WATER TABLE AND RAINFALL CORRELATIONS

RAINFALL AND THE WATER TABLE IN MUCK OF THE EVERGLADES

Figure 48 shows rises of the water table in observation well G 72 correlated with hourly precipitation data from a recording rain gage located about 1.5 miles to the south. Well G 72 is near the Dade-Broward County line on the east side of State Highway 25. This well is in the glades about 12 miles west of the coastal ridge and 20 miles west of the Atlantic Ocean. The well is 4.6 ft deep and the land surface is 6.0 ft above U. S. Coast and Geodetic Survey mean sea level datum. The upper 3 ft consists of muck overlying hard permeable limestone.

Figure 48 shows a 2.25-ft rise of the water table following 4.50 in. (0.375 ft) of rainfall between 12 m. and 6 p. m. on April 18, 1943. On this date only a fraction of an inch of rainfall was recorded by gages at, and south of, Miami. Therefore, this was probably a local shower having a high intensity over only a relatively small

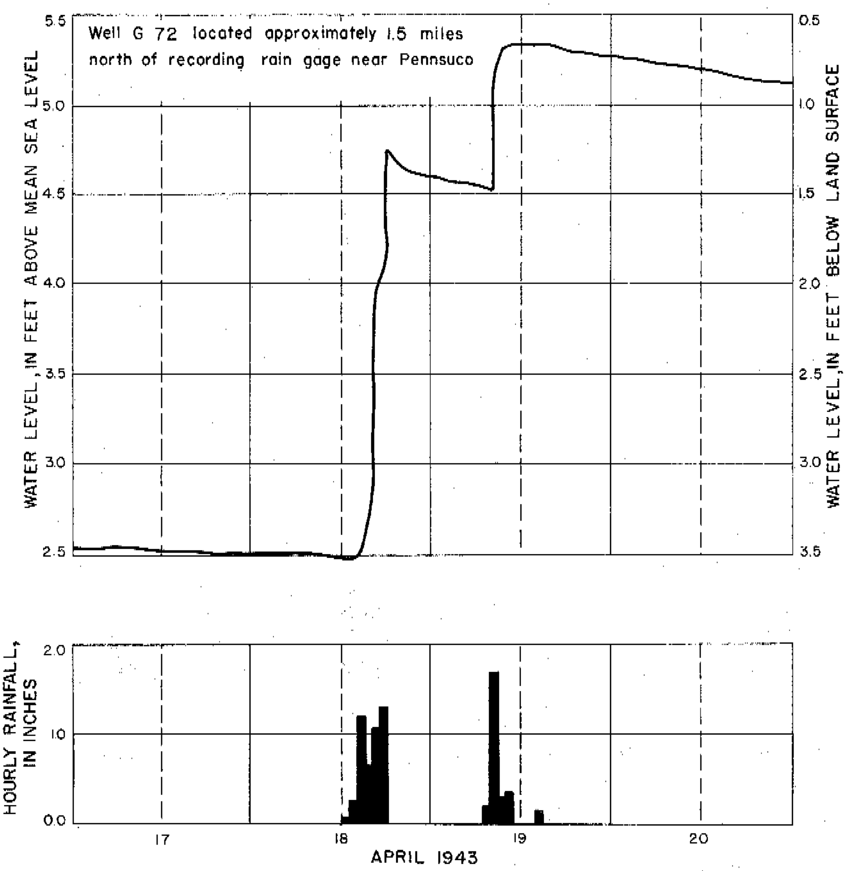

Figure 48. —Graph showing fluctuations of water level in well G 72 following periods of heavy rainfall.

area, and the rainfall recorded by the Pennsuco gage may even have been appreciably different from the actual rainfall at the well 1.5 miles distant. Another rainfall of 2.70 in. (0.225 ft) between 7 a.m. and 11 a.m. the following day, caused a rise of the water table of 0.82 ft. These figures give a value of approximately 0.27 for the ratio between precipitation and the water-level rise in the muck between depths of 0.65 and 1.47 ft below land surface.

RAINFALL AND THE WATER TABLE IN SAND OF THE ATLANTIC COASTAL RIDGE

Figure 49 shows the rise of the water level in well G 86 on April 16 and 17, 1942, following very heavy rainfall. This well is 0.4

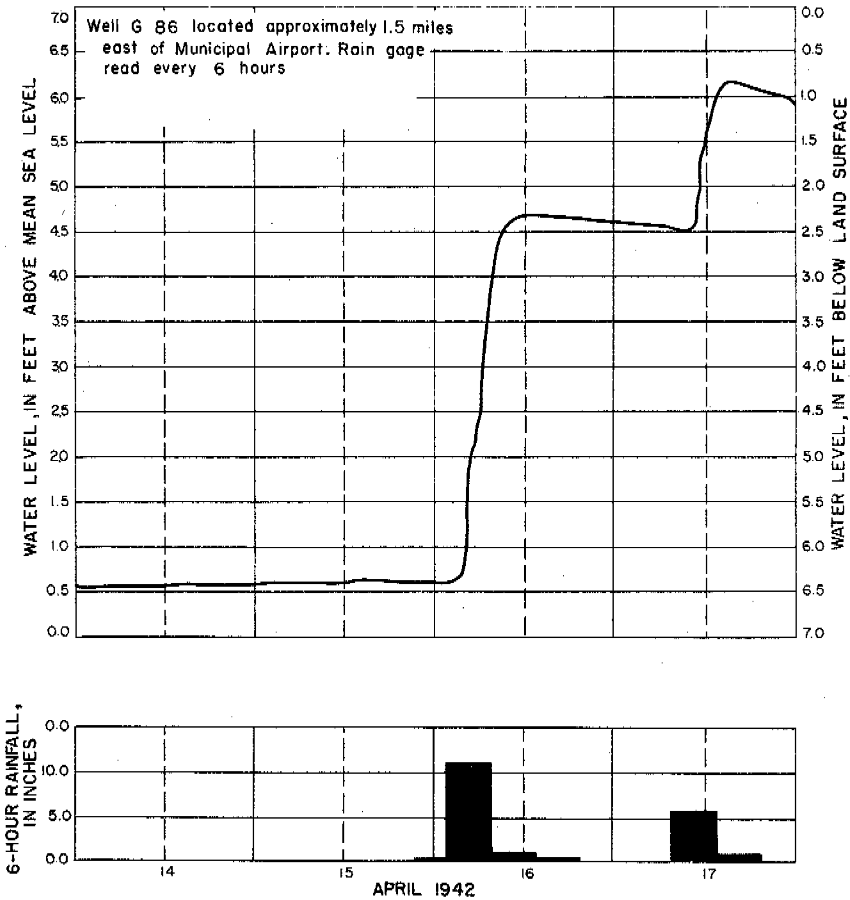

Figure 49. —Graph showing fluctuations of water level in well G 86 following periods of heavy rainfall.

mile north of Little River Canal on the east side of NW. 27th Avenue, and 1.5 miles southeast of NW. 111th Street and LeJeune Road, where the Miami Airport rain gage was located (it has since been moved to the NW. 36th Street Airport). During the morning of April 16, 1942, 12.16 in. of rain caused a 4.1-ft rise in the water table. This well is only 9 ft deep and penetrates fine to medium quartz sand. Previous to April 16, only 0.10 in. of rain had fallen in the preceding 15 days; therefore the sand above the capillary fringe probably contained little moisture. Assuming that the water table in the area rose about the same amount as in well G 86, that the rainfall at the well was the same as at the rain gage, that the 2 ft of sand above the water table at the end of the first rise retained an estimated 2/3 in. of rainfall, and that the moisture held in the capillary fringe is the same in both positions of the water table, then the approximate value for the ratio between recharge (water reaching the water table) and the water-level rise due to increased storage in the aquifer, between the depths of 2.4 and 6.5 ft below land surface, is calculated as 0.23. This ratio is somewhat higher than the specific yield because a part of the water would be retained and would not be released from storage under gravity drainage; also, it is somewhat less than the porosity because the sand probably contained a small amount of moisture before the rain fell and because appreciable amounts of air may have become trapped, thus preventing water from completely filling the void spaces.

A second rainfall in the area, 6.64 in. on April 17, 1942, between 9 a. m. and 3 p. m., caused a 1.65-ft rise of the water table. At the peak of this rise the water table was within 0.85 ft of the land surface. Assuming that the soil above the water table had been thoroughly moistened by the heavy rains that ended less than 12 hours previously, and that all the rainfall infiltrated to the water table, a value of 0.34 for the ratio between recharge and water-table rise was calculated for the formation between the depths of 0.85 and 2.50 ft below land surface.

The area surrounding the well is level and sandy, and it is believed that the water table did not get above the surface in the area. After the rain ceased the water table began to drop at the rate of 0.6 ft per day. Probably one of the greatest sources of error in the above line of reasoning is that for individual storms the rainfall in the area near the well may be appreciably different from that recorded by the gage, 1.5 miles away. As the period of observation is lengthened, however, the total precipitation recorded by the rain gage becomes a more reliable measure of total precipitation at the well.

RAINFALL AND THE WATER TABLE IN THE OOLITE OF THE ATLANTIC COASTAL RIDGE

Figure 50 shows rises of the water level in response to rainfall at well S 182, near Peters. No recording rain gages were maintained near this well. A standard U. S. Weather Bureau rain gage at Peters, which is read daily, is about 0.5 mile east of well S 182.

On September 15-16, 1945, the water level in well S 182 rose 2.54 ft, and on these same dates 6.95 in. of rainfall was recorded at Peters. The 10 previous days were without rain, and it is estimated that the 4.5 ft of material above the water table absorbed about 1 in. of water from this rainfall. The remainder of the rain, if it is assumed to have caused the rise in water levels, would give a ratio between recharge and water-table rise of about 0.19 on these dates.

All the wells shown in figures 48, 49, and 50 are typical wells of the Miami area, and their water-level behavior indicates that a large percentage of the rain that falls in heavy showers or storms reaches the water table in a very short time.

The amount of rainfall that becomes recharge after any particular storm depends on many factors, most important of which are: the infiltration capacity (ability of water to percolate from the surface

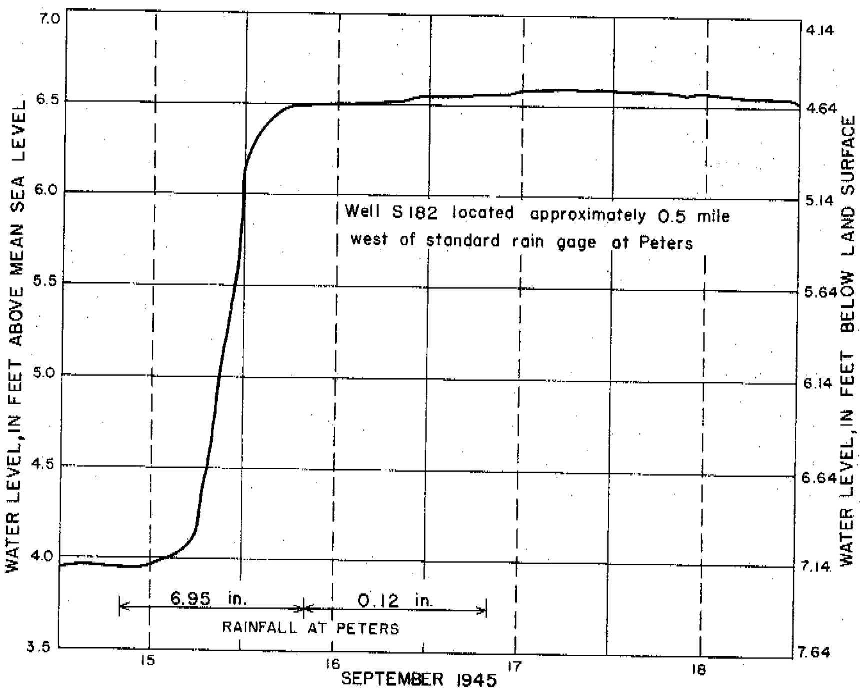

Figure 50. —Graph showing fluctuation of water level in well S 182 following periods of heavy rainfall.

to the water table), the porosity and the specific retention of the soil and rock above the water table; the intensity and duration of the rainfall; and the antecedent weather conditions, which may result in dry, moist, or wet materials above the water table.

Under certain conditions it is possible to calculate approximately the amount of recharge reaching the water table. The following conditions are important: The area should have a high infiltration capacity, the land surface should be nearly level, and the water table should have a gentle slope and should not be unduly influenced by nearby canals (thus, to limit ground-water discharge during the storm and to make an estimation of discharge possible with a fair degree of accuracy).

It is necessary, of course, to have fairly reliable values for the ratio between recharge and the water-table rise for that part of the aquifer over which the water table fluctuates. Wells S 182, near Peters, and S 196, 3 miles north-northwest of Homestead at the University of Florida Sub-Tropical Experiment Station, are in areas that appear to satisfy these conditions. Well S 182 is equipped with an automatic water-stage recorder, and a rain gage is 0.5 mile east, near the intersection of U. S. Highway No. 1 and Quail Roost Drive. Well S 196 has a float gage that is read daily; the rain gage is about 100 ft east of the well.

From field work done during the early part of this investigation, Cross (Cross, Love, Parker, and Wallace, 1940, p. 80) estimated the specific yield of the aquifer in the vicinity of Hialeah to be 0.22 and estimated the specific yield near Opa Locka to be 0.15.

Relating amounts of recharge to the response of the water table in well S 196, a ratio of about 0.15 for the Miami oolite was determined by the writers. When a shower followed a dry period, recharge was calculated as precipitation minus 0.25 in. for each foot of rock above the water table at the end of a rise, thus allowing for water retained by the rock. Recharge was assumed to equal precipitation if heavy showers occurred on succeeding days when the rise caused by the second shower could be correlated with the corresponding rainfall.

Examination of the Miami oolite in borrow pits and canal bank cuts shows that the porosity and permeability of the formation change considerably in relatively short distances, both horizontally and vertically, but even with these changes it is believed that values of 0.15 to 0.20 are the best average figures to use in relating recharge to water-table rises for these parts of the coastal area.

Figure 51 shows the annual sum of the individual rises of the water table in well S 182 during the calendar years 1943–46, plotted against annual precipitation at Peters, about 0.5 mile to the east. Each

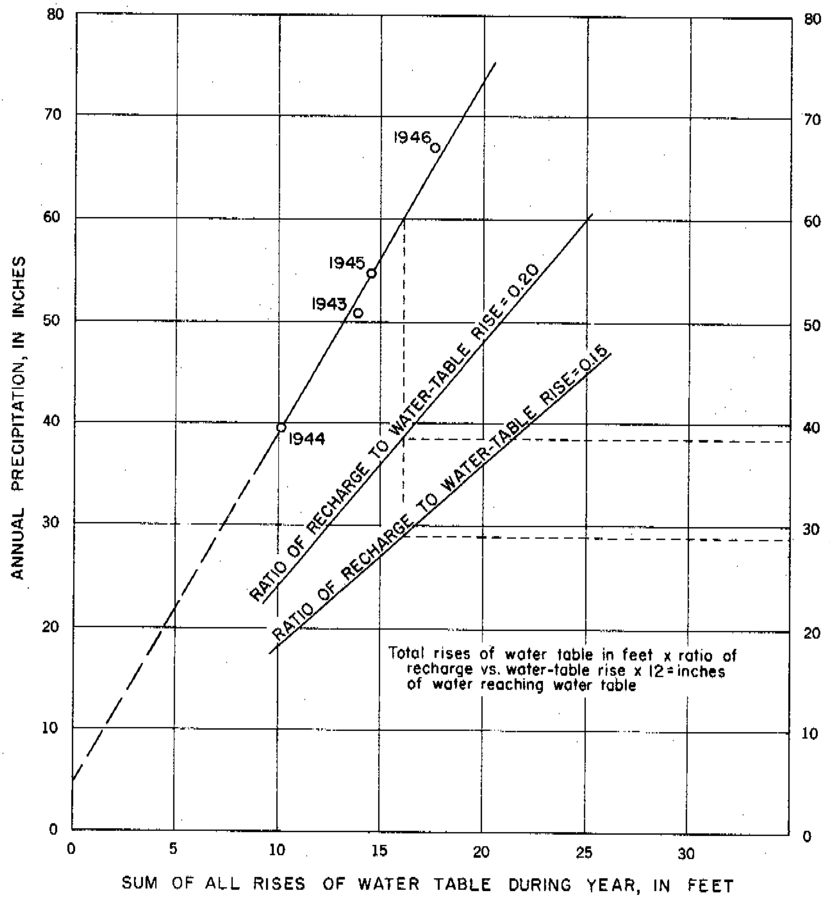

Figure 51. —Graph showing total yearly rise of water level in well S 182 compared with precipitation at Peters for 1943-46.

point plotted indicates the total rise of the water level for that year. Only the years 1943 to 1946, inclusive, were used because the rain gage at Peters was not installed until May 1942. A straight line was drawn as closely as possible to the four plotted points, which gave a fair degree of alinement. This graph indicates that, with an average annual rainfall of 60 in., an average total rise of about 16 ft will occur in the water table. If the ratio of recharge to water-table rise for the material saturated by the rising water table were 0.20, the amount of recharge would be 38.5 in., or 64 percent of the rainfall.

A similar study, made for the 10 years of record for well S 196, showed that a 60-in. annual rainfall would cause a total rise during the year of about 20.5 ft. Assuming a ratio of recharge to water-table rise of 0.20, 40 in. or 81.5 percent of the rainfall would reach the water table. If this ratio were 0.15 the recharge would be 36.8 in., or 61.5 percent.

It is recognized that results of the above method are of limited accuracy because of the impossibility of accurately determining the true specific yield of the material saturated by the rising water table; but by this method it is possible to obtain relative values of the ratio of recharge to water-table rises, and it shows that about two-thirds of the annual rainfall reaches the water table in southern Dade County.

Wells S 182 and S 196 are between Howard and Homestead in typical areas of the coastal ridge. Well G 86, however, is in the north Miami area, where the formation above the water table consists largely of sand; recharge to the water table averaging about 50 percent of the annual rainfall appears to be somewhat less than that of the area to the south.

GROUND-WATER DISCHARGE

GENERAL STATEMENT

In this discussion, the discharge of ground water refers only to the water that has infiltrated to the water table, although the rate of extraction from the aquifer may, in some instances, be related to the amount of moisture retained in the formation above the water table.

Ground water may be removed from the aquifer by several means. Gravity will cause it to flow into streams, canals, and wells when the water table in the area is above the water surface in these exits. From the stream or canal, it may flow by gravity out of the area; from the well, it may be removed by pumping. The amount of ground water moving by gravity depends on the slope of the water table (also referred to as the hydraulic gradient) and the transmissibility of the aquifer. Where the water table is at or near the surface, a large percentage of the ground water may be removed by the process of evapotranspiration. If the water table is below the land surface, evaporation may take place where the capillary fringe reaches the land surface or where the air can circulate through the pores or interstices of the soil to sufficient depth to reach the capillary fringe.

Water rises in the capillary fringe above the water table in response to the same molecular forces that cause a liquid to rise in a wick. In fine-grained geologic materials the capillary fringe may reach to a height of several feet above the water table; in silty or clayey materials it may exceed 10 ft; but in coarse sand or gravel it may be only a few inches or even a fraction of an inch. The height of the capillary fringe above the water table also varies with the temperature, and with the rise or fall of the water table itself.

EVAPORATION AND TRANSPIRATION

Transpiration is the exhalation of water vapor by organisms, but the ground-water investigator is concerned only with transpiration by plants, whose roots remove water from the soil and whose leaves return most of this water to the atmosphere. Transpiration is a process that is concerned with the metabolism of plants, and if the supply of water becomes too small, the plants wilt, wither, and die. Plants can extract water from the water table only so long as their roots extend into the capillary fringe or to the water table itself.

In many ground-water investigations it is difficult or impossible to separate evaporation from transpiration, and the total of both is referred to as evapotranspiration, or total evaporation. Evapotranspiration varies widely, depending on the topography, soil, weather conditions, depth to water table, height of capillary fringe above the water table, and character of the vegetation.

Over the Everglades and over most of the coastal ridge in Dade County the water table is so close to the land surface that evapotranspiration accounts for a large percentage of the total water removed from the aquifer. The land surface in the glades in Dade County ranges from about 2 to 8 ft above mean sea level, and over most of the coastal ridge it is less than 10 ft above mean sea level. The water table in the coastal ridge ranges from about 10 ft above mean sea level to 1 ft below, varying with location, weather conditions, pumpage, and several other factors.

(See the section on climate for descriptions of experiments on evapotranspiration at Belle Glade and West Palm Beach.)

EVAPOTRANSPIRATION AND DISCHARGE STUDIES IN DADE COUNTY

An attempt was made by the writers to correlate the rate of decline of the water level in two wells, G 72 and G 218, with the depth to the water table. These two wells are in the eastern part of the Everglades of northern Dade County, where the muck is about 3 ft thick and where the water table seldom drops below the base of the muck. The hydrographs of these two wells indicate that the rate of decline of the water table during rainless periods does not decrease noticeably as the water table drops. This suggests that evapotranspiration rates, for the limited range of water levels considered, do not decrease materially as the water table declines.

Figure 52 shows typical fluctuations of the water table in wells G 72 and G 218 during rainless periods. The maximum daily rate of decline due to evapotranspiration losses appears to be about 0.15 ft, and the minimum is about 0.03 ft. An average decline of 0.10 ft per day is at the rate of 3 ft per month. Assuming that the

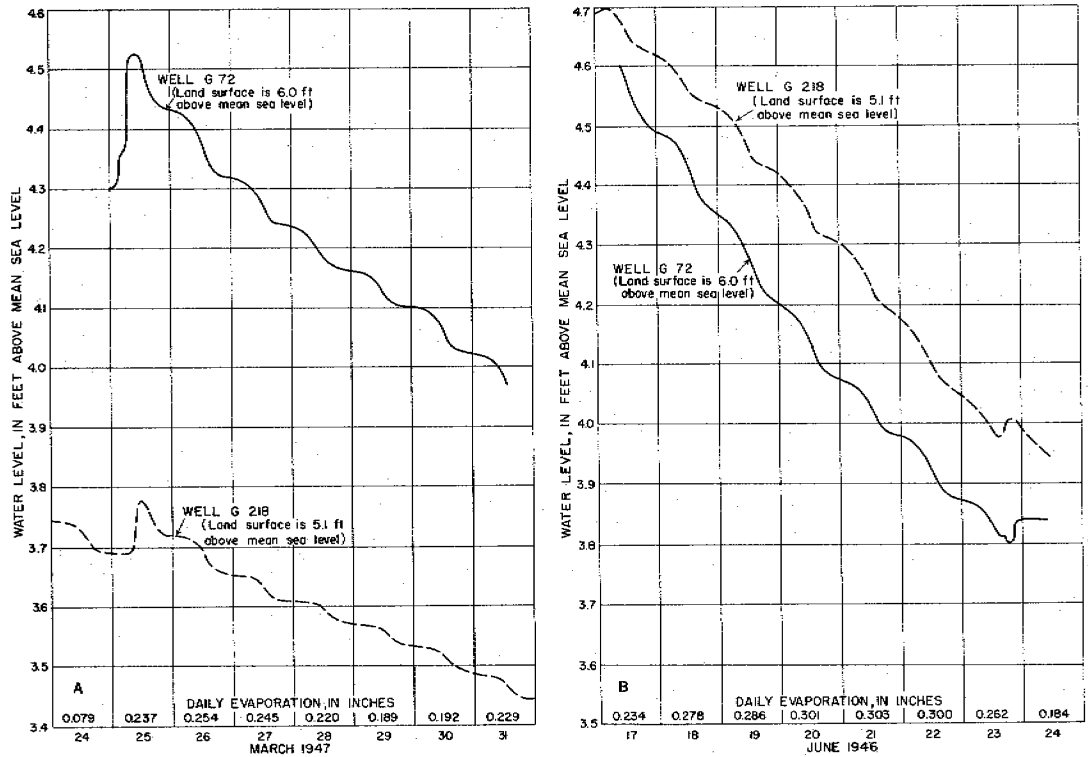

Figure 52. — Typical fluctuations of water levels in wells G 72 and G 218 during rainless periods.

water coming out of storage occupied 20 percent of the volume of the material dewatered, this would give 0.60 ft or 7.20 in. of water per month and 86.4 in. per year. A decline of 0.03 ft per day would be equivalent to 2.16 in. per month or 25.9 in. per year.

The water taken by evapotranspiration from below the water table is only a part of the total evapotranspiration loss from the muck, because the latter has a high specific retention and plants can extract water held against gravity drainage above the water table. From Clayton's published results (Clayton, Neller, and Allison, 1942, p. 10-14), it is concluded that a saturated cubic foot of muck soil in its original condition is about 80 percent water, but the specific yield is only about 0.20.

Thus, it appears that in an area where the muck is 2 ft or more thick, the water table could be lowered readily by pumping but very little salvage would be obtained and evapotranspiration losses would be reduced only slightly. While the water table is in the muck, the capillary fringe reaches the land surface and evapotranspiration rates are high; when the water table is lowered below the muck, the moisture content of this material is so high that evapotranspiration rates can be maintained for long periods without appreciable falling off. Subsequent rainfall is largely absorbed by the muck, and only a small part of it percolates through to the water table. Thus, very little can be done to reduce the priority of plants on rainfall.

Where the muck and marl mantle is about 1 ft or less in thickness above the limestone, lowering of the water table would probably produce conditions more favorable for salvaging some water from loss by evapotranspiration.

The hydrographs of wells S 182 and S 196 show rainless periods as smooth curves of decreasing slope similar to the recession curve illustrating runoff for a surface stream. This decline generally starts from an hour to a day or more after the rain ceases, depending on depth to the water table, characteristics of the soil, and formation above the water table. Wells S 182 and S 196 are typical wells for the section of the coastal ridge south of the Tamiami Canal and north of Homestead; they are situated more than 3.5 and 9.5 miles respectively from the nearest Biscayne Bay shore points and thus, as indicated in figures 39 and 41, they are unaffected by normal variations in Bay levels. Furthermore, these wells are in the general area of weather observation (rainfall) stations (see p. 219). They are the only wells in this area with daily or continuous records of water levels over a period of more than 6 years.

In an attempt to correlate the rate of decline of the water level in well S 182 with the stage of the water table, during rainless periods a graph was made (fig. 53) of rates of decline (obtained from re-

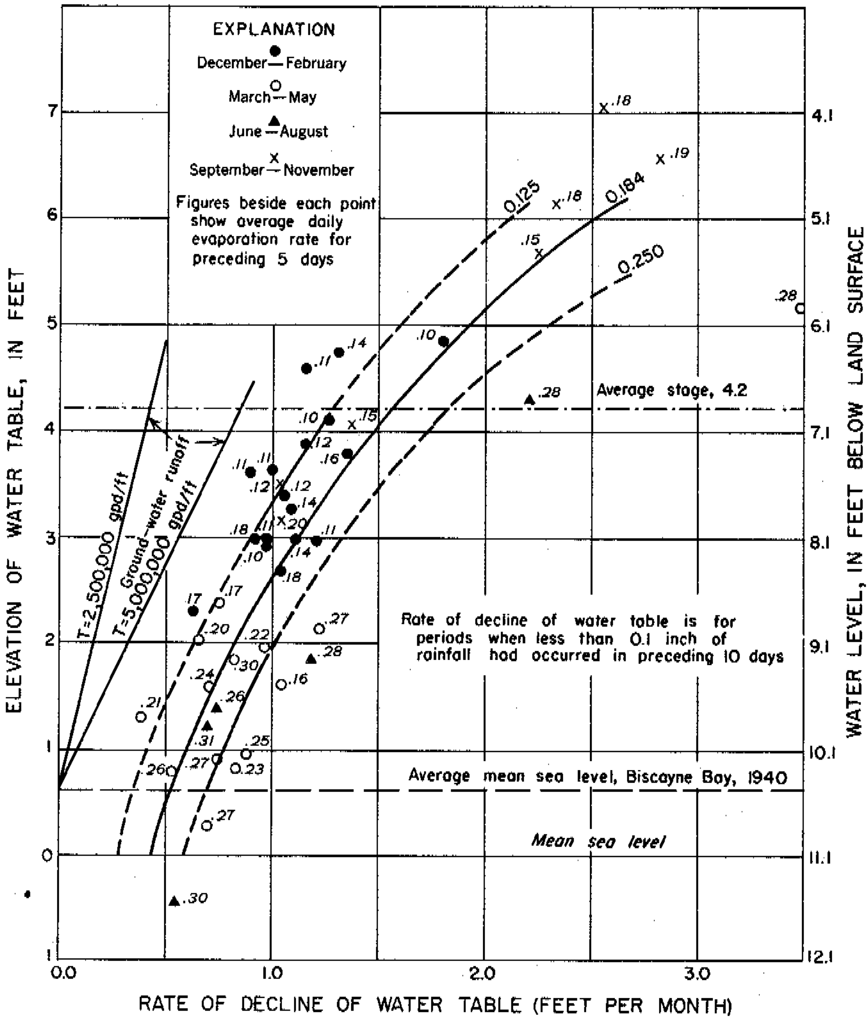

Figure 53. —Graph showing rate of decline of water level in well S 182 versus stage of water table correlated with average evaporation rate for preceding 5 days.

corder charts) plotted against the corresponding water-level stages for times when less than a total of 0.10 in. of rain had fallen in 10 preceding days. Note that a decline of the water table, indicating removal of water from storage in the aquifer, is evidently accompanied by a decreasing rate of ground-water removal. The daily fluctuations in this well are slight during rainless periods, and the departures from a smooth curve drawn through the water level at noon each day seldom exceed 0.01 ft. These small daily fluctuations, superimposed on the recession curve for the well, appear to be due mostly to changes in atmospheric pressure, but at times they may be due partly to transpiration.

Forty-one points are plotted on figure 53, the data having been obtained from records for the period from May 1942 (date rain gage was installed) through December 1946. The season of the year when the rainless periods occurred is indicated by symbols. Points for the lower stages, less than 2.5 ft above mean sea level, are for periods that occurred principally during the spring and summer months when evapotranspiration rates were high (pl. 3). Above the 2.5-ft stage, most of the 10-day rainless periods occurred in fall and winter when evapotranspiration rates were somewhat below the yearly average. High water-table stages occurred frequently during the summer but a rainless period of 10 days, or more, did not occur often.

The scattering of points on figure 53 shows that the rate of decline of the water table is influenced by factors other than the stage alone. If evapotranspiration processes are effective in withdrawing water from the water table, the quantity of water withdrawn will vary with depth to the water table and with plant growth and weather conditions. These factors, together with the character of the soil and the type of vegetation, may cause evapotranspiration to range from zero to as much as 0.4 in. of water per day, and it can easily be seen that if water is being extracted from the aquifer by evapotranspiration, then the rate of decline is being influenced by the weather at the same time. Beside each point on figure 53, the average daily evaporation rate is given in inches for the preceding 5 days (from Class A pan at Hialeah). Curves for three arbitrarily selected evaporation rates are drawn to show the general manner in which differences in evaporation rates displace the water-table recession curve.

Another factor affecting the rate of decline is the intensity, duration, and areal distribution of the preceding rainfall. Heavy and highly localized showers build up isolated water-table mounds, which gradually flatten out during the period following the shower. Thus, a higher local rate of decline of the water table occurs in one of these temporary mound areas than would follow a rainfall of nearly uniform intensity and duration over a large area. As a ground-water mound spreads out to other areas, not covered by the shower, it tends to retard the decline of the water table in those areas.

Also shown on figure 53 are the approximate rates of decline of the water table caused by ground-water runoff. These rates were determined for two values of the coefficient of transmissibility by using the average water-table slope and assuming a ratio of 0.20 between recharge and water-table rise (see p. 218). If the entire ground-water loss were by percolation through the aquifer to discharge areas in Biscayne Bay, the rate of decline would gradually approach zero as the water table declined to the same level as that in the bay, after which no further decline would occur. However, other factors are effective, for, as shown in figures 41 and 45, the water table at times falls below sea level in certain coastal areas.

The average water level in Biscayne Bay during this investigation (1941–46), as determined from the recording gage at Coconut Grove, was found to be approximately 0.6 ft above mean sea level, U. S. Coast and Geodetic Survey datum, adjustment of 1929 (the plane of reference for this investigation). This average is significant when studying water-level records for observation wells located in areas unaffected by transient tidal variations. The measurable effects of transient variations apparently are restricted to a coastal strip no wider than about 7,000 ft. Thus, for such observation wells as S 182 and S 196, the longer term averages of sea level, or water level in Biscayne Bay, are more significant than daily or monthly tidal variations.

For average, and below average, water-table stages Biscayne Bay is the only natural outlet (other than by evapotransipiration) for the ground water moving from the area in which well S 182 is located. If all the ground-water flow were discharged into the bay, then the average gradient between the well and the bay would vary with the height of the water level in well S 182 above some relatively long-term average water level in Biscayne Bay. However, the data indicate that at a stage of 0.6 ft above mean sea level (using 0.6 ft as the average water level in Biscayne Bay, base level for ground-water flow) the water table is still declining at a rate of about 0.5 ft per month. Assuming that the water coming out of storage occupied 0.20 of the volume of material unwatered, the loss from storage is at the rate of 1.20 in. per month, or 14.4 in. per year, for this stage of the water table.

Above the average stage, the rate of decline increases rapidly as the stage rises. This is due to several causes, among which are the following: (1) As the stage rises, the hydraulic gradient toward the sea steepens and ground-water flow increases in proportion. (2) At higher stages there are additional outlets for ground-water flow. (3) As the water table approaches the land surface, the evapotranspiration loss increases.

Figure 47 shows a high stage of the water table, and the contours indicate discharge from the western part of the coastal ridge toward the Everglades and toward Biscayne Bay. At extremely high stages, certain springs that formerly flowed perennially commence flowing in places along the shoreline at elevations slightly above sea level. These springs flow only for a short time, but they are additional and somewhat freer exits than those that discharge into the bottom of the bay. Also, at extremely high stages, considerable ground-water flow is directed into the transverse glades (see the section on Geomorphology) that discharge (by surface flow) into the bay or connecting canals.

The recorder charts for well S 182 generally do not show daily fluctuations similar to those for wells in the glades. The land surface at S 182 is about 11 ft above mean sea level, but within half a mile there are areas less than 7 ft above sea level. Because the Biscayne aquifer is very permeable, the evapotranspiration loss in nearby areas, where the water table is closer to the land surface, would have some effect on the water in this well.

Figure 46 shows contours on the water table for March 17, 1941, at a time when the water table was very near the average stage computed for the entire period of record (1940–46). The rain gages in Dade County show no rainfall other than slight traces for the period March 10–19, inclusive, but on March 8–9 a rainfall ranging from 1.04 in. to 2.28 in. was recorded in all gages (the first 7 days of the month were rainless). The average evaporation rate from the Hialeah pan during the 5 days preceding March 17, 1941, was 0.18 in. per day (about average for the year).

GROUND-WATER DISCHARGE STUDIES IN THE ATLANTIC COASTAL RIDGE SOUTH OF KENDALL

If an average value for the coefficient of transmissibility (see p. 237) for a selected area around well S 182 were known, the ground-water runoff from the area for an average stage, based on the shape of the water table (see fig. 46), could be computed. If a value for the average annual recharge to the aquifer in the area were known, it would be possible to compute evapotranspiration losses from the water table because the part of the recharge not accounted for by ground-water runoff would have to return to the atmosphere. Figure 51 indicates that about 38 in. of an average annual rainfall of 60 in. reaches the water table, assuming that the average ratio between recharge and water-table rise is 0.20 (see p. 218).

Three pumping tests, at locations respectively 4 miles north-northeast, 7 miles north-northwest, and 4 miles west-northwest of well S 182 (fig. 61), gave values for the coefficient of transmissibility (T) of about 4, 15, and 6 mgd per ft respectively (p. 270). This is a rather wide range for the coefficient of transmissibility, the maximum of the three determinations being about four times the minimum. Therefore, it was decided to compute theoretical ground-water runoff, using an average water-table stage and values of T equaling 2.5, 5, and 10 mgd per ft in each case. These computations should therefore yield results indicative of orders of magnitude of the ground-water discharge.

The area selected for making these computations centered around well S 182 and lay between the 2.0-ft and the 5.5-ft water-table contours. At the average water-table stage in this area (fig. 46) the location of the 2.0- and 5.5-ft contours on the water table approximately coincides with the boundaries of the coastal ridge, because, at the average stage, the Everglades generally begin west of the 5.5-ft contour on the water table and the mangrove swamp and marl flats begin on the east near the 2.0-ft contour on the water table. The width of the selected area was about 5 miles, measured along the 4.0-ft contour, 2.5 miles on each side of well S 182. The north and south boundaries were arbitrarily chosen flow lines between the 2.0- and 5.5-ft contours.

The average length of, and the average water-table gradient across, each of the contours in the selected area was determined, and by assuming a value of *T*, the ground-water flow past each contour was computed, and the flow into, and out of, the area was obtained.

Because the size of the area enclosed within the contours and the north and south boundaries was known, the amount of increase in flow between the two contours could be expressed in inches per year for the area.

This method was used for various combinations of contours to "average-out" errors in mapping the water table and to compensate for variances in the transmissibility. On the basis of these studies, a value of *T* equal to 10 mgd per ft appeared to be too high, because it would require about 50 in. (of an average annual of 60 in.) of rainfall as ground-water discharge to maintain the water table at the average stage. Given a value of *T* equal to 5 mgd per ft, the ground-water discharge would be about 25 in. per year. Assuming that 38 in. per year would reach the water table (fig. 51), 13 in. would be discharged as evapotranspiration from the water table, and the total evapotranspiration would be 35 in. of water per year, provided that the only other disposition of rainfall were ground-water runoff.

A logical continuation of the above method shows:

38 in. recharge to water table
-25 in. ground-water discharge
13 in. evapotranspiration from water table

60 in. average annual rainfall
-38 in. recharge to water table
22 in. evapotranspiration of rain not reaching water table

22 in. evapotranspiration of rain not reaching water table
+13 in. evapotranspiration loss from water table
35 in. total evapotranspiration loss, or

60 in. annual rainfall
-25 in. ground-water discharge
35 in. total evapotranspiration loss

Inasmuch as there are no canals within these selected areas, surface runoff would be inconsequential. Given a value of *T* equal to

2.5 mgd per ft, the ground-water runoff would be 12.5 in. per year; evapotranspiration from the water table would be 25.5 in., and total evapotranspiration would be 47.5 in.

Assuming that atmospheric conditions causing high evaporation rates from a free water surface are also conducive to high evapotranspiration rates from the water table, there should be some relationship between the rate of evaporation and the rate of decline of the water table. This relationship would hold true only when the water table is close enough to the surface for plants to draw freely upon it.

The water table under most of the coastal ridge appears to decline as a result of evapotranspiration, but at a lower rate than in the glades to the west. Figure 53 shows the water table still declining at the rate of about 0.5 ft per month when the water table is down to 0.6 ft above mean sea level, which is approximately the average level of the water surface in Biscayne Bay (see p. 227). Because ground-water discharge would be zero from a flat water table at this elevation, and because there is no large-scale artificial withdrawal in the area, evapotranspiration must account for the decline below this stage. Even if the evapotranspiration in the immediate vicinity of a well on high ground is small, loss from adjacent lower ground (possibly within a 2- or 3-mile radius) will affect the water table in the higher area, and it will lower the water level there at a rate in excess of that caused by the local evapotranspiration.

Figure 53 was prepared in an attempt to correlate stage and rate of decline of the water table with evaporation rates for the 5 preceding days from a standard Class A, U. S. Weather Bureau evaporation pan at Water Plant, Hialeah. It is realized that evaporation rates as determined from an evaporation pan cannot be directly applied to evaporation from land surfaces, and in this section the estimates are based on changes in evaporation rates. The average evaporation rates are shown in figure 53.

The highest evaporation rates tend to fall to the right side of the group of points, and the lowest evaporation rates tend to fall to the left side of the group, thus indicating that higher evaporation rates tend to cause a more rapid decline of the water table at a given level. There is a considerable scattering of points for equal evaporation rates. This is probably due to several factors, the two principal ones being: (1) Local rainfall creates water-table mounds (see p. 226), which, even after 10 to 20 days following a heavy shower, may still be dissipating into the water table. (2) The evaporation pan at Water Plant, Hialeah, is 16.5 miles north-northeast of the well, and average 5-day pan evaporation rates at the two locations may vary enough to introduce some error.

This scattering of points makes it impossible to draw curves accurately correlating the rate of decline versus stage for constant evaporation rates. However, lines have been drawn to indicate average evaporation rates of about 0.25, 0.184 (the average evaporation rate for the Hialeah pan for the period of record, 1941–46) and 0.125 in. per day.

A study similar to the above was made for well S 196 for the period 1938–46, inclusive. This well is at the University of Florida Sub-Tropical Experiment Station, 3 miles north-northwest of Homestead and 10.5 miles southwest of well S 182.

The evapotranspiration loss in the area of well S 196, at a stage of 0.6 ft above mean sea level (at this stage the water table is nearly flat and at the same level as the average water surface in Biscayne Bay), causes the water to decline at the rate of about 0.85 ft per month. In the vicinity of S 196, a ratio of recharge to change in ground-water level of 0.15 seems a reasonable value (see section on Ground-water recharge, p. 219). Using this ratio, the evapotranspiration is equivalent to about 1.5 in. of water per month or 18 in. per year. At an average stage of 3.5 ft above mean sea level, the water table declines at the rate of 1.6 ft per month. Using the ratio of 0.15, this is equivalent to about 35 in. per year.

After deducting losses for evapotranspiration, the ground-water discharge at average water-table stage is sufficient to cause the water table to drop at the rate of 0.60 ft per month, which is equivalent to about 13 in. of runoff per year.

In the area of well S 196 the value of the cofficient of transmissibility required to carry off 13 in. per year, based on the shape of the water table at average stage, appears to be between 15 and 20 mgd per ft. The map of the water table in this area (fig. 46) shows that the gradients are rather flat; however, the control for mapping the water table in this area is not complete enough to allow accurate contours to be drawn.

The analysis of the behavior of the water levels in wells S 182 and S 196 indicates that, for the coastal ridge between Kendall and Homestead, about 35 to 40 in. of the annual rainfall of 60 in. reaches the water table, and of this amount, about 15 to 20 in. is lost by ground-water runoff and 20 to 25 in. is lost by evapotranspiration from the water table.

The coastal ridge area between Kendall and Homestead consists of about 120 square miles. A 15-in. annual ground-water runoff from this area is equivalent to an average daily runoff of 85 mgd; and a 20-in. runoff is equivalent to 114 mgd. It is estimated that, at an average stage of the water table, the ground-water flow past the 2.0-ft contour, which in this area is about 18 miles long, is from 100 to 140 mgd.

WATER-LEVEL DECLINE IN SELECTED WELLS OF THE ATLANTIC COASTAL RIDGE

Figure 54 shows the recession curves for several wells in the coastal ridge. From these curves may be determined the approximate length of time required for the water table to decline from

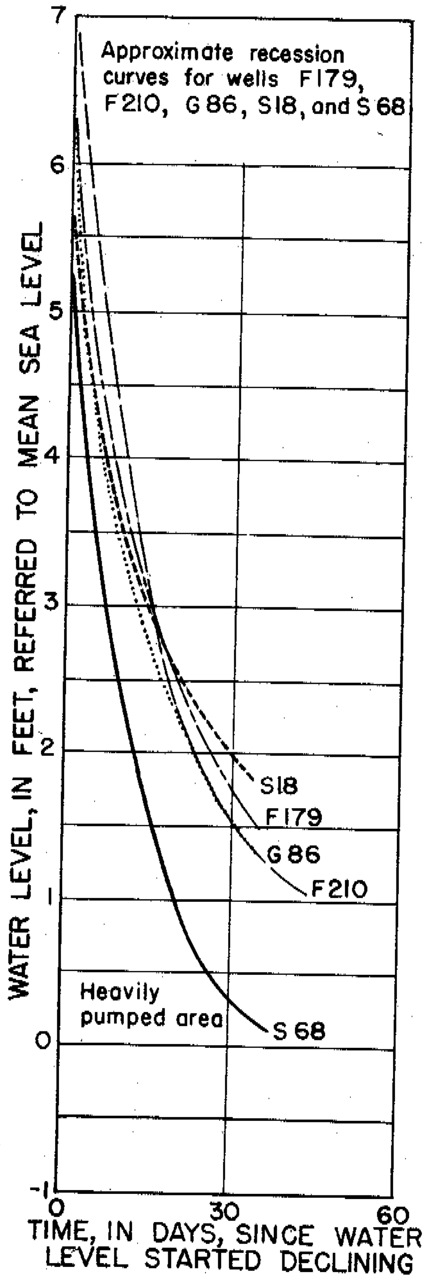

Figure 54. —Recession curves for five wells in the coastal ridge.

one stage to another during periods of no rainfall. The curves were traced from the hydrographs of wells F 179, F 210, G 86, S 18, and S 68, following very heavy rainfall on April 16–17, 1942.

On April 14, the water level in these five wells was 0.5 to 0.9 ft below their average stage. Ten to twenty in. of rainfall fell on the area in which these wells are located during April 16 and 17, 1942. At this time, the water levels in the five wells rose to their highest stages on record (1940–46); they reached peak stages of 5.3 to 6.9 ft above mean sea level, 1 to 7 hours after the rainfall stopped. The rise in the five wells during these 2 days ranged from 4.8 to 6.1 ft. By May 14, the water level in these wells had declined to stages only 0.1 to 0.6 ft above their average stage. Thus, within a 30-day period the water level had risen from relatively low stages to the highest stages on record and had declined to a stage only slightly above normal.

These five wells are all located within 1.5 miles of either the Biscayne Bay shoreline or a major canal, and the water levels in the wells react accordingly.

Well S 68 is an observation well in the Miami well field near Hialeah, about 0.6 mile southwest of the Miami Canal. On April 14, the stage in the well was about mean sea level, and in the Miami Canal at Hialeah it was 1.2 ft higher, thus indicating influent conditions. After about 18 in. of rainfall on April 16 and 17, the water level in the well reached a peak stage of 5.3 ft above mean sea level within an hour after the rain ceased. The corresponding peak stage in the canal was 2.6 ft above mean sea level, a difference of 2.7 ft, and the establishment of effluent conditions had taken place. Fifteen days later, the stage in well S 68 was 1.6 ft above mean sea level, and in the canal it was 1.7 ft above mean sea level. Thirty days after the heavy rainfall, the stage in the well was about 0.4 ft above mean sea level and 0.6 ft below the canal stage, thus indicating that influent conditions had been reestablished.

Well S 18 is about 0.5 mile north of Biscayne Canal, and well G 86 is about 0.5 mile north of Little River Canal; both wells are on, or near, NW. 27th Avenue. After about 18 in. of rainfall, these wells rose to peak stages of 5.6 and 6.1 ft above mean sea level. Thirty days later, they had declined to stages of 2.0 and 1.6 ft above mean sea level, respectively. The differences between the water levels in the wells and those in the canals probably never exceed 3.5 ft. Well F 179 is located at SW. 24th Terrace and 32d Avenue, and well F 210 is located at NW. 62d Street and Miami Court. Well F 179 is about 1.3 miles northwest of Biscayne Bay shoreline and 1.7 miles northeast of Coral Gables Canal. Well F 210 is about 1 mile west of Biscayne Bay shoreline and 1.3 miles south of Little River Canal.

GROUND-WATER DISCHARGE STUDIES IN THE ATLANTIC COASTAL RIDGE NORTH OF KENDALL

The northern part of the coastal ridge of Dade County is dissected by several tidal drainage canals; the principal ones are Snake Creek, Biscayne, Little River, Miami, Tamiami, Coral Gables, and Snapper Creek. The recession curves for five wells in this area show that these canals quickly drain off water reaching the water table when the area is not flooded. Although no quantitative data are available for ground-water runoff through canals from the coastal ridge, the average discharge was determined for several selected days at two or three locations on each major canal. Continuous discharge records are available for the years since 1940 for several stations on the Miami and Tamiami Canals (see section on Surface water). From these limited data it is impossible to arrive at a figure for total average annual ground-water runoff through canals from the coastal ridge.

From 1 mile below Kendall to the Dade—Broward County line to the north, the coastal ridge includes an area of about 120 square miles. In this area, it is estimated that more than half of the average annual rainfall reaches the water table, and of this amount, the larger part leaves by ground-water runoff. If the average annual ground-water runoff were 25 in., this would be equivalent to an average daily ground-water runoff of 143 mgd, or 221 cfs. However, this is not the total ground-water runoff, because, in addition to recharge from rainfall on the coastal ridge, there is ground-water runoff due to underground flow into the area from the Hialeah, Opa Locka, and adjacent areas, just west of the coastal ridge. This additional ground-water runoff appears to be less than one-quarter of the above figure (221 cfs), but it would be difficult to determine the amount with any degree of accuracy.

A large part of the area just west of the coastal ridge is drained by a network of lateral canals. Most of the discharge from these canals is into the Miami Canal when the 36th Street dam is open. However, some of it goes into Biscayne, Little River, Tamiami, and Snapper Creek Canals. On the basis of the total area contributing ground-water flow directly to the bay and on the basis of the slope and shape of the water table at the average elevation above sea level, it is estimated that three-quarters, or more, of the ground-water runoff from the northern part of the coastal ridge of Dade County (beginning at the south near Kendall) is discharged into canals and thence finds its way into the sea. The remainder of the ground water discharges directly into the bay.

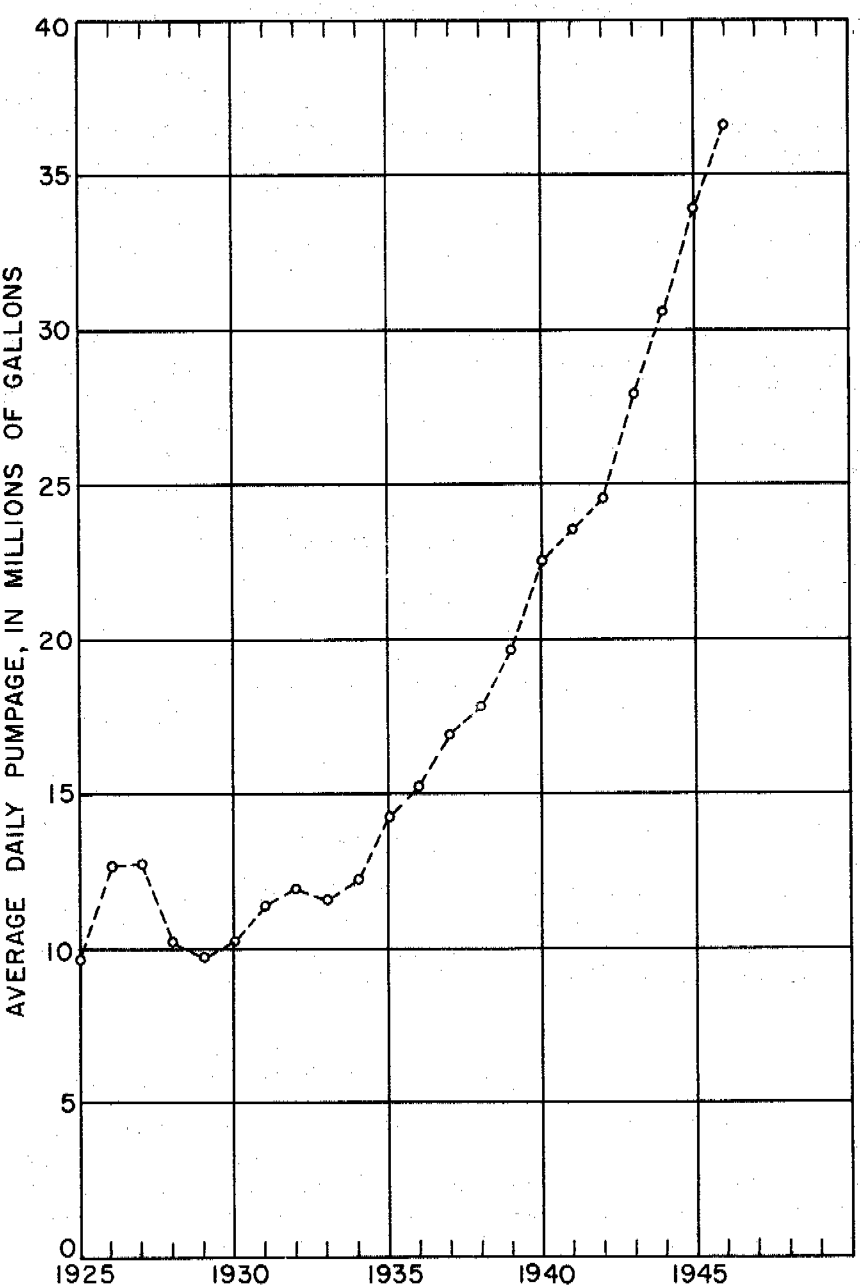

Figure 55. —Pumpage from Miami well field at Hialeah, 1925–46.

ESTIMATED GROUND-WATER PUMPAGE IN DADE COUNTY

The total amount of water pumped from wells in Dade County during 1945 was estimated (Parker, 1947, p. 72–88) to be 21,310 million gallons, of which 4,240 million gallons was for industrial use and 4,140 million gallons was for rural and agricultural purposes. The remaining 12,930 million gallons was pumped for the various municipal water supplies in the county. These figures are equivalent to an average daily pumpage of 58 million gallons for all purposes.

The average daily pumpage in 1945 from the Miami municipal well field was about 34 million gallons. The pumpage from this well field has increased steadily from an average of about 11.5 mgd in 1933 to 36.6 mgd in 1946 (see fig. 55 and Appendix). Not all of this water enters the well field as normal ground-water flow. Instead, a large part of the water pumped from the Miami municipal well field percolates into the area from nearby canals; the most heavily pumped area is completely surrounded by canals.

Of the more than 21,000 million gallons of ground water pumped annually, a considerable part is returned to the aquifer by seepage from irrigation systems, septic tanks, and drainage wells. As pointed out elsewhere in this report, this recharge helps to maintain high water levels in that part of the coastal ridge near the shore of Biscayne Bay.

HYDROLOGIC PROPERTIES OF WATER-BEARING MATERIALS

The two principal hydrologic properties of an aquifer are its capacities to store and to transmit water. All aquifers have both properties to a certain extent, but there is a wide range of degree of efficiency among them.

COEFFICIENT OF PERMEABILITY

The capacity to transmit water through the pores and interstices of an aquifer is often referred to as permeability, and it is expressed by various combinations of units of space and time. It is now commonly referred to as the coefficient of permeability. Meinzer and Wenzel (1942, p. 452–454) discuss permeability as follows:

"The standard coefficient of permeability used in the hydrologic work of the United States Geological Survey is defined as the rate of flow of water at 60° F., in gallons a day, through a cross section of 1 square foot, under a hydraulic gradient of 100 percent. A related coefficient, which may be called the 'field coefficient of permeability,' is defined as the rate of flow of water, in gallons a day, under prevailing conditions, through each foot of thickness of a given aquifer in a width of 1 mile, for each foot per mile of hydraulic gradient. The standard coefficient of permeability can generally be computed very closely by multiplying the field coefficient by the ratio of (1) viscosity of the water in the stratum to (2) the viscosity of water at 60° F."

COEFFICIENT OF TRANSMISSIBILITY

Transmissibility is a measure of the capacity of an aquifer to transmit water. To quote further from Meinzer and Wenzel: "Recently Theis (1935, p. 519–524) introduced the very convenient term 'coefficient of transmissibility,' [*T*] which is the 'field coefficient of permeability' multiplied by the thickness, in feet, of the saturated part of the aquifer. Thus the [standard] coefficient of permeability denotes a characteristic of the water-bearing material, whereas the coefficient of transmissibility denotes the analogous characteristic of the aquifer as a whole.

"Natural earth materials that have been tested in the hydrologic laboratory of the United States Geological Survey have been found to have coefficients of permeability ranging from about 0.0002 to about 90,000—that is, the most permeable material carries water at a rate about 450,000,000 times that of the least permeable material. However, most water-bearing materials utilized by wells have coefficients that are whole numbers of two or more figures, generally between 10 and 5,000."

COEFFICIENT OF STORAGE

The coefficient of storage (*S*) of an aquifer is defined as the volume of water released from, or taken into, storage per unit surface area of aquifer, per unit change in the component of head normal to that surface. To visualize this concept, imagine a decrease in head on an elemental vertical prism extending from top to bottom of a horizontal elastic aquifer of uniform thickness. The volume of water thereby released from this prism, divided by the product of the cross-sectional area and the decline in head, determines the storage coefficient. For artesian conditions, the water released is attributed solely to the compressibility of the aquifer material and of the water. Obviously, under these conditions, *S* is dependent on the thickness of the aquifer. For water-table conditions the water released is attributed partly to gravity drainage of the zone through which the water-table declines and partly to compressibility of the water and aquifer material in the saturated zone. Usually, the volume of water attributable to compressibility is a negligible proportion of the total volume of water released and can be ignored. The storage coefficient then is sensibly equal to the specific yield and is independent of the thickness of the aquifer.

SOLVING FOR *T* AND *S* BY USE OF THE THEIS NONEQUILIBRIUM FORMULA

The coefficients of transmissibility (T) and storage (S) may be determined from a mathematical analysis of the shape and rate of expansion, with respect to time since pumping started, of the cone of depression that develops around a pumped well. As in any mathematical analysis, it is necessary to assume certain basic conditions and relationships, and the reliability and accuracy of these coefficients, for practical application, is determined by how closely actual field conditions fit the assumptions on which the formulas are based.

The nonequilibrium formula, now widely used in quantitative studies of ground water was introduced by Theis (1935, p. 519–524) in 1935 for determining the coefficients of transmissibility and storage of a theoretical aquifer. It is called the nonequilibrium formula because the element of time (time since well started pumping) enters into it.

This formula is as follows (Wenzel, 1942, p. 87):

$$s = \frac{114.6q}{T} \int_{\frac{1.87r^2S}{Tt}}^{\infty} \frac{e^{-u}}{u}\, du$$

in which:

s = the drawdown in feet at any point in the vicinity of a well discharging at a uniform rate.
q = the discharge of the well in gallons per minute.
T = coefficient of transmissibility of the aquifer, in gallons per day per foot.
S = coefficient of storage.
t = time the well has been pumped in days.
r = distance in feet from discharging well to the point of observation.

According to Theis and Wenzel, the nonequilibrium formula is based on the following assumptions: (1) The water-bearing formation (aquifer) is homogeneous and isotropic; (2) the aquifer has an infinite areal extent; (3) the discharge well penetrates the entire thickness of the aquifer; (4) the coefficient of transmissibility is constant at all places and times; (5) the discharge well has an infinitesimal diameter; and (6) water taken from storage is discharged instantaneously with the decline in head. The formula also assumes that Darcy's Law is effective; that is, that the velocity of groundwater flow varies directly as the slope of the hydraulic gradient.

The integral can be called the well function of u, $W(u)$, and can be evaluated by the series:

$$W(u) = -0.577216 - \log_e u + u - \frac{u^2}{2 \cdot 2!} + \frac{u^3}{3 \cdot 3!} - \frac{u^4}{4 \cdot 4!} \cdots$$

in which: $$u = \frac{1.87r^2S}{Tt}$$

The nonequilibrium formula may then be written:

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

By means of a special type curve, a graphical solution for values of T and S may be made (Wenzel, 1942, p. 88).

Several attempts were made by pumping tests to determine the coefficients of transmissibility and storage for the aquifer in the Miami area. The four most important of these tests are known as: (1) S 1; (2) G 551; (3) G 552; and (4) G 553. They will be discussed on the following pages.

LARGE-SCALE PUMPING TESTS

S 1 PUMPING TEST

From November 19 to November 26, 1946, a large-scale pumping test was made in the lower Miami Springs well field. The pumped well was S 1, one of the regular city supply wells; its location with respect to the other wells and the canals in the area is shown in figure 56. An idealized geologic section of the lower well field, with location and depth of the test wells plotted with respect to each other and to land surface, is shown in figure 57.

Well S 1, the pumped well, is 61 ft deep and has 14-in. casing to a depth of 48.5 ft. The rated capacity of the pump is about 4,100 gpm. The deeper observation wells in the area, which were measured during the test, are approximately the same depth as the pumped well and have about the same amount of casing. The shallow observation wells are 10 to 12 ft deep and have about 2 to 5 ft of slotted casing or open hole at the bottom (see fig. 57).

All the supply wells, S 1 to S 8 (see fig. 56), in the lower well field were idle on November 14, except S 7, which was in operation continuously from November 14 through November 26. For the period November 14 to 18, inclusive, all the Miami water supply was obtained from the upper well field (wells S 11 to S 22, inclusive), except that supplied by well S 7. Well S 7 has a rated capacity of 2,600 gpm, or 3,744,000 gpd. The average daily pumpage from wells S 11 to S 22 during this period was 30.9 mgd.

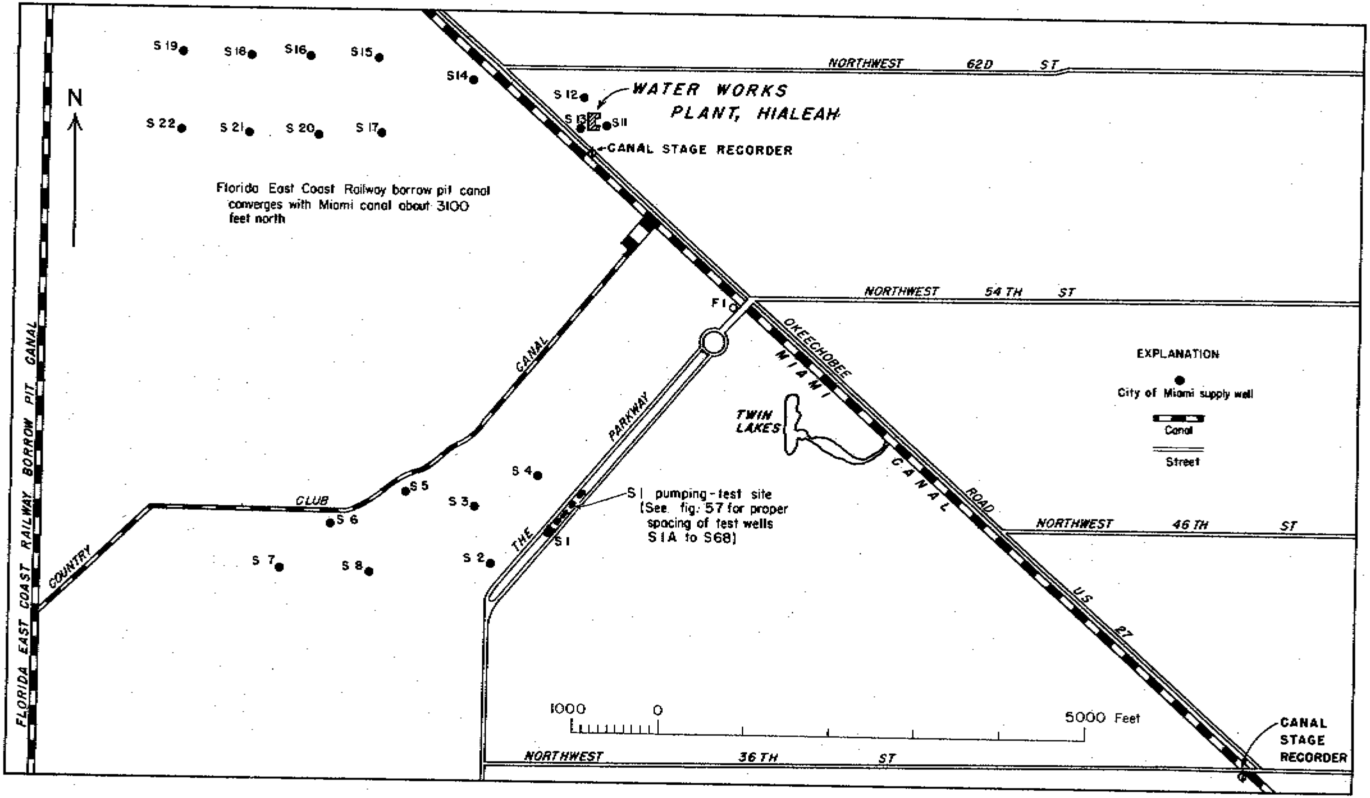

Figure 56.—Map of the Miami well-field area.

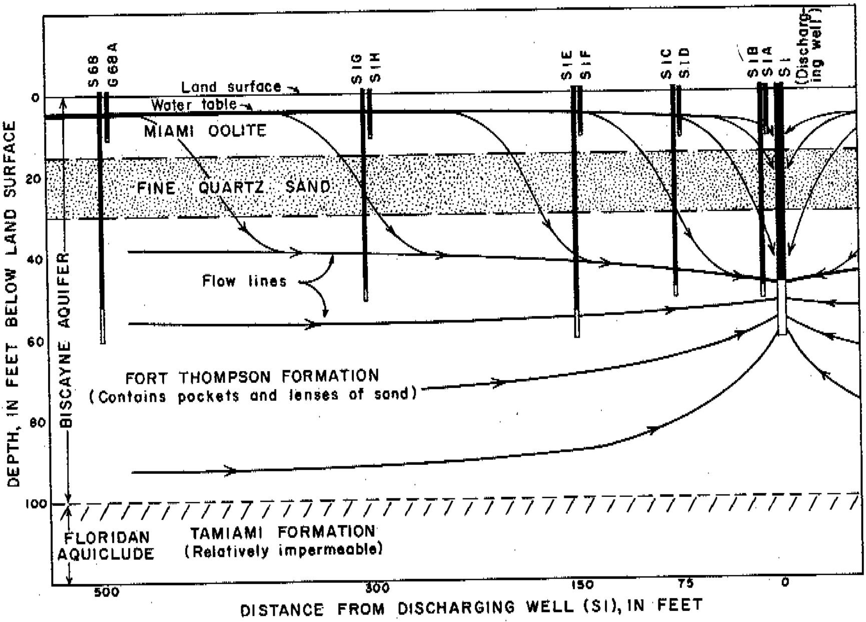

Figure 57. —Generalized cross section through wells S 1 to S 68 in the lower Miami well field.

On November 19 at 10:30 a. m., supply well S 1 was placed in operation and continued operating until 11 a. m. November 26. The capacity of this well is 4,030 gpm, or 5.8 mgd. From November 19 to 25, inclusive, the total pumpage to the water works plant averaged 36 mgd, of which 9.55 mgd was pumped from wells S 7 and S 1; the remainder was pumped from wells S 11 to S 22 in the upper field.

Well S 7 is 3,150 ft west of S 1. Of the supply wells S 11 to S 22, S 13 is the closest to S 1 and is about 4,750 ft north of the Miami Canal. Wells S 11, S 12, and S 13 are near the water works plant on the north side of the Miami Canal. Wells S 14 to S 22, in the upper field, are 5,100 to 7,100 ft from well S 1 and on the opposite side of the Country Club Canal (a shallow canal just northwest of the lower well field). At the closest point, well S 1 is about 1,500 ft from the Country Club Canal.

Records from a recording gage on the Miami Canal at the Water Works Plant at Hialeah for the period November 17 to November 25 showed that the daily mean stage of the Miami Canal ranged from 2.6 to 2.5 ft above mean sea level. The average daily range of fluctuation in the canal, during this period, was 0.90 ft; this fluctuation was caused mostly by tides. At the upper 36th Street gage on the Miami Canal, 2.1 miles downstream from the Hialeah gage (fig. 56), the daily mean stage in the canal for the same period ranged from 1.80 to 1.68 ft above mean sea level, and the average daily range of fluctuation for the 9 days was 1.75 ft (owing almost entirely to tides).

Well F 1, which is 52.7 ft deep and about 80 ft from the southwest bank of the Miami Canal and 3,400 ft northeast of well S 1, was equipped during the test with an automatic water-stage recorder that showed a daily mean stage of 2.26 ft above mean sea level on November 19, and 1.94 ft above mean sea level on November 26. The average daily range caused by tides was 0.07 ft. The daily fluctuations caused by tides in well S 68 (fig. 57) during the period November 17 to November 26 was about 0.03 ft.

From a study of the measurements made of the water level in all observation wells within 6,000 ft of well S 1 (during, and 6 days prior to, the test), it was concluded that a regional decline of about 0.10 ft in water levels had occurred during the period November 19 to November 26. Three observation wells equipped with automatic water-stage recorders, 4.4 to 6.2 miles from well S 1, showed declines during this period of 0.06, 0.08, and 0.15 ft. Therefore, corrections varying from 0.00 to 0.10 ft, increasing uniformly with time during the period November 19 to November 26, were applied to the observed drawdowns. The observed drawdowns were then computed on the assumption that no decline of the water level would have occurred if well S 1 had not been pumped.

Pairs of shallow and deep observation wells were constructed at distances of 10, 75, 150, and 300 ft northeast from well S 1 (the pumped well) in line with S 68 and G 68A (a shallow well), which are 500 ft northeast of well S 1 (fig. 57). The shallow wells are numbered respectively S 1A, S 1D, S 1F and S 1H, and the deep wells are numbered S 1B, S 1C, S 1E and S 1G. Shallow wells had been installed about 10 ft from the other supply wells.

In the lower well field the water levels in these supply wells and their companion shallow wells were measured during the test (except for S 7, which had been in continuous operation during and for 5 days before the tests). Wells S 2, S 3, and S 4 are the only supply wells within 1,000 ft of well S 1.

Referring to the Theis nonequilibrium formula (p. 239) as long as the values of u are less than 0.02, values of $W(u)$ versus u will plot as a straight line on semilogarithmic coordinates. This indicates that in the theoretical aquifer, the drawdowns (for a given time in that part of the cone of depression around the pumped well that has stabilized in shape) will vary as the logarithm of the distance from the pumped well. After the shape of the cone of depression has stabilized, the water levels may still slowly decline, but they will decline the same amount in all wells in that part of the cone of depression that has stabilized. This condition makes it possible to solve the coefficient of transmissibility (T) by a semilogarithmic plot of drawdown versus distance of observation well from the pumped well, so long as the observation well used is in that part of the cone of depression that has stabilized in shape. This method has been discussed by Cooper and Jacob (1946, p. 526–534). The following formula is used:

$$T = 2.303\, Q/2\pi\, \Delta s$$

in which:

Q = discharge of well.

Δs = change in drawdown, in feet, for one log cycle on the straight-line plot.

Figure 58 shows a solution for T by this method. Using the drawdowns in the deeper wells after 1 day of pumping, the value of T (as obtained from the slope of the straight line fitted to the plotted points as closely as possible) is 3.4 mgd per ft; after 6 days of pumping, the value of T is 3.25 mgd per ft. Using the drawdowns in the shallow wells, the value of T, after 6 days of pumping, is 4.3 mgd per ft.

Figures 59 and 60 show semilogarithmic plots of drawdown versus time (after well S 1 started pumping) for the five pairs of observation wells at 75, 150, 300, 500, and 900 ft from S 1. The drawdown in all shallow wells was less than in the deeper wells during the early part of the test, but, after 2 days of pumping, the water level in each pair of shallow and deep wells beyond 300 ft was very nearly the same, and for all pairs the rate of decline of the water level in the shallow and deep wells was about the same. As far as 300 ft from the pumped well, the water level in a given pair

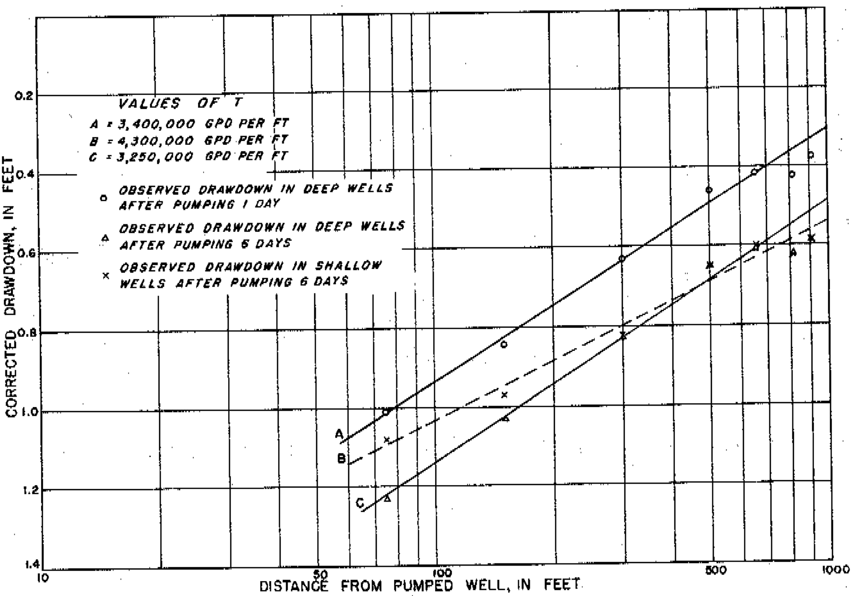

Figure 58.—Graph showing transmissibility coefficient as determined by distance-drawdown relationships from S 1 pumping test.

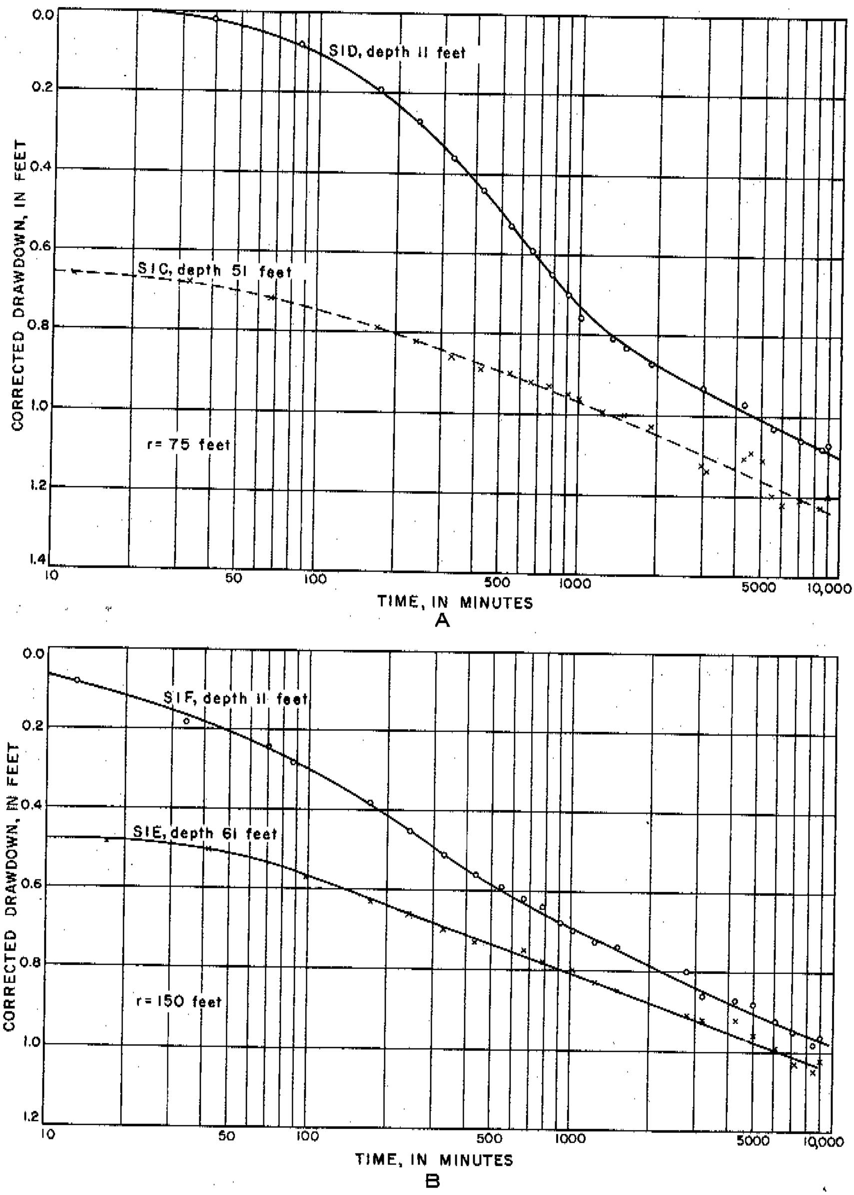

Figure 59. —Time-drawdown graphs for pumping test of S 1 for wells 75 and 150 feet distant.

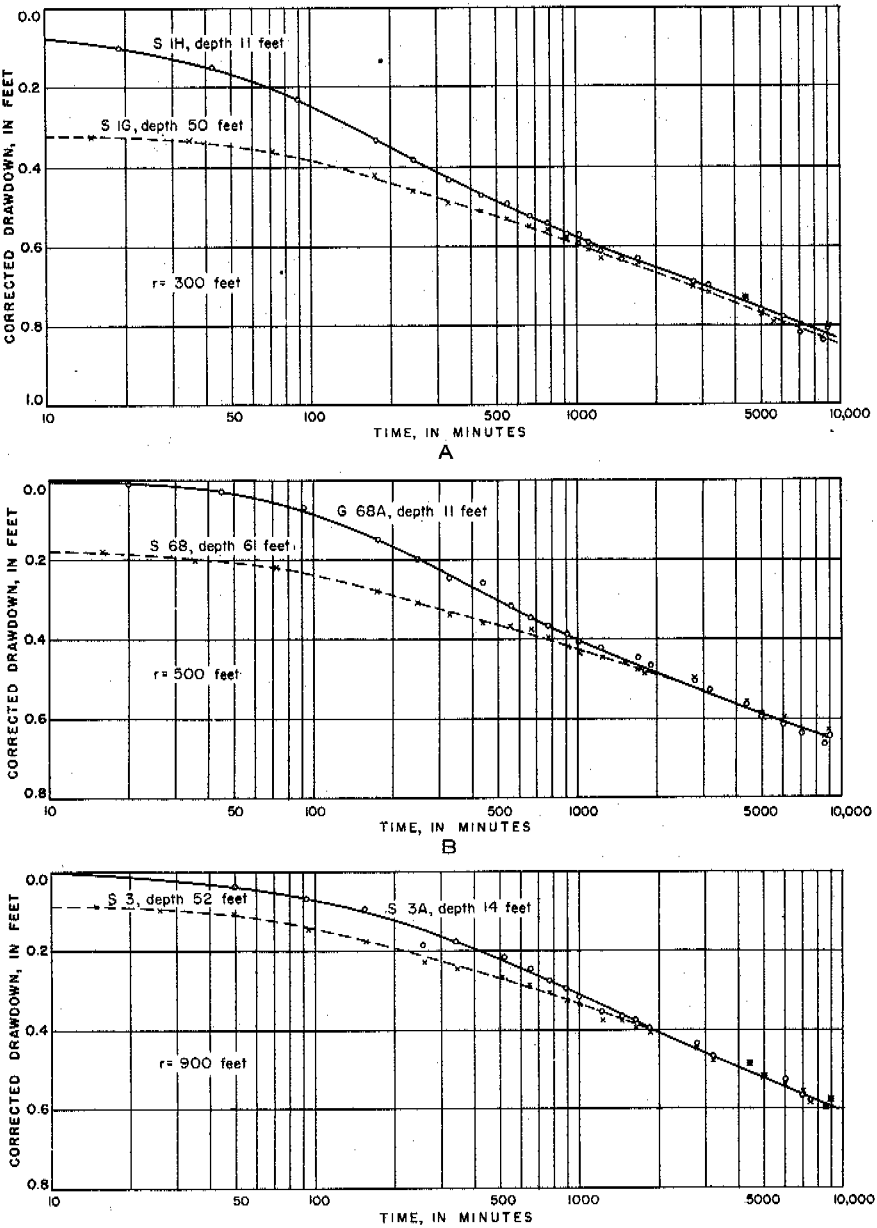

Figure 60. — Time-drawdown graphs for pumping test of S 1 for wells 300, 500, and 900 ft distant.

of deep and shallow wells differed by almost a constant during the last 3 or 4 days of the test. For the wells at 10, 75, 150, and 300 ft from S 1 the difference was about 1.08, 0.15, 0.07 and 0.015 ft, respectively. This difference was still decreasing slightly in these wells during the latter part of the test, thus indicating that a slight amount of water was still infiltrating from the upper part to the lower part of the aquifer.

The uncased hole in well S 1 is between 48.5 and 61 ft below the land surface (fig. 57) and it was through this 12.5 ft of open hole that the pumped 4,030 gpm entered the well. As water approached the pumped well, the flow lines gradually converged to this zone, thus causing higher velocities and greater drawdowns in the deeper wells that are open to the aquifer only in this zone. The deeper observation wells and supply wells are so constructed that they would be subject to this effect, and, near the end of the test, most of the differences in drawdown between the deep and shallow wells of each pair are believed to be due to this cause.

The geologic section of the lower well field (fig. 57) includes a layer of fine quartz sand, 10 to 20 ft thick, between the Miami oolite (in which the water table is located) and the Fort Thompson formation (from which the water is directly pumped). This layer of sand is less permeable than the limestones, and the vertical velocity of water from the upper to the lower part of the aquifer is small compared to the horizontal velocity in the Fort Thompson formation. Thus, during the early part of the pumping test the aquifer manifests temporary artesian characteristics, which allow the cone of decreased pressure to spread more rapidly through the lower part of the aquifer than through the upper part. These same characteristics are also responsible for a more rapid recovery in the lower part of the aquifer immediately after pumping ceases. After a day or two of pumping, enough water infiltrates through the sand from the upper part of the aquifer to supply most of the water pumped by the well; the aquifer then evidences water-table characteristics. Well S 68 responds markedly to sudden shocks (such as those caused by earthquakes) and thus gives further evidence of the semiconfining effect that the sand layer exerts in the aquifer.

Using a value of the coefficient of transmissibility (T), determined by distance-drawdown relationships for that part of the cone of depression that has stabilized (fig. 58), values for the coefficient of storage (S) may be computed on the basis of the drawdown observed in a well. Values of S computed in this manner will be large for shallow wells that are close to the pumped well during the early part of the test. The value of S computed from drawdown in shallow wells will (in general) decrease with time and distance from the pumped well until the water levels in both the shallow and deep wells are the same. Values of S computed from the drawdowns observed in the deeper wells will (in general) increase with time and distance from the pumped well until a fixed value is reached upon stabilization of the cone of depression.

If the water table is recharged from above or by influent canals, the value of *S*, computed in this manner by the Theis nonequilibrium formula, may continue to increase indefinitely, rising above 1.00. A value of *S* greater than 1.00 is a physical impossibility under the conditions assumed for deriving the formula, because more than 1 cu ft of water cannot be extracted from 1 cu ft of saturated material. However, one of the basic assumptions of the formula is that the water pumped by the test well comes entirely from storage in the aquifer and the aquifer receives no recharge during the test.

Another basic assumption of the Theis nonequilibrium formula is that the pumped well completely penetrates the aquifer and is capable of receiving water throughout the saturated thickness of the aquifer. In this pumping test, the deeper wells are open and capable of receiving ground water only for a few feet in the pumped zone; the well does not directly receive water from that part of the aquifer in which the shallow wells end. Thus, it is believed that the deeper wells gave greater drawdown and the shallow wells gave less drawdown than if the pumped well had drawn directly from all parts of the aquifer. In making computations for coefficient of storage, therefore, an average value of 3.8 mgd per ft for *T* was used.

Table 16 shows values of *S* computed from drawdown in the deeper wells using a value of *T* equal to 3.8 mgd per ft for both 1 and 6 days after the test started. The drawdowns used in the computations were those taken from smooth curves drawn through the plotted points as shown in figures 59 and 60. Table 17 shows values of *S* computed from drawdowns in the shallow wells, using a value of *T* equal to 3.8 mgd per ft for both 1 and 6 days after the test started.

The coefficients derived from the S 1 pumping test indicate that *T* is about 4 mgd per ft and that *S* is about 0.10. The coefficient of storage is more likely to be in error because a 10-percent error in the value of the coefficient of transmissibility, used in making the computation of *S*, may nearly double or halve the coefficient of storage, depending on whether *T* is greater or smaller. Also, slight errors or differences in drawdown, due to irregularities in the aquifer, cause relatively large differences in the calculated values for the coefficient of storage.

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Table 16.—*Values of S computed from drawdowns in deep wells*

[Coefficient of transmissibility equal to 3.8 mgd per ft]

Time since pumping started (days)	Well S 1C r = 75 ft	Well S 1E r = 150 ft	Well S 1G r = 300 ft	Well S 68 r = 500 ft	Well S 4 r = 650 ft	Well S 2 r = 820 ft	Well S 3 r = 900 ft	Average
1	0.050	0.048	0.071	0.105	0.090	0.052	0.063	0.068
6	.047	.063	.083	.130	.107	.063	.066	.080

Table 17.—*Values of S computed from drawdowns in shallow wells*

[Coefficient of transmissibility equal to 3.8 mgd per ft]

Time since pumping started (days)	Well S 1D r = 75 ft	Well S 1F r = 150 ft	Well S 1H r = 300 ft	Well G 68A r = 500 ft	Well S 4A r = 650 ft	Well S 2A r = 820 ft	Well S 3A r = 900 ft	Average
1	0.247	0.113	0.080	0.119	0.111	0.054	0.072	0.114
6	.168	.104	.089	.130	.117	.060	.066	.105

G 551, G 552, AND G 553 PUMPING TESTS

One of the chief objectives of the ground-water investigation begun in 1939 was to determine a potentially safe area for the development of future municipal supplies. This area was described and mapped by Parker (Parker, Ferguson, and Love, 1944, p. 26–28), but detailed and large-scale quantitative investigations outside the present well-field area, where test S 1 was made, necessarily awaited the selection of a general area for development. This choice, based on engineering economics as well as on the data collected from the geologic and ground-water investigations, was made in the spring of 1947 by Messrs. W. A. Glass and Frederick Weed, of the Department of Water and Sewers of the city of Miami, and arrangements were made to obtain detailed quantitative ground-water data on the chosen area.

Previous pumping tests in the Miami area were not too successful because of the necessity for pumping and disposing of very large quantities of water. In most parts of the world, pumping tests using quantities of water ranging from about 100 to about 1,000 gpm would suffice. In Dade County, however, the Biscayne aquifer is so permeable that rates of pumping in this range produce a cone of depression too shallow to be used to accurately determine the drawdowns caused by the pumping, or to satisfactorily determine the depth, shape, rate of spread, and decline of the cone of depression. In previous pumping tests, the wells had been pumped at approximately 1,000 and 1,500 gpm; these tests had indicated that the coefficients of transmissibility were very high and that exceedingly large volumes of water could be pumped with a very small drawdown, but it was felt that they did not give accurate values of the hydrologic coefficients needed for the planning of a well field. Therefore, it was decided to run tests in the potential well-field area at a rate of approximately 3,500 gpm, or 5 mgd. It had been shown in the fall of 1946 (in the lower Miami well-field area, where the pumping test on well S 1 was conducted) that such a quantity would be large enough.

The selected test area lies south and west of Miami (see fig. 61). It is of triangular shape and contains approximately 11.3 sq miles. A test well occupies each angle of this area: G 551 is in sec. 36, T. 54 S., R. 39 E., a distance of approximately 5.4 miles northeast of G 552; G 552 is in sec. 27, T. 55 S., R. 39 E., a distance of approximately 5.8 miles west-southwest of G 553; G 553 is in sec. 16, T. 55 S., R. 40 E., a distance of approximately 4.6 miles southeast of G 551.

Most supply wells in Dade County are drilled or driven. Most drilled wells have solid casing driven into hard, solution-holed limestone, with an open hole extending from 1 or 2 ft to about 15 ft beyond the end of the casing; driven wells generally have solid

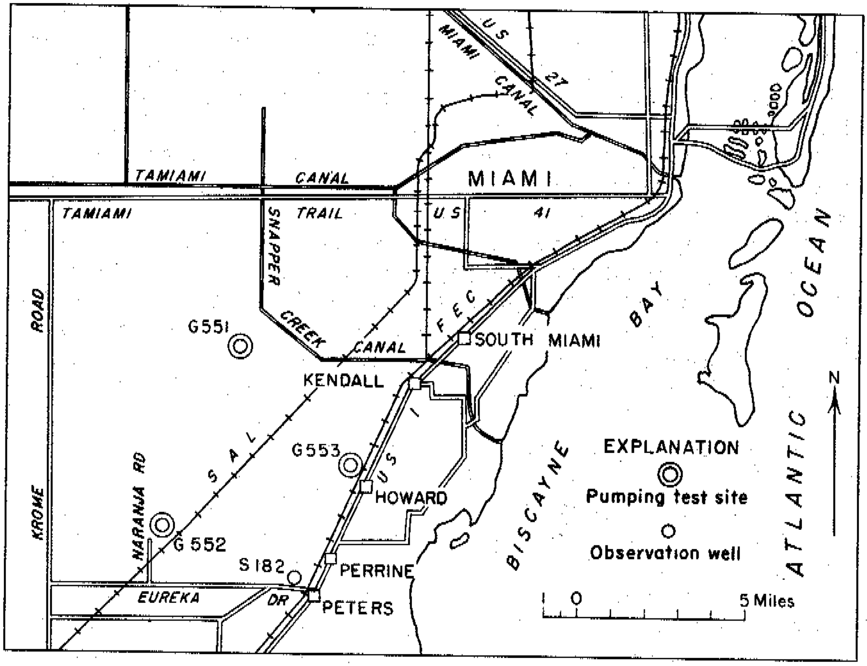

Figure 61. — Index map showing general locations of wells G 551, G 552, and G 553.

casing finished with a well point in sand. The driven wells seldom are more than $2\frac{1}{2}$ in. in diameter and are used chiefly for small domestic,supplies. All large supply wells, most fire wells, and large-capacity drainage wells are drilled wells ranging in diameter from 6 to 18 in. The average 6-in. well yields as much as 1,500 gpm with a drawdown of 2 to 4 ft.

In these wells, all water must enter the casing only through the open hole or screened parts, thus causing large deflections in the ground-water flow lines, especially near the well (see fig. 57). In order to avoid this condition it was decided to try a type of well construction new to the Miami area, which, insofar as geologic conditions permitted, would minimize the deflection of ground-water flow lines and permit a more nearly horizontal flow through the entire thickness of the aquifer toward the well.

Such conditions could be fulfilled at the least expense by a slotted well, which would draw water through relatively large openings in the casing at all depths below the water table. However, it was obvious that if such slots were opposite a thick sand layer the well would fill with sand; therefore, the geologic section at the exact site of each test well was explored by core borings before the well was drilled. The placement of the slots then was determined from the geologic interpretation of the cores and cuttings.

The slots, 3/8 in. wide and 12 in. long, were burned in the 18-in. casing with an acetylene torch; they were cut parallel to the length of the casing and were arranged spirally around the circumference. Enough slots were cut to provide a total intake area through the casing walls of about 10 sq ft (see fig. 64).

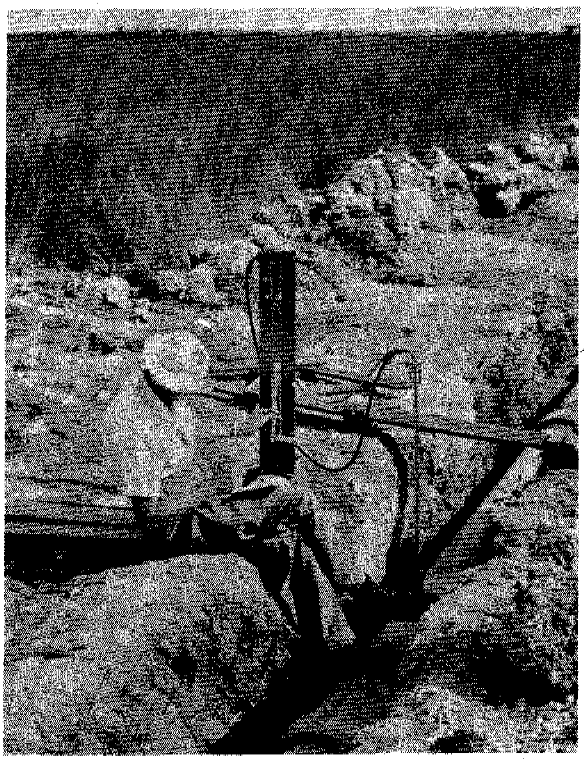

Figure 62. —Reading the mercury manometer of the pitometer used to measure discharge from the pumped well.

In each of the three pumping tests to be described, several short runs were made, generally on the day preceding the final test run, to ascertain that the physical and mechanical setup was satisfactory and to allow for any necessary repairs or adjustments. Then a 2-hour test run was made to correlate the throttle of the motor with the desired rate of pumping, which was determined by pitometer readings (fig. 62). These were made in a straight section of the Transite discharge line at a point far enough from the pump, valves, and pipe bends so that flow disturbances due to these features would be unimportant. When this rate was established for each well, the throttle was marked to facilitate setting when the actual test run was made. Pitometer readings were made at frequent intervals during the test runs to ascertain that a flow of approximately 5 mgd (3, 500 gpm) was maintained.

Descriptions of the tests and results obtained are given in the following sections.

G 551 PUMPING TEST

On April 9, 1947, a 5-hour pumping test was run on test well G 551, the first of three such wells constructed for test purposes for the Department of Water and Sewers of the city of Miami. Figure 63 shows the location of this well with respect to tne neighboring roads and observation wells, and figure 64 shows the method of construction of G 551, and the logs of G 551 (prepared from cores, drill cuttings, and the action of the drill) and the five observation wells.

G 551, the pumped well, has solid 24-in. casing to 29 ft, slotted 18-in. casing from 29 to 71 ft, and an open hole in the cavernous limestone from 72 to 84 ft. The four observation wells, OW 1-10, OW 1-50, OW 1-200, and OW 1-500, at respective distances of 10, 50, 200, and 500 ft from the pumped well, have solid 4-in. casing to 72 ft with an open hole in the cavernous rock to 84 ft. Well OW 1-10S, a second well 10 ft north of G 551, is 72 ft deep and has perforated 4-in. casing to 72 ft. The casing in this well is perforated with 3/8-in. holes drilled at 1-ft intervals for the entire length.

The test-well site is relatively free from canals or supply wells that might affect the water table. The nearest waterway of any importance is Snapper Creek Canal, which, at its closest point, is about 1.5 miles northeast of G 551 and is fairly well clogged with debris and water plants; there are no pumped wells of any consequence within several miles. Therefore, there could be no appreciable recharge from canals nor any interference from other pumping wells during a 5-hour test.

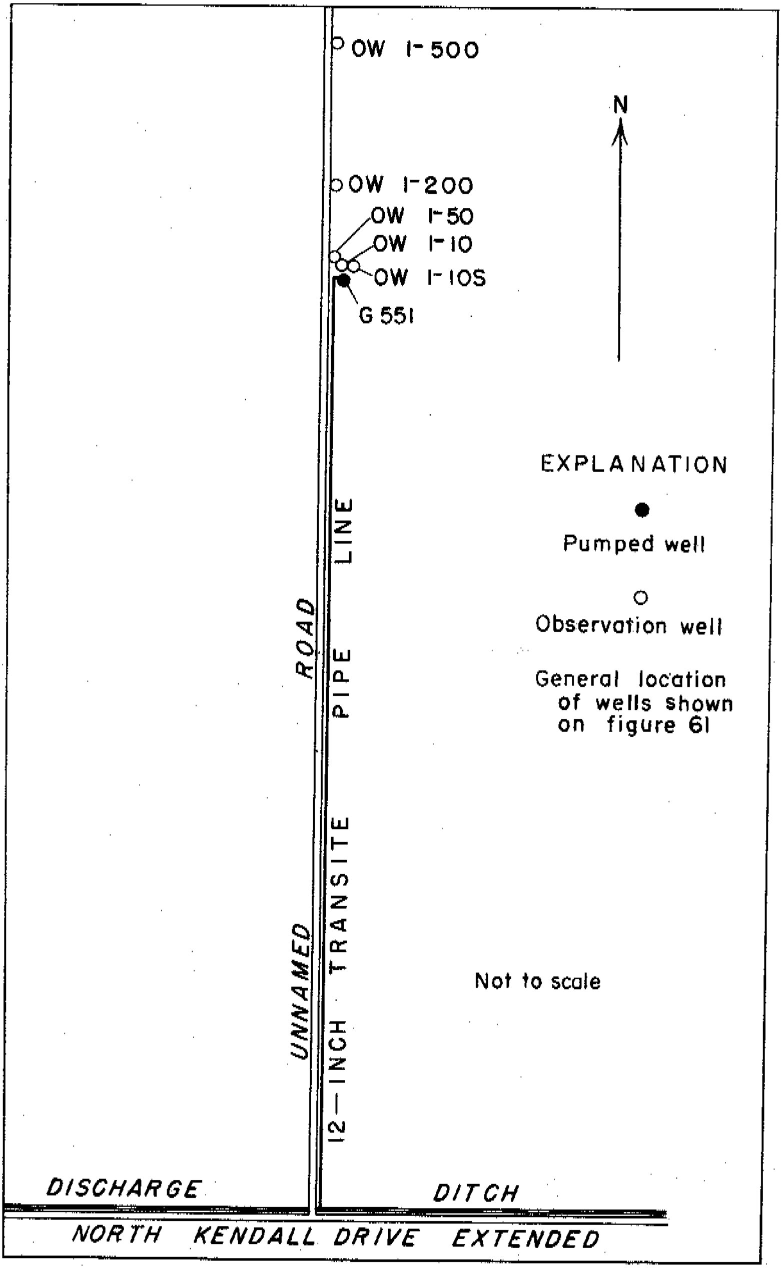

Figure 63. —Sketch showing locations of well G 551 and observation wells.

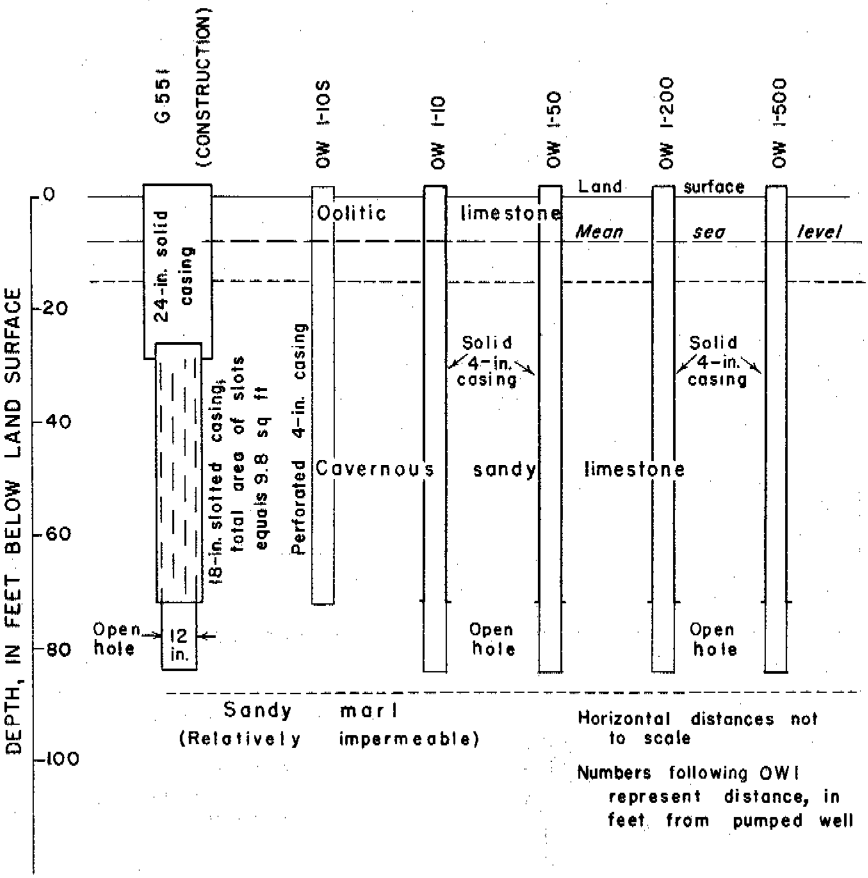

Figure 64. —Drawing showing method of construction of G 551, and logs of G 551 and observation wells.

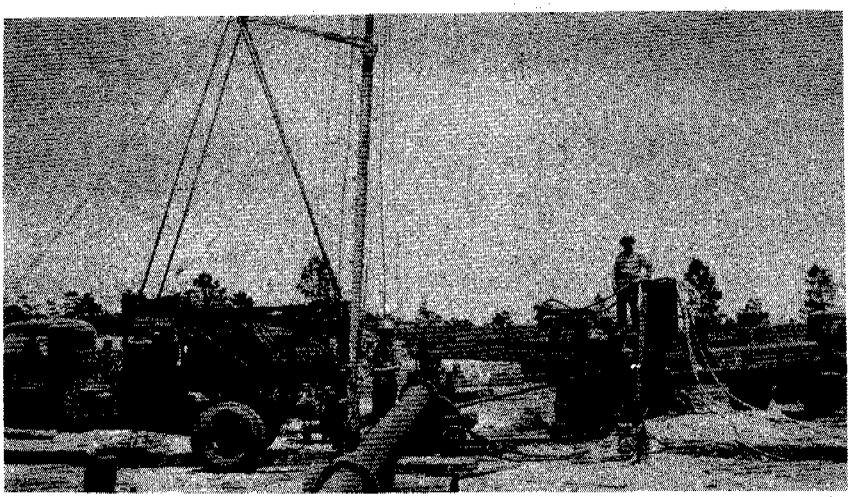

Figure 65. —Pump used in aquifer tests.

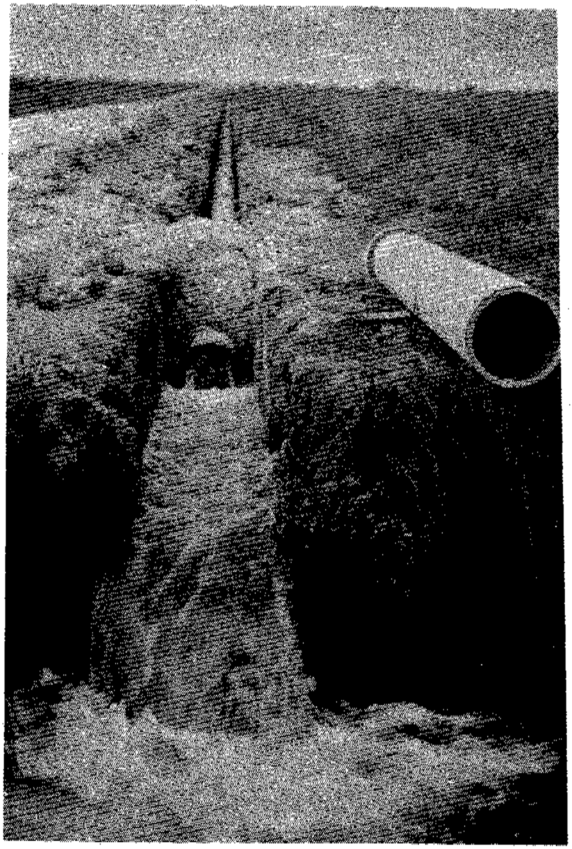

Figure 66.—Water discharging at a rate of 5 mgd from G 551 at the end of the 2,000-ft Transite line.

The pumped well was equipped with a deep-well turbine pump set at 32 ft, having a rated capacity of 3,520 gpm and powered by a 225-hp diesel motor (fig. 65). The water pumped from the well was discharged to the south through 2,000 ft of 12-in. Transite pipe into the western end of a shallow, narrow borrow ditch cut into the Miami oolite (fig. 66). The ditch, extending 1,320 ft east, is closed at both ends, and there was no surface flow into, or out of, the ditch during the test.

The pump was started at 10:05 a. m. and run until 12:05 p. m., when it was necessary to shut it down because of motor trouble. After waiting for the effects of the pumping to disappear, the test was started again at 2:15 p. m. This time the pump operated satisfactorily, and at 7:15 p. m. it was shut down.

The drawdowns used in making the computations were obtained during the period from 2:15 p. m. to 7:15 p. m. A small adjustment, varying from zero at the start to 0.013 ft at the end of the test, was made to correct for the residual drawdown caused by operating the well (G 551) from 10:05 a. m. to 12:05 p. m.

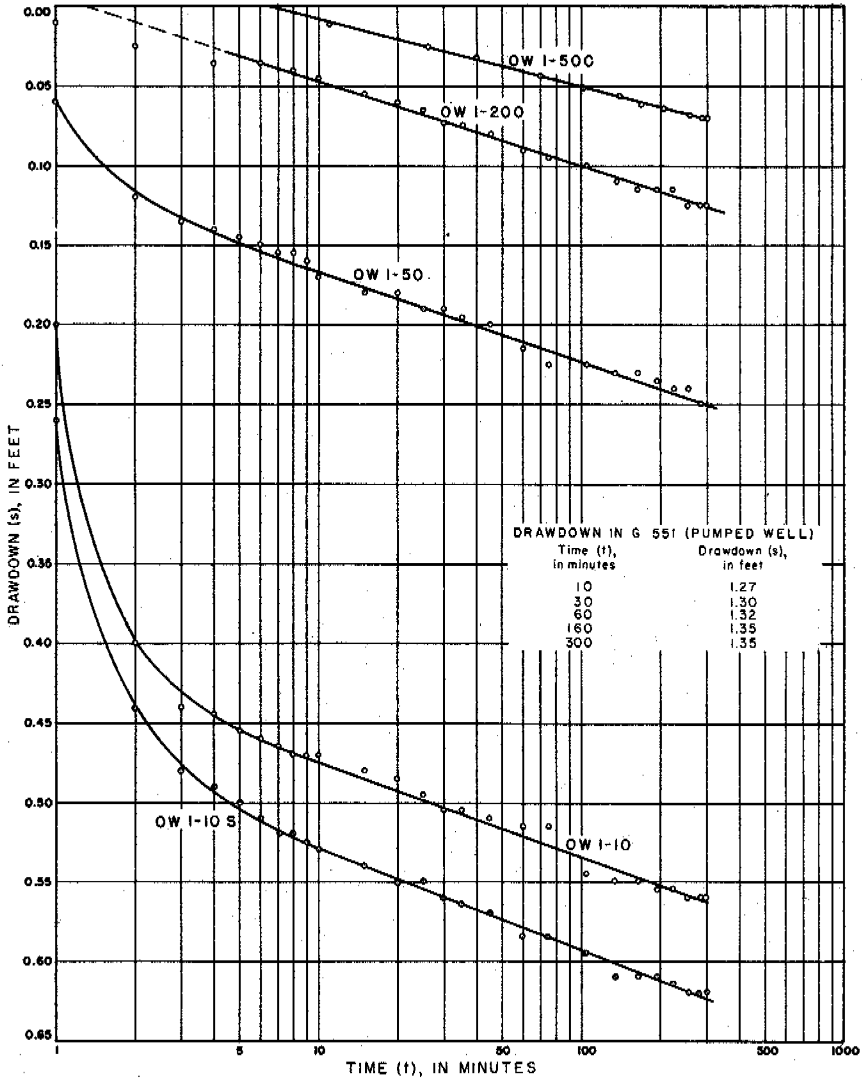

Figure 67. —Time-drawdown graph for pumping test of G 551.

Figure 67 shows a semilogarithmic plot of the corrected drawdowns versus time after pumping started in G 551. The difference in drawdown between wells OW 1-10 and OW 1-500 remained nearly constant from 20 minutes after the pump was started until the end of the test; this indicates that the cone of depression had approximately stabilized to a distance of at least 500 ft in the first 20 minutes of pumping.

Figure 68 shows a semilogarithmic plot of corrected drawdowns versus distance from the pumped well for 150 and 300 minutes

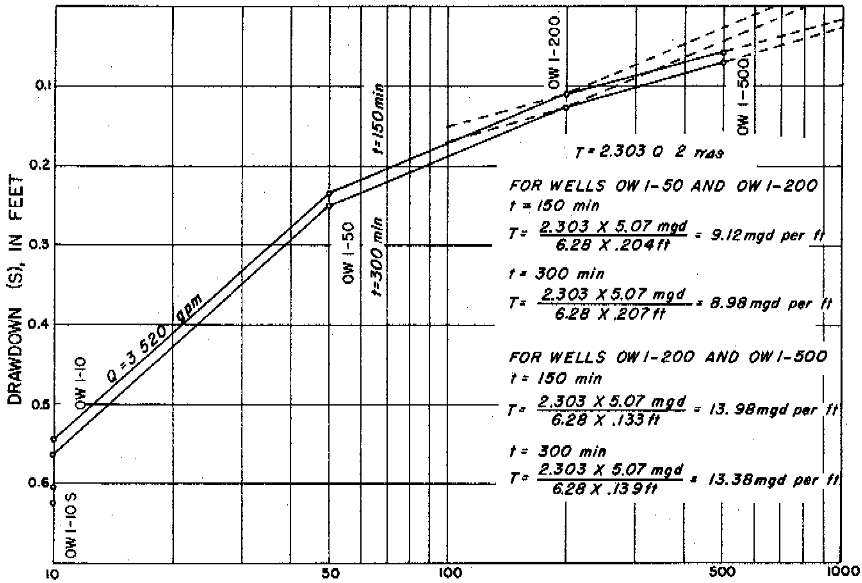

Figure 68.—Distance-drawdown graph for pumping test of G 551.

after pumping started. The points for the four wells do not fall upon a straight line as they should do for the theoretical aquifer composed of materials that are homogeneous, isotropic, and of indefinite areal extent. This indicates that the field conditions do not closely approximate the conditions upon which the test formula is based. The difference in conditions is believed to be due largely to the following causes: (1) Numerous solution holes and cavities occur in the aquifer, the largest of which is shown by the log of well G 551 to have a vertical dimension of 5 ft or more. The shape, size, and extent of these cavities are unknown; however, it is not uncommon for the interconnecting passages or solutional areas to range in cross section from less than a quarter of a square inch to many square feet. These interconnecting passages are rather indirect and tortuous, and many large cavities begin and end abruptly in a series of small tubular ones. Probably, most of the larger ones extend laterally for considerable distances from the pumped well and act as natural pipelines or collectors of ground water in a manner similar to laterals driven out from a collection gallery. Thus, the largest percentage of the pumped water probably moves into the well through the solution channels, not uniformly through the aquifer. Obviously, the location of any well with respect to a large cavity would have a marked effect on the drawdown occurring in the well. (2) Even the small solution holes are sufficiently large, and the velocities in them are so high, relatively, that they cause turbulent ground-water flow in the aquifer.

Assuming that the water entering the well approaches it through a section of aquifer 60 ft thick, that this section has an effective porosity of 20 percent, and that the flow is radial toward the well, pumping 5 mgd would give rise to ground-water velocities of about 0.12 ft per minute (nearly 180 ft per day) at a point 50 ft from the well. If the interconnecting passages are indirect and approach the well by tortuous routes, the actual velocities might be several times this amount. As the pumped well is approached the actual observed drawdowns increase at a faster rate than they should for a theoretical aquifer in which laminar flow occurs. These anomalous drawdowns indicate that turbulent flow is probably occurring at distances greater than 100 ft from the pumped well.

Computations based on the drawdowns in wells 50 and 200 ft from G 551 (fig. 68) give values for T of 9.12 and 8.98 mgd per ft after 150 and 300 minutes of pumping. If the difference between the drawdowns in the two wells were as little as 0.01 ft in error the calculated value of T would change about 8 percent. Computations based on the drawdowns in wells 200 and 500 ft from G 551 give values for T of 13.98 and 13.38 mgd per ft after 150 and 300 minutes of pumping. An error of 0.01 ft in the drawdown in these two wells would cause an error of 18 percent in the calculated value of T.

Using the drawdown in the wells 200 and 500 ft from G 551 and a value of T equal to 13.5 mgd per ft, the value of S after 150 minutes pumping is 0.27. This value of S appears to be too high, and for it to fall near the expected value, T would have to be higher.

From an analysis of all the data obtained from this pumping test (G 551) the coefficient of transmissibility is about 15 mgd per ft; however, the field conditions appear to differ too much from the assumptions on which the formula is based to allow a precise value of S to be calculated.

G 552 PUMPING TEST

On April 16, 1947, a 5-hour pumping test was made on well G 552 in an effort to determine the coefficients of transmissibility and storage of the aquifer for the general area in which this well is located.

G 552, constructed for the Department of Water and Sewers of the city of Miami, is about 1.5 miles north of Eureka Drive and about 300 ft east of Naranja Road (see fig. 61). It is in an area that is free from pumping effects of wells, or recharge or discharge effects from tidal canals. Figure 69 shows the location of this well with respect to nearby roads and observation wells.

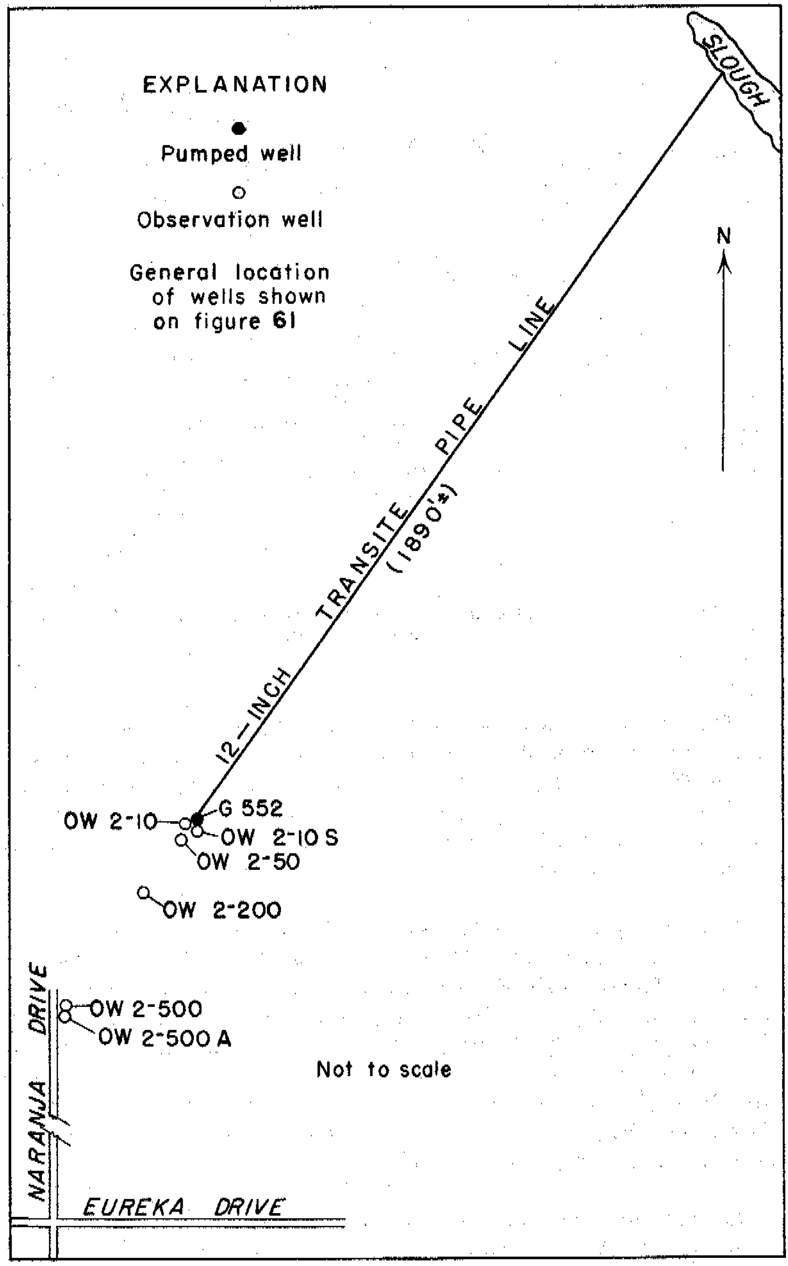

Figure 69. —Sketch showing location of well G 552 and observation wells.

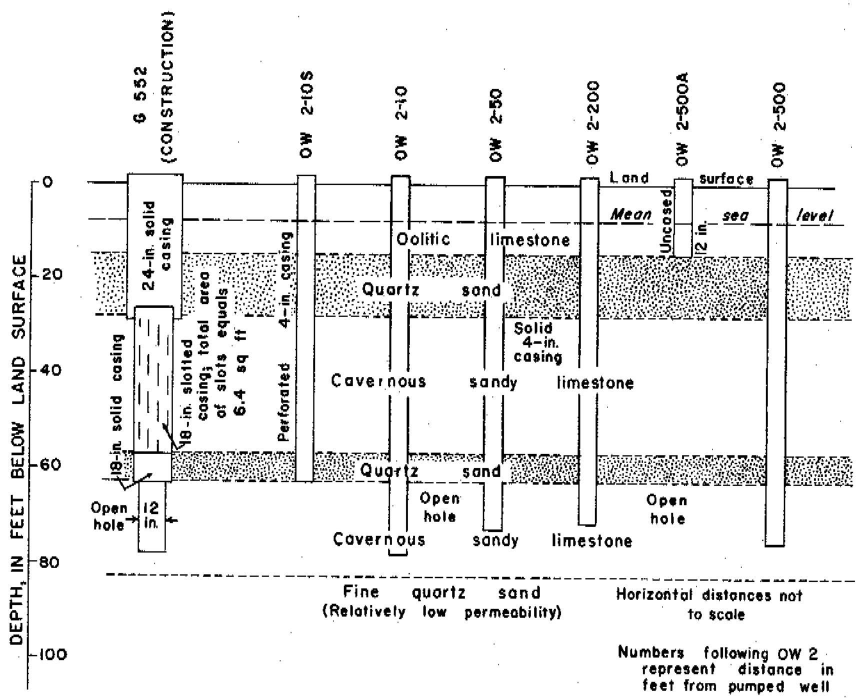

Figure 70. —Drawing showing method of construction of G 552, and logs of G 552 and observation wells.

Figure 70 shows the method of construction of well G 552 and contains the logs of G 552 and observation wells. Well G 552 has solid 24-in. casing to 29 ft, slotted 18-in. casing from 29 to 57 ft (the slots have a total area of approximately 6.4 sq ft), solid 18-in. casing from 57 to 63 ft, and about a 12-in. open hole in cavernous limestone from 63 to 79 ft.

Observation wells OW 2-10, OW 2-50, OW 2-200, and OW 2-500 are respectively 9.8, 50.0, 199.2, and 497.1 ft southwest of well G 552. These observation wells have solid 4-in. casing to 63 ft, and below this is 11 to 15 ft of open hole in the limestone. Well OW 2-10 S, 10.3 ft southwest of well G 552, has perforated 4-in. casing to 64 ft; the well is open to a depth of 63 ft. The 4-in. casing has 3/8-in. holes drilled at 1-ft intervals. Well OW 2-500 A, 504 ft southwest of G 552, is an uncased hole, about 12 in. in diameter and about 15 ft deep.

G 552 was pumped in the same manner and with the same equipment as G 551; the pump bowl was set at 32 ft. The average rate of pumping during the test was 3,540 gpm and the actual rate never varied more than 2 percent from this figure. The water pumped from the well was discharged to the northeast through 1,890 ft of 12-in. Transite pipe into a transverse glade. It is believed that no recirculation of water occurred.

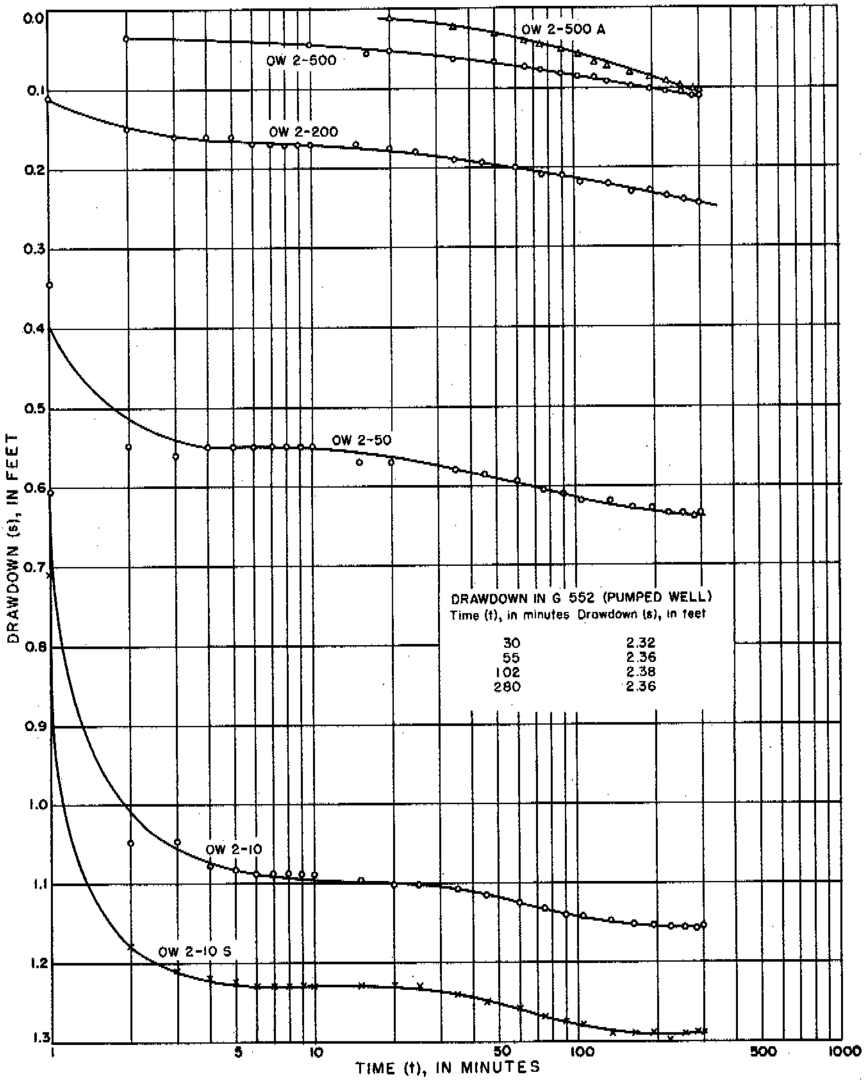

Figure 71.—Time-drawdown graph for pumping test of G 552.

Figure 71 shows a semilogarithmic plot of drawdown in G 552 versus time after pumping was started. The drawdown curves for wells 200 ft, and less, from the pumped well show that the water levels were nearly stationary for about 6 to 15 minutes after the test. This may have been caused by the removal of sand from solution channels, which thus increased their capacity to transmit water and decreased the actual drawdown enough to compensate for the anticipated increase in drawdown.

Figure 72 is a semilogarithmic plot of drawdown versus distance from the pumped well. Drawdowns in wells OW 2-50 and OW 2-200 give a value of T of about 2.84 and 2.85 mgd per ft, but drawdowns in wells OW 2-200 and OW 2-500 give a value of T of about 5.72 and 5.65 mgd per ft. The smaller value of T, obtained by using drawdowns in the nearer wells, appears to be too low. This may be due to turbulent flow that develops in the solution channels of the aquifer, and which requires a steeper gradient (greater Δs) than would be necessary if the flow were laminar (see p. 257). The values of T required to give a value of S equal to 0.20 (which appears to be the most reasonable value of S for the Miami area in general) for the observed drawdowns in wells OW 2-200, OW 2-500, and OW 2-500A, after 5 hours of pumping, range from about 6 to as much as 10 mgd per ft.

The data from this test corroborate the fact that the field conditions do not conform closely to the assumptions on which the test formula is based. Although the aquifer is composed largely of sandy limestone, it is not homogeneous or isotropic; pockets of sand and dense layers of limestone, together with the numerous solution cavities and channels, give rise to conditions impossible to account for mathematically. The solution holes and channels, which give rise to turbulent flow in the aquifer, are especially confusing. It is conceivable that if another line of observation wells had been available, the drawdowns observed (especially in the closer wells) would not have agreed precisely with those used and would have given a different calculated value of T.

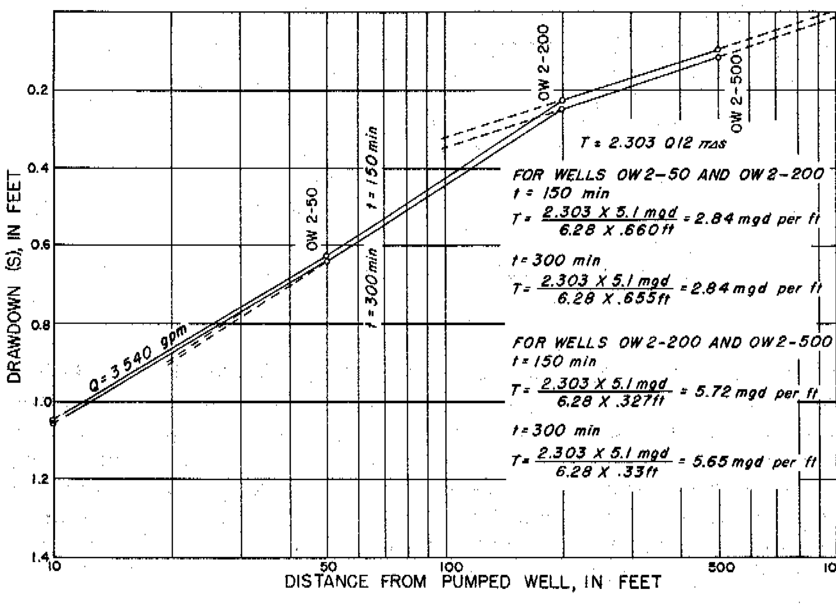

Figure 72. —Distance-drawdown graph for pumping test of G 552.

G 553 PUMPING TEST

On May 1, 1947, a 5-hour pumping test was conducted on well G 553, the third test well constructed for the Department of Water and Sewers of the city of Miami. This well is about 0.4 mile northwest of the little community of Howard (see fig. 61), in an area where no extraneous influences are felt, such as pumping from other wells or recharge or discharge due to canals. Figure 73 shows the location of this well on the south side of Motu Drive with respect to the observation wells. Figure 74 shows the method of construction of G 553, the logs of G 553, and the observation wells.

G 553 has solid 24-in. casing to 36 ft, a slotted 18-in. casing from 36 to 80 ft, and an open hole about 12 in. in diameter in the cavernous limestone between 80 and 91 ft. The total area of the slots in the slotted casing is approximately 10 sq ft. The 4-in. observation wells OW 3-10, OW 3-25, OW 3-50, and OW 3-100 are respectively 10, 25, 50, and 100 ft east of G 553. These wells were drilled to a depth of 85 ft and have perforated casing to a depth of 81 ft. The perforations are 3/8 in. holes drilled at intervals of 1 ft. The observation wells OW 3-200 and OW 3-500, located 200 and 500 ft east of well G 553, respectively, were drilled to a depth of 85 ft and have 4-in. perforated casing to 70 ft. The uncased portion of the hole in each of these two wells filled with sand shortly after construction, thus reducing their effective depth to about 70 ft. Observation well OW 3-250, located 250 ft east of G 553, was an uncased hole about 12 in. in diameter and about 16 ft deep; it was drilled to a depth of 34 ft, but the bottom 18 ft filled with sand.

Well G 553 was pumped in the same manner and with the same equipment as in the two preceding tests (see fig. 65). The water was pumped east from G 553 through 1,700 ft of 12 in. Transite pipe into the eastern end of a ditch emptying into a transverse glade. It is believed that no recirculation of the pumped water occurred.

On April 30, 1947, between 11:30 a. m. and 1:00 p. m., G 553 was pumped at a rate of 3,540 gpm for 90 minutes; then it became necessary to shut down the pump to replace a bearing. The next day, May 1, between 9:30 a. m. and 2:30 p. m., a 5-hour pumping test was made; the pumping rate was 3,540 gpm and did not vary more than 2 percent.

In the area of G 553, a layer of fine to medium quartz sand intervenes between the upper limestone (Miami oolite) and the lower limestone (Fort Thompson formation) in a geologic relationship similar to that at the site of S 1 in Miami Springs. (See p. 246.) The permeability of this sandy layer is lower than that of the limestone, and pumping causes a difference in recovery of water levels in deep and shallow wells.

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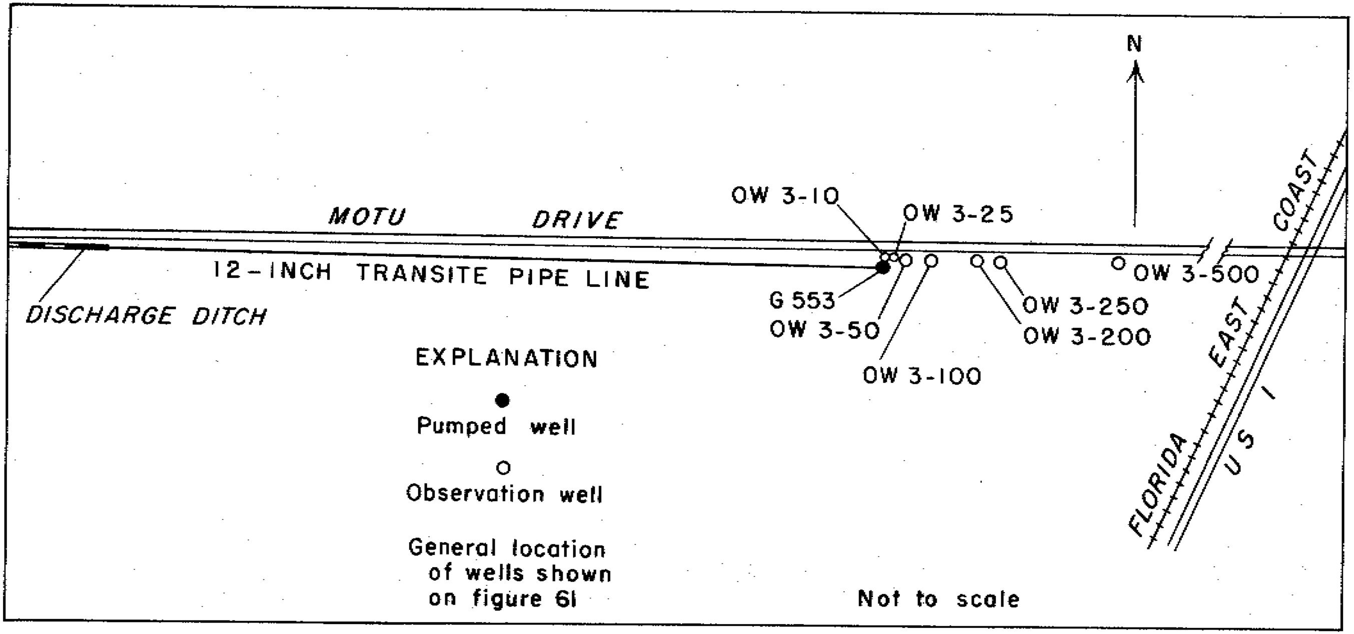

Figure 73.—Sketch showing location of G 553 and observation wells.

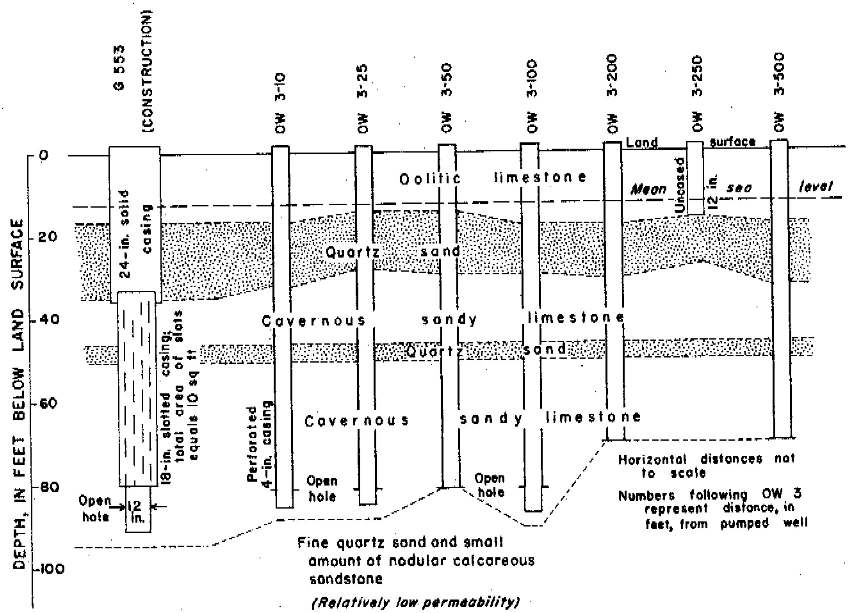

Figure 74. —Drawing showing method of construction of G 553, and logs of G 553 and observation wells.

Inasmuch as well G 553 was constructed with nonperforated casing to 36 ft (which excludes the sand layer and overlying oolite), all water pumped had to enter the well between depths of 36 and 91 ft. Thus, the water in the upper limestone, where the water table was located, reached the pumped well by infiltrating through the sandy layer.

Figure 75 shows a semilogarithmic plot of the drawdown versus time after pumping started for each observation well during the 5-hour test on May 1. From 10 minutes after the pump started until the end of the test the difference in drawdown between OW 3-10 and OW 3-500 was nearly constant; this indicates that the cone of depression, as observed in the deeper wells, had approximately stablized to a distance of at least 500 ft in the first 10 minutes of pumping.

The water level in the upper limestone, in which the water table is located, did not decline as much as that in the deeper limestone. The drawdown in the shallow well (OW 3-250) for the first 4 minutes was less than that for the deeper well (OW 3-500), which is twice as far from the pumped well. However, after the first couple of minutes the rate of decline in OW 3-250 was greater than that for any of the deeper wells. This indicates that water was still infiltrating from the water table through the sand to the lower part of the aquifer (in this portion of the cone of depression) at a greater rate than the lateral flow of water through the oolite.

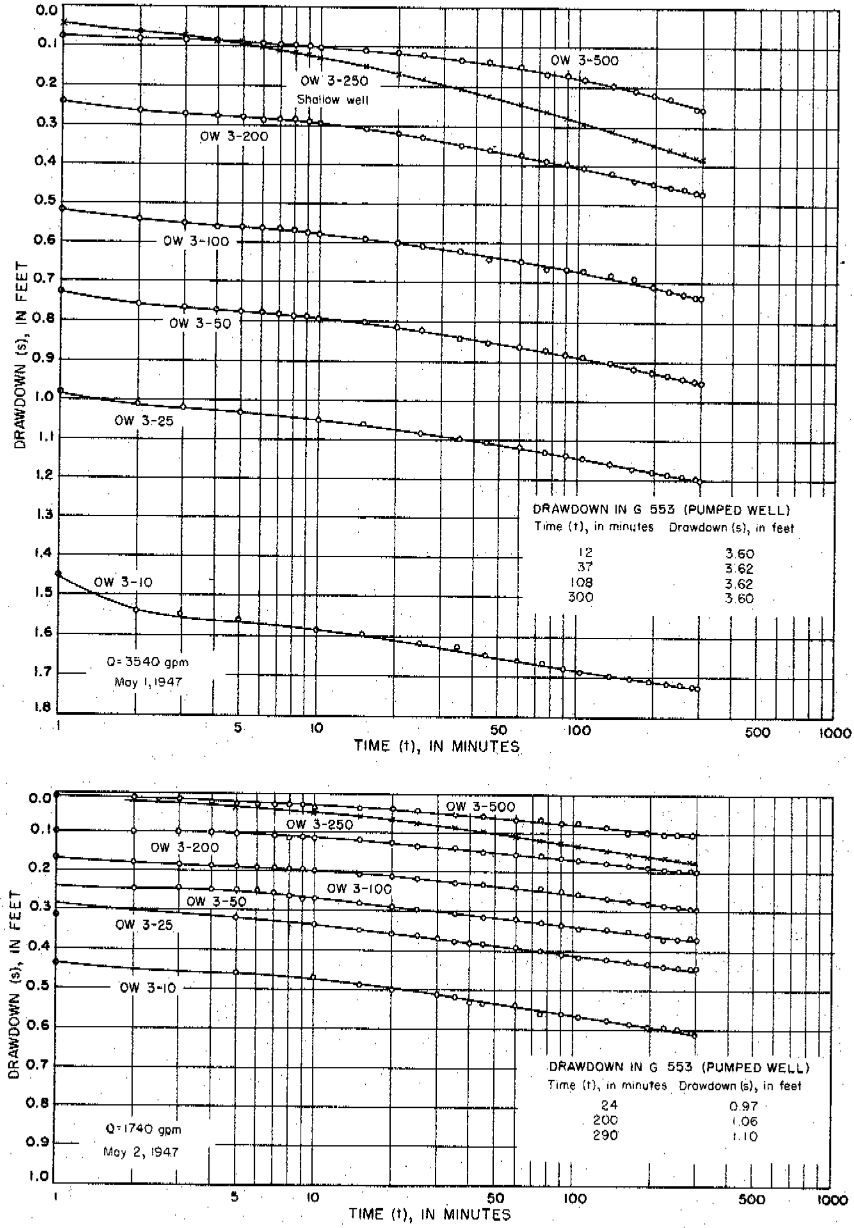

Figure 75. — Time-drawdown graphs for pumping tests at G 553.

On May 2, 1947, between 8:00 a. m. and 1:00 p. m., a second 5-hour pumping test was made on well G 553. For this test the rate of pumping was 1,740 gpm, or 49 percent of the rate on the preceding day. Figure 75 also shows the drawdown in the observation wells during this test.

The drawdowns at the two rates of pumping for both 5-hour tests are compared in table 18. The computed drawdowns at 3,540 gpm were obtained by multiplying the observed drawdowns at 1,740 gpm by the ratio of the two rates (3,540/1,740 = 2.03).

Table 18.—*Drawdown, in feet, for well G-553 and observation wells at end of 5-hour pumping test*

Drawdown	Well G 553	OW 3-10	OW 3-25	OW 3-50	OW 3-100	OW 3-200	OW 3-250	OW 3-500
Observed at 3,540 gpm............	3.60	1.73	1.20	0.95	0.74	0.47	0.38	0.26
Observed at 1,740 gpm............	1.10	.61	.45	.37	.30	.20	.18	.11
Computed at 3,540 gpm (= 2.03 x drawdown at 1,740 gpm)....	2.23	1.24	.91	.75	.61	.41	.37	.22
Observed minus computed, at 3,540 gpm.....	1.37	.49	.29	.20	.13	.06	.01	.04
Ratio of computed to observed, at 3,540 gpm.....	.62	.72	.76	.79	.82	.87	.97	.85

The ratio of computed to observed drawdown increases with distance away from the pumped well, but it is appreciably less than 1.00 at a distance of 500 ft from the pumped well. The fact that the ratio is less than 1.00 indicates that the actual conditions are not in strict accord with the assumptions on which the test formula is based, and it appears to corroborate previous data indicating that the ground-water flow is not strictly laminar, even at distances as great as 500 ft from the pumped well, when the rate of pumping is as high as 3,500 gpm.

Figure 76 is a semilogarithmic plot of drawdown versus distance from the pumped well for 150 and 300 minutes after the test started for both the 1,740 gpm rate and the 3,540 gpm rate. The value of *T* obtained from the 3,540 gpm rate, using all wells 25 to 500 ft from the pumped well, is about 2.5 mgd per ft and that for the 1,740 gpm rate is about 3.5 mgd per ft. The value of *T*, computed from the drawdowns in OW 3-200 and OW 3-500 after 5 hours of pumping at the 3,540 gpm rate, was 3.2 mgd per ft. After 5 hours of pumping at 1,740 gpm rate the value of *T*, determined from the same two wells, was 3.9 mgd per ft. Using a value of *T* equal to 3.5 mgd per ft, values of the coefficient of storage (S) were com-

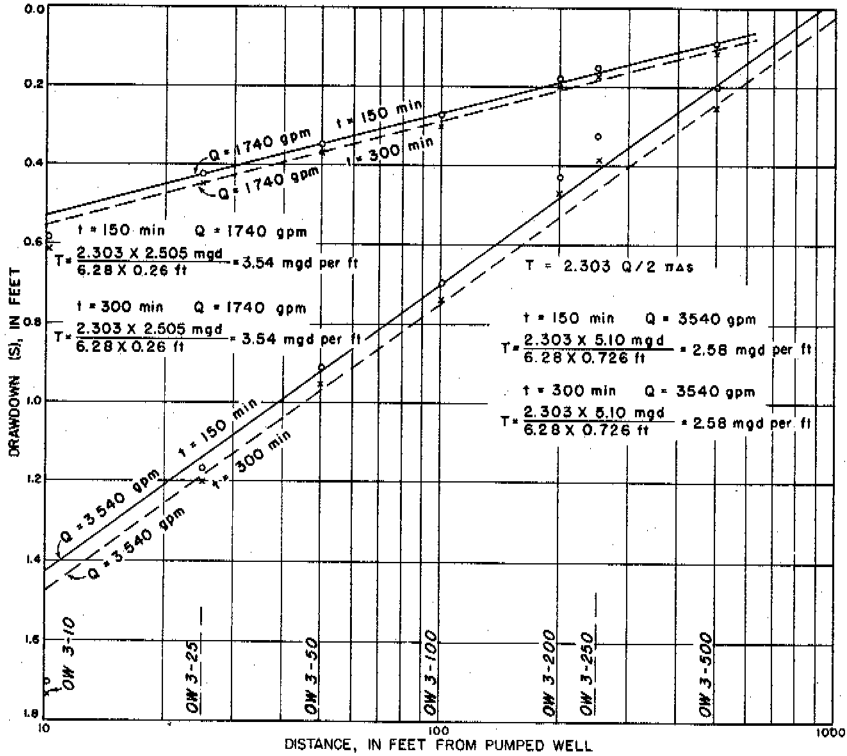

Figure 76. —Distance-drawdown graphs for pumping tests of G 553.

puted from the drawdown in wells located 100, 200, 250, and 500 ft from the pumped well, as shown in table 19.

Table 19.—*Computed values of S based on drawdowns after 5 hours of pumping and a value of T equal to 3.5 mgd per ft*

Rate of pumping (gpm)	Well OW 3-100	Well OW 3-200	Well OW 3-250	Well OW 3-500	Average
3,540	0.037	0.089	0.131	0.103	0.090
1,740	.134	.166	.164	.152	.154

The analysis of the data obtained from these tests on G 553 indicates that the value of *T* in that area is about 3.5 to 4 mgd per ft and that the value of *S* is equal to about 0.15. The fact that the full-rate pumping test produced drawdowns considerably more than twice that of the half-rate pumping test indicates that turbulent flow toward the well is occurring through the natural limestone tubes or solution channels in the aquifer, and that this flow is highly complicated and not subject to precise mathematical analysis. Nonetheless, the data are useful and give a general value for hydrologic coefficients.

CONCLUSIONS REGARDING T AND S FROM DATA OBTAINED FROM LARGE-SCALE PUMPING TESTS

The pumping tests on wells S 1, G 551, G 552, and G 553 are of value chiefly in indicating the productiveness of the Biscayne aquifer in several different locations. These tests, and other preceding smaller-scale tests, show that the aquifer in Dade County is highly productive everywhere along the coastal ridge and for a considerable distance into the Everglades; however, its productivity appears to vary considerably from one locality to another within this area.

The pumping tests discussed in the preceding pages indicate that actual field conditions under which the tests were made do not conform closely to the basic assumptions from which the Theis nonequilibrium test-pumping formula is derived. The differences are principally these: (1) The Biscayne aquifer is not homogeneous. Sand pockets or lenses, dense limestone layers, and cavities or solution channels occur. These cavities may extend considerable distances and may influence a pumping well in a manner similar to that of laterals supplying a collection gallery. (2) Even when no large cavities are encountered, the solution passages, common nearly everywhere in the Miami oolite and Fort Thompson formation, are large enough to allow turbulent flow to develop at distances as great as 500 ft from the pumped well (when the rate of pumping is as high as 5 mgd). (3) In some areas, layers of sand or dense limestone of much lower permeability intervene between the water table in the upper limestone (Miami oolite) and the lower limestone (Fort Thompson formation), from which water is pumped. It is assumed in the formula that the aquifer has the same permeability throughout its entire thickness and that, during pumping, the water level in a well ending in the top of the aquifer is the same as the level in a well ending near the middle or bottom of the aquifer, providing that all wells are the same distance from the pumped well. (4) In the formula it is also assumed that water in storage is released immediately as soon as the water table declines, whereas actually, when there is a sudden lowering of the water table, water may drain for several hours or days from the part of the aquifer that was saturated before pumping began. (5) Also, it is assumed that the pumped well penetrates the entire thickness of the aquifer and is able to receive water throughout the entire saturated thickness of the aquifer. Thus, it is assumed that there is no vertical convergence of flow lines as the pumped well is approached.

These divergences in the Biscayne aquifer from the basic assumptions on which the formula is derived make it difficult to obtain reliable values for the coefficient of storage (S). The values derived for the coefficient of transmissibility (T) are approximate, and T appears to range in value considerably from one location to another so that it is difficult to determine what the average value of *T* would be for an area of 100 or 200 sq miles.

From the four pumping tests just discussed S 1 and G 553 gave values of *T* equal to about 4 mgd per ft. The values of *T* at G 551 and G 552 are about 15.0 and 6.0, respectively. The higher value of *T* obtained from tests of well G 551 probably is due in large measure to the fact that most of the water moves into the well through solution channels in the limestone that are large enough to more than offset the added resistance of turbulent flow. When high pumping rates are used, water does not move uniformly through the aquifer to the well with laminar flow.

It is estimated on the basis of analysis of the water levels in well S 182 that annual ground-water runoff in that area is about 21 in., which, at an average stage of the water table for the area surrounding the well, gives a computed value of *T* equal to about 4 mgd per ft. This figure compares closely with the value obtained from the G 553 test, which was conducted about 3.8 miles away.

In view of the wide ranges in value of *T* (4 to 15 mgd per ft) obtained from these tests, the median (5 mgd per ft) was adopted as the best single value of *T* to use for the coastal ridge from Peters north to Hialeah.

A coefficient of transmissibility of 5 mgd per ft indicates a highly productive aquifer, which would compare favorably with an aquifer of the same thickness composed of clean, coarse, well-sorted gravel. As a matter of comparison, an 18-in. well in the Miami area, penetrating less than 100 ft of the Biscayne aquifer and pumping 5 mgd, has a drawdown of only about 2 to 3 ft; at Jacksonville, a typical well of equal diameter, penetrating about 500 ft of water-bearing limestone in the Floridan aquifer (p. 188), when pumped at the rate of 5 mgd, would have a drawdown about 10 times as great as that of the Miami well; and a well of equal diameter at Savannah, Ga., penetrating about 500 ft of water-bearing limestone in the Floridan aquifer where the coefficient of transmissibility is about 250,000 gpd per ft, would have a drawdown of about 50 ft. It is concluded that the Biscayne aquifer at Miami compares favorably in productivity with any aquifer in the world.

QUANTITATIVE STUDIES OTHER THAN LARGE-SCALE PUMPING TESTS

F 85 PUMPING TEST

The first effort to determine hydrologic coefficients of the Biscayne aquifer at Miami by means of a pumping test was made February 29, 1940. The well selected was a fire well, F 85, located at NW. 77th Street and 15th Avenue. It was 6 in. in diameter and 58.6 ft deep, with an open hole of unknown length in sandy limestone below the bottom of the casing. The well was pumped at a rate of 400 gpm over a $7\frac{1}{2}$-hour interval. Other fire wells in the vicinity were measured periodically, and it was found that the pumping had no effect on water levels in these wells.

As a result of this pumping it was decided that if any appreciable drawdown of the water table were to be measured, a considerably higher pumping rate would have to be employed, and measurements of drawdown would have to be made much closer to the pumped well. Accordingly, plans were made to obtain a pump capable of discharging 1,000 gpm (1.44 mgd) and to install 20 observation wells within a radius of 100 ft. Ten of these observation wells were to be of the same depth as the pumped well and the other 10 were to be about 15 ft deep, located beside each of the deeper observation wells. The water was to be discharged through 300 ft of 6-in. pipeline into a weir box and thence into a large rock pit west of the well. Two staff gages were to be installed in the rock pit, and one gage was installed in Little River Canal at the point closest to the pumped well. All measuring points were to be tied by levels into U. S. Coast and Geodetic Survey mean sea level datum.

Preparations were completed so that preliminary readings of water levels in the new observation wells, all fire wells in the vicinity, and staff gages in the rock pit and Little River, could be made on May 8, beginning at 11:00 a. m. Pumping started on May 9, but engine trouble caused a shut-down after 20 minutes of operation. Readings were discontinued until pumping started again at 6:00 a. m., May 10. Again the pump did not operate properly and readings were discontinued in the afternoon. On May 11, readings were begun again at 6:00 a. m., pumping was started at 10:35 a. m. and except for three shut-downs of about 5 minutes each, to allow for pump oiling, continued for 24 hours. Readings were discontinued at noon, May 12.

Discharge from the pumped well was approximately 950 gpm and, except for the period of the three shut-downs, varied only slightly. Vacuum-gage readings on the intake side of the pump fluctuated very little and showed no definite trend, being approximately the same just before pumping ceased as they were just after pumping started.

Drawdown data from all wells were plotted as hydrographs. They indicate that the water table was practically a plane surface in the vicinity of the pumping test and that it fluctuated slightly owing to oceanic tides in Biscayne Canal. The hydrographs also indicate that accidental errors, though small, are highly significant because of the small drawdowns involved. They are especially significant for those wells being used to determine the slope of the cone of depression, because the difference in water level (between distances of 50 and 100 ft from the pumped well) was only 0.015 ft.

The water levels in the 10 deep observation wells dropped a few hundredths of a foot almost instantaneously when the pump started and they rose the same amount when the pumping ceased. There was no apparent increase in drawdown in the 24 hours of the pumping test. The shallow wells responded more slowly and with even less drawdown. No effect of pumping was noted in any of the other fire wells in the vicinity, nor was there any conclusive evidence that the water discharged into the rock pit raised water levels adjacent to the pit; water-table profiles between the pit and the pumped well (F 85) showed no gradient from the pit.

Recovery at the end of the pumping test was plotted on semilogarithmic paper against distance from F 85. Of the 10 deep observation wells, six of them plotted in approximately straight lines; the other four wells showed such scattered points that they were disregarded. From the recovery curves of these six wells, the coefficient of transmissibility (T) was computed to be approximately 9.5 mgd per ft. This figure seemed excessively high, but it was realized that even very small observational errors would result in large errors in the computed values. However, large-scale pumping tests since that time have shown that considerably higher values for T may be valid for the limestone aquifer of this area (p. 270). Therefore, the 9.5 mgd per ft, obtained at F 85 is probably correct.

G 218 PUMPING TEST

G 218, an exploratory test well 202.5 ft deep, had been drilled in August 1941, in the Everglades, 6 miles west of the Miami well field. As with all such test wells, G 218 was pumped at successively deeper levels as drilling proceeded to obtain data on the quality and quantity of water at different depths. The data showed that this well penetrated the Biscayne aquifer at a point where it is typically composed of permeable sandy limestone and calcareous sandstone with occasional beds or pockets of quartz sand. The most permeable part was between about 60 and 75 ft below the land surface, where the water was very similar, chemically, to that obtained from the Miami well field.

Early in 1945 it was decided that a pumping test should be made in that area, not only to determine the hydrologic coefficients of the aquifer, if possible, but also to pump the well long enough and at a high enough rate that ground water would be drawn in from a considerable distance; thus, if any change in quality were to occur, it could be ascertained by chemical analyses of the collected samples.

Accordingly, plans were made to plug the well at about 72 ft, blast the casing open slightly above that depth, then surge and pump the well to develop maximum efficiency. This was done in March 1945, and an excellent well was obtained. Following this, 10 observation wells, $2\frac{1}{2}$ in. in diameter, were drilled to the same depth as G 218, that is, to a depth of 72 ft. All wells were finished with open holes below the casing, ranging from about 5 to 10 ft. One line of wells extended north-south along the east bank of Snapper Creek Canal, which was then practically dry. These observation wells were drilled at the following distances from the pumped well: G 358, 1 ft; G 359, 25 ft; G 360, 50 ft; G 361, 100 ft; G 362, 200 ft; and G 363, 2,200 ft. The other line of wells extended east-west through G 218 with one well, G 364, being 25 ft west of G 218 in Snapper Creek; the rest of the wells of this line extended east of G 218 as follows: G 365, 25 ft; G 366, 50 ft; and G 367, 100 ft.

To make certain that no recirculation of pumped water occurred, a solid earth dam was constructed across Snapper Creek Canal at a point slightly in excess of 2,200 ft north of G 218, and a 12-in. Transite pipeline was laid to discharge the water beyond (north of) the dam. Staff gages were then set in Snapper Creek Canal on both sides of the dam and these, together with all observation wells, were tied by levels to U. S. Coast and Geodetic Survey mean sea level datum.

A 6-in. centrifugal pump, rated at 1,450 gpm when 20 ft above water and pumping against a total head of 50 ft, was attached to the well casing. Discharge was into the 12-in. pipeline where it was measured by a pitometer located far enough from the pump that flow disturbances would be unimportant.

Pumping began on April 7, 1945, at 9:50 a. m., and continued, except for occasional interruptions due to engine trouble, until April 25 at 9:30 p. m. During this period, the pump operated efficiently for a total of 189 hours and pumped a total of approximately 14.2 million gallons. The average rate of pumpage was 1,250 gpm, although at times the rate was as high as 1,350 gpm.

Because the pump operated discontinuously, the frequent starts and stops probably did not always allow the water table to regain equilibrium, and because the observed drawdowns were very small, and because occasional light showers had some recharging effect on the water table, it is believed that the values obtained for the coefficient of transmissibility are not precise, even though corrections were applied.

The most satisfactory part of the test occurred from 9:50 a. m., April 16, to 10:00 p. m., April 18. During this time, pumping was continuous for 60 hours and 10 minutes. No pumping had been done previously since 6:00 p. m., April 13, and the water table on

April 16 was a plane surface apparently not affected by previous pumping. Distance-drawdown graphs gave values for *T* equal to 4.4 mgd per ft for the north-south leg between wells G 359 and G 361, and 3.9 mgd per ft for the east-west leg between wells G 365 and G 367. Thus, it is believed that *T* is about equal to 4.0 mgd per ft in the G 218 area. No satisfactory value for *S* could be computed.

The chemical analyses of the water, made by chemists of the city of Miami Department of Water and Sewers, showed that no change in the quality of the water occurred with pumping. The dissolved chemical constituents, temperature, pH, and color remained essentially uniform throughout the test and were very similar to those of raw water from the Miami well field in Miami Springs.

STUDIES OF CONE OF DEPRESSION IN MIAMI WELL FIELD

Pumping from the Miami well field in Miami Springs produces a rather wide, shallow cone of depression averaging about a mile in radius and 2 to $2\frac{1}{2}$ ft in depth at the center during the dry season. (See figures 32–34.) Water from this field is measured by venturi meters in the Water Works Plant in Hialeah, and a record of pumping time is kept for each individual well. By use of average water-table slopes at different distances from the center of the cone, the coefficient of transmissibility was computed using the Thiem formula (Wenzel, 1942, p. 81). These computations were made throughout the year, under both wet and dry conditions, so that deep, intermediate, and shallow cones of depression were represented. Values of *T* ranged from 2.35 to 4.9 mgd per ft and averaged about 3.5 mgd per ft. This compares with the value for *T* equal to about 4 mgd per ft obtained from data gathered at the S 1 pumping test (p. 239-248). S 1 is one of the wells in the Miami well field. These values give a check on the methods used for determining *T* in this area and indicate a fairly close correlation.

CANAL INFLOW STUDIES

Darcy's law states that the rate of flow through porous media is directly proportionate to the hydraulic gradient. This law is utilized in the expression $Q = PIA$, in which Q equals the quantity of water discharged in unit time, P equals the permeability factor, I equals the hydraulic gradient, and A equals the cross-sectional area through which the water is discharged.

In field conditions, discharge through an aquifer past a line, such as a water-table contour, may be reckoned by the formula $Q = TIW$. In this formula, Q equals the ground-water discharge in gallons per day; T equals the coefficient of transmissibility; I equals the hydraulic gradient in feet per mile; and W equals the length, in miles, of the line across which the ground-water flow is measured.

A reach of canal suitable for an inflow study would be one that is fairly straight, with an appreciable inflow between upstream and downstream measuring points, and with all the inflow contributed by ground-water seepage. In the Miami area, suitable sites for such studies are very limited because of lateral canals, surface inflow, small ground-water inflow, and tidal influences. The only suitable area is in the vicinity of Opa Locka between LeJeune Road and NW.

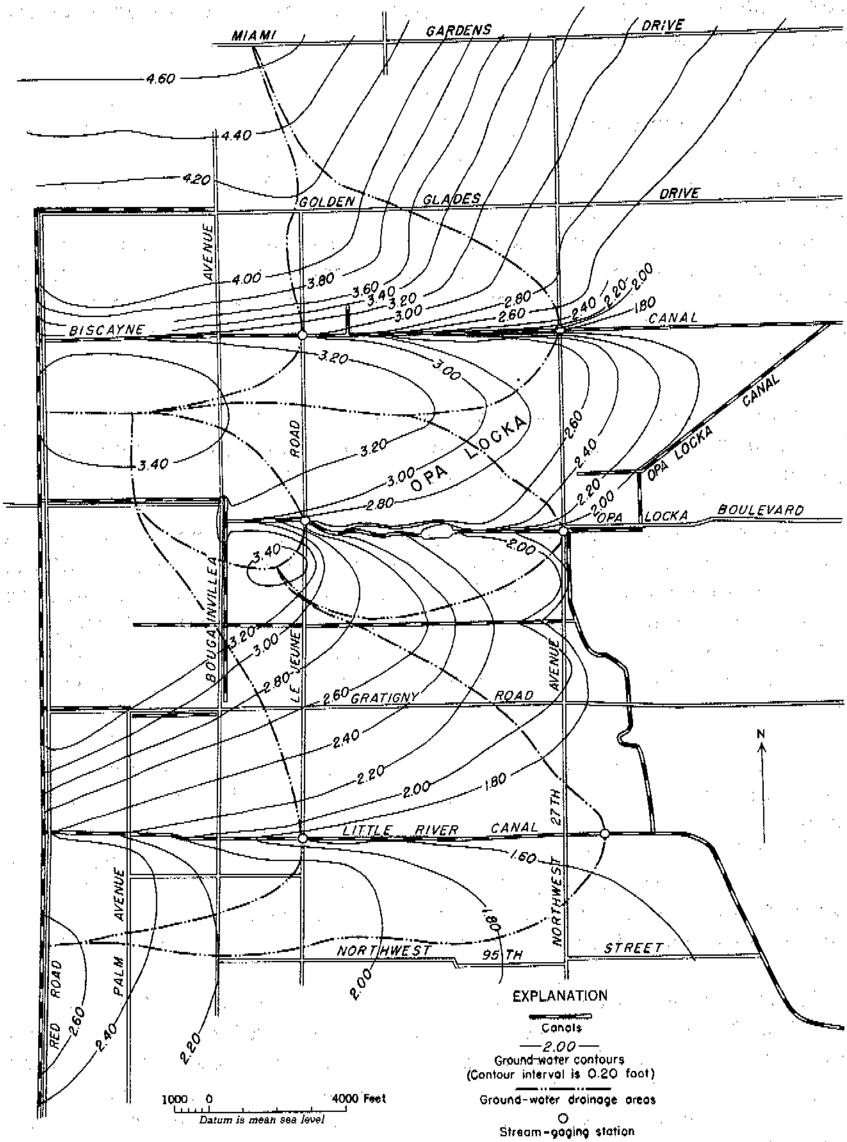

Figure 77.—Map of Opa Locka and vicinity showing water-table contours and ground-water drainage areas on August 29, 1940.

27th Avenue on the Biscayne, Opa Locka, and Little River Canals. However, this area is not ideal (see fig. 77) because low flow, tides, and weeds in the canals make stream gaging difficult; furthermore, these canal stretches occur in an area that is largely underlain by quartz sand instead of the usual highly permeable limestone. Such materials would have a lower value of T than would be found elsewhere in the Miami area.

All observation wells, measuring points, and staff gages were tied in to mean sea level datum and were measured at hourly intervals over a 13-hour tidal cycle. Streamflow measurements were made hourly at both the upper and lower ends of each of the canal reaches used in these studies.

Mean flow past each gaging station was computed. Water-table maps were constructed (fig. 77), based on mean water-level readings over the tidal cycle corrected to mean sea level datum. Drainage boundary lines, located by means of contours on the water table, were then drawn on the water-table map. These delineated drainage areas were then planimetered, and areas between the contours of each drainage area also were determined. The average distance of each contour from the canal was measured and the length of each contour within the drainage areas was measured also.

From these data, an average profile of each drainage area was drawn, and diagrams of the width of the drainage areas were plotted against distances from the canal.

Studies of these ground-water profiles indicate large head losses near the canals. These losses are the tangible evidence of the crowding of the flow lines as ground water moves from the aquifer out into the canals. It should be recalled that the water-table profile used is not an actual one taken normal to a given canal, but instead, it is built up by using average distances of each water-table contour from the canal.

The flow past any given contour is: $Q = TIW$. From the profile and width diagrams, the water-table slope, I, and the element of drainage area width, W, can be determined, and a diagram of slope-width factor can be drawn. Since the total discharge, Q, is known only at the canal, it is necessary to assume that ground-water discharge is uniform throughout the area. This assumption makes the slope-width factor related linearly to the drainage area.

These studies give values of T ranging from 1,458,000 to 1,970,000 gpd per ft. It is believed that these values may be fairly reliable for the areas tested; they are somewhat lower than values for T of the highly permeable limestone areas. However, this is to be expected because the sand is not so permeable as the limestone.

FACTORS TO BE CONSIDERED REGARDING THE PROPOSED WELL FIELD IN THE MIAMI AREA

GENERAL ANALYSIS

The Department of Water and Sewers of the city of Miami plans to develop a well field with a capacity of 50 mgd in sec. 36, T. 54 S., R. 36 E. This site is about $\frac{1}{2}$ mile west of the western border of the coastal ridge on the eastern border of the Everglades. The center of the proposed well field will be about 1.5 miles southwest of Snapper Creek Canal near the site of well G 551 (see fig. 61). In this area, an average stage for Snapper Creek Canal is approximately 5 ft above sea level. In its present condition, the canal just west of the Seaboard Air Line Railroad is not very effective as a drainage canal because it is relatively shallow and contains a heavy growth of aquatic plants (mosses and rushes) that offer considerable resistance to the flow of water. From the vicinity of Miller Drive, Snapper Creek Canal flows northward into the Tamiami Canal, and from the vicinity of the Seaboard Air Line Railroad the canal flows eastward into Biscayne Bay.

The land surface in the area of the proposed well field averages about 6 to 8 ft above mean sea level and is about 3 to 5 ft lower than the coastal ridge to the east. At an average stage, the water table is about $5\frac{1}{2}$ ft above mean sea level and about $1\frac{1}{2}$ to 2 ft below the land surface.

It appears that the water pumped from a well field in this area would be derived from the following sources: (1) Water salvaged from evapotranspiration due to lowering of the water table substantially below the base of any muck present; (2) water supplied to the well field by infiltration from canals and by salvaging ground water that would have been discharged into canals, and thence into the sea, if the well field had not been in existence; and (3) water diverted from natural ground-water discharge areas. A part of the water pumped will be put back into the ground through drainage wells, septic tanks, and irrigational uses in the areas in which it is consumed. This will tend to decrease part of the total quantity of water diverted from natural discharge and will assist in maintaining a fresh-water head to prevent salt-water encroachment.

The question of primary importance is whether sufficient fresh-water head can be maintained in the Biscayne aquifer between the bay and the well field, at an average stage of the water table, to prevent salt water encroaching westward at depth in the aquifer.

Under conditions of high water levels (shown by fig. 47) when the glades are flooded, water will be readily available to recharge the aquifer because more than half the cone of depression created by pumping in the well field will be under the flooded glades area.

As there are no impermeable layers to prevent this surface water from recharging the aquifer, the cone of depression cannot expand to any great extent, because, as the water near the well field moves downward, water from adjacent flooded areas will flow in to replace it. It appears obvious that if the glades were continuously flooded, the water table would be high enough (because of a continuously high recharge to the aquifer) to prevent any encroachment of salt water from the sea. If a wet period followed a dry period, when the water taken from storage in the aquifer would cause the cone of depression to expand for a distance of several miles, most of the water taken from storage would be quickly replaced by downward infiltration from the flooded glades. The recharge of the aquifer in this manner would result in the salvage of some surface water that otherwise would have been discharged into the sea.

During times of drought, such as occurred in 1945 (see fig. 45), the water table over most of southern Dade County did not have sufficient fresh-water head above the average water level in Biscayne Bay to prevent sea water from encroaching inland at depth in the aquifer (see Ghyben-Herzberg principle, section on Salt-water encroachment, p. 591-592). Although the salt water moves inland at depth in the aquifer under low water-table conditions, the rate of advance, owing to the extremely low gradient causing encroachment, is so slow that the total advance of the salt-water front during 3 or 4 months of extremely low water-table conditions is not likely to exceed several hundred feet. As the water table rises (a result of recharge from rainfall), the rate of advance is decreased, and if recharge continues, the advance of the salt-water front will be stopped; if high water-table conditions are maintained for several months, the salt-water front may be flushed seaward beyond its original position. Thus, so long as hydrologic conditions are not greatly changed by artificial means, the inland and seaward movements of the salt-water front in the permeable limestone aquifer probably will be restricted to a zone less than ½ mile wide. However, ground-water conditions can be changed by artificial means, such as the construction of new canals or the deepening of existing ones (which would drain off additional large quantities of ground water into the sea), or by heavy pumpage from wells. Under the new conditions, if the average yearly stage of the water table becomes appreciably lower, the salt-water front will move inland until it reaches a new position that will be in equilibrium with the average water-table stage.

The water table in the area of the proposed well field ranges between 1 and 9 ft above mean sea level. As the stage changes, the balance between the fresh- and salt-water heads changes, thus causing the salt-water front to fluctuate back and forth through its mean position. Because of the slow movement of water through the Biscayne aquifer (estimated to be about 2,000 ft per year when the gradient is 1 ft to the mile), the salt front never has sufficient time to attain a position of equilibrium for either the high or the low water-table stages. Figure 78 indicates the position of the contact between fresh and salt water for the average water-table stage, according to the Ghyben-Herzberg principle.

Figure 78 also illustrates estimated average conditions after the well field has been in operation for several years. This profile was obtained by subtracting from the average water-table profile,

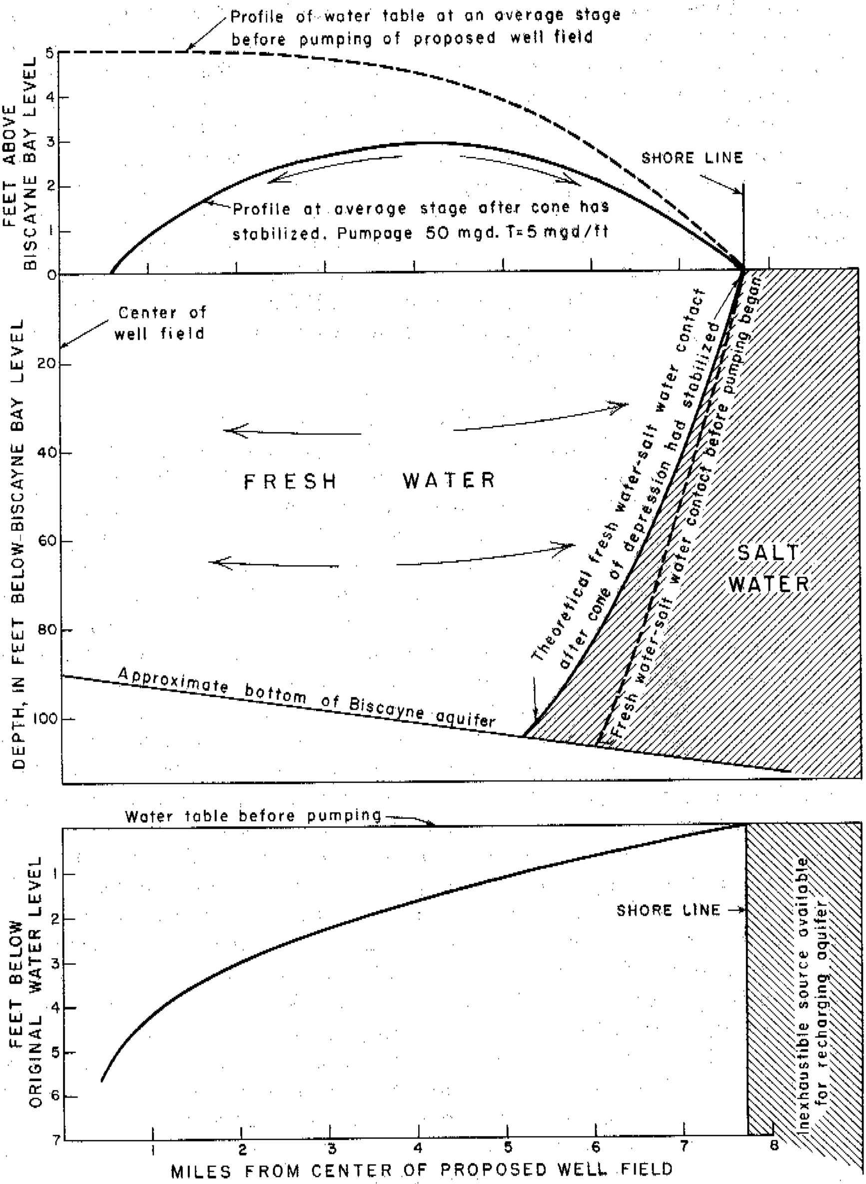

Figure 78. —Stabilized profiles of water table between shoreline and center of proposed well field.

prior to the development of the proposed well field (dashed line), the equilibrium drawdowns due to pumping 50 mgd in an aquifer having a coefficient of transmissibility of 5 mgd per ft. The center of pumping is a perpendicular distance of 7.7 miles from an infinite line source of recharge (Theis, 1935, p. 519–524; Muskat, 1937, p. 175), which is approximated in reality by Biscayne Bay water level (see p. 227) along the shoreline. This is illustrated by diagram in the lower portion of figure 78.

Under these conditions, which were established by pumping from the proposed well field, the maximum fresh-water head above Biscayne Bay level (between the well field and the shore) is about 3 ft and occurs about halfway between these two locations. A fresh-water head of 3 ft is sufficient to displace the salt water to a depth of 120 ft. At a depth of 100 ft, relatively impermeable material (Floridan aquiclude), which offers much greater resistance to the movement of water, is encountered in this area. Figure 78 shows the theoretical position of the contact between fresh and salt water that would be in equilibrium with the average water-table profile after the proposed well field has been operating for several years. The dashed line shows its position prior to development of the well field.

For several reasons it is believed that the drawdowns shown are greater than would occur in actuality: (1) The center of the well field is approximately 1.5 miles from Snapper Creek Canal at its nearest point. As mentioned previously, the flow in Snapper Creek Canal is to the north and southeast from this area, but after the new well field is in operation it is expected that the water table will average 1 to 1.5 ft lower in the vicinity of the bend in Snapper Creek Canal. The water surface in the canal will lower as the area water table lowers, and thus the canal will salvage much of the water that formerly flowed out of the area and probably it will reverse its flow. Because Snapper Creek Canal is part of an extensive network of canals, water can be fed into the area by canal flow from considerable distances. It is essential however, that there be suitable gates and locks on these canals to prevent inland flow of salt water from the bay during dry periods when the water table is below that in Biscayne Bay. If Snapper Creek Canal should be cleaned out and deepened, it is especially important to prevent salt water from flowing into the eastern part of the canal from the bay or ocean. (2) A second reason for smaller drawdowns than are shown in figure 78 is that lowering of the water table, by pumping, substantially below the base of any muck present, will reduce evapotranspiration losses. (3) During periods of drought the cone of depression about the well field continues to expand. The rainy periods that follow must refill the cone of depression before glades in this area can become flooded; this will salvage some of the water that formerly flowed out of the area by surface runoff through transverse glades and canals.

It is believed that the water salvaged from evapotranspiration, water salvaged from out-flowing canals, and water now being fed into the area through the canal system, will account for more than 50 percent of the 50 mgd that is expected to be pumped from the new well field, thus requiring less than half of this amount to be diverted from natural discharge areas in Biscayne Bay. Drawdowns in figure 78 are based on the total pumpage being diverted from natural discharge areas in Biscayne Bay. Therefore, the drawdowns would be expected to be considerably less than those indicated, which should increase the maximum fresh-water head (as shown by the profile) from 3 to 5 ft, or more, above the average Biscayne Bay water level.

THEORETICAL DRAWDOWNS IN AN INFINITE AQUIFER

It is possible by means of the Theis nonequilibrium formula (p. 238) to compute the theoretical drawdowns that would occur for a pumpage of 50 mgd in an aquifer of infinite extent having a coefficient of transmissibility of 5 mgd per ft and a coefficient of storage of 0.20. As discussed on p. 270, the above coefficients appear to be about average values for the general area of the proposed well field.

Figure 79 shows the theoretical drawdowns at various distances from the pumped well and for various times after pumping has started. This figure may be of value in estimating the drawdown that would occur during a rainless period after pumping starts, before the cone of depression has spread out to recharge areas (canals and flooded-glade areas). If turbulent flow develops in the aquifer as the pumping well is approached, the drawdowns will exceed those shown. Turbulent flow may develop as far as 500 ft from a well pumped at the rate of 5 mgd. For a single well with a capacity of 5 mgd, the drawdown would be one-tenth of those indicated by figure 79.

CONE OF DEPRESSION IN AN AREA IN WHICH THE AQUIFER IS UNIFORMLY RECHARGED

It is possible to compute the theoretical drawdown for a stabilized cone of depression in an ideal aquifer that is continuously recharged at a uniform rate. The transmissibility of the aquifer, the pumpage from the well, and the rate of recharge will determine the shape and extent of the cone of depression. The cone will spread until its circle of influence has expanded sufficiently to divert toward the well a quantity of water equal to that pumped from the well. For conditions to stabilize, the effective recharge within the circle of influence must equal the pumpage from the well. The effective recharge may be defined as the recharge to the water table minus evapotranspiration from the water table. For a given

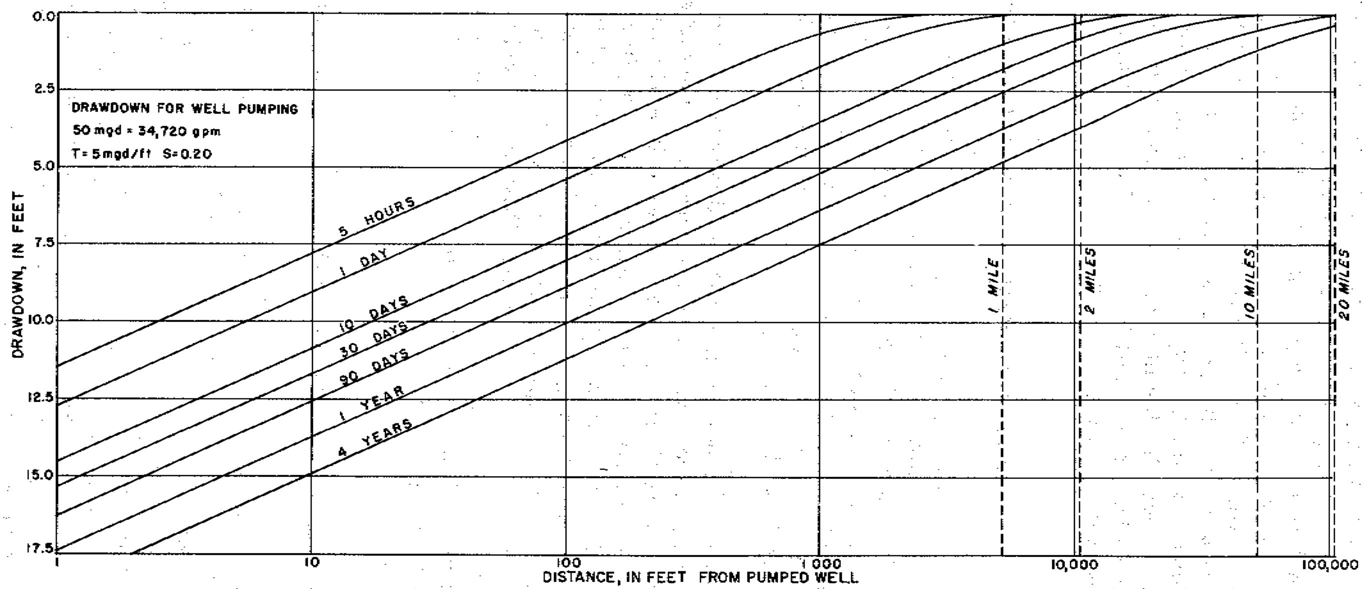

Figure 79. — Theoretical drawdowns in relation to distance from pumped well and time after start of pumping.

pumpage, the size of the circle of influence may be assumed and the recharge (in inches per year) may be calculated, or, if the effective recharge is known or assumed, the size of the circle of influence may be calculated.

As the radius of the cone of depression is increased from a well that has been continuously pumped at a steady rate for a long time, a point is reached at which the drawdown, due to pumping from this well, is zero. This distance is said to be the radius of influence. Beyond the circle of influence the drawdown is zero, but within this circle of influence the drawdown increases as the well is approached. A level water table, prior to the time pumping began, is assumed.

This drawdown curve may be approximated by the following method. Determine the increase in drawdown for various decrements in radius from the pumping well, using the following formula and a decrement that is a small percentage of the radius of influence.

$$s_1 - s_2 = \frac{527.7 q \log \frac{r_2}{r_1}}{T} \left[\frac{R_0^2 - r_1^2}{R_0^2} \right]$$

where:

r_2 = the distance in feet from the pumping well to point in question on the cone of depression;
r_1 = the distance in feet from the pumping well to a point on the cone of depression such that r_1 is less than r_2;
R_0 = radius of influence in feet;
s_2 = drawdown in feet at r_2;
s_1 = drawdown in feet at r_1;
q = well discharge in gallons per minute;
T = coefficient of transmissibility in gallons per day per foot (assumed to be constant);
$s_1 - s_2$ = equals the increase in drawdown between r_2 and r_1,

$\frac{R_0^2 - r_1^2}{R_0^2}$ equals the percentage of the circle of influence, over which recharge is uniformly occurring, that is farther from the well than r_1. By starting with r_2 equal to R_0 and making the increment between r_2 and r_1 equal to 1/10 or 1/20 of R_0, the drawdown curve to the point in question may be closely approximated by summation of the increments of drawdown from R_0 to the point in question.

Assuming a circle of influence 7 miles in radius, the drawdowns for various distances from the pumping well or well field were

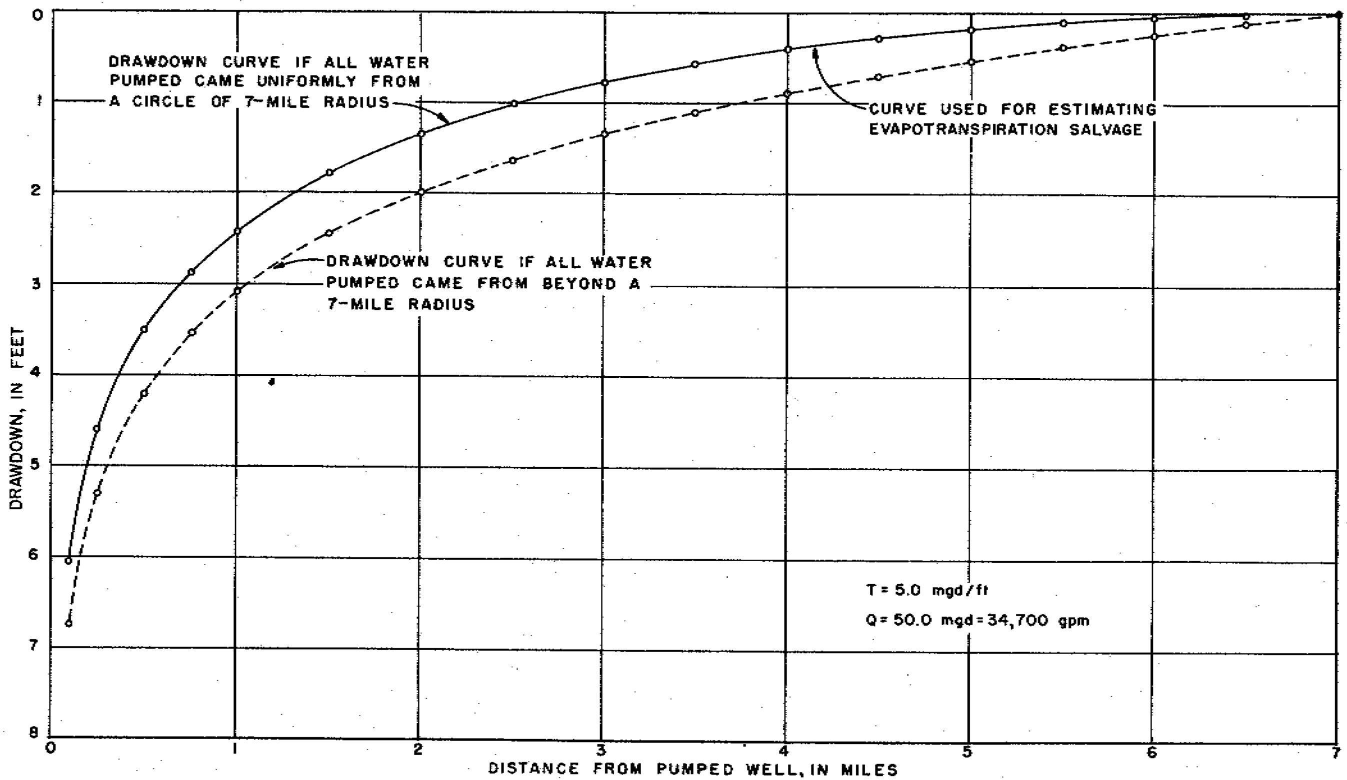

Figure 80. —Graph showing drawdowns resulting from pumpage within and beyond a circle of 7-mile radius.

computed for a coefficient of transmissibility of 5 mgd per ft and a pumpage of 50 mgd. The curve resulting from a plot of these drawdowns is shown as a solid line in figure 80, and for comparison there is also shown, as a dashed line, the drawdown curve that would have occurred if all the water pumped from the well had moved in from distances greater than 7 miles. The decrement of change for the radius was 0.5 mile. A well field pumping 50 mgd continuously, having a circle of influence 7 miles in radius, would require an effective recharge of 6.82 in. of water per year. A 6-mile radius of influence would require an effective recharge of 9.28 in. per year, and a 5-mile radius of influence would require 13.36 in. of water per year.

From studies of the glades area (see section on Surface water, Hydrologic studies), it has been determined that the runoff through canals and transverse glades has averaged 5 to 14 in. of water per year. It is possible that 6.82 in. of water per year could be salvaged by decreased runoff through canals and transverse glades, and by reduction of evapotranspiration loss due to lowering of the water table. The salvage of water over the area would not be uniform; near the center of the cone of depression, conditions would be favorable for salvaging a larger percentage of the water than could be salvaged near the border of the cone of depression. Also, the size of the cone of depression would vary with the rainfall over the area and with the elevation of the water level in the glades.

Figure 81 shows the estimated cone of depression in the proposed well-field area resulting from superimposing the drawdowns caused by a uniformly recharged circle of influence of 7-mile radius (see fig. 80) upon the water table of March 17, 1941, when the stage was about average for this area.

EVAPOTRANSPIRATION SALVAGE

Declines of the water table due to evapotranspiration were discussed on pages 222-231. Figure 53 indicates that evapotranspiration is less at lower stages of the water table than at higher stages. This would generally be expected, because, as the water table declines farther below the surface, more energy is required to raise the water to the surface. Also, the water table and the capillary fringe may be beyond the reach of some plant roots. The middle curve on figure 53 is for average atmospheric conditions, which would cause about 0.19 in. of water per day to evaporate from a standard class A evaporation pan.

Figure 82 shows the estimated reduction in evapotranspiration under average atmospheric conditions for stages below average, as indicated by the decline of water levels in wells S 182 and S 196 (see fig. 53). Using the evapotranspiration salvage curve, devel-

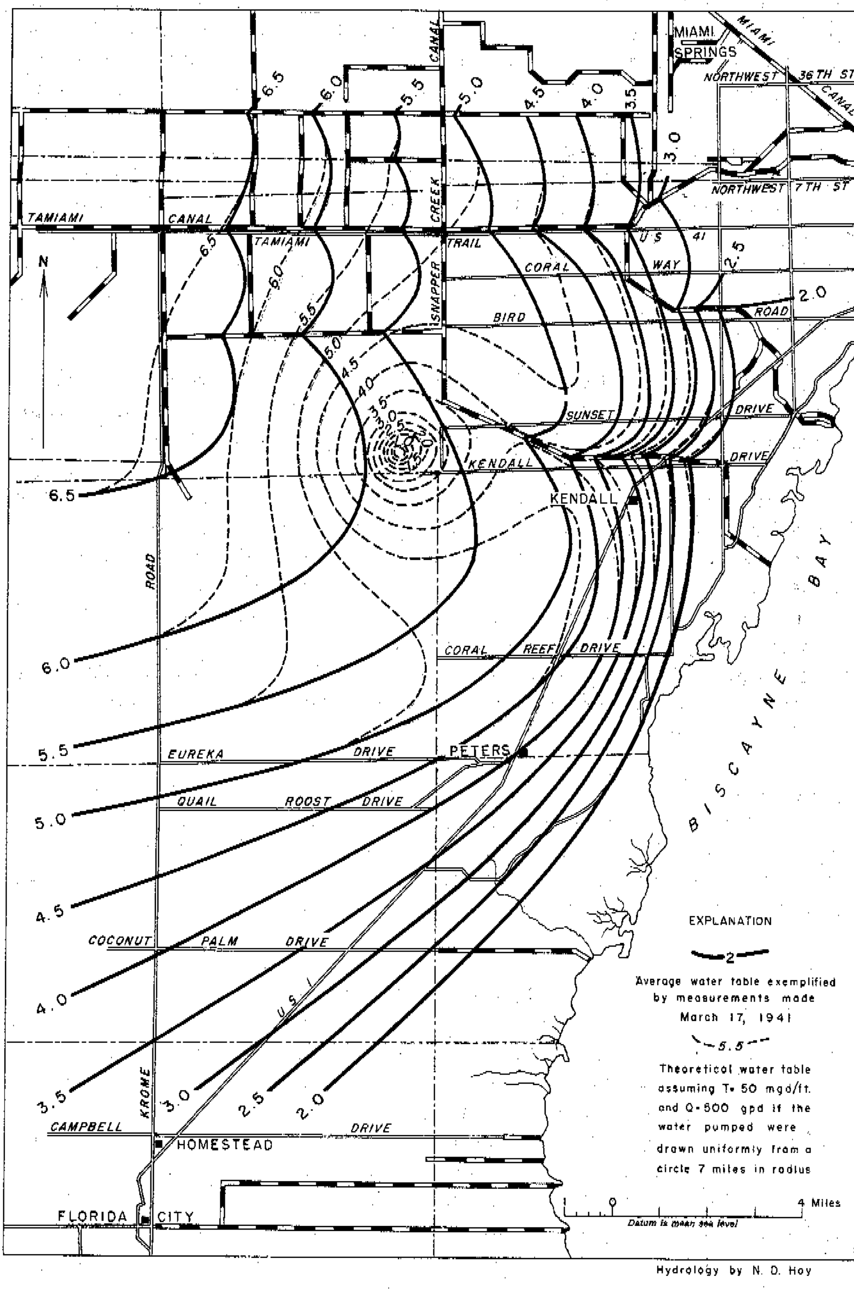

Figure 81. —Map of proposed well-field area showing theoretical cone of depression after drawdowns for a uniformly recharged circle of 7-mile radius are superimposed on the average water table.

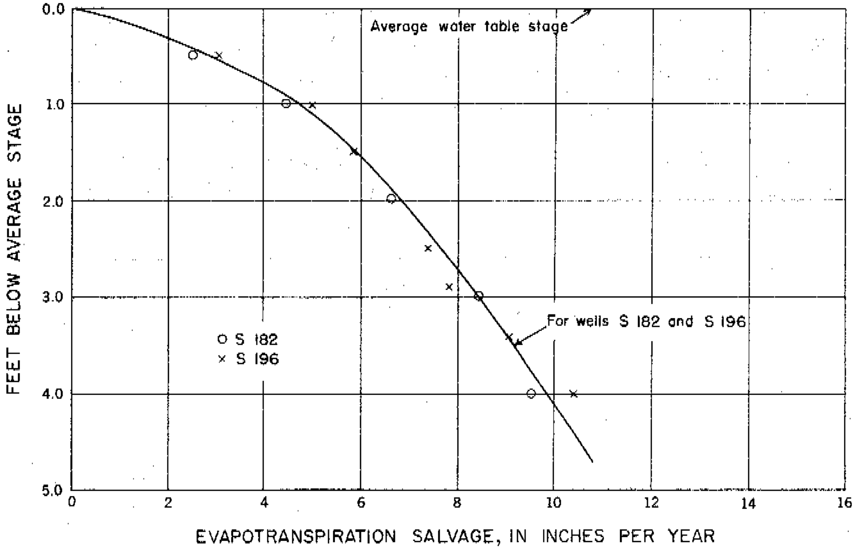

Figure 82. —Curve showing estimated evapotranspiration salvage resulting from lowering the water table.

oped for wells S 182 and S 196 (fig. 82), and the drawdown curve for a pumping well that has developed a cone of depression of 7 miles' effective radius under uniform recharge conditions (upper curve, fig. 80), it is possible to compute, by the summation process, what the apparent evapotranspiration salvage would be. The computation suggests that about 15 mgd could be salvaged. The average stage of the water table in the area in which the proposed well field is to be established is only $1\frac{1}{2}$ to 2 ft below the surface; this places the stage 2 or 3 ft closer to the surface here than in the coastal ridge. It is believed that this would make the evapotranspiration salvage in the proposed well-field area greater than that indicated by wells S 182 and S 196.

However, in areas of permanently lower average water-table conditions, the roots of some plants may tend to grow deeper and thus somewhat offset potential salvage. The writers know of no way to determine this effect, but it is believed that evapotranspiration salvage might be about 10 mgd where the well field is surrounded by a cone of depression 7 miles in radius.

INFILTRATION FROM CANALS

By assuming certain conditions it is possible to approximate the amount of induced recharge to an aquifer from a canal using the principle of recharge from an infinite line source, as outlined by Muskat (1937, p. 175). It is assumed that the canal is freely connected with the water-bearing formation, that the water level in the aquifer bordering the canal is the same as the water level in the canal, and that these water levels will rise and fall together, even though the change is rather rapid. It is further assumed that the canal is long and straight, and that canal flow into the area is sufficient to maintain a constant water level in the canal.

Figure 83 shows the stabilized cone of depression and the water-table profile for a well pumping 50 mgd in an ideal aquifer that has

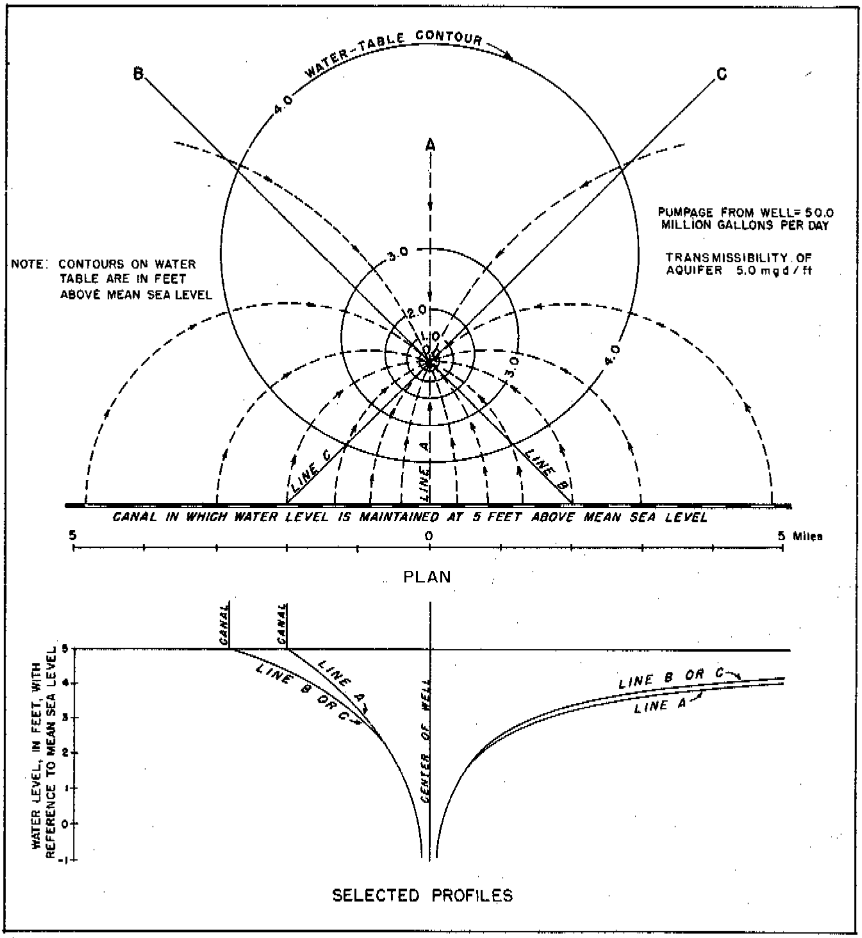

Figure 83.—Profiles of the water table and plan of the stabilized cone of depression in an ideal aquifer of semi-infinite extent with reduced recharge from a canal.

an infinite areal extent away from the canal. A long straight canal, 2 miles from the well and freely connected to the aquifer along its entire length, supplies essentially all the water pumped by the well. The water level in the canal is maintained at 5 ft above mean sea level throughout its entire length. The dashed flow lines show that influent seepage from the 4 miles of canal nearest the well supplies 50 percent of the water pumped by the well, and the 10 miles of canal nearest the well supply 75 percent of the water. The water-table profile along lines A, B, and C show the steepness of the gradients between the canal and the pumping well.

Snapper Creek Canal passes within 1.5 miles of the center of the proposed well field, and Bird Drive extension canal is 2.5 miles to the north. It appears that the effect of these canals would be approximated by a straight canal trending northwest–southeast and passing at a perpendicular distance of 2.0 miles from the center of the proposed well field. Pumping test data for well G 551 (p. 252–258) indicate that the Biscayne aquifer is very permeable in this area; the test gives a value of T equal to about 15 mgd per ft. The limestone in this vicinity is rather cavernous, and the layer of quartz sand present in some areas between the Miami oolite and the limestone of the Fort Thompson formation is absent or much less prominent. There appear to be no layers with low permeability to restrict the movement of the water from the Miami oolite to the Fort Thompson formation, from which the wells will draw their water. The rock walls of the canals are cut into the Miami oolite.

The average water level in Snapper Creek Canal is about 5 ft above mean sea level in that part of the canal nearest to the proposed well field. At the junction of Snapper Creek Canal and Tamiami Canal the average water level is about 4.5 ft above mean sea level. If the water table should decline 1 or $1\frac{1}{2}$ ft in the area bordering the canal at its nearest point to the well field, the direction of flow in this part of Snapper Creek Canal would reverse, and the water surface would have a slope of about 0.2 ft per mile toward the well field and away from the Tamiami Canal. It is believed that this slope would cause a sufficient flow of water in the canal to replenish the water that would infiltrate to the aquifer from the canal. A lower water surface in the canal would tend to lower the whole cone of depression (to a lesser degree) and would cause the infiltration from the canal to be a little less effective. During severe dry periods, as in 1945, Snapper Creek Canal can become dry and no water will be delivered from other areas for infiltration to the well field. In such periods, simple water-table conditions will prevail. Infiltration from the canal to the aquifer will not account for all the water pumped from the well field, because the southern part of the cone of depression will intercept considerable quantities of ground water moving toward the natural discharge area of Biscayne Bay. In view of the high permeability of the limestone in this area, it is believed that the water salvaged (including that which formerly flowed into canals and out of the area), plus the influent seepage from canals caused by pumping, will amount to a minimum of 25 to 50 percent of the total pumpage (data for 1 day, Mar. 28, 1946, p. 484, indicate that 78 percent of pumpage can come from seepage from canals). Water-table maps of the Miami area and the northern part of the coastal ridge (figs. 42–44) show the effectiveness of deepened sections of canals in the lowering of the water table. If the situation were reversed, so that the water table was lower than the stage of the canal, the canals would be just as effective in raising the water table.

CONCLUSIONS

Because of variations in water-bearing properties of the Biscayne aquifer within relatively short distances, because the network of canals that intersect the area have a pronounced effect upon the water table, and because of the delicate balance between the fresh- and salt-water heads, it is not possible to obtain exact mathematical solutions to hydrologic problems. After carefully analyzing the data available, it is believed that about one-half or more, of the desired quantity of 50 mgd will be obtained through infiltration from canals to the cone of depression, by the reduction of runoff to canals from the area, and by evapotranspiration salvage due to lowering of water levels by pumping. It is believed that the remaining amount of water that is diverted from natural discharge areas in Biscayne Bay will not lower the fresh-water head sufficiently to allow salt water to encroach into the well field at depth in the aquifer. During a series of dry seasons, or years, the salt-water front may advance slowly inland, but this advance should be compensated by the seaward movement during wet periods.

Particular attention should be given to the control of canals to insure that they do not allow salt water to flow inland from the bay or from areas where the aquifer is salty. If the new well field is established at the location proposed, and if Snapper Creek Canal is cleaned out or deepened between Biscayne Bay and Tamiami Canal, special attention should be given to the eastern section of these canals to protect against inland flow of salt water, because the canals would provide a direct access for salt water into the well-field area.