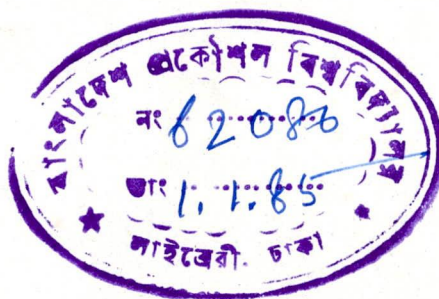


A STUDY ON THE PERFORMANCE OF DEEP
TUBEWELL AND AQUIFER CHARACTERISTICS
INSIDE
DHAKA CITY

MD. SERAJUDDIN

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE
OF MASTER OF SCIENCE IN ENGINEERING (WATER RESOURCES)



BANGLADESH UNIVERSITY OF ENGINEERING AND TECHNOLOGY, DHAKA



SEPTEMBER, 1984

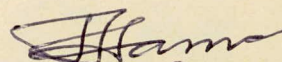
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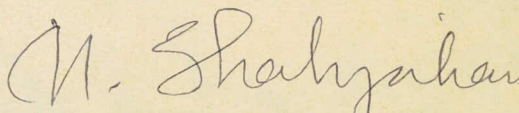
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WE HEREBY RECOMMEND THAT THE THESIS PREPARED BY
MD. SERAJUDDIN
ENTITLED "A STUDY ON THE PERFORMANCE OF THE DEEP
TUBEWELL AND AQUIFER CHARACTERISTICS INSIDE DHAKA CITY"
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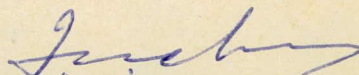
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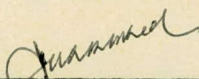
Member


(Dr. M. Shahjahan)

Member


(Dr. J. U. Chowdhury)

Member


(Mr. S. A. N. M. Wahed)

Head of the Department


(Dr. A. Hannan)

Hydrogeologic properties of an aquifer form the basis of evaluating the aquifer potentiality in respect of storing and transmitting water. (Efficient withdrawal of ground water is dependent on tubewell performance which is in turn governed by the design and construction of tubewell fixtures). (In this context the performance of some of the Dhaka WASA wells in Dhaka sunk by WASA which are not upto expected performance and the aquifer characteristics inside Dhaka city has been studied.) The related informations and soil samples have been collected from WASA offices and construction sites.

The mode and type of failure of some of the early defunct tubewells have been noted carefully. Detailed sieve analysis of the aquifer samples obtained during drilling of a number of wells at different locations of Dhaka city and the gravel packing material used at the working sites have been done to get a clear picture of the characteristics of aquifer and packing materials. The formation constants, well loss constants, specific capacities etc have also been calculated. It is concluded that the constant and fixed slot size (50"/1000) of screen as used by Dhaka WASA might have created problems in some cases specially when the screen is placed in finer aquifer layer. The design of gravel pack as used by WASA need to be reviewed for better functioning of the tubewells. It has further been observed that development of most of the wells studied is insufficient. The Development procedure and method used for development may be reviewed for

its improvement. The accuracy of development may be checked by the calculation of well loss constant. The specific capacity is dependent on well loss as well as aquifer loss. Specific capacity calculated from well data may be found low even for a relatively permeable aquifer due to excessive well loss.

The uniformity co-efficient of the aquifer material inside Dhaka city is around 2. The maximum and minimum effective grain size (d_{50}) of the aquifer material are around 0.73 mm and 0.25 mm respectively with most of the values in between 0.3 mm and 0.48 mm. It is necessary to do proper sieve analysis of aquifer material before installing the well fixtures for proper selection of the layers for screening. The procedure of putting gravel packing material around the well may further be improved to minimize the possibility of damage of the screen.

The coefficient of transmissibility and co-efficient of storage of the study area vary from 0.35×10^5 gpd/ft to 1.46×10^5 gpd/ft and 0.013 to 0.129 respectively.

To study the problem of mining it is necessary to study the variation of highest water level year to year. Study relating to lowest water level is necessary keeping the withdrawal rate constant. This may help us to decide whether the rate of withdrawal can be increased if necessity arises.

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NOTATION

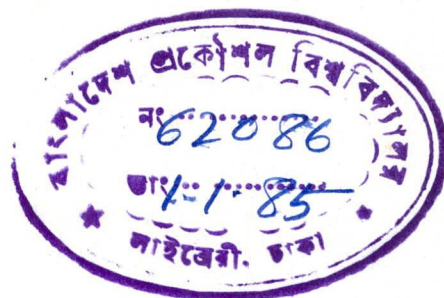
<u>Symbols</u>	<u>Definition</u>
A_p	Ratio of slot area to the total area of the screen
b	Thickness of Aquifer
C	Well loss constant
C_c	Contraction co-efficient
C_s	Screen co-efficient
C_v	Velocity co-efficient
C_u	Uniformity co-efficient
D	Screen diameter
d_{10}	Size such that 10% of the total sample is finer by weight
DTW	Deep Tubewell
d_{50}	Size such that 50% of the total sample is finer by weight
e	Void ratio
K	Co-efficient of permeability
$\log_e$	Logarithm to the base e
$\log_{10}$	Logarithm to the base 10
PHE	Public Health Engineering
Q	Discharge
r	radial distance from well center/radius of influence
r_w	well radius
SWL	Static water level
S_r	Specific retention
S_y	Specific yield
S	Storage co-efficient
SW_L	well loss

NOTATION .(Contd.)Symbols

s_i	Drawdown
ΔS	Drop in piezometric surface
S_L	Optimum length of screen
T	Co-efficient of Transmissibility
t	time elapsed since discharge began
t_0	time intercept on the zero drawdown axis
v	volume of rock
WASA	Dhaka Water Supply and Sewerage Authority
w	volume of water
$W(u)$	Well function
W_y	Volume of water drained
Δh	Drawdown difference per log cycle of time
α	Porosity

CHAPTER ONE

INTRODUCTION



1.1 General

Water, from the time immemorial, has played very important role in the life of living beings. Indeed, water is, with the exception of air, the most important single substance to man's survival-man can survive much longer without food than he can without water.

All water come in the form of precipitation. It is evaporated from the ocean, river, pond etc., condenses to form clouds and finally precipitates over land. When the water falls on the ground a portion of it runs off across the surface of the ground and ^{another} / portion of it ^{percolates} / into the ground. Thus the source of supply is of two types, i) surface water supply and ii) ground water supply. Actually, surface and ground water sources are not always separate. A source of supply may appear as surface water at one point on the earth's surface and may become ground water at another and then again as surface water at a third point and so on. Hydraulic interconnections make this possible.

Ground water is found in very large quantity and is extremely widely distributed. According to Nace (9) the total volume stored under land areas may be around 80 million cubic meter, half of which may be at depths less than 800 meter. This is about 35

times the quantity of fresh water available on the surface of earth and is about one third of the volume stored as ice in polar region and the mountain ranges.

With the increase in population, the demand for water is also increasing throughout the world. Effective management of ground water system is essential to meet the increasing demand for water. Ground water can be used as a reliable source of water supply irrespective of the climate. Surface water is generally easy and economical to harness, but its availability varies with the season. Ground water source yields clear water at almost constant temperature and it is preferred as compared to surface water for municipal water supply.

An assured supply of water can be obtained from the underground sources even in the desert areas if the system is properly designed and maintained. New Valley Project of Egypt is a good example of proper use of ground water where surface water was not available. There are numerous such examples. 94% of all the water works use ground water as source of water and they supply 77% of the total water consumed.

History of the water wells is as old as the history of ground water itself. Egyptians were known to have used water wells in 2100 B.C. Wells are also known to have been in use in ancient Greece, Italy and India and artesian wells were

sunk in China in very early times. Ancient China possessed the deepest well in the world and it was known to be 1500 ft deep. In ancient Greece, Italy, Egypt and India wells as deep as 300 ft were used to collect ground water for various purposes.

Though tremendous quantity of water is stored in earth crust and its extraction is generally easy and in many areas where it is used the stock replenish themselves, yet, this raw material has associated problems which can prohibit or limit its use.

Ground water is a vital source of water supply for Bangladesh. Bangladesh is almost entirely underlain by water bearing formation at depths varying from zero to forty feet below the ground surface. Almost entire supply of drinking water and a good proportion of the irrigation water come from the under ground sources.

The city of Dhaka has an extensive water supply system dependant mainly on ground water source. Dhaka WASA is supplying potable water to the metropolis area by operating about 100 deep tubewells sunk at different places inside the city.

The present population of Dhaka urban area is about 3500 thousands and the potable water demand is 144 million gallon per day (U.S.Gallon). At present Dhaka WASA supplies 85 million

gallons per day (MGD) of potable water. Out of this 85 MGD, 80 MGD is by deep tubewells and rest from surface water source. The demand of potable water is increasing day by day and WASA shall have to depend mostly on tubewells for its water supply for a number of years to come. Even after the installation of a large scale surface water treatment plant, as planned by WASA, WASA will have to use ground water source as supplementary one to be utilized according to necessity.

Existing tubewells are generally of gravel packed design and drilled to depths of 220-550 feet. Most of them are over 350 feet deep. Typically they have an upper pump chamber section of diameter 16 to 18 inches upto a depth of 100 to 160 feet and a lower intermittently screened section of 8 inches diameter steel casing.

The theoretical specific capacity of a tubewell should average about 30 gallons per minute per foot in Dhaka. Any existing well with a specific capacity less than about 18 gallon per minute per foot is excessively costly to operate and should be either redeveloped or abandoned (38).

From records it has been found that many wells are becoming defunct much earlier than its anticipated life, while a large number of wells are giving discharge less than the design discharge. The total cost of one deep tubewell installed by

WASA is around taka 25 lakhs. If we add running, maintenance and operation cost then the figure reaches to a still higher value. So, an early failure and a lower discharge of wells causes tremendous financial loss as well as sufferings of the inhabitants due to scarcity of potable water. So far the author's knowledge goes no systematic study has yet been made to identify the probable causes of the unsatisfactory performance of WASA tubewells. But such a study may help in understanding the probable reasons for unsatisfactory functioning of the tubewells and the obtained result may be used in the future program for ground water utilization by tubewell.

1.2 Objective of the study

The present study has been taken up with the following main objectives:

1. To study the present performance of WASA tubewell and to identify the probable causes of unsatisfactory functioning of the wells which are not upto expected performance and to suggest probable remedial measures.
2. To study the effect of ground water extraction by Dhaka WASA on ^{sub-surface} / water level at different places inside Dhaka city and on existing WASA wells.
3. To evaluate the aquifer characteristics at different places of Dhaka city.

CHAPTER TWO

GROUND WATER OCCURRENCE AND MOVEMENT

2.1 Introduction

Though ground water development dates back from ancient times yet the concept of ground water hydrology has come to the modern form after passing through many imagination and assumption made by ancient philosophers. Ground water hydrology evolved as a definite science in the 19th century. The publication of Darcy's law in 1856 added a great leap in the development of this science. Jules Dupuit and A. Theim developed formula for flow towards wells. Towards the end of this century ph.Forchheimer and C.S.Slitchee applied the potential theory and the Laplace equation to flow through porous media (7,9,13,14,24).

In the last few decades ground water flow has received a thorough mathematical treatment.F.J.de Glee, C.V.Theis, M.Muskat, C.Jacob, N.S.Boulton and M.S.Hantush have been some of the pioneers in this field (7,9,12,13,14,24).

The increasing demand for potable water in the country has made necessary the development of ground water resources. The essential feature of ground water development is the installation of ground water well. For the proper assesment of ground water resources its development and subsequent fruitful use

through tapping by water well, an understanding of the processes and factors affecting the origin, occurrence and movement of ground water and the factors related with well design, construction and maintenance are essential. The basis for development of this resource, as it is seen, is a long history of practical experiences plus relatively recent developments of theory. In this context a brief review on the related processes and factors are made in the following.

2.2 Origin and Occurrence of Ground Water

Almost all ground water can be thought of as a part of the hydrologic cycle, including surface and atmospheric (meteoric) waters. Relatively minor amounts of ground water, however, may enter this cycle from other origins.

Ground water occurs in permeable geologic formations known as aquifers, i.e., formations which permit appreciable water movement yielding water in sufficient quantity when the formation is tapped. Ground water reservoir and water bearing formation (also bed, stratum or deposit) are commonly used synonymous. In contradistinction an aquiclude is an impermeable formation which may contain water but is incapable of transmitting significant water quantities and an aquifuse is an impermeable formation neither containing nor transmitting water. Again the term aquitard is used for a saturated bed, formation or group of formation which yields

inappreciable quantities of water compared to an aquifer but through which appreciable leakage of water is possible.

Most rocks contain numerous open spaces, not occupied by solid mineral matter, but may be occupied by ground water. These spaces are known as voids, interstices, pores or pore space. Because interstices can act as ground water conduits, they are of fundamental importance to the study of ground water. Typically, they are characterized by their size, shape, irregularity, and distribution. Original interstices were created by geologic processes governing the origin of the geologic formation and are found in sedimentary and igneous rocks. Secondary interstices developed after the rock was formed; examples include joints, fractures, solution openings and openings formed by plants and animals. With respect to size, interstices may be classed as capillary, super capillary and sub capillary. Capillary interstices are sufficiently small so that surface tension forces will hold water within them, super capillary interstices are those larger than capillary ones, and subcapillary interstices are so small that water is held primarily by adhesive forces. Depending upon the connection of interstices with others, they may be classed as communicating or isolated.

2.3 Index Properties of Soils

Soils consist of mixture of various sizes. A ground water engineer is mostly concerned with the coarser fractions of soil,

for which grain size classification is the most pertinent.

In trying to describe the fineness or coarseness of a granular material, we speak of fine sand, coarse sand, fine gravel and the like. Unfortunately we seldom think in terms of specific particle size limits for each of these materials. The result is that what one person may call coarse sand might be referred to as fine gravel by another.

To provide a specific definition for each descriptive term, several different grain size classifications have been developed. Each of these systems has been adopted for application in the special fields where they seem to fit the best. Soil classification according to Massachusetts Institute of Technology (MIT), Indian Standards Institution (ISI), ASTM and U.S. Bureau of soil are given in table no. 2.1.

The U.S. geological survey formerly followed the Bureau of Soils classification when describing sizes of sands and gravels. In recent years, however, the ground water Branch of the U.S. Geological Survey has used the classification described in Table 2.2.

These size limits were set-up by a sub-committee on sediment terminology of the National Research Council.

uniformity co-efficient is larger when the degree of uniformity is low. A uniform material is defined as that for which the uniformity co-efficient is equal to or less than two. A well graded material is defined as that having a uniformity co-efficient of more than two.

Effective Grain Size

The effective grain size of aquifer material is designated by d_{50} indicating that the cumulative weight of all grains smaller than this size is 50 percent of the total weight of the sample.

Uniformity Co-efficient

Uniformity Co-efficient of a soil sample is given by C_u

$$C_u = \frac{d_{60}}{d_{10}}$$

Where

C_u = uniformity Co-efficient

d_{60} = size such that 60% of the total sample is finer by weight.

d_{10} = size such that 10% of the total sample is finer by weight.

For uniform soils where there is not much variation in the grain sizes, there is little difference between the d_{60} and d_{10} size and the uniformity co-efficient is, therefore, low. A soil with all grains of practically the same size has a uniformity co-efficient of unity. A higher uniformity co-efficient represents greater variation in sizes of soil grains. Thus the uniformity co-efficient is larger when the degree of uniformity is low. A uniform material is defined as that for which the uniformity co-efficient is equal to or less than two. A well graded material is defined as that having a uniformity co-efficient of more than two.

Porosity

Porosity is a measure of the interstitial space of a rock and is expressed (quantitatively) as percentage of the total volume of rock to the interstices. If α is the porosity then

$$\alpha = \frac{100 W}{V}$$

Where W is the volume of water required to fill or saturate all of the pore space and V is the total volume of the rock or soil. In general, a porosity greater than 20 percent is considered large, a porosity between 5 and 20 percent is considered medium and a porosity less than 5 percent is considered small. The highest porosity known is 80 to 95 percent, which has been reported for freshly deposited alluvium of the Mississippi delta.

The porosity of a sedimentary deposit depends chiefly on the shape and arrangement of its constituent particles, the degree of assortment of its particles, the cementation and compacting to which it has been subjected since its deposition, the removal of mineral matter through solution by percolating waters and the fracturing of the rock, resulting in joints and other interstices. In well-sorted and well rounded deposits, the size of grains has no influence on porosity; thus, a deposit of boulders may have the same porosity as a deposit of clay. The porosity of many deposits is increased by the irregular angular shapes of its constituent grains. Porosity decreases with increase in the variety of size of grains; small grains fill interstices between large grains. Representative porosity ranges for sedimentary

materials are listed in Table 2.3.

2.4 Ground Water

The sub-surface occurrence of water may be divided into zones of saturation and aeration. In the zone of saturation all interstices are filled with water under hydrostatic pressure. The zone of aeration consists of interstices occupied partially by water, called vadose water and partially by air. The saturated zone is bounded at the top by either a limiting surface of saturation or overlying impermeable strata, and extends down to underlying impermeable strata such as clay beds or bed rocks. In the absence of overlying impermeable strata, the upper surface of the zone of saturation is the water table, or phreatic surface. Actually saturation extends slightly above the water table owing to capillary attraction; however, water is held here at less than atmospheric pressure. Water occurring in the zone of saturation is commonly referred to simply as ground water.

Since ground water fills all of the interstices in the saturated zone, the porosity is a direct measure of the water contained per unit volume of soil. But not all the water of a saturated rock can be withdrawn by drainage or pumping from a well. A part of water is always retained in interstices largely by the forces of molecular attraction, adhesion and cohesion.

This water retention varies (1) directly as the aggregate surface of the interstices and (2) indirectly as the size of the interstices and also depends on time of drainage, temperature, surface tension, viscosity and specific gravity of water.

The specific retention of a rock or soil is the ratio expressed as a percentage of the volume of water it will retain after saturation against the force of gravity to its own volume. If S_r is the specific retention, then

$$S_r = \frac{W_r \times 100}{V}$$

Where W_r is the volume occupied by retained water, and V is the gross volume of the rock or soil. On the other hand, the water which can be drained is expressed as the specific yield S_y .

The term effective porosity has a synonymous meaning. It may be defined as the ratio expressed as a percentage of the volume of water which, after being saturated, can be drained by gravity to its own volume.

Therefore, $S_y = 100 W_y / V$

Where W_y is the volume of water drained. Because $W_r + W_y = W$, it is apparent that, $\alpha = S_r + S_y$.

Values of specific yield depend upon grain size, shape and distribution of pores and compaction of the stratum. For uniform sand, specific yield may equal upto 30 percent, but most alluvial aquifers give values in the range of 10 to 20 percent. Representative specific yield ranges for selected rocks are given in table 2.4.

For most rocks, gravity drainage of interstices is not instantaneous and the water yielding capacity increases at a diminishing rate as the time of drainage increases, gradually approaching the Specific yield. Gravity yield has been defined as the percentage of the total volume of rock occupied by ground water that will drain under the action of gravity during a given period of drainage.

2.5 Types of Aquifer

Most aquifers are of large areal extent and may be visualized as underground storage reservoirs. Water enters a reservoir from natural or artificial recharge, it flows out under the action of gravity or is extracted by wells. Aquifer may be classed as unconfined or confined depending upon the presence or absence of a water table.

An unconfined aquifer is one in which a water table serves as the upper surface of the zone of saturation. It is also known as a free, phreatic water table or non artesian aquifer. The water table varies in undulating form and in slope, depending upon areas of recharge and discharge, pumpage from wells and permeability

Confined aquifers, also known as artesian or pressure aquifers occur where ground water is confined under pressure greater than atmospheric by overlying relatively impermeable strata. Water enters a confined aquifer in an area where the confining bed rises to the

surface or ends underground and the aquifer becomes unconfined. A region supplying water to a confined aquifer is known as a recharge area. Rises and falls of water in wells penetrating confined aquifers result primarily from changes in pressure rather than changes in storage volume. Hence confined aquifers have only small changes in storage and serve mainly as conduits for conveying water from recharge areas to location of natural or artificial discharge. The piezometric surface of a confined aquifer is an imaginary surface coinciding with the hydrostatic pressure level of the water in the aquifer. The water level in a well penetrating a confined aquifer defines the elevation of the piezometric surface at that point. Should the piezometric surface lie above ground surface a flowing well results.

A special case of an unconfined aquifer is the perched aquifer. This occurs wherever a ground water body is separated from the main ground water by a relatively impermeable stratum of small areal extent and by the zone of aeration above the main body of ground water. Well tapping perched aquifers yield only temporary or small quantities of water.

2.6 Storage Co-efficient

Water recharge to or discharge from an aquifer represents only a change in the storage volume within the aquifer.

For an un-confined aquifer it is simply expressed by the product of the volume of aquifer lying between the water table at the beginning and at the end of a period of time and the average specific yield of the formation. In confined aquifer, however, assuming the aquifer remain saturated, changes in pressure produce only small changes in storage volume. Thus the hydrostatic pressure within an aquifer partially supports the weight of the over burden while the solid structure of the aquifer provides the remaining support when hydrostatic pressure is reduced, such as by pumping water from a well, the aquifer load increases. A compression of aquifer results in forcing some water from it. In addition, lowering of the pressure causes a small expansion and subsequent release of water. The water yielding capacity of a confined aquifer can be expressed in terms of its storage co-efficient.

Storage coefficient 'S' is defined as the volume of water that an aquifer releases from or takes into storage per unit surface area of aquifer per unit change in the component of head normal to that surface. For a vertical column one foot by one foot extending through a confined aquifer, the storage co-efficient 'S' equals the volume of water in cft, released from the aquifer when the piezometric surface declines one foot. In most confined aquifers, values falls in the range 0.0005 to 0.005, indicating that large pressure changes over extensive areas are required to produce substantial water yield.

The storage co-efficient of an unconfined aquifer corresponds to its specific yield and its volume ranges from 0.02 to 0.30.

2.7 Ground Water Basins

A ground water basin may be defined as a physiographic unit containing one large aquifer or several connected and interrelated aquifer. The term basin is used very loosely in practice and, because of its vagueness, has no clear general definition; however it implies an area confining a ground water reservoir capable of furnishing a substantial water supply. The concept of ground water basin has become important in recent years with the realization that over development of ground water in one portion of a basin directly affects water supplies throughout the remainder of the basin.

CHAPTER THREE

WELL HYDRAULICS

3.1 Introduction

The study of well hydraulics is a fascinating subject and has occupied the attention of many capable investigators. Like all engineering sciences, however, it is not an exact science and the mathematical assumption which often have to be made represent only limiting cases of real conditions. For instance one interested in theoretical well hydraulics usually finds he must postulate at least some of the following conditions which, are not usually fully correct.

1. The aquifer is homogeneous and isotropic and is of infinite areal extent and of uniform thickness.
2. Well penetrates the entire aquifer and flow is laminar.
3. At the beginning of pumping the piezometric surface and/or phreatic surface are horizontal.
4. The aquifer is pumped at a constant rate.

In spite of the necessity for introducing some of these unrealities excellent progress has been achieved in theoretical well hydraulics. Useful tools for taking into account specific deviation of natural condition from the idealized conditions assumed have also been devised in many instances. Well hydraulics, especially in recent years has contributed much towards a better and more economic development of the important ground water

resource .

3.2 Flow formulas

3.2.1 Equilibrium case

Dupuit, (20) is credited with first combining Darcy's law of laminar flow through sands and gravels with the statement of continuity in order to derive an equation for well discharge. Dupuit assumed complete axial symmetry, steady flow through an infinitely extending aquifer, and ignored the effects of curvilinear motion. He deduced

$$Q = K \frac{(h_2^2 - h_1^2)}{\log_e r_2/r_1} \dots \dots (3.1)$$

Where, Q is discharge ($L^3 T^{-1}$), K is the permeability or, more properly the hydraulic conductivity (LT^{-1}), and h_1 and h_2 are the elevations of the water surface above a level, impermeable bed bounding the lower side of the aquifer at radial distances r_1 and r_2 (Fig. 3.1).

Dupuit's equation implies a well discharging from a water table condition. However, all of the common steady flow formulas for fully penetrating wells stem from the Dupuit's concept and may readily be deduced by algebraic manipulations of equation 3.1.

Thus from equation 3.1 we may get equation 3.2 in terms of drawdown S_1 and S_2 and average aquifer thickness b of water transmitting formation.

$$Q = \frac{2 \pi K b (S_1 - S_2)}{\log_e r_2/r_1} \dots \dots (3.2)$$

Equation 3.2 may also be used to describe a well discharging from an artesian aquifer of thickness b . Common practice of Geological survey is to introduce the term transmissibility T into these equations. Transmissibility is the product Kb and is widely used in Characterizing an aquifer. With this substitution equation 3.2 Yields

$$T = \frac{Q \log_e r_2/r_1}{2 \pi (S_1 - S_2)} \dots \dots (3.3)$$

In geologic survey practice T is ordinarily expressed in gallon per day per foot, Q in gallons per minute and length in feet and by introducing the necessary unit conversion constants for this equation 3.3 yields

$$T = \frac{527.7 Q \log_{10} r_2/r_1}{S_1 - S_2} \dots \dots (3.4)$$

Equation 3.1 through 3.4 are useful for determining K or T of water bearing formations. These equations are also useful in predicting well yields, ordinary practice is to plot drawdown against $\log r$, then the quantity $(S_1 - S_2)/\log_{10} r_2/r_1 = \Delta S$ is simply the drop in piezometric surface for one log cycle of radi from the well centre. T is directly proportional to Q and inversely proportional to S , i.e., $T = 527.7 Q / \Delta S$.

3.2.2 Non equilibrium case

In 1935, Theis utilizing mathematical methods developed in the study of heat flow, proposed a means for analyzing the non steady flow of water into a well. Because the steady flow or equilibrium condition is a limiting case in well hydraulics rather than the rule, this advance has led to much greater understanding of well performance and to improved well engineering. It takes into account the water drawn from storage in the aquifer, as a result of unwatering in the water table case, or from volume changes associated with changes in pressure under pumping in the artesian case. An additional aquifer characteristics, the storage coefficient S , was introduced for confined aquifer condition.

When a well in an extensive confined aquifer is pumped at a constant rate, water is removed from storage within the aquifer due to decompression as the piezometric head is reduced. This reduction has to be progressive to enable a continuous release of water and the radius of influence must extend farther and farther with time. If it is assumed that there is no component of velocity in the vertical direction, the differential equation in cylindrical coordinates governing unsteady flow of water in a confined aquifer can be written as

$$\frac{\partial^2 h}{\partial r^2} + \frac{1}{r} \frac{\partial h}{\partial r} = \frac{S}{T} \frac{\partial h}{\partial t} \quad \dots \quad (3.5)$$

where T is the coefficient of transmissibility and t is the time since beginning of pumping. Theis obtained the following solution for equation 3.5 by assuming that $h = h_0$ at $t = 0$ and after

pumping is begun $h \rightarrow h_0$ as $r \rightarrow \infty$

$$s = h_0 - h = \frac{Q}{4\pi T} \int_u^\infty \frac{e^{-u} du}{u} \dots \quad (3.6)$$

$$\text{where } U = \frac{r^2 S}{4Tt} \dots \dots \dots (3.7)$$

and Q is the constant well discharge. Equation 3.6 is known as the nonequilibrium, or Theis, equation. The integral is a function of the lower limit and is known as an exponential integral and tables of its values have been published. It has also been called well function, $W(u)$, Rewriting equation 3.6 as

$$h_0 - h = \frac{Q}{4\pi T} W(u) \dots \dots \dots (3.8)$$

$$\text{and recalling } r^2/t = \frac{4T u}{S} \dots \dots \dots (3.9)$$

one may substitute these specific values to solve for S and T for the formation. The equation 3.6 can be expanded so that it becomes

$$h_0 - h = \frac{Q}{4\pi T} W(u) = \frac{Q}{4\pi T} \left[-0.5772 - \log_e u + u - \frac{u^2}{2.2!} + \frac{u^3}{3.3!} - \frac{u^4}{4.4!} + \dots \right] \quad (3.10)$$

3.3 Methods of determining Aquifer Constants by using Non-Equilibrium Test Pumping Data (10,14,33).

The following are the methods of determining the values of Co-efficient of transmissibility and co-efficient of storage.

1. Theis' non-equilibrium method
2. Jacob's modified non-equilibrium method.
3. Theis' recovery method
4. Chow's method
5. Method using distance-draw down data

The assumptions those are made in these methods are as follows:

- i. The aquifer is infinite in extent
- ii. The aquifer is homogeneous, isotropic and of uniform thickness
- iii. At the begining of pumping the piezometric surface and/or phreatic surface are horizontal
- iv. The aquifer is pumped at a constant discharge rate.
- v. The pumped well penetrates the entire aquifer, so that the flow towards the well is horizontal
- vi. The water removed from storage is discharged instantaneously with reduction of head.
- vii. The diameter of the pumped well is small, i.e. the storage in the well can be neglected.

THEIS' NON-EQUILIBRIUM METHOD

The eqn 3.8 can be written in the gallon-day foot system of unit as

$$h_o - h = \frac{114.6Q}{T} W(u) \quad \dots \quad 3.11$$

$$u = \frac{1.87 r^2 S}{Tt} \quad \dots \quad 3.12$$

Where,

$h_0 - h$ = draw down in feet

r = distance from pumped well to observation point
in feet

t = time after pumping started

The values of $W(u)$ versus u are plotted on logarithmic paper and this plotting is known as type curve. Values of draw down are plotted against values of r^2/t on logarithmic paper of same size as for the type curve. The two curve are then superimposed with their axes parallel and adjusted until a position is found by trial whereby most of the plotted points of the observed data fall on a segment of the type curve. An arbitrary point is selected on the coincident segment of the type curve. The values of $W(u), u, h_0 - h, r^2/t$ of the match point is used to determine T and S by the equation -3.11 and 3.12. The data used in this method is time draw down data.

Walton has suggested two important means for reduction of calculations.

(a) Draw down is plotted against t instead of r^2/t . In this case, the type curve should be turned over to obtain coincidence and a match point, but the results are identical.

(b) The match point is selected at the intersection of two major axes of the type curve.

It is important to note that in applying this method, as also all type curve methods, less weight should be given to the early part of the data, since these data may not be closely represented by the theoretical development on which the type curve is based. Initially there may be a time-lag between the decline of head and the release of stored water, and the well discharge may vary as the pump adjusts itself to the changing head. As the time of pumping increases the efforts of such factors are minimised.

JACOB'S MODIFIED NON-EQUILIBRIUM METHOD

Jacob pointed out that for small values of u only the first two terms of equation 3.10 are significant and $W(u) \simeq -0.5772 - \log_e u$. Substituting in equation 3.8 we get

$$h_0 - h = s = \frac{Q}{4\pi T} \left(\log_e \frac{1}{u} - 0.5772 \right) \quad \dots \quad 3.13$$

Taking into account the value of u from equation (3.7) for a well at a particular radial distance one can demonstrate from equation 3.13 that

$$s_2 - s_1 = \frac{Q}{4\pi T} \log_e t_2/t_1 \quad 3.14$$

or in geological survey units

$$s_2 - s_1 = \frac{264Q}{T} \log_{10} \frac{t_2}{t_1} \quad 3.15$$

If t_1 and t_2 are so selected that $t_2/t_1 = 10$ i.e., one log cycle apart, then equation, 3.15 gives.

$$T = \frac{264Q}{\Delta h} \quad \dots \quad \dots \quad 3.16$$

Where Δh is the change in draw down in feet or recovery over one log cycle. Similarly using geological survey unit we get from equation 3.13

At $h_0 - h = 0$, $t = t_0$

$$S = \frac{0.3T t_0}{r^2} \quad \dots \quad \dots \quad 3.17$$

Draw down $h_0 - h$ versus logarithm of time t , if plotted, forms a straight line. The slope of the line per log cycle, Δh , and the time reading where the line cuts the zero drawdown t_0 , is taken from the plotting and with the help of equations 3.16 and 3.17 values of T and S are calculated.

The non-equilibrium time-drawdown data is used in this method.

The straight line approximation for this method should be restricted to values of u less than about 0.01 to avoid large errors. The coefficient of storage cannot be determined with any degree of accuracy from data for the pumped well because the effective radius of the pumped well is seldom known and drawdowns in the pumped well are often affected by well losses which cannot be determined precisely.

THEIS' RECOVERY METHOD

If a well is pumped for a known period of time and then shut down, the draw down thereafter will be the same as if the discharge had been continued and a recharge well with the same flow were superposed on the discharged well at the instant the discharge is shut down. The residual draw down, $h_0 - h'$ where h' is the head during the recovery period, can be given by

$$h_0 - h' = \frac{Q}{4\pi T} \left[\int_{r^2 S/4tT}^{\infty} \frac{e^{-u}}{u} du - \int_{r^2 S/4Tt'}^{\infty} \frac{e^{-u}}{u} du \right] \quad 3.18$$

Where, t = time since pumping started

t' = time since pumping stopped

for small r and large t' , the exponential integrals can be approximated by the first two terms of equation 3.10 and the co-efficient of Transmissibility is given by

$$T = \frac{2.3Q}{4\pi(h_0 - h')} \log_{10} t/t' \quad \dots \quad \dots \quad 3.19$$

In this method for obtaining a convenient solution, residual drawdown $h_0 - h'$ should be plotted on linear scale against t/t' on a logarithmic scale. Then T , similar to Jacob's modified non-equilibrium method, reduces to

$$T = \frac{264Q}{h'} \quad \dots \quad \dots \quad \dots \quad 3.20$$

Where, T is in gpd/ft. Q in gpd, and h' = the change in residual drawdown in feet per log cycle of time.

To calculate S , the calculated recovery should be plotted against log of time after pumping stopped. Then, similar to Jacob's modified non-equilibrium method.

$$S = \frac{0.3 T t_0}{r^2}$$

This calculated S can have a smaller value than that obtained during pumping period.

The data used is time recovery data. An advantage of a recovery analysis of a pumped well is that it provides an easy check on pumping test results; also, it implies a constant discharge Q , which often is difficult to control accurately in the field.

CHOW METHOD

V.T. Chow developed a method of solution which avoids curve fitting and is not restricted in its application. Measurements of drawdown in an observation well near a pumped well are made. The observational data are plotted on a semilogarithmic paper as in the Jacob method. On the plotted curve an arbitrary point is chosen and the co-ordinate t and $h_0 - h$ are noted. A tangent is drawn to the curves at the chosen point and the drawdown difference Δh in feet per cycle of time is determined. $F(u)$ is computed from $F(u) = \frac{h_0 - h}{\Delta h} \dots \dots \dots 3.21$

The corresponding value of $W(u)$ and u are determined from a logarithmic plot of $F(u)$ versus u on which values of $F(u)$ are also given.

For $F(u) \Rightarrow 2.0$, $W(u) = 2.30 F(u)$ and u is obtained from Wenzel table.

The co-efficients T and S are then computed from

$$T = \frac{114.6Q}{h_o - h} \quad W(u) \text{ and } \dots \dots \dots 3.22$$

$$S = \frac{uTt}{1.87r^2} \quad \dots \dots \dots 3.23$$

METHOD USING DISTANCE DRAWDOWN DATA

If the drawdown at observation wells of known distances from the pumped well is known, the drawdowns can be plotted against logarithm of distances. The slope of the line is calculated. It is known that the slope of the line in Jacob's modified non-equilibrium method is exactly half of the slope of the line obtained in this method. So, Jacob's modified non-equilibrium formula can be used to solve for T and S . The formulae in this case are as follows:

$$T = 528 \frac{Q}{\Delta h} \text{ and } \dots \dots \dots 3.24$$

$$S = \frac{0.3 Tt}{r_o^2}$$

Where,

S = Coefficient of storage

T = Coefficient of transmissibility in gpd per ft.

t = time since pumping started in days

r_0 = intercept as zero drawdown of extendent straight line in ft.

The straight line method is popular largely because of its simplicity of application and interpretation; however, as pointed out by Cooper and Jacob "the method is not applicable in some cases and it supplements, rather than supersedes, the type curve method". They state further that "the method is designed expecially for artesian conditions, but it may be applied successfully to tests of nonartesian aquifers under favorable circumstances".

The straightline method is based on the fact that when u becomes small a plot of drawdown against the logarithm of time after pumping started or distance from the pumped well describes a straight line. A semilogarithmic graph of values of $W(u)$ and u does not describe a straight line until $u \leq 0.01$. Deviation from a straight line becomes apreciable when u exceeds about 0.02. Therefore, a plot of drawdown against the logarithm of time after pumping started or the logarithm of distance from the pumped well cannot describe a straight line until u becomes small (say less than 0.02).

The above examples refer to a confined aquifer, but the non-equilibrium equation can be applied also to unconfined aquifer when the drawdown is small compared to the total saturated

thickness. In the derivation of the equation it was assumed that the water released from storage by lowering the head was released instantaneously. In unconfined aquifers, however, the storage coefficient is equivalent to the specific yield, a value which is attained only after the dewatered portion of the aquifer has drained for some time. Hence, to apply the non-equilibrium equation in unconfined aquifers T is first determined and S is approximated by plotting values determined during the test period against $\log t$ and extrapolating the trend to an ultimate value. Jacob suggested that more accurate values of T and S are obtained by subtracting $(h_0 - h)^2 / 2 h_0$ from each drawdown.

3.4 Well Loss

The drawdown inside a pumping well is greater than that given by the equation 3.10. The excess drawdown, which is known as well loss, is caused by head losses due to strainer characteristics while the drawdown given by equation 3.10 is due to the head loss in the aquifer and it is known as aquifer loss. Hence, drawdown inside a well is equal to the sum of aquifer loss and well loss as illustrated in Fig. 6.18. Well loss is governed by tubewell fixtures, construction and condition of the tubewell. Well loss is an indicator of well performance.

METHODS OF DETERMINING WELL LOSS CONSTANT (C):

Values of well loss constant can be determined by the following methods:

METHOD - 1: Trial method as suggested by Rorabaugh depends on the following equation:

$$D_w = BQ + CQ^n \quad \dots \quad \dots \quad \dots \quad 3.25$$

Where,

D_w = drawdown at the well

Q = Discharge

C = Well loss constant

$$B = \frac{2.30}{4\pi T} \log_{10} \frac{2.25 Tt}{r_w^2 S} = \text{Formation loss constant}$$

r_w = Well radius.

From equation 3.25,

$$\frac{D_w}{Q} = B + CQ^{n-1} \quad \dots \quad \dots \quad \dots \quad 3.26$$

Rorabaugh suggested that $(D_w/Q - B)$ be plotted versus Q on logarithmic paper. Because B is unknown, it is necessary to assume various values of it. The correct value will be that giving a linear plot.

From a consideration of equation 3.26 it is evident that the slope of the line will be $(n-1)$, and that the value of $(D_w/Q - B)$ for $Q = 1$ will be equal to C . All unknown constants are thereby determined.

In plotting $(D_w/Q-B)$ against Q for the turbulent flow condition, the first assumption should be that $B = 0$. With good test data this should yield a curve that is concave upward. The assumption of larger and larger possible values of B will then eventually lead to the desired linear plot.

METHOD-2:

Jacob suggested that for equation 3.26 a value of $n = 2$ may be reasonably assumed. Then $D_w = BQ + CQ^2$ from which we get,

$$\frac{D_w}{Q} = B + CQ \quad \dots \quad \dots \quad \dots \quad \dots \quad 3.27$$

which is the equation of a straight line.

If we plot D_w/Q at equilibrium condition and at various discharge versus Q on normal graph paper, a straight line is obtained.

$$\text{At } Q = 1, \quad \frac{D_w}{Q} = B + C$$

and the value of B can be found from the graph paper (intercept on D_w/Q axis).

From these two conditions, the value of C can be determined.

METHOD-3:

As stated earlier if the equifer is fairly permeable and the production well can be pumped at rates exceeding 25 gpm the value of C may often be completed from the data collected during a variable rate well production test by using the following

equation

$$C = \frac{\Delta D_w^i / \Delta Q_i - \Delta D_w^{i-1} / \Delta Q_{i-1}}{\Delta Q_{i-1} + \Delta Q_i} \quad \dots \quad \dots \quad 3.28$$

Increment of drawdown ΔD_w are determined by taking the difference between the observed water level and the extension of the preceding water level curve. The value of C for one set of steps may vary from another set of steps. The average value of C is taken.

METHOD-4:

In this method the value of well loss constant C, is determined by using multiple step drawdown data. For each period of test an equation may be written as

$$\Delta S_i = B \Delta Q_i + C (Q_i^n - Q_{i-1}^n) \quad \dots \quad \dots \quad 3.29$$

$$\text{or } \frac{\Delta S_i}{\Delta Q_i} = C \frac{Q_i^n - Q_{i-1}^n}{\Delta Q_i} + B \quad \dots \quad \dots \quad 3.30$$

$$B = \frac{2.30}{4 \pi t} \log_{10} 2.25 T t / r_w^2 S \quad \dots \quad 3.31$$

Where, ΔS_i = increment of drawdown in the ith step which can be determined by taking the difference between the **observed** water level and the extension of the preceding water level curve after the respective change in discharge.

ΔQ_i = increment of discharge of the i th step. By plotting $\Delta S_i / \Delta Q_i$ versus $(Q_i^n - Q_{i-1}^n) / \Delta Q_i$ the equation can be solved graphically, the value of n ordinarily being determined by trial as that value giving the straightest line. C is then found from the slope of the line and B from its intercept. If the accuracy of discharge measurements, particularly at low rates, is not good enough to permit a determination of n , it is assumed to be 2.

In these cases the test should be run following a period in which the well has been inactive, beginning at as fraction of the capacity of the pump and increasing the discharge in steps, each of which is a fair fraction of the pump capacity. During each period of test the metered discharge should remain essentially constant.

During each period of the drawdown test, frequent readings of the drawdown be made, and all reading should be made accurately as far as possible.

Significance of well loss co-efficient

Small values of C denotes an efficient well. If C does not change during a step drawdown test it means that the well is stable. In newly constructed or not fully developed wells, the

value of C is affected by changes in pumping rates due to movement of sand and gravel under the influence of high rates of pumping. If the value of C for steps say 2 and 3 is considerably less than that for steps 1 and 2, ^(Eqn. 3.28) it is probable that development has occurred during the variable-rate well production test. A large increase in C with higher pumping rates indicate clogging of the production well screen or wall. If development during the pumping period is large, $\Delta DW_2 / \Delta Q_2$ will be greater than $\Delta DW_3 / \Delta Q_3$ and solution of Equation 3.17 will be impossible.

CHAPTER FOUR

WATER WELL DESIGN, CONSTRUCTION AND FAILURE

4.1 Water Well Design

4.1.1 Introduction

A well is a hydraulic structure which, when properly designed and constructed permits the economic withdrawal of water from a water bearing formation. Tubewell design should ensure an efficient and economical well with a service life of more than a decade, or period of run of 50,000 to 70,000 hours. Though conflicting design criteria have been enunciated from time to time by those working in the field of well design, a stage has now been reached when it is possible to design an optimum well for almost all the aquifer conditions met within nature provided the materials are available in hand.

Designing a water well involves selecting the proper dimensional factors for the well structure and choosing the material to be used in its construction. Good design aims to assure an optimum combination of performance, long service life, and reasonable cost. Selection of the design feature of a production well and a particular method of construction depends upon the quantity of water required, economic factors, the well's objective, and hydrogeologic conditions.

In designing a tubewell cost and technical factors must be properly analyzed. But it is not good engineering to use inadequate sizes of well casing and well screen, or to choose materials of inferior quality, merely to cut first cost. This only saddles the owner with higher pumping and maintenance costs, as well as reduced useful life of the well. Any additional investment for a properly designed, efficient well will, in long run, usually produce maximum economy.

A municipal well must usually be designed to obtain the highest yield available from the aquifer, and the highest efficiency in terms of specific capacity. These factors bear directly upon operating costs. Another important cost factor in these types of installation is the economic loss that may result from interruption of service in large water supply developments. Good design minimizes this danger by building into the water well the features that will assure long and trouble free life.

The well structure may be considered to consist of two main elements. One element is the part of the well that serves as a housing for the pumping equipment and as a vertical conduit through which water flows upward from the aquifer to the level where it enters the pump. This is commonly the cased portion of the well, although some of its length may be uncased where

the well is constructed in consolidated rock materials.

The other main element is the intake portion of the well. Since this is where water enters the well from the aquifer, the design of this element requires careful consideration of the hydraulic factors that influence well performance. This applies particularly to a well that derives water from an unconsolidated water-bearing sand. In such a case, a well screen is employed, and it functions as the intake portion of the well structures.

A properly constructed well screen allows water to enter the well freely at low velocity, prevents sand from entering with the water, and serves as the structural retainer to support the loose formation material.

The aim of both screened wells and gravel packed wells is to draw clear water from the aquifer without excessive head loss and at the same time to keep the aquifer material out. Both are developed in place. Development removes the finer material from the aquifer surrounding the well so that only coarser material is left adjacent to the screen. The aquifer material around the well becomes more uniform in grain size and holds back the finer material of the aquifer further beyond, so that it can not clog the screen. Of course,

when the well is gravel packed, much of the same purpose has been accomplished, although development is still beneficial.

The choice of whether a well is to be provided with a screen or with a screen with gravel pack depends primarily upon the effective grain size d_{50} (50 percent finer) and the uniformity co-efficient (d_{60}/d_{10}) of the aquifer material.

4.1.2 Gravel packed wells

Gravel packed wells are used under the following circumstances:

- a. When the aquifer is homogeneous, has a uniformity co-efficient of less than 3.0 and has an effective grain size less than 0.25 m.m.
- b. When well graded aquifers having large percentage of fines require to be stabilized in order to prevent their removal during pumping and consequent settlement of the overlying strata.
- c. When large-sized screen slots are to be used.
- d. When the aquifer is a thick artesian aquifer.
- e. When the aquifer is loosely cemented sand stone.
- f. When the aquifer is extensively laminated formation.

4.1.3 Well diameter

Choice of proper well diameter is important because it affects significantly the cost of the structure. The well may or may not be of the same diameter from top to bottom. After starting construction with casing of a given size, drilling conditions or other factors sometimes make it necessary to reduce the diameter at some depth and complete the lower portion of the well in a smaller size.

The well diameter must be chosen to satisfy two requirements:

1. The well casing must be large enough to accommodate the pump with proper clearance for installation and efficient operation.
2. The diameter of the intake section of the well must be such as will assure good hydraulic efficiency of the well.

In choosing the size of the casing, the controlling factor is usually the size of the pump that is expected to be required for the desired or potential yield of the well. The diameter of the well casing should be two sizes larger than the normal diameter of the pump. Under no circumstances should it be chosen less than one nominal size larger than the pump bowls.

4.1.4 Well Depth

The expected depth of a well is usually determined from the log of a test hole, from logs of other nearby wells in the same aquifer, or during the drilling of the production well. Generally, a well is completed to the bottom of the aquifer. This is desirable for two reasons.

- i. More of the aquifer thickness can be utilized as the intake portion of the well, resulting in higher specific capacity.
- ii. More drawdown can be made available permitting greater well yield.

One exception to this rule is made where the well screen is centered between top and bottom of the aquifer, a practice sometimes followed to make more efficient use of a given length of screen in a uniform artesian aquifer.

Another situation that requires departure from the basic rule is where water of poor quality is found in the lower part of an aquifer. In such a case, the well should be completed to a depth which will avoid the undesirable water and obtain the better quality water that is available. Any part of the hole that may have been drilled into the part of the aquifer containing poorer quality water should be

carefully backfilled so that this water will not migrate upward when the well is pumped. The backfill material should be relatively impermeable. It should be tamped solidly so that it will support the well screen without settlement.

4.1.5 Well screen length

To reduce entrance velocities and the effects of partial penetration of the aquifer by the production well as long a screen as possible should be used under water table conditions. Optimum production-well specific capacity and yield are generally obtained by screening the lower 0.5 to 0.3 of the aquifer. The pumping level must be kept above the top of the screen, thus the longer the screen, the less the available drawdown. The factors that affect the choice of screen length include : the open area per foot of screen, the character of the aquifer, the cost of the screen, the desired yield from the production well, and the desired service life of the production well. Screen length selection is often a compromise between the factors of cost and hydrogeology.

The screen length is based in part on the effective open area of a screen and an optimum screen entrance velocity. When a screen is placed in an aquifer, sediment will settle around it and partially block the slot openings. The amount

of blocking depends largely upon the shape and type of slot and the shape, size and sorting of the aquifer or pack material. On the average about one half of the open area of the screen will be blocked by aquifer material. Thus the effective open area averages about 50 percent of the actual open area of a screen.

To ensure that the production well has a long service life by avoiding migration of fine materials toward the screen and subsequent clogging of the well wall and screen openings, screen length is based on velocities between 2 and 12 fpm. In general aquifers of low permeability are composed of finer grained material than aquifers of high permeability. The possibility of clogging openings depends in large part upon the grain size of the finer aquifer materials, and there is a relationship between optimum screen entrance velocities and the coefficient of permeability of the aquifer. A study was made of several actual case histories of production well failures due to the partial clogging of the well walls and screen openings resulting in the values in Table 4.1.

The permeability of the aquifer is determined from aquifer tests. To prevent rapid clogging of the screen and formation, the length of the screen for a natural pack production well is selected using Table 4.1 and the following

equation:

$$S_L = \frac{Q}{7.48A \cdot V} \dots \dots (4.1.1)$$

Where

S_L = Optimum length of screen, in feet

Q = discharge of production well, in gpm

A = effective open area per foot of screen, in
square feet

V = optimum screen entrance velocity, in fpm.

The same procedure is followed in selecting the optimum length of screen for an artificial-pack production well except that the average of the permeabilities of the aquifer and pack is used to determine the optimum entrance velocity.

4.1.6 Screen open area

The open area of the screen should be so selected that the well loss is small. Corey observed that little or no increase in well efficiency results from actual open areas greater than 15 percent of the total surface area of the screen. In actual practice sometimes larger open areas are found desirable in order to hold the screen entrance velocities within optimum limits.

In a study well losses were seen to depend upon the following parameters:

- i. A dimensionless screen parameter $C_s L/D$
- ii. A dimensionless friction factor fL/D
- iii. Co-efficient of permeability of the aquifer
- iv. The head loss through the aquifer

Where, $C_s = 8 \sqrt{2} C_c C_v A_p$ = Screen Co-efficient,
dimensionless

C_c = Contraction Co-efficient, dimensionless,
Value for square slots = 0.626

A_p = Ratio of the slot area to the total area
of the screen, dimensionless.

L = Screen length, in meter

D = Screen diameter in meter

f = Darcy - weisbach resistance Co-efficient.

C_v = Velocity co-efficient dimensionless,
usual value = 0.97.

It was found that for low values of the screen parameter $C_s L/D$ the well loss is very high and as the value of the screen parameter increases, the well loss gets progressively reduced, till a value of the screen parameter equal to 10 is reached beyond which the well loss does not reduce further. In nature about half of the slot area of a screen gets blocked by aquifer grains. In course of time the blockage may further increase due to incrustation and rearrangement of grains. Hence in actual practice the value of $C_s L/D$ was recommended to be kept more than 60, i.e.,

$$\frac{8\sqrt{2} C_c C_v A_p L}{D} > 60 \quad \dots \quad \dots \quad (4.12)$$

In wells the ratio L/D is generally larger than 60. If this value is substituted into the screen parameter, putting $C_s = 8\sqrt{2} C_c C_v A_p = 8\sqrt{2} \times .626 \times .97 A_p$, the inequality of equation 4.1.2 above is satisfied if the value of A_p is at least 0.15. Hence the open area should generally not be less than 15 per cent of the total surface area of the screen.

4.1.7 Screen diameter

The effect of the screen diameter on the aquifer loss is not very large, because in the well equation the logarithm of the well radius is used. Diameter of the intake portion of a well can be varied without greatly affecting specific

capacity or well yield. Doubling the diameter of the intake portion the well screen can be expected to increase the specific capacity only about 10 percent, other things being equal.

However, the well diameter influences the well loss to a large extent, and has to be selected so that the total loss i.e., the aquifer loss plus the well loss, is kept minimum in conjunction with the cost of the screen, the boring and the pumping costs required. If the well diameter selected is too large, the cost of installation would be high but the running costs would be low. If the well diameter selected is too small, the cost of installation would be low but the head loss would be high resulting in high running costs.

Screen diameter is selected to satisfy an essential basic principle enough total area of screen openings must be provided so that the entrance velocity of the water will not exceed the design standard. Screen diameter is a factor that can be varied after the length of the screen and size of the screen openings have been selected.

Laboratory tests and field experience show that if the screen entrance velocity is equal to or less than 0.1 ft per second, the following will result:

- i. Friction losses in the screen openings will be negligible;
- ii. Rate of incrustation will be minimum;
- iii. Rate of corrosion will be minimum.

When the calculated entrance velocity is less than 0.1 ft per sec say for example, 0.05 ft per sec the screen diameter may be reduced somewhat. A well screen with greater per cent open area offers a decided cost advantage where different types of screens are compared at the same entrance velocity. Screen diameter should be as large as possible, however, to keep the entrance velocity safely below the 0.1 ft per sec limit.

The foregoing statements assume that the pump will be set above the well screen and that the head loss associated with the vertical flow of water upward through the screen will be small (which is also usually true).

4.1.8 Well screen slot openings

For naturally developed wells, the optimum slot size is selected principally on the basis of the grain size distribution of the aquifer, but hydrogeological condition also play an important role. Slot size are selected which will allow a

definite proportion of fine materials to pass into the production well. The fines are removed from the production well with a bailer, and such methods as surging are used to draw the finer material out of the aquifer around the production well and thereby provide improved permeability in the immediate vicinity of the screen.

In gravel packed wells, with homogeneous aquifer, the slot size should be equal to the d_{10} size of the pack material so that it can retain 90 percent of the gravel pack. This criteria is accepted by all authority except U.S. Bureau of reclamation. The latter recommended that the slot size should be half the d_{85} (85% Finer) size of the pack material. The slot size obtained by this criteria is also almost equal to the d_{10} size of the pack material.

A well sometimes encounters several layers of sand and gravel having different grain size and gradations. If the d_{50} grain size of the Coarsest aquifer is less than four times the d_{50} grain size of the finest aquifer, the slot size or the pack should be based on the finest aquifer. If the difference is more than four times, the slot size or the pack should be tailored to individual layers.

In formations where the bottom layers are coarse and the top layers are fine, it is necessary to keep the slot size in this portion (bottom layers) in accordance with the pack designed for the upper finer aquifer so that if the finer gravel moves down it is retained on the slot.

4.1.9 Design of Gravel Pack

The relationship between the aquifer grain size and the pack grain size has been determined on the basis of practical experience and laboratory experiments by Various agencies concerned with water wells. All have agreed that different criteria are required for uniform aquifer material ($Cu \leq 2$) and for graded aquifer material ($Cu > 2$). The most widely used, pertaining to uniform pack material, are summarised below.

1. U.S. Bureau of Reclamation adopted the following criteria

(a) Uniform aquifer ($Cu \leq 2$)

$\frac{d_{50} \text{ of pack}}{d_{50} \text{ of aquifer}}$ should lie between 5 and 10.

The average slope of the filter material gradation curve should be the same as that of the aquifer material.

(b) Graded aquifer ($Cu > 2$)

$\frac{d_{50} \text{ of pack}}{d_{50} \text{ of aquifer}}$ should lie between 12 and 58

and

$\frac{d_{15} \text{ of pack}}{d_{15} \text{ of aquifer}}$ should lie between 12 and 40.

ii. U.S. Department of Agriculture, after Kruse's model experiments, laid down the following criteria

(a) Uniform aquifer

$$\frac{d_{50} \text{ of pack}}{d_{50} \text{ of aquifer}} = 9.5$$

(b) Graded aquifer

$$\frac{d_{50} \text{ of pack}}{d_{50} \text{ of aquifer}} = 13.5$$

iii. Johnson recommended as follows

(a) Uniform aquifers

$$\frac{d_{30} \text{ of pack}}{d_{30} \text{ of aquifer}} = 4$$

(b) Graded aquifers

$$\frac{d_{30} \text{ of pack}}{d_{30} \text{ of aquifer}} = 6$$

iv. The Central Board of Irrigation and Power recommended after an extensive series of laboratory experiments that the following criteria ensure least sand movement and loss of head through the gravel pack

(a) Uniform aquifer ($C_u \leq 2$)

$$\frac{d_{50} \text{ of pack}}{d_{50} \text{ of aquifer}} \text{ should lie between 9 and 12.5}$$

Two lines approximately parallel to the central portion of the aquifer gradation curve should be drawn through the ~~two~~ points thus obtained and the gravel used should lie within the zone thus defined

(b) Graded aquifer ($C_u > 2$)

$$\frac{d_{50} \text{ of pack}}{d_{50} \text{ of aquifer}} \text{ should lie between 12 and 15.5.}$$

In order to obtain the gradation curve of the gravel pack, two lines should be drawn through the two points thus obtained such as to give a uniformity co-efficient of 2. The gravel used should lie within the zone thus defined.

Thickness of Gravel Pack

Since the design theory of gravel pack gradation is based on Mechanical retention of the formation particles, a

pack thickness of only two or three grain diameter is all that is actually needed to retain and control the formation sand. However, it is impractical to place in a well a gravel pack only a fraction of an inch thick and expect the material to completely surround the well screen. To ensure that an envelope of gravel will surround the entire screen, therefore, a thickness of 3" is the minimum that is considered practical for installation in the field. The upper limit of this thickness is usually taken as 8".

A thicker envelope does not materially increase the yield of the well and thickness, in itself, does nothing to reduce the possibility of sand pumping, because, the controlling factor is the ratio of the grain size of the gravel pack material to the formation material. Too thick a gravel packing can make final development of the well more difficult.

Walton recommended a thickness of gravel pack of 15 to 23 cm. U.S. Bureau of Reclamation suggested a range of 10 to 23 cm. Johnson recommended a thickness of 7.6 cm with a maximum limit of 20 cm. U.P. Irrigation Research Institute determined after model experiments that the minimum thickness necessary to keep out sand movement is 12.5 cm. It is suggested that the gravel thickness should preferably be between 13 and 20 cm.

4.1.10 Selection of Materials

Three factors govern the choice of material of which the screen should be fabricated. These are:

- (a) Mineral content of the water
- (b) Presence of bacterial slimes
- (c) Strength requirement of the screen

The mineral content of the ground water is obtained from a chemical analysis. A study of the analysis usually can reveal if the water is corrosive or inscrusting or both.

4.2 Construction of Tubewells

4.2.1 Introduction

The construction of a tubewell consists of a number of operations such as drilling, well fixture installation, development and commissioning of the well. Each of these operations is described briefly in this article.

4.2.2 Methods of Drilling Deep Wells

Most large, deep high-capacity wells are constructed by drilling. Three basic methods of construction are employed (1) cable tool (also known as percussion or standard (2) hydraulic rotary, and (3) reverse rotary. Each method is particularly suited for drilling in certain formations and not in others. Recognizing this fact, many experienced drillers arrange their drilling rigs to be able to switch methods as required for drilling in formations of a variable nature. The construction procedure of a successful well is dependent upon local conditions encountered in drilling; hence each well should be treated as an individual projects.

4.2.3 Installation of Screen

4.2.3.1. Alignment of well :

Before installation of the screen it is essential to check the hole for verticality. A bore containing kinks and bends may create difficulties in installation and operation of the pump. It may cause undue wear on the pump shaft, bearings and casing and in a severe case might make it impossible to get the pump in or out. In the case of deep well turbine pumps, for casings less than 35 cm in diameter, the verticality should not deviate more than 10 cm per 30 m depth and the deviation should be in one direction and plane only. If a suction air lift or submersible pump is installed, the alignment is not so important but it is still desirable to follow the above criterion. Normally a well should be tested for verticality after drilling is completed. However, in the case of gravel packed wells, verticality should be tested after installation of the well assembly but prior to commencement of gravel filling.

The alignment of a tube well should be tested by use of heavy plunger 6 mm smaller in diameter than the inside diameter of the well casing. The plunger is suspended by a line running over a pulley at least 3 m above the top of the well so that the wire passes through the centre of the top of the casing.

The plunger is lowered in steps of 3 m and deviations of the line from the centre of the casing are observed. The drift at any depth is given by the deviation multiplied by the length of the line and divided by the height of the pulley above top of well casing .

If the eccentricity of a bore is seen to be more than that permitted, it can be corrected by loosening earth on one side of the pipe and forcing the pipe back by applying jacks on the other side. If the hole is badly eccentric, it may have to be rebores.

4.2.3.2. Installation of well screen:

For wells drilled by the cable tool percussion method, the pull back method is employed for installation of well screen. The well assembly comprising the casing pipe and the screen is lowered into the well and after this the casing pipe is extracted. Rotary holes are not cased and so the well assembly can be lowered directly. Centering guides (short iron strips) are attached to the screen to centre it in the hole. It is desirable to suspend the assembly from the top instead of allowing it to rest at the bottom till filling up of gravel, otherwise it may buckle due to its excessive slenderness. If a hole has to stand for some time, a casing pipe is lowered till such time that the well assembly is lowered. The casing pipe is extracted after lowering of the assembly.

Placement of gravel pack around a well has to be done carefully to avoid segregation or bridging. A 5 cm diameter pipe is lowered in the annular space that is to be gravel packed. Gravel is fed through a hopper provided at the top. The feeding pipe is gradually pulled up as the annular space is filled. Water is fed into the pipe so as to aid in the downward movement of gravel. For very deep wells, the reverse circulation system of gravel feeding is more suitable. A pipe is lowered inside the well assembly and is connected to a pump. A circulation of water is established downwards in the annular space and upward through the lowered pipe. Gravel is fed in the annular space and is carried downwards by the force of the water.

4.2.4 Development

Development is essential in order to obtain an efficient and long lasting well. The fundamental purpose of development is to cause reversal of flow through the screen openings so that formation particles are loosened, the fines are drawn out, and the remaining coarser particles get rearranged. Development results in the following advantage:

1. Creation of a highly permeable zone around the well screen. Development pulls out and removes finer material from the aquifer thereby cleaning out, opening up and enlarging

passages in the vicinity of the well screen so that water can enter the well more freely. In screened wells development is essential in order to create a coarse highly permeable zone around the well screen. In the case of artificial gravel packed wells development helps in removing the finer particles of the aquifer through the gravel pack. The efficiency of development for increasing the permeability of the surrounding aquifer is best when the aquifer sand is non-uniform.

ii. Stabilisation of the sand around the well so as to ensure sand free delivery of water. In a zone just outside the hole, development removes all particles smaller than the screen slot size and only the coarsest material is left in place. Further away the effect of development is progressively diminished and the aquifer material progressively grades back to the original form of the water bearing stratum. By creating this succession of graded zones, the formation is stabilised so that no further sand movement would take place. In case of gravel packed wells all particles which can pass through the pores of the gravel pack are removed. It has been observed that fine particles resist movement due to the characteristic mechanism of bridging of the pores. They do not move from their position even on increasing the velocity of flow through the aquifer by over-pumping. Reversal of the direction of flow, causes such particles to move after breaking the bridges if the reversal is followed by an inward flow, the fine particles

move towards the screen and into the well.

iii. Removal of clogging of the sides of the hole caused by the drilling operation. Every method of drilling plugs the pores of the water bearing formation around the bore hole to a greater or smaller extent. In the direct rotary method, where drilling mud is used, some mud deposits on the sides of the bore and seals the same. In reverse circulation drilling water is lost into the formation due to excess fluid pressure that must be maintained in order to keep the hole open. In this process the silts, clays and fine sandy material picked up by the drilling water from the formation are deposited on the walls of the hole. Though this material is comparatively easier to remove than the mud cake formed in the rotary method, it has nevertheless to be completely removed. In case of drilling by pipe driving, a reduction of porosity is caused in the surrounding aquifer due to compaction and vibrations. The surrounding aquifer has therefore to be loosened.

Development should be started slowly and gently and as development progresses the energy should be increased in steps to the full capacity of the equipment. Development can be started at the top of the screen and worked down or started at the bottom of the screen and worked up. Some advocate the use

of the former procedure, particularly in the case of development by a surge block, probably in order to minimise the possibility of material coming in above the surge block, which may result in 'sand locking' of the block. However, the upper layers compacted as a result of development may have a tendency to bridging and cavities may form around the screen when the lower formations are being developed. By starting development at the bottom of the screen the compaction takes place as work progresses upwards so that the overlying material can move downwards without much possibility of bridging, and should a bridge develop, the development action would usually break it up.

The following methods of development listed in order of their effectiveness, are commonly used:

- i. High velocity jetting,
- ii. Surging,
- iii. Backwashing,
- iv. Over pumping,
- and v. Acid treatment.

4.2.4.1 High velocity Jetting

The most recently improved method of development is high velocity jetting. The device used is a tool that concentrates a fluid jet stream on the face of the bore hole and

penetrates a short distance beyond into the aquifer. Fluid leaves the jet nozzle with a velocity of at least 150 fps. When such a force is strikes, it breaks down dense compacted materials, rotary drilling muds, and even some chemical precipitaty, and rather violent localized agitation is set up. Fine materials are dispersed in solution, and these can be parged by pumping the well at the time jetting is taking place. The rate of pumping is 15 to 20 percent greater than the rate at which fluid is applied at the jets, which keeps water in the formation moving toward the well. Pumping is easily carried out by the air lift method or if the static level is near surface, by a suction lift type pump.

The jets are rotated at elevations from 6 inch to 1 ft apart throughout the water entry area of the well. Several passes are made vertically throughout the entry section until the pump discharge is clean.

Chemical additives are often used where silt, clays or drilling muds are known to be present. Wetting agents used with hexametaphosphates in concentrations of 5 to 6 lbs/100 gal. of jetting fluid have been shown to be particularly effective. In well reconditioning, where biological or chemical precipitation has resulted in plugging, chlorine and acids have been used with considerable success.

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4.2.4.2 Surging

Surging is probably the most frequently employed development method. It may be used as part of the clean up procedure after explosive work, in conjunction with chemical agents, or as the sole means of moving formation fines into the well bore. In its simplest form, surging employs a plugger on the tool string which is moved up and down in the well casing to cause flow first in one direction and then the other at the well face. The result is that fine materials settle into the bottom of the well and are bailed out as they accumulate. The surge block used either may be a solid and tight fitting or may be constructed with a check-valve-type arrangement that give a strong inward surge as it is raised in the well and a weaker backwash action as it is lowered. When the static water level is near the surface, the valved surge may actually pump the well while surging and thereby remove small suspended fines as they enter the well.

Surging may also be accomplished by using compressed air or alternate pumping and backwashing with water. With air, provision is usually made to close in the top of the casing so that air pressure can be used to drive the water in the well bore out through screen or bore face. Then, by proper valving, water can be rapidly pumped by air lift to remove fines. Care must be exercised not to drive air into the formation because it may collect in pockets and reduce the aquifer production by air

locking. Also, if the surging is too violent, water and air mixtures may tend to move upward along the outside of the well casing and rupture through to the surface. Such ruptures cause many problems and may result in loss of the well.

Surging with water is usually accomplished by rapid on off operation of a pump. The pump is turned on until water just starts to be discharged at the surface, and then it is stopped, allowing the water to wash back down and out of the well face. When the cycle is repeated several times, the pump is allowed to operate until the water is clear. Unless the static level is a great distance below ground level and the pump employed is of much greater capacity than the one to be used for ordinary operation, the degree of development attained is not very satisfactory.

Backwash surging with chemical solution has been applied with a greater degree of success. The most effective procedure, requires a large tank for holding the mixed solution pumped from the well and adequate piping to permit rapid dumping back into the well when the pump is stopped. For the best effects, the rate of return to the well should be greater than the rate at which the well recovers, which is usually quite difficult to accomplish.

4.2.4.3 Backwashing

In this the surging effect or reversal of flow required to develop the formation is produced by alternately lifting the water to the surface by a pump, and then stopping the latter so that water in the column pipe falls back into the well. This produces an alternate inflow and outflow through the screen openings. After a few cycles of surging, the well is pumped to remove any accumulated sand. Back washing method generally used is not very effective because the surging effect produced by this is not vigorous enough to obtain good result. And by overpumping the aquifer is only partially stabilized and creation of a high permeability zone around the screen is not fully achieved because many sand grains may be left bridged in the formation. Furthermore, in general artificial gravel packing poses a special problem in well development which is enumerated below in few lines.

When a well is drilled for artificial gravel packing, a thin skin of relatively impervious material is plastered on the wall of the hole as has been described. When the gravel has been placed around the well screen, this skin or wall cake becomes sandwiched between the gravel and the face of the natural formation. The principal object of the development work is to break up and remove this material that is caught between the gravel and the wall of the hole. No matter what

method of drilling is employed, the thin layer of material sandwiched at the outer face of the gravel pack must be removed. The presence of the gravel envelope creates some difficulty in accomplishing this job. The thickness of the gravel pack and the grading of the material used for the gravel pack both have a considerable effect on what can be done by development to bring the well to maximum efficiency. Surging operation are less effective than for wells that are designed for natural development. One of the reasons for this is that a properly designed gravel pack is so permeable that there is more tendency for water to slosh up and down in the gravel envelope rather than to move into or out of the natural formation at places where it may be partially clogged. The thinner the gravel pack the surer is the driller is about removing of the finer sand, silt and clay for proper development. Polyphosphate dispersing agent effectively assist removal of the silt and clay.

4.2.4.4 Overpumping

In this the well is pumped at a much larger rate than its ultimate discharge until the sand discharge becomes negligible. In this method the aquifer is only partially stabilized and creation of a high permeability zone around the screen is not fully achieved because many sand grains may be left bridged in the formation. It requires a pumping unit much larger than that ordered for the well, which may not

always be available. AWWA manual of water supply practices stated regarding the high-rate pumping procedure of development that "Although this procedure has been satisfactory in many consolidated formations, it is not usually adequate where a screen is used. It should be noted, however, that regardless of what other development procedures are utilized, Over pumping should be accomplished as the final step to clean up the well and to demonstrate that the performance will be satisfactory at a lower rate".

4.2.4.5 Acid Treatment

In some cases a compound containing polyphosphates is used along with a method of development in order to remove the electrochemical bonds that hold the clay particles together. It is an effective way to remove clay but is expensive and the required acid is difficult to procure.

4.2.5 Commissioning of the well

After the development is complete, the well may be overpumped to verify if the discharge is sand free. Recommended standards for sand content are 20 ppm for municipal and industrial use, 50 ppm for sprinkler irrigation and 150 ppm for other irrigation uses. Before testing, the well should be permitted to recover at least for the period of overpumping. The well should then be pumped in four or five approximately equal steps. The first step should be run until a plot of the draw-

down versus log time fall on a straight line over a minimum 2-hour period. Succeeding steps should be run for the same length of time. The resulting data are used to determine the pump setting for the design capacity and similar data are collected annually and analysed to evaluate well performance.

4.3 Failure of Tubewells

4.3.1 Introduction

A tubewell is expected to give good service for around 50,000 to 70,000 hours. Some tubewells, however, fail much earlier. The most common causes of such failures are:

- i. Incorrect design, for instance, use of incorrect size of screen, gravel pack.
- ii. Poor construction, e.g., the bore may not be vertical, the joints may be leaky, slots may be non-uniform in size, etc.
- iii. Depletion of ground water, in such cases wells can generally be pumped with considerably reduced discharge.
- iv. Collapse of the screen or the casing due to movement or subsidence of the soil. In unconsolidated formations this type of failure often occurs in wells used for dewatering foundations if the screen and casing are too weak or are not rigid enough.
- v. Inadequate development

- vi. Chemical action of water to produce incrustation or corrosion of the screen.

The situations described in (i) to (iv) above can generally not be easily corrected.

4.3.2 Inadequate Development

In adequate development means that the aquifer was not flushed back to the extent that its fines were removed, and a good backgrading was not achieved so as to retain the fines which could not be flushed. These fines progressively move out with pumped water. Cavities are created due to this loss of material, turbulence pockets are generated, and sand movement increases with time. The aquifer or the gravel pack material slips vertically downwards to fill these cavities. In this process the graded filter effect is lost fully or partially, excessive sand pumpage begins and the screen may choke resulting in failure of the well. There have been cases in which the entire pump house along with the motor foundations subsided by a few metres into the ground due to formation of such cavities.

In the early stages the trouble can sometimes be corrected by re-developing the well and feeding around the screen sand or gravel pack material, 90% of which is retained by the screen.

4.3.3 Incrustation and Corrosion

Incrustation or corrosion occurs due to the chemical action of water on the screen or the casing. Incrustation is a condition of clogging of the water bearing formation just outside the well bore and clogging of the openings in the aquifer or the well screen. This occurrence results from extraneous material accumulating or being deposited in the openings or pores of the water bearing sand and on the surface of the well screen.

Corrosion is a chemical action upon metals that results in their being "eaten" away by the water. Incrustation, on the other hand, occurs when certain undesired materials collect in and about the screen openings or in the voids of the water bearing formation. Incrustation, in most cases, is caused by factors different from those that cause corrosion.

Incrustation is usually hard and cement like though in cases it may be soft and gummy. It clogs the screen openings and pores of the aquifer due to which the yield of the well is reduced and pumping costs mount up. Corrosion, on the other hand, leads to excessive discharge and sand pumping.

Although incrustation and corrosion are essentially opposite in effect, they can occur together under certain conditions. The products of corrosion of iron can cause

incrustation. For example, bimetallic construction of pipe base screens promotes corrosion of the perforated pipe. The iron oxide, or rust like material, from the corrosion of the steel pipe then plugs the openings in the bronze or stainless steel jacket.

CHAPTER FIVE

STUDY AREA AND DATA COLLECTION

5.1 Study Area

Wells owned by Dhaka WASA in side Dhaka city have been selected for the present study. Study area is therefore more or less Dhaka city area, which is gradually expanding .

The built-up area of Dhaka city is at elevation of 20-25 feet above sea level, although many low-lying flooding areas are scattered throughout the Metropolitan Dhaka from which fill materials have been taken to raise the built-up area. At the time of the partition and birth of Pakistan in 1947, Dhaka became the capital of the East Pakistan. In the last three decades, it has experienced much physical growth. In the 1950, Dhaka grew to include Tejgaon, Mohakhali, Gulshan and Bannani. In the late 1960, its boundaries were expanded to incorporate such areas as Gulistan, Topkhana, Motijheel, Ramna, Moghbazar, Dhanmondi, Mohammedpur and Mirpur. After the liberation war in 1971, Dhaka's role as capital of the new nation further stimulated its growth as commercial and industrial center.

WASA presently operates about 100 tubewells in the study area. A check list regarding the existing WASA tubewells is given in Table 5. The well depth, length of housing,

blind pipe and strainer with diameter are shown in this Table. In fig.1 the location of the tubewells are shown. The history of some of the WASA well installation has been collected from the authority.

5.2 Types of Data Collected

Following data regarding the selected deep tubewells have been collected for the present study from different offices of Dhaka WASA and WDB.

- a. The location of the deep tubewells.
- b. The design and construction procedures of deep tubewells as used by Dhaka WASA.
- c. The recorded lithology of aquifer at different tubewell sites.
- d. The sample of aquifer materials at interval of 5 ft depth, bailed out at different well site.
- e. The sample of gravel packing materials as used at different sites.
- f. Total length and diameter of boring, blind pipe, and of housing pipe/, strainer of all the existing tubewells of Dhaka WASA.
- g. Characteristic features such as percent open area, slot size, material etc. of strainer, housing pipe and blind pipe.

- h. Available information regarding static ground water level as observed in different tubewells inside Dhaka city.
- i. Available information regarding the ground water withdrawal for Dhaka since 1964 till date.
- j. Specific capacity of the existing WASA wells.
- k. Test pumping data of different wells just after installation of the wells.

Besides collection of above data the actual field procedures of construction of deep tubewells as adopted by the working contractor of Dhaka WASA was observed carefully and important features were noted.

5.3 Determination of grain size distribution of aquifer and packing materials

Detailed sieve analysis of the aquifer samples obtained during drilling of a number of wells at different area of Dhaka city, and the packing material used at the working site was done to know the characteristics of aquifer and packing materials. The results provide the basis for decision about several factors controlling the design of a screened well.

The Hydraulics and River Engineering Laboratory of the Department of Water Resources Engineering of Bangladesh University of Engineering and Technology, Dhaka, provided required facilities for sieve analysis.

The ASTM standard sieves, the automatic electric oven, balance sensitive upto 0.1 gm and the electric vibrator were used in this analysis. The standard and established procedures were used for the sieve analysis. The results of sieve analysis were plotted on semilogarithmic graph paper with percent finer as ordinate and size of sieve opening as abscissa. One curve for each sample taken at 5 ft. interval of aquifer depth from each well was drawn to get a clear picture of the stratification of the aquifer materials. From each ^{of} these curves d_{10} , d_{30} , d_{50} , d_{60} size were noted and the uniformity co-efficient C_u was calculated.

5.4 Determination of aquifer constants and well loss constant

Jacob's modified non-equilibrium method was used for determining the value of co-efficient of transmissibility and co-efficient of storage. The well loss constant was determined by using multiple step drawdown data. Data from observation wells also was used to determine the formation constants.

CHAPTER SIX

DATA ANALYSIS, RESULTS AND DISCUSSIONS

6.1 Introduction

To study the performance of the tubewell and aquifer characteristics inside Dhaka city the related informations have been collected from the concerned departments and analyzed. The study shows that the design history of WASA wells over the past 30 years has developed in response to the gradually increasing municipal demand for the easily obtainable ground water and the increasing stress on the aquifer supplying the ground water. Dhaka PHE Directorate began installing water wells in Dhaka city in 1949, and for the next 10 years the department installed on an average one tubewell each year. Since 1959, PHE and later on WASA after its formation has been installing wells at an average rate of about 3 to 4 wells per year (Fig.6.2). During the period 1949-1959 wells were drilled and constructed to depths between 200-350 feet using 70-100 feet length of slotted brass screens. The pump chamber was 10-12 inches in diameter and about 70 feet deep. Between 1959-1969, the well depth was increased to 240-450 feet using 100-150 feet of slotted brass screens. Depth of pump chambers during this period was increased to 90-110 feet and its diameter to 14 inches (Fig. 6.3 and 6.4). From 1969 WASA wells has been drilled progressively deeper ranging from 330 feet

to 540 feet and WASA has been using a standard and more or less fixed design parameters and construction procedures for tubewells inside the Dhaka city.

6.2 Design and Construction Procedure

Since 1969 WASA has been installing gravel packed wells drilled by reverse rotary method. As per WASA's design the following criteria is used in selecting gravel size for the pack material:

<u>Screen size</u>	<u>Percent by weight retained by the screen sizes specified</u>
$\frac{3}{8}$ "	0
4	20 - 30
8	40 - 60
16	20 - 40
Pan	0 - 5

Other design parameters are as follows:

Thickness of the gravel envelope	= 7 inches
Slot size	= 50/1000 inches
Strainer length	= 120 to 200 feet
Strainer diameter	= 8 inches
Screen open area	= 30 square inches (minimum) per foot run of the strainer.

The designed diameter of the well is however not same from top to bottom. The diameter of housing pipe as used now is 16"-18", and the diameter of blind pipe and strainer is 8". The designed borehole diameter is 22 inches and it is drilled to the depth specified by Engineer during drilling operations, but not exceeding 600 feet. The housing and blind pipe are made of mild steel and the strainer of stainless steel or brass. The housing pipe length as in use now is 160 feet.

The WASA wells, as stated earlier, are drilled by reverse circulation method. The concerned contractors is expected to drill the hole in one continuous operation. The contractor, is instructed to keep the drilled hole full of water from the start of drilling through completion of the gravel shrouding operation.

Instruction is also given to keep an accurate log of material encountered during drilling to know the size and type of material at different depths. A minimum of half pound of representative sample of each material encountered during drilling is generally collected at each change of formation and also at five feet depth intervals. Each sample is placed in a clean compartmented sample box which is

constructed in such a way that the samples does not get mixed. The sample are placed in sequence and leveled to identify their source properly. As soon as the boring operation is complete, the concerned Engineers check the log of material along with the representative sample, and depending upon their experience they select the most potential stratum of the aquifer and the strainer is placed accordingly in that regions of the aquifer.

Centralisers are generally installed in the 8" casing pipe and the screen at an interval not exceeding 30 feet. The centralizers are made of mild steel bars not less than $\frac{3}{8}$ " diameter or $\frac{1}{8}$ " x 1" strap steel. Centraliser is to provide a well clearance of not more than one inch in a 22 inches hole. A bail plug 5 feet long and made of the same material as the 8 inches blind pipe and having its bottom end securely sealed is installed at the bottom of the well. Gravels are placed immediately after the completion of casing installation. The gravels are generally put by labourers basket in the drilled hole around the tubewell upto a level 50 feet below ground surface. Prior to the completion of gravelling two G.I. pipe each of 3 inches nominal diameter and of sufficient length to reach from 1 foot above the pump house floor to 2 feet below the top of the gravel pack are installed. The annular space between the housing pipe and the hole above the gravel are back filled with paddled clay upto a level 20 feet below ground surface. The remaining space is filled with cement grout (1:1:2) as per direction of site Engineer.

The contractor supplies a development and test pump, with a capacity of not less than 4 cusec which is equipped with suitable orifice and manometers for accurately measuring the well discharge. The development procedure for wells as used by WASA consists of step pumping and back washing. The step pumping starts from an initial rate of 75% of design discharge and the rate is gradually increased upto a minimum of 150% of the design discharge. Development pumping is continued for atleast six hours with intermittent back washing at an interval of 10 minutes. Development for a period of more than 6 hours is continued if the water is not free of turbidity and contain more than 100 ppm of sand 10 minutes after back washing. Following development the test pumping is started. However, the test is not commenced until the static water level has recovered to within 0.1 feet of the static water level prior to development.

The well test pumping is generally conducted as follows:

Pumping at 75% of Capacity for 6 hours

Pumping at 100% of Capacity for 6 hours

Pumping at 125% of Capacity for 6 hours

Pumping at 150% of Capacity for 6 hours

The water level record are collected during the test pumping.

6.3 Mode of Failure

The wells which have been found to be defunct much earlier than anticipated time showed that their failure occurred more or less in the same way. At the beginning of failure the well started yielding sand in traces and progressively in large quantities with muddy water at the design discharge. In such a situation generally the high capacity pump is replaced by a lower capacity pump and under this condition water is found to be clear for some days. After running for few days at this lower capacity the well again starts sand pumping like before even at this lower capacity and ultimately failure occurs. The following case studies will show more clearly how failure occurred in these cases.

6.3.1 Shamoli well

The shamoli well was completed on 1.5.77. Below are given some of the design features of this tubewell:

<u>Description of item</u>	<u>Length</u>
i. 16" dia M.S. seamless casing pipe	140'-0"
ii. 16" x 8" dia reducing cone	2'-0"
iii. 8" dia blind pipe	120'-0"
iv. 8" dia stainless steel strainer	170'-0"
v. 8" dia bail plug	3'-0"

The tubewell was commissioned on 21.4.78 with a 2.5 cusec capacity turbine pump and 75 H.P. motor.

From log book record it is learnt that the sand gravel and muddy water was coming out on 21.8.81. A turbine pump of capacity 2.5 cusec and a 75 H.P. Motor was in use. The pump operator then opened the wash out and washed the water for some time, but, no improvement was found. On the following day i.e., on 22.8.81 the higher capacity motor and pump were replaced by low powered motor (38 H.P.) and pump (1 cusec submersible pump). After completion of the installation of new pump and motor the pump was put into operation at 2-10 P.M. on 22.8.81. But as there was not much improvement it was decided to shut down the pump. Again on 23.8.81 at 10 A.M. the pump was put into operation and after few minutes of washing the water was found clear and then it was supplied to the distribution system.

The well with this low capacity was operated for two years and in September 1983 it again started sand pumping and ultimately it was declared defunct.

6.3.2 Lalmatia well

As per record the Lalmatia well was installed in March 1977. Type and quantity of materials used in this tubewell are as follows:

<u>Description</u>	<u>Length</u>
i. 16" dia M.S. seamless casing pipe	= 140'-0"
ii. 16" x 8" dia reducing cone	= 2'-0"
iii. 8" dia blind pipe	= 138'-0"
iv. 8" dia stainless steel strainer	= 170'-0"
v. 8" dia bail plug	= 5'-0"

As per recort it is learnt that the fine sand was coming out from 3.9.83. To improve the situation washing was done for 5 hours, but no improvement was found. The 2 cusec capacity pump was replaced by a 1 cusec pump but even then the sand pumping was not stopped and ultimately it was declared defunct.

6.4 Probable Causes of Failure

From the fixture of Shamoli well it is seen that the strainer was fixed at depth 260'-430'. The aquifer sample from the whole depth was collected and sieve analysis was done. The d_{30} size of aquifer material was found around 0.3 mm in most of the cases with Cu more than 2 but less than 3. For depth 405'-410' and 420'-425' the Cu for the aquifer material was found to be less than 2, the d_{30} size of grain was found to be 0.23 mm. Depending upon this grain size distribution a gravel pack was designed by Johnson method and it was found that the gravel should have the following characteristics (Figure 6.5).

$$d_{10} = .75 \text{ mm}, d_{30} = 0.92 \text{ mm}, d_{50} = 1.1 \text{ mm}, d_{60} = 1.2 \text{ mm}$$

$d_{90} = 1.9 \text{ mm}$ and accordingly the slot size for the above

condition come out to be to 30"/1000 whereas the installed screen had slot opening of 50"/1000.

Further more the sample of gravel pack practically used were approximately of the following sizes, $d_{10} = 1.4 \text{ mm}$, $d_{30} = 2.5 \text{ mm}$, $d_{50} = 3.45 \text{ mm}$, $d_{60} = 3.8 \text{ mm}$, $d_{90} = 5 \text{ mm}$.

It is apparent from the above data that the slot size used in the depth 405'-410' was larger than the design size and further more the size of gravel used were much larger than the designed gravel size. It might have so happened that through this big slot size with larger gravel packs the fines progressively moved out with pumped water. Cavities were formed in the depth 405'-410' due to this loss of material and turbulence pockets were created and sand movement increased with time. This resulted in movement of aquifer material together with gravel pack material and thus in this process the graded filter effect might have been lost fully or partially and as a result excessive sand pumpage occurred.

From the fixture of Ialmatia well it is seen that the strainer was installed at depth 278'-448'. Aquifer sample from depths of 275'-405' was collected and sieve analysis was done. At depth 280'-285', d_{30} of aquifer materials was found to be 0.26 mm and C_u was found to be equal to 2. Depending upon this grain size distribution a gravel pack size was designed as before (Fig. 6.6). According to this design the different size of gravels

are: $d_{10} = 1.15$ mm, $d_{30} = 1.56$ mm, $d_{50} = 2.0$, $d_{60} = 2.4$, $d_{90} = 3.5$ mm and accordingly design slot size stands to $45\frac{1}{1000}$ ". The gravel pack material used in this case was of the same size and gradation as used in shamoli, So, the same phenomenon of failure as discussed in case of shamoli well might have taken place in this case also.

In Hazaribagh well, installed in the year 1976, the strainer was placed from 242'-402' depth. At depth 240'-245' and 270'-275' the d_{30} size of aquifer material was found to be 0.25, Cu greater than 2. For this aquifer material the design slot size comes out to be $43\frac{1}{1000}$ " and the gravel pack design should be as follows:
 $d_{10} = 1.1$ mm, $d_{30} = 1.45$ mm, $d_{50} = 2.0$ mm, $d_{60} = 2.3$ mm, $d_{90} = 3.1$ mm (Fig. 6.7). The gravel pack material used in this case was more or less of the same size and gradation as used in shamoli. So, the same phenomenon of failure as discussed in case of shamoli well might have taken place in this case also.

From the comparative study of the above three cases it may be discussed here why the life of Shamoli well was only 4 years whereas Lalmatia and Hazaribagh wells had a life of 7 years each although all had some drawbacks in the light of theoretical design. The reasons for such a small life span of Shamoli well with respect to the other two wells may be explained in the following way:

The difference between the two designs, one from the theoretical considerations as per aquifer characteristics and

the other practically used in field, are relatively more in case of Shamoli well than in other two tubewells (Table 6.1). The theoretical design slot size in case of Shamoli well is $30''/1000$ and for other two cases is $45''/1000$. Whereas the actual slot size used in field was $50''/1000$ in all the cases. Similarly in case of Shamoli well there was more difference between gravel pack size actually used and as obtained from standard design procedure. So, it is clear that since Lalmatia and Hazaribagh wells had design parameters more closer to the theoretical design, than that of Shamoli well, these wells got a longer life span than the Shamoli well. This shows that a good design can ensure a longer life and satisfactory performance of a well.

According to the standard design of gravel pack as used in WASA, a gravel size distribution graph is drawn, using approximately the average value from the maximum and minimum limits (Fig.6.7). According to this graph the gravel size distribution comes out to be as follows (in mm):

$$d_{10} = 1.9, d_{30} = 2.7, d_{50} = 4.0, d_{60} = 4.6, d_{90} = 8.0.$$

On the other hand using the approximate average d_{30} size of aquifer material (0.35mm) a gravel pack was designed (Fig.6.7). This design has the size distribution as follows (in mm):

$$d_{10} = 1.6, d_{30} = 2.1, d_{50} = 2.8, d_{60} = 3.2, d_{90} = 4.8$$

From this comparison it is apparent that size of the gravel pack material used was much larger than what should have been used as per aquifer material size.

The size distribution of gravel pack material as plotted in Fig. 6.7 using the average value from the maximum and minimum limits of WASA design criteria has been used for comparison. According to this plotting the d_{30} of pack material is 2.7 mm. For this value of packs material the d_{30} size of aquifer material should be 0.45 mm. But it is interesting to note that in case of Lalmati well only 5.55% of the aquifer depth screened had d_{30} equal to 0.45 mm and above. Similarly, in case of Shamoli and Hazaribagh well this size of d_{30} was found only in 10% and 12% of the aquifer depth screened.

So, it may be concluded that the size of gravel pack to be used should be adjusted for better well performance.

The factors discussed above may be one of the probable causes of unsatisfactory well performance. It is obvious that if the finer portion of aquifer material could have been avoided during screening of the aquifer, then much more satisfactory performance could have been achieved. As there is no provision for grain size analysis of aquifer material just before fixture installation in the field

in case of WASA well there always remains a probability of screening a portion of an aquifer having particle size smaller compared to the fixed slot size. Generally in the field as soon as the drilling is completed, the suitable aquifer to be screened are selected on the basis of eye estimation and experience and the fixtures are placed very hurriedly to avoid caving. Under this condition it is obvious that no attempt is made to analyze the grain size of the aquifer and as a result proper gravel pack design becomes out of question. Effort should be made to improve this aspect of tubewell construction for proper functioning and longer life.

The smallest d_{10} grain size of aquifer material is found 0.12 mm. Depending upon this size the optimum design length of screen stands to 100' for 2 cusec design discharge with the standard screen as used by WASA. So, it is better to avoid the tendency of installing larger length of screen rather it is better to install required screen in the coarser aquifer avoiding finer particle. This is only possible with in-field grain size analysis. This will minimize the threat of sand pumping as discussed earlier.

6.5 Development of WASA Well

Analysis of test pumping records of WASA wells reveals that another probable cause of un-satisfactory well performance may be inadequate development. It has been discussed earlier that by inadequate development it is meant that the aquifer is not

flushed back to the extent that all its fines are removed, and a good back grading has not been achieved so as to retain the fines which could be flushed. Under this condition these fines progressively move out with pumped water and ultimately causes failure of the tubewell.

The calculation of well loss constant from the test pumping data in case of Lalmatia well gives the following values (Fig. 6.8) of well loss constant.

For step I and II	$-2(C_1)$
For step II and III	$-7.32(C_2)$
For step III and IV	$-2.93(C_3)$

The above well loss constants were computed using test pumping data and the test pumping had been performed after developing the well by overpumping and backwashing. The results indicate that the incremental drawdown decreased with the increase in discharge. This indicates that the development was not complete. Hence the well was not stable that the development was not satisfactory.

In case of Hazaribagh well the value of C_1 , C_2 and C_3 are found respectively to be -1.83, -2.5 and +6.16. The values indicates that the well was neither fully developed nor stable (Fig.6.9). At step four the value of C jumps to 6.16 indicating partial clogging of the screens.

From Table 6.2 it is apparant that for most of the tubewells the development was not at all satisfactory. The procedure used for development and the interest of the persons involved in the development operation play a very vital role in the proper development of the well. The back washing procedure which is generally used in WASA tubewell development is not very effective and perhaps due to this development has not been upto requirement. Efforts made to get proper development by any procedure also plays an important role. So, even in the back washing procedure there remains scope for better result by ensuring more attention of the persons involved in the development work. In this context, it may be worth mentioning that in order to breakup the wall cake of relatively impervious material (sandwitched between the gravel pack and the face of the natural formation) polyphosphate dispersing agent may be used to assist the development procedure.

6.6 Formation Constants of the Aquifer Material inside the study area.

Test pumping data from observation wells were used to determine the values of storage co-efficient at different locations of study area (Fig. 6.10). These values are given in Table no.6.3. The 'S' values shows an wide range of variation from one well location to another. The maximum and minimum values of 'S' as obtained are 0.013 and 0.129 respectively. Depending upon grain size of aquifer material the 'S' values should be between 0.1 to 0.2 (Table 2.4). Under such a situation it is better to rely more

on the values obtained from test pumping data of observation wells as it takes into account the spacial variation of grain size distribution, its arrangement and degree of compaction. The average value of 'S' as obtained from test pumping data of observation well (Table 6.3) is same as that reported by Karim (15).

The radius of influence has been calculated using equation 3.11 and 3.12. The results are given in Table 6.3 which shows that the radius varies from 4700' to 14500'. While applying the equation, the limit of the radius of influence has been assumed where the drawdown was 3" after 300 days of pumping at the rate of 900 gpm.

The calculated values of radius of influence seems to be higher as regards to the aquifer soil characteristics. The probable causes of such higher value may be due to the fact that there are several wells which are functioning near the vicinity of WASA wells but the effect of these wells could not be considered during the investigation. The existance of such well has been mentioned in the paper of Wahed (32).

From table 6.3 it is worthy to note that when the 'S' values are relatively higher the values of 'r' are relatively lower. But in case of Rajarbagh and Mirpur area it is seen that both 'S' value and r value are higher in case of Rajarbagh area than Mirpur area. This may be due to the presence of more closer tubewells in large numbers in Rajarbagh area than in Mirpur area.

Test pumping data from observation well were also used to determine the values of Transmissibility co-efficient at eight locations of the study area. These values are given in Table 6.3. Depth of the tubewells varies from 375' to 505' and length of strainer varies from 136' to 200'. It is apparant from table 6.3 that a co-relation is not possible between the values of T with the depth of tubewell and the length of strainers. The maximum and minimum values of T as obtained are 1.46×10^5 gpd/ft and 0.35×10^5 gpd/ft respectively. Data from Balda Garden well were analyzed by distance drawdown method while those from rest seven locations by Jacob Method.

The hydraulic characteristics such as S and T of the underground water bearing formation show a considerable degree of variability. The ratio of the maximum and minimum values of co-efficient of transmissibility is 4.17 and that of S is 9.92. This variation of the values of the formation constants necessitates designing of tubewell components in conformity with the formation characteristics in order to economise and to optimize the development of ground water resources of the study area. Designing without consideration of the formation constants will surely lead to over design or under design of the tubewells.

6.7 Well loss and Specific Capacity of WASA Well

The analysis shows that most of the tubewells have very high well loss values which signify that the performance of these wells is poor. 33% of selected tubewells studied have

well losses greater than 30% of total drawdown at the well. The maximum well loss value has been obtained as high as 70%.

High percentage of well loss causes increase in pumping cost. It also indicates unsatisfactory performance of tubewells. Study of the designed fixtures such as screen length, diameter and percent open area used in the field reveals that the length of screen which are generally used are more than sufficient to ensure minimum entrance velocity so as to keep the well loss at a minimum value. But the calculated well loss values obtained from the field data are very high. Such a high values of well loss may be a result of partial clogging of screen beyond the accepted limit due to insufficient development of the well. Another cause for such high values of well loss may come from improper verticality of the well fixtures resulting high friction of flow through the riser.

The specific capacity as calculated from test pumping data has been found to vary from 12 gpm/ft to 46 gpm/ft (Table 6.4). Generally higher specific capacity indicates a higher co-efficient of transmissibility. But a comparison of 'T' and specific capacity shows that in some cases specific capacity has relatively low value when compared with corresponding 'T' values. In these cases it is notable that the well loss values have been found to be relatively high and this has resulted in low specific capacity even though 'T' was high. Thus it is apparent that the specific capacity

is also dependant on well loss which may increase due to faulty well construction and development. Test pumping data from pumping well and observation well which is generally within a distance of 20 meters from pumping well were collected and analyzed. The value of transmissibility coefficient as obtained from pumping well and observation well data are worth noting.

At Mill Barrak area the 'T' value obtained from well data is $.964 \times 10^5$ gpd/ft and that from observation well data is 1.46×10^5 gpd/ft and they differ considerably. At Mirpur area the 'T' values from pumped and observation wells are found to be $.548 \times 10^5$ gpd/ft and 0.44×10^5 gpd/ft respectively, and these values are quite closer. Another point of interest is that the value of 'T' calculated from test pumping data from two pumped wells drilled at the same area but at different times differs considerably. As for example value of 'T' obtained from test pumping data of Lalmatia well in the year 1977 and 1983 are respectively 0.58×10^5 gpd/ft and 1.15×10^5 gpd/ft. Similarly for Hazaribagh area the calculated 'T' from data of well constructed in the year 1976 and 1983 are $.678 \times 10^5$ gpd/ft and 1.42×10^5 gpd/ft. respectively.

To find reasons for such variation, detailed study of the lithological characteristic of the aquifer, the depth and thickness of aquifer tapped and the characteristics of strainer are required.

Changes in specific capacity with time in some good running wells and in some early defunct wells are shown in figure 6.11 and 6.12 respectively. The wells with good performance shows that their specific capacity has gradually increased though they have exceeded their anticipated life time. But in case of early defunct well it is seen that the specific capacities gradually decreased with time since they started functioning.

Study of variation of specific capacity of a well with time will thus help to know how good the tubewell is.

6.8 Index Properties of the Aquifer Material

The bore log data of different wells reveal that the aquifer, from where WASA well are fed extends up to a depth of about 450' to 500'. For the top 20 to 30' ft sometimes upto 50', the aquifer materials is mostly clay. Then with depth aquifer material changes to silts, fine sand, medium sand and in some cases coarse sand with traces of gravel (Fig.6.13 - 6.15). The deeper portion, therefore naturally tend to be the most productive part of the aquifer.

The grains size analysis of aquifer materials at different tubewell locations reveal that in the most productive zone where generally strainer is placed the maximum and minimum values of effective grain size are 0.73 mm. and .25 mm respectively whereas most of the particle are within a size of 0.3 mm to 0.48 mm indicating a medium sand. The values of d_{10} size mostly ranges from 0.2 mm to 0.28 mm. The values of uniformity co-efficient are found to vary from around 2 to 2.5.

6.9 Lowering of the lowest recorded Ground Water Level and its effects on WASA wells.

From record it is seen that ground water withdrawal in Dhaka city has gradually increased since the first use of ground water in 1949. Table 6.5 shows how annual withdrawal rates have increased over last 20 years and Figure 6.16 has been drawn based on this table.

Due to this increase rate of withdrawal the lowest recorded ground water level in Dhaka WASA's well during the period 1964 to 1984 has gone down considerably and Table 6.6 shows how it has gone down over last 17 years. It is worth mentioning here that the lowest water level data as recorded in different years were not recorded in the same time in each year. It might so happen that the data recorded as lowest for a year may not be the actual lowest for that year. However, from the data used in this study at least an idea may

be formed about the lowering of the water level in different wells. Figure 6.17 shows the change of lowest recorded ground water level with time as observed in four WASA wells. From this figure it appears that around upto 1968 there was possibly no lowering of recorded lowest static water level in Dhaka city area although there was difference in water level at different locations. After 1968 the recorded lowest water level has gradually gone down year after year. Upto 1975 the lowering rate was relatively small and then it started increasing with increasing rate of withdrawal. From 1968 to 1975 i.e. in a period of 7 years the lowering of the lowest recorded water level in the three wells namely Mohakhali, Bangladesh Math and High Court were approximately 7', 3' and 8.75' respectively with an average rate of 1', 0.43' and 1.25' per year respectively. From 1975 to 1983 that is in the next 8 years the lowering in the same three wells were about 14', 16' and 21.5' with the average rate per year 1.75', 2' and 2.68' respectively.

(i) In Khilgaon well where data are available since 1975 lowering as much as 22 feet has been recorded during last eight years and this is equivalent to an average annual rate of lowering of three feet.

It is worthy to note that the rate of annual withdrawal has increased gradually and the lowering of lowest recorded water level has also increased gradually. It is not clear with

this data whether this lowering of water table is due to mining or due to higher withdrawal rate. To study this aspect it is necessary to observe the change of lowest recorded water level with a constant withdrawal rate. The data relating to highest water level from year to year will also help to study this aspect of the problem. If it is found that there is no mining due to present withdrawal rate and the lowest recorded water level is more or less at such a level that the present installed tubewells do not suffer due to such lowering, the present rate of withdrawal may be continued. If the study shows that further increase of withdrawal rate will not cause any mining and at the same time further lowering due to further increase of withdrawal will not make the installed tubewells inoperative or expensive from operation point of view, suggestion of higher withdrawal rate may be made.

Based on data from an observation well of BWDB a hydrograph has been prepared (Fig.6.19). From this figure it is seen that the water level is gradually going down with time. However, further study with more data from different observation wells will be necessary to comment on the trend as observed in Fig.6.19.

A study of the existing running wells of WASA shows that 21% of the wells have housing pipe length of 160', 44% of the wells have 140', 24% of wells have 120' and 10% of the wells have from 110' to 115'. Accordingly the R.L. of the bottom

of the housing pipes are placed between -80' to -140'.

From figure 6.17 it may be assumed that the average GWL is now at about -30' R.L. If the present rate of lowering of recorded lowest water level is continued the average GWL will go down to -44 ft by 1990. Assuming a drawdown of about 45 feet for a 2 cusec DTW the water level in the tubewell will be at -89 ft R.L. This will make the tubewells which has the R.L. of the bottom of the housing pipe above -89' level inoperative. About 10% of the WASA tubewells have their housing pipe length of 110 to 115 ft. Considering an average ground level of +25 feet these wells have the lowest point of housing pipe at an elevation of about -85 feet. From this it may be concluded that if the present rate of lowering of recorded lowest water level is continued about 10% of WASA wells will be inoperative by 1990 and by 2000 another 35% will become inoperative. However, it may be expected that if the rate of withdrawal is kept constant even at this value, the rate of lowering may gradually decrease and become ^{negligible} negligible// if there is really no mining. Under such a condition most of the tubewells, may remain operative.

It is important to mention here that the data relating to test pumping of different wells were collected manually by WASA contractors. The result obtained from the analysis of these test pumping data is therefore dependent much on the degree of correctness of these data. One should ^{keep} keep this ⁿ in mind while using the data and result from this study.

CHAPTER SEVEN

CONCLUSION AND RECOMMENDATION

7.1 Conclusion

From the study of the data relating to WASA tubewells and aquifer characteristics the following conclusion are drawn:

1. The uniformity coefficient of the aquifer material feeding the Dhaka WASA wells is around 2.
2. The maximum and minimum values of effective grain size (d_{50}) of the aquifer material are around 0.73 mm and 0.25 mm with most of the values ranging from 0.3mm to 0.48 mm.
3. The slot size (50/1000)" which is generally used by Dhaka WASA in the well screen may sometimes create problems when the screen is placed in finer aquifer having d_{30} less than 0.25 mm.
4. It is better to installed optimum length of screen in the coarser part of the aquifer and to avoid the use of longer screen covering relatively finer aquifer layer.
5. It is necessary to have sieve analysis of the aquifer material for correctly selecting the aquifer layers to be screened. The present practice of selecting the aquifer layers to be screened from visual observation of the aquifer material may not give good result always.

6. The gravel pack as used now may not be ideal for all places. Depending upon size of aquifer material to be screened different gravel pack should be used in different places.
7. The procedure, as used now for placing gravel packing material around the well screen if improved may improve the overall performance of the tubewell.
8. Gravel packed screen may be used in field on experimental basis.
9. Development of most of the wells seems to be insufficient. It is necessary to have better development of satisfactory well performance. The accuracy of development may be checked by calculating well loss constants.
10. The coefficient of transmissibility of the study area varies from 0.35×10^5 gpd/ft to 1.46×10^5 gpd/ft.
11. The storage coefficient of the study area varies from 0.013 to 0.129.

12. 'T' values obtained from data of pumped wells may not always give a correct picture as it is dependent on well loss percentage.
13. Low well loss values is an indication of better well development and construction. Under such condition, 'T' values calculated from test pumping data from pumped well and observation well comes out to be quite close.
14. The water bearing formation from where WASA well are fed extends upto a depth of about 450 to 500 feet.
15. The specific capacities of the wells studied are found to vary from 12 gpm/ft to 46 gpm/ft during the time of test pumping which is generally done just after the construction of the wells.
16. The specific capacity is dependent on well loss as well as aquifer loss. The specific capacity of a well extracting water from a relatively more permeable aquifer may be low due to excessive well loss because of faulty construction.
17. Because of lowering of lowest recorded water level due to excessive withdrawal of water a number of existing wells will be inoperative soon as the pumped water level will go below the bottom of the housing pipe if the present trend is continued.

18. An investigation for possible artificial recharge into the study area aquifers should be taken up to see upto what extent artificial recharge may help to get more ground water.
19. For the study of the problem of mining it is necessary to study the variation of highest water level year to year. The study of variation of lowest water level at a constant withdrawal will help in the determining the possible lowest water level for that withdrawal rate.

7.2 Recommendations for future study:

1. Similar study should be taken up in future by using data collected systematically. Dhaks WASA may be requested to collect necessary data for this type of study.
2. Laboratory study may be taken up to see how a variation in gravel pack design or a variation in selection of screen slot opening affect the performance of a well.
3. Laboratory study may also be taken up to see how a fine layer screened by a slot size as per design of the most of the layers screened affect the performance of a well.
4. A study may be taken on the effect of corrosion, if any, in the existing WASA wells.

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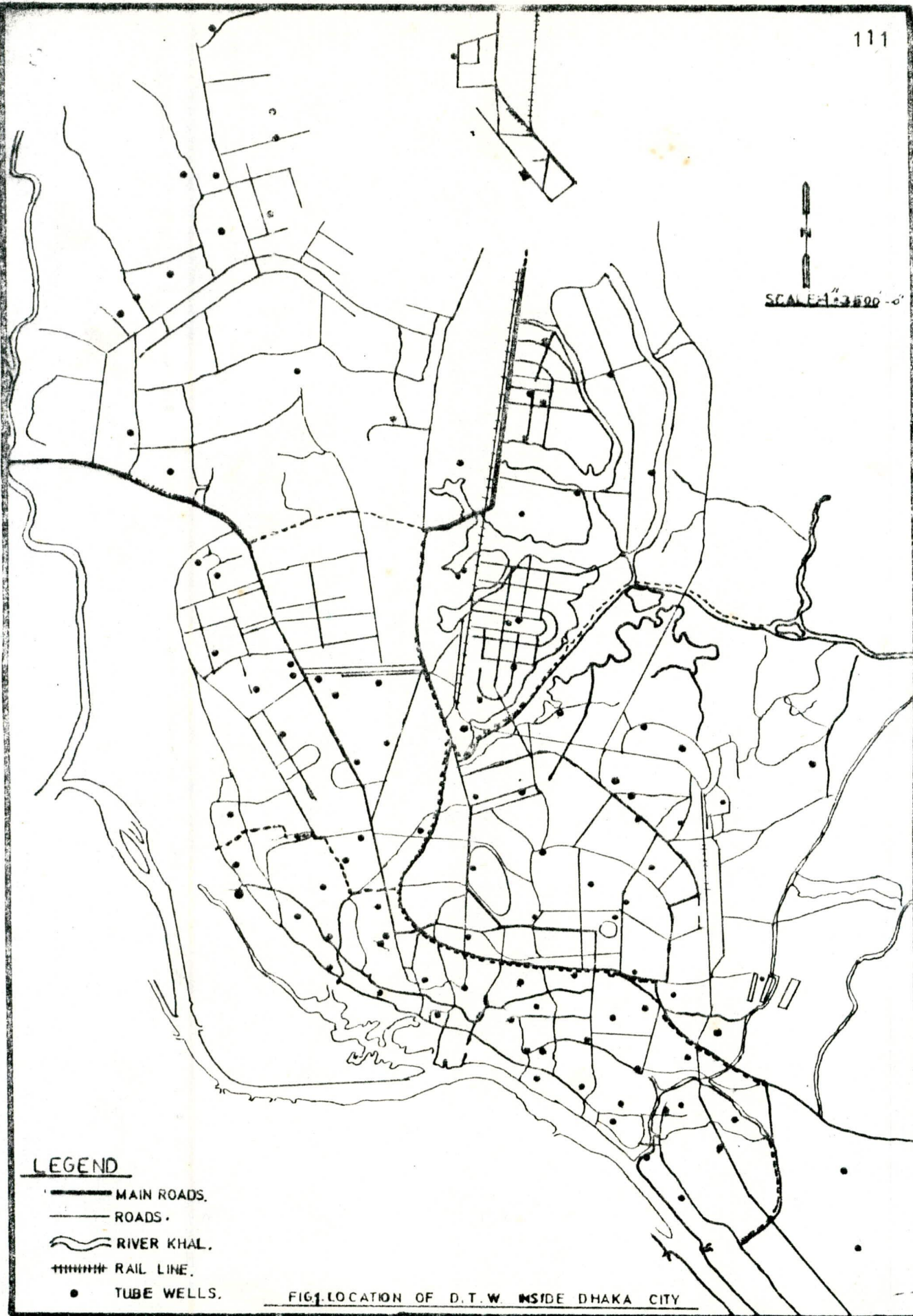
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APPENDICES



LEGEND

- MAIN ROADS.
- ROADS.
- ~ RIVER KHAL.
- + + + + + RAIL LINE.
- TUBE WELLS.

FIG. 1. LOCATION OF D.T.W. INSIDE DHAKA CITY

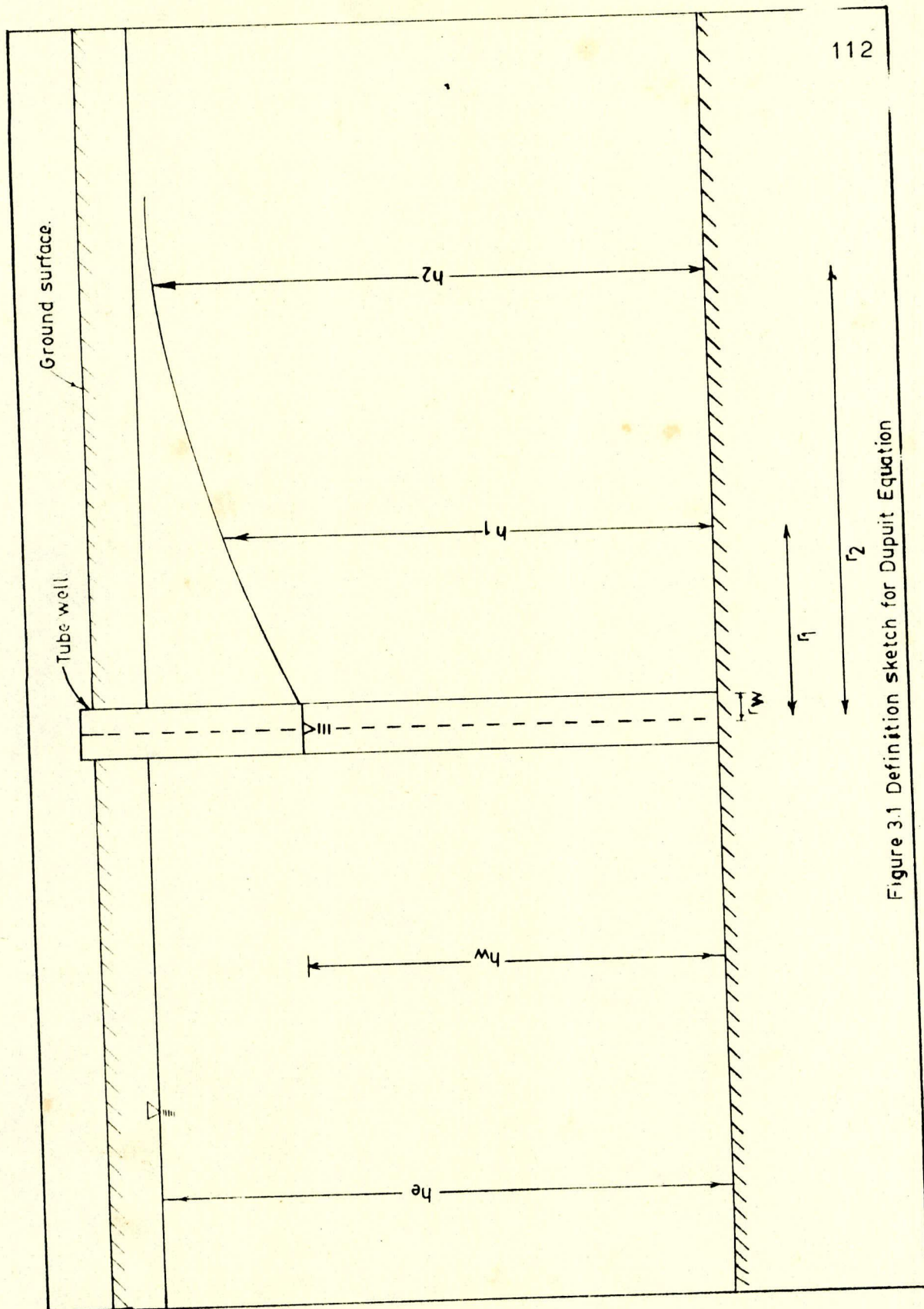


Figure 3.1 Definition sketch for Dupuit Equation

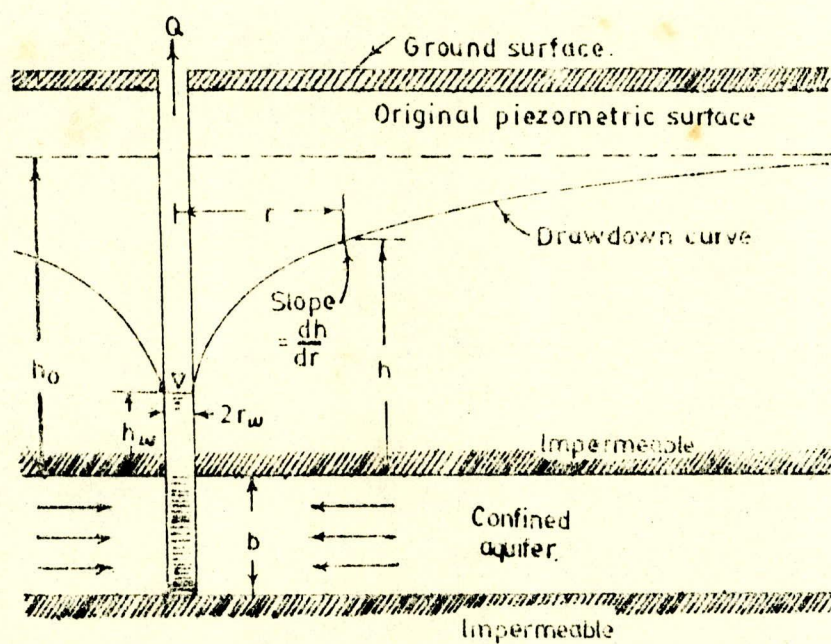


Figure 3.2. Radial flow to a well penetrating an extensive confined aquifer.

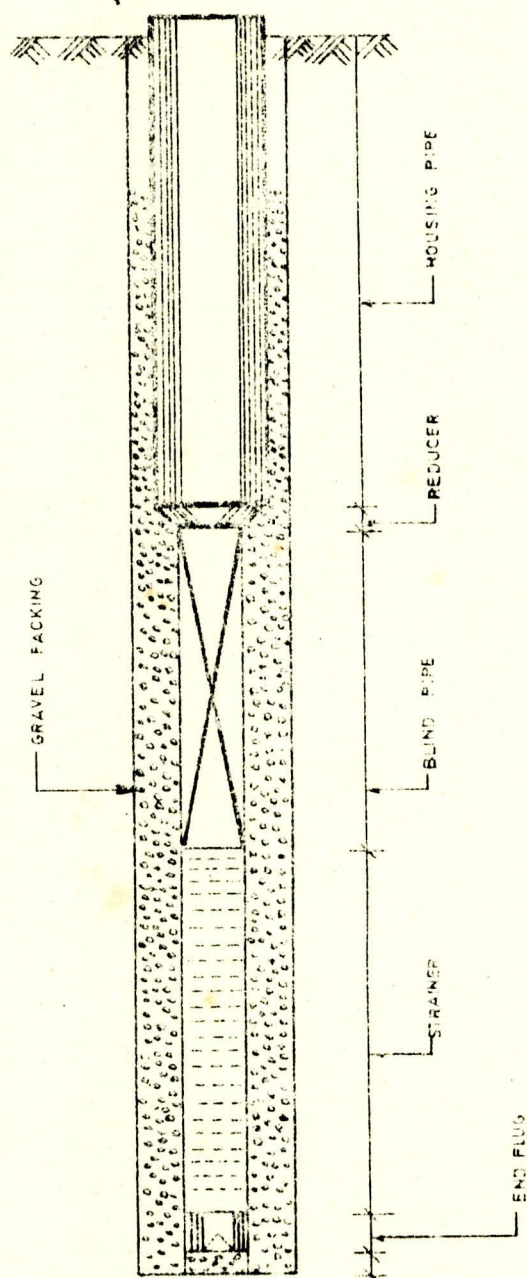


FIGURE 6-1 TYPICAL X-SECTION OF A WASA WELL

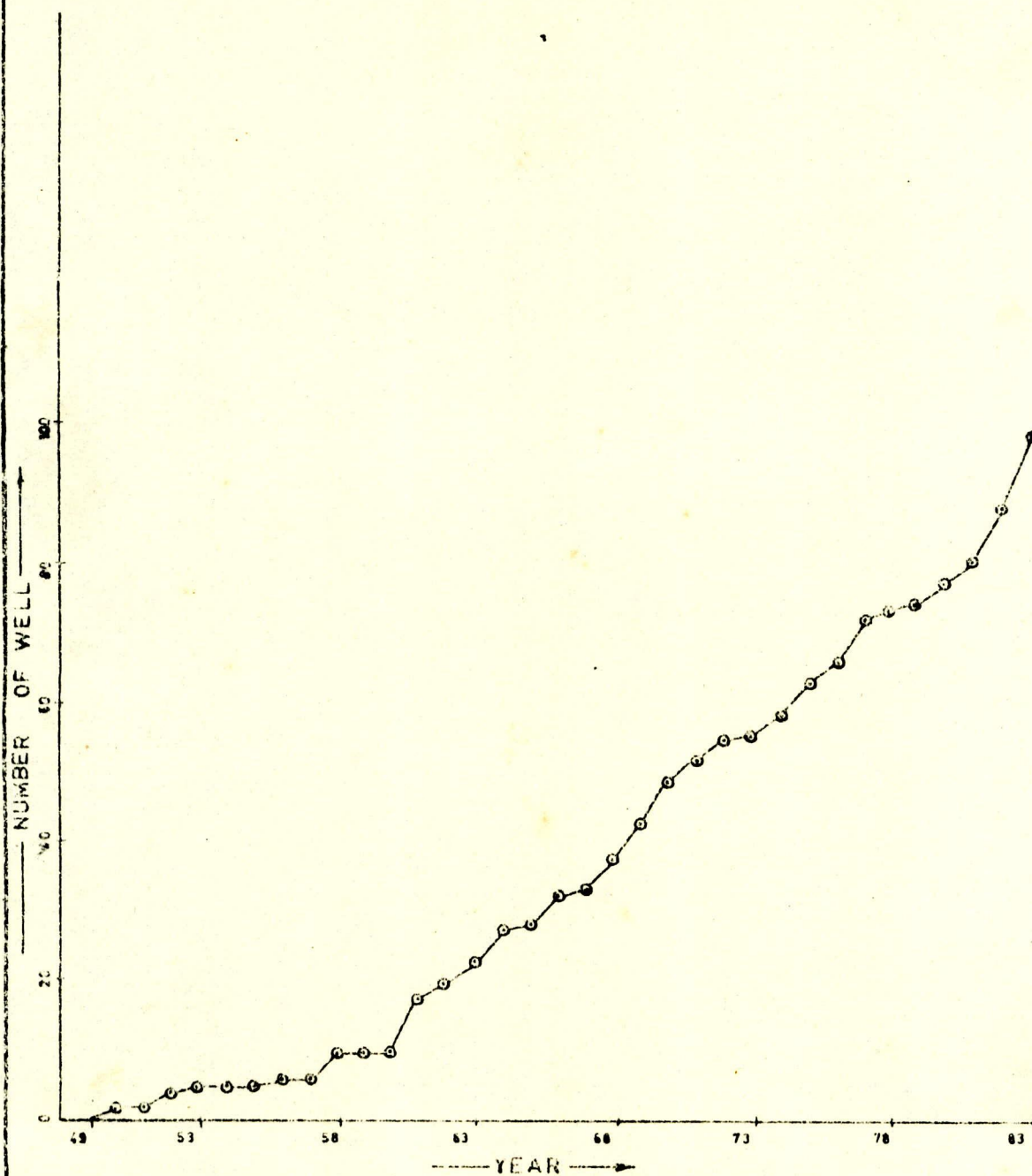


FIGURE 6-2 NUMBERS OF WELL INSTALLATION WITH TIME.

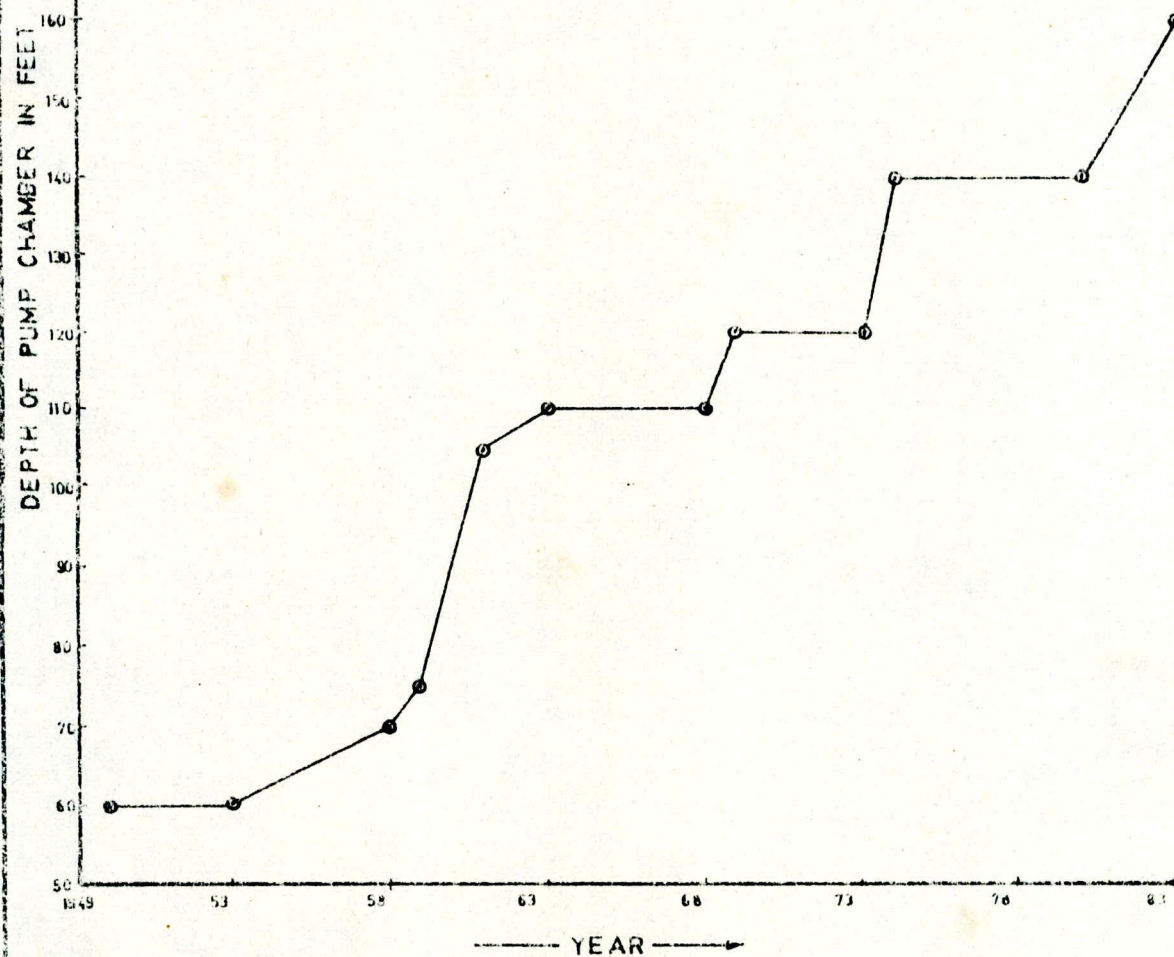


FIGURE 6-3 INCREASE IN THE DEPTH OF PUMP CHAMBER
IN WASA WELLS WITH TIME.

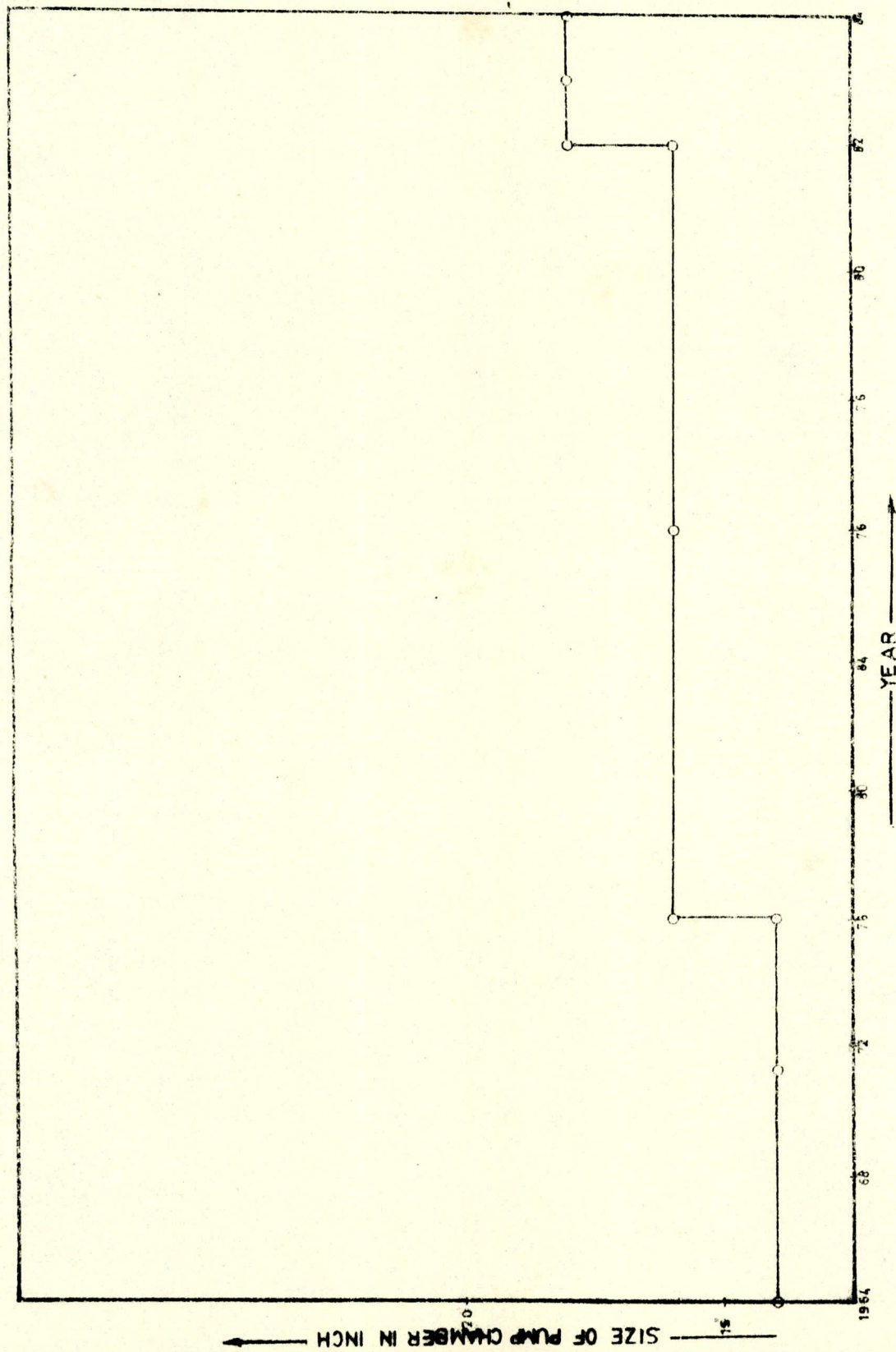


FIGURE 6.4. INCREASE IN THE SIZE OF PUMP CHAMBER WITH TIME.

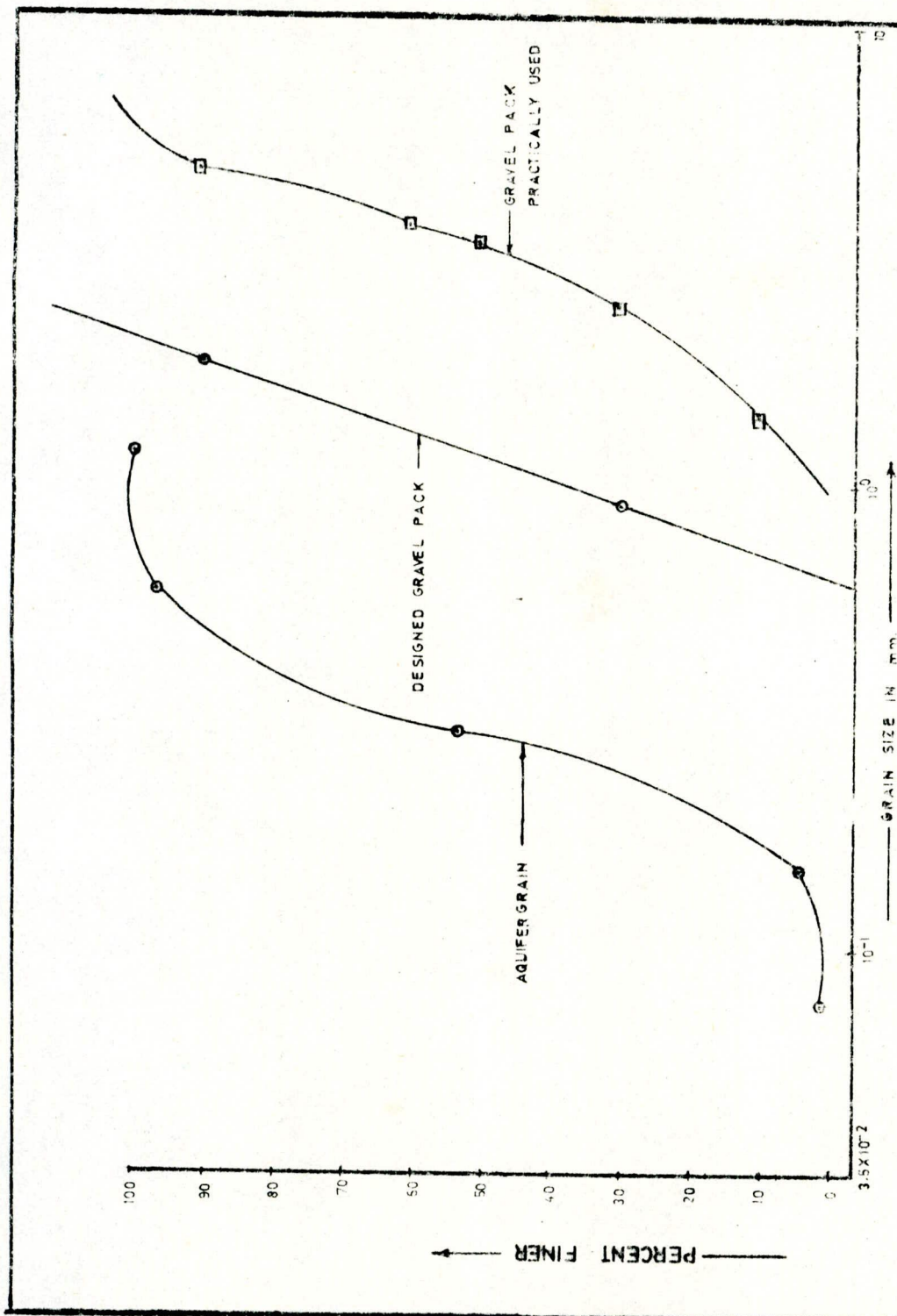


FIGURE 6.5 GRAIN SIZE DISTRIBUTION (AT DEPTH 405'-410') AND CORRESPONDING GRAVEL PACK DESIGN AT SHAMOLI WELL.

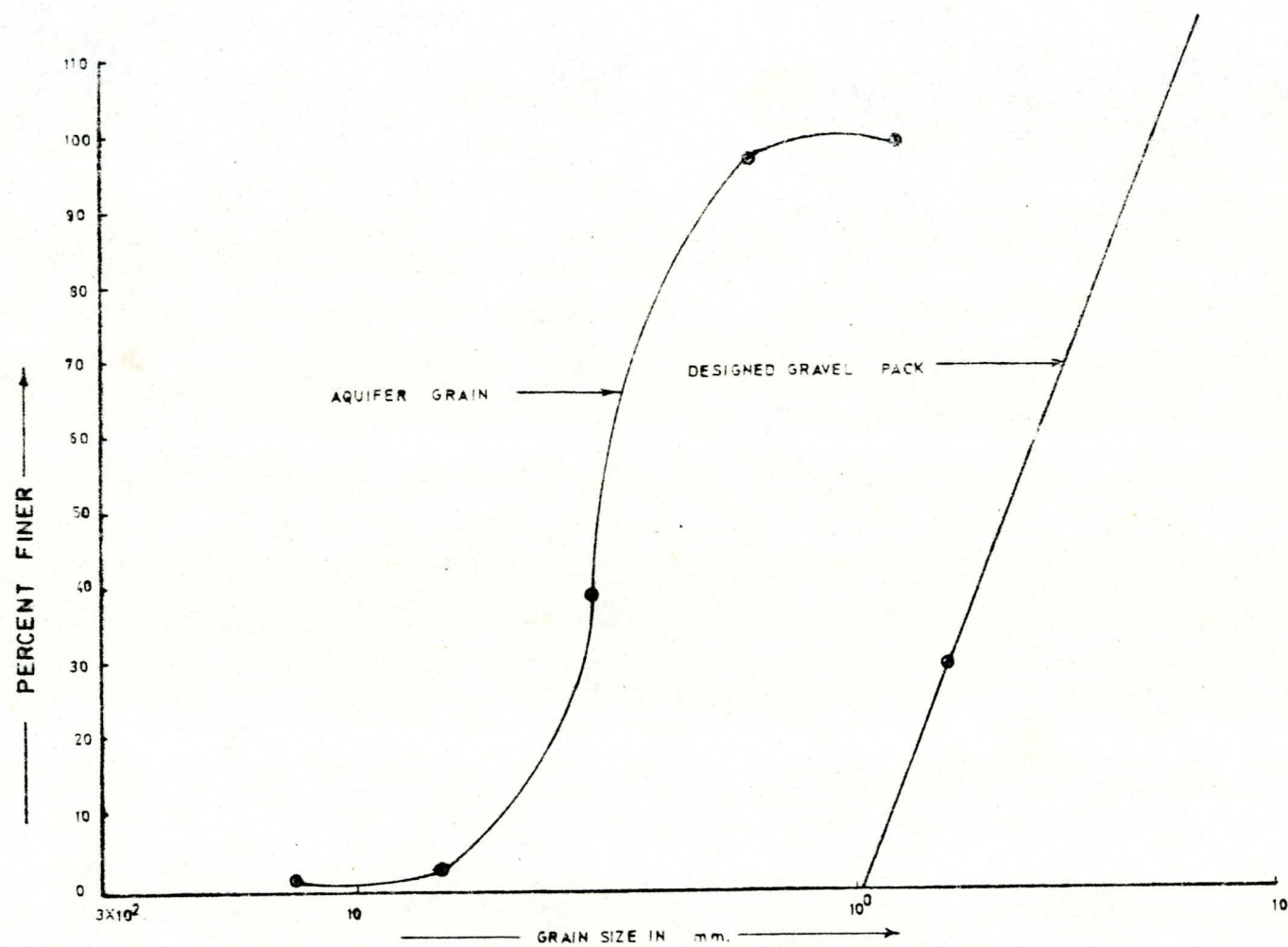


FIGURE 6-6 GRAIN SIZE DISTRIBUTION (AT DEPTH 280-285) AND CORRESPONDING GRAVEL PACK DESIGN AT LAMATIA WELL.

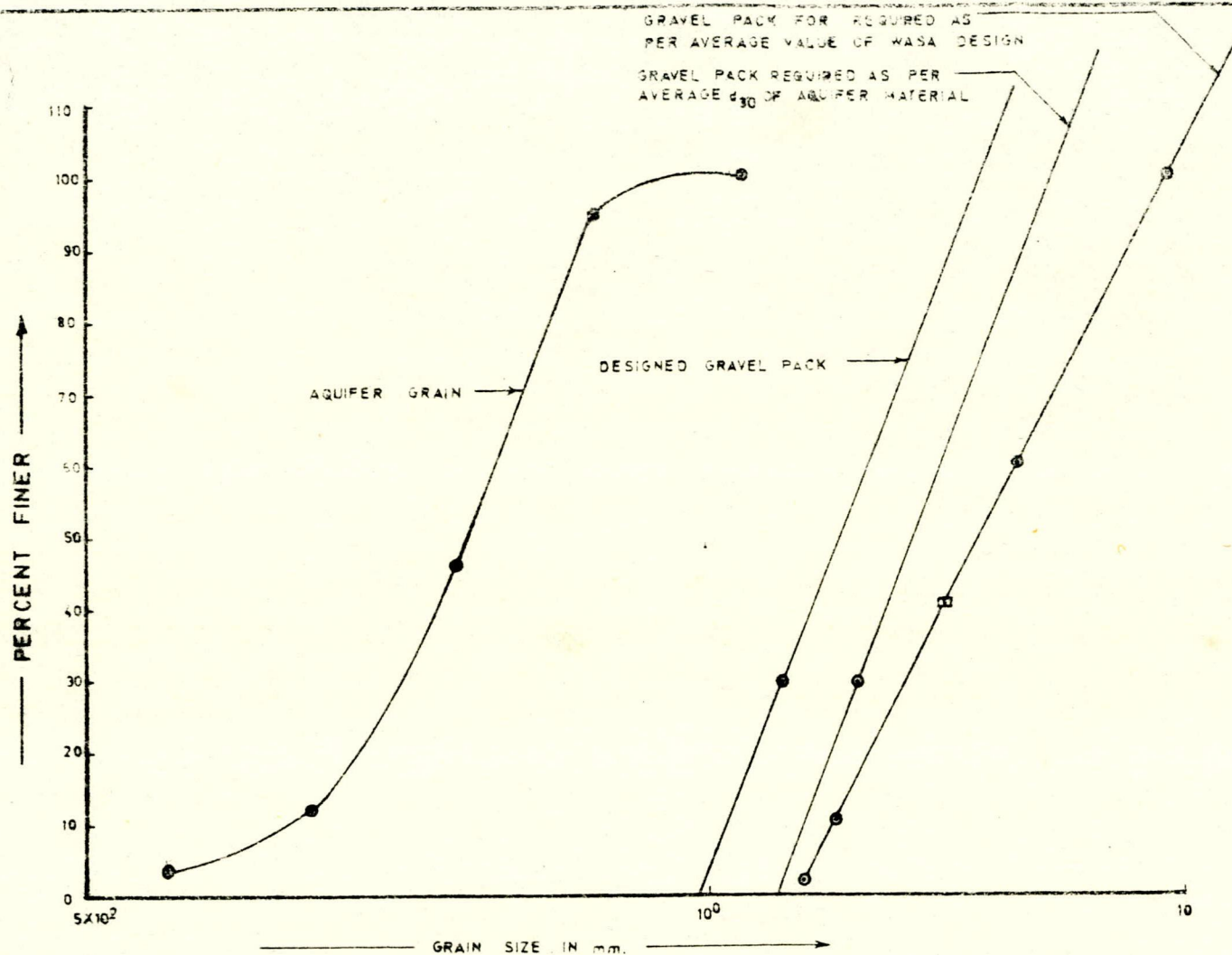


FIGURE 6.7 GRAIN SIZE DISTRIBUTION (AT DEPTH 270'-275') AND CORRESPONDING GRAVEL PACK DESIGN AT HAZARIBAGH WELL

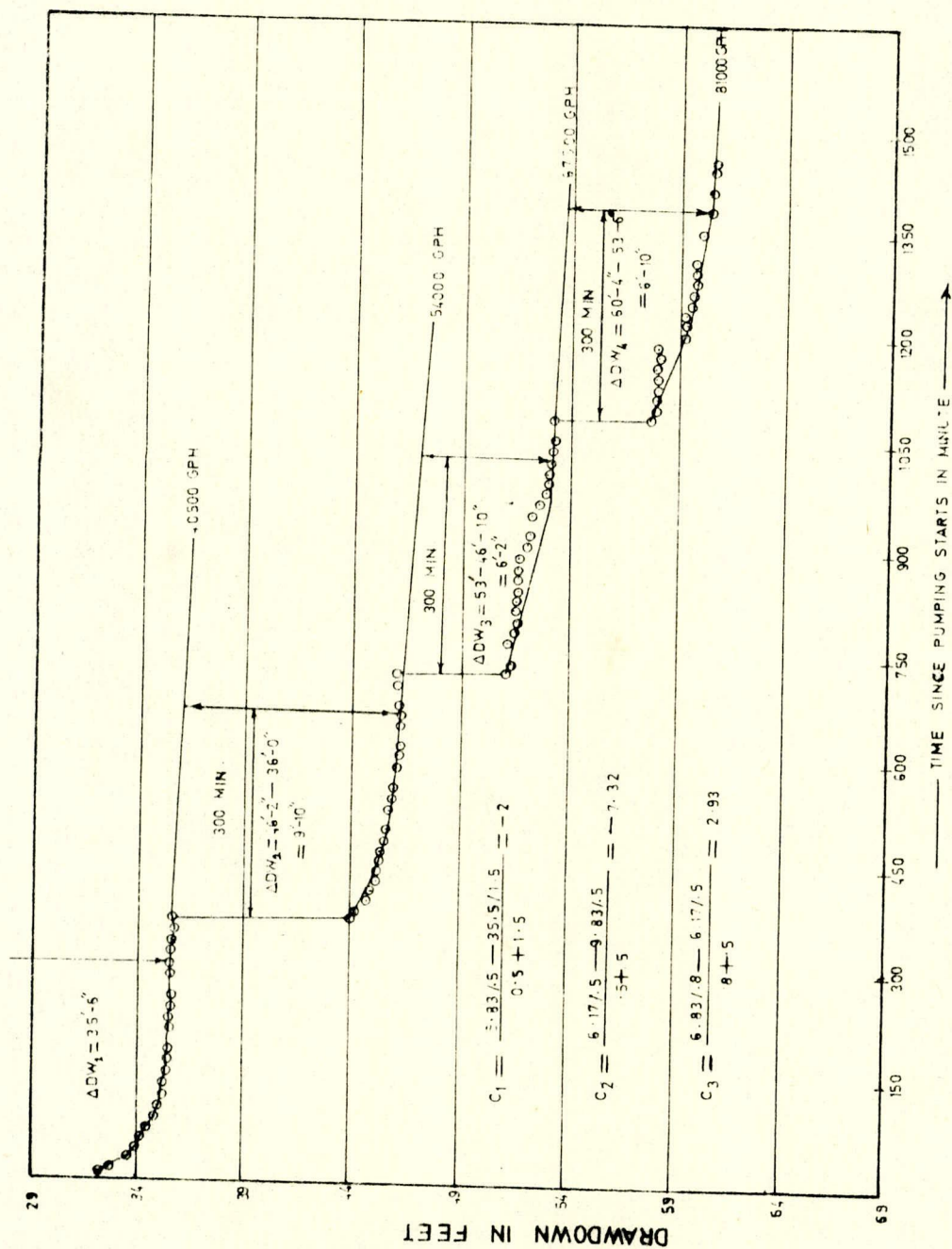


FIGURE 6.8 DETERMINATION OF WELL LOSS CONSTANT AT LALMATIA WELL (1977)

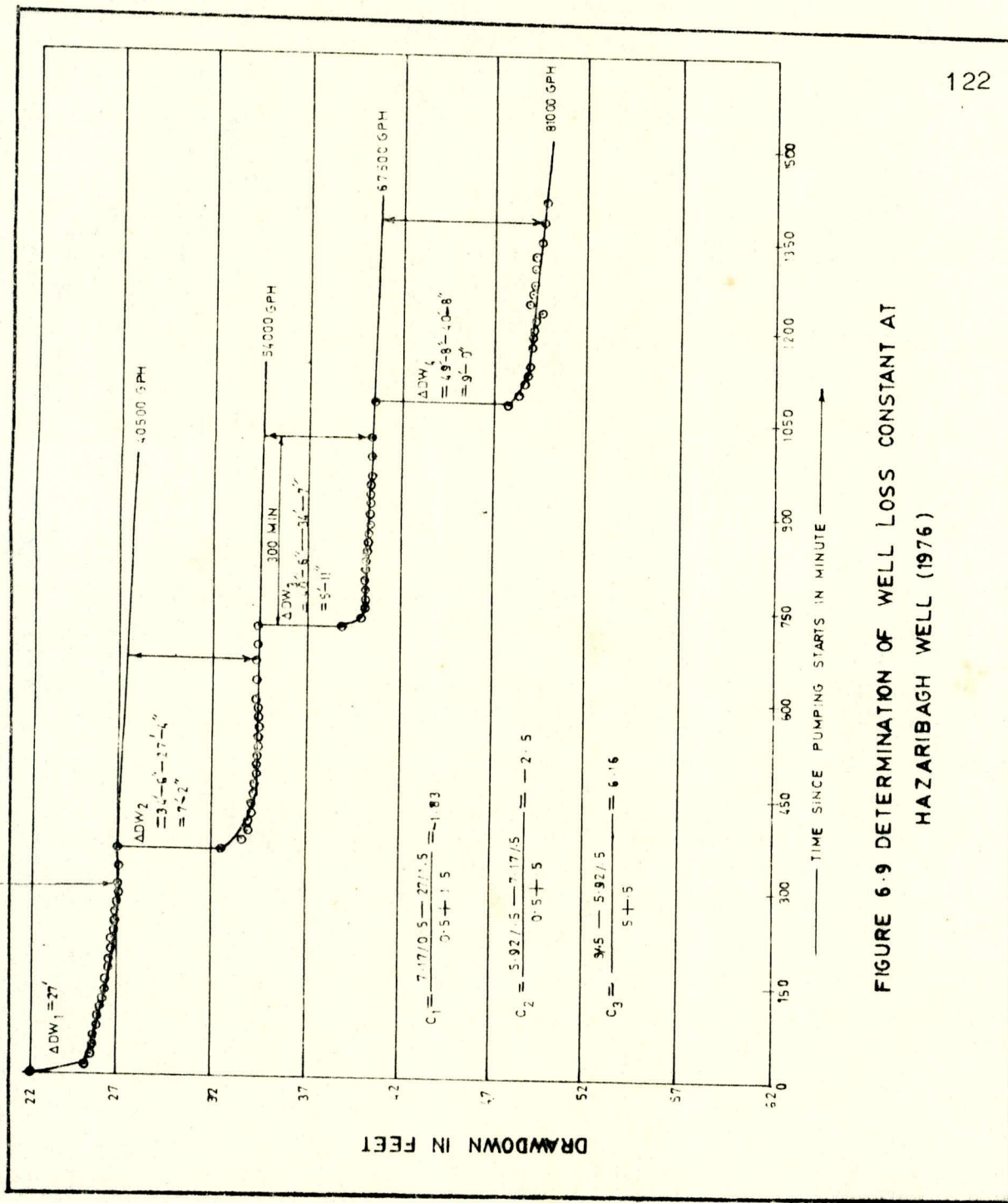


FIGURE 6.9 DETERMINATION OF WELL LOSS CONSTANT AT HAZARIBAGH WELL (1976)

DRAW DOWN IN FEET

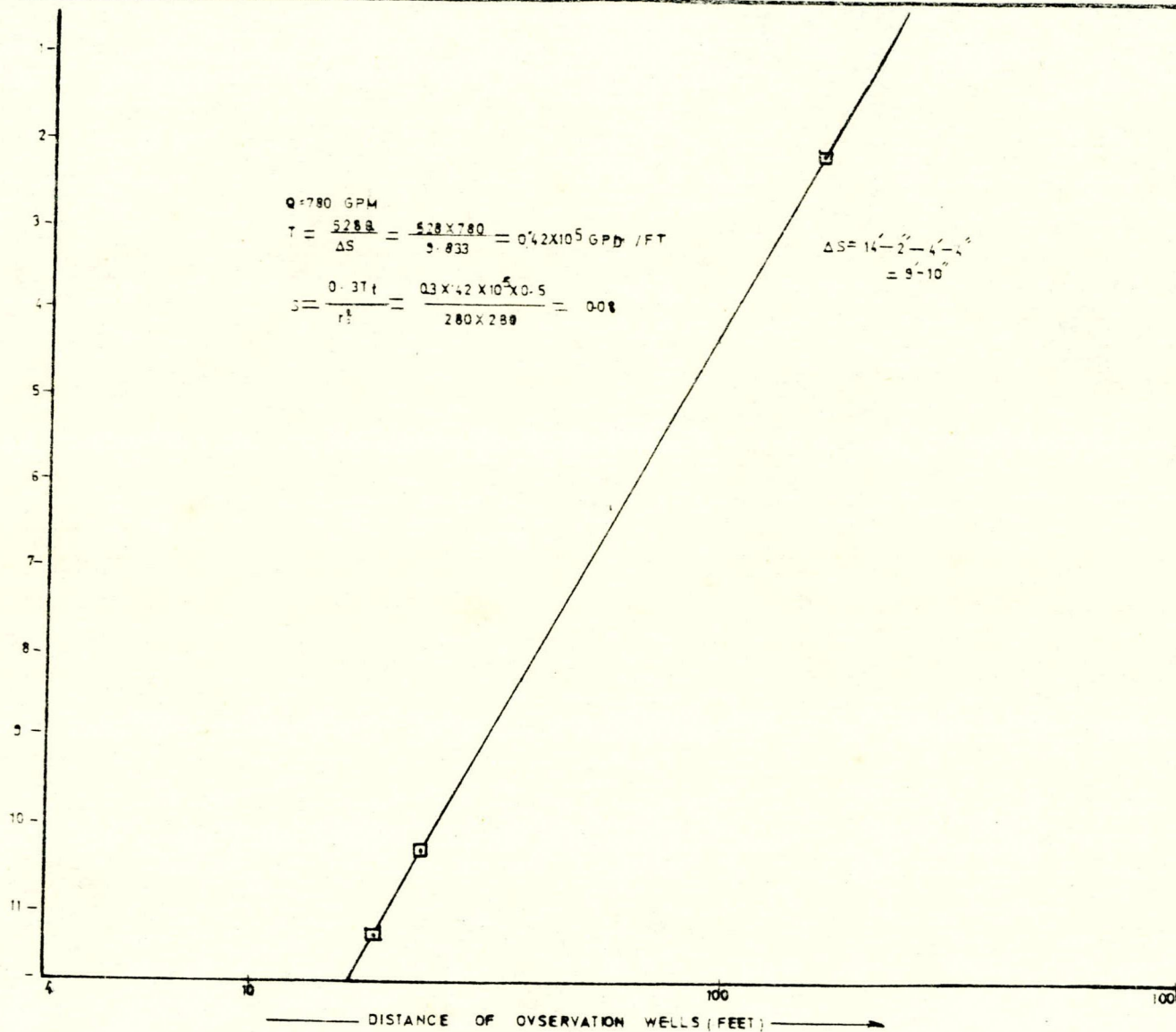


FIGURE 6.10 DETERMINATION OF AQUIFER CONSTANTS AT BALDAGARDEN WELL.

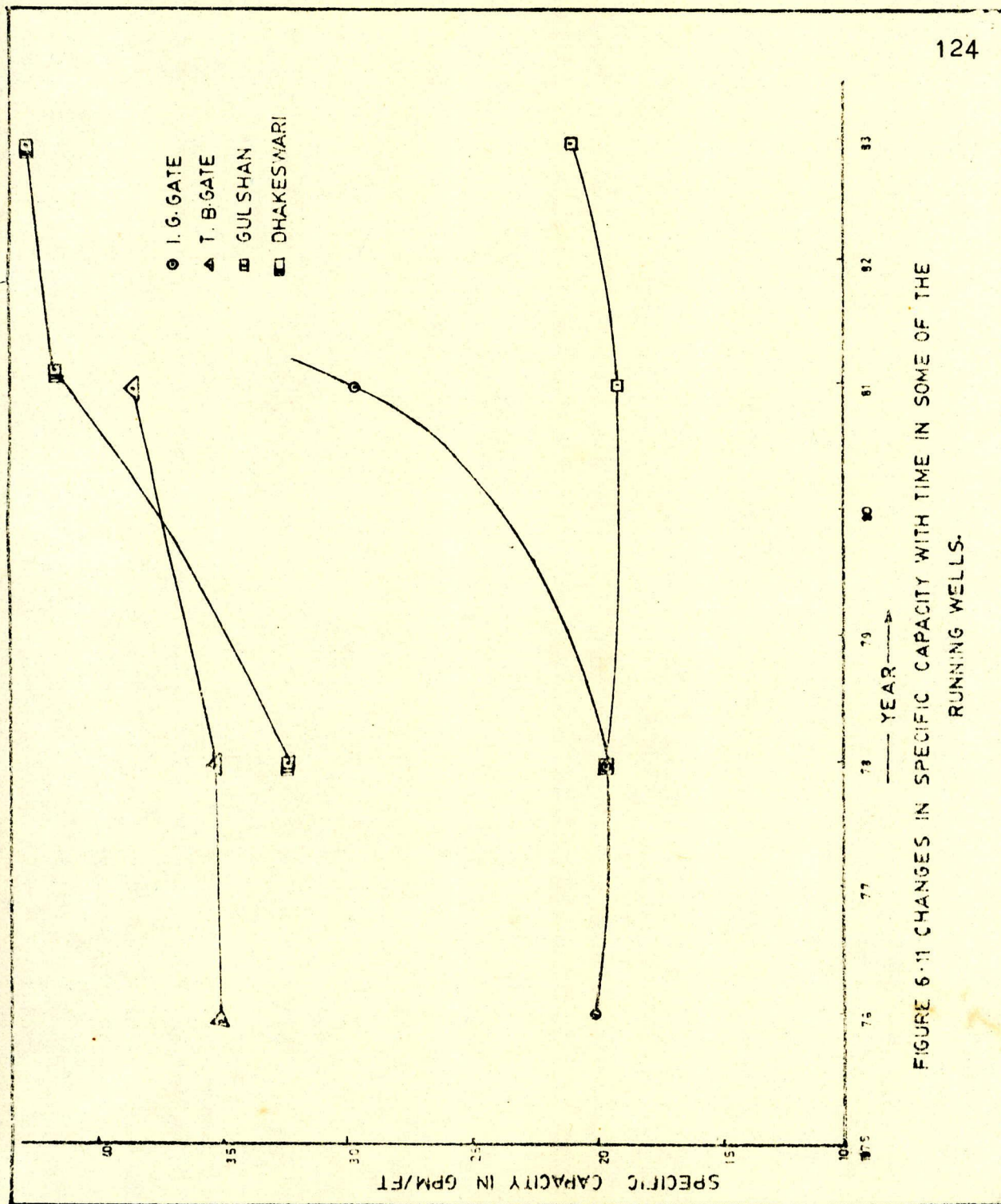


FIGURE 6-11 CHANGES IN SPECIFIC CAPACITY WITH TIME IN SOME OF THE RUNNING WELLS.

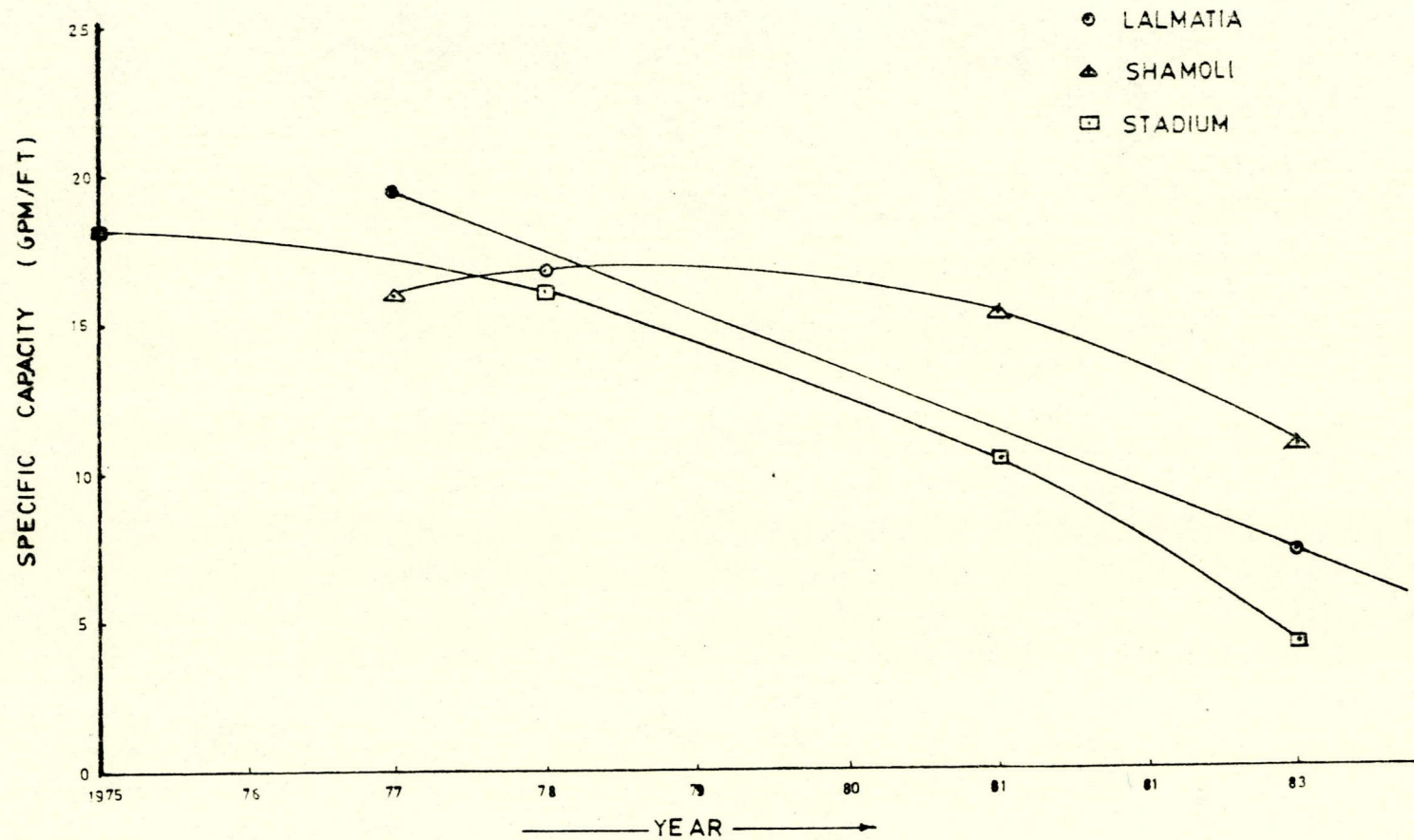
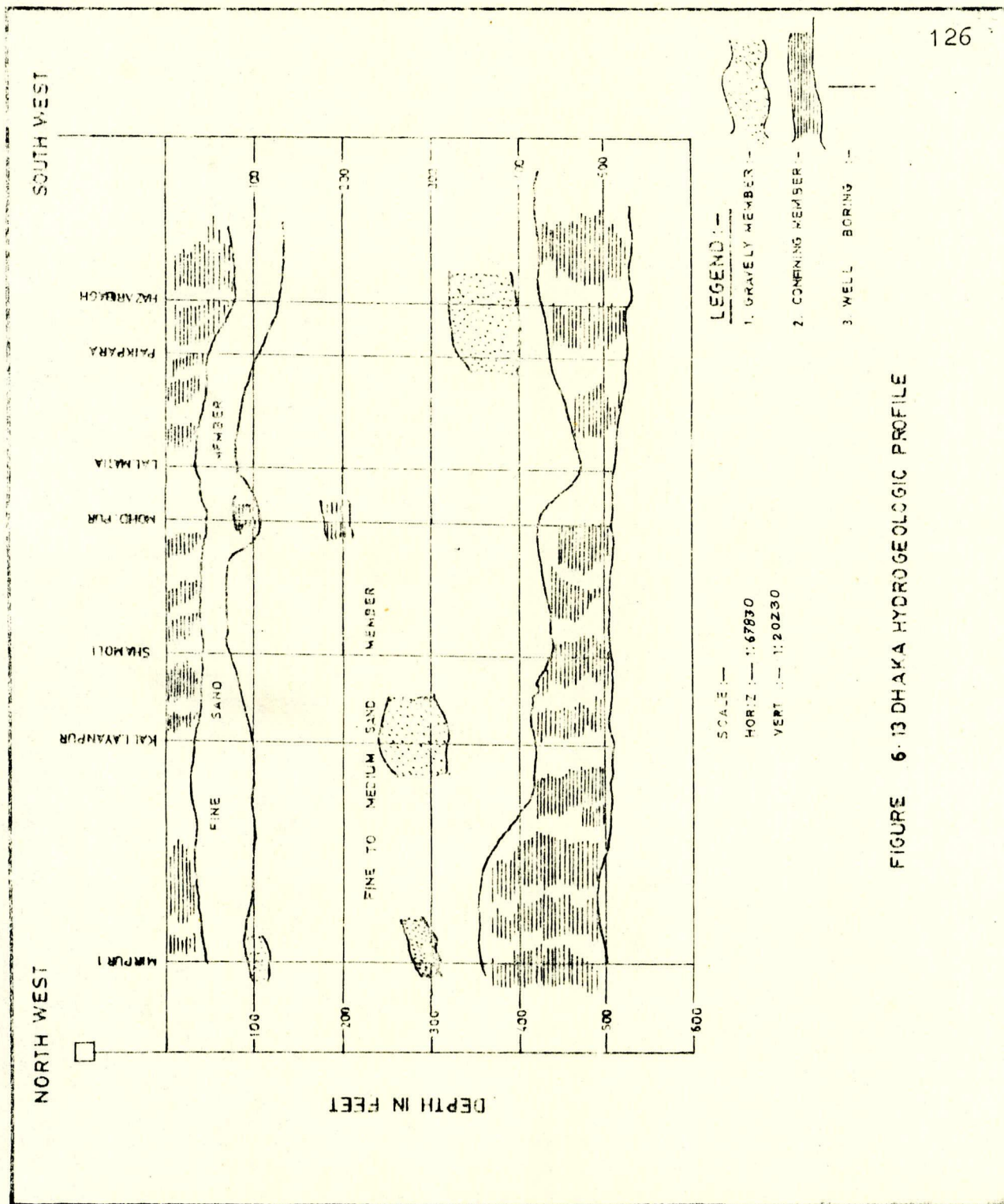


FIGURE 6-12 CHANGES IN SPECIFIC CAPACITY WITH TIME IN SOME OF THE EARLY DEFUNCT WELLS.



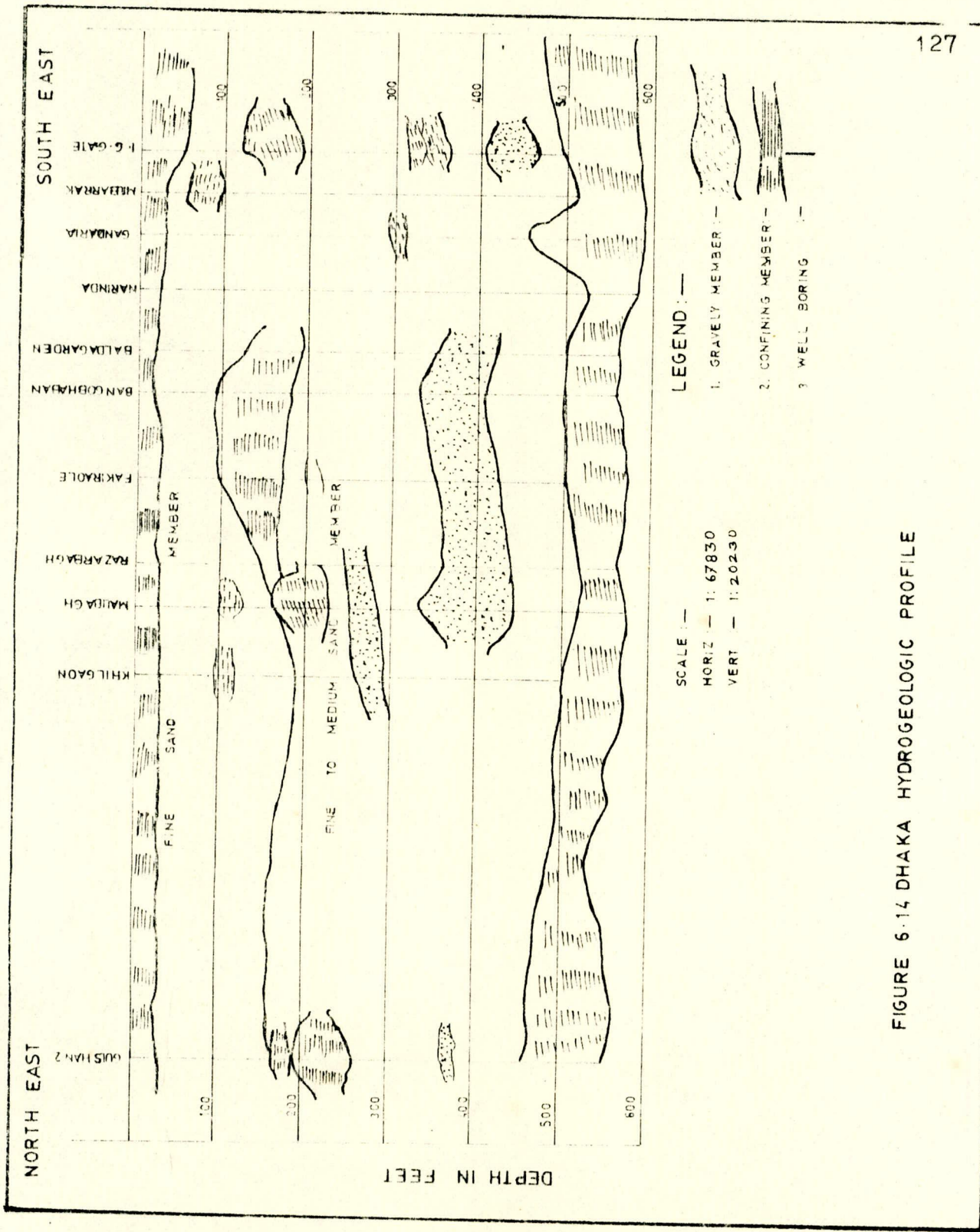
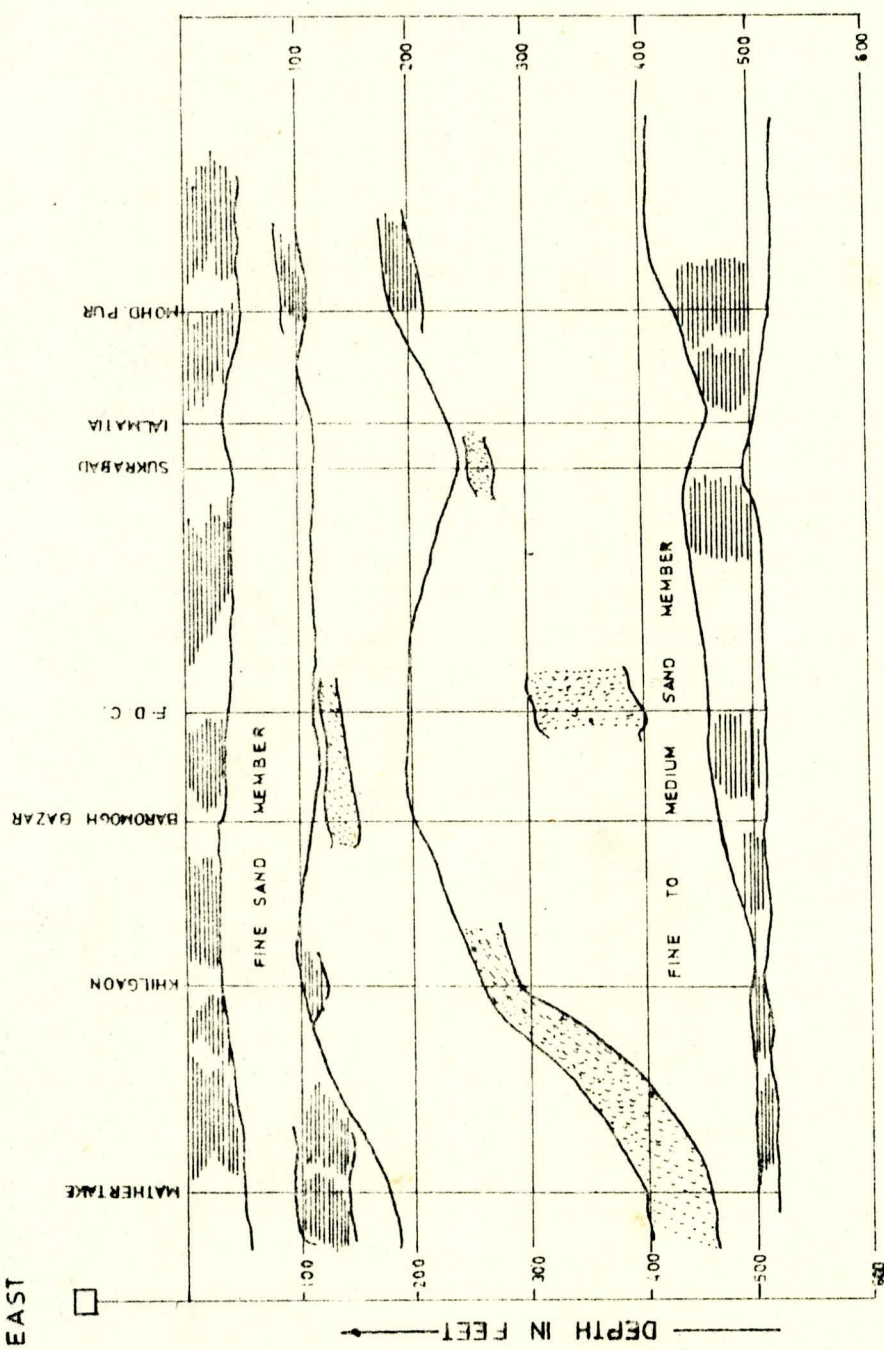


FIGURE 6.14 DHAKA HYDROGEOLOGIC PROFILE

WEST

EAST



SCALE —
HORIZ — 1:67830
VERT — 1:20230

LEGEND —

- 1. GRAVELLY MEMBER —
- 2. CONFINING MEMBER —
- 3. WELL BORING —

FIGURE 6.15 DHAKA HYDROGEOLOGIC PROFILE

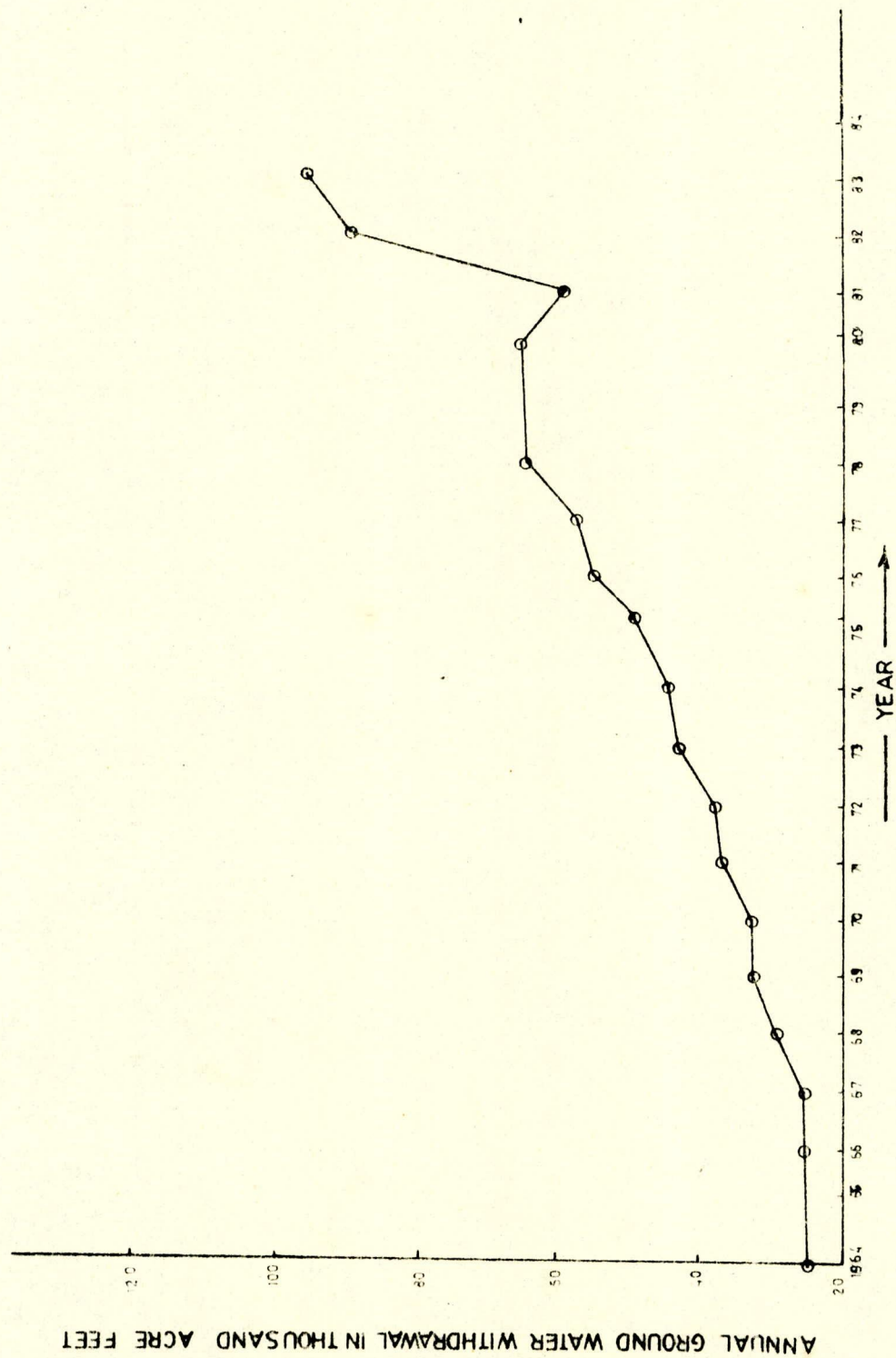


FIGURE 6.16. GROUND WATER WITHDRAWAL BY DHAKA WASA IN DIFFERENT YEAR.

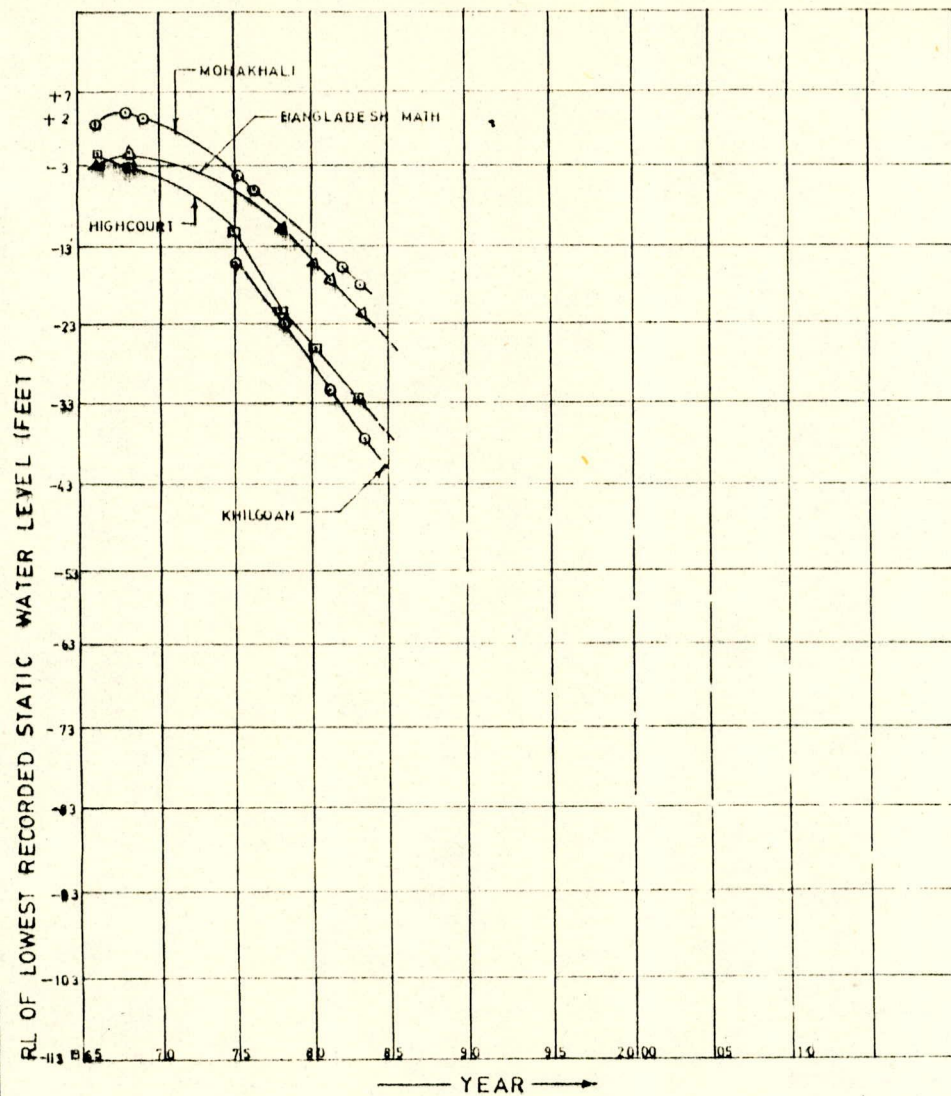


FIG. 6.17 LOWEST RECORDED STATIC GROUND WATER LEVEL DECLINE

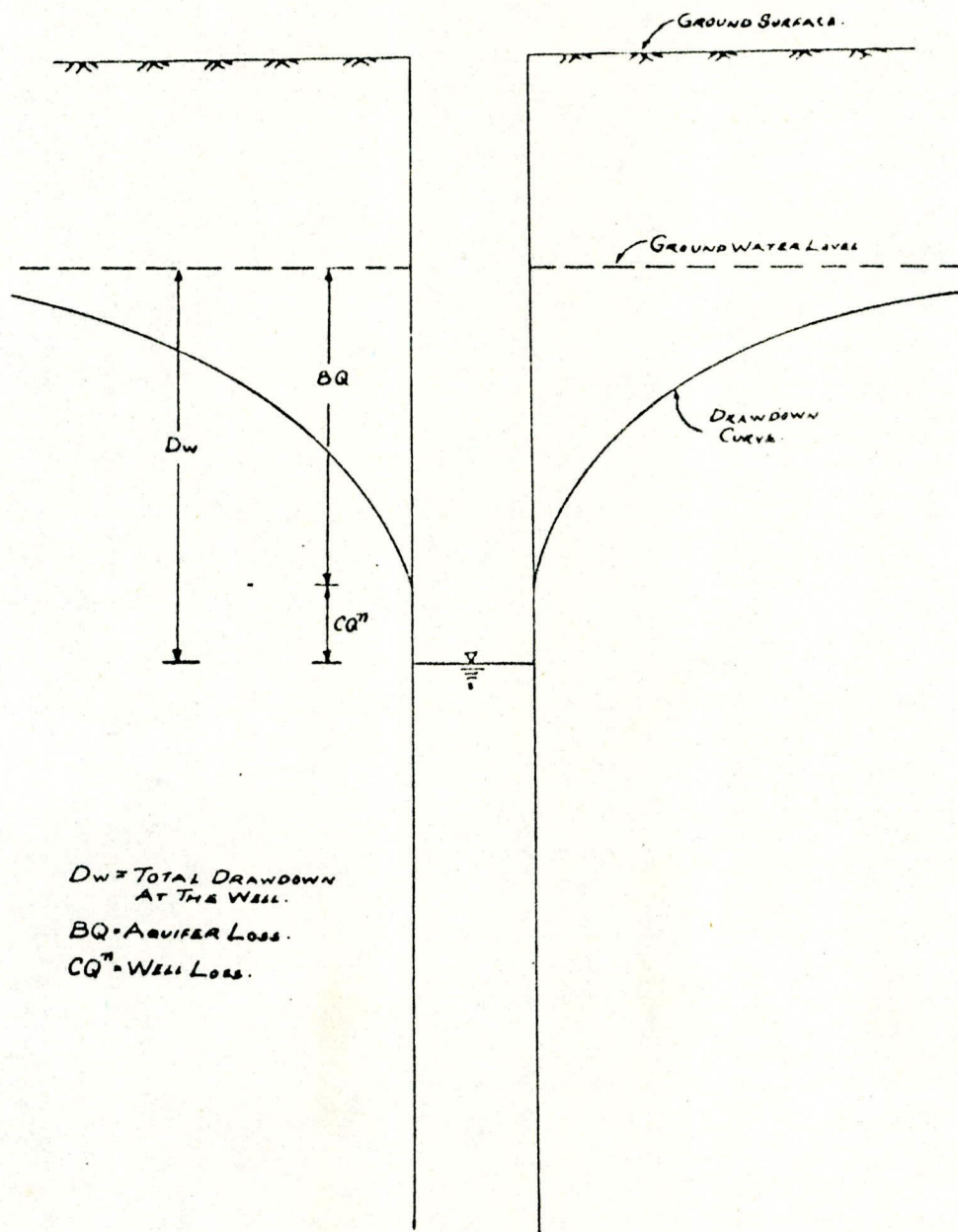


Fig: 6.18 WELL LOSS.

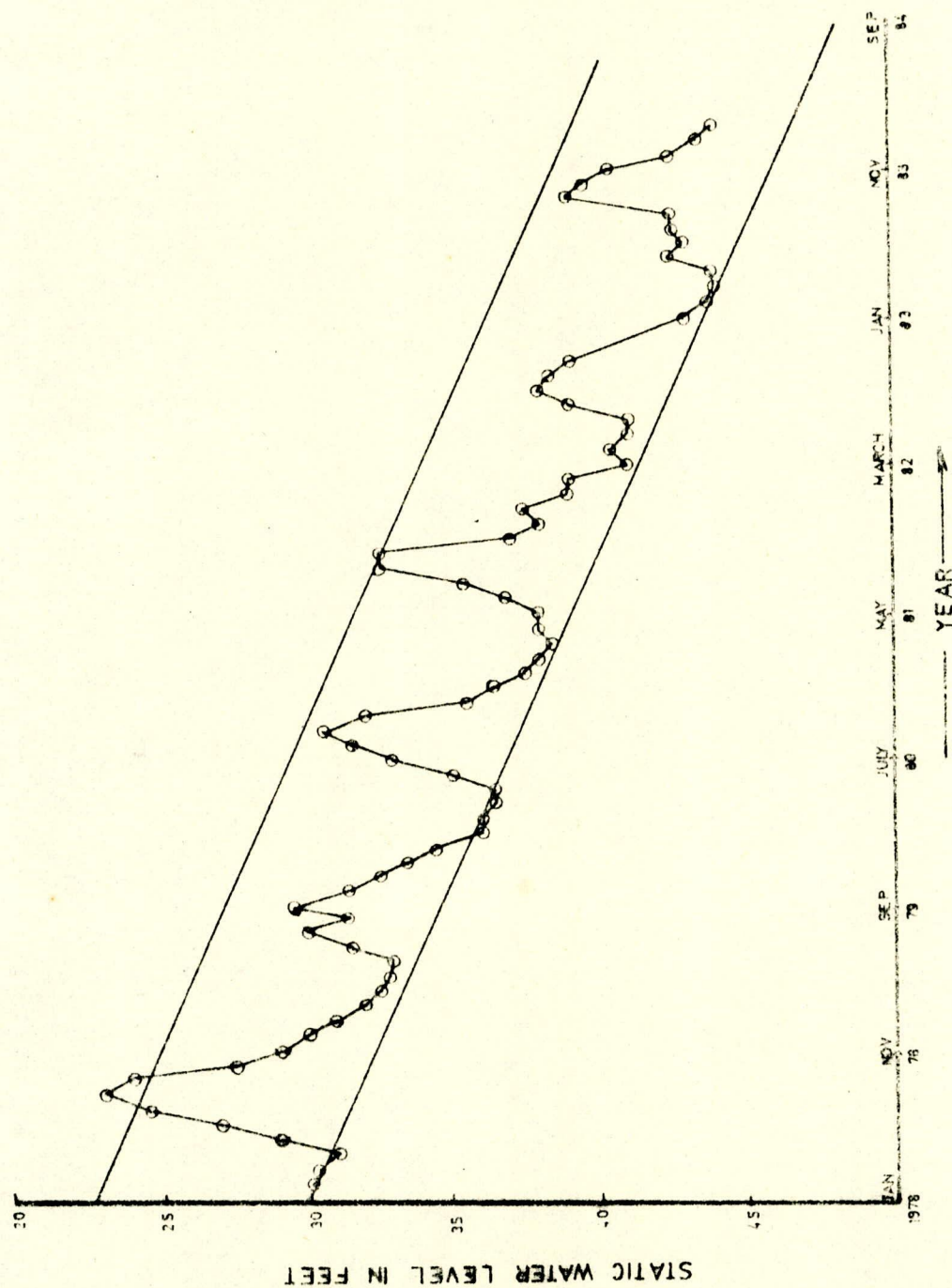


FIGURE 6-19 OBSERVATION WELL HYDROGRAPH AT MANIKHAGAR

TABLE - 2.1

SOIL CLASSIFICATION ACCORDING TO MIT, ISI,
ASTM AND U.S. BUREAU OF SOILS

Soil classification ISI AND MIT.				Size in m.m.
Gravel	...	...	...	72
Sand				
Coarse	...	...	...	2-0.6
Medium...	...	...	...	0.06-0.2
Fine ...		...	...	0.2-0.06
Silt				
Coarse...	...	...	...	0.06-0.02
Medium...	...	...		0.02-0.006
Fine ...	...	...	...	0.006-0.002
Clay	...	...	...	0.002
Gravel		ASTM		>4.75
Sand				
Coarse	...	...	...	4.75-200
Medium	...	...	...	2.00 to 0.425
Fine	...	...	...	0.425 to 0.075
Fines (combined silt and clay)	...	...	...	0.075
<u>U.S. Bureau of soils</u>				
Gravel	...	...	...	>4.75
Sand				
Coarse	...	...	...	4.75 - 2.00
Medium	...	...	...	2.0 - .42
Fine	...	...	...	0.42 - 0.074
Clay (Plastic) to silt (non-plastic)				<0.074

TABLE - 2.2

SOIL CLASSIFICATION ACCORDING TO GROUND
WATER BRANCH OF THE U.S. GEOLOGICAL SURVEY

Classification				Size
Gravel	...	...	...	0.080" and up
Very coarse sand	...	...	...	0.040" to 0.080"
Coarse sand...	...	...	...	0.020" to 0.040"
Medium sand...	...	...	...	0.010" to 0.020"
Fine sand ...	...	...	...	0.005" to 0.010"
Very fine sand	...	...	...	0.003" to 0.005"
Silt and clay	...	...	...	below 0.003"

TABLE - 2.3REPRESENTATIVE POROSITY RANGES FOR SEDIMENTARY MATERIALS

<u>Materials</u>			<u>Porosity per cent</u>
Soils	...	...	50-60
Clay	...	...	45-55
Silt	...	...	40-50
Medium to coarse	...	...	35-40
Mixed sand...	...	...	
Uniform sand	...	...	30-40
Fine to medium mixed sand..	...	...	30-35
Gravel	...	...	30-40
Gravel and sand	...	...	20-35
Sand stone...	...	...	10-20
Shale	...	...	1 -10
Lime stone...	...	...	1 -10

TABLE 2.4REPRESENTATIVE SPECIFIC YIELD RANGES
FOR SELECTED ROCKS

<u>Rocks</u>	<u>Specific Yield %</u>
Clay	1 - 10
Sand	10 - 30
Gravel	15 - 30
Sand and Gravel	15 - 25
Sand stone	5 - 15
Shale	0.5 - 5
Lime stone	0.5 - 5

TABLE 4.1OPTIMUM SCREEN ENTRANCE VELOCITIES

Coefficient of permeability, gpd/sq.ft.	Optimum screen entrance velocities fpm
6,000	12
6,000	11
5,000	10
4,000	9
3,000	8
2,000	7
2,000	6
1,000	5
1,000	4
500	3
500	2

1	2	3	4	5	6	7	8	9
15.	B.D.R. III. July/77	419'-0"	117'-0"	142'-0"	160'-0"	8	16	8
16.	Zikatola Aug./74	440'-0"	132'-0"	140'-0"	168'-0"	8	17	8
17.	Razar Bagh, Oct./74	481'-0"	170'-0"	139'-0"	172'-0"	8	17	8
18.	Mali Bagh, Apr./79	463'-0"	153'-0"	140'-0"	170'-0"	8	16	8
19.	Khilgaon IV, May/77	482'-0"	182'-0"	140'-0"	160'-0"	8	16	8
20.	Khilgaon V, March/80	507'-5"	221'-0"	140'-0"	146'-0"	8	16	8
21.	Elephant Road II, Oct./79	457'-3"	137'-0"	140'-0"	180'-0"	8	16	8
22.	Green Road III Aug./74	440'-0"	140'-0"	140'-0"	160'-0"	8	16	8
23.	Green Road IV Aug./77	240'-0"	100'-0"	140'-0"	180'-0"	8	17	8
24.	Mohammadpur, Nov/73	405'-0"	140'-0"	120'-0"	144'-0"	8	17	8
25.	Mohakhali P.H.Lab., Oct./70	443'-0"	153'-0"	120'-0"	170'-0"	8	16	8
26.	Lalmatia IV, Sept/83	410'-0"	170'-0"	160'-0"	180'-0"	8	16	8
27.	Shahjahanpur Feb/72	435'-0"	145'-0"	120'-0"	170'-0"	8	16	8
28.	Gulshan IV June/65	438'-0"	153'-0"	107'-0"	178'-0"	8	14	8
29.	High Court, Aug./74	460'-0"	160'-0"	120'-0"	160'-0"	8	16	8
30.	Bangobhobon II Jan./81	407'-0"	127'-0"	140'-0"	140'-0"	8	16	8
31.	Zimkhana, Apr./75	441'-0"	135'-0"	140'-0"	166'-0"	8	16	8
32.	Tejgaon VIII, Sept./75	484'-0"	164'-0"	140'-0"	180'-0"	8	16	8
33.	Bannani III, Oct/70	428'-0"	138'-0"	120'-0"	170'-0"	8	16	8
34.	Assembly House, Nov/79	436'-0"	124'-0"	142'-0"	170'-0"	8	16	8

1	2	3	4	5	6	7	8	9
35.	Dhanmandhi VI, Oct/68	382'-0"	129'-0"	101'-0"	152'-0"	8	15	8
36.	Dhupkhala Math, Oct/80	410'-0"	110'-0"	140'-0"	160'-0"	8	16	8
37.	Indira Road, Jul./80	425'-0"	145'-0"	140'-0"	140'-0"	8	16	8
38.	Hazari Bagh, April/80	467'-0"	147'-0"	140'-0"	180'-0"	8	16	8
39.	Lab. School, Nov./80	405'-0"	155'-0"	140'-0"	110'-0"	8	16	8
40.	Fulbaria, Sep./81	474'-0"	165'-0"	149'-0"	160'-0"	8	16	8
41.	Fakira pool, Aug/81	455'-0"	125'-0"	160'-0"	170'-0"	8	16	8
42.	Saidabad, Jan/81	468'-0"	148'-0"	140'-0"	180'-0"	8	16	8
43.	Bangla College, Nov./80	320'-0"	60'-9"	140'-0"	120'-0"	8	16	8
44.	Balda Garden, May/82	505'-0"	145'-0"	160'-0"	200'-0"	8	16.75	8
45.	Madertake Feb./82	415'-0"	85'-9.5"	159'-2½"	170'-0"	8	16	8
46.	Simson Road, Aug./82	463'-0"	123'-0"	160'-0"	180'-0"	8	15.75	8
47.	Gandaria, Jun/82	445'-0"	150'-0"	155'-0"	140'-0"	8	16	8
48.	Sen Para East, Jan/82	425'-11½"	106'-0"	159'-11½"	160'-0"	8	16	8
49.	Kallayenpur, Sept./82	410'-0"	80'-0"	160'-0"	170'-0"	8	16	8
50.	Urdo Road, Oct/82	447'-0"	107'-0"	160'-0"	180'-0"	8	16	8
51.	Uttara Model Town, Oct./82	445'-0"	106'-0"	167'-0"	180'-0"	8	16	8
52.	Razar Bagh, Sept/83	518'-0"	198'-0"	160'-0"	160'-0"	8	16	8
53.	Narinda, Dec./64	438'-0"	132'-0"	107'-0"	179'-0"	8	14	8
54.	.I.G. Gate, Dec./61	387'-0"	50'-7"	107'-0"	230'-0"	8	14	8
55.	Mill Barake, Dec./76	480'-0"	160'-5"	140'-0"	180'-0"	8	16	8
56.	Armanitola, Feb/77	445'-0"	135'-0"	140'-0"	170'-0"	8	16	8
57.	Bangladesh Math, Feb/77	445'-0"	135'-0"	140'-0"	170'-0"	8	16	8
58.	Rahmathullah School, April, /77	437'-0"	127'-0"	140'-0"	170'-0"	8	16	8

1	2	3	4	5	6	7	8	9
59. Dhakeshwari, Jun/61	387'-0"	150'-0"	107'-0"	230'-0"	8	16	8	
60. B.D.R. II, Oct/70	399'-0"	119'-0"	110'-0"	170'-0"	8	16	8	
61. Rayer Bazar, Mar/79	430'-0"	110'-0"	140'-0"	180'-0"	8	16	8	
62. Tali Road, Oct/70	374'-6"	84'-0"	120'-0"	170'-0"	8	16	8	
63. Asad Gate, Feb/71	376'-7"	86'-0"	120'-0"	170'-0"	8	16	8	
64. Khilgaon, Jun/65	438'-0"	152'-0"	107'-0"	179'-0"	8	16	8	
65. Kakrail (O.H.T), Dec/71	408'-5"	159'-0"	118'-0"	131'-0"	8	15	8	
66. Dhanmindi VIII, Jun/72	375'-0"	95'-0"	120'-0"	160'-0"	8	16	8	
67. Mohammadpur VIII, Nov/73	405'-0"	140'-0"	120'-0"	144'-0"	8	17	8	
68. Medical College, Nov/67	450'-0"	215'-0"	91'-0"	144'-0"	8	14	8	
69. T.B. Gate, Oct./70	443'-0"	153'-0"	120'-0"	170'-0"	8	16	8	
70. Lalmatia II, Jan/75	405'-0"	206'-0"	139'-09"	160'-0"	8	16	8	
71. Bashaboo, Sept/70	415'-0"	125'-0"	120'-0"	170'-0"	8	16	8	
72. Lichu Baghan, Sept/70	435'-0"	145'-0"	120'-0"	172'-0"	8	16	8	
73. Kawran Bagar, Apr/70	349'-0"	155'-0"	132'-0"	162'-0"	8	16	8	
74. Lulshan V, Aug./72	402'-0"	134'-0"	114'-0"	164'-0"	8	16	8	
75. Tajgaon VI, Oct/70	378'-0"	88'-0"	120'-0"	170'-0"	8	14	8	
76. Bannani IV, May/77	443'-0"	123'-0"	140'-0"	180'-0"	8	16	8	
77. Lalashorai, Jun/80	432'-0"	72'-0"	160'-0"	200'-0"	8	16	8	
78. Uttara I, Apr/77	445'-0"	149'-0"	136'-0"	160'-0"	8	16	8	
79. Dhonia, Sept/81	435'-0"	118'-0"	137'-0"	180'-0"	8	16	8	
80. Jurain, Nov/81	485'-0"	187'-0"	158'-0"	130'-0"	8	16	8	
81. Bijoy Nagar,	428'-0"	118'-0"	140'-0"	180'-0"	8	16	8	
82. Khilgaon, Oct./70	330'-0"	93'-0"	120'-0"	177'-0"	8	16	8	
83. Hatir pool, Oct/70	391'-0"	101'-0"	120'-0"	170'-0"	8	16	8	

1	2	3	4	5	6	7	8	9
84.	Babar Road, Sept/70	410'-0"	120'-0"	120'-0"	170'-0"	8	16	8
85.	Boro Mogh Bazar, II June/72	346'-6"	74'-0"	120'-0"	152'-0"	8	16	8
86.	Sen Para West, Nov/82	410'- $\frac{1}{2}$ "	70'-10"	160'-0"	180'-0"	8	16	8
87.	Boro Mogh Bazar, April/82	505'-0"	144'-0"	161'-0"	200'-0"	8	16.75	8
88.	Cantonment, Mar/82	468'-0"	128'-0"	140'-0"	200'-0"	8	15.75	8
89.	Azimpur V, Jul/73	425'-0"	155'-0"	120'-0"	150'-0"	8	14	8
90.	Pallabi II, Dec./80	405'-0"	109'-0"	140'-0"	136'-0"	8	16	8
91.	Goran, Oct/80	425'-0"	117'-0"	140'-0"	168'-0"	8	16	8
92.	Stadium, Aug./75	483'-9"	181'-9"	140'-0"	162'-0"	8	16	8
93.	Lalmatia III, Feb/77	445'-0"	135'-0"	140'-0"	170'-0"	8	16	8
94.	Mirpur Sec.13, Jun/83	441'-0"	101'-0"	160'-0"	180'-0"	8	18	8
95.	F.D.C., Jan/83	468'-0"	148'-0"	160'-0"	160'-0"	8	18	8
96.	Mirpur Sec.8, Apr/83	351'-3"	41'-3"	160'-0"	150'-0"	8	18	8
97.	Pagla, Mar/83	523'-5"	183'-5"	160'-0"	180'-0"	8	18	8
98.	Ulon, Jan/83	461'-0"	141'-0"	160'-0"	160'-0"	8	18	8

TABLE 6.1

DIFFERENCE BETWEEN THEORITICAL DESIGN AND ACTUAL
CONSTRUCTION IN SOME OF WASA WELL

Sl. No.	Name of well	d ₃₀ at depth	Grain size distribution of gravel pack as per design in m.m.					Slot size as per design	Approximate distribution of gravel pack used in m.m.					Slot size used	Year of ins- talla- tion	Year of fail- ure
			d ₁₀	d ₃₀	d ₅₀	d ₆₀	d ₉₀		d ₁₀	d ₃₀	d ₅₀	d ₆₀	d ₉₀			
1.	Shamoli	d ₃₀ at depth 405'-410' & 420'-425' = 0.23 [Cu < 2]	.75	.92	1.1	1.2	1.9	30"/1000	1.4	2.5	3.45	3.8	5 50"/1000		1978	1981
2.	Lalmatia	d ₃₀ at 280'-285' 0.26 m.m. [Cu = 2]	1.15	1.56	2.0	2.4	3.5	45"/1000		Ditto				,,	1977	1983
3.	Hazari bagh	d ₃₀ at 240'-245' & 270'-275' ft are 0.25 m.m. [Cu > 2]	1.10	1.45	2.0	2.3	3.1	44"/1000		Ditto				,,	1976	1982

TABLE 6.2

VALUES OF WELL LOSS CONSTANT (C) AT DIFFERENT
WASA DEEP TUBEWELLS

Sl. No.	Name of Well with year of installation	Value of 'C'			Average value of C	Well loss as % of total loss at 2 cfs rate of pumping
		1st step	2nd step	3rd step		
1.	Abul Hasnat Road (1983)	-1.96	+ 4.168	+ 1.16	+ 2.664	29.8
2.	Banani-4 (1977)	- .585	+ 2.17	+ 1.66	+ 1.92	29.55
3.	Baily Road(1983)	+1.04	+ 1.34	- 5.84	+ 1.19	15.69
4.	Farashganj (1980)	+1.27	- 4.32	- 1.34	+ 1.27	17.17
5.	Hazaribagh (1976)	-1.83	- 2.5	+ 6.16	+ 6.16	71
6.	Hazaribagh (1983)	+1.24	- 1.07	+ 1.16	+ 1.2	24
7.	Fallanpur (1982)	+0.83	+ 8	-17.20	+ 4.42	23.89
8.	Ialmatia-III (1977)	-2	- 7.32	-2.93	- ve	- ve
9.	Ialmatia-III (1983)	+0.4	+ 0.16	+5.34	+ 1.96	24
10.	Mill Barrak (1977)	- .165	+ 2.34	-10.68	+ 2.34	19.1
11.	Nawabpur (1976)	-1.22	+ 6.32	-0.5	+ 6.32	67.4
12.	National assembly (1973)	+ .55	- 1	+1	+ .775	11.8
13.	Pallabi (1980)	-3.77	-10.07	+4.07	+ 4.07	32.65
14.	Rajarbagh (1983)	+2.75	-3.84	+4.34	+ 3.545	48.89
15.	Shenpara(East)(1982)	+3.35	-4.66	+1.66	+2.5	16
16.	Shanoli (1977)	-10.44	-0.32	+1	+ 1	06
17.	Shenpara(West)(1982)	+ .69	-1	+1	+ .845	6.89
18.	Stadium (1975)	-0.06	+ .68	+1.32	+ 1.0	08
19.	Simson Road(1982)	+0.58	+0.28	+0.22	+ 0.36	06
20.	Tejgaon Industrial area	+3.78	-11.84	+7	+ 5.39	48.91

TABLE 6.3

STORAGE COEFFICIENT (S), COEFFICIENT OF
TRANSMISSIBILITY AND RADIUS OF INFLUENCE
AT DIFFERENT PLACES INSIDE THE STUDY AREA

Sl. No.	Name	Depth of Tube-well	Total Length of strainer	Storage Coefficient (S)	Coefficient of Transmissibility (T) in gpd/ft	Approximate radius of influence (r), in feet
1.	Mohakhali	443'-0"	170'-0"	0.129	0.83×10^5	4770
2.	Balda Garden	505'-0"	200'-0"	0.080	0.42×10^5	5460
3.	Mill Barrak	480'-0"	180'-0"	0.06	1.46×10^5	7210
4.	AZimpur	425'-0"	150'-0"	0.0566	0.35×10^5	6200
5.	Rajar Bagh	518'-0"	160'-0"	0.045	1.1×10^5	8300
6.	Mirpur	405'-0"	136'-0"	0.036	0.44×10^5	8100
7.	Zymkhana	441'-0"	166'-0"	0.022	0.55×10^5	10800
8.	Dhanmandi	375'-0"	150'-0"	0.013	0.52×10^5	14500

TABLE 6.4

SPECIFIC CAPACITY AND COEFFICIENT OF TRANSMISSIBILITY
(T) AS OBSERVED IN DIFFERENT WASA WELLS

Sl. No.	Name of the well with year of installation	Specific Capacity (gpm/ft) during test pumping just after construction	Coefficient of Transmissibility 'T' (gpd/ft)
1.	Abul Hasnat Road(1983)	25.00	0.87×10^5
2.	Bannani-4 (1977)	34.72	0.712×10^5
3.	Baily Road (1979)	30.85	1.65×10^5
4.	Farashganj (1980)	23.42	0.864×10^5
5.	Hazaribagh (1976)	26.00	0.648×10^5
6.	Hazaribagh (1983)	45.56	1.42×10^5
7.	Kallanpur (1982)	11.95	0.324×10^5
8.	Lalmatia-III(1977)	19.56	0.58×10^5
9.	Lalmatia-III(1983)	28.27	1.15×10^5
10.	Mill Barrak (1977)	18.32	1.46×10^5
11.	Nawabpur (1976)	24.00	0.264×10^5
12.	National Assembly (1977)	34.28	0.264×10^5
13.	Pallabi (1980)	18.44	0.95×10^5
14.	Rajarbagh (1983)	31.03	3.56×10^5
15.	Shenpara East (1982)	18.36	1.01×10^5
16.	Shamoli (1977)	13.30	0.31×10^5
17.	Stadium (1975)	18.18	0.61×10^5
18.	Shenpara West (1982)	18.55	0.548×10^5
19.	Simson Road (1982)	37.64	1.78×10^5
20.	Tejgaon Industrial Area	20.42	0.218×10^5

TABLE 6.5ANNUAL GROUNDWATER WITHDRAWAL BY DHAKA WASA

Year	Annual withdrawal (Acre-feet)
1964	24200
1966	25000
1967	24800
1968	29200
1969	32100
1970	32200
1971	36300
1972	37800
1973	42300
1974	44000
1975	49000
1976	54400
1977	57000
1978	64500
1980	64200
1981	58250
1982	89620
1983	95200

TABLE 6.6

CHANGE IN ELEVATION OF GROUND WATER LEVELS, LOWEST RECORDED
(IN FEET) WITH TIME (MEAN SEA LEVEL DETUM)

Sl. No.	Name of well	1966	1968	1975	1978	1980	1981	1982	1983
1.	Mohakhali	+2.3	+ 3	- 4	- 4.8	- 12	- 14	- 16	- 18
2.	Bangladesh Math	-2.5	- 1.3	- 5	-10.4	- 16	- 17	- 20	- 22
3.	High Court	-2.0	- 2.5	-11.5	-21.5	- 26.3	- 28	- 31	- 34
4.	Khilgeon	-	-	-15	-23	- 28	- 31	- 34	- 37

TABLE 6.7

ANNUAL WITHDRAWAL AND CROSSPONDING DECLINE OF GROUND WATER (LOWEST RECORDED) LEVEL (SWL)
IN FOUR DIFFERENT DEEP TUBEWELL IN SIDE THE STUDY AREA

Year	Annual with drawal in acre ft	Average annual with- drawal in acre ft	High Court D.T.W.		Bangladesh Math		Mohakhali D.T.W.		Khilgaon D.T.W.		Consolidated average decline in SWL in inch
			RL.of S.W.L. in ft.	Average decline in S.W.L. in inch	RL.of S.W.L. in ft.	Average decline in S.W.L. in inch	RL.of S.W.L. in ft.	Average decline in SWL in inch	RL.of SWL in ft.	Average decline in SWL in inch	
1964	24200	-	-	-	-	-	-	-	-	-	-
1966	25000	24600	- 2	-	- 2	-	+ 2	-	-	-	-
1967	24800	24900	- 2.1	+ 1.2	- 2	- 0	+ 3	- 12	-	-	-
1968	29200	27000	- 2.75	+ 7.8	- 2	- 0	+ 3	- 0	-	-	-
1969	32100	30650	- 4	+15	- 2	+ 0	+ 3	+ 0	-	-	5
1970	32200	32150	- 5	+12	- 2	0	+ 2	+ 12	-	-	8
1971	36300	34250	- 6	+12	- 2.75	+ 9	+ 1	+ 12	-	-	11
1972	37800	37050	- 7.25	+15	- 3.1	+ 4.2	0	+ 12	-	-	10.4
1973	42300	40050	- 8.5	+15	- 4	+ 10.8	- 1	+ 12	-	-	12.6
1974	44000	43150	- 10	+18	- 4.5	+ 6	- 2.5	+ 18	-	-	14
1975	49000	46500	- 11.5	+18	- 5	+ 6	- 4	+ 18	- 15	-	14
1976	54400	51700	- 14	+30	- 7	+ 24	- 6	+ 24	- 17.5	30	27
1977	57000	55700	- 18	+48	- 8.75	+ 21	- 7.5	+ 18	- 20	30	29.25
1978	64500	60750	- 21	+36	- 10.5	+ 21	- 9	+ 18	- 22	24	24.95
1980	64200	64350	- 26.5	+33	- 16	+ 33	- 12	+ 18	- 28	36	30
1981	58250	61225	- 30	+42	- 17	+ 12	- 14	+ 24	- 31	36	28.5
1982	89620	73935	- 31	+12	- 21	+ 48	- 16	+ 24	- 35	48	33
1983	95200	92410	- 33	+12	- 21	+ 0	- 18	+ 24	- 37	24	15

TABLE 6.8

AVERAGE RATE OF ANNUAL WITHDRAWAL AND
CORRESPONDING RATE OF LOWERING OF THE
LOWEST RECORDED WATER LEVEL IN FOUR
DIFFERENT TUBEWELLS

Sl. No.	Year	Average annual withdrawal (Acre/feet)	Average annual mining (Inch)			
			High Court D.T.W.	Bangladesh Math.D.T.W.	Mohakhali D.T.W.	Khil gaon D.T.
1.	1964-68	25640	1.8	0	0	-
2.	1969-73	36140	13.8	4.8	9.6	-
3.	1974-78	53780	30.0	15.6	19.2	24
4.	1979-83	75484	28.8	25.2	21.6	36

