CORPS OF ENGINEERS, U. S. ARMY

CONTROL OF UNDERSEEPAGE BY RELIEF WELLS

TROTTERS, MISSISSIPPI

FIELD INVESTIGATION

CWI PROJECT NO. 478

TECHNICAL MEMORANDUM NO. 3-341

PREPARED FOR

THE PRESIDENT, MISSISSIPPI RIVER COMMISSION

VICKSBURG, MISSISSIPPI

AND

THE OFFICE, CHIEF OF ENGINEERS

WASHINGTON, D. C.

BY

WATERWAYS EXPERIMENT STATION

VICKSBURG, MISSISSIPPI

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ARMY-MRC VICKSBURG, MISS.

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PREFACE

A general study of underseepage and its control along levees in the lower Mississippi River valley has been in progress since 1940 and is still under way. The investigation has included a review of underseepage conditions observed during previous high waters, exploration and geological studies of sites where underseepage has been a problem, model studies, installation of piezometers to measure substratum pressures beneath and landward of levees, installation of relief wells along levees to intercept underseepage and relieve excess substratum pressures, and measurement of natural seepage landward of certain levees during a high water.

One of the sites studied along the Mississippi River, and at which piezometers and relief wells were installed in 1942 and 1943, was at Trotters (Mi 54) in the Memphis District, CE. These wells were found to be too small during the 1943 high water and were plugged. However, valuable information was obtained from this well system and was used in the design of a new well system described in this report.

In 1950 the Office, Chief of Engineers, authorized the Mississippi River Commission to install and study a new relief well system at Trotters and to make a more detailed study of its efficacy in controlling underseepage. The study is an item of the Civil Works Investigations program of the Office, Chief of Engineers, and is designated Civil Works Investigation No. 478, "Investigation of Underseepage Control by Relief Wells." The Waterways Experiment Station in cooperation with the Memphis District is performing the study under the general direction of the

Mississippi River Commission.

The new well system was designed by the Waterways Experiment Station and installed by the Layne-Central Well Company, Memphis, Tennessee, under the supervision of the Memphis District. Tests of the well system during its first period of operation in March and April 1951 were made by personnel of the Memphis District under the direction of Mr. J. M. Pollock, Memphis District, and Mr. C. I. Mansur, Waterways Experiment Station.

This memorandum constitutes a final report on the geological study of the site, characteristics of the foundation, design of the well system, and analysis of all piezometer and well flow data obtained during the 1943, 1950, and 1951 high waters, with particular reference to the data obtained from the new (1950) wells and piezometers. However, both the piezometer and well systems will continue to be observed during future high waters. Data obtained from such future observations will be analyzed and reported in the form of appendices to this report.

Engineers of the Soils Division of the Waterways Experiment Station connected with this study were Messrs. W. J. Turnbull, Chief, Soils Division; S. J. Johnson and W. G. Shockley, Chief and Assistant Chief, respectively, Embankment and Foundation Branch; C. I. Mansur, Chief, Design and Analytical Section; T. B. Goode, Chief, Inspection and Exploration Section; and W. J. Emrich and R. I. Kaufman, engineers, Design and Analytical Section. This report was prepared by Mr. Mansur.

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SUMMARY

The control of underseepage and sand boils is a serious problem along various reaches of Mississippi River levees, particularly when the river stage creates a head of water in excess of about 15 ft. Excessive underseepage at certain sites during the 1937 flood occasioned the initiation of a general study of this problem. A part of this study included a detailed investigation of the underseepage problem at Trotters, Mississippi, in the Memphis District, CE. This investigation embodied: a geological study of the site; the installation in 1942 of piezometers to measure hydrostatic pressures in the pervious substratum beneath and landward of the levee during periods of high water and to determine the effective source of seepage; and the installation in 1943 of a line of 2-1/2-in. relief wells along the landside toe of the levee to provide pressure relief and controlled seepage outlets. Data obtained and observations made during the 1943 high water indicated that these relief wells would intercept a considerable amount of the underseepage and would reduce substratum pressures but that the system was somewhat underdesigned. These wells were plugged in the fall of 1943. In 1950 a new system of 6-in. ID wells was installed and its performance was observed during the 1951 high water. The results of the investigations at the Trotters site to date are summarized in the following paragraphs.

<u>Geology and underseepage</u>

Foundation conditions at the site consist of a top stratum of clay and silt approximately 9 ft thick underlain by a fine sand stratum about 35 ft thick, which in turn is underlain by a medium and coarse sand stratum

about 85 ft thick. The combined pervious stratum has an equivalent thickness of about 90 ft in terms of the coarser stratum which has a permeability of 1250 x 10^{-4} cm/sec. The top stratum has a permeability of about 1 to 5 x 10^{-4} cm/sec. The Mississippi River is approximately 3000 ft from the levee; however, riverside borrow pits excavated to sand provide a ready entrance for seepage into the pervious substratum. The "effective" seepage entrance, as determined from piezometers, is about 600 ft riverward of the center line of the levee or 1000 ft from the well system.

Very heavy underseepage and numerous sand boils occurred at the site during the 1937 flood. The estimated natural seepage passing beneath the levee at the crest of the 1950 high water was about 150 gpm per 100 ft of levee with a net head of 13.5 ft against the levee; 55 gpm per 100 ft of levee was measured in a strip 100 ft wide immediately landward of the levee.

With no wells in operation, the maximum artesian pressures developed above the average ground surface, and above the water elevation in the drainage ditch, amounted to 35 and 50 per cent, respectively, of the net head on the levee for a head of about 10 ft. The maximum possible pressure at the levee toe apparently is about 3 to 5 ft. There is very little time lag in substratum pressures with changes in river stage.

Well system

The piezometer readings and seepage observations made during the 1951 high water indicate that the 6-in. ID well system will reduce substratum pressures landward of the levee to a small fraction of the head on the levee, and will also intercept a large proportion of natural seepage which otherwise would rise to the surface landward of the levee. The head between wells, with the well system operating on 50-ft centers, was about 8 per cent of the net head on the system. The average flow from the wells in the central portion of the system was about 10 gpm per well per ft of net head. The total flow from the system was about 350 gpm per ft of net head.

The wells as installed performed entirely satisfactorily. Very little material passed through the filter gravel and well screens during the period of operation and there was very little head loss through the filter and screen, or in the well. Approximately 50 per cent of the total flow entering the wells entered through the bottom 10 ft of the screen.

CONTROL OF UNDERSEEPAGE BY RELIEF WELLS

TROTTERS, MISSISSIPPI

Field Investigation

PART I: INTRODUCTION

1. Underseepage in the form of sand boils and piping landward of levees pose a threat to the safety of any levee during periods of high water. Such sand boils and piping are the result of excessive hydrostatic pressures and seepage in and through pervious substrata which provide a means for transmission of pressure and seepage from the river or borrow pits to the landside of the levee. Thus, prevention of sand boils and control of underseepage require some means of intercepting the seepage beneath the levee and reducing excess pressures beneath the top stratum landward of the levee. One such means is to tap the underlying pervious stratum with a properly designed series of relief wells. These wells will provide pressure relief and controlled seepage outlets offering less resistance to flow than any other path, and at the same time preventing erosion of the foundation soils.

Early Observations and Studies

Field investigations

2. No crevasses along the lower Mississippi River levees have been positively attributed to sand boils, piping, or blowouts since 1913. However a failure occurred at Weecama in 1922, which is believed to have been the result of underground piping, and a levee crevasse almost occurred at

Greenville, Mississippi, in 1929 as a result of subsurface piping. Excessive seepage and sand boils also occurred along numerous reaches of these levees during the flood of 1937. As a result the Mississippi River Commission, in September 1940, initiated a general study of underseepage and its control at several locations along the lower Mississippi River levees where heavy underseepage and sand boils had been observed during floods. One of the sites studied was located on the east bank of the Mississippi River at Trotters (Mi 54) in the Yazoo Delta Mississippi Levee District (see plate 1).

3. The first part of the study at this site consisted of making a number of borings along the toe of the levee and on lines perpendicular to it in order to ascertain the subsurface conditions at the site. The borings made at that time (1940) are the B borings shown on plates 1-3. The results of this study* and of a geological investigation of the site**, authorized in October 1941, have been published. The second phase of the study of underseepage at the Trotters site consisted of installing some piezometers to measure the hydrostatic pressure in the pervious substratum beneath and landward of the levee. The Memphis District installed seven of these piezometers in April 1942 and eight more in 1943 in connection with a proposed field permeability test which was subsequently abandoned.

4. Then, in January 1943, the Waterways Experiment Station

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* Waterways Experiment Station Technical Memorandum No. 184-1, "Investigation of Underseepage, Lower Mississippi River Levees," October 1941.

** H. N. Fisk, "Geological Report on the Trotters, Mississippi, Underseepage Area," September 1942.

installed 81 relief wells along the landside toe of the existing seepage berm between sta 53/31+84 and sta 54/34+00 to study the practicability of relief wells for controlling underseepage at Trotters. These wells were installed on 75-ft centers and consisted of a 24-ft screen made of 2-1/2-in. ID, flush joint, perforated (3/32-in. holes), clay tile pipe with 4-in. ID porous concrete pipe filter, and 3-in. ID, clay tile riser pipes. This well system first operated during the high water of May and June 1943. A report describing the piezometer and well system and its operation during this high-water period has been published*.

5. Observations made during the 1943 period of operation indicated that the wells were too small and spaced too far apart for the degree of pressure relief desired. Also the joints in the riser pipes permitted undesirable leakage around the wells when closed. Because of these considerations it was decided in November 1943 that the wells should be plugged.

Model experiments

6. In the interval between 1943 and 1950, the control of underseepage by means of relief wells was studied in large-scale sand models at the Waterways Experiment Station. The results of these studies have been published** and use was made of the data obtained in designing the new experimental relief well system for Trotters.

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* Waterways Experiment Station Technical Memorandum No. 3-316, "Investigation of Underseepage and Its Control by Relief Wells -- Commerce and Trotters, Mississippi," June 1950.

** Waterways Experiment Station Technical Memorandum No. 3-304, "Relief Well System for Dams and Levees on Pervious Foundations," November 1949.

Current Investigation

7. In the spring of 1950 the Office, Chief of Engineers, and the Mississippi River Commission decided to install a new relief well system at the Trotters site. Also, a considerable number of additional piezometers were to be installed to measure the distribution and amount of hydrostatic pressures in the pervious substratum. The location of the new well system and piezometers is shown on plates 1-3; details of the piezometer installation are given in table 1.

Purpose

8. The primary purpose of installing the new well system and additional piezometers was to make a full-scale field test of the efficacy of an adequately designed relief well system, and to obtain more knowledge concerning the action of relief well systems in general. Other purposes of the study were:

- a. To obtain more complete information on foundation conditions at the site and to determine the permeability of the top stratum and underlying pervious substratum.
- b. To investigate by means of piezometers the distribution and amount of hydrostatic pressures in the pervious substratum with and without a well system in operation, including the head between and immediately adjacent to the wells, and landward and beyond the ends of the well system.
- c. To estimate from piezometer data the "effective" source of underseepage and the rate of underseepage with and without relief wells in operation.
- d. To determine the maximum excess hydrostatic pressure which can exist landward of the levee at the Trotters site.
- e. To determine the effect of different well spacings on the reduction of substratum pressure and well flow.

Investigational procedures

9. In carrying out the above studies, the following steps were taken: (a) Thirty 6-in. ID wooden wells were installed along the toe of the levee at Trotters, Mississippi, from levee sta 53/51+55 to 54/8+25. These wells were installed on 50-ft centers and have screens which penetrate 50 per cent of the principal pervious aquifer. The wells are equipped with flap valves which permit closing the wells so as to obtain a system with different well spacings. (b) Some new borings were made both riverward and landward of the levee to obtain more information on surface and subsurface conditions. One undisturbed sand boring was also put down at the center of the well installation. Mechanical analyses and permeability tests were performed on samples obtained from these borings to determine the permeability of the foundation sands. The permeability of the pervious substratum at the site was also estimated from pumping tests made on the new relief wells and from the flow from the well system during the 1951 high water. (c) The distribution and amount of hydrostatic pressures in the pervious substratum were determined, for different well operating conditions, by piezometers. (d) The location of the effective seepage entrance was estimated from readings of piezometers located beneath the existing levee and landside berm. (e) The rate of underseepage with and without the relief wells in operation was estimated from model data* converted to the Trotters site conditions, and from the hydraulic gradient beneath the levee and berm, the permeability of the

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* Waterways Experiment Station Technical Memorandum No. 3-304, "Relief Well Systems for Dams and Levees on Pervious Foundations," November 1949.

foundation, and its depth. The natural seepage emerging along the toe of the seepage berm was measured during the 1950 high water when no wells were in existence at the site. (f) The maximum excess hydrostatic pressure that probably can exist along the landside toe of the levee at the Trotters site was estimated from piezometer readings obtained during the 1950 high water.

10. Detailed discussions and analyses of the above-described data are contained in subsequent parts of this report.

Future Studies

11. Because of the experimental nature of this project, the following measurements and data will be obtained during each significant high water:

- a. Individual well flow, and flow at different elevations in the well screen.
- b. Total flow in collector ditch.
- c. Natural seepage in existing landside drainage ditch at points opposite the quarter points of the well system.
- d. Head between wells and in the pervious stratum at various depths in the foundation both riverward and landward of the levee.

These data will be analyzed and published in the form of appendices to this report.

PART II: DESCRIPTION OF THE SITE

Selection of Site

12. The site at Trotters was selected for the new experimental relief well installation because of:

- a. Its previous record of underseepage and sand boils.
- b. The existence of a fairly uniform top stratum of clay approximately 9 ft thick.
- c. A thick substratum of pervious sands approximately 120 ft deep.
- d. Entry for seepage into the substratum provided by open riverside borrow pits near the levee and the Mississippi River approximately 3,000 ft from the levee.
- e. Available well-flow and piezometer data obtained during the 1943 high water on a well system installed that year.

Geology and Foundation Conditions

13. The levee at the site of the well system crosses an old river channel which extends from approximately levee sta 53/37 to 54/18. The sediments within this area consist of a comparatively thin, relatively impervious top stratum and a thick, pervious substratum extending to a depth of about 130 ft (see plates 4-7). From about sta 53/16 to 53/51 the top stratum consists of a fairly uniform clay layer about 15 to 20 ft thick overlain by a layer of natural levee deposits 4 to 7 ft thick. The top stratum thins to a rather uniform clay stratum, only about 9 ft thick, which extends along the toe of the levee from sta 53/51 to 54/18. The top stratum apparently thickens landward of the levee. A stratum of extremely uniform fine and very fine sand about 20 to 40 ft thick lies

immediately beneath the top stratum of clay and silts. The principal pervious stratum, approximately 85 to 90 ft thick, consists of uniform medium to coarse sands that increase in coarseness with depth. (Mechanical analyses of samples of these sands are shown in fig. (E), plate 13.) At the lower depths of this stratum a considerable amount of gravel is interspersed in the sand. Tertiary materials underlie this lower sand stratum. This condition of an upper stratum of relatively impervious clays and silts underlain by a stratum of fine sand, in turn underlain by a more massive stratum of coarser sands and gravels, is typical of many sites along the lower Mississippi River levees, except that the thickness of the very fine sand stratum at Trotters (Mi 54) is somewhat greater than usual.

14. The Mississippi River is approximately 3,000 ft from the center line of the levee at the Trotters site. However, the entire reach of levee is bordered by riverside borrow pits which, in certain areas, penetrate through the top stratum into the top of the pervious substratum and facilitate the entrance of seepage into the pervious foundation.

History of Underseepage at Site

15. The levee at Trotters was constructed in its present location in the summer of 1933 (see plate 1). The first high water of any consequence against this levee occurred in the spring of 1937. At that time heavy underseepage and many boils occurred along the toe of the levee, particularly in the area near sta 54/1. Because of this underseepage a 200-ft landside berm was constructed along the Trotters levee in 1939.

1937 high water

16. Very heavy underseepage was observed at the site during the

1937 high water, extending from the levee toe to a distance approximately 1000 ft landward between sta 53/34 to 54/26. About 300 to 500 relatively small sand boils occurred in this area. No substratum pressures were measured during the high water.

<u>1943 high water</u>

17. A considerable amount of data regarding substratum pressures beneath and landward of the levee and the distance to the "effective" source of seepage was obtained from the first system of piezometers and relief wells during the 1943 high water, which extended from about 20 May to 20 June. During this high water, the well system was kept closed until 1 June, on which date the wells were opened and left open, except for the day of 9 June when the river was 1 ft below the crest stage. During this period the maximum net head on the levee was 9.8 ft.

18. <u>Well flow data.</u> A bar graph of the well flow for 7 June is plotted in fig. (A) on plate 21. The generally lower flows of wells 1 through 24 were due in part to the outlets for these wells being 2 to 4 ft higher than the outlets for the remainder of the wells. Well 12 had a low outlet and consequently a considerably higher discharge than the other wells in this group. The well flows were also affected by the traverses left perpendicular to the levee across the riverside borrow pits. Wells landside of these berms usually showed somewhat lower flows than other wells nearby. The low flow from wells 65 and 66 was due to the well screens being placed in the upper, very fine sand stratum, whereas the screens for the other wells were placed in the deeper, more pervious sands.

19. The generally lower flows of wells 58 through 81 as compared to

wells 25 through 57 may be the result of a more pervious landside top stratum. The geological investigation indicated that the landside top stratum at approximately sta 54/17 changes from impervious clayey channel filling to relatively pervious deposits. Therefore, considerable pressure relief was probably afforded by the top stratum landward of wells 58 to 81 resulting in lower well flows.

20. The average flow from wells 23-49 in the central portion of the system at the crest stage of the river was 41.7 gpm with a net head on the levee of about 9.8 ft, or about 4.5 gpm per ft of net head on the levee.

21. Pressure relief. As the wells were opened several days in advance of the crest of the river, the amount of pressure reduction resulting from operation of the wells at the maximum river stage cannot be obtained directly. However, the hydrostatic pressure in the pervious stratum at the floodcrest is estimated at 1.2 to 1.5 ft less than it would have been if the wells had not been opened. This reduction in artesian head is borne out by the effect demonstrated by closing the wells on 9 June (see piezometric data on plate 8). With the well system closed, piezometers M-3-X and M-4-W, located at the toe of the seepage berm, both recorded artesian pressures equal to approximately 33 per cent of the net head on the levee. Opening the wells reduced the excess head at the levee toe by about 32 per cent. Before the wells were opened a number of small sand boils had occurred in the bottom of the drainage ditch paralleling the line of wells. When the wells were opened about half of these sand boils disappeared and the activity of the remainder was diminished.

<u>1950 high water</u>

22. <u>Seepage data.</u> During the high-water period from 26 January to 28 February 1950, approximately 100 sand boils ranging in size from 3 to 10 in. occurred in a shallow drainage ditch parallel to and about 50 ft from the toe of the existing seepage berm between sta 53/36 and 54/34 (see plate 2). Considerable seepage was also noted emerging from the ground along the toe of the berm through crayfish holes and other fissures. The natural seepage between the berm toe and 100 ft landward was measured at this site between sta 53/48+15 and 54/10+25 during the 1950 high water. This seepage amounted to 55 gpm per 100 ft of levee for a net head of 13.5 ft, or 4.0 gpm per 100 ft of levee per foot of net head on the levee. The computed total natural seepage passing beneath the levee was estimated at about 10 gpm per 100 ft of levee per foot of net head on the levee from the 1950 piezometer data (piezometer no. 5) and river stages.

23. <u>Piezometer data.</u> Piezometer data obtained during the 1950 high water, when no wells were in existence at the site, are shown on plate 9. During the 1950 high water the river stage reached an elevation of 193.8 or about 13.8 ft above the average ground surface landward of the levee and 15.7 ft above the water elevation in the drainage ditch paralleling the toe of the seepage berm. The maximum piezometer reading observed at the toe of the seepage berm was 183.0 in piezometer M-4-W. Thus, the maximum hydrostatic pressure observed at the toe of the seepage berm was 3.0 ft above the average ground surface or 4.9 ft above the tailwater in the drainage ditch along the toe of the berm. Fig. (A) on plate 10 is a plot of the readings for piezometer M-4-W vs river stage.

This plot shows that the maximum excess pressure which can probably exist at sta 54/1 is about elev 183.0. The maximum hydrostatic pressure at the toe of the berm, as measured by piezometer M-4-W, checks closely the computed maximum head, based on $i_c^* = 0.85$, which can develop beneath the top stratum along the drainage ditch (see plate 10). Also, from fig. (B) of this plate it may be noted that the maximum heads ever developed above the average ground surface and along the drainage ditch were approximately 35 and 50 per cent, respectively, of the net heads above the ground surface and above the tailwater in the drainage ditch. At higher river stages, the percentage will decrease considerably, due to the limiting amount of head which can develop beneath the top stratum landward of the levee toe.

24. Piezometric gradients in the pervious substratum along line M are shown on plate 11 for river stages of 186.3 and 193.8, the crest of the high water. At the crest of the flood, excess pressures above the ground surface were observed as far as 3500 ft landward of the toe of the seepage berm. The steep gradient from the river to a point approximately 280 ft landward from the berm and the flat hydraulic gradient beyond show that most of the seepage passing beneath the levee was rising to the surface in a strip about 300 ft wide along the toe of the seepage berm.

25. At the crest of the river stage, the readings of piezometers 5 and M-4-W, at the toe of the seepage berm, differed by approximately

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* i_c: The hydraulic gradient required to cause heaving or flotation of the top stratum. It is the ratio of the submerged unit weight of the soil to the unit weight of water.

2.2 ft. The lower head observed in shallow piezometer M-4-W as compared to that in piezometer 5 in the deep, more pervious sand indicates vertical seepage at the toe of the seepage berm.

Characteristics of Foundation

Top stratum

26. Thickness. Within the limits of the 1950 well system, the thickness of the relatively impervious top stratum of silts and clays ranges from 9 to 11 ft, except along the bottom of the drainage ditch previously referred to where the thickness is only 6.5 ft (see plates 6, 7, and 16). An average value of 10 ft was used in computing the permeability of the top stratum.

27. Permeability. During the 1950 high water, 55 gpm of natural seepage per 100 ft of levee was observed emerging in a strip approximately 100 ft wide immediately landward of and parallel to the existing seepage berm. On the basis of this flow measurement and piezometer readings taken at the toe of the berm, the coefficient of permeability of the 100-ft wide strip of top stratum (k_b) was estimated to be about 5×10^{-4} cm per sec. Using Barron's* formula for a leaking top stratum and the 1950 piezometer readings, k_b was determined to be about 1×10^{-4} cm per sec, which agrees fairly well with the value computed using observed flows. Although these values are rather high for a clay material, it is pointed out that the area in which the seepage was

* R. A. Barron, "The Effect of a Slightly Pervious Top Blanket on the Performance of Relief Wells," Proceedings, Second International Conference on Soil Mechanics, IV (1948).

measured contained numerous sand boils, crayfish holes and other fissures that were emitting considerable amounts of flow. These considerations probably account for the rather high quantity of measured seepage noted above. A value for k_b of 1 to 5 x 10^{-4} cm per sec was selected for use in the design computations discussed subsequently.

Upper fine sand stratum

28. Thickness. A stratum of fine to very fine sands approximately 35 ft thick exists immediately beneath the upper stratum of silts and clays. The thickness of this stratum of fine sand is shown on plate 6.

29. Permeability. A number of mechanical analyses were run on samples of the fine and very fine sands taken in 1940. An approximate value of the coefficient of permeability of these fine sands was determined from a correlation of effective grain size (D_{10}) and coefficient of permeability of sands found in the valley of the lower Mississippi River (see plate 12). The average value as determined by this method was approximately 75 x 10^{-4} cm per sec. Several permeability tests also were run on undisturbed samples in both horizontal and vertical directions. The averages of these values (see plates 4 and 5) were about 50 x 10^{-4} and 40 x 10^{-4} cm per sec in horizontal and vertical directions, respectively. For design purposes, a permeability of 50 x 10^{-4} cm per sec was assigned to this fine to very fine sand stratum. This stratum has an effective carrying capacity equivalent to about a 2-ft thickness of the underlying medium and coarse sand stratum.

Medium and coarse sand stratum

30. Thickness. The principal seepage-carrying stratum of medium and coarse sands averages about 85 to 90 ft in thickness, but varies with the

thickness of the upper fine sand stratum and the top of underlying Tertiary formation. The fine sand stratum overlying the medium sand stratum adds an additional effective thickness to the principal seepage-carrying stratum of about 2 ft. In the original design of the 1950 well system the pervious sand stratum was assumed to have a thickness of 88 ft. On the basis of another boring made the summer of 1950 and other studies, the pervious sand strata are believed to have an effective thickness of about 90 ft.

31. Permeability. The permeability of the sand stratum at the Trotters site, as determined from permeability tests on undisturbed and remolded samples and from correlations of effective grain size with permeability, is shown adjacent to the borings on the soil profiles, plates 4-7. From these data the permeability of the sand stratum at Trotters was estimated to be about 400 x 10^{-4} cm per sec.

32. The permeability of the pervious stratum (k_f) at Trotters (Mi 54) was also estimated from flow measurements made on the 1943 relief well system during the 1943 high water (fig. (A), plate 21), the natural seepage measurements made during the 1950 high water, and piezometer data collected during both high waters (plates 8 and 9). This estimate was made in the manner described in the following paragraph.

33. On 7 June 1943 the average flow from wells 28 to 46 within the limits of the proposed well system was 59.8 gpm per 100-ft station. The pressure gradient beneath the levee as determined by piezometers M-2-X and M-4-W was 0.0077, and the excess head above the well outlets at the toe of the berm was about 2.6 ft. On 9 February 1950 the natural seepage along a 100-ft-wide strip paralleling the seepage berm amounted to 55 gpm

per 100-ft station with an excess head at the toe of the berm of 5.4 ft. On the assumption that the natural seepage at the toe of the berm is proportional to the artesian head, the natural seepage emerging along the above-mentioned 100-ft-wide strip during the 1943 high water with the 1943 wells flowing would be $55 \times \frac{2.6}{5.4} = 26$ gpm per 100-ft station. The flow passing beyond the 100-ft-wide strip between the toe of the berm and the drainage ditch was estimated, on the basis of the following assumptions and computations, to be 43.5 gpm per 100 ft of levee on 7 June 1943*.

<u>a</u>. The actual permeability of the sand foundation was assumed, from the pumping tests on the wells, to be 1250×10^{-4} cm per sec or 0.25 ft per min.

<u>b</u>. The average hydraulic gradient from the toe of the berm to the drainage ditch was taken as 1/2 of that beneath the levee on the assumption that the wells were intercepting 50 per cent of the total seepage passing beneath the levee.

<u>c</u>. The hydraulic gradient between the drainage ditch and piezometer M-6-X was estimated to be 0.00257.

<u>d</u>. Q (passing beyond drainage ditch) = kiA = $0.25 \times 0.0257 \times 90 \times 100 \times 7.5 = 43.5$ gpm.

Thus, the total flow passing beneath the levee would be 26 + 59.8 + 43.5 = 129 gpm per 100-ft station. Using the formula $k_f = \frac{Q}{i\,A}$ and an equivalent thickness of pervious stratum of 90 ft, $k_f = \frac{129}{7.5 \times (0.0077)\ (90 \times 100)} =$ 0.25 ft per min = 1250×10^{-4} cm per sec.

34. In 1943, field pumping tests were made at Commerce, Mississippi, (sta 23/24+75) in the Memphis District, and at Wilson Point, Louisiana,

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* The flow beyond the drainage ditch was inadvertently omitted in the original computations of the permeability of the pervious foundation from the 1943 well flow data. As a result of this omission the permeability of the foundation was computed to be 825×10^{-4} cm per sec instead of the more correct value of 1250×10^{-4} cm per sec.

(sta 852) in the Vicksburg District, to determine the permeability of the pervious substratum at these sites*. These pumping tests indicated an average permeability of 700×10^{-4} cm per sec at Commerce and 600×10^{-4} cm per sec at Wilson Point.

35. A permeability of $k_f = 800 \times 10^{-4}$ cm per sec was used in the original design of the new well system at Trotters on the basis of the pumping tests at Commerce and Wilson Point and the incorrectly computed permeability from the field measurements at Trotters in 1943 and 1950.

36. During the installation of the 1950 well system, rather comprehensive field permeability pumping tests were made at wells 1, 15 and 30. The permeability of the pervious substratum as determined from these pumping tests is summarized in table 2. In addition to the pumping tests on wells 1, 15, and 30, for which drawdowns were observed at considerable distances from the well, pumping tests were also performed on each well in which only the well flow and drawdown in the well were observed. The permeability of the foundation then was computed from the pumping tests on the individual wells. The permeabilities as determined from these tests are also shown in table 2. The data presented in this table indicate that the over-all permeability of the sand foundation at Trotters is about 1250×10^{-4} cm per sec. The pumping tests and determination of the permeability of the sand foundation will be described subsequently in more detail.

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* Waterways Experiment Station Technical Memorandum No. 3-299, "Field Permeability Tests, Commerce Landing, Mississippi, and Wilson Point, Louisiana," August 1949.

Source of seepage

37. Seepage can enter the pervious substratum riverward of the levee through the bank of the Mississippi River, approximately 3000 ft from the levee, and through river borrow pits from which much of the impervious top stratum was removed down to the top of the pervious sands during construction of the levee and seepage berm. The effective source of seepage computed from the 1943, 1950, and 1951 piezometer data is shown for various river stages in fig. (C) on plate 22. The distances shown were based on the hydraulic gradient in the pervious substratum beneath the levee. The effective source of seepage was taken as the distance from the line of wells to the point where the hydraulic grade line beneath the levee extended intersects the stage of the river.

38. Pressure gradients given by piezometers M-2-X and M-3-X, and by M-2-X and M-4-W during the 1943 high water showed the effective seepage entrance to be about 950 to 1350 ft from the well system. During the 1950 high water, piezometers 2 and 4 indicated the effective seepage entrance to be about 975 to 1250 ft from the well system. The distance from the effective source of seepage to the well line was taken to be 1000 ft in the original design of the well system. (The distance to the effective seepage entrance during the 1951 high water ranged from 925 to 1285 ft.)

39. It can be noted from fig. (C), plate 22, that the distance to the effective seepage entrance decreased considerably with increasing river stages. This is attributed to scouring away of silt, deposited in the bottom of riverside borrow pits during previous high waters, by the increasing velocity of the water as the river rises. As the river

is approximately 3000 ft from the line of wells, it is apparent that most of the seepage is entering the pervious foundation through the riverside borrow pits and top stratum.

PART III: DESIGN OF WELL SYSTEM AND APPURTENANCES

Well System

Criteria of design

40. The primary requirements of a relief well system for the control of underseepage are:

- a. The wells should penetrate into the principal water-carrying strata and be placed sufficiently close together to intercept the seepage and reduce the pressure which otherwise would act beyond the wells.
- b. The wells must offer little resistance to water flowing into and out of them, they must prevent infiltration of sand into the well after initial pumping, and they must resist the deteriorative action of the water and soil.

41. The primary criteria used in determination of the well penetration, spacing, and size was the amount of artesian head considered allowable midway between the wells with the project flood against the levee. The well system was designed on the basis that the maximum artesian head midway between the wells should be little, if any, above the average ground surface landward of the levee when the water surface in the collector ditch is held, by pumping, to elev 178.0. As the normal ground surface is at about elev 180.5, this would allow approximately 2.5 ft of residual net head between the wells, or 8.4 per cent of the maximum net head on the levee. The critical net substratum pressure along the drainage ditch at the Trotters site is about 5.5 ft. The maximum artesian heads which have been measured along the existing berm during the 1943 and 1950 floods range from 3 to 5 ft (see plates 8-10). These heads produced numerous sand boils varying in size from 3 to 10 in. and discharged from 0.1 to 2 cu yd of sand.

42. The new well system at Trotters was designed for two different cases. Case A was based on the assumption that all flow from the well system would be pumped over the levee and the tailwater over the wells held to elev 178.0, or that natural flow conditions would be such that the water in the collector ditch would not rise above elev 178.0. In this case, where the artesian head landward of the levee would not be above the ground surface, there should be little natural seepage landward of the wells, and thus it could be assumed that the landward top stratum would be analogous to the impervious top stratum case. For this case the maximum allowable head between wells (P) would be 8.4% H.

Stage of project flood	=	Elev 207.7
Tailwater above well outlets in collector ditch	=	178.0
Maximum net head on well system (H)	=	29.7 ft

Maximum allowable head between wells = 2.5 ÷ 29.7 = 8.4% of the river head above tailwater in collector ditch.

43. Case B was based on the assumption that the flow from the wells would not be pumped and that the tailwater over the wells would be at elev 180. In checking the design of the system for case B, the net head between wells was assumed to be not more than 2.0 ft above the natural ground surface or 7.2 per cent of the maximum net head on the levee. In this case the top stratum landward of the wells was assumed to be only relatively impervious and to have a permeability of about 1 to 5 x 10^{-4} cm per sec.

Stage of project flood	=	Elev 207.7
Approximate tailwater	=	180.0
Maximum net head on levee (H)	=	27.7 ft

Maximum assumed head between wells = 2.0 ÷ 27.7 = 7.2% of the river head above the tailwater over the wells.

44. The well system was designed primarily for reduction of artesian pressure at the landward toe of the existing seepage berm. However, in addition to the reduction of landside artesian pressures, the well system will reduce the amount of seepage emerging landward of the well system. An estimate was made of the amount of reduction for both cases A and B mentioned above, but this factor was not used as a criterion of design. An estimate was also made for case B of the amount that the total flow (well flow plus seepage with wells open) would be increased over the natural seepage with no well system.

45. Most formulas and model test data available for the design of relief well systems are based on an infinite line of wells. In the design of the experimental Trotters relief well system, it was decided to make the system only long enough to provide a central section of about 1000 ft which, on the basis of approximate computations presented later, would be affected relatively little by end effects. The well system as finally designed consisted of 30 wells spaced on 50-ft centers, giving a line of wells 1450 ft long.

<u>Bases of design</u>

46. The following assumptions and design values were used in designing the Trotters well system. Values used in re-analysis of the well system based on data obtained during and after installation of the system are shown by "#" in the following paragraphs.

a. Well diameter = 20 in., well radius (r_w) = 10 in. = 0.833 ft.

b. ID of well screen and riser pipe = 6 in.

c. Thickness of gravel filter = 6 in.

d. Perforations in well screen, 3/16 in. to 1/4- x 2-in. slots;

minimum slot area = 18 sq in. per ft of well screen.

e. Infinite line source of seepage, parallel to an infinite line of relief wells, assumed in computing well flow and head between wells. End effects were considered in computing total flow from system.

f. Distance from source of seepage to line of wells (s) = 1000 ft, #(s = 900 ft).

g. Thickness of top stratum (z) = 10 ft.

h. Thickness of pervious stratum (d) = 88 ft, #(d = 90 ft).

i. Permeability of top stratum (k_b) = 1 to 5 x 10^{-4} cm/sec.

j. Permeability of pervious stratum (k_f) = 800 x 10^{-4} cm/sec, #(k_f = 1250 x 10^{-4} cm/sec).

k. Penetration of well screen based on 50 per cent penetration into medium and coarse sand stratum.

47. The following formulas and model data were used in designing the well system:

Case A: Impervious top stratum; tailwater above wells at or below elev 178.0.

(a) Muskat formula (100% penetration)*.

(b) Jervis design curves (partial penetration)**.

(c) WES, Model A-a-1 (all penetrations)***.

(d) WES, Model B-a (100% penetration)***.

Case B: Semipervious top stratum; flow from wells to natural drainage ditches and tailwater above wells at elev 180.0.

(a) Model A-a-2 (all penetrations)***.

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* Morris Muskat, "The Flow of Homogeneous Fluids Through Porous Media" (New York: McGraw - Hill Book Co., Inc., 1937).

** T. A. Middlebrooks and W. H. Jervis, "Relief Wells for Dams and Levees," Transactions (ASCE) Vol. 112, 1947.

*** Waterways Experiment Station models described in T.M. 3-304, "Relief Well Systems for Dams and Levees on Pervious Foundations," November 1949.

All model data were adjusted to the foundation conditions existing at Trotters and the size of well used at Trotters.

48. Computations of head losses in well screen and riser pipe, control culvert, and collector ditch were based on commonly accepted hydraulic formulas.

Design computations*

49. Theoretical. Before making a selection of well spacing and penetration, the head between wells and well flow were computed for various well spacings (a) and screen penetration (W) of 50 and 100 per cent. The results of these computations, together with finally selected design curves, are shown in figs. (A), (B), and (C) on plate 13. Examples of the computations made in deriving these curves, using a well spacing of 50 ft,** are given in the following paragraphs.

50. Case A. Impervious top stratum.

a. Muskat formula (W = 100%, a = 50 ft). (See p A4 and fig. A2, WES TM 3-304.)#

$$\frac{a}{r_w} = \frac{50}{0.833} = 60; \quad \frac{s}{a} = \frac{900}{50} = 18; \quad \text{from fig. A2,}$$

$$\frac{P}{H} = 0.022 \quad \text{or} \quad P = 0.022 \times 29.7 = 0.65 \text{ ft}$$

$$\frac{Q_w}{k\ H\ d} = 0.0545 \quad \text{or} \quad Q_w = 0.0545 \times 0.25 \times 1 \times 90 \times (7.5)$$

$$= \#9.2 \text{ gpm/ft H} \quad \text{or} \quad Q_w = \#9.2 \times 29.7 = 273 \text{ gpm, for project flood.}$$

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* All values given in design computations regarding well flow, landward pressure, and seepage are based on an infinite line of wells unless otherwise stated.

** Well spacing and penetration for the Trotters system are 50 ft and 50%, respectively.

b. Jervis' curves (W = 50%, a = 50 ft). (See fig. A4, WES TM 3-304.)

$\frac{a}{r_w} = 60$; $\frac{d}{a} = \frac{88}{50} = 1.8$; from fig. A4,

$\frac{Q_w}{H} = \frac{k\ a\ d}{S + (EL)}$ where $\frac{(EL)}{a} = 0.82$

or extra length (EL) = 0.82 x 50 = 41 ft

$Q_w = \frac{0.16 \times 50 \times 88}{1000 + 41}(7.5) = 5.05$ gpm/ft H, #(8.96 gpm/ft H)

or $Q_w = 5.05 \times 29.7 = 150$ gpm, #(266 gpm) for project flood.

$\frac{P}{H}\left(\frac{s}{a} + \frac{EL}{a}\right) = 0.77$

$\therefore \frac{P}{H} = \frac{0.77}{\left(\frac{1000}{50} + 0.82\right)} = 0.037$, or $P = 0.037 \times 29.7 = 1.1$ ft

for project flood.

#($\frac{P}{H} = 0.041$, or P = 1.2 ft for project flood)

c. WES Model A-a-1 (W = 50%, a = 50 ft).

Notes: (1) See fig. 15, WES TM 3-304 for model data.

(2) Model data adjusted to Trotters site and well system by means of Jervis curves for partial penetration - see fig. A4, TM 3-304.

(3) Subscripts P and M refer to prototype and model, respectively.

From the model for W = 50%; a = 50 ft; $r_w = 1.5$ ft;

d = 150 ft; $k_f = 0.10$ ft/min; s = 1000 ft:

$$\left(\frac{Q_W}{H}\right)_M = 5.5 \text{ gpm} \quad \text{and} \quad \left(\frac{P}{H}\right)_M = 0.040$$

$$\left(\frac{Q_w}{H}\right)_P = \frac{d_P}{d_M} \times \frac{k_{f-P}}{k_{f-M}} \times \frac{(s + EL^*)_M}{(s + EL^*)_P} \times \left(\frac{Q_w}{H}\right)_M , \quad \text{or}$$

$$\left(\frac{Q_w}{H}\right)_P = \frac{88}{150} \times \frac{0.16}{0.10} \times \frac{(1000 + 37)}{(1000 + 41)} \times 5.5 = 5.1 \text{ gpm}, \#(9.05 \text{ gpm}) \text{ or}$$

$$Q_{w-P} = 29.7 \times 5.1 = 151 \text{ gpm}, \#(Q_{w-P} = 268 \text{ gpm}) \text{ for project flood.}$$

$$\left(\frac{P}{H}\right)_P = \frac{\left[\frac{P}{H}\left(\frac{s}{a} + \frac{EL}{a}\right)\right]_P^* \left[\left(\frac{s}{a}\right)_M + \left(\frac{EL}{a}\right)_M^*\right]}{\left[\frac{P}{H}\left(\frac{s}{a} + \frac{EL}{a}\right)\right]_M^* \left[\left(\frac{s}{a}\right)_P + \left(\frac{EL}{a}\right)_P^*\right]} \times \left(\frac{P}{H}\right)_M , \quad \text{or}$$

$$\left(\frac{P}{H}\right)_P = 0.040 \times \frac{0.77}{0.55} \times \frac{\left[\frac{1000}{50} + 0.75\right]}{\left[\frac{1000}{50} + 0.82\right]} = 0.055 \ \#(0.062)$$

or $P = 0.055 \times 29.7 = 1.63$ ft, #(1.83 ft) for project flood.

d. <u>WES Model B-a (W = 100%).</u> Only data for 100% well penetration in Model B-a (fig. 26, WES TM 3-304) were applicable to the Trotters site because of the stratification of the foundation in the model. The model data, for W = 100%, were adjusted to the Trotters site in a manner similar to that described in c

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* From Jervis curves for partial penetration of 50 %.

above, and then plotted in fig. (B) on plate 13.

51. Case B. Semipervious top stratum.

WES Model A-a-2 (W = 50%, a = 50 ft).

Notes: (1) See fig. 16 and 18, WES TM 3-304 for model data.

(2) Data for Model A-a-2 considered applicable to Trotters site because both model and prototype have approximately the same s = 1000 ft and the same artesian pressure (36% H) at landside toe of levee with no well system in operation.

(3) Model well flow and head between wells adjusted to Trotters site by the method described in the previous paragraph.

From the model, $\left(\frac{Q_w}{H}\right)_M = 4.6$ gpm and $\left(\frac{P}{H}\right)_M = 0.040$

Natural seepage (no wells), $\left(\frac{Q_s}{H}\right) = 4.0$ gpm/50 ft of levee

Natural seepage (with wells), $\left(\frac{Q_{s-w}}{H}\right) = 0.6$ gpm/50 ft of levee

Adjusted to Trotters, $\left(\frac{Q_w}{H}\right)_P = 4.3$ gpm or $Q_w = 4.3 \times 27.7 =$ 119 gpm for project flood.

$$\# \left[\left(\frac{Q_w}{H}\right)_P = 7.6 \text{ gpm or } Q_w = 210 \text{ gpm} \right]$$

$$\left(\frac{P}{H}\right)_P = 0.055 \quad \text{or} \quad P = 0.055 \times 27.7 = 1.52 \text{ ft}$$

$$\# \left[\left(\frac{P}{H}\right)_P = 0.062 \quad \text{or} \quad P = 1.7 \text{ ft} \right]$$

Natural seepage (no wells), $\left(\frac{Q_s}{H}\right) = 4.0 \times \frac{88}{150} \times \frac{0.16}{0.10} = 3.75$ gpm/50 ft of levee

$$\# \left[\left(\frac{Q_s}{H}\right) = 6.6 \text{ gpm/50 ft of levee} \right]$$

$Q_s = 3.75 \times 27.7 = 104$ gpm/50 ft of levee for project flood.

$$\# \left[Q_s = 183 \text{ gpm/50 ft of levee} \right]^{*}$$

Natural seepage (with wells), $\left(\frac{Q_{s-w}}{H}\right) = 0.6 \times \frac{88}{150} \times \frac{0.16}{0.10} = 0.56$ gpm

$$\# \left[\frac{Q_{s-w}}{H} = 1.0 \text{ gpm/50 ft of levee} \right]$$

$Q_{s-w} = 0.56 \times 27.7 = 15$ gpm/50 ft of levee for project flood.

$$\# \left[Q_{s-w} = 28 \text{ gpm/50 ft of levee} \right]^{**}$$

Total flow (well + seepage), $Q_{s+w} = 119 + 15 = 134$ gpm/50 ft of levee.

$$\# \left[Q_{s+w} = 210 + 28 = 238 \text{ gpm/50 ft of levee} \right]$$

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* The computed total natural seepage passing beneath the levee with no wells during the 1950 high water and river stage of 193.7 was

$$Q_s = 0.25 \times \frac{(13.7 - 5.4)}{900} \times 90 \times 50 \times 7.5 = 78 \text{ gpm per 50 ft of levee,}$$

or 5.7 gpm per ft of net head for this river stage. However, as the maximum net head that can develop in piezometer 5 at the toe of the seepage berm is about 8 ft, the total natural seepage passing beneath the levee at the project flood would be about 185 gpm per 50 ft of levee. These values agree closely with those obtained from the model data.

** Natural seepage in 100-ft strip landward of levee with wells estimated from 1950 piezometer data (piezometer 5) and seepage measurements =

$$\frac{55}{2} \times \frac{1.7 + 0.5 \text{ head loss in wells (head between wells)}}{5.4 \text{ (piezometer 5)}} = 11 \text{ gpm per}$$

50 ft of levee, estimated seepage passing beyond drainage ditch = 20 gpm, or total natural seepage with wells = about 30 gpm per 50 ft of levee for project flood, or about 1 gpm per ft of net head on levee per 50 ft of levee.

(For Case B and H = 27.7 ft, Q_W = 210 gpm based on Model A-a-2)

Increase in total flow due to wells $= \frac{238 - 185}{185} = 29\%$.

52. An examination of the curves for head between wells (figs. (B) and (C), plate 13) indicates the following:

a. The Muskat formula and Models A-a-1 and B-a give almost identical results for 100 per cent well penetration.

b. The values of head between wells as computed from Model A-a-1 and the Jervis curves for partially penetrating wells agree reasonably well for 50 per cent penetration.

c. The values determined from Model A-a-2 (semipervious top stratum) are slightly lower than corresponding values for the impervious case except for close well spacings with 50 per cent or more penetration. This slight exception is of course, impossible and is probably the result of discrepancies in the model tests.

53. The final design curves selected from the curves shown for computing the head between wells for the Trotters system are also shown for cases A and B in fig. (C) on plate 13. The curves shown do not include any screen or hydraulic head losses in the well.

54. The design curves for well flow are shown for both cases A and B in fig. (A) on plate 13. These curves are averages of individual curves computed from the various methods available, all of which give consistent and close results.

55. Well spacing and penetration. From the design curves in fig. (A), plate 13, a well spacing of 50 ft and a penetration of 50 per cent were selected as being the best combination of spacing and penetration which would result in the desired pressure relief. A summary of well flow and head between wells as computed from the various available methods for the above spacing and penetration is given on the following page.

Case	Method or Model	Theoretical Head between Wells (P)		Well Flow in gpm		Natural Seepage in gpm /50 Ft
		P in % H	P in Ft	$(Q/H)_w$	Q_w	
A - Imperious Top Stratum H = 29.7	Jervis	3.7 #4.1	1.1 #1.2	5.0 #9.0	148 #266	-
	A-a-1	5.5 #6.2	1.6 #1.8	5.1 #9.1	151 #270	-
	Design	5.6 *7.2	1.7 *2.1	5.0 *10.0	148 *300	-
B - Semi-pervious Top Stratum H = 27.7	A-a-2	5.5 #6.2	1.5 #1.7	4.3 #7.6	119 #210	15 #28
	Design	5.6 *7.2	1.6 *2.0	4.3 *8.7	119 *240	15 *25
No Wells	A-a-2			0	0	#183
	Piez 5					#185

* Based on 1951 well flow and piezometer data.

Screen and hydraulic head losses in the well must be added to the above head between wells. It is also pointed out that the above values are based on a well line of infinite length. As the Trotters system is only 1450 ft long, the performance of the wells near the ends of the system will be affected by end effects. However, as subsequently discussed, the above design values should be applicable to the wells in the center portion of the system.

56. Screen and well losses. Screen entrance and hydraulic head losses in the well were computed and added to the computed head between the wells as described in the following paragraphs.

57. The filter and screen entrance losses were estimated from well tank tests made on a slotted wooden well screen and gravel filter described in Appendix A. The open slot area for the screen tested was

10.5 sq in. per linear ft of well screen whereas the slot area of most of the Trotters well screens is 18 sq in. However, no allowance was made for the increased area in estimating the filter and screen entrance losses.

58. Friction losses in the well were computed from tables in the "Wood Pipe Handbook" published by the National Tank and Pipe Company. Velocity head losses were computed from the formula

$$h_v \doteq \frac{V^2}{2\,g}$$

59. Hydraulic head losses in the wells for the revised flow based on 1950 and 1951 pumping and well flow data are computed for both cases A and B on the following pages. Screen entrance and hydraulic losses in the well are shown graphically for various well flows in figs. (B) and (C), plate 20.

<u>Case A. Impervious top stratum (H = 29.7 ft)</u>

Well flow	$\# Q_w$ = 300 gpm
Computed P	2.10 ft
Entrance and well loss, h_w	<u>0.91</u>
Total head between wells	= #3.01 ft

Thus for case A, the maximum artesian head between the wells in the central portion of the system should not exceed #3.0 ft above the tailwater in the collector ditch. For river stages less than the project flood, the net head between wells should range between #7 and #10 per cent of the river head above the water elevation in the collector ditch.

Case B. Semipervious top stratum (H = 27.7 ft)

Well flow	$\#Q_w$ = 240 gpm
Computed P	= 2.00 ft
Entrance and well loss, h_w	0.53
Total head between wells	#2.53 ft

For case B, the maximum artesian head between wells should not exceed #2.5 ft above the average ground elevation of approximately 180.0. As for case A, the head between wells for river stages less than the project flood should range between 7 and 9 per cent of the net head on the levee.

60. Length of system. Since the length of the relief well system is 1450 ft and the effective seepage entrance only about 1000 ft from the line of wells, the effects of seepage passing around the ends of the system and entering the wells from the landside will be appreciable. This effect was estimated by means of a flow net (shwon in fig. (D) on plate 22) in which two line sinks were assumed parallel to the source of seepage and fully penetrating the pervious stratum. For these flow conditions a flow net for a horizontal section may be constructed. One sink, which represented the well system, is 1450 ft in length and located 1000 ft from the source of seepage; the other, representing the effective seepage exit, is situated 1600 ft from the source of seepage and is infinite in extent. The effective seepage exit was estimated in a manner similar to the effective seepage entrance previously discussed. From piezometer readings at the site, it is known that natural landward seepage reduces the artesian head at the toe of the berm to approximately 40 per cent of the net head on the levee. Since the effective seepage

entrance was estimated from piezometer data to be 1000 ft from the line of wells, the "effective" seepage exit would be located approximately 600 ft landward from the wells. From the flow net on plate 22, end effects were estimated to increase the total flow from the 30 wells approximately 40 per cent over that expected from the same number of wells in an infinite line of wells; actually end effects increased the total flow from the wells by only about 10 per cent (see paragraph 92). Thus, the total flow from the system, on the basis of the 1951 observations, would be

<u>Case A:</u> $\#Q_W = 10 \times 30 \times 1.10 = 330$ gpm/ft head, or

$\#Q_W = 330 \times (29.7 - 0.9) = 9500$ gpm or 21.1 cfs for project flood.

<u>Case B:</u> $\#Q_W = 8.7 \times 30 \times 1.10 = 287$ gpm/ft head, or

$\#Q_W = 287 \times (27.7 - 0.7 - 0.5) = 7600$ gpm or 16.9 cfs for project flood.

Flow around the ends of the system may affect appreciably the head between and the flow from the end wells. However, flow and head in the central portion of the system should correspond reasonably well to those computed for an infinite line of wells, as can be seen from figs. (D) and (E), plate 22.

Wells and Appurtenances

61. The design of the wells and appurtenances to the well system included the following features:

- a. A gravel filter to drain the foundation sands with a minimum loss of head.
- b. Placement of the well outlets in the bottom of the collector ditch at the lowest elevation possible without preventing natural drainage from the collector ditch into the existing outfall ditch.

c. Means for closing or throttling the wells at any time.

d. A collector ditch for collecting the flow from the wells and surface water from the levee and berm.

e. A sump from which water in the collector ditch can be pumped over the levee.

f. A 30-in. control culvert at the downstream end of the collector ditch with a gate on the upstream end to contain water in the collector ditch and sublevee basin when desired.

g. A sublevee around the wells and collector ditch to prevent flow of seepage and surface water from adjoining areas into the collector ditch when the wells are being pumped, and to provide a means for impounding water over the well system if desired.

h. An overflow spillway for the sublevee basin.

Details of these various design features are shown on plates 14-16.

Well screen, gravel filter, and riser pipe

62. The well screen consists of 6-in. ID redwood pipe slotted with 3/16- to 1/4- by 2-in. slots. The slots have an open area of 18 sq in., or about 8 per cent of the inside area of the pipe, per linear ft of pipe. The screen portion of the well is 40 ft long. It is surrounded with a 6-in. layer of filter gravel (see fig. (E), plate 13) extending from 1 ft below the plug in the bottom of the screen to 1 ft above the top of the screen. The design of this filter was primarily based on the filter criterion $\frac{D_{15} \text{ Filter}}{D_{85} \text{ Sand}} < 5.0$. The filter has an estimated permeability of about 5000×10^{-4} cm/sec. The minimum D_{85} of the filter is 1.2 times the slot width. The riser pipe is also of 6-in. ID redwood pipe. The backfill around the riser pipe above elev 154, the elevation of the minimum low water table, consists of concrete. Below this elevation the backfill consists of fine sand. Details of the well screen and riser pipe are shown on plate 15.

63. The top of the riser pipe is capped with a brass flap valve (see plate 15) which may be used to close the wells or throttle the flow if desired. The wells will be kept closed when there is no head against the levee.

Collector ditch

64. The collector ditch for the wells is about 3.5 ft deep and has a bottom width of 3 ft, side slopes of 1 on 2, an average bottom elevation of 177.1, and an invert grade of 0.00025. It is paved with porous concrete underlain with a 2-in. blanket of well-graded coarse sand. The paving has a thickness of 6 in. in the bottom of the ditch and 4 in. on the side slopes. Details of the collector ditch are shown on plates 14 and 16. With a grade of 0.00025 and $n = 0.0225$, the ditch has a capacity of about 35 cfs when flowing full. Thus, its capacity should be adequate to carry the estimated maximum well flow (Q_W) of #17 cfs for case B when running full, as will be the case when the control culvert is passing 17 cfs.

Control culvert

65. The control culvert through the sublevee at the downstream end of the well system (plates 2 and 14) consists of a 30-in. asphalt-coated, corrugated pipe of No. 12 gage metal 58 ft long with the invert paved with 1/8 in. of asphalt. The culvert will pass the total estimated maximum flow from the well system, including a 40 per cent allowance for end effects, plus the runoff from the levee into the sublevee basin, without the water within the sublevee rising above elev 184.0, the net grade of the sublevee. The hydraulics of the control culvert were checked on the basis of the following assumptions and well flow

data obtained during the 1951 high water:

a. Well flow for project flood (case B), #Q_W = 8.7 gpm/ft head.

b. Total well flow increased 10% to allow for end effects, Q_W.

c. Runoff area, A = 14.7 acres.

d. Runoff formula, $Q_R = A\ I\ R$ (cfs) where I = 0.15 and

R = 6 in./hr (1-in-10-yr rainfall) or I = 0.45 and R = 2 in./hr.

e. Time of concentration of surface runoff = 5 min.

f. Control culvert: 30-in.-diameter corrugated metal pipe, length = 58 ft, coefficient of roughness (n) = 0.021.

g. Culvert design flow $Q_C = (Q_W + Q_R)$.

h. Elevation of tailwater downstream of control culvert = 180.0 msl.

The hydraulic computations made are summarized below.

Runoff = 14.7 x 0.15 x 6, or 14.7 x 0.45 x 2, = 13.2 cfs

Well flow (assuming average water elev in sublevee = 182.5, H = 207.7 - 181.6 - 0.5 = 25.6 ft)

$$Q_W = \frac{25.6 \times 8.7 \times 30}{7.5 \times 60} + 10\% \text{ (for end effects)} = \underline{16.4} \text{ cfs } \#$$

Total runoff and well flow = 29.6 cfs #

Culvert design flow, For Q_C = 30 cfs and the culvert outlet submerged,

Friction losses in culvert = 0.015 x 58 = 0.87 ft

Entrance and velocity head losses = $1.5\,\frac{v^2}{2g} = 1.5 \times \frac{4.8^2}{64.4} = 0.54$

Loss through Calco 101 gate = 0.0

Total head losses through culvert = 1.41 ft

Required elev water at control culvert = 180.0 + 1.4 = 181.4

Elev water at center of collector ditch = 181.4 + 0.2 = 181.6. Thus, the control culvert should pass the estimated maximum runoff and well flow (30 cfs) without causing the water in the sublevee basin to rise above elev 181.6. With water in the sublevee basin up to the net grade of the sublevee, elev 184.0, the control culvert will pass 45 cfs.

66. The control culvert will pass the maximum well system flow for case B, $Q_W = \frac{(207.7 - 180.7 - 0.5) \times 8.7 \times 30}{7.5 \times 60} + 10\,\% = 16.9$ cfs, with a head of 0.5 ft or water elevation at the middle of the system of 180.7.

Overflow spillway

67. The overflow spillway for the sublevee basin consists of a 30-in. riser from the control culvert with a flared, removable top at elev 183.0 (see plate 14). It has a capacity of about 40 cfs with the landward tailwater at elev 180.0 and water in the sublevee at elev 184.0. This should be adequate to handle the maximum flow from the well system plus runoff into the sublevee basin.

Road culverts

68. The two 30-in. road culverts at the end of the outfall ditch (plate 15) will pass approximately 50 cfs with a downstream tailwater elevation of 180.0 and an upstream water elevation of 181.0, the crown of the road.

PART IV: INSTALLATION AND CONSTRUCTION OF WELLS AND APPURTENANCES

Installation of Wells and Pumping Tests

69. The holes for the wells at Trotters were made by the reverse-rotary method. This method consists of recirculating a drilling fluid in the hole as it is advanced by fishtailing. As the fishtailing bit is advanced the drilling fluid is pumped up through the drill rod into a stilling basin in which the sand settles out but the fines in the drilling fluid flow back into the drill hole. The drilling fluid was made by adding natural top stratum soil to the drilling water. After advancing the hole approximately 25 ft, a 21-ft section of 24-in. pipe was installed. After this, the hole was advanced to a depth of approximately 45 ft and a 20-in. casing installed extending from the bottom of the previously installed 24-in. casing down to the top of the medium sand stratum. The remainder of the hole was then advanced to the required depth, an additional 40 ft, and the well screen and filter installed without casing. The filter material was placed around the well screen by washing it down through a 2-in. tremie pipe.

70. After placing the filter up to an elevation approximately 1 ft above the well screen, the well was cleaned of any filter material which entered it during placement of the filter. The filter material removed usually amounted to about 10 gal or 2-1/2 per cent of the total volume of filter placed around the screen. Photographs of the installation operations are shown on plate 31.

71. The procedure used for developing the wells was as follows:

a. The well was surged for approximately 15 minutes with a plunger device in the riser pipe. This device consisted of two sets of rubber washers mounted on a steel pipe and spaced approximately 3 ft apart. The pipe between the washers, which closely fitted the well screen and riser pipe, was perforated with holes which permitted pumping the well as it was being surged. This was accomplished by means of an air line inside the pipe on which the washers were mounted. While surging was in progress, the well was pumped with air to keep a stream of water flowing out of it, thus flushing out any dirt or turbid water entering the well during the surging operation.

b. The surging device was then lowered into the upper part of the screen and the upper 5 or 6 ft of the screen surged. After this the surging device was brought to rest in the upper 3 ft of the screen and the well pumped until the discharge became free and clear of any sand or gravel. Then the next several feet of well screen was similarly pumped and surged until the entire screen had been subjected to the surging-pumping operation. Upon completion of the surging-pumping operation, any sand or gravel pulled into the well by the surging process was removed.

72. After development of the well, a pumping test was run to determine the flow for various drawdowns in the well, head loss through the filter and well screen, and the permeability of the pervious sand stratum. Piezometers to measure the head loss through the filter and well screen were installed at the outer periphery of the filter at the top and midpoint of the well screen at wells 1, 8, 15, 22, and 30. Piezometers were also installed on fairly close intervals out from these wells in order to measure the drawdown during the pumping tests (see plate 2 and Section T-T on plate 6). Only the drawdown in the well itself was measured in the pumping tests on the other wells.

73. The pumping tests extended over a period of 6 hours. Each well was pumped for 2 hr at drawdowns of about 2, 4, and 6 ft. The discharge from the well was measured with a calibrated orifice in the end of the discharge line from the pump. The well flows for these drawdowns

usually ranged from 200 to 500 gpm. No sand was observed in the effluent from any of the wells during the pumping tests.

Construction of Collector and Outfall Ditches, Control Culvert, and Sublevee

Collector and outfall ditches

74. The collector ditch for the well system was constructed in accordance with the drawings on plates 14 and 16. Photographs of the ditch are shown on plate 32. The porous concrete for paving this ditch was mixed in proportion of 1 part of sulfate-resisting portland cement to 5 parts of aggregate. The amount of water added was such that the resulting cement paste did not fill the voids in the aggregate but thoroughly coated and bound the aggregate particles together. A mechanical analysis of the aggregate used for the porous concrete is shown in fig. (E) on plate 13. Laboratory tests made on the porous concrete used showed that it would pass water with very little loss of head. The ditch lining was made in sections 20 ft long with bituminous expansion joints between each section. The outfall ditch running landward from the well system was deepened slightly for a distance of about 300 ft from the previously mentioned road culverts and cleaned for about 800 ft.

Control culvert

75. The control culvert through the sublevee at the end of the well system was constructed as shown on plate 14. Photographs of the culvert and gate are shown on plate 32.

Sublevee

76. A sublevee as shown on plates 2 and 14 and in the photograph

on plate 32 was constructed around the well system. This sublevee has a crown width of 8 ft and side slopes of 1 on 2.75. The sublevee was constructed of relatively impervious waste materials obtained from required excavation and from a riverside borrow pit across the levee. The fill material was placed in uniform layers not exceeding 12 in. and compacted by three coverages of a crawler-type tractor. After completion, the sublevee was planted with Bermuda grass.

Pump platform

77. A pump platform consisting of a concrete slab 15 ft by 45 ft and 4 in. thick was constructed on the adjacent seepage berm at the center of the well system (see plates 2 and 14). This platform will serve as a base for any pumps which may be needed to pump the water from the well system over the levee.

PART V: ANALYSIS OF WELLS AND PIEZOMETER DATA

Analysis of Pumping Tests on 1950 Wells

78. The results of the pumping tests made on the 1950 relief wells are plotted on plates 17 and 18. Drawdown curves which were obtained from the pumping tests on wells 1, 15, and 30 are shown on plate 19. Fig. (A), plate 19, shows the water elevation in the wells for three different rates of pumping, the radius of the well screen, the radius of the filter, elevation of the water in the piezometers at the outer periphery of the filter at the top and middle of the well screen, the water table before initiation of pumping, and the elevation of water in the piezometers out from the test wells. The drawdown curves shown on plate 19 indicate that the radius of influence for the wells tested ranged from about 400 to 500 ft.

Head loss through filter and screen

79. The difference in the readings of the piezometers at the top and middle of the screen for wells 1, 15, and 30 showed that a significant amount of head loss was occurring in the screen and riser portion of the well up to the bottom of the suction pipe from the pump. Therefore, the true specific yield for the wells was obtained by correcting the drawdown in the wells for hydraulic head losses* using the data shown

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* The head loss through the filter and well screen used in correcting the drawdown in the wells was obtained from the well tank test results (Appendix A) as follows: The flow measurements in the wells made during the 1951 high water, at different depths in the screen, show that the inflow at the midpoint of the screen is approximately 75 per cent of the average screen inflow for the entire length of screen (see fig. (A), plate 20). The resulting inflow per ft at the midpoint of the screen was computed for various assumed well flows and the resulting head loss obtained from fig. (C), plate A-1. These head losses were then plotted against the assumed well flows in figs. (B) and (C), plate 20.

in figs. (A) and (B), plate 20, and the corrected data plotted on fig. (B), plates 17 and 18. The head loss through the filter gravel and screen at the center of the screen corresponded closely to that estimated from the well tank tests (Appendix A). The filter and screen loss as determined from the laboratory tests was 0.12 ft for a well flow of 5 gpm per ft of screen; the head loss through the filter and screen as measured by piezometers in the field for a rate of flow through the screen of 5 gpm per ft ranged from about 0.05 to 0.15 ft. Thus the filter and screen appear to have functioned as anticipated from the standpoint of allowing the entrance of water with a minimum of frictional resistance.

Specific yield of wells

80. The specific yield of each well was taken as the flow occurring for 1 ft of drawdown as read from the corrected well discharge versus drawdown curves, fig. (B), plates 17 and 18. The specific yields are plotted for each well in fig. (B) on plate 21. The average specific yield for all 30 wells was 100 gpm. The average specific yield for wells 6 through 25, the wells used in analyzing the performance of the well system, was 105 gpm. The yield of wells 1 through 24 was extremely uniform. For some unknown reason the specific yield of wells 25 through 30 was approximately 25 per cent less than the remainder of the wells in the system. The flow from these latter wells under normal operating conditions was also considerably less than the flow from the other wells (see fig. (C), plate 21).

Permeability of foundation

81. The permeability of the pervious substratum as determined from the previously described pumping tests is summarized in table 2. The

permeability of the sand stratum was computed from the drawdown curves obtained at wells 1, 15, and 30 (fig. (A), plate 19) in two different ways. One method (B) was based on a secant drawn through the upper flat portion of the drawdown curve assuming the flow lines in the foundation to be parallel and horizontal beyond 50 ft from the well. In this method the permeability was computed from the formula

$$k = \frac{2.30\ Q \log_{10} \frac{r_2}{r_1}}{2\pi d\ (h_2 - h_1)} \quad .$$

The permeability at wells 1, 15, and 30 was also computed from a secant drawn through the steeper portion of the drawdown curve adjacent to the well. The permeability by this method (A) was computed from the same formula given above except that a correction factor of 0.63* was put in the denominator to correct for the partial penetration of the well screen into the pervious substratum. The data shown in table 2 reveal fair agreement between the above-described methods for computing the permeability.

82. The permeability of the foundation was also computed from the drawdown in each of the wells pumped. This permeability was computed using the specific yields shown on plate 21 and from the general equation for artesian flow, given above, with Muskat's correction factor of 0.63. The results of these computations are also summarized in table 2.

83. The average permeability of the foundation computed from the

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* M. Muskat, "The Flow of Homogeneous Fluids Through Porous Media," New York: McGraw - Hill Book Co., Inc. 1937.

drawdown curves for wells 1, 15, and 30 was 890×10^{-4} cm/sec; the average permeability based on the specific yield of the 30 individual wells was 1180×10^{-4} cm/sec, the average permeability of the central portion of the well system based on wells 6 through 25, was 1240×10^{-4} cm/sec.

84. The permeabilities selected from the field pumping tests for subsequent averaging to obtain the best estimate of the foundation permeability were the permeabilities computed from the drawdown curve for well 15 using method B and the permeability computed from the specific yields of wells 6-25.

85. Well flow data obtained during 1951 high water. The permeability of the pervious stratum at Trotters was also computed from the well flow and piezometric data obtained during the 1951 high water. From these data the permeability of the foundation required to give the flows measured was computed to be 1500×10^{-4} cm/sec.

86. Summary. The permeability of the pervious foundation along the central portion of the well system at Trotters was taken to be the average of the permeabilities as computed from the following data:

a. 1943 well data and 1950 piezometer and seepage measurements, $k = 1250 \times 10^{-4}$ cm/sec.

b. Drawdown curves from pumping tests on well 15, $k = 1070 \times 10^{-4}$ cm/sec.

c. Specific yields obtained from pumping tests on wells 6-25, $k = 1240 \times 10^{-4}$ cm/sec.

d. 1951 well flow and piezometer data, $k = 1500 \times 10^{-4}$ cm/sec.

The average of the above values indicates that the pervious substratum at Trotters has a permeability of approximately 1250×10^{-4} cm/sec.

Analysis of 1951 Well Flow and Piezometer Data

87. The first opportunity to observe the operation of the 1950 well system occurred during the high-water period from 20 February to 30 April 1951; on 6 March the river crested at a stage of 185.8 ft mGl with a net head on the well system of about 7.5 ft. During this period, the river stage and resulting piezometer readings and well flows were observed at frequent intervals for different well operating conditions.

Well flow

88. All wells in the system were open from the beginning of overbank river stages until 2 March at which time the even numbered wells were closed thereby creating a well system with a well spacing of 100 ft. The well flows for this condition were determined on 5 March. All wells were opened after the 5 March observation and were left open until 8 March on which date they were closed. After reading the piezometers on 10 March, all the wells were opened and left open during the remainder of the high water.

89. The individual well flows were measured by a special well flow meter, constructed at the Waterways Experiment Station, which could be lowered into the wells. This device measured the velocity of the flow which in turn was converted into well flow. The flow in the collector ditch was measured at the downstream end of the ditch by cross-sectioning the stream and measuring the velocity of flow by means of a midget Gurley flow meter. (When the flow from the well system is pumped over the levee, it will be measured by means of 5- or 7-in. sharp-edged circular orifices in the discharge lines from the pumps. These orifices have the following

flow formulas $Q_{5\ in.} = 310\sqrt{h}$ gpm and $Q_{7\ in.} = 674\sqrt{h}$ gpm.)

90. The individual well flows were determined several times during the 1951 high-water period. The flows measured on 4 different days are shown by the bar graph in fig. (C) on plate 21. For comparative purposes, the specific yields for the 1950 wells and the well flows observed from the 1943 well system during the 1943 high-water period are also shown on plate 21.

91. The average flows from wells 6-25 per ft of net head on the system are also tabulated at the bottom of plate 21 for both well spacings (a) of 50 and 100 ft. The flows for $a = 50$ ft vary between 7.9 and 10.5 gpm per well and compare fairly well with the flow predicted in paragraph 55 for case A on the basis of $k = 1250 \times 10^{-4}$ cm/sec for the sand foundation. On the basis of the flows observed during the 1951 high water it is believed that the flow from the wells in the central portion of the system will average about 10 gpm per ft of net head on the system. The total flow from the system is estimated at about 330 gpm per ft of net head on the system or about 9,150 gpm (20 cfs) for the project flood. The average individual well flows from the 1943 well system on 75-ft centers and from the 1950 well system on 50- and 100-ft centers are plotted, for various heads on the system, in fig. (A), plate 22. The total flow from the 1950 well system is plotted in fig. (B), plate 22.

92. The bar diagram on plate 21 shows that, in general, the wells in the central portion of the system discharged less than the end wells. This general trend is attributed to end effects which were previously discussed under "Design of Well System and Appurtenances." The increased flow caused by end effects as determined from a flow net and as

measured along the well system are compared in fig. (E) on plate 22. End effects actually increased the flow from the system by only about 10 per cent rather than the 40 per cent estimated from the flow net study. Part of the discrepancy between the observed and the computed increase can be attributed to the lower capacity wells (25-30) on the downstream end of the system and well 1 on the upstream end of the system indicated by the lower specific yields of these wells as compared to the yield of the remainder of the wells in the system.

93. All of the well flow measurements made during the 1951 high water are presented in table 3 together with the river stage, elevation of the water in the collector ditch, elevation of the water in the drainage ditch immediately landward of the sublevee around the well system, and seepage conditions observed on the days of observation.

94. Flow measurements at different depths in the well screen were made in wells 1, 5, 10, 15, 20, 25, and 30 to determine the variation in screen inflow with depth below the top of the well screen (see table 3). The measured flows in the screen at various depths in terms of per cent of total discharge are shown in fig. (D) on plate 20. The variation in "flow in screen" and "flow through screen" with depth is shown in fig. (E), on plate 20. This figure shows that the flow through the top, middle, and bottom of the screen is approximately 60, 75, and 300 per cent, respectively, of the average flow through the screen, and that almost 50 per cent of the total well flow enters through the bottom 10 ft of screen. With the flow distribution observed, it was found that the frictional head loss up through the screen portion of the well could be approximated by computing the frictional loss resulting

from 90 per cent of the well flow through one half the length of the screen. The computed head losses in the wells are shown for both the "pumping" and "natural flow" conditions in figs. (B) and (C) on plate 20.

95. Although provision was made to pump the flow from the well system over the levee, such pumping was never found necessary because the outfall ditch at the end of the system and landward from the levee had ample capacity to contain all of the flow from the well system without overflowing its banks at any point. The control culvert through the sublevee at the lower end of the well system was left wide open throughout the period of testing.

Substratum pressure

96. The piezometric pressures obtained during the 1951 high water are plotted on plates 23-26. The effect of well operation on substratum pressures along the toe of the seepage berm is illustrated by the data plotted on plates 26 and 30.

97. The piezometer data plotted on plate 26 show that very little substratum pressure developed along the line of wells when the system was operating on 50-ft centers. At no point landward of the well system was there any excess head above the ground surface (see plates 27-29). However, on 8 March when all the wells were closed, the substratum pressures rose rapidly as can be seen from the readings of piezometers R-7, T-10, and T-11; the substratum pressures continued to rise until 10 March on which date they had almost stabilized and the wells were again opened after the piezometers had been read. On 10 March, when all the wells were closed, the river stage was 185.5, the water elevation in the collector ditch was 177.4, and the average piezometric head

in the pervious substratum along the well system was about 181.8. On this date the net hydrostatic pressure in the pervious substratum was about 3.7 ft, or 50 per cent of the net head on the levee, above the elevation of the water in the drainage ditch immediately landward of the well system. With the wells open on 50-ft centers and approximately the same head on the levee, the maximum head midway between the wells in the center of the system was only 178.8 ft msl or 0.6 ft above the elevation of the water in the collector ditch or 8.2 per cent of the net head on the well system. In other words, opening the wells reduced the hydrostatic pressure at the landside toe of the seepage berm by approximately 85 per cent of the pressure without any wells in operation.

98. The river stage, the hydrostatic pressure along the line of wells when closed, the average ground surface, the head between wells for well spacings of 50 and 100 ft, and the elevation of the water in the collector ditch and drainage ditch back of the sublevee are all shown for certain selected days during the test period on plate 30. The curves and data shown on this plate are believed to be self-explanatory. However, it is pointed out that not only was the substratum pressure along the line of wells very materially reduced by the well system but also the hydrostatic head beyond the ends of the well system was significantly reduced for a distance of 200 ft.

99. A plot of the maximum head between wells along the well system is shown in fig. (D) on plate 13 for well spacings of both 50 and 100 ft. This figure indicates that the curves of best fit for the a = 50 ft data, corrected for head losses in the wells, give a value

of 7.2 per cent (H) for the head between the wells at the center of the system. This value agrees closely with the original design value of 5.6 and the new computed design value of 6.2. With the well system operated on 100-ft centers the curve of best fit for the data, corrected for well losses, gives a value of 11.5 per cent for the head between wells at the center of the system as compared to the original design value of 9.7 per cent. A comparison of computed (original) values of head between the wells and those observed, corrected for head losses in the wells, is shown in fig. (C), plate 13.

100. The elevation of the water in the collector ditch was that which naturally built up as a result of the flow through the control culvert, and was about 178.2 between 2 and 12 March except that on 10 March, when all the wells were closed, the water in the ditch dropped to elevation 177.4 (see plate 30). Thus, the water in the collector ditch was usually 2 to 3 ft below the adjoining natural ground surface. As the substratum pressure at and landward of the well line was below the natural ground surface, except when all the wells were closed, it follows that the well condition tested corresponds to case A (see "Design of Well System and Appurtenances") in which it was assumed that the top stratum was impervious.

101. Piezometric gradients in the foundation along lines M, R, Q, and T for the conditions of all wells open and all wells closed near the crest of the flood are shown on plates 27 to 29. Excess pressures above the ground surface were observed as far as 2200 ft from the well system when the wells were closed. The flatness of the hydraulic gradient landward of the well system when it is in operation shows that very little

seepage is passing landward beyond the line of wells. This is borne out by the field observations described in the following section and given in table 3.

102. The distance from the line of wells to the effective seepage entrance was determined along piezometer lines Q, M, and R from the piezometer data for rising river stages during the 1951 high water. The average distances determined from these lines for various river stages are shown in fig. (C) on plate 22. The distances ranged from 925 to 1285 ft and decreased as in previous high waters with increasing river stages. Although a linear variation is shown in the figure, it is believed that for the project flood the average distance to the effective seepage entrance will not be less than about 900 ft. The distance was approximately the same along each piezometer line, being 990, 1040, and 890 ft for lines Q, M, and R on 8 March when the stage of the river was 185.7 ft.

103. The conclusion that most of the seepage passing beneath the levee originates somewhere between the main-line levee and the old abandoned levee approximately 2000 ft riverward is strengthened by the fact that piezometers down through the old levee on lines M and R show substratum pressures almost exactly equal to the stage in the main channel of the river. If the river bank were the principal source of seepage, the hydraulic gradient on lines M and R from the river to the old levee would have essentially the same slope as the gradient beneath the main levee, and the piezometers down through the old levee would have to read about 4 ft below the stage in the river.

104. The effective source of seepage was determined from the

intersection of the hydraulic grade line beneath the levee extended to the water surface riverward of the levee. However, the true hydraulic gradient riverward of the levee probably has the configuration shown by the dotted lines on plates 27 and 28. The curved shape of the gradient is due to the combined effect of seepage through the borrow pit and the top stratum riverward of the main levee.

105. The short time lag in substratum pressures with changes in river stage and operation of the well system, as shown by plates 8, 9, and 23 to 26, is indicative of very rapid transmission of substratum pressure which in turn is indicative of a very pervious foundation and a relatively close effective seepage entrance.

Seepage control

106. An inspection on 1 March 1951 of the landside drainage ditch for a distance 1000 ft upstream of well 1 disclosed several pin boils in the bottom of the ditch. Downstream of well 30 the same condition existed for a distance of about 1000 ft. No boils were found in the ditch immediately landward of the well installation. Seep water was standing in the field 600 to 900 ft landward of sta 54/15 (about 500 ft downstream of well 30). The outfall ditch carrying the well flow had about 2 ft of freeboard for a distance of approximately 1000 ft landward of the levee and 3 to 4 ft for the next mile. At no point was the ditch overflowing. On 1 March the river stage was 184.8, or approximately 5 ft above the average ground elevation landward of the levee.

107. On 2 March, when there was a net head of about 5.2 ft on the levee, the seepage flow in the drainage ditch immediately landward

of the sublevee was measured opposite wells 6 and 25. The seepage amounted to about 1 gpm per 100 ft of levee. On the basis of the seepage measurements made along the same section of levee during the 1950 high water, without any relief wells in operation, the seepage in the same drainage ditch would have amounted to approximately 20 gpm per 100 ft of levee. Seepage conditions in the areas adjacent to the well system were similar to those on 1 March.

108. On 6 March, when the river crested at a stage of 185.8, seepage conditions landward and adjacent to the well system were also the same as those on 1 March. On this date the water against the levee was approximately 5.8 ft above the average ground surface landward of the well system.

109. On 8 March all of the wells in the system were closed. Two days later, on 10 March, when the river stage was 185.5, the following seepage observations were made. (On this date the river stage was 8.05 ft above the pool elevation in the collector ditch and 7.4 ft above the elevation of the water in the drainage ditch landward of the sublevee.)

- a. With the wells closed, the flow from the collector ditch was only 145 gpm, or 1 gpm per 100 ft of levee. Seepage entering the drainage ditch immediately landward of and parallel to the sublevee between wells 6 and 25 amounted to 4.4 gpm per 100 ft of levee. In other words, closing the well system increased the amount of seepage entering the drainage ditch about 4 to 5 times over that observed when the well system was in operation.

- b. Seep water began appearing in the field 400 to 600 ft landward of the well installations 24 hours after closure of the wells. After the wells had been closed for two days, seep water was observed landward of the well system for a distance of 1100 ft. Numerous pin boils were active in the outfall ditch which runs perpendicular to the levee at the downstream end of the well system. The boils extended for a distance of 1200 ft landward of the levee.

> Numerous pin boils also were active in the drainage ditch parallel to and landward of the well system. Seepage was entering the collector ditch through the slopes and bottom. Seepage was also observed emerging on the landside slope of the sublevee around the well system. It is pointed out that there was no water impounded in the sublevee basin when this seepage was observed.

110. After observing the substratum pressures and seepage conditions on 10 March with the wells closed, the wells were opened and left open for two days. The following observations were made on 12 March. (On this day the river stage was 185.2; the net head on the well system was 6.9 ft; the total flow from the collector ditch was 2370 gpm; and the elevation of the water in the drainage ditch landward of the wells was 178.0.) The seep water in the field immediately landward of the installation had disappeared, and the field was dry for a distance of 700 ft landward of the installation but was still moist from 700 and 1100 ft landward. No water was ponded anywhere in the field back of the well system. The sand boils in the drainage ditch landward and parallel to the system and in the outfall ditch perpendicular to the downstream end of the system had disappeared. Seepage through the bottom and slopes of the collector ditch was no longer evident. Seepage on the landside slope of the sublevee had also stopped.

111. The piezometer readings (plate 10) and observations made during the 1951 high water indicate that the new well system at Trotters not only will reduce substratum pressures landward of the levee to a small fraction (7 to 10%) of the head on the levee but will also intercept a large proportion of natural seepage which otherwise would rise to the surface landward of the levee.

112. Very little material washed through the filter and screen while the wells were in operation during the 1951 high water. Approximately 1 ft of sediment was observed in the bottom of the wells on 10 April after the wells had been in operation for almost two months (see table 3).

113. Photographs of the well system and adjacent seepage conditions, taken on 20 April 1951 when conditions were similar to those described for 1 March, are shown on plate 33.

PART VI: SUMMARY AND CONCLUSIONS

Geology and Underseepage

114. The levee paralleling the well system crosses an old abandoned channel which has been buried by a widespread clay blanket approximately 9 ft thick. Immediately beneath the top stratum of clay and silt lies a stratum of extremely uniform fine and very fine sand about 20 to 40 ft thick. This fine sand is underlain by a stratum of uniform medium to coarse sand with some gravel, approximately 88 ft thick. This latter stratum is the principal seepage carrier at the site. The effective thickness of the pervious strata is about 90 ft.

115. The bank of the main channel of the Mississippi River is approximately 3000 ft from the center line of the levee. However, the entire reach of levee is bordered by riverside borrow pits which penetrate the pervious substratum in various areas and facilitate the entrance of seepage into the pervious foundation.

116. Very heavy underseepage was observed at the well site during the 1937 high water, extending from the levee toe to a distance approximately 1,000 ft landward. During this high water many sand boils occurred along this reach of the levee. Numerous sand boils and heavy underseepage were also observed at the site during the 1950 high water. Considerable seepage was noted emerging from the ground along the toe of the seepage berm through crayfish holes and other fissures. Natural seepage between the berm toe and a drainage ditch 100 ft landward amounted to 55 gpm per 100 ft of levee for a net head of 13.5 ft or 4.0 gpm per 100 ft of levee per ft of net head on the levee.

117. Although the top stratum at the site of the well system is of predominantly clay materials, it is not as impervious as the type of soil would indicate. The top stratum was estimated to have a permeability of about 1 to 5 x 10^{-4} cm/sec on the basis of seepage and piezometer measurements made during the 1950 high water. The permeability of the pervious foundation was estimated from field pumping tests and observations on both the 1943 and 1950 well systems to be about 1250 x 10^{-4} cm/sec.

118. The effective source of seepage estimated from piezometer readings ranges from about 900 to 1300 ft riverside of the line of relief wells. The distance to the effective source of seepage apparently decreases with increasing river stages, and will probably be about 900 ft for a project flood. This is attributed to the scouring away of silt, deposited in the bottom of the riverside borrow pits during previous high waters, by the increasing current velocities as the river rises.

Substratum Pressures without Well System

119. The piezometers along the landside toe of the levee at Trotters indicated artesian pressures, with no wells in operation, of approximately 35 to 50 per cent of the net head on the levee for a head of about 10 ft. Data obtained during the 1950 high water indicate that the maximum hydrostatic pressure that can exist at the toe of the seepage berm is about 3.0 ft above the average ground surface or 4.9 ft above the tailwater in the drainage ditch along the toe of the berm. This head agrees closely with the computed maximum head which can develop beneath the top stratum along the drainage ditch. A river stage of approximately

190, or 10 ft of water on the levee, creates the maximum pressure possible landward of the levee toe. River stages above this will not increase the substratum pressure at the toe but will only cause increased seepage and more sand boils.

120. The short time lag in substratum pressures with changes in river stage and operation of the well system is indicative of very rapid transmission of substratum pressures which in turn is indicative of a very pervious foundation and a relatively close effective seepage entrance. With no well system in operation, the piezometer data obtained during the 1950 and 1951 high waters show that most of the seepage passing beneath the levee rises to the surface in a strip about 300 ft wide along the toe of the existing seepage berm.

1950 Well System

121. Analysis of the data from the well and piezometer systems at Trotters shows the following:

- a. The new well system at Trotters functioned entirely satisfactorily during the 1951 high water. It significantly reduced the hydrostatic pressure which otherwise would have existed at the landside toe of the seepage berm and practically eliminated any evidence of seepage or sand boils landward of the levee for the river stages experienced. Of course, some seepage and minor sand boils must be expected for a project flood, but if the system is maintained in good operating condition the severity of underseepage with a project flood should be no greater than what would occur with a river stage of 190 and no well system. The lack of any backwater in the outfall ditch during the period of well operation facilitated drainage of the flow from the well system and helped maintain a low tailwater over the well outlets.

- b. Very little substratum pressure developed along and landward of the line of wells when they were operated on 50-ft centers. With all the wells operating the maximum head midway between the wells was about 8 per cent of the net head on the well system. This agrees closely with the computed value. Opening the wells reduced the hydrostatic pressure at the toe of the seepage berm by 85 per cent or to 15 per cent of the pressure without any wells in operation. With the wells on 100-ft centers, the head between the wells was about 14 per cent of the net head on the system.

c. The average flow from the wells in the central portion of the system with a spacing of 50 ft was about 10 gpm per well per ft of net head on the system. This flow compares fairly well with the flow predicted on the basis of $k = 1250 \times 10^{-4}$ cm/sec for the sand foundation. The flow per well when the system was on 100-ft centers was approximately 16 gpm per well per ft of net head on the system or 60 per cent more than the flow per well for a well spacing of 50 ft.

d. The total flow from the system with the wells on 50-ft centers was about 350 gpm per ft of net head on the system.

e. Wells at the end of the system flowed at a somewhat greater rate than those in the central portion of the system, as the result of seepage passing around the ends of the system and entering the wells from the landside. Flow through the top, middle and bottom of a well screen is approximately 60, 75, and 300 per cent, respectively, of the average flow through the screen, and almost 50 per cent of the total well flow enters through the bottom 10 ft of the screen.

f. Although provision was made to pump the well flow over the levee, such pumping was not necessary because the outfall ditch had ample capacity to handle all the flow from the system without overflowing its banks at any point.

g. The head loss through the filter gravel and screen at the center of the screen, as observed in the field, corresponded closely to that estimated from the well tank tests made in the laboratory. Relatively little material washed through the filter and well screen during either the pumping tests or during the 1951 high water. Therefore, it is concluded that the well screen and filter are satisfactory, and that the hydraulic head losses through the filter and well screen and up through the well should not exceed 1.0 ft for well flows that would accompany a project flood.

h. The collector ditch and control culverts are believed adequate to handle the flow from the well system for the project flood. The outfall ditch will also handle all the flow from the well system for river stages considerably in excess of those occurring during the 1951 high water, as long as seepage from other areas and rainfall have not raised surface water conditions to such an elevation that backwater in the outfall ditch away from the well system causes the ditch to overflow.

i. From the observations made and the data presented in this report, it is concluded that an adequately designed and properly installed and maintained relief well system will control underseepage and reduce detrimental artesian pressures landward of levees along the lower Mississippi River where these phenomena are a serious problem. However, it is pointed out that a satisfactory well system must be designed to fit the particular foundation conditions existing at the site and must be of sufficient capacity to effect the pressure reduction and seepage interception desired.

122. Observed and estimated performance data pertaining to the Trotters well system are summarized below. All values given regarding well flow, landward pressure, and seepage are based on an infinite line of wells, unless stated otherwise.

a. Estimated well flow = 10 gpm per ft of head above water elevation in collector ditch when the head between the wells is at or below the natural ground surface (case A). Estimated well flow = 8.7 gpm per ft of river head above elev 180.0 when the head between the wells is above the natural ground surface (case B).

b. Maximum net head between wells = 7 to 10 per cent of net head on well system.

c. Estimated natural seepage per 100 ft of levee without well system = 300 to 400 gpm for project flood.

d. Estimated natural seepage landward of wells with well system in operation = 50 gpm per ft of levee for project flood if the tailwater over the wells is allowed to reach elev 180. Increase in total flow caused by wells = approximately 30 per cent of the project flood stage.

e. End effects are estimated to increase the total flow from the well system by 10 per cent as compared to the flow from the same number of wells in an infinite well line. Therefore, the total flow from the system is estimated at about 330 gpm per ft of net head on the system for case A and about 300 gpm per ft of net head for case B.

f. The control culvert and overflow spillway will pass the total flow from the well system including runoff from the levee into the sublevee basin without the water in the sublevee rising above the net grade of the sublevee.

TABLES

Table 1

PIEZOMETER AND WELL INSTALLATION DATA

Piez. No.	Levee Station	Distance from Center Line of Levee, Ft	Ground Elev at Piez.	Elev Top of Riser	Elev Center of Piez. Screen	Remarks
M-1-X	54/1+05	2086 RS	202.6	204.1	162.5	
M-2-X	54/1+05	120 LS	193.5	195.9	170.0	
M-3-X	54/1+05	388 LS	180.4	183.0	160.5	
M-4-W	54/1+05	393 LS		183.0	169.0	In collector ditch
M-5-W	54/1+00	1050 LS	178.5	181.0	163.0	
M-6-X	54/1+05	1050 LS	178.5	181.0	158.0	
M-7-X	54/1+05	3215 LS	180.6	183.0	164.0	
M-8	54/0+99	2086 RS	202.3	205.35	125.1	
M-9	54/0+99	500 RS	171.1	177.2*	165.1*	
M-10	54/1+05	500 RS	171.1	177.2*	125.0*	
M-11	54/0+99	50 RS	199.3	202.28	164.5	
M-12	53/0+99	393 LS		180.08	167.5	In collector ditch
M-13	54/1+05	393 LS		179.98	124.8	In collector ditch
M-14	54/0+99	730 LS	180.4	183.09	166.0	
M-15	54/1+02	1050 LS	178.8	181.57	126.0	
M-16	54/1+05	1948 LS	180.0	183.21	150.0	
1	54/0+99	170 RS	179.0*	191.5	136.5	
2	54/0+99	205 LS	191.1*	193.6	127.6	
3	54/0+99	280 LS	189.1*	192.3	126.3	
4	54/0+99	355 LS	184.1*	188.9	122.9	
5	54/0+99	380 LS		183.0	117.0	In collector ditch
6	54/0+99	505 LS	180.2*	182.8	137.8	
7	54/0+99	580 LS	180.2*	183.0	138.0	
8	54/0+99	655 LS	180.2*	182.7	137.7	
P-1	53/47+55	50 RS	199.1	202.22	168.0	
P-2	53/47+55	393 LS	182.5	185.60	157.1	
Q-1	53/54+30	50 RS	200.3	203.30	168.0	
Q-2	53/55+80	393 LS		180.24	167.8	In collector ditch
Q-3	53/55+86	393 LS		180.30	125.3	In collector ditch
Q-4	53/55+80	465 LS	181.1	183.95	168.0	
Q-4-Z	53/56+80	465 LS	180.7	183.41	168.0	
R-1	54/3+97	2090 RS	202.6	205.55	162.0	
R-2	54/4+03	2090 RS	202.6	205.55	125.0	
R-3	54/3+97	500 RS	171.5	178.4*	165.0*	
R-4	54/4+03	500 RS	171.5	177.2*	125.0*	
R-5	54/4+00	50 RS	199.2	202.60	168.0	
R-6	54/5+00	393 LS		179.80	167.7	In collector ditch

* Approximate elevation

Table 1 (continued)

Piez. No.	Levee Station	Distance from Center Line of Levee, Ft	Ground Elev at Piez.	Elev Top of Riser	Elev Center of Piez. Screen	Remarks
R-7	54/4+00	393 LS		179.74	128.9	In collector ditch
R-8	54/4+00	465 LS	179.6	182.54	167.9	
S-1	54/13+25	50 RS	199.3	202.07	167.8	
S-2	54/13+25	393 LS	180.3	183.41	168.0	
T-1	53/50+77	393 LS	181.3	184.29	166.0	
T-2	53/51+79	393 LS		179.94	166.9	In collector ditch
T-3	53/52+82	393 LS		179.94	167.9	In collector ditch
T-4	53/55+95	393 LS		*	168.0	In collector ditch
T-5	53/56+00	387 LS		177.89	168.3	In collector ditch
T-6	54/0+89	387 LS		177.65	167.8	In collector ditch
T-7	54/1+09	393 LS		180.05	167.6	In collector ditch
T-8	54/3+85	387 LS		177.39	167.9	In collector ditch
T-9	54/3+92	393 LS		179.65	167.9	In collector ditch
T-10	54/7+10	393 LS		179.94	166.2	In collector ditch
T-11	54/8+00	393 LS		179.95	168.0	In collector ditch
T-12	54/9+00	393 LS	180.7	179.37	167.8	
V-1	53/51+54	387 LS		178.00	122.0	In collector ditch
V-2	53/51+54	387 LS		177.95	102.0	In collector ditch
V-3	53/51+97	387 LS		177.79	121.5	In collector ditch
V-4	53/51+80	387 LS		177.69	121.5	In collector ditch
V-5	53/55+04	387 LS		177.93	122.0	In collector ditch
V-6	53/54+97	387 LS		177.72	122.3	In collector ditch
V-7	53/54+80	387 LS		177.72	122.1	In collector ditch
V-8	54/0+74	387 LS		177.74	122.0	In collector ditch
V-9	54/0+74	387 LS		177.76	102.0	In collector ditch
V-10	54/0+67	387 LS		177.67	121.5	In collector ditch
V-11	54/4+24	387 LS		177.52	130.0	In collector ditch
V-12	54/4+17	387 LS		177.42	129.8	In collector ditch
V-13	54/8+25	387 LS		181.25	132.0	In collector ditch
V-14	54/8+25	387 LS		181.23	115.0	In collector ditch
V-15	54/8+33	387 LS		177.93	131.8	In collector ditch
V-16	54/8+45	372 LS	181.4	184.52	132.3	
V-17	54/8+83	372 LS	181.4	184.62	131.9	
V-18	54/9+23	372 LS	180.9	185.28	131.4	
V-19	54/10+30	393 LS	180.7	184.03	132.2	
Well 1	53/51+55	387 LS				
Well 30	54/8+25	387 LS				

Wells on 50 ft centers 387 ft LS

* Missing, piezometer covered by ditch paving.

Table 2

PERMEABILITY OF MEDIUM TO COARSE SAND STRATUM

(a) Permeability from Pumping Tests on Wells 1, 15, and 30

Well No.	Well Discharge in gpm	$k \times 10^{-4}$ cm/sec Method A	$k \times 10^{-4}$ cm/sec Method B
1	220	1067	1005
	326	806	766
	427	840	902
	Average	905	890
15	202	730	----
	351	886	1121
	457	911	1024
	Average	840	1070*
30	196	878	770
	320	796	815
	422	711	938
	Average	795	840
Total Average		845	935
Average of Both Methods		890	

Notes:

(1) Method A consists of using straight line portion of drawdown curve adjacent to well with factor for partial penetration = 0.63.

(2) Method B consists of using a chord of drawdown curve on straight line portion of drawdown curve at some distance from the well.

(3) Permeabilities were computed from the following formula:

$$k = \frac{2.30\, Q \log_{10} r_2/r_1}{2\pi d\, (h_2 - h_1)\, 0.63}$$

wherein:

- k = coefficient of permeability
- Q = well discharge
- r_1 = distance from center of well to point 1
- r_2 = distance from center of well to point 2
- d = thickness of aquifer
- $h_2 - h_1$ = difference in hydrostatic pressure at points 2 and 1
- 0.63 = artesian well flow correction factor for 50 per cent penetration of aquifer.

(b) Permeability from Pumping Tests on Individual Wells

Well No.	Specific Yield gpm/ft	$k \times 10^{-4}$ cm/sec
1	90	1058
2	112	1316
3	106	1245
4	98	1152
5	110	1293
6	98	1152
7	112	1316
8	113	1328
9	100	1175
10	123	1445
11	116	1363
12	88	1034
13	117	1375
14	110	1293
15	113	1328
16	108	1269
17	98	1152
18	112	1316
19	102	1199
20	108	1269
21	98	1152
22	100	1175
23	104	1222
24	109	1281
25	77	905
26	84	987
27	74	870
28	77	905
29	80	940
30	78	917
	Average	1180
Average, Wells 6-25		1240*

(c) Average Permeability from Laboratory Tests and Mechanical Analyses = 400×10^{-4} cm/sec.

(d) Permeability Computed from 1943 Well Data and 1950 Piezometer and Seepage Measurements = 1250×10^{-4} cm/sec.*

(e) Permeability Computed from Measured Well Flows during 1951 High Water = 1500×10^{-4} cm/sec.*

* Average permeability from starred values about 1250×10^{-4} cm/sec.

Table 3

WELL FLOW MEASUREMENTS, TEST DATA, AND OBSERVATIONS, 1951

Date (1951)	Well No.	Well Distance from Bottom of Screen, Ft	Measured Well Discharge, gpm Total	Measured Well Discharge, gpm Above Bottom of Screen	Well Discharge Based on Avg of Flow from Collector Ditch and Sum of Individual Well Flow, gpm	Remarks
23 Feb	-	-	-	-	-	River stage = 179.6; elev of water in collector ditch = 177.55; net head on well system = 2.05 ft; total flow from collector ditch = 785 gpm.
26 Feb	1	-	53	-	51	River stage = 182.4; elev of water in collector ditch = 177.90; net head on well system = 4.50 ft; total flow from collector ditch = 1085 gpm; elev of water in drainage ditch landside of collector ditch = 177.92.
	2	-	60	-	57	
	3	-	49	-	47	
	4	-	40	-	38	
	5	-	31	-	30	
	6	-	42	-	40	
	7	-	42	-	40	
	8	-	38	-	36	
	9	-	43	-	41	
	10	-	45	-	43	
	11	-	39	-	37	
	12	-	31	-	30	
	13	-	43	-	41	
	14	-	44	-	42	
	15	-	31	-	30	
	16	-	25	-	24	
	17	-	30	-	29	
	18	-	30	-	29	
	19	-	29	-	28	
	20	-	37	-	35	
	21	-	31	-	30	
	22	-	38	-	36	
	23	-	37	-	35	
	24	-	45	-	43	
	25	-	43	-	41	
	26	-	47	-	45	
	27	-	33	-	31	
	28	-	38	-	36	
	29	-	46	-	44	
	30	-	54	-	51	
Total			1194		1140	
27 Feb	1	-	63	-	61	River stage = 183.25; elev of water in collector ditch = 177.94; net head on well system = 5.31 ft; total flow from collector ditch = 1330 gpm; elev of water in drainage ditch landside of collector ditch = 177.98.
		10	-	28	-	
		20	-	36	-	
		30	-	50	-	
	2	-	71	-	68	
	3	-	58	-	56	
	4	-	50	-	48	
	5	-	41	-	40	
		10	-	12	-	
		20	-	22	-	
		30	-	33	-	
	6	-	53	-	51	
	7	-	53	-	51	
	8	-	49	-	47	
	9	-	47	-	45	
	10	-	58	-	56	
		10	-	24	-	
		20	-	37	-	
		30	-	46	-	
	11	-	50	-	48	
	12	-	41	-	39	

(Continued)

Table 3 (Continued)

Date (1951)	Well No.	Well Distance from Bottom of Screen Ft	Measured Well Discharge, gpm Total	Measured Well Discharge, gpm Above Bottom of Screen	Well Discharge Based on Avg of Flow from Collector Ditch and Sum of Individual Well Flow, gpm	Remarks
27 Feb	13	-	44	-	42	
	14	-	49	-	47	
	15	-	43	-	41	
		10	-	22	-	
		20	-	33	-	
		30	-	33	-	
Est Total 1-30			1438		1384	Well flow meter failed.
1 Mar	1	-	88	-	86	River stage = 184.8; elev of water in collector ditch = 178.15; net head on well system = 6.66 ft; total flow from collector ditch = 1810 gpm; elev of water in drainage ditch landside of collector ditch = 178.0.
		10	-	39	-	
		20	-	51	-	
		30	-	71	-	
	2	-	88	-	86	
	3	-	75	-	73	
	4	-	63	-	61	
	5	-	55	-	54	An inspection of landside drainage ditch for 1000 ft upstream of well 1 disclosed several pin boils in the bottom of the ditch. Downstream of well 30 the same condition existed for a distance of about 1000 ft. No boils were found in the ditch immediately landward of the installation. Seep water was standing in the field 600 to 900 ft landward of sta 54/15, 500 ft downstream of well 30.
		10	-	20	-	
		20	-	36	-	
		30	-	49	-	
	6	-	73	-	71	
	7	-	56	-	55	
	8	-	64	-	62	
	9	-	59	-	57	
	10	-	75	-	73	
		10	-	31	-	
		20	-	46	-	
		30	-	58	-	The drainage ditch carrying the water landward from the collector ditch had about 2 ft freeboard for a distance of approximately 1000 ft landward of the levee and 3 to 4 ft for the next mile. At no point was the ditch overflowing.
	11	-	60	-	59	
	12	-	50	-	49	
	13	-	63	-	61	
	14	-	64	-	62	
	15	-	52	-	51	
		10	-	27	-	
		20	-	42	-	
		30	-	47	-	
	16	-	43	-	42	
	17	-	49	-	48	
	18	-	48	-	47	
	19	-	44	-	43	
	20	-	60	-	59	
		10	-	35	-	
		20	-	45	-	
		30	-	54	-	
	21	-	53	-	52	
	22	-	63	-	62	
	23	-	60	-	59	
	24	-	71	-	69	
	25	-	71	-	69	
		10	-	32	-	
		20	-	48	-	
		30	-	59	-	
	26	-	74	-	72	
	27	-	59	-	57	
	28	-	59	-	57	
	29	-	80	-	78	
	30	-	86	-	84	
		10	-	34	-	
		20	-	59	-	
		30	-	74	-	
Total			1905		1858	
2 Mar	1	-	94	-	95	River stage = 185.2; elev of water in collector ditch = 178.19; net head on well system = 7.02 ft; total flow from collector ditch = 2110 gpm; elev of water
		10	-	42	-	
		20	-	60	-	
		30	-	79	-	

(Continued)

Table 3 (Continued)

Date (1951)	Well No.	Well Distance from Bottom of Screen, Ft	Measured Well Discharge, gpm Total	Measured Well Discharge, gpm Above Bottom of Screen	Well Discharge Based on Avg of Flow from Collector Ditch and Sum of Individual Well Flow, gpm	Remarks
2 Mar	2	-	101	-	102	in drainage ditch landside of collector
	3	-	82	-	83	ditch = 177.99.
	4	-	68	-	69	
	5	-	62	-	63	Flow in landside drainage ditch 25 ft
		10	-	24	-	downstream of well 6 = 15.5 gpm and
		20	-	43	-	opposite well 25 = 25.5 gpm. (Distance
		30	-	58	-	between measuring points was 925 ft.)
	6	-	70	-	71	The seepage flow in the ditch was
	7	-	70	-	71	measured with submerged floats and a
	8	-	68	-	69	small wooden flume. The seepage amounted
	9	-	60	-	61	to 1.1 gpm per 100-ft sta of levee.
	10	-	76	-	77	
		10	-	34	-	Seepage conditions in areas adjacent to
		20	-	53	-	installation similar to those reported
		30	-	64	-	for 1 March.
	11	-	66	-	67	
	12	-	54	-	54	
	13	-	71	-	72	
	14	-	69	-	70	
	15	-	59	-	60	
		10	-	30	-	
		20	-	46	-	
		30	-	55	-	
	16	-	47	-	47	
	17	-	53	-	53	
	18	-	50	-	50	
	19	-	48	-	48	
	20	-	63	-	63	
		10	-	36	-	
		20	-	48	-	
		30	-	57	-	
	21	-	58	-	58	
	22	-	66	-	67	
	23	-	65	-	66	
	24	-	76	-	77	
	25	-	76	-	77	
		10	-	34	-	
		20	-	52	-	
		30	-	64	-	
	26	-	80	-	81	
	27	-	61	-	62	
	28	-	68	-	69	
	29	-	88	-	89	
	30	-	96	-	97	
		10	-	34	-	
		20	-	62	-	
		30	-	71	-	
Total			2065		2088	
5 Mar	1	-	No observation		-	Even numbered wells were closed on 2
	3	-	No observation		-	March and system was allowed to stabilize
	5	-	134	-	144	48 hours prior to the 5 March observa-
	7	-	133	-	143	tions.
	9	-	124	-	133	
						River stage = 185.77; elev of water in
	11	-	122	-	131	collector ditch = 178.24; net head on
	13	-	71	-	76	well system = 7.55 ft; total flow from
	15	-	82	-	88	collector ditch = 2075 gpm; elev of water
	17	-	110	-	118	in drainage ditch landward of collector
	19	-	77	-	83	ditch = 178.01.
	21	-	126	-	136	Flow in landside drainage ditch 25 ft
	23	-	136	-	146	downstream of well 6 = 21 gpm and
	25	-	140	-	151	opposite well 25 = 33 gpm. The seepage
	27	-	88	-	95	amounted to 1.3 gpm per 100-ft sta of

(Continued)

Table 3 (Continued)

Date (1951)	Well No.	Well Distance from Bottom of Screen, Ft	Measured Well Discharge, gpm Total	Measured Well Discharge, gpm Above Bottom of Screen	Well Discharge Based on Avg of Flow from Collector Ditch and Sum of Individual Well Flow, gpm	Remarks
5 Mar	29	-	148	-	159	levee. Slight increase in seepage was observed in areas adjacent to installation.
Est Total			1491		1804	
6 Mar	1	-	111	-	115	All wells were opened after 5 March observations and system allowed to stabilize 24 hours prior to these observations. Well spacing = 50 ft.
		10	-	No reading	-	
		20	-	69	-	
		30	-	87	-	
	2	-	118	-	122	
	3	-	99	-	104	River stage = 185.77; elev of water in collector ditch = 178.30; net head on well system = 7.48 ft; total flow from collector ditch = 2610 gpm; elev of water in drainage ditch landward of collector ditch = 178.02.
	4	-	85	-	88	
	5	-	79	-	82	
		10	-	30	-	
		20	-	52	-	
		30	-	70	-	
	6	-	82	-	85	
	7	-	82	-	85	The well flows measured on 6 March are believed to be slightly high because of the relatively short period of stabilization since the system was on 100-ft well spacing.
	8	-	78	-	81	
	9	-	74	-	77	
	10	-	88	-	91	
		10	-	37	-	
		20	-	56	-	
		30	-	71	-	
	11	-	77	-	80	
	12	-	63	-	66	
	13	-	82	-	85	
	14	-	78	-	81	
	15	-	71	-	74	
		10	-	31	-	
		20	-	52	-	
		30	-	60	-	
	16	-	55	-	57	
	17	-	60	-	62	
	18	-	60	-	62	
	19	-	60	-	62	
	20	-	71	-	74	
		10	-	38	-	
		20	-	52	-	
		30	-	61	-	
	21	-	70	-	73	
	22	-	75	-	78	
	23	-	77	-	80	
	24	-	88	-	91	
	25	-	84	-	87	
		10	-	36	-	
		20	-	44	-	
		30	-	70	-	
	26	-	88	-	91	
	27	-	72	-	75	
	28	-	79	-	82	
	29	-	99	-	104	
	30	-	117	-	121	
		10	-	27	-	
		20	-	63	-	
		30	-	83	-	
Total			2422		2515	
8 Mar	-	-	-	-	-	All wells open and system allowed to stabilize 72 hours prior to these observations. River stage = 185.7; elev of water in collector ditch = 178.28; net head on well system = 7.43 ft; total flow from

(Continued)

Table 3 (Continued)

Date (1951)	Well No.	Well Distance from Bottom of Screen, Ft	Measured Well Discharge, gpm Total	Measured Well Discharge, gpm Above Bottom of Screen	Well Discharge Based on Avg of Flow from Collector Ditch and Sum of Individual Well Flow, gpm	Remarks
8 Mar	-	-	-	-	-	collector ditch = 2340 gpm; elev of water in drainage ditch landward of collector ditch = 178.01. Flow in landside drainage ditch 25 ft downstream of well 6 = 21 gpm and opposite well 25 = 21 gpm. The seepage entering the ditch between these two points was too small to detect. Seepage conditions in areas adjacent to installation were about the same as reported 1 March.
10 Mar	-	-	-	-	-	All wells had been closed for 44 hours when these observations were made. River stage = 185.49; elev of water in collector ditch = 177.44; net head above pool in collector ditch = 8.05 ft; flow from collector ditch = 145 gpm; elev of water in drainage ditch landward of collector ditch = 178.1. Flow in drainage ditch landward of collector ditch 25 ft downstream of well 6 = 39 gpm and opposite well 25 = 79 gpm. The seepage entering the ditch between these two points amounted to 4.4 gpm per 100 ft of levee station. Seep water began appearing in the field 400 to 600 ft landward of installation 24 hours after closing the wells. After the wells were closed 44 hours, seep water was observed landward of the well system for a distance of 1100 ft. Numerous pin boils were active in the drainage ditch which runs approximately perpendicular to the levee at the downstream end of the installation. The boils extended for a distance of 1200 ft landward of the levee. Numerous pin boils were also active in the drainage ditch parallel to and landward of the well system. Seepage was entering the collector ditch through the slopes and bottom. Seepage was also observed emerging on the landside slope of the sub-levee around the well system.
12 Mar	-	-	-	-	-	All wells were opened 48 hours prior to these observations. River stage = 185.15; elev of water in collector ditch = 178.21; net head on well system = 6.93 ft; total flow from collector ditch = 2370 gpm; elev of water in drainage ditch landward of installation = 178.0. Seep water had disappeared in the field immediately landward of installation. (Field was dry for a distance of 700 ft landward of installation; 700 to 1100 ft landward the field was still wet but there was no water ponded on the surface.) Sand boils in the drainage ditch landward

(Continued)

Table 3 (Continued)

Date (1951)	Well No.	Well Distance from Bottom of Screen, Ft	Measured Well Discharge, gpm Total	Measured Well Discharge, gpm Above Bottom of Screen	Well Discharge Based on Avg of Flow from Collector Ditch and Sum of Individual Well Flow, gpm	Remarks
12 Mar	-	-	-	-	-	and parallel to system and in the ditch perpendicular to the levee at the downstream end of the system had stopped. Seepage through the bottom and slopes of the collector ditch was not evident. Seepage on the landside slope of the sub-levee had also stopped.
16 Mar	1	-	96	-	94	All wells had been open since 10 March.
		10	-	43	-	
		20	-	55	-	River stage = 183.60; elev of water in collector ditch = 178.10; net head on well system = 5.48 ft; total flow from collector ditch = 1935 gpm; elev of water in drainage ditch landward of collector ditch = 178.0.
		30	-	79	-	
	2	-	101	-	99	
	3	-	84	-	82	
	4	-	71	-	70	
	5	-	66	-	64	
		10	-	27	-	
		20	-	43	-	
		30	-	59	-	
	6	-	65	-	64	
	7	-	69	-	67	
	8	-	65	-	64	
	9	-	66	-	64	
	10	-	73	-	71	
		10	-	32	-	
		20	-	50	-	
		30	-	59	-	
	11	-	62	-	61	
	12	-	52	-	51	
	13	-	67	-	65	
	14	-	66	-	64	
	15	-	58	-	57	
		10	-	31	-	
		20	-	47	-	
		30	-	51	-	
	16	-	53	-	52	
	17	-	52	-	51	
	18	-	51	-	50	
	19	-	51	-	50	
		10	-	28	-	
		20	-	35	-	
		30	-	44	-	
	20	-	56	-	55	Meter would not pass well screen connection.
	21	-	56	-	55	
	22	-	62	-	61	
	23	-	63	-	62	
	24	-	72	-	71	
	25	-	72	-	71	
		10	-	30	-	
		20	-	41	-	
		30	-	55	-	
	26	-	71	-	70	
	27	-	59	-	58	
	28	-	65	-	64	
	29	-	82	-	80	
	30	-	90	-	88	Well 30 had some silt in lower part of screen.
		10	-	52	-	
		20	-	67	-	
		30	-	89	-	
Total			2016		1975	

(Continued)

Table 3 (Continued)

Date (1951)	No.	Well Distance from Bottom of Screen, Ft	Measured Well Discharge, gpm Total	Measured Well Discharge, gpm Above Bottom of Screen	Well Discharge Based on Avg of Flow from Collector Ditch and Sum of Individual Well Flow, gpm	Sediment in Well, Ft	Remarks
10 Apr	1	-	102	-	101	2.4	River stage = 184.4; elev of water in collector ditch = 178.16; net head on well system = 6.24 ft; total flow from collector ditch = 2120 gpm; elev of water in drainage ditch landward of installation = 177.91.
		10	-	44	-	-	
		20	-	60	-	-	
		30	-	76	-	-	
	2	-	110	-	109	1.1	
	3	-	89	-	88	0.9	
	4	-	79	-	78	0.8	
	5	-	74	-	73	1.4	
		10	-	29	-	-	
		20	-	49	-	-	Seepage conditions adjacent to installation were similar to those reported for 1 March 1951.
		30	-	64	-	-	
	6	-	73	-	72	1.0	
	7	-	72	-	71	1.0	
	8	-	74	-	73	1.4	
	9	-	66	-	65	2.6	All wells were sounded to determine the amount of sediment that had accumulated in screen sections.
	10	-	83	-	82	1.1	
		10	-	32	-	-	
		20	-	52	-	-	
		30	-	65	-	-	
	11	-	71	-	70	1.4	
	12	-	56	-	56	2.4	
	13	-	75	-	74	2.4	
	14	-	74	-	73	1.4	
	15	-	64	-	63	1.5	
		10	-	37	-	-	
		20	-	48	-	-	
		30	-	55	-	-	
	16	-	56	-	56	1.0	
	17	-	57	-	57	1.0	
	18	-	55	-	55	1.0	
	19	-	52	-	52	0.5	
	20	-	61	-	60	0.6	
		10	-	38	-	-	
		20	-	48	-	-	
		30	-	56	-	-	
	21	-	59	-	59	0.4	
	22	-	64	-	63	0.3	
	23	-	67	-	66	0.7	
	24	-	76	-	75	1.0	
	25	-	74	-	73	1.2	
		10	-	41	-	-	
		20	-	55	-	-	
		30	-	65	-	-	
	26	-	75	-	74	0.9	
	27	-	60	-	59	1.3	
	28	-	70	-	69	1.4	
	29	-	83	-	82	2.9	
	30	-	98	-	97	3.3	
		10	-	43	-	-	
		20	-	79	-	-	
		30	-	96	-	-	
Total			2169		2145		

(Continued)

Table 3 (Concluded)

Date (1951)	Well No.	Well Distance from Bottom of Screen, Ft	Measured Well Discharge, gpm Total	Measured Well Discharge, gpm Above Bottom of Screen	Well Discharge Based on Avg of Flow from Collector Ditch and Sum of Individual Well Flow, gpm	Remarks
30 Apr	1	-	105	-	103	River stage = 184.4; elev of water in collector ditch = 178.18; net head on well system = 6.22 ft; total flow from collector ditch = 2100 gpm; elev of water in drainage ditch landward of installation = 178.04.
		10	-	50	-	
		20	-	65	-	
		30	-	89	-	
	2	-	110	-	108	
	3	-	92	-	90	
	4	-	81	-	79	
	5	-	77	-	75	Seepage conditions adjacent to installation were similar to those reported for 10 April 1951.
		10	-	30	-	
		20	-	51	-	
		30	-	66	-	
	6	-	75	-	73	
	7	-	75	-	73	
	8	-	75	-	73	
	9	-	67	-	65	
	10	-	80	-	78	
		10	-	35	-	
		20	-	53	-	
		30	-	64	-	
	11	-	70	-	68	
	12	-	57	-	56	
	13	-	74	-	72	
	14	-	75	-	73	
	15	-	64	-	63	
		10	-	32	-	
		20	-	48	-	
		30	-	55	-	
	16	-	56	-	55	
	17	-	58	-	57	
	18	-	55	-	54	
	19	-	52	-	51	
	20	-	61	-	60	
		10	-	35	-	
		20	-	46	-	
		30	-	53	-	
	21	-	63	-	62	
	22	-	66	-	65	
	23	-	71	-	70	
	24	-	75	-	73	
	25	-	74	-	73	
		10	-	38	-	
		20	-	54	-	
		30	-	63	-	
	26	-	76	-	74	
	27	-	61	-	60	
	28	-	69	-	67	
	29	-	86	-	84	
	30	-	96	-	94	
		10	-	39	-	
		20	-	62	-	
		30	-	78	-	
Total			2196		2148	

Notes: 1. Well spacing was 50 ft unless specifically stated otherwise.
2. Flow from collector ditch was measured with a "midget" Price current meter.
3. Flow from individual wells was measured with a well velocity meter lowered into the well.
4. Drainage gate through sub-levee around well system was open during entire observation period.

APPENDIX A

WELL TANK TESTS

1. The well screen and gravel filter proposed for use in the relief wells to be installed at Trotters, Mississippi, were tested in the well-testing apparatus at the Waterways Experiment Station to check the design of the filter and perforations in the wooden well screens. The tests included determination of the head loss through the filter and well screen, the stability of the gravel filter with respect to the slots in the well screen, and the ability of the filter gravel to prevent excessive infiltration of the foundation sand.

Description of Tests

Well screen

2. The wooden pipe used for the well screen was made of untreated white pine staves and had an inside diameter of 6 in. with a wall thickness of approximately 1 in. The wooden pipe staves were bound with a specially wound steel wire and the screen had 28 longitudinal 2- by 3/16-in. slots per lin ft (total area of slots = 10.5 sq in. or 4.65 per cent of the inside area of the pipe). The area of the slots in the screen tested was somewhat less than the area of slots used in the well installation, which was about 18 sq in. per ft. A photograph of the screen used during testing is shown in fig. (D) on plate A-1.

Filter gravel

3. The filter gravel used in the well system was designed on the basis of the following filter criteria:

$$\frac{D\ 85\ F}{\text{Slot width}} > 1.2;\quad \frac{D\ 15\ F}{D\ 85\ S} < 5.0;\ \text{and}\ \frac{D\ 15\ F}{D\ 15\ S} > 5.0.$$

The permeability of the filter was about 5000 x 10^{-4} cm/sec. The allowable range of filter material specified for the Trotters wells and the range of the pervious stratum sands found at that site are shown on plate 13. Mechanical analyses of the filter gravel and foundation sand used in the well tank tests are shown on fig. (E), plate A-1.

Equipment

4. The apparatus* used for making these experiments consisted principally of a circular steel tank 5 ft in diameter and 2 ft deep. A circular 60-mesh screen, 4 ft 4 in. in diameter, which formed the outer container for the foundation sand was mounted inside the tank. A 4-in. space between the screen and the tank wall afforded a chamber for distributing the flow uniformly around the sand and for maintaining uniform pressure conditions. Photographs of the apparatus are shown in figs. (A) and (B), plate A-1.

5. After the well screen to be tested had been installed in the center of the tank, a metal form for the filter gravel, 15 in. long and of 22 in. diameter, was placed concentrically around the well. Enough water then was admitted into the tank to permit the materials to be placed under water. The foundation sand was raked as it was placed to eliminate entrapped air. No other compaction was used. The filter gravel was kept at approximately the same level as the sand and was

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* Detailed description of apparatus is contained in Waterways Experiment Station T.M. 195-1, "Field and Laboratory Investigation of Design Criteria for Drainage Wells," October 1942.

rodded lightly as it was placed. The filter was installed in clean water. When the sand and gravel had been placed to a depth of about 1 ft, the shell was removed and the piezometer tips were placed in position. These piezometers, which were in duplicate on opposite sides of the well, were located in the space between the tank wall and the screen enclosing the sand foundation; at the interface between the sand foundation and the filter; at the intersection of diagonals drawn between four adjacent slots and about 1/4 in. from the outer wall of the well screen; and in the stilling basin. The head loss through the foundation sand, the filter, and the well screen was measured by means of these piezometers. The shell was replaced after the piezometer tips were set in position and the placing of the sand and gravel resumed. When the level of the sand and gravel was about 1/2 in. from the top of the tank the shell was again removed and about 1/4 in. of sand was placed over both the foundation sand and gravel filter. The water level was then lowered to approximately 1/8 in. below the surface of the sand and a 1/2-in. layer of bentonite was laid on the surface. A photograph of the well tank with the sand, filter, and well screen in place is shown in fig. (D), plate A-1.

6. With the materials to be tested in position, a rigid watertight lid with a packing collar for the well screen was bolted to the tank. The neck of the well was packed with cotton and "Plastilina" where it extended through the top plate. Water was admitted to create a small pressure within the tank to wet the bentonite, thereby causing it to swell and seal the space between the top of the sand and the plate. An air-relief valve in the top plate allowed any entrapped air to escape

from the tank. A vacuum line was used to clear the piezometer lines of any entrapped air. A stilling basin served to catch any material which might wash out of the well during testing and also created a constant tailwater condition.

7. The water supply, obtained from a large overhead tank, was maintained at a constant elevation during the tests. Pressure heads in the outer portion of the well tank were controlled by a valve in the supply line. Discharge measurements were made with a calibrated container and a stop watch. Sand inwash was removed from the bottom of the well by a vacuum line attached to a sand and water trap.

Testing procedure

8. After the well was installed in the test tank the pressure head at the outer circumference of the sand foundation was raised to an initial net head of 0.2 ft of water. When the flow from the well became stabilized at this pressure the piezometers were read and the flow measured. Before raising the pressure another increment, the sand and gravel which washed into the well at this rate of flow were removed from the stilling basin and well for weighing. This procedure was continued with net pressure heads of 0.5, 1.0, 1.5, 2.0, and 3.0 ft.

9. At the end of the above cycle of pressure readings the well was surged by plunging a 5-in. steel disk attached to the end of a rod inside the well for 50 strokes. The sand and gravel inwash was then collected and the cycle of pressure readings as described in the preceding paragraph was repeated. The well again was surged with the disk plunger for 20 minutes, the inwash material collected, and the cycle of pressure readings repeated. The head losses through the filter and screen as

measured for the various rates of flow used in the tests are plotted in fig. (C) on plate A-1. The following discussion is based on a flow of 5 gpm per foot of well screen.

Test Results

10. Immediately after the material had been placed the combined head loss through both filter and screen was 0.24 ft, and through the screen alone was 0.15 ft for a flow of 5 gpm. These losses were reduced to 0.13 and 0.05 ft, respectively, by surging with the 5-in. disk plunger 50 times. Additional surging for 20 minutes with the plunger did not change the results materially. These latter losses are considered satisfactory and check reasonably well the head losses observed during the pumping tests on the wells as installed.

11. Additional tests were performed to determine whether the use of bentonitic mud in the process of drilling the holes for the wells would result in a satisfactorily functioning well. The results of these tests indicated that even after violent surging it was not possible to remove the bentonite mud from the filter and that the resulting head losses were about two to three times those occurring after surging the filter placed in clear water.