#7

BARKER, OSHA & ANDERSON, INC.
ENGINEERS - PLANNERS
860 U.S. HIGHWAY ONE
NORTH PALM BEACH, FLORIDA 33408
305/626-4653

April 20, 1979

Honorable Mayor and City Council
City of Riviera Beach
600 West Blue Heron Boulevard
Riviera Beach, Florida 33404

Attention: Mr. Ronald A. Davis
City Manager

Subject: Transmittal and Submittal
"Report and Analyses
Wellfield Exploration Program
in the Turnpike Aquifer".
Project No. 78-1033

Gentlemen:

We submit and transmit to you herewith 10 copies of the above document, which supplements our "Engineering Plan and Feasibility Report, Waterworks Expansion and Improvements Program", December 1, 1978.

As stated in the report, it is our considered opinion that the "Turnpike Aquifer" located in the partially incorporated Reserve Annexation Area west of the City proper, provides a sufficient, reliable and economically the best raw water source to sustain the City's future needs.

In order to use this source, it is necessary that the City's current Water Use Permit (No. 50-00460-W) issued by the South Florida Water Management District (SFWMD) (expiring July 14, 1979), be amended and extended. In view of this, we have coordinated our exploratory work in the new aquifer with their Staff. We are also submitting copies of this Report directly to that agency on behalf of the City as of this date, and requesting a staff review of the total concept and material submitted, and a resulting recommendation to and action by the Board of Governors to permit the City to proceed with the very necessary Waterworks Expansion Program.

We will continue to work with the SFWMD staff to a conclusion of the matter, and advise you from time to time of our progress.

We wish to thank members of the City's Staff, and Mr. Jack Walden, Director of Utilities, and members of his department for their invaluable assistance and cooperation in carrying the Program this far.

Please advise us of any questions you may have or of further service we may be at this time.

Respectfully submitted,

BARKER, OSHA AND ANDERSON, INC.

James H. Angel

JAMES H. ANGEL
Project Engineer

JAH/thw

Enclosure

REPORT AND ANALYSES

WELLFIELD EXPLORATION PROGRAM
IN THE
TURNPIKE AQUIFER

FOR THE CITY OF
RIVIERA BEACH, FLORIDA

APRIL, 1979

PREPARED BY

BARKER, OSHA & ANDERSON, INC.
ENGINEERS-PLANNERS
NORTH PALM BEACH, FLORIDA

REPORT AND ANALYSES
WELLFIELD EXPLORATION PROGRAM
IN THE
TURNPIKE AQUIFER
FOR THE CITY OF
RIVIERA BEACH, FLORIDA

<u>MAYOR</u>

MR. BOBBIE E. BROOKS

<u>CITY COUNCIL</u>

Mr. Gary R. Nikolits, Chairman
Mr. Robert H. Dodd
Mr. Clem Guider
Mr. Cornelius Lawrence
Mr. Claude T. Tolbert, Jr.

City Manager	Mr. Ronald A. Davis
City Engineer	Ronald P. Glorsky
Director of Utilities	Jack Walden

Prepared By
Barker, Osha & Anderson, Inc.
Engineers-Planners

April 1979

ABSTRACT AND FOREWORD

It is necessary that the City of Riviera Beach supplement their present raw water supply for the public water system which the City operates, due to movement of saltwater toward the present wellfield, which is located in an aquifer of limited capacity. Groundwater continues to be the desirable source, and a hydrologic zone in the City's Reserve Annexation Area known as the "Turnpike Aquifer" appears to provide an additional and substantial source. Geologic and hydrologic data have been compiled and analyzed, and test and observation wells constructed and pumped to measure the hydraulic reactions of groundwater to stress (pumping) in this aquifer, all designed and correlated to evaluate its capacity to meet present and future water demands of the City, and justify it as the source to supply the currently planned expansion and improvement of the City's water treatment, storage and distribution system.

These explorations tend to establish the adequacy of the Turnpike Aquifer for the intended use, and this Report sets forth the findings and recommendations thus developed. If the Report appears to the lay reader to be burdened with technical discussion, please bear in mind that this information must be presented to and reviewed by the State agencies who regulate withdrawal and use of groundwater, and this Report is intended to serve that purpose as well.

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INTRODUCTION

For many years, the City of Riviera Beach has accepted and met the responsibility of providing its residents and seasonal population surges with an ample supply of properly treated and otherwise excellent quality water for drinking and other general public and domestic use. The source of this water has been a wellfield of some 20 wells located between about 1 and 2 miles west of the tidal waters of the Atlantic Ocean, in a hydrologic region locally known as the "Sandy Ridge Aquifer".

This aquifer is naturally recharged principally by rainfall occurring in the area between Lake Worth on the East, and the Earman River (Canal C-17) on the West. During intense rainfall periods, typical of the regional climitological pattern, most of this rainfall runoff is diverted through an excellent system of drainage inlets, conduits and canals, back to the ocean to the East. Over the years, with more development and improved drainage systems, less and less water from the seasonal rainfall has been returned to the Sandy Ridge Aquifer through natural percolation and absorption, and its annual average level has receded, aggravated by increased wellfield pumpage to satisfy the demands of a growing population.

During the early years of metropolitan growth, the Sandy Ridge Aquifer was able to sustain itself through the annual dry winter season by means of its capability to "store" the surges of the wet summer season, characteristic of the regional climate. Eventually, the critical stage developed, when pumpage from the aquifer during the dry season (when irrigation demands are high) brought the formerly high water tables in the wellfield down to and sometimes statically below sea level. At this point, seawater, which is heavier than fresh water, began to penetrate westward into the lower levels of the Sandy Ridge Aquifer, from which most of the present raw water supply is drawn.

There is substantial evidence that salt water (seawater) intrusion induced in this manner has permeated the major water-bearing zone as far west as Broadway (U.S. Route 1), raising the chloride content of the water to unacceptable levels for human consumption. It may be assumed that this intrusion will progress westward to the present wellfield in time under present demands, and be accelerated by the demands of population growth in the Service Area of the system, unless pumpage from the aquifer is reduced.

It thus becomes evident that the City must find a new source of raw water supply which has the capacity, not only of satisfying predictable growth demands, but of replacing the demands which have been placed on the Sandy Ridge Aquifer in excess of its sustained capacity.

In recent years, severe growth rates have developed in the Singer Island area, which now approaches predictable saturation, both as to population and resultant water demand. Also, the present City proper is some 80% fully developed, and an upper limit can be placed on ultimate water demands in that area with fair accuracy.

Less predictable, but perhaps more subject to future control, is the growth and future water demands of the Riviera Beach "Reserve Annexation Area", which, it is assumed, will depend on the City System for its utility services in the foreseeable future. Significant portions of this area have been annexed into the corporate limits in recent years, and this pattern of political expansion may be expected to continue until the entire area falls within the jurisdiction of the City, at least with respect to utility services.

All studies to date indicate ground water to be the most reliable and economically feasible source of raw water presently available to the City. Previous explorations have also indicated the presence of an excellent shallow aquifer in the Riviera Beach zone of the area between Interstate 95 and Florida's Turnpike, which thus becomes the logical source of raw water to alleviate the presently excessive demands on the Sandy Ridge Aquifer, and provide for the demands of future population growth in the Service Area.

The purpose of this project is to make a definitive evaluation of the capability of this aquifer as to its sustained capacity, and to develop parameters and guidelines for the design of an efficient and cost-effective wellfield in this aquifer, and a water withdrawal and aquifer resource management plan for the total system.

This constitutes the objectives to which this Report is addressed.

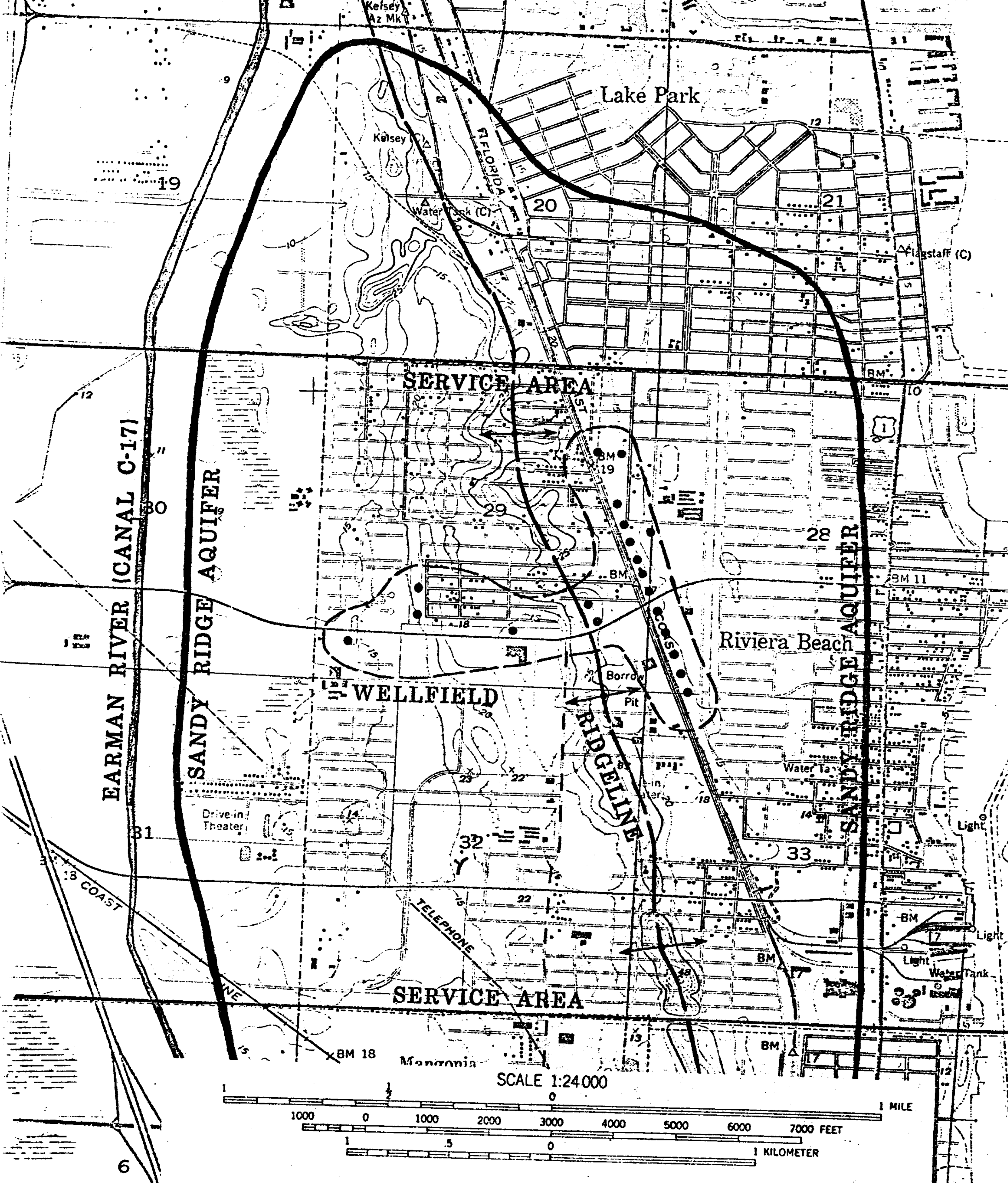


Fig. 1. Topography vicinity of Sandy Ridge Aquifer and present wellfield.

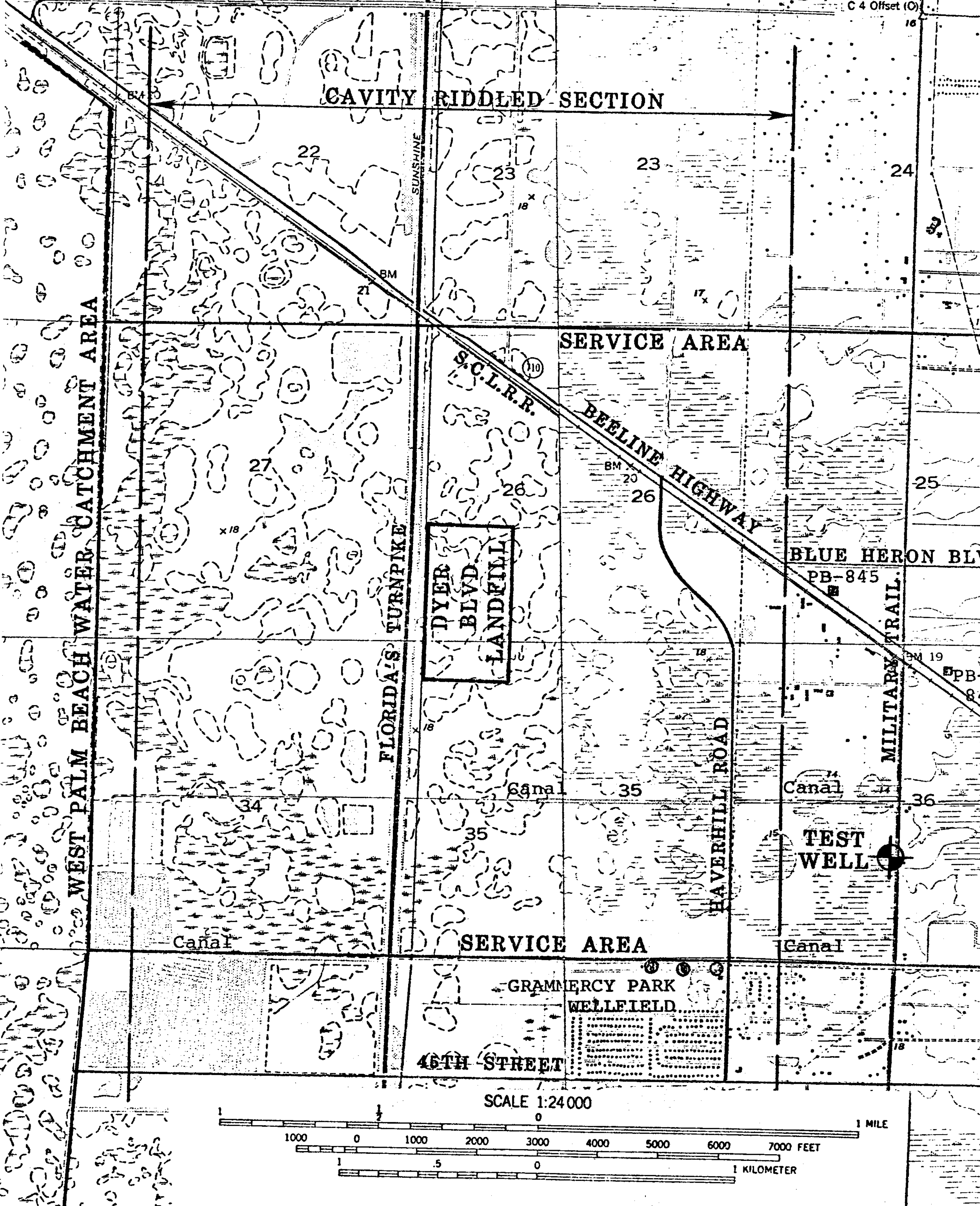


Fig. 2 . Topography near Test Well.

WELL	RADIUS	REF. (1) ELEV.	SCREENED DEPTH	COORDINATES E-W N-S
TW	0.0	18.76 (2)	95'-135'	70.75W-225.75N
TH-1	314.5'	17.66	144'-149'	169.42E- 22.44N
TH-2	606.8'	15.35	144'-156'	501.46E- 23.10N
OW-1	40.0'	15.52	30'- 35'	70.75W-185.75N
OW-2	120.7'	16.03	30'- 35'	71.67W-105.00N
OW-3	240.1'	15.55	34'- 39'	71.04W- 14.37S

(1) TOP OF 2" COUPLING, M.S.L. DATUM

(2) TOP OF 1½" PLATE

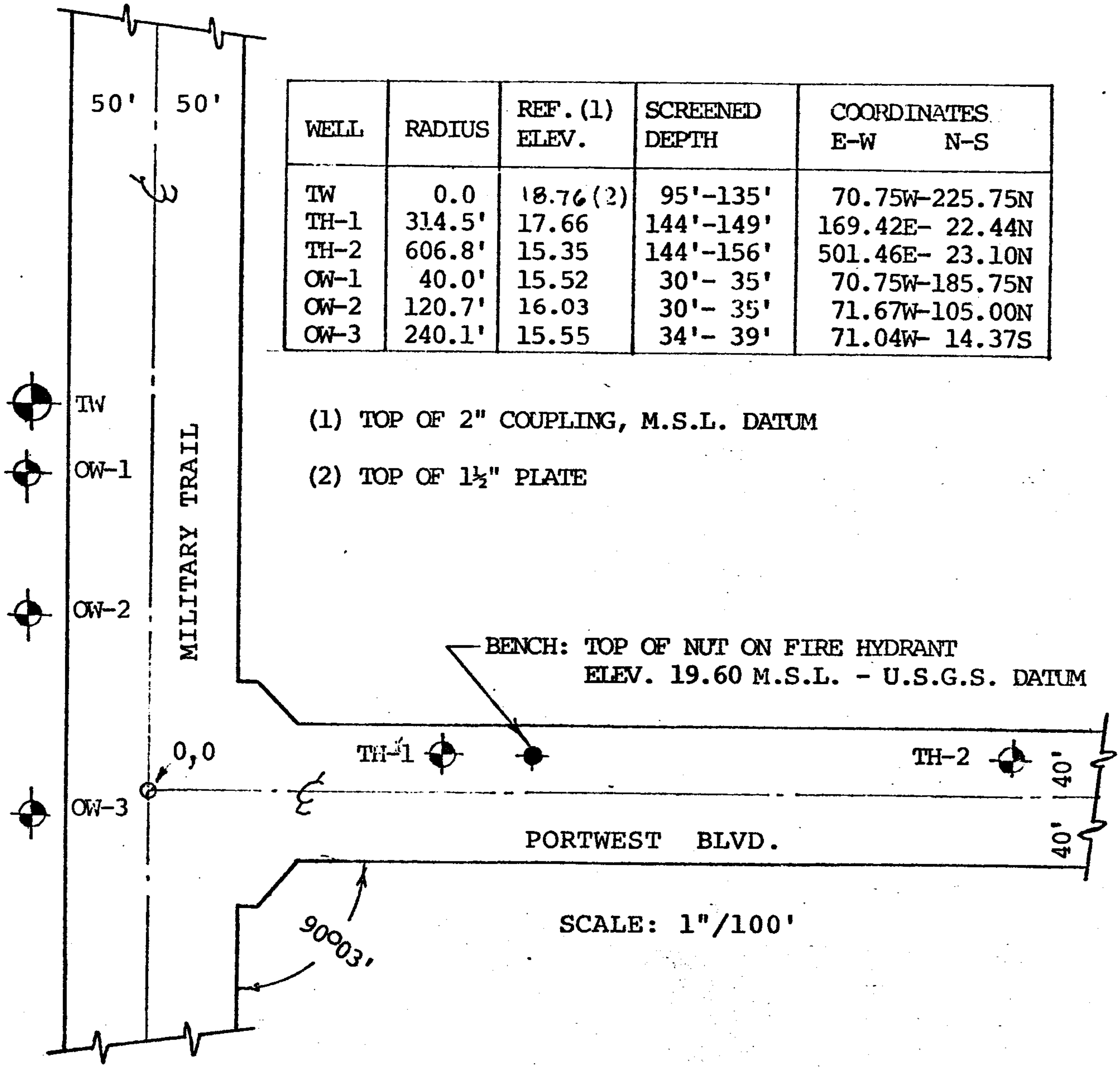


FIG. 3. SKETCH SHOWING GEOMETRICS OF TEST SITE

GEOLITHIC SUMMARY OF TEST BORINGS

GENERAL

Three test holes were drilled at the test site located as shown in Fig. 3 . The prime purpose of these borings was to provide a geologic profile of the formations comprising the aquifer in the vicinity of the Test Well, and thus aid in its proper design and founding. A secondary purpose was to provide holes for two deep observation wells (at Test Holes 1 and 2) which would provide drawdown data during subsequent aquifer stress testing.

Each hole was drilled to a depth of 200 feet by the rotary method using a 6" diameter roller bit with direct circulation of Bentonite drilling fluid. Through unconsolidated formations, vertical speed of the bit was limited to approximately two feet per minute, and through harder formations by the weight of the drilling column only, but not over two feet per minute.

Formation samples were washed on a 60-mesh (.017") screen, and four composite sets were prepared for each 20-ft. increment of depth. These samples were distributed as follows:

1. Florida State Bureau of Geology.

2. South Florida Water Management District.

3. City of Riviera Beach

4. Consultants

Additional samples from the 100-140 ft. zone of the test hole at the Test Well were sieve-analyzed to assist in the design and selection of the gravel packing material used in the screened zone of the Test Well.

Additional samples were inspected during the drilling phase. These were rapidly rinsed on a coarse (.080") sieve to clear them of drilling fluid. In this manner, it was possible to retain cuttings (lumps) of fine cohesive material which otherwise dissolved and passed through the finer screen without detection.

Comparision of the geolithic logs recorded during the drilling operations indicates that four very distinct zones exist within the depth of the test holes, and are quite consistent in depth and texture from hole to hole. These zones are as indicated on the generalized profile shown in Fig. 4 . An analysis and condensation of the geolithic logs shows these following characteristics of the materials and formations found in each of the zones of the aquifer.

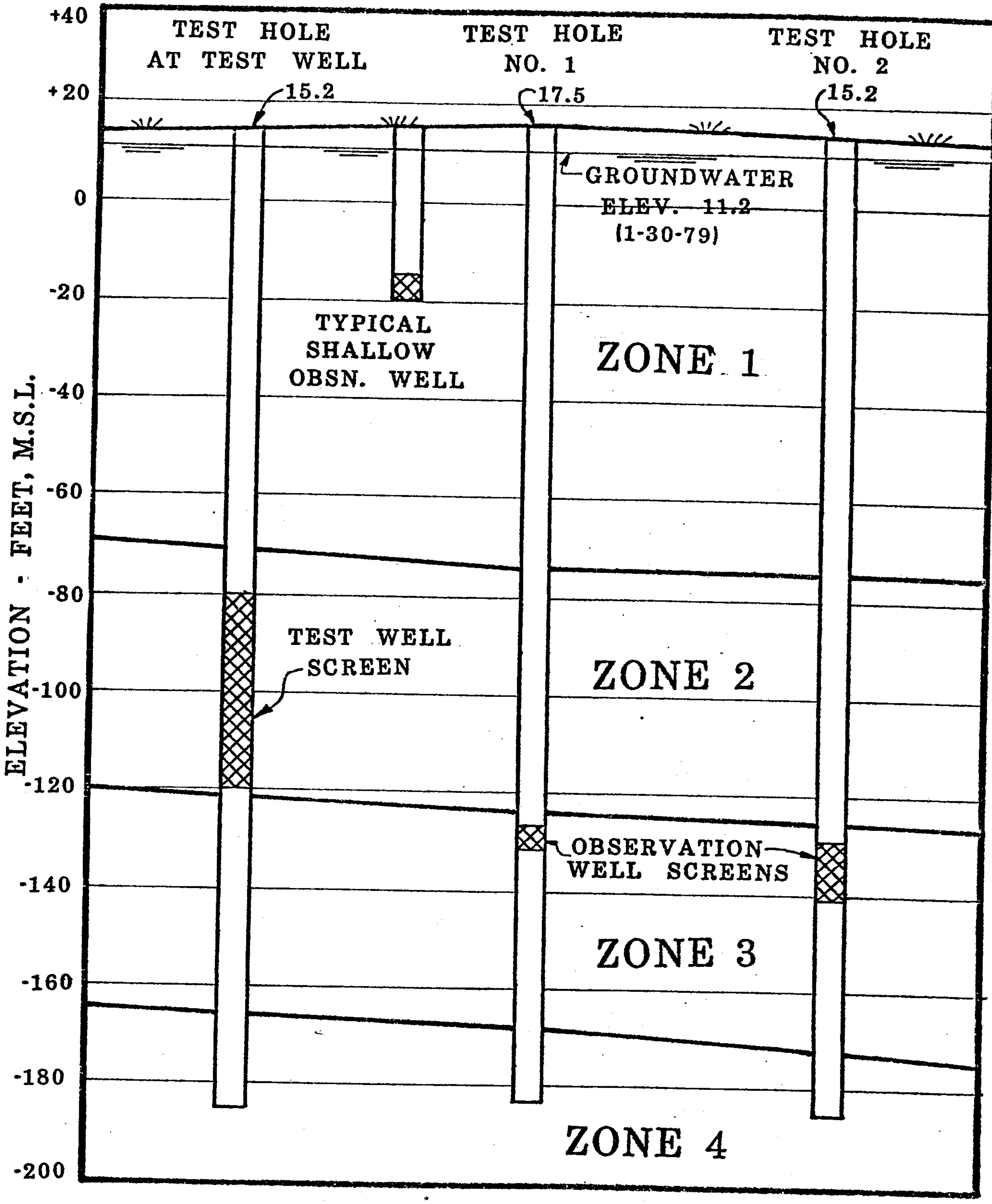


FIGURE 4 . GENERALIZED PROFILE OF GEOLOGIC FORMATIONS AND WELL ARRANGEMENT AT THE TEST SITE.

ZONE 1: Surface (15'-17' M.S.L.) to about 90 feet depth. This zone is composed mainly of fine sand and small (coquina) shell fragments. A temporary driven surface casing prevented accurate sampling of the top 20 feet. However, some evidence of red saturated cohesive material appeared in the gleanings from the casing in all three holes, indicating the presence of a layer of clayey sand in the top 20 feet (probably around 8 to 12 feet deep), locally referred as "hardpan". Shell content and size appeared to increase moderately with depth, as did effective size of sand particles, which varied between fine/medium-fine. Sand color varied from tannish to grayish with depth. A few random and inconsistent thin lenses of lightly cemented shell/sand were encountered below about 40 feet. Also, small amounts of saturated cohesive lumps were brought up from random levels, usually immediately above a slightly consolidated level. The color of these cohesives varied from white to gray.

Sand
Tan → Gray
with depth

ZONE 2: About 90 to 140 feet depth.

This zone consists of about 8 layers of hard material about 2 to 3 feet in thickness vary~~ing~~ from hard porous cemented sand/shell to fairly dense sand~~stone~~. These layers are generally separated by cemented sand/shell, some 4 to 5 feet in thickness. Mostly white, but occasionally gray, cohesives were usually returned from the levels just above the more firmly consolidated strata.

The stratification in this zone appears most pronounced and uniform from layer to layer at the Test Well hole, noticeably diminishing easterly through Test Holes Nos. 1 and 2. The layers correlate well between the Test Well hole and Test Hole No. 1, but poorly with Test Hole No. 2, where fewer layers were detected, and where more cementing was found in the softer layers. However, Zone 2 is uniformly marked by dense sandstone lenses at the top and bottom at all three holes.

An inspection of the settled cuttings from this zone at the Test Well hole indicated about 30% sandstone, 60% cemented shell and sand, 10% loose shell and sand.

Zone 3: About 140 to 180 feet depth.

Zone 3 is similar to Zone 1 in composition, but contains much coarser sand and shell, particularly in the upper 20 feet. The top of Zone 3 is marked by a layer of coarse loose shell fragments embedded in medium sand, about three or four feet in thickness. Below 160 feet, a few thin lenses of sandstone were encountered, usually overlaid by finer sands and white to gray cohesives. In general, the proportion of finer sands and cohesives increases with depth. Some coquina shell is present, but severely abraded fragments of larger shell predominate. The bottom of Zone 3 is marked by a hard layer of cemented shell and sandstone, overlaid by very fine sand and cohesives.

ZONE 4: Below about 180 feet depth.

All three test holes penetrated into Zone 4, from about 20 feet at the Test Well hole to about 12 feet at Test Hole No. 2. Two to three feet of loose shell/sand underlying a relatively thick hard lens forms the top of the zone, and is quite permeable, as evidenced by a moderate drilling fluid loss at two holes, and a complete loss of return at Test Hole No. 1 at 186 feet depth. Below this shell layer and to 200 feet depth, the formation consists of small shell fragments, very fine sand, and abundant amounts of cohesive material which combine into a relatively hard mass and cut similar to cemented material. However, very little cementing was noted in sampling this zone, which, it is believed, forms the effective bottom of the aquifer.

ELECTRIC AND GAMMA RAY LOGGING

Upon completion of drilling each test hole, the formation was logged for electric self-potential and resistivity, as well as gamma ray radiation. Logging services and analyses were provided by the firm of Geraghty & Miller, Inc. consulting Hydrologists, of West Palm Beach. Charts of these logs are shown in Figs. 5 a, 5 b, and 5 c. These logs and analyses provide valuable information when coupled with the geolithic (driller's) log and examination of the formation samples.

Negative shifts in the self-potential charts (very pronounced at Test Hole No. 1) correlates well with the increase in clayey material found in the zone below about 170 feet depth, as well as lesser but significant amounts at other various elevations.

Increases in resistivity generally between about 90 and 140 feet depth most likely indicate more free water as would be expected in the coarser materials found in this level during drilling.

Sharp increases in gamma radiation between about 70 and 180 feet are probably associated with minor amounts of phosphate materials in the cohesives usually found overlying the harder strata. Both resistivity and gamma radiation decrease below 180 to 185 feet depth, the former probably due to more clays and less free water, and the latter to less phosphatic material.

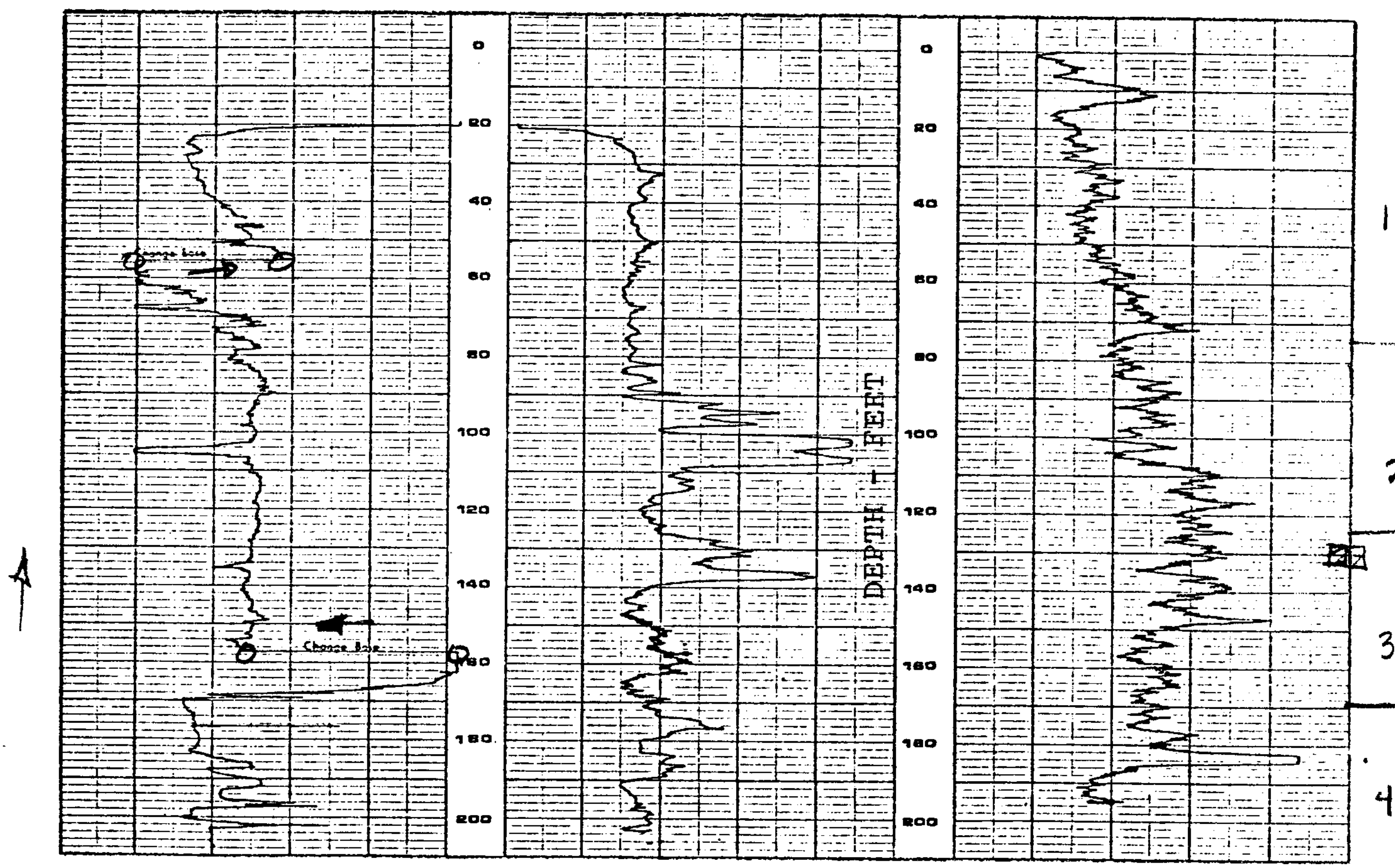
Due to the temporary surface casing, self potential and resistivity could not be logged in the top 20 feet. Gamma radiation is not affected by the casing, however, and the sharp "kicks" at the 5 to 10 feet levels are probably associated with the "hardpan" material found in the driven casings.

Not only do these logs correlate well with the geolithic logs, but also within themselves and from hole to hole. The resistivity log bears out the difference noted earlier in the lower part of Zone 2 at Test Hole No. 2, as compared to uniformity of this zone at the Test Well and Test Hole No. 1. The greatest consistency is evidenced in the uniformity of the gamma ray logs marking the elevation of the phosphatic deposits from hole to hole, particularly the bed found at the bottom of Zone 3. It appears possible that these markers could be used to trace formation strata through other holes drilled in the future in the general area.

GERAGHTY & MILLER, INC. GEOPHYSICAL BOREHOLE LOG

TEST HOLE NO. 1

Logged Jan. 16, 1979
Ground Elevation: 17.4 ft. M.S.L.



SELF-POTENTIAL

SCALE: 50 mv

RESISTIVITY

SCALE: 25 ohms

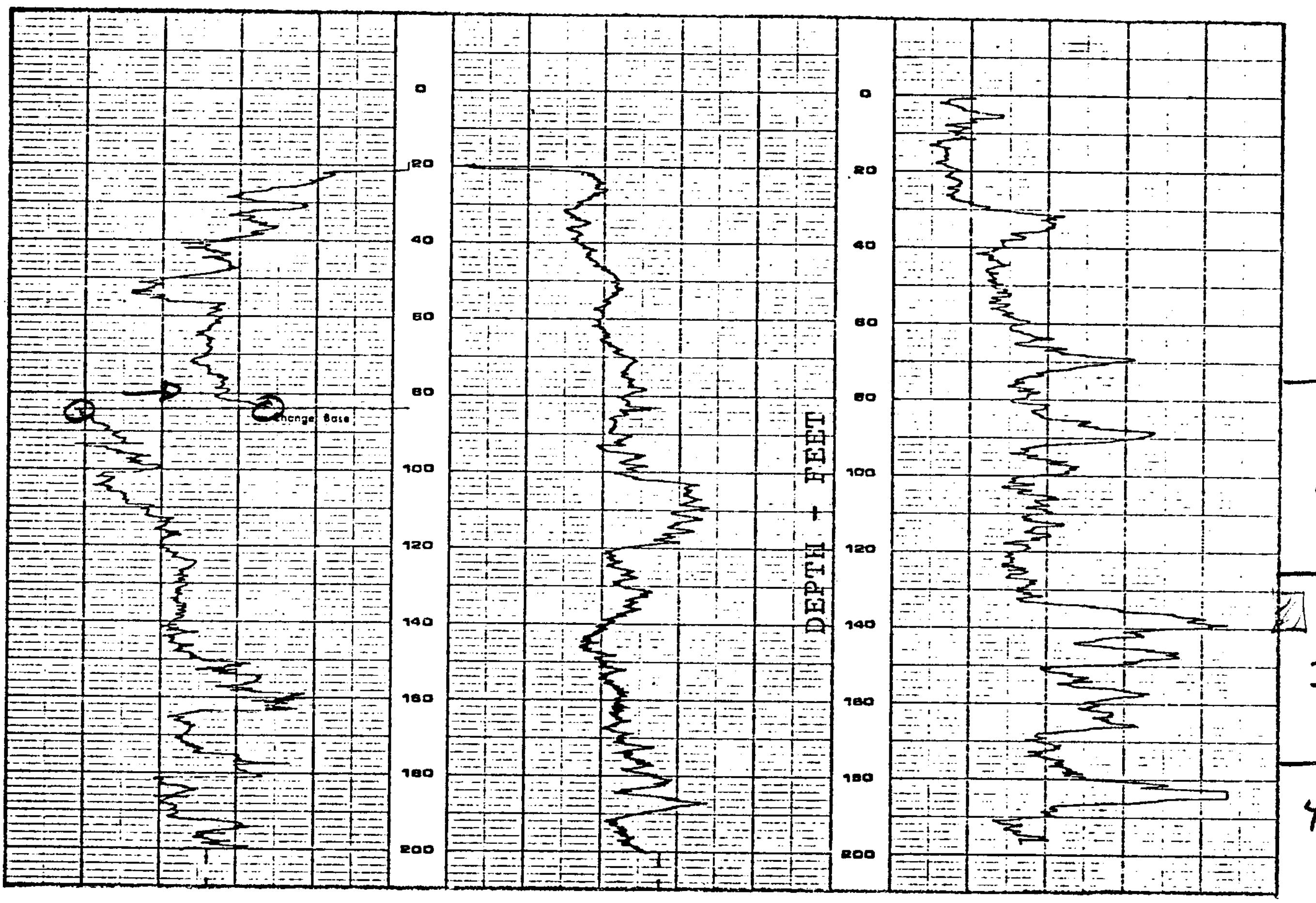
GAMMA RAY

SCALE: 0.5 TC
0.01 MR/H
24. Ft/Min

FIG. 5a. ELECTRIC AND GAMMA RAY LOGS OF TEST HOLE NO. 1.

TEST HOLE NO. 2

Logged Jan. 10, 1979
Ground Elevation: 15.2 ft. M.S.L.



SELF-POTENTIAL
SCALE: 20 mv

RESISTIVITY
SCALE: 25 ohms

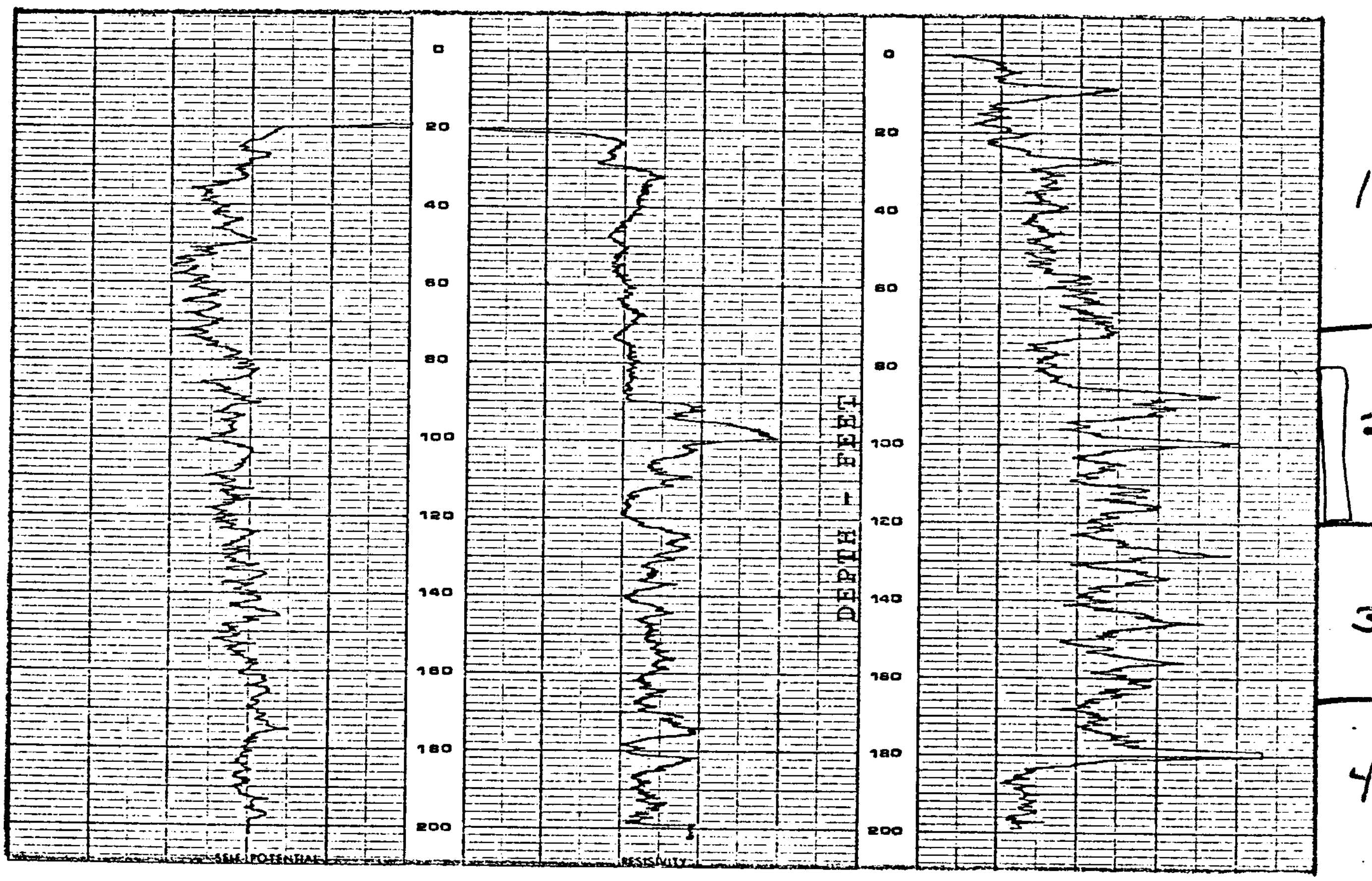
GAMMA RAY
SCALE: 0.5 TC
0.01 MR/H
28. Ft/Min

FIG. 5b. ELECTRIC AND GAMMA RAY LOGS OF TEST HOLE NO. 2.

GERAGHTY & MILLER, INC. GEOPHYSICAL BOREHOLE LOG

TEST HOLE AT TEST WELL

Logged Jan. 23, 1979
Ground Elevation: 15.2 ft. M.S.L.



SELF-POTENTIAL

SCALE: 100 mv

RESISTIVITY

SCALE: 25 ohms

GAMMA RAY

SCALE: 0.5 TC
0.01 MR/H
24. Ft/Min

FIG. 5c. ELECTRIC AND GAMMA RAY LOGS OF TEST HOLE AT TEST WELL.

COMPARISON WITH EXISTING GEOLOGIC DATA

Reference is made to "Ground Water Resources of the Riviera Beach Area", published by U.S. Geological Survey in September 1977 (Water Resources Investigations 77-47) by L.F. Land, which reports on what is considered the most comprehensive and detailed geological and hydrological exploration and analysis of the fresh water aquifer in this area presently available.

Land profiled a geologic section along the East-West line of Blue Heron Boulevard about one mile north of the present test site, extending from Lake Worth westward into the West Palm Beach Water Catchment Area, based on 12 test borings, some extending down to 400 feet, and all well into the confining formations at the bottom of the fresh water aquifer. Some of Land's test holes were at the approximate latitude and others at the same general longitude as the test site, but none closer than about 3/4-mile away. Land's map and geologic section are shown in Figs. 6a and 6b, respectively.

Land demonstrated that greater variations exist in the level and texture of the aquifer formations and performance along an east-west axis than in a north-south direction, where a fairly high degree of uniformity was found to exist. It would therefore be expected that the vertical section at the Test Site would closely resemble his section in the vicinity of Military Trail. With minor exception, this was found to be the case.

The main similarities lie in what Land shows as Beds 1, 2, 3 and part of 4 (Fig. 6 b) lying above about 90 feet depth, which are lumped into Zone 1 in the Test Site profile (Fig. 4). Zone 2 appears to be well defined and distinctly different from the balance of the aquifer, and probably extends more westerly than easterly from the Test Site, since it seemed to "fade" at Test Hole No. 2 between 120 and 140 feet depth.

Zone 2 very likely contains the "cavity riddled section" underlying Florida's Turnpike shown in Fig. 6 b. This particular zone of very high transmissivity has been reported by others to lie at this general longitude and depth and to extend as far north as the Juno Area and south to the Boynton Area. Unofficially, it has been dubbed the "Turnpike Aquifer".

Zone 3 (Fig. 4) comprises the balance of Bed 4 (Fig. 6 b), which agree as to formation materials and effective depth of the bottom of the aquifer in this area, which is about -170 ft. M.S.L. Land reports this bed as "cemented". Test Site drilling indicates mostly loose but stable sand and shell, with some cemented lenses. The presence of marl or clay (cohesive) layers in Bed No. 4 also agrees with the Test Site findings.

Although not conclusive, there is some evidence that the "sandy clay" stratum near the surface west of Military Trail (Fig. 6b) may extend through the Test Site, as indicated by the gamma ray logs and color of material gleaned from the casings. Also noticeable in both instances is a general eastward "tilt" of the strata of about .005 (0.5% gradient).

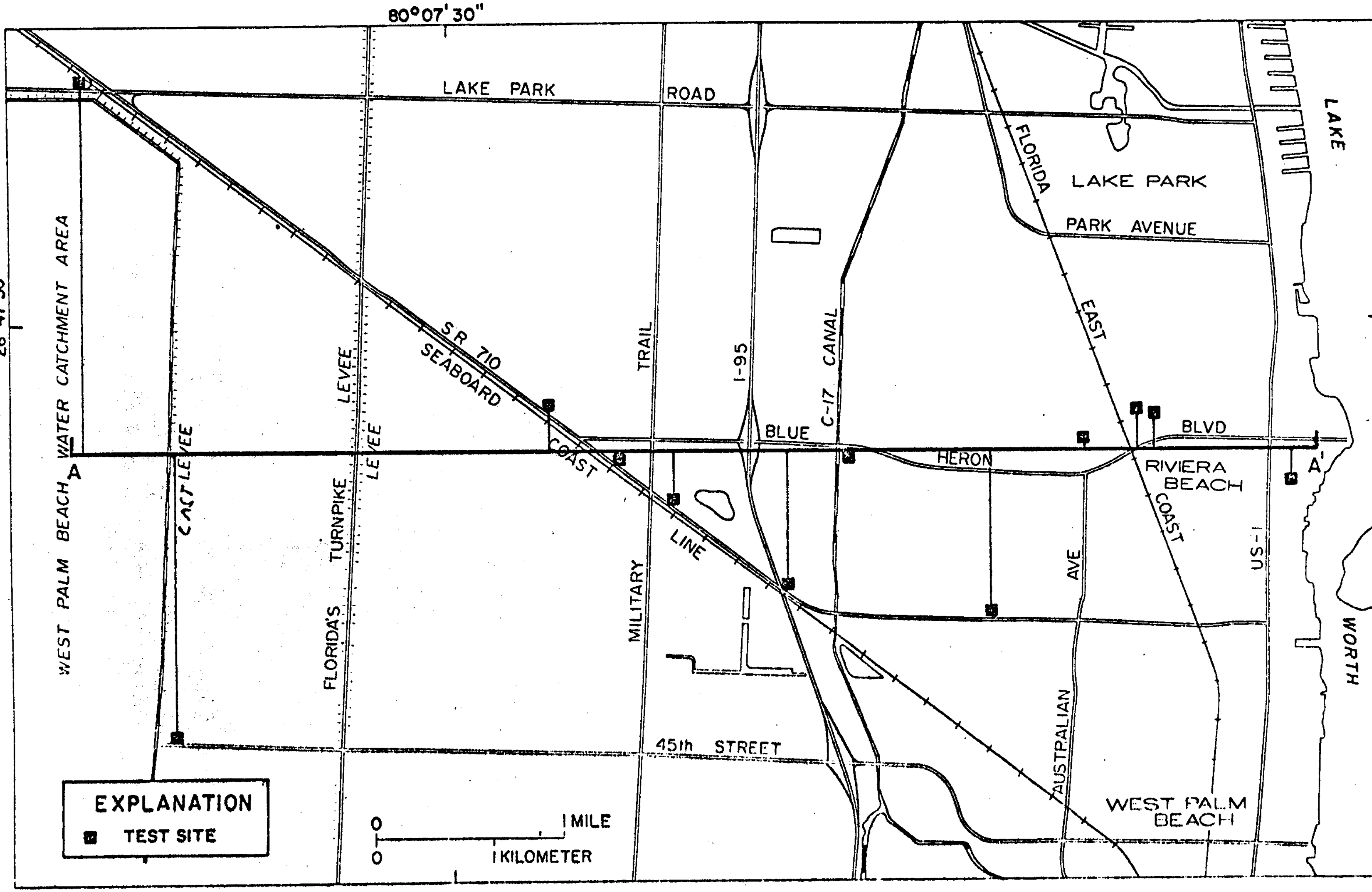


Fig. 6 a. U.S.G.S. (LAND-1977) Test drilling locations and geologic section A-A'.

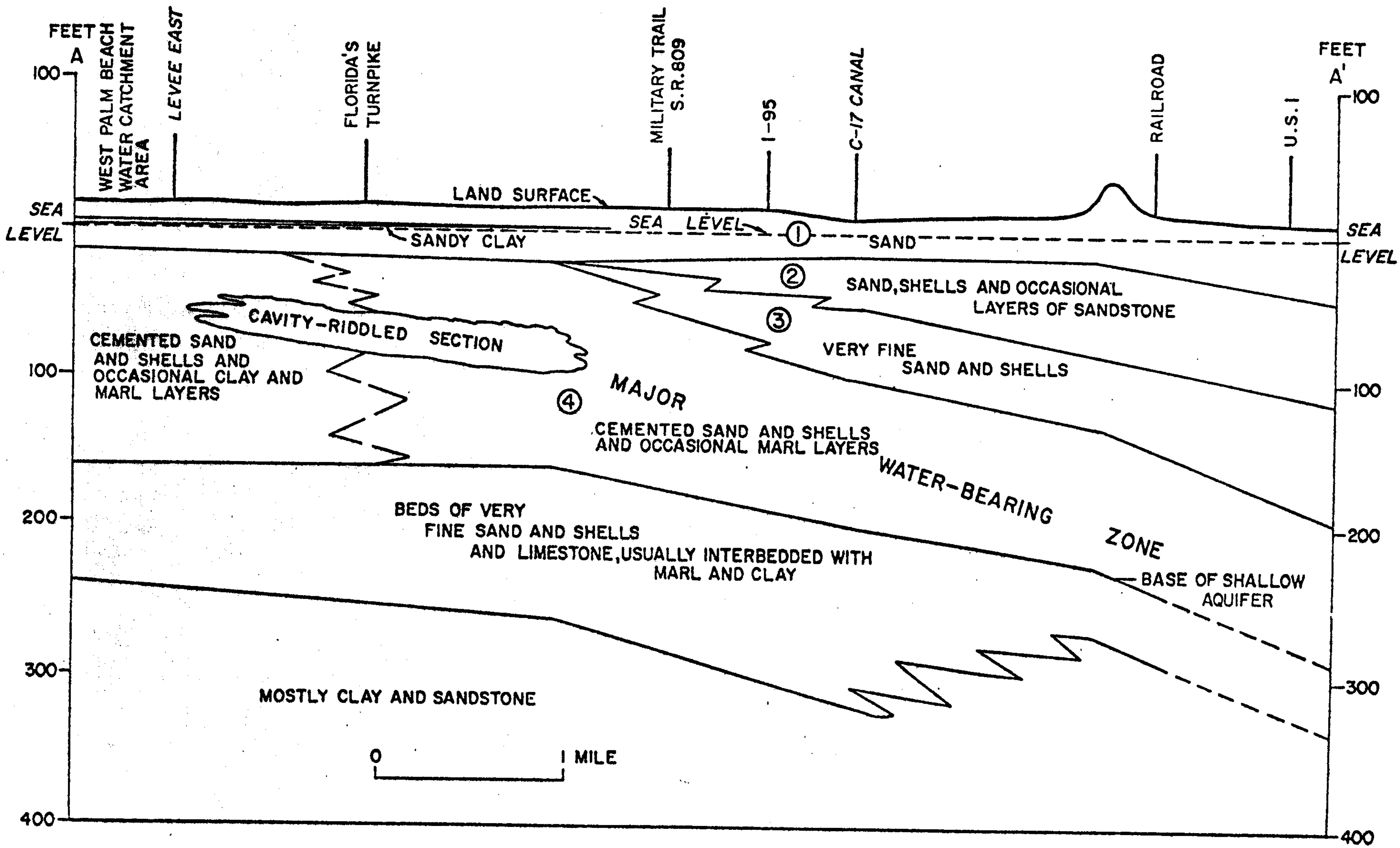


Fig. 6 b. U.S.G.S. (LAND-1977) Geologic section of the shallow aquifer along line A-A' in Fig. 6a , showing bed identification numbers.

HYDRAULIC EVALUATION OF AQUIFER ZONES

ZONE 1: Although some variations were noted in the character of materials found in this portion of the local aquifer, there appears to be much fewer and less distinct laminations (and thus probably less vertical confinement) here than in the deeper zones. Recharge and gravitational discharge induce some northeastward movement of water, particularly when the water table is high. Most of this natural movement in the total aquifer probably occurs in the top half of this zone. The coefficient of storage at levels near the groundwater surface is estimated to be on the order of 0.2 and average at least .05 for the zone. This zone would be expected to react locally as a homogenous unit of the aquifer under stress, and it serves as the sole source of recharge of the lower zones.

ZONE 2: This distinctly laminated zone contains layers of coarse loose material and very pervious cemented materials of apparent higher hydraulic conductivity than other levels of the local aquifer. It also is bounded by and contains layers of rather dense limerock, cohesive clay and marl embedded in fine sands, all of comparatively poorer permeability. Because of this confining structure, water movement caused by hydraulic stress induced in this zone would meet with far less horizontal than vertical impedence. Such stress would be relieved by recharge from Zone 1, and would extend over a rather broad area.

Since it appears that Zone 2 may actually contain (or is at the least intimate with) the "cavity riddled" Turnpike Aquifer some distance to the west, some degree of stress could be expected to be relieved, or perhaps equalized, by this formation if it were intercepted.

ZONE 3: Except at its upper and lower extremities, Zone 3 is relatively free of confining intermediate layers. The materials comprising this zone appear to have, on the average, much lower values of conductivity and storage capacity than the two higher zones, both of which values become negligible as the bottom is approached, thus defining the lower extremity of the available fresh water aquifer. Due to its confined nature, Zone 3 would not be expected to contribute any significant yield (flow) to higher zones if they were independently stressed.

ZONE 4: Except for a thin lens of coarse material and perhaps small cavities confined at the immediate top of Zone 4, it is considered sufficiently impermeable as to be excluded from the aquifer.

WELL DESIGN CONSIDERATIONS

It was therefore decided that the proper elevation at which to locate the screened portion of the Test Well would be in Zone 2, where well efficiency and capacity would be greatest, due to the layers of high conductivity. The zone appears to be about 50 feet thick at this point, and the area of highest permeability appears to be between 95 and 135 feet depth (-80 to -120 feet M.S.L.), which is the screened elevation of the Test Well.

-50 -144

Due to the variable permeability of the materials found in the screened zone, a gravel packed design was considered more appropriate than a "naturally developed" well, since it would result in a more constant velocity through the face of the screen. This design also permits use of a coarser screen, which would be relatively free of the choking effect of iron bacteria encrustation characteristic of the finely screened naturally developed wells common to the area after prolonged use.

In designing the deep observation wells to be used in the test of the aquifer, it was not considered practical to screen the same entire 40-ft. zone as the Test Well screen. It was also considered that screening only a small portion of the zone could result in some erratic reactions under stress, and not be indicative of the aquifer as a whole. The deep observation well screens were therefore located in Zone 3, just below the lowest confining layer of Zone 2. At this elevation (about -130 to -140 ft. MSL), the average elastic effects of Zone 2 stress should be readily evidenced.

The purpose of the shallow observation wells is to monitor ground water surface during aquifer tests, as opposed to piezometric reaction. Screens in these three wells were thus located well above the confining formations (-15 to -20 ft. MSL), and only sufficiently deep to preclude possible dewatering during testing.

SUMMARY

In designing the test program, a review of available geologic and hydrologic data strongly indicated that a partially - (but far from totally -) confined aquifer condition would be encountered at the Test Site. Test boring data reinforces these indications. The general design, arrangement and extent of the Test and observation well construction is considered sufficient for monitoring not only aquifer stress performance tests with respect to transmissivity, storage and radius of influence, but also to determine to a great extent the probable manner and rate at which the lower semi-confined zones are recharged by vertical movement of water within the total aquifer.

PUMPAGE TESTS.

GENERAL

Installation of the pump, meter and discharge piping at the Test Well was completed late March 12, and a half-hour calibration test at approximately 500 g/m began at 8:00 PM that date. Pumped water was discharged via an 8" pipeline into the lateral drainage canal about 850 feet north of the Test Well, so as to avoid unnatural influence of water levels in the observation wells during pumping. The pump used was a 12" X 5-stage turbine suspended on an 8" discharge column with internal drive shaft, with suction inlet set about 58 feet below static water level.

Drive power was furnished by a 150 HP diesel engine connected through a 2:1 ratio right-angle gearbox. Engine speed was 2200 RPM versus 1100 RPM pump speed at 800 GPM under the applied condition during Test X-2, at an estimated 25 SHP at the engine coupling. A 6" tube flow meter was connected at the end of 20 feet of a 6" pipe immediately downstream from the discharge head at the Test Well. Meter accuracy is estimated at plus or minus 1% (.99 to 1.01). Meter readings were made periodically during Test X-2 to assure constant discharge. Pump shaft speed was checked hourly by means of a tachometer.

TEST X-1

Pumpage Test X-1 was begun at 10:00 AM on March 13. Through some misunderstanding, shaft speed had been set higher than planned, and difficulty with the electric probe prevented accurate drawdown measurements at the Test Well during the first 20 minutes. The flow meter also exhibited some erratic behavior, possibly due to a high gradient on the discharge line causing a partial vacuum at the meter. It appeared that the high pumping rate could possibly induce drawdown to a level very near the pump inlet depth if continued for several hours.

Test X-1 pumping was therefore stopped at 10:30 AM, and recovery in all observation wells was logged for the next half-hour. The probe was repaired and a 4½" diameter orifice plate was installed in the discharge line downstream from the meter, and a pressure gauge upstream, which indicated a positive constant pressure of about 4 psig during later pumping. Comparison of drawdown data of Tests X-2 and X-1 indicate a pumping rate of 925 g/m during Test X-1.

Results of Test X-1 proved useful in observing the time-lag reactions of drawdown in the deep observation wells, which have been used in evaluating the coefficient of storage (S) of the aquifer.

TEST X-2

A period of 1.5 hours after Test X-1 shut down was allowed, by which time the aquifer appeared to have very nearly recovered the static levels observed in all observation wells prior to Test X-1.

Test X-2 pumpage was begun at precisely 12:00PM on March 13 at a pump shaft speed of 1100 RPM, and continued for three days, terminating March 16 at exactly 1:00 PM (after 73 hours = 4380 minutes pumpage).

PUMPING RATE CONTROL

On the morning of March 15, a slight "drift" in the pumping rate was noted, from the 794 g/m observed early in the test to 787 g/m, and most probably due to a gradually receding water level in the pumped well. At this time, pump shaft speed was intentionally increased to 1125 RPM, which resulted in a pumping rate of 804 g/m, which was maintained for the last 27 hours of Test X-2. This change in pumping rate (17 g/m, or about 2%) showed an immediate and distinct reaction in drawdown in the Test Well (about +1.1 ft.) and in the two deep observation wells (about +0.11 ft.).

Based on timed flow meter readings, the average pumping rate during Test X-2 was 795 g/m, with a maximum variation from average in the range of plus or minus 1% (.99 to 1.01). For simplicity, this value has been rounded to a constant 800 g/m in subsequent computations.

TIME AND DRAWDOWN MEASUREMENT CONTROL

Carefully calibrated electric probe type water level sensors were used to measure drawdown at each of the six wells. During the early critical phase of drawdowns and recoveries (when frequent measurements are required), a competent observer was stationed at each separate well. Times of critical observations were synchronized by visual signals from a central point. Time was controlled by a single digital electric watch reading to the nearest second. The maximum error in time of critical observations is estimated to be less than two seconds, and in drawdown measurements, about .005 ft.

Two separate closed differential leveling circuits of second order accuracy were run thru all six well reference elevations from a USGS benchmark located near Haverhill Road and Dyer Boulevard. These circuits also traversed two remote observation wells (PB-845 and PB-844) previously established by USGS. The observed accuracy of these two circuit closures indicates a probable error of about .015 ft. between the wellfield datum and the USGS/MSL datum, and less than .005 ft. between any two points within the test area.

Computed water level observations referred to this common datum therefore appear to be accurate to within about .01 ft within the test area, and to within about .02 ft with respect to the USGS/MSL datum and the remote wells PB-844 and PB-845.

CASING STORAGE EFFECT

Unless a pumped well is 100% efficient, a significant volume of water is removed not only from the aquifer but also from the casing of the pumped well during the first few minutes of pumpage, and this condition continues until the rate of drawdown in the pumped well subsides to the approximate rate of drawdown in the aquifer at its interface with the effective diameter of the pumped well. In the case of an inefficient well, this phenomenon exhibits rather inconclusive drawdown data, at least until a degree of stabilization is reached in the casing-versus-aquifer drawdown pattern, usually after about 5 to 20 minutes of pumpage, depending upon the applied condition.

In the instance of Test X-2, computation shows that higher pump performance (at a constant 1100 RPM) at the shallower early drawdowns almost exactly counterbalanced the quantity of water removed from the casing. The net result was a relatively constant stress of 800 g/m on the aquifer during the first 40 minutes of the test, which was the time required for the pumped well to establish a constant rate of drawdown. Early drawdown observations were therefore distorted only to a negligible extent by variable pumpage during casing storage removal.

AQUIFER REACTIONS - ANALYSES AND DISCUSSION

GENERAL - METHODS AND ABERRATIONS

Time and distance drawdown and recovery measurements in all six wells at the test site observed during Test X-2 have been plotted on semi-logarithmic graph paper, and are as shown on Plates I through VII. It is readily noted that the shapes of the deep well curves are indicative of a confined aquifer. This is further evidenced by comparison of the relatively higher drawdowns in the deep wells to that in the shallow observation wells. The fact that there is an immediate and continued recession in the shallow wells, albeit rather modest, is indicative of a marked degree of vertical water movement through the confining bed(s), thus defining what is known as a "leaky artesian aquifer".

In this type of aquifer, attempts to evaluate performance under stress by use of the Theis formula can lead to misleading conclusions. In its simplest form, the Theis formula is:

$$s = \frac{114.6\ Q}{T} W(u)$$

Where:

s = drawdown, in feet, at any point in the vicinity of a well discharging at a constant rate

Q = Pumping rate, in g/m

T = coefficient of transmissibility of the aquifer, in g/d/ft.

W(u) is read "well function of u".

This formula is quite applicable to wells founded in homogenous (unconfined) or artesian (totally confined) aquifers. It also provides the basis from which the more sophisticated equations applicable to an infinite leaky confined aquifer have been derived.

The method chosen for this analysis is what may be known as the Hantush-Jacob-Cooper Method, as discussed and elaborated by S.W. Lohman in "Ground-Water Hydraulics" (1972), published as U.S. Geological Survey Professional Paper 708, beginning on page 30. "Family type" curves included with this publication have also been utilized and found most helpful.

Behavior of this particular section of the areal aquifer with time and stress appears to agree quite well with the standard model and the observed deviations appear to correlate significantly with previously detected conditions which would be expected to cause some degree of aberration from the normal reaction of an infinite aquifer. The known complexities which induce these aberrations are:

1) Neither the static water table or potentiometric surface within the radius of influence are level, and to the west of the test site, they do not appear to be planar. In the static condition, there exists an eastward gradient of about .0007 in both the groundwater and potentiometric surface. At

the test site, the potentiometric surface remains poised about 0.2 ft. above the groundwater level. Some 1000 feet west of the site, the potentiometric surface rises to about 2.0 feet above a projection of the easterly gradient. No reliable observations were available of groundwater levels in the westerly area, but it is estimated they remain somewhere near the westerly projection of the easterly gradient, due to relatively high transmissivity in the unconfined zone, and relatively low upward leakance from the confined zone. Undocumented observations have indicated differentials of up to +2 feet between adjacent shallow and deep will water levels in this general area, lending some substance to this conjecture, which is based primarily on the consistency of 6 years of monthly observations in U.S.G.S. Wells PB-799 and PB-844 (shallow), and PB-845 (deep). (See Fig. 7)

This condition can be attributed to the relatively high transmissivity and coefficient of storage in the "cavitied" zone of the confined aquifer, as reported by Land (USGS-1976). The combined effect of these conditions would be expected to induce a greater radius of influence, and hence a greater flow of groundwater toward the pumped well in the western quadrants than in their eastern counterparts. (See Fig. 8).

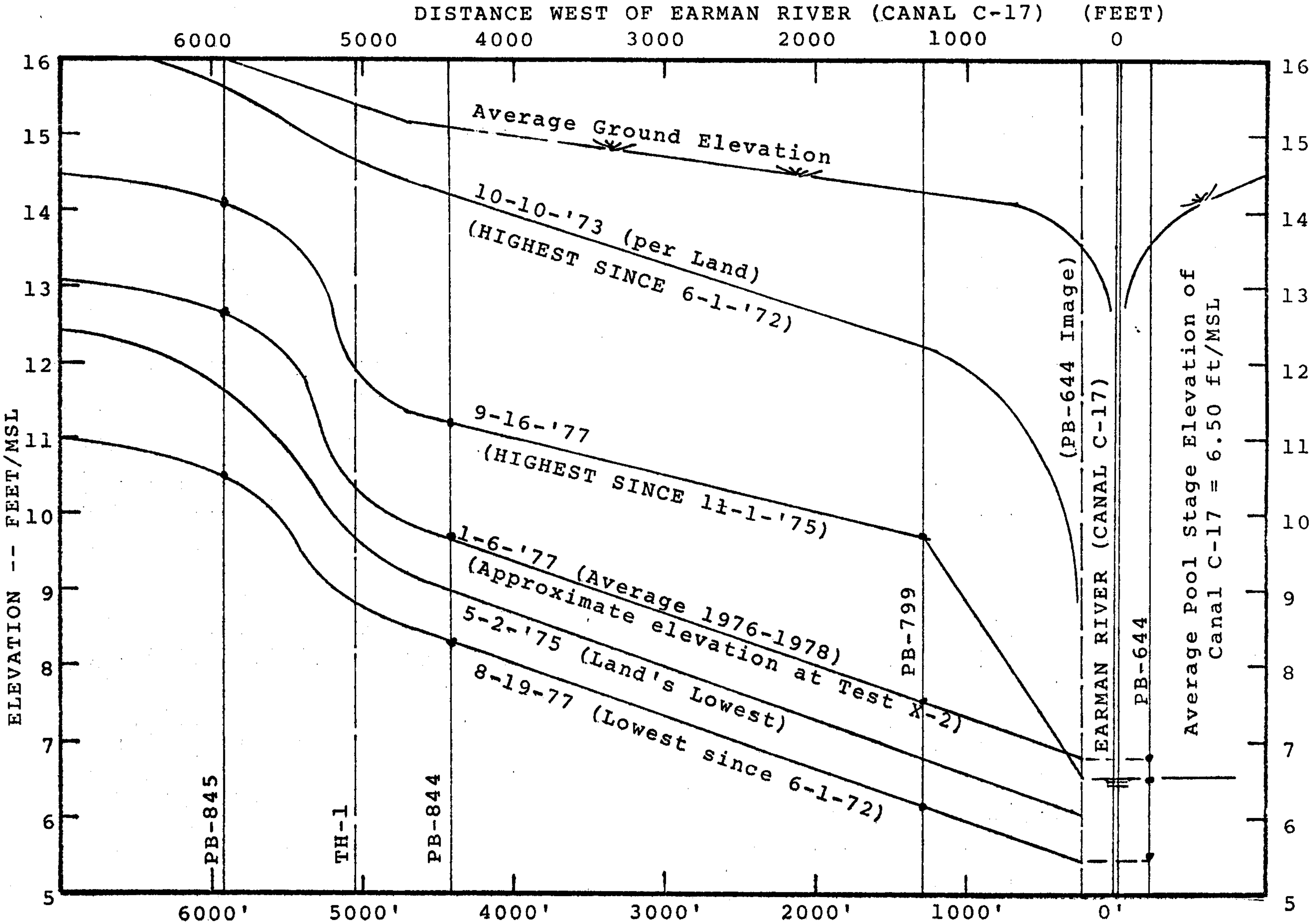
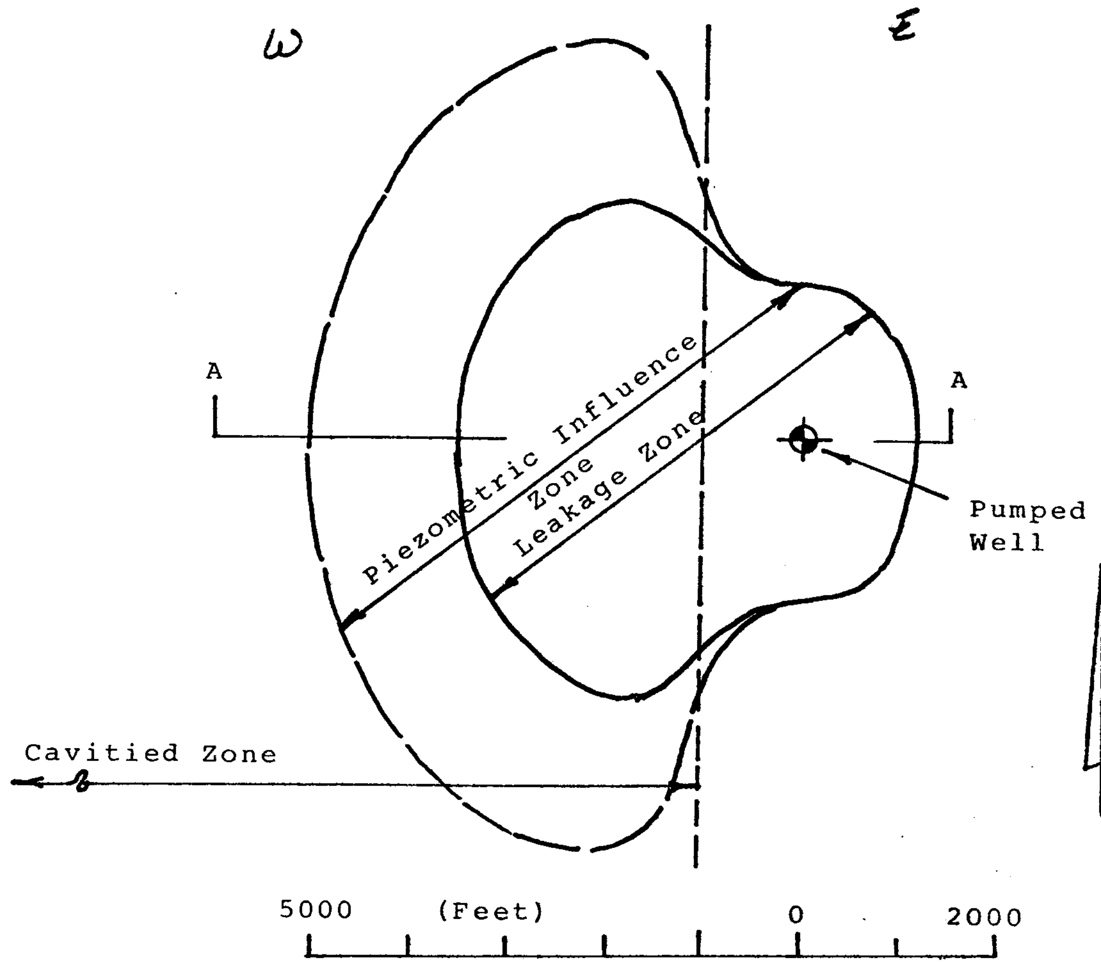


FIG. 7 . Profile of ground water levels in the vicinity of the Test Site at critical times during the period 1972-1978, based on monthly observations of the monitoring wells shown, and indicating the influence of the Earman River on the aquifer in this general area.



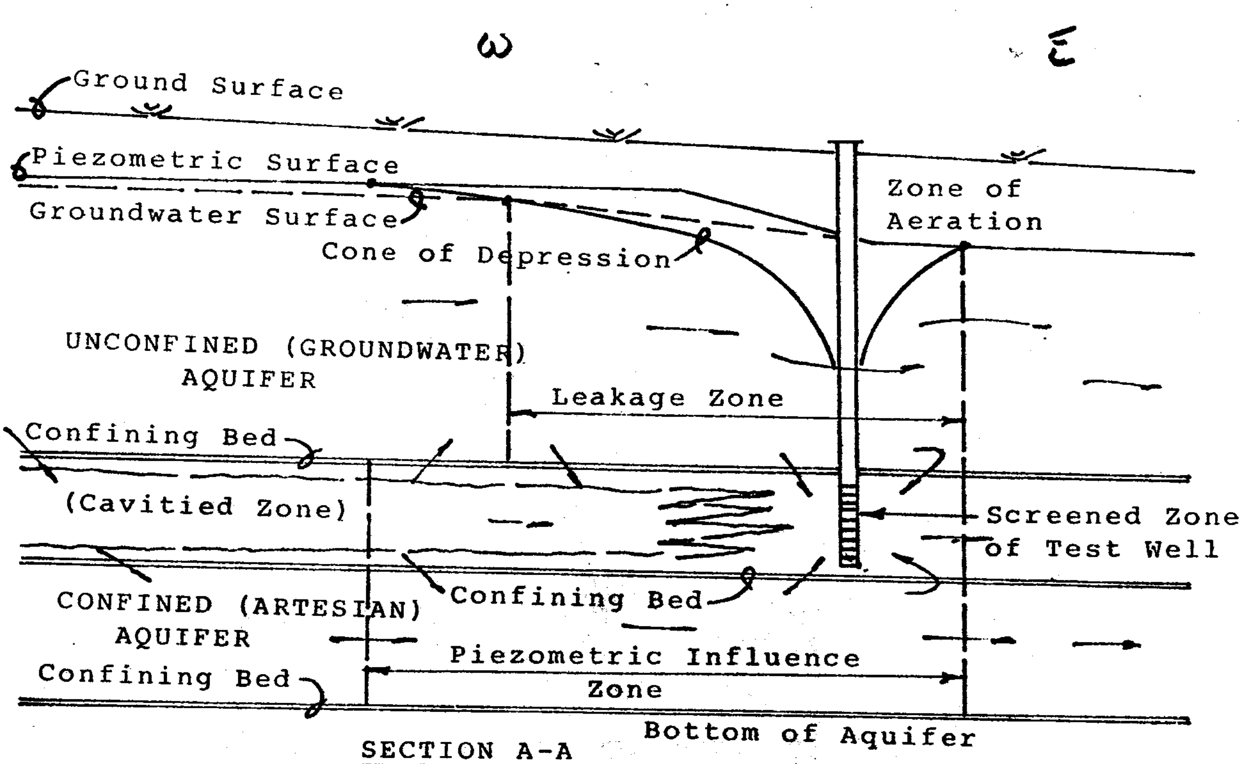


FIG. 8 . Sketch of most probable aquifer reactions at the end of Test X-2.

2) Based on an appraisal of the geophysical, electric and gamma ray logs at the test site, as well as Land's observations and the test data, the degree of confinement diminishes proceeding easterly, but can be assumed to remain fairly constant to the west, at least within the radius of influence of a 100-day stress.

3) The static ground-water table in the radius of influence is neither level nor planar. It has the general eastward tilt of .0007 along the E-W axis previously discussed. Along the N-S axis it is intersticed by two drainage canals, one about 800 feet north, and one about 1800 feet south of the test well. Under average conditions, water level in these canals is about 2.5 feet lower than groundwater along a line halfway between. Flow in these canals during the test period indicated a relatively constant discharge from the shallow aquifer, apparently unaffected by test pumpage.

4) Drainage from the shallow aquifer when it is near average levels, induces a recession in both groundwater level and potentiometric surface of almost exactly .02 ft/day. (See Fig. 9).

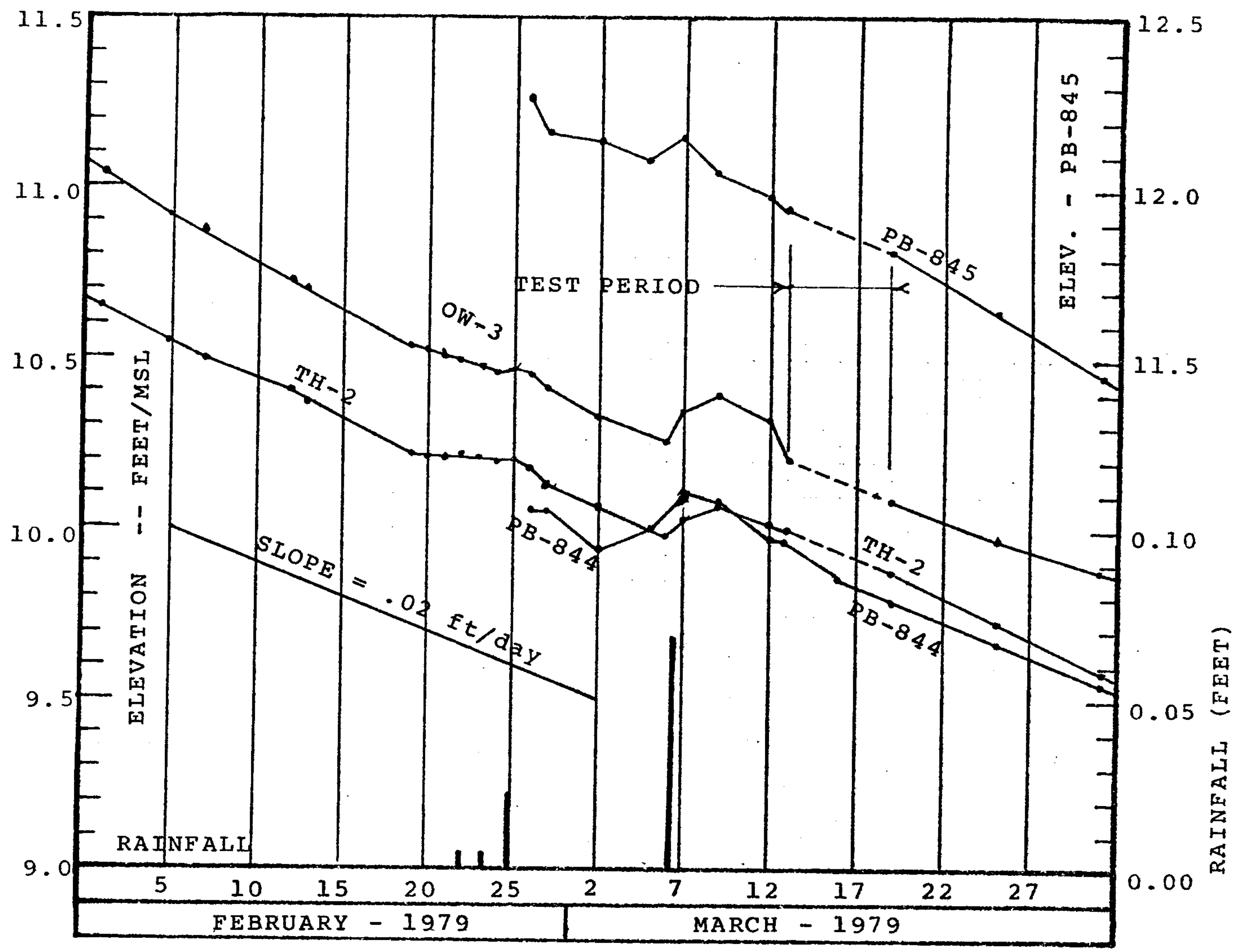


FIG. 9 . Chart showing groundwater level fluctuations in both the unconfined aquifer (OW-3 and PB-844) and the confined aquifer (PB-845 and TH-2), and the relatively constant recession of .02 ft/day during periods of no rainfall.

RAINFALL

Good fortune prevailed in that there was no detectable rainfall to aberrate the data during the entire test, including the recovery period. The last rainfall prior to the Test occurred in the early morning hours of March 7. At that time, 0.8" measured by raingauge at the test site produced a surge of about 0.20 ft. in both the deep and shallow wells, indicating a coefficient of storage on the order of 0.3 in the formation near the groundwater surface.

TEST WELL EFFICIENCY

It is evident that the Test Well is somewhat lacking in efficiency with respect to its performance as compared to the total confined aquifer capability. The maximum possible yield of the Test Well is about 1440 g/m, based on drawdown to the top of the screen. The actual stabilized yield at 800 g/m is 20 g/m ft. At the end of 2 days pumping, the aquifer drawdown is 16.40 feet at the gravel pack/aquifer interface (r=11"), and the casing drawdown is 40.30 feet. The well efficiency is therefore 16.4/40.3 = .407 = 41%.

Geophysical, electric and gamma ray logs all suggest the presence of many confining beds within the confined zone, thus subdividing the confinement. It is inferred that under these conditions, maximum attainable well efficiency would be about equal to the screen length divided by the aquifer thickness. In this light, the Test Well appears to achieve a satisfactory yield, in that 40'/95' = .42.

It is estimated that wells of construction similar to that of the Test Well, but with fully penetrating screens could yield about 48 g/m per foot of drawdown, which appears to be the capacity of the confined aquifer discharging into a 22" diameter well.

DEEP WELL TIME/DRAWDOWN CURVES (Test X-2)

Examination of the pumped well curve (Plate I) indicates that the Test Well required about 40 minutes to rid itself of the "casing storage effect" and attain a declining drawdown rate similar to that evidenced by TH-1 and TH-2 (Plate II), whose curves have a characteristic "leaky artesian" shape. All three curves indicate a condition near equilibrium is reached in about 24 hours. It is also likely that the potentiometric cone of depression had developed sufficient interface with the more transmissive cavitied zone that it was "feeding" on that source to some extent by that time.

As a final comment on the interpretation of the time/drawdown curves, the slight adjustment (17 g/m) in the pumpage rate at 45 hours has no attributable effect on the drawdown trend, but does demonstrate the predicted reaction in all three deep wells. Also, the small undulations in the curves (most noticeable in the more severely stressed test well), are capable of being associated with barometric fluctuations, nearby golf-course irrigation, and daily pumpage in the Grammercy Park Wellfield (some 4400 feet distant) between 8:AM and noon. Golf course irrigation from 6PM March 14 until 6AM March 15 produced a surge in the shallow wells, most noticeable in OW-3, which was located nearest a sprinkler. This irrigation water was pumped at a relatively high rate from a small lake about 1500 feet west of the site for a period of 12 hours. The increase in recession of the levels in the shallow wells beginning at about 2000 minutes is attributed to recharge to the lake, and is reflected in the Test Well at about 2400 minutes.

Time does not permit, nor does the situation appear to warrant a more rigorous exploration of these rather insignificant vagaries.

TRANSMISSIVITY (T)

The distance versus drawdown graph (Plate III) demonstrates the constancy of the shape of the cone of depression near the pumped well, as well as its modest growth rate after about 40 minutes. The interception of P.B. 845 at about 2300 minutes (See Plate XI) into test X-2 also agrees quite well both graphically as well as with respect to computed values. Transmissivity is therefore computed on the basis of a Δs of 5.0 feet per $\log_{10}$ cycle of radius, as indicated on Plate III:

(34) (p.12)

$$T = \frac{2.30\,Q}{2\pi\,\Delta s_r}$$

$$= \frac{(2.30)(1.54 \times 10^5)}{(2\pi)\ (5.0)}$$

$$= 11{,}270\ (ft^2/day)$$

COEFFICIENT OF STORAGE (S)

Test X-1 yields what appears to be reliable and consistent data relative to the computation of the coefficient of storage (S), in the form of effective time of zero drawndown (t_0) at the beginning of the pumpage, observed in TH-1 and TH-2, as 1.03 minutes and 3.75 minutes, respectively. (See Plate 8).

(Eq. 73, p. 24) $S = 2.25\ T\ (t/r^2)$

$S = (2.25)(11{,}270)(1.03/1440)/314.5^2)$

$= .000183$ (TH-1)

$= (2.25)(11{,}270)(3.75/1440)/(606.8^2)$

$= .000179$ (TH-2)

A value of $S = 1.8 \times 10^{-4}$ was thus used to compute a trial match point value of t/r^2 versus $1/u$ for the leakance analysis (See Plate IX). A slightly better curve fit was found for a match point value of $t/r^2 = 5.0 \times 10^{-9}$. Substituting this value into

(Eq.88, p.31) $$S = 4\ T\ \frac{t/r^2}{1/u}$$

$$S = (4)(11{,}270)(5.0 \times 10^{-9})/(1.0)$$

$$= .00023$$

A reasonable value of the coefficient of storage for the confined aquifer therefore appears to be in the range of

$$S = 2 \times 10^{-4}$$

<u>LEAKANCE CONSTANT (L)</u>

Since transmissivity appears well defined, a trial match point value of s versus L(u,v) = 1.0 was computed as

(Eq. 87, p.31)

$$s = \frac{Q}{4\pi T} L(u,v) \quad ; \; L(u,v) = 1.0$$

$$= \frac{(154{,}000)(1.0)}{(4\pi)(11{,}270)}$$

$$= 1.09 \text{ ft.}$$ ok

The values of time (t) and t/r^2 were then computed (See Table I) for Test X-2 at TH-1 and TH-2. The drawdown (s) versus these values was then plotted on full logarithmic paper, and superimposed on a plot of the "type curves" from the referenced USGS paper, the result being as shown on Plate IX. Although the "fit" is somewhat "forced" it appears to be totally reasonable, yielding about the same leakance (L) at TH-1 and TH-2, computed as follows, based on the indicated values of "v".

(Eq. 89, p. 31)

$$L = \frac{K^1}{b^1} = 4T \frac{v^2}{r^2}$$

$$L = (4)(11{,}270)(.12^2)/(314.5^2)$$ TH-1

$$= .0066 \quad \text{(TH-1)}$$

$$= (4)(11{,}270)(.24^2)/606.8^2)$$ TH-2

$$= .0070 \quad \text{(TH-2)}$$

The best apparent value from Test X-2 for leakance appears to be:

$$L = 6.8 \times 10^{-3} \text{ day}^{-1}$$

SHALLOW WELL REACTIONS

Drawdown and recovery observed in the shallow wells (OW-1, -2 & -3) during Test X-2 is shown on Plates IV, V and VII. Although data from these wells cannot be used directly in the aquifer analyses, it does serve to confirm the "leaky artesian" syndrome.

The drawdown in the shallow wells appears to vary as the logarithm of time, and the equation $s = (1.17 \times 10^{-6})\ Q \log t$ (minutes) appears to approximate the drawdown in OW-1. A 100-day projection of this approximation is shown on Plate X.

DEEP WELL RECOVERY

Recovery curves for TH-1, TH-2 and the Test Well from Test X-2 are shown on Plate VI. These curves appear to be the reverse image of the drawdown curves, and are included as information only, since no analysis based on recovery data is made in this report.

EXTRAPOLATIONS

In the "infinite leaky artesian aquifer" concept, the application of a constant stress (Q) at a point (well) in the confined zone generates a cone of depression in the potentiometric surface, which continues to expand until a condition of equilibrium is reached. During the period of expansion, water is removed from storage in the confined zone at a diminishing rate, approaching zero at time of equilibrium, and at which time leakage through the confining bed exactly balances the pumping rate. This condition may be expressed as

(1) $Q = L\forall$,

where Q = stress (pumping rate), L = leakance constant of the confining bed, and $\forall$ = the total force acting on the confining bed.

The leakage rate at any point within the radius of influence varies as the product of s (feet of drawdown of potentiometric surface below groundwater level), L (leakance x day^{-1}), and A (ft^2, area), the dimensions of the product being $ft^3 day^{-1}$. The total force, $\forall$, is equal to the volume of the cone of depression, which, by integration, is found to be

(2) $\forall = 0.682 \Delta s\, R^2$

where R = the radius of influence of the potentiometric cone, and Δs = the $\log_{10}$ slope of drawdown versus distance from the pumped well, per cycle. These following relationships also become evident:

(3) $\Delta s = (s' - s'')/\log (r''/r')$, and

(4) $s_o = s' + (\Delta s \log r')$, or

$= s'' + (\Delta s \log r'')$, and

(5) $\text{Log } R = s_o/\Delta s$

where s' and s'' are the drawdowns observed in two observation wells located at radii of r' and r'' from the pumped well, and s_o = the drawdown at $r = 1.00$ ft.

This group of equations has been evolved by the writer to examine linear and volumetric dimensions of aquifer reactions in general.

Substituting the value Δs = 5.0 (from Plate III), Eq. (2) becomes

(2) $\forall = (.682)(5.0)(R^2) = 3.41\ R^2$ (when $Q = 154{,}000\ ft^3\ day^{-1}$).

Combining Eqs. (1) and (2) produces

(6) $Q/L = 3.41\ R^2$, or $R^2 = Q/(3.41\ L)$

$$R^2 = (154{,}000)/(3.41)/(6.8 \times 10^{-3}) = 6.64 \times 10^6$$

R = 2,577 feet (constant for the aquifer), and

(5) $s_o = \Delta s \log R = (5.0)(3.41) = 17.05$ ft, when

$$Q = 154{,}000\ ft^3\ day^{-1}$$

These appear to be "average" values of R (radius of influence) for any reasonable pumping rate, and s (drawdown at r=1.00 feet) at the time of apparent equilibrium, and agree reasonably well with the Test X-2 observed conditions.

It has been shown that the value of Δs varies directly as Q in a given aquifer, and from Test X-2, (in this aquifer).

$$\Delta s/Q = (5.0)/(154{,}000) = 3.25 \times 10^{-5}, \text{ or}$$

$$\Delta s = Q\ (3.25 \times 10^{-5})$$

for any value of Q.

WATER DEMANDS

The City of Riviera Beach is currently engaged in a program of expansion of its entire public water supply facility to meet the demands of a growing population. The existing treatment facilities are presently stretched to very near their capacity on maximum days, and with all presently available wells pumping, the supply falls short of damand on occasion.

Appendix A to this Report is extracted from the "Engineering Plan and Feasibility Report" for the expansion program, and details to a great extent the growth of the Service Area, and resultant water demands. The following table is extracted from page VII-13 of Appendix A:

YEAR	POPULATION	AVG. DAY MGD.	MAX. DAY MGD.	MAX. HOUR MGD.	FIRE GPM.
1930	811	-	-		-
1940	1,981	-	-		-
1950	4,065	-	-		-
1960	13,046	-	-		-
1970	21,401	-	-		-
1975	33,075	5.754	9.206	12.947	5528
1980	43,000	7.482	11.971	16.835	6249
1985	48,000	8.352	13.362	18.792	6576
1990	55,000	9.570	15.312	21.533	7003
1995	57,500	10.005	16.008	22.511	7149
2000	60,000	10.440	16.704	23.490	7287
Ultimate	83,000	14.442	23.107	32.495	8445

Year 1985 demands have been underscored as the minimum feasible capacity of raw water source which can be considered in the initial expansion phase, the "maximum day" demand of 13.4 MGD (million gallons per day), being the significant figure.

As stated earlier, present demands approaching 10.0 MGD on maximum days, and averaging over 6 MGD on an annual basis are over taxing the capacity of the "Sandy Ridge Aquifer", the present sole source of system raw water.

To preclude further advancement of the salt water front toward the wellfield, it is desirable to reduce pumpage from this aquifer to about 3.4 MGD average, and 5.5 MGD maximum.

It thus becomes evident that about 8.0 MGD raw water capacity (13.4 - 5.5 = 7.9) must be supplied from some new source; most feasibly, a new wellfield in the area of the Turnpike Aquifer.

CAPACITY AND NUMBER OF WELLS

Based on the transmissivity of the subject aquifer, the yield of a well of optimums efficiency (cost effectiveness) would be about 1.0 MGD (700g/m). Thus, eight (8) wells of this capacity would be required in the initial construction phase, and would probably satisfy demands until at least 1985.

LIMITING FACTORS

The Dyer Boulevard Landfill operated by the Palm Beach County Solid Waste Authority, located about 7,500 feet northwest of the Test Site, and about 6,700 feet west of Military Trail, is recognized as a potential source of aquifer contamination in this area. The extent to which leacheate migration from the landfill would eventually affect the useability of the adjacent aquifer as a raw water source for a public supply system remains largely a matter of conjecture.

It appears safe to draw four very general conclusions at this time, however:

1) Induced stress (wellfield pumpage) will probably cause an earlier migration of leacheate than would occur naturally.

2) The effect of the leacheate migration, should it reach the wellfield, will most positively be negative.

3) It would be wise to tickle this dragon's tail as gently and from as great a distance as possible until more is known of his disposition.

4) This dragon is less fearsome than continued induction of salt water into the Sandy Ridge Aquifer.

If this slight editorial digression can be excused, it should be observed that the U.S. Geological Survey (Miami office and staff resource) is presently engaged in evaluating the possible effects of the subject landfill on the areal aquifer. Their Report is currently pending, and is hoped that the test results documented in this report will assist to some extent in that study.

It must be taken into account that salt water aggression in the Sandy Ridge aquifer is fairly well documented, and poses very real and somewhat more predictable consequences than the "in-time-it-may" threat of landfill leacheate on the Turnpike Aquifer.

From the more technical viewpoint, the comparatively low transmissivity and high leakance in the aquifer found at the Military Trail longitude limits the "average" radius of influence of a pumped well to about 2600 feet (½ mile). The higher potentiometric surface west of Military Trail (and lower level east) induces the lip of the cone more westerly (and less easterly), but to not more than about 5000 feet (1 mile). (See Plate III, and Figs. 7 and 8). This is about 1750 feet (1/3 mile) short of the landfill. It can thus be inferred (with some degree of safety that wells located east of a line (say) 1000 feet west of Military Trail will not affect the potentiometric surface at the landfill, which in turn, would not induce a downward movement of the leacheate in that area.

It is rationalized that in the "leaky artesian" condition, once the pumping from a well has developed the conditions of equilibrium in the aquifer, the leakage becomes equal to the pumpage. In the model condition, this leakage is replaced by an inward flow (radially toward the well) from the area outside the radius of (confined) influence, in the (unconfined) zone of the aquifer lying above the confining bed. This horizontal movement (with a small vertical component) could induce some eastward migration of the leacheate, and it would be useful to know what the speed of this movement would be, or the "travel time" from the landfill to the well. This requires assigning some values to the stress and other factors, which is the next subject discussed, after which this problem can be revisited.

LOCATION AND SPACING OF WELLS

The "Demand Table" on page 48 shows the "Ultimate Maximum Day Demand" to be on the order of 23 MGD, of which, as explained earlier, only about 5 MGD can be furnished from the Sandy Ridge Aquifer, leaving about 18 MGD to be supplied by the Turnpike Aquifer. It is assumed that ultimately a series of wells of 18 MGD total capacity could be located at equal intervals along Military Trail in the two miles (10,600 feet) between the north and south boundaries of the Riviera Beach Reserve Annexation Area.

This would result in the ultimate demand being distributed along this axis at the rate of 18(MGD)/10,600(ft) = 1700 (g/ft/d) and, 1700/7.48 = 227 $ft^3/ft/d$, which reduces to 227 ft^2/d, which we shall call the "unit demand" (Q_b). As stated earlier, the optimum well capacity (Q) would be about 700 g/m or 1.0 MGD, of which eight would be initially required. The optimum spacing of these wells would therefore be equal to Q/Q_b, or 1,000,000/1,700 = 588 (say) 600 Feet.

It is noted at this point that Q_b/T = 227/11,270 = .02 indicating that this unit demand is equal to about 2% of the aquifer transmissivity. Since the radius of an individual well screened in the confined zone of this aquifer is about 2600 feet, the suggested spacing of 600 feet indicates that the cones of depression of a series of pumping wells along a straight line will create a "dimpled trough" effect in the potentiometric surface, resembling a channel. This suggests that a "line sink" as opposed to a "point sink" method of analysis would be more appropriate to use in approximating the equilibrium state with all wells pumping, which also simplifies the arithmetic to a great extent.

For this aquifer, to recapitulate, these values have been determined:

$T = 11{,}270\ (ft^2 day^{-1})$

$S = 2.3 \times 10^{-4}$ (dimensionless)

$L = 6.8 \times 10^{-3}\ (day^{-1})$

At page 40 in the referenced text, the "line sink" conditions are equated in these terms, with some new (slightly rearranged) dimensions added. These are:

s = Drawdown, as before (ft).

s' = Drawdown at drain

Q_b = Stress, or base flow of the drain per unit of length, which is the same as "unit demand" and

Q_b = 227 $ft^2\ day^{-1}$, in this instance.

x = Distancce from drain to point of observation.

x' = Distance from drain to nearest point of zero drawdown.

(Eq. 115, p.40) $$T = \frac{Q_b\, x}{2s} D(u);\ D(u) = 1.00$$

Rearranged (7) $$\frac{s}{x} = \frac{Q_b}{2T} = G \text{ (gradient)}$$

$$= (227)/(2)/(11{,}270)$$

$$= 1.00 \times 10^{-2}, \text{ or}$$

$$s = .01\ x$$

At this point, it is necessary to determine the "line sink" leakage, induced by $\forall$ (total unit force on confining bed, per foot of drain), and in this instance

(Eq. 1, p.46) $\forall L = Q_b \; ; \; \forall = Q_b/L = x's'$ (8)

$$\forall = 227/6.8 \times 10$$

$$= 3.34 \times 10^4 \; (ft^3)$$

$$x's' = .01 \; (x^{12}) = 3.34 \times 10^4$$

$$x' = 1830', \; s' = 18.3',$$

which is the distance (x') from the drain to the edge of the trough, and 18.3' is the drawdown at the drain (average). This would approximate the reaction along an east-west section at (say) wells nos. 4 and 5 in a series of 8 wells built along a north-south line, but does not allow for half the pumpage of each of the end wells being supplied from beyond the zone of the model section. This could be roughly corrected by reducing the value of Q_b by $1 - \frac{N-1}{N}$, N being the number of wells in the series. However, in practice, it would merely result in less drawdown in the end wells, if all were pumping at the same rate.

Prediction of drawdown at the pumped wells in this arrangement becomes complex, but based on interference at r = 300' (half the spacing), it is approximated as follows (at r = 1.00 ft):

$$\Delta s = Q\,(3.25 \times 10^{-5}) \qquad \text{(p.47)}$$

$$= (1.34 \times 10^{5})(3.25 \times 10^{-5})$$

$$= 4.34 \text{ ft}$$

$$S_o = (\log 300)(4.34) \qquad \text{(5,p.47)}$$

$$= 10.8 \text{ ft. (conic drawdown)}$$

$$s' = \frac{s}{x}(x' - 300)$$

$$= (.01)(1530)$$

$$= 15.3 \text{ ft (channel drawdown)}$$

$$s_o + s' = 26\text{ft } (\pm) \text{ (or about 27 g/m/ft)}$$

This method of approximation is without substantiation, and perhaps somewhat tenuous, but a more reliable method of predicting yield versus drawdown under the applied condition could not be discovered by the writer. It can be rationalized that the yield per foot of drawdown would be somewhat less than for an isolated well (about 45 g/m, but greater than for a "point-sink" superimposed on a "line-sink" condition, (about 700/(18 + 17) = 20 g/m). An aquifer yield of 27 g/m ft @ r = 1.00 ft is probably sufficiently accurate for sizing the pumping equipment on the proposed wells, which is the critical question in this regard.

PEAKING FACTORS

It must be taken into account at this time that the computations of aquifer performance, demand and required capacity up to this point have been based on the "maximum day" condition, which is the situation of "worst condition" of stress ever expected to be placed on the wellfield. (Please refer to page VII-11 of Appendix A). On an annual average basis, these demands are reduced to .62% of maximum, and on a quarterly basis, to about 72% of maximum. The 1985 (Phase I) demands on the Turnpike Aquifer are predicted as follows:

PERIOD	MGD
Maximum Day	8.00
Maximum Quarter	5.75
Average Day	5.00
Minimum Quarter	4.50
Minimum Day	3.35

LEACHATE MOVEMENT

It is now possible to return to the task of estimating the length of time it would most likely take for leachate to move from the landfill area to the wells, which problem may be approached by two different methods.

The first method could be called the "point sink" type of analysis, and is based on the equation

$$t' = (e^{(r^2\theta/4Tt)}-1)(4\pi bTt/Q)$$

where t' = the "travel time" in days, t = pumping time in days (say, 365), r = distance (about 7,000 feet), θ = porosity, say about .01 to .02, which is the estimated average storage of the aquifer, b = the thickness, and the other units and values are those previously determined. This equation yields a travel time (t') of about 400 to 800 days, depending upon which value of the θ function is used. It is believed that this equation is probably fairly accurate for totally confined or totally unconfined aquifers, but there may be some doubt as to its applicability in the more complex leaky artesian. (In this computation, the value of "Q" was assumed to originate with a single well pumping at a constant rate of 5.0 MGD.)

The second method of analysis could be called the "line sink" type. In this particular case, it is presupposed that all horizontal movement in the aquifer is limited to the zone above the confining bed, and velocity is equal at all points along an infinite horizontal axis perpendicular to the channel and projecting to both sides of the channel. The velocity (average) would then be:

$$v = Q_b/(2b\theta), \text{ and}$$

$$t' = x/v = 2xb\theta/Q_b$$

where t' = travel time in days

Q_b = unit stress per foot of channel, in this instance, the annual average is $Q_b = (227)/(1.6) = 142$ ($ft^2 day^{-1}$)

x = distance travelled (7000 ft)

b = aquifer thickness (85 ft)

θ = average porosity (say, S = .05 for shallow zone), then

$$t' = (2)(7000)(85)(.05)/(142) = 419 \text{ days.}$$

As before, t' is sensitive to the roughly estimated values of θ.

In this manner, about 400 days is estimated as the shortest time in which the leachate could move to the well field after pumpage is begun, and there are several factors which could delay it to perhaps 800 or 1200 days, or even years. Some of these factors are:

1) The Q_b factor is for 1985, and initial pumpage would be lower.

2) Assuming an east-west movement of water would be induced by pumping, and that the observed natural movement is west to east, it is noted that the proposed location of the wells is not directly downstream, requiring a diagonal vector in the leachate movement.

3) "Cavitied zone" recharge of the areal aquifer would tend to relieve a part of the stress.

The only reasonable conclusions that can be drawn from the available evidence appears to be:

1) Pumpage at the proposed well locations may, or perhaps will, induce an eastward movement of the landfill leachate toward the wells.

2) The movement will be sufficiently retarded as to allow time for its evaluation as to quality and intensity, and to take whatever measures are necessary to protect the public health.

3) That the phenomenon cannot be more precisely evaluated until it has been caused to occur.

4) That it should be given some further consideration at this time.

POSSIBLE EFFECTS OF LEACHATE ON WATER QUALITY

There appear to be several factors which would tend to mitigate the possible negative effects of leachate contamination, if and when it reached the raw water supply. Among these are:

1. Distance. The 7000 feet between the nearest point of the landfill and the nearest well provides an excellent barrier against the conductance of any bacteriological or particulate matter.

2. Time. The estimated relatively slow migration provides not only lead time for making corrective provisions, but also reaction times for chemical changes in the leachate, such as oxidation or pH adjustment, or other natural reactions.

3) Vertical Separation. The leacheate interface with the aquifer is at the surface of the groundwater zone, while the wells are proposed to pump from the confined zone some 90 to 150 feet below the groundwater surface. Any induced movement of leachate would remain near the surface over most of the distance, and over the period of a year or more, much could be lost to vegetative transpiration on its way to the wellfield.

4) _Dilution_. The landfill site covers only 1/20 of the horizon of the wellfield, and less than one percent of the visible recharge area of the aquifer, thus indicating a high degree of dilution of any leachate which could reach a well.

5) _Drainage_. During the past six years, not a single instance could be found when the water table in this area dropped to the level of the bottoms of the drainage canals, or to below about five or six feet above the pool stage of the Earman River (which drains the area) at the longitude of Haverhill Road, east of the landfill. This indicates that there was a constant skimming of the aquifer by the drainage canals in the area, and that this natural (man-induced) action could be expected to skim off much of the leachate which might be expected to leak into the aquifer in the future. It also raises the interesting possibility that this means could be used to "corral" the leachate, assuming, of course, that some means could be found to then dispose of it.

6) _Treatment_. It is assumed that any substance originating from the landfill and penetrating 7000 feet of natural filter media plus some rather tight confining beds, would necessarily have to be in total solution in the raw water, the same as the natural elements

and compounds found in the groundwater, and to some extent would be amenable to the normal treatment used in the softening process, or could be reduced to acceptable levels (perhaps) without special treatment.

7) Strength. Although very little control appears to have been exercised in what was allowed to be put in the landfill over previous years, it is perhaps fortunate that there are few industries or activities indigenous to the area which would be expected to contribute large amounts of toxic materials to the landfill. Hopefully, then, time will not produce in the aquifer the heavy metals, pesticide and petrochemical residues which would be expected in a more industrialized area.

PREVENTIVE MEASURES

The most obvious preventive measure with respect to aquifer contamination by the leachate lies in the construction and operation of the landfill itself. The existing damage is perhaps irreversible, but this must not be allowed to become the excuse for further fouling of the valuable groundwater resource. Recommendations for action in this respect are beyond the scope of this Report, but it is felt that as a minimum, the responsible agencies should actively enforce the laws and regulations governing construction and operation of expansions of this particular landfill, with particular respect to proper on-site containment and disposal of leachate.

Beyond this, and as part of the wellfield program, it is recommended that a gallery of four observation wells be provided to monitor any leachate movement toward the wellfield. These wells should be located no further east than Haverhill Road, and approximately on a line between the landfill and the wellfield. One of the wells would be screened near the groundwater surface, one near the middle and one near the bottom of the unconfined zone, and one near the top of the confined zone.

Water samples from these wells should be collected and analyzed on a monthly basis, in order to detect, monitor and evaluate any leachate migration and effect.

It is also recommended that appropriate provisions be made in the proposed expansion of the water treatment plant as to not preclude the convenient addition of activated carbon filtration, ozonation equipment or other process, as may be found necessary in the future.

AQUIFER CAPACITY

Comparison of rainfall to groundwater levels for the last three years as shown in Figures 10 and 11, tends to indicate that a much higher proportion of rainfall is captured by the aquifer in this area than in the better drained coastal area.

Assuming that the coefficient of storage (S) is about 0.25 in the formation near the groundwater, and that the .02ft/day recession in groundwater (See Figures 7 and 9) in dry weather is due in the main to evapotranspiration, would account for about 22 inches of annual rainfall passing through the aquifer in that manner. The shallow static hydraulic gradients (about .0007) in both the shallow and deep wells indicates very minimal discharge to surface drainage, when the groundwater is at its average levels, estimated at less than (Q_b =) 20 $ft^2 day^{-1}$.

Comparing selected periods shown on Figures 10 and 11 indicates that

1. The aquifer discharges rapidly when it is surcharged by periods of heavy rainfall.
2. When it is low, the aquifer responds rapidly to rainfall.

Based on these two indicators, plus the known poor drainage in this area (as compared to the coastal region) it is estimated that probably in the range of 50% to 75% of the annual rainfall passes through the aquifer.

Also from these indicators, it can be deduced that the only time wellfield pumpage could have a severe effect on the aquifer would be after a prolonged period of not just less than average, but of nearly no rainfall whatsoever. Historically, these periods seem to seldom exceed 3 months, and for this reason, statistics are developed for a 100-day drought, and complete as follows:

Total Pumpage:

$$5.75 \text{ MGD} \times 100 = 5.75 \times 10^{8} \text{ gal.}$$
$$= 7.69 \times 10^{7} \text{ ft}^{3}$$
$$= 1765 \text{ Acre-feet}$$

Aquifer Volume: (S = .25)

$$1765/.25 = 7060 \text{ Acre-feet}$$
$$= 3.08 \times 10^{8} \text{ ft}^{3}$$

The significance of these figures is strictly statistical, but they do demonstrate the relative insignificance of the volume of the aquifer dewatered during a 100-day drought, compared to the volume of the aquifer as a whole. Based on an extrapolation of shallow well reactions during Test X-2, it appears that there could be a recession of groundwater level of up to four or five feet in the immediate vicinity of a constantly pumped well after about 100 days, two feet of which would be due to natural evapotranspiration.

Due to the relatively high transmissivity of the unconfined (groundwater)zone of the aquifer, the source of the pumped water extend over an area many times the size of the potentiometric zone of influence, and under these conditions, the "cavitied" zone would provide a great leveling influence.

In a more specific vein, it is noted that transmissivities and coefficients of storage appear to be substantially higher in The Turnpike Aquifer, than in the present wellfield, which has tolerated much higher pumpage stress than that proposed for the new wellfield, and which, it is also noted, is located in an aquifer whose areal extent is several times that of the Sandy Ridge Aquifer, whose main limitation is due to its proximity to the ocean.

In any event, the Turnpike aquifer appears to provide much more than adequate capacity to provide not only for the 1985 demands, but also the ultimate demands of the Riviera Beach Water System.

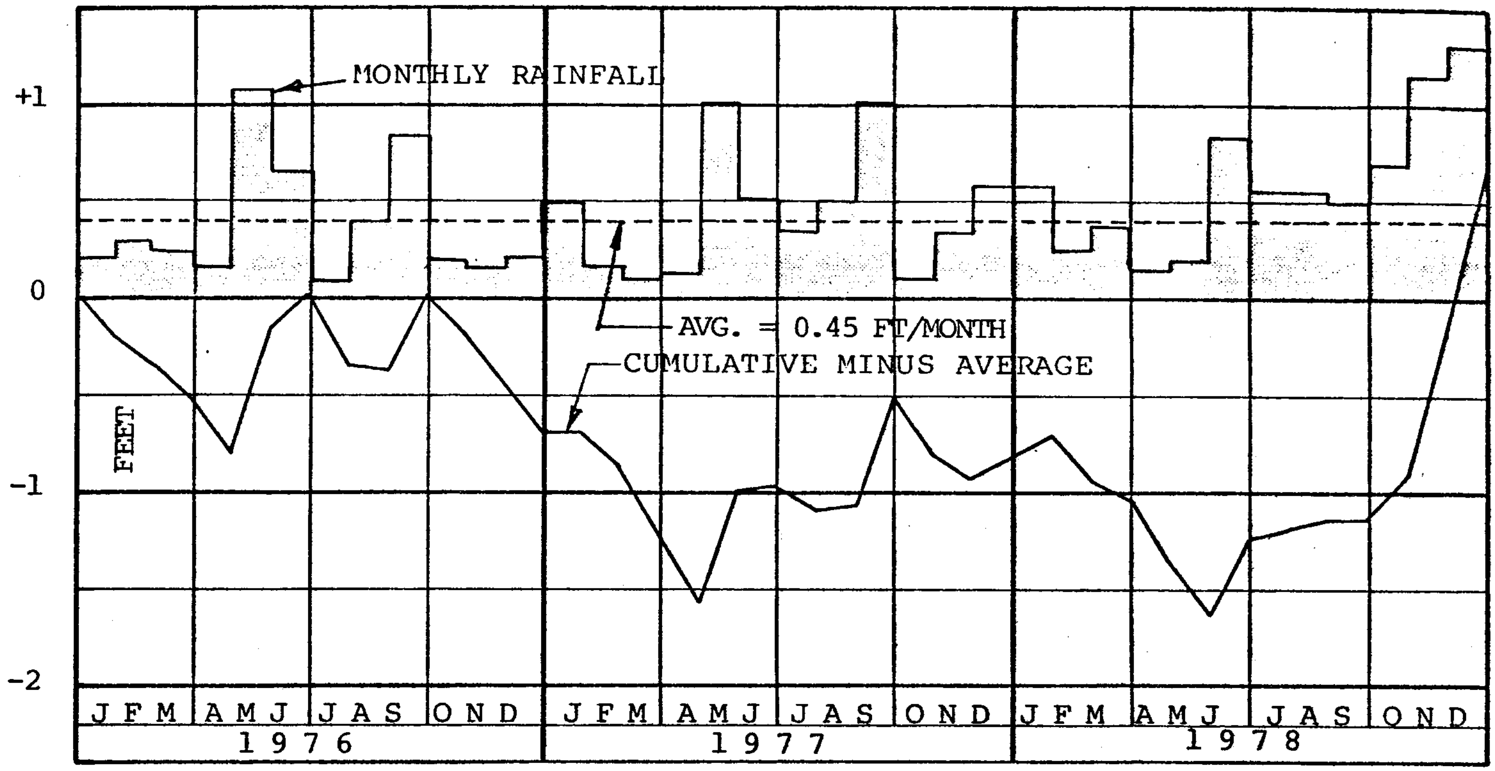


FIG.10 Actual rainfall pattern in Riviera Beach Area from Jan. 1, 1976 through Dec. 31, 1978.

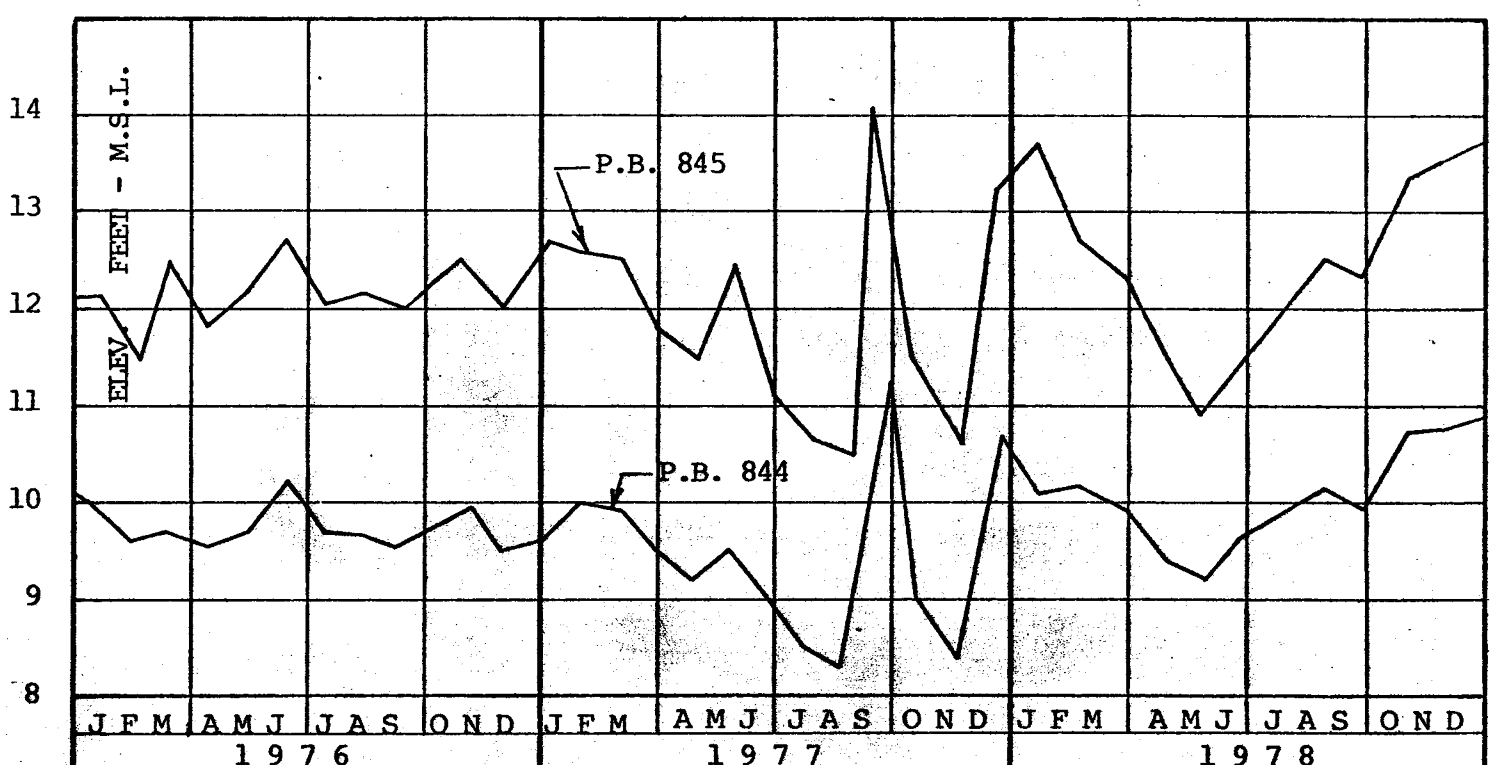


FIG. 11 Groundwater level fluctuations in the vicinity of Military Trail during 1976, 1977 and 1978.

RAW WATER QUALITY

After completion of construction of the observation wells at TH-1 and TH-2, and at the conclusion of the development operation, chemical analyses of groundwater samples from these two wells exhibited noticeably different characteristics (See Cols. 2 and 3, Fig. 12). These wells were resampled and independently analyzed, confirming the validity of the earlier tests.

Comparison of these samples to those analyzed by Land in 1975 show a striking resemblence between TH-1 and PB-845 (Cols. 2 and 4) and between TH-2 and PB-844 (Cols 3 and 5). No conclusion is drawn from this, and it is mentioned only as a matter of passing interest.

Comumn 1 represents the analysis of a sample taken from the Test Well after over 24 hours' pumping. This sample appears to be a "blend" of the others, and is thought to be representative of the probable quality of the long-term yield of wells founded in this aquifer.

The Column 1 sample was not analyzed for trace metals and nutrients since significant concentrations of these are not characteristic of the aquifer. Typical values found by Land in 1972 in the Turnpike Aquifer are as follows:

ELEMENT	CONCENTRATION	% MCL *
Metals		
Aluminum (Al)	0 Mg/1000 l	0 (%)
Zinc (Zn)	10 "	.2
Lead (Pb)	20 "	40
Copper (Cu)	10 "	1
Chromium (Cr^{+6})	0 "	0
Arsenic (As)	10 "	20
Mercury (Hg)	0 "	0
NUTRIENTS		
Nitrite (N)	0.08 mg/l	8
Nitrate (N)	0	0

(* Raw water concentration expressed a percentage of "Maximum Contaminant Level" per EPA Interim Primary Drinking Water Water Regulations.)

It is apparent that the Turnpike Aquifer provides water of suitable quality for public drinking water supply after treatment by the conventional lime-alum softening process.

CHEMICAL ANALYSES OF SELECTED GROUNDWATER SAMPLES IN THE TURNPIKE AQUIFER

	COL. 1	COL. 2	COL. 3	COL. 4	COL. 5
Total Dissolved Solids	510	350	520	356	537
Total Hardness ($CaCO_3$)	266	260	320	260	330
Alkalinity ($CaCO_3$)	258	250	290	260	367
Non-carbonate Hardness ($CaCO_3$)	8	10	30	0	0
Bicarbonate (HCO_3)	315	250	290	352	448
Iron (Fe^{++})	.10	.05	.05	.88	3.3
Sulphate (SO_4)	1	8	2	1.5	3.6
Chloride (Cl)	44	32	16	32	93
Calcium (Ca)	--	96	124	98	120
Magnesium (Mg)	10	20	10	3.8	6.4
Fluoride (F)	0.2	0.2	0.2	0.3	0.5
Sodium (Na)	73	58	29	25	68
Carbon Dioxide (CO_2)	38	18	26		
Bicarbonate ($CaCO_3$)	258	250	290		
Color	18	25	30	100	40
Turbidity	0.2	0.5	0.2		
pH	7.1	7.3	7.3	7.4	7.4
pHs	7.0	7.1	7.0		

COLUMN 1: Test Well, 3/14/79, after 24 hours pumping, confined zone.
COLUMN 2: Test Hole No. 1, 1/29/79 (not stressed), confined zone.
COLUMN 3: Test Hole No. 2, 1/29/79 (not stressed), confined zone.
COLUMN 4: PB-845, 1/23/ 75, (not stressed), cavitied zone. (per Land).
COLUMN 5: PB-844, 2/11/75, (not stressed), unconfined zone.

Fig.12. Water quality analyses, Turnpike Aquifer Groundwater.

WELL DESIGN AND LOCATION

Prior to the aquifer tests, it was estimated that the aquifer could perhaps efficiently sustain wells pumping in the 1000 to 1500 GPM range. As stated in the previous section, efficient single well capacity based on the tests appears to be more in the 700 GPM range. The Test Well was constructed to be able to accommodate the higher capacity, and will thus perform satisfactorily at the 700 GPM rate.

Based on data gathered from testing, it appears possible to construct a slightly more efficient well for less probable cost, however. The apparent higher hydraulic conductivity and greater isolation from possible surface contamination indicate the confined zone of the aquifer as the desirable level from which the raw water should be withdrawn. This zone begins at a depth of approximately 90 feet and extends downward to about 180 feet.

The high degree of confinement found at the Test Well appears to restrict efficiency to a high degree, and indicate the necessity of a longer screen. Optimum pump and discharge column sizes dictate 12" as the minimum casing diameter.

The dimensional aspects of the proposed well construction is therefore:

1) 12" diameter x 95' long steel casing cemented into 20" diameter x 92' borehole.

2) 60 l.f. of 12" telescope diameter screen set in 12" x 60' deep borehole.

3) Total depth of well: about 150 feet.

A total of 8 wells are proposed to be located as shown in Fig. 13.

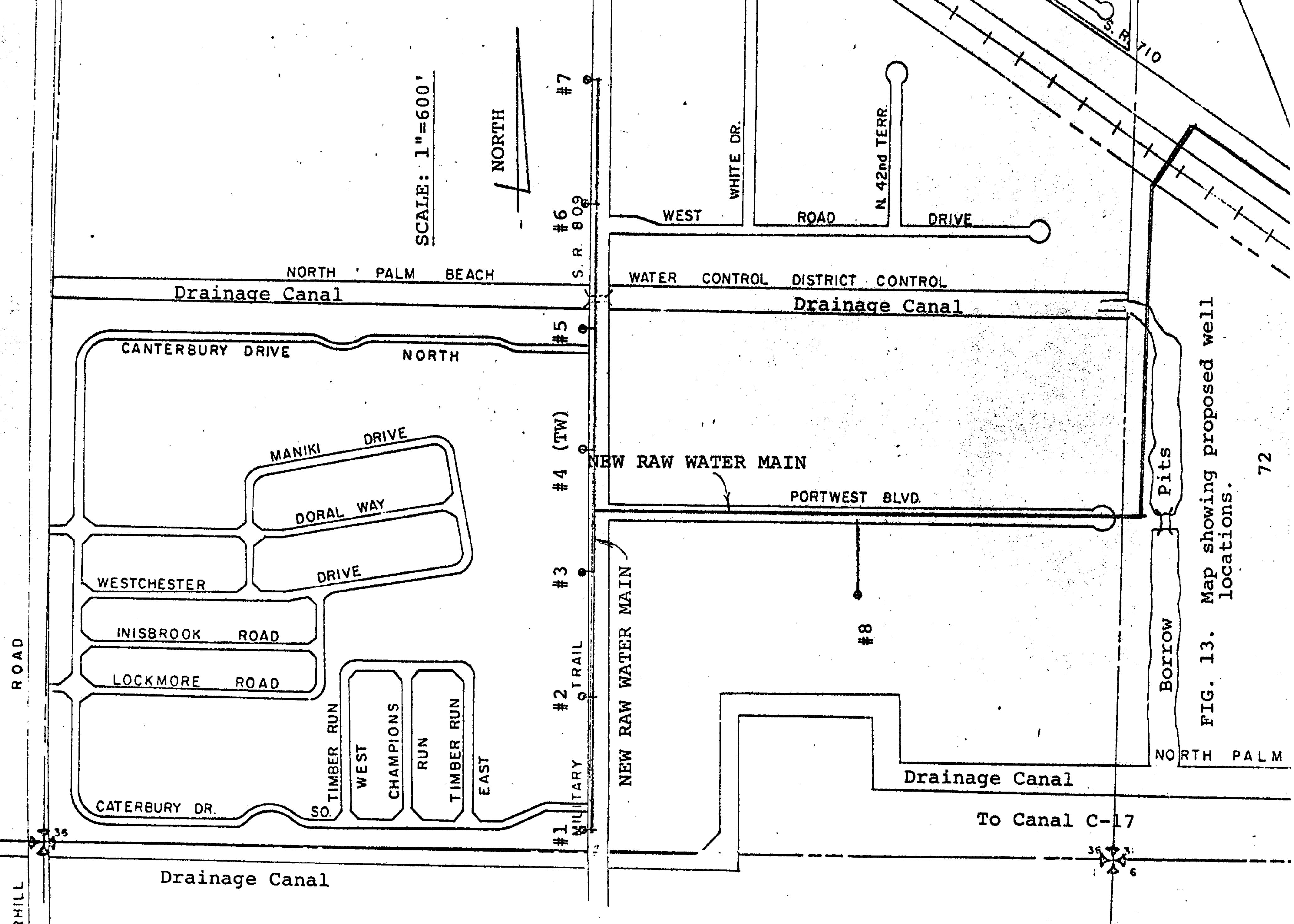


FIG. 13. Map showing proposed well locations.

OTHER USER IMPACT

Only two known groundwater users of any demand consequence could be found in the area, these being the Grammercy Park Utility Company and the Lone Pine Golf Course irrigation system.

The Grammercy Park wells are about 3600 feet west of Military Trail (see fig. 2), and pump about 100,000 gallons daily from the confined zone of the Turnpike Aquifer. It is estimated that pumpage from the proposed wellfield would increase drawdown in the Grammercy Park wells by less than one foot, due to the shallow overlap of the radii of influence.

The Lone Pine Golf Course is irrigated by pumpage from a small surface lake about 1500 feet west of Military Trail. This lake is recharged by local storm water runoff in rainy seasons, and by unconfined groundwater seepage in dry seasons. Pumpage from the confined zone by the proposed wellfield would have no appreciable effect on the levels in this lake, or on any random domestic irrigation wells in the area, since ground water drawdown at the pumped wells is estimated to be on the order of only two or three feet after a 100 day drought.

Therefore, it is doubtful that there would be any consequential impact detected by any other user within the radius of influence of the proposed wellfield due to withdrawals from that source.

PUMPING TIME		TH-1 (r=314.5')		TH-2 (r=606.8'	
Minutes	Days	s	t/r^2	s*	t/r^2
1.0	.000694	.28	7.02×10^{-9}		
2	.00139	.78	1.41×10^{-8}		
3	.00208	1.11	2.10		
4	.00278	1.37	2.81	.39	7.55×10^{-9}
5	.00347	1.56	3.51	.47	9.42
10	.00694	2.12	7.02	.77	1.88×10^{-8}
15	.0104	2.43	1.05×10^{-7}	.97	2.82
20	.0139	2.62	1.41	1.13	3.77
25	.0174	2.75	1.76	1.25	4.72
30	.0208	2.84	2.10	1.33	5.65
40	.0278	2.99	2.81	1.45	7.55
60	.0417	3.11	4.21	1.60	1.13×10^{-7}
90	.0675	3.23	6.32	1.73	1.70
120	.0833	3.31	8.42	1.81	2.26
160	.111	3.36	1.11×10^{-6}	1.88	3.01
220	.153	3.43	1.55	1.97	4.16
360	.250	3.45	2.52	2.00	6.79
600	.417	3.52	4.22	2.08	1.13×10^{-6}
1000	.694	3.55	7.01	2.12	1.88
1440	1.00	3.56	1.01×10^{-5}	2.13	2.71
3600	2.50	3.64	2.52	2.22	6.79
4380	3.04	3.65	3.07	2.24	8.26

* Measured below TH-1 static water level.

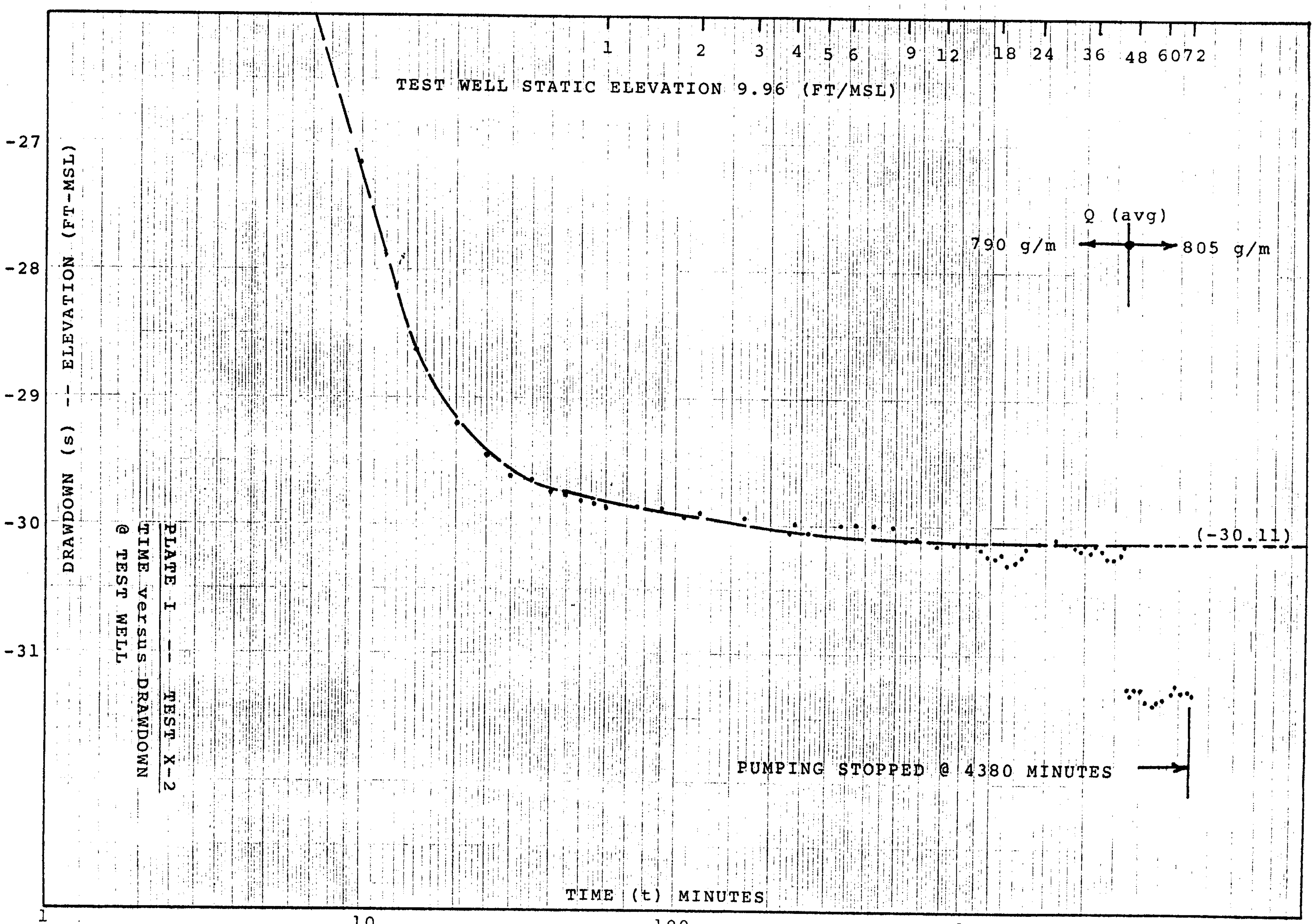
TABLE I. Computed values of t, s and t/r^2 (values of drawn down versus time/radius squared) based on observations at TH-1 and TH-2 (deep observation wells) during Test X-2.

DATE	12AM	6AM	12PM	6PM
3/13	30.23	30.23	30.27	30.20
3/14	30.21	30.15	30.18	30.00
3/15	30.10	30.12	30.21	30.19
3/16	30.25	30.25	30.35	30.33
3/17	30.24	30.33	30.39	30.32
3/18	30.28	30.24	30.23	30.17
3/19	30.18	30.14	30.16	30.10

TABLE II. Barometric pressures recorded at Palm Beach International Airport Test period (in - Hg).

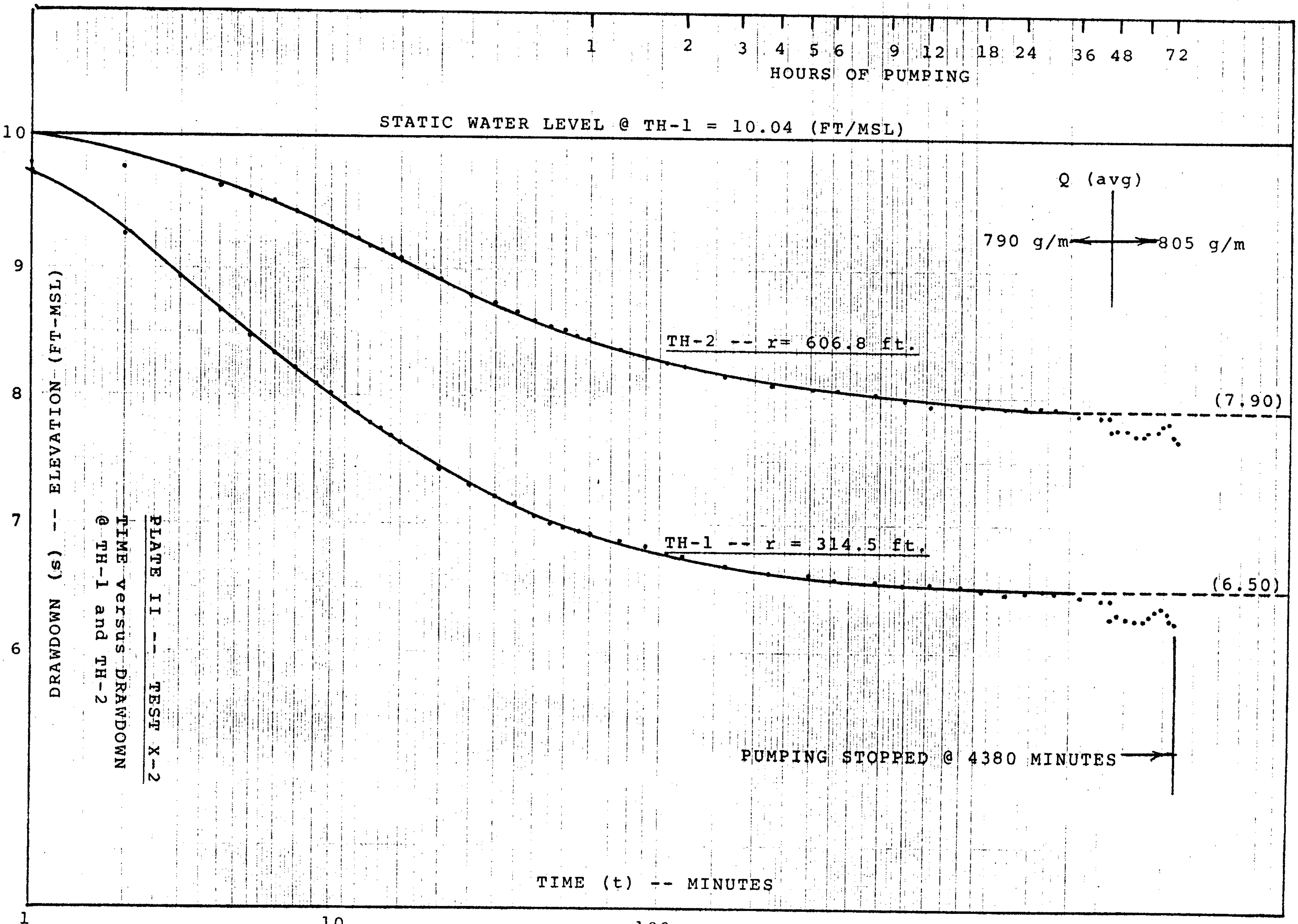
SEMI-LOGARITHMIC 4 CYCLES X 70 DIVISIONS
KEUFFEL & ESSER CO.

46 6013



K+E SEMI-LOGARITHMIC 4 CYCLES X 70 DIVISIONS
KEUFFEL & ESSER CO. MADE IN U.S.A.

46 6013



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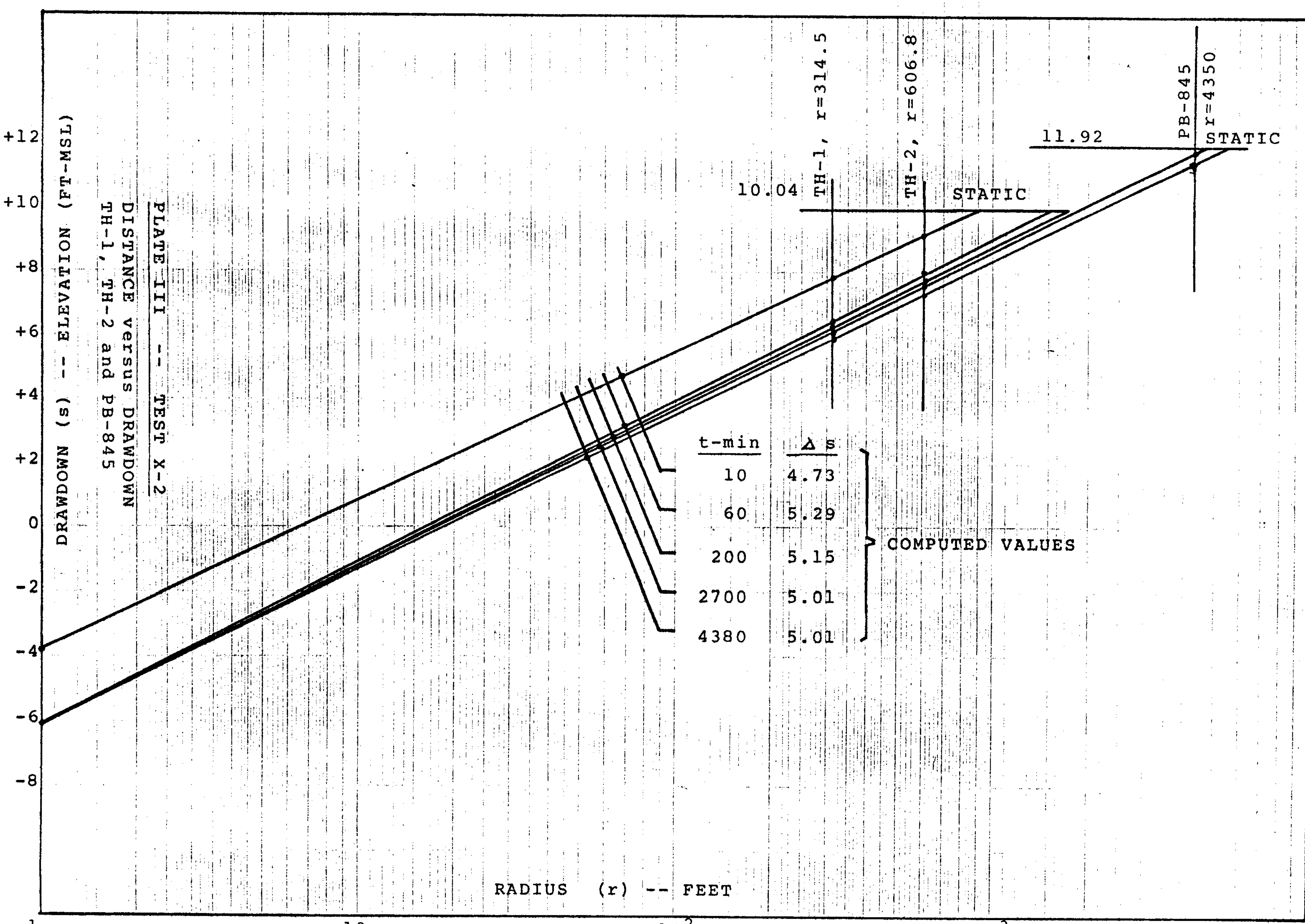


PLATE III -- TEST X-2
DISTANCE versus DRAWDOWN
TH-1, TH-2 and PB-845

46 6013

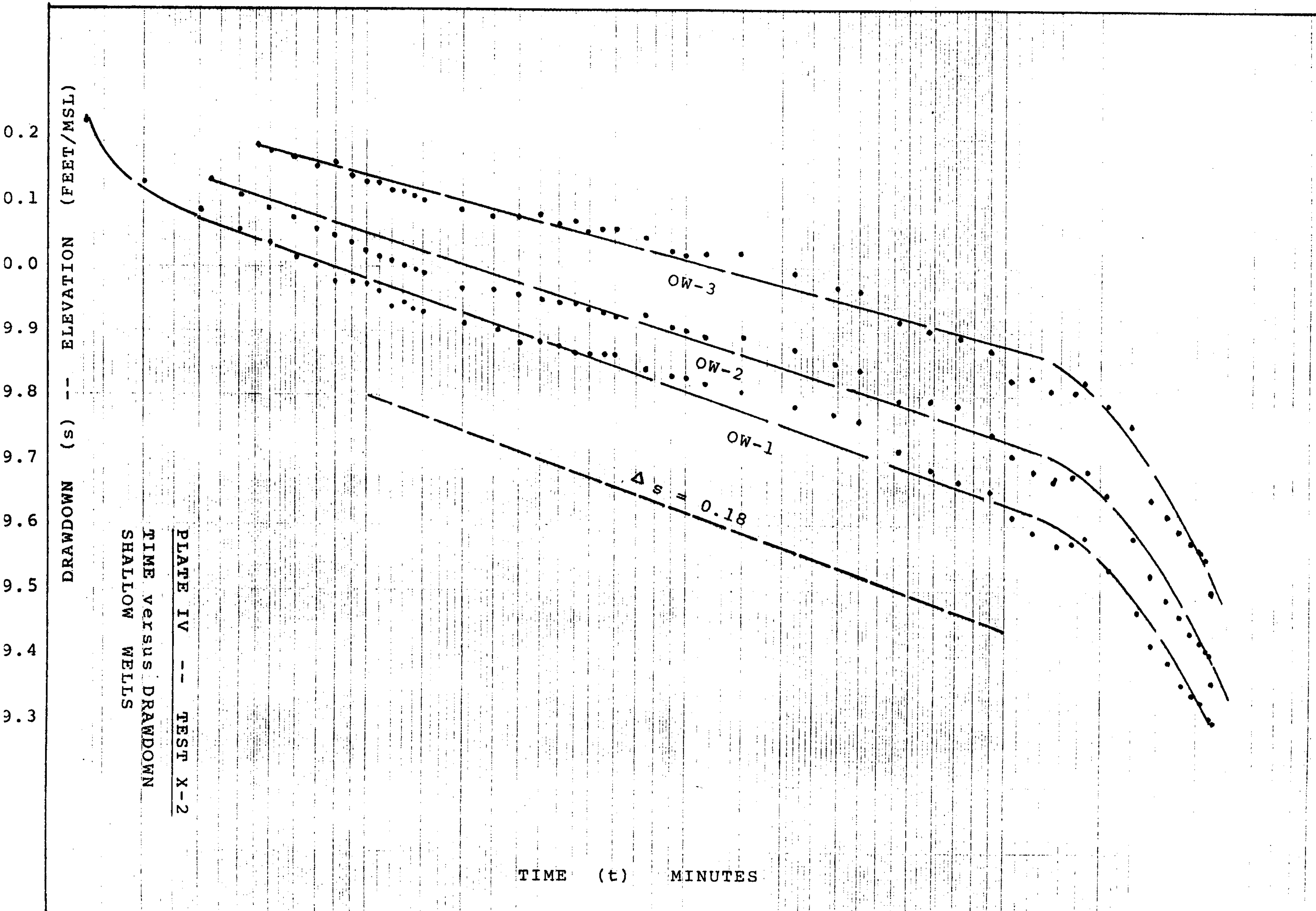


PLATE IV -- TEST X-2
TIME versus DRAWDOWN
SHALLOW WELLS
DRAWDOWN (s) -- ELEVATION (FEET/MSL)
0.2
0.1
0.0
9.9
9.8
9.7
9.6
9.5
9.4
9.3
OW-3
OW-2
OW-1
$\Delta s = 0.18$
TIME (t) MINUTES

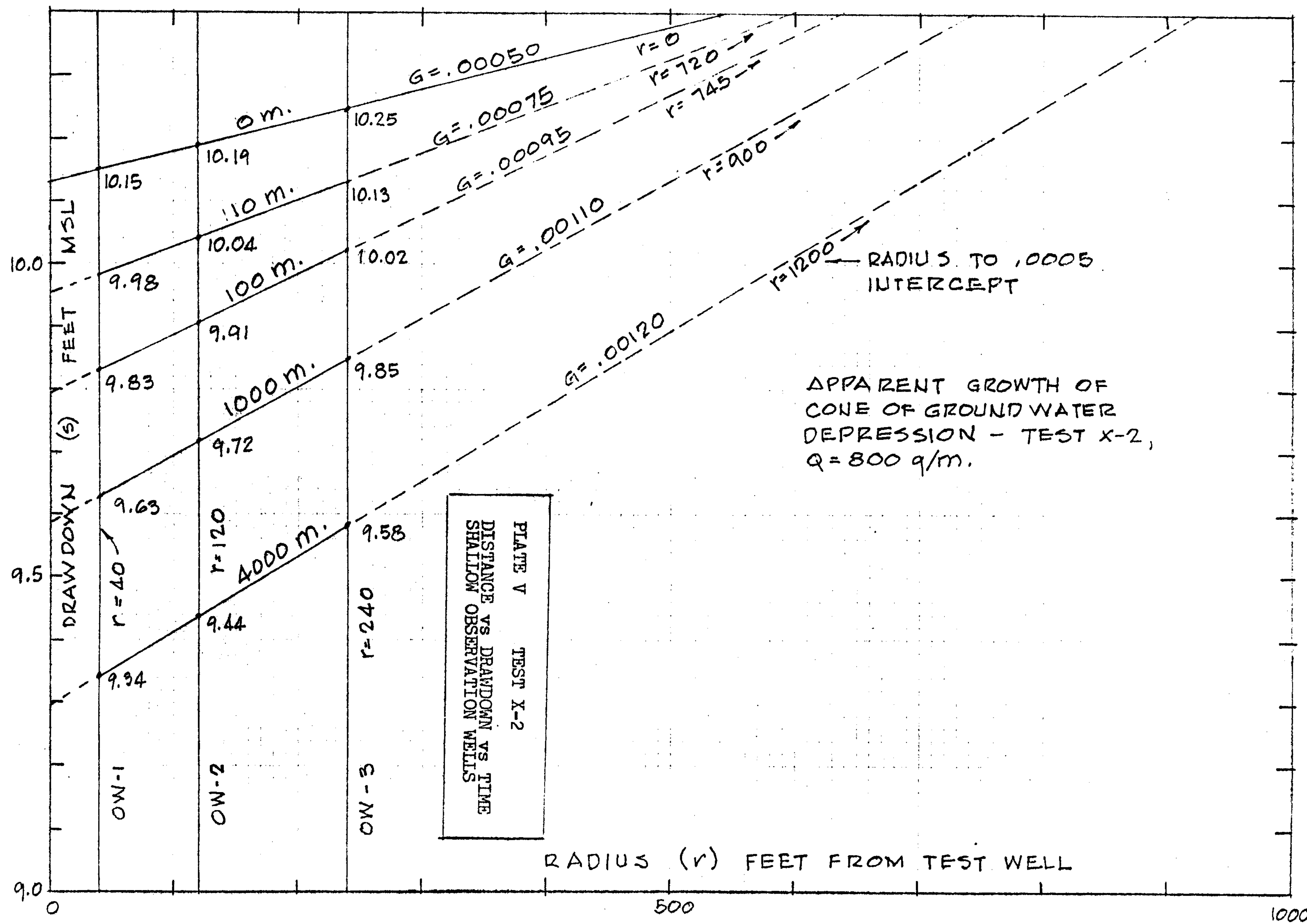


PLATE V TEST X-2
DISTANCE vs DRAWDOWN vs TIME
SHALLOW OBSERVATION WELLS

SEMI-LOGARITHMIC 4 CYCLES X 70 DIVISIONS
KEUFFEL & ESSER CO.

46 6013

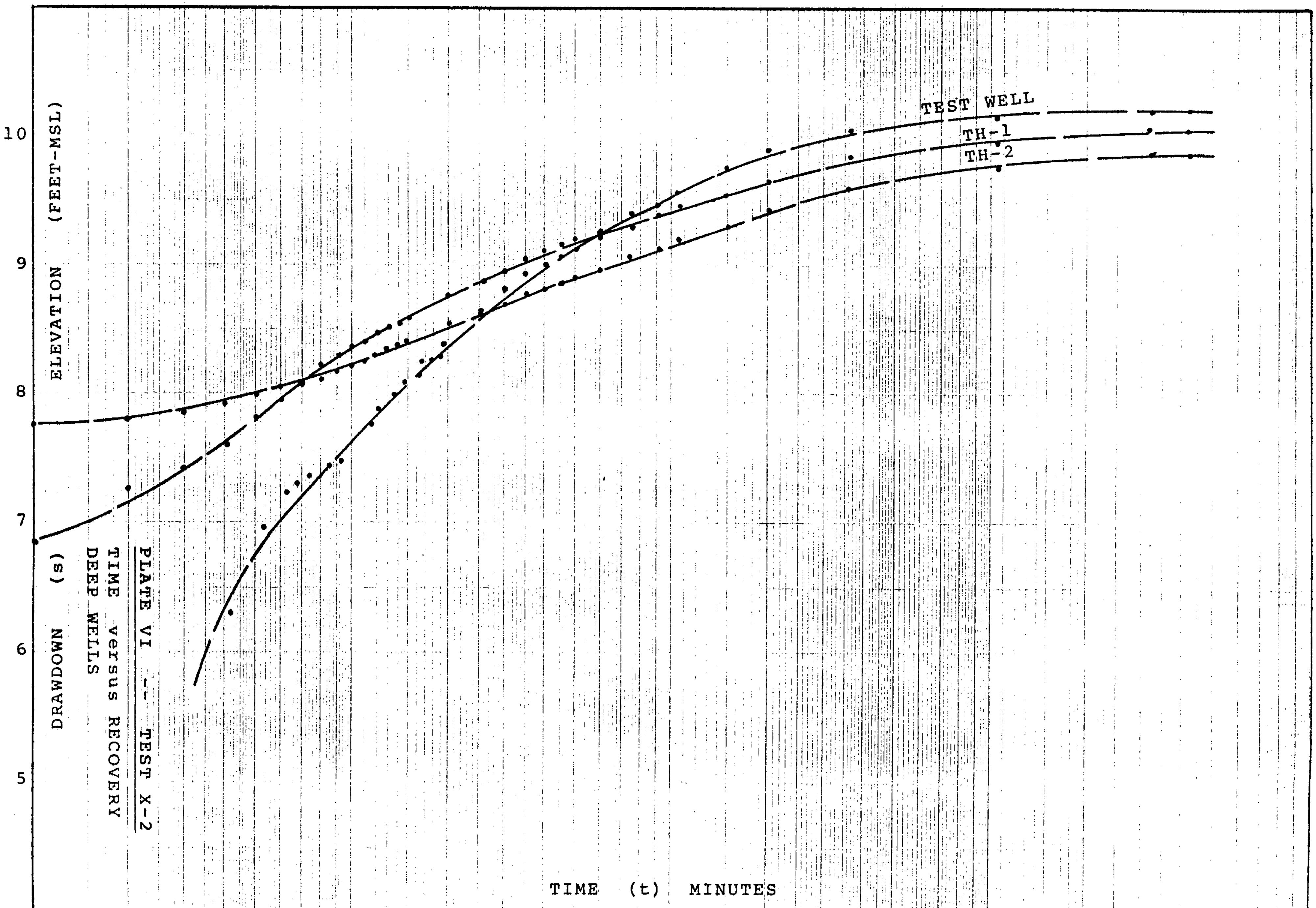


PLATE VI -- TEST X-2

TIME versus RECOVERY
DEEP WELLS

46 6013

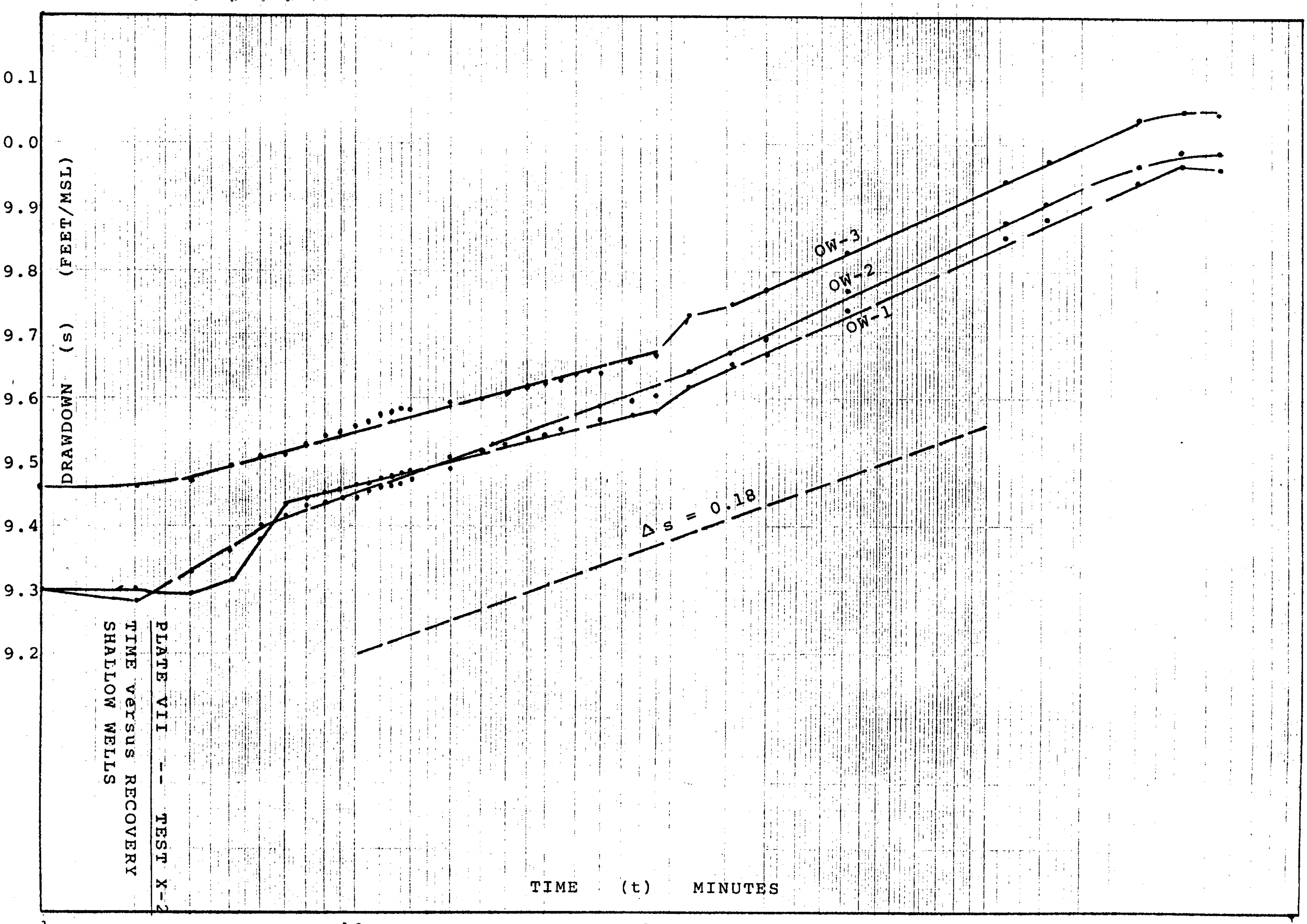


PLATE VII -- TEST X-2
TIME versus RECOVERY
SHALLOW WELLS

46 6013

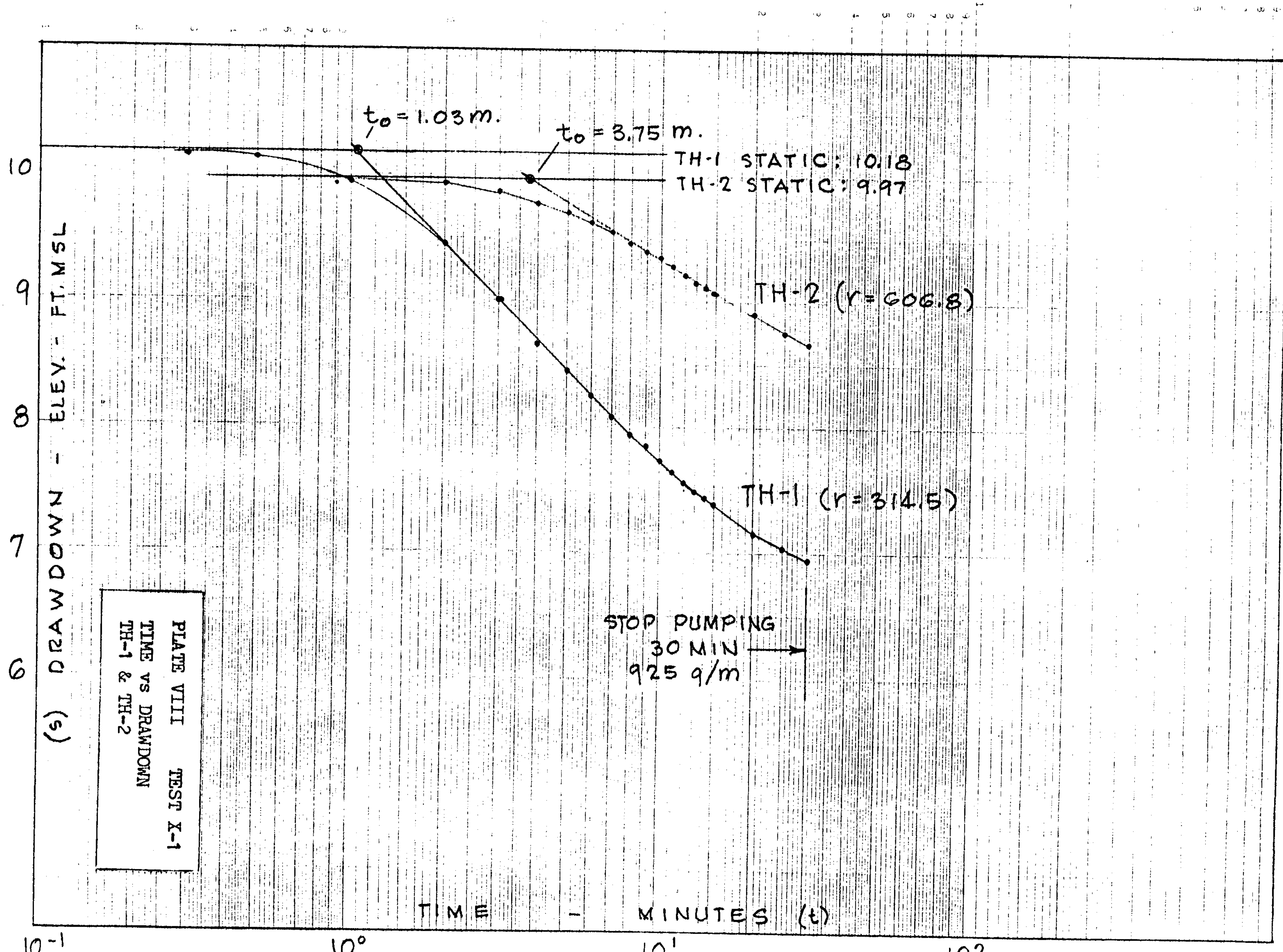


PLATE VIII TEST X-1
TIME vs DRAWDOWN
TH-1 & TH-2

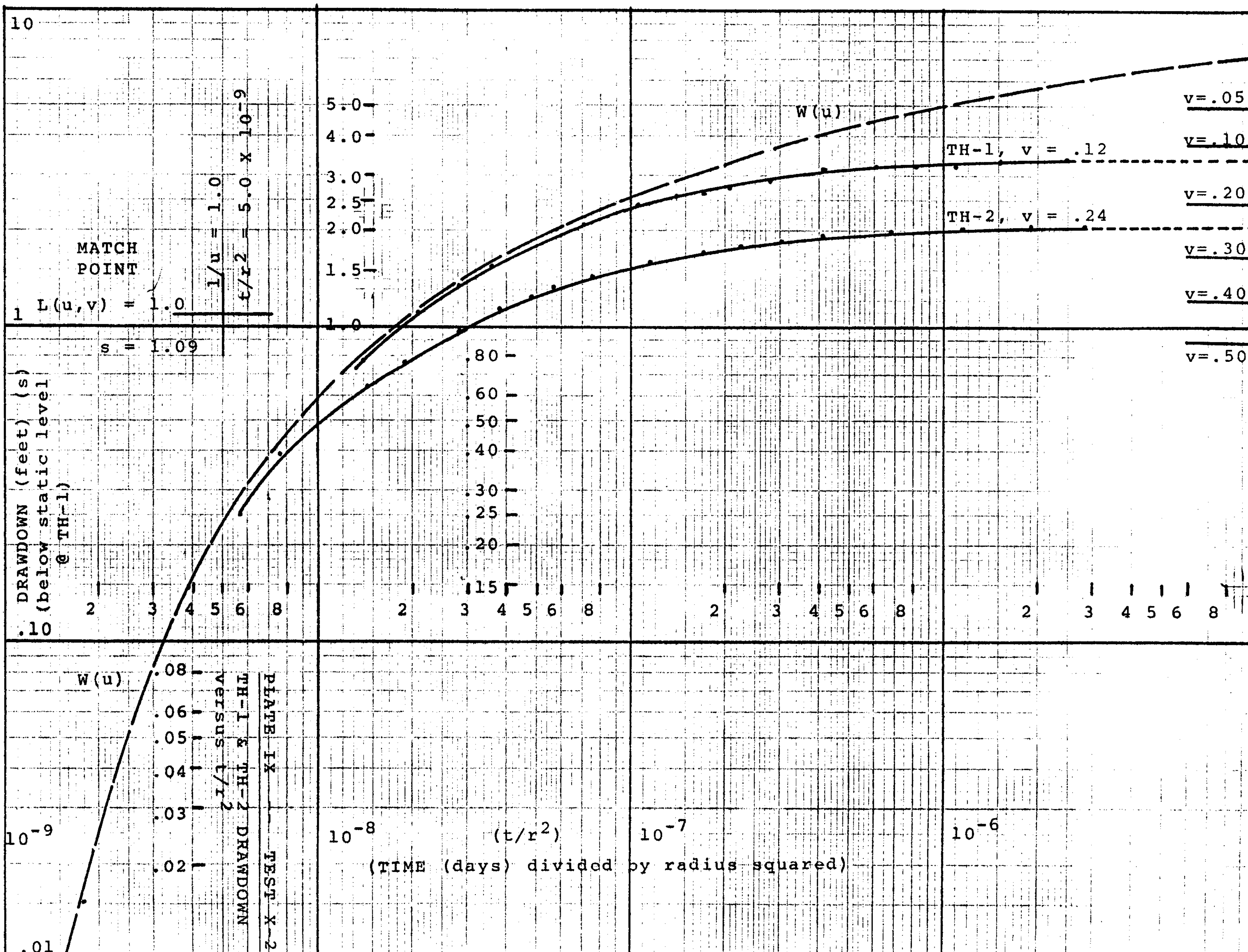
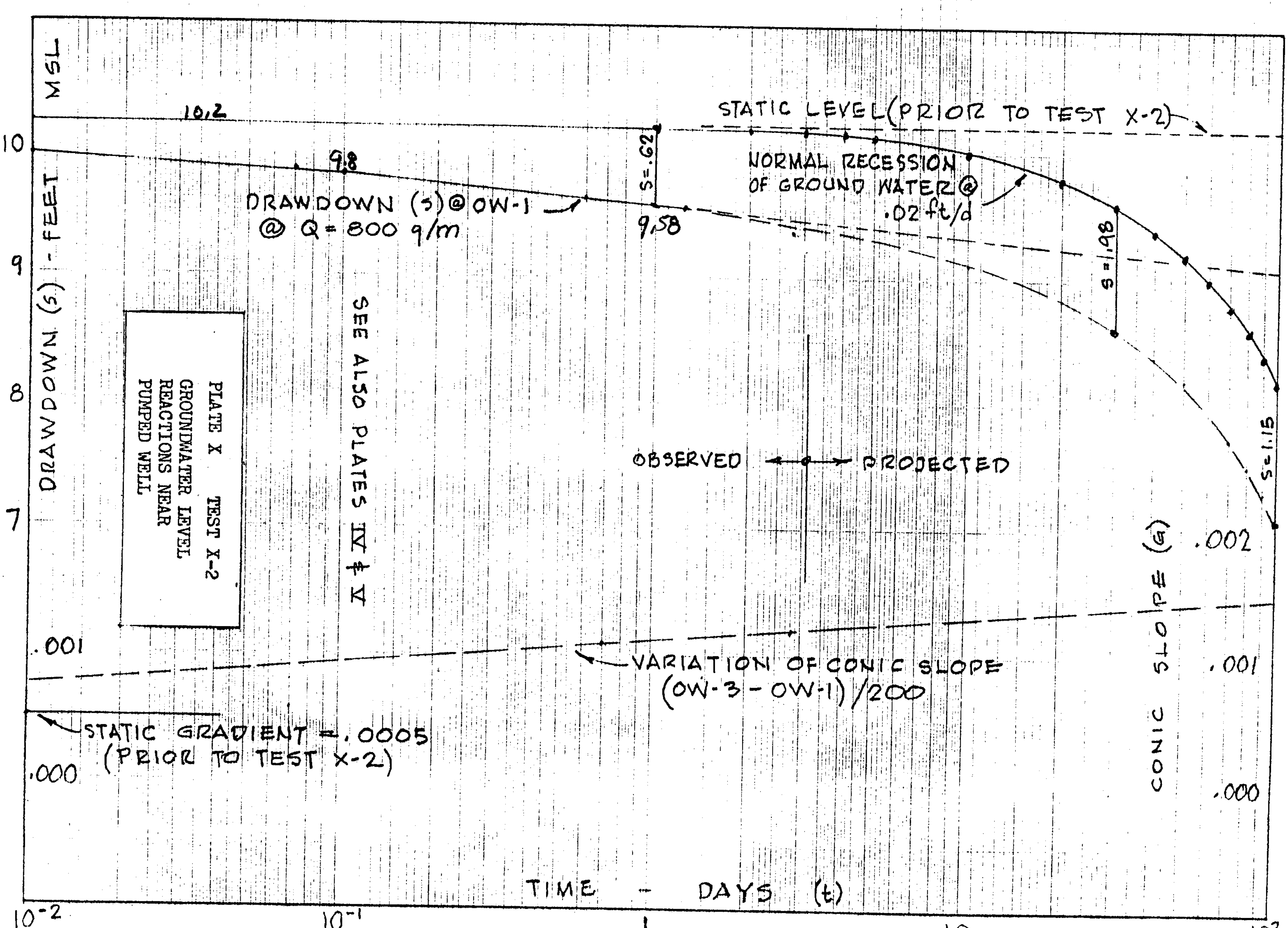


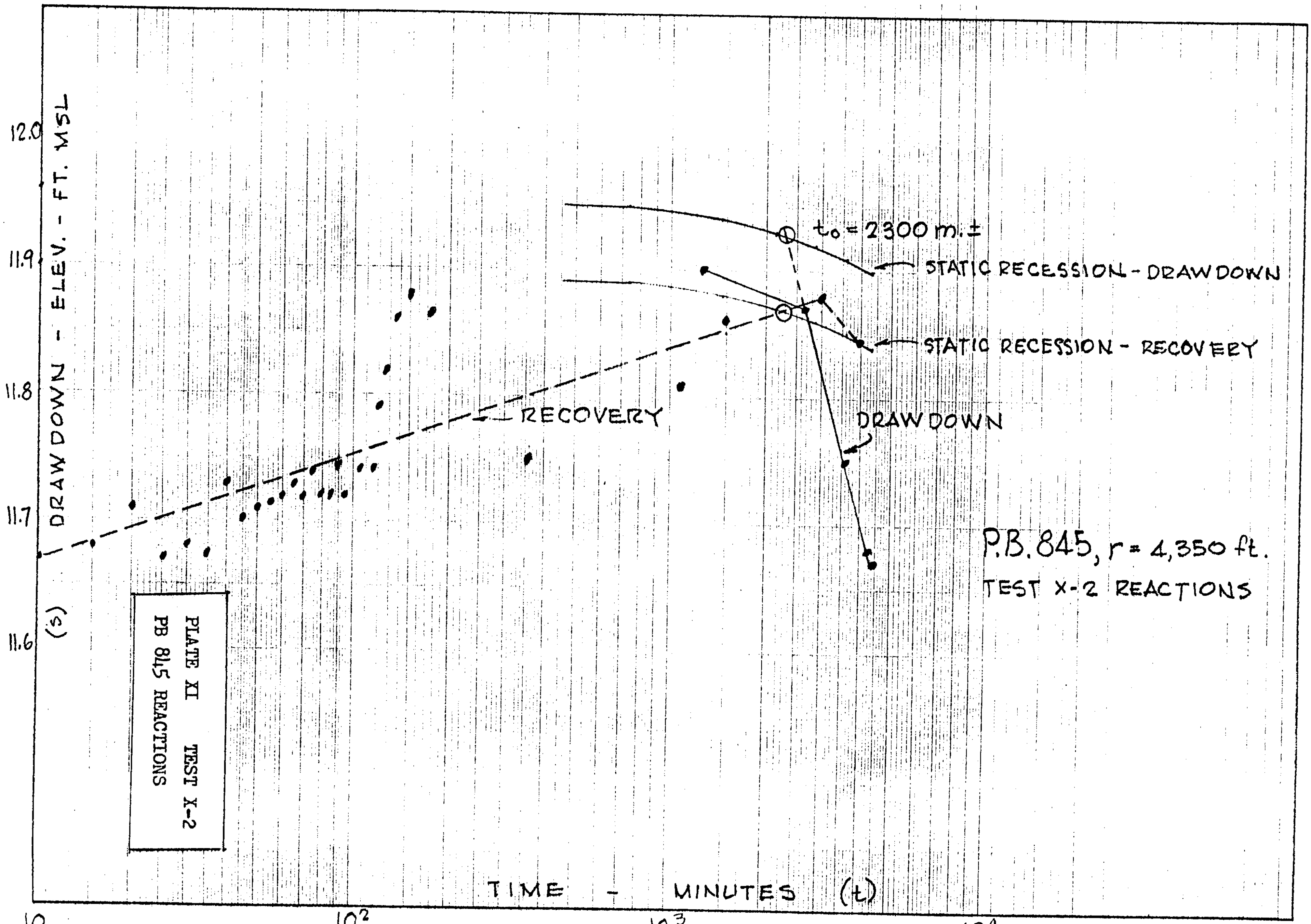
PLATE IX — TEST X-2
TH-1 & TH-2 DRAWDOWN
versus t/r^2

K+E SEMI-LOGARITHMIC 3 CYCLES X 70 DIVISIONS

46 6013



46 6013



12.0
11.9
11.8
11.7
11.6
DRAWDOWN - ELEV. - FT. MSL
(s)
$t_0 = 2300$ m. ±
STATIC RECESSION - DRAWDOWN
STATIC RECESSION - RECOVERY
RECOVERY
DRAWDOWN
P.B. 845, r = 4,350 ft.
TEST X-2 REACTIONS
PLATE XI TEST X-2
PB 845 REACTIONS
TIME - MINUTES (t)
10
10^2
10^3
10^4

VII EVALUATION OF WATER DEMANDS:

Water demands to be placed on the City of Riviera Beach water system are based on the cumulative demands of the service area for a specific time period. These demands may be defined as follows:

A. Fire Protection Water Demand

B. Domestic Water Demand

C. Miscellaneous Water Demand

All of these demands are related to the projected service population, type of consumers, type and density of structures, and rate of growth. The proper consideration of growth and growth rate is important in waterworks analysis because certain elements of the waterworks facility must be provided initially to satisfy anticipated demands for a given number of years without requiring duplication of construction. Otherwise, it may be expected that optimum overall economy will not be achieved. This requires consideration of the following factors:

1. The number of years which the waterworks improvements, including component structures, are to be adequate. This is referred to as, "Period of Design."
2. The number of people to be served and the cumulative water demands generated thereby.
3. The financial capability of the system to fund capital improvements adequate to serve the period of design.

With reference to the period of design, the following criteria are considered for major components of the waterworks facility.

TABLE XIII

PERIOD OF DESIGN

CITY OF RIVIERA BEACH WATERWORKS FACILITIES

1.	Raw Water Supply Facilities	5-10 years
2.	Raw Water Transmission Facilities	15-20 years
3.	Water Treatment Facilities	15-20 years
4.	Treated Water Transmission Facilities	15-20 years
5.	Water Storage Facilities	10-15 years
6.	Water Distribution Facilities	30-40 years

Thus, for the components noted, the water treatment facilities should provide capacity to accomodate estimated future demands for a 15-20 year period without major additions. The treated water transmission system should provide capacity to accommodate future demands for a 15-20 year period without major additions, and water storage facilities to be provided should be adequate for some 10-15 year period.

A. Population Growth & Projections

Service area of the City of Riviera Beach water system has been described in a previous section of this report and the population growth and projections are based on the described area. In February, 1976, the Engineers prepared a "Report & Analysis of Water Resources for the City of Riviera Beach, Florida", as part of an application submitted to the

South Florida Water Management District for additional ground water withdrawal capability. That report reviewed historical population data and statistically analyzed existing and proposed land use within the entire water service area. Data included in that analysis are referred to and utilized in this report.

The past growth pattern of the City proper with respect to the surrounding area is demonstrated in the following summary from the U.S. Decennial Census:

TABLE XIV
POPULATION TRENDS

YEAR	RIVIERA BEACH		PALM BEACH COUNTY	
	POPULATION	%INCREASE	POPULATION	%INCREASE
1930	811		51,781	
1940	1,981	144.2%	79,981	54.4%
1950	4,065	104.9	114,688	43.3%
1960	13,046	211.2	228,106	98.8
1970	21,401	64.0	348,753	52.8

It is noted that the growth rate has been considerably higher in the City each decade than in the balance of the County, where the great majority of the population resides in similar coastal municipalities.

The inference may be drawn that some factors exist in the City of Riviera Beach which induce a higher rate of residential development and resultant population growth than in the remaining portion of Palm Beach County. Without identifying these factors, it is advisable to assume that their combined effect still exists and the condition will probably continue some distance into the future.

During the 1960-1970 decade, the average exponential growth rate within the City computes to be 5.1% per annum. Using a reliably developed population of 28,462 in 1975, the comparable rate appears to have been 5.9% in the first half of the current decade. A survey of new residential units now complete and awaiting occupancy, plus those under construction or permit, indicates only slight abatement of the current impetus before perhaps 1980.

The 1975 Riviera Beach population of 28,462 persons was determined by projecting unit occupancy since the 1970 Decennial Census. This figure correlates well with the 1975 population of 27,872 exponentially interpolated from the "Riviera Beach Comprehensive Development Plan" population projections. Since the Plan considers in great depth the aspects affecting growth, and has proved quite accurate to date, it is accepted verbatim as to population projections used in this Report.

TABLE XV

CITY OF RIVIERA BEACH PRESENT POPULATION PROJECTED BY OCCUPANCY

STUDY AREA	POP'N 1970	HOUSING UNITS 1970	PERSONS/ UNIT 1970	HOUSING UNITS 1975	POP'N 1975
NW	6703	1685	3.98	1958	7793
SW	5330	1476	3.61	1948	7032
NE	3895	1719	2.26	1974	4461
SE	3950	1770	2.23	1739	3878
SI	1523	951	1.60	2492	3987
TOTAL	21401	7601	(2.815)	10111	28462

As noted, population projections of the City are extracted from the Comprehensive Development Plan (CDP). Likewise, those in the unincorporated (Reserve Annexation) area are extracted from projections of the Palm Beach County Area Planning Board. The Town of Palm Beach Shores is not distinctly identified in the APB projection, having been included as part of the Singer Island zone, which includes part of the City of Riviera Beach. A modest growth of 2%/ annum has therefore been assumed for the Town. The CDP projections were chosen over those of APB for the City proper since they were 1) based upon a more rigorous analysis, and 2) developed from more recent data.

For purposes of waterworks evaluation, projected population figures have been developed for the Service area. The first figure is for the year 1986, and is used to determine the relatively short-term growth. The second figure is the theoretical ultimate population based upon land use saturation under existing zoning.

These population projections are summarized as follows:

1) <u>POPULATION - 1975</u>

RIVIERA BEACH	28,850	(C.D.P. PROJ.)
PALM BEACH SHORES	1,600	(TOWN SURVEY)
RESERVE ANNEXATION	2,625	(A.P.B. PROJ.)
SERVICE AREA	33,075	

2) <u>POPULATION - 1986</u>

RIVIERA BEACH	45,360	(C.D.P. PROJ.)
PALM BEACH SHORES	1,920	(20% GROWTH)
RESERVE ANNEXATION	3,200	(A.P.B. PROJ.)
SERVICE AREA	50,480	

3) <u>POPULATION - ULTIMATE MAXIMUM</u>

RIVIERA BEACH	69,503
PALM BEACH SHORES	7,574
RESERVE ANNEXATION	5,701
SERVICE AREA	82,778

Exhibit IV, included in the Appendix of this report, is a graphic representation describing projected population of the water service area. The following table provides in tabular form, the population projections described in Exhibit IV:

TABLE XVI

POPULATION PROJECTIONS
CITY OF RIVIERA BEACH WATER SERVICE AREA

YEAR	POPULATION
1930	811
1940	1,981
1950	4,065
1960	13,406
1970	21,401
1975	33,075
1980	43,000
1985	48,000
1990	55,000
1995	57,500
2000	60,000
ULTIMATE	83,000

B. DESCRIPTION OF WATER DEMANDS

It has been noted that demands placed on a waterworks facility are based on the cumulative demands of the water service area during a specific time period. These cumulative demands are domestic water demand, miscellaneous water demand, and fire protection water demands.

Provision of municipal waterworks facilities to accomodate domestic water demand during fire demand periods is essential, since it is not reasonable to assume that public consumption will cease during fire periods. Domestic demand which includes residential, commercial, and industrial flow should, therefore, be superimposed on fire demands.

1. Domestic Water Demand

There are three different domestic demand rates which are of significance in evaluating waterworks facilities. These are:

a. Average Annual Demand
b. Maximum Day Demand
c. Peak Hourly Demand

a. Average Annual Demand

The average annual demand represents the total quantity of water required on an average annual basis over a time period in excess of one year. The primary significance of this demand rate is in relationship to annual load to be placed on raw water resources and in determining annual revenue. This demand rate has little significance in establishing the size of various waterworks components considered in this report because of the other greater rates of water demand.

b. Maximum Day Demand

The maximum day demand represents a total maximum quantity of domestic and miscellaneous water demand required during a 24 hour period, excluding fire protection demand. The primary significance of this demand rate is in relationship to sizing various waterworks components such as raw water supply capacity, raw water transmission mains capacity, water treatment plant capacity, service pumping capacity, and treated water transmission mains capacity in those instances where adequately sized and properly located water storage facilities are available. Where treated water transmission mains and water storage facilities are not adequately sized and properly located, other components of the waterworks facility should be appropriately increased.

c. Peak Hourly Demand

The peak hourly demand represents the maximum rate of water consumption during a 24 hour period, excluding fire demand. The primary significance of this demand rate is in relationship to sizing certain waterworks components such as service connections and water distribution facilities. In those instances where adequately sized and properly located treated water transmission and water storage facilities are not available, the peak hourly demand will exert influence on the size of all other major components of the waterworks facilities.

2. Miscellaneous Water Demand

Miscellaneous water demand includes water demand created by illicit water usage through non-metered installations, pipe leakage, that amount of water actually used for hydrant flushing, but excluding internal requirements of the water treatment facility. On occasion, miscellaneous water demands, or portion thereof is referred to as unaccounted for water. For a properly operated waterworks facility, this demand should not exceed 10% of the average annual demand.

3. Fire Protection Water Demand

Water requirements for fire defense systems are described in the Grading Schedule for Municipal Fire Protection, published by the Insurance Services Office (I.S.O.).

Requirements for fire flow are based on the population and type of fire hazards district with respect to the following formula. This formula was developed by the National Board of Fire Underwriters but is not included in the I.S.O. publications. The formula is generally accepted for use in determining requirements for fire flows.

$G = 1020 \times \sqrt{P}\,(1.0-0.1\sqrt{P})$ where
G = Required fire flow in gallons per minute
P = Population in thousands

C. Projection of Water Demands

Future water demand estimates have been projected from the current usage rates on the basis of both population growth and land use. Land use is considered to be more reliable basis for projecting ultimate demands at the point of land use saturation. Additionally, it is assumed that commercial/industrial demands will increase at the same rate as population growth. It follows that either basis should produce substantially the same answer for short-term demand projections.

The following table is provided to reflect historical water pumpage data for years 1973-74 and 1974-75 in the City of Riviera Beach water systems:

TABLE XVII

CITY OF RIVIERA BEACH WATER SYSTEM
HISTORICAL WATER PUMPAGE DATA

MONTH YEAR	AVERAGE DAY M.G.D.	MAXIMUM DAY M.G.D.	TOTAL MONTH B.G.	PERCENT OF ANNUAL
FY 73-74	(5.465)			
OCT	4.91	6.10	.1522	7.6
NOV	5.34	6.00	.1602	8.0
DEC	4.91	6.56	.1522	7.6
JAN	4.58	5.14	.1420	7.1
FEB	5.54	6.49	.1551	7.8
MAR	6.51	7.24	.2018	10.1
APR	6.57	7.59	.1971	9.9
MAY	6.06	7.68	.1880	9.4
JUN	4.45	7.01	.1335	6.7
JUL	5.20	6.68	.1612	8.1
AUG	5.64	7.42	.1748	8.8
SEP	5.87	7.86*	.1761	8.9
TOTAL			1.9942	100.0
FY 74-75	(5.588)			
OCT	4.98	6.16	.1544	7.6
NOV	5.25	6.31	.1575	7.7
DEC	5.12	5.85	.1587	7.8
JAN	5.56	6.48	.1724	8.5
FEB	5.60	6.46	.1568	7.7
MAR	6.21	7.07	.1925	9.4
APR	6.18	7.37	.1854	9.1
MAY	5.19	6.82	.1609	7.9
JUN	4.85	6.05	.1455	7.1
JUL	5.21	7.36	.1615	7.9
AUG	6.80	7.56*	.2108	10.3
SEP	6.10	7.44	.1803	9.0
TOTAL			2.0394	100.0

Based upon a historical average day per-capita consumption of 174 gallons per day and with parameters previously discussed, the future water demands are estimated as follows:

TABLE XVIII
CITY OF RIVIERA BEACH WATER SYSTEM
PROJECTED AVERAGE WATER DEMAND

LOCALITY	AVERAGE DEMAND (M.G.D.) 1975	ULTIMATE
RIVIERA BEACH	4.706	9.828
PALM BEACH SHORES	0.581	1.280
RESERVE ANNEXATION	0.466	2.012
PEANUT ISLAND	0.001	0.104
ENTIRE SERVICE AREA	5.754	13.224

The only area of serious doubt in the water demand projection is in the ultimate demand in the Reserve Annexation Area. It is not only possible, but quite likely, that a substantial portion of the land now zoned for Agriculture could be rezoned to a higher use, thus creating an ultimate demand much higher than 2.012 M.G.D. as computed. However, based on data available at this time, the estimated average water demand will be 8.35 M.G.D. in 1985, and ultimately approximately 14.5 M.G.D.

A review of the City of Riviera Beach water system follows:

TABLE XIX

CITY OF RIVIERA BEACH WATER SYSTEM
WATER DEMAND PEAKING FACTORS

	RATE	EXPERIENCED	EXPECTED
MAXIMUM	HOUR	1.95	2.25
	DAY	1.42	1.60
	WEEK	1.25	1.40
	MONTH	1.20	1.35
	QUARTER	1.08	1.15
MINIMUM	DAY	0.72	0.67
	WEEK	0.77	0.75
	MONTH	0.83	0.80
	QUARTER	0.93	0.90

It must be noted that the "experience" factors apply only to a particular year, and as applied to the annual average for that year. Discussion with waterworks operating personnel discloses that on "peak" days, operating pressures intentionally have been allowed to drop so as limit the depletion of storage. This in turn has had some effect on the maximum/average day ratio, which would likely have approached 1.6 without interference. Continuation of this practice cannot be depended upon, and is not considered in the projections.

Although domestic irrigation has a strong affect on a particular day, the total effect is substantially absorbed in the annual average. Comparison of maximum/minimum weeks (1.25/.77 - 1.62) indicated minimal domestic usage varying from 77% of total on the average, to as low as 38% during period when irrigation and other seasonal demands drives consumption upward. By comparing 1975 (with fairly normal rainfall) to 1974 (a relatively dry year), and after adjusting for growth, it would appear that a "dry" year increases annual consumption by something in the order of only 2 to 3 percent. This is probably due to the fact that each year has a characteristic dry season of about 6 months, during which period irrigation is near maximum regardless of the total rainfall that particular year.

Using current unsurance underwriters policies and grading schedules, the quantity of water demanded be a maximum fure must be superimposed on maximum day demand previously estimated at 1.6 x average. This has been noted among other factors to be influenced by population.

For the City of Riviera Beach water system, the following table described estimate fire protection demand requirements with respect to time:

TABLE XX

CITY OF RIVIERA BEACH WATER SYSTEM
FIRE PROTECTION DEMAND

YEAR	POPULATION	FIRE FLOW	DURATION
1975	33,075	5528 gpm	10 hrs.
1980	43,000	6249 gpm	10 hrs.
1985	48,000	6576 gpm	10 hrs.
1990	55,000	7003 gpm	10 hrs.
1995	57,500	7149 gpm	10 hrs.
2000	60,000	7287 gpm	10 hrs.
Ultimate	83,000	8445 gpm	10 hrs.

The following table summarizes projected domestic and miscellaneous water demands estimated to be imposed on the City of Riviera Beach water system in subsequent years.

TABLE XXI

CITY OF RIVIERA BEACH WATER SYSTEM
WATER DEMAND SUMMARY

		WATER DEMAND RATES			
YEAR	POPULATION	AVG. DAY MGD.	MAX. DAY MGD.	MAX. HOUR MGD.	FIRE GPM.
1930	811	-	-		-
1940	1,981	-	-		-
1950	4,065	-	-		-
1960	13,046	-	-		-
1970	21,401	-	-		-
1975	33,075	5.754	9.206	12.947	5528
1980	43,000	7.482	11.971	16.835	6249
1985	48,000	8.352	13.362	18.792	6576
1990	55,000	9.570	15.312	21.533	7003
1995	57,500	10.005	16.008	22.511	7149
2000	60,000	10.440	16.704	23.490	7287
Ultimate	83,000	14,442	23.107	32.495	8445

IX. EVALUATION OF REQUIRED WATERWORKS FACILITIES:

Previous sections of this report have described the condition and capacity of existing waterworks facilities of the City of Riviera Beach, together with projected future water demands of the service area. It is the purpose of this section of the report to evaluate these data in terms of future waterworks facilities requirements and to determine the additions and improvements required in order to meet projected demands.

A. Source of Supply

1. Primary Facilities

Design period recommended for provision of a water supply source has been noted to be not less than five years following initial operation. Total maximum day and fire demand requirements projected for the year 1990 is 15.3 M.G.D. Projected average annual water demand for the same time period is 9.6 M.G.D. This primary supply would be derived from two sources including existing ground water supply facilities, together with future additions made in a proposed westerly wellfield constructed in or adjacent to the highly porous cavity sandstone aquifer in the reserve annexation area of the City. In this regard, it is to be noted that while the existing source of supply has a peak delivery capacity of approximately 10 M.G.D., it is necessary that this withdrawal be substantially reduced if salt water migration into the present wellfield is to be properly deterred. Prior studies completed by the U.S. Geological Survey have indicated the magnitude of the reduced withdrawal should be in the range of 50% of present levels based on average annual demand, thereby reducing the annual safe yield from the current average withdrawal rate of approximately 6.0 M.G.D. to 3.0 M.G.D. with a maximum day withdrawal limited to approximately 5.0 M.G.D. This would require the development of an additional 8.0 M.G.D. capacity from the new westerly well-field in order to meet projected maximum day water demands in the year 1985.

Preliminary data developed from separate studies indicate that a properly constructed and developed water supply well penetrating the cavity sandstone aquifer may be expected to yield 750 gpm to 1000 gpm, the specific yield depending on characteristics of each well. Pending completion of detailed hydrologic and geologic studies in the proposed westerly wellfield area, it is recommended that the anticipated yield for each well be limited to approximately 750 g.p.m., or 1.0 M.G.D. On this basis, initial construction in the proposed new westerly well field would include 8 wells, each twelve inches diameter, and constructed to a depth of approximately 125 feet. Each well would be provided with an electrically driven pump and equipped with right angle gear drive, together with housing and individual auxilliary power facilities. Exhibit VII including in the appendix of this report, is a map generally defining the area anticipated for construction of the new raw water supply facilities. As noted previously, it is anticipated this initial wellfield construction would provide adequate capacity thru the year 1985. At approximately that time, additional wells would be added to accomodate raw water demands for future years.

2. Auxilliary Facilities

A previous section of this report has discussed the alternative of providing auxilliary raw water supply facilities to act as an emergency backup for the primary water supply source. The raw surface water supply of the adjacent city of West Palm Beach has been described as the auxilliary or backup raw water source for the City of Riviera Beach water system. This report has previously noted the reasons that the surface supply is not recommended as the primary source.

Exhibit VII, included in the appendix of this report, describes the location of the required auxilliary supply intake structure, and auxilliary raw water transmission main which will interconnect with the primary raw water transmission main at the southerly limits of the City of Riviera Beach facilities to be provided for the auxilliary raw water supply will include the intake structure, two 10,000 gpm each electric motor driven raw water pumps with variable output, and auxilliary power facilities. Fiscal limitations of the City at the present time indicate the need to defer this construction for future years.

SYMBOLS AND DIMENSIONS

SYMBOL	DIMENSIONS	MEANING
A	ft^2	Area
C	---	Constant
G	(ft/ft)*	Linear gradient
H	ft	Static thickness of aquifer
K	ft/d	Hydraulic Conductivity
L	C/d	Leakance
P	$g/ft^2/d$	Permeability
Q	ft^3/d	Hydraulic stress
R	ft	Radius of influence
S	$(ft3/ft^3)$*	Coefficient of Storage
T**	ft^2/d	Transmissivity
T**	g/ft/d	Transmissibility
V	ft^3	Volume
W(u)	(integral)	Well function of u
d	---	Differential
h	ft	Stressed thickness of aquifer
r	ft	Radius, general
s	ft	Drawdown
t	d	Time (days)
v	ft/d	Velocity
x,y,z	ft	Coordinate components
Δ	ft/cycle	Log_{10} Slope
/	---	Per, divided
ft	---	Foot (feet)
ft	---	Square feet
ft	---	Cubic feet
g	$ft^3/7.48$	Gallons (s)
m	t/1440	Minute (s)
d	1440 M	Day(s)

* Usual units, otherwise dimensionless.

** Identified in Text.

APPENDIX B

DEFINITIONS

AQUIFER: Geologic formations or strata in the saturated zone from which ground water can be obtained through wells at a sufficient rate as to provide a beneficial and practical source of water supply; in this instance, for public use.

COEFFICIENT OF STORAGE (S): The volume of water released from or taken into storage per unit of surface area per unit change in depth of ground water; the same as the "specific yield" of the material dewatered during pumping from a homogenous (water table) aquifer.

CONE OF DEPRESSION: The shape of the surface of the ground water or piezometric surface surrounding a pumped well.

DELTA"s" (Δs = ft): The "logarithmic slope" of time-drawdown or distance-drawdown curves, equal to the change in value of "s" (drawdown) between any two values of time or distance whose ratio is 10. For a given set of conditions, the value Δs_d (distance) = 2 Δs_t (time).

APPENDIX B

DEPTH OF AQUIFER (H=ft): The saturated thickness of the unstressed aquifer in feet (before pumping). The symbol "h" is used to denote the stressed thickness, measured at some point in the radius of influence, such as at an observation well. Thus, H-h = s.

DRAWDOWN (s=ft): The recession of the water level in feet during period of stress (pumping) measured at some particular time and point within the radius of influence.

LEAKANCE (L): The "leakance constant", which is the hydraulic conductivity of the confining bed of an artesian aquifer, divided by its thickness. Thus, K/b = L.

HYDRAULIC CONDUCTIVITY (K): The unit rate of discharge of a particular formation in an aquifer, expressed in cubic feet per square foot per day (which reduces to feet per day) under a hydraulic gradient of unity (100% or 1.00).

HYDRAULIC GRADIENT (G): The slope (dz/dx) of the ground water or piezometric surface within an area of a static aquifer or at some given radius on axial lines within a cone of depression.

HYDRAULIC STRESS (Q): The pumping rate employed during a pumping test, or the theoretical well yield for a given set of dimensional conditions.

PIEZOMETRIC SURFACE: The "pressure head" level of ground water in a confined or "artesian" aquifer; similar to the ground water level in an unconfined or "homogenous" aquifer; synonomous with potentiometric surface.

RADIUS OF INFLUENCE (R=ft): The distance (radius) of the outer edge of the cone of depression, measured from the pumped well after some period of stress. The symbol "r" is used to denote the radius of some reference point where drawdown is determined within the cone of depression.

TIME (t=d): Time of applied stress or recovery during pumpage tests, expressed in days. The symbol "m" is used to denote time expressed in minutes. For example, 3 hours: 36 minutes = 216 m/1440 = t = 0.15 (days).

TRANSMISSIBILITY (T = g/ft/d): Transmissivity expressed in gallons per foot per day.

TRANSMISSIVITY (T = ft^2/d): The rate of discharge of an aquifer (or zone of an aquifer), expressed in cubic feet per foot per day (which reduces to square feet per day) under a hydraulic gradient of unity (100% or 1.00).

VOLUME: (V=ft^3): The product of xyz, expressed in cubic feet. Also, 7.48 V = g(gallons).

PERMEABILITY ($P=g/ft^2/d$): Hydraulic conductivity expressed in gallons per square foot per day ($P=7.48\ K$).

RADIUS OF INFLUENCE ($R=ft$): The distance (radius) of the outer edge of the cone of depression, measured from the pumped well after some period of stress. The symbol "r" is used to denote the radius of some reference point where drawdown is determined within the cone of depression.

TIME ($t=d$): Time of applied stress or recovery during pumpage tests, expressed in days. The symbol "m" is used to denote time expressed in minutes. For example, 3 hours: 36 minutes = 216 m/1440 = t = 0.15 (days).

TRANSMISSIBILITY ($T = g/ft/d$): Transmissivity expressed in gallons per foot per day.

TRANSMISSIVITY ($T = ft^2/d$): The rate of discharge of an aquifer (or zone of an aquifer), expressed in cubic feet per foot per day (which reduces to square feet per day) under a hydraulic gradient of unity (100% or 1.00).

VOLUME: ($V=ft^3$): The product of xyz, expressed in cubic feet. Also, $7.48\ V = g$(gallons).