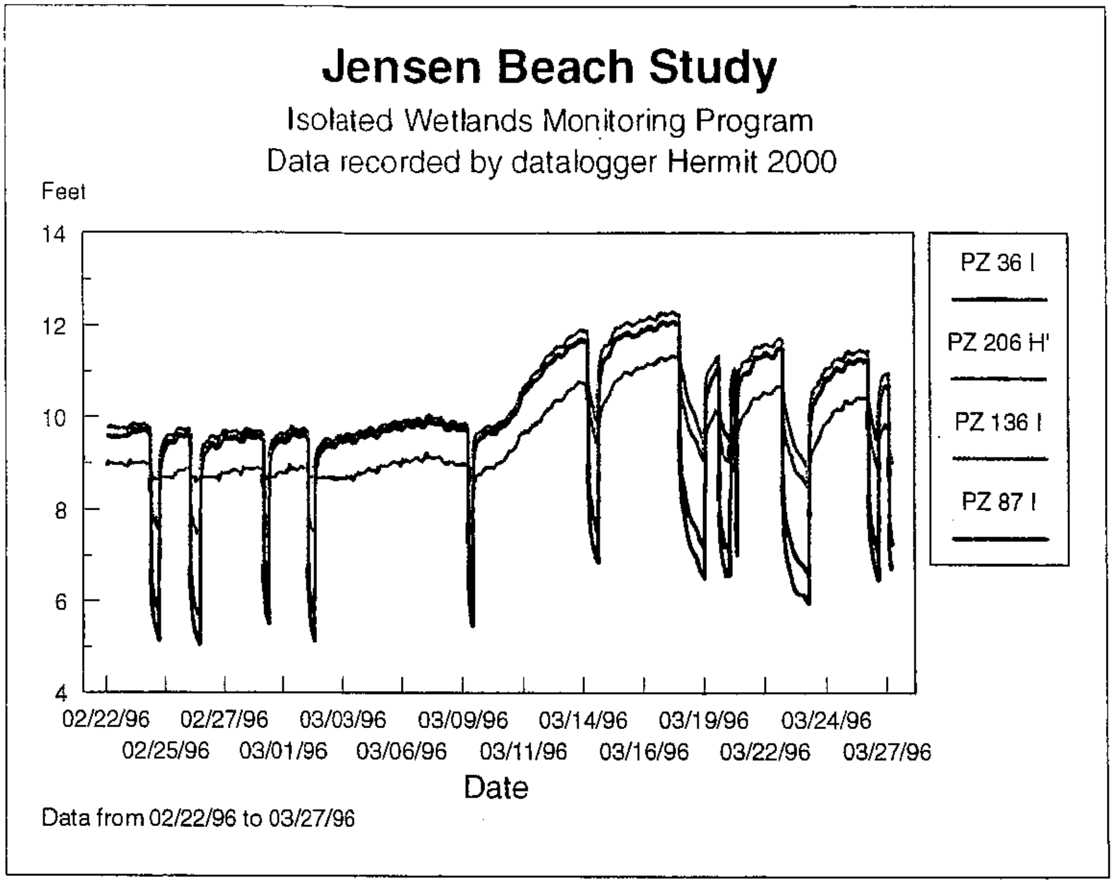
Jensen Beach Study
Isolated Wetlands Monitoring Program
Data recorded by datalogger Hermit 2000
Feet
14
12
10
8
6
4
2
0
02/22/96
02/25/96
02/27/96
03/01/96
03/03/96
03/06/96
03/09/96
03/11/96
03/14/96
03/16/96
03/19/96
03/22/96
03/24/96
03/27/96
Date
PZ 87 D
PZ 87 I
Data from 02/22/96 to 03/27/96



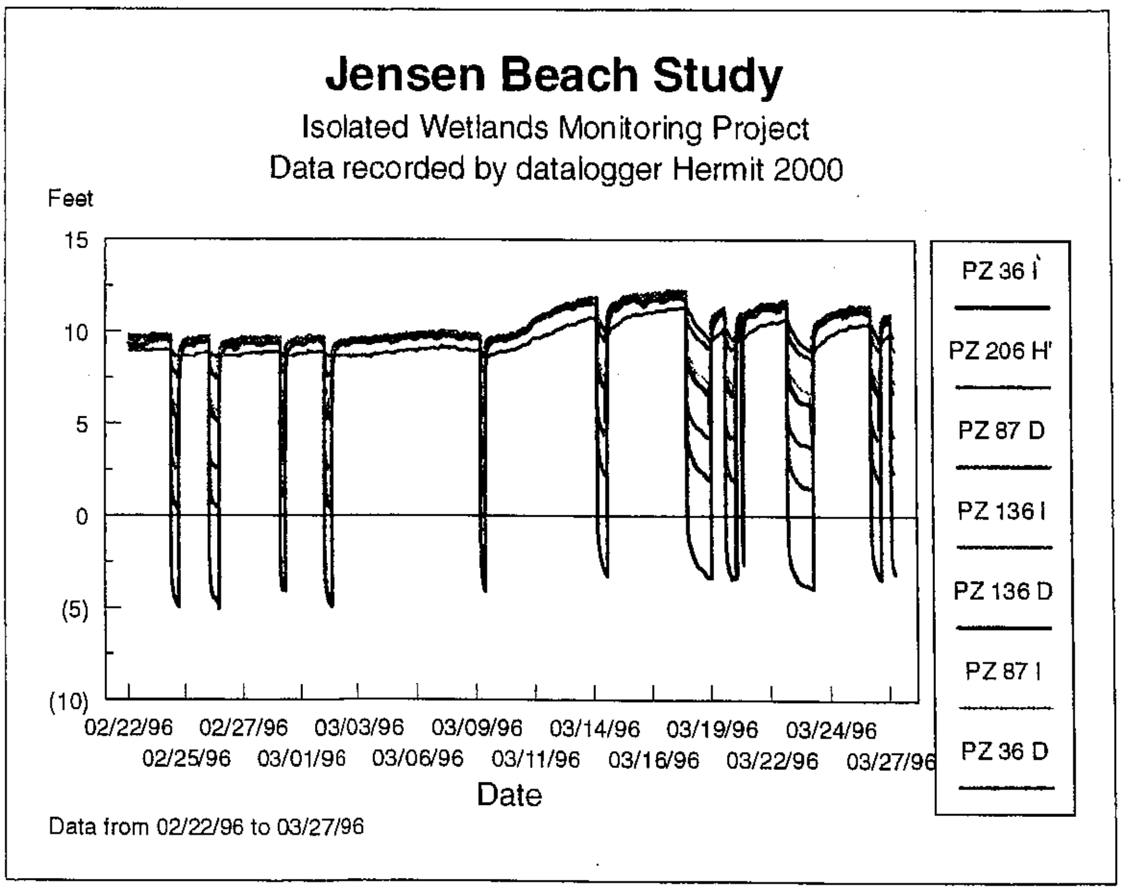
Jensen Beach Study
Isolated Wetlands Monitoring Program
Data recorded by datalogger Hermit 2000
Feet
14
12
10
8
6
4
2
0
PZ 136 I
PZ 136 D
02/22/96
02/25/96
02/27/96
03/01/96
03/03/96
03/06/96
03/09/96
03/11/96
03/14/96
03/16/96
03/19/96
03/22/96
03/24/96
03/27/96
Date
Data from 02/22/96 to 03/27/96



Jensen Beach Study
Isolated Wetlands Monitoring Program
Data recorded by datalogger Hermit 2000
Feet
15
10
5
0
(5)
(10)
PZ 87 D
PZ 136 D
PZ 36 D
02/22/96
02/25/96
02/27/96
03/01/96
03/03/96
03/06/96
03/09/96
03/11/96
03/14/96
03/16/96
03/19/96
03/22/96
03/24/96
03/27/96
Date
Data from 02/22/96 to 03/27/96



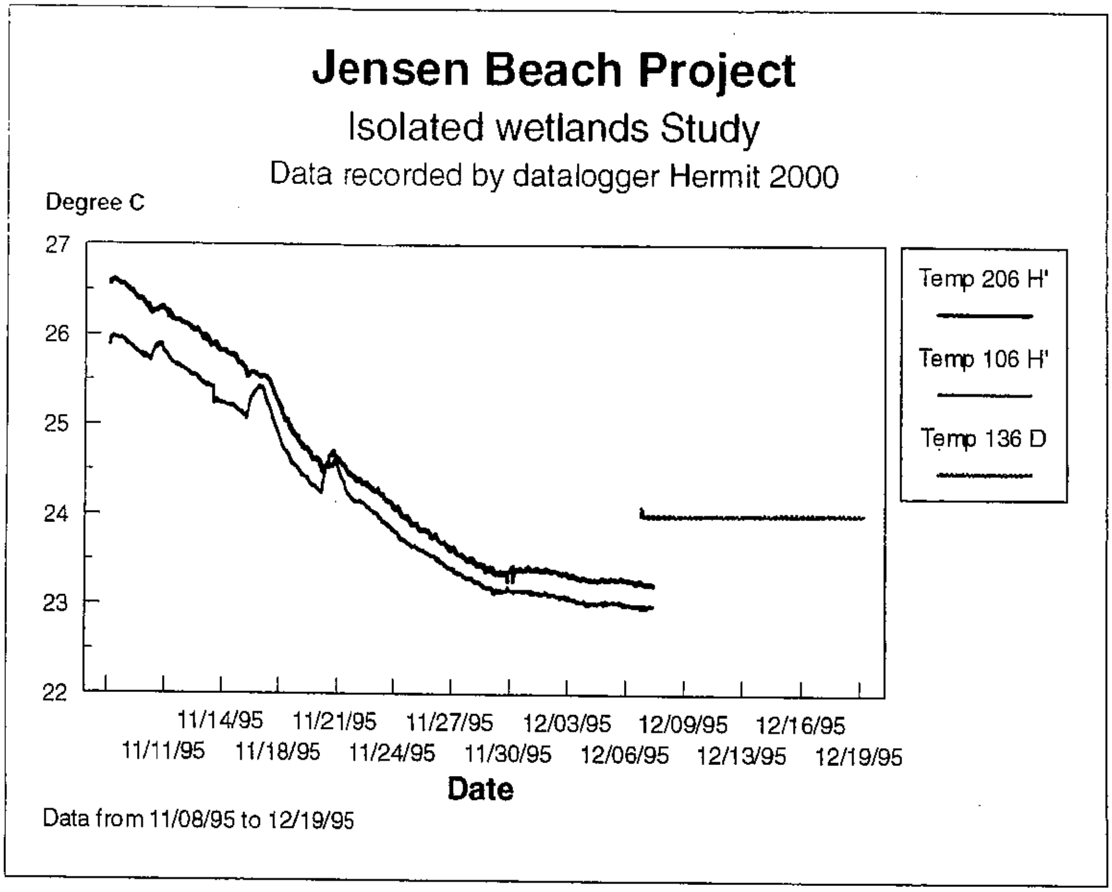
Jensen Beach Study
Isolated Wetlands Monitoring Program
Data recorded by datalogger Hermit 2000
Feet
14
12
10
8
6
4
02/22/96
02/25/96
02/27/96
03/01/96
03/03/96
03/06/96
03/09/96
03/11/96
03/14/96
03/16/96
03/19/96
03/22/96
03/24/96
03/27/96
Date
Data from 02/22/96 to 03/27/96
PZ 36 I
PZ 206 H'
PZ 136 I
PZ 87 I



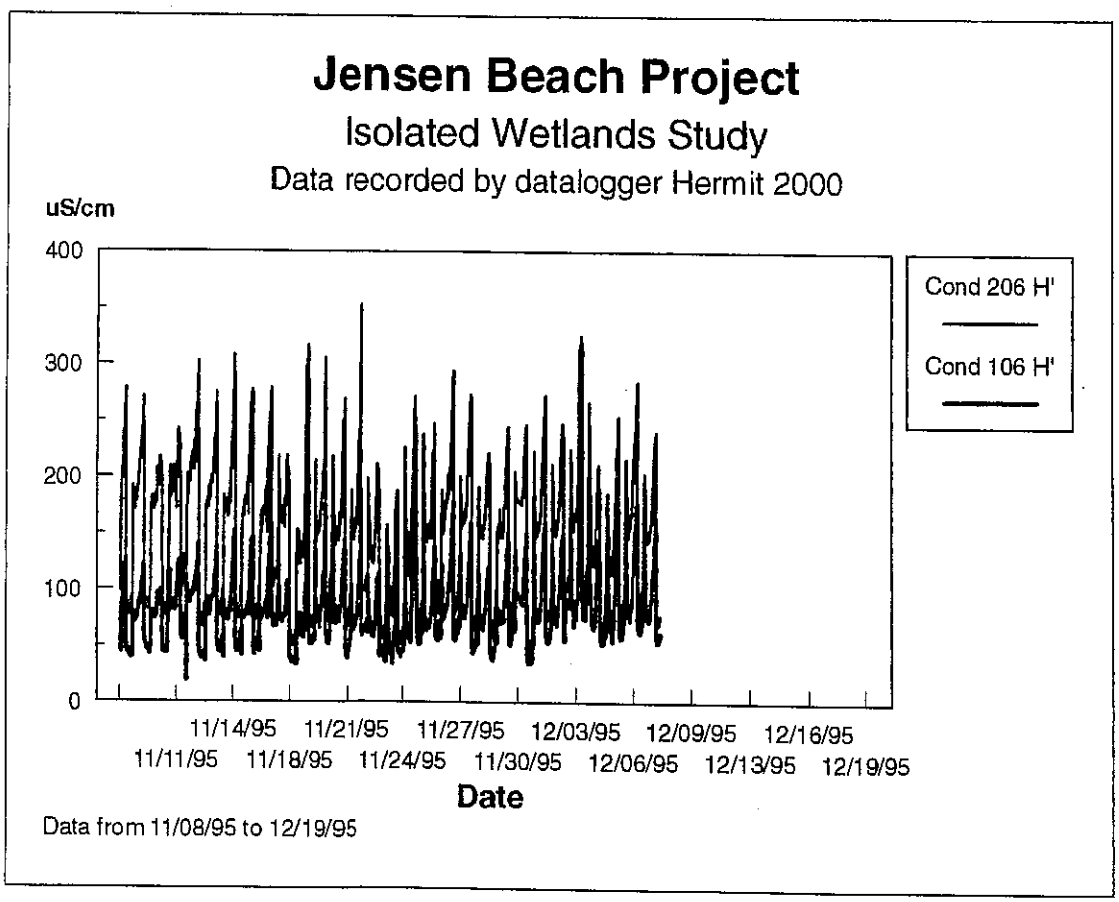
Jensen Beach Study
Isolated Wetlands Monitoring Project
Data recorded by datalogger Hermit 2000
Feet
15
10
5
0
(5)
(10)
02/22/96
02/25/96
02/27/96
03/01/96
03/03/96
03/06/96
03/09/96
03/11/96
03/14/96
03/16/96
03/19/96
03/22/96
03/24/96
03/27/96
Date
Data from 02/22/96 to 03/27/96
PZ 36 I
PZ 206 H'
PZ 87 D
PZ 136 I
PZ 136 D
PZ 87 I
PZ 36 D

APPENDIX 10

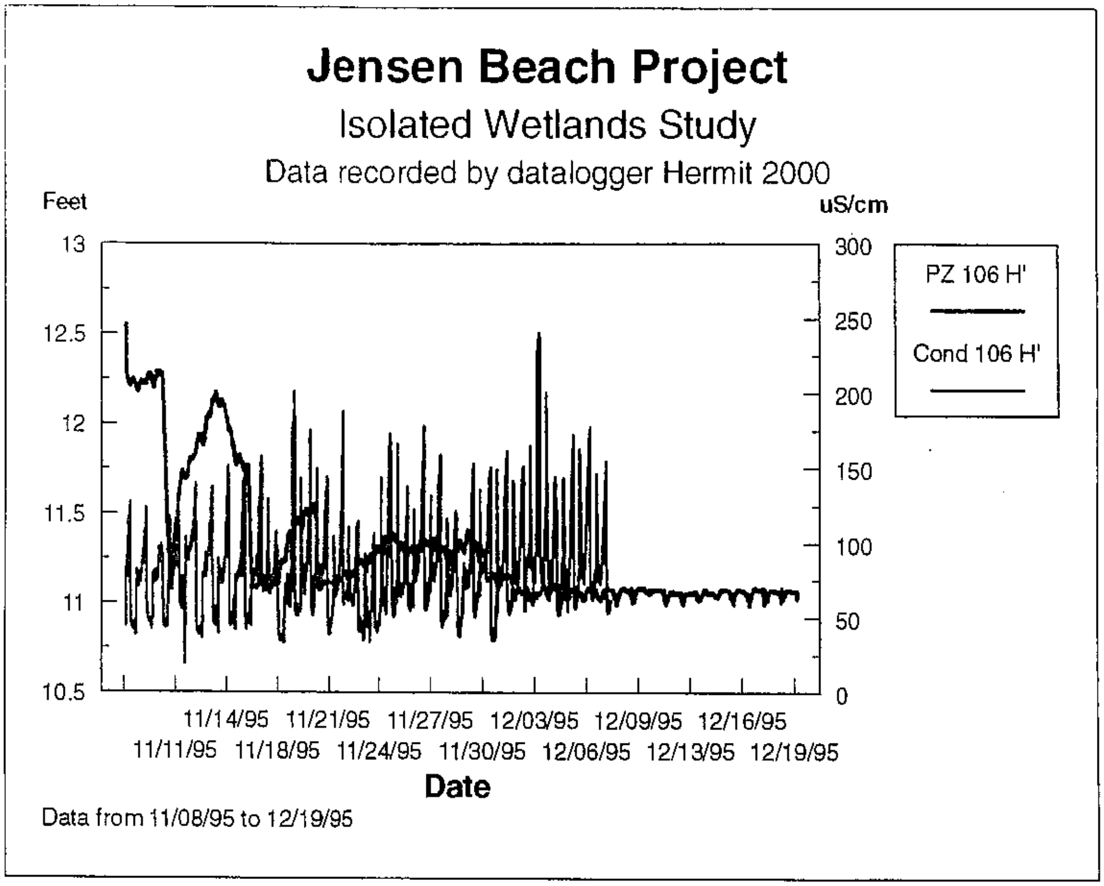
GRAPHS OF CONDUCTIVITY AND TEMPERATURE DURING THE OBSERVATION PERIOD

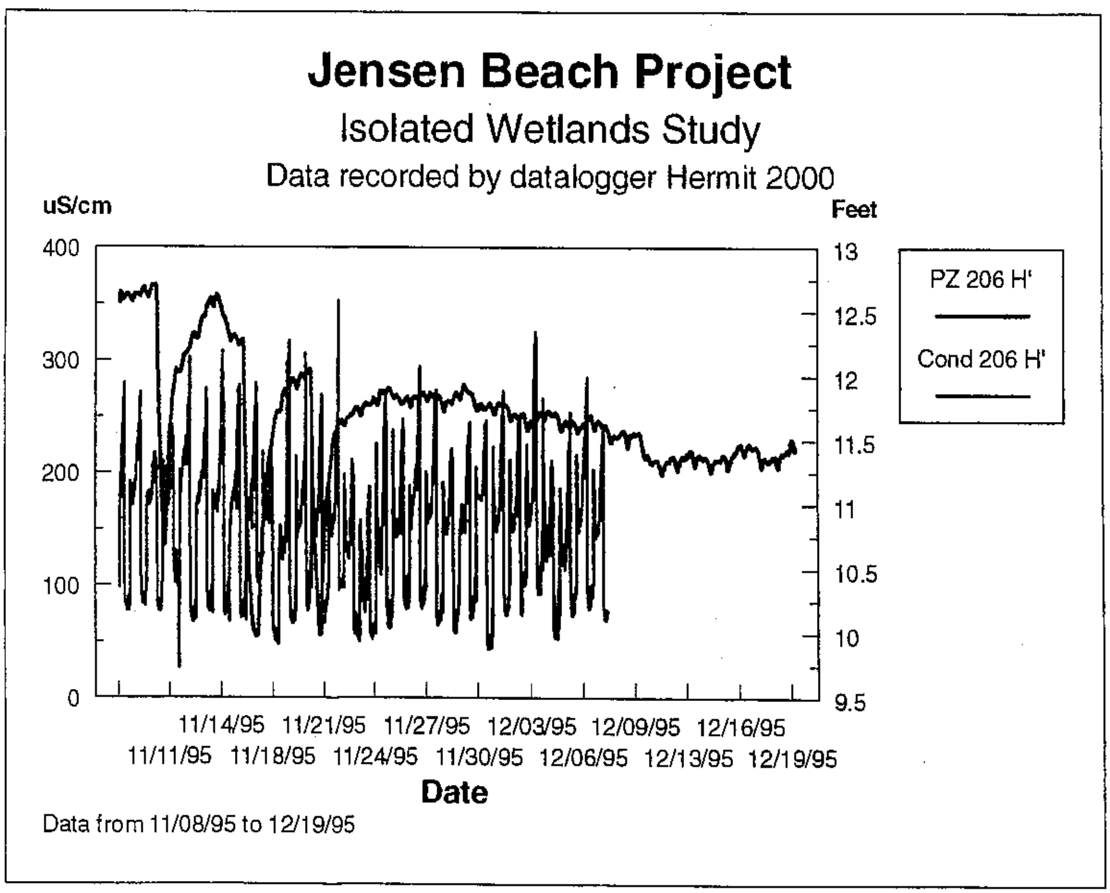


Jensen Beach Project
Isolated wetlands Study
Data recorded by datalogger Hermit 2000
Degree C
27
26
25
24
23
22
Temp 206 H'
Temp 106 H'
Temp 136 D
11/14/95
11/21/95
11/27/95
12/03/95
12/09/95
12/16/95
11/11/95
11/18/95
11/24/95
11/30/95
12/06/95
12/13/95
12/19/95
Date
Data from 11/08/95 to 12/19/95



Jensen Beach Project
Isolated Wetlands Study
Data recorded by datalogger Hermit 2000
uS/cm
400
300
200
100
0
Cond 206 H'
Cond 106 H'
11/14/95
11/21/95
11/27/95
12/03/95
12/09/95
12/16/95
11/11/95
11/18/95
11/24/95
11/30/95
12/06/95
12/13/95
12/19/95
Date
Data from 11/08/95 to 12/19/95


Jensen Beach Project
Isolated Wetlands Study
Data recorded by datalogger Hermit 2000
Feet
uS/cm
13
12.5
12
11.5
11
10.5
300
250
200
150
100
50
0
PZ 106 H'
Cond 106 H'
11/14/95 11/21/95 11/27/95 12/03/95 12/09/95 12/16/95
11/11/95 11/18/95 11/24/95 11/30/95 12/06/95 12/13/95 12/19/95
Date
Data from 11/08/95 to 12/19/95


Jensen Beach Project
Isolated Wetlands Study
Data recorded by datalogger Hermit 2000
uS/cm
Feet
400
300
200
100
0
13
12.5
12
11.5
11
10.5
10
9.5
PZ 206 H'
Cond 206 H'
11/14/95 11/21/95 11/27/95 12/03/95 12/09/95 12/16/95
11/11/95 11/18/95 11/24/95 11/30/95 12/06/95 12/13/95 12/19/95
Date
Data from 11/08/95 to 12/19/95

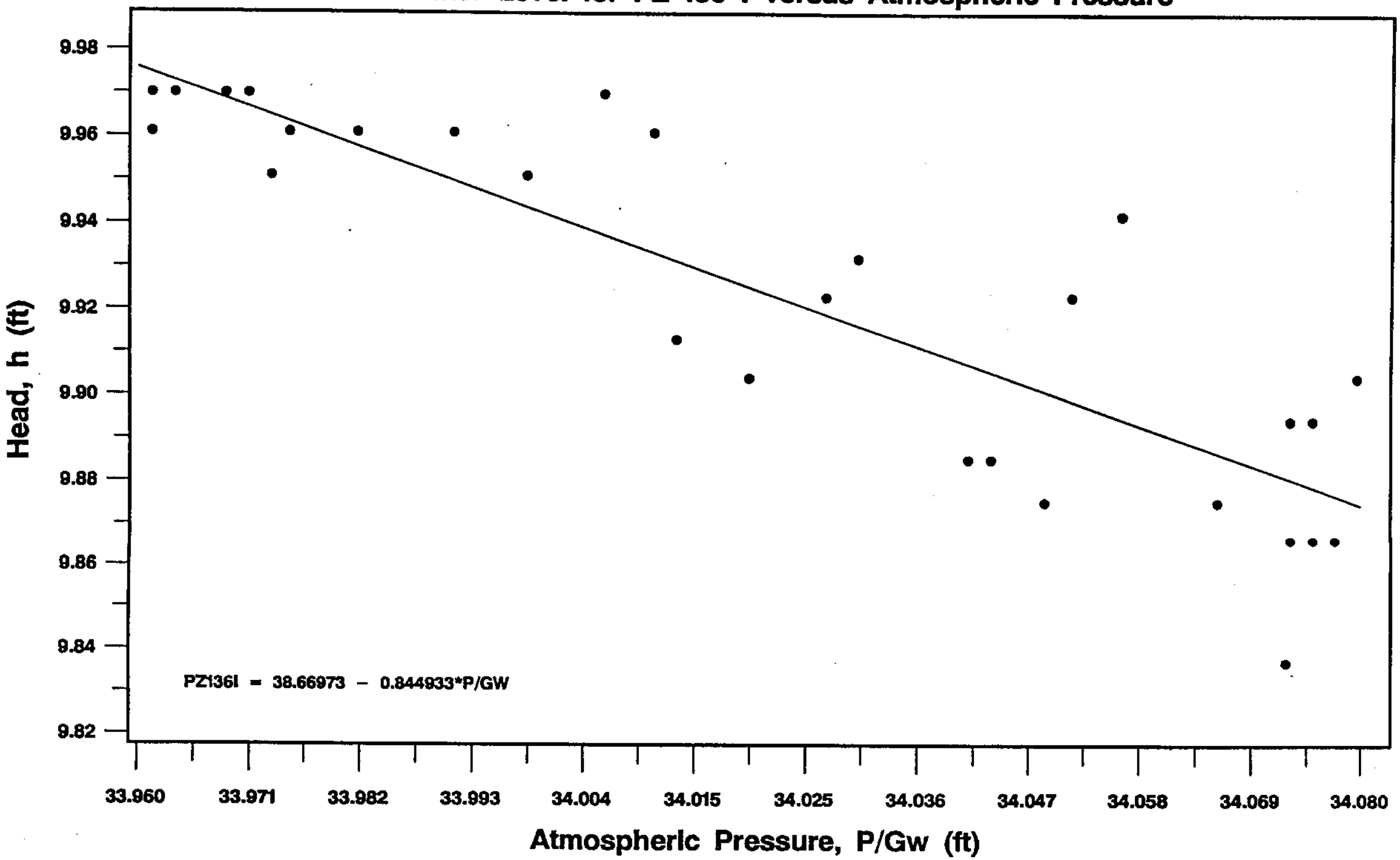
APPENDIX 11

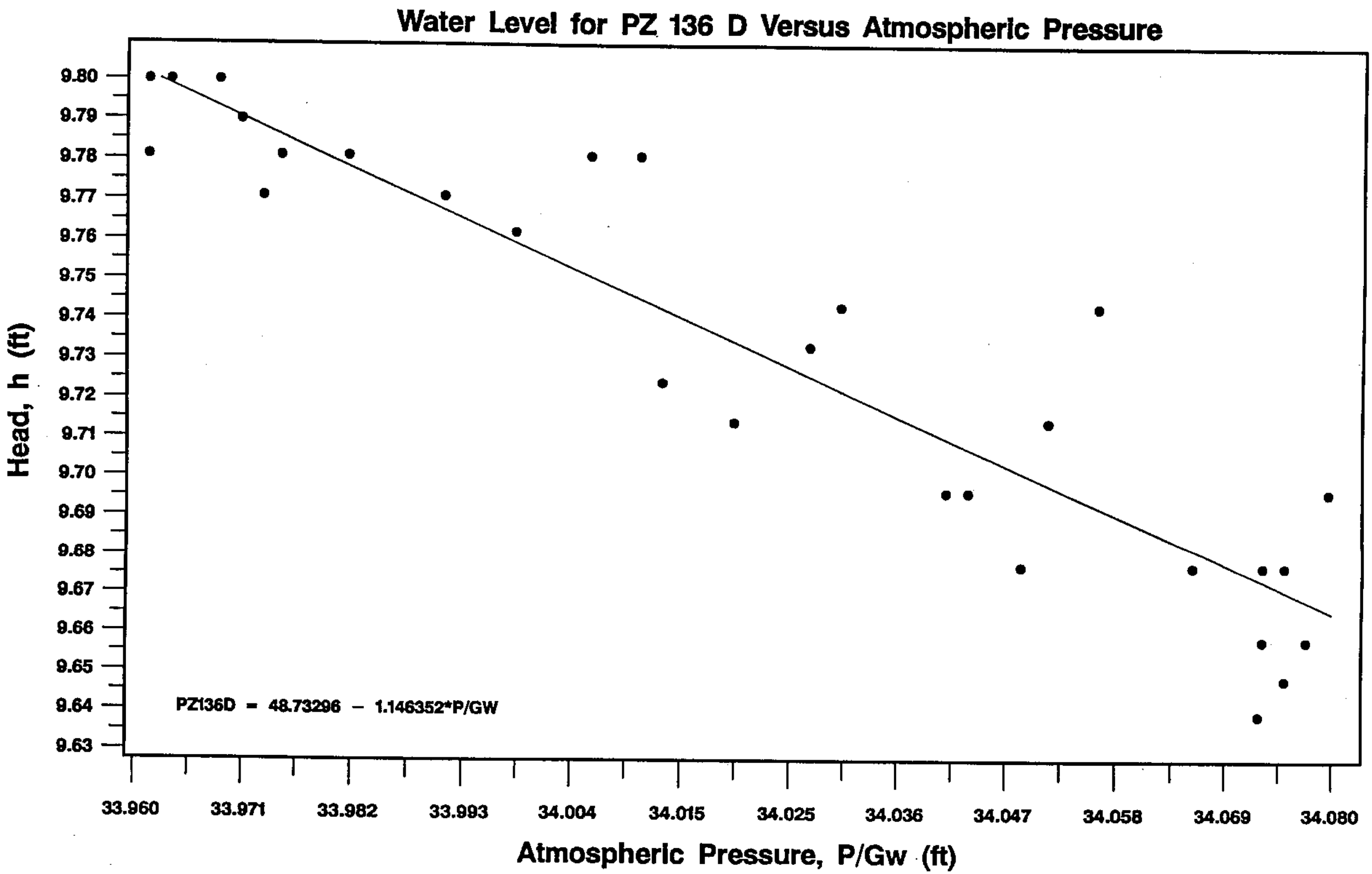
SUPPORTING INFORMATION FOR THE DETERMINATION OF THE BAROMETRIC EFFICIENCY

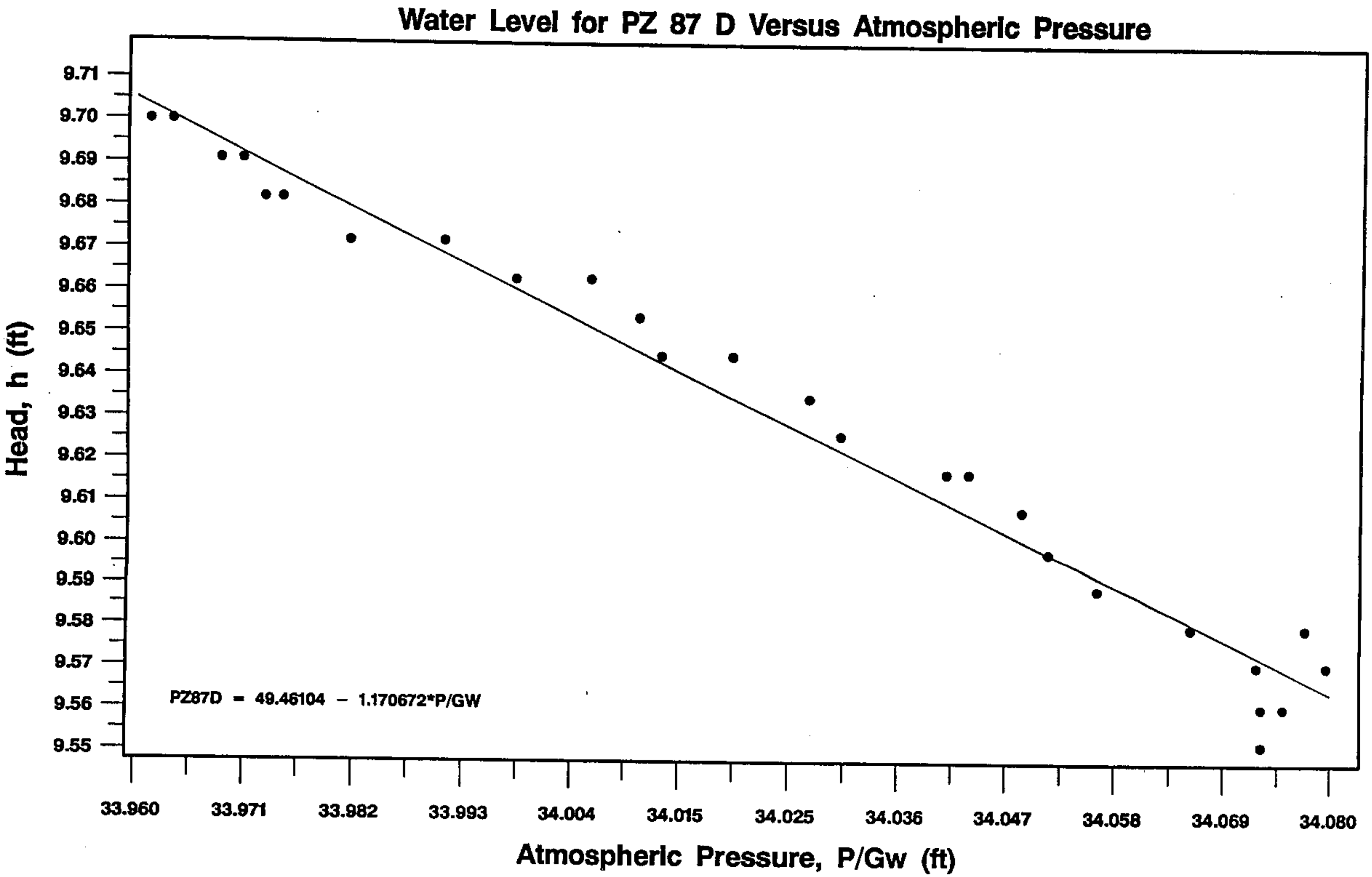
Data collected for 7 hours to determine barometic efficiency

P/Gw	PZ 136 I	PZ 136 D	PZ 87 I	PZ 87 D
FT	MSL IN FT	MSL IN FT	MSL IN FT	MSL IN FT
33.961379	9.97	9.8	9.84	9.7
33.961379	9.961	9.781	9.83	9.7
33.9636106	9.97	9.8	9.83	9.7
33.96852	9.97	9.8	9.84	9.691
33.9707516	9.97	9.79	9.83	9.691
33.9729832	9.951	9.771	9.821	9.682
33.9747684	9.961	9.781	9.83	9.682
33.9814631	9.961	9.781	9.821	9.672
33.9908357	9.961	9.771	9.821	9.672
33.9979768	9.951	9.762	9.811	9.663
34.0055641	9.97	9.781	9.811	9.663
34.0104736	9.961	9.781	9.811	9.654
34.0127051	9.913	9.724	9.792	9.645
34.0198462	9.904	9.714	9.772	9.645
34.0274335	9.923	9.733	9.792	9.635
34.0305577	9.932	9.743	9.782	9.626
34.0412692	9.885	9.696	9.763	9.617
34.0435008	9.885	9.696	9.763	9.617
34.0488566	9.875	9.677	9.753	9.608
34.0515345	9.923	9.714	9.772	9.598
34.0564439	9.942	9.743	9.763	9.589
34.0658165	9.875	9.677	9.743	9.58
34.0725112	9.837	9.639	9.724	9.571
34.0729575	9.866	9.658	9.734	9.561
34.0729575	9.894	9.677	9.743	9.552
34.0751891	9.894	9.677	9.743	9.561
34.0751891	9.866	9.648	9.724	9.561
34.0774207	9.866	9.658	9.734	9.58
34.0796522	9.904	9.696	9.753	9.571

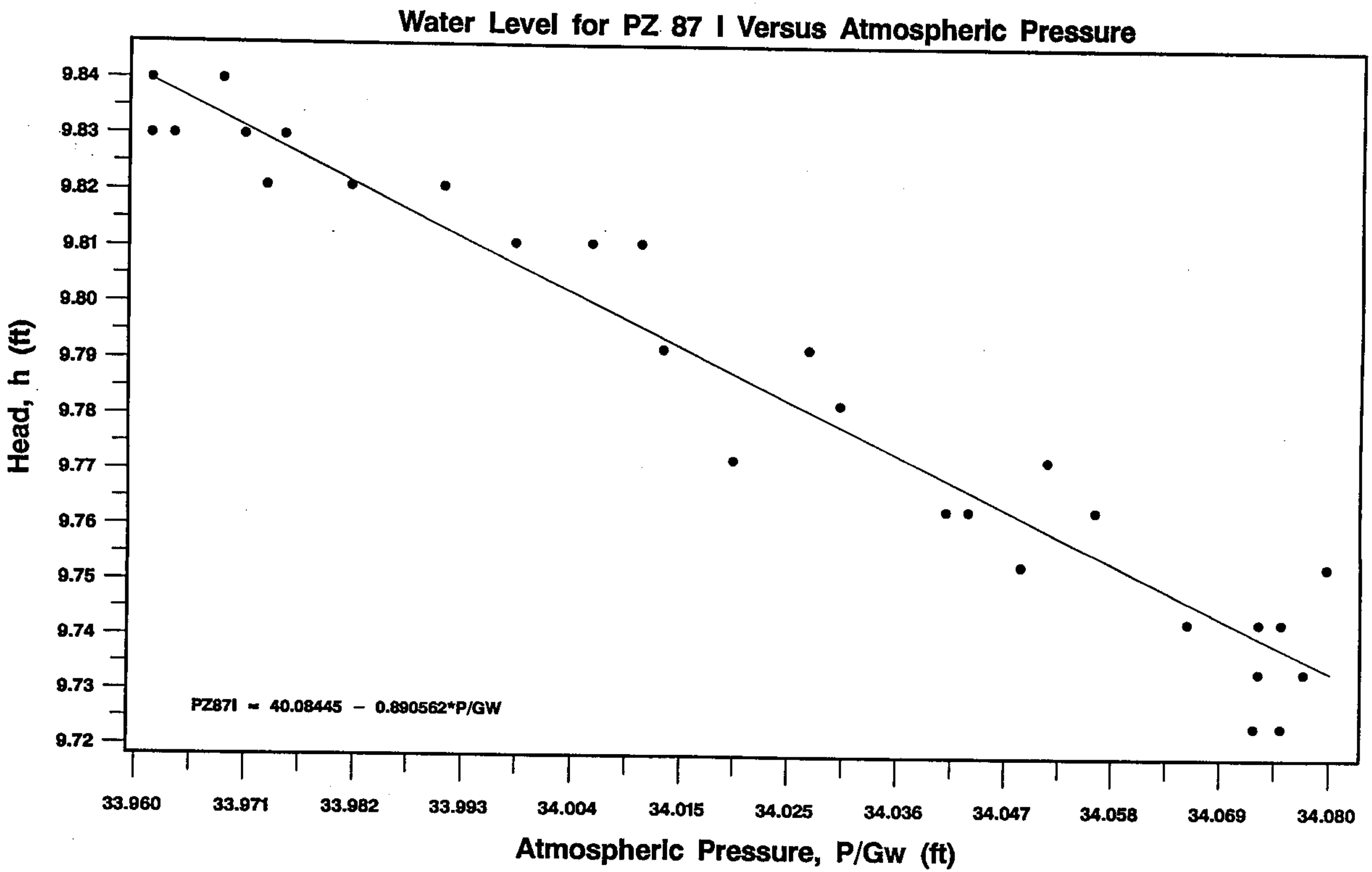
Water Level for PZ 136 I Versus Atmospheric Pressure




Water Level for PZ 136 D Versus Atmospheric Pressure
Head, h (ft)
9.80
9.79
9.78
9.77
9.76
9.75
9.74
9.73
9.72
9.71
9.70
9.69
9.68
9.67
9.66
9.65
9.64
9.63
PZ136D = 48.73296 – 1.146352*P/GW
33.960
33.971
33.982
33.993
34.004
34.015
34.025
34.036
34.047
34.058
34.069
34.080
Atmospheric Pressure, P/Gw (ft)



Water Level for PZ 87 D Versus Atmospheric Pressure
Head, h (ft)
9.71
9.70
9.69
9.68
9.67
9.66
9.65
9.64
9.63
9.62
9.61
9.60
9.59
9.58
9.57
9.56
9.55
PZ87D = 49.46104 - 1.170672*P/GW
33.960
33.971
33.982
33.993
34.004
34.015
34.025
34.036
34.047
34.058
34.069
34.080
Atmospheric Pressure, P/Gw (ft)


Water Level for PZ 87 I Versus Atmospheric Pressure
Head, h (ft)
9.84
9.83
9.82
9.81
9.80
9.79
9.78
9.77
9.76
9.75
9.74
9.73
9.72
PZ87I = 40.08445 - 0.890562*P/GW
33.960
33.971
33.982
33.993
34.004
34.015
34.025
34.036
34.047
34.058
34.069
34.080
Atmospheric Pressure, P/Gw (ft)

Determination of the barometric efficiency

The plots of the water level versus the atmospheric pressure for the four wells give us the following results :

Name of the well	Slope of the line
PZ 136 D	1.17
PZ 87 D	1.14
PZ 136 I	0.844
PZ 87 I	0.89

For the deep aquifer, the slope of the lines are superior to 1. So, we can conclude that the **barometric efficiency is 100 %** (it can not be more than 100 %)

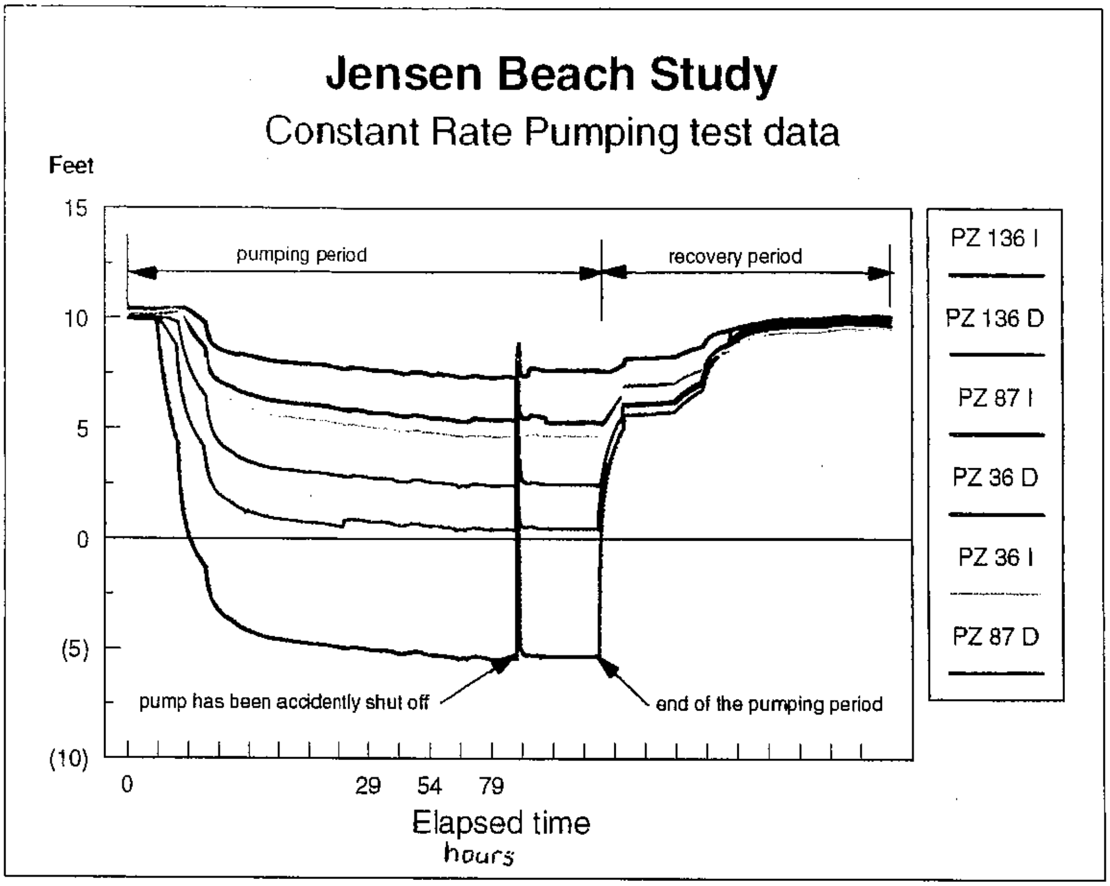
B.E.(deep aquifer) = 100 %

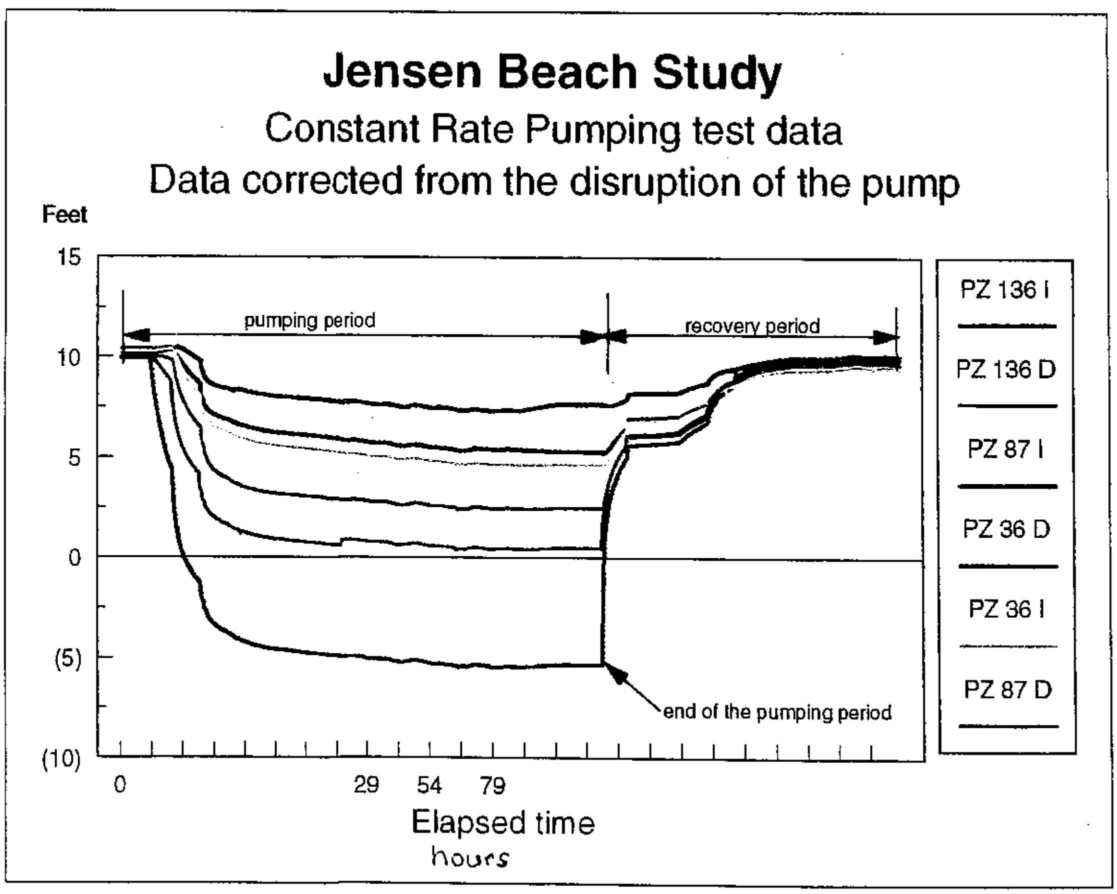
For the intermediate aquifer, the slope of the lines are 0.844 and 0.89. So we can take the average value 0.867. **The barometric efficiency is around 86 %.**

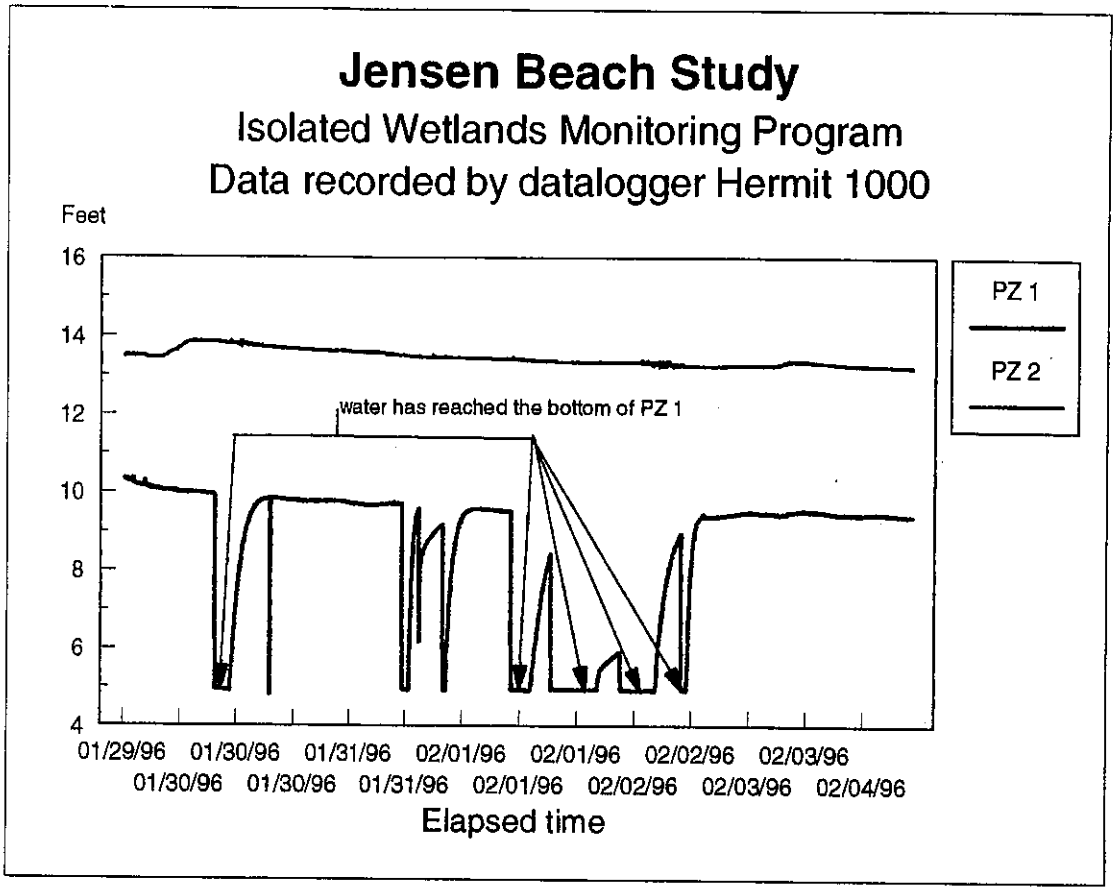
B.E.(intermediate aquifer) = 86 %

APPENDIX 12

HYDROGRAPHS OF THE MONITOR WELLS DURING THE CONSTANT RATE PUMPING TEST


Jensen Beach Study
Constant Rate Pumping test data
Feet
15
10
5
0
(5)
(10)
pumping period
recovery period
pump has been accidently shut off
end of the pumping period
0
29
54
79
Elapsed time
hours
PZ 136 I
PZ 136 D
PZ 87 I
PZ 36 D
PZ 36 I
PZ 87 D

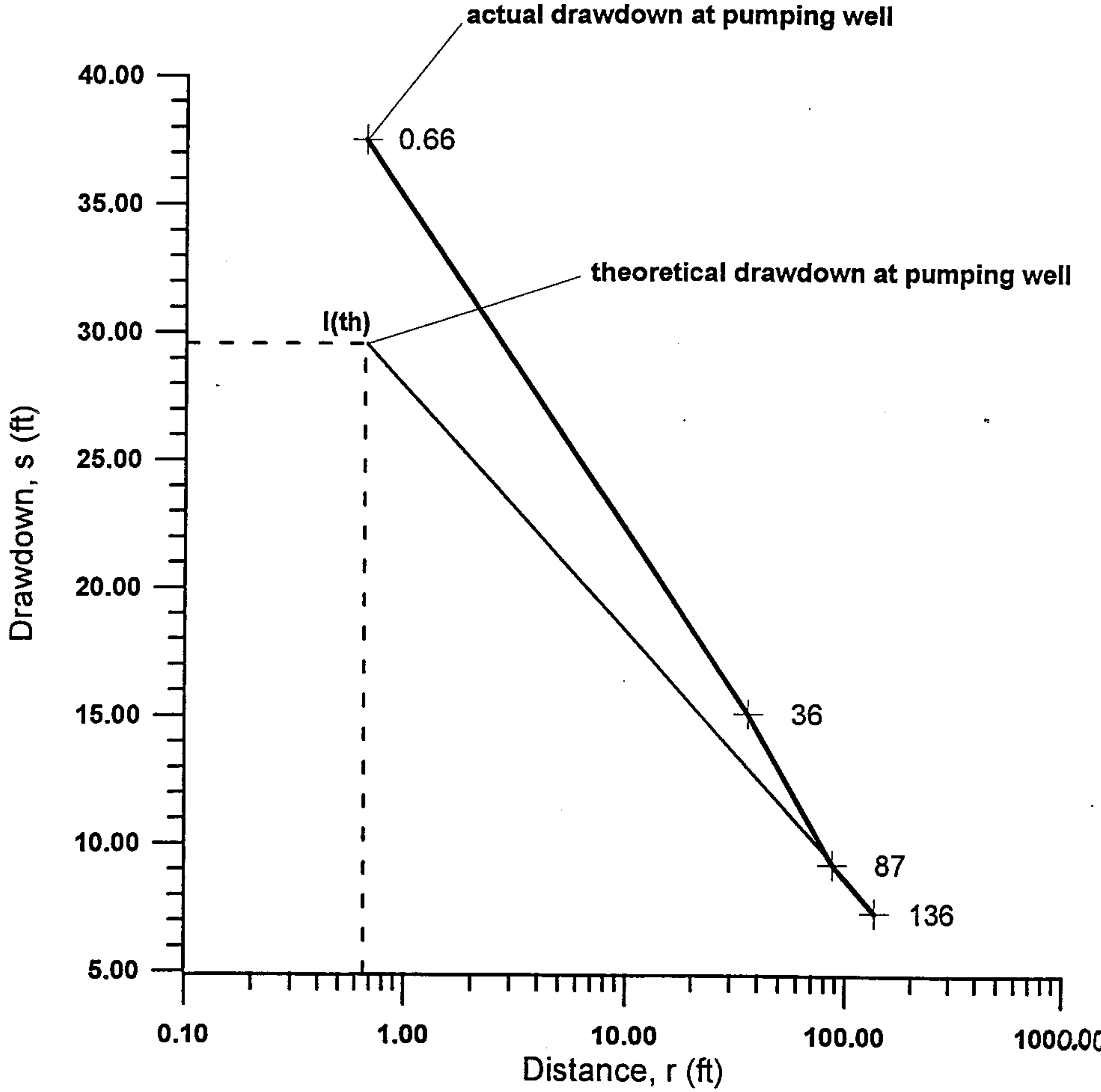

Jensen Beach Study
Constant Rate Pumping test data
Data corrected from the disruption of the pump
Feet
15
10
5
0
(5)
(10)
pumping period
recovery period
end of the pumping period
0
29
54
79
Elapsed time
hours
PZ 136 I
PZ 136 D
PZ 87 I
PZ 36 D
PZ 36 I
PZ 87 D


Jensen Beach Study
Isolated Wetlands Monitoring Program
Data recorded by datalogger Hermit 1000
Feet
16
14
12
10
8
6
4
water has reached the bottom of PZ 1
PZ 1
PZ 2
01/29/96 01/30/96 01/31/96 02/01/96 02/01/96 02/02/96 02/03/96
01/30/96 01/30/96 01/31/96 02/01/96 02/02/96 02/03/96 02/04/96
Elapsed time

APPENDIX 13

SUPPORTING INFORMATION FOR THE DETERMINATION OF THE WELL EFFICIENCY PW-7

Distance-drawdown data used to compute well efficiency 1435 minutes after the pumping began



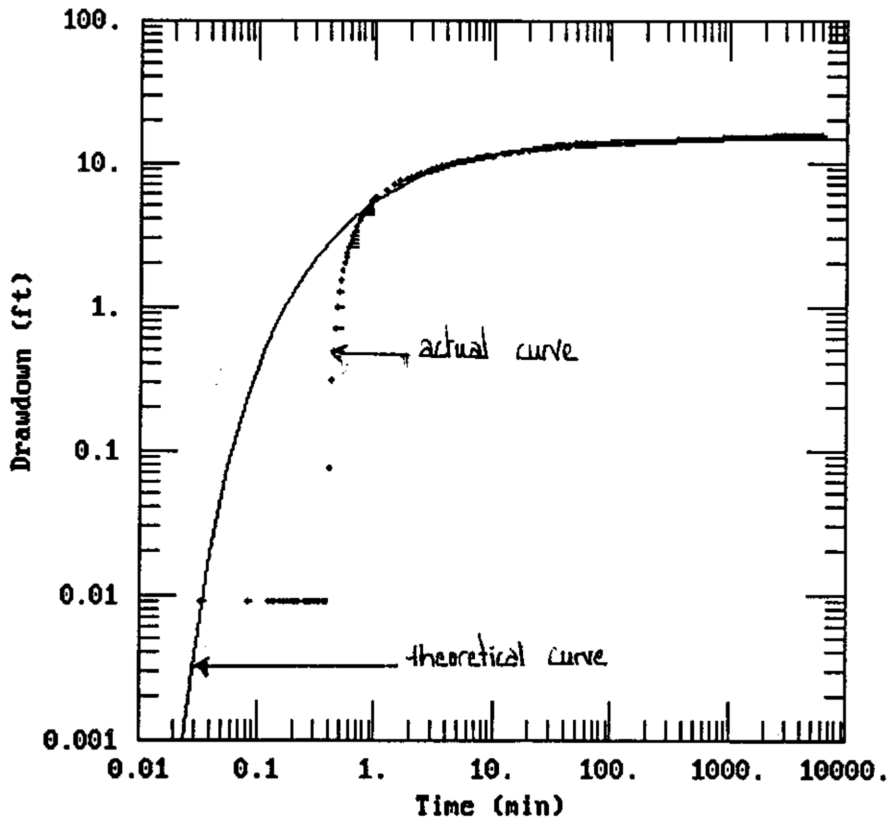
I(th) = (29.6, 0.66)

APPENDIX 14

HANTUSH-JACOB PLOTS FOR THE DEEP AQUIFER DURING THE CONSTANT RATE PUMPING TEST

CLIENT: ENGEES	COMPANY: SFWMD
LOCATION: Jensen Beach	PROJECT: 01

Jensen Beach isolated wetlands study



DATA SET:
PZ36D.DAT
05/29/96

AQUIFER MODEL:
Leaky

SOLUTION METHOD:
Hantush (no stor.)

PROJECT DATA:
test date: 01/29/96 to 02/04/96
test well: PW-7
obs. well: PZ 36D

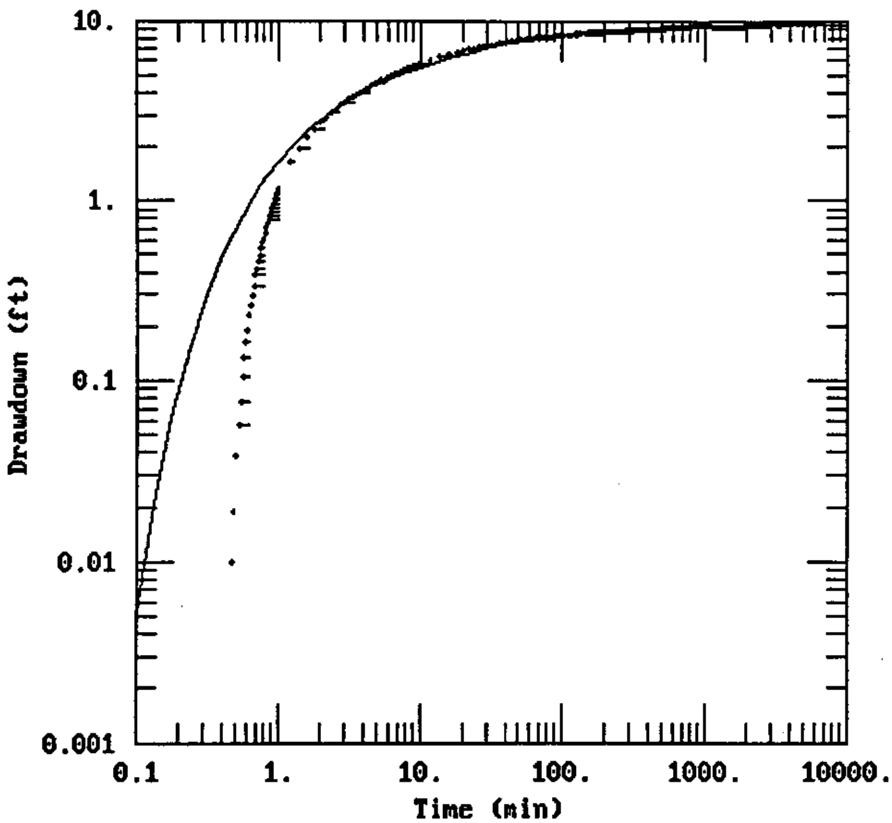
TEST DATA:
Q = 46.8 ft^3/min
r = 36. ft
r_c= 0.333 ft
b = 150. ft
Pumping Well Screen Depth:
top = 71. ft
bot.= 147. ft
Obs. Well Screen Depth:
top = 105. ft
bot.= 110. ft

PARAMETER ESTIMATES:
T = 2.132E+04 gal/day/ft
S = 0.0009072
r/B= 0.09

AQTESOLV

CLIENT: ENGEES	COMPANY: SFWMD
LOCATION: Jensen beach	PROJECT: 2

Jensen beach isolated wetlands study



DATA SET:
P287DCO.DAT
05/29/96

AQUIFER MODEL:
Leaky
SOLUTION METHOD:
Hantush (no stor.)

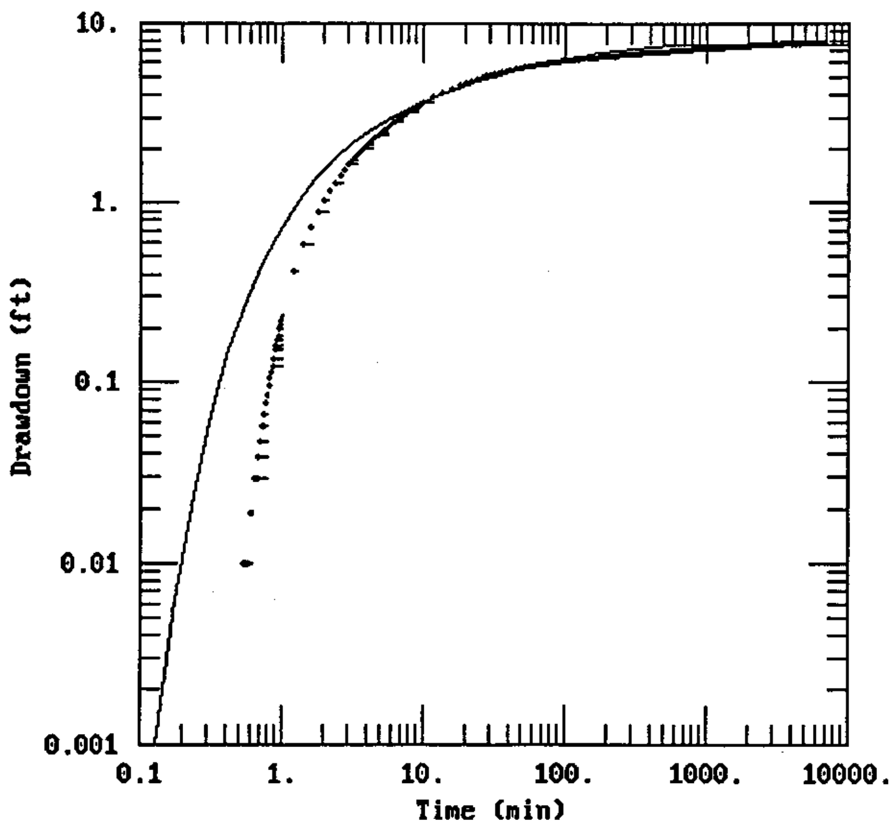
TEST DATA:
Q = 46.8 ft^3/min
r = 87. ft
r_c= 0.333 ft
b = 150. ft
Pumping Well Screen Depth:
top = 71. ft
bot.= 147. ft
Obs. Well Screen Depth:
top = 105. ft
bot.= 110. ft

PARAMETER ESTIMATES:
T = 2.743E+04 gal/day/ft
S = 0.0006227
r/B= 0.09

AQTESOLV

CLIENT: engees	COMPANY: SFWMD
LOCATION: Jensen beach	PROJECT: 2

Jensen beach isolated wetlands study



DATA SET:
PZ136DCO.DAT
05/29/96

AQUIFER MODEL:
Leaky

SOLUTION METHOD:
Hantush (no stor.)

TEST DATA:
Q = 46.8 ft^3/min
r = 136. ft
r_c= 0.333 ft
b = 150. ft
Pumping Well Screen Depth:
top = 71. ft
bot.= 147. ft
Obs. Well Screen Depth:
top = 105. ft
bot.= 110. ft

PARAMETER ESTIMATES:
T = 3.075E+04 gal/day/ft
S = 0.000479
r/B= 0.09

AQTESOLV

APPENDIX 15

LAKE DRAWDOWN SIMULATION

Storativity =	0.0002		Model Input Data		
Transmissivity =	3342.014	ft^2/day			
Vert.Conductivty =	0.038767	ft/day			
Aquitard (D)=	10	ft			
Well Q =	67375	ft^3/day			
r =	100	pts. from	10 to	1000	ft
t =	100	pts. from	0.1 to	720	min

Radial Distance From Pump Feet	Time (Min) 0.10	7.20	14.40	21.60	28.80	36.00	43.20
10	5.84	11.98	12.93	13.44	13.77	14.00	14.17
20	3.68	9.76	10.71	11.22	11.54	11.77	11.94
30	2.50	8.46	9.41	9.92	10.24	10.47	10.64
40	1.73	7.55	8.49	1.00	9.32	9.55	9.72
50	1.20	6.84	7.78	8.29	8.61	8.84	9.01
60	0.83	6.26	7.20	7.71	8.03	8.26	8.43
70	0.56	5.77	6.71	7.22	7.54	7.77	7.94
80	0.38	5.35	6.29	6.79	7.12	7.35	7.52
90	0.25	4.99	5.92	6.42	6.75	6.97	7.14
100	0.16	4.66	5.59	6.09	6.41	6.64	6.81
110	0.10	4.37	5.29	5.79	6.11	6.34	6.51
120	0.06	4.10	5.02	5.52	5.84	6.07	6.24
130	0.04	3.86	4.77	5.27	5.59	5.82	5.98
140	0.02	3.64	4.55	5.04	5.36	5.59	5.75
150	0.01	3.43	4.34	4.83	5.15	5.37	5.54
160	0.01	3.24	4.14	4.63	4.95	5.18	5.34
170	0.00	3.06	3.96	4.45	4.76	4.99	5.15
180	0.00	2.90	3.79	4.27	4.59	4.81	4.98
190	0.00	2.74	3.62	4.11	4.43	4.65	4.81
200	0.00	2.60	3.47	3.96	4.27	4.49	4.66
210	0.00	2.46	3.33	3.81	4.12	4.35	4.51
220	0.00	2.34	3.20	3.67	3.99	4.21	4.37
230	0.00	2.22	3.07	3.54	3.85	4.07	4.24
240	0.00	2.10	2.95	3.42	3.73	3.95	4.11
250	0.00	2.00	2.83	3.30	3.61	3.83	3.99
260	0.00	1.90	2.72	3.19	3.49	3.71	3.88
270	0.00	1.80	2.61	3.08	3.38	3.60	3.77
280	0.00	1.71	2.51	2.97	3.28	3.50	3.66
290	0.00	1.62	2.42	2.88	3.18	3.40	3.55
300	0.00	1.54	2.33	2.78	3.08	3.30	3.46
310	0.00	1.46	2.24	2.69	2.99	3.21	3.36
320	0.00	1.39	2.16	2.60	2.90	3.12	3.27
330	0.00	1.32	2.08	2.52	2.82	3.03	3.18
340	0.00	1.25	2.00	2.44	2.73	2.95	3.11
350	0.00	1.19	1.92	2.36	2.65	2.87	3.02
360	0.00	1.13	1.85	2.29	2.58	2.79	2.94
370	0.00	1.07	1.78	2.21	2.50	2.71	2.87
380	0.00	1.02	1.72	2.14	2.43	2.64	2.80
390	0.00	0.97	1.66	2.08	2.36	2.57	2.73
400	0.00	0.92	1.60	2.01	2.30	2.50	2.66

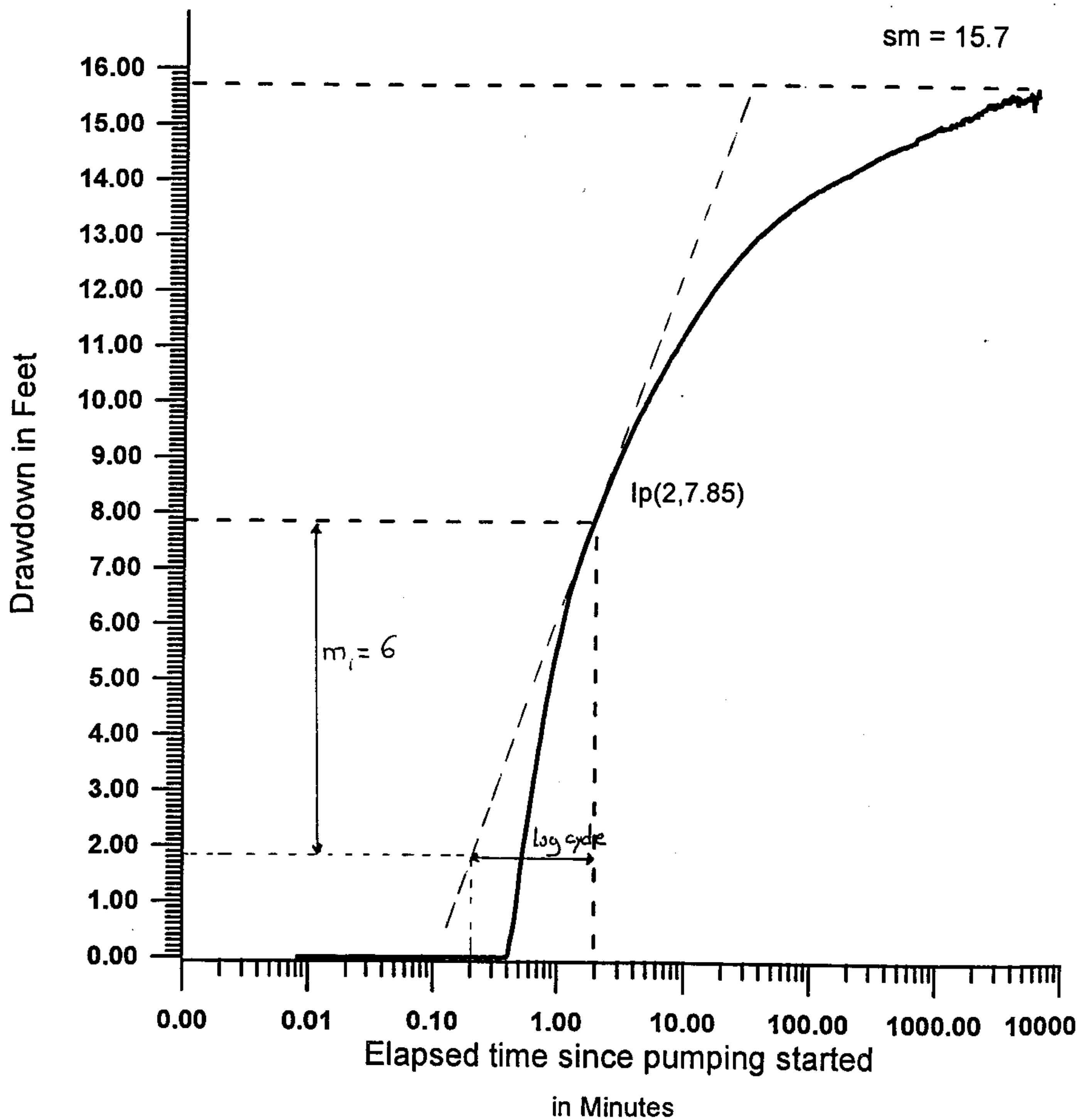
Radial Distance From Pump Feet	Time (Min) 0.10	7.20	14.40	21.60	28.80	36.00	43.20
410	0.00	0.87	1.54	1.95	2.23	2.44	2.59
420	0.00	0.82	1.48	1.89	2.17	2.37	2.53
430	0.00	0.78	1.43	1.83	2.11	2.31	2.47
440	0.00	0.74	1.37	1.77	2.05	2.25	2.41
450	0.00	0.70	1.32	1.72	1.99	2.20	2.00
460	0.00	0.67	1.28	1.67	1.94	2.14	2.29
470	0.00	0.63	1.23	1.62	1.89	2.08	2.24
480	0.00	0.60	1.18	1.57	1.83	2.03	2.18
490	0.00	0.57	1.14	1.52	1.78	1.98	2.13
500	0.00	0.54	1.10	1.47	1.74	1.93	2.08
510	0.00	0.51	1.06	1.43	1.69	1.88	2.03
520	0.00	0.48	1.02	1.38	1.64	1.83	1.98
530	0.00	0.45	0.98	1.34	1.60	1.79	1.94
540	0.00	0.43	0.95	1.30	1.56	1.74	1.89
550	0.00	0.41	0.91	1.26	1.51	1.70	1.85
560	0.00	0.38	0.88	1.22	1.47	1.66	1.80
570	0.00	0.36	0.84	1.19	1.43	1.62	1.76
580	0.00	0.34	0.81	1.15	1.39	1.58	1.72
590	0.00	0.32	0.78	1.11	1.36	1.54	1.68
600	0.00	0.30	0.75	1.08	1.32	1.50	1.64
610	0.00	0.29	0.72	1.05	1.29	1.46	1.60
620	0.00	0.27	0.70	1.02	1.25	1.43	1.57
630	0.00	0.26	0.67	0.98	1.22	1.39	1.53
640	0.00	0.24	0.65	0.95	1.18	1.36	1.50
650	0.00	0.23	0.62	0.93	1.15	1.33	1.46
660	0.00	0.21	0.60	0.90	1.12	1.29	1.43
670	0.00	0.20	0.57	0.87	1.09	1.26	1.40
680	0.00	0.19	0.55	0.84	1.06	1.23	1.36
690	0.00	0.18	0.53	0.82	1.03	1.20	1.33
700	0.00	0.17	0.51	0.79	1.01	1.17	1.30
710	0.00	0.16	0.49	0.77	0.98	1.14	1.27
720	0.00	0.15	0.47	0.74	0.95	1.12	1.24
730	0.00	0.14	0.45	0.72	0.93	1.09	1.22
740	0.00	0.13	0.44	0.70	0.90	1.06	1.19
750	0.00	0.12	0.42	0.68	0.88	1.04	1.16
760	0.00	0.11	0.40	0.65	0.85	1.01	1.14
770	0.00	0.11	0.39	0.60	0.83	0.83	0.99
780	0.00	0.10	0.37	0.61	0.81	0.96	1.08
790	0.00	0.09	0.36	0.60	0.79	0.94	1.06
800	0.00	0.09	0.34	0.58	0.77	0.92	1.04
810	0.00	0.08	0.33	0.56	0.74	0.89	1.01
820	0.00	0.08	0.32	0.54	0.72	0.87	0.99
830	0.00	0.07	0.30	0.52	0.70	0.85	0.97

At 800 feet (250 meters) from the pumping well, the drawdown observed after 7 minutes of pumping is 0.08 feet (2.4 cm) and after 14 minutes of pumping the drawdown is 0.34 feet (10 cm)

APPENDIX 16

SUPPORTING INFORMATION FOR THE HANTUSH INFLECTION-POINT METHOD FOR THE DEEP AQUIFER DURING THE CONSTANT RATE PUMPING TEST

Hantush Inflection-Point Method
PZ 36 D



Hantush Inflection-Point Method for observation well PZ 36 D

- From the plot of s versus log(t), the following inflection point values were determined.

  $s_m = 15.7$ ft
  $s_i = 7.85$ ft
  $t_i = 2$ min
  $m_i = 6$ ft

- Using the Hantush equation (1956) gives :

$$\exp(r/B)K_o(r/B) = 2.3 \times \frac{s_i}{m_i} = 2.3 \times \frac{7.85}{6} = 3.0091$$

From table 9.3 from Aquifer Testing, Karen J. Dawson and Jonathan D. Istok 1991,

when $\exp(x)\,K_0(x) = 3.0091$ than $x = 0.0677$ and $K_0(x) = 2.814$

Therefore, $r/B = 0.0677$ and **B = 36 / 0.0677 = 531.7 ft.**

- Using equation (3) we obtain :

$$T = \frac{Q \times K_0(r/B)}{4\pi \times s_i} = \frac{350\frac{gal}{day} \times 2.814 \times 1440\frac{\min}{day}}{4\pi \times 7.85(ft)} = 14384\frac{gal}{day \cdot ft}$$

T = 14 384 gpd/ft = 178 m²/d

• Using equation (1) we obtain

$$S = \frac{2Tt_i}{rB} = \frac{2(14384\frac{gal}{day\cdot ft})\times(2\min)\times(\frac{1day}{1440\min})\times(\frac{1ft^3}{7.48gal})}{36(ft)\times 531.7(ft)} = 0.000558$$

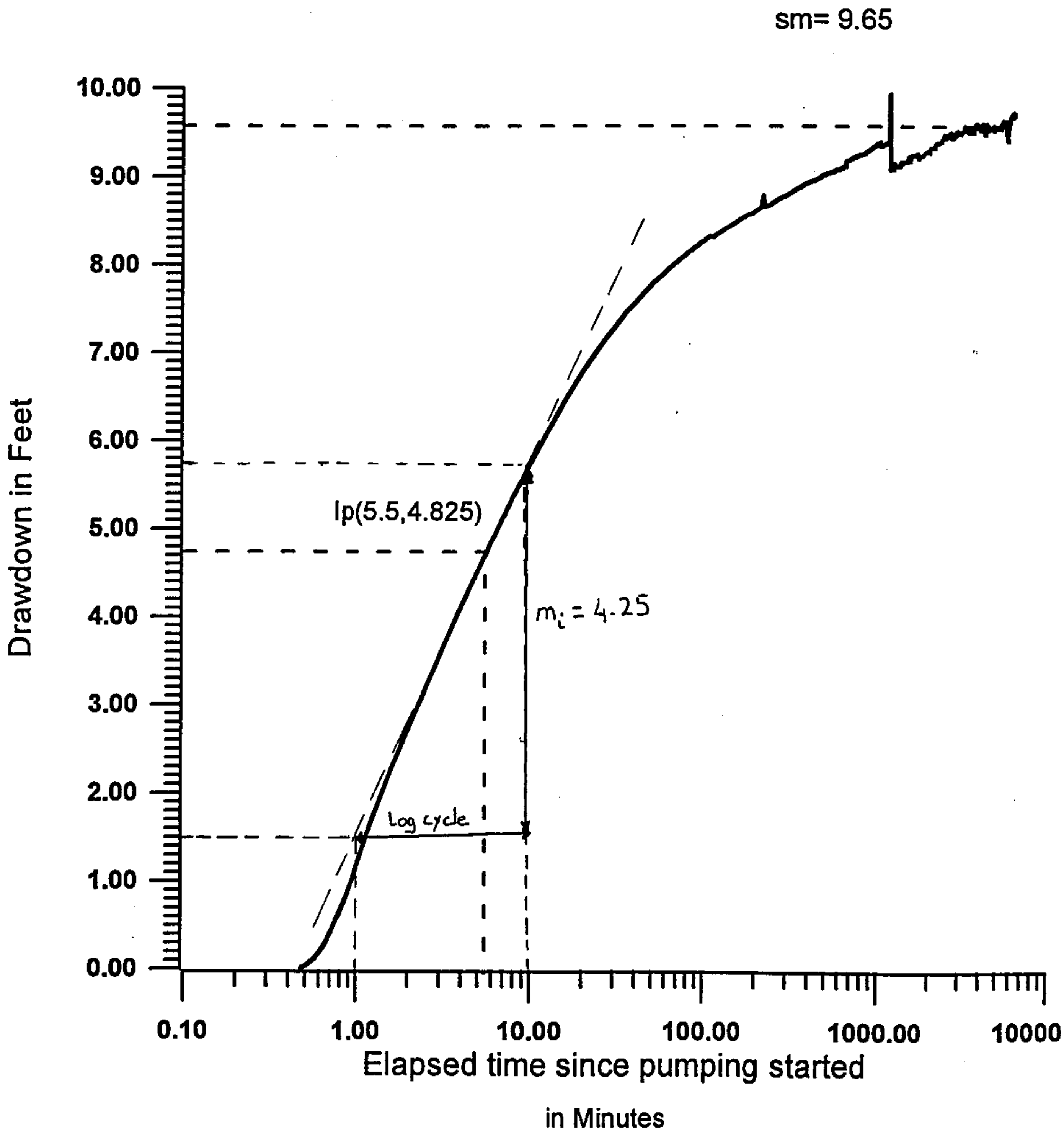
S = 0.000558

• We have r/B = 0.067>0.05. Therefore we can use this equation to obtain K'

$$K' = \frac{Tm'}{B^2} = \frac{14384(\frac{gal}{day\cdot ft})\times 10(ft)}{(531.7)^2(ft)} = 0.51\frac{gal}{day\cdot ft^2} = 0.51\frac{1}{3.28\cdot 7.48}\frac{m}{day}$$

K' = 0.51 gpd/ft² = 0.02 m/d.

Hantush Inflection-Point Method
PZ 87 D



Hantush Inflection-Point Method for observation well PZ 87 D

- From the plot of s versus log(t), the following inflection point values were determined.

  $s_m = 9.65$ ft
  $s_i = 4.825$ ft
  $t_i = 5.5$ min
  $m_i = 4.25$ ft

- Using the Hantush equation (1956) gives :

$$\exp(r/B)K_o(r/B) = 2.3 \times \frac{s_i}{m_i} = 2.3 \times \frac{4.825}{4.25} = 2.6112$$

- From table 9.3 from Aquifer Testing, Karen J. Dawson and Jonathan D. Istok 1991,

when $\exp(x)\ K_0(x) = 2.6112$

$\Rightarrow$ than $x = 0.109$ and $K_0(x) = 2.342$

Therefore, $r/B = 0.109$ and **B = 87 / 0.109 = 798.16 ft.**

- Using equation (3) we obtain :

$$T = \frac{Q \times K_0(r/B)}{4\pi \times s_i} = \frac{350 \frac{gal}{day} \times 2.342 \times 1440 \frac{\min}{day}}{4\pi \times 4.825(ft)} = 19477 \frac{gal}{day \cdot ft}$$

T = 19477 gpd/ft = 242 m²/d

• Using equation (1) we obtain

$$S = \frac{2Tt_i}{rB} = \frac{2(19477\frac{gal}{day \cdot ft}) \times (5.5\min) \times (\frac{1day}{1440\min}) \times (\frac{1ft^3}{7.48gal})}{87(ft) \times 798.2(ft)} = 0.0002864$$

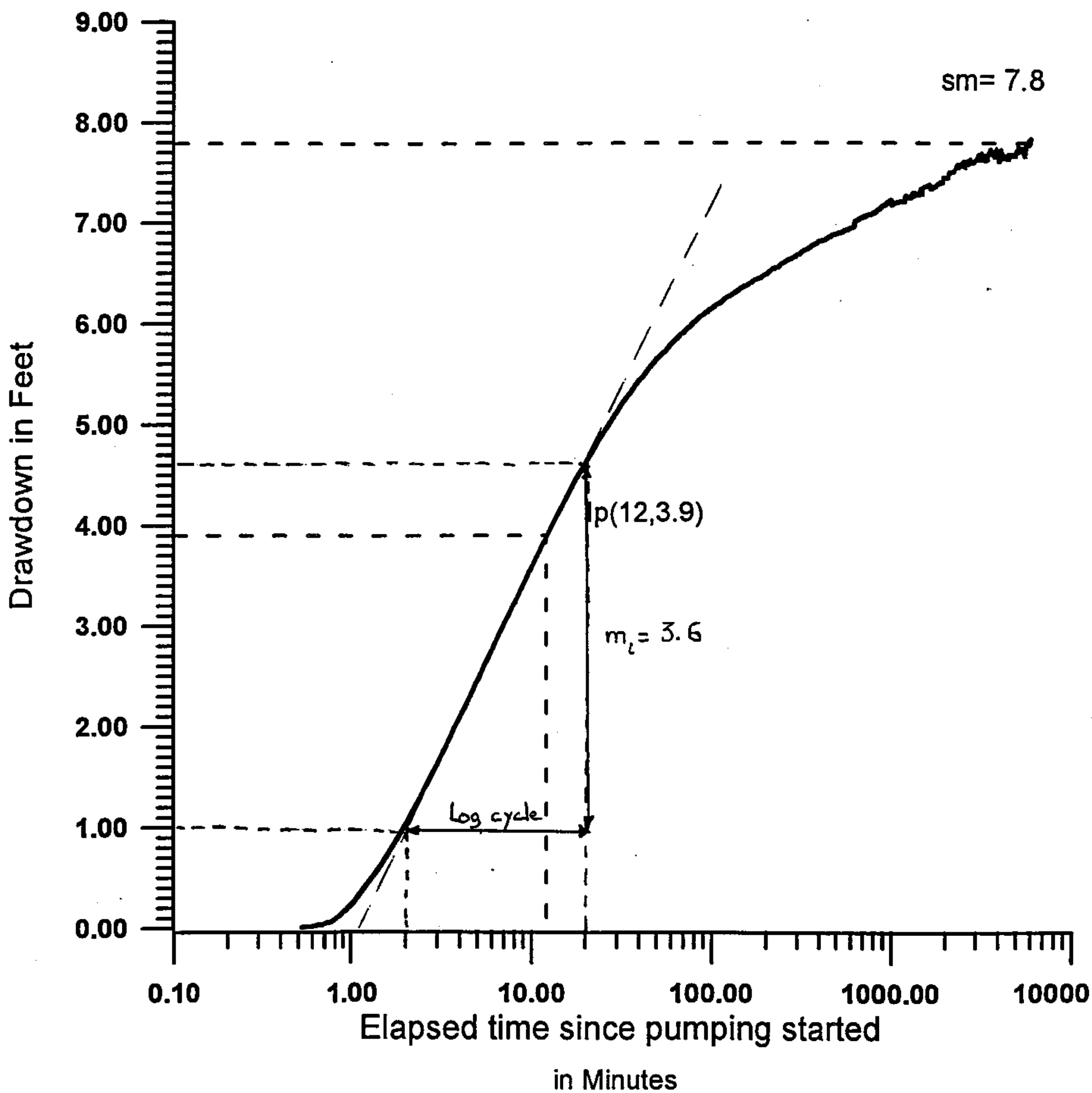
S = 0.0002864

• We have r/B = 0.109>0.05. Therefore we can use this equation to obtain K' :

$$K' = \frac{Tm'}{B^2} = \frac{19477(\frac{gal}{day \cdot ft}) \times 10(ft)}{(798)^2(ft)} = 0.305\frac{gal}{day \cdot ft^2}$$

K' = 0.305 gpd/ft² = 0.012 m/d

Hantush Inflection-Point Method
PZ 136 D



Hantush Inflection-Point Method for observation well PZ 136 D

- From the plot of s versus log(t), the following inflection point values were determined.

$s_m = 7.8$
$s_i = 3.9$ ft
$t_i = 12$ min
$m_i = 3.6$ ft

- Using the Hantush equation (1956) gives :

$$\exp(r/B)K_o(r/B) = 2.3\times\frac{s_i}{m_i} = 2.3\times\frac{3.9}{3.6} = 2.491$$

- From table 9.3 from Aquifer Testing, Karen J. Dawson and Jonathan D. Istok 1991,

when $\exp(x)\,K_0(x) = 2.491$

$\Rightarrow\ x = 0.1269$ and $K_0(x) = 2.193$

Therefore, r/B = 0.1269 and **B = 136 / 0.1269 = 1071.7 ft.**

- Using equation (3) we obtain :

$$T = \frac{Q\times K_0(r/B)}{4\pi\times s_i} = \frac{350\frac{gal}{day}\times 2.193\times 1440\frac{\min}{day}}{4\pi\times 3.9(ft)} = 22564\frac{gal}{day\cdot ft}$$

$$\boxed{\mathbf{T = 22564\ gpd/ft = 280\ m^2/d}}$$

• Using equation (1) we obtain

$$S=\frac{2Tt_i}{rB}=\frac{2(22564\frac{gal}{day\cdot ft})\times(12\,\text{min})\times(\frac{1day}{1440\,\text{min}})\times(\frac{1ft^3}{7.48gal})}{136(ft)\times1071.7(ft)}=0.0003449$$

$$\boxed{\mathbf{S = 0.0003449}}$$

• We have r/B = 0.1269 > 0.05. Therefore we can use this equation to obtain K' :

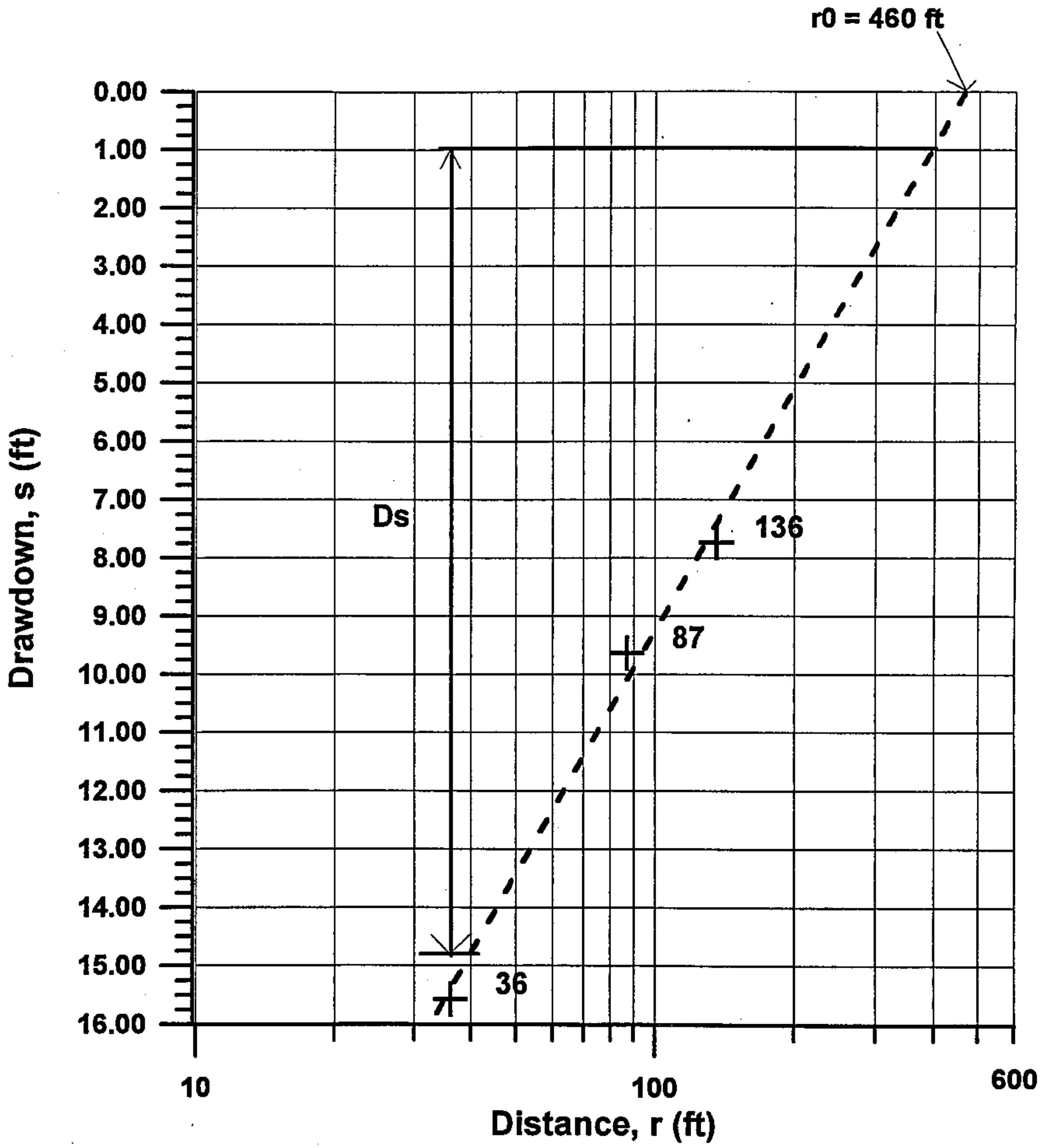
$$K'=\frac{Tm'}{B^2}=\frac{22564(\frac{gal}{day\cdot ft})\times10(ft)}{(1071.7)^2(ft)}=0.196\frac{gal}{day\cdot ft^2}=0.196\frac{1}{7.48\cdot3.28}m/d$$

$$\boxed{\mathbf{K' = 0.1976\ gpd/ft^2 = 0.008\ m/d}}$$

APPENDIX 17

PLOT OF THE DISTANCE-DRAWDOWN ANALYSIS OF THE DEEP AQUIFER FOR THE CONSTANT RATE PUMPING TEST

Distance-drawdown method for t = 1435 min

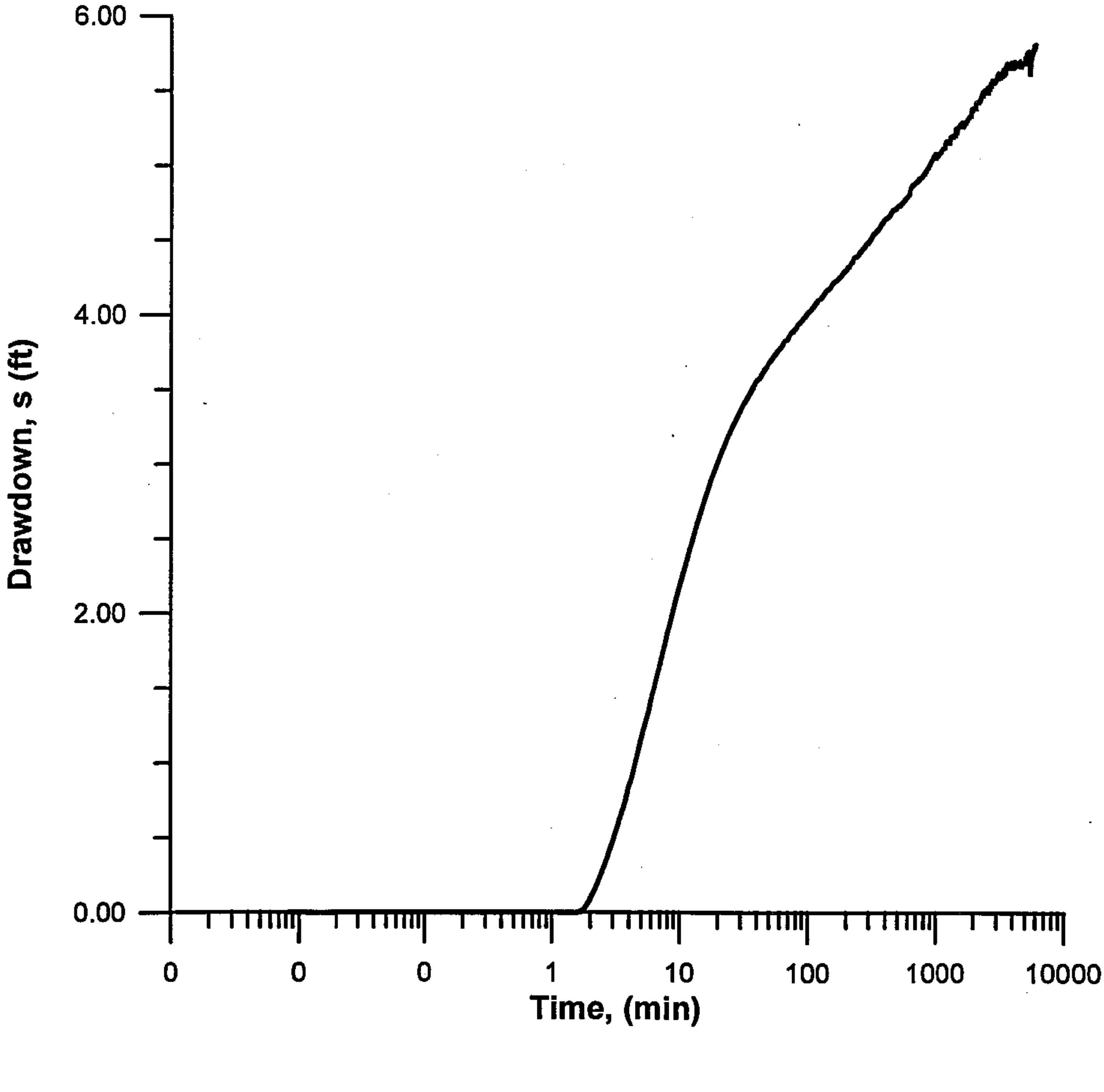


Ds = 14.75-1 = 13.75 ft

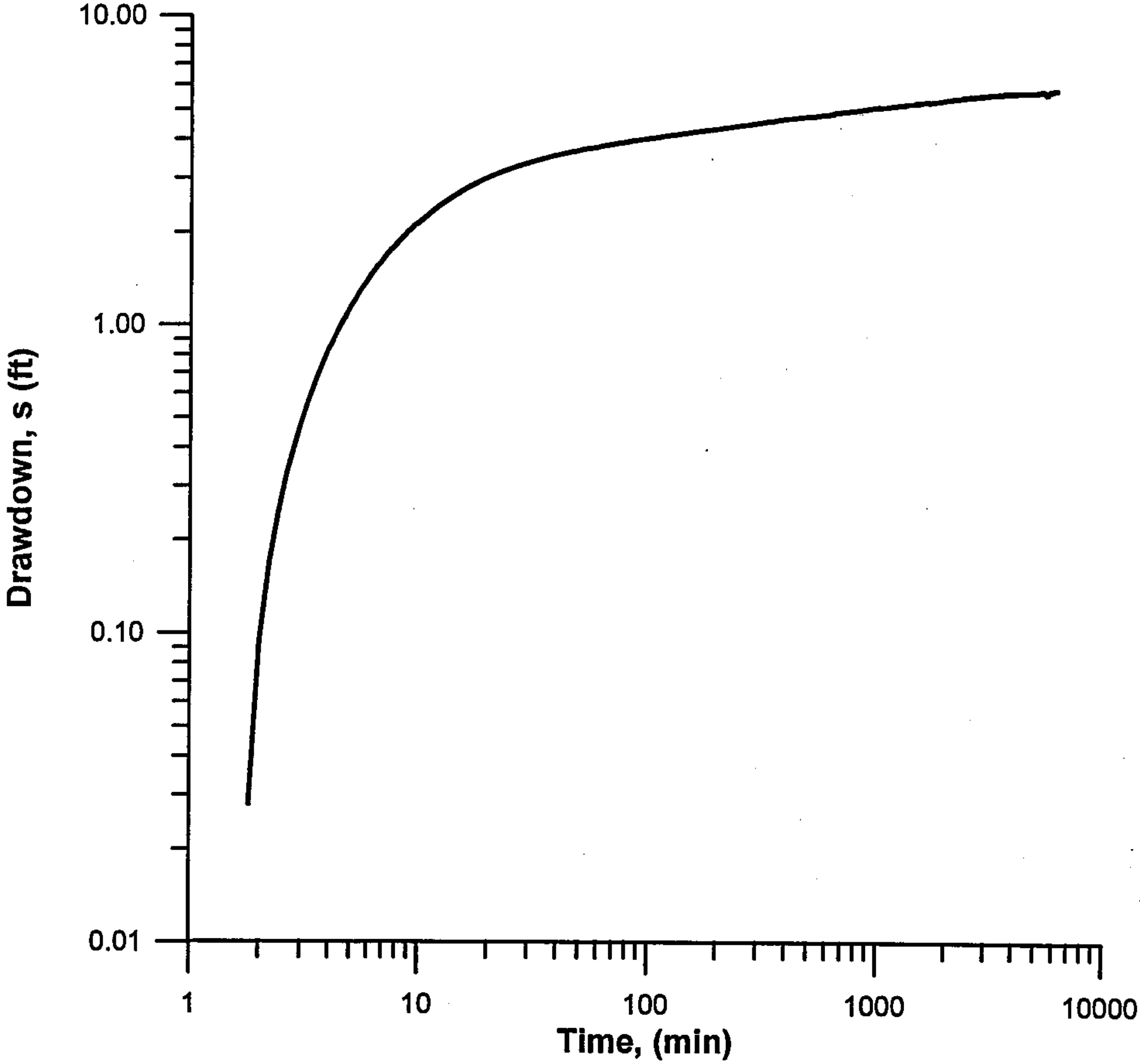
APPENDIX 18

LOG-LOG AND SEMI-LOG PLOTS OF THE TIME-DRAWDOWN RELATIONSHIPS FOR THE INTERMEDIATE AQUIFER DURING THE CONSTANT RATE PUMPING TEST

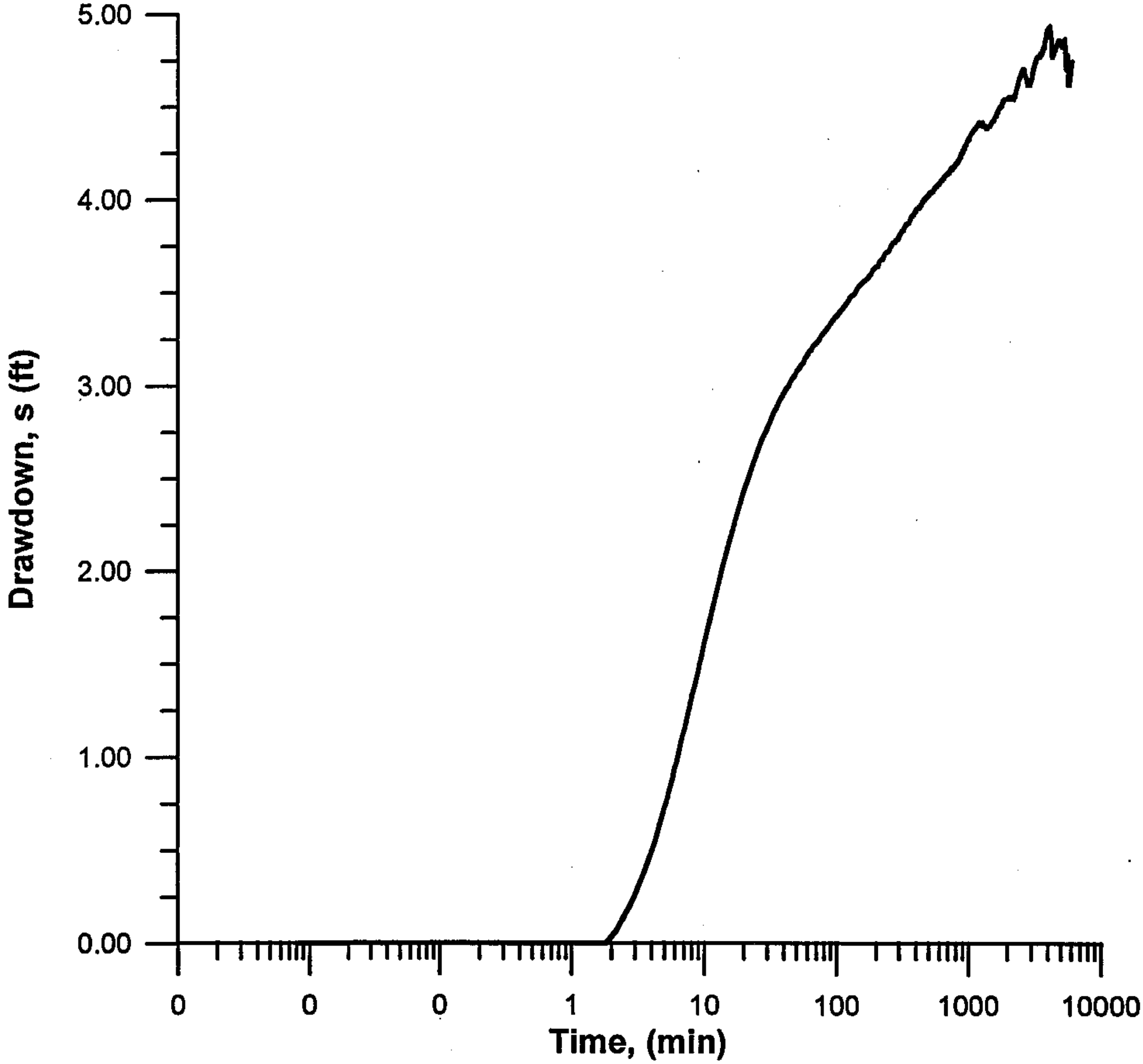
Semi-log plot of the time-drawdown relationships for well PZ 36 I



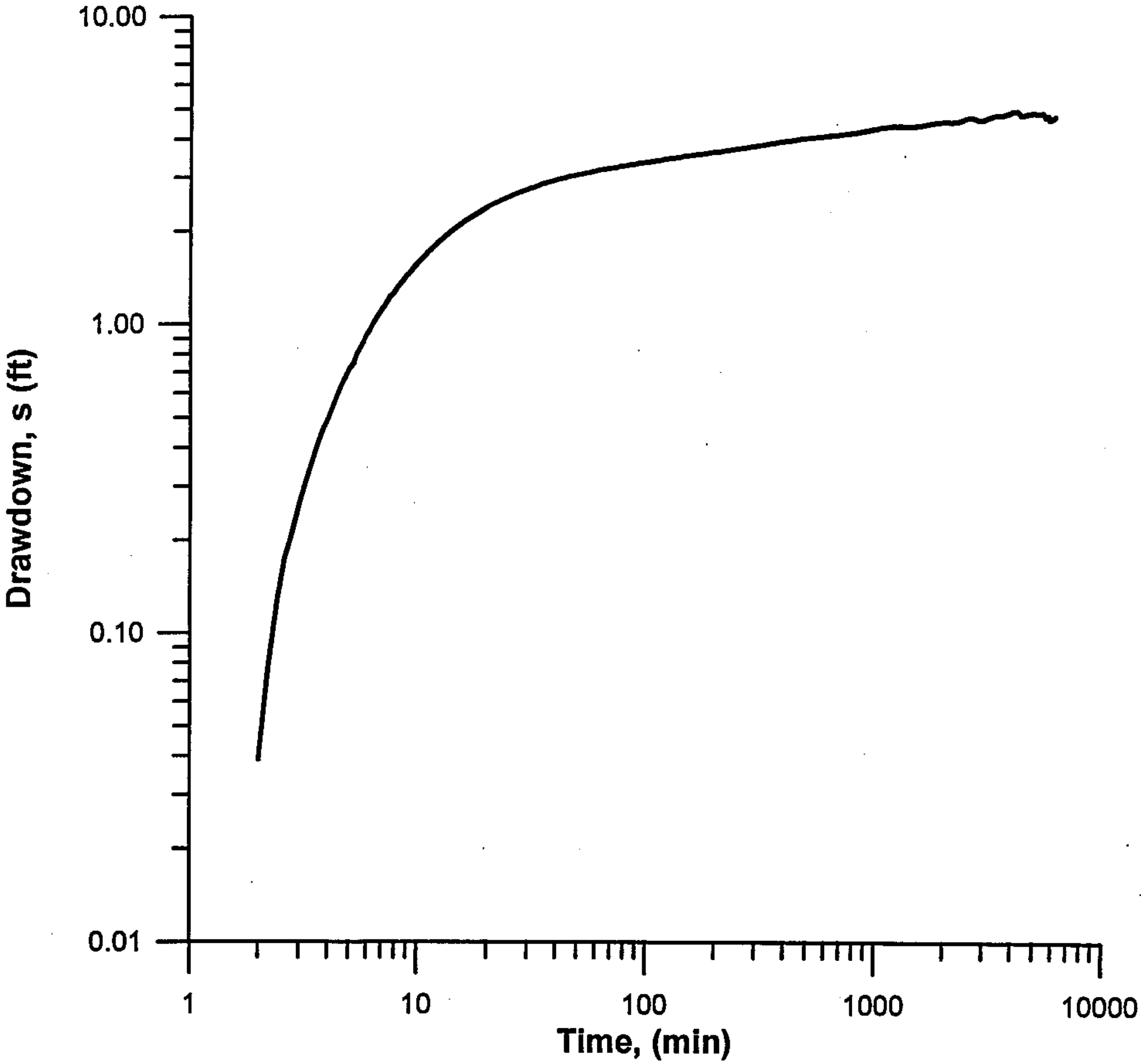
Log-log plot of the time-drawdown relationships for well PZ 36 I



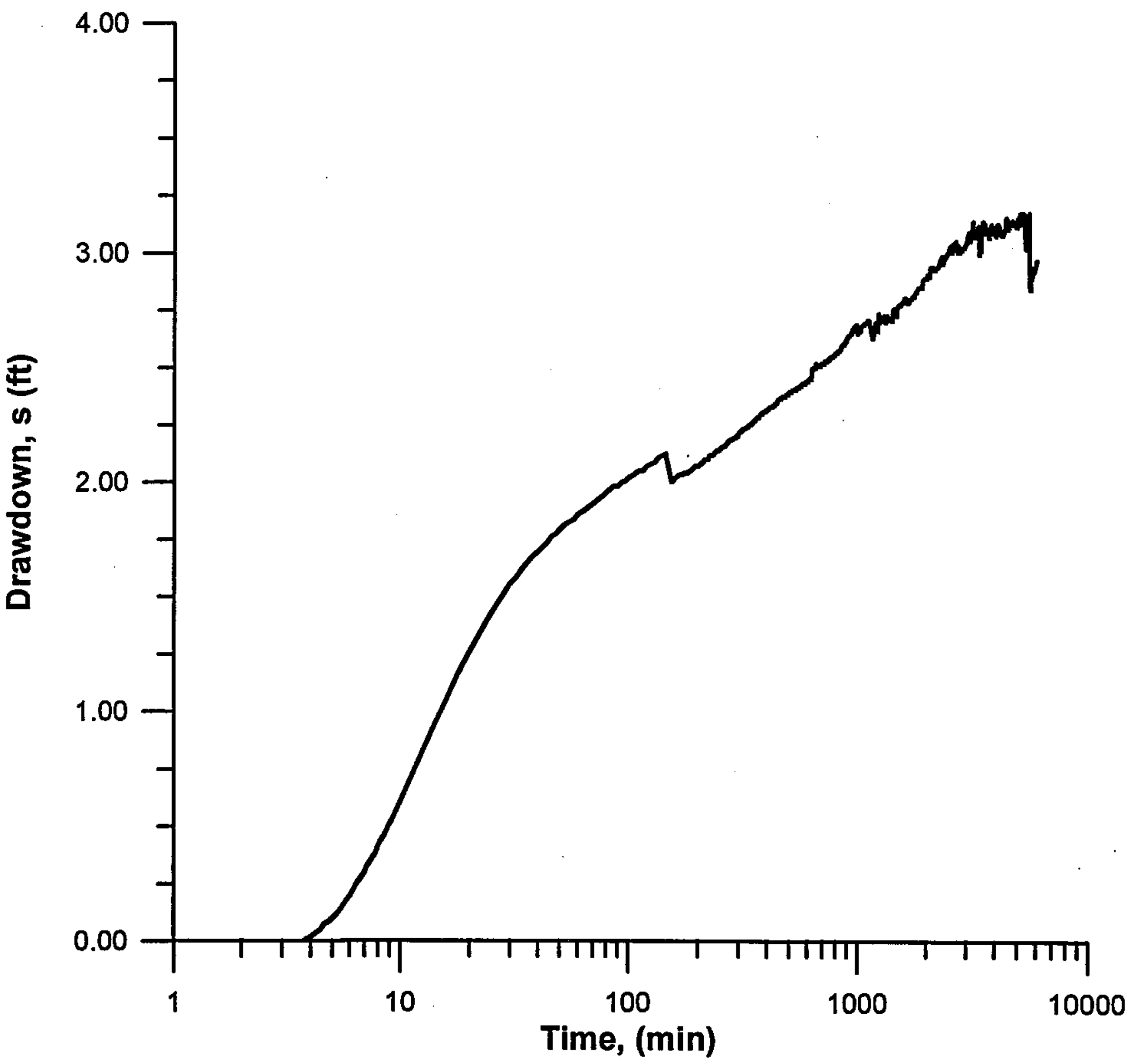
Semi-log plot of the time-drawdown relationships for well PZ 87 I



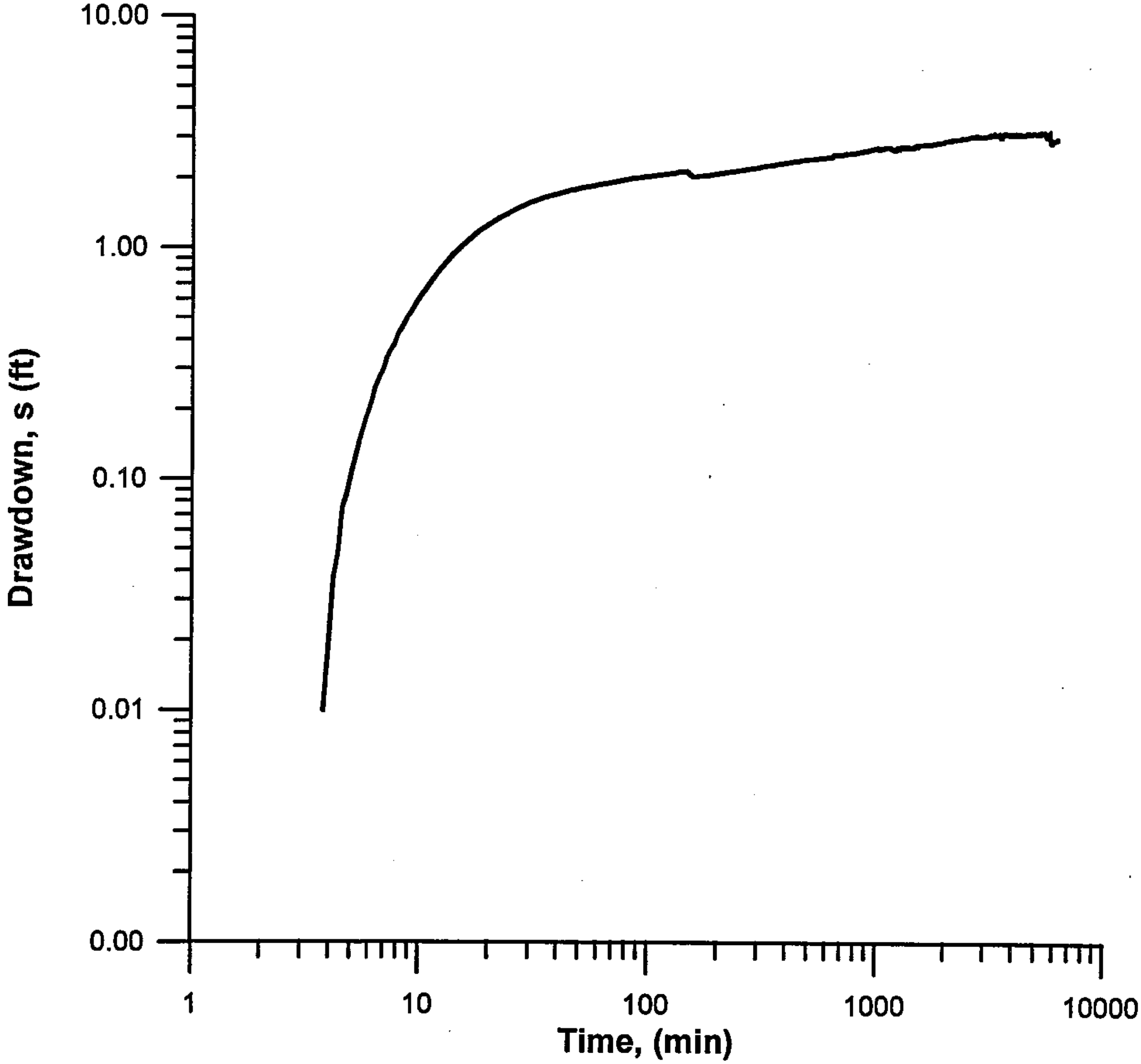
Log-log plot of the time-drawdown relationships for well PZ 87 I

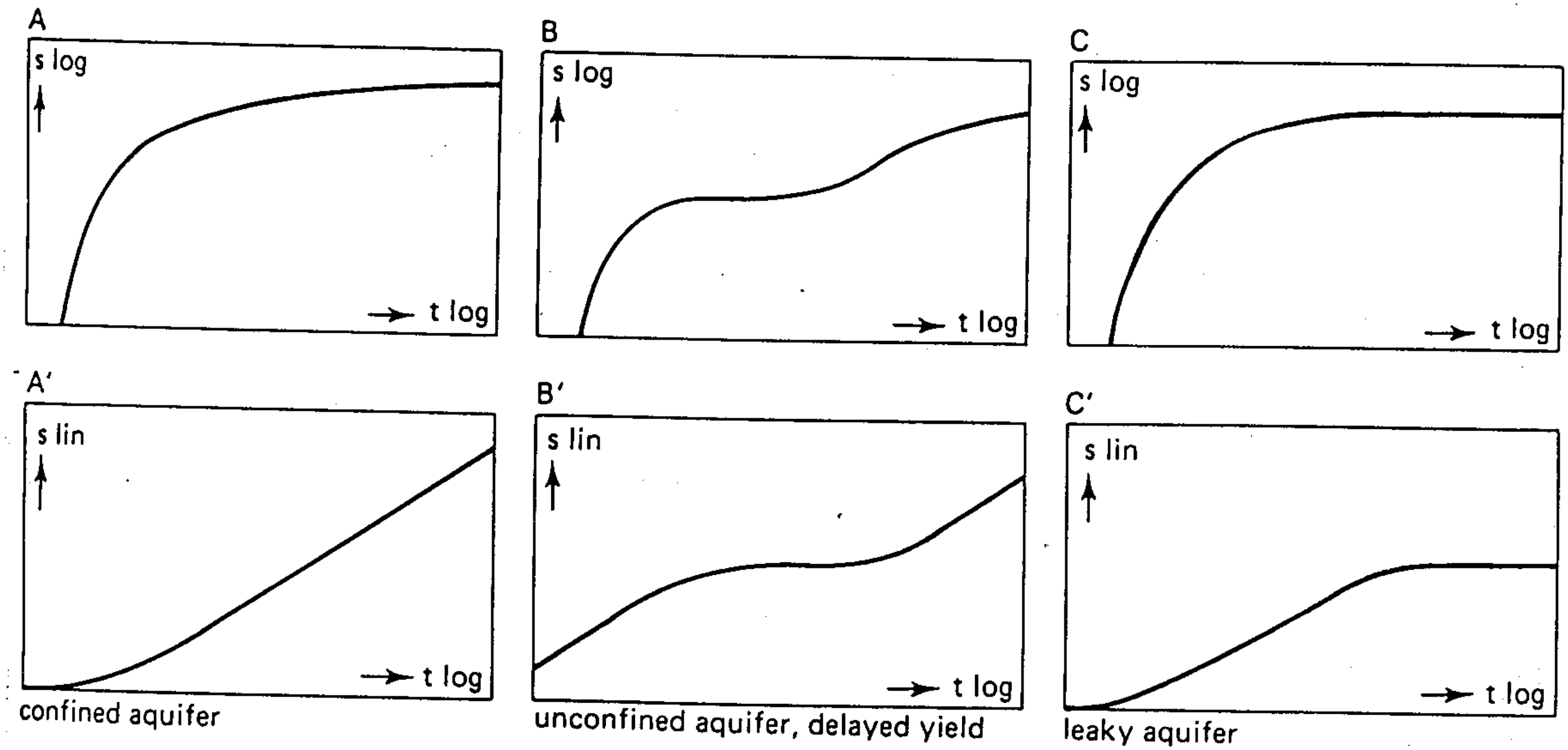


Semi-log plot of the time-drawdown relationships for well PZ 136 I



Log-log plot of the time-drawdown relationships for well PZ 136 I





Log-log and semi-log plots of the theoretical time-drawdown relationships of unconsolidated aquifers:

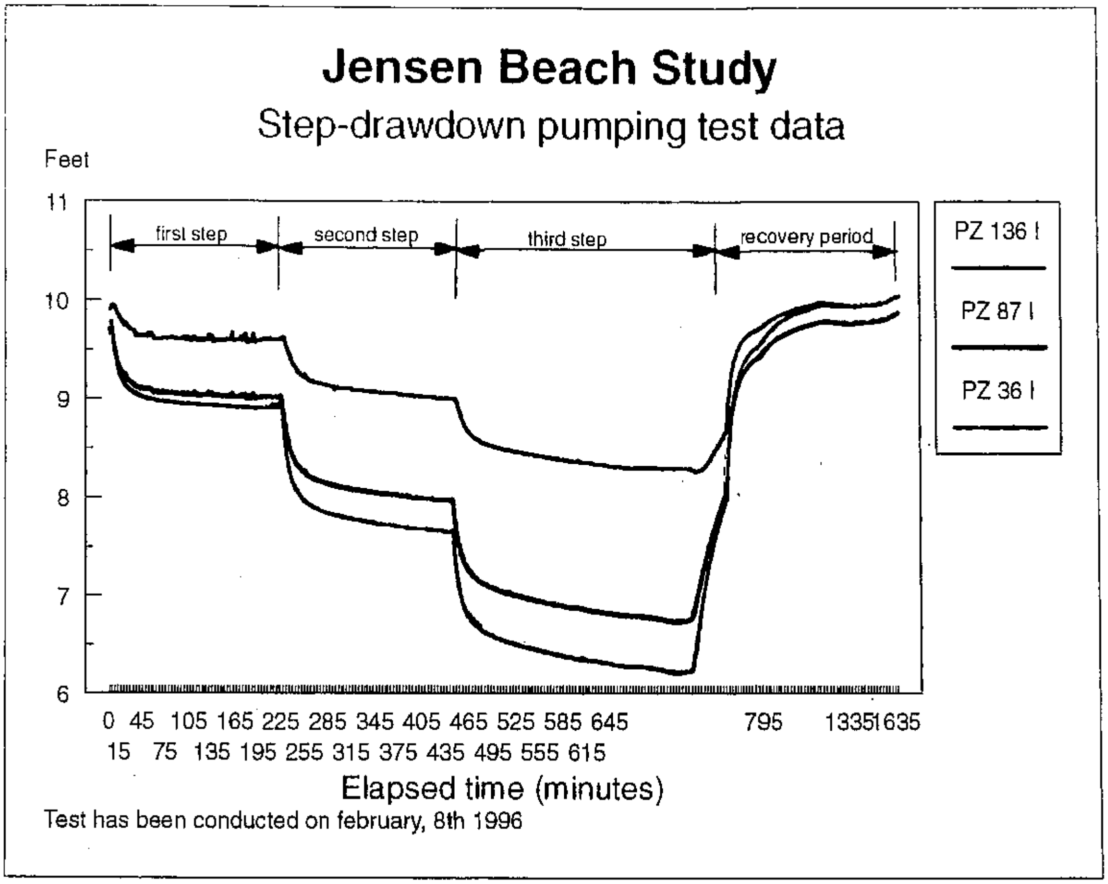
Parts A and A′: Confined aquifer

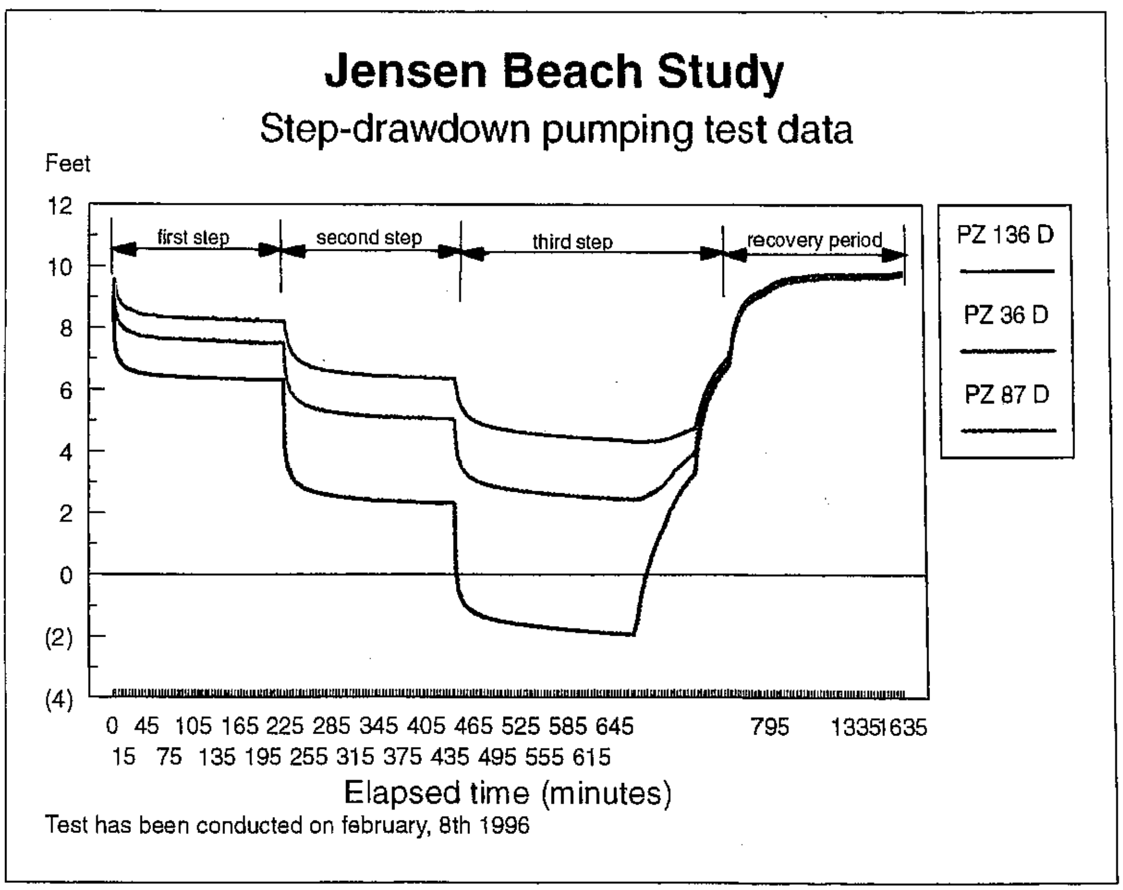
Parts B and B′: Unconfined aquifer

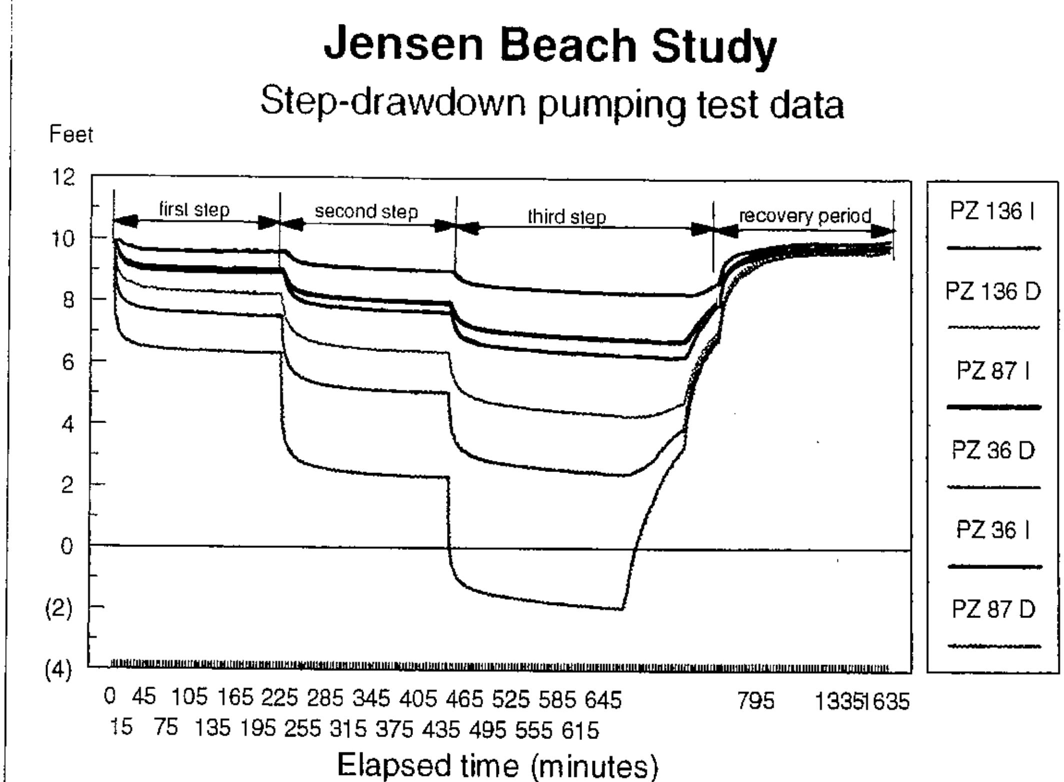
Parts C and C′: Leaky aquifer

APPENDIX 19

HYDROGRAPHS OF THE MONITOR WELLS DURING THE STEP-DRAWDOWN PUMPING TEST


Jensen Beach Study
Step-drawdown pumping test data
Feet
11
10
9
8
7
6
first step
second step
third step
recovery period
PZ 136 I
PZ 87 I
PZ 36 I
0 45 105 165 225 285 345 405 465 525 585 645 795 13351635
15 75 135 195 255 315 375 435 495 555 615
Elapsed time (minutes)
Test has been conducted on february, 8th 1996


Jensen Beach Study
Step-drawdown pumping test data
Feet
12
10
8
6
4
2
0
(2)
(4)
first step
second step
third step
recovery period
PZ 136 D
PZ 36 D
PZ 87 D
0 45 105 165 225 285 345 405 465 525 585 645 795 13351635
15 75 135 195 255 315 375 435 495 555 615
Elapsed time (minutes)
Test has been conducted on february, 8th 1996



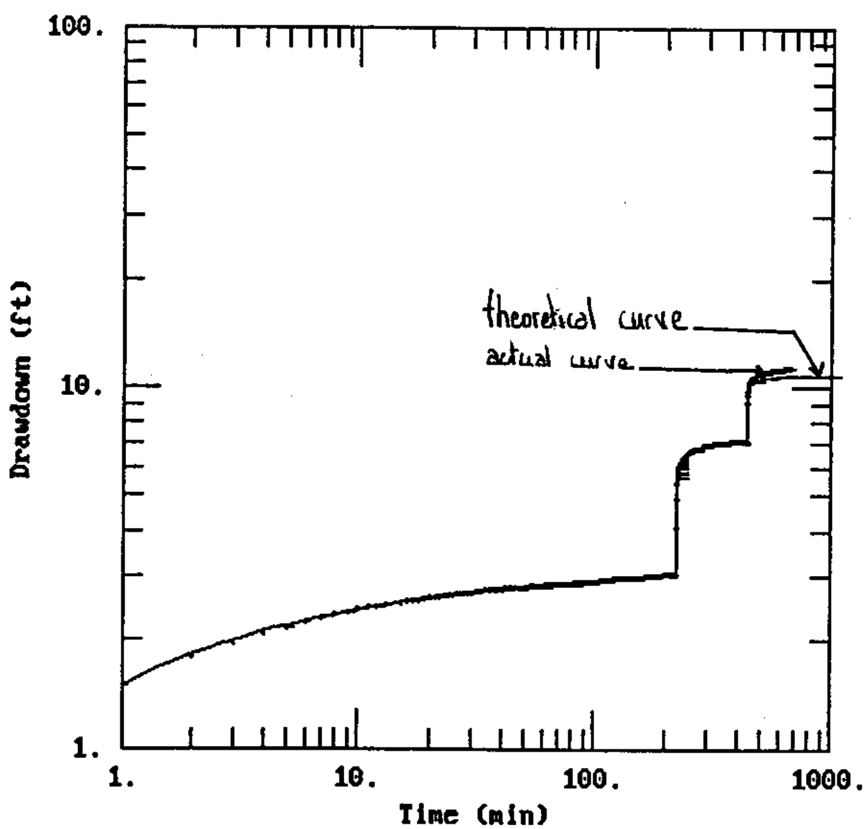
Test has been conducted on february, 8th 1996

APPENDIX 20

HANTUSH-JACOB PLOTS FOR THE DEEP AQUIFER DURING THE STEP-DRAWDOWN PUMPING TEST

CLIENT: Sandrine DIAZ	COMPANY: SFWMD
LOCATION: Jensen Beach	

Jensen Beach Project Step Drawdown Test



DATA SET:
PZ36DPP.DAT
05/24/96

AQUIFER MODEL:
Leaky

SOLUTION METHOD:
Hantush (no stor.)

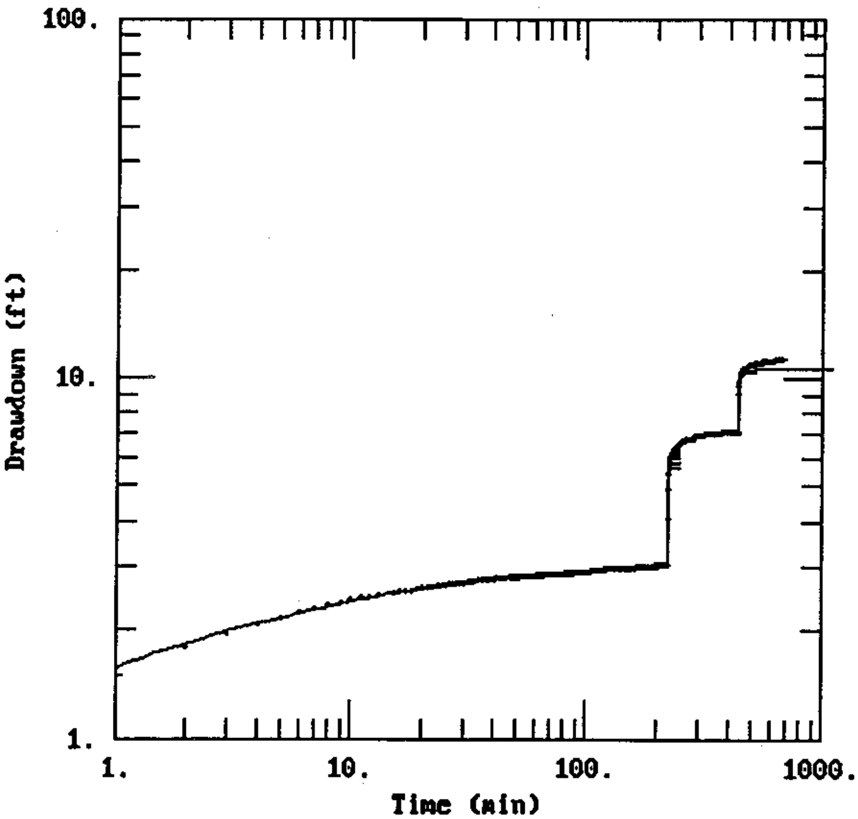
TEST DATA:
Q = 10.96 ft^3/min
r = 36. ft
b = 150. ft
Pumping Well Screen Depth:
top = 71. ft
bot.= 147. ft
Obs. Well Screen Depth:
top = 105. ft
bot.= 110. ft

PARAMETER ESTIMATES:
T = 2.869E+04 gal/day/ft
S = 0.0004211
r/B= 0.04551

AQTESOLV

CLIENT: ENGEES	COMPANY: SFWMD
LOCATION: Jensen Beach	

Jensen Beach Project Step Drawdown Test



DATA SET:
PZ87DPP.DAT
05/24/96

AQUIFER MODEL:
Leaky

SOLUTION METHOD:
Hantush (no stor.)

TEST DATA:
Q = 10.96 ft^3/min
r = 87. ft
b = 150. ft
Pumping Well Screen Depth:
top = 71. ft
bot.= 147. ft
Obs. Well Screen Depth:
top = 105. ft
bot.= 110. ft

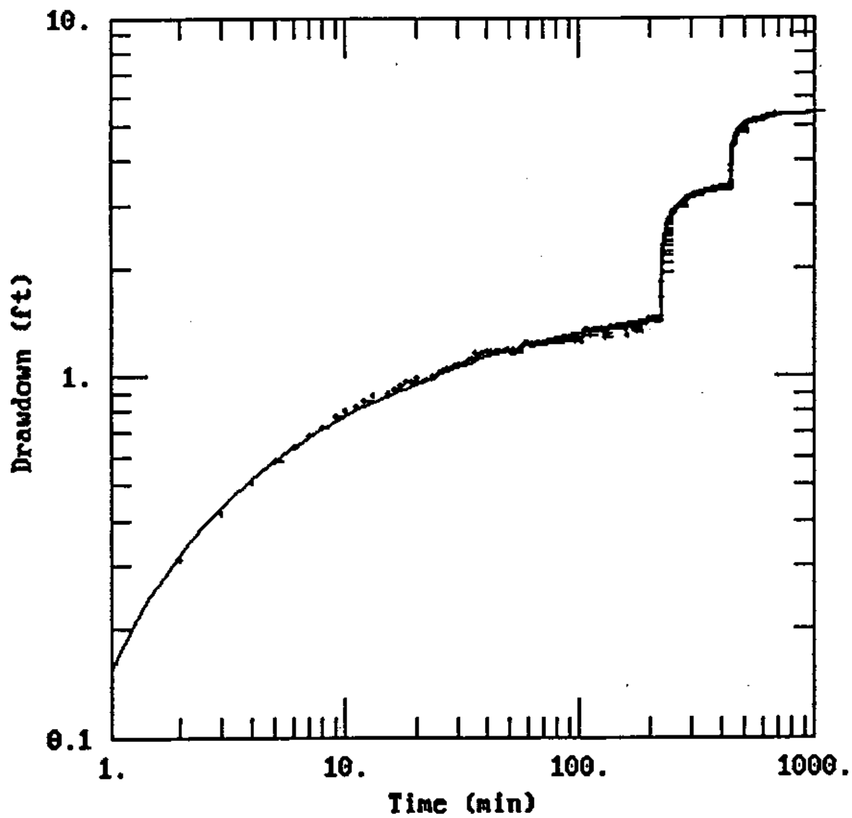
PARAMETER ESTIMATES:
T = 2.443E+04 gal/day/ft
S = 4.408E-05
r/B= 0.0451

AQTESOLV

CLIENT: ENGEES | COMPANY: SFWMD

LOCATION: Jensen Beach

Jensen Beach Project Step Drawdown Test



DATA SET:
PZ136DPP.DAT
05/24/96

AQUIFER MODEL:
Leaky

SOLUTION METHOD:
Hantush (no stor.)

TEST DATA:
Q = 10.96 ft^3/min
r = 136. ft
b = 150. ft
Pumping Well Screen Depth:
top = 71. ft
bot.= 147. ft
Obs. Well Screen Depth:
top = 105. ft
bot.= 110. ft

PARAMETER ESTIMATES:
T = 3.329E+04 gal/day/ft
S = 0.0005356
r/B= 0.1108

AQTESOLV

APPENDIX 21

SUPPORTING INFORMATION FOR THE SLUG TEST CONDUCTED IN THE INTERMEDIATE WELLS

CH2M HILL - SLUG TEST FROM RECOVERY DATA

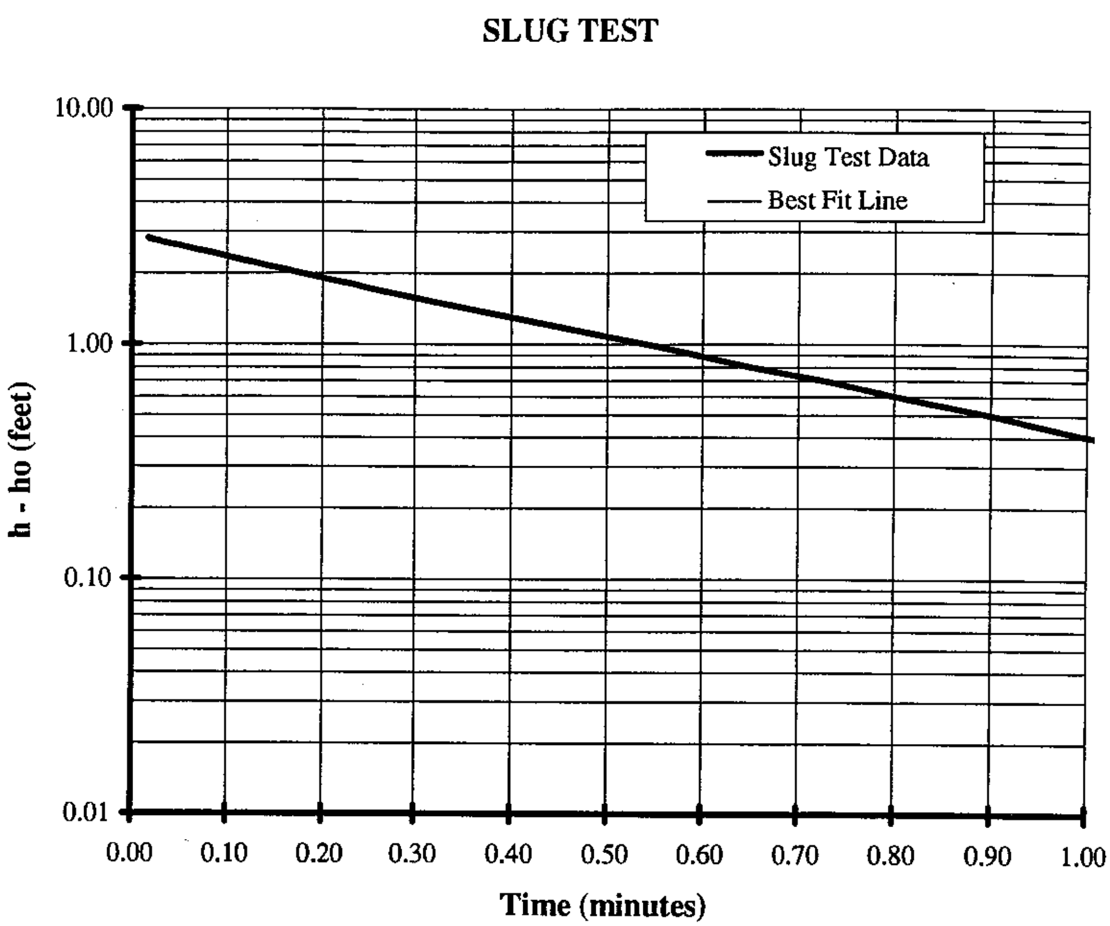
Project Number: Date of Test: 6/12/96

Client: SFWMD Well Number: PZ 136I

Incremental Values		H/Ho	Incremental Values		H/Ho	Incremental Values		H/Ho	Incremental Values		H/Ho
Time (min)	Head (feet)	(-)	Time (min)	Head (feet)	(-)	Time (min)	Head (feet)	(-)	Time (min)	Head (feet)	(-)
0.000	2.94	1.00									
0.017	2.83	0.96									
0.033	2.71	0.92									
0.050	2.62	0.89									
0.067	2.52	0.86									
0.083	2.44	0.83									
0.100	2.35	0.80									
0.117	2.27	0.77									
0.133	2.20	0.75									
0.150	2.12	0.72									
0.167	2.05	0.70									
0.183	1.98	0.67									
0.200	1.91	0.65									
0.217	1.85	0.63									
0.233	1.79	0.61									
0.317	1.52	0.52									
0.400	1.30	0.44									
0.483	1.11	0.38									
0.567	0.95	0.32									
0.650	0.80	0.27									
0.733	0.69	0.23									
0.817	0.58	0.20									
0.900	0.50	0.17									
0.983	0.42	0.14									
1.067	0.36	0.12									
1.150	0.30	0.10									
1.233	0.24	0.08									
1.317	0.20	0.07									
1.400	0.16	0.05									
1.483	0.13	0.04									
1.567	0.10	0.03									
1.650	0.07	0.02									
1.733	0.05	0.02									
1.817	0.02	0.01									
1.900	0.00	0.00									

CH2M HILL - SLUG TEST FROM RECOVERY DATA

Project Number:		Date of Test:	#####
Client:	SFWMD	Well Number:	PZ 136I



WELL CONSTRUCTION DATA

R casing =	0.08	feet
R gravel pack =	0.21	feet
Porosity of the aquifer =	0.20	
Porosity of gravel pack =	0.30	
Effective radius of casing =	0.10	feet (Rc)
Casing radius for analysis =	0.11	feet (Rw)
Radius of well for analysis =	0.25	feet
Casing Stickup =	0.80	feet als
Depth of water =	7.95	feet btoc
Depth of well =	50.00	feet btoc
Depth of aquifer =	150.00	feet bls
Depth to top of filter pack =	38.00	feet bls
Length of screen =	5.00	feet
Length of gravel pack =	12.00	feet
L (input) =	21.00	feet
Case =	1	

SLUG TEST DATA/RESULTS
(Bouwer Rice Method)

L/Rw =	84	
A =	3.98	
B =	0.58	
C =	3.53	
H =	42.05	feet
ln[(D-H)/Rw] =	6.00	
Ln (Ri/Rw) =	3.29	
R influence =	6.73	feet (Ri)

Line Fit Range and Parameters

t minimum =	>.10	minutes
t maximum =	<1	minutes
r 2 =	1.000	
Estimated K h =	3	feet/day

CH2M HILL - SLUG TEST FROM RECOVERY DATA

Project Number:
Date of Test: 6/12/96

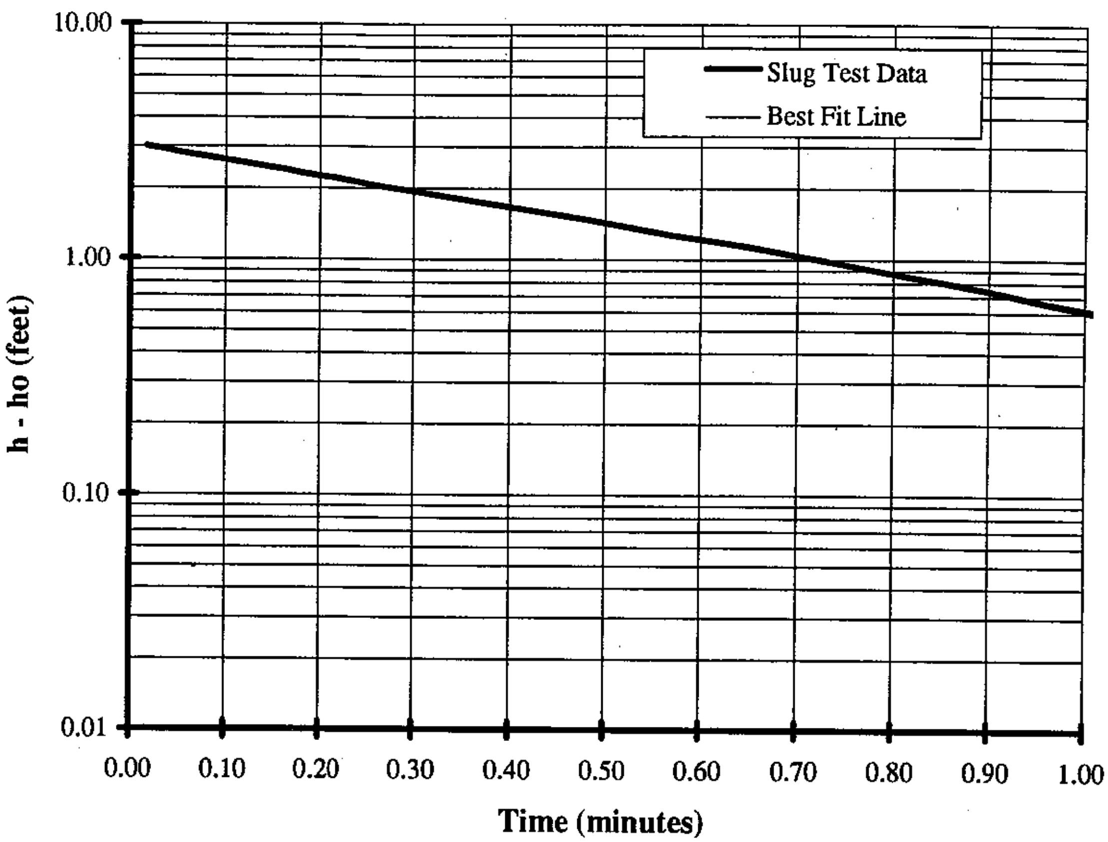
Client: SFWMD
Well Number: PZ 36I

Incremental Values		H/Ho	Incremental Values		H/Ho	Incremental Values		H/Ho	Incremental Values		H/Ho
Time (min)	Head (feet)	(-)	Time (min)	Head (feet)	(-)	Time (min)	Head (feet)	(-)	Time (min)	Head (feet)	(-)
0.000	3.11	1.00									
0.017	3.01	0.97									
0.033	2.93	0.94									
0.050	2.85	0.92									
0.067	2.78	0.89									
0.083	2.71	0.87									
0.100	2.64	0.85									
0.117	2.57	0.83									
0.133	2.51	0.81									
0.150	2.44	0.78									
0.167	2.38	0.77									
0.183	2.32	0.75									
0.200	2.26	0.73									
0.217	2.21	0.71									
0.233	2.15	0.69									
0.317	1.89	0.61									
0.400	1.67	0.54									
0.483	1.47	0.47									
0.567	1.28	0.41									
0.650	1.13	0.36									
0.733	0.98	0.32									
0.817	0.85	0.27									
0.900	0.74	0.24									
0.983	0.63	0.20									
1.067	0.54	0.17									
1.150	0.45	0.14									
1.233	0.38	0.12									
1.317	0.31	0.10									
1.400	0.25	0.08									
1.483	0.20	0.06									
1.567	0.15	0.05									
1.650	0.11	0.04									
1.733	0.07	0.02									
1.817	0.03	0.01									
1.900	0.00	0.00									

CH2M HILL - SLUG TEST FROM RECOVERY DATA

Project Number:		Date of Test:	#####
Client:	SFWMD	Well Number:	PZ 36I

SLUG TEST



WELL CONSTRUCTION DATA

R casing =	0.08	feet
R gravel pack =	0.21	feet
Porosity of the aquifer =	0.20	
Porosity of gravel pack =	0.30	
Effective radius of casing =	0.10	feet (Rc)
Casing radius for analysis =	0.11	feet (Rw)
Radius of well for analysis =	0.25	feet
Casing Stickup =	0.80	feet als
Depth of water =	7.73	feet btoc
Depth of well =	50.00	feet btoc
Depth of aquifer =	150.00	feet bls
Depth to top of filter pack =	38.00	feet bls
Length of screen =	5.00	feet
Length of gravel pack =	12.50	feet
L (input) =	21.00	feet
Case =	1	

SLUG TEST DATA/RESULTS
(Bouwer Rice Method)

L/Rw =	84	
A =	3.98	
B =	0.58	
C =	3.53	
H =	42.27	feet
ln[(D-H)/Rw] =	6.00	
Ln (Ri/Rw) =	3.30	
R influence =	6.74	feet (Ri)

Line Fit Range and Parameters

t minimum =	>.10	minutes
t maximum =	<1	minutes
r 2 =	0.999	
Estimated K h =	2	feet/day

CH2M HILL - SLUG TEST FROM RECOVERY DATA

Project Number: **Date of Test:** 6/12/96

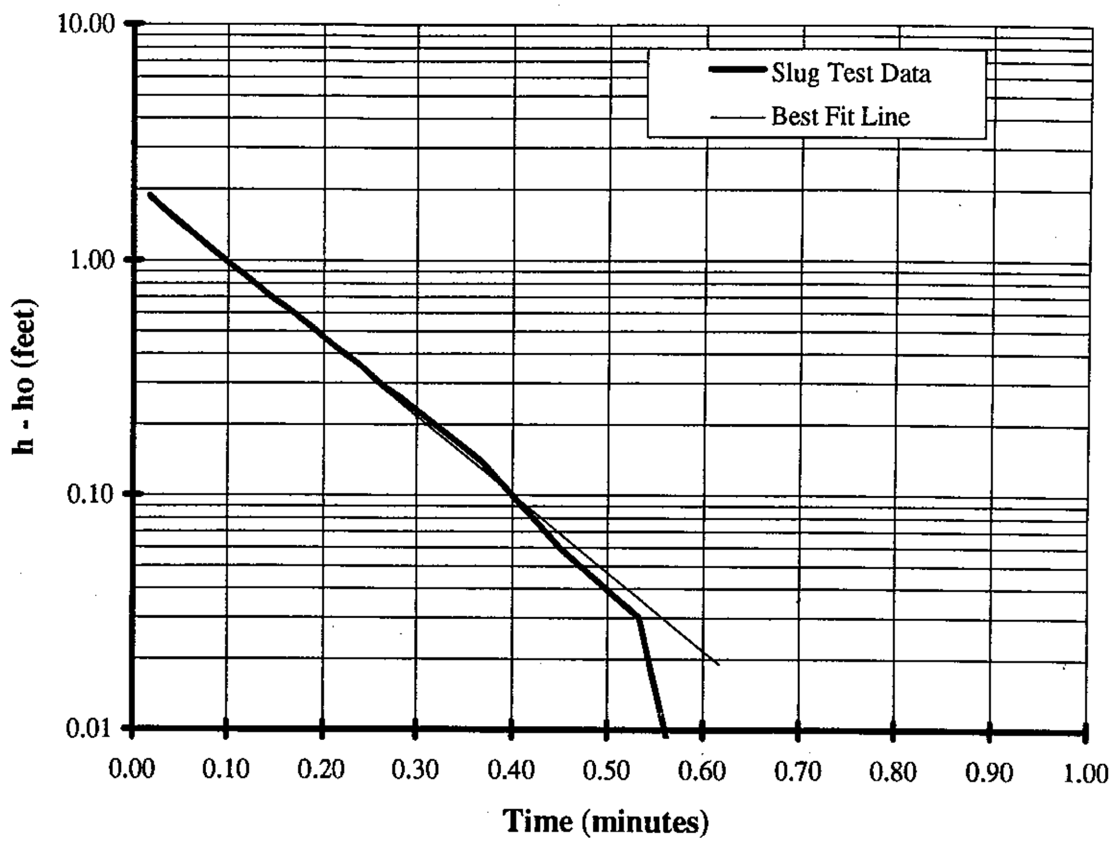
Client: **Well Number:** PZ 87I

Incremental Values		H/Ho	Incremental Values		H/Ho	Incremental Values		H/Ho	Incremental Values		H/Ho
Time (min)	Head (feet)	(-)	Time (min)	Head (feet)	(-)	Time (min)	Head (feet)	(-)	Time (min)	Head (feet)	(-)
0.000	2.12	1.00									
0.017	1.88	0.89									
0.033	1.63	0.77									
0.050	1.43	0.67									
0.067	1.27	0.60									
0.083	1.11	0.52									
0.100	0.98	0.46									
0.117	0.87	0.41									
0.133	0.77	0.36									
0.150	0.68	0.32									
0.167	0.61	0.29									
0.183	0.54	0.25									
0.200	0.47	0.22									
0.217	0.42	0.20									
0.233	0.37	0.17									
0.250	0.33	0.16									
0.267	0.29	0.14									
0.283	0.26	0.12									
0.367	0.14	0.07									
0.450	0.06	0.03									
0.533	0.03	0.01									
0.617	0.00	0.00									

CH2M HILL - SLUG TEST FROM RECOVERY DATA

Project Number:	0	Date of Test:	6/12/96
Client:	0	Well Number:	PZ 87I

SLUG TEST



WELL CONSTRUCTION DATA

R casing =	0.08	feet
R gravel pack =	0.21	feet
Porosity of the aquifer =	0.20	
Porosity of gravel pack =	0.30	
Effective radius of casing =	0.10	feet (Rc)
Casing radius for analysis =	0.11	feet (Rw)
Radius of well for analysis =	0.25	feet
Casing Stickup =	0.80	feet als
Depth of water =	7.89	feet btoc
Depth of well =	50.00	feet btoc
Depth of aquifer =	150.00	feet bls
Depth to top of filter pack =	38.00	feet bls
Length of screen =	5.00	feet
Length of gravel pack =	12.00	feet
L (input) =	21.00	feet
Case =	1	

SLUG TEST DATA/RESULTS
(Bouwer Rice Method)

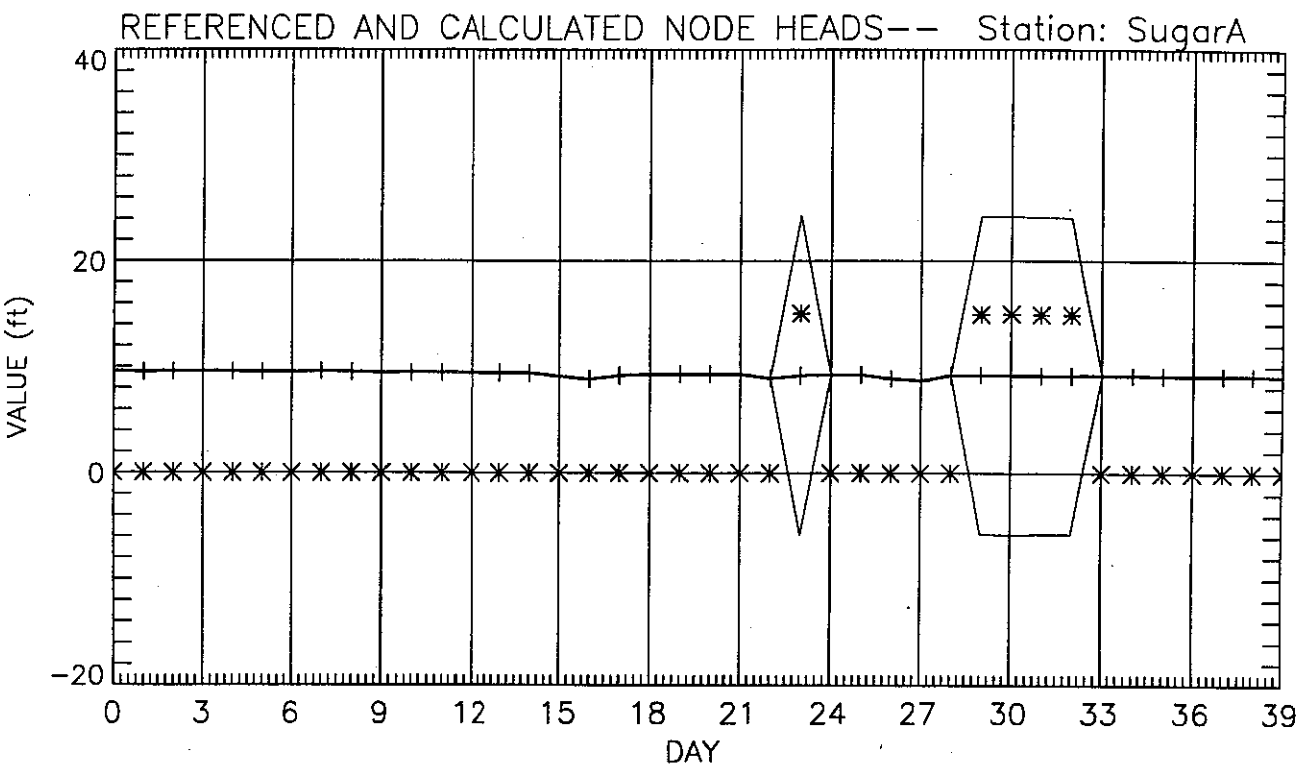
L/Rw =	84	
A =	3.98	
B =	0.58	
C =	3.53	
H =	42.11	feet
ln[(D-H)/Rw] =	6.00	
Ln (Ri/Rw) =	3.29	
R influence =	6.73	feet (Ri)

Line Fit Range and Parameters

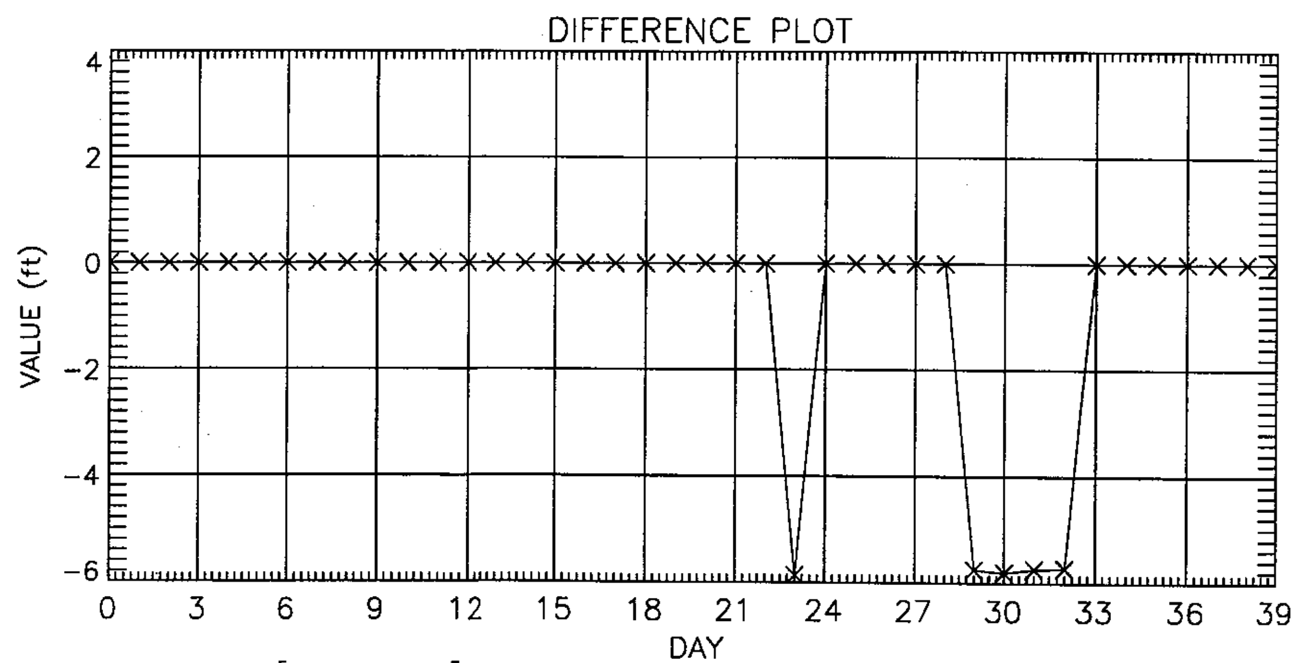
t minimum =	>.05	minutes
t maximum =	<0.5	minutes
r 2 =	0.997	
Estimated K h =	10	feet/day

APPENDIX 22

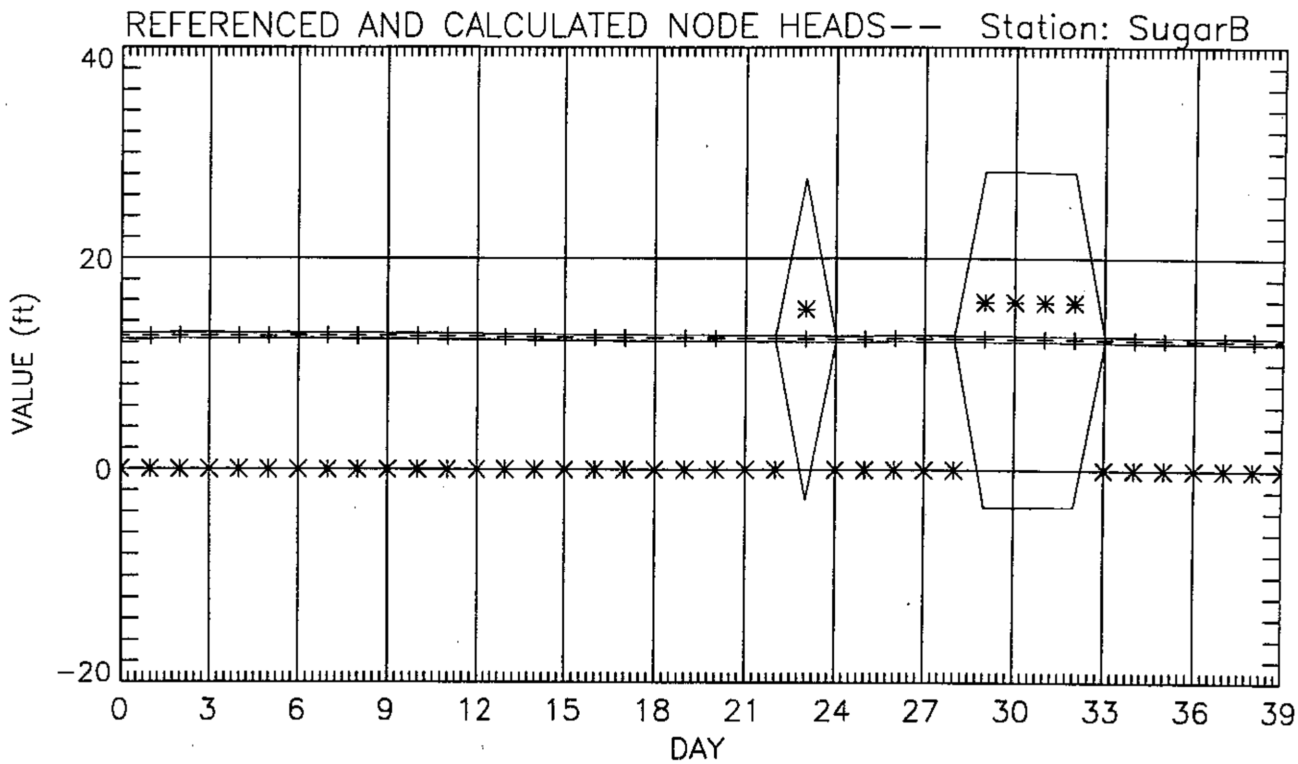
MODEL CALIBRATION HYDROGRAPHS



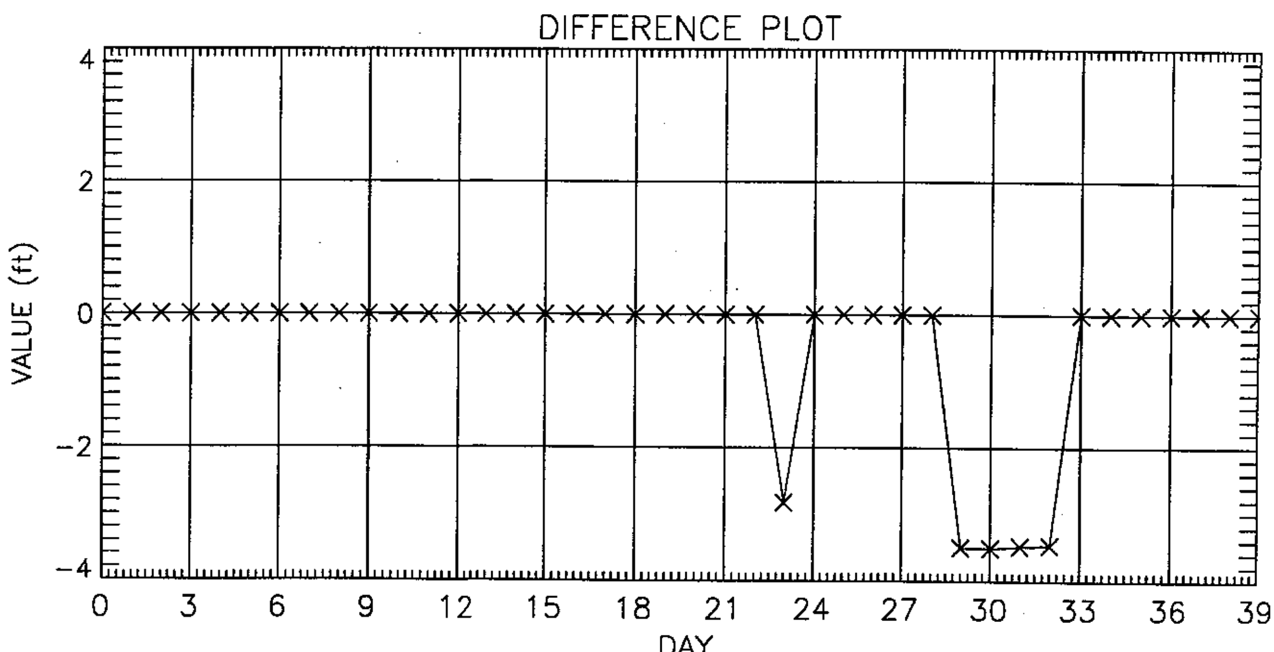
Layer 2 Row 37 Column 74 Hist.Stnd.Dev. 0.06 C.I.Year 0.13 C.I. DAY 0.00



Extreme Errors [-5.9, 0.0] Average Absolute Error 0.72 Std.Error 0.06

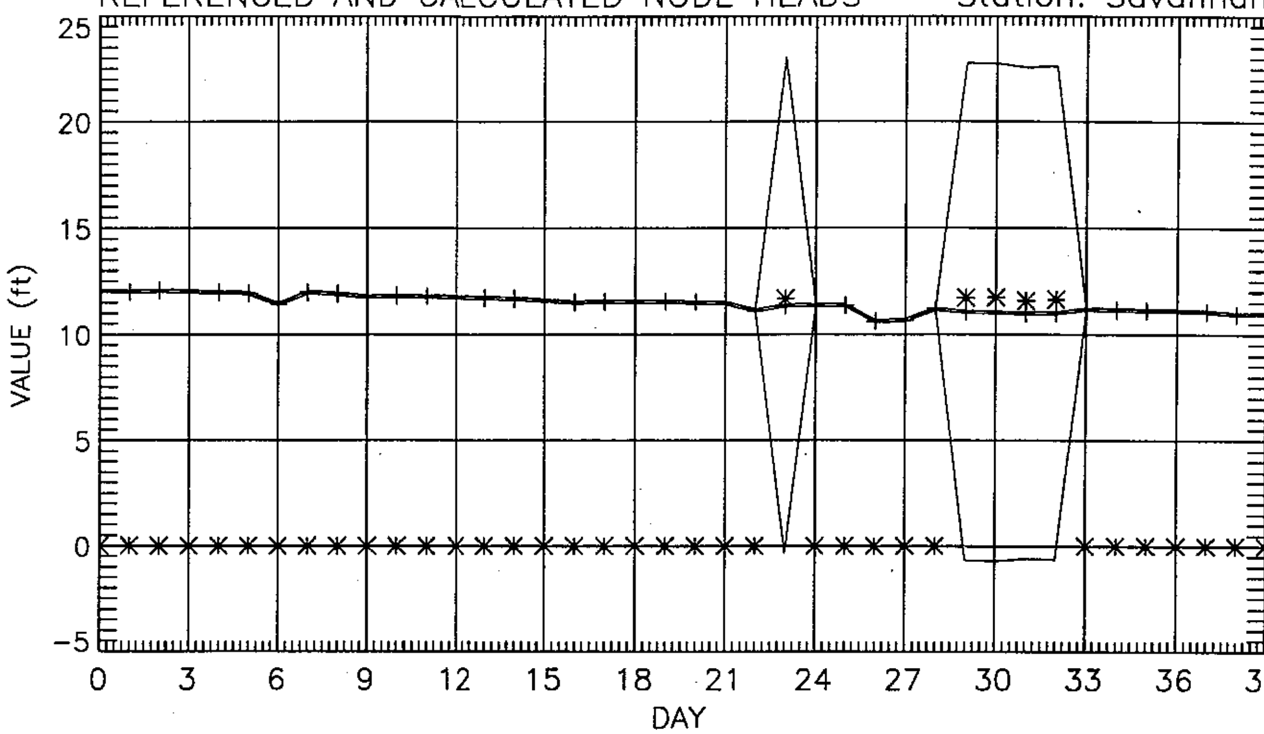


Layer 1 Row 37 Column 74 Hist.Stnd.Dev. 0.29 C.I.Year 0.13 C.I. DAY 0.00



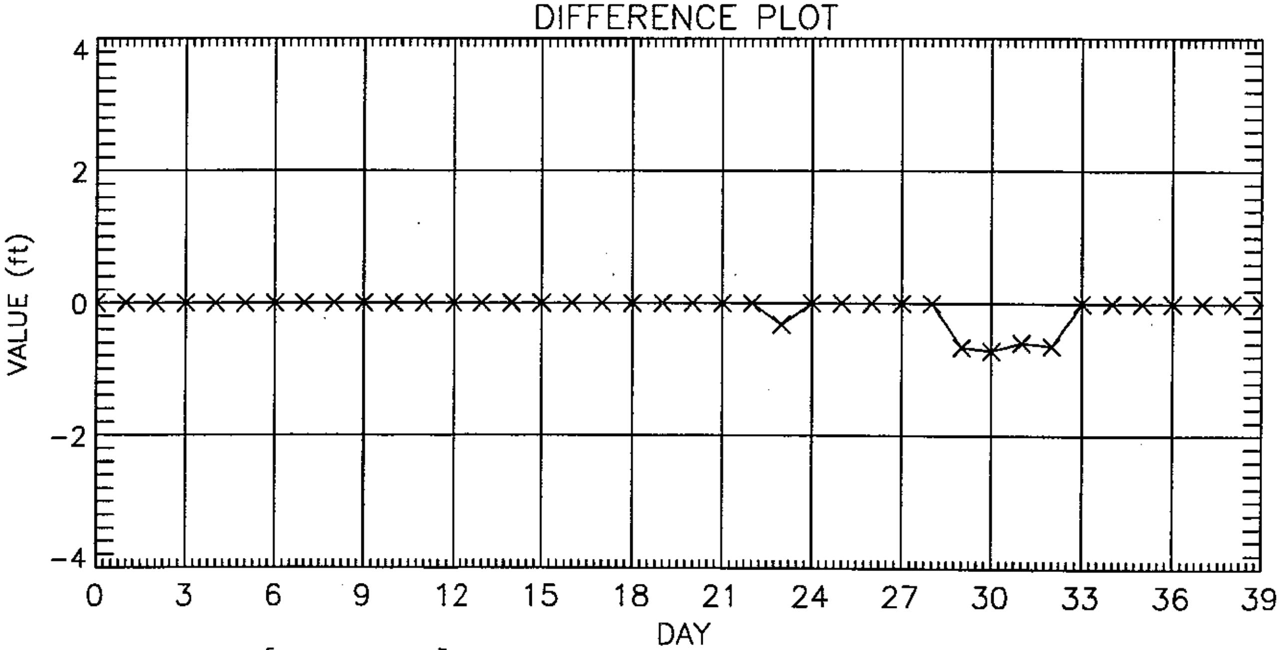
Extreme Errors [−3.5, 0.0] Average Absolute Error 0.42 Std.Error 0.30

REFERENCED AND CALCULATED NODE HEADS-- Station: Savannah



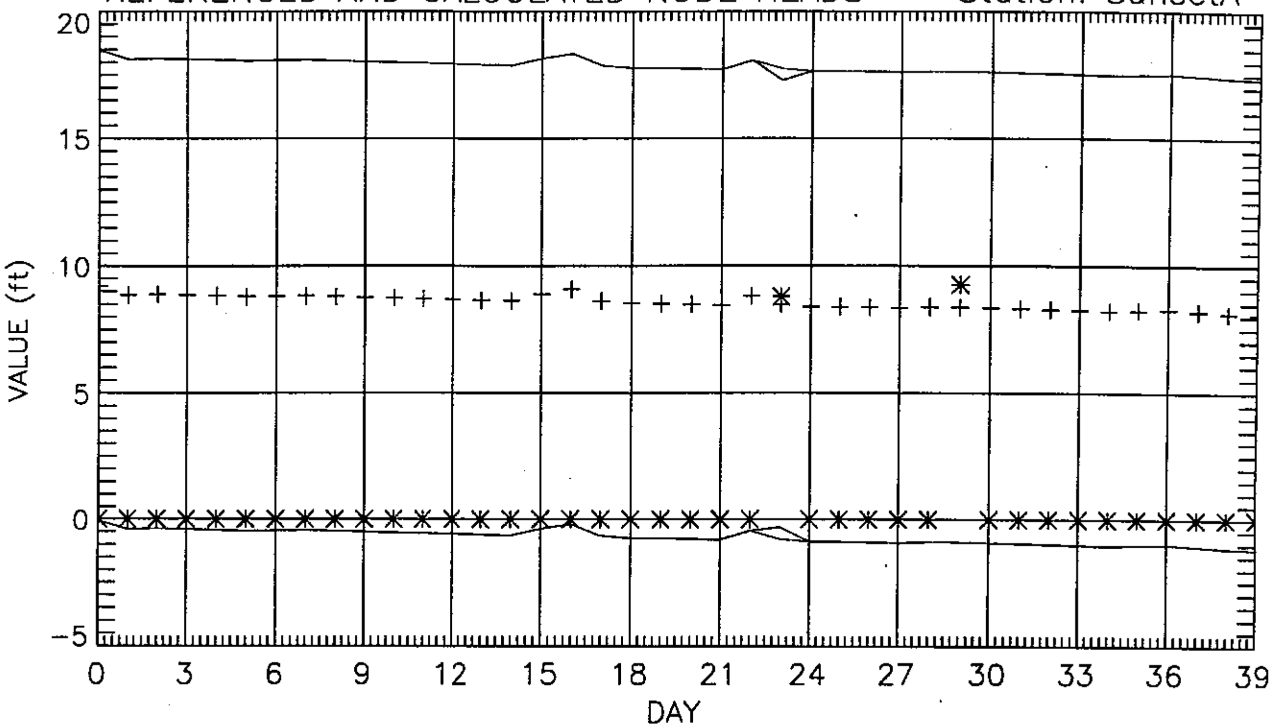
Layer 2 Row 37 Column 60 Hist.Stnd.Dev. 0.06 C.I.Year 0.13 C.I. DAY 0.00

DIFFERENCE PLOT



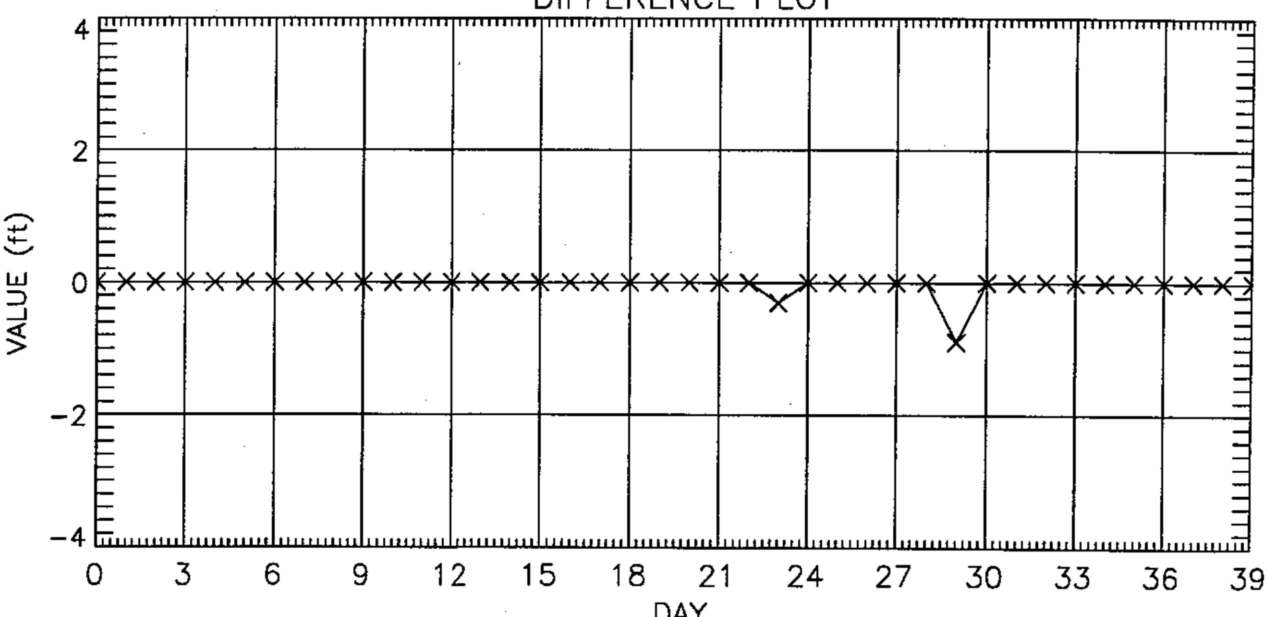
Extreme Errors [-0.7, 0.0] Average Absolute Error 0.07 Std.Error 0.15

REFERENCED AND CALCULATED NODE HEADS-- Station: SunsetA

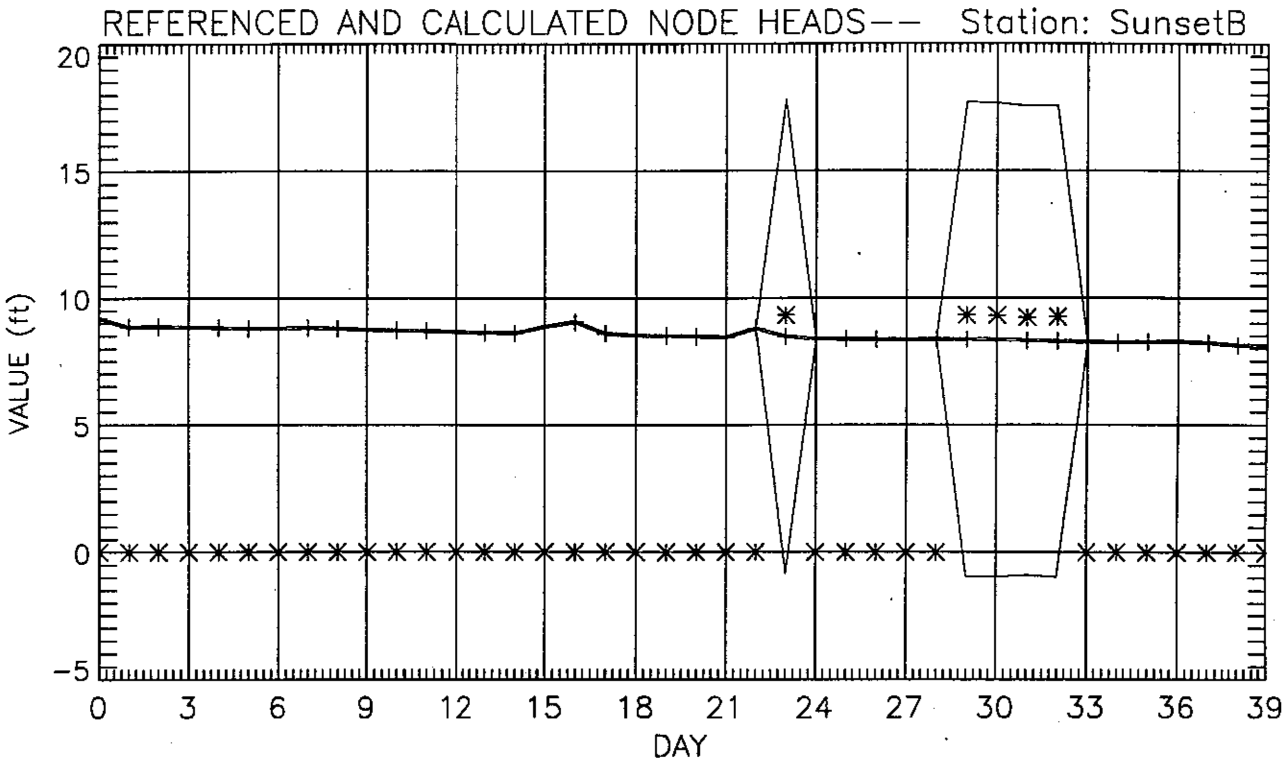


Layer 2 Row 24 Column 34 Hist.Stnd.Dev. 9.26 C.I.Year 0.00 C.I. DAY 0.00

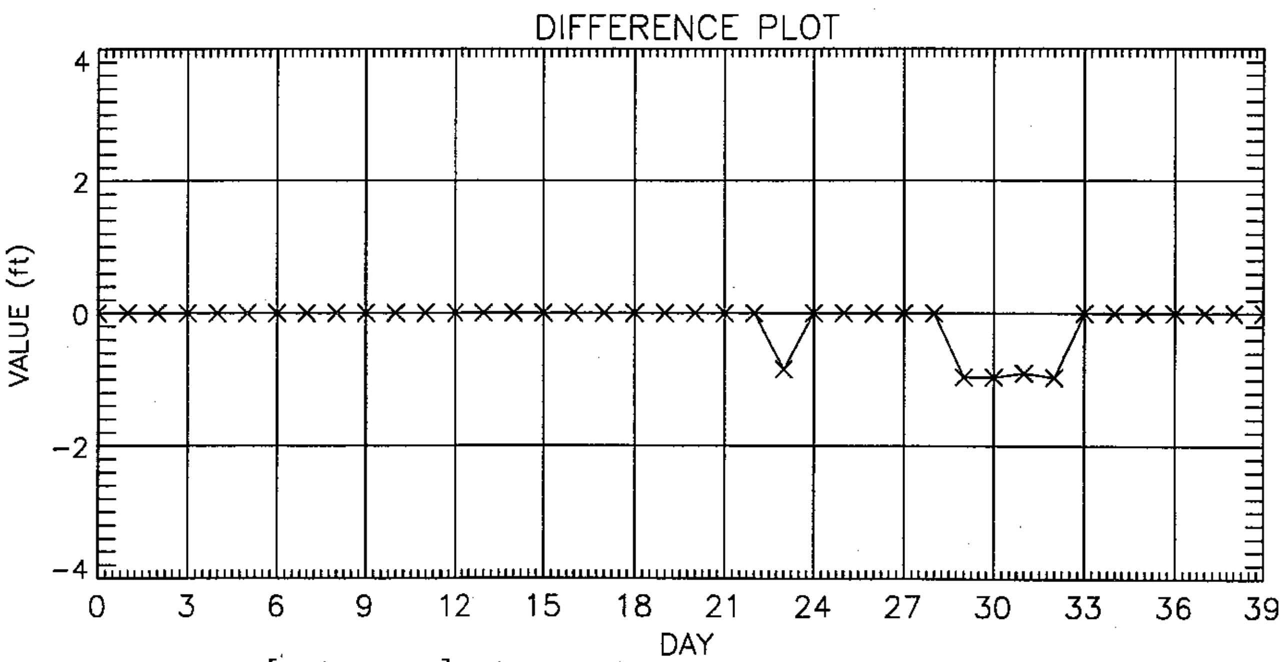
DIFFERENCE PLOT



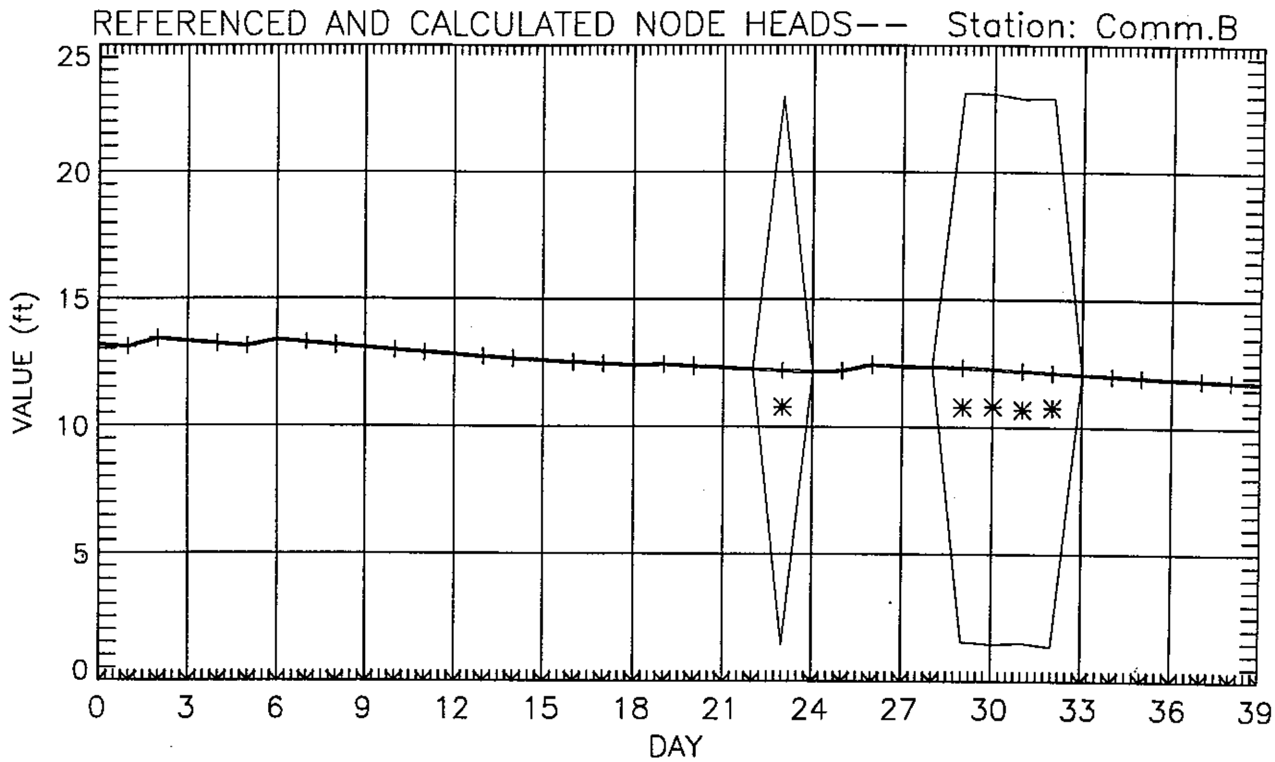
Extreme Errors [-0.9, 0.0] Average Absolute Error 0.03 Std.Error 0.41



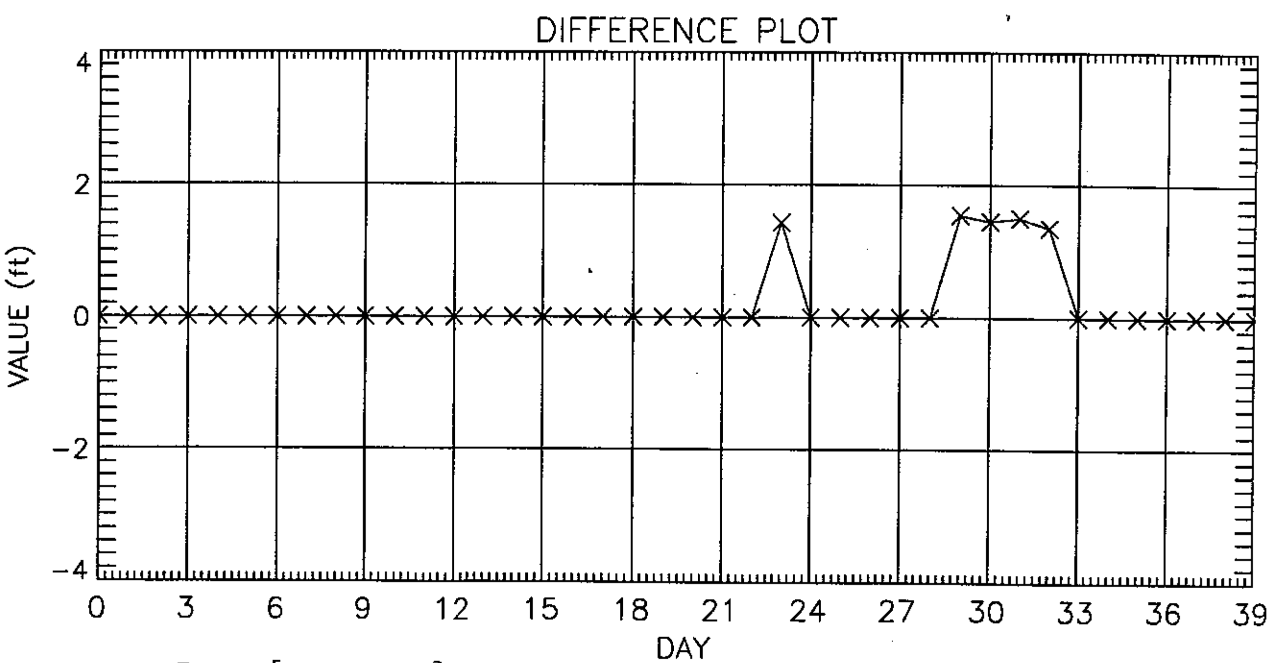
Layer 2 Row 24 Column 34 Hist.Stnd.Dev. 0.05 C.I.Year 0.13 C.I. DAY 0.00



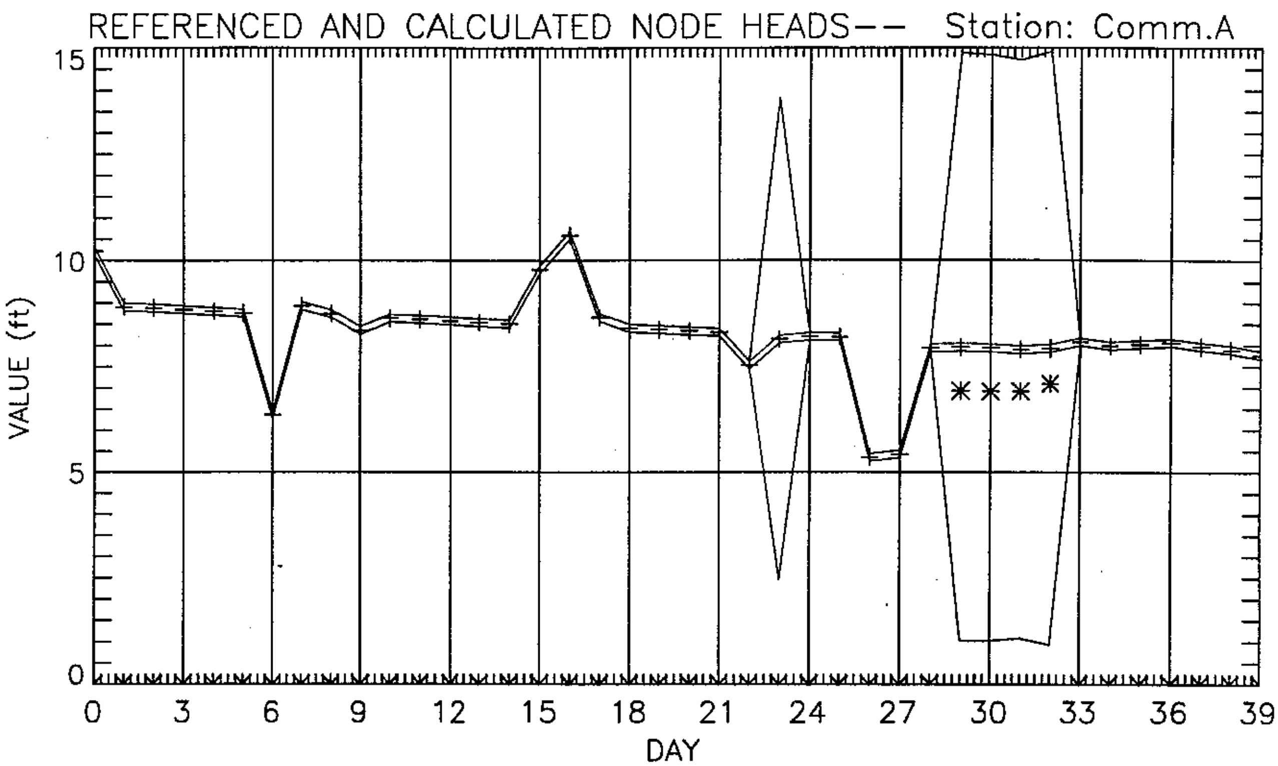
Extreme Errors [−1.0, 0.0] Average Absolute Error 0.12 Std.Error 0.05



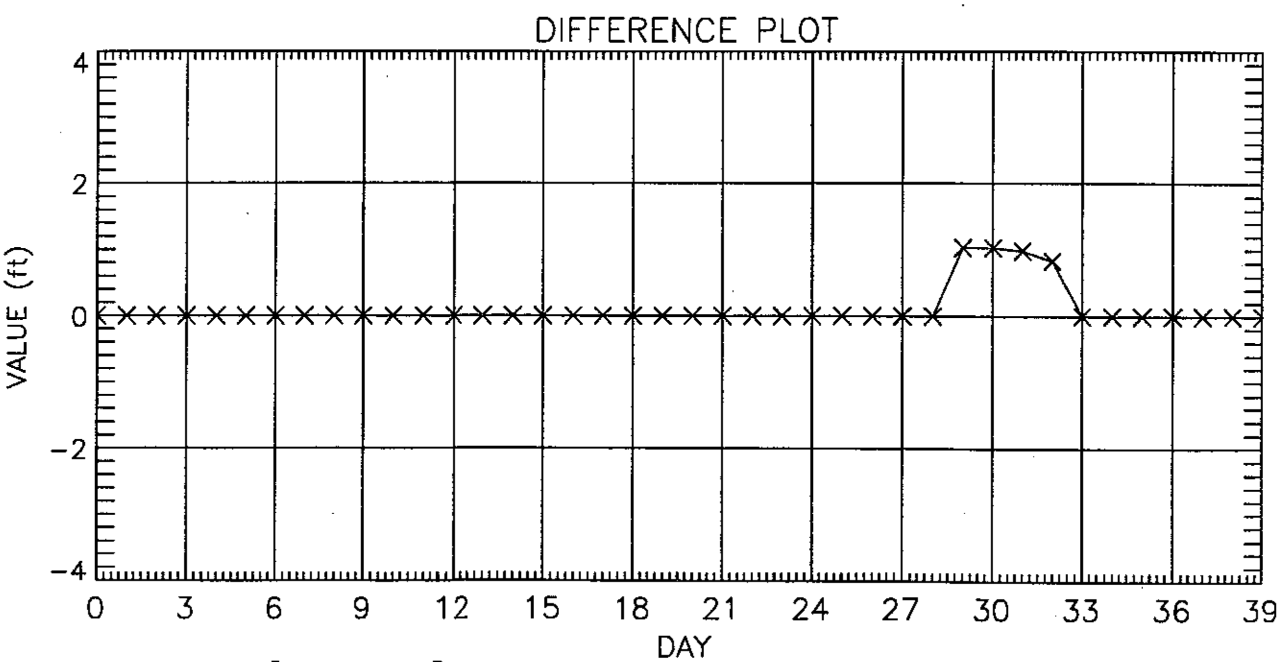
Layer 1 Row 38 Column 51 Hist.Stnd.Dev. 0.04 C.I.Year 0.13 C.I. DAY 0.00



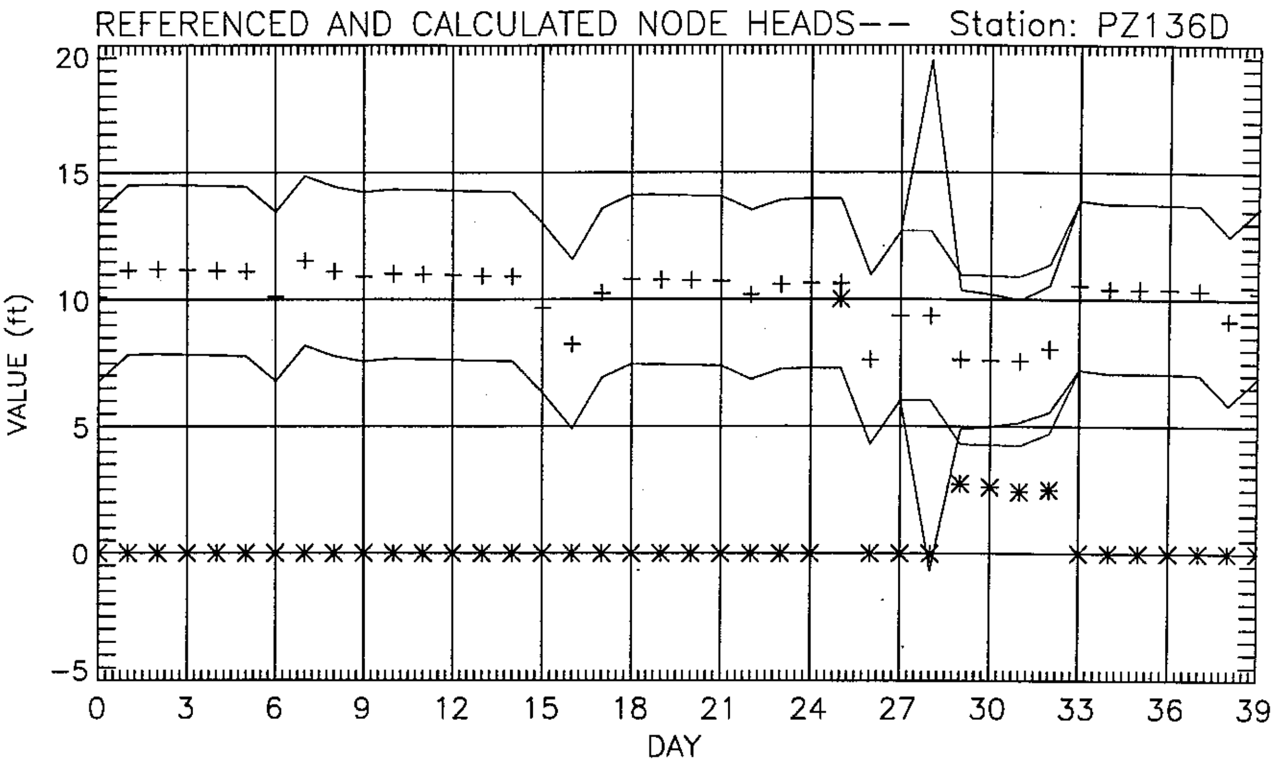
Extreme Errors [0.0, 1.5] Average Absolute Error 0.18 Std.Error 0.07



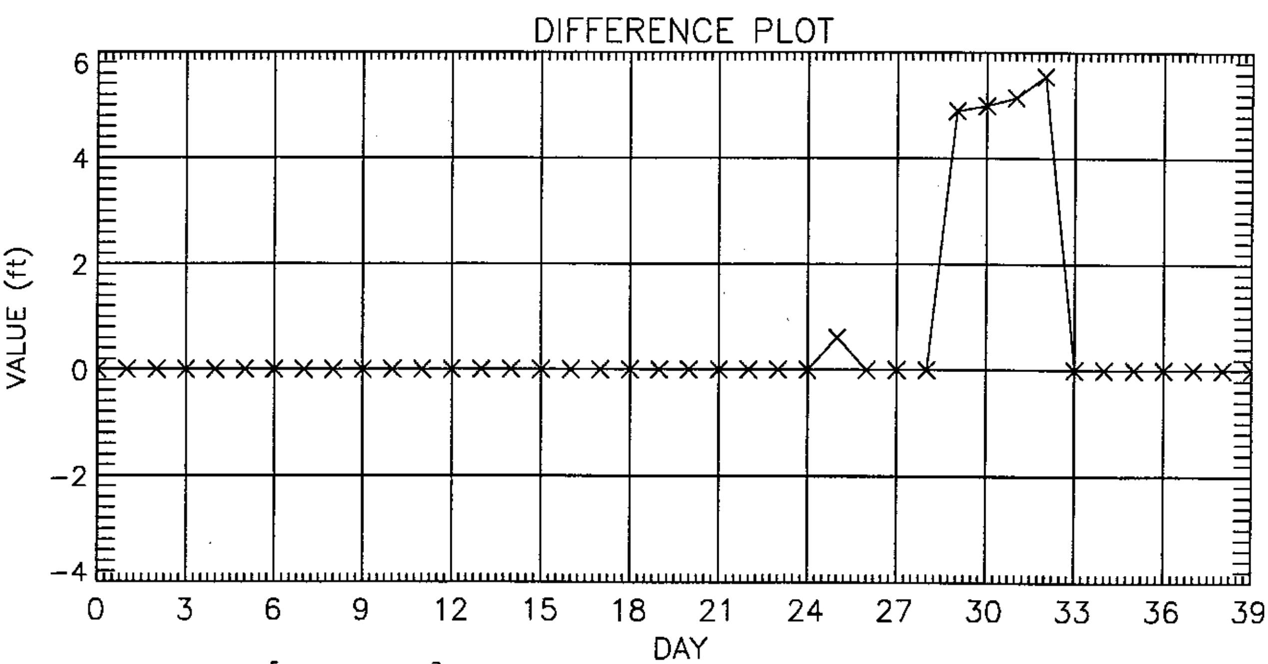
Layer 2 Row 38 Column 51 Hist.Stnd.Dev. 0.09 C.I.Year 0.10 C.I. DAY 0.00



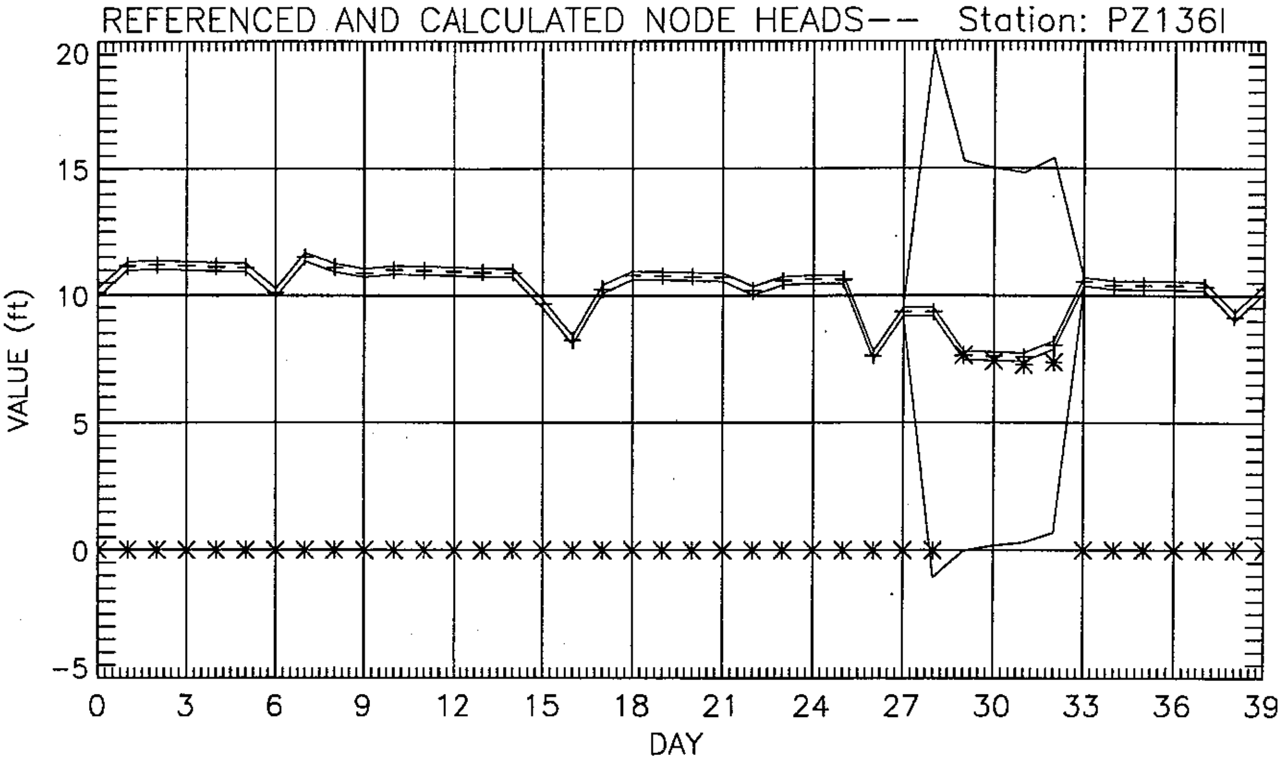
Extreme Errors [0.0, 1.0] Average Absolute Error 0.10 Std.Error 0.09



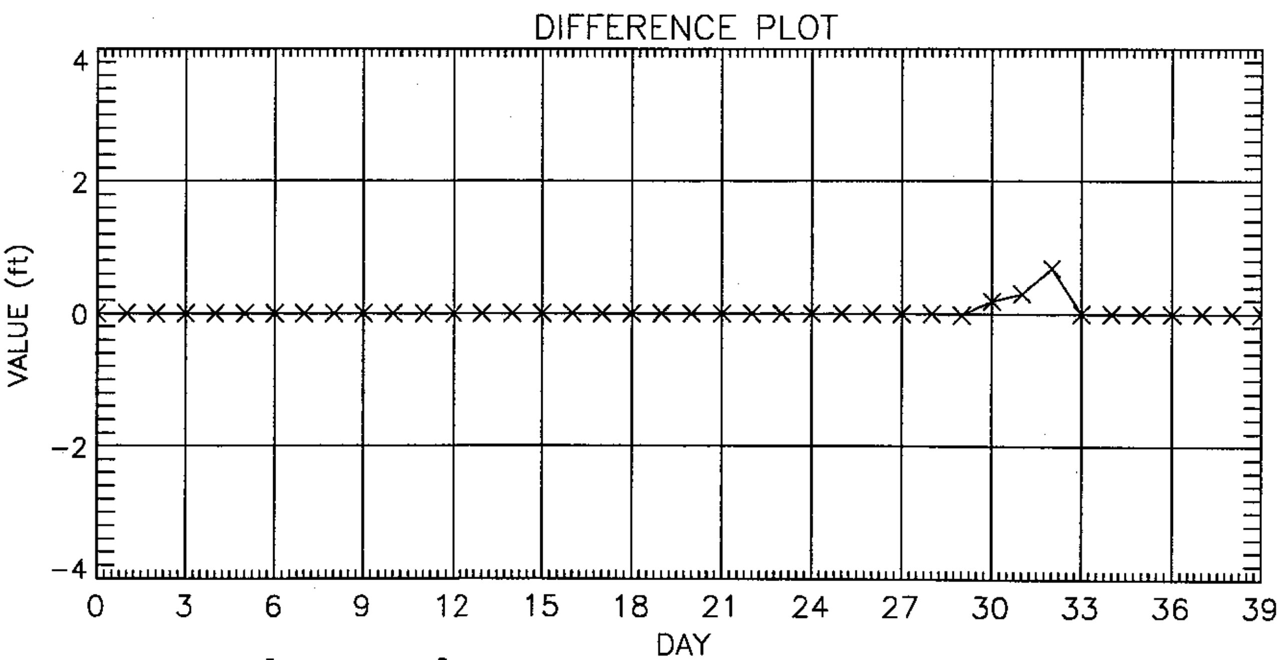
Layer 2 Row 43 Column 56 Hist.Stnd.Dev. 3.34 C.I.Year 0.10 C.I. DAY 0.10



Extreme Errors [0.0, 5.5] Average Absolute Error 0.53 Std.Error 2.04

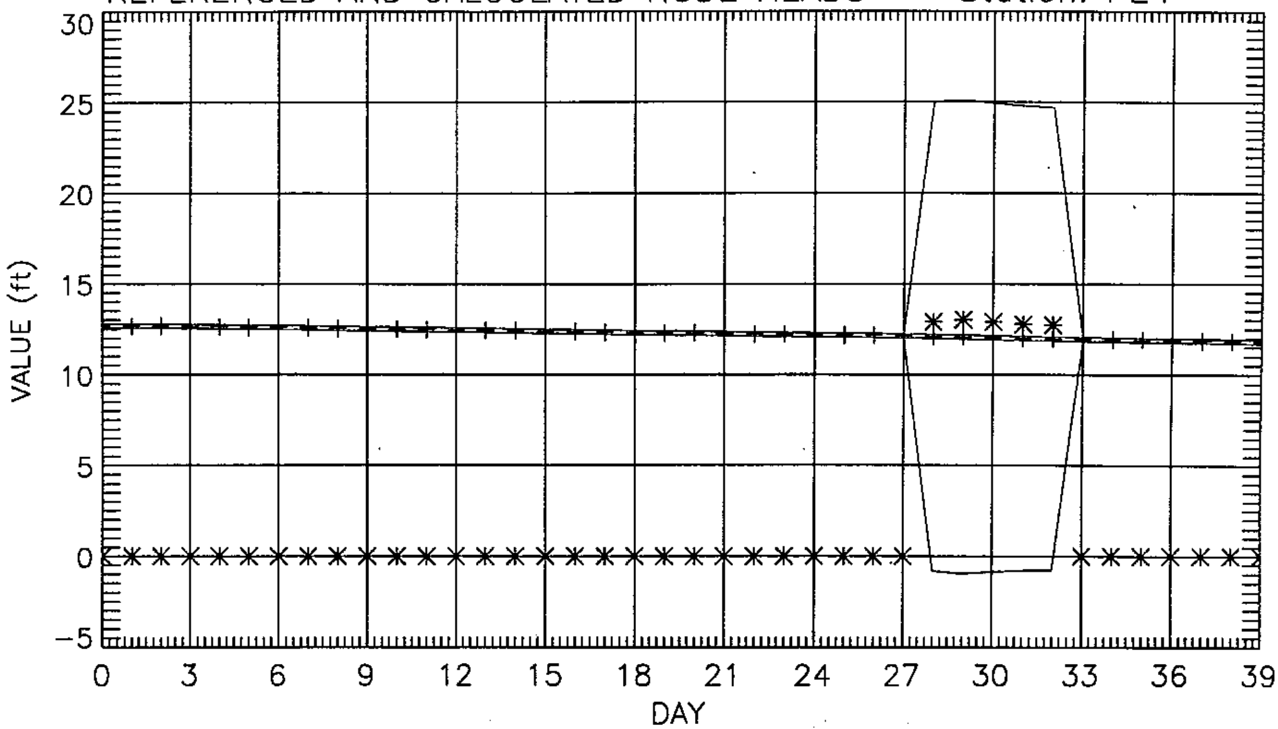


Layer 2 Row 43 Column 56 Hist.Stnd.Dev. 0.17 C.I.Year 0.08 C.I. DAY 0.00



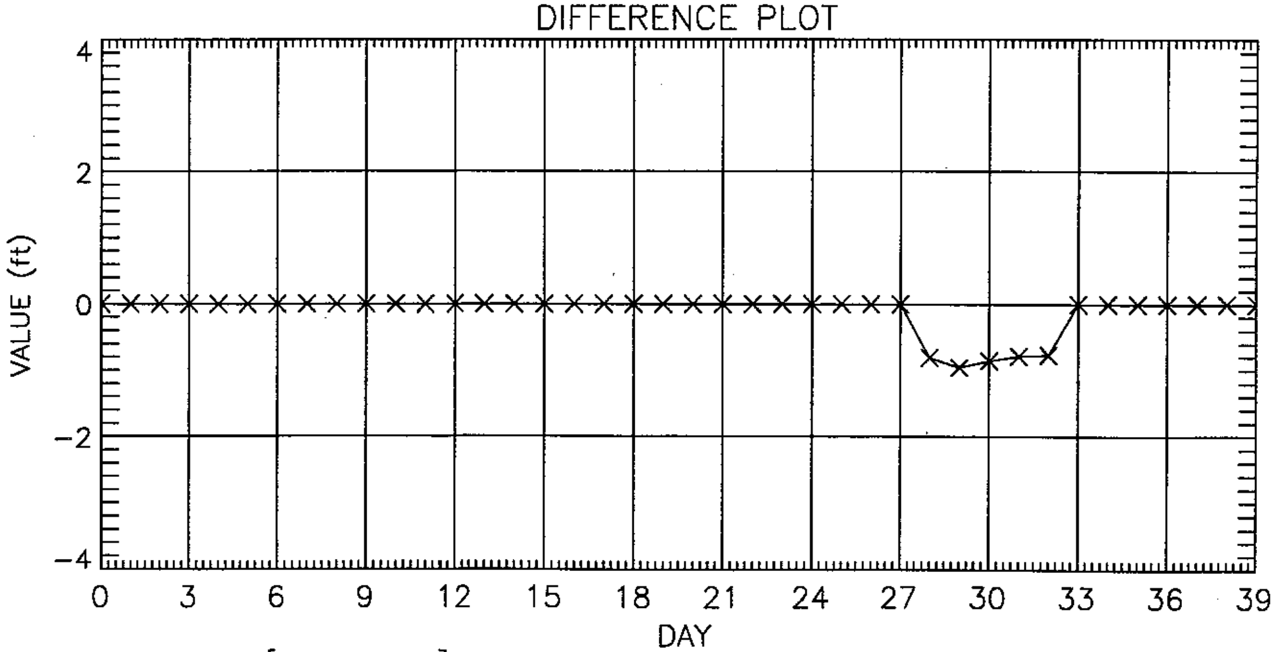
Extreme Errors [−0.0, 0.7] Average Absolute Error 0.03 Std.Error 0.30

REFERENCED AND CALCULATED NODE HEADS-- Station: PZ4

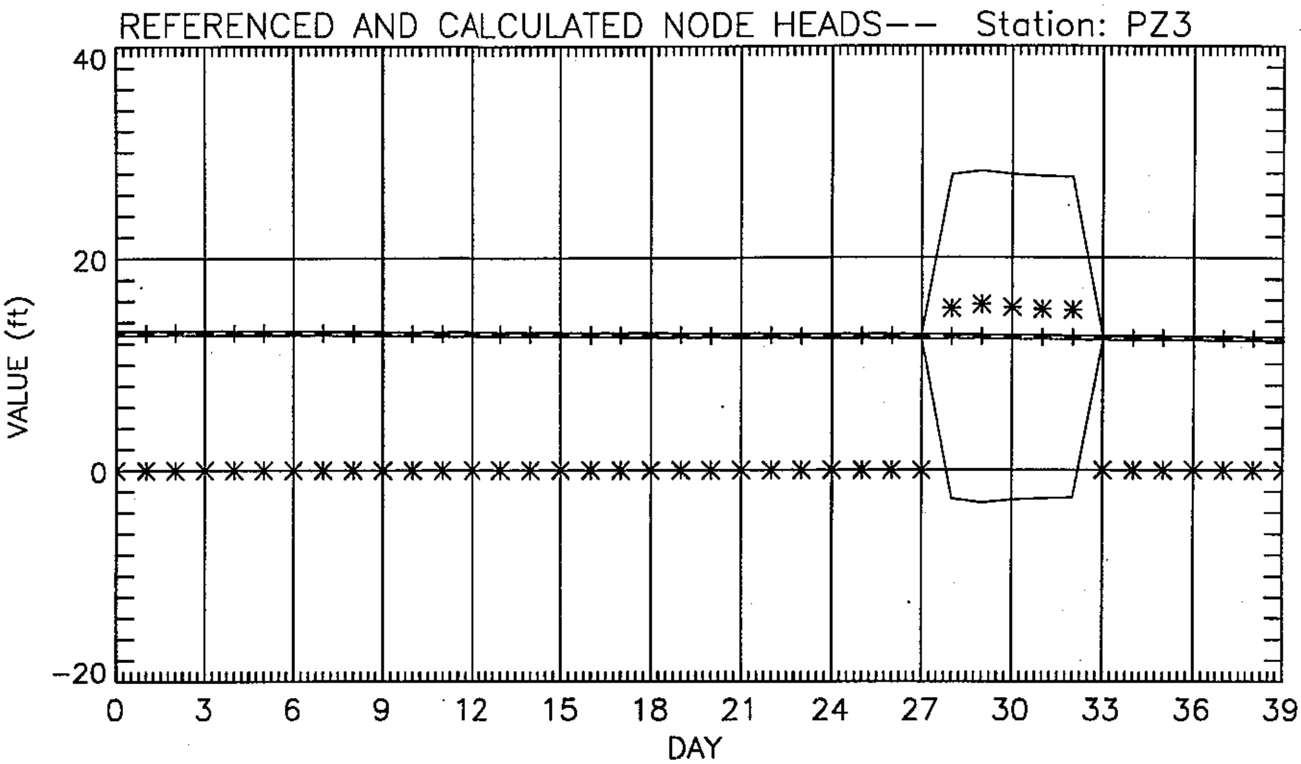


Layer 1 Row 43 Column 56 Hist.Stnd.Dev. 0.11 C.I.Year 0.13 C.I. DAY 0.00

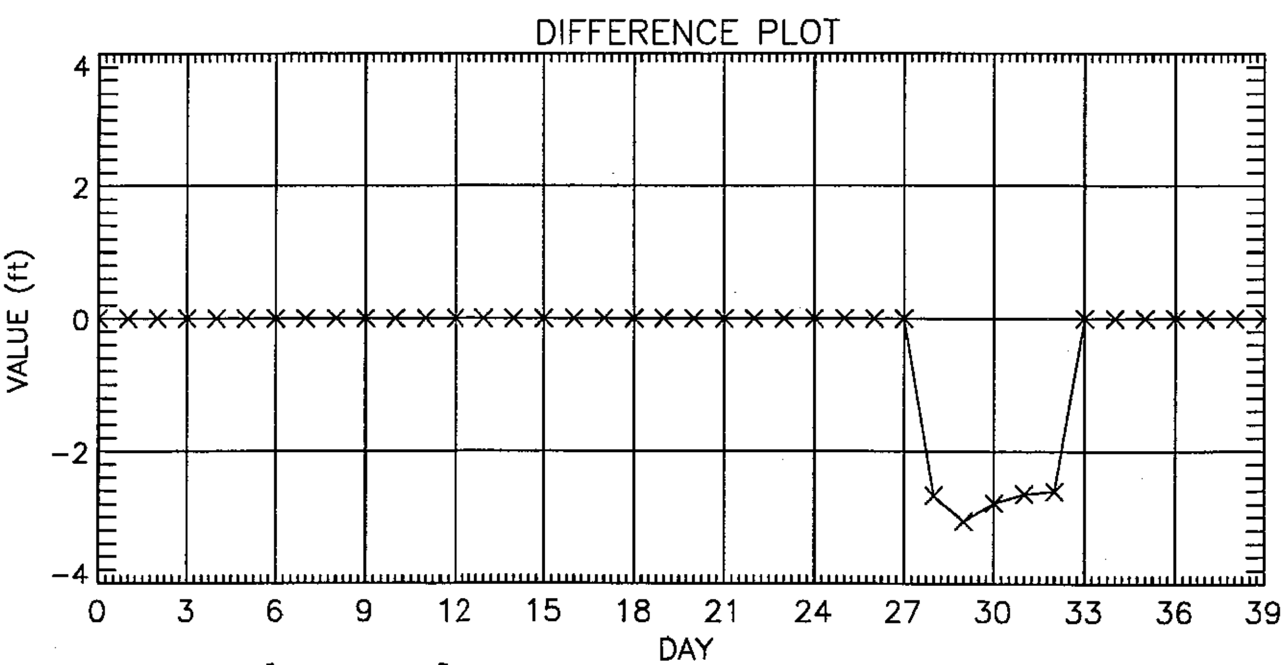
DIFFERENCE PLOT



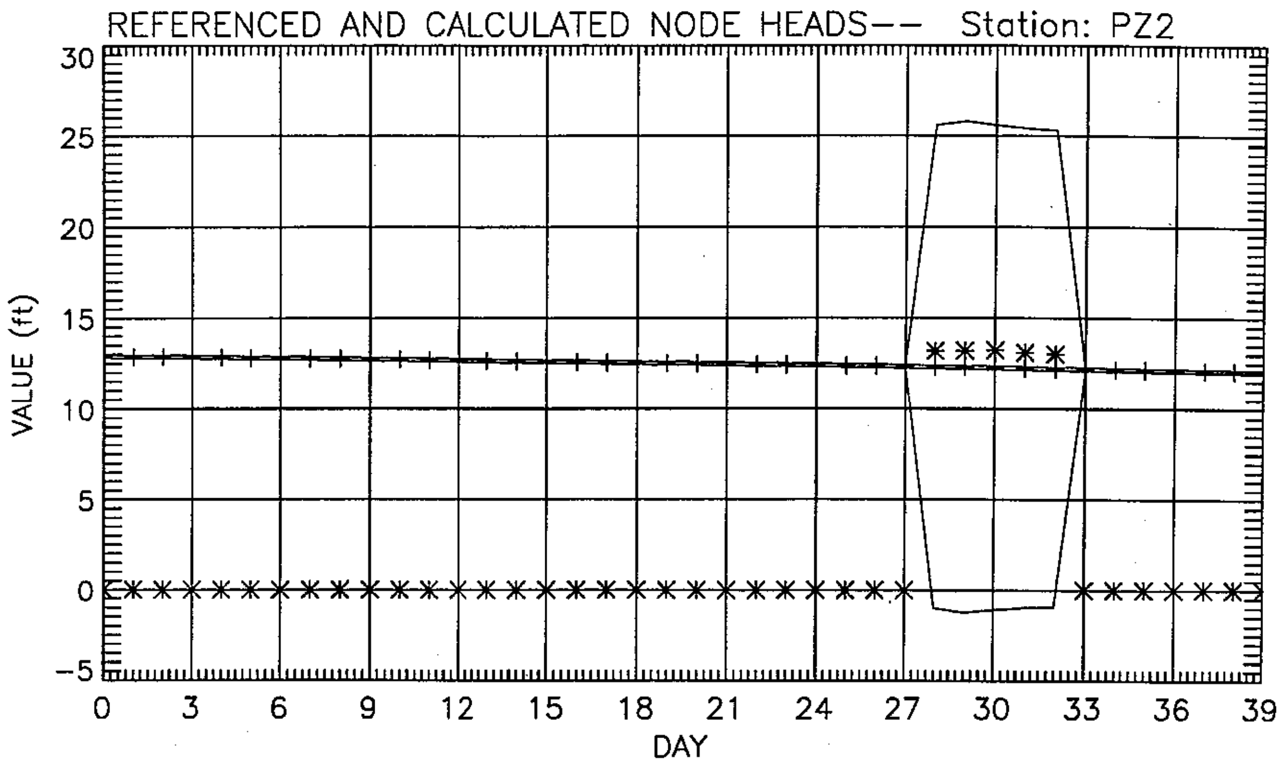
Extreme Errors [-1.0, 0.0] Average Absolute Error 0.10 Std.Error 0.08



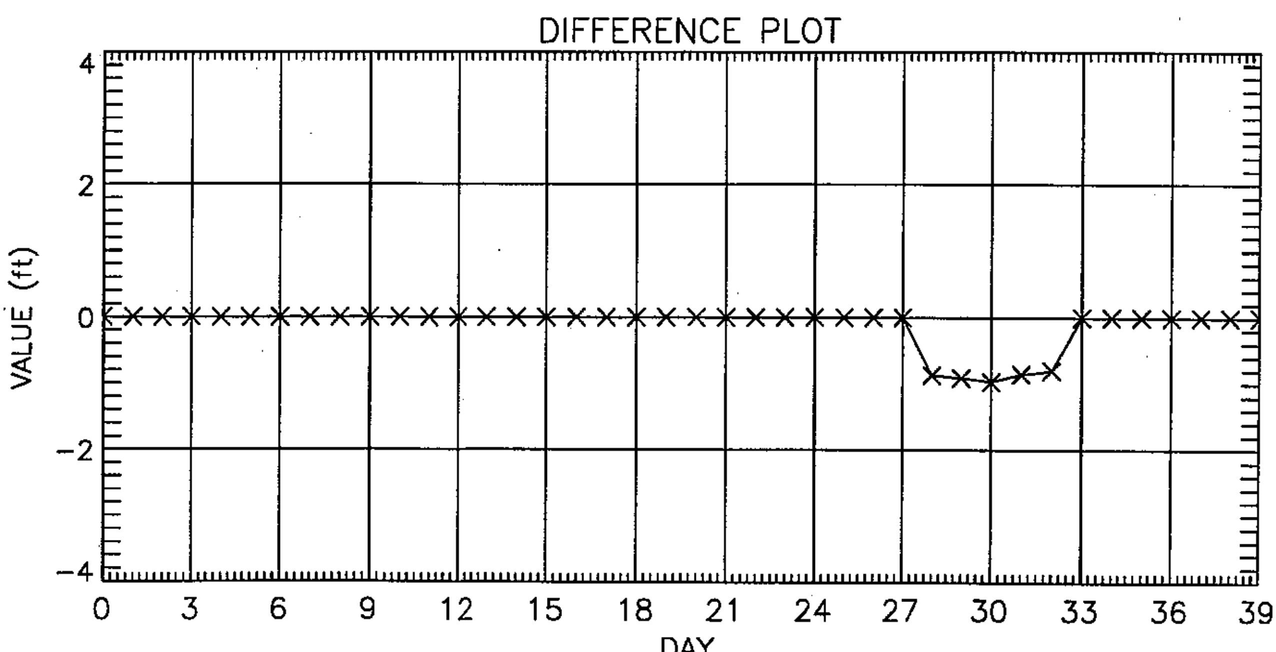
Layer 1 Row 43 Column 56 Hist.Stnd.Dev. 0.20 C.I.Year 0.13 C.I. DAY 0.00



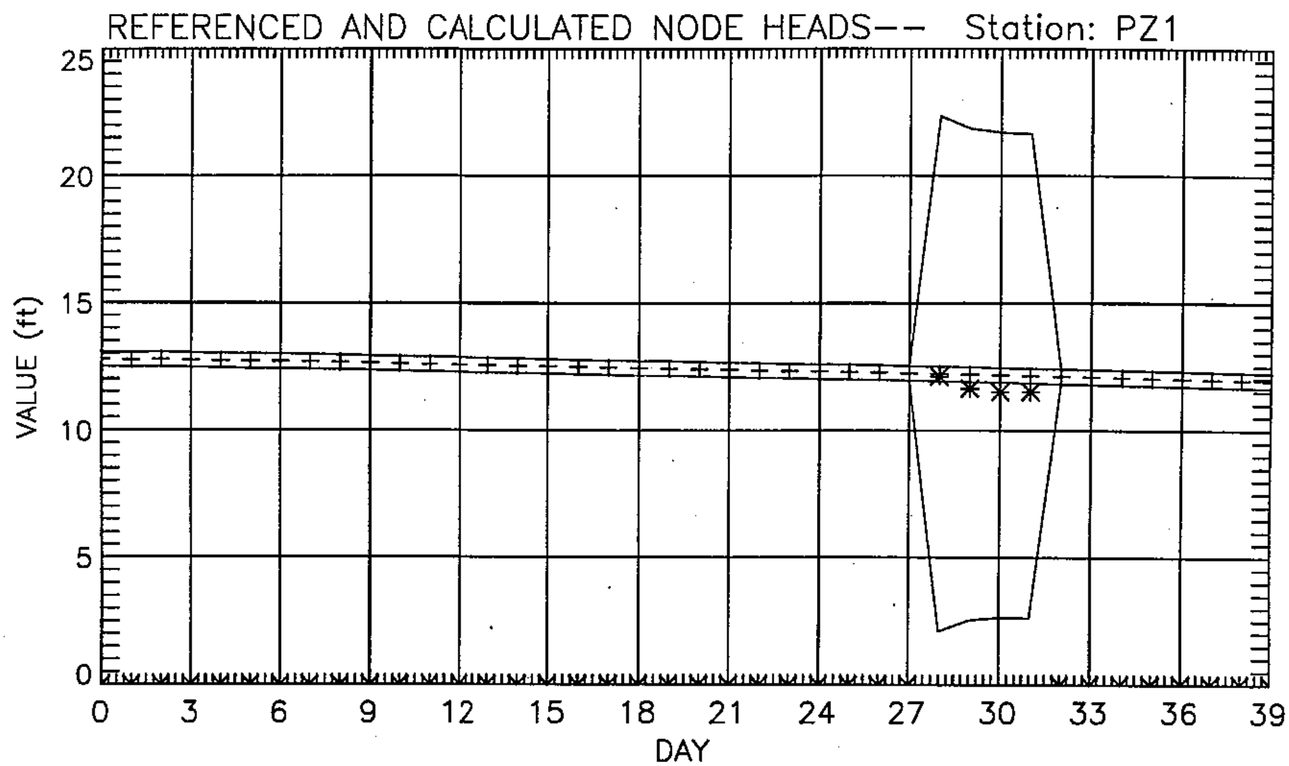
Extreme Errors [-3.1, 0.0] Average Absolute Error 0.34 Std.Error 0.18



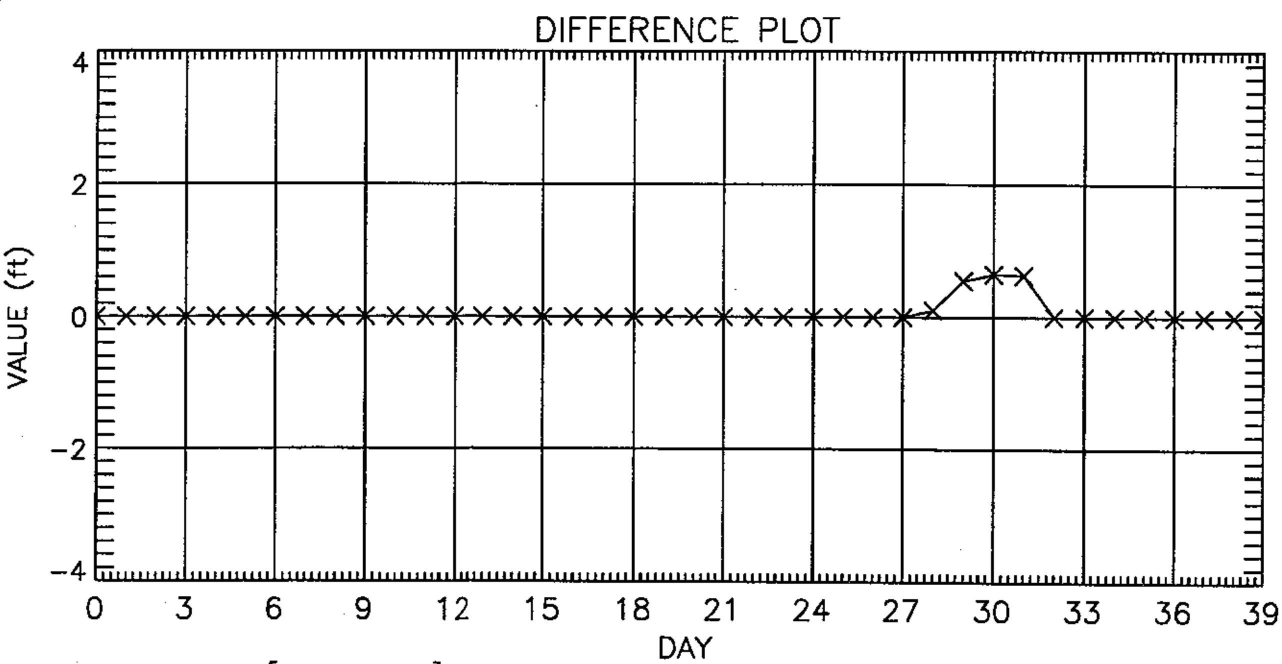
Layer 1 Row 43 Column 56 Hist.Stnd.Dev. 0.09 C.I.Year 0.13 C.I. DAY 0.00



Extreme Errors [−1.0, 0.0] Average Absolute Error 0.11 Std.Error 0.06

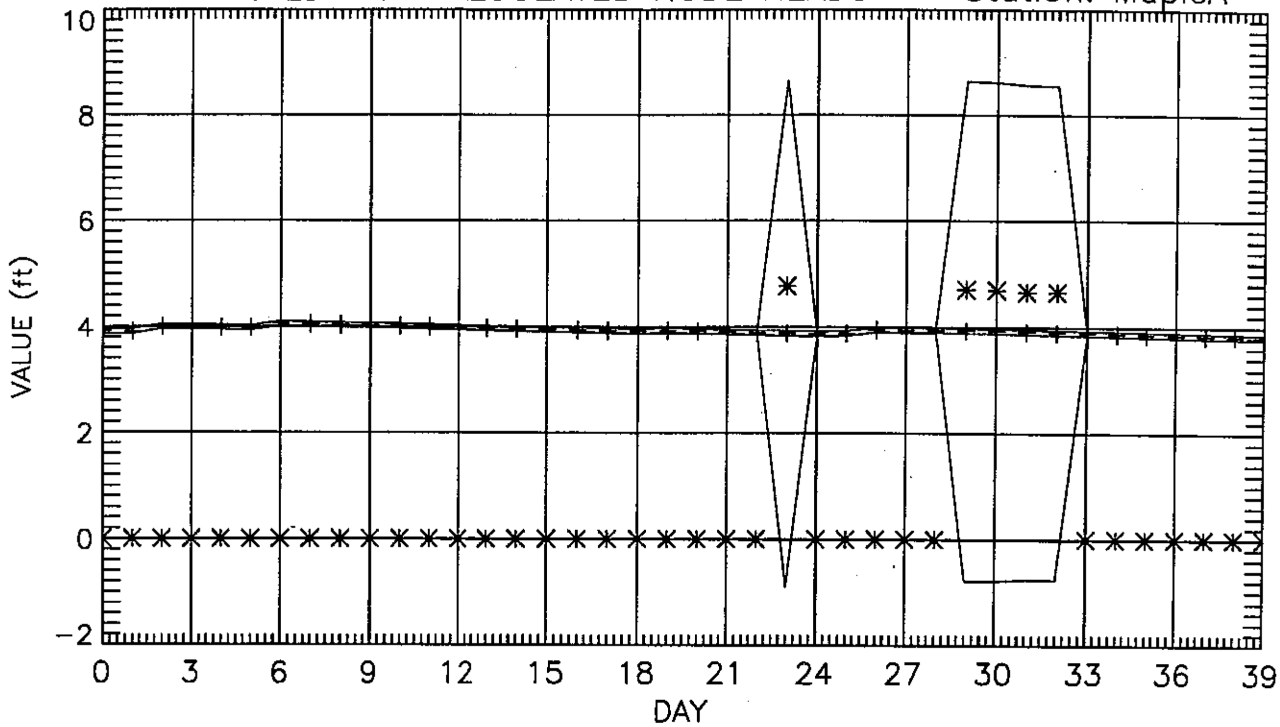


Layer 1 Row 43 Column 56 Hist.Stnd.Dev. 0.29 C.I.Year 0.08 C.I. DAY 0.00



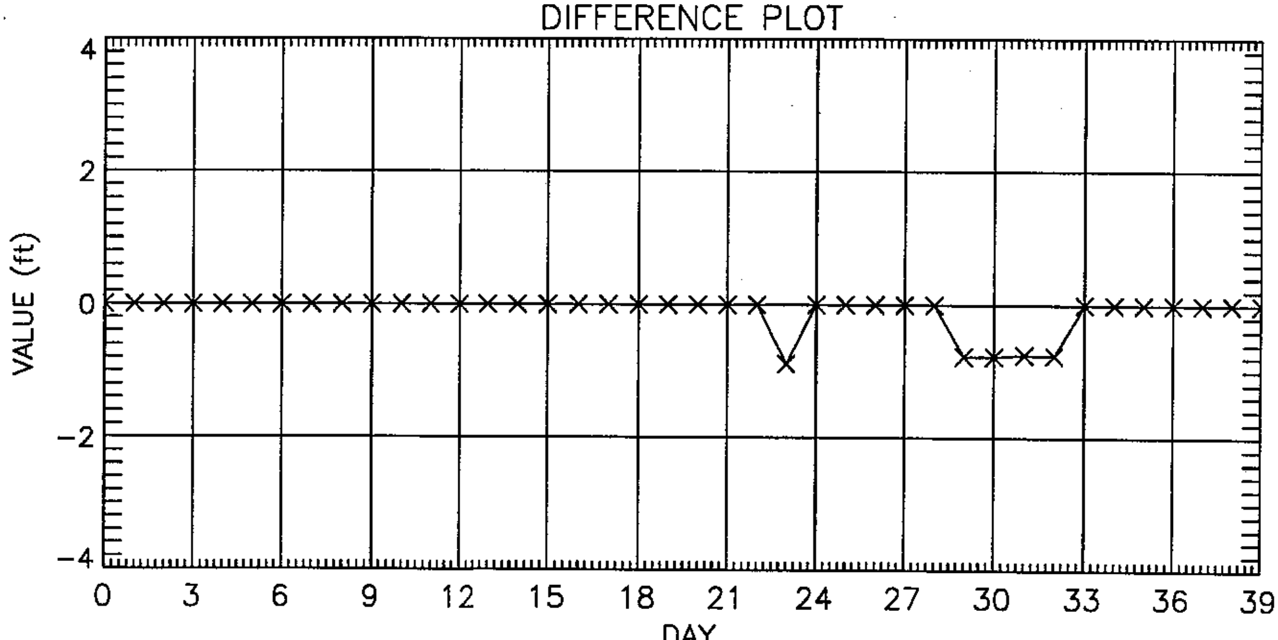
Extreme Errors [0.0, 0.6] Average Absolute Error 0.05 Std.Error 0.26

REFERENCED AND CALCULATED NODE HEADS-- Station: MapleA



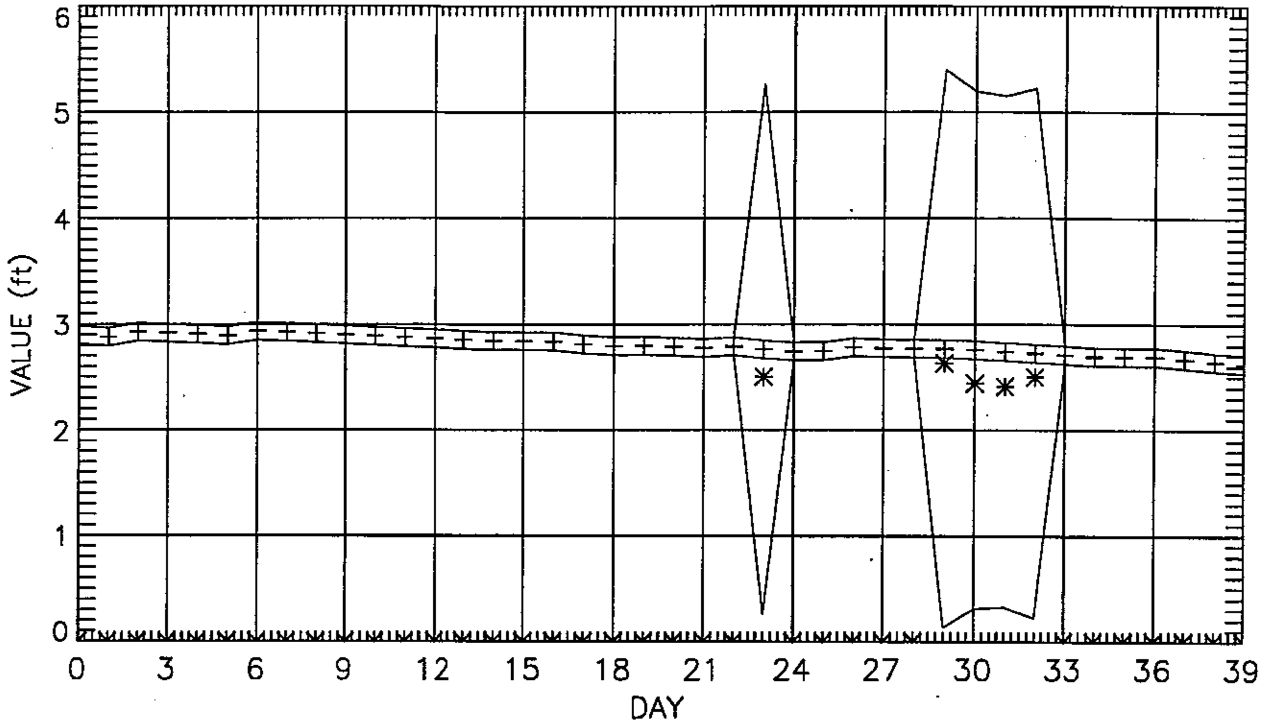
Layer 1 Row 61 Column 88 Hist.Stnd.Dev. 0.05 C.I.Year 0.13 C.I. DAY 0.00

DIFFERENCE PLOT



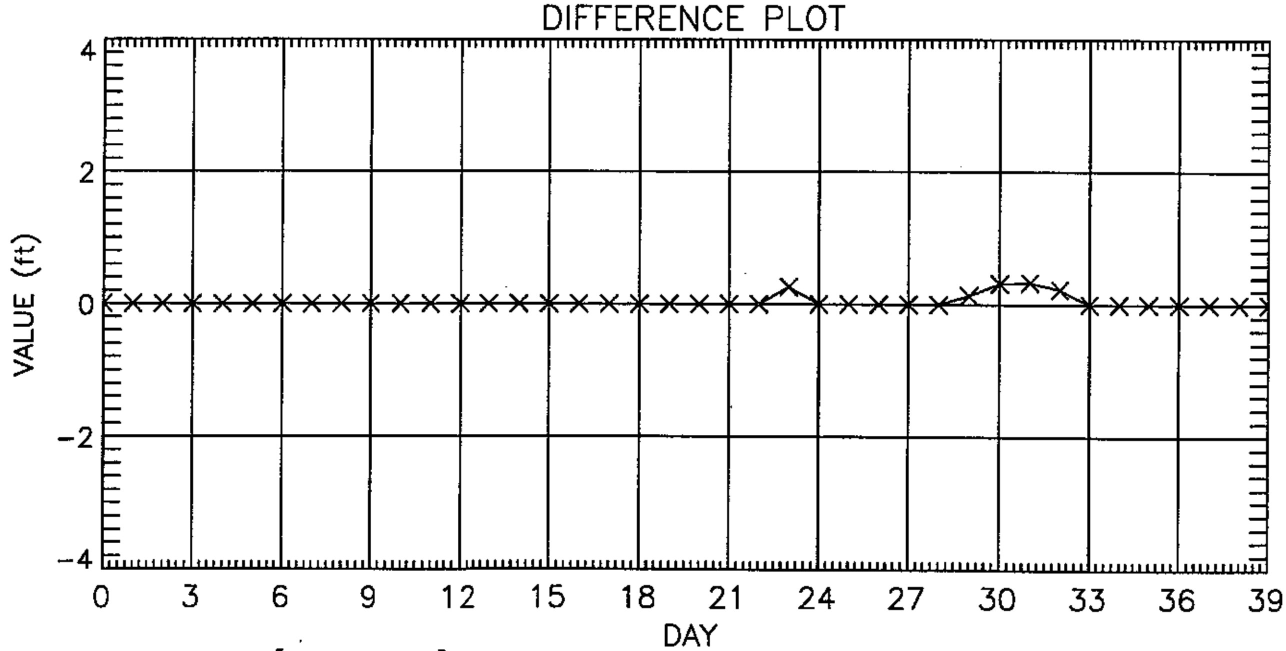
Extreme Errors [-0.9, 0.0] Average Absolute Error 0.10 Std.Error 0.06

REFERENCED AND CALCULATED NODE HEADS-- Station: HowardC

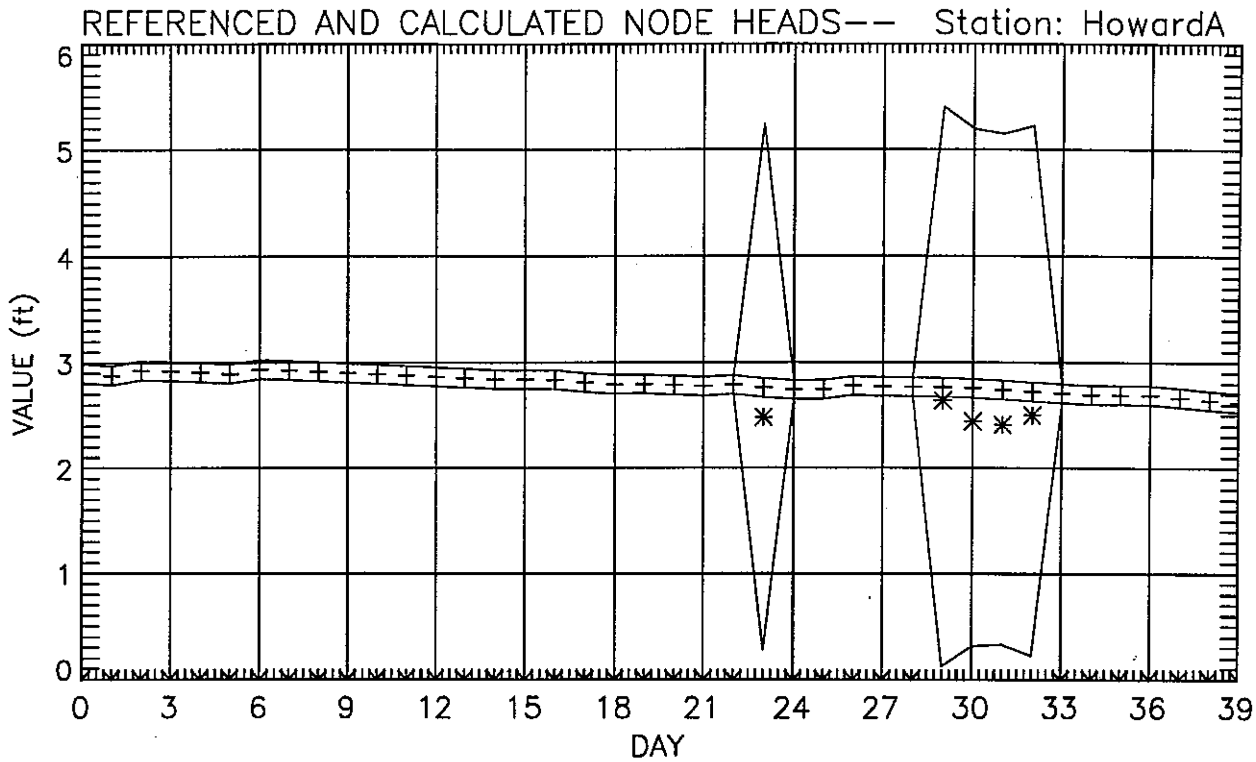


Layer 2 Row 25 Column 12 Hist.Stnd.Dev. 0.08 C.I.Year 0.13 C.I. DAY 0.00

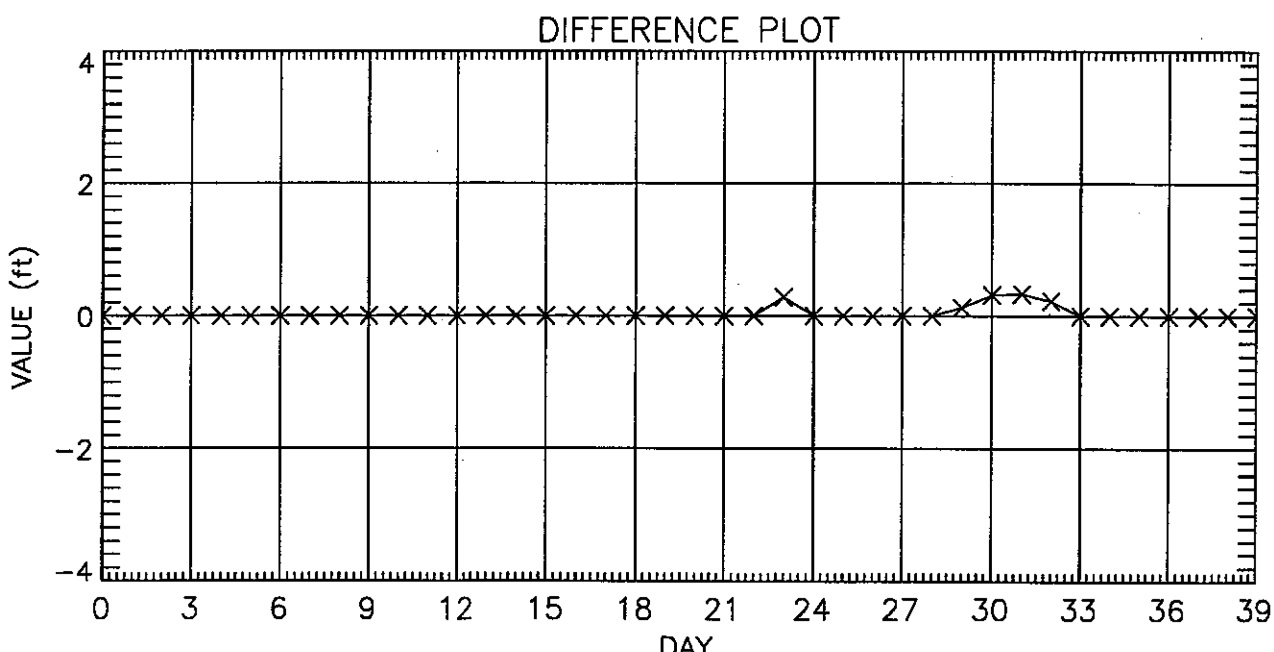
DIFFERENCE PLOT



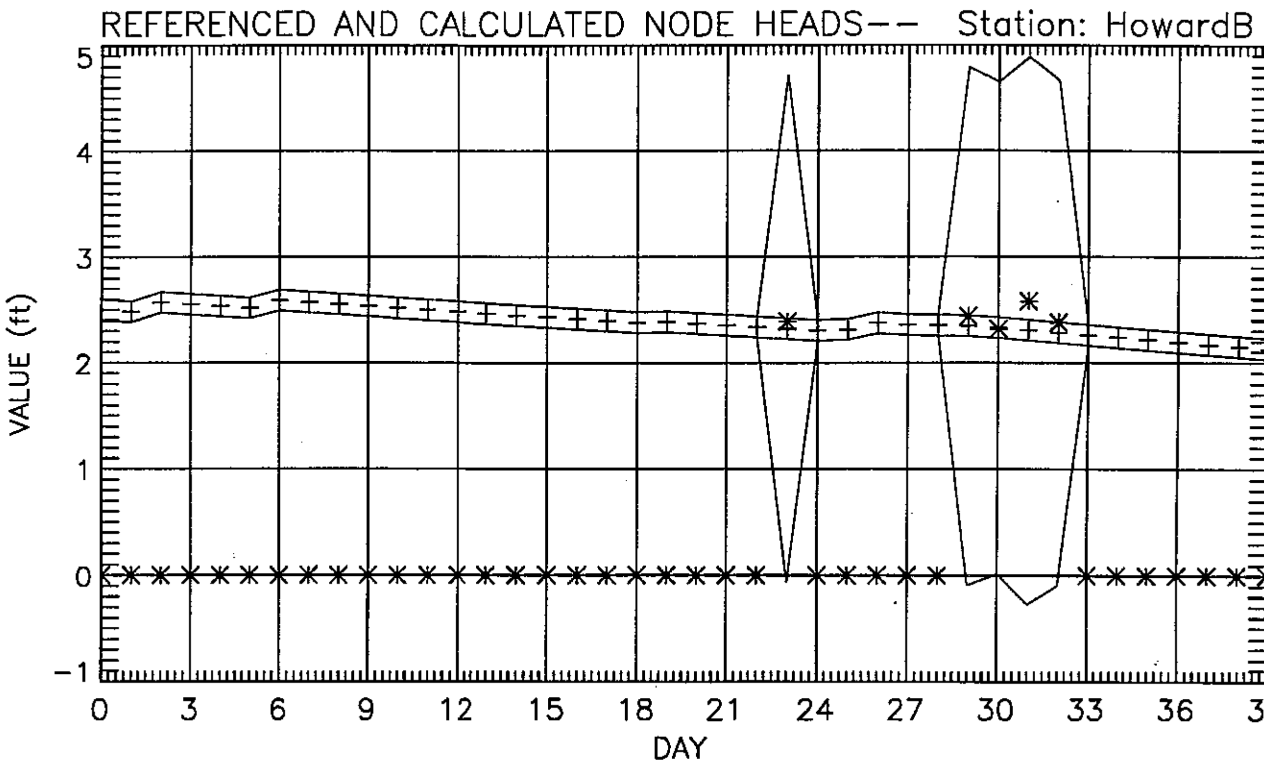
Extreme Errors [0.0, 0.3] Average Absolute Error 0.03 Std.Error 0.08



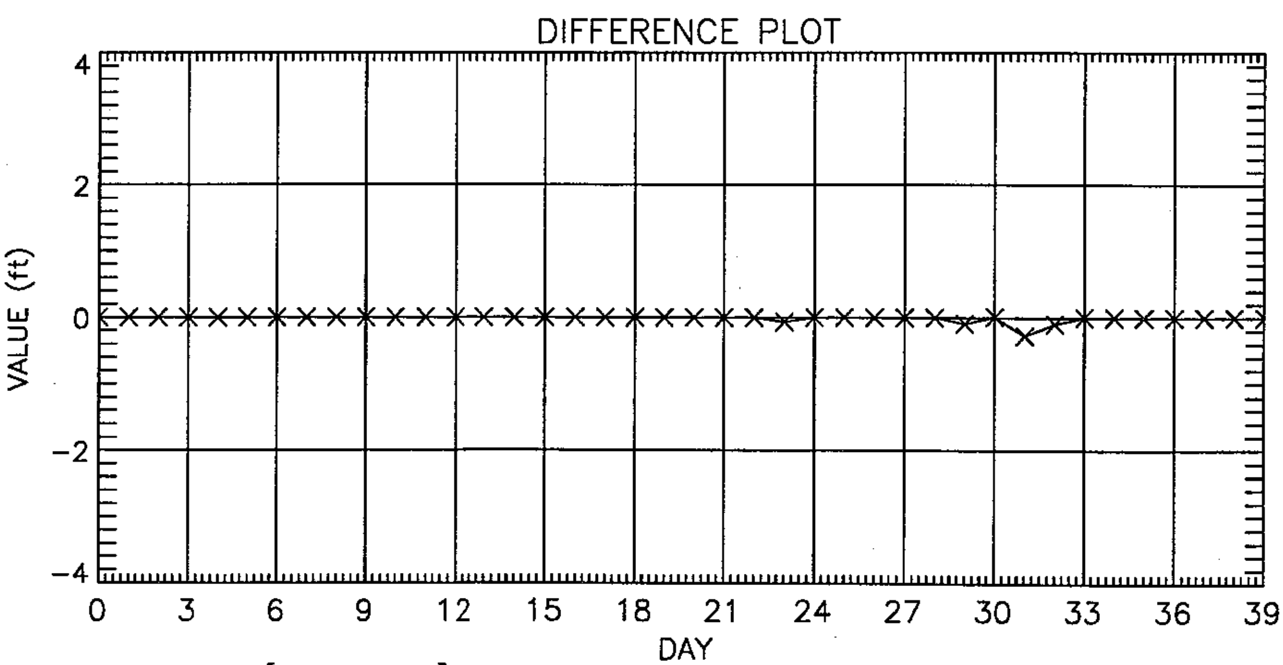
Layer 2 Row 25 Column 12 Hist.Stnd.Dev. 0.09 C.I.Year 0.13 C.I. DAY 0.00



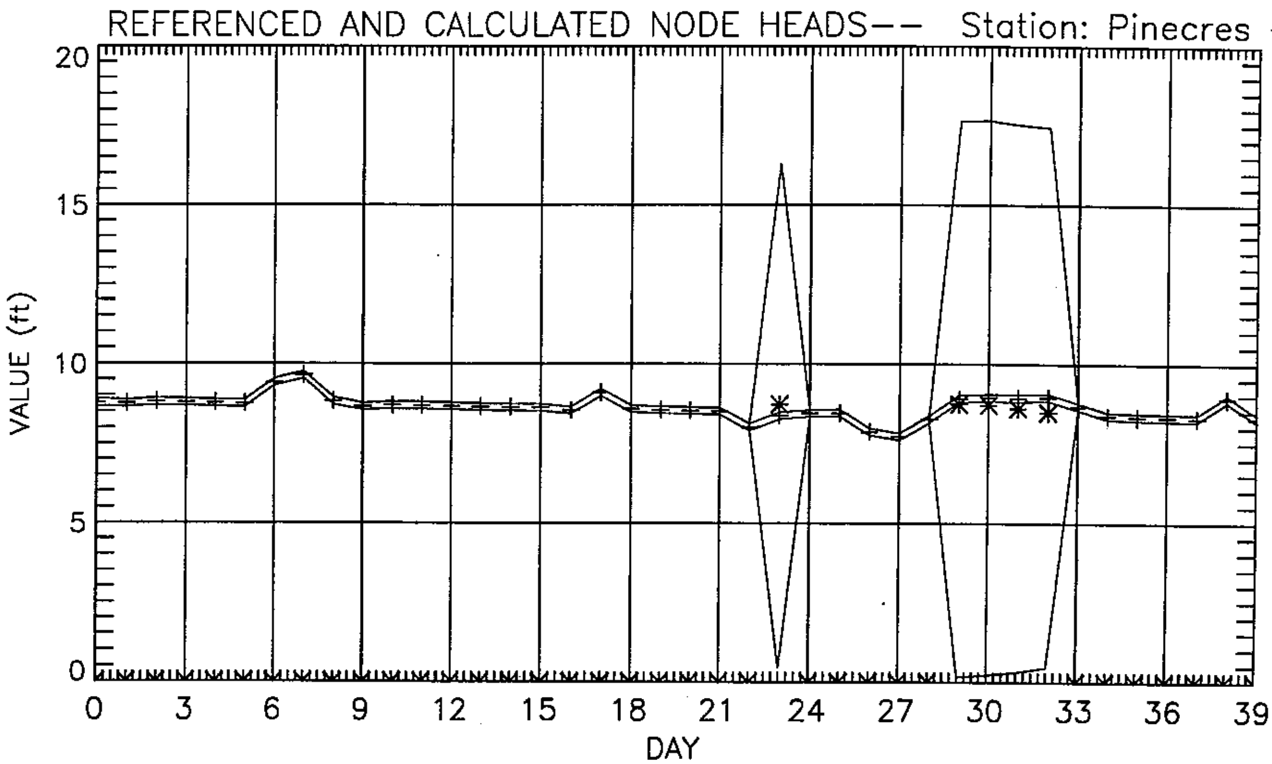
Extreme Errors [0.0, 0.3] Average Absolute Error 0.03 Std.Error 0.08



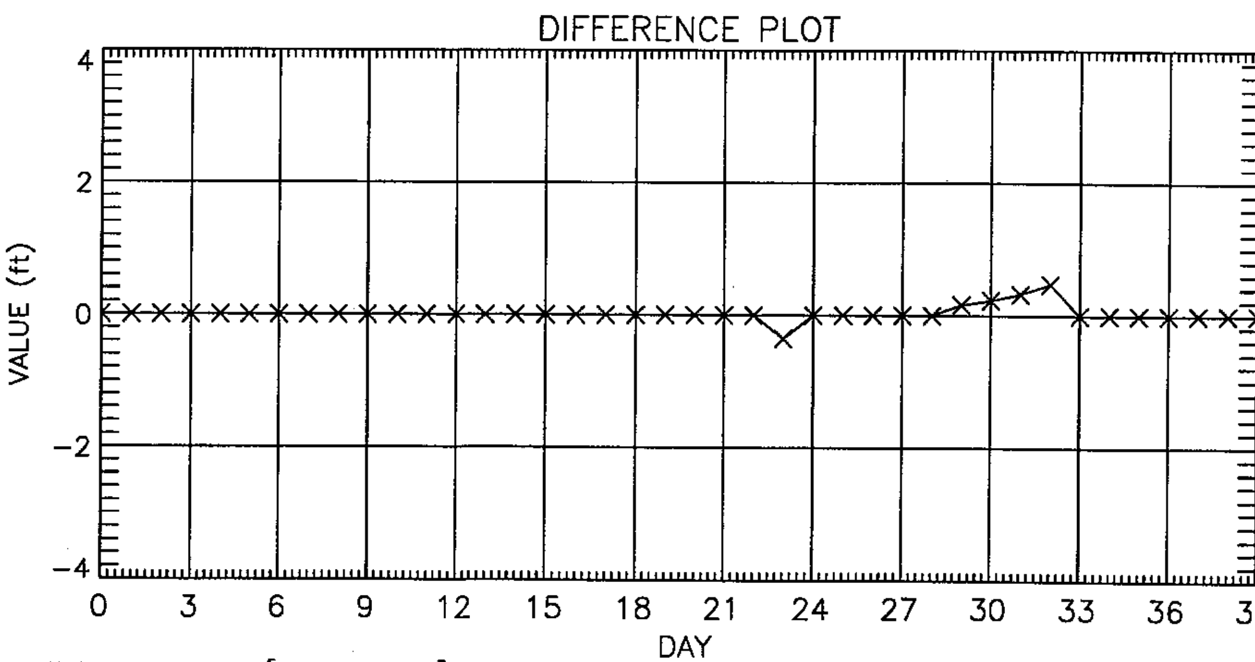
Layer 1 Row 25 Column 12 Hist.Stnd.Dev. 0.10 C.I.Year 0.03 C.I. DAY 0.00



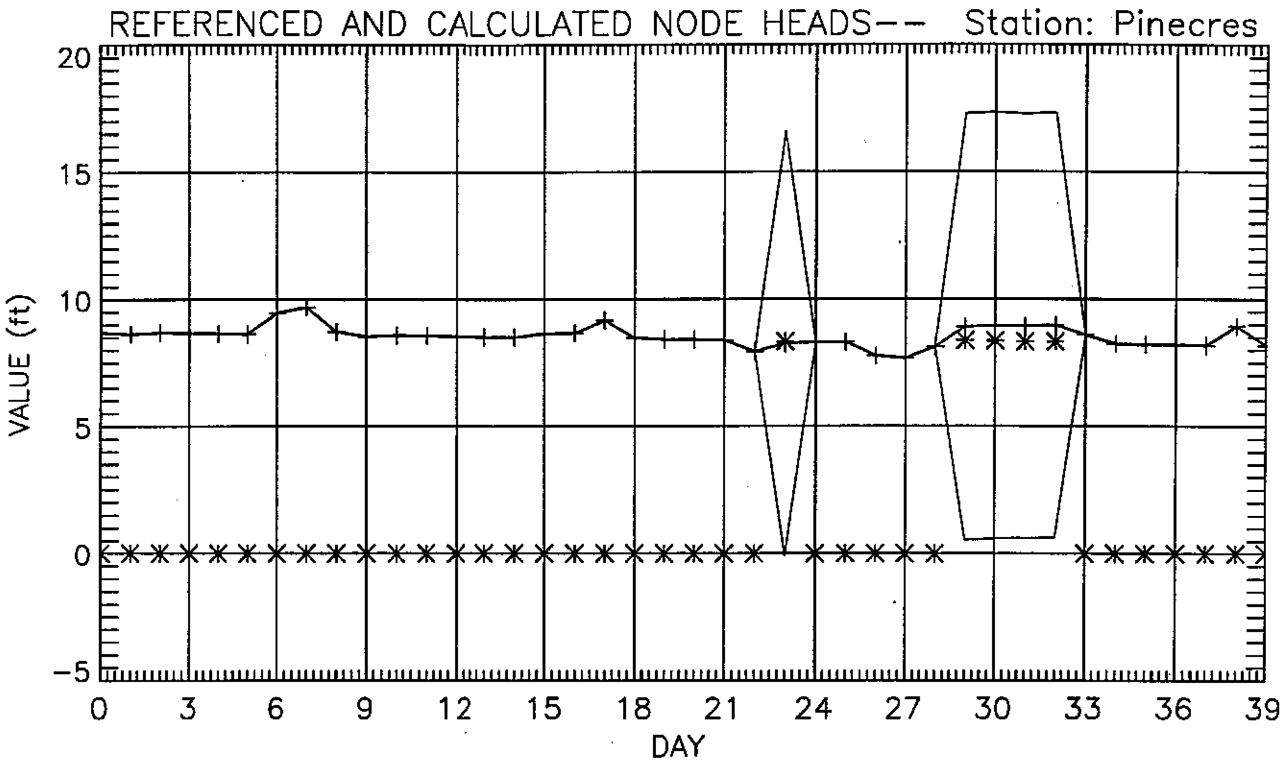
Extreme Errors [-0.3, 0.0] Average Absolute Error 0.01 Std.Error 0.10



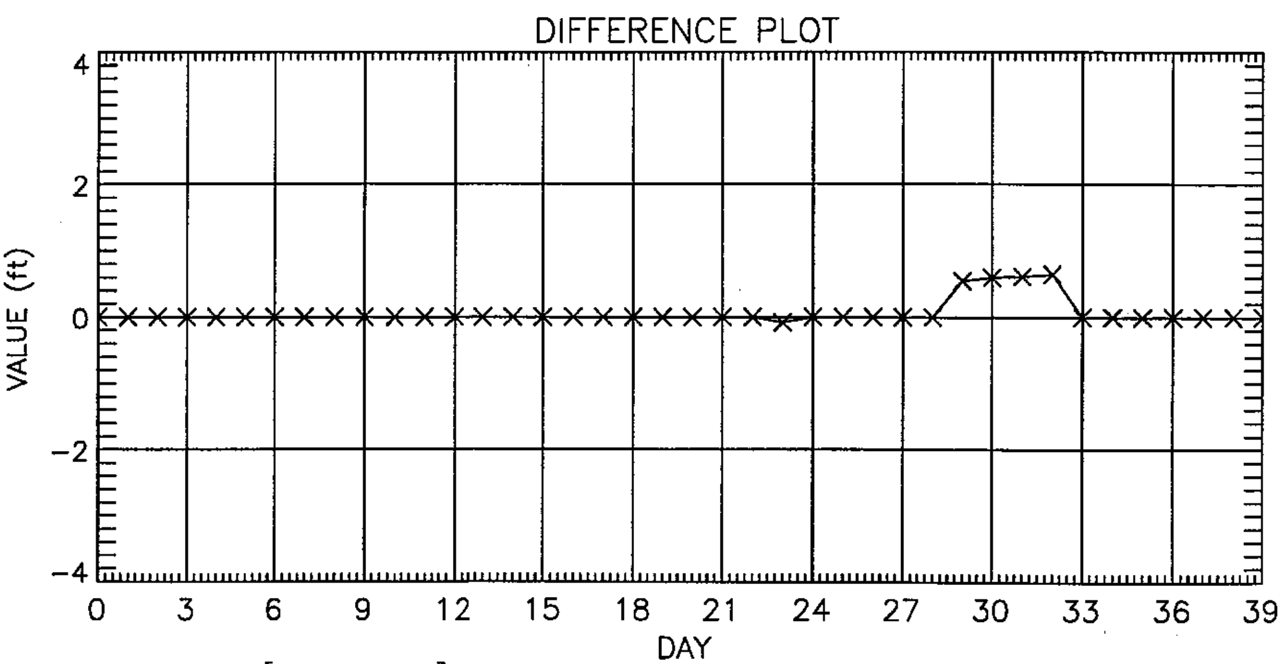
Layer 2 Row 55 Column 62 Hist.Stnd.Dev. 0.11 C.I.Year 0.13 C.I. DAY 0.00



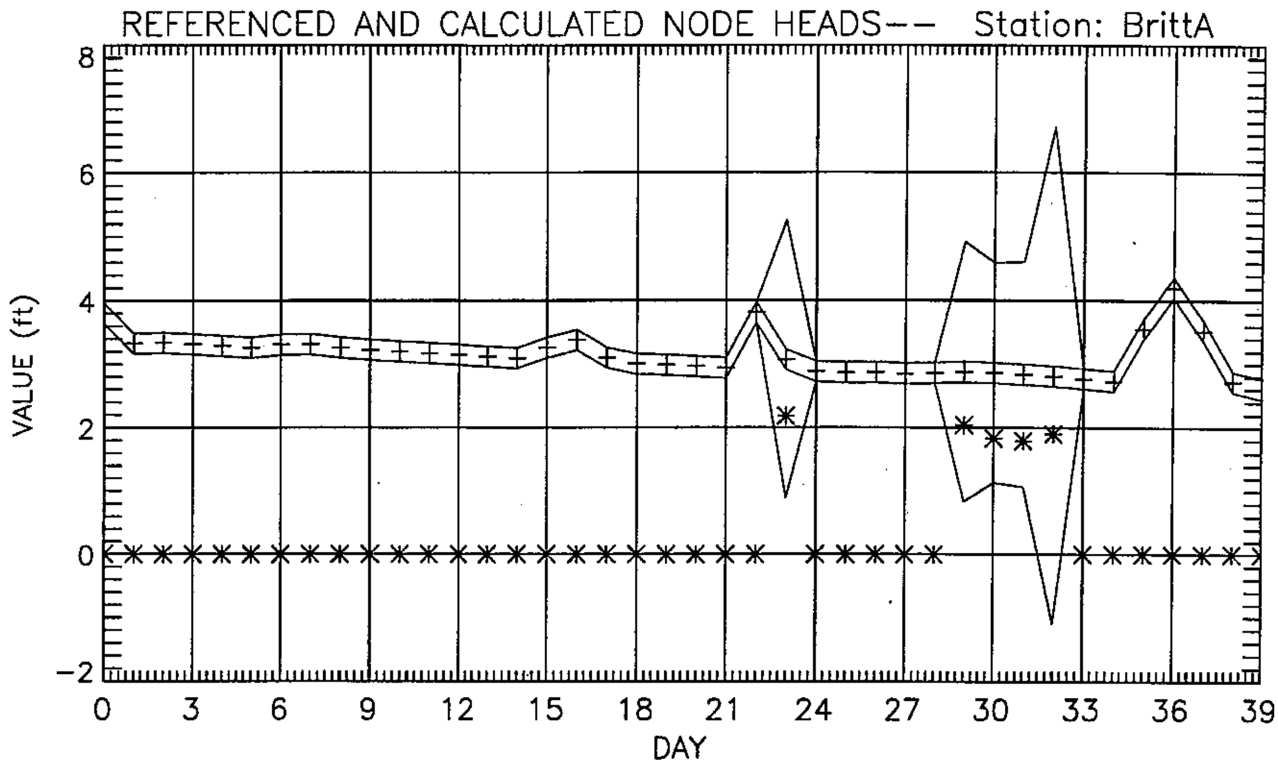
Extreme Errors [−0.4, 0.5] Average Absolute Error 0.04 Std.Error 0.31



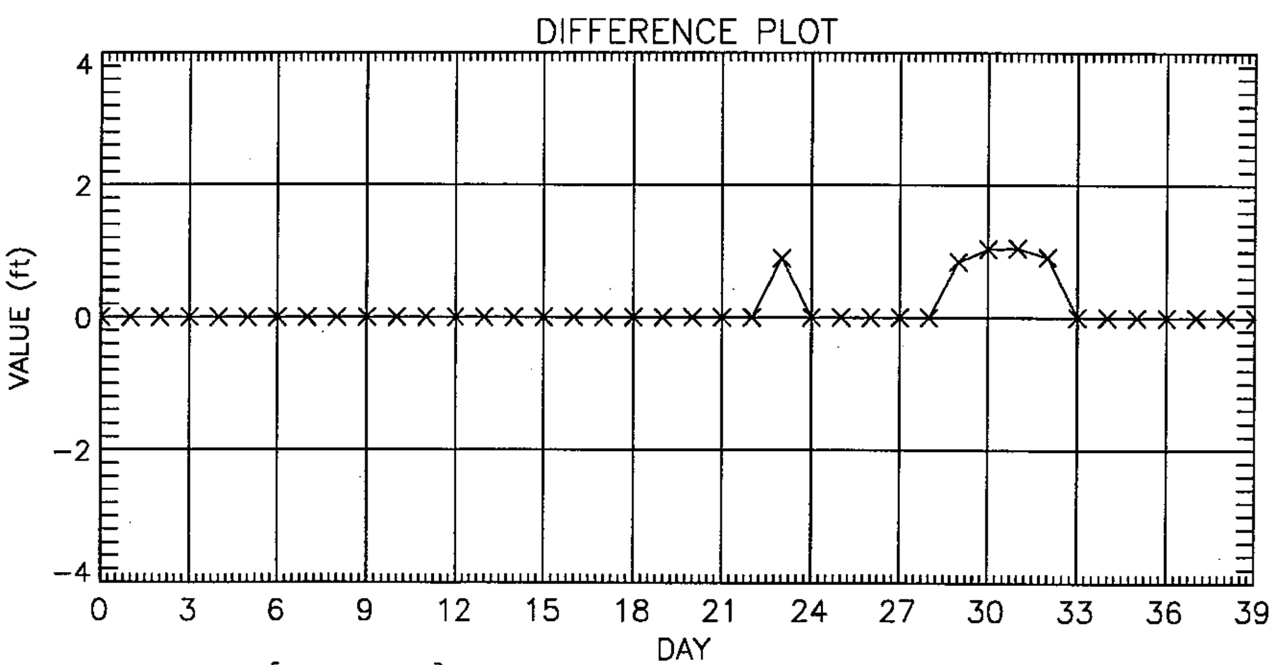
Layer 2 Row 55 Column 61 Hist.Stnd.Dev. 0.02 C.I.Year 0.13 C.I. DAY 0.00



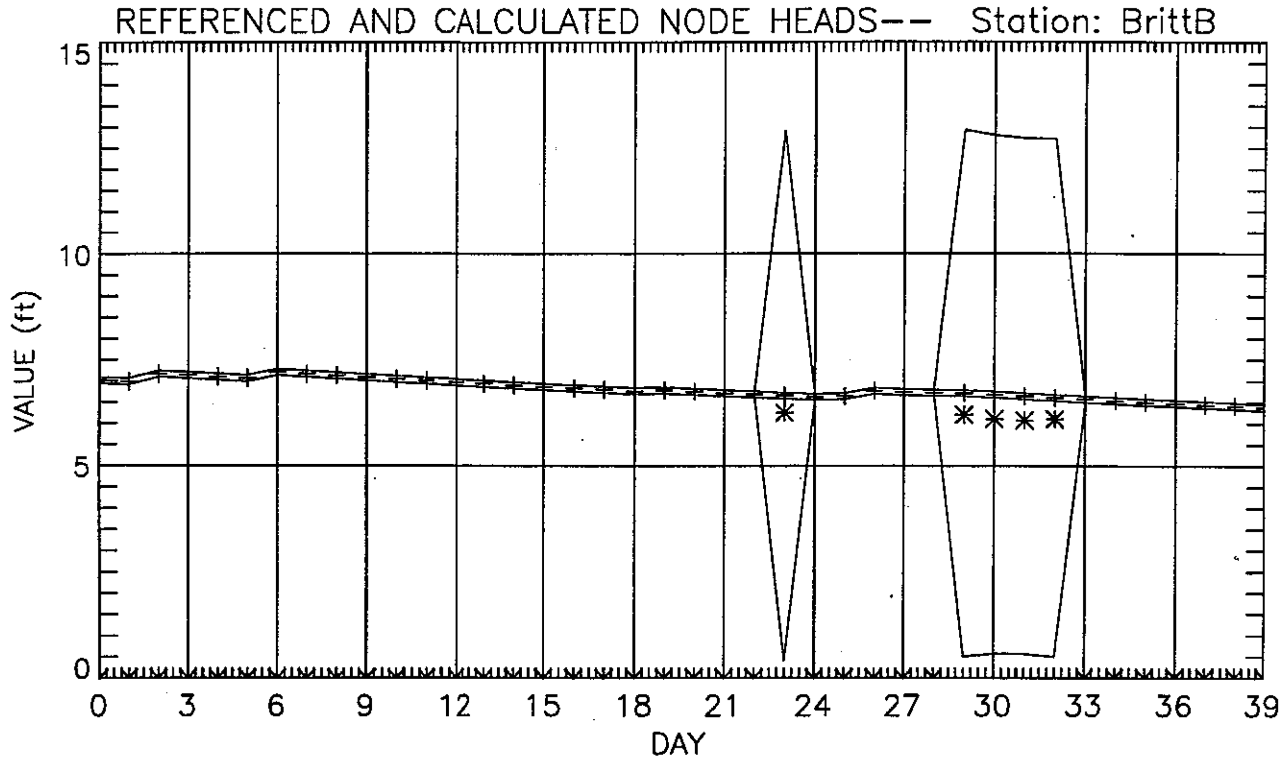
Extreme Errors [-0.1, 0.6] Average Absolute Error 0.06 Std.Error 0.31



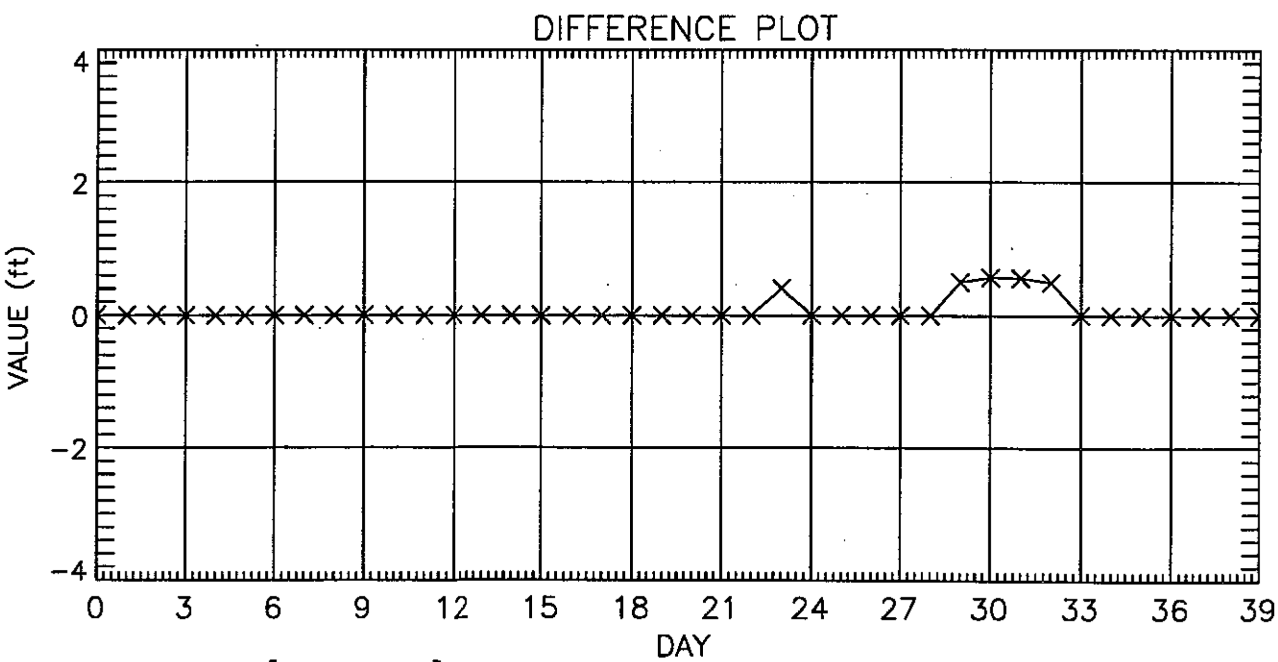
Layer 2 Row 43 Column 26 Hist.Stnd.Dev. 0.16 C.I.Year 0.13 C.I. DAY 0.00



Extreme Errors [0.0, 1.0] Average Absolute Error 0.12 Std.Error 0.09

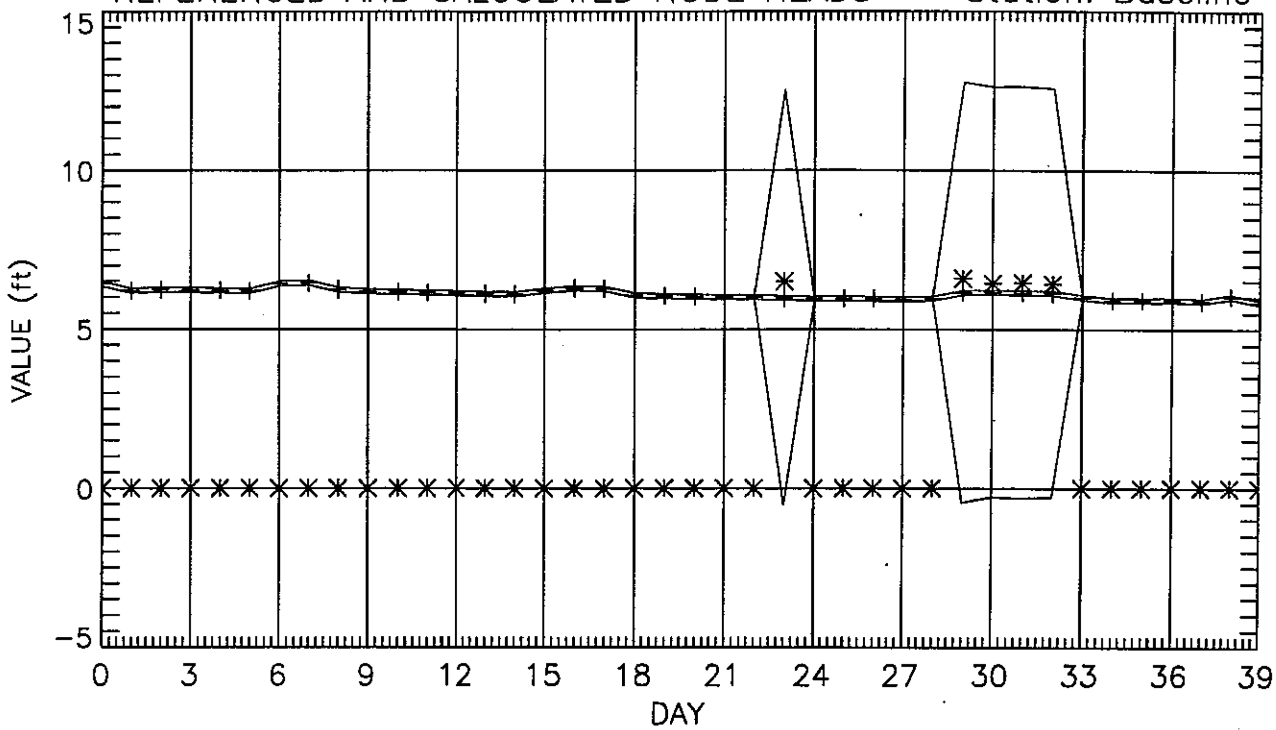


Layer 1 Row 43 Column 26 Hist.Stnd.Dev. 0.07 C.I.Year 0.13 C.I. DAY 0.00



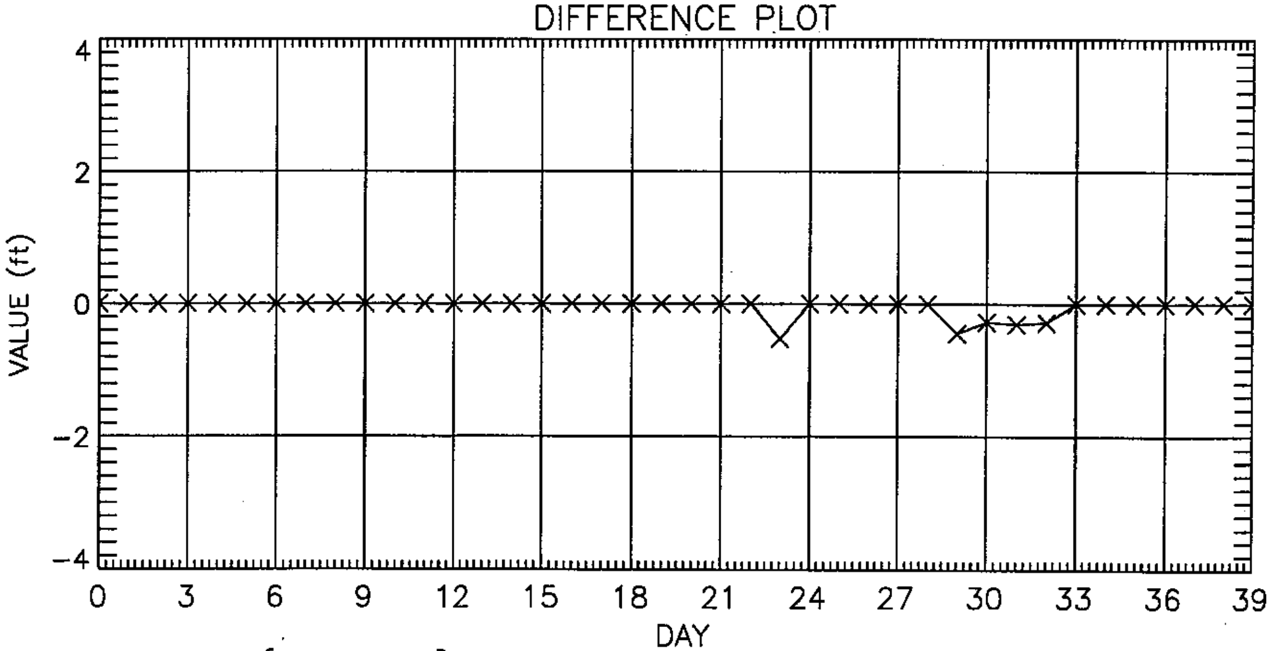
Extreme Errors [0.0, 0.6] Average Absolute Error 0.06 Std.Error 0.07

REFERENCED AND CALCULATED NODE HEADS-- Station: Baseline

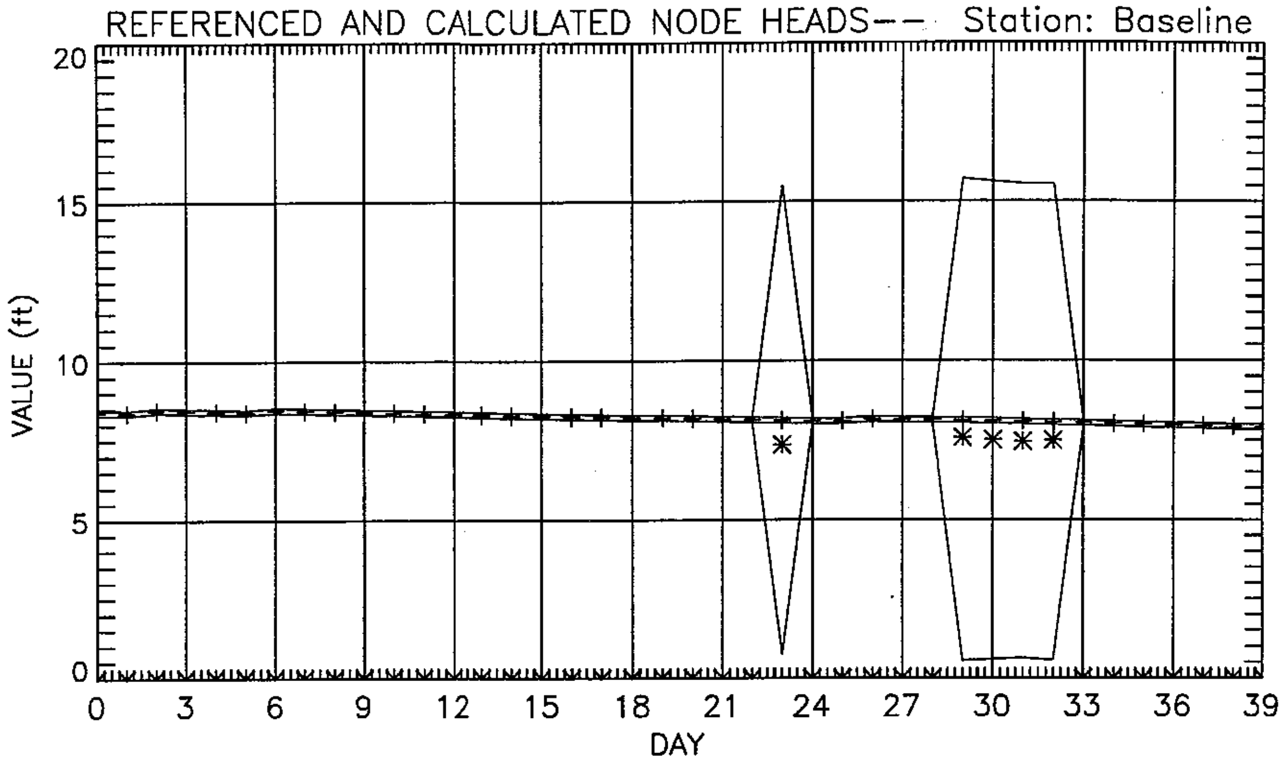


Layer 2 Row 68 Column 44 Hist.Stnd.Dev. 0.07 C.I.Year 0.13 C.I. DAY 0.00

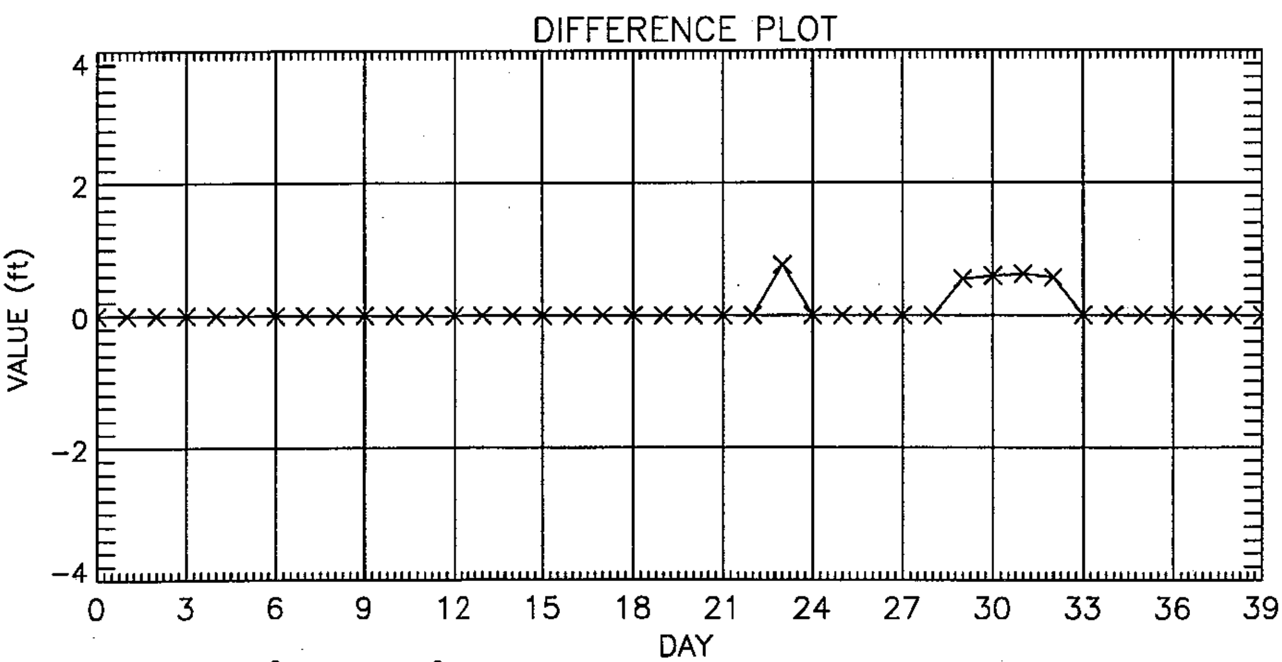
DIFFERENCE PLOT



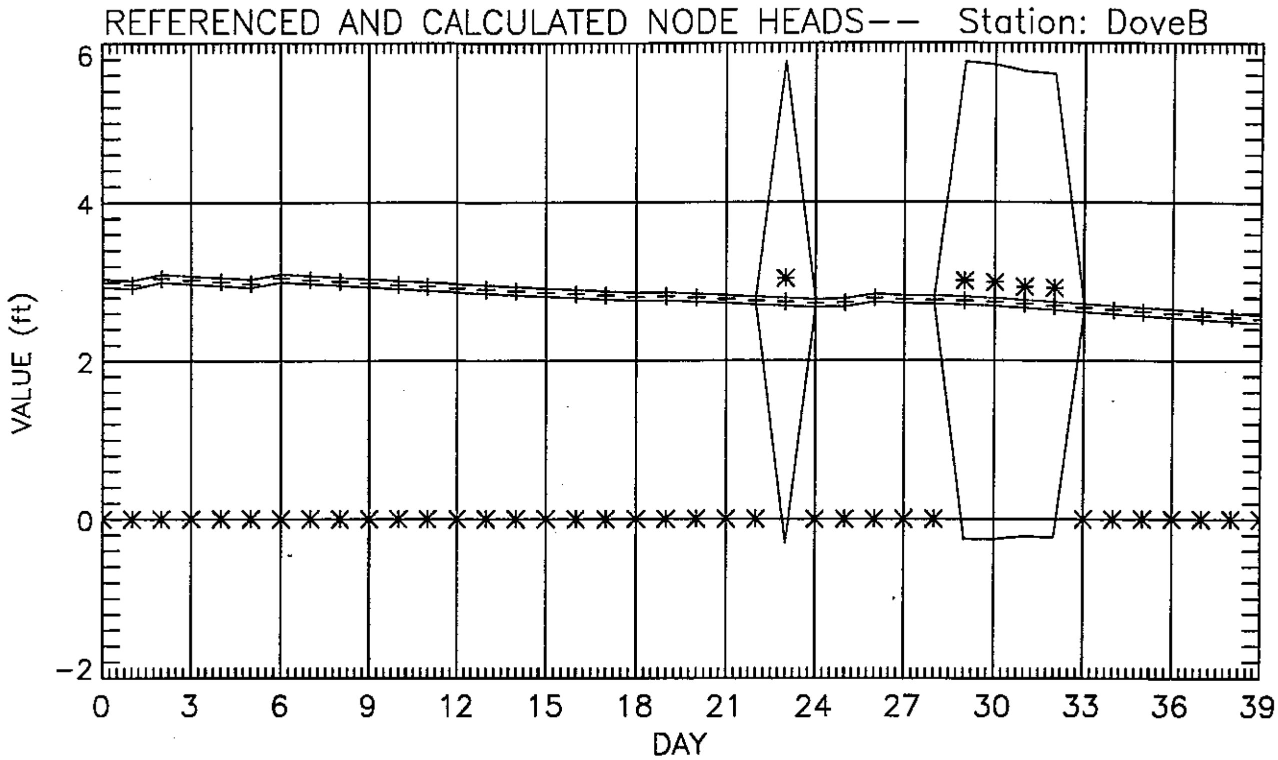
Extreme Errors [-0.5, 0.0] Average Absolute Error 0.05 Std.Error 0.11



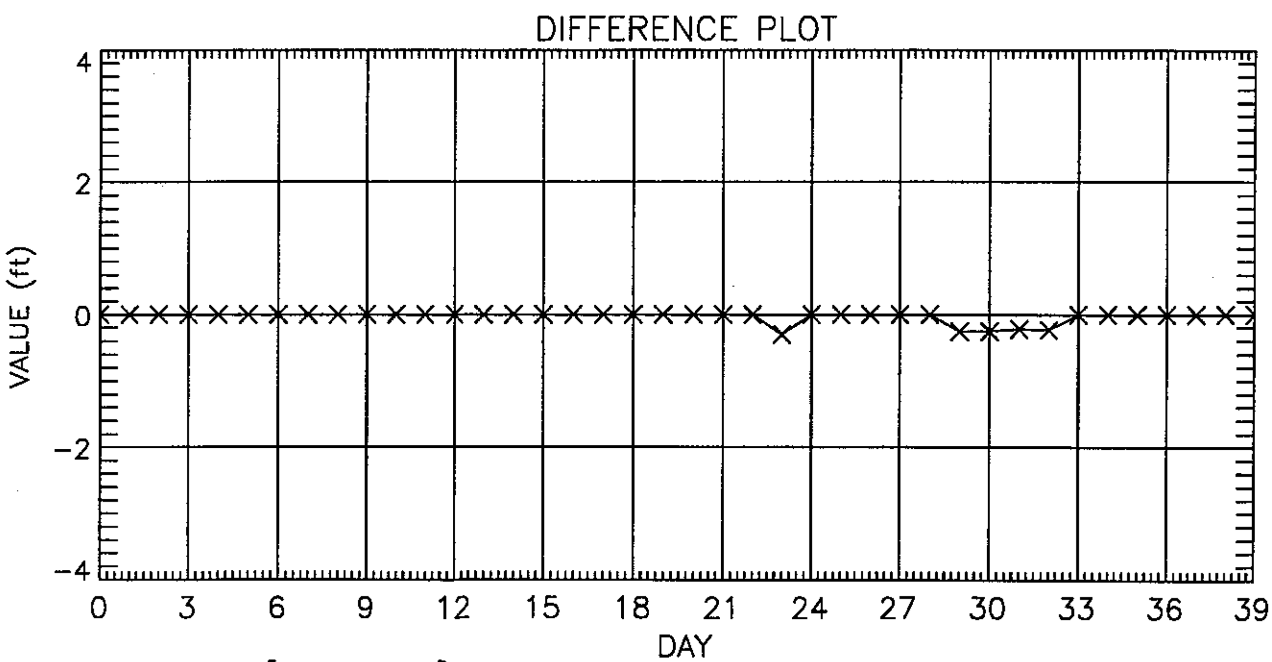
Layer 1 Row 68 Column 44 Hist.Stnd.Dev. 0.08 C.I.Year 0.13 C.I. DAY 0.00



Extreme Errors [0.0, 0.8] Average Absolute Error 0.08 Std.Error 0.08

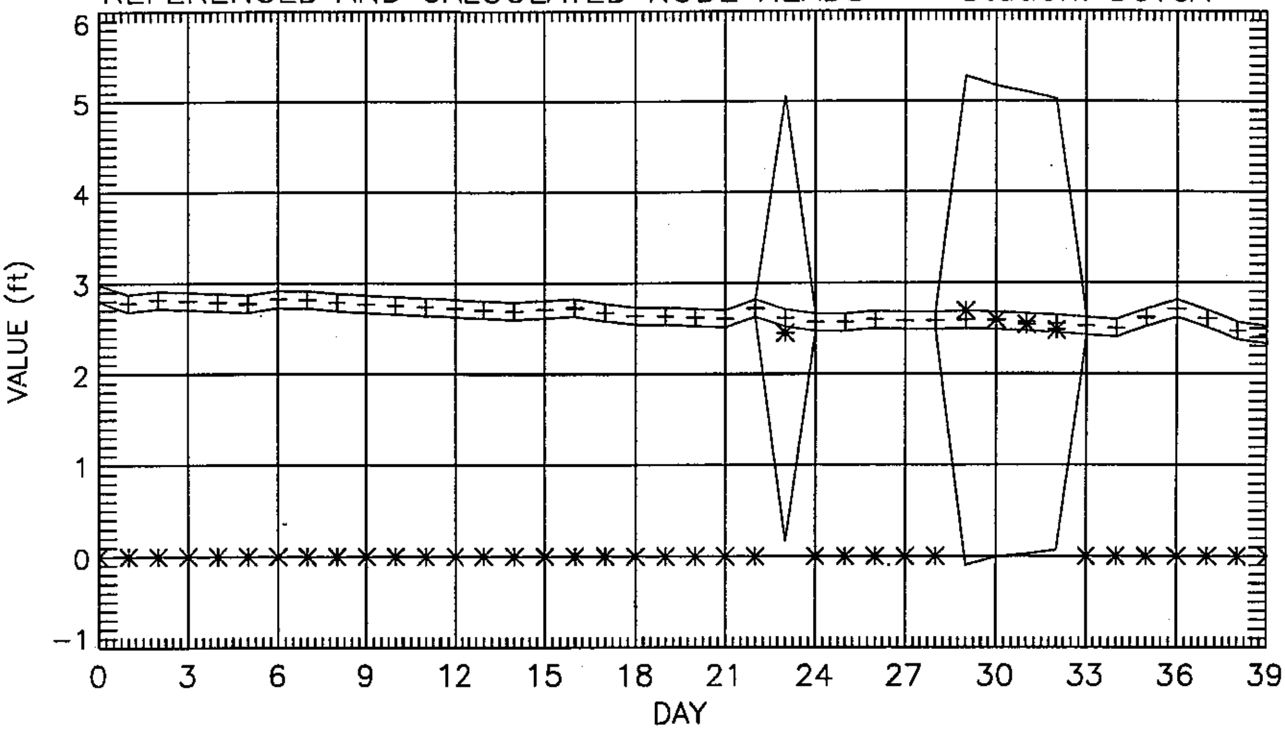


Layer 1 Row 56 Column 20 Hist.Stnd.Dev. 0.05 C.I.Year 0.13 C.I. DAY 0.00



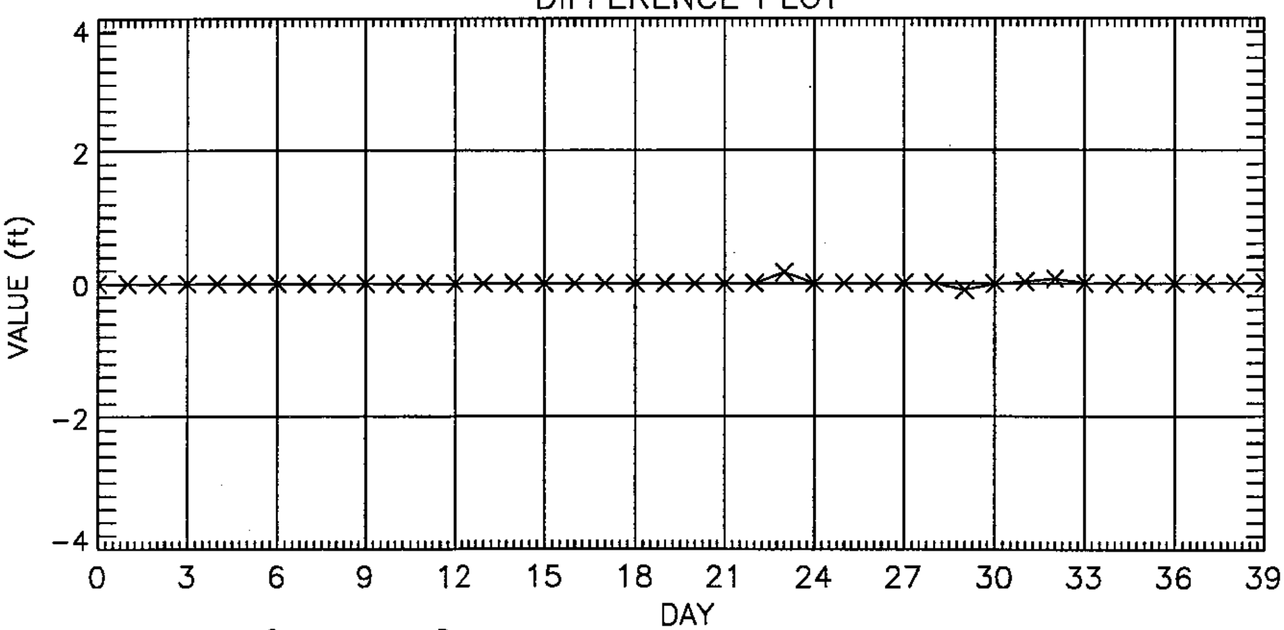
Extreme Errors [−0.3, 0.0] Average Absolute Error 0.03 Std.Error 0.03

REFERENCED AND CALCULATED NODE HEADS-- Station: DoveA

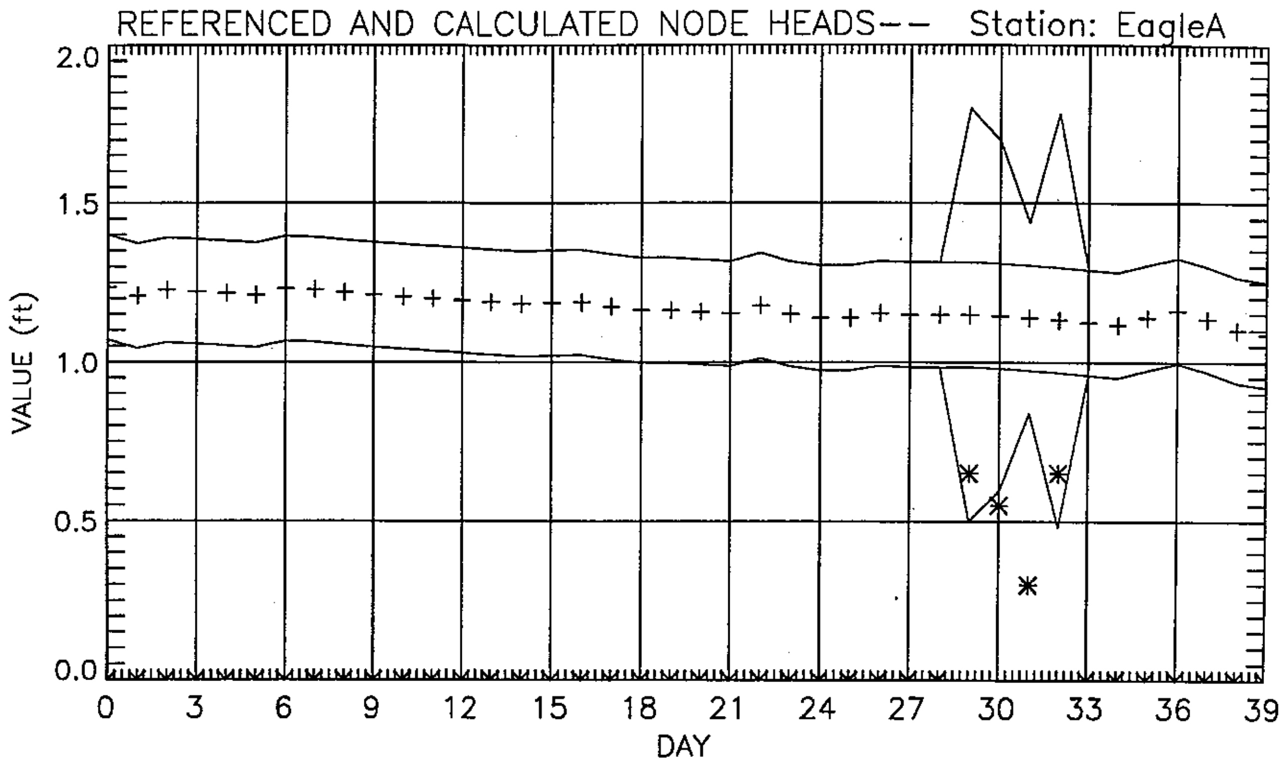


Layer 2 Row 56 Column 20 Hist.Stnd.Dev. 0.10 C.I.Year 0.05 C.I. DAY 0.00

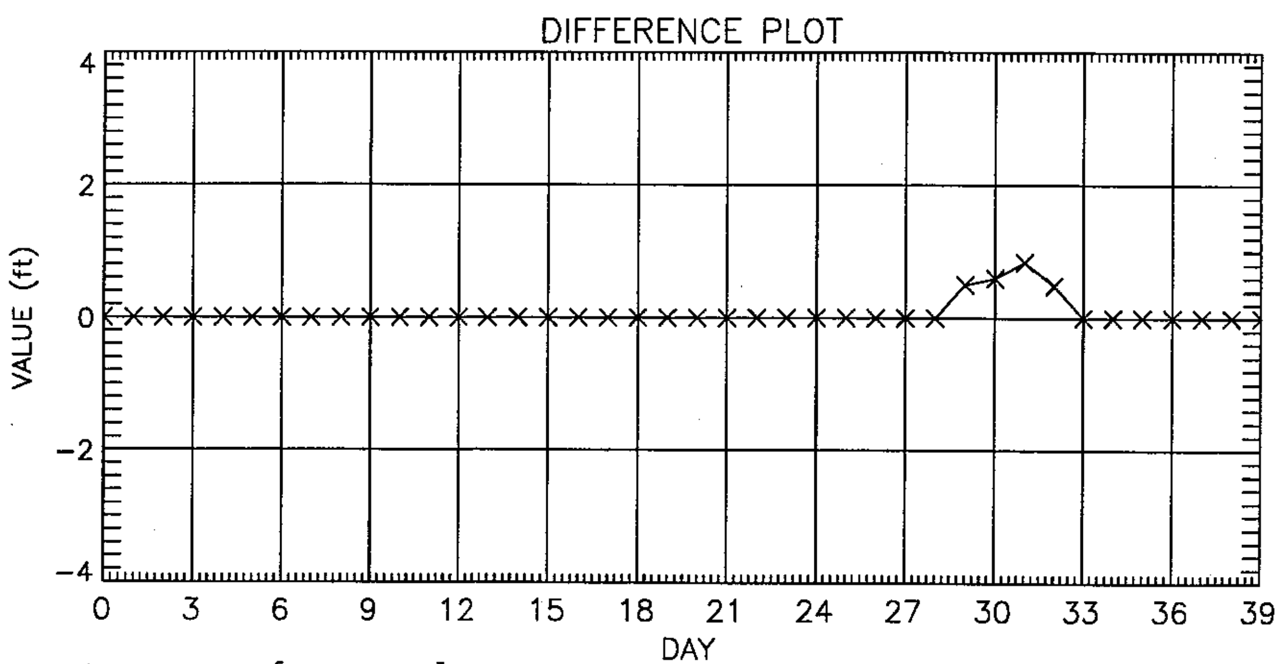
DIFFERENCE PLOT



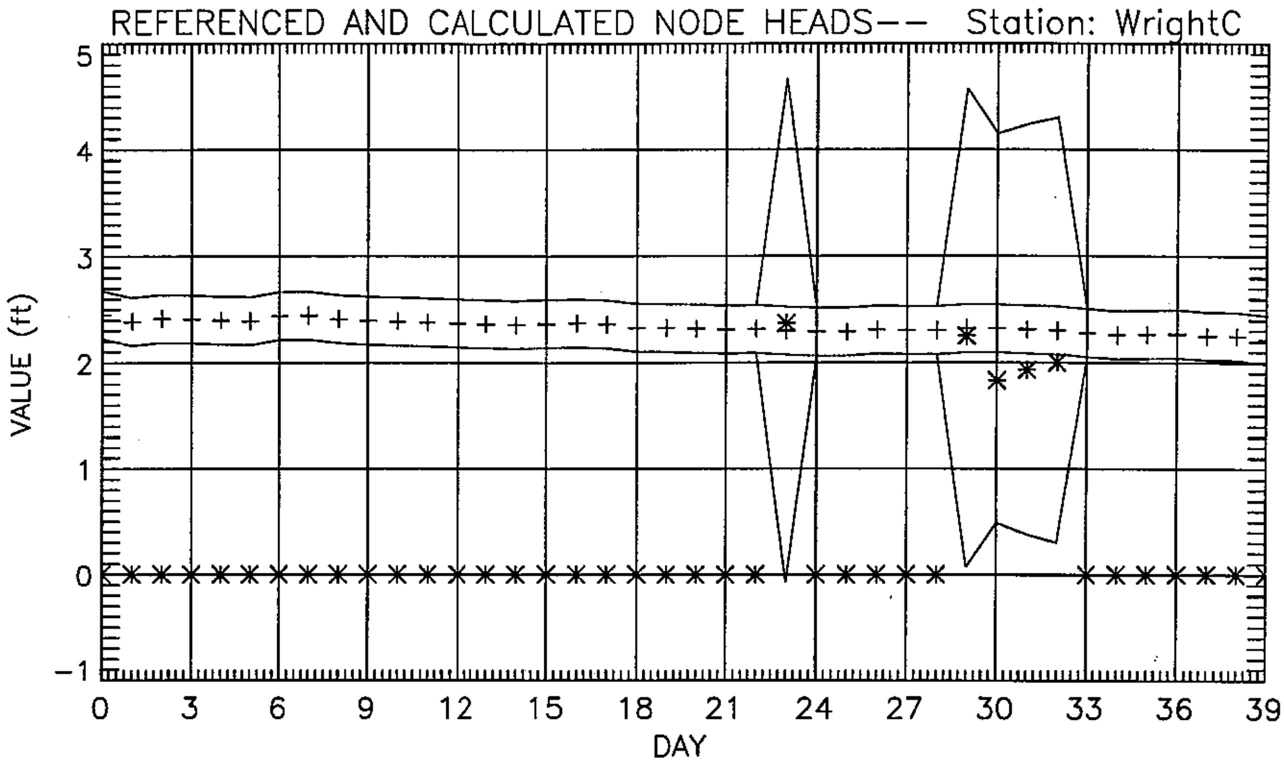
Extreme Errors [-0.1, 0.2] Average Absolute Error 0.01 Std.Error 0.10



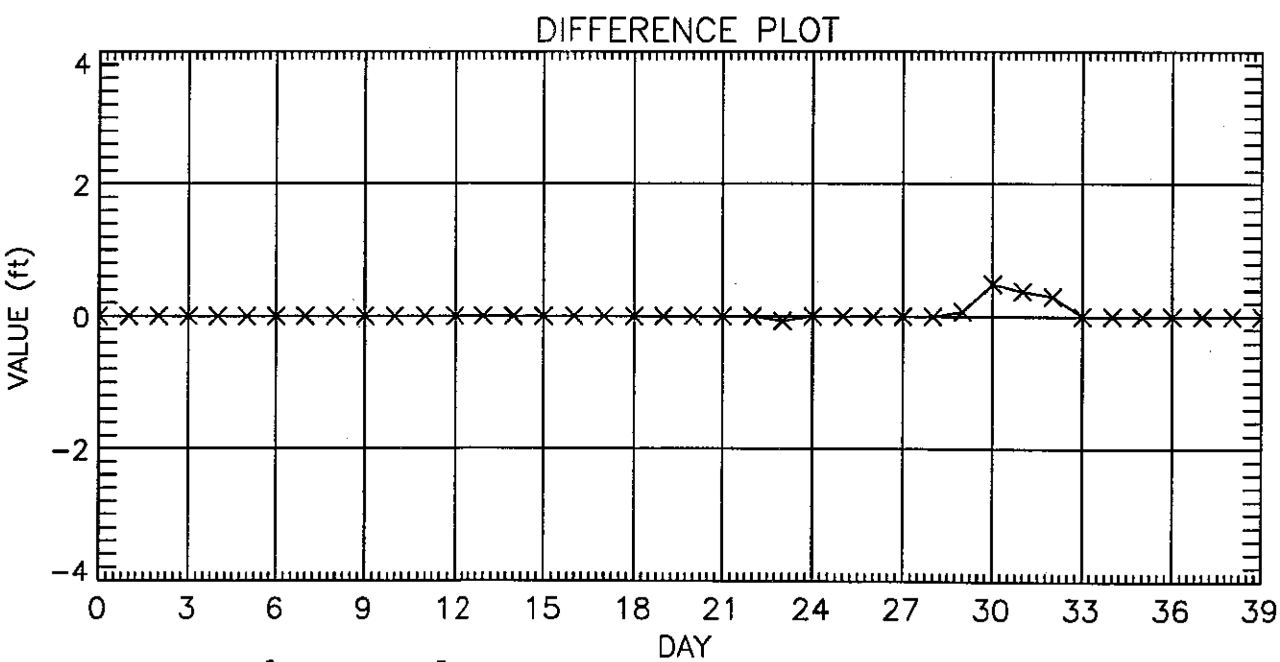
Layer 2 Row 56 Column 12 Hist.Stnd.Dev. 0.17 C.I.Year 0.10 C.I. DAY 0.05



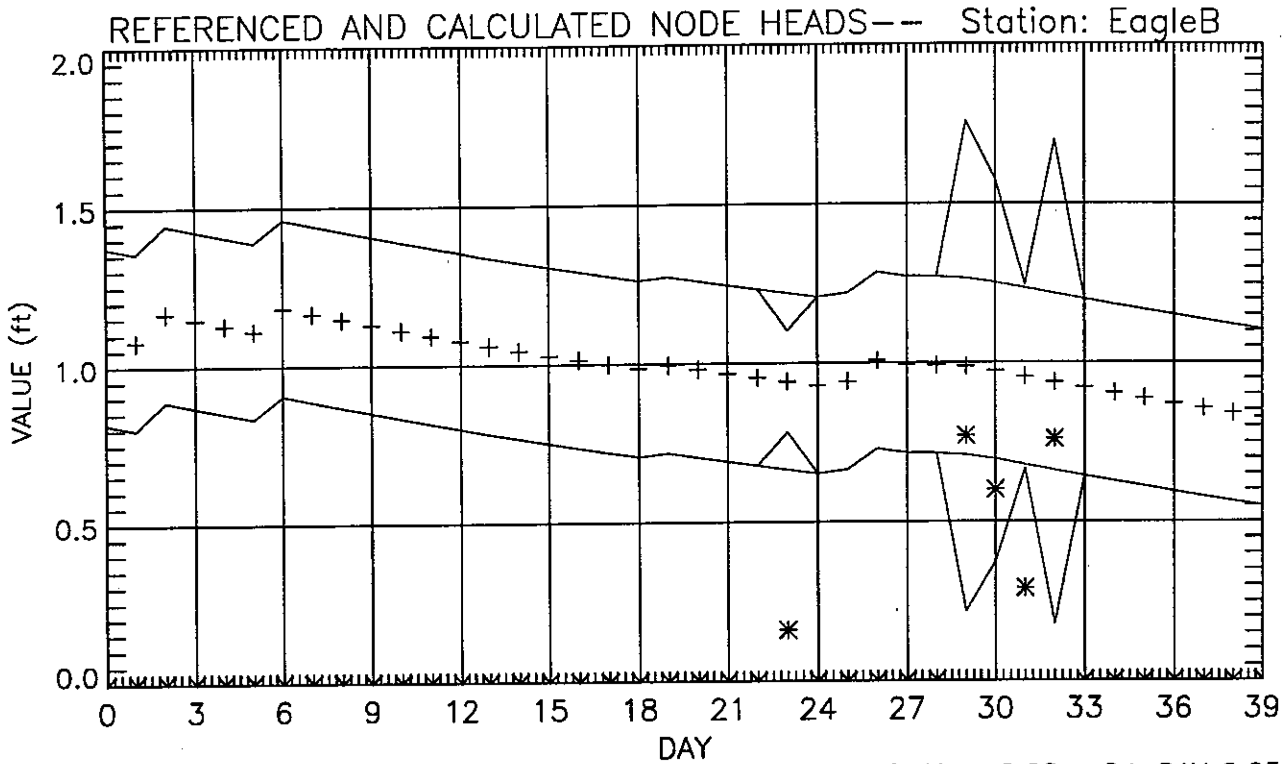
Extreme Errors [0.0, 0.8] Average Absolute Error 0.06 Std.Error 0.16



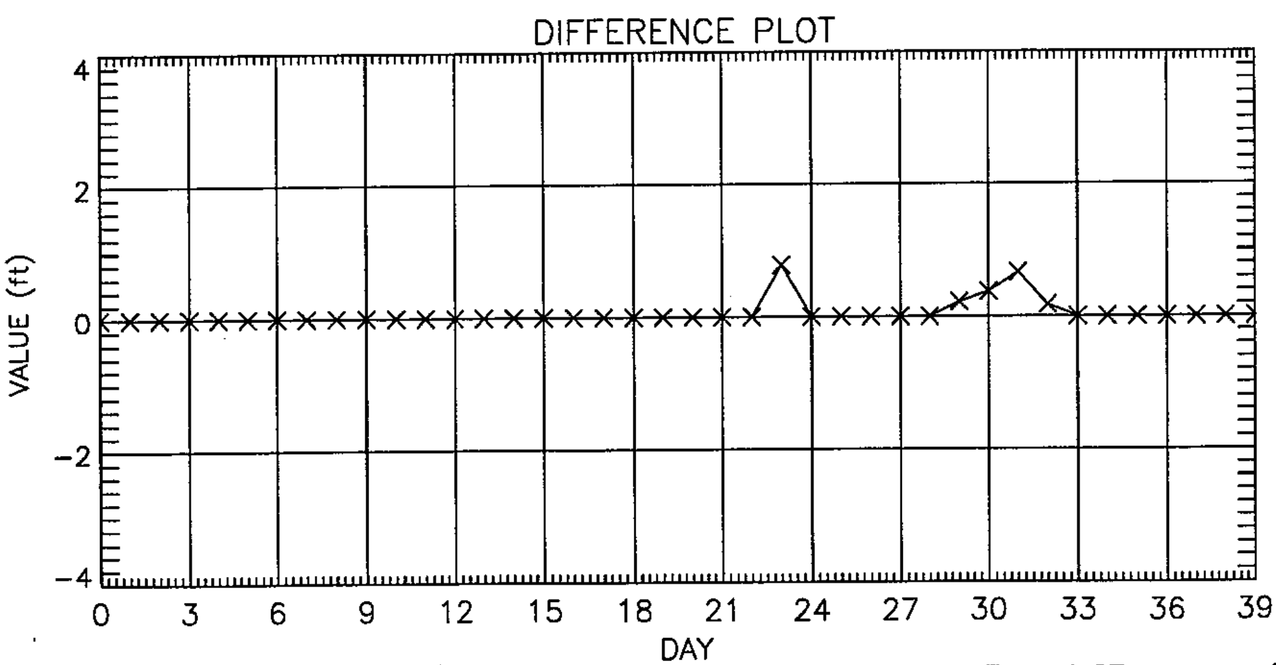
Layer 2 Row 75 Column 31 Hist.Stnd.Dev. 0.23 C.I.Year 0.08 C.I. DAY 0.00



Extreme Errors [−0.1, 0.5] Average Absolute Error 0.03 Std.Error 0.23



Layer 1 Row 56 Column 12 Hist.Stnd.Dev. 0.28 C.I.Year 0.08 C.I. DAY 0.05



Extreme Errors [0.0, 0.8] Average Absolute Error 0.06 Std.Error 0.27

ECOLE NATIONALE DU GENIE DE L'EAU
ET DE L'ENVIRONNEMENT DE
STRASBOURG

DISSERTATION

Author : DIAZ Sandrine	Promotion : Guadeloupe
Title : "Effects of groundwater withdrawals and surface water management systems on the groundwater levels within the isolated wetlands in Jensen Beach area, Florida."	Year of publication : 1996

Number of volumes : 1 Number of pages : 111 Number of references : 30

Location of internship :

South Florida Water Management District

3301 Gun Club Road, West Palm Beach, FLORIDA 33406

Abstract :

The purpose of this study is to assess the effects of groundwater withdrawals and surface water management systems on groundwater levels within the Jensen Beach wetlands.

The aquifer parameters were determined by conducting two pumping tests. The influence of lakes on groundwater levels within the wetlands was observed during the constant rate pumping test. The lakes drained the local groundwater which decreased the hydroperiod of the wetlands.

The aquifer parameters were used in a pre-existing groundwater model to simulate the field conditions during the constant rate pumping test. The calibration of the model has confirmed the influence of the lakes and has shown the weaknesses of the model. It is necessary to improve the conceptualisation of the model before doing future simulations.

Key word : Aquifer analysis, hydroperiod, pumping test, groundwater modeling.