A Study of Land Drainage by Pumping from an Experimental Drainage Well in the Delta Area, Utah

Ellaf Arni Olafson

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A STUDY OF LAND DRAINAGE BY PUMPING
FROM AN EXPERIMENTAL DRAINAGE WELL IN THE DELTA AREA, UTAH

by

Ellaf Arni Olafson

A thesis submitted in partial fulfillment of the requirements for the degree of
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in
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1948

Utah State Agricultural College
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Logan, Utah
May 1948
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A STUDY OF LAND DRAINAGE BY PUMPING FROM AN EXPERIMENTAL DRAINAGE WELL IN THE DELTA AREA, UTAH

INTRODUCTION

Although it is now generally accepted that, in the west, irrigation and drainage are necessarily complementary practices, the realization has been slow in developing. Recent estimates indicate that about 8 million acres of land under irrigation in the 17 western states require drainage. (8) For most irrigated lands a depth to groundwater of at least five to ten feet is desirable. Very high capital as well as annual maintenance costs would be involved in meeting this minimum requirement with the usual types of tile and open drains. Indeed, in most instances it cannot economically be accomplished. The purpose of this study is to determine the degree to which pumping groundwater, so successful in certain other areas, can contribute to the solution of the drainage problem in the Delta Area, Utah. The data presented herein have been collected in meeting one of the objectives of a cooperative research agreement between the four Millard County drainage districts, the U. S. Regional Salinity Laboratory, and Utah Agricultural Experiment Station, namely, "to study the feasibility and costs of drainage by pumping from wells in the area". (15)

Location and Extent of Area

The Delta Area, in Millard County, Utah, comprises about 115,000 acres, of which about 80,000 acres are included in drainage districts, and of which approximately 35,000 acres are irrigated. The average annual rainfall is approximately eight inches and irrigation is essential to satisfactory crop production. The Sevier River, which winds through the region in a south-westerly direction, is the source of irrigation water and also provides the only natural drainage. For the most part the area is a desert plain about 4600 feet in elevation with comparatively little slope.
Delta Area Drainage Experiences

"Irrigation was begun in the area about 1860 and gradually expanded until about 1905 when it was given a new impetus." (6) The groundwater, usually comparatively near the ground surface due to the geological structure of the valley and plain as described later, gradually rose to points near the land surface over large areas. The high evaporation losses from these moist soils resulted in surface concentration of alkaline salts and decreased productivity. The flat terrain, the relatively heavy and impermeable soils, and the high salt content of the irrigation water complicate the drainage problem.

The four drainage districts which are still in operation were organized between 1914 and 1918. A large tile drainage construction program was carried out mainly during the period of World War I high prices. The cycle of low prices for agricultural products and inadequate water supplies which followed resulted in abandonment of lands and a disregard for the need of drainage. With the return of adequate water supplies and the re-occurrence of waterlogging during the last decade, has come a more general realization that drainage is a vital problem.

The recent construction of open drains, in many cases cutting the existing tile lines at right angles to make them function again, has made it possible to continue farming operations in the Delta Area. Since open drains and tile lines can only be constructed at shallow depths, the groundwater cannot always be maintained below the minimum desired level, which in this area is considered by local officials to be 5 feet. (16) It is believed, therefore, that the present drainage facilities in the Delta Area are inadequate.

Advantages of Pumping Groundwater

Further objections to open drains are the unsightliness, the value of the land which they withhold from cultivation, and the high maintenance
costs due to weed growth and instability of sideslopes. Where drainage by pumping is feasible, it offers the following advantages:

1. The value for irrigation use of the water pumped materially lowers, if it does not completely offset, the cost of drainage provided the quality is such that the water can be used for irrigation. Further, and subject to the same provision, the extra water made available by pumping constitutes a natural resource which should be utilized. Particularly is this the case in areas where the amount of water available is the limiting factor in the acreage of land irrigated.

2. The water table can be more closely controlled than by any other method.

3. Maintenance costs are low.

4. Power, the chief annual cost, has steadily become cheaper over the years. The trend promises to continue. This is a stabilizing economic factor in pumping.

5. Experience in areas which have pumped groundwater extensively indicates that capital costs are materially lower for drainage by pumping than for any other method.

6. Almost no land is withheld from cultivation.

Experiences of Other Areas

Experience in other parts of the western United States has demonstrated that drainage is seldom a problem where an appreciable part of the irrigation water is pumped from wells in the immediate area. (16) Several areas which began pumping as a drainage measure now pump almost solely for irrigation water supplies. Drainage has been automatically taken care of except in minor areas where pumping primarily for drainage has had to be continued. Examples are the Salt River Valley in Arizona and the San Joaquin Valley in California.
Around Phoenix, Arizona, drainage became a problem about 1918, some seven years after the completion of Roosevelt Dam greatly expanded irrigation in the area. The Salt River Valley Water Users' Association decided to pump groundwater. (8) Figure 2 shows the water table at its highest point at the beginning of 1920, and its steady decline with increased pumping since that date. In 1946 the association pumped one-third of its water supplies, and the depth to groundwater was greater than fifty feet. Indeed, the present concern is not drainage but the recharge of the groundwater reservoir.

The experiences of the Modesto Irrigation District in California are illustrated in the following summary:

"The total cost of construction for wells, pumps and discharge pipe lines, up to and including 1939, was $159,000 as compared with $308,000 previously cited as cost of gravity drains. The cost of operation and maintenance for the pump system totaled $60,050 and similar cost for gravity drains equaled $148,700. The power cost for operating pumps totaled $393,100.

"On the basis of 50,000 acres, which is the area subject to a high water table, the cost per acre, including $4.38 for construction, maintenance and operation, and $7.86 for power cost, is $12.24 for drainage pumps as compared with $9.13 per acre expended for gravity drains. During the period in which the District has operated the pumps, a total of 602,000 acre-feet of water has been pumped, equivalent to more than twice the capacity of Don Pedro Reservoir. Approximately 75% of all pumped water has been used for irrigation. At the rate of $1.36 per acre-foot, the pumped water which was used instead of gravity water for irrigation would have a potential value of $612,150, which would entirely offset all drainage pump costs." (3)

"The Modesto experience leads to the conclusion that the operation of deep-well pumps is not only a most satisfactory method of sub-surface drainage, but also a self-liquidating project." (8)

Study of Geological Formation Important

In order to pump successfully from wells, there must be water-bearing strata coarse enough to yield water readily. It becomes of prime importance then to study the geology of the Delta Area and thus determine the areas
in which pumping from wells is most likely to be feasible from this standpoint. The U. S. Geological Survey has made such a study of Millary County which is described by O. E. Meinzer, and summarized in the following section. (12)
GEOLGY OF THE DELTA AREA

General

Utah lies in two major physiographic regions—the Plateau and Basin provinces. Millard County lies in the Basin province consisting of a desert plain interrupted by more or less parallel and isolated mountain ranges. (See Figure 3) Weathering and stream and lake action have combined to gradually wear down the mountains and fill the intervening basins.

Stream Deposits

When a stream escapes from its canyon and its carrying power diminishes, it drops the coarsest part of its load first and conveys the finest sediments farthest into the valley or desert. Hence the upper parts of the alluvial slopes consist largely of gravel and boulders, and the parts most remote from the mouths of the canyons are underlain by beds of fine clay and sand associated with little gravel and no boulders. In most localities the stream deposits include beds of sand and gravel which are capable of yielding water freely. As the distance from the mountains increases, the number and thickness of these beds decrease, the constituent particles become smaller, and their yield of water becomes less copious; but fairly abundant supplies can generally be obtained even in the valley flats.

Lake Deposits

In the quiet waters of a lake, the gravel and sand brought by streams are deposited near the shore and only very fine particles which remain in suspension for a long period of time reach points remote from the shore. Hence lake deposits are likely to consist so largely of beds of clay and fine-grained quicksand that they will yield only meagre supplies of water. This condition is imminent in the region designated the Sevier Desert.

A Typical Valley and Desert Flat

A typical valley of this region consists of a rock trough partly
filled with sediments so dispersed as to form alluvial slopes on each side with a flat between. Stream deposits underlie the alluvial slopes but lake deposits may occur at the centre. Water coming from the mountains is readily absorbed by the porous gravel of the alluvial slopes and transmitted to the pervious strata underlying the valley ground surface. On the central flats and the lower parts of the alluvial slopes the groundwater is therefore near the ground surface and can easily be obtained by sinking wells into the unconsolidated sediments. The bedrock serves as a confining basin.

The deserts differ from the valleys mainly in that they have more extensive central flats with a more important development of lake deposits of non-water-bearing clay and quicksand. In many cases salty water is encountered in the water-bearing materials. (See Figure 4)

The Sevier Desert

The Delta Area lies in just such a desert plain nearly surrounded by mountain ranges which form a closed basin and designated by Meinzer as the Sevier Desert. As defined by Meinzer, the area is bounded on the north by the Juab-Millard County line, on the east by the Canyon Range, on the south by a line passing through Pavant Butte and the north end of Crickett Mountains, and on the west by the Little Drum Mountains and Swasey Wash. (See Figure 3)

Lynn Bench

When the Sevier Desert was submerged beneath Lake Bonneville during the Provo stage, the Sevier River, on emerging from its canyon at Leamington, built its deposits into an extended and somewhat complex delta. The upper portion, consisting of a relatively level upland tract, has been named Lynn Bench. It consists largely of sandy and gravelly material which absorbs much of the rainfall and is likely the principal
hydrologic contributor to the groundwater of the lower areas. The southwestern border of the bench is a clearly recognizable gradual slope approximately one-half mile north of the town of Delta and one and one-half miles northeast of the location of the experimental drainage well at Sutherland.

The Desert Flat

The adjacent flat is a vast featureless tract with little slope for the most part. The Sevier River empties into Sevier Lake far to the southwest and there are other minor, shallow, isolated depressions which serve as local drainage areas.

At Deseret and other settlements not too far removed from the base of the bench, many successful wells have been drilled. The sediments consist mainly of clay with very little gravel. There are, however, numerous beds of sand all of which are charged with water which rises nearly to, or slightly above, the ground surface. The theory of sedimentation previously described is borne out for this general area by the log of wells at Lynn (now Lynndyl), Oasis, Swan Lake Farm, Neels, and Goss. (See Figure 5) These show that conditions become more unfavorable for groundwater supplies as sediments become finer with increasing distance in a southwesterly direction from Lynn bench.
GROUNDWATER

Definitions

The pore spaces in soils and alluvial materials, and the fissures, and solution channels in rocks, make up a vast underground reservoir. (14) The water that occurs in the zone of saturation beneath the land surface and fills these pores is commonly called groundwater. The level to which the reservoir is filled constitutes the water table (17) and is defined as the contact plane between free groundwater and the capillary fringe, or as the upper surface of the zone of saturation.

"If a well is sunk into a permeable material just to the point where water enters the well and forms a water surface in it, that surface marks the hydrostatic level of the water at the top of the zone of saturation; by definition, it indicates the position of the water table at this place." (13)

Forms of Groundwater

Although conditions in nature are highly complex, in general groundwater may be classified as free, confined or fixed. In the ideal case, free groundwater is free to move through pervious material under the control of the slope of the water table, and is unhampered by any confining impervious material. It occurs beneath the water table and is bounded by the first effective confining stratum. Pumping for drainage will usually be most concerned with this form. Confined water occurs beneath confining strata which are sufficiently impervious to sever free hydraulic connection with all overlying groundwater except at the upper edge of the confining stratum. There the confined water connects with free groundwater. (See Figure 4) In cases where the hydrostatic level in a well piercing such a formation rises above the level of the water table, an artesian condition is said to exist and the water is called artesian water. The hydrostatic level marks the piezometric surface of the confined water.
Fixed groundwater is held in small openings by molecular forces and does not move readily.

Bodies of groundwater, called perched groundwater, can also exist in nature and are held above the main zone of saturation by impermeable material. Their upper surfaces are perched water tables. (17)

Sources of Groundwater

Widespread observations of water fluctuations in wells, correlated with rainfall and runoff data, have demonstrated the influence of the hydrologic cycle on groundwater levels. Reference has previously been made (see section on geology) to the manner in which particular geological formations can facilitate contributions to groundwater of low-lying areas. In most irrigated areas, however, it is likely that the major contribution to groundwater comes from the irrigation water applied in excess of plant needs and evaporation losses. The history of most of these areas shows that during the early years following irrigation the water table rose, often from appreciable depths, and remained near the ground surface. In the majority of cases artificial drainage has been necessary to carry off excess groundwater.

Availability

Not all of the water in the zone of saturation is available for pumping. The term 'specific yield' is used to denote the amount of water which the formation will yield as a percentage of a given volume of the formation. Similarly 'specific retention' is the amount which the formation retains. For example if 100 cubic feet of a saturated formation when drained by gravity will yield 20 cubic feet of water, the specific yield is 20%. Similarly if 12 cubic feet is retained in the formation, its specific retention is 12%. The sum of the two is the porosity. (2)
Movement of Groundwater

General

In recent years the study of the movement of groundwater has removed much of the mystery formerly surrounding this subject, and shown it to be governed by, and as amenable to, laws quite as definite as those governing surface flow.

Water table maps or profile plottings of the water table and of the piezometric surfaces of artesian aquifers form the basis for the study of groundwater movement. These show the hydraulic gradient and the direction in which the groundwater is moving in that locality, and hence to a great extent, they show the source and destination of groundwater flow. Miniature cased wells or piezometers as they are called, are normally used in gathering such data. Investigations have shown that vertical as well as lateral hydraulic gradients often occur.*

Darcy's Law

Tests have shown that turbulent flow is not likely to occur in materials with grain size smaller than three to four millimeters even with unit hydraulic gradient, and that a Reynolds number of ten (2) indicates the inception of turbulent flow through granular materials. (17) Most groundwater flow is laminar.** The simple law of laminar flow developed by the French investigator Poiseuille and applied to water-bearing material by the French hydrologist Darcy, (13) is the basic law of groundwater movement.

* Measurements in Cache Valley, Utah, have shown a piezometric surface thirty-two feet above ground surface. This represents an appreciable vertical hydraulic gradient.

** In turbulent flow, losses vary as the square of the velocity. It seems likely that turbulent flow occurs in the immediate area of the well. (17)
Darcy's law says that in any given material at a given temperature, the rate of flow is directly proportional to the hydraulic gradient. Expressed mathematically,

\[ v = ki \]  

where \( v \) is the gross velocity of flow over the cross section including both soil particles and voids, \( L/T \) units, 

\[ i = \text{hydraulic gradient} = \frac{h}{L} \]  

\( h \) = difference in hydraulic head, a dimensionless ratio, and \( k \) is a constant of proportionality for the given material called permeability.

The quantity of water flowing through any aquifer in unit time can then be calculated as the product of the area of the soil mass through which flow takes place, the permeability, and the hydraulic gradient of the water table. Expressed as an equation,

\[ Q = AV = kAi \]  

where \( Q \) is the discharge in unit time, \( L^3/T \) units.

Because of the fundamental importance of Darcy's law, the range of its validity has been questioned and has been thoroughly studied by the United States Geological Survey and other organizations. These studies show that the law holds for all gradients up to the critical velocity and inception of turbulence, and also for gradients as low as two or three inches to the mile. (17) (13)

Units for \( k \) and Factors Affecting

Since \( i \) in Equation 1 is a dimensionless quantity, \( k \) in Equation 1 has the dimension of velocity in engineering practice. Actually it is a volume per unit time per area, i.e. \( \frac{L^3}{TM} = \frac{L}{T} \).

Several factors affect the value of \( k \): (9)

(a) The size and grading of particles.
(b) The density of the material as measured by porosity (or void ratio).

(c) The temperature of the water.

(d) The presence of organic matter.

(e) The presence of colloidal material.

(f) Air in water.

(g) Bacterial action.

**Relation Between Engineer's k and Physicist's K**

Since physicists have been mainly responsible for developments in permeability studies, the relationship between permeability $K$, as used by the physicists, and that used by the engineer, $k$, becomes important.

Expressed mathematically these are related by

$$k = \frac{K}{\mu} \rho g$$  \hspace{1cm} (3)

$K$ depends only on the properties of the media, i.e. $d^2$, where $d$ is a length dimension depending on the diameter of the pores. $K$ therefore has dimensions of $L^2$.

$p = \text{mass density of the fluid flowing, } \frac{M}{L^3}$

$g = \text{acc. of gravity, } \frac{L}{T^2}$

$\mu = \text{coefficient of viscosity of the fluid flowing, } \frac{M}{LT}$

As a check on units

$$k = \frac{L}{T} = L^2 \cdot \frac{M}{L^3} \cdot \frac{1}{M/LT} \cdot \frac{L}{T^2} = \frac{L}{T}$$

The engineer's $k^*$ then is a function of:

1. The media through which flow occurs (the effective $d^2$).

2. The viscosity and density of the fluid.

In engineering practice, permeability measurements are standardized for the viscosity and density of water at $20^\circ C$. By reference to Equation 2

* In the literature, several other units for $k$ are proposed. In the interests of standardization the two forms mentioned herein seem most likely to achieve wide acceptance.
then, $k$ can be considered as the discharge through unit area under unit hydraulic gradient.

**The Significance of $k$**

The permeability, $k$, is a characteristic of the formation, and thus beyond control. Its influence on the flow through a water-bearing stratum is best shown by a study of Equation 2, which shows that, under a constant hydraulic gradient, $Q$ varies directly with $k$. Thus, if one formation is twice as permeable as another, other things being equal, twice the flow may be expected through the more permeable formation. In the field, $k$ is found to be a highly variable factor for different materials* and has come to be considered the limiting factor in much drainage work.

---

* In drainage investigations in Cache Valley, Utah, materials 100,000 times as permeable as others have been encountered. Permeability ratios of 10,000 to 1 have been measured in the Delta Area.
THE HYDRAULICS OF WELLS

Wells as Engineering Structures

Wells can be considered as engineering structures designed to make available for economical pumping the largest possible quantity of water from the formation in which they are constructed. (2) Many factors affect the achievement of this aim. Among these are:

1. Methods and materials of construction.
2. Well diameter and depth.
3. The amount of development work done.
4. Size, shape, and number of casing perforations or screen openings.

Methods of construction vary widely with the formation encountered and with the use to which the well is to be put. A discussion of the effect of geological formation and well use on construction design is beyond the scope of this thesis and will not be discussed here.

Influence of Well Diameter on Discharge

Although it seems clearly apparent that the well must penetrate the full depth of the water-bearing stratum for maximum yield, the relation between well diameter and discharge is not so obvious.

For steady radial flow in a stratum of uniform thickness, under the artesian conditions encountered in the area under study, the discharge is given by:

\[ Q = \frac{2\pi k t (Y_2 - Y_1 \log_{10} R_2)}{2.3 \log_{10} \frac{R_2}{R_1}} = \text{Constant} \times \frac{1}{R_2} \log_{10} \frac{R_2}{R_1} \]  

\( \text{(4)} \)
in which

\[ k = \text{permeability, feet per second,} \]
\[ Q = \text{discharge, cubic feet per second,} \]
\[ R_1 \text{ and } R_2 = \text{radii to selected points on the piezometric surface, feet,} \]
\[ t = \text{thickness of water bearing stratum, feet,} \]
\[ Y_1 \text{ and } Y_2 = \text{elevations of the piezometric surface at radii } R_1 \text{ and } R_2 \text{ respectively, feet.} \]  

(See Figure 6)

From Equation 4, it is evident that \( Q \) varies as \( \frac{1}{R_2} \), and since values of \( \frac{1}{\log R_2} \) change are small compared to changes in either \( R_2 \) or \( \log R_1 \), \( Q \) changes slowly also. (2) Although the values in practice may be somewhat higher, doubling the well diameter from 12 to 24 inches, for example, increases the discharge only about 10 percent if a radius of influence of 1000 feet is assumed. On this basis comparisons of yield for various diameters from 4 inches to 240 inches are shown in Table 2 which is based on Equation 4.

Table 2. Relation of Well Diameter to Yield (1, p. 152)

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<th>Well Diameter Inches</th>
<th>Relative Yield</th>
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<td>4</td>
<td>1.00</td>
<td>24</td>
<td>1.27</td>
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<tr>
<td>6</td>
<td>1.04</td>
<td>48</td>
<td>1.42</td>
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<td>8</td>
<td>1.08</td>
<td>120</td>
<td>1.65</td>
</tr>
<tr>
<td>12</td>
<td>1.15</td>
<td>240</td>
<td>1.92</td>
</tr>
</tbody>
</table>

* Radius of influence assumed 1,000 feet for all wells.
It appears, then, that "the only time that it is necessary to increase the diameter of well beyond that necessary to make the formation yield available, is when space is needed for the installation of pumping equipment." (2)

**Well Development**

The lower end of the well in the water-bearing stratum has either perforations cut in the casing, or a section of well screen connected to the end of the casing. A gravel envelope may or may not be added depending on the formation. The operation of developing consists of surging or agitating the water in the formation for the purpose of removing the finer material near the casing and preventing "bridging" of the sand particles. This is accomplished by alternately reversing the flow of water through the well casing. Some of the common methods used are backwashing (alternately starting and stopping the pump), and surging with a surge block or with compressed air. The materials brought into the well may be removed by bailing or by pumping.

The importance of well development can be shown by a consideration of the theory involved. Equation 3 has shown that $Q$ varies directly with permeability $k$. In turn, $k$ has been shown to be a function of the effective particle diameter (i.e. $d^2$). For full development and maximum discharge, it is desirable to have a material as coarse as possible in the vicinity of the casing by removing the fines.

**Perforation or Screen-opening Area**

To understand the relationship of casing perforations or screen openings to the well structure, certain fundamentals must be considered. When water is pumped from a water table well a hydraulic gradient toward the well from all directions is established. (2) The water table is lowered at the well and assumes a form comparable to an inverted cone.
(See Figures 8, 9 and 10) The distance that the water table is lowered outside the well casing with pumping is termed drawdown. Experience indicates that for non-artesian wells, in the majority of cases encountered, 90 percent of the available yield from the formation is obtainable with a drawdown of two-thirds of the water depth. (2) In any case it seems undesirable to pump for long periods at maximum drawdown.

If velocities, and thus entrance losses, through the well casing are to be kept at a minimum, the problem becomes one of determining the required area of screen openings or casing perforations to be provided in a length of one-third the water depth. After making allowance for the percentage of openings which are likely to be blocked by formation particles, a conservative value of permissible entrance velocity of 0.10 feet per second has been suggested. (2) Velocity calculations can be made by use of the equation of continuity transposed, in which:

\[ V = \frac{Q}{A} \]  

(5)

and

\[ V = \text{Velocity, feet per second,} \]

\[ Q = \text{Pump discharge, cubic feet per second,} \]

\[ A = \text{Total open area of screen or perforations through which flow takes place, square feet.} \]

A similar calculation of velocities may be made for artesian conditions. Theoretically in the latter case, the casing would be fully perforated throughout the depth of the water-bearing formation. In practice, particularly in a shallow well, it is likely that somewhat less than this length, based on experience and particular conditions, would be perforated or screened.

For a given slot opening, the entrance velocity of the water can be reduced by:

1. Increasing length of screen or perforated section of casing.
2. Increasing the screen or casing diameter.

In the latter case for example, since circumferential area varies directly as the diameter, doubling the diameter would double the circumferential, and thus the screen opening area. The velocity of flow through the casing openings is thus inversely proportional to well diameter, or, doubling the diameter reduces velocity one-half, and friction loss one-quarter. The sand carrying capacity of water depends on the velocity, and once the entrance velocity has been reduced sufficiently to prevent sand carrying, no further reduction is necessary.

**Effectiveness of Well**

"The 'effectiveness' or efficiency of a well is defined by Wenzel (19, p. 118) as

\[
E_W = \frac{100 S_C}{S_1} \tag{3}
\]

where

- \( E_W \) = the effectiveness, percent
- \( S_C \) = the theoretical drawdown at the outside of the casing, feet
- \( S_1 \) = drawdown in the well, feet

This formula provides an excellent method of representing losses through the well casing and envelope and has significance ordinarily comparable to efficiency. Under very favorable conditions of development, the 'effectiveness' of a well may exceed 100 percent due to removal of finer materials adjacent to the well and to increase of permeability in the area of high velocity flow." (7)
WELL SPECIFICATIONS AND CONSTRUCTION

Introduction

Preliminary studies of drainage by pumping from a well were conducted in 1946. The methods of procedure, and the results are presented in the 1946 report of the cooperative drainage studies. (7) A new well was drilled in August, 1947. Details concerning the location, design, development, testing and operation of this well are presented in the following pages.

Specifications and Construction Contract - 1947 Well

Proposal and Contract

Specifications for the 1947 well were written during the early spring. On June 21, the invitation to bidders was released calling for bids on July 3. Only one bid was received. This bid was rejected because it was too high and because the driller would not commit himself as to starting date. Negotiations were initiated with two other contractors who failed to bid, either because they did not fully understand the terms of the proposal, or for some other reason. A contract was finally negotiated on July 9, with R. L. Halterman of Parowan, Utah.

Specifications

The principal features of the specifications which differed from the method used in 1946 were:

(1) The hydraulic rotary method of drilling was excluded as unsuitable to the local conditions.

(2) A double-cased well was required. The driller exercised the option of either installing the outside casing to full depth and then pulling to the top of the gravel after the envelope
was installed or of installing only to the top of the gravel in the first instance. Mr. Halterman elected the latter.

Every effort was made to utilize the experience of the driller. Due to shortage of materials and limited drilling equipment in the area, the detailed specifications* were necessarily quite broad in their scope in order to invite competition and not rule out any qualified drillers.

Location of Well

The 1947 well is located 97.2 feet East and 80.5 feet North of the West one-fourth corner of Sec. 34, Township 16 S., Range 7 W., Salt Lake Base and Meridian. Its location with respect to the 1946 well is approximately 36 feet East and 6 feet North and in the same southwest corner of a field owned by Mr. W. H. Walker. Quoting from the 1946 annual report, (7)

"This location is one-half mile south of the village of Sutherland, along the east side of the main highway to Topaz. Across the road, west from the well, an irrigation canal runs north and south parallel to the road. The well is also adjacent to an open drain which runs west to the road and then turns south along the east side of the road."

Piezometer Location and Use

Location

The piezometers installed in 1946 were used for determinations of the water table levels in 1947, during both the non-pumping and the pumping period. The 1946 annual report gives the details of the piezometer location. (See Figure 7) Piezometers of 3/8-inch diameter pipe were installed in pairs located on four radial lines running in cardinal directions from the well with a fifth line running in a northeasterly direction. Of the pairs of piezometers, the shallower one, usually 10.5 feet in length, terminated in the upper clay stratum, and the longer one

* These specifications are on file in the office of the Irrigation and Drainage Department, Utah Agricultural Experiment Station.
was of such length as to extend well into the gravel stratum. In most cases piezometer stations were located at distances of 50, 100, 200, 400, and 800 feet from the well. Maintenance and replacement of bent and missing piezometers has been necessary, but on the whole, the piezometers have performed quite satisfactorily.

**Water Table Profiles**

The location of the 1946 well remains the central point of the piezometers and correction for the changed location of the 1947 well has been made in plotting the water table profiles. These profiles, determined at various times prior to, during, and following pumping, are shown in Figures 8 to 10. (See page 39 for further discussion of Figures 8 to 10)

**Control Piezometers**

To provide a check on major water table fluctuations due to general hydrographic conditions outside of the probable area of influence of the well, but in the immediate general area, on December 26, 1947, check piezometers were installed about one-half mile from the well in the four cardinal directions. The period of observation has been so short that it has not been found possible to correlate fluctuations in these piezometers with those in the well area. Descriptions of these locations are as follows:

1. East of well 1/2 mile near cross road and beside fence along north side of road.
2. South 1/2 mile and east 1/8 mile along fence line on north side of paved road and opposite farmstead.
3. West of well 3/8 mile along fence line south of A. Ogden field lane.
4. North of well 1/2 mile, 100 yards west, near power line pole opposite Ferrel Walker residence in Sutherland.
Well Construction

Outside Casing

Drilling of the 1947 well was begun on August 11. A 26-inch auger was used to bore through the surface 15-foot layer of heavy clay. The hole was made to a depth of four feet the first day, and completed on the second day. The 26-inch hole through the clay stood without support for four hours while the driller had several sections of 26-inch diameter outside casing welded in Delta to make the required length of 15 feet. The hole had sufficient clearance for the outside casing to be installed without driving. The casing was supported temporarily by wiring to two railroad ties set horizontally on the ground and then fine sand was packed around it so as to hold the top 4 inches above the ground. A driving shoe was welded to the lower end as provided in the specifications. As an aid in centering the 12-inch casing to be installed later, 3" x 5" wooden guide blocks were bolted inside before lowering into the hole.

Inside Casing

The 41-foot length, 12-inch diameter inside casing was of 1/4-inch material, except for an upper 14-foot section of 5/16-inch thickness. Its lower end was fitted with a heavy shoe. By bailing from the inside with a suction-type sand bucket, the unperforated casing was rapidly lowered through the gravel stratum. The driller reported gravel from 15 to 31 feet, clay from 31 to 33 feet, and fine sand from 33 to 37 feet, with a heavy clay layer below 37 feet. No driving of the 12-inch casing was necessary until the 37-foot depth was reached. The casing was driven two feet into the solid clay and thus securely anchored at the 39-foot depth. A cross-sectional elevation of the completed well casing and strata is presented in Figure 11.
During bailing, an ample supply of graded gravel was kept between the casings for replacing soil material which might be removed. Approximately 1/3 cubic yard of gravel was fed down during the process. Representative strata samples were obtained during bailing and kept for laboratory analyses.

**Perforations**

The 12-inch casing was perforated with a heavy duty Mills knife. Thirteen holes per row and 26 rows in all were made from the 17- to 26-foot depth. This averages slightly less than 3 rows per foot. Later, 9 rows of 13 holes per row were made from the 27.0- to the 29.5-foot depth. A test perforation through the casing extending above the ground measured 7/16-inch wide and 3 inches high with the uppermost portion tapering to a point. Its area is approximately 1 square inch. The perforated area per foot of pipe is about 8.5%. The minimum considered desirable is 6%.

**Power Supply and Pump**

The well was completed August 15 with the permanent installation of timbers to spread the casing load and support I-beams welded to the casings.

The Telluride Power Company installed a pole and transformer to bring power to the site at 60 cycles, 3 phase, and 220 volts.

The test-pump unit rented from the driller was an 8-inch turbine type driven by a 10-H. P. electric motor. Only 24 feet of column was initially installed.

**Gravel Envelope**

Gravel for the envelope was obtained from the Delta pit and was hand-screened over a 1/2-inch mesh screen. Approximately 9 cubic yards was stock-piled. Fortunately, there were few stones over 1 inch in size,
but the high percentage of fines at the pit made the handling of a large volume of material necessary. This, together with the fact that many of the stones have an undesirable flat plate-like shape, would seem to justify bringing in graded gravel from outside the Delta Area in future operations. No pit in the Delta, Topaz, or Oak City Area seems to have favorable gravel.

**Summary of Well Data**

**Depth:** 39 feet

**Outside Casing:** 26-inch diameter; 1/4-inch thickness; 15 feet length; not perforated.

**Inside Casing:** 12-inch diameter; 1/4-inch thickness except top 1/4 feet which is 5/16-inch; 40 feet long; perforated.

<table>
<thead>
<tr>
<th>Soil Formation</th>
<th>Depth, Feet</th>
<th>Description of Formation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Topsoil, lighter clays</td>
<td>0 - 10</td>
<td>Topsoil, lighter clays</td>
</tr>
<tr>
<td>Dense, brick-like clays</td>
<td>10 - 15</td>
<td>Dense, brick-like clays</td>
</tr>
<tr>
<td>Gravel and sand</td>
<td>15 - 31</td>
<td>Gravel and sand</td>
</tr>
<tr>
<td>Clay</td>
<td>31 - 33</td>
<td>Clay</td>
</tr>
<tr>
<td>Very fine sand</td>
<td>33 - 37</td>
<td>Very fine sand</td>
</tr>
<tr>
<td>Very dense clay</td>
<td>Below 37</td>
<td>Very dense clay</td>
</tr>
</tbody>
</table>

**Method of Installation:**

Rotary drill with 30-foot tower. Upper 15-foot clay augured out with 26-inch auger. Hole completed and left unsupported before installation of outside casing. Unperforated 12-inch casing bailed from inside with suction-type sand bucket to 38-foot depth.

**Perforations:**

Perforated in place with Mills knife.

17- 26-foot depth: 26 rows, 13 holes per row.

27- 29.5-foot depth: 9 rows, 13 holes per row.

**Calculated Area:** 8.5 percent of circumferential area.
Gravel Envelope:

7-inch thick gravel envelope 3/8- to 1-inch stones.

Development:

Limited development by surging pump and backwashing.

Pump:

8-inch, two-stage, deep well turbine with direct connected, vertical, 10-horsepower electric motor.

Underground Formation

General Description

The 1946 annual report describes the underground formation as determined during piezometer installations which indicated a depth to gravel, through a surface clay stratum, of 15 to 17 feet at the well and points north and east, with the depth decreasing to 12 feet at a distance of 800 feet west of the well. Likewise, the thickness of the gravel stratum is noted as 10 to 15 feet in north, east, and south directions from the well and decreasing to 6 feet at 800 feet west of the well. That a clay stratum underlies the gravel stratum at all points in the well area seems evident.

The 1947 drilling operations support the 1946 findings in regard to the formation at the well site. Samples of the formation were obtained for laboratory analyses which reveal more accurately the geological structure. Details of the analyses are given on page 37.

Water-Bearing Stratum

The 16-foot gravel stratum at the well seems rather sharply divided into two layers from the standpoint of texture and permeability. The top 9 feet is a mixture of pebbles and sand and the bottom 7 feet is sand with a sprinkling of stones. The decreased permeability to water of the
lower part of the water-bearing stratum places even greater limitation on the water-yielding capacity of the shallow stratum than was anticipated in 1946.

TEST DATA

Operation and Testing of the Well

Initial Pumping

Pumping was begun August 16. Although no water measurement was made, the flow was quite clear and estimated at 100 g.p.m. After preliminary pumping the casing was perforated between the depths 27.0 and 29.5 feet.

By means of a 90-degree, V-notch weir, the flow was determined as 135 g.p.m. with 13-foot drawdown. The specific capacity was therefore 10.4 g.p.m. per foot of drawdown. Additional sand pumping was noted. As was to be expected, the high initial rate of flow was not maintained when the steady flow conditions were approached, and by 6 P.M., the flow was down to 107 g.p.m. Sand was still being pumped.

Test work was begun on August 18. Since no air line was available, a 40-foot insulated wire designed for use with the electrical sounder was used to measure the depth to water inside the 12-inch casing. Piezometers were installed between the 12- and 26-inch casings in an effort to determine the water level there, and thus measure the hydraulic head loss near the well and through the casing. Data gathered at this time indicated the loss through the casing to be about 4 feet for a flow of 100 g.p.m., with a specific capacity of about 7.3 g.p.m. per foot of drawdown.

The low flow prompted a decision to add a 10-foot length pump column in order to determine the flow that could be obtained with greater drawdown. One obstacle was that the bottom 10 or 12 feet of the hole had
sanded in and no well driller or equipment was available to clean out the well. By alternately pumping and stirring up the sand with an air compressor, the hole was cleared to 31 feet. The pump was pulled with the aid of a tripod and the 10 feet of column added. Difficulty was encountered in lowering the unit due to the fact that the hole had partially refilled with sand. By rotating the pumping unit back and forth while the pump was in operation, it was possible to pump sand and clear the hole sufficiently to allow the unit to be lowered to the desired level. Initial flows on resumption of pumping at this time were 140 to 150 g.p.m. Throughout the intermittent pumping and testing during the following ten days, a flow of approximately 120 g.p.m. was maintained.

**Development**

During the period of initial pumping and development comparatively little sand was pumped—much less than from the 1946 well. As a result, there was little opportunity to add graded gravel from above. With no equipment for further development available, the backwashing method was used. To provide an adequate source of water for downward flow a pond was built near the pump delivery pipe. By alternately pumping to fill the pond, and then stopping the pump and letting the ponded water flow back down the well and through the pump, approximately 1/2 cubic yard of sand was removed. The operation was continued for about 10 hours over a period of 3 days. Since approximately 1/3 cubic yard of graded gravel was added as replacement of the pumped sand, this method of feeding gravel to the stratum was demonstrated as positive. Settlement of the gravel between the casings was most noticeable during the period of reversal or downward water flow.

**Drawdown–Discharge Curve**

Further test work included determination of the drawdown-discharge
curve for the well and an attempt to measure the hydraulic head loss through the casing. The discharge was varied to give successive drawdowns of 4, 8, 12, 16, and 20 feet. Sufficient time was allowed in each case for steady flow to be established as nearly as possible as evidenced by constant water levels in the piezometers. A 4-inch gate valve on the end of the discharge pipe made close control of discharge possible. The resulting curve of Figure 12 is typical of the kind obtained from an artesian well. As long as the drawdown is less than the pressure head measured at the top of the water-bearing stratum (providing hydraulic losses at the well are not excessive), discharge is directly proportional to drawdown, and a straight line is obtained. Beyond this point discharge increases less and less rapidly with drawdown until no measurable increase in discharge is obtained for the last one or two feet.

The drawdown-discharge curve of Figure 12 is applicable only to the well in its state of development at time of test. Further development work will likely alter the curve.

**Direct Measurement of Head Losses at the Well**

In an effort to determine water levels outside the 12-inch well casing, and thus measure directly the hydraulic head loss through the gravel envelope and through the casing, three types of piezometers—(a) open end, (b) pointed end with openings in the side wall near the lower end, and (c) well points—were installed between the 12- and 26-inch casings.

The results were erratic and disappointing. No definite reason for this can be assigned. It may be due, in part, to the uncertainty of determining the exact location of the lower end of the piezometer which perhaps "drifts" during driving.
Long-Time Pump Test

On September 14, the supervision of the pumping unit was turned over to Mr. Jay Bennet, of Sutherland. With only minor stoppages, pumping has been continuous since that date. Daily records of discharge have been kept and these are summarized in Figures 13 and 14. Piezometers have been read monthly.

Effect on Water Table

Figures 8 to 10, inclusive, show the water table elevations on each of the five radial lines, on various dates during the non-pumping period, December 1946 to August 15, 1947, and during the period of almost continuous pumping, September 1, 1947 to January 24, 1948. In order to keep the plotted data to a minimum only representative elevations have been shown for each of the summer months. As noted in the 1946 investigations and report, the elevations of the pairs of piezometric surfaces are so nearly identical that only one surface is plotted in each case. Quoting from the 1946 report, (7)

"The small differences between the elevation of the water surface in the short piezometers, and in the piezometers based in the gravel stratum, are particularly significant. The rapid lowering of the free water table due to the reduction of pressure in the gravel stratum, indicates there is no danger that the water table will be 'perched' due to the clay overlying the gravel. In spite of the density of this clay, it will pass water readily enough, either due to fracture, or for some other reason, that the water table is instantly lowered by pumping the gravel stratum."

This is also borne out by the 1947 water table readings. Water table elevations' and depths during May and July are so nearly identical that only one profile has been plotted as representative for the two months. The plotted ground water level profile of September 14 represents the water table at the beginning of the continuous long-time pumping test.

Figure 13 shows the fluctuation of the ground water level at each of the five points 400 feet from the well, and Figure 14 at points 800
feet during both the non-pumping and the pumping period of 1947. They show that the water table is normally highest in the latter part of June when it is at a depth of 4 to 5 feet in this area. The daily pumping rate is shown in the upper right part of the figure. The flow of 95 g.p.m. is equivalent to 12.5 acre-feet per 30-day month.

Table 1 shows the ground surface elevations at points approximately 400 feet and 800 feet distant from the well.

Table 1. Ground Surface Elevations at Points (or stations)

<table>
<thead>
<tr>
<th>Station Designation</th>
<th>Ground Surface Elev. Feet Above Datum</th>
<th>Station Designation</th>
<th>Ground Surface Elev. Feet Above Datum</th>
</tr>
</thead>
<tbody>
<tr>
<td>NE 371</td>
<td>109.3</td>
<td>NE 712</td>
<td>109.3</td>
</tr>
<tr>
<td>E 400</td>
<td>109.4</td>
<td>E 800</td>
<td>109.4</td>
</tr>
<tr>
<td>S 400</td>
<td>109.1</td>
<td>S 800</td>
<td>109.1</td>
</tr>
<tr>
<td>W 414</td>
<td>108.9</td>
<td>W 800</td>
<td>108.9</td>
</tr>
<tr>
<td>N 400</td>
<td>108.5</td>
<td>N 800</td>
<td>108.5</td>
</tr>
</tbody>
</table>

The depth of the water table below the ground surface at any of these points may be determined by subtracting the ground water level elevation, as given by Figures 13 and 14, from the appropriate ground surface elevation, given in Table 1. For example, referring to Figure 13, on July 22, 1947, the water surface at Station NE 370.7 was at elevation 104.3 feet and, since the ground surface elevation at this point is 109.3, the depth to water table on this date was 5.0 feet. On November 26, 1947, the elevation of the water surface at the same point was 102.3, making the depth to water table 7.0 feet.
Winter Pumping

Figures 8, 9, and 10 show a progressive lowering of the water table from September 14, 1947 to March 7, 1948 (except for the rise following early December irrigation) due to pumping and to seasonal changes. The March 7, 1948 plottings from data taken three days after pump stoppage due to a burned-out pump motor form a basis of comparison with the March 1, 1947 data. The 1948 plottings for March show the water table to be generally 0.8 feet lower than in March, 1947. It is difficult to say what credit may be given the single-unit pumping program for this situation.

It is believed that total winter precipitation was lower during 1947 and 1948 than during 1946 and 1947. October to December, inclusive, precipitation at the Delta airport was 6.91 inches for 1946 and 3.45 inches for 1947. A comparison of March 1947, and 1948 groundwater levels determined from piezometer readings at six other sites in the Delta area may be of interest. (See Figure 1) Sites C and D west and north of the well show no change and a rise of 0.3 feet for the 1948 readings respectively. At site F to the southwest, and at sites A and Oasis, south of Delta, the ground water is 0.4 to 1.0 feet lower and up to 0.5 feet lower respectively. It is likely, however, that at the sites compared the elevation of the water table is influenced more by the elevation of the bottom of the drain than by winter precipitation.

The effect of pumping on the ground water gradient is evident from a comparison of the profiles during pumping and non-pumping. The steepness near the well during pumping shows the desirability of keeping head losses at the well to a minimum in order to have the maximum head available for moving water through the stratum. Figure 13 shows that the ground water levels at points 400 feet from the well are affected more uniformly and to a greater extent than those 800 feet distant shown in Figure 14.
Figure 14 shows for October 25 a very favorable lowering of the water table due to pumping—approximately 1 to 1-1/2 feet over the area. Station 8+00 East is in the direction of the source of ground water and hence there was less lowering at that station. The general rise on December 2 is probably due to somewhat heavier local irrigation near the well during 1947. In spite of the high December 2 water table, the readings of December 26 showed a lowering of 1 to 2 feet in the three-week period which brought levels well below those of October 25. Winter pumping had, by January 24, 1948, lowered the water table to approximately one foot below the minimum recorded natural water table of March 1, 1947. The lack of water infiltration from precipitation during the winter evidently causes the well to draw from an extremely wide area. A widespread pumping program would undoubtedly cause a greater lowering of the water table.

**Effectiveness of Well**

**The 1947 Well**

Percent effectiveness has been calculated and shown for each of the flows in Figures 15 to 19. Most recent calculations based on the long-term pump test show a value of 37%. With reference to Figure 19 using 103.1 as the normal water-table elevation, the theoretical drawdown at the casing is 103.1 - 94.9 = 8.2 feet. Elevation 94.9 is the intersection of the logarithmic straight line plotting of the water-table surface and the outside of the casing of radius 0.5 feet. The drawdown in the well at this time was 22.0 feet. The effectiveness is then

\[
\frac{8.2}{22.0} \times 100 = 37\% \text{ approximately}
\]
Indirect Measurement of Head Losses at the Well

The total drawdown is equal to the hydraulic head which causes the water to move from remote points, through the stratum and into the well. It is especially important to know the hydraulic head losses in the vicinity of the well. The data of Figures 15 to 19 were used to make an analysis of these losses as a basis for determining the need of further well development. In each case, the discharge Q, the actual drawdown in the well, and theoretical drawdown in the stratum are the significant items. Figure 20 summarizes the results.*

There are two scales on the left-hand side of Figure 20. The outer gives the calculated elevation of the water level outside the casing and the inner, the calculated drawdown in feet. At each elevation the two scale values are complementary—their sum is 103.0—the elevation of the natural water table at the time the data were collected. The right-hand scale gives the head loss at the well in feet as the difference between the actual drawdown in the well and the calculated drawdown in the stratum.

Hydraulic Head Loss in Water-Bearing Stratum

Curve A of Figure 20 shows the relation of the head loss in the aquifer to the well discharge. The following example illustrates how this curve was developed: From Figure 15 for \( Q = 0.15 \) c.f.s., (67 g.p.m.) theoretical drawdown 5.1 feet. From Figure 16 for \( Q = 0.20 \) c.f.s., (90 g.p.m.) theoretical drawdown = 7.6, etc. Other points are determined in the same way. Curve A is determined purely by the characteristics of the formation. It is indicative of the flow to be expected for each drawdown for steady streamline flow when only the formation is considered. Its slope cannot be altered if the thickness of the aquifer is constant and if the permeabilities at all points in the aquifer are of the same value. (See Equation 4)

* Analysis suggested by D. F. Peterson, Jr.
Hydraulic Head Loss at Well

Curve B is obtained by plotting Q against the head loss in the well, i.e., Q against the difference between the theoretical or the calculated and the actual drawdowns. For example, from Figure 16, for \( Q = 0.2 \text{ c.f.s.} \) (90 g.p.m.) the theoretical head loss is \( 12.0 - 7.6 = 4.4 \) feet. Other points are obtained in a similar manner. The steepness of curve B for the discharges near 100 g.p.m. represents a rapidly increasing loss of hydraulic head in the vicinity of the well with increase in discharge. The characteristics of curve B are subject to some measure of control. For example, further successful well development would reduce the head loss and flatten the curve. The head loss in the well should be very small.

The data presented in curve B of Figure 20 quite definitely show the need for further well development.

Permeability Determinations

Field Measurements

Figures 15 to 19, plotted from data collected during drawdown discharge tests, indicate an artesian condition by the fact that the water surface outside the casing was above the top of the gravel stratum during the tests.

For steady radial flow in a stratum of uniform thickness, under artesian conditions, the permeability is equal to

\[
k = 2.30 Q \log_{10} \frac{R_2}{R_1} \frac{R_2}{2\pi t (y_2 - y_1)} \tag{4a}
\]

from Equation (4).

In solving Equation (4a) the average water surface elevations can best be determined by plotting against the logarithm of the radius. Such
Plotting have been made in Figures 15 to 19, and a calculation of permeability made in each case. Using the average stratum thickness of 10.9 feet as determined in the 1946 analyses, the values of k as recorded in the upper left-hand corner of each figure are of the order of $3 \times 10^{-3}$ feet per second as compared to the 1946 value of nearly $4 \times 10^{-3}$ feet per second. This is a close agreement considering the meagre data collected during 1946. This value of k is in the range ordinarily attributed to a medium to coarse sand. (9, p.649)

As an example of the calculation of k, reference is made to Figure 17. For convenience, $R_2$ and $R_1$ are chosen as 100 feet and 1 foot respectively. Corresponding values of $y_2$ and $y_1$ at $R_2$ and $R_1$ from the figure are 100.9 and 95.2. Substitution of $Q = 0.235$ c.f.s. for this test, and $t = 10.9$ feet in the formula gives $k = 2.77 \times 10^{-3}$ feet per second.

**Laboratory Measurements**

It has not been found possible to check these values by laboratory permeability tests on the disturbed samples. Determinations made at the college soil mechanics laboratory using a constant-head permeameter gave an average value of k of $6.4 \times 10^{-4}$ feet per second for the 15- to 24-foot stratum. Values ranging from $0.85 \times 10^{-4}$ to $3.0 \times 10^{-4}$ feet per second, were obtained for the 24- to 31-foot stratum. In the U. S. Regional Salinity Laboratory tests the values of k varied from $6 \times 10^{-4}$ to $18 \times 10^{-4}$ feet per second for the coarser material and from $1 \times 10^{-4}$ to $2.5 \times 10^{-4}$ feet per second for the fines. The upper layer is therefore 6 to 7 times as permeable as the lower.

Tests on samples from the 34- to 37-foot stratum gave average permeability values of $0.050 \times 10^{-4}$ feet per second. These values closely approximate the average permeability for coarse silt.
Assuming the ratio of the permeabilities of the strata to be 7 to 1 and noting that the ratio of their thickness is 9 to 7, the ratio of the flows under the same hydraulic gradient would be 9 to 1. It appears, therefore, that 90% of the flow occurs through the upper part of the permeable stratum which is from the 15- to 24-foot depth.

Before beginning each test, carbon dioxide gas was fed through the sample in the permeameter to displace air in the voids, since entrapped air has been found to affect permeability. The laboratory tests showed rather wide variation in permeability values. The degree of compaction of the samples, and perhaps other factors, seemed to affect permeability to such an extent as to make the effect of the carbon dioxide treatment unnoticeable.

**Mechanical Analyses of Soils**

**Size Gradation Curves**

Figure 21 shows representative curves resulting from mechanical analyses of samples from various depths. Soil classification is sometimes determined from such curves on the basis of the 20% size. The 20% size is that size for which 20% of the sample is finer and 80% coarser. On the basis of the United States Bureau of Soils Classification (9, p. 649) the material from the 15- to 24-foot depth is in the medium to coarse sand range. Likewise, the 24- to 31-foot layer is classified as a fine sand and the 33- to 37-foot layer as a very fine sand or a coarse silt. The field determinations of permeability approximate the usual values for a medium to coarse sand whose 20% size is 0.50 mm. (9)

From these studies it appears that the water-bearing stratum which has previously been referred to as a gravel, is in reality a medium to fine sand. The slope of the plotted curves indicates that there is a rather complete gradation from fines to larger particles. Such a condition imposes serious limitations on the water yield that may be expected from the formation. The finer particles fill the pore spaces and thus decrease the area through which flow can take place, and greatly reduce the permeability.
It becomes of special importance, therefore, to reduce losses at the well to a minimum in order that the maximum flow through the stratum may be realized.

**Suitability for Well Development**

The curves for the 15- to 24-foot samples of Figure 21 indicate that by bringing into the well the fine sands from these depths of the stratum, a uniformly graded envelope, with greatly increased pore space diameter would be developed. The curves for the 25- to 31-foot samples indicate that much sand might be brought out from the layer. With a positive method of getting replacement gravel down, difficulty from caving, due to failure to replace removed fines, is unlikely.
COST ANALYSIS

Costs of Well Unit

The cost of the 1947 well, the prices quoted for a pump unit designed for the well capacity and lift, and the assumptions noted, are used as the basis for the following analysis of annual costs of a single well and pump.

Capital Costs

1. Well and extras ........................................ $ 869.00
2. One and one-half H. P. baby turbine to discharge 120 g.p.m. at 30-foot head ........ 398.50
   Total .................................................. $1,267.50

Annual Costs

1. Power costs for pumping 12 months
   1-1/2 H. P. at $6.30 .................................. $ 113.50
2. Maintenance at $10 per year .......................... 10.00
3. Capital costs reduced to uniform annual cost
   on basis of 20-year life and 4 percent interest
   (1267.50) (0.0736) .................................... 93.25
4. Total annual costs ..................................... $ 216.65

Costs per Acre

It is assumed that the pump will operate twelve months per year with a discharge of 100 g.p.m., and that eight acre-inches per acre per year must be removed by artificial drainage. One hundred g.p.m. is equivalent to 161 acre-feet per year, or the excess water from 242 acres of land. On this basis, one well unit would furnish drainage for 242 acres at a cost of $216.65 per annum, or 90.90 per acre.
A more effective well would reduce the annual costs per acre because of lower capital investment, but not to the extent that might be assumed upon casual investigation. Suppose a well and pump having twice the discharge could be obtained for the same investment; and, therefore, that one unit would drain twice the area, or 484 acres. On the basis of an eight-inch artificial drainage requirement, the total annual cost for one unit would then be:

<table>
<thead>
<tr>
<th>Description</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Power</td>
<td>$226.80</td>
</tr>
<tr>
<td>Maintenance</td>
<td>10.00</td>
</tr>
<tr>
<td>Capital Costs</td>
<td>93.25</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>$330.05</strong></td>
</tr>
</tbody>
</table>

which is $0.68 per acre as compared to $0.90, a reduction of about 24%.

The most uncertain figure in the foregoing analysis is the amount of water that must be removed annually by artificial drainage. Assuming that for alfalfa hay two and one-half feet is the average depth of water applied, (6) the excess will be the two and one-half feet plus available rainfall during the growing season, less evaporation and consumptive use. The average summer rainfall is rather low, approximately four inches for April to September inclusive, and it is believed that the consumptive use must be between one and one-half and two feet. (11) (10) (4) This leaves an excess of between six and twelve inches to be removed by natural or artificial causes, provided the evaporation equals the rainfall. There is reason to believe that there is considerable natural underground flow from the cultivated area. It appears, therefore, that eight inches is a reasonable estimate of the amount that would need to be removed by artificial drainage.

* As experience in drilling wells in the local formation is gained, less expensive wells producing more water will doubtless be obtained, but it is not believed that a reduction in capital investment to the extent herein assumed is possible in the Sutherland formation.
Quality of Water as a Factor in Pumping Costs

A factor of vital importance in costs of drainage by pumping is the value of the pumped water for irrigation purposes. In the Delta Area this value depends largely on the salt content of the irrigation water. The salt content of the water pumped from the experimental drainage well during the winter of 1947-48 has ranged from 4,000 to 6,000 p.p.m. High salt content in the drainage water is to be expected as long as Sevier River water, which contains about 1,500 p.p.m. salts, is used in irrigating. Indeed, as long as Sevier River water is applied in the area, it will likely be advantageous in the long term view to waste the drainage water pumped and thus continue to remove salt from the irrigated area. Should better water, with much less salt, be made available for irrigation at some later date, it is possible that the drainage water pumped during the irrigation season might be of such quality that it could be used to supplement irrigation water supplies. The pumped water would be of particular value during dry years.

Assuming that 80 acre-feet would be pumped from one well and utilized during the irrigation season, its value at $1.50 per acre-foot would be $120. This amount is more than sufficient to cover the annual power bill or 55% of the total annual costs for wells of the present effectiveness.
CONCLUSIONS

The 1947 Well

With regard to the 1947 well, the following conclusions are presented:

1. **Effectiveness.** The 'effectiveness' of the 1947 well is quite low. Further development should be undertaken in order to determine if specific capacity can be increased.

2. **Specific Capacity.** The specific capacity of the well is somewhat variable. For a discharge of 80 g.p.m., the drawdown is 10 feet giving a specific capacity of 8 g.p.m. per foot of drawdown. (See Figure 12) At a discharge of 105 g.p.m., the drawdown is 16 feet, corresponding to a specific capacity of 6.6 g.p.m. per foot of drawdown.

3. **Water Table Lowered.** Continuous pumping for four and one-half months prior to February 1, 1948, has successfully lowered the water table in the general area of well influence by from two to three feet during the winter months.

4. **Continuation of Test.** In order to obtain complete information regarding the effectiveness of the well for drainage, test pumping should be continued for at least one full year.

General

With regard to the general problem of drainage by pumping the following conclusions are drawn.

5. **Limited Pumping Feasible.** Limited areas in the Delta region are underlain by a relatively thin gravel stratum at shallow depth in which wells may be constructed for the purpose of obtaining better drainage. (See Figure 1)
6. **Method of Construction.** The method used for construction of the 1947 well produces a stable well. It is believed that much more effective wells will be produced when further experience is gained in the particular formation.

7. **Cost.** Assuming that eight inches of water per year must be removed from the soil by artificial means it is estimated that drainage can be accomplished for approximately $0.90 per acre per annum if wells of the same degree of effectiveness as the 1947 well are constructed.

8. **Increased Effectiveness.** Construction of more effective wells would decrease the annual cost of drainage.

9. **Winter Pumping.** Limited data indicate that pumping a single well during the winter season does not produce a rapid lowering of the water table beyond a certain depth. General pumping throughout the area might be expected to be considerably more effective in this regard, however.

10. **Well Screens.** If further test wells are to be constructed, consideration should be given to using a properly designed well screen, especially in the more sandy formations.

11. **Further Exploration.** Consideration should be given to constructing test wells in other favorable areas and to constructing small diameter exploratory wells in areas where presently available information indicates favorable formations may exist.

12. **Local Gravels Not Suitable.** Local gravels are flat-shaped and plate-like and are not suitable for the envelope. Importation of gravels in which the particles are not flat, for use as envelope material, is desirable.

13. **Careful Planning Recommended.** Any extensive program of drainage by pumping should be undertaken cautiously since areas where
pumping is feasible are limited. A slow, well-planned program would provide an opportunity to apply the experience gained from previous drilling and pumping as the development progresses. Careful observations and complete records of all work done, performances, and costs should be maintained in order to evaluate the effectiveness of variations in technique.
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887. 1942.
FIGURE 1
SKETCH MAP SHOWING UNDERGROUND CONDITIONS DELTA AREA, UTAH
2-8-47
D.F.P.
FIG. 2. EFFECT OF PUMPING ON GROUND WATER — SALT RIVER PROJECT
FIG. 3. TOPOGRAPHIC MAP OF JUAB AND MILLARD COUNTIES

REPRODUCED FROM PLATE I, U.S.G.S. WATER SUPPLY PAPER 277, 1911
**Fig. 4. Diagrammatic Cross-section of a Typical Valley in the Basin Province.** (From Fig. 5, U.S.G.S. Water Supply Paper 277).
**Fig. 5. Well Logs in Sevier Desert and Lower Beaver Valley.**

Reproduced from U.S.G.S. Water-Supply Paper 277 Plate III.
FIG. 6. VERTICAL CROSS-SECTION OF A TYPICAL WELL IN AN ARTESIAN FORMATION.
Fig. 7.
PIEZOMETER LOCATION
SUTHERLAND WELL
SHOWING
APPROXIMATE CONTOURS
Scale 1" = 200'
8-15-46  D.F.P.
Piezometer Station
Note
Open Drain W.S. levels
9-14-47 and 12-2-47 = 103.3
10-25-47 = 103.2

Profile of groundwater surface
North & South lines from well

FIG. 8

Water intake through perforations below here
PROFILE OF GROUNDWATER SURFACE
RADIAL LINE NORTHEAST FROM WELL

Fig. 10.
Delta Well No. 2

Perforations
13 slots per row
3 rows per foot

1/4" thick Casing
Slot Size

CLAY
3"

Outside Casing
Gravel

Graded Gravel
(1" pebbles to fine sand)

Sand with some large Particles

Clay

Sand

Clay

Elevation

Fig 11. Well & Stratum Profile
DRAWDOWN - DISCHARGE CURVE

1947 DELTA WELL

Discharge - c.f.s.

0.04 0.06 0.08 0.1 0.12 0.14 0.16 0.18 0.2 0.22 0.24 0.26

Discharge - g.p.m.

Drawdown - feet

Fig. 12.
$$K = \frac{2.3 \cdot \left( \frac{150}{\ln \frac{100}{T}} \right)}{20 \cdot (0.27) / (101.7 - 98.5)} = 3.15 \times 10^{-2} \text{ ft/sec}$$

Effectiveness = \frac{103.1 - 98.0}{103.1 - 95.0} = 65 \%

Q = 0.150 \text{ c.f.s.}

Theoretical D.D. = 5.1'

Actual D.D. = 7.8'

Note: Data from 6' drawdown test 1500 hrs - 3/5/47 4-hr. test

Permeability Analysis 1947 Delta Well 6 Sep 47

Fig. 15
\[ k = \frac{2.3(0.200) \ln 100}{2 \pi (0.9)(101.2-96.2)} = 2.72 \times 10^{-3} \text{ ft/sec} \]

Effectiveness = \[ \frac{103.1-95.5}{103.1-91.1} = 63\% \]

\[ Q = 0.20 \text{ c.f.s.} \]

Theoretical D.D. = 7.6''
Actual D.D. = 12.0''

Clay

Top of gravel stratum

\[ \text{at well} \]

Note:
Data from 12'' D.D. test 1947 Delta Well
11:00 AM, 9-6-47
24-hr. test

PERMEABILITY ANALYSIS

Fig. 16.
\[ k = \frac{2.3 \times 0.235}{2\pi (10.3)(100.9-95.2)} \ln \left( \frac{104}{101} \right) = 2.77 \times 10^{-3} \text{ ft/sec} \]

Effectiveness = \frac{103.1 - 94.3}{103.1 - 87.0} = 55\%

\[ Q = 0.235 \text{ c.f.s.} \]

Theoretical D.D. = 8.8'
Actual D.D. = 16.1'

Top of gravel at well

Note: Data from 16' D.D. test
1:00 A.M., 9-5-47
24-hr. test.

PERMEABILITY ANALYSIS
1947 Delta Well
Sept 5, 1947

Fig. 17
\[ K = \frac{(2.3)(0.252) \ln \frac{100}{1}}{(2\pi)(10.9)(101.05 - 94.8)} = 2.71 \times 10^{-3} \text{ ft sec} \]

Effectiveness = \frac{103.1 - 93.8}{103.1 - 82.9} (100) = 46% 

Q = 0.252 c.f.s.

Theoretical D.D. = 93'

Actual D.D. = 20.2'

Note: Data from 20'
drawdown test,
2:00 P.M. 9-4-47
28-hr. test

PERMEABILITY ANALYSIS
1947, DELTA WELL
Sept. 4, 1947

Fig. 18.
The diagram illustrates the permeability analysis for the 1947 Delta well. The equation for the permeability coefficient $K$ is given as:

$$K = \frac{23(0.209)/\ln 190}{2\pi(10.9)(10865-982)} = 2.86 \times 10^{-3} \text{ ft/s}.$$ 

The effectiveness is calculated as:

$$\text{Effectiveness} = \frac{103.1 - 94.9}{103.1 - 81.1} = 37\%.$$ 

The theoretical D.D. is 82', and the actual D.D. is 220'.

The data for the permeability analysis is from 10/25/47 at well D.D. 220'.

The top of the gravel stratum at well 7 is indicated on the graph.
MECHANICAL ANALYSIS CURVES
1947 DELTA WELL
Fig. 21.

U.S. Std. Sieve No. 2 4 8 14 20 40 60 100 140 200
Tyler Sieve No. 3/4" 3/8" 1/4" 1/8" 1/16" 1/32" 1/64" 1/128" 1/256" 1/512"

Per Cent Finer
0 10 20 30 40 50 60 70 80 90 100

Per Cent Coarser
0 10 20 30 40 50 60 70 80 90 100

(1/2-inch) Casing Perforation Width
Gravel Envelope
1947 Well

U.S. Bureau of Soil Classification

Grain Size in Millimeters
0.05 0.025 0.01 0.005 0.000