THE QUANTITY OF STORMWATER ENTERING THE DRAINAGE WELLS OF ORLANDO, FLORIDA

BY

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THESIS

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ABSTRACT

An extensive literature survey revealed there have been no in-depth studies of the quantity of water entering Orlando area drainage wells. Previous values ranging from 30 to 85 MGD were based on water supply withdrawal information or gross drainage area estimates. This paper presents a detailed study of the quantity of water entering 208 drainage wells in the Orlando Urban Stormwater Management Manual (OUSWMM) area. Extrapolation of results to the remaining wells in Orange County is discussed briefly. Field experiments on one 20 inch drainage well yielded a mathematical relationship that was reasonable for estimating acceptance rates for drainage wells of all sizes.

One hundred seventeen drainage sub-basins have been identified in the 54,000 acre OUSWMM area. Seventy-four of these sub-basins contain or contribute flow to one or more of the 208 drainage wells. Weighted mean daily runoff in the 74 sub-basins was estimated between 39.1 and 53.4 MGD. Storage effects reduce this to 17 to 31 MGD, which is the maximum quantity of runoff available to the drainage wells. Other effects could reduce this more.

A well hydraulics estimate of the inflow quantity was 18 to 47 MGD. This agreed reasonably well with the estimate of available runoff. Deviation between the two could be due to the limited amount of data on heads on the well.

Maximum hydraulic capacity of all the wells was estimated to range from 90 to 226 MGD. Total aquifer transmissivity capacity for the wells was estimated to range from 356 to 534 MGD. These values show that the wells are sufficient to easily handle the average available runoffs, but may not be capable of handling extreme event flood flows.

DEDICATION

Work is an important part of our lives, and completing this thesis has definitely been that - work!

Knowing God as my Father is becoming more meaningful to me day by day. With this in mind, I thought it would be interesting (and humorous) to see some of the things His Word says about work.

First, as recorded by King Solomon:

> "All hard work brings a profit,
> but mere talk leads only to poverty."

Proverbs 14:23 NIV

But later Solomon had this to say:

> "A fool's work wearies him,
> he does not know the way to town."

Ecclesiastes 10:15 NIV

Finally, the Words of the Lord Jesus Christ. Jesus Himself said that He was greater than Solomon, greater than anyone, and so He deserves our attention.

"All things have been committed to me by my Father."

"No one knows the Son except the Father, and no one knows the Father except the Son and those to whom the Son wishes to reveal him."

"Come to Me, all you who are weary and burdened and I will give you rest. Take my yoke upon you and learn from me, for I am gentle and humble in heart, and you will find rest for your souls. For my yoke is easy and my burden is light."

Matthew 11:27-30 NIV

It is my prayer that all men (myself included) - and women - will come to know Jesus as He really is. He will give us rest.

It is to Him that this work is dedicated.

James Michael McBee
July 23, 1985

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1

INTRODUCTION

The first Orlando area drainage wells (also called drainwells) were dug back in 1905 to correct a flooding problem. A sinkhole which helped drain stormwater runoff became clogged; trying to unclog the sinkhole was useless, so as an experiment, a two-inch diameter test drainage well was drilled. The high transmissivity of the aquifer resulted in this well being very effective for storm drainage, so it was decided to dig other, larger drainage wells. During the hurricane periods of the 1920s, 1940s, and 1959 to 1961 many drainage wells were dug because they seemed so effective in accepting large quantities of stormwater runoff and in controlling lake levels.

Flood Problems

The City of Orlando has had many problems with localized floodings. Drainage wells were considered from 1905 to the 1960s to be the best solution to these problems because they are so much more cost effective than drainage ditches, storm sewers, swales, and force mains. A cost estimate for a new complete drainage system for the Southeast Lakes regions of Orlando is $7,000,000 which is at a rate of $1.85 million per mile of storm sewer. This estimate was made in 1984 by Dyer, Riddle, Mills and Precourt. A cost estimate in current 1985 dollars for the total 48 wells in the 14 sub-basins in the

Southeast Lakes region is $1,920,000, or $40,000 per well. This estimate was provided by Mr. Richard Potts, formerly of Jammal and Associates, Inc.

As the developed area of Orlando grew larger, longer and longer distances of open channel drainage storm sewers would have been required to reach satisfactory final disposal areas. So the cost of storm sewers and ditches would grow geometrically. On the other hand, drainage wells were found to be capable of accepting ever-increasing flows. Hence, until the mid-1960s drainage wells were the traditional solution to drainage problems.

Quality Problems

One popular belief about drainage wells is that there have not been well documented water quality problems for wells in the Orlando area. Yet there have been a few instances recorded of contamination of water supply wells because of drainage wells. The first serious problems were encountered in Live Oak, Florida, in 1948 where a drainage well, along with some sanitary sewage disposal wells, caused contamination of chlorinated water supplies. In the Orlando area a drainage well on Lake Pleasant was shown to cause "muddying" and bacteria contamination of a public supply well in Orlando in 1961. Then in 1972, W.F. Lichtler of the United States Geological Survey mentioned a 1300' deep supply well that for two years produced water with high bacteria counts. Presumably this was due to contamination caused by a drainage well. These instances of contamination of public water supplies are discussed in greater detail in Chapter III, Literature Review.

Current Status

As far as is known, no new drainage wells are being constructed or planned. According to John Armstrong of the Orlando office of the Florida Department of Environmental Regulation (DER), the current policy of the Florida DER is that no permits will be issued for drilling of new drainage wells.

The Environmental Protection Agency has entrusted the DER with primary jurisdiction over all injection wells in Florida, including drainage wells. The DER has had jurisdiction for only two years. No permits have been issued since sometime in the mid 1960s for new drainage wells. Permits were allowed for constructing replacements for damaged or defected drainage wells until the early 1970s. Now this, too, is no longer allowed. No drainage well construction of any kind is allowed under current policy. Final disposition of the remaining wells is still being argued.

Objectives and Scope

Objectives

The purpose of the research effort is to determine a reasonable estimate of the quantity of water entering drainage wells within the City of Orlando. This estimate can be used in future studies in at least three ways: to determine the magnitude of flooding if the wells are closed; the effects on recharge if the wells are closed;

and along with water quality data, to calculate the total loading of pollutants entering the drainage wells.

Scope

The study is limited in scope to using available data on predicted land use patterns, topography, known drainage basins and systems, and estimated runoff coefficients, to estimate the quantity of stormwater runoff generated in the immediate Orlando vicinity known as the Orlando Urban Stormwater Management Area. This estimate was done using the rational formula. This estimate was then modified by considering lake storage to yield an estimate of the quantity of runoff available to the drainage wells. The study is supplemented by field measurements of flow and head variations for a few specified wells, and field calibration of the acceptance rate for one well.

The field measurements of flow and head variation was used to estimate transmissivity of the aquifer. This estimation was evaluated to determine if it was reasonable to expect high or low quantities of acceptance of water through drainage wells. More importantly, the field measured flow was extended to consider other wells and the reasonableness of expecting a high or a low quantity of acceptance for all wells based on well hydraulics.

CHAPTER II

WHAT IS A DRAINAGE WELL?

Before getting too deeply into the subject, perhaps it is wise to describe in detail exactly what a drainage well is. It has been the author's experience that not many people had previously heard of or knew what drainage wells were. The lay public often does not know of the existence, much less the purpose and mode of operation, of these wells.

A drainage well is simply a pipe which drains water down into the ground. That is all it is. The water can be stormwater runoff, lake water, or standing water in depressions and swamps. It can be excess irrigation water in an agricultural area, or it can be excess groundwater from very soggy ground in areas with a high water table. The water could be sewage and industrial wastewater, as it was in the past in the Orlando area and many other places across the State. Wells can even be used to drain water from one aquifer to another, as documented in "Geohydrologic Reconnaissance of Drainage Wells in Florida," (Kimrey and Fayard, 1984, p. 1-2). Interaquifer connector wells are used extensively by the phosphate mining industry in Florida. Wells are even being proposed which will pump wastewaters (under pressure) as deep as a 3000 feet or more into the ground.

Wells such as these are known by such various names as injection wells, recharge wells, disposal wells, and drainage wells, and for interaquifer drainage or recharge they are called connector wells. However, for this paper and also in the general case, a drainage well is strictly a well which drains, by gravity, stormwater or other excess surface water (lake or swamp water) into the ground, usually into the Upper Floridan aquifer. Drainage wells are also called drainwells in the literature. The two terms are used interchangeably in this thesis.

Why are Drainage Wells Necessary, and How do They Work?

Orlando has 92 lakes completely or partially within its borders. Most of these lakes were probably sinkholes at their formation and then became closed off. Very few of these now have a natural outlet such as a creek or stream. Therefore, any water that is removed from these lakes must be by evaporation, exfiltration through the lake bottom, or through some artificially created outlet. Also, Orlando is now largely an urban area. There are many buildings located densely together, and there are, of course, many streets and parking lots. Thus there are many large impervious areas. When the rain falls on impervious areas it cannot penetrate so it must run off into adjacent areas, and eventually into the storm sewers. Storm sewers are usually routed to the nearest natural water body; which in Orlando is usually one of 92 lakes.

If the rainfall is not of a high volume and intensity, then this method of stormwater management is sufficient to handle the quantity of water. But, if the storm is intense or of such high volume that the parking lots and streets are flooded, the storm sewers are full to overflowing and the lakes are beginning to flood, then it is obvious something has to be done.

Since no natural drainage exists for the lakes, artificial drainage must be provided. This can be done either by digging canals or pumping the water to the nearest suitable creek or river; or by digging a well on the side of the lake (or even in a storm sewer itself), and letting the water drain by gravity into the ground. As previously discussed this is what a drainage well does.

Wells which are located in a storm sewer itself are generally just pipes in a collection basin. Figure 1 is an example of a storm sewer well.

Wells on a lake usually have some sort of weir arrangement located somewhere upstream of them so that the lake has to rise to a certain level before flow to the drainage well can occur. An example of this is shown in Figure 2.

Regardless of whether the well is a lake level control well or storm sewer well, the water is conducted by gravity down into the ground. The well itself is a hole, usually dug down into the first cavern large enough to accept a large amount of water. Steel or iron pipe is placed into the hole, down as far as the top of the first competent (hard) limestone rock. This pipe is called the well casing, and it prevents water from seeping out of (or into) the well until it reaches the aquifer. Enough of the bottom of the well is

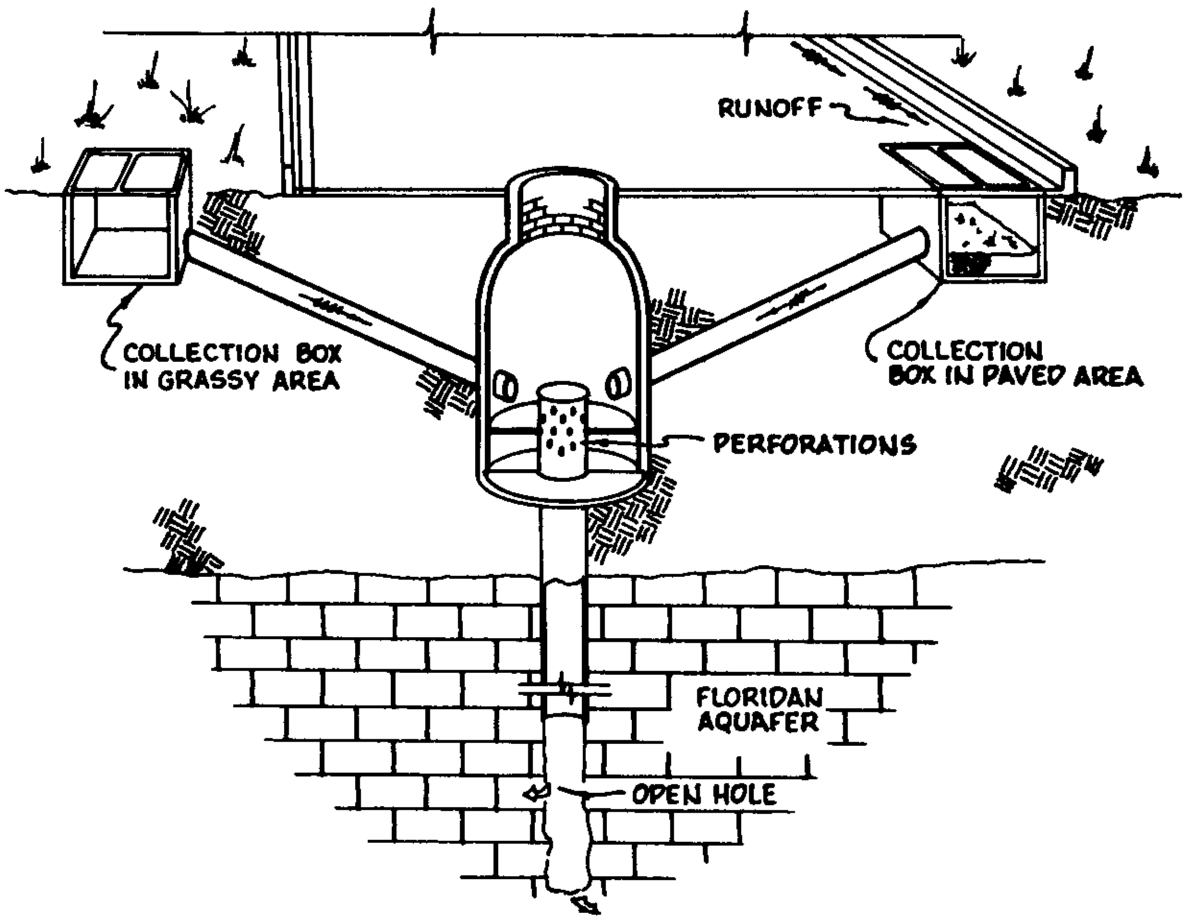

FIGURE 1 STORM SEWER DRAINWELL
WITH CONNECTING INLETS

SOURCE : DR.M.P. 1984

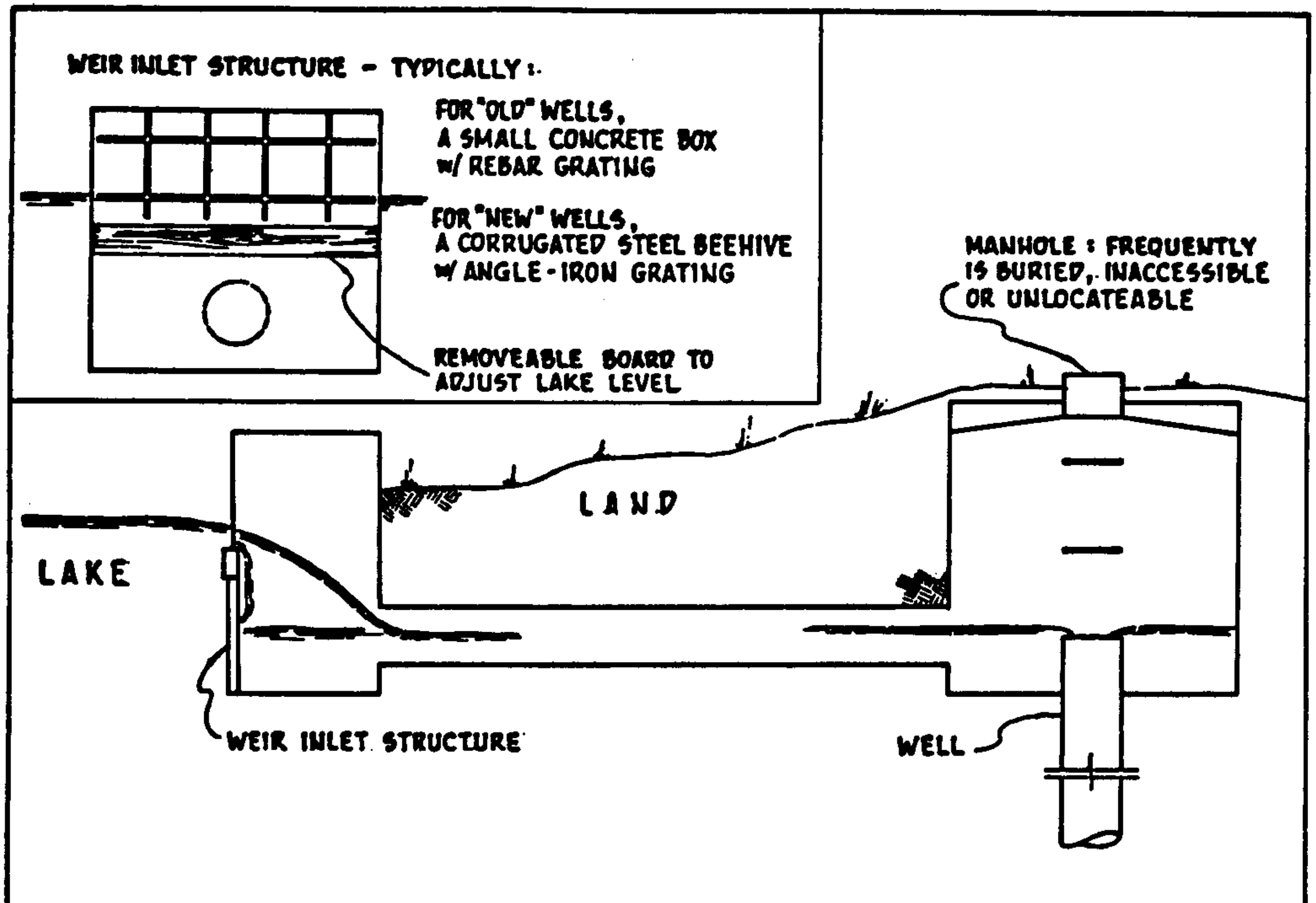

FIGURE 2 "TYPICAL" LAKE CONTROL DRAINWELL left uncased so that it will dispose of the necessary amount of water.

Where Does the Water Go When it Reaches the Aquifer?

We don't completely know. Obviously it leaves the well and mixes with the groundwater. But its movement after that, and the effects of dilution and rock filtration, are only known in general terms. Most of the wells penetrate into the Upper Floridan aquifer which is a limestone layer about 450 feet thick, and from about 150 feet deep to 600 feet deep (Kimrey, 1978, p. 7, Figure 2). Some of the wells penetrate only to the upper surficial aquifers; others were drilled too deep and extend into the relatively impermeable intervening zone between the Upper and Lower Floridan aquifers (which is inefficient for discharging the water). A few drainage wells may extend down into the Lower Floridan Aquifer which is the layer from which the cities of Winter Park and Orlando obtain their water supply. Figure 3 shows a simplified geologic cross section of drainage wells and supply wells.

The low permeability intervening zone (called the aquitard) between the Upper Floridan and Lower Floridan aquifers acts as a separating barrier, and slows down the movement of water from the one aquifer into the other. However since there is generally a higher potential in the upper aquifer than in the lower aquifer, there usually is some movement between aquifers. The rate and mode of movement of water is not well documented, but it is known to exist (Kimrey and Fayard, 1984, p. 36). So there is a possibility that deep supply wells could eventually become contaminated.

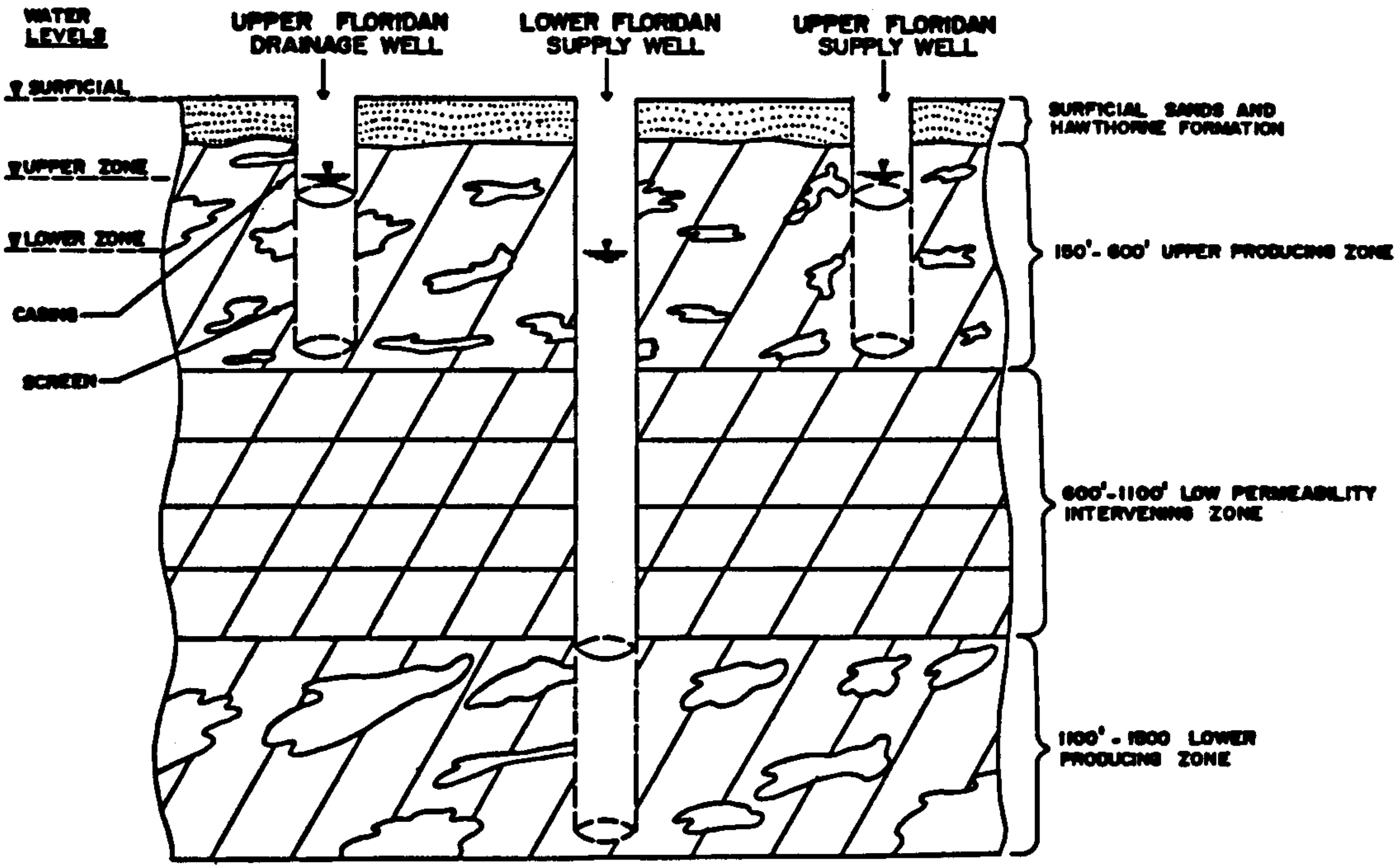

FIGURE 3 SIMPLIFIED GEOLOGICAL CROSS SECTION

In addition, there is a possibility that water transmitted to the Upper Floridan aquifer could be transported to some of the many shallower supply wells located outside of Orlando. It should be pointed out that while the public water supplies of Orlando and Winter Park are taken from the Lower Floridan aquifer, making up 65% of the withdrawals for supplies, 35% of the local water supplies are withdrawn from the Upper Floridan aquifer (Kimrey and Fayard, 1984, p. 36). In fact, Table 1 shows that the median depth of 424' of 314 drainage wells in the Orlando area is almost the same as the median depth of 420' of 186 supply wells in the Orlando, Orange county area (Schiner and German, 1982, p. 11). So it is very possible that contamination of these supply wells could occur.

An excellent description of the geology of the receiving zones for drainage wells can be found in much of the United States Geological Survey (USGS) literature and also in the East Central Florida Regional Planning Council (ECFRPC) 208 study of 1977 (ECFRPC, 1977, P. 1-3).

How Long Have There Been Drainage Wells?

Many people are understandably disturbed when they hear about wells which transmit "dirty" street water into the aquifer, and want to know when did "they" start doing this.

The answer is that drainage wells are nothing new. "We" first dug them in Orlando in 1905, and they also existed at that time in other states as well, as documented by E.H. Sellards (see Chapter III, Literature Review). Then, every time it rained after that,

TABLE 1

SUMMARY OF DATA ON DEPTHS OF DRAINAGE WELLS AND PUBLIC-SUPPLY WELLS

WELL TYPE	NUMBER OF WELLS	RANGE OF DEPTH (FEET)	PERCENT OF WELLS IN WHICH INDICATED DEPTH IS EXCEEDED				
			90	75	50	25	10
Drainage	314	120-1,049	196	334	424	484	600
Public supply	186*	94-1,500	200	324	420	558	1,300

* Includes four non-public supply wells used to expand the statistical base for quality of water interpretations.

SOURCE: United States Geological Survey 1983, WRI 82-4094 "they" wanted to dig some more wells to keep the lakes from flooding. Table 2 presents a chronological list of cumulative known drainage wells. In 1939 permits became required to dig drainage wells, and so records are a little better since then. In 1965, the Florida State Board of Health stopped granting permits for the construction of new drainage wells, but still allowed replacement of existing defective drainage wells. Finally, in the 1970s the Board of Health stopped granting permits for construction of any drainage wells.

TABLE 2

CHRONOLOGICAL GROWTH OF KNOWN NUMBER OF DRAINAGE WELLS

YEAR	NUMBER OF WELLS	DEPTHS	DIAMETERS	LITERATURE REFERENCES
1906	6	140'-340'	8" and 12"	E.H. Sellards, 1908, p. 63
1936	120	160'-800'	6" - 16"	V.T. Stringfield, 1936, p. 162
1943	200	141'-1049'	5" - 18"	A.G. Unklesbay, 1944, p. 21
1977	412			J.O. Kimrey, 1978, P. 10
1981	392	120'-1049'	4" - 26"	Schiner and German, 1983, p. 12
1985	413	20'-1070'	2" - 26"	Latest USGS Computer Listing

Note: For more detailed information on the above, refer to text in Chapter III - Literature Review

CHAPTER III

LITERATURE REVIEW

Introduction

A significant amount of literature has been generated about the drainage well situation in Florida. It always makes an interesting topic. The leaders in this research and publishing effort have been the United States Geological Survey, and before them, the Florida Geological Survey. However, some useful and very pertinent reports have been written by other groups, notably John S. Telfair's 1948 report for the Florida State Board of Health regarding the health hazard posed by sanitary sewage drainage wells in Live Oak, and also Orlando, Florida. Also, some engineering consulting firms have published material which is useful in estimating quantity and/or quality of water reaching drainage wells. Dyer, Riddle, Mills, and Precourt have done much work in this regard, also Jammal and Associates, and Black Crow Eidsness/CH2M Hill have made contributions in this regard.

Most of the literature so far has centered on the quality aspect of drainage wells - particularly, do they pose a threat to our water supply and how can we determine how much of a threat? This quality emphasis of the literature is understandable. The quantity aspect of drainage wells is no problem, the more water the better (for aquifer recharge) as long as the increased quantity of drained water does not interfere with the quality of water in the public supply. However, considerations of quantity have appeared consistently in the literature from the very beginning of the use of drainage wells. This chapter will review chronologically almost all literature directly relating to Orlando area drainage wells. Useful sources of practical engineering information appear towards the end of the chapter. Finally, as an aid to the reader, Table 3 at the very end of the chapter summarizes the pertinent literature.

Literature

The very first reference to the quantity of water entering drainage wells appeared in 1908, just three years after the drilling of the first drainage wells in Orlando. Dr. E.H. Sellards, the new state geologist for Florida, prepared Florida State Geological Survey (FGS) Bulletin No. 1, entitled "A Preliminary Report on the Underground Water Supply of Central Florida." In this report, Sellards established the two major conditions that must exist for wells to be used for drainage:

1) the static head of water at the surface, entering the well, must be greater than the static head of the underground water in the stratum the well discharges to.

2) the water must be able to move freely out of the well and into the underground stratum, i.e., the receiving stratum must be very permeable, preferably cavernous.

This is why most operating wells discharge into hard fissured limestone. Clay or silt will not do. Sellards theorized that if a well penetrated a zone suitable for discharging water quickly it would probably also supply large quantities of water when pumped (Sellards, 1908, p. 61).

Sellards also stated four factors influencing the efficiency of any given well. These factors were:

(a) The well's size. Increasing diameter results in a squared increase in area, and the relative hindering effects of friction on velocity are less in a large versus a small pipe.

(b) Well intake construction. Sellards noted that most drainage wells do not have bell-shaped mouth entrances, they are just straight cutoff pipes. This significantly decreases their potential capacity. This fact was confirmed in the field experiment performed on the Lake Angel well (see Chapter IV).

(c) The "head" on the well, i.e., the height of water above the well entrance. Sellards stated that the intake to the mouth of the pipe would be proportional to the square root of the head on the pipe, which is true in the limiting case of orifice flow entering the pipe.

(d) The depth to the static underground water level. Sellards stated that the greater this distance, the more draft tube suction would be available to increase the flow rate, up to a maximum possible of 32.8 feet of suction head.

For all of the four factors Sellards is careful to note that he assumes free movement of water out of the bottom of the well, in other words a highly transmissive receiving zone (Sellards, 1908, p. 61-62).

In the FGS Bulletin No. 1, Sellards also recounted the early history of drainage wells in Orlando, Florida, noting that the first wells were dug in response to flooding caused by heavy rains and loss of natural drainage through a sinkhole in southeast Orlando. Sellards notes that by February 1906, there were four 12-inch diameter and one 8-inch diameter drainage wells in existence, plus one 2-inch diameter test well. Four of the wells (sizes not specified) were 140 feet deep, the fifth was 340 feet deep. All of the wells at that time were relatively close to the sink, which historians believe to be Lake Greenwood.

Sellards stated that county authorities had made level checks to see if surface canal drainage was possible; it was not. It is important to realize that digging drainage wells was not completely a panic response to flooding, it just seemed at the time the most judicious thing to do (Sellards, 1908, p. 62-63).

Sellards also emphasized the possibility of pollution of the water supply because of drainage wells, especially sewage disposal wells. He refers to salt-tracing tests conducted in 1907 under Mr. M.L. Fuller's direction which pointed out the interconnectedness of the aquifers. Sellards also correctly points out that underground rivers do not exist, and so underground water movement is generally slow (Sellards, 1908, p. 65).

The next known literature reference to drainage wells was in 1910, again by E.H. Sellards, along with Herman Gunter, in the <u>Third Annual Report (1909-1910) of the Florida Geological Survey</u>.

Sellards devotes nine pages of the annual report to drainage wells, repeating again the previously described four conditions governing drainage well capacity, and also the history of drainage wells in Orlando. He adds some equations for estimating drainage well intake, saying that velocity of flow in the well may be measured by means of a pitot tube. Referencing R.E. Horton's U.S.G.S. Water Supply Paper #145 of 1905, Sellards derives that

$$V = \text{velocity (feet/sec)} = \sqrt{2gh} = 2.32\sqrt{h}$$

$$Q = \text{flow (cfs)} = 0.0055\, d^2V = (d^2\sqrt{h})/80$$

where d is the diameter of the well in inches and h is the pitot tube head in inches (Sellards and Gunter, 1910, p. 70-71).

But the most fascinating addition in the 1910 report is Sellards' account of the first "spouting" well (Sellards and Gunter, 1910, p. 72-74). The well in question had existed for three years before it began spouting September 26, 1910. It was a 12-inch diameter well, drilled to 260 feet and cased to 60 feet, located on a small lake three miles north of Orlando (probably Lake Fairview). T.P. Robinson took an excellent photograph of the well in action. The spout appears to reach 20 to 25 feet in the air (Sellards and Gunter, 1910, Plate 9). Since that time many spouting or gassing wells have been reported. What is unusual about that well was that it was spouting at regular intervals every few minutes for over a week, during a non-storm period. Also, when heavy rains caused the lake level to rise by 2 feet, the well stopped spouting. This is the opposite of the "usual" case; well spouting or backflow usually occurs only during or right after a heavy storm event, when greatly increased inflows to the well cause it to entrain air under pressure or some other phenomenon occurs to cause the spouting. Sellards attributes the well spouting to the fact the well was not carrying water at its full capacity, that air was being sucked down by the water into the well. The air accumulated under pressure until it forced itself and the water out of the pipe. Thus, Sellards says the well "could be classified as self pumping" (Sellards and Gunter, 1910, p. 73). Sellards reports a similar well existed in Albany, Georgia, as documented by McCallie in Science, Vol. 24 (1905), p. 694.

Sellards also notes that by 1910 "a number of" drainage wells had been drilled in the Orlando area for various purposes by private individuals as well as the city.

The next scientific literature regarding drainage wells was by V.T. Stringfield in 1933, on pages 19-24 of the "Groundwater Investigations in Florida," Florida Geological Survey Bulletin, No. 11. The author of this thesis has not personally reviewed this document, but it is cited in several of the U.S. Geological Survey publications. Stringfield also authored another article relating to drainage wells.

In "Contributions to Hydrology," 1936, p. 161-62, Stringfield gives an inventory of then existing drainage wells. He states that over 120 wells existed in or near Orlando at that time, about 90 owned by the city, and 30 owned by the county. The well diameters ranged from 6 to 16 inches, depths from 160 to 800 feet. The wells were located in such varying topographic areas so that the static water depth below ground surface varied from a few feet to 60 feet. Stringfield also gives the first estimate of drainage well capacities, from less than 100 gpm up to a reported 9500 gpm. The maximum capacity was for only one well, of unknown size, located 4 miles northeast of Orlo Vista. Stringfield records that practically all the sewage as well as storm runoff in the city was disposed of into drainage wells (Stringfield, 1936, p. 161-62).

The next significant report dealing with quantity of water entering Orlando drainage wells was in 1944 by A.G. Unklesbay of the U.S.G.S. for the Florida Geological Survey, Report of Investigations No. 5, entitled "Ground-water Conditions in Orlando and Vicinity, Florida." This report was actually devoted entirely to investigations of drainage wells. Most helpful in this report are its detailed well record inventory, and also the report of Unklesbay's well water level measurement project. Also very interesting is the velocity of water in drainage wells investigation.

Unklesbay reports that by August 1943, there were at least 182 wells in Orlando/Orange County areas. Ninety storm drainage wells owned by the city, 40 storm and lake drainage wells owned by the county, 12 drainage wells owned by the Orlando Army Air Base, 40 drainage wells owned privately or by other municipalities, and used for various purposes including citrus packing waste disposal (Unklesbay, 1944, p. 21).

In additon, Unklesbay counted at that time 18 drainage wells used to dispose of sanitary sewage. Seventeen of these were city-owned, the other was at the Orlando Army Air Base. The sanitary wells alone ranged in bored depth from 231 to 863 feet, and cased depths from 67 to 400 feet. Diameters were from 8 to 12 inches.

In his appendix, Table 1, Unklesbay gave a detailed inventory of 246 wells in Orange County, probably all the wells he could find. The well number, location, owner, driller, construction date, well depth and casing depth, well diameter, well water level, measuring point and altitude of measuring point, and usage (drainage, sanitary drainage, or supply) were included, if known, for each well. Because Unklesbay included the altitude above sea level of his measuring points for water levels, this made possible the construction of piezometric surface maps for Orlando at that time. In fact, Unklesbay included such maps showing on them the location of every known existing well, storm drainage, sanitary drainage, or supply wells. Figure 4 in his report plots well water levels versus monthly rainfall for 3 wells for the years 1930-34 and 1943. In general, there was no significant correlation between the two, except for June 1934 when a very high rainfall of 16 inches was recorded. Water levels jumped rapidly during that month. Prior to that, for the years 1930-33, the water level had all shown a slow, steady decline corresponding to all of those years of low rainfall.

Finally, Figure 11 of his report contained the summary of the velocity investigations in the eight wells he had chosen. Unfortunately, Unklesbay expressed his results in terms of rpms of the meter's current propeller, not in terms of feet per second, so the total flow of water entering the well cannot be determined. However, the results are useful for determining the depth of the best receiving zone. Unklesbay's description of the current meter device and its usage and results are excellent, written clearly and simply, and helpful in understanding this type of well logging. Also of some interest is that Unklesbay recorded the occurrence of both gassing wells and artesian flowing drainage wells, especially in the summer of 1930. Unklesbay reported that a well northwest of Orlo Vista had artesian flow so that the highway was flooded two feet deep. Other wells ceased to receive water because their own static water levels were too high. So, drainage wells are not always dependable.

Unklesbay did not dwell much on water quality issues. He mentioned that pollution of limestone aquifers and quarries in Minnesota had caused typhoid and gastroenteritis. Unklesbay also pointed out deep supply wells were not necessarily safe from pollution, because deep drainage wells also existed (Unklesbay, 1944, p. 30-31).

Some of Unklesbay's later inflow estimates were reported in David Todd's Groundwater Hydrology. Inflow estimates to recharge wells from around the country are compared. Orlando had the highest rates (Todd, 1980, p. 469).

In January 1948, the Florida State Board of Health, Bureau of Sanitary Engineering, planned a study on the pollution aspects of drainage wells in Florida. Spurred on by bacterial pollution of water supply wells in Live Oak, Florida, the work proceeded rapidly and an interim report was issued by J.S. Telfair on December 29, 1948. The report was entitled "Pollution of Artesian Groundwaters in

Suwannee and Orange Counties, Florida by Artificial Recharge Through Drainage Wells."

This report focused almost exclusively on the quality aspects of drainage wells. It included a bacteriological survey of the wells in both Orlando and Live Oak, and a report of groundwater tracing efforts at Live Oak, Florida. Both Uranine-B organic dye and common salt (NaCl) were used to try to trace flow from the drainage wells. The results indicated direct contamination of a public water well from a drainage well (Telfair, 1948, p. 10). Updated piezometric surface maps for both Live Oak and Orlando were prepared, showing the locations of both drainage wells and supply wells.

No specific attention was paid to quantity of water entering drainage wells, although Telfair seemed to have the opinion that drainage wells often were counter productive in that they would subvert the normal natural drainage patterns of those lakes which were joined together by canals. In fact, he was almost scathing in his criticism of their use for drainage or any other purposes. As a case in point he cites Shingle Creek, whose upper headwaters flow was reversed from Lake Tyler back north to Lake Catherine, due to unnaturally low water levels caused by drainage wells. Another case was Lake Lawne in the Upper Wekiva River, where the water level was made so low that surface drainage from Lake Lawne did not occur (Telfair, 1948, p. 21).

What is most delightful about Telfair's work is its honest, indignant tone. When complaining about the lack of a topographic map of Orange County, he comments that, "Some parts of the New Guinea

Jungle are more accurately mapped." He goes on to state that, "There is no overall plan or program of drainage for Orange County. There never has been." (Telfair, 1948, p. 21).

Telfair gives a general description of the topography of Orange County and of the surface drainage caused by five streams in the area: the Upper (Little) Wekiva River; Howell Creek, (also called Howell Branch); Shingle Creek; Boggy Creek; and Reedy Creek. He omits the Econlockhatchee River, possibly because most of it is in Seminole County. Sometimes the water table of the surficial aquifer is low enough so that both streams and lakes lose water by seepage. Telfair believed that more effort should have been made to take advantage of this. Also, he clearly saw the importance of protecting the underground waters. He made the first recommendation that a comprehensive program of drainage, flood control and pollution abatement be developed and implemented for the county and the city. He recommended that all sanitary sewage drainage wells be abolished and plugged, and further, that stormwater drainage wells be used only as a last possible resort, and then only after pretreatment of the discharged water. He further recommended that zoning prohibited construction of residences in flood prone areas (thus eliminating the need for drainage wells in those areas) and also that the rural suburbs of Orlando develop adequately treated water supply sources. First of all, he recommended a complete topographic map for the county (Telfair, 1948, p. 32-34). Telfair made similar recommendations for Live Oak (Telfair, 1948, p. 10-11).

The next literature reference to drainage wells also concerned quality, and reported on some salt tracing tests. This was on pages 128 to 133 of Florida Geological Survey Report of Investigations No. 50, "Water Resources of Orange County, Florida," by W.F. Lichtler, B.F. Joyner, and Warren Anderson.

Lichtler noted a case occurring in September 1960 of pollution of a supply well by a drainage well. A lake level control well of Lake Pleasant was draining "somewhat muddy water" into the aquifer; and the water in the Northcrest Public Supply Well was discovered to suddenly become muddy, odorous and high in bacteria count. Shutting down the drainage well caused the pollution to clear up, while reopening the drainage well caused the pollution to return. Salt tracing tests that were conducted later confirmed that the drainage well was the cause of the pollution. Unfortunately, Lichtler does not say what corrective action was taken.

Lichtler also noted contamination of a rural domestic supply well as a result of pollution from a cattle dairy's drainage well. The drainage well received the flushings from the cow barns, and as a result the supply well had elevated mineral concentrations. A similar case occurred with a drainage well in a citrus grove area. Lichtler stated that for 1959 to 1964 the Orange County Health Department recorded about 50 wells which had high bacteria counts (Lichtler et al., 1968, p. 129-133).

Lichtler and others possessed more imagination than many persons in pointing out the problems with drainage wells. One of the possibilities they mentioned was increased chance of sinkhole formations due to a possible increased rate of solution of the limestone aquifer by the injected water. Another possibility was permanent contamination of the aquifer after a nuclear attack if the radioactive fallout was washed into the drainage wells.

The next appearance of quantity of flow into drainage wells in the literature was probably by W.F. Lichtler, in 1972, in a U.S. Geological Survey Open File Report prepared for the East Central Florida Regional Planning Council entitled "Appraisal of Water Resources in the East Central Florida Region." He noted that the annual recharge into Orange County totaled 210 MGD and that since no appreciable cone of depression existed in the groundwater surface below Orlando, it was possible that then existing groundwater supply withdrawals of 50 MGD were being balanced by recharge through drainage wells. Thus the first estimate for total quantity of water entering Orange County drainage wells was 50 MGD (Lichtler, 1972, p.74). Lichtler also noted that injection rates of 5 to 10 cfs (2250 to 4490 gpm) were common for many gravity recharge wells (drainage wells) in the East Central Florida Region (Lichtler, 1972, p. 76).

He also noted pollution of a deep aquifer supply well by an unknown source. A 1300 foot well, cased to 1200 feet, had for two years produced raw water high in bacteria. Presumably the source of pollution was from a drainage well, but this was not ascertained in Lichtler's report (Lichtler, 1972, p. 74). On page 76, Lichtler stated that any recharge water should be of a quality at least as good as that of the aquifer water to avoid contamination because cavernous limestone did not provide much filtering action. Lichtler suggested holding basins and other facilities for pretreatment of rain water recharge into the aquifer through wells.

The East Central Florida Regional Planning Council 208 study performed in 1977 by E.E. Shannon of Black, Crow, and Eidsness/CH2M-Hill, Inc., in collaboration with Dr. Martin P. Wanielista of the University of Central Florida, was probably the first documented effort to backpump some drainage wells in an effort to determine if the immediately surrounding aquifer had been polluted. No specific estimates of quantity entering the wells were made, but the drawdowns in the two backpumped wells were measured versus time. This permits calculation of transmissivities. This is done in Chapter VIII of this thesis. A significant amount of water sampling and quality work was done in this report. One conclusion was that "there is evidence that drainage wells have caused contamination of upper aquifer water supply wells" (B.C.E./CH2M-Hill, 1977, p. 51).

Notable among the recommendations made in the East Central Florida Regional Planning Council report was that a study of 15 to 20 or more drainage wells be made to estimate the actual loadings of pollutants into the drainage wells. While this recommendation has not yet to this day been fully carried out, two other important recommendations have been (B.C.E./CH2M-Hill, 1977, p. 5-2 & 3). These are that the drainage well inventory be substantially updated (which has been significantly accomplished by the USGS and also by Carla Palmer and her colleagues at Dyer, Riddle, Mills, and Precourt) and that a study be made of a significant number of upper aquifer water supply wells and recharge wells in order to establish the background water quality of the upper aquifer. This has been largely done by the USGS, as documented in the USGS Water Resources Investigation Report No. 82-4094, "Effects of Recharge from Drainage Wells on Quality of Water in the Floridan Aquifer in the Orlando Area, Central Florida".

The next report to deal specifically with drainage wells was by Joel O. Kimrey of the USGS. His "Preliminary Appraisal of the Geohydrologic Aspects of Drainage Wells, Orlando Area, Central Florida," Water Resources Investigations, 78-37, while not containing any new specific information an quantity, does contain an excellent review of the history of drainage wells use, and a description of the geologic setting of both drainage wells and deep aquifer supply wells.

Kimrey noted that 1960 was the most active year for drainage well construction since 35 wells were constructed in that year (Kimrey, 1978, p. 9). The reason was that 1959 and 1960 were very wet years. However, because of the high aquifer pressure due to the rain, the wells were not able to adequately handle the elevated inflows. As a result flooding occurred. In fact, Kimrey reported that some wells in 1960 commenced artesian flow and were equipped with pressure injection pumps to force the water back into the aquifer; sort of an ironic twist of events that points out drainage wells do not always work, even when they are needed the most.

Kimrey also noted that gassing of wells was still occurring, due to two different causes. One was the entrainment of air in the water entering the well, the other was methane gas produced within the well, apparently due to decomposition of algae and other organics in lake water introduced into the wells.

Finally, Kimrey noted that, without question, some hydraulic connection exists between the Upper Floridan (drainage well zone) and the Lower Floridan (supply well zone) aquifers. Kimrey used potentiometric maps of the Upper Floridan and Lower Floridan Aquifer to illustrate this point.

In 1979 R.W. Hull and M. Yurewicz of the USGS published an Open File Report #79-1073 entitled "Quality of Storm Runoff to Drainage Wells in Live Oak, Florida." This report is important because of its analysis of constituents of the stormwater runoff.

In 1980 the first documented calibration of an Orlando drainage well was performed by Timothy B. Walsh as part of his master's thesis work at the University of Central Florida. The well calibrated was the 20-inch lake level control well located on the north side of Lake Eola, which was then the only lake level control well operating on Lake Eola. Walsh calibrated the well from lake level measurements made during the dry non-storm periods following two reasonably intense storm periods. From the data he obtained an exponential function expressing lake stage above the drainage well inlet versus time as $S = 13.6e^{-0.00877t}$, where S = head in cm, and t = time in hours after cessation of the storm event (Walsh, 1981). From this relationship Walsh derived an equation for the rate of inflow into the drainage well as

$$Q_{dw} = (ds/dt)(A) = bSA = (0.01 \times 0.00877)(S)\ (109300m^2)$$

where:

ds/dt = acceptance rate in m^3/hr

S = lake stage above drainage well inlet (cm)

A = Lake Eola's surface area = $109300m^2$

b = time decay constant of 0.00877 hr^{-1} derived from the data.

Walsh's work also contains some bar graphs giving estimates of the quantity of water entering the drainage well based on a mass balance approach. However, the measurements of seepage and evaporation and certain other parameters were uncertain, so the quantity entering the drainage well should be taken as only an estimate. The total quantity of water was estimated to be 25.70 x $10^6 ft^3$/year (192 million gallons), with the maximum monthly flow of 68.8 million gallons occurring in the month of May 1980. The minimum flows were zero in January, and 0.15 million gallons in February 1981. The average daily flow is calculated as 0.526 MGD.

A summary of some aspects of Walsh's work was given in "Impact of Stormwater Runoff on Lake Eola Water Quality," a paper that was presented at the Second International Conference on Urban Storm

Drainage, Urbana, Illinois, June 14-19, 1981. The co-authors of this report were Yousef A. Yousef, Martin P. Wanielista and Harvey H. Harper, III, (all from the University of Central Florida), and Richard P. Traver (from the U.S. Environmental Protection Agency).

In 1983, George R. Schiner and Edward R. German of the USGS published "Effects of Recharge from Drainage Wells on Quality of Water in the Floridan Aquifer in the Orlando Area, Central Florida," USGS Water Resources Investigation Report, 82-4094. This report contained the first table of theoretical acceptance rates of drainage wells for the range of existing sizes and for varying sizes of wells (Schiner and German, 1983, p. 16). These rates were based upon the orifice equation; and so for the smaller heads and larger well diameters the rates are probably low. Schiner and German observed that an 18-inch diameter well on Lake Adair was receiving an estimated 3400 gpm, with an estimated 0.75 head, on July 17, 1979. The recent prior rainfall was 6 inches from July 6 to July 17. Because of this large amount of rain in a short period of time the flow regime on the well was probably orifice flow.

Schiner and German then noted that the estimated groundwater supply withdrawals had increased to 85 MGD for 1980, but that the "balance of recharge and discharge probably still (1981) exists for the most part." They thus estimated that the quantity of water entering Orlando area drainage wells could be as high as 85 MGD and 40% of the total recharge of 210 MGD into Orange County could be due to drainage wells (Schiner and German, 1983, p. 15).

Schiner and German stated that records of 392 drainage wells occur in the USGS files. They categorize these as 50% storm drainage wells (direct discharge), 35% lake level control wells, and 15% other uses.

Schiner and German continue to do significant sampling of many drainage wells and supply wells for quality analysis.

The latest of the published United States Geological Survey reported on drainage wells is Water Resources Investigations Report 84-4021, by Joel D. Kimrey and Larry D. Fayard. This report, covering drainage wells and interaquifer connector wells throughout Florida, was issued in 1984 and is aptly entitled, "Geohydrological Reconnaissance of Drainage Wells in Florida." This report showed the water quality sampling results for Floridan aquifer drainage wells in Live Oak, Ocala and Orlando; for Biscayne aquifer drainage wells in the Miami-Fort Lauderdale area; and for interaquifer connector wells in Polk and Hillsborough Counties. Three generalized maps of well locations is given for the wells in Ocala, Orlando and Live Oak. Some sample well logs also appear.

Kimrey and Fayard mention work by C.H. Tibbal that suggested Orlando area drainage well recharge is 30 to 35 MGD. According to Kimrey and Fayard, about 35 to 40 square miles in Orlando are drained solely by drainage wells, and some of the surrounding area is partially drained by wells (Kimrey and Fayard, 1984, p. 43).

Another significant contribution of this document to obtaining a quantity estimate was that it stated that a total of 392 wells appeared in the Orlando/Orange County area and this number has been confirmed by field inventory (Kimrey and Fayard, 1984, p. 36).

Ed German of the USGS, Orlando Office, has conducted another study on drainage wells to establish the quantity of water entering lake level control drainage wells. His study, which may be released sometime in 1985 or 1986, focuses especially on field measurements made at Park Lake in Orlando and at Lake Midget in Winter Park, Florida.

Jammal and Associates, Inc., a geotechnical and hydrogeological consulting firm located in Winter Park, Florida, has done work on drainage wells. One of their reports, written by Robert Oros in 1983, covered some hydrogeological data for drainage wells in Ocala (Jammal, 1983, p. 3). Oros gave some rough estimates of maximum drainage well capacity for various size wells. These ranged from 2000 gpm for 6-inch wells to 8000 gpm for 24-inch wells. Twelve-inch wells were estimated at 4000 gpm (Jammal, 1983, p. 3).

Oros also calculated the estimated rise in well water level for various steady state well inflows and various assumed aquifer transmissivities. This appears in Figure 2 of his report.

In January 1984, the Orlando Urban Storwmater Management Manual (OUSWMM) Plan took effect. This was the first comprehensive inventory of the stormwater management system in Orlando, and included maps of the watersheds for each lake in Orlando. This study proved very helpful to the calculations made in this thesis and is described in greater detail in Chapter V.

In addition, Dyer, Riddle, Mills and Precourt have recently issued several reports dealing directly or indirectly with drainage wells. Most important among these is the "Drain Well Monitoring Plan - City of Orlando, 1985" for the City of Orlando, written by Carla N. Palmer. This plan describes the steps to be taken by the city to meet Florida Department of Environmental Regulation requirements that drainage wells be monitored. The monitoring program is scheduled to last five years. During the first year five wells are to be monitored for both quantity and quality of influent water. The specific five wells are described in Chapter 5 of the plan. This data will be of great value in improving the estimate of quantity of water entering the wells.

Some other helpful points of interest in this document include its present and projected land use maps for the city, maps showing the location of both drainage wells and supply wells within the city, and other maps.

What is particularly important about the monitoring plan is its list of active drainage wells within the city limits of Orlando. Palmer lists 175 of these in Table 3-1 of the Plan along with their location, diameter and depths. Most of these were field checked by Dyer, Riddle, Mills, and Precourt personnel. This number corresponds well with the 208 wells counted in the OUSWMM study area.

Also helpful is Table 3-2 which lists the City of Orlando supply wells and which drainage wells are located within one mile of each supply well.

Other helpful reports prepared by Dyer, Riddle, Mills and Precourt include "Lake Level Determinations - Downtown Orlando," DRMP #82-198R issued on January 30, 1984, and "Southeast Lakes Interconnects - Preliminary Engineering Report," DRMP #84-295H which was issued in November 1984. Both of these reports, particularly the first one, contained hydrologic data such as runoff curve numbers, time of concentration, etc., for the lake sub-basins in downtown Orlando. This data could be used later for a supplemental calculation of the runoffs for those basins, however the results may not be accurate due to the nature of the runoff curve number. Also, extensive interconnecting of lakes and modifications of the drainage structure of southeast downtown Orlando is planned. This will modify the hydrologic characteristics of that area, and thus, of course, modify the quantity of water entering the drainage wells in that area. The "Southeast Lakes Interconnects" report outlined the proposed changes and what the estimated final hydrologic characteristics would be, so an estimate of the quantity of runoff entering drainage wells from that area could be recalculated.

Finally, there is a wealth of other data available relating to drainage wells in the Orlando area. The City of Orlando and Orange County both possess "drainage well notebooks" describing the wells they own and their locations. The City of Orlando's book is maintained by the Streets and Drainage Department; a copy is available for viewing in the Records section of City Hall. Orange County's notebook is maintained by the County Engineers Office located on 33rd Street, Orlando. It includes photographs and location maps of most of their wells. In addition, Orange County has recently (April 1985) completed videotape logging of about 40 of the almost 130 wells which they own. These loggings showed that the drainage well casings were generally in good shape. These videotapes are also kept at the 33rd Street office.

The City of Orlando surveys and records monthly the elevation of every lake within the city limits. This data is useful for storage estimates and is available for viewing at the Records Department in City Hall. They also have a computerized list of every lake, it's shoreline and area, and elevations of flooding.

Lastly, the United States Geological Survey Office in Orlando, located on S. Hughey Avenue, has a computerized listing of every known drainage well in Florida, by county. The wells are precisely identified by their latitude and longitude, and information such as owner, size, depth, and types of available geophysical logs are included.

TABLE 3

SUMMARY TABLE OF DRAINAGE WELL LITERATURE

AUTHOR	YEAR	ORGANIZATION AND REPORT NUMBER*	TOPICS	QUANTITY REF.	QUALITY REF.	WELL LOGS	POTENTIOMETRIC SURFACE MAPS	OTHER MAPS
E.H. Sellards	1908	F.G.S. Bulletin #1	Factors Affecting Inflow Rate, Early Drainage Well History, Pollution Considerations	p. 61-63	p. 64-65	None		
E.H. Sellards and H. Gunter	1910	F.G.S. Third Annual Report	Theoretical Inflow Formula, Spouting Wells	p. 69-71, 74	None	None		
V.T. Stringfield	1933	F.G.S. Bulletin #11	Inflow Rate Estimate	p. 22	p. 19-24			
V.T. Stringfield	1936	USGS WSP #773-C	Estimate of Number of Wells, Inflow Rate Estimates	p. 161-162	None	None		
A.G. Unklesbay	1944	FGS RI #5	Number of Wells, Geological Description, Velocity Measurements for Receiving Zones	Throughout	p. 31-31	Yes	Yes	Yes
J.S. Telfair	1948	Florida State Board of Health No report number	Bacteriological Survey, Groundwater Tracing, Generalized Drainage Basin Descriptions	None	Throughout	Yes	Yes	Yes
Lichtler, Joyner, Anderson	1968	FGS	Pollution Aspects, Salt Water Tracing, Pumping Tests to Estimate Transmissivity	p. 134-136	p. 128-134	Yes		
W.F. Lichtler	1972	USGS Open File Report also FGS RI #61	Reports Pollution of Deep Aquifer Supply Well, Estimates Total Recharge by Wells at 50 MGD	p. 74	p. 74, 76			
ECFRPC	1977	ECFRPC 208 Study	Drainwell Pumping Tests, Chemical Analysis, Drainwell Location Map, Stormwater Analysis	p. 3-3, 3-6 p. 1-8	Throughout	Missing	Yes	Yes
Joel O. Kimrey	1978	USGS WRI #78-37	Drainwell History, Geological Overview, Possible Aquifer Interconnections	p. 13, 15, 21	p. 15, 18, 21	None	Yes	Yes
R.W. Hull and M. Yurewicz	1979	USGS OF #79-1073	Quality of Storm Runoff to Drainwells in Live Oak, FL		Throughout			
Timothy B. Walsh	1981	UCF Master's Thesis	Attempted Calibration of Lake Eola Drainwell	Throughout				
E.G. Schiner and E.R. German	1983	USGS WRI #82-4094	Theoretical Inflow Rates for Individual Wells. Estimate Total Acceptance Possibility = 85 MGD, Extensive Water Quality Sampling/Analysis	p. 15-16	Throughout	Yes	Yes	Yes

TABLE 3 -- CONTINUED

AUTHOR	YEAR	ORGANIZATION AND REPORT NUMBER	TOPICS	QUANTITY REF.	QUALITY REF.	WELL LOGS	POTENTIOMETRIC SURFACE MAPS	OTHER MAPS
J.O. Kimrey and L.D. Fayard	1984	USGS WRI #84-4021	Estimate Total Acceptance 30 to 35 MGD for Orlando Area, Field Verified Well Locations Map, Information on All of Florida Drainage Wells.	p. 36, 43	Throughout	Yes	No	Yes
Robert Oros	1983	Jammal & Associates 83-727	Rough Estimates of Maximum Rates for Individual Wells. Water Level Mounding in Drainwells.	p. 3	None	Yes	Yes	Yes
Dyer, Riddle, Mills, Precourt	1984	City of Orlando OUSWMM	Drainage Basin Maps, Land Uses for City of Orlando, Other Stormwater Information.	N/A	N/A	N/A	N/A	Yes
Carla N. Palmer	1984	DRMP 83-141	City of Orlando Drainwell Monitoring Plan - Preliminary Draft	p. 6-10	Throughout	Yes	Yes	Yes
Dyer, Riddle, Mills, Precourt	1984	DRMP 84-295H	City of Orlando-Southeast Lakes Interconnects-Details Proposed Drastic Changes to Southeast Orlando Drainage Pattern Which Will Affect Drainwell Recharge-Preliminary Draft.	N/A	N/A	N/A	N/A	Yes
City of Orlando	1985	City Drainwell Notebook		Yes	Yes	Yes	No	No
Orange County	1985	County Drainwell Notebook and Videotape logs		Yes	Yes	Yes	No	Yes
U.S. Geological Survey	1985	USGS Orlando Office	Computerized Inventory of Florida Drainwells	No	No	Yes	N/A	N/A

*Explanation of Abbreviations:

FGS	- Florida Geological Survey
USGS	- United States Geological Survey
ECFRPC	- East Central Florida Regional Planning Council
UCF	- University of Central Florida
City/SDD	- City of Orlando - Streets and Drainage Department
County/WMD	- Orange County - Water Management Engineering Department
WSP	- Water Supply Paper (USGS)
RI	- Report of Investigations (FGS)
WRI	- Water Resources Investigations (USGS)
OF	- Open File Report (USGS)
OUSWMM	- Orlando Urban Stormwater Management Manual

CHAPTER IV

FIELD DATA

Importance of Field Data

All engineers do, or should, understand the importance of field data. In the field of hydrology, especially, field data is needed for accurate calculations. Yet field data is often the most difficult information to obtain. This is especially true for this particular problem of estimating the quantity of aquifer recharge through drainage wells in the Orlando area.

Just finding and visiting once all of the 208-some-odd drainage wells in the study area would be a major task, requiring perhaps 200 to 400 or more man hours. Obtaining flow rates for each well would be even harder since each well has its own individual characteristics. For the lake level control wells, weirs would have to be tailor-made for each site; for the street drainage wells, flow meters would have to be installed for each site at which that an estimate was desired. Even then the results, if only from a single event for each site, would be only partially useful since rainfall varies markedly through the year, and also may vary from site to site during any one storm event. The time, labor, and money necessary for such a task are just not available.

Scope/Objective for Taking Field Data

Yet some field data is still desirable and useful, and so for that reason it was decided to perform a limited project of obtaining water level fluctuations in a few wells and to supplement this with a field calibration of the inflow rate for one well. It was believed that observing these variations during both dry times and storm events could be useful for deducing general guidelines about how much a given well could receive. The idea was to try to get a general idea on how much water a specific well could take by observing the response of the well's water level to increased inflow of water. The goal was to try to relate water level to inflow.

To achieve this goal both the inflow and the water level had to be accurately estimated. Estimating the former proved to be easier and was valuable for making generalizations about how much water all the wells could accept.

Overview of Sites

There were two sites upon which significant field work was done. These were a 20-inch diameter drainage well located on Lake Angel in Orlando, Florida, and a 12-inch diameter well located on the Department of Transportation's (DOT) retention pond at the intersection of US 441 and SR 436 near Apopka, Florida.

In addition, some water level measurements were made on a 6-inch diameter drainage well behind the New England Heights Baptist Church in Apopka, and a 12-inch diameter drainage well located at the intersection of Avondale and 18th Streets, adjacent to Interstate 4

TABLE 4

GROUNDWATER LEVEL FLUCTUATIONS IN DRAINAGE WELLS

		SITES			
		CHURCH	441-436	LAKE ANGEL	RAINFALL AT ORLANDO JETPORT (INCHES)
GROUND SURFACE ELEV. (APPROX. FT - MSL)		150	85	109	
DATE-TIME		GROUNDWATER ELEVATION (FEET - MSL)			
9/21/84	11:00		58.0		0.00
	15:00			60.0	
	15:45			60.0	
9/22/84	14:00			59.0	0.00
	18:00		57.5		
9/23/84	17:00		57.5		Trace
9/24/84	15:00			59.4	
	16:00				0.00
9/26/84	14:45		59.0		
	15:15	54.0			0.00
	16:00			59.3	
9/27/84	10:30			58.8	1.94
	12:30			109.0	Lake Angel
	13:00			83.0	well filled
	14:45		57.5		to the lake
	15:15	54.0			surface
9/28/84	13:30			58.0	Trace
	15:45	54.0			441-436 well
	17:00		71.0		flowing (local rain)
10/3/94	16:00			59.3	0.00
10/4/84	15:30			59.6	0.00
10/5/84	16:15			59.5	
	17:45		57.4		0.00
10/9/84	11:00			59.3	Trace

TABLE 4 -- CONTINUED

	SITES			
	CHURCH	441-436	LAKE ANGEL	RAINFALL AT ORLANDO JETPORT (INCHES)
GROUND SURFACE ELEV. (APPROX. FT - MSL)	150	85	109	
DATE-TIME	GROUND WATER ELEVATION (FEET - MSL)			
10/10/84 15:30			59.0	Trace
10/19/84 18:00			57.9	Trace
19:30	53.8			
20:00		56.5		
10/20/84 16:40		56.4		0.00
17:00	53.3			
10/26/84 15:00			57.7	0.53
10/27/84 11:40	53.1			
12:30		56.5		Trace
11/9/84 11:45			57.7	0.14
11/13/84 15:15		56.0		0.00
16:33	53.0			
11/14/84 15:00			57.3	0.00
11/21/84 11:00			57.0	0.50
17:00			57.1	
11/22/84 8:30	52.6			0.86
9:00		56.1		
11:00			57.7	
12/15/84 10:35	52.8			0.00
11:00		55.9		
13:40			58.1	

in Orlando. Water level measurements were recorded on a periodic basis for all of these wells. The results are summarized in Table 4.

Lake Angel

Lake Angel is located in southwest Orlando. It is bound by Interstate 4 on the east, Parramore Avenue on the west, and Harding Avenue on the north. Lake Angel is a small lake, probably originally a marsh that was dug out for fill and to serve as a retention pond for Interstate 4's construction. The drainage well is about 50 feet from the lake's northwest corner.

As it turns out, the Lake Angel site is probably the most valuable. One of the reasons for this is that water was continuously flowing into the well, and more importantly the flow rate entering the well increased greatly during storm periods. In order to insure accurate water level measurements during storm events a 60' stilling well was constructed of three 1" diameter by 20' PVC pipes and secured into the drainage well.

The other major reason for the value of the Lake Angel site is the manner of construction of the well box, with a ready made rough broadcrested weir upstream of the well.

A more detailed description of Lake Angel, its well and the experiments performed on it can be found later in this chapter. A map of the lake, a map of the lake's drainage basin and a cross section of the well are also included. Table 4 shows water levels.

Avondale Avenue/18th Street Well

This is a 12" diameter well within the drainage basin for Lake Angel. The well is on the east side of Avondale Avenue at the T-intersection of Avondale Avenue and 18th Street, underneath a rather tricky-to-remove manhole cover. This well acts as an upgradient overflow relief for the 36" trunkline, that parallels Interstate 4, leading down to Invert #1 at Lake Angel. The well also receives some water from residential storm sewers.

The Avondale well receives direct discharge of storm waters and fits into the classification of a "storm sewer" well as described in Chapter V. It has a storm sewer system feeding it, so it is not an "alone" well; because it is not located directly on a lake and used for lake level control it is obviously not a "lake" well. As a "storm sewer" well it does not take water continuously, but only while draining the storm sewer during and after an event.

No stilling well was ever needed for the Avondale well, because water level measurements were only taken during no flow or very low flow periods. The few measurements that were taken correlated well with the Lake Angel well measurements, and so are not shown here. Measurements of observed heads on the well are included in Table 7 in Chapter VII.

Apopka US 441/SR 436 Pond

The second major site was the Florida DOT retention pond on the southwest side of the Y intersection of SR 436 and US 441, just south of Apopka. This pond does not have a name, but it can be found on the extreme western edge of Section 10, Township 21 South, Range 28 East of the Forest City USGS Quadrangle map. The pond is about one acre or less in size, and is the low point in a depression in the pasture and sloping ground around it. The mean elevation of the pond is about 78 feet. Highway runoff from US 441 is collected in a gutter along the highway; this is discharged through a 15" pipe and spillway into the east side of the pond. This side of the pond is very overgrown with water oak trees. On the west side of the pond a weed-choked ditch leads off to Sheeler Avenue, which is a low-density residential area. Three large culverts pass under Sheeler Avenue and empty into the ditch, indicating the possibility of a much higher amount of residential runoff than highway input. Tracing two of these culverts showed that both of them were used to drain only one home's front yard, and the third culvert was the 5' diameter main trunk line along Sheeler Avenue. No significant flow was ever observed in any of the culverts during the study period.

The well system at the Apopka US 441/SR 436 site has a somewhat unusual design. A 24" concrete pipe leads from the lake into a round deep cylindrical manhole box. On the opposite side of the manhole from the pipe is a steel grating and another pipe leading to a similar cylindrical manhole box. The casing cutoff of the well is flush with the bottom of the floor of the second manhole. A cross section of the system is shown in Figure 4. The drainage well is 400' deep, and cased to 125'. The casing is 12" I.D. There is no grating over the well casing itself.

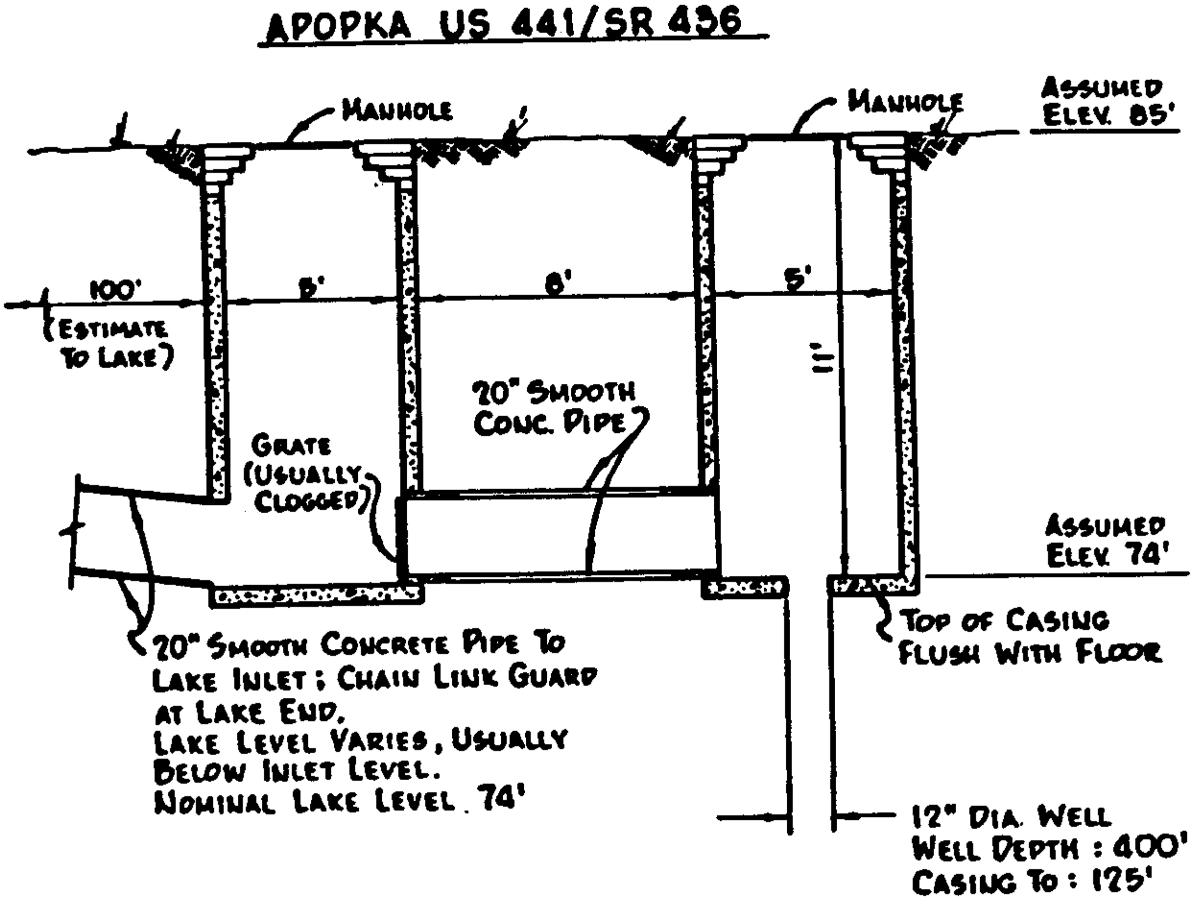

FIGURE 4 CROSS-SECTION OF 441/436 WELL SITE

A stilling well consisting of two 20' x 1" PVC pipes was assembled and placed in the drainage well. The normal depth from the manhole rim to the static water level in the well is about 30'. The depth of the manhole box is about 10', but because there is no grating over the well, the stilling well had to be lashed up higher and the bottom of the stilling well only extended about 10' into the static water.

No weir existed for this well, and though it was planned to install a cylindrical insert into the well which would create a weir, this was never necessary. The grating in the first manhole was usually clogged with plastic garbage bags or similar debris which limited flow into the well to about 10 gpm or so. More important, in October and November the pond stage declined rapidly due to lack of rain. The drop in head available caused inflow to the well to be almost zero. By the end of October, the pond stage was below the inlet pipe, and so flow was zero. As recently as February 1985 the pond was observed to be almost dry. These facts support the conclusion that this pond tends to act as a wet/dry retention area, with the water normally being lost by seepage exfiltration out of the pond, and the drainage well serving only as an emergency overflow and stage control during very wet periods.

The water level readings recorded for this site are found in Table 4.

Apopka Church Well

The final site at which static water level readings were made was also in Apopka, behind the New England Heights Baptist Church on Hawthorne Avenue. This well is also owned by the Florida DOT.

The well is located in a stand of brush and trees behind the church and accepts overflow from a small lake. The lake is almost hidden from view, when looking from the well, by the density of the thorns and brush. The exact inlet from the lake to the drainage well could not be found because of this. However, the fact that the well was always receiving a small amount of water helped to indicate that the well was a lake level control well.

This well is 12" in diameter, drilled to a depth of 430' and cased to 131'. The normal land elevation of this area is high, about 150', and so the depth to standing water in the well was great, averaging 97' to 98'. The inlet pipe to this well was probably partially clogged; the amount of water visually observed entering the well never seemed to be very great, probably at the most five gallons per minute. This flow rate was not enough to merit installing a stilling well. No flow-measuring device was ever installed on this well, either. The static water levels recorded for this well are found in Table 4.

Lake Angel Site -- Detailed Description, Discussion

Description

Lake and Drainage Basin. Lake Angel was probably originally a marshy area. When Interstate 4 was constructed, the lake was dug out. The

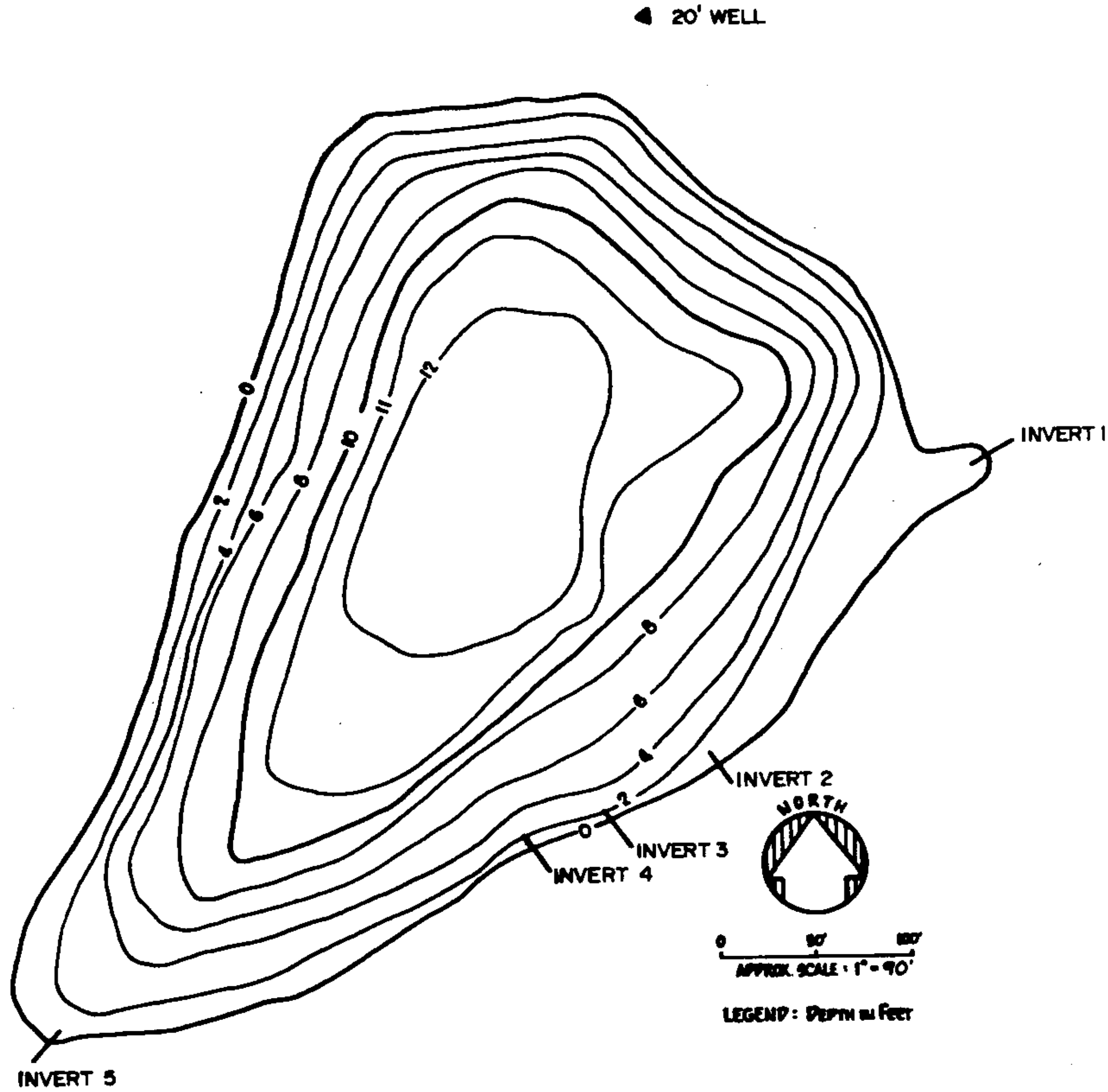

FIGURE 5 LAKE ANGEL WATER DEPTH MAP

present shape of the lake is roughly that of an isoceles triangle, with the base at the north end of lake. The lower half of the west side is rough and uneven, marshy and covered with brush. A map of the lake, along with the inlet pipes, is shown in Figure 5.

The north and west sides of the lake are bounded by a chain link fence. The eastern boundary is Interstate 4. Presently the lake serves mainly as a stormwater detention pond for runoff from Interstate 4. The lake is now regarded as joint City of Orlando and Florida Department of Transportation property, with lake maintenance being performed by the city.

The only verified existing positive outfall from the lake is the subject drainage well, located on the northwest edge of the lake. This well is owned and maintained by the Florida Department of Transportation (FDOT). The well has a 20" inside diameter (I.D.) casing, and FDOT records indicate the well was drilled to 172' and cased to 145'.

According to Jack Sellers of the Streets and Drainage Department of Orlando (formerly of the FDOT), the well was constructed in 1969 or 1970 as a replacement for a previous 20" diameter well (constructed in 1959 on the northeast corner of the lake) that was irreparably damaged when a sink formed around it. This damaged well was grouted and dirt was backfilled over it so it is no longer visible.

The records of the City of Orlando's Engineering Department indicate that a 8" diameter drainage well was once located on the west edge of the lake within the marshy area. The existence of this well was confirmed by some long term residents of the area; however, repeated field searches failed to locate this well. It is possible that, during the construction of Interstate 4 the well was buried by fill and lost. If so, the well may or may not still be partially operating, taking in water by groundwater seepage and sending it to the aquifer. It should be noted that many wells have possibly been destroyed or "lost" by highway construction, particularly by the construction of Interstate 4.

Some other maps by the City of Orlando indicate the existence of a 24" well on the northwest edge of the lake, along with a 42" residential sewer outfall. This is totally erroneous, as neither field investigations nor other historical data confirm the existence of such systems.

In summary, the water inputs to Lake Angel appear to be the five storm sewers and one swale on the east side of the lake, which essentially handle only highway runoff; some slight amount of overland flow and direct precipitation; and groundwater infiltration. Groundwater infiltration may be a significant input, since the lake level was maintained high enough to have the well flowing continuously through the study period, even during the very dry months of October and early November when there was no rain. There are no natural positive outfalls from Lake Angel. The only manmade outfalls are the 20" drainage well which is being studied and possibly a lost 8" drainage well on the west side of the lake.

According to the maps of the FDOT, the drainage sub-basin for Lake Angel includes a total of 130 acres, primarily from Interstate 4, and being drained by the 36" pipe going to INVERT #1. This area is also partially drained by the 12" basin well at the intersection of Avondale and 18th streets.

The Lake Angel drainage sub-basin area, as corrected from the Orlando Urban Stormwater Management Manual (OUSWMM), is shown in the map in Figure 6. It should be noted that while not all of this area drains into Lake Angel itself and then into the well, all of this area is drained primarily by drainage wells, and their operation could have an impact on the Lake Angel well. A list of all the wells in the sub-basin, their locations, diameters and depths is given in Table 5.

<u>Drainage Well and Weir</u>. The drainage well at Lake Angel is located on the northwest corner of the lake, very near the intersection of Harding Avenue and Parramore Avenue. A 36" diameter corrugated pipe leads 50' from a concrete abutment on the edge of the lake into a concrete well box. This pipe was always observed to be submerged and flowing full. Water exits the pipe into a small weir tank and spills over the weir into the other half of the well box. Water pools up on this side of the box and overflows down into the drainage well. The casing cutoff of the well is about 7" to 8" above the floor of the

TABLE 5

INVENTORY OF DRAINAGE WELLS IN LAKE ANGEL SUB-BASIN

WELL NUMBER	WELL SIZE	WELL LOCATION	WELL DEPTH	CASING DEPTH	WELL CLASS
1	20"	Lake Angel	172'	145'	Lake Control
2	12"	Avondale and 18th Street	513'	110'	Storm Sewer
3	20"	Conroy St. between Parramore and Avondale	Unknown	Unknown	Lake Control (small retention pond)
4	6"	Indiana St. and Westmoreland Drive	451'	146'	Alone well (located in private driveway)

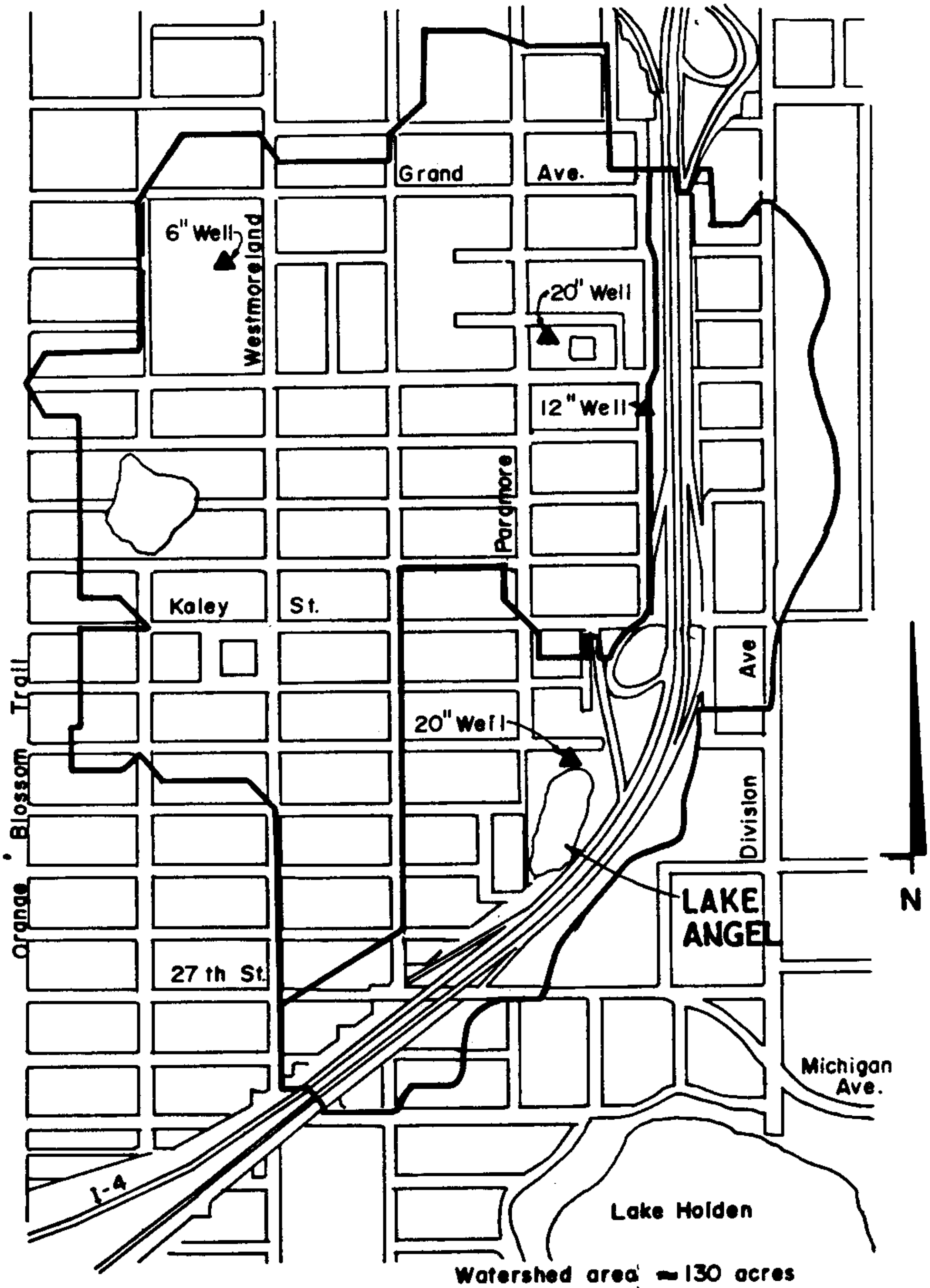

FIGURE 6 LAKE ANGEL WATERSHED

well box. The well casing itself is steel, with an inside diameter of 20 inches. The well is 172' deep, cased to 145'.

The weir itself was originally probably just a concrete wall, part of a grating system designed to prevent trash from entering the well. Almost all of the grating has long since rusted away, leaving only a few stubs of steel rebar which poke out of the top of the weir, slightly obstructing flow over the weir. The weir is broadcrested, 6" thick and 47.5" long. The floor of the weir tank is very rough and pitted, but it appears that the design height of the weir wall was about 44". The top of the weir is not exactly level, as it tilts about 3/4 inch across its length of 47.5". This required taking three measurements, one at each side and one in the middle, for the head on the weir ("water over the weir") and averaging them. A cross section of the weir and well box is given in Figure 7.

Some time into the data-gathering process the question of the effects of contraction of the water over the weir came up. A staff gage was constructed and placed on the back of the weir tank, and "water over the weir" head readings were correlated to the staff gage readings. Under normal flow conditions, it was discovered that head contraction is minimal, less than approximately 1/8 inch. Taking the staff gage readings required crawling down into the manhole with a flashlight, which was difficult at best, so generally just the average of the "water over the weir" measurements was taken for the head on the weir.

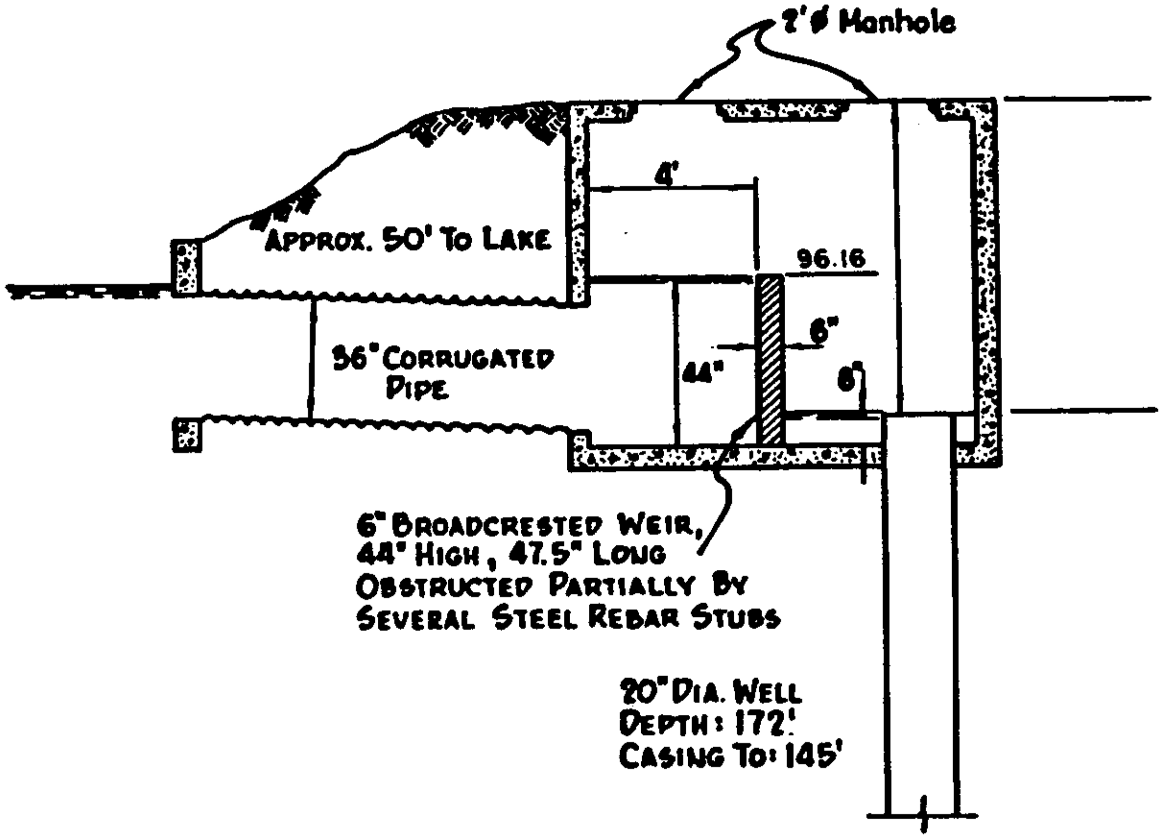

FIGURE 7 CROSS-SECTION OF LAKE ANGEL WELL SITE

Measurements

Weir Calibration and Well Calibration. The weir was calibrated during the week of Monday, February 4, 1985 to Monday, February 11, 1985. The average velocity of the water in the 36" inlet pipe was measured using a Montedoro-Whitney Corporation PVM-2 Fluid Velocity Meter. The flowrate Q was calculated by multiplying the velocity by the area of the pipe (7.0685 ft^2). At the same time average head measurements were made on the weir and the well. The results of these measurements are presented in Table 6.

The weir equation is of the form

$$Q = CLH^n \qquad \text{(Equation 6-1)}$$

where:

Q = flow over weir

L = length across the weir

H = head on weir

C,n = coefficients to be determined for each weir

C and n can be found (given Q, L and H) by making a logarithmic transformation of the equation:

$$\log Q = \log C + \log L + n \log H \qquad \text{(Equation 6-2)}$$

The weir head data of Table 6 is plotted on a 3 x 3 cycle log-log plot in Figure 8. It is immediately noted that this plot is very straight. Referring again to Equation 6-2, we see that it is of

TABLE 6

LAKE ANGEL WEIR AND WELL CALIBRATION

DATE	TIME	AVERAGE HEADS WEIR (ft)	WELL (ft)	36" INLET PIPE VELOCITY (fps)	INFLOW Q (cfs)
2/4/85	10:00	0.0521	--	0.02	0.141
2/6/85	18:00	0.2917	0.250	0.25	1.767
2/8/85	15:00	0.0625	0.0417	0.025	0.177
2/11/85	18:00	0.0833	0.0625	0.04	0.283
3/21/85*	21:00	0.5833	--	0.80	5.656

*(Not included in regression analysis)

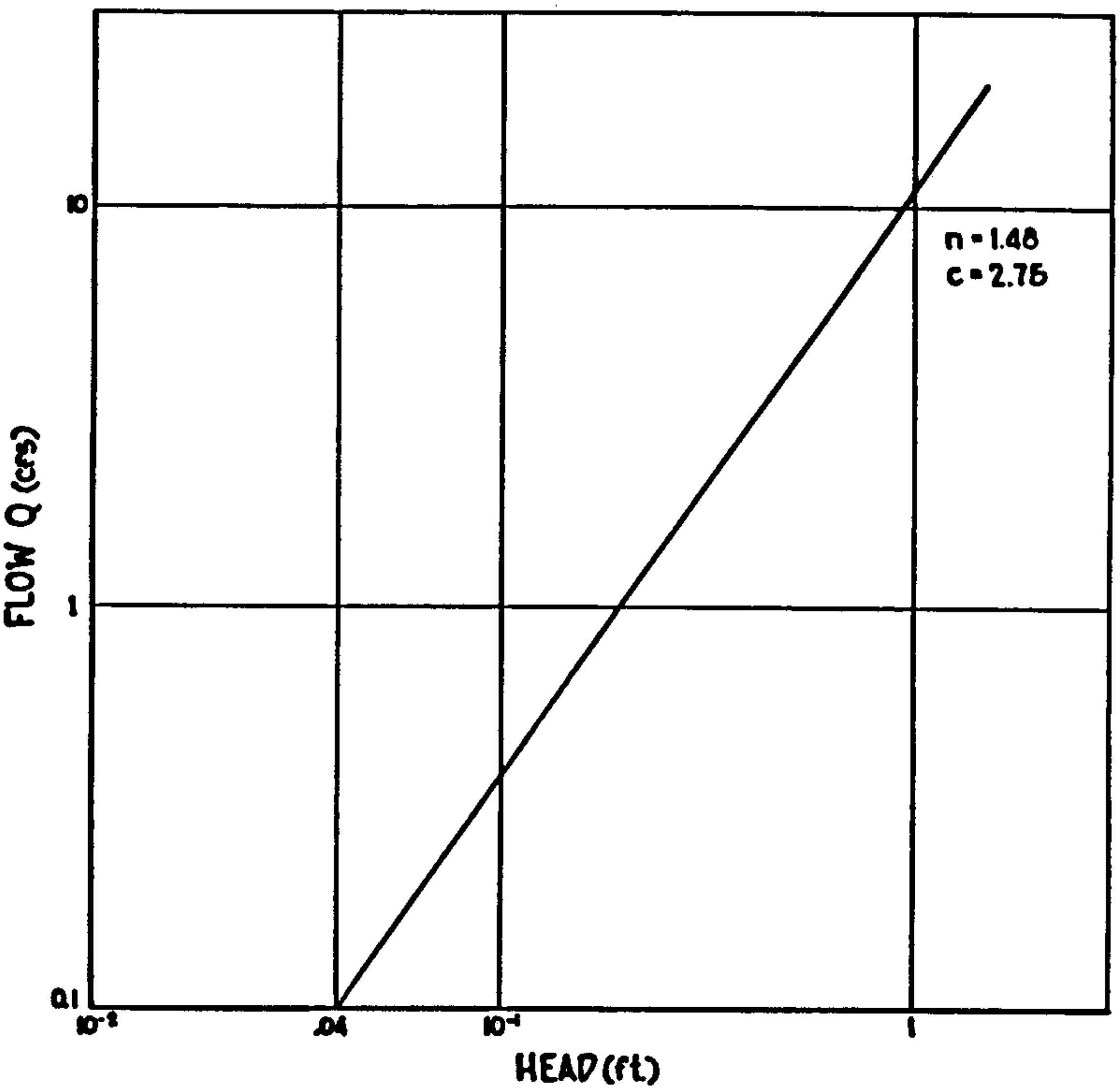

FIGURE 8 LAKE ANGEL WEIR CALIBRATION: Q vs H

the same form as the equation of line $y = mx + b$, with log Q corresponding to y, n corresponding to the slope m, log C + log L corresponding to the intercept b, and log H corresponding to x.

Logarithmic least squares curve fitting was performed and the parameters n and C solved for.

The slope n was calculated as

$$n = \frac{\log Q_2 - \log Q1}{\log H_2 - \log H_1} = 1.48$$

knowing n permits solving for C:

$$\log C = \log Q - \log L - n \log H$$
$$= 2.75 \text{ by regression analysis}$$

In a manner similar to that done for the weir at Lake Angel, the 20" drainage well itself was calibrated. The head H on the rim of the well was measured and correlated to the flow rate Q into the well. The flow rate Q was determined by measuring the average velocity in the 36" inlet pipe to the well box. This data is also in Table 6.

Presently, due to lack of significant storm events, only three points exist. These three points have been plotted on a log-log plot in Figure 9.

The slope n of the resulting line of best fit is

$$n = \frac{\log Q_2 - \log Q_1}{\log H_2 - \log H_1} = 1.295$$

The coefficient C of the equation is $C = Q/LH^n = 2.02$

Static Water Level Measurements. Static water levels in the well were measured on a periodic basis. Since the well was continuously taking water, it was necessary to construct a stilling well within

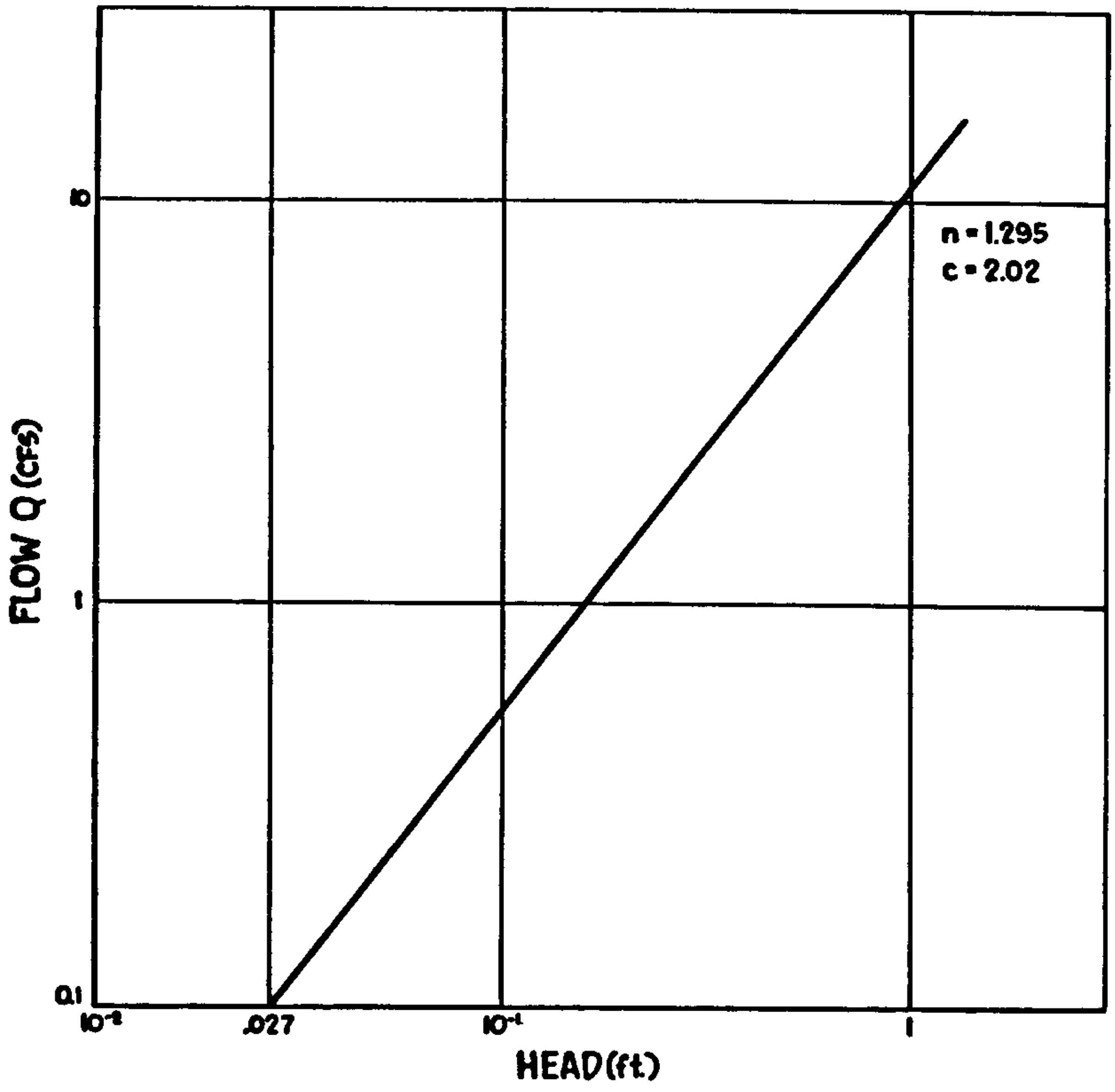

FIGURE 9 LAKE ANGEL WELL CALIBRATION: Q vs H

which the depth indicator probe could be lowered. This was done by assembling three pieces of 1" diameter by 20' long PVC pipe. At the lower end of the pipe, steel wire coils were attached to keep the pipe centered. The upper end of the pipe was attached by rope to some grating level with the casing cutoff. The static water level had been previously estimated to be about 50 feet below the rim of the manhole. Since the distance from manhole rim to top of casing cutoff was 8 feet, this would allow a drop of static water level of almost 60 + 8 - 50 = 18 feet and still insure that the depth indicator probe would be within the stilling well. This insures the protre would be reading only the standing water level and not be affected by any falling water in the drainage well. The recorded water level measurements have been given in Table 4.

Backflow Event

An unusual phenomenon occurred on Thursday, September 27, 1984, at the Lake Angel drainage well. A heavy rainstorm occurred on this date in Orlando; the total rainfall recorded at the McCoy Airport weather station was 1.94". The flow of water entering the well was increased substantially. The head on the weir increased from about 2 inches to over 4 inches, which would correspond to a flow increase from 360 gpm to 990 gpm, or 175%.

It was observed that the water level in the well fluctuated wildly. It is possible the well was, or is, partially clogged. Water would fill the well to a certain level until sufficient head could be developed to open the clog, then the well would drain down, the clog would again close, and the well would refill.

This cycle of well filling and then draining was repeated periodically until at one point the apparent well clogging became so severe that the well filled up completely to the top of the casing cutoff. The water continued to fill up to the level of the weir, and then beyond, so that the water level downstream of the weir was higher than that upstream of the weir; so in essence water was flowing backwards, towards the lake. The filling continued until water was spouting from the manhole over the drainage well. This lasted for a period of ten seconds, and then the flow subsided.

Soon after this event occurred, the researcher removed his measuring equipment, replaced the manhole cover, and traveled to another site to take measurements. Upon returning to the Lake Angel site the next morning, it was observed that the 85-pound manhole cover had been lifted up and set lightly to the side of the manhole. This was probably done by a subsequent refilling of the well manhole box. According to Mr. Hardin, who resides at 645 W. Harding Street, he and his son have frequently found the manhole covers to be off after a heavy storm, and have had to replace them. Mr. Hardin's house is on the corner of Parramore Avenue and Harding Avenue, immediately across the street from the well.

What is most unusual, or puzzling, about this event is that the water level downstream of the weir momentarily exceeded the water level upstream of the weir. This indicates that the head of the water in the well was greater than the head of the lake, which was supplying the water. This gives the appearance of a negative head loss having occurred, which is contrary to the laws of physics.

What may have happened is that the well became so tightly clogged that the force of the water entering the well was like a water hammer, causing the water level to jump. But this does not match what was observed. The filling of the well, though quite rapid, was steady and not jumpy. Another possibility is that the well became tightly clogged and filled up even with the weir, yet, because the sheer inflow of water from the lake was so rapid and so large the water level did not immediately stabilize. However, this explanation does not account for why the water level did increase to the point where it was spouting out of the manhole cover. Also, why did the water level rapidly subside after reaching its maximum point? If the well were truly that tightly clogged, why would it then become unclogged so quickly? More importantly, how does the well become repeatedly very tightly clogged and then unclogged for every heavy storm event?

The most likely explanation to the author is that the aquifer itself is experiencing localized flooding. Some nearby drainage wells which are of a higher elevation than the Lake Angel well may be injecting enough water into the aquifer during heavy storms so that it must temporarily escape out of lower elevated drainage wells, such as Lake Angel. This could explain the dynamic nature of this back-flow event, since the inflow to these drainage wells would be somewhat variable, and partial clogging of some wells could be occurring.

Artesian flow of drainage wells has been recorded in the literature, particularly during seasons of heavy rainfall. Unklesbay recorded that, in the summer of 1930, artesian flow of a well near Orlo Vista caused highway flooding, while other wells simply stopped accepting water (Unklesbay, 1944, p. 10). Joel Kimrey of the USGS reported that, in the fall of 1960, some drainage wells "flowed at land surface and had to be equipped with pressure injection pumps to allow their continued use" until the Floridan aquifer's potentiometric surface declined (Kimrey, 1977, p.10). 1959 and 1960 were years of very heavy rainfall, resulting in record high potentiometric levels for the upper and lower zones of the Floridan aquifer, as shown in Kimrey's report.

While no explicit mention is made of well backflowing or artesian flowing occurring as a result of a single rainfall event, it seems plausible that it could happen on a very localized basis, if the conditions were right. The concept is very interesting -- a very dynamic, responsive aquifer.

Another phenomenon that is well documented in the literature is that of gassing wells, either due to escape of air that was originally entrained in the turbulent water entering the well, or gassing due to escape of methane and other byproducts of the decomposition of organic matter in the well. Kimrey reported on some occurrences of well gassing (Kimrey, 1977, p.15). Well gassing could be an explanation of the particular backflow event that happened at Lake Angel on 9/27/84. However, well gassing would have been like a geyser going off; what occurred at Lake Angel was a rapid, brief, but steady filling and subsiding of water in the well and well box.

Maximum Flow Event

On Thursday evening, March 21, 1985, a very hard rain of 4.06 inches occurred. The runoff was sufficient to raise the lake level by about 6 inches to 96.75 feet. The rate of inflow, of course, was very greatly increased; and it appears from the behavior of the well to have attained the maximum possible for that well.

What was observed to occur was that the well and the downstream weir tank were periodically filling up, although only to a level even with the upstream weir tank's water level, and draining to a level about 3 inches above the well casing. The well itself was never visible because of the rapidity of the inflow of water. The filling and draining cycles took about 30 seconds to complete. Average velocity measurements were taken in the 36" inlet pipe for many cycles; they varied from about 1.0 foot/second when the downstream weir tank was draining, to about 0.6 foot/second when it was full, even with the upstream weir tank. The weir tank staff gage reading increased by about 6.5 to 7 inches.

The depth of water over the weir increased greatly too, to about 6 inches, although this varied with the cycle of filling and draining. Contraction of water over the weir was now very apparent. The depth of water over the weir was generally about 1/2 inch to 1 inch less than the increased water level as read on the staff gage.

The average velocity, as read by the Montedoro-Whitney Velocity Meter, was estimated at 0.8 feet/second. This corresponds to a flow rate of 5.2 cfs, or 2540 gpm. Judging from the violent rapid filling and draining behavior, this is very nearly the maximum possible capacity for this well.

The apparent cause of the filling and draining action is an air bubble being entrained by the filling action, and then being released immediately prior to the draining action. The draining action was much more rapid than the filling action. Draining took perhaps 3 to 5 seconds; filling took about 10 seconds. Possibly supplementing the effect of the air bubble on limiting flow was a new flow regime transition between pipe and orifice. When the weir tank was full, with about 44 inches of water above the well casing cutoff, the inflow could have been pipe flow. Since this would be slower than orifice flow, the incoming upstream flow would be slowed; as the downstream weir tank drained and the flow changed to orifice flow, incoming flow would be speeded back up. However, it seems likely that the cause of the cycling would have to be an entrained air bubble periodically being released. Otherwise the flow rate would be steady; either pipeflowing or more probably orifice flow.

This "maximum flow" event, with its violent filling and draining action, prompts more curiosity and questions of the earlier "backflow" event of September 1984. Was the "backflow" event actually caused by an entrained air bubble rather than localized aquifer flooding? The rain was twice as severe for the March 1985 event compared to the September 1984 event. Why didn't backflow occur in March 1985 as well? These and other questions shall be the enigmas of future researchers.

CHAPTER V

ORLANDO URBAN STORMWATER MANAGEMENT MANUAL

Purpose and Introduction

The Orlando Urban Stormwater Management Manual (OUSWMM) was completed in January 1984 by the consulting firm of Dyer, Riddle, Mills and Precourt, Inc., for the City of Orlando.

The Orlando Stormwater Management Manual covers a study area that is actually slightly larger than the 1982 city limits. It delineates the estimated watershed areas for every lake within the study area. Drainage well locations and diameters are shown for every then-known drainage well within the study area. Because the OUSWMM delineates the watershed for every sub-basin and lake in Orlando, and because it contains information on land use patterns and maps the location of drainage wells, it was chosen as the vehicle through which to make calculations for Chapters VI and VII. Without the watershed areas being defined in some way it would have been impossible to make any meaningful calculations.

Description of Manual

The OUSWMM Phase One is composed of basically two parts

1) a 500-plus page set of maps and graphs on 11 x 17 inch paper called OUSWMM - Phase One - Inventory.

2) a 350-plus page handbook, called Volume I Facility.

The "Volume 1 - Facility" handbook is about 355 pages long, including tables and figures, which are both quite numerous and interesting. Chapters in the OUSWMM include: "Context of Stormwater Management," "Rainfall," "Runoff," "Metropolitan Stormwater Characteristics," and "Basin Analysis." It was this chapter on basin analysis that was most helpful to the work done in this thesis. The basin analysis chapter contains a listing of the general land use for each individual sub-basin within the study area, although it does not total them. The land uses have been compiled for presentation in this thesis. The land uses for each sub-basin are listed in Appendix A. Also helpful in the basin analysis section is the description of each individual sub-basin. This gives clues to whether any outfalls exist from each specific lake, or whether it is a totally landlocked lake.

The Phase 1 - Inventory maps are approximately 520-plus pages long, on 11x17 inch colored paper. There are three types of maps: Water Quality Maps for each lake; Drainage Watershed Basin Maps for each lake, and the Flood Prone Maps for the Orlando area. The only maps of interest for this thesis were the Drainage Basin Watershed maps. These maps detail the probable watersheds for each lake within the OUSWMM study area. They are primarily based on the stormwater drainage systems into the lakes, and also show the locations of all known drainage wells. Although these maps contain numerous errors in detail, such as slightly misplaced wells or misrouted sewer lines, they are surprisingly accurate for the broad scope of work they encompass.

How the OUSWMM Drainage Basins Were Determined

The study area for the OUSWMM encompasses a total area of 53,499 acres. All of the lakes fully or partially in the city of Orlando were covered. The city limits of Orlando at that time encompassed fewer acres than this so that the study area was about 20% larger than the existing city limits. The reason for using a larger study area than the city limits was to include those urban areas immediately adjacent to the city limits which might possibly be annexed into the city. The happy effect of this decision, although not one of the express purposes for it, is that it includes many of the urban lake watersheds that start out of the city and drain into it. In some instances this is not the case but generally it was true. A map of the limits of the OUSWMM study appears in Figure 10.

The Orlando study area is divided into two major watersheds. The northern portions of Orlando tend to drain into the St. John's River, which flows north, and the southern portions of Orlando tend to drain into the Kissimmee River, which flows south. Neither of these two rivers themselves pass through Orlando. Instead, five tributary waterways (Howell Branch Creek, the Little Econlockhatchee River, the Little Wekiva River, Shingle Creek, and Boggy Creek) form inside the Orlando area and flow out to the St. John's River and the Kissimmee River. The headwaters of all five of these tributaries start inside the Orlando study area, no major drainage systems or

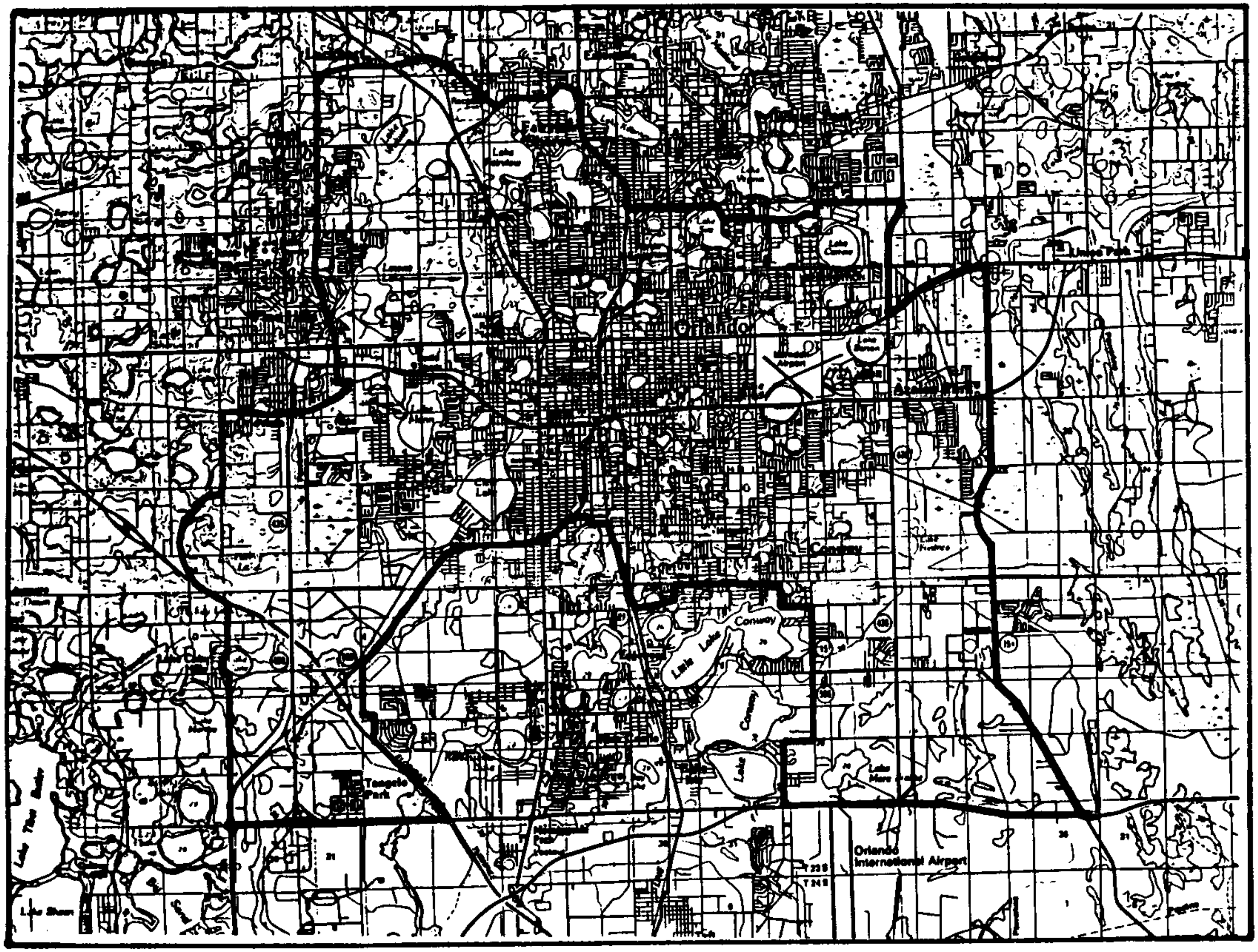

FIGURE 10 LIMITS OF OUSWMM STUDY AREA

waterways flow into the Orlando area from outside, and so from this we know that Orlando is not affected by flood problems originating outside of its boundaries. Any lake stage increase, creek or river stage changes or surface runoff which occurs within the Orlando area is due solely to rainfall and runoff occurring in Orlando.

The three tributaries of Howell Branch Creek, Little Econlockhatchee River, and Little Wekiva River flow into the St. John's River. The two other tributaries, Shingle Creek and Boggy Creek, flow into the Kissimmee River. For the purposes of the OUSWMM study, each tributary and the topographic area surrounding it was classed as a drainage basin, or sometimes simply a "basin." So we have the Howell Branch Basin, the Little Econlockhatchee River Basin, the Little Wekiva River Basin, etc.

Each drainage basin was broken up into a varying number of individual sub-basins, depending on the number of lakes within the basin. Each lake and the surrounding land which drained into it either by overland flow or through gravity storm sewers, formed an individual sub-basin. After the individual sub-basins for each lake were considered, the remaining land area within each basin which drained directly into the tributaries was classified as a sub-basin. These sub-basins were given the same name as their basins, i.e., the Little Wekiva River sub-basin was the land area within the Little Wekiva River basin which discharged directly into the Little Wekiva River without first passing through a lake. The Howell Branch Basin does not have a Howell Branch sub-basin since there apparently is not any land area within the City of Orlando which discharges directly into Howell Branch without first passing through a lake.

It should also be noted that not all lakes within a basin actually do drain into that basin's tributary; in fact, most Orlando lakes are landlocked sinkholes and do not drain anywhere except into the ground by seepage and drainage wells, but because the lakes are geographically located within a certain basin's area it becomes convenient to consider them a part of that basin.

A further description of the two watersheds and the five basins can be found in pages 223 to 224 of the OUSWMM; Volume 1 Facility. The geographic boundaries for each basin are given on those pages.

After the basins and sub-basins were determined, they were systematically analyzed and discussed in pages 225 through 348 of Volume 1 Facility. First, the major (or non-lake) sub-basin was analyzed, starting from downstream and working upstream, and then each lake within the basin was analyzed, starting with the most downstream sub-basin and working upstream. Sub-basin numbers were assigned for each sub-basin, they have the form "SJ-HB-20" or "KR-SC-19." For example "KR-SC-19" is the Lake Angel sub-basin, it is the 19th sub-basin analyzed within the Shingle Creek (SC) basin, which is part of the Kissimmee River (KR) Watershed. "SJ-HB-20" is Lake Sue, which is sub-basin #20 within the Howell Branch (HB) basin which is part of the St. John's River watershed.

There were a total of 117 sub-basins for the entire OUSWMM study area. By drainage basin the numbers are:

Drainage Basin	Number of Sub-Basins
Howell Branch	20
Little Econlockhatchee River	40
Little Wekiva River	20
Shingle Creek	19
Boggy Creek	18
	117

Land Use Within the OUSWMM Study Area

The land use within each sub-basin was determined and classified into one of nine categories. These categories and the total acres in the whole study area for each are:

Residential	19,995.5
Commercial/Industrial	12,901
Lake and Open Water	4,508.5
Swamp	581
Citrus	2,617
Grassland	5,680
Forest	6,355
Park/Recreation	92
Other	769
	53,499

The land uses for each individual sub-basin appear in Appendix A, Tables 28-32. Not all of the sub-basins had drainage wells or contributed to drainage wells; the sub-basins which are believed to contribute to drainage wells are indicated in the tables. The rules for determining if a sub-basin contributes to drainage wells are given later in Chapter VI of this thesis. Appendix A, Table 27 summarizes the total land uses by basin. A summary table similar to this appears in Table 19, page 220 of Volume 1 - Facility of the OUSWMM. However, the values given in the OUSWMM are incorrect.

Number of Drainage Wells within the OUSWMM Study Area

Drainage wells are indicated on the OUSWMM maps as red triangles, with the words 'XX" well' next to the triangle. XX" indicates the well diameter in inches.

No summary or table of the total number of drainage wells for the study area is given in the OUSWMM. The total number of wells was tabulated directly from the maps by the author and his capable assistants.

Drainage wells can fall into four broad categories:

1) Lake Wells - wells which are used to control lake stage levels. These wells do not receive direct discharge of storm runoff, the storm runoff is first detained by passing through the lake.

2) Storm Sewer Wells - a term created by the author. These wells do receive direct discharge of storm runoff, which is fed to the well through storm sewers, or possibly swales.

3) Alone Wells - another term created by the author. These wells also receive direct discharge of storm runoff, however they receive this only by overland flow. They do not have any connecting storm sewers or swales.

4) Combinations of the Above - it is possible for some wells to have been constructed in such a way that they function both as lake stage control wells and receive direct discharge of storm runoff from storm sewers. However no wells like this have yet been identified by the author.

Even though categories 2 and 3 are both direct discharge wells, a distinction is made between the two because it is felt that "Alone" wells will receive much less runoff than "Storm Sewer" wells since "Alone" wells are without a supporting storm drainage system.

Appendix B, Table 33 summarizes the total number of wells in the study area by type (lake, storm sewer or alone) and size within each basin.

Appendix B, Tables 34-38 summarize the total number of wells by type and size for each sub-basin.

How to Obtain the OUSWMM

The OUSWMM is available for sale by the City of Orlando, City Engineering Department. As of this writing (4/3/85) the cost for Volume 1 - Facility and Phase One Inventory books is $100.

According to James Hunt, a consulting engineer for Dyer, Riddle, Mills and Precourt, Inc., future additions to the OUSWMM are planned. These include Phase Two - Design Criteria, which has been released (its cost is $50). This phase contains little new information on land uses and/or drainage basins. Phase Three - Flood Control Options, scheduled for release in 1985 or 1986, will consist of general proposals on flood control options.

CHAPTER VI

QUANTITY ESTIMATE - RAINFALL RUNOFF METHOD

Purpose for This Calculation

The purpose of this chapter is to detail and describe the calculations necessary to set an upper bound and a probability distribution on the quantity of water entering Orlando area drainage wells by estimating the amount of rainfall runoff that is available to drainage wells. The total land use within the OUSWMM study area will be calculated, then those individual sub-basins which appear not to contribute significantly to drainage wells will be eliminated and the total land use for the remaining sub-basins will be calculated. Knowing the land use will permit estimation of runoff using the rational method.

Since both rainfall and runoff are stochastic in nature, historical records of precipitation from 1943-1983 will be used to determine the probability of a given annual rainfall, and thus the probability of a given total annual runoff for both all of the sub-basins and for only those sub-basins that contribute to drainage wells. This will be done for high and low values of the runoff coefficient "C."

Some studies have indicated that the runoff coefficent increases with the intensity of rainfall. Since average monthly rainfall varies through the year, some crude estimates of possible monthly variation in available runoff will be made by assuming low C factors for low rainfall months and higher C factors for high rainfall months. The stages of the 92 lakes in Orlando also vary during the year. Lower stages in the winter, when below the level of the drainage well inlets, allow for some extra storage of water before drainage wells start operating. An attempt has been made in estimating what this storage is.

Finally there is a discussion of various pertinent topics:

1. Does all of the sub-basin contribute to drainage wells, or only part of the sub-basin? What effects do evaporation, transpiration, infiltration, and outfall to other basins or natural channels such as the Little Econlackhatchee River have?

2. Does the fact that some wells are "lake" wells, some are "storm sewer " wells (supported by a storm sewer system) and some are "alone" wells (with no supporting systems) have a bearing on the quantity of runoff available to them?

Land Use in the OUSWMM

The total land use in the study area of the OUSWMM was determined and a table showing the land use by categories for each individual sub-basin is presented in Appendix A.

Not every sub-basin contributes to drainage wells. Many sub-basins are self-contained with no outfall to any other sub-basin and yet also are without drainage wells, so it is obvious that those sub-basins do not contribute runoff to the wells. Those sub-basins have been eliminated in the calculation of the available runoff to the drainage wells.

Some individual sub-basins are very large, and yet contain few "storm water" drainage wells and no "lake" drainage wells, and perhaps are even waterways carrying water out of Orlando, and so would seem not to contribute significantly to drainage well input.

Some particular examples of these are the four sub-basins of Little Econolockhatchee River, Little Wekiva River, Shingle Creek, and Boggy Creek. These four sub-basins, as defined in the OUSWMM, contain a total of 18,792 acres, but no lakes and only a total of 11 wells. Therefore, they were totally eliminated when calculating the total runoff available to drainage wells. It was decided it was more practical to ignore them than to try to calculate what small proportion of each of those sub-basins contributed to drainage wells. Probably the contributing areas of these sub-basins are less than 5% of the total area of those sub-basins. Adjusting for this would require more precision than the accuracy of estimating the "C" factors allows.

The guidelines for determining which sub-basins to include were as follows:

1. If the sub-basin did not contain any drainage wells and also had no outfalls given in the OUSWMM, that sub-basin was not included.

2. If the sub-basin did contain at least one "lake" well or "storm sewer" well, it was considered.

3. If the sub-basin did not contain any drainage wells, but did have an outfall to a sub-basin with "lake" wells, then runoff from that sub-basin was considered.

4. If the sub-basin did not contain any drainage wells, and its outfall was to a sub-basin which contains no "lake" wells, but only "storm sewer" wells, that sub-basin was not included, because the outfalls usually were from lake to lake and so could not reach "storm sewer" wells.

5. The sub-basins of Little Econolockhatchee River (SJ-LE-1); Little Wekiwa River (SJ-LW-1); Shingle Creek (KR-SC-1); and Boggy Creek (KR-BC-1) were eliminated, as previously mentioned.

A list of the sub-basins appears in the Appendix A, Tables 28-32, along with their total area, and the area of each of the nine types of land use. Also a note is made if that sub-basin does or does not contribute runoff to drainage wells.

Table 9 gives the various C's used for each given land use. The nine land uses described in the OUSWMM are

1) residential,
2) commercial/industrial combined,
3) open water (lakes),
4) swamps,
5) grassland,
6) forest and flatwoods,
7) citrus,
8) parkland and recreation areas,
9) other uses -- undeveloped lots, vacant land, etc.

Table 9 also indicates the total area within the study area that contributes to drainage wells, and the equivalent impervious area given the assumed runoff coefficients.

Stochastic Nature of Rainfall

Rainfall is stochastic, varying from moment to moment. However, as human beings, and particularly as water resources specialists, we tend to think of this stochasticism in structured ways, i.e., this is a very "dry" month or "wet" month, or a very "dry" year or "wet" year. In fact, it is more correct to say that rainfall occurred during this time period and not during this one, and leave it at that. Unfortunately this approach does not allow us to make plans, and as engineers that is what we want to do, make plans.

All of the annual rainfalls from 1943 to 1983 are shown in Table 7. These rainfalls are classified into nine groups and a probability of the occurrence of that range of rainfall is given. This is shown in Figure 11. The median rainfall of each range was used in calculating the runoffs.

Calculation of Runoffs

Total Runoff for the Entire OUSWMM Study Area

The total runoff for all of the OUSWMM study area is calculated in Table 8-B. This ranges from an annual daily average 99.3 MGD for High CAREA to 72.2 MGD for low CAREA. The total equivalent impervious area CAREA is calculated first in Table 8-A and then Average Daily Runoff is calculated for the various occurring rainfalls by the formula:

$$Q = (\text{CAREA}) * (\text{RAIN inches/year}) * 1\text{ ft}/12\text{ in.} * 43560\text{ ft}^2/\text{acre} * 7.481/10^6\text{ gallons/ft}^3 * 1\text{ yr}/365\text{ days}$$

$$= \text{CAREA} * \text{RAIN} * 7.44 \times 10^{-5}\text{ MGD} \qquad \text{(Equation 6-1)}$$

where:

CAREA = equivalent impervious area (acres),

RAIN = annual rainfall (inches)

TABLE 7

MONTHLY RAINFALLS FOR ORLANDO, FLORIDA: 1943-1983

Year	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	Annual
1943	1.19	0.50	3.92	1.53	5.42	3.66	5.17	5.85	7.18	3.04	0.87	1.28	39.61
1944	2.14	0.10	3.69	4.07	2.83	6.43	11.04	5.39	4.52	8.53	0.11	T	48.85
1945	3.86	3.86	0.54	1.47	2.93	13.70	7.06	5.28	15.87	1.61	1.00	2.52	55.95
1946	2.24	2.96	1.15	0.81	4.24	7.78	8.57	10.06	7.75	3.32	0.97	0.28	50.13
1947	0.87	4.78	5.55	4.98	2.81	11.61	13.90	6.71	8.87	4.83	1.90	0.66	67.47
1948	6.44	1.84	4.05	1.08	0.97	1.97	8.76	12.30	10.81	2.55	0.45	1.31	52.53
1949	0.31	0.47	0.29	3.02	2.54	7.97	6.05	8.83	8.25	1.51	1.22	3.82	44.28
1950	0.15	0.48	3.44	4.82	2.93	5.55	8.27	3.48	7.93	14.51	0.09	4.30	55.95
1951	0.52	2.28	0.96	5.99	1.40	5.08	14.51	7.84	9.34	3.08	4.86	2.06	57.92
1952	0.70	5.47	6.67	2.88	2.45	2.32	4.43	6.51	4.94	3.69	0.74	0.65	41.45
1953	2.86	2.89	3.03	6.18	1.87	6.28	6.85	15.19	8.84	3.50	4.78	3.58	65.85
1954	0.45	1.16	0.99	4.44	3.55	5.81	13.64	4.39	3.99	5.07	2.68	1.80	47.97
1955	2.00	1.12	1.59	1.36	3.13	4.73	6.88	6.65	6.97	4.10	2.17	1.56	42.26
1956	1.66	0.90	0.16	4.03	3.70	5.41	5.88	6.10	6.27	8.24	1.26	0.30	43.91
1957	0.91	1.93	3.76	4.74	8.58	4.39	4.35	9.45	7.47	1.68	0.82	2.85	50.93
1958	4.49	2.83	6.16	3.79	2.68	3.83	9.93	3.40	1.65	7.27	2.48	2.69	51.20
1959	2.78	4.55	7.69	4.91	4.44	7.95	8.02	6.77	8.33	5.97	0.99	1.37	63.77
#1960	1.49	5.64	10.54	2.55	0.50	9.50	19.57	3.20	11.21	3.17	0.30	1.07	68.74
1961	1.75	2.82	2.21	0.28	0.43	8.08	9.93	6.99	4.84	2.87	0.92	0.66	41.78
1962	1.11	2.08	3.55	1.58	2.74	3.11	12.77	5.11	12.24	1.90	2.46	1.70	50.35
1963	3.17	4.76	2.69	1.23	3.56	6.67	3.83	3.54	6.72	0.46	6.39	2.26	45.28
1964	6.18	3.42	4.65	2.14	2.74	6.11	6.68	9.00	9.47	1.64	0.45	1.91	54.39
1965	1.79	3.67	3.02	0.66	0.52	7.36	11.55	5.49	5.99	4.06	1.06	2.23	47.40
1966	4.45	6.31	2.57	1.92	6.57	9.77	6.73	7.76	6.25	1.98	0.09	0.99	55.39
1967	0.48	5.49	1.31	0.28	1.69	11.16	4.63	6.83	5.88	0.35	0.03	2.42	40.91
1968	0.65	2.76	2.27	0.30	3.72	18.28	5.60	3.44	5.91	5.47	2.82	0.88	52.10
1969	2.22	3.30	5.52	2.38	1.40	5.04	6.73	7.17	6.44	9.45	0.87	4.66	55.18
1970	4.05	6.77	3.66	0.45	4.08	4.92	5.97	5.91	3.25	2.60	0.24	2.06	43.96
1971	0.45	2.98	1.46	1.52	4.31	4.39	8.29	7.51	2.98	3.06	1.21	1.93	40.09
1972	0.99	4.96	5.06	1.39	3.76	6.33	3.98	16.11	0.43	2.34	4.11	1.89	51.35
1973	4.82	2.73	4.13	2.82	4.74	6.63	6.24	7.33	11.53	1.10	0.74	2.56	55.37
#1974	0.18	0.63	3.67	1.17	2.69	15.28	6.01	6.56	5.78	0.48	0.31	1.62	44.38
1975	0.98	1.49	1.10	1.36	7.52	9.70	9.26	4.75	4.97	4.74	0.66	0.51	47.04
1976	0.37	0.83	1.72	2.16	10.36	9.93	7.05	3.25	5.87	0.74	2.03	2.77	47.08
1977	1.81	1.76	1.82	0.14	1.47	4.47	6.61	6.28	7.03	0.43	2.60	3.70	38.12
1978	2.49	5.45	2.14	0.61	3.16	10.00	11.92	5.13	4.31	1.51	0.18	3.69	50.59
1979	6.48	1.45	3.24	1.08	7.66	4.00	7.95	5.88	9.19	0.43	1.93	0.94	50.23
1980	2.45	1.64	1.51	4.07	6.96	5.25	5.14	2.92	3.70	0.55	6.55	0.47	41.21
1981	0.21	4.36	1.85	0.18	2.02	12.49	3.5	5.60	8.26	3.13	2.50	2.97	47.10
1982	1.72	1.34	4.85	6.27	5.29	6.06	11.81	5.03	6.96	0.74	0.53	1.01	51.61
1983	2.08	8.32	5.37	3.21	1.77	7.82	6.49	4.83	5.16	3.78	1.36	5.33	55.52
Mean	2.11	2.92	3.25	2.44	3.57	7.24	8.09	6.58	6.91	3.40	1.65	1.98	50.14

#Indicates a station move or relocation of instruments.

SOURCE: United States Department of Commerce, National Climatic Center, 1982 and 1983 Annual Summaries of Local Climatological Data for Orlando, Florida

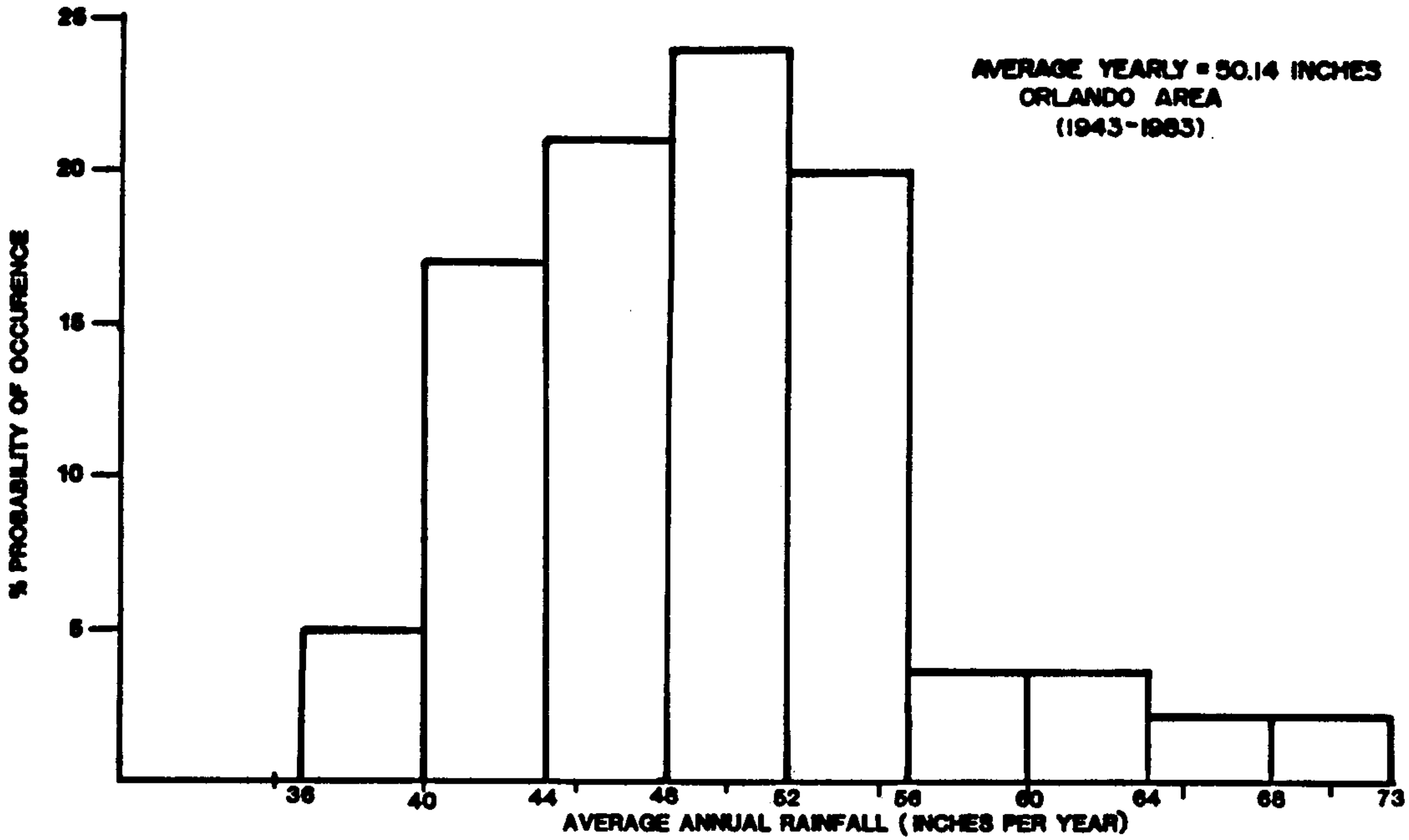

FIGURE 11 PROBABILITY OF OCCURRENCE OF A CERTAIN ANNUAL RAINFALL

TABLE 8

A) TOTAL LAND USE FOR ENTIRE OUSWMM STUDY AREA

TYPE OF LAND USE	TOTAL ACRES	HIGH ESTIMATE "C"	HIGH ESTIMATE "CAREA" (ACRES)	LOW ESTIMATE "C"	LOW ESTIMATE "CAREA" (ACRES)
Residential	19,995.5	0.5	9,997.8	0.3	5,998.7
Commercial/Industrial	12,901	0.8	10,320.8	0.6	7,740.6
Lakes	4,508.5	1.0	4,508.5	1.0	4,508.5
Swamp	581	1.0	581.0	1.0	581.0
Citrus	2617	0.1	261.7	0.05	130.9
Grassland	5,680	0.1	568.0	0.05	284.0
Forest	6,355	0.1	635.5	0.05	317.8
Park/Recreation	92	0.2	18.4	0.10	9.2
Other	769	0.1	76.9	0.05	38.5
	53,499		26,968.6		19,609.2

B) ESTIMATED TOTAL RUNOFF FOR ENTIRE OUSWMM STUDY AREA

RANGE OF RAINFALL	MEAN ANNUAL RAINFALL	PROB. OF OCCUR.	HIGH ESTIMATE AVERAGE DAILY RUNOFF (MGD)	HIGH ESTIMATE WEIGHTED AVERAGE (MGD)	LOW ESTIMATE AVERAGE DAILY RUNOFF (MGD)	LOW ESTIMATE WEIGHTED AVERAGE (MGD)
< 40	38.87	0.05	77.99	3.900	56.71	2.835
40.01-44	42.0	0.17	84.27	14.326	61.27	10.417
44.01-48	46.0	0.22	92.30	20.305	67.11	14.764
48.01-52	50.0	0.24	100.32	24.078	72.95	17.507
52.01-56	54.0	0.20	108.35	21.670	78.78	15.756
56.01-60	58.0	0.045	116.38	5.237	84.62	3.808
60.01-64	62.0	0.035	124.40	4.354	90.45	3.166
64.01-68	66.0	0.02	132.43	2.649	96.29	1.926
> 68.01	68.74	0.02	137.92	2.758	100.29	2.006
				99.277		72.185

Total Runoff for Contributing Sub-basins

Next, the probable Average Daily Runoffs (ADR) for those sub-basins contributing to drainage wells is calculated. First the equivalent impervious area of the contributing sub-basins is Table 7 calculated. The results are shown in Table 9. Then the average daily runoff during the year is calculated by the formula Q = CAREA * RAIN * 7.44 x 10^{-5} and the weighted average daily runoff for all the years is:

$$Q = \Sigma(P_i * Q_i) \text{ for all } i$$

where

P_i = probability of a certain flow Q_i occurring for all frequency values.

The weighted daily average runoff is 53.4 MGD for high CAREA and 39.1 MGD for low CAREA as is shown in Table 10. A plot of the resulting numbers is given in Figure 12, for low runoff potential (low CAREA), and high runoff potential (high CAREA).

Runoff Variation During the Year

Various studies have been conducted showing that the runoff coefficient C is not a constant, but that in fact it varies as the intensity and volume of the rainfall increases. Notable among these studies is "Rainfall-Runoff Mechanics for Developed Urban Basins, South Florida," by Bob Miller of the USGS. He showed that the runoff coefficient increases for volumes greater than 1.5 inches.

Therefore, to at least crudely reflect this variation in runoff for varying C factors, some calculations of resulting average daily

TABLE 9

CALCULATION OF EQUIVALENT IMPERVIOUS AREAS
FOR OUSWMM SUB-BASINS CONNECTED TO DRAINAGE WELLS

LAND USE	TOTAL WATERSHED AREA	AREA CONTRIBUTING TO DRAINWELLS (ACRES)	C FACTORS HIGH EST.	C FACTORS LOW EST.	EQUIVALENT IMPERVIOUS AREA (ACRES) HIGH EST.	EQUIVALENT IMPERVIOUS AREA (ACRES) LOW EST.
Residential	19995.5	12119	0.5	0.30	6059.5	3635.7
Commercial/ Industrial	12901	6783	0.8	0.60	5426.4	4069.8
Lakes	4508.5	2752	1.0	1.00	2752	2752
Swamp	581	84	1.0	1.00	84	84
Citrus	2617	395	0.1	0.05	39.5	19.75
Grass	5680	559	0.1	0.05	55.9	27.95
Forest	6355	691	0.1	0.05	69.1	34.55
Park/ Recreation	92	92	0.2	0.10	18.4	9.2
Other	769	10	0.1	0.05	1.0	0.5
Total	53499	23485	0.62	0.45	14505.8	10633.45

TABLE 10

PROBABLE ANNUAL RAINFALLS AND RESULTING AVERAGE DAILY RUNOFFS TO DRAINAGE WELLS IN THE OUSWMM STUDY AREA (1943-1983)

			HIGH ESTIMATE[1]		LOW ESTIMATE[2]	
LOWER LIMIT ANNUAL RAINFALL	MEAN ANNUAL RAINFALL	PROB. OF OCCUR.	AVERAGE DAILY RUNOFF (MGD)	WEIGHTED AVERAGE (MGD)	AVERAGE DAILY RUNOFF (MGD)	WEIGHTED AVERAGE (MGD)
≤ 40	38.87	0.05	41.95	2.098	30.75	1.537
40.01	42.0	0.17	45.33	7.706	33.23	5.648
44.01	46.0	0.22	49.65	10.922	36.39	8.006
48.01	50.0	0.24	53.96	12.951	39.55	9.493
52.01	54.0	0.20	58.28	11.656	42.72	8.544
56.01	58.0	0.045	62.60	2.817	45.88	2.065
60.01	62.0	0.035	66.91	2.342	49.05	1.717
64.01	66.0	0.02	71.23	1.425	52.21	1.044
≥ 68.01	68.74	0.02	74.19	1.484	54.38	1.088
TOTALS				53.401		39.142

[1] Contributing Impervious Area = CAREA = 14,506 Acres

[2] Contributing Impervious Area = CAREA = 10,633 Acres

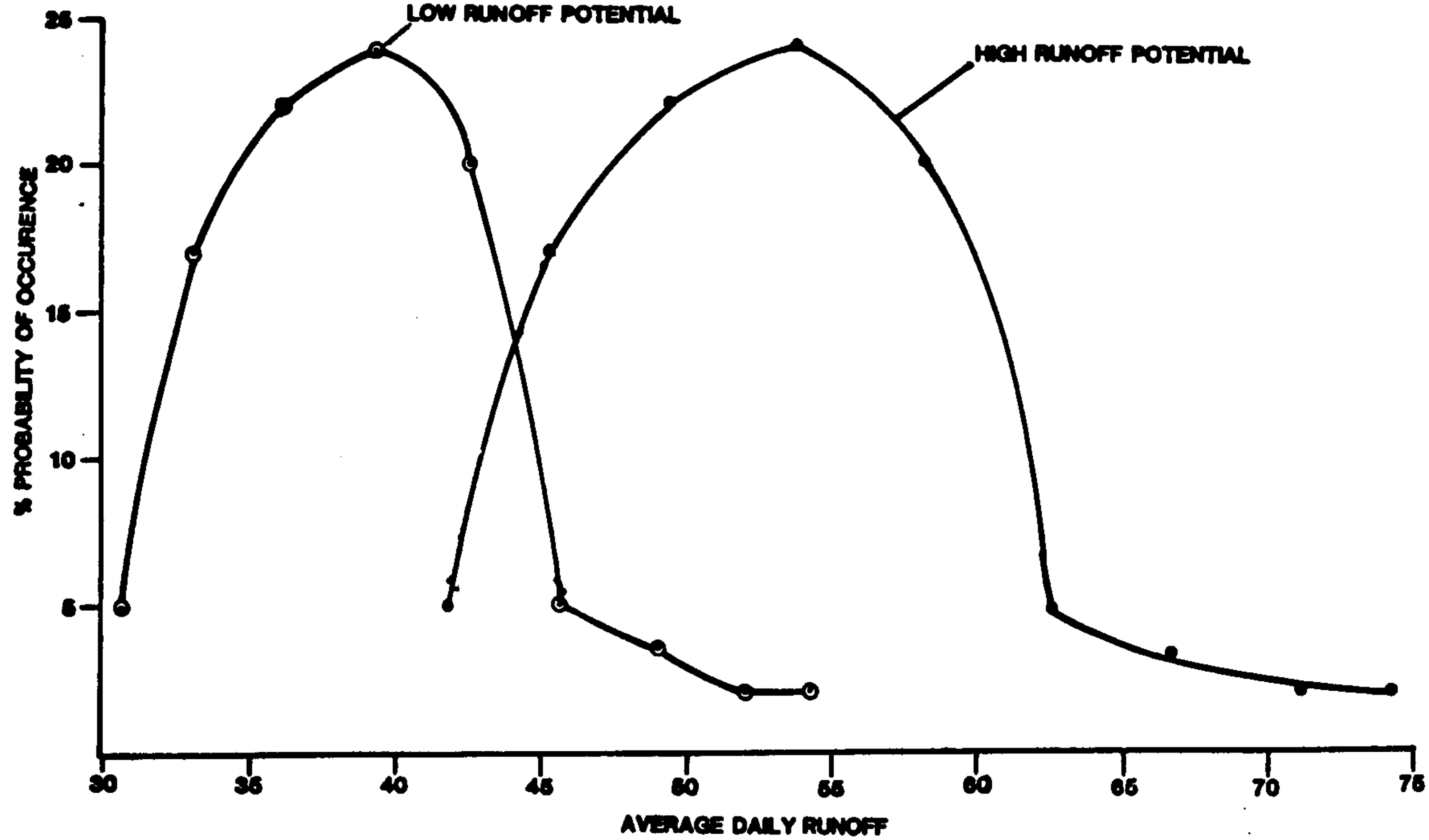

FIGURE 12 PROBABILITY OF AVERAGE DAILY RUNOFF FOR A GIVEN YEARLY RAINFALL

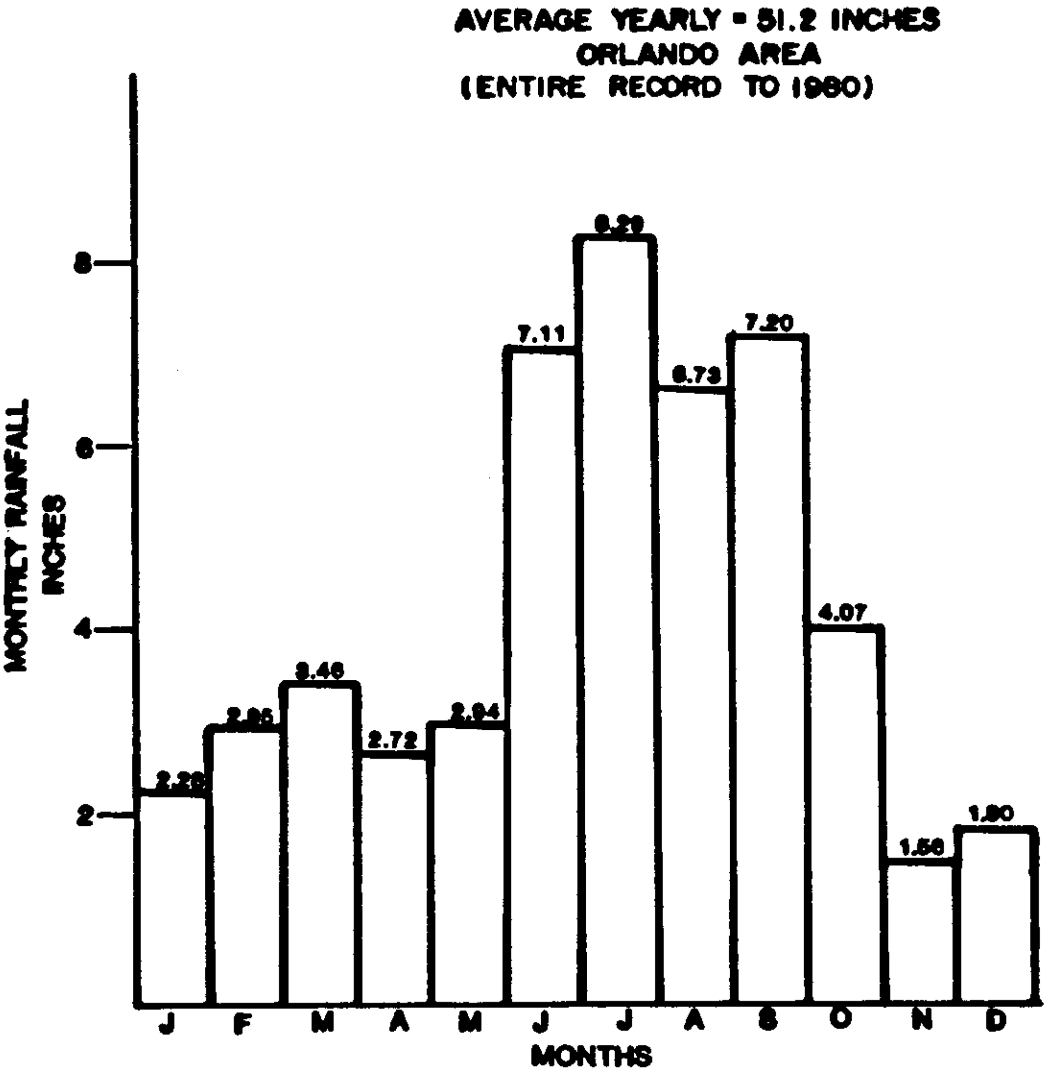

FIGURE 13 AVERAGE MONTHLY RAINFALL

SOURCE: QUSWMM

runoff (ADR) for the various months have been made by assuming different C's for different months. The C's chosen depended on the amount of rain occurring in that individual month.

A graph of the average monthly precipitation during a year is shown in Figure 13.

As can be seen from the graph, the "average" monthly rainfalls seem to fit into two definite groups:

1. the wet season of June through September, with all the monthly rainfalls being greater than 5"
2. the dry season for the rest of the year, October through May, with all of the monthly rainfalls being less than 5"

As an initial assumption, it was decided to assume that high runoff potentials existed in the wet season (i.e., "high" CAREAs were used) and low runoff potentials existed in the dry season ("low" CAREAs were used). The ADR to the wells was calculated to be 48.96 MGD, as shown in Table 11. The total (unweighted) average annual flow is 17871.79 MG.

The resulting monthly average rainfall and average daily drainage well discharge is shown in Figure 14.

If it is assumed that there are three classes of rain: low CAREA for rain $\leq$ 3", medium CAREA for 3" $\leq$ rain $\leq$ 5" and high CAREA for rain $\geq$ 5"; then it can be seen that the total annual runoff is increased slightly, from 17871.19 MG to 18267.71 MG and the resulting average daily runoff increases from 48.96 MGD to 50.05 MGD, increases of 8.8%. The pattern of runoffs through the year remains the same, as can be seen in Table 12. The only two months which would show a difference are March and October.

TABLE 11

AVERAGE DAILY RUNOFF FOR EACH MONTHLY RAINFALL
(BASED ON ASSUMPTION I)

MONTH	RAIN (INCHES)	CAREA (ACRES)	Q (MGD)	TOTAL MONTHLY FLOW (MILLION GALLONS)
January	2.28	10633.45	21.65	658.38
February	2.95	10633.45	28.01	851.85
March	3.46	10633.45	32.85	999.12
April	2.72	10633.45	25.82	785.43
May	2.94	10633.45	27.91	848.96
June	7.11	14505.8	92.08	2800.77
July	8.29	14505.8	107.36	3265.60
August	6.73	14505.8	87.16	2651.08
September	7.20	14505.8	93.25	2836.22
October	4.07	10633.45	38.64	1175.26
November	1.56	10633.45	14.81	450.47
December	1.90	10633.45	18.04	548.65
			TOTAL	17,871.79 MG/YEAR

ANNUAL AVERAGE Q = 17,871.79/365 = 48.96 MGD

Formula:
Q (in MGD) = CAREA*MONTHLY RAIN*(1'/12")(0.3259 MG/acre-ft)(12/365)

CAREA = 14505.8 acres if RAIN > 5"
CAREA = 10633.45 acres if RAIN <= 5"

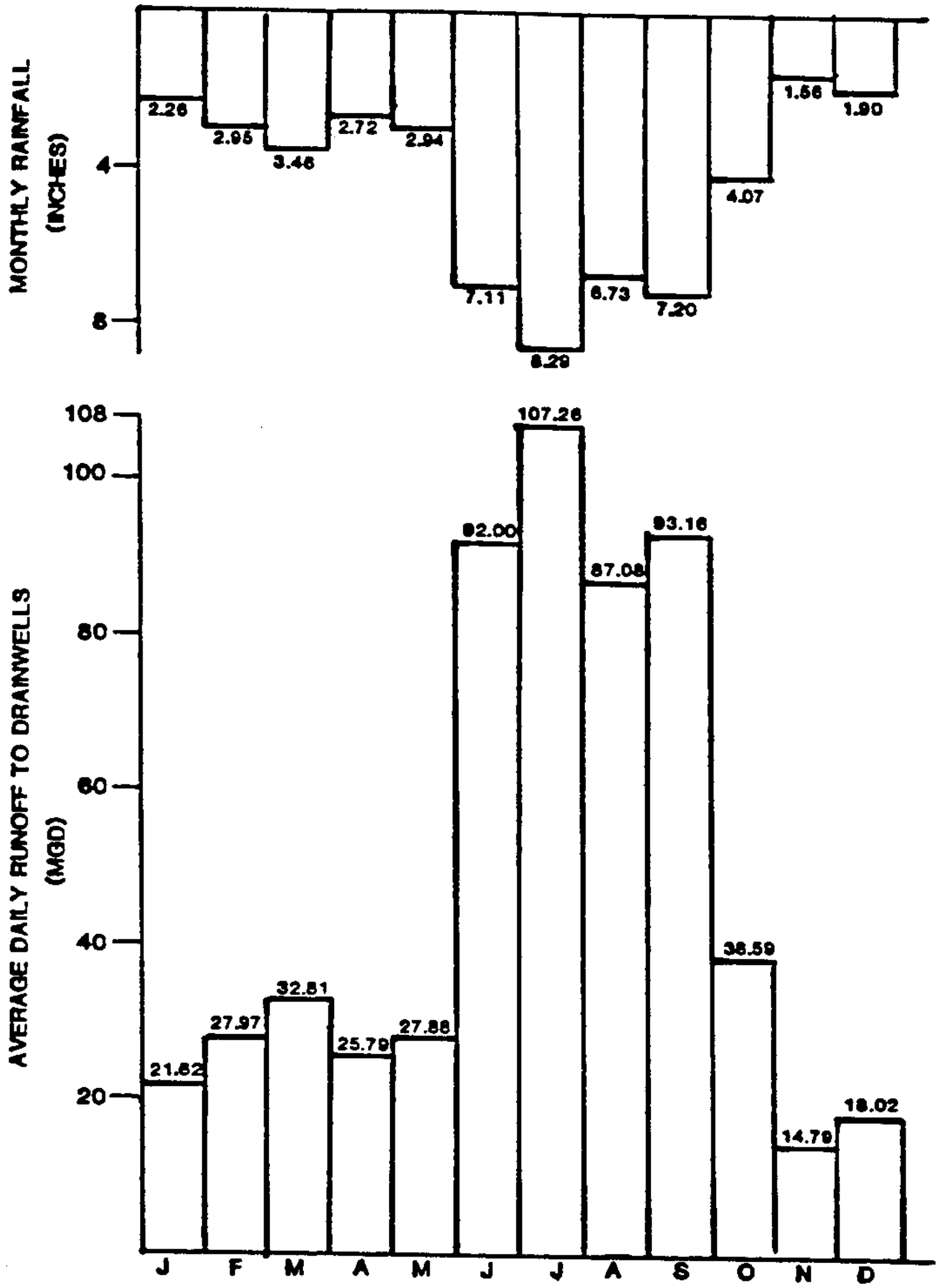

FIGURE 14 MONTHLY AVERAGE RAINFALL AND DRAINAGE WELL DISCHARGE

TABLE 12

AVERAGE DAILY RUNOFF FOR EACH MONTHLY RAINFALL
(BASED ON ASSUMPTION II)

MONTH	RAIN (INCHES)	CAREA (ACRES)	Q (MGD)	TOTAL MONTHLY FLOW (MILLION GALLONS)
January	2.28	10633.45	21.62	658.38
February	2.95	10633.45	27.97	851.85
March	3.46	12569.6	38.83	1181.04
April	2.72	10633.45	25.79	785.43
May	2.94	10633.45	27.88	848.96
June	7.11	14505.8	92.00	2800.77
July	8.29	14505.8	107.26	3265.60
August	6.73	14505.8	87.08	2651.08
September	7.20	14505.8	93.16	2836.22
October	4.07	12569.6	45.68	1389.26
November	1.56	10633.45	14.79	450.47
December	1.90	10633.45	18.02	548.65
			TOTAL	18,267.71 MG/YEAR

ANNUAL AVERAGE Q = 18,267.71/365 = 50.05 MGD

Formula:
Q (in MGD) = CAREA*MONTHLY RAIN*(1'/12")(0.3259 MG/acre-ft)(12/365)

CAREA = 14505.8 acres if RAIN > 5"
CAREA = 12569.6 acres if 3" < RAIN < = 5"
CAREA = 10633.45 acres if RAIN < 3"

It should be remembered that these are the "average" daily runoffs, available to drainage wells, for each month during the average year. The average year is the average rainfall for the period 1943 to 1983 and that does not necessarily reflect what would happen next year or in any given year. But they do give an indication of possible trends - available runoffs during the wet season (June through September) could be four or five times as high as available runoffs in the dry season. It is interesting to note that the resulting average daily runoffs calculated by this method (48.96 and 50.04 MGD) are within the range calculated in Table 10 for the weighted average of average daily runoffs (39.142 to 53.401 MGD).

Storage

Until now we have assumed that all of the runoff generated within a sub-basin with drainage wells was available to the drainage wells. Obviously, this is a conservative assumption; much of the runoff could be diverted or disposed of in some way other than drainage wells. One of the important causes (but not the only cause) of reduction of available runoff is the storage available in the lakes.

Lake levels are not constant. They obviously vary from lake to lake, and also they vary through the year. It is commonly thought that in the summer the lake levels are persistently above the well inlet levels, so that the wells operate in the summer, while in the non-summer months the lake levels are usually lower than well inlets, so that wells cease operating. The estimation of the variation in average daily runoff available through the year seems to support this. Figure 14 showed that 7.25 times as much runoff could be available in wet July as in dry November. However, the data which will be presented here indicates this may not have a significant effect, that in fact the relative degree of urbanization may possibly be a more important factor. But in either case, if storage is available in a lake (i.e., if the lake level is below the well inlet level), then the quantity of water entering the drainage well may be further reduced due to the extra losses of evapotranspiration and seepage, thus creating more storage volume, and more volume the runoff must fill up before the well operates.

To calculate the storage available in any given lake, three important facts must be known:

1) the lake's area

2) the lake level elevations

3) the drainage wells' elevations

The City of Orlando has a computerized listing of all lakes fully or partly within the city limits, and their areas and shore line lengths. Here, however, information on lake, or open water, areas was obtained from the OUSWMM. Since 1961, the City of Orlando has recorded monthly the lake levels of almost all of its lakes. This information is available in the Engineering Department at City Hall. Lake level recordings from January 1979 to December 1984 were used here to estimate storage.

Information on the actual drainage well inlet elevations is more difficult to come by. Well inlet elevations for only eight wells could be found even after consulting various sources such as the city's drainage well notebook, the city's lake level notebook (some lake control levels are given in it), Florida DOT records for Lake Angel, and University of Central Florida research work on Lake Eola. Some of the sources are very old, and it would be very beneficial if the elevations could be field verified; however this work is out of the scope of this thesis. It is highly recommended to the city that they perform this task for all of their lakes and drainage wells.

The eight lakes and their associated wells are given in Table 13. The table also gives the total number of lake level readings made between January 1, 1979 to December 31, 1984 for each lake, the number of readings that were above or below the well inlet, and the mean storage and the mean head.

Using a LOTUS 1-2-3 spreadsheet program, the differences between lake level and well inlet level were calculated for each reading. The mean storage was calculated by summing all of the absolute values of only those differences which were negative (lake level < well inlet level) and dividing that sum by the total number of readings that were made. In a similar fashion, the mean head was calculated by dividing the sum of positive differences by the total number of readings. In this way, the most expected head or storage can be obtained.

TABLE 13

MEAN STORAGE AND HEAD FOR SEVEN OUSWMM LAKES, 1979-1984

LAKE	SUBBASIN #	INLET LEVEL	TOTAL # OF READINGS	# READINGS > INLET	# READINGS < INLET	MEAN HEAD	MEAN STORAGE	LAKE AREA	BASIN AREA	HIGH CAREA	RATIO CAREA/LAKE AREA
1 Angel	KR-SC-19	96.16	73	72	1	0.18	0.0	10	356	221.1	22.11
2 Arnold	SJ-LE-12	96.5	73	0	73	0.0	3.10	26	265	144.6	5.56
3 Clear	KR-SC-10	94.25	73	22	51	0.10	0.95	360	1764	1147.6	3.19
4 Emerald	SJ-LE-22	70.0	73	0	73	0.0	1.75	2	27	14.5	7.25
5 Eola	SJ-HB-32	88.0	73	64	9	0.57	0.06	27	340	222.4	8.24
6 Giles	SJ-LE-11	103.4	73	12	61	0.04	0.97	27	267	143.6	5.32
7 Lurna	SJ-LE-30	86.81*	72	63	9	0.49	0.03	7	126	83	11.86
8 Underhill	SJ-LE-10	97.45	72	64	8	0.58	0.08	141	1259	959.2	6.80

* For well 2 (a 20" well)
well 1, elev. 88.42', had only one lake level reading to exceed it.

Table 13 shows that some wells operate quite extensively while others almost never operate. In fact, only Clear Lake has close to the 2:1 ratio (not operating:operating) that one would expect if season were the determining factor in when drainage wells operate. Otherwise, for the other lakes it seems that they are either always or never working.

The eight lakes are rank ordered below according to available storage and CAREA/Lake ratios.

Storage (Least Greatest)	High CAREA/Lake (Greatest Least)
Angel	Angel
Lurna	Lurna
Eola	Eola
Underhill	Emerald
Clear	Underhill
Giles	Arnold
Emerald	Giles
Arnold	Clear

There is a good rank order correlation between highest CAREA/Lake ratios and least available storage for the four lakes of Angel, Lurna, Eola, and Underhill. If Lake Emerald is not considered, which is justifiable because it is a special case geohydrologically, requiring frequent filling, then the rank ordering correlation is improved.

Figure 15, a plot of available storage versus High CAREA/Lake ratio, makes it clear why this is. All of the lakes Angel, Lurna, Eola, and Underhill have mean storages near zero. For the remaining three lakes Arnold, Clear, and Giles, the pattern is less clear. It is noted that Clear and Giles both have mean storages of about 1'.

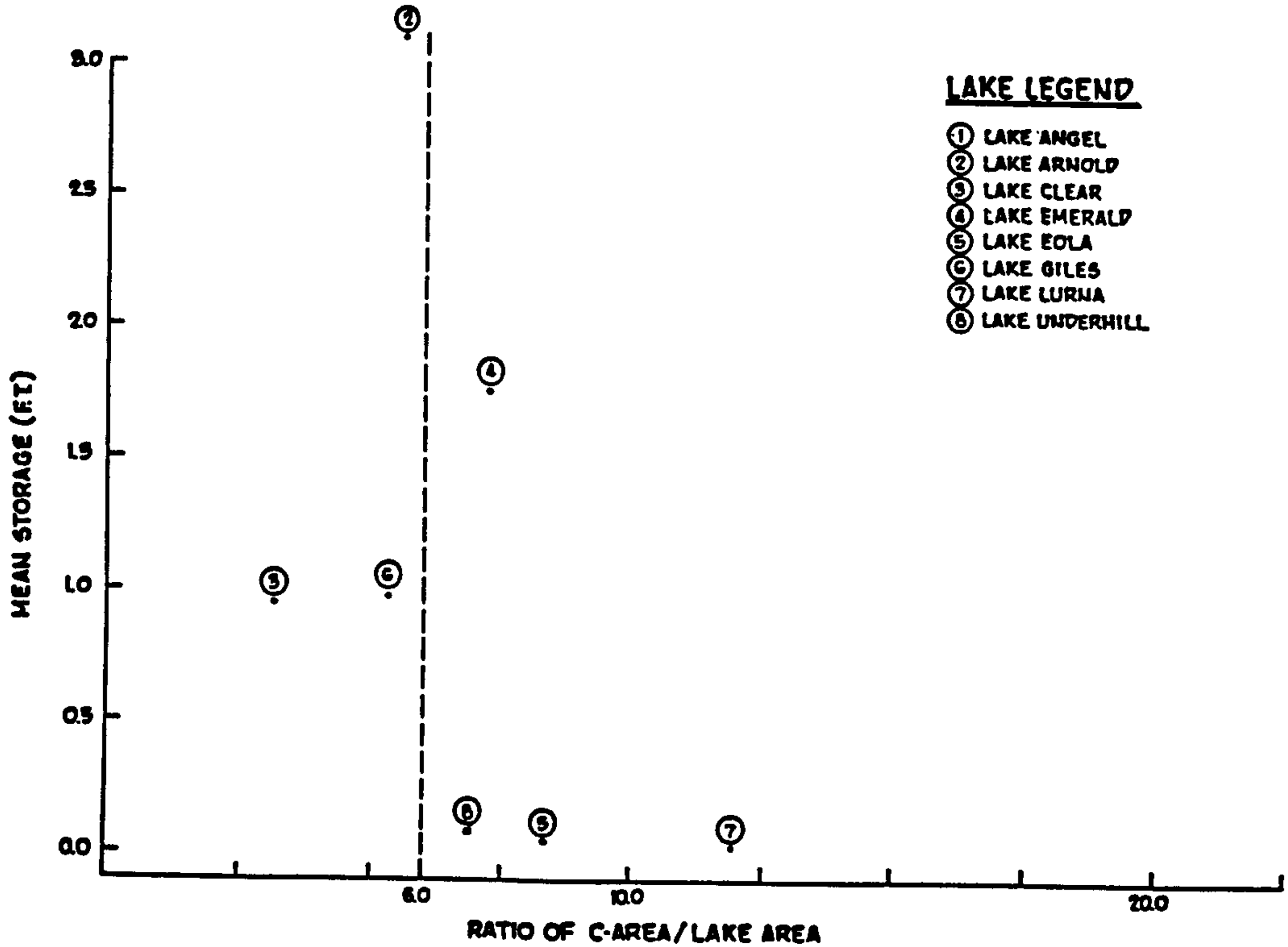

FIGURE 15 STORAGE VS C-AREA/LAKE RATIO FOR SEVEN ORLANDO LAKES

This is quite remarkable because otherwise they are quite different: Clear Lake is a very large lake, with a large drainage sub-basin and a great deal of impervious area, however its impervious area/lake area ratio is the lowest of them all. Lake Giles is a medium-sized (27 acre) lake in a moderate sized sub-basin. Its total impervious area is of course much less than Clear Lake's, but the CAREA/Lake ratio is higher for Lake Giles. Considering only the available data, it is intuitively best to describe the relationship between mean storage and High CAREA/Lake ratio as a step function, with a value of 1 foot for High CAREA/Lake < 6 and zero for High CAREA/Lake > 6.

In other words, in sub-basins where the CAREA/Lake ratio is less than six, we can expect storage to exist, and its value will average 1 foot of depth. In sub-basins where CAREA/Lake is greater than six we do not expect any storage. This rule of thumb is conservative for the available limited data.

Using LOTUS 1-2-3, the CAREA/Lake ratios for all of the sub-basins in the OUSWMM were determined. A total of 54 sub-basins were found to have High CAREA/Lake ratios < 6; but only 33 of these were sub-basins that contributed to drainage wells. The 33 sub-basins were broken down into 9, 10, 5, 5, and 4 each in the Howell Branch, Little Econlockhatchee River, Little Wekiwa River, Shingle Creek, and Boggy Creek basins respectively. The total lake area of these 33 sub-basins is 2,061 acres. Then, assuming an average storage depth of 1 foot for each of these sub-basins, there is 2,061 acre-ft = 89.7 million ft^3 = 671.6 MG storage for each interval of measurement (i.e., monthly). The available runoff to these exact same 33 sub-basins is calculated to be 27.37 MGD or 832.0 MG/month for High CAREA and mean rain. Subtracting the storage leaves 160.4 MG/month = 5.28 MGD. Assuming no storage at all for the remaining 84 sub-basins, the total available runoff to drainage wells would then be 31.3 MGD. Thus, storage is quite a significant factor in determining the available amount of water to drainage wells.

Other Considerations

The calculations in this chapter have been stringently conservative by assuming that all runoff in a sub-basin with drainage wells is routed to and disposed of solely by the drainage wells. This allows estimating an "upper bound" or "worst case" quantity of recharge. Actually, however, not all of the runoff in any sub-basin is routed solely to drainage wells. Many of the lakes are interconnected and water is routed offsite to the various creeks and rivers. Much of the runoff is retained in surface depressions or abstractions. Water is also retained on streets and parking lots. This abstracted water then either evaporates or infiltrates into the ground. Many of the lakes exfiltrate water horizontally through their banks into the shallow groundwater table. All of these factors combine to reduce the amount of water hydrologically available to the drainage wells. Analyzing accurately the system to determine the reduction is out of the scope of this thesis. It would require much more field work to determine such things as lengths and types of all interconnections between each sub-basin, weir sizes, relative elevations of all sub-basins, field data to estimate the abstraction for each sub-basin and the seepage for each lake, and various other parameters. However, as a rough guess, reducing the available runoff amount by half would seem reasonable. So then the estimated available runoff is 8.55 to 15.7 MGD, or 3132 to 5731 MG/year.

Another important consideration is the location of the well: i.e., is the well a "lake" well, a "storm sewer" well or an "alone" well. Most probably lake level control wells will receive the greatest amount of water per well, while "alone" wells, those wells without a supporting swale or storm sewer system, will receive the least quantity of runoff per well. "Storm sewer" wells would probably be in the middle. In the next chapter, "Well Hydraulics," some calculations are made to rate the relative importance of lake wells versus nonlake wells. The exact relationship of runoff available to lake wells versus nonlake wells is site specific, and not considered in this thesis.

Summary

The quantity of runoff available to drainage wells is dependent on the quantity of rainfall and the land use. The annual rainfall averages 52.1" and varies from 38.87 to 68.74 inches. The total equivalent impervious area "CAREA" could vary from 10,633 to 14,505 acres. Thus the annual runoff available to drainage wells could vary from 30.75 to 74.19 MGD and the weighted average annual runoff would be 39.1 or 53.4 MGD, depending on land use. The available runoff generally varies markedly through the year. The available runoff for the "average" summer month (2885 MG) is almost four times as much as the available runoff for the "average" winter month. However, reviewing some of the available lake level data indicates there may not be as marked effect on the relative quantity of water disposed of through the year.

Storage is probably available in 33 of the sub-basins. This storage volume is calculated as 22 MGD (669 MG/Month). This reduces the available runoff to 17.1 to 31.4 MGD (520 to 955 MG/month).

Finally, not all of the runoff in a drainage well containing sub-basin is necessarily disposed of by the drainage well. Some is lost by evaporation, transpiration, infiltration into ground, exfiltration in the lakes, or off-site routing into creeks, rivers, and swamps. Determining this quantity is out of the scope of this thesis. The author's estimate is that it could reduce the available runoff by half. Thus 8.55 to 15.7 MGD (260 to 478 MG/month) would be available to drainage wells within the Orlando Urban Stormwater Management Manual (OUSWMM) study area.

CHAPTER VII

QUANTITY ESTIMATE - WELL HYDRAULICS CONSIDERATIONS

Purpose for this Calculation

The purpose of this chapter is to detail and describe the calculations necessary to estimate the maximum theoretical capacity and the average annual capacity of the drainage wells. In this chapter this will be done by consideration of well hydraulics; in other words - what is the maximum probable inflow of water if weir flow governs, or if orifice flow governs? In a later chapter (Chapter VIII) consideration will be given to well capacity as determined by aquifer transmissivity.

If the flow into the wells is limited hydraulically such that they cannot accept the amount of runoff estimated in Chapter VI, then it is obvious that the estimate of water entering the well needs to be reduced. If the acceptance inability is only temporary, such as being not quite able to handle maximum stormflows, then perhaps the estimate would need to be only slightly reduced. The water would still drain down the well but would be delayed in extra storage in the lake, perhaps long enough to slightly increase evaporation of the stored runoff water. If the acceptance ability of the wells of a particular subbasin is significantly less than the estimated runoff for that subbasin, then perhaps a large quantity of the runoff overflows or is diverted into off-site drainage, away from the drainage wells. This would mean a significant modification to the estimated runoff.

Total Number of Wells

Perhaps the most obvious questions to ask when considering how much water could enter the Orlando area drainage wells are how many wells are there, what sizes are they, and where are they located? These questions have been answered in Chapter V of this thesis together with Appendix B. A synopsis of the pertinent points is presented here.

There are a total of 208 operating wells identified in the OUSWMM report. They range in size from 6" diameter to 24" diameter. Four wells are of an unknown or unlisted size. The wells are separated into three general categories:

1) "Lake" wells, which are wells located on or near lakes and control the lake stage

2) "Storm sewer" wells, which are wells connected directly to storm sewers or swales

3) "Alone" wells, which are wells neither located on a lake nor connected to storm sewers or swales. Their drainage area is only that immediately adjacent area that can drain by overland flow into the well. Alone wells may be located in a depression in someone's yard, such as the 6" alone well off of Westmoreland Drive in the Lake Angel sub-basin.

After reviewing all of the OUSWMM drainage maps a total of 84 lake wells, 108 basin wells, and 16 alone wells were counted. Table 14 summarizes the number of wells of each type and their sizes. Appendix B lists the type and size of each well for each individual

TABLE 14

TOTAL NUMBER OF DRAINAGE WELLS IN THE OUSWMM STUDY AREA BY BASIN AND BY SIZE

A. BY BASIN

BASIN NAME	LAKE[1]	STORM[2]	ALONE[3]	BASIN TOTAL
Howell Branch	15	39	9	63
L. Econ River	41	41	2	84
L. Wekiva River	9	12	2	23
Shingle Creek	13	15	3	31
Boggy Creek	6	1	0	7
TOTALS	84	108	16	208

B. BY SIZE

WELL DIAMETER (INCHES)	LAKE[1]	STORM[2]	ALONE[3]	SIZE TOTAL
6"	2	6	3	11
8"	7	8	3	18
10"	4	12	3	19
12"	35	57	5	97
14"	3	1	1	5
16"	1	1	0	2
18"	10	8	1	19
20"	19	13	0	32
24"	0	1	0	1
Not Given	3	1	0	4
TOTALS	84	108	16	208

[1] LAKE - lake level control wells

[2] STORM - stormwater drainage wells connected to stormsewers or swales, not located on a lake. Elsewhere referred to as "storm sewer" wells.

[3] ALONE - stormwater drainage wells which apparently, from OUSWMM maps, are alone, not connected to a supporting stormsewer or swales and also are not located on a lake.

sub-basin, and then summarizes this for each basin. This information will be useful later in this report.

Field Data of Flow on Wells

Table 15 contains data on observed heads of wells in various places. For some wells a notation of duration of the elevated flow is made, if the well is a lake well. If the well is a basin well, the duration is from that of first observed flow until flow was observed to have ceased.

All heads are measured at the rim of the well. It is realized this is not the most accurate place to measure the head, because of possible contraction of the flow over the rim. But for practical purposes, measuring the head at the well rim was the best place for quick measurement. Also, visual observation of the wells made it appear that contraction over the weir was not significant.

Theory of Flow Conversion from Weir to Orifice

A major problem faced when trying to estimate hydraulic capacity of drainage wells is that for any given well, when is the flow regime weir flow, and when is it orifice flow? It was decided that the best way to solve this problem was to consult the literature. Since very little experimental work has been done on drainage wells per se, it was necessary to search for similar, but perhaps larger-scale systems.

One somewhat similar system is the morning glory spillways used for dams. These are large circular drop inlet pipes, set upstream of

TABLE 15

FIELD OBSERVED HEADS ON VARIOUS DRAINAGE WELLS

A. STREET WELLS

WELL	DATE/TIME	OBSERVED HEAD (@ WELL RIM)	DURATION OF FLOW
Avondale/18th	2/11/85 17:14	0.25 inches	perhaps 2 hrs (street well)
Avondale/18th	6/12/85 20:10	1.0 inches	steady - 2 to 3 hours
Avondale/18th	6/15/85 11:30	1.0 inches	steady - 2 hours
	6/15/85 13:30	zero	flow stopped
Atlanta St.	6/15/85 12:00	0.5 inches	steady - 2 hours (street well)

B. LAKE WELLS

WELL	DATE/TIME	OBSERVED HEAD (@ WELL RIM)	DURATION OF FLOW
Lake Angel	9/28/84 13:30	1.6 inches	steady
	10/4/84 15:30	0.75 inches	steady
	10/9/84 10:25	0.88 inches	steady
	10/10/84 15:30	0.50 inches	steady
	10/19/84 18:00	0.80 inches	steady

TABLE 15 -- CONTINUED

WELL	DATE/TIME	OBSERVED HEAD (@ WELL RIM)	DURATION OF FLOW
	10/28/84 14:15	0.80 inches	steady
	11/21/84 11:10	0.75 inches	steady
	11/22/84 11:00	1.50 inches	steady
	2/6/85 18:00	3 inches	2+ hrs - perhaps 6 hrs (lake well)
	2/11/85 18:00	0.75 inches	N/A
Lake Lurna	1/28/85 2:00	-2 inches	not flowing
	6/15/85 12:47	10 inches	strong steady flow
Lake Eola	1/28/85 3:00	zero inches (even with weir)	not flowing
	6/12/85 21:26	2 inches	steady flow
Lake Giles	1/28/85 4:30	-2 foot (or more)	not flowing (storage)
	6/15/85 2:26	-2 foot (or more)	not flowing (storage)
Lake Underhill	6/15/85 2:48	10 inches	strong steady flow

the dam, which allow excess discharge to overflow into them and be carried under the dam for downstream disposal. Many researchers have done studies of the hydraulics of morning glory spillways, the most practical for use here was Design of Small Dams, pp. 414 to 418, by the U.S. Department of Interior.

Figure 281 on p. 414 of Design of Small Dams shows the general transition in flow from weir to orifice flow to pipe flow. Pipe flow occurs in morning glory spillways when both the vertical drop inlet pipe and the horizontal discharge pipe become completely full. Drainage wells are obviously different as they do not have horizontal discharges. Instead, they are vertical holes extending 100 to 1000 feet straight down into the ground. To acheive full pipe flow in them would probably require an immense head. Therefore, this possible flow regime was not considered. Also, drainage wells differ from morning glory spillways by not having bell mouth inlets.

In order to "bridge the gap" between weir flow and orifice flow, the authors of Design of Small Dams devised a modification of the weir equation $Q = CLH^{1.5}$ which could work for both flow types by using a variable C factor. The C factor varied depending mainly on the ratio of head to spillway radius (Ho/Rs), and to a lesser extent on the ratio of approach depth to radius (P/Rs).

Figure 16, which is a reproduction of their Figure 283, shows the results of their calculations. It should be noted that even large variations in the P/R ratio do not cause significant (for our

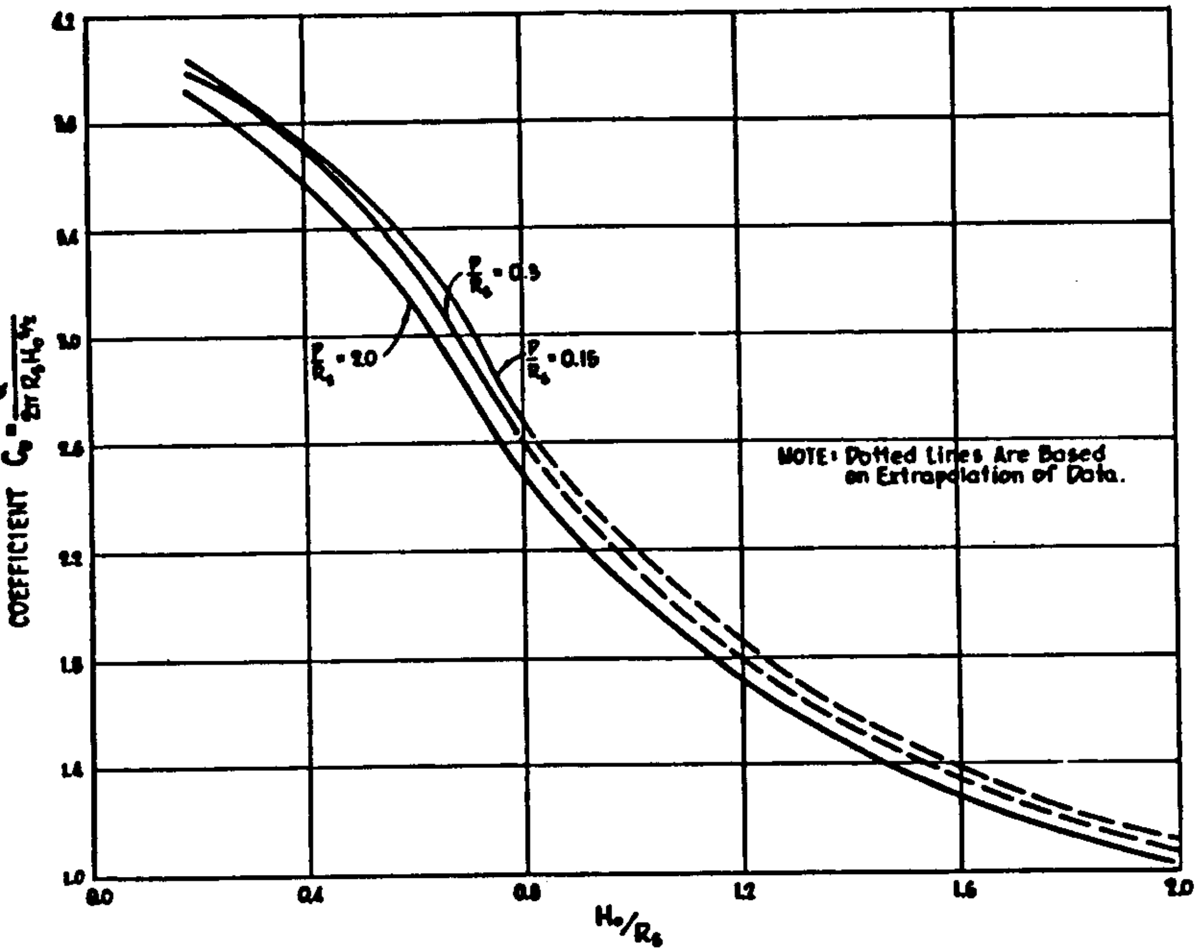

FIGURE 16 RELATIONSHIP OF CIRCULAR CREST COEFFICIENT C_0 TO H_0/R_s FOR DIFFERENT APPROACH DEPTHS (AERATED NAPPE)

SOURCE: DESIGN OF SMALL DAMS, P.417 U.S. DEPARTMENT OF INTERIOR purposes) variations in Co. The significant factor is the head variation.

Theoretical Acceptance Rates for Individual Wells

In order to facilitate computations, a chart of values of Co for various assumed heads on the existing wells sizes was calculated in Table 16-A. These Co values are used in the equation $Q = Co2\pi R_s H^{1.5}$ to calculate the theoretical maximum acceptance rate Q in gpm for each size well. These acceptance rates are given in Table 16-B. It is interesting to compare these rates with the actual field measured rates of flow for the Lake Angel 20" diameter well, and also with the findings of other researchers. The final equation derived for the Lake Angel well itself was:

$$Q = 2.02 \times LH^{1.295} = 2.02 \times 2\pi \times 20/12 \times H^{1.295}$$
$$= 21.15\ H^{1.295}$$

Table 17 is a comparison of this equation, as well as Walsh's (1980) equation for Lake Eola's 20" well, and the values tabulated by Schiner and German (1983) for a 20" well. As can be seen, there is good agreement between the actual field data and the Design of Small Dams formula for the lesser heads. As expected, the field values are somewhat less than the Small Dams formula values, due to:

1) absence of bell mouth inlet
2) roughness of field pipe
3) other reasons

TABLE 16-A

VALUES OF Co TO USE IN THE WEIR/ORIFICE EQUATION FOR VARIOUS Ho/Rs RATIOS

(For Po/Rs=0.15 conservative choice)

WELL DIAMETER IN INCHES (=2Rs)	HEAD Ho 0.05'	0.10'	0.18'	0.25'	0.50'	1.00'
6	4.0	3.7	2.95	2.2	1.1	≤ 1.1
8	4.1	3.85	3.45	2.8	1.48	≤ 1.1
10	4.1	3.95	3.7	3.3	1.85	≤ 1.1
12	4.2	4.0	3.75	3.55	2.2	1.1
14	4.2	4.1	3.8	3.7	2.5	1.3
16	4.2	4.1	3.9	3.75	2.8	1.4
18	4.2	4.1	4.0	3.83	3.1	1.65
20	4.2	4.1	4.1	3.85	3.3	1.85
24	4.2	4.1	4.1	3.95	3.55	2.2

TABLE 16-B

VALUES OF THEORETICAL ACCEPANCE RATES FOR DRAINAGE WELLS (BASED ON DROP-INLET HYDRAULICS)

$$Q=(Co2\ \pi RsHo^{1.5}\ \text{cfs})(448.86\ \text{gpm/cfs})$$

WELL DIAMETER IN INCHES (=2Rs)	HEAD Ho 0.05'	0.10'	0.18'	0.25'	0.50'	1.00'
6	31.5	82.5	159	194	274	776
8	42.7	113.3	245	326	487	1024
10	54.3	148.0	335	489	775	1303
12	66.2	178.4	404	626	1097	1551
14	76.8	212.1	475	757	1446	2126
16	88.7	245.0	554	886	1843	2606
18	99.3	274.2	646	1013	2318	3490
20	109.9	303.5	733	1127	2731	4330
24	132.4	365.7	883	1393	3540	6205

TABLE 17

COMPARISON OF ACCEPTANCE RATE CALCULATIONS
FOR 20" DIAMETER DRAINAGE WELLS

	ACCEPTANCE RATE (GPM)				
	HEAD (FT)				
SOURCE	0.05'	0.1'	0.25'	0.50'	1.00'
Lake Angel Field Data (February 1985)	98.1	240.7	788.5	1935	*
Design of Small Dams Formula	109.9	303.5	1127	2731	4330
Schiner and German (1983) p. 16	*	1500	2300	3300	4700
Lake Eola Field Data (head on weir) (Walsh, 1981 p. 48-49)	*	128.6	321.5	643.0	*

* No value measured or calculated for this head value.

The agreement between field data and Schiner and Germans values is not as good, possibly their orifice coefficient was a little high.

Walsh's values are lower. His equation was derived based on observed declines in lake stage, perhaps other factors such as seepage or evaporation had a greater effect than expected. Also, the weir at Lake Eola is much narrower than at Lake Angel. Lake Angel has a 47.5 inch weir, Lake Eola's is 33 inches.

In any event, the Design of Small Dams formula gives the best results and will be used in the rest of this analysis.

Estimation of Maximum Hydraulic Capacity for all OUSWMM Wells

Knowing what the theoretical expected acceptance rates are for various heads permits some speculative calculations. In Chapter VI we calculated the probable annual runoff available for discharge to drainage wells. This figure ranged from 14,300 to 19,500 million gallons, or an a daily average basis 39.1 MGD to 53.4 MGD, respectively. The question is, are the 208 drainage wells hydraulically capable of handling that immense amount of water?

Tables 18 through 21 can help answer that question. After knowing how many drainage wells there are and their sizes (from Appendix B) and what some typical head values to expect are (and thus the theoretical expected acceptance rates), all that needs to be known to calculate the total annual acceptance of water is what proportion of the year do the wells operate: all of the year, some of the year, none of the year, or all three depending on the well?

The answer to this question is purely a guess; no actual surveys have been made to establish these numbers. Some references in the literature state that drainage wells only operate in the summer months. These were probably specifically referring to lake wells. The author's field observations confirm that most lake wells do not appear to operate during the winter months, however the lake stages at Lake Angel, Lake Eola, and some of the other larger lakes are high enough so that their lake wells have been continuously accepting water. In fact, data presented in Chapter VI showed that the relative ratio of impervious area to lake area may be more important than season in determining whether the wells operate at all. However this data is limited. So for now it is reasonable to assume the 84 lake wells on the average operate between four months to year-round.

Table 18 calculates the annual acceptance of water for all lake wells of each size, for various assumed heads. The total annual acceptance of water for all the wells is given along with what the daily average flow would be. Table 19 is the same except that all the lake wells are assumed to operate only for the summer (1/3 of the year). The MG/summer figures on this table are equivalent to MG/year quantities since now the annual draining time is assumed to be only summer. The MGD annual figures are what the average daily flow would be throughout the year, which of course is 1/3 of the average daily summer flow. Table 21 calculates the annual acceptance of water for all non-lake wells of each size, for various assumed heads. The totals are Mg/year and the daily average MGD.

TABLE 18

ANNUAL TOTAL THEORETICAL ACCEPTANCE OF WATER BY LAKE CONTROL DRAINAGE WELLS IN THE OUSWMM STUDY AREA (YEAR-ROUND OPERATION)

NOTE: TOTAL ANNUAL DRAINING TIME = 1 YEAR = 5.256 * 10^5 MINUTES

WELL DIAMETER (INCHES)	TOTAL # LAKE WELLS	HEAD 0.05'		HEAD 0.1'		HEAD 0.18'		HEAD 0.25'		HEAD 0.5'		HEAD 1.0'	
		1*	2**	1	2	1	2	1	2	1	2	1	2
6	2	31.5	33	82.5	87	159	167	194	204	274	288	776	816
8	7	42.7	157	113	416	245	901	326	1199	487	1792	1024	3768
10	4	54.3	114	148	311	335	704	489	1028	775	1629	1303	2739
12	35	66.2	1218	178	3274	404	7432	626	11516	1097	20180	1551	28532
14	3	76.8	121	212	334	475	749	757	1194	1446	2280	2126	3352
16	1	88.7	47	245	129	554	291	886	466	1843	969	2606	1370
18	10	99.3	522	274	1440	646	3395	1013	5324	2318	12183	3490	18343
20	19	110	1099	304	3036	733	7320	1127	11255	2731	27273	4330	43241
24	none												
unknown***	3	66.2	104	178	281	404	637	626	987	1097	1730	1551	2446
TOTAL	84	(MG/yr)	3415		9308		21596		33173		68324		104607
		(MGD)	9.36		25.50		59.17		90.88		187.2		286.6

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*rate (gpm/well)

**total (MG/yr)

***assumed 12-inch diameter

(calculated by Equation 28, p.415, Design Of Small Dams, Dept. of Interior)
(Q per well=Co2 πRsHo$^{1.5}$, where Co is given in Figure 283, p.417, IBID, and varies as required to consider weir => orifice transition)

TABLE 19

ANNUAL TOTAL THEORETICAL ACCEPTANCE OF WATER BY LAKE CONTROL DRAINAGE WELLS
IN THE OUSWMM STUDY AREA (SUMMER ONLY OPERATION)

NOTE: TOTAL ANNUAL DRAINING TIME = 1 SUMMER = 1.752 * 10^5 MINUTES

WELL DIAMETER (INCHES)	TOTAL # LAKE WELLS	HEAD 0.05'		HEAD 0.1'		HEAD 0.18'		HEAD 0.25'		HEAD 0.5'		HEAD 1.0'	
		1*	2**	1	2	1	2	1	2	1	2	1	2
6	2	31.5	11	82.5	29	159	56	194	68	274	96	776	272
8	7	42.7	52	113	139	245	300	326	400	487	597	1024	1256
10	4	54.3	38	148	104	335	235	489	343	775	543	1303	913
12	35	66.2	406	178	1091	404	2477	626	3839	1097	6727	1551	9511
14	3	76.8	40	212	111	475	250	757	398	1446	760	2126	1117
16	1	88.7	16	245	43	554	97	886	155	1843	323	2606	457
18	10	99.3	174	274	480	646	1132	1013	1775	2318	4061	3490	6114
20	19	110	366	304	1012	733	2440	1127	3752	2731	9091	4330	14414
24	none												
unknown***	3	66.2	35	178	94	404	212	626	329	1097	577	1551	815
TOTAL	84	(MG/summer)	1138		3103		7198.7		11059		22775		34869
		(MGD annual)	3.12		8.50		19.72		30.30		62.40		95.53
		(MGD summer)	9.35		25.50		59.17		90.90		187.2		286.6

----

*rate (gpm/well)

**total (MG/yr or MG/summer)

***assumed 12 inch diameter

(calculated by Equation 5.28, p.415, Design Of Small Dams, Dept. of Interior)
(Q per well=Co2 πRsHo$^{1.5}$, where Co is given in Figure 283, p.417, IBID, and varies as required to consider weir => orifice transition)

TABLE 20

MEAN HEADS FOR SEVEN OUSWMM LAKES, 1979-1984

LAKE	SUBBASIN #	INLET LEVEL	TOTAL # OF READINGS	# READINGS > INLET	# READINGS < INLET	MEAN HEAD (FT)	MEAN STORAGE (FT)	LAKE AREA (ACRES)	BASIN AREA (ACRES)	WELL DIAMETER (INCHES)	WEIR/INLET STRUCTURE (IF KNOWN)
1 Angel	KR-SC-19	96.16	73	72	1	0.18	0.0	10	356	20	Submerged 36" pipe to a 47.5 broadcrest weir
2 Arnold	SJ-LE-12	96.5	73	0	73	0.0	3.10	26	265	20	Spillway
3 Clear	KR-SC-10	94.25	73	22	51	0.10	0.95	360	1,764	20	----
4 Eola	SJ-HB-32	88.0	73	64	9	0.57	0.06	27	340	20	Beehive weir/33"
5 Giles	SJ-LE-11	103.4	73	12	61	0.04	0.97	27	267	20	Beehive weir/33"
6 Lurna	SJ-LE-30	86.81*	72	63	9	0.49	0.03	7	126	20	weir/21"
7 Underhill	SJ-LE-10	97.45	72	64	8	0.58	0.08	141	1,259	20	Beehive weir/33"
					Average:	0.28	0.74				

* For well #1
(20" diameter well)

TABLE 21

ANNUAL TOTAL THEORETICAL ACCEPTANCE OF WATER BY NON-LAKE DRAINAGE WELLS IN THE OUSWMM STUDY AREA

NOTE: TOTAL WELL OPERATING TIME (ASSUMED) = 3 * annual storm duration time
= $5.8173 * 10^4$ minutes/year

WELL DIAMETER (INCHES)	TOTAL # NON-LAKE WELLS	HEAD 0.05'		HEAD 0.1'		HEAD 0.25'		HEAD 0.5'		HEAD 1.0'		HEAD 2.0'	
		1*	2**	1	2	1	2	1	2	1	2	1	2
6	9	31.5	16	82.5	43	194	102	274	143	776	406	2194	1149
8	11	42.7	27	113	72	326	209	487	312	1024	655	2896	1853
10	15	54.3	47	148	129	489	427	775	676	1303	1137	3685	3216
12	62	66.2	239	178	642	626	2258	1097	3957	1551	5594	4387	15823
14	2	76.8	9	212	25	757	88	1446	168	2126	247	5089	592
16	1	88.7	5	245	14	886	52	1843	107	2606	152	5879	342
18	9	99.3	52	274	143	1013	185	2318	424	3490	638	6580	3445
20	13	110	83	304	230	1127	852	2731	2065	4330	3275	7283	5508
24	1	132	8	366	21	1393	81	3540	206	6205	361	8775	510
unknown***	1	66.2	4	178	10	626	36	1097	64	1551	90	4387	255
TOTAL	124	(MG/yr)	490		1329		4290		8122		12555		32693
		(MGD)	1.34		3.64		11.75		22.25		34.40		89.57

*rate (gpm/well)

**total (MG/yr)

***assumed 12 inch diameter

(calculated by Equation 28, p.415, Design of Small Dams, Dept. of Interior)
(Q per well=$Co2\pi RsHo^{1.5}$, where Co is given in Figure 283, p.417, IBID, and varies as required to consider weir => orifice transition)

Estimating the duration of flow for lake wells is easy compared to estimating the duration of flow for the non-lake wells (storm sewer and alone wells).

Since these wells all receive direct discharge of storm runoff, it is reasonable to expect that their period of operation is a function of the total annual storm duration. In other words, storm wells operate a certain multiple of the time that it rains. But what this multiple is, is unknown. Here it has been assumed to be three times the annual storm duration, which is reasonable and conservative.

The annual storm duration was calculated based on data presented in David Anderson's thesis. Anderson presented a probability distribution of storm duration for a total of 617 storms over the five-year period of 1975 to 1979 (Anderson, 1982, p. 141). The most probable duration of any one storm was found to be 2.325 hours, or 139.5 minutes. Since there was an average of 139 storms per year, the annual storm duration is 139 storms x 139.5 min = 19391 minutes. If the annual draining time is assumed to be thrice the storm duration, then that is 5.8173×10^4 minutes per year.

Assuming the above operating times, the total maximum hydraulic capacity of all the wells operating simultaneously under the same head can be estimated. The maximum flow event of March 1985 at Lake Angel (see Chapter IV) suggests a maximum possible well rim head of about 0.5 feet. The total hydraulic capacity of all lake wells at this head is from 62.42 MGD (summer only flow) to 187.3 MGD (year-round flow). The total hydraulic capacity for non-lake wells at 0.5 feet head is 16.29 MGD. Thus the maximum total hydraulic capacity of all the OUSWMM wells would range from 78.71 to 203.59 MGD. This is more than sufficient to handle all of the average available runoff of 39.1 to 54.3 MGD calculated in Chapter VI.

Estimation of Actual Inflow for all OUSWMM Wells

In addition to estimating the maximum hydraulic capacity of the drainage wells, the tables can also be used to estimate the actual quantity of flow into the drainage wells, based on well hydraulics.

It is realized that it is very unrealistic to expect that all lake wells will have the same amount of head applied to them. So the tables may be misleading, but they do provide a framework within which to explore the reasonableness of the well's ability to handle the expected stormwater runoffs. An important question is how much head can be expected, on the average, on lake wells? Table 20 presents the average head calculated to be on the weir or level-control structures for seven lakes in the OUSWMM area. The arithmetic mean head is 0.28'. This is the overall mean annual head. Reviewing the same data shows that the heads during the summer months also average 0.28 feet. If we assume that the "typical" inlet structure is a 33 inch weir, with weir coefficients C = 2.75, n = 1.48 as described in Chapter IV, and the typical drainage well on these lakes is a 20" well with C = 2.02 and n = 1.295, then the equivalent well rim head is 0.18 feet.

From Tables 18 and 19, the average daily flow for the lake wells ranges from 19.72 to 59.17 MGD at this head. However, this is assuming morning glory spillway hydraulics hold. For actual drainage wells the rate is lower. From Table 17, the correction factor for 0.18 feet head can be interpolated as 0.745. Applying this to the lake well data yields a range from 14.7 to 44.1 MGD. This is the estimate of actual quantity of water entering lake drainage wells based on well hydraulics. This is also called the total hydraulic inflow for lake wells.

Table 15 presented a limited number of observed heads on storm sewer drainage wells. These values range from 0 to 0.25 feet, a reasonable average assumption is 0.10 foot. Thus, from Table 21 it is calculated that the direct discharge wells could receive only 2.43 MGD on the average. But the total hydraulic inflow for the lake drainage wells was calculated as 14.7 to 44.1 MGD. The total hydraulic inflow for all drainage wells would then be 17.1 to 46.5 MGD. This agrees fairly well with the estimate of available runoff ranging from 17 to 31 MGD calculated in Chapter VI.

Summary

The total number of working drainage wells shown in the OUSWMM area is 208. The normal flow regime into most drainage wells is probably weir flow because of the relatively small heads normally encountered. The Design of Small Dams formula agrees best with actual field data of well acceptance rates. Although all of these acceptance rates are rather low, even for high heads, the wells are quite capable of handling hydraulically the average quantity of rainfall runoff made available to them. They are able to do this because of their long durations of flow. The maximum hydraulic capacity of all 208 wells operating simultaneously ranges from about 78.7 to 203.6 MGD. This is for the maximum (probably) achievable head of 0.5 feet.

The calculated range of actual hydraulic inflow of water into the drainage wells is 17.1 to 46.5 MGD. This is based on a very limited set of data on heads on drainage wells. Still, it agrees fairly well with the estimate of available runoff, and increases confidence that the total quantity entering Orlando wells is less than 50 MGD.

CHAPTER VIII

QUANTITY ESTIMATE - WELL TRANSMISSIVITY LIMIT

Purpose for this Calculation

The purpose of this chapter is to detail and describe the calculations necessary to estimate the quantity of water entering Orlando area drainage wells by estimating the receiving zone's ability to accept the water available to it. This acceptance ability of the receiving zone is known as its transmissivity, or how easily aquifer transmits away from the well the water discharged into it.

This transmissivity is reflected in pumped wells by the drawdown or water-level decline that occurs in the well as pumping occurs. Very transmissive aquifers will have little or no drawdown, non-transmissive aquifers will have a large drawdown.

An attempt is made to extend to drainage wells the concept of relating water level change to aquifer transmissivity. Aquifer transmissivity is the final determining factor in estimating the ability of a given drainage well to accept water. If the aquifer cannot receive the flow of water discharged into it, then the well will back up; the water level in the well will rise and eventually match the water level of the source and inflow to the well will stop. If the aquifer can receive the water discharged to it, then it is not the limiting factor on the acceptance rate for that well. Data from actual pumping tests of drainage wells will be used to calculate the aquifer's theoretical ability to accept the water discharged into it by drainage wells. This value will be compared to the available runoff calculated using surface water hydrology (Chapter IV) and to the well hydraulic limit (Chapter VII) to draw a final conclusion on the total quantity of water entering the drainage wells in the Orlando Urban Stormwater Management study area.

Calculation of Upper Floridan Aquifer Transmissivity

Definitions of Transmissivity and Hydraulic Conductivity

In determining the aquifer's ability to receive the water discharged to it, the parameters of paramount importance are transmissivity, and a related parameter, hydraulic conductivity. These parameters are defined by David Todd (Todd, 1980, p. 69) as follows:

> Hydraulic Conductivity. For practical work in groundwater hydrology, where water is the prevailing fluid, hydraulic conductivity K is employed. A medium has a unit hydraulic conductivity if it will transmit in unit time a unit volume of groundwater at the prevailing kinematic viscosity through a cross section of unit area, measured at right angles to the direction of flow, under a unit hydraulic gradient. The units are
>
> $$K = \frac{v}{dh/dL} = \frac{-\ m/day}{m/m} = m/day$$
>
> indicating that hydraulic conductivity has units of velocity.

<u>Transmissivity</u>. The term transmissivity T is widely employed in groundwater hydraulics. It may be defined as the rate at which water of prevailing kinematic viscosity is transmitted through a unit width of aquifer under a unit hydraulic gradient. It follows that

$$T = Kb = (m/day)(m) = m^2/day$$

where b is the saturated thickness of the aquifer.

The corresponding English system units for aquifer transmissivity are ft^2/day, or more commonly, gpd/ft (gallons per day per foot). Because transmissivity has these units, it would be easy to mistakenly assume that a well's ability to receive water could be calculated by multiplying the transmissivity by the uncased length. This is wrong. Notice from the definition of transmissivity that this dimension has already been taken into account by the aquifer thickness. Instead, to correctly calculate a fully penetrating well's yield or receiving ability, transmissivity should be multiplied by the well's circumference, i.e.

$$Q_{well} = T \pi D_{well} = (Kb)(\pi D_{well})$$

$$D_{well} = \text{well diameter in feet}$$

Field Measurements of Transmissivity

There are many ways to calculate hydraulic conductivity and/or transmissivity: by theoretical or empirical grainsize formulas, by laboratory tests, or by various field measurements. The best of these methods is from drawdown versus time data obtained while actually pumping a well in the field. Preferably the drawdown should be measured in an observation well a known distance from the pumped well, and not in the pumped well itself. Marginally useful results are still obtainable from drawdowns measured in the pumped well.

Lichtler, Joyner, and Anderson document some pumping tests done in the early 1960s of two drainage wells and one shallow supply well in Orange County (Lichtler et al. 1968, p. 134-138). The first drainage well was on Lake Davis in downtown Orlando. The well was 12 inches in diameter, 364 feet deep and cased to 77 feet. The average transmissibility obtained in four surrounding observation wells was 596,000 gpd/ft. The second drainage well was located on Long Lake, six miles northwest of Orlando. The well was 20 inches in diameter, 387 feet deep and cased to 47 feet. The average transmissivity for this well from two observation wells was 271,000 gpd/ft. The supply well was located in eastern Orange County in the water supply well fields for the City of Cocoa. Its diameter is not given; its depth was 509 feet, cased to 244 feet. Average of three observation well transmissivities was 482,000 gpd/ft. All transmissivities were calculated by Lichtler and others using the leaky aquifer method (Lichtler et al. 1968, p. 134). The overall average of the nine measurements was 485,000 gpd/ft.

Lichtler estimated the general transmissivity of the upper Floridan Aquifer to be 500,000 gpd/ft. The range of transmissivities they obtained varied from 130,000 gpd/ft to 745,000 gpd/ft. All transmissivities were calculated by Lichtler and others using the leaky aquifer method and Hantush's equation. One important assumption is that the wells are open to the entire thickness of the aquifer. This is not true in the case of the pumping tests nor in the case of drainage wells in general. They rarely penetrate the entire aquifer. If the median depth is 420 feet and the Upper Floridan is assumed to extend from 150 feet to 600 feet deep, then most drainage wells would terminate somewhere in the middle of the Upper Floridan. They would be half penetrating. Thus, the effective transmissivity of those wells would only be half that of the actual transmissivity of the aquifer. However, since the pumping tests were done on actual wells of representative depths (364 feet to 509 feet), the transmissivities obtained are the effective transmissivities.

Some more recent pumping tests of Orlando area drainage wells are documented in the East Central Florida Regional Planning Council 208 Study of 1977 (ECFRPD, 1977, Chapter 3).

Two drainage wells were pumped out and the actual drawdown in the well versus time was recorded. The data obtained for one of the wells (the 20-inch well at Englewood Park) was unusable. The data for the other well (a 24-inch well draining Lake Sherwood, 320 feet deep and cased t0 120 feet) is usable and is shown in Table 22. This data is plotted in Figure 18. Figure 17 is a plot of the well function W(u) versus u. Figures 17 and 18 yielded a value of

$$T = \frac{Q}{4 s} W(u) = \frac{1,100}{4 \times 0.5} \times 2 = 350 \text{ gpm/ft} = 504,000 \text{ gpd/ft}$$

by Theis' method of superposition.

An alternative solution by the Cooper Jacob method (Todd, 1980, p. 129), with least square curve fitting, yielded a T of 726,000 gpd/ft for the Lake Sherwood Park well. The data is plotted semilogarithmically in Figure 19.

TABLE 22

FIELD DATA OBTAINED WHILE PUMPING THE LAKE SHERWOOD DRAINAGE WELL[a]

TIME	DISSOLVED SOLIDS (PPM)	WATER TEMPERATURE (°F)	UNADJUSTED DRAWDOWN[b] (FEET)
11:00 a.m.	(Started Pump)		0.0
11:05	265	74	1.81
11:10	245	74	1.81
11:20	230	74	2.00
11:30	220	74	2.17
12:00 noon	215	74	2.29
12:30 p.m.	200	74	2.29
1:00	220	74	2.38
1:30	215	74	2.40
2:00	215	74	2.42
2:30	225	74	2.42
3:00	225	74	2.42
3:30	225	74	2.42

[a] 1/19/77: withdrawal rate = 1,100 gpm

[b] Measuring point was manhole cover rim elevation = 87 feet msl.

Source: B.C.&E./CH2M Hill, 1977

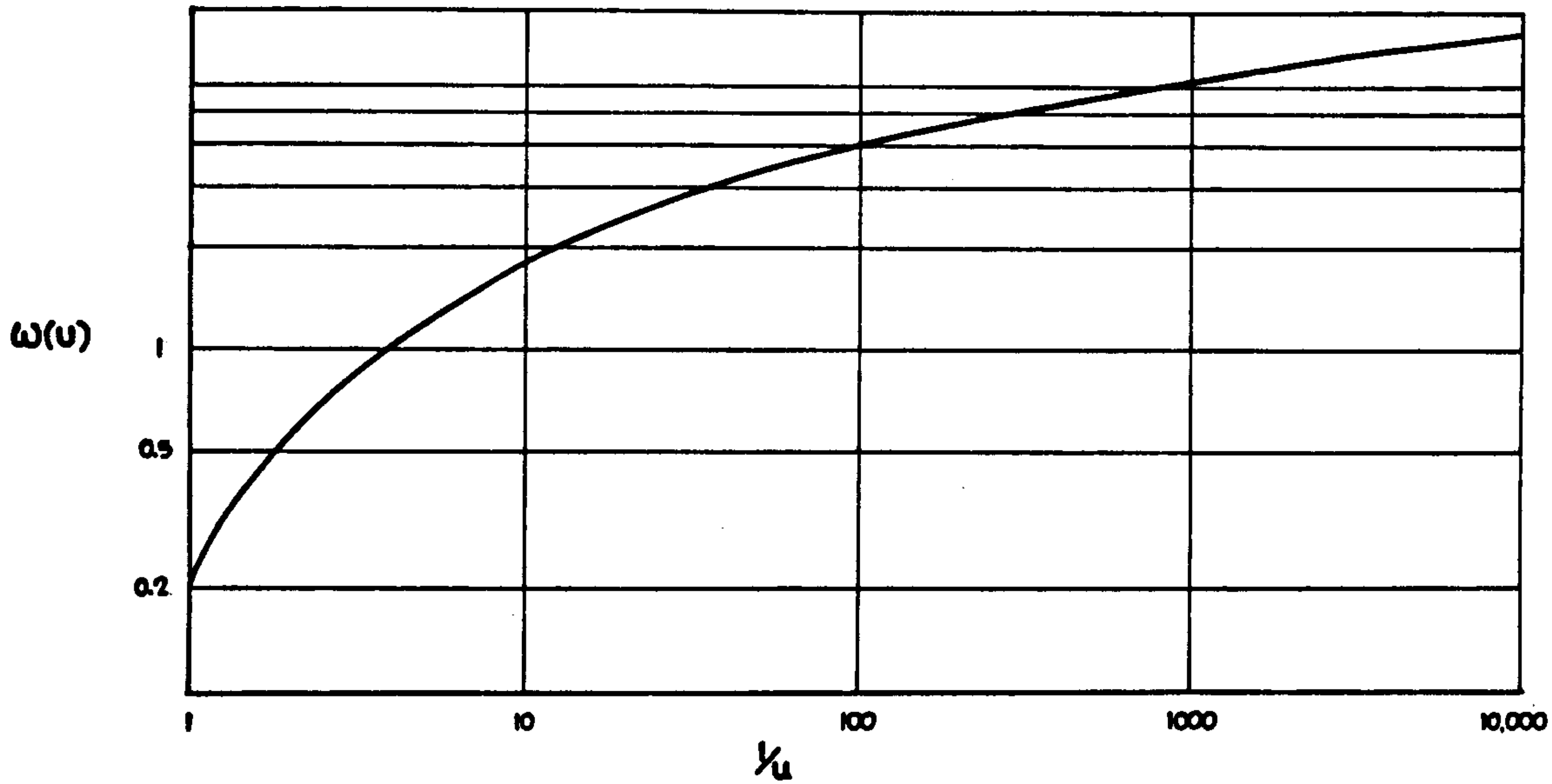

FIGURE 17 WELL FUNCTION ω(u) FOR THEIS SUPERPOSITION SOLUTION

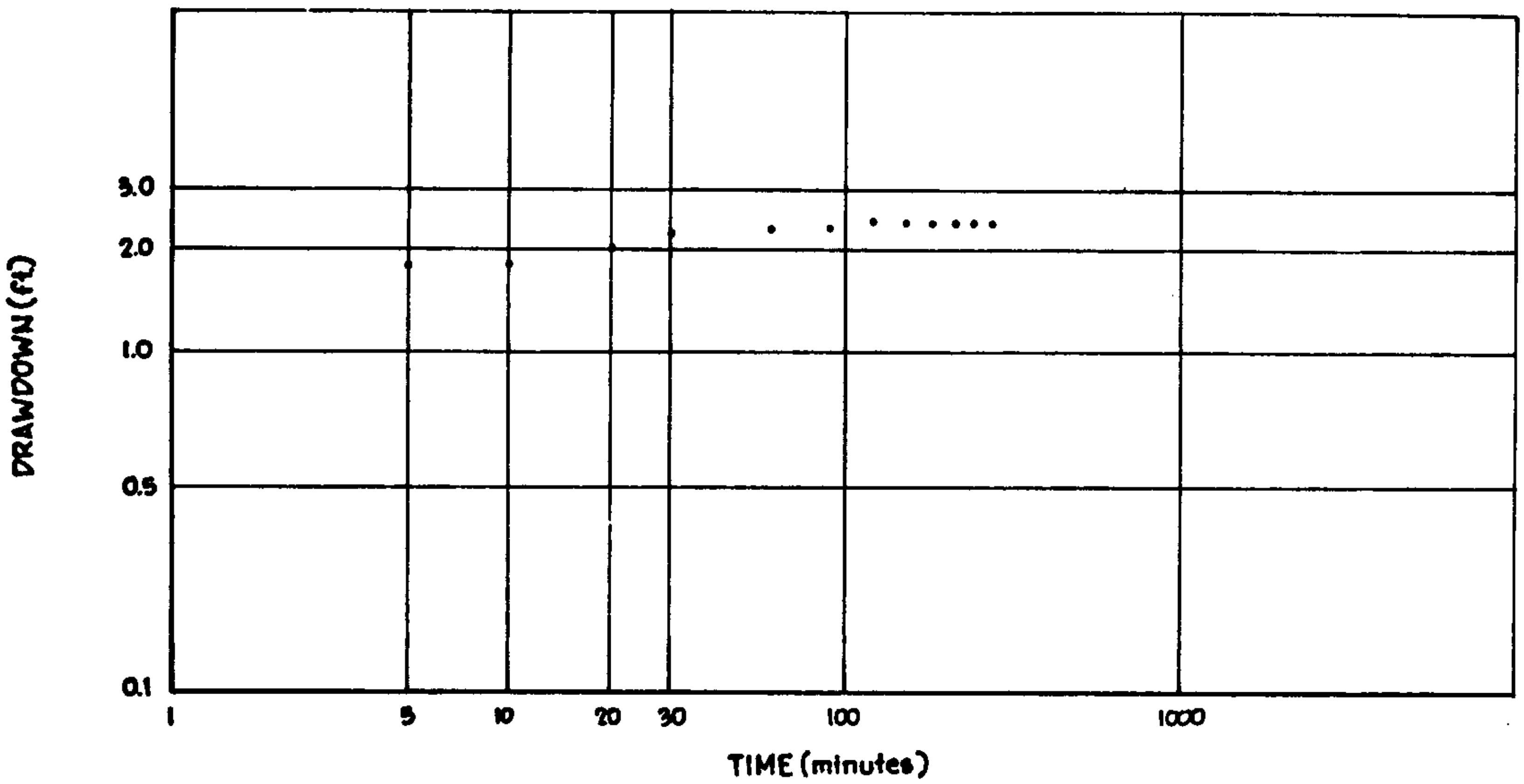

FIGURE 18 LAKE SHERWOOD DRAINAGE WELL PUMPING TEST JAN. 19, 1977
(DATA FROM ECFRPC 208 STUDY ; P. 3-3)

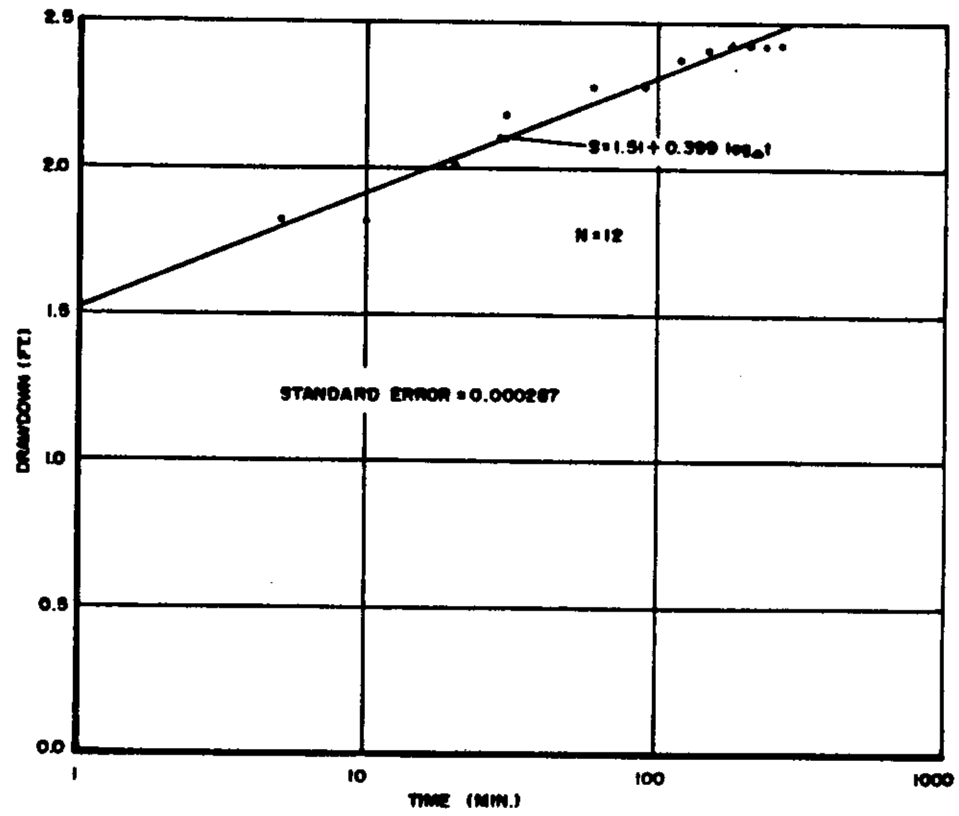

FIGURE 19 COOPER-JACOB SOLUTION FOR LAKE SHERWOOD WELL

Statement of Transmissivity and Calculation of Hydraulic Conductivity

The transmissivity obtained from actual pumping tests of drainage wells, as given in the previous section, ranged from 130,000 gpd/ft to 745,000 gpd/ft. It seems reasonable to assume that the actual transmissivity of the Upper Floridan aquifer is probably 500,000 gpd/ft, possibly as high as 750,000 gpd/ft. These two values are used later to calculate theoretical acceptance ability of drainage wells.

If the thickness of the Upper Floridan aquifer is assumed to be 450 feet, the aquifer being assumed to start at 150 feet down and end 600 feet down, then the hydraulic conductivity can be calculated. This is a very big assumption because, of course, the aquifer is very non-homogeneous as well as being of variable thickness and depth. However, the assumption of depth and thickness is a necessary one and probably also a reasonable one. Therefore, assuming an aquifer thickness b of 450 feet, the hydraulic conductivity $K = T/b$ would range from 148 ft/day (1,111 gpd/ft^2) to 225 ft/day (1,667 gpd/ft^2) for transmissivities of 500,000 and 750,000 gpd/ft, respectively.

Calculation of Acceptance Rates and Quantities

Theoretical Acceptance Rates for Various Diameter Wells

Knowing the aquifer transmissivity permits an easy calculation of the theoretical acceptance for a individual well. This rate is simply $Q_{well} = (\pi D)T$, if the well fully penetrates the aquifer. If the well does not fully penetrate the aquifer, then it has been assumed here to accept a directly proportional amount. Table 23 shows the acceptance rates in gallons per minute and million gallons per day for transmissivities of 500,000 gpd/ft and 750,000 gpd/ft respectively.

The acceptance rates in Table 23 are then used to calculate the total theoretical acceptance ability of water, assuming full penetration of the aquifer, for all the 208 wells in the OUSWMM study area. This total quantity is 357 MGD (for T = 500,000 gpd/ft) and 534 MGD (for T = 750,000 gpd/ft), as shown in Table 24. So it would seem that the aquifer is easily capable of handling the water discharged to it. However, this is for wells of comparable depths to those which were pumped. As has been previously discussed, not all drainage wells are this way.

Information on cased depths of wells is not given in the OUSWMM study; however, Carla Palmer and others have compiled well diameters, depths, and cased depths for 172 of the 175 City of Orlando owned wells (Palmer, et al., 1984, Table 3-1). This information is presented in Appendix C of this thesis. These 175 wells comprise the vast bulk of the 208 OUSWMM wells.

Table 25 shows the categorization of the 175 City of Orlando wells according to their total depth, cased depth, and uncased length. As can be seen from this table, the most common total depth of these drainage wells range from 400 to 500 feet. This agrees well with the median depth of Orange County drainage wells of 420 feet (Schiner and German, 1982, p.11). The most common cased depths are

TABLE 23

THEORETICAL ACCEPTANCE RATES FOR DRAINAGE WELLS
(BASED ON AQUIFER TRANSMISSIVITY)

A. T = 500,000 gpd/ft = 0.5 MGD/ft

SIZE (INCHES)	FULL PENETRATION (p = 1.0)		60% PENETRATION (p = 0.60)		6% PENETRATION (p = 0.06)	
	(GPM)	(MGD)	(GPM)	(MGD)	(GPM)	(MGD)
6	545	0.79	327	0.47	32.7	0.05
8	727	1.05	436	0.63	43.6	0.06
10	909	1.31	545	0.79	54.5	0.08
12	1,091	1.57	654	0.94	65.4	0.09
14	1,273	1.83	764	1.10	76.4	0.11
16	1,454	2.09	873	1.26	87.3	0.13
18	1,636	2.36	982	1.41	98.2	0.14
20	1,818	2.62	1,091	1.57	109	0.16
24	2,182	3.14	1,309	1.88	131	0.19

B. T = 750,000 gpd/ft = 0.75 MGD/ft

SIZE (INCHES)	FULL PENETRATION (p = 1.0)		60% PENETRATION (p = 0.60)		6% PENETRATION (p = 0.06)	
	(GPM)	(MGD)	(GPM)	(MGD)	(GPM)	(MGD)
6	818	1.18	491	0.71	49.1	0.07
8	1,091	1.57	654	0.94	65.4	0.09
10	1,364	1.96	818	1.18	81.8	0.12
12	1,636	2.36	982	1.41	98.2	0.14
14	1,909	2.75	1,145	1.65	115	0.16
16	2,182	3.14	1,309	1.88	131	0.19
18	2.454	3.53	1,473	2.12	147	0.21
20	2,727	3.93	1,636	2.36	164	0.24
24	3,272	4.71	1,963	2.83	196	0.28

TABLE 24

TOTAL TRANSMISSIVITY LIMITED ACCEPTANCE OF WATER
BY OUSWMM STUDY AREA DRAINAGE WELLS
(FULL AND PARTIAL PENETRATION)

		ACCEPTANCE QUANTITIES			
		T = 500,000 GPD/FT		T = 750,000 GPD/FT	
WELL SIZE (INCHES)	# OF WELLS	RATE (MGD/WELL)	TOTAL ACCEPTANCE (MGD)	RATE (MGD/WELL)	TOTAL ACCEPTANCE (MGD)
6	11	0.79	8.64	1.18	12.98
8	18	1.05	18.85	1.57	28.26
10	19	1.31	24.87	1.96	37.24
12	97	1.57	152.37	2.36	228.92
14	5	1.83	9.16	2.75	13.75
16	2	2.09	4.18	3.14	6.28
18	19	2.36	44.77	3.53	67.07
20	32	2.62	83.78	3.93	125.76
24	1	3.14	3.14	4.71	4.71
Sizes Unlisted (Assume 12")	4	1.57	6.28	2.36	9.44
	TOTAL 208	TOTAL 356.04 MGD		TOTAL 534.41 MGD	
Assuming 60% Penetration:			213.62 MGD		320.65 MGD

TABLE 25

CITY-OF-ORLANDO-OWNED DRAINAGE WELLS
CATEGORIZED BY TOTAL DEPTH, CASED DEPTH, AND UNCASED LENGTHS

RANGE OF DEPTH (FEET)	NUMBER OF WELLS W/TOTAL DEPTH IN GIVEN RANGE	NUMBER OF WELLS W/CASED DEPTH IN GIVEN RANGE	NUMBER OF WELLS W/UNCASED LENGTHS IN GIVEN RANGE
0-49	0	3	6
50-99	0	20	14
100-149	2	56	18
150-199	8	34	9
200-249	11	24	16
250-299	5	16	40
300-349	11	6	29
350-399	11	3	7
400-449	35	6	4
450-499	38	0	9
500-549	15	0	5
550-599	10	0	5
600-649	7	0	2
650-699	3	0	0
700-749	5	0	2
750-799	2	0	0
800-849	1	0	1
850-899	6	0	0
900-949	1	0	0
Depth Unknown	4	7	8
Total	175	175	175

from about 100 feet deep to about 250 feet. The most common uncased length (starting at any depth) is 250 feet to 350 feet. This is substantially less than the estimated average thickness, b, of 450 feet for the Upper Floridan aquifer.

The most common cased depths are from about 100 feet to about 250 feet deep. Statistically the most frequent depth of casing is 178 feet, which extends into the assumed top of the Upper Floridan aquifer. The statistically most frequent uncased length, hs, is 274 feet. The total depth of 452 feet is less than 600 feet, so most wells only partially penetrate the aquifer. The average degree of penetration, $p = hs/b = 274/450 = 0.609$, or about 60 percent for the city-owned wells. Thus total acceptance of water for the OUSWMM wells was calculated using partial penetration and would range from 214 MGD to 321 MGD (for T = 500,000 and 750,000 gpd/ft, respectively).

Transmissivity Limit of Lake Angel Well

The 20-inch diameter drainage well at Lake Angel is 172 feet deep and cased to 145 feet deep. Its uncased length is equal to 27 feet and thus the degree of penetration of the well is, at best, $p = hs/b = 27/450 = 0.06$.

The theoretical maximum capacity of the Lake Angel drainage well is thus $Q_{well} = p \times T \times \pi \times D_{well} = 0.06 \times 500{,}000 \times \pi \times 20/12 =$ 0.157 MGD or a paltry 109 gpm. However, the actual maximum capacity of the Lake Angel drainage well was measured at 2,450 gpm after the 4-inch rainstorm of Thursday, March 21, 1985. This points out the folly of trying to apply partial penetration theory to Florida wells.

Better correlations are obtained assuming full penetration. These are: 1,818 gpm for T = 500,000 gpd/ft, and even nicer, 2,727 gpm for T = 750,000 gpd/ft. So it seems quite possible that the actual limiting factor of the Lake Angel well was not well hydraulics but well transmissivity.

This points out that while drainage wells are easily capable of accepting the average daily runoff of between 39 and 54 MGD, they are not always capable of handling the total flow resulting from intense storm events.

The total runoff for the Lake Angel drainage basin for the 4-inch storm event would be about 24 million gallons. Six days would be required to dissipate all of this water through the drainage well. The usefulness of drainage wells to control rainfalls greater than 4-inches is limited. In particular, they should not be solely relied on to control flooding from hurricane-type conditions. Surface storage for runoff before discharge should be made available. The Orlando area was fortunate that the March 21 storm was preceded by a very long dry spell. The additional storage available in the lakes was able to prevent major flooding.

Summary

In this chapter, calculations have been made which show that the Upper Floridan aquifer is transmissive enough to receive water at the rate of from 214 MGD to 321 MGD through the existing 208 partial penetrating drainage wells. This is more than sufficient to handle the probable average daily runoffs of 39 to 54 MGD for the OUSWMM area. However, this transmissivity is not necessarily sufficient to immediately handle the runoff from large, intense storm events. This is evidenced by the Lake Angel well which reached the limits of its disposal capacity as a result of a 4-inch storm. Thus it is necessary to not rely solely on drainage wells for flood control, but to also provide some surface water storage and disposal of storm water.

CHAPTER IX

EXTRAPOLATION OF RESULTS TO ENTIRE METROPOLITAN ORLANDO/ORANGE COUNTY AREA

All of the calculations in this thesis have been restricted to the wells in the Orlando Urban Stormwater Management (OUSWMM) area. The scope of the thesis made this clear. The reason for this is that drainage basins or watersheds for drainage wells have been clearly defined only for the OUSWMM area. They have not been defined outside this area. So meaningful, accurate calculations cannot be made for the non-OUSWMM drainage wells.

However, these wells are still important. All of the wells together recharge the Floridan aquifer. All of the wells together introduce poor quality, contaminated water into the Floridan aquifer, with potential quality effects. The total quantity entering all of the approxiamtely 413 wells must be determined to properly estimate the impact drainage wells have on the area.

The latest U.S. Geological Survey (March 1985) computer listing of drainage wells shows a total of 413 wells in Orange County as shown in Table 26. A few wells probably also exist in southern Seminole County. This total is almost double the total number of OUSWMM wells (208). Thus as an upper bound we can estimate that total quantity of water entering all of the metropolitan Orlando area wells is 34 to 62 MGD on the daily average basis. This is double the runoff (after storage) estimated for OUSWMM wells in Chapter VI. But

TABLE 26

TOTAL NUMBER AND SIZES OF DRAINAGE WELLS IN ORANGE COUNTY
(FROM USGS COMPUTER LISTING)

A. TOTAL NUMBER OF WELLS (BY OWNER) IN ORANGE COUNTY

OWNER	NUMBER OF WELLS	DIAMETER RANGE (IN)	TOTAL DEPTH RANGE (FT)	CASED DEPTH RANGE (FT)
Private	67	2-20	20-1070	?-292
U.S. Government	5	6-18	283-512	150-280
State (FDOT)	14	6-20	370-977	100-405
Orange County	117	4-26	123-777	14-383
City Orlando	194	5-20	109-1049	12-436
City Winter Park	10	4-20	314-507	52-200
City Maitland	1	20	403	117
City Apopka	2	12	315-423	94-124
Unknown	2	8-12	863-?	?
Total	413	2-26	20-1070	12-436

B. TOTAL NUMBER OF WELLS (BY SIZE) IN ORANGE COUNTY

DIAMETER (INCHES)	NUMBER OF WELLS
2	1
3	1
4	10
5	1
6	43
8	53
10	36
12	166
13	1
14	3
16	5
18	21
20	49
24	2
26	1
unknown	20
Total	413

it is unlikely that the runoff to the wells outside the OUSWMM area will be as high as the runoff to the OUSWMM wells, because the county is relatively less densely developed. It has a relatively lower ratio of impervious area to total area, so less runoff is generated. So perhaps a better guess of the quantity of water entering the total wells is 1.5 times the OUSWMM range, or 25.5 to 46.5 MGD, but this is only a guess.

Some, perhaps many, of the above wells are clogged. This is one factor which could seriously reduce the total inflow quantity. Other factors to be determined are what the drainage sub-basins are for each well. It would be most helpful if Orange County, and also the City of Winter Park, would develop drainage programs similar to Orlando's OUSWMM, defining the various drainage areas within their jurisdiction. This would be beneficial in many ways besides just refining the estimate of the quantity entering the drainage wells.

Another program which could be very effective in improving the estimate would be for Orange County, Winter Park, etc., to regularly record on a monthly basis the lake levels of the lakes in their jurisdiction. This would give an indication of the times of flow, and the quantity of storage of these lakes. This option would initially be much cheaper than the first one.

Neither of the above two recommendations should be carried out solely for the purpose of estimating the inflow to all of the drainage wells. Drainage wells are not that important in the overall scheme of things. But there are considerations such as flood control planning, growth management and water quality protection that might warrant carrying out the above recommendations.

Table 26, which summarized the latest USGS listing of drainage wells in Orange County, also pointed out other interesting items. Although most drainage wells are government owned, a substantial number (72) of wells are still privately owned. These range in size from a 2-inch diameter, 20 foot deep well owned by Mr. Joe Zink, to the 20-inch diameter 1070 foot deep well of the Plymouth Citrus Cooperative North of Apopka. Many private individuals names are listed as being owners of wells, although some of the major corporations such as K-mart, Southern Bell Utilities, etc., also own wells. There is even one well located on the property of a McDonald's restaurant!

All of the State-of-Florida-owned wells are Department of Transportation property. Of the six U.S. government wells, five are owned by the U.S. Air Force, the other is a U.S. Post Office well. Perhaps what is most important from this list is that all of the various entities, private and public, own a large number of wells that cover a wide range of depths. This suggests a broadly distributed introduction of drainage well water into every zone of aquifer in the county, with an especially high concentration in Orlando. This could be very significant from a water quality standpoint; the potential for contamination is distributed throughout the county.

CHAPTER X

COMPUTER PROGRAM

Most of the calculations in this thesis were either made by or verified by using the LOTUS 1-2-3 spreadsheet system on an IBM PC computer. A special tailor-made spreadsheet was developed which could be helpful as an inventory device and also for modifying future calculations.

This spreadsheet program stored data files information, on land use areas, and number and sizes of well, for every individual sub-basin within the OUSWMM study area. The program allows the calculation of stormwater runoff for any sub-basin by means of the rational method. Summaries of all parameters (land use, number of wells, and runoff quantities) are available for each entire drainage basin (i.e., Howell Branch, Little Econlockhatchee, etc.) and also for the entire study area. Calculations of storage for every lake in the OUSWMM area, and head on every lake well, are possible by using the City of Orlando lake level records. A well calibration routine was also included. It is also hoped to eventually include computer graphic representation of each drainage basin and the land use within that basin.

CHAPTER XI

FINAL SUMMARY, CONCLUSIONS, RECOMMENDATIONS AND POSTSCRIPT

Final Summary

Drainage wells (also called drainwells) are metal cased holes in the ground which carry excess surface water into the aquifer. They do this by gravity; normally no pumps are necessary to force the water into the aquifer. The first Orlando area drainage wells were dug in 1905 and their numbers grew rapidly until the 1970s when construction of any new drainage wells was absolutely prohibited.

It has been documented in the literature that chemicals and bacteria from sewage and stormwaters have been found in the Upper Floridan aquifer in two different locations in Florida. The extent of the contamination is not known. The median depths for all Orange County drainage wells and most public supply wells are both about 420 feet, and there have been documented isolated cases of pollution of Upper Floridan aquifer public supply wells. Contamination of the Lower Floridan aquifer may have occurred in the past, and it is documented that hydraulic interconnections exist that allow some flow from the Upper Floridan aquifer into the Lower Floridan aquifer.

Therefore, it is justifiable to be concerned about any potential for widespread contamination of drinking water supplies. Research to quantify this potential and develop any necessary remedial plans should be performed. As a first step, the quantity of water discharged through the drainage wells should be estimated. This thesis was an attempt to do this.

There are four basic limits to the quantity of water entering the aquifer through drainage wells. These four limits are:

1) the quantity of runoff available to the wells

2) the hydraulic capacity of the weir or inlet structure leading to the drainage well

3) the hydraulic capacity of the drainage well (pipe) itself, especially of the mouth

4) the aquifer transmissivity at the well location.

Of course these four factors are interrelated, especially the last three. Analyses of all factors were done. Factors 2 and 3 were explored by field experiments described in Chapter IV.

Slightly over half of all the drainage wells in Orange County occur within the Orlando Urban Stormwater Management area. This area contains 208 wells within its 53,499 acres. It is subdivided, according to lake watersheds, into 117 individual sub-basins. The total runoff generated within these 117 sub-basins could vary from 72 to 99 MGD, on a daily average basis, depending on the choice for the runoff coefficient "C". However, only 74 of these 117 sub-basins appear to be able to contribute to drainage wells. The average daily runoff within these 74 sub-basins could vary from 39.1 to 53.4 MGD.

Storage was found to exist in 54 of the sub-basins, 33 of which were drainage well contributors. The mode storage was approximately 1 foot per month per lake, or a total of 2061 acre-ft/ month for all the lakes. This is equivalent to 22 MGD, so the total available runoff is reduced by at least this much to be 17.1 to 31.4 MGD. Thus, from 24% to 31% of all the runoff in the OUSWMM area enters drainage wells. Probably other effects would reduce this further. These losses would apply particularly to the forty-one sub-basins which contribute to drainage wells but are estimated not to possess any significant storage volumes.

The average rate of runoff in the wet summer season is almost four times as high as during the dry non-summer season. However, this does not appear to have a proportional effect on drainage well flows. Based on a limited set of data from seven lakes, the critical factor is the ratio of impervious area to lake area. If this ratio is greater than 6, the well operates continuously; if it is less than 6, the well almost never operates. Instead it has storage available, usually 1 foot deep on the average.

The 20-inch diameter well at Lake Angel was observed to have a maximum capacity of approximately 2500 to 3000 gpm (3.6 to 4.3 MGD). This occurred with an average head over the weir of about 0.65 feet. The average head on the well rim itself could not be measured, but it theoretically must have been less than 0.65 feet. It probably averaged 0.5 feet or less.

If it is assumed that the maximum allowable head on any drainage well is 0.5 feet, and that head is applied uniformly on all the OUSWMM wells for their expected operating duration, then the total maximum hydraulic capacity would be 203.59 MGD. If somehow the maximum possible head is 1.0 feet, then the total hydraulic capacity is 312.1 MGD. In either case this is more than sufficient to handle the average daily runoff from 17.1 MGD to 31.4 MGD.

The actual head on the weirs of seven OUSWMM lakes was found to average 0.28 foot, regardless of season. This corresponds to a 0.18 foot head on the wells themselves, which would mean an average hydraulic inflow of 44.1 MGD for all eighty-four OUSWMM lake wells. The average head on storm sewer wells appears to be 0.1 foot or less. This head would produce a hydraulic inflow of 2.4 MGD. The total estimated hydraulic inflow is then 46.5 MGD. This agrees somewhat with the estimate of 31.4 MGD average daily runoff available to drainage wells.

The transmissivity of the Upper Floridan aquifer was calculated to be 500,000 gpd/ft^2. The corresponding maximum capacities for the wells would range from 356 to 534 MGD. This, too, is sufficient to handle the average daily runoff. However, it was calculated that this is probably not sufficient to efficiently dispose of runoff from extreme storm flows.

The total number of drainage wells in Orange County is about 413. It could be assumed that the total flow for all of Orange County is double that for the 208 OUSWMM wells, or 34.2 to 62.8 MGD. However, most of Orange County is not nearly so densely developed as Orlando itself, so the scale-up factor must be less than double. A more reasonable estimate might be 50% more, which yields 25.7 to 47.1 MGD.

Finally, some points of interest from the literature. For at least two different time periods, drainage wells were subjected to sustained artesian flow which made them counter-productive. On the other side of the spectrum, some wells are located at such an elevation that they never flow; they are almost always useless. The literature mentions Lake Sherwood to be such a well, others the author has observed include Lakes Giles, Arnold, and frequently the DOT Pond at the intersection of U.S. 441 and S.R. 436.

Conclusions

The following are the major conclusions to be drawn from this research effort.

1) Contamination of drinking water supplies by drainage wells has occurred in the past and the potential still exists for such contamination. Drainage wells should be studied in-depth with the goal of completely removing this potential.

2) The maximum capacity of the larger drainage wells is about 2500 to 3000 gpm. The maximum functioning head is probably about 0.5 feet to 1.0 foot, depending on well size and design.

3) The major factor limiting inflow to drainage wells is usually the quantity of water available to them (i.e. available runoff). During heavy storm events the limiting factor switches to either aquifer transmissivity or well hydraulic capacity. It is not certain which.

4) The weighted mean average daily runoff available to the 208 drainage wells in the OUSWMM area, adjusted for storage, is 17.1 to

31.4 MGD. The total quantity to all of Orange County would be something less than twice this quantity. These values should probably be reduced considerably to account for off-site routing, lake seepage, and additional evapotranspiration or infiltration effects occurring in certain sub-basins.

5) Lake wells accept much more water than do non-lake wells. They are able to do so because of their much longer theoretical operating time, and the higher heads they maintain due to the detention capacity of the lakes.

Summary of Recommendations

This study has several serious shortcomings. Among these is the lack of sufficient field data to arrive at categoric answers. The author realizes this.

The following are the author's personal recommendations:

1) No reasonable estimate can be made for the non-OUSWMM wells because their drainage basins are not defined. Orange County should undertake a project to delineate these watersheds. Also, the land use, and if possible the runoff coefficients, for each sub-basin should be determined.

2) All drainage wells need to be surveyed and have the casing cutoff elevation, the surrounding land elevation, and the elevation of the weir/inlet determined. This is especially true for City of Orlando wells. Knowing these values will permit an accurate calculation of lake storage, mean head on wells, and even the actual quantity of water entering the lake wells.

3) For more accurate estimates of inflow, more field surveys of heads on both lake wells and on non-lake wells should be done. Also a survey of the duration of flow after storm events for non-lake wells. Eventually a pattern or consensus range of values for head will emerge.

4) More wells should be field calibrated to coordinate drainage well inflow to well rim head, or better still, to weir heads. In particular several 12-inch diameter wells should be done, and at least one of each of the other sizes.

5) Many lake wells almost never receive flow. When they do receive flow it is a result of a major flood event, and then their contribution to flood abatement is insufficient. These useless lake wells should be properly abandoned by grouting and capping. Examples of these include the Lake Giles and Lake Arnold wells.

6) "Storm sewer" and "alone" drainage wells do not accept a significant quantity of water. Probably they are not truly useful for avoiding street flooding under any circumstances, because of their poor hydraulic design. As a test case several of these wells should be capped. Eventually all of these non-lake wells could probably be grouted and properly abandoned without any major street flooding problems.

7) Since lake drainage wells receive most of the water, it would be beneficial to concentrate water quality research on these wells.

8) Finally, the author has observed many drainage wells in the field, and so offers this as a most important recommendation,

acknowledging that it does not directly relate to the subject of the thesis. Drainage wells are dangerous. They are large, open holes extending down hundreds of feet. Their manhole structures are typically deep and narrow, and often the wells are almost centered underneath the manhole, making it easy for someone to fall directly into. While the manhole covers are usually quite heavy, and are often designed to be difficult to remove, they are rarely locked down. Thus, creative children can remove them. In fact, the wells themselves often "blow off" their covers by overflowing during heavy rains. To make matters worse, many of the wells do not have gratings actually over the well mouth itself. Small children could be lost down 12-inch or smaller diameter wells, and grown men may fit nicely in 20-inch diameter wells. There needs to be a concerted effort made to find all of the drainage wells, grout and cap the many useless ones, and for the others, place secure grating over their well mouths and padlock the manhole covers. Signs saying "DANGER - DRAINAGE WELL" are not sufficient and only invite trouble. Specific examples of wells that need this treatment are the 12-inch Avondale well and the 20-inch Lake Underhill well. But there are probably also others as well.

Postscript

As a final thought, here are some of the author's dreams, or visions, if you will.

Imagine clean cities and streets. Imagine stormwater being purified before discharging into the lakes. Imagine the lake waters and sediments having been treated so that swimming and fishing are possible, even the downtown lakes.

Imagine some drainage wells still existing on some of the lakes. They are equipped with pollution control and safety devices so that they discharge pure water into the aquifer, helping to recharge it.

These are some of the dreams and goals that the author shares with many others, for the Orlando area.

APPENDICES

APPENDIX A

Land Use For Each Sub-basin in the OUSWMM Study Area

TABLE 27

SUMMARY OF LAND USE FOR ALL OUSWMM BASINS

BASIN NAME	CONTRIBUTING ACRES TO DRAINAGE WELL?	RESIDENTIAL	COMMERICAL/ INDUSTRIAL	LAKE	SWAMP	CITRUS	GRASSLAND	FOREST	PARK/ RECREATIONAL	OTHER	TOTAL
Howell Branch	5,680	3,453.5	1,463	684.5	10	3	0	0	92	10	5,716
Little Econlockhatchee River	5,896	5,522	3,734	1,020	37	527	1,017	2,572	0	16	14,445
Little Wekiva River	5,942	4,025	2,888	980	53	33	865	790	0	0	9,634
Shingle Creek	4,432	4,996	3,900	1,352	195	1,305	1,685	2,419	0	0	15,852
Boggy Creek	1,535	1,999	916	472	286	749	2,113	574	0	743	7,852
	23,485	19,995.5	12,901	4,508.5	581	2,617	5,680	6,355	92	769	53,499

TOTAL ACRES IN BASIN: 53,499

TOTAL ACRES CONTRIBUTING TO WELLS: 23,485

TABLE 28

LAND USE FOR EACH SUB-BASIN IN THE HOWELL BRANCH BASIN

SUB-BASIN NAME	SUB-BASIN NUMBER	CONTRIBUTE TO DRAINAGE WELL?	RESIDENTIAL	COMMERCIAL/ INDUSTRIAL	LAKE	SWAMP	CITRUS	GRASSLAND	FOREST	PARK/ RECREATION	OTHER	TOTAL
Sue	SJ-HB-20	YES	286	5	146	0	0	0	0	0	0	437
Rowena	SJ-HB-21	YES	584	203	57	0	0	0	0	0	0	844
Estelle	SJ-HB-22	YES	96	49	45	0	0	0	0	0	0	190
Winyah	SJ-HB-23	YES	503	96	18	0	0	0	0	0	0	617
Formosa	SJ-HB-24	YES	88	60	18	0	0	0	0	0	0	166
Ivanhoe	SJ-HB-25	YES	392	177	122	0	0	0	0	0	0	691
Concord	SJ-HB-26	YES	70	226	65	0	0	0	0	0	0	361
Adair	SJ-HB-27	YES	193	34	24	0	0	0	0	0	0	251
Spring Lake	SJ-HB-28	YES	322	13	37	0	3	0	0	72	0	447
Dot	SJ-HB-29	YES	137	145	6	0	0	0	0	10	0	298
Highland	SJ-HB-30	YES	161	100	33	0	0	0	0	0	0	294
Park	SJ-HB-31	YES	55	34	9	0	0	0	0	0	0	98
Eola	SJ-HB-32	YES	170	133	27	0	0	0	0	10	0	340
Druid	SJ-HB-33	YES	54	69	19	10	0	0	0	0	0	152
Caydee	SJ-HB-34	YES	66	30	11	0	0	0	0	0	0	107
Shannon	SJ-HB-35	YES	60	0	11	0	0	0	0	0	0	71
DOT Ret. Pond	SJ-HB-36	YES	87	87	27	0	0	0	0	0	0	201
Theresa	SJ-HB-37	YES	9	2	2	0	0	0	0	0	10	23
Forest	SJ-HB-38	YES	87	0	5	0	0	0	0	0	0	92
Chelton	SJ-HB-39	NO	33.5	0	2.5	0	0	0	0	0	0	36
			3,453.5	1,463	684.5	10	3	0	0	92	10	5,716

TOTAL ACRES IN BASIN: 5,716 TOTAL ACRES CONTRIBUTING TO WELLS: 5,680

TABLE 29

LAND USE FOR EACH SUB-BASIN IN THE LITTLE ECONLOCKHATCHEE RIVER BASIN

SUB-BASIN NAME	SUB-BASIN NUMBER	CONTRIBUTE TO DRAINAGE WELL?	RESIDENTIAL	COMMERCIAL/ INDUSTRIAL	LAKE	SWAMP	CITRUS	GRASSLAND	FOREST	PARK/ RECREATION	OTHER	TOTAL
Little Econ River	SJ-LE-01	NO	2,020	814	35	37	220	852	2,365	0	0	6,343
Lake Baldwin	SJ-LE-02	NO	5	303	201	0	0	0	13	0	0	522
Susannah	SJ-LE-03	NO	125	139	100	0	0	0	41	0	0	405
Spier	SJ-LE-04	YES	30	7	24	0	0	0	7	0	0	68
Compton	SJ-LE-05	NO	4	0	1	0	0	0	9	0	0	14
Gear	SJ-LE-06	YES	10	14	6	0	0	12	0	0	0	42
Overstreet	SJ-LE-07	NO	0	0	1	0	0	0	0	0	16	17
Barton	SJ-LE-08	NO	156	407	144	0	0	0	22	0	0	729
Little Barton	SJ-LE-09	YES	77	1	16	0	0	0	0	0	0	94
Underhill	SJ-LE-10	YES	205	892	141	0	9	12	0	0	0	1,259
Giles	SJ-LE-11	YES	221	6	27	0	13	0	0	0	0	267
Arnold	SJ-LE-12	YES	221	9	26	0	9	0	0	0	0	265
DOT Retention	SJ-LE-13	YES	108	41	7	0	72	0	0	0	0	228
Greenwood	SJ-LE-14	YES	275	210	14	0	0	28	0	0	0	527
Como	SJ-LE-15	YES	115	30	2	0	0	0	0	0	0	147
Hourglass	SJ-LE-16	YES	227	30	13	0	34	18	0	0	0	322
Crystal Lake	SJ-LE-17	YES	68	1	6	0	0	0	0	0	0	75
Silver	SJ-LE-18	YES	26	0	1	0	0	0	0	0	0	27
Wade	SJ-LE-19	YES	141	43	4	0	0	0	0	0	0	188
Lancaster	SJ-LE-20	YES	271	5	42	0	0	0	0	0	0	318
Weldona	SJ-LE-21	YES	63	16	8	0	0	0	0	0	0	87
Emerald	SJ-LE-22	YES	25	0	2	0	0	0	0	0	0	27
Fern	SJ-LE-23	NO	3	0	1	0	0	0	0	0	0	4
Lawsona	SJ-LE-24	YES	64	14	9	0	0	0	0	0	0	87
Olive	SJ-LE-25	YES	74	8	4	0	0	0	0	0	0	86
Davis	SJ-LE-26	YES	89	4	18	0	0	3	0	0	0	114
Cherokee	SJ-LE-27	YES	22	21	12	0	0	21	0	0	0	76
Lucerne	SJ-LE-28	YES	19	265	17	0	0	0	0	0	0	301
Copeland	SJ-LE-29	YES	30	5	15	0	0	0	0	0	0	50
Lurna	SJ-LE-30	YES	64	55	7	0	0	0	0	0	0	126
Beauty	SJ-LE-31	YES	2	46	2	0	0	0	0	0	0	50
Lake of the Woods	SJ-LE-32	YES	3	167	4	0	0	0	0	0	0	174
Azalea	SJ-LE-33	YES	91	26	2	0	0	12	0	0	0	131
Monterey	SJ-LE-34	YES	144	2	1	0	0	0	0	0	0	147
Ret. Pond/Lido St.	SJ-LE-35	YES	188	50	9	0	25	0	21	0	0	293
Dover	SJ-LE-36	YES	61	14	2	0	0	11	0	0	0	88
Stillinger	SJ-LE-37	YES	37	3	1	0	0	0	0	0	0	41
Rabama	SJ-LE-38	YES	126	32	6	0	20	7	0	0	0	191
Fredrica	SJ-LE-39	NO	71	53	86	0	125	41	94	0	0	470
Turnbull	SJ-LE-40	NO	41	1	3	0	0	0	0	0	0	45
			5,522	3,734	1,020	37	527	1,017	2,572	0	16	14,445

TOTAL ACRES IN BASIN: 14,445 TOTAL ACRES CONTRIBUTING TO WELLS: 5,896

TABLE 30

LAND USE FOR EACH SUB-BASIN IN THE LITTLE WEKIVA RIVER BASIN

SUB-BASIN NAME	SUB-BASIN NUMBER	CONTRIBUTE TO DRAINAGE WELL?	RESIDENTIAL	COMMERCIAL/ INDUSTRIAL	LAKE	SWAMP	CITRUS	GRASSLAND	FOREST	PARK/ RECREATION	OTHER	TOTAL
Little Wekiva River	SJ-LW-01	NO	475	500	35	0	0	180	0	0	0	1,190
Lake Wekiva	SJ-LW-02	NO	481	479	180	6	5	384	204	0	0	1,739
Bay Lake	SJ-LW-03	NO	155	6	35	0	28	50	52	0	0	326
Fairview	SJ-LW-04	YES	455	457	372	0	0	0	9	0	0	1,293
Little Fairview	SJ-LW-05	YES	257	244	78	23	0	0	11	0	0	613
Fair	SJ-LW-06	YES	48	28	3	0	0	0	0	0	0	79
Fairhope	SJ-LW-07	YES	14	6	1	0	0	0	0	0	0	21
Sarah	SJ-LW-08	YES	13	0	12	0	0	0	0	0	0	25
Daniel	SJ-LW-09	YES	56	5	8	0	0	0	0	0	0	69
Silver	SJ-LW-10	YES	382	37	70	0	0	0	0	0	0	489
Emerald Springs	SJ-LW-11	NO	40	5	1	0	0	0	0	0	0	46
Kelly	SJ-LW-12	NO	0	0	3	0	0	54	0	0	0	57
Kristy	SJ-LW-13	NO	70	0	4	0	0	0	0	0	0	74
Kasey	SJ-LW-14	NO	39	0	3	0	0	35	0	0	0	77
Lawne	SJ-LW-15	YES	1,075	1,012	160	19	0	90	486	0	0	2,842
Divot	SJ-LW-16	NO	24	17	2	0	0	0	0	0	0	43
Marilyn	SJ-LW-17	NO	66	1	3	0	0	0	0	0	0	70
White Heron	SJ-LW-18	YES	245	18	3	0	0	24	23	0	0	313
Connie	SJ-LW-19	NO	47	0	3	0	0	20	0	0	0	70
DOT Pond	SJ-LW-20	YES	83	73	4	5	0	28	5	0	0	198
			4,025	2,888	980	53	33	865	790	0	0	9,634

TOTAL ACRES IN BASIN: 9,634 TOTAL ACRES CONTRIBUTING TO WELLS: 5,942

TABLE 31

LAND USE FOR EACH SUB-BASIN IN THE SHINGLE CREEK BASIN

SUB-BASIN NAME	SUB-BASIN NUMBER	CONTRIBUTE TO DRAINAGE WELL?	RESIDENTIAL	COMMERCIAL/ INDUSTRIAL	LAKE	SWAMP	CITRUS	GRASSLAND	FOREST	PARK/ RECREATION	OTHER	TOTAL
Shingle Creek	KR-SC-01	NO	2,435	2,015	167	168	336	1,175	2,099	0	0	8,395
Pat	KR-SC-02	NO	0	5	5	0	0	46	46	0	0	102
Sandy Lake	KR-SC-03	NO	0	178	24	0	0	0	0	0	0	202
Florida Center	KR-SC-04	NO	0	165	10	0	0	0	0	0	0	175
Turkey Lake	KR-SC-05	NO	4	287	310	0	636	255	178	0	0	1,670
Cane	KR-SC-06	NO	124	0	82	0	15	8	18	0	0	247
Cathy	KR-SC-07	NO	9	0	7	0	289	31	0	0	0	336
Geyer	KR-SC-08	NO	108	0	1	0	0	0	7	0	0	116
Richmond	KR-SC-09	YES	88	0	35	0	0	39	0	0	0	162
Clear Lake	KR-SC-10	YES	851	428	360	8	29	28	60	0	0	1,764
Walker	KR-SC-11	NO	37	0	4	0	0	0	0	0	0	41
Beardall	KR-SC-12	YES	12	123	2	0	0	0	0	0	0	137
Lorna Doone	KR-SC-13	YES	20	129	16	0	0	0	0	0	0	165
Rock Lake	KR-SC-14	YES	109	55	43	0	0	0	0	0	0	207
Kozart	KR-SC-15	NO	118	10	8	0	0	0	0	0	0	136
Mann	KR-SC-16	YES	689	234	236	0	0	90	11	0	0	1,260
Sunset	KR-SC-17	YES	83	33	29	0	0	0	0	0	0	145
Notasulga	KR-SC-18	YES	90	111	3	19	0	13	0	0	0	236
Angel	KR-SC-19	YES	219	127	10	0	0	0	0	0	0	356
			4,996	3,900	1,352	195	1,305	1,685	2,419	0	0	15,852

TOTAL ACRES IN BASIN: 15,852 TOTAL ACRES CONTRIBUTING TO WELLS: 4,432

TABLE 32

LAND USE FOR EACH SUB-BASIN IN THE BOGGY CREEK BASIN

SUB-BASIN NAME	SUB-BASIN NUMBER	CONTRIBUTE TO DRAINAGE WELL?	RESIDENTIAL	COMMERCIAL/ INDUSTRIAL	LAKE	SWAMP	CITRUS	GRASSLAND	FOREST	PARK/ RECREATION	OTHER	TOTAL
Boggy Creek	KR-BC-01	NO	286	574	0	286	0	1,718	0	0	0	2,864
Mare Prairie	KR-BC-02	NO	140	4	123	0	21	245	458	0	743	1,734
Warren	KR-BC-03	NO	0	0	12	0	53	0	58	0	0	123
Anderson	KR-BC-04	NO	121	26	15	0	5	6	0	0	0	173
Inwood	KR-BC-05	NO	32	0	8	0	16	0	0	0	0	56
George	KR-BC-06	YES	90	32	67	0	129	115	35	0	0	468
Farrar	KR-BC-07	NO	0	0	7	0	83	13	0	0	0	103
Porter	KR-BC-08	NO	0	0	36	0	307	0	0	0	0	343
Ret. Ponds	KR-BC-09	NO	139	0	1	0	2	5	0	0	0	147
Tennessee	KR-BC-10	NO	54	4	10	0	68	3	0	0	0	139
Condel	KR-BC-11	NO	54	0	3	0	6	0	0	0	0	63
Margaret	KR-BC-12	YES	65	0	10	0	1	0	23	0	0	99
LaGrange	KR-BC-13	YES	85	0	17	0	0	0	0	0	0	102
Bass	KR-BC-14	YES	220	9	24	0	23	8	0	0	0	284
Boot	KR-BC-15	NO	101	12	2	0	0	0	0	0	0	115
Jennie Jewel	KR-BC-16	NO	170	64	66	0	7	0	0	0	0	307
Pinelock	KR-BC-17	YES	353	142	59	0	28	0	0	0	0	582
Willisara	KR-BC-18	NO	89	49	12	0	0	0	0	0	0	150
			1,999	916	472	286	749	2,113	574	0	743	7,852

TOTAL ACRES IN BASIN: 7,852 TOTAL ACRES CONTRIBUTING TO WELLS: 1,535

APPENDIX B

Number and Type of Drainage Wells For Each Sub-basin in the OUSWMM Study Area

TABLE 33

TOTAL NUMBER OF WELLS BY BASIN AND BY SIZE

A. BY BASIN

BASIN NAME	LAKE[1]	STORM[2]	ALONE[3]	BASIN TOTAL
Howell Branch	15	39	9	63
L. Econ River	41	41	2	84
L. Wekiva River	9	12	2	23
Shingle Creek	13	15	3	31
Boggy Creek	6	1	0	7
TOTALS	84	108	16	208

B. BY SIZE

WELL DIAMETER (INCHES)	LAKE[1]	STORM[2]	ALONE[3]	SIZE TOTAL
6"	2	6	3	11
8"	7	8	3	18
10"	4	12	3	19
12"	35	57	5	97
14"	3	1	1	5
16"	1	1	0	2
18"	10	8	1	19
20"	19	13	0	32
24"	0	1	0	1
Not Given	3	1	0	4
TOTALS	84	108	16	208

NOTES:

1. LAKE - lake level control wells
2. STORM - stormwater drainage wells connected to stormsewers or swales, not located on a lake. Elsewhere referred to as "storm sewer" wells.
3. ALONE - stormwater drainage wells which apparently, from OUSWMM maps, are alone, not connected to a supporting stormsewer or swales and also are not located on a lake.

TABLE 34

DRAINAGE WELLS (BY SIZE AND TYPE) IN EACH SUB-BASIN OF THE HOWELL BRANCH BASIN

SUB-BASIN NAME	SUB-BASIN NUMBER	EXHIBIT (MAP) #	NUMBER OF WELLS			TOTAL
			LAKE WELLS	STORM SEWER	ALONE WELLS	
Lake Sue	SJ-HB-20	104	None	1-10"	None	1
Rowena	SJ-HB-21	108	1-12"	6-12"	1-6", 1-8"	9
Estelle	SJ-HB-22	116	None	None	None	0
Winyah	SJ-HB-23	119	None	1-10", 7-12", 1-18", 4-20"	1-10", 1-12"	15
Formosa	SJ-HB-24	124	1-8"	None	None	1
Ivanhoe	SJ-HB-25	126	1-12"	1-6", 1-12"	1-10"	4
Concord	SJ-HB-26	133	None	3-12"	None	3
Adair	SJ-HB-27	137	1-20"	None	None	1
Spring	SJ-HB-28	140	1-18"	None	1-14"	2
Lake Dot	SJ-HB-29	145	2-8"	2-6", 1-10", 3-12", 3-20"	1-6"	12
Highland	SJ-HB-30	148	None	1-10", 1-12"	None	2
Park	SJ-HB-31	151	1-12"	None	None	1
Eola	SJ-HB-32	153	2-12"(unused); 1-20"	1-6", 2-12"	None	6
Druid	SJ-HB-33	156	None	None	1-8", 1-12"	2
Caydee	SJ-HB-34	158	1-18"	None	None	1
Shannon	SJ-HB-35	160	None	None	None	0
DOT Pond	SJ-HB-36	162	None	None	None	0
Theresa	SJ-HB-37	165	1-12"	None	None	1
Forest	JS-HB-38	167	2-12"	None	None	2
Chelton	SJ-HB-39	169	None	None	None	0
			15	39	9	63

TABLE 35

DRAINAGE WELLS (BY SIZE AND TYPE) IN EACH SUB-BASIN OF THE LITTLE ECONLOCKHATCHEE RIVER BASIN

SUB-BASIN NAME	SUB-BASIN NUMBER	EXHIBIT (MAP) #	NUMBER OF WELLS			
			LAKE WELLS	STORM SEWER	ALONE WELLS	TOTAL
Little Econ River	SJ-LE-01	173-180	1-18" (small pond)	1-12"	1-10", 1-18"	4
Baldwin	SJ-LE-02	181	None	None	None	0
Susannah	SJ-LE-03	186	None	None	None	0
Spier	SJ-LE-04	190	1-18"	None	None	1
Compton	SJ-LE-05	192	None	None	None	0
Gear	SJ-LE-06	194	1-8"	None	None	1
Overstreet	SJ-LE-07	196	None	None	None	0
Barton	SJ-LE-08	198	1-12"	1-20"	None	2
Little Barton	SJ-LE-09	203	1-12"	None	None	1
Underhill	SJ-LE-10	205	1-20"	1-6", 8-12", 1-18", 2-20"	None	13
Giles	SJ-LE-11	213	1-20"	None	None	1
Arnold	SJ-LE-12	216	1-20"	None	None	1
DOT Pond	SJ-LE-13	220	1 - size not listed	1-8", 1-18", 1-20"	None	4
Greenwood	SJ-LE-14	222	4-12"	1-8", 3-12", 4-18", 1-20"	None	13
Como	SJ-LE-15	225	None	None	None	0
Hourglass	SJ-LE-16	228	1-8", 1-12"	1-12"	None	3
Crystal	SJ-LE-17	231	1-12"	None	None	1
Silver	SJ-LE-18	234	1-10"	None	None	1
Wade	SJ-LE-19	235	1-18", 1-20"	1-10"	none	3
Lancaster	SJ-LE-20	237	3-12"	1-10", 1-16"	None	5
Weldona	SJ-LE-21	240	None	None	None	0
Emerald	SJ-LE-22	243	1-12"	None	None	1
Fern	SJ-LE-23	245	None	None	None	0
Lawsona	SJ-LE-24	247	1-20"	1-10"	None	2
Olive	SJ-LE-25	249	1-20"	1-8"	None	2
Davis	SJ-LE-26	251	2-12"	2-12"	None	4
Cherokee	SJ-LE-27	253	1-10", 2-12"	1-18"	None	4
Lucerne	SJ-LE-28	255	1-20"	1-10", 3-12"	None	5
Copeland	SJ-LE-29	258	1--10"	None	None	1
Lurna	SJ-LE-30	260	1-12", 1-20"	None	None	2
Beauty	SJ-LE-31	263	None	None	None	0
Lake of Woods	SJ-LE-32	265	2-20"	1-12"	None	3
Azalea	SJ-LE-33	267	1-20"	None	None	1
Monterey	SJ-LE-34	269	1-18"	None	None	1
Retention Pond	SJ-LE-35	271	None	None	None	0
Dover	SJ-LE-36	274	None	1-12"	None	1
Stillinger	SJ-LE-37	276	1-18"	None	None	1
Rabama	SJ-LE-38	278	1-6", 1-20"	None	None	2
Fredrica	SJ-LE-39	280	None	None	None	0
Turnbull	SJ-LE-40	281	None	None	None	0
			41	41	2	84

TABLE 36

DRAINAGE WELLS (BY SIZE AND TYPE) IN EACH SUB-BASIN OF THE LITTLE WEKIVA RIVER BASIN

SUB-BASIN NAME	SUB-BASIN NUMBER	EXHIBIT (MAP) #	NUMBER OF WELLS: LAKE WELLS	STORM SEWER	ALONE WELLS	TOTAL
Little Wekiva River	SJ-LW-01	285-86	1-6" (small pond)	2-12"	1-12"	4
Lake Wekiva	SJ-LW-02	287	None	1-12"	None	1
Bay	SJ-LW-03	299	None	None	None	0
Fairview	SJ-LW-04	302-03	3-12", 1-14"	1-12",	1-12"	6
Little Fairview	SJ-LW-05	311-312	1-18"	None	None	1
Fair	SJ-LW-06	318	1-12"	None	None	1
Fairhope	SJ-LW-07	320	None	None	None	0
Sarah	SJ-LW-08	322	None	None	None	0
Daniel	SJ-LW-09	324	None	None	None	0
Silver	SJ-LW-10	326	None	1-10", 2-12", 1-24"	None	4
Emerald Springs	SJ-LW-11	329	None	None	None	0
Kelly	SJ-LW-12	331	None	None	None	0
Kristy	SJ-LW-13	333	None	None	None	0
Kasey	SJ-LW-14	335	None	None	None	0
Lawne	SJ-LW-15	334-37	1-14", 1-18"	1-10", 1-size unlisted	None	4
Divot	SJ-LW-16	353	None	None	None	0
Marylin	SJ-LW-17	355	None	None	None	0
White Heron	SJ-LW-18	358	None	1-12"	None	1
Connie	SJ-LW-19	361	None	None	None	0
DOT Pond	SJ-LW-20	364	None	1-14"	None	1
			9	12	2	23

TABLE 37

DRAINAGE WELLS (BY SIZE AND TYPE) IN EACH SUB-BASIN OF THE SHINGLE CREEK BASIN

SUB-BASIN NAME	SUB-BASIN NUMBER	EXHIBIT (MAP) #	NUMBER OF WELLS			
			LAKE WELLS	STORM SEWER	ALONE WELLS	TOTAL
Shingle Creek	KR-SC-01	368-378	1-8" (small pond)	1-8", 1-10"	None	3
Lake Pat	KR-SC-02	379	None	None	None	0
Sandy	KR-SC-03	381	None	None	None	0
DOT Pond	KR-SC-04	383	None	None	None	0
Turkey	KR-SC-05	385	None	None	None	0
Cane	KR-SC-06	387	None	None	None	0
Cathy	KR-SC-07	388	None	None	None	0
Geyer	KR-SC-08	389	None	None	None	0
Richmond	KR-SC-09	391	1-20"	None	None	1
Clear	KR-SC-10	393-95	1-size unlisted	1-6", 2-8", 1-10", 3-12", 1-20"	1-8"	10
Walker	KR-SC-11	405	None	None	None	0
Beardall	KR-SC-12	407	1-10"	None	None	1
Lorna Doone	KR-SC-13	409	1-12"	1-8", 1-12"	None	3
Rock	KR-SC-14	411	1-20"	None	None	1
Kozart	KR-SC-15	413	None	None	None	0
Mann	KR-SC-16	415-16	1-16"	1-12"	1-12"	3
Sunset	KR-SC-17	424	2-12"	None	None	2
Notsaluga	KR-SC-18	426	1-size unlisted 1-20" (small pond)	1-12"	None	3
Angel	KR-SC-19	429	2-20"	1-12"	1-6"	4
			13	15	3	31

TABLE 38

DRAINAGE WELLS (BY SIZE AND TYPE) IN EACH SUB-BASIN OF THE BOGGY CREEK BASIN

SUB-BASIN NAME	SUB-BASIN NUMBER	EXHIBIT (MAP) #	NUMBER OF WELLS			
			LAKE WELLS	STORM SEWER	ALONE WELLS	TOTAL
Boggy Creek	KR-BC-01	434-37	None	None	None	0
Lake Mare Prairie	KR-BC-02	438	None	None	None	0
Warren	KR-BC-03	440	None	None	None	0
Anderson	KR-BC-94	441	None	None	None	0
Inwood	KR-BC-05	444	None	None	None	0
George	KR-BC-06	446	1-12"	None	None	1
Farrar	KR-BC-07	447	None	None	None	0
Porter	KR-BC-08	448	None	None	None	0
Retention Ponds	KR-BC-09	449	None	None	None	0
Tennessee	KR-BC-10	451	None	None	None	0
Condel	KR-BC-11	453	None	None	None	0
Margaret	KR-BC-12	455	1-14"	None	None	1
Lagrange	KR-BC-13	457	1-8"	1-8"	None	2
Bass	KR-BC-14	459	1-12"	None	None	1
Boot	KR-BC-15	462	None	None	None	0
Jennie Jewel	KR-BC-16	464	None	None	None	0
Pineloch	KR-BC-17	467	1-12", 1-18"	None	None	2
Willisara	KR-BC-18	471	None	None	None	0
			6	1	0	7

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