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CORPS OF ENGINEERS, U. S. ARMY

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INVESTIGATION OF UNDERSEEPAGE MISSISSIPPI RIVER LEVEES, ALTON TO GALE, ILL.

TECHNICAL MEMORANDUM NO. 3-430

PREPARED FOR

ST. LOUIS DISTRICT, CORPS OF ENGINEERS

BY

WATERWAYS EXPERIMENT STATION

VICKSBURG, MISSISSIPPI

ARMY-MRC VICKSBURG, MISS.

IN TWO VOLUMES VOL. 1

APRIL 1956

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PREFACE

In 1950 the St. Louis District initiated an investigation of seepage beneath the levees along the Mississippi River between Alton and Gale, Illinois. This investigation indicated that numerous reaches of levee might be critical with respect to underseepage. In October 1952 the Waterways Experiment Station was authorized by the Office, Chief of Engineers, to assist the St. Louis District with the investigation and to design the required seepage control measures. This report presents the results of that investigation, and includes all field, laboratory, and design studies made in connection with the project; geological studies of the alluvial valley; construction procedures for installing and testing relief wells and piezometers; as-built information on relief wells and piezometers installed to date; and a description of maintenance required for wells and piezometers.

Field exploration, pumping tests, and laboratory testing were conducted by the Waterways Experiment Station and the St. Louis District. The Tulsa, Omaha, Mobile, and Louisville Districts furnished drill crews and rigs to assist with the field exploration. Surveys for the investigation were made by the St. Louis District. Most of the piezometers were installed by the St. Louis District and the Waterways Experiment Station, although some piezometers were installed by contract. Relief wells were installed and seepage berms were built by contract under the supervision of the St. Louis District. Personnel of the Experiment Station also served in an advisory capacity on procedure regarding installation of relief wells. Investigation and design of remedial measures for scour in riverside borrow pits were made by the St. Louis District. The underseepage control measures were designed by the Waterways Experiment Station.

New air-photo maps of the levees from Alton to Gale, Illinois, showing levee appurtenances and seepage control measures designed and constructed as a part of this project were prepared by the St. Louis District and the Waterways Experiment Station and have been published in a separate volume.

At the beginning of the project, an advisory board was set up to assist in the planning of the investigation. Members of this board were Messrs. T. A. Middlebrooks (deceased) and R. A. Barron, Office, Chief of Engineers; W. J. Turnbull, Waterways Experiment Station; P. T. Bennett, Missouri River Division; and K. V. Taylor, formerly of the Kansas City District.

Engineers in the St. Louis District actively connected with the investigation and installation of seepage control measures were Messrs. W. F. Lawlor, D. S. Blair, J. J. Zebas, C. J. Weaver, S. E. Hoffman, Jr., H. N. Abele, J. E. Fuhrmann, H. F. Louvall, H. L. Gray, and A. E. Fedder. Mr. Harry Leibovich, formerly with the Upper Mississippi Valley Division, was also connected with the investigation.

Planning of the investigation, design of control measures, and preparation of this report for the Waterways Experiment Station were carried out by Messrs. C. I. Mansur, R. I. Kaufman, and W. M. Nichols. Field explorations were accomplished under the direction of Mr. T. B. Goode. Laboratory testing was supervised by Messrs. J. E. Mitchell and A. L. Boggs. Other personnel at the Experiment Station who assisted in the project were Messrs. G. W. Leese, R. R. Webb, engineers, and R. D. McCary and A. L. Sullivan, draftsmen. Activities of the Experiment Station were under the general direction of Messrs. W. J. Turnbull and W. G. Shockley.

Other engineers or geologists formerly at the Waterways Experiment Station who were directly connected with the investigation were Messrs. S. J. Johnson, J. R. Schultz, J. A. Focht, Jr., J. L. McCall, Jr., and G. D. Jones.

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NOTATIONS

a Well spacing

a_c Computed well spacing corrected for head loss in well

c A constant, $c = \sqrt{\frac{k_b}{k_f\ z\ d}}$, for natural top stratum

DD Effective drawdown in well during a pumping test

D_n Grain size, n per cent of grains smaller than stated size

D_{10} Effective grain size, 10 per cent of grains smaller than stated size

d Effective thickness of pervious substratum for aquifer

e_n Natural void ratio of soil

e_R Void ratio of laboratory test specimen

F Factor of safety against uplift at landside toe of levee

G Ratio of flow from a partially penetrating well to that from a fully penetrating artesian well; also specific gravity of soil

H Total net head on levee

H_m Head (net) beneath top stratum midway between relief wells

$H_{m\infty}$ Head (net) beneath top stratum midway between wells in an infinite line of wells

H_{m_N} Head (net) beneath top stratum midway between wells at center of a finite line of wells

H_w Total head loss in a well including loss through filter, screen, in well, and elevation head loss

h_a Allowable (net) head beneath landside top stratum

h_e Head loss through filter and well screen

h_o Head (net) beneath top stratum at landside toe of levee (without seepage control measures) assuming top stratum capable of withstanding such a head

h'_o Head beneath top stratum at landside toe of levee (measured above

natural ground surface or tailwater) with a landside seepage berm

h_{o_1} Head beneath top stratum and berm (measured above natural ground surface) $X_s/2$ distant landward from landside toe of levee

h_r Friction loss in well riser pipe

h_s Friction loss in well screen

h_v Velocity head loss in relief well

h_x Head (net) beneath top stratum at distance x landward from landside toe of levee

$h_{x(c)}$ Net head above ground surface or tailwater at time sand boils or heaving of top stratum occurs

h_1, h_2 Substratum heads (above ground surface) at two piezometers on a line perpendicular to the levee at distances ℓ_1 and ℓ_2, respectively, from landside toe of levee.

i Upward gradient through top stratum (and berm) landward of levee

i_c Critical upward gradient through top stratum landward of levee

i_o Allowable upward gradient at landside toe of levee

i_1 Allowable upward gradient at toe of landside seepage berm

k Coefficient of permeability

k_b Vertical permeability of top stratum landward of levee

k_f Horizontal permeability of effective thickness of pervious substratum

k_n Coefficient of permeability of a laboratory test specimen at its estimated natural void ratio, e_n

k_H Horizontal permeability of a pervious stratum

k_R Permeability of remolded soil sample as void ratio e_R as determined from laboratory tests

k_{v-n} Vertical permeability of a stratum

L_2 Base width of levee, and berm if present

L_3 Landward (effective) extent of top stratum

ℓ Distance between two piezometers installed on a line perpendicular to the levee

ℓ_1, ℓ_2 Respective distances from landside toe of levee (or berm) to piezometers 1 and 2, installed on a line perpendicular to the levee

M Slope of hydraulic grade line in pervious substratum beneath the levee

Q_s Total seepage flow (with or without wells) per unit length of levee per unit of time

Q_w Flow from a single relief well

$Q_{w(100)}$ Well flow per 100 ft of levee

q Well or seepage flow in gpm per 100 ft of levee as determined in model studies

R Radius of influence of a well during a pumping test

r Ratio of allowable upward gradient through top stratum at toe of levee to that at toe of seepage berm = i_o/i_1

r_w Effective radius of a relief well

s Distance from landside toe of levee (or berm) to effective source of seepage entry

t Required thickness of landside seepage berm at toe of levee

W Penetration of well screen into pervious aquifer expressed as a decimal

X Width of landside seepage berm

X_p Width of a free-draining berm

X_s Required width of sand berm

X_{sp} Required width of a semipervious berm

x_3 Distance from landside toe of levee (or berm) to effective seepage exit

z_n Thickness of individual layers comprising top stratum

z_{b-n} Transformed thickness of a stratum comprising the top stratum

z Effective thickness of top stratum landward and riverward of levee

z_t Total thickness of landside top stratum

γ'_t Submerged unit weight of seepage berm

γ'_z Submerged unit weight of landside top stratum

γ_w Unit weight of water

$\mathcal{F}$ Shape factor, the ratio in a flow net of the number of flow channels to number of equipotential drops from the seepage source to exit

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SUMMARY

This report presents the results of an investigation of underseepage along Mississippi River and tributary levees from Alton to Gale, Illinois. It also includes the design and construction of seepage control measures, and recommendations for the maintenance and observation of such measures during high water.

The alluvial valley of the Mississippi River from Alton to Gale, Illinois, is about 3 to 10 miles wide and is confined by resistant rock bluffs which roughly parallel the river. Numerous tributary streams emerge from the bluffs and flow through the flood plain into the Mississippi River. The areas between the tributary streams are protected by levees along those streams and the Mississippi River. These areas are subdivided into 16 levee districts which are protected by about 240 miles of levee.

The alluvial valley of the Mississippi River between Alton and Gale generally consists of a top stratum of silts or clays about 3 to 30 ft thick underlain by a pervious substratum about 75 to 150 ft thick comprised of fine to coarse sands with some gravel and cobbles. The permeability of the pervious substratum ranges from about 750 to 2000×10^{-4} cm per sec. The alluvial fill generally overlies limestone rock.

The levees are constructed of top stratum material generally obtained from borrow pits riverside of the levee. In some borrow pits the top stratum has been either completely or largely removed, thereby creating a ready source for entry of seepage into the underlying pervious substratum.

To determine whether seepage might endanger the levee system during high water, a field investigation was first made of foundation conditions along the levees. Data from this investigation were used in the design of control measures where considered necessary. The field investigation consisted of a reconnaissance of all the levees, and extensive studies to delineate soil and foundation conditions along the levees.

Relief wells or seepage berms were considered warranted wherever it appeared that the upward gradient through the top stratum landward of the levee would equal 0.85 with a river stage at the net grade of the levee. On the basis of this criterion, seepage control measures including about 2100 relief wells and 14 landside seepage berms were considered warranted along about 70 miles of the 240 miles of levees.

The relief wells consist of 8-in.-ID wood-stave pipe installed to depths of about 70 to 100 ft and spaced on about 75- to 300-ft centers. A 6-in. sand gravel filter was used around the screens for the relief wells. The wells were designed on the basis of 50 per cent penetration of the pervious substratum. From special pumping tests, it was found that a penetration of 60 per cent of the pervious substratum on a depth basis was necessary to achieve an effective penetration of 50 per cent because of increased permeability of the pervious substratum with depth. The relief wells are provided with a check valve and rubber gasket to prevent their being backflooded with muddy surface water, a metal guard to protect the wells against animals and vandalism, and removable standpipes to prevent the wells from flowing at low flood stages. Each well was subjected to a pumping test to check its stability and to determine its specific yield. In general, the wells had specific yields ranging from about 50 to 150 gpm per ft of drawdown.

Landside seepage berms generally were built of random soil except for one sand berm to be constructed in the East St. Louis levee district. Landside seepage berms generally were designed in accordance with criteria derived from electrical analogy studies by the Kansas City District, CE. These studies indicated that landside seepage berms should be sufficiently thick to withstand uplift pressures yet be thin enough to permit seepage relief. They should be sufficiently wide, of course, to prevent the development of critical uplift pressures at the berm toe.

Piezometers were installed between wells and along the toe of seepage berms to check on their adequacy and performance during high water, and also along reaches of levee where underseepage might be a problem but the need for control measures was questionable. Some piezometers were installed beneath the levee to determine the distance to the

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effective source of seepage entry and exit. If piezometric data obtained during future high waters indicate that critical uplift pressures will develop at a project flood stage in presently unprotected areas, control measures will then be designed and installed.

In addition to areas where relief wells and berms were necessary, areas exist where the levee is founded directly on relatively thin strata of pervious materials. In these reaches shallow cutoffs of clay compacted in trenches excavated at the riverside toe of the levee are being constructed to prevent piping in the pervious strata directly beneath the levee.

At the time of this report, installation of relief wells is essentially complete in 13 of the 16 levee districts. Sixteen hundred and twenty-six of 2113 relief wells have been installed and pump-tested. Eight of the 14 landside seepage berms have been constructed. Installation of piezometers is essentially complete in 13 levee districts.

It is also planned to observe the flow from selected relief wells. The relief well systems and piezometers are to be maintained and inspected and each well will be subjected to pump-testing about every five to eight years.

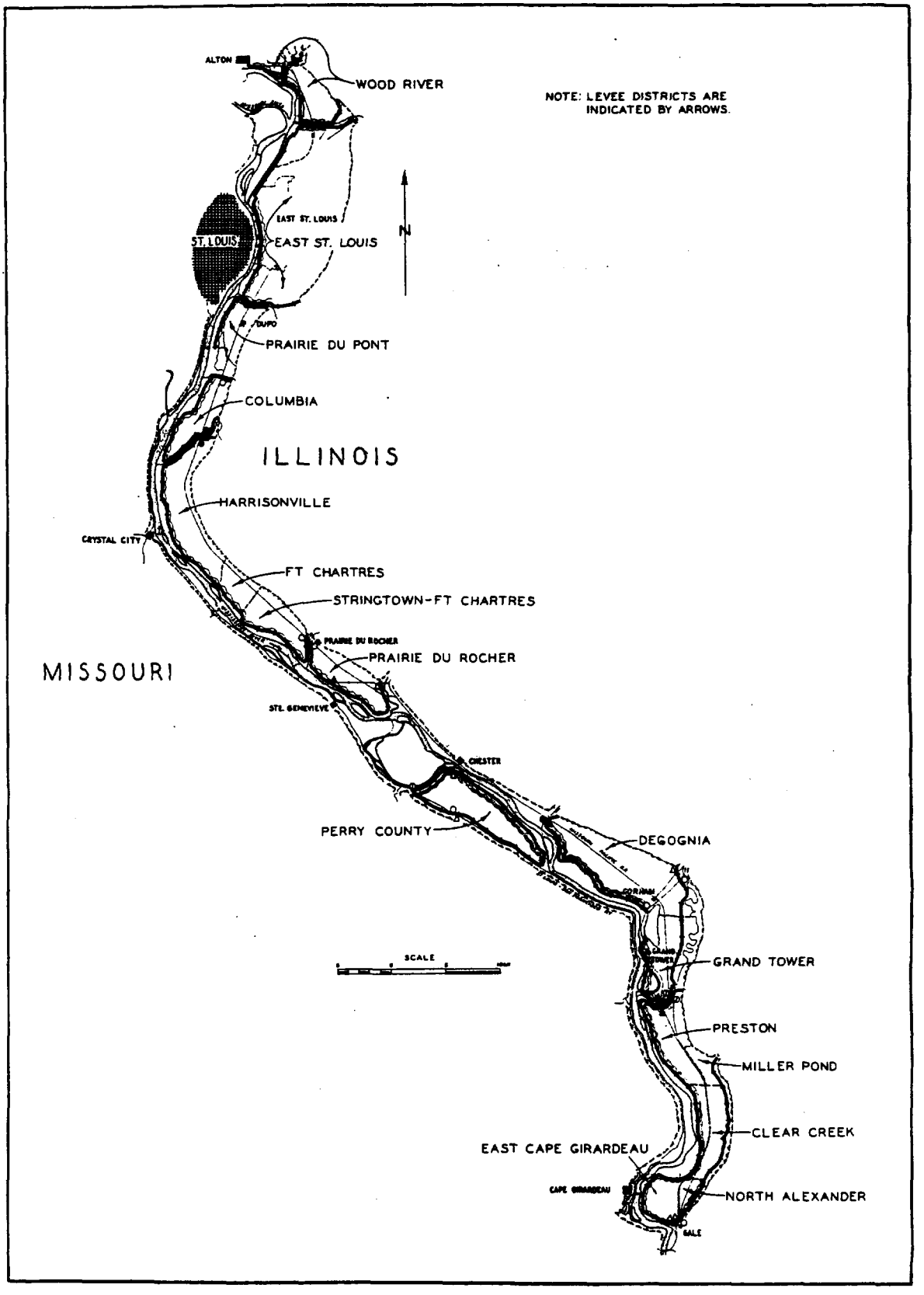

FIG. 1. PLAN OF LEVEES AND LEVEE DISTRICTS ALONG MISSISSIPPI RIVER, ALTON TO GALE, ILL.

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INVESTIGATION OF UNDERSEEPAGE
MISSISSIPPI RIVER LEVEES, ALTON TO GALE, ILLINOIS

PART I: INTRODUCTION

Location and Statement of Problem

1. This report contains the results of an investigation of underseepage and the design and construction of seepage control measures for levees along the Mississippi River from Alton to Gale, Illinois.

2. In the reach of the Mississippi River between Alton and Gale, Illinois, the alluvial valley is 3 to 10 miles wide and is confined by resistant rock bluffs which roughly parallel the river. Generally, the river flows adjacent to the bluffs on the west (Missouri) side and the flood plain is on the east (Illinois) side. Numerous tributary streams flow across the flood plain and empty into the Mississippi River between Alton and Gale. Therefore the levee system along the Mississippi is supplemented by levees on the tributary streams that originate at the bluffs and parallel the tributary streams toward the Mississippi River until they join the main stream system (see fig. 1). The total length of the levees is about 240 miles. Levees along the Mississippi River are termed "river-front" levees, whereas levees along the tributary streams are called "flank" levees. An "upper" flank levee is that portion at the upstream end of the protected area, and a "lower" flank levee is the portion at the downstream end of the protected area. The area so protected is either in itself a levee district or is subdivided into several levee districts. Locations of the levees and levee districts considered in this report are shown in fig. 1. Detailed maps of the levee system are included in the publication entitled "Air-photo Maps of Mississippi River Levees, St. Louis District, Alton to Gale, Illinois."[10*]

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* Raised numbers refer to the list of references at the end of this report.

3. Although a project flood has not yet occurred in the St. Louis District, underseepage has been a problem during previous high waters (see plates 1 and 2). Inasmuch as the levees from Alton to Gale had not previously been sufficiently high to withstand the project flood, many of them were overtopped before very high river stages occurred. In addition, certain areas had not been protected by levees until recently.

4. Under the Flood Control Acts of 1936, 1938, and 1946 the St. Louis District was authorized to complete the levee systems and to raise the grade of existing levees so that they could withstand the project flood. This program created a problem which had not existed previously, i.e., because greater heads against the levees could now develop, the safety of the levees against underseepage along certain reaches became questionable. The St. Louis District initiated a preliminary investigation of underseepage for the levee system. The results of these studies indicated that numerous reaches of the levee system might be subject to dangerous underseepage and that seepage control measures were necessary. Some seepage berms were constructed, a limited number of relief wells were installed along several critical reaches of levee, and some piezometers were installed to determine the pressures beneath the top stratum in other levee reaches.

Purpose and Scope of Investigation

5. The purposes of the investigation were to (a) delineate soil and foundation chracteristics along the levees; (b) locate sections of levee considered critical with respect to underseepage; (c) determine other factors related to seepage beneath the levees and the design of seepage control measures; (d) design seepage control measures where considered necessary; and (e) lay out piezometers along reaches of levee where underseepage might be a problem but control measures did not appear warranted on the basis of available data.

6. The investigation included: the making of numerous borings along the landside toe of the levees to determine the character and thickness of the top stratum, some shallow borings in riverside borrow

pits, some borings to delineate certain geological features, some deep borings to obtain samples of the pervious substratum and to determine its depth; studies of aerial mosaics; electrical resistivity and seismic surveys to determine the depth to rock; field pumping tests to determine the permeability of the underlying sand stratum; and evaluation of the need for underseepage control measures. The study also included the design of relief wells and seepage berms to control underseepage where considered necessary.

Scope of Report

7. This report summarizes the investigation, design, and installation of seepage control measures along the Mississippi River levees from Alton to Gale, Ill., and is divided into text (volume I), plates and tables relating to the individual levee districts (volume II), and a folio of air-photo maps of the levee system[10].

8. A description of the seepage problem and general discussion of methods for control of underseepage are discussed in part I of this report. A discussion of the geology of the Mississippi River valley as related to underseepage, including definition and description of various types of geological features encountered in the alluvial valley in the St. Louis District having an effect on underseepage, is presented in part II. Field reconnaissance and investigations, borings, geophysical investigations, the surveying program, and field pumping tests are discussed in part III. Laboratory investigations conducted for the design of the seepage control measures are described in part IV.

9. Evaluation of the seepage problem and formulas for seepage analyses are presented in part V. Methods used to design seepage control measures, and a general discussion of the principles involved in the design of such control measures, are contained in part VI. Formulas and graphs for the design of relief wells and seepage berms are also presented.

10. The design and installation of relief wells and seepage berms for the individual levee districts are presented in part VII. Sections

in this part include a detailed description of the geology of each levee district and soil conditions as revealed by aerial mosaics and borings, the history of seepage conditions, details of the design of well systems and berms, special features concerning the installation of piezometers and relief wells, permeability of the pervious substratum as determined from the pumping tests, and a summary of observations to be made during high-water periods.

11. The results of special pumping tests on certain wells in the Harrisonville, Fort Chartres, and East St. Louis levee districts are discussed in part VIII. The design and installation of relief wells and installation of piezometers are described in part IX. Part X contains procedures to be followed in maintaining the relief wells and piezometers, and in observing the piezometers and relief wells during flood periods. The system used for classifying sands and gravels is described in appendix A. The results of freeze-thaw tests on porous concrete for collector ditch lining are presented in appendix B. Special tests on check valves for relief wells are described in appendix C. Construction costs of various underseepage control measures are presented in appendix D. Locations of river gages along the Mississippi River from Alton to Gale, Ill., are given in appendix E.

12. Volume II of this report consists of the following plates for each levee district: soil profiles along the landside toe of the levee, cross sections to develop certain topographical features and soil conditions other than those at the levee toe, tables showing the design of relief wells and seepage berms, cross sections of seepage berms and piezometer lines, and a summary of all well and piezometer installation data available to date. The soil profiles show the net grade of the levee, ground profiles, logs of borings, stratification, elevation of rock, elevation of screens of piezometers, elevations of screens and filters for wells that have been installed, specific yield of wells, and the proposed locations of wells and piezometers which were not installed prior to 1 November 1955. Volume II also contains photographs of seepage conditions during past high-water periods and all plates pertinent to parts VIII-IX.

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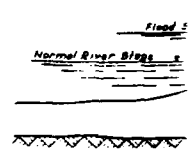

13. A volume of air-photo maps[10] of the levee system from Alton to Gale, Ill., was also prepared in connection with this study. These maps show the levees and the levee stationing and include the locations of cities, towns, highways, rivers, drainage structures, river gages, abatis dikes, relief wells, piezometers, seepage berms, cutoff trenches, and borrow pit fills. Both as-constructed and as-designed items are shown on the maps, but a distinction is made between such items. In this report, all references to maps and map numbers refer to these air-photo maps.

14. The St. Louis District will prepare an additional appendix to this report containing the installation data for relief wells and piezometers not installed in time to be described herein.

Development of Underseepage and Sand Boils

15. Whenever a levee is subjected to a differential hydrostatic head of water as a result of river stages being higher than the adjacent land, seepage enters the pervious substratum beneath the levee through the bed of the river, riverside borrow pits, and/or the riverside top stratum, and creates an artesian head and hydraulic gradient in the sand stratum under the levee. This gradient causes a flow of seepage beneath and landward of the levee as illustrated in fig. 2. The seepage from the pervious substratum that emerges at or landward of the levee toe is generally termed underseepage; concentrated underseepage that carries sand up to the surface through a more or less open channel in the top stratum

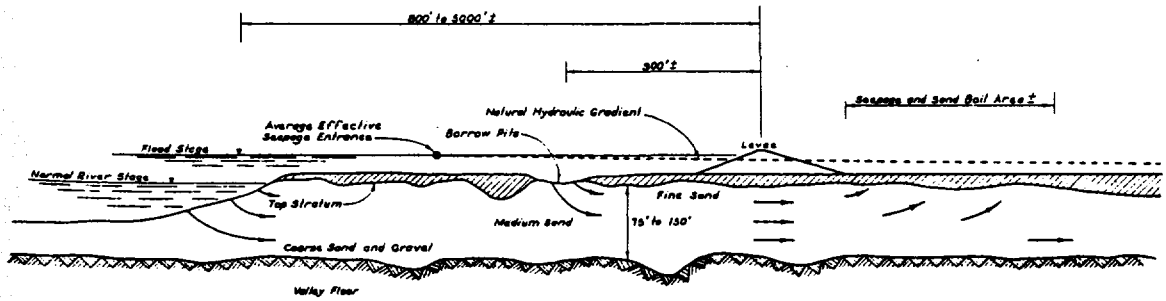

FIG. 2. CROSS SECTION OF LEVEE AND ALLUVIAL VALLEY

is called a sand boil. Piping is the active erosion of sand or other soil from under the top stratum as a result of substratum pressure and concentration of seepage in localized channels. Photographs of sand boils which have occurred along the Mississippi River levees from Alton to Gale are shown on plates 1 and 2.

16. If the hydrostatic pressure in the pervious substratum landward of a levee becomes greater than the submerged weight of the top stratum, the excess pressure will cause heaving of the top blanket, or will cause it to rupture at one or more weak spots with a resulting concentration of seepage flow in the form of sand boils. Where the foundation and top strata are heterogeneous in character, as is usually the case, seepage tends to appear at localized spots instead of causing the entire top stratum to heave or become "quick."

17. The hydraulic gradient required to cause heaving is called the "critical hydraulic gradient"; it is the ratio of the submerged unit weight of the soil to the unit weight of water, and is usually about 0.7 to 0.85. Any tendency for the hydraulic gradient to increase above the "critical" gradient only causes additional sand boils or increased percolation. In nature, high exit gradients and concentrations of seepage are usually found along the landside toe of the levee, at thin or weak spots in the top stratum, and adjacent to clay-filled swales or channels. In these cases the exit gradient may become "critical" at singular points, while the average gradient through the top stratum is still considerably less than the "critical" value.

18. Where seepage is concentrated to the extent that turbulent flow is created, the flow will cause erosion in the top stratum and development of a channel down into the underlying silts and fine sands which frequently exist immediately beneath the base of the top stratum. As the channel increases in size and/or length, there is a progressively greater concentration of flow into it, with a still greater tendency for erosion to progress beneath the levee. This is especially true if the top stratum is cohesive and the underlying soils are susceptible to erosion. Shrinkage cracks, root holes, and holes made by man or burrowing animals form ready channels in the top stratum for development of

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localized flows and attendant piping. Not only is underseepage a hazard from the standpoint of underground erosion, but it may also saturate and reduce the stability of the landside slope of a levee to such an extent that the slope may slough with attendant dangers to the levee.

19. If the landside stratum overlying the pervious foundation were completely impervious and there were no flow of water landward, the hydrostatic pressure head beneath the landside blanket would equal the river stage. This condition seldom exists, as there is generally some water flowing landward through the pervious foundation and upward through the surface stratum with a consequent landward decrease of pressure head (see fig. 2). This dissipation of head partially explains why sand boils have not occurred in some areas where, without such head loss, subsurface pressures during high-water stages would be sufficiently great to cause heaving or sand boils in the landside blanket. The reduction of the pressure beneath the top stratum as a result of natural seepage should not be considered as a guarantee against sand boils or subsurface erosion. Even with such reduction in pressure due to natural seepage, the residual substratum pressure may still be sufficient to cause sand boils and piping.

20. Characteristics of the top stratum landward of the levee that affect the underseepage problem in the general sense can be classified into the following three types:

- a. No significant top stratum.
- b. Top stratum of insufficient thickness to withstand the substratum pressures that will develop at the design flood stage.
- c. Top stratum of sufficient thickness to withstand any substratum pressures that may develop at the design flood stage.

21. Where the top stratum is very thin or nonexistent landward of the levee, seepage will emerge in the area adjacent to the levee toe even at moderately low heads; such seepage may also saturate the lower portion of the levee slope and cause it to slough. If the soil conditions are homogeneous, the amount of seepage and intensity of seepage pressure immediately below the surface will be uniform throughout the

seepage area. If the levee has adequate stability such seepage is probably not dangerous from the standpoint of subsurface erosion provided the levee has sufficient base width for the head involved to insure an acceptable escape gradient for the seepage at the toe of the levee. For this case, substratum pressures in the upper part of the sand at or landward of the levee toe are usually relatively low. However, natural stratification in the upper part of the sand may cause the development of some artesian pressure and concentration of seepage flow in the form of small, and probably insignificant, sand boils.

22. Cases b and c are those most representative of average conditions existing along the Mississippi River levees from Alton to Gale. Case b is potentially the most dangerous in that the resistance to seepage flow through the top stratum is so great in comparison to the carrying capacity of the substratum sands that appreciable artesian pressures are built up beneath the top stratum landward of the levee toe. (These artesian pressures above the ground surface at the levee toe frequently range from 25 to 75 per cent of the net head on the levee and may extend appreciable distances landward of the levee.) Where the hydrostatic pressure exceeds the weight of the top stratum along the levee toe or at some thin or weak point, the top stratum will rupture (frequently without warning) and allow a concentration of seepage at the point of rupture which creates a sand boil and a potential point for the initiation of piping. The development of a sand boil by rupture of the top stratum is more dangerous than development as a result of erosion through some previous or existing channel in the top stratum as there frequently is no prior indication or warning in the field. The quantity of seepage may not be significant until the top stratum is suddenly ruptured.

23. The third condition presents no underseepage problem except at localized spots where the top stratum may not be entirely continuous and locations where channels and drainage ditches have been excavated immediately landward of the levee toe. However, where such ditches are present, "critical" substratum pressures may rupture the bottom of the ditch and serious sand boils develop because the thick blanket on each

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side of the ditch will cause the subsurface seepage to concentrate in the form of sand boils in the bottom of the ditch.

24. The amount of underseepage and excess hydrostatic pressure that may develop landward of a levee is related to the river stage; location of effective seepage entrance; extent, thickness, and perviousness of the landside top stratum; carrying capacity of the pervious substratum; and geological features. (The effect of these foundation characteristics on underseepage and excess pressure is subsequently discussed in parts II and V to VII.) Other factors related to the seriousness of sand boils caused by seepage and excess hydrostatic pressure are the increase of river stage above that required to start piping, the degree of seepage concentration, the velocity of flow emerging from the boils or "pipes," and the existence of a stratum of fine cohesionless soil readily susceptible to piping. When the pressure head that has caused a sand boil is sufficiently dissipated by the increased flow through the foundation, erosion is usually slow. However, when there is enough pressure and the supply of water from the pervious strata is sufficient, piping may develop beneath the levee.

Methods of Control of Underseepage

25. Several methods may be used for controlling seepage beneath levees, such as landside berms, relief wells, impervious riverside blankets, sublevees, cutoffs, drainage blankets, and drainage trenches. The choice depends on a number of factors, including the character of the foundation, cost, permanency, availability of right-of-way, maintenance, and disposal of seepage water. Only landside berms and relief wells were considered practicable for control of deep underseepage along the levees in the St. Louis District. The manner in which all of the methods mentioned control underseepage is described in the following subparagraphs.

a. Landside berms. Landside berms have been used extensively for the control of underseepage. They strengthen the top stratum, increase the base width of the levee, reinforce the landward slope of the levee against sloughing during

high water, and move the point of emergence of seepage or sand boils and piping away from the levee proper. In the Alton to Gale Reach, landside berms were used along levees wherever they were the most economical control measure and where the top stratum landward of a levee was very thin or nonexistent so that relief wells would have had to be installed excessively close together. Landside berms were also used where a levee was relatively low, but yet a certain amount of protection against underseepage and possible sloughing of the landside slope appeared desirable.

b. Relief wells. One method of controlling underseepage is to tap the underlying pervious strata with a series of relief wells which provide pressure relief and controlled seepage outlets that offer little resistance to flow and, at the same time, prevent erosion of the foundation soils. This method has been used successfully by the Corps of Engineers at a number of dams and along numerous levees underlain by a pervious foundation. Relief wells offer an advantage as compared to gravel toes, pervious blankets, and other surface measures, where the foundation consists of stratified deposits of pervious materials, in that they penetrate the more pervious strata in which pressure relief is necessary. With proper spacing and penetration, relief wells will reduce hydrostatic pressures landward of levees underlain by impervious foundations for a wide range of seepage entrances, foundation stratification, and landward top strata. In addition, seepage which formerly emerged landward of a levee in an uncontrolled manner without relief wells will be materially reduced, although the total of well flow plus seepage with wells will be increased somewhat[7, 15]. The tendency for increased total seepage at low river stages was minimized by installing removable standpipes on the tops of the wells, as discussed later in this report.

c. Impervious riverside blanket. Frequently the natural top stratum riverside of levees has been removed as a result of borrow operations for construction of the levee, thereby exposing the pervious substratum and providing a ready source of seepage entry. No artificial impervious blankets were placed riverward of the levees as a means of seepage control. However, deep holes or channels scoured by the river in riverside borrow pits have been filled, either by dredged or hauled material, and abatis dikes have been constructed to promote silting and the growth of willows in the pits (see plates 263 and 264).

d. Sublevees. Areas known to be susceptible to dangerous underseepage may be encircled by one or more sublevees in which water can be impounded during high water to reduce

the net effective head acting in this critical area. Construction of sublevees requires considerable right-of-way landward of a levee, skillful operation during high water to insure proper balancing of heads to prevent dangerous sand boils or blowouts, adequate spillways and adequate section to insure that the sublevee itself will not fail during maximum flood stages owing to a head of water in the sublevee basin. For these reasons sublevees were not considered practicable as permanent underseepage control measures in the St. Louis District.

e. Cutoffs. Where practicable, the most positive method of underseepage control is to cut off all pervious strata beneath a levee by means of an impervious barrier which will eliminate both excess substratum pressures and the problem of seepage water landward of a levee. However, completely cutting off pervious strata 80 to 200 ft deep along extensive reaches of levees is not economically feasible. The installation of partially penetrating cutoffs will not reduce seepage and excess pressures significantly unless the cutoff penetrates 95 per cent or more of the pervious aquifer. This fact has been demonstrated by both mathematical and theoretical studies[4]. No deep cutoffs were attempted along the levees for these reasons; however, a few shallow cutoffs along the riverside toe of the levee have been constructed to cut off relatively thin layers of either natural levee or crevasse sands which lie immediately under the base of the levee and which in turn are underlain by more impervious strata.

f. Drainage blankets. Drainage blankets may be used for controlling underseepage where the levee is built on exposed sands and gravels of fairly homogeneous character. However, they are not effective for controlling seepage in deep substrata where the drain would be underlain by impervious top strata or where stratified fine sands would exist between the drain and the deeper, more pervious sand. No drainage blankets were used to control deep underseepage in the St. Louis District.

g. Drainage trenches. Drainage trenches may be used to control underseepage where the pervious foundation is of limited depth so that the trench will substantially penetrate the formation. Where the pervious foundation is deep a drainage trench of any practicable depth would attract only a small portion of underseepage, its effect would be local, and detrimental underseepage would bypass the trench. Because of the depth of the pervious substratum along the Mississippi River levees, drainage trenches were not considered feasible for this project.

PART II: GEOLOGY OF ALLUVIAL VALLEY

26. Detailed geological studies of several sites along levees in the Lower Mississippi Valley where underseepage had been a problem during previous high waters show a definite correlation between the distribution of alluvial deposits of sand, silt, and clay and the location and occurrence of underseepage and sand boils. Thus, before it was possible to make an intelligent investigation of underseepage and to design control measures, an understanding of the alluvial fill in the Upper Mississippi Valley from Alton to Gale was necessary. The geological studies made for this project included field reconnaissances, studies of aerial mosaics, and a review of logs of borings.

27. A few miles below St. Louis, the Mississippi River cuts across an eastward extension of the Ozark Plateaus and, for a distance of about 100 miles, flows in a narrow (3 to 10 miles wide), rock-walled, alluviated valley incised into ancient, predominantly limestone bedrock. The river emerges from the plateaus through a narrow gorge known as Thebes Gap into the broad Lower Mississippi Valley. The depth of the alluvial deposits in the St. Louis District is rather variable, ranging from about 75 to 200 ft with an average depth of about 125 ft.

The Alluvial Fill

28. Melting of the glaciers during late or postglacial times caused the entrenched valley to become filled with a series of sandy gravels, sands, silts, and clays which can be grouped into two broad units: (a) a sand and gravel substratum, and (b) a fine-grained top stratum. As these units are of basic importance to the underseepage problem, they are described in some detail below.

The pervious substratum

29. During the gradual rise of sea level accompanying the final retreat of glacial ice, the Mississippi River became an overloaded braided stream in which large quantities of gravel-bearing sands were deposited. As sea level continued to rise and the deposits on the

entrenched valley floor continued to thicken, stream slopes were progressively reduced and both the quantity and grain size of the material transported by the river decreased. The gravel-bearing sands were succeeded by coarse sands grading upward into progressively finer materials and terminating in deposits of very fine sand. This upward gradation from coarse to fine materials reflects the gradual adjustment between the transporting capacity of the river and the available load. Fine to very fine sands were not deposited until sea level had reached essentially its present stand and the river began to change from a braided to a meandering stream. The substratum sands generally vary in thickness from about 75 to 150 ft, are quite pervious, and have a high seepage-carrying capacity as subsequently discussed.

30. Excavation for the Chain of Rocks lock into the East St. Louis levee district completely penetrated the alluvial fill. Examination of this excavation revealed the pervious substratum to be composed of sands varying in gradation from fine to coarse. Boulders and cobbles of glacial origin were interspersed throughout the sand, and occasional lenses of cobbles were found wherein the interstices between the cobbles were filled with coarse sand and pea gravel. Fairly continuous strata of cobbles and gravel were found to overlie the fissured limestone bedrock. The interstices between these cobbles and gravel were also filled with coarse sand and pea gravel.

The top stratum

31. The predominantly fine-grained top stratum deposits have been laid down by the Mississippi River as it meandered across the flood plain. Meander belt deposits are characterized by marked lateral and vertical discontinuities, and by wide variation in grain size and permeability. They are therefore of primary importance in the localization of underseepage and the design of control measures. Meander belt deposits may be grouped as point-bar, channel-fill, natural levee, and backswamp deposits. Such deposits can usually be identified in the field and on aerial mosaics.

32. Point-bar deposits. As meander loops increase in radius by erosion, deposition takes place on the inside bank where low sandy ridges

are built up with intervening elongated depressions which usually become filled with fine-grained deposits. Ridges and swales formed in this manner are known as point bars. Long sandy ridges separated by clay- and silt-filled depressions more or less paralleling the margins of former river courses are also prominent features of shifting river valleys and are especially common in the St. Louis District. These features appear to have been formed in migrating channels rather than on points and might well be referred to as channel bars and slough fillings. The upper part of these ridge-and-swale deposits are usually covered with clay silts and silty clays which are laid down during gradual migration of the river channel from its former banks. The slough fillings normally are about 10 to 20 ft deep.

33. Channel-fill deposits. When the river abandons its former course as a result of a cutoff, the central and lower portions of the old cutoff channel usually become filled with silts and clays which are relatively impermeable. Such deposits in the area studied may be 40 to 80 ft deep.

34. Natural levees. When the river overtops its banks the water spreads out, the velocity is checked, and deposition of a portion of the sedimentary load results. In this manner, long ridges known as natural levees are formed on the outside of meander loops and along both banks in straight reaches. Natural levee deposits found in the Upper Mississippi Valley appear to be mainly sandy in composition and are seldom over 200 yd wide and about 5 ft high. Clearly distinct natural levees are relatively rare in the St. Louis District.

35. Backswamp deposits. Low-lying areas on the landside of natural levees are known as backswamps. These areas receive only quiet flood waters and, as a result, the sediments deposited consist of fairly uniform silts and clays. Backswamp deposits create an almost impervious block to the emergence of subsurface seepage. There do not appear to be many, if any, true backswamp deposits in the St. Louis District; however, some thick clay top stratum deposits that resemble backswamp deposits were found adjacent to the valley walls.

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Influence of Geology on Underseepage

36. The emergence of seepage landward of a levee is influenced by: (a) configuration of geological features such as swale fillings and clay plugs and their relation to the levee; (b) characteristics and thickness of the top stratum; (c) cracks and fissures formed by drying or other natural causes; (d) borrow pits, post holes, seismic shot holes, and other works of man; and (e) decay of roots, uprooting of trees, animal burrows, crayfish, and other organic agencies. The severity of underseepage along a reach of levee is frequently dependent upon the configuration of geological features in the area, as discussed in the following subparagraphs.

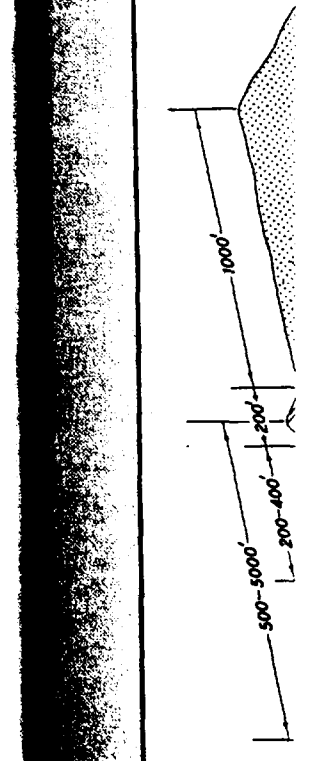
a. Point-bar deposits. Point-bar deposits are the most heterogeneous of all the materials forming the top stratum and are the predominant type of surface deposit along the Mississippi river-front levees from Alton to Gale. Fine-grained swale fillings of highly variable thickness separating sandy ridge deposits are the chief discontinuities found in point-bar materials. Inasmuch as the ridges are considerably more pervious, practically all of the underseepage flows up in the ridges regardless of the orientation of the swales to the levee. The greatest concentration of seepage always occurs along the edges of swales and at the landside levee toe, as illustrated in fig. 3. In instances where the long dimension of the swales is parallel to or intersects the levee toe at a small angle, seepage is particularly concentrated in the sandy ridges where the edges of the swales intersect the levee toe (see fig. 4). Sharp inside angles or bends in levee alignment cause a greater concentration of flow, as a three-dimensional flow pattern results with all flow lines directed toward the angle at the landside toe.

b. Clay plugs and channel fillings. Clay plugs and channel fillings differ from swale fillings mainly in their greater width and thickness. They are also generally longer, and are often somewhat more uniform in composition. Their effect on the distribution of underseepage is similar to that of swale fillings but, owing to their greater thickness and width, is considerably accentuated (see figs. 5 and 6). The worst possible underseepage condition is found where a deep or wide clay-filled channel exists parallel to and immediately landward of the levee toe. In a case of this type, all of the underseepage is concentrated in the small area between the levee toe and

the clay plug. However, clay-filled channels that parallel the levee reduce underseepage if the landside toe of the levee overlaps the filled channel. Such channels immediately riverward and overlapped by the levee also tend to reduce underseepage by forming a relatively impervious riverside blanket.

c. Natural levees. Where the levee is founded on top of a continuous natural levee deposit of silts or silty sands underlain by clay, the natural levee deposit will act as a semipervious aquifer for shallow underseepage. However, such deposits do add to the weight of the underlying clay and thereby help resist lifting of the top stratum by excessive substratum pressures in underlying clean sands. Sandy or silty crevasse fillings may also act as seepage channels, but because of their narrow cross section and irregular shape any underseepage originating in these fillings is very restricted in distribution.

d. Backswamp deposits. As previously mentioned, these materials are generally thick and practically impervious, and no significant underseepage is known to penetrate them. However, if for any reason the continuity of backswamp clays is broken, or their thickness sufficiently reduced by excavations such as drainage ditches or borrow pits, underseepage and possibly sand boils may be expected to develop.



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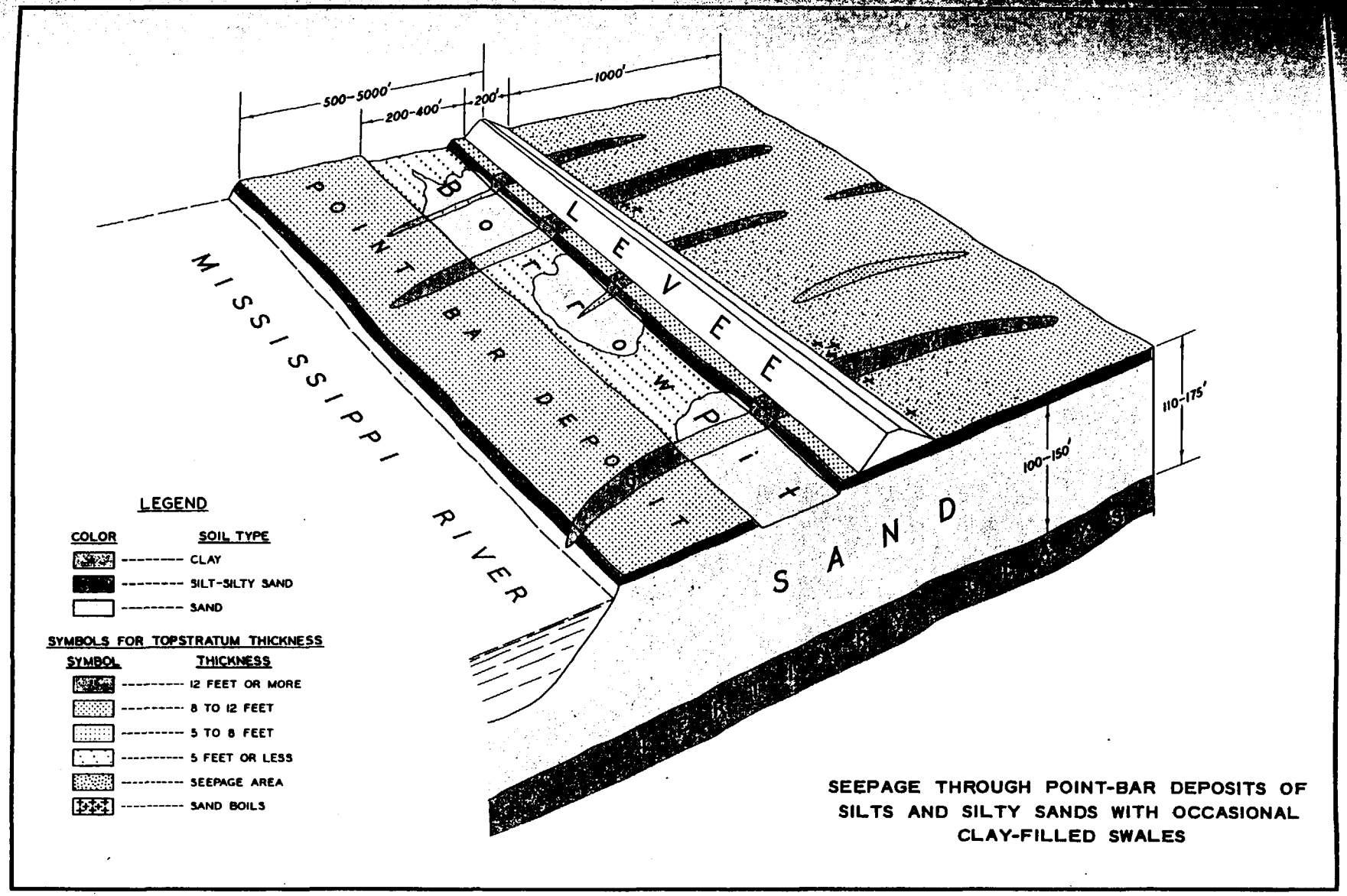
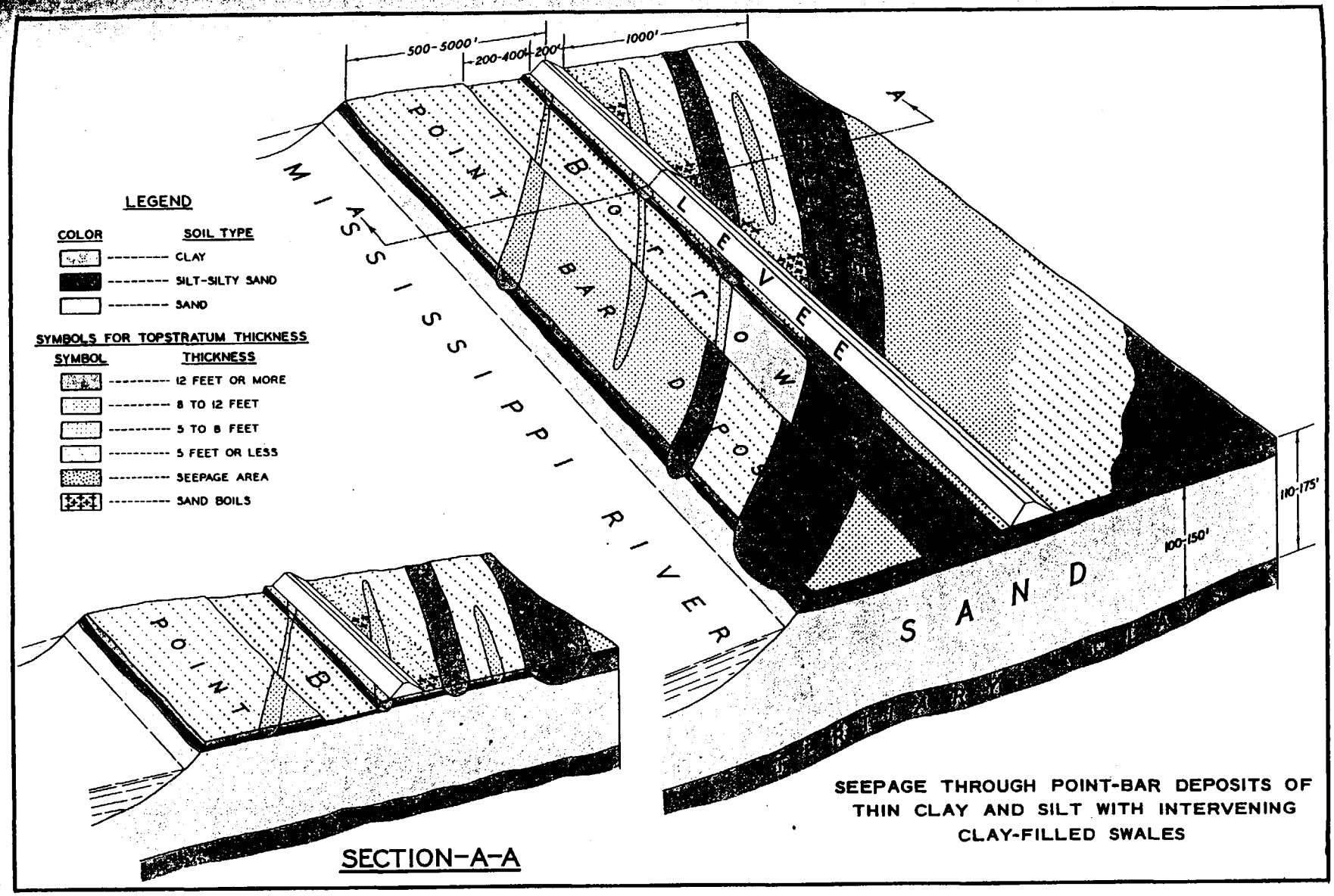


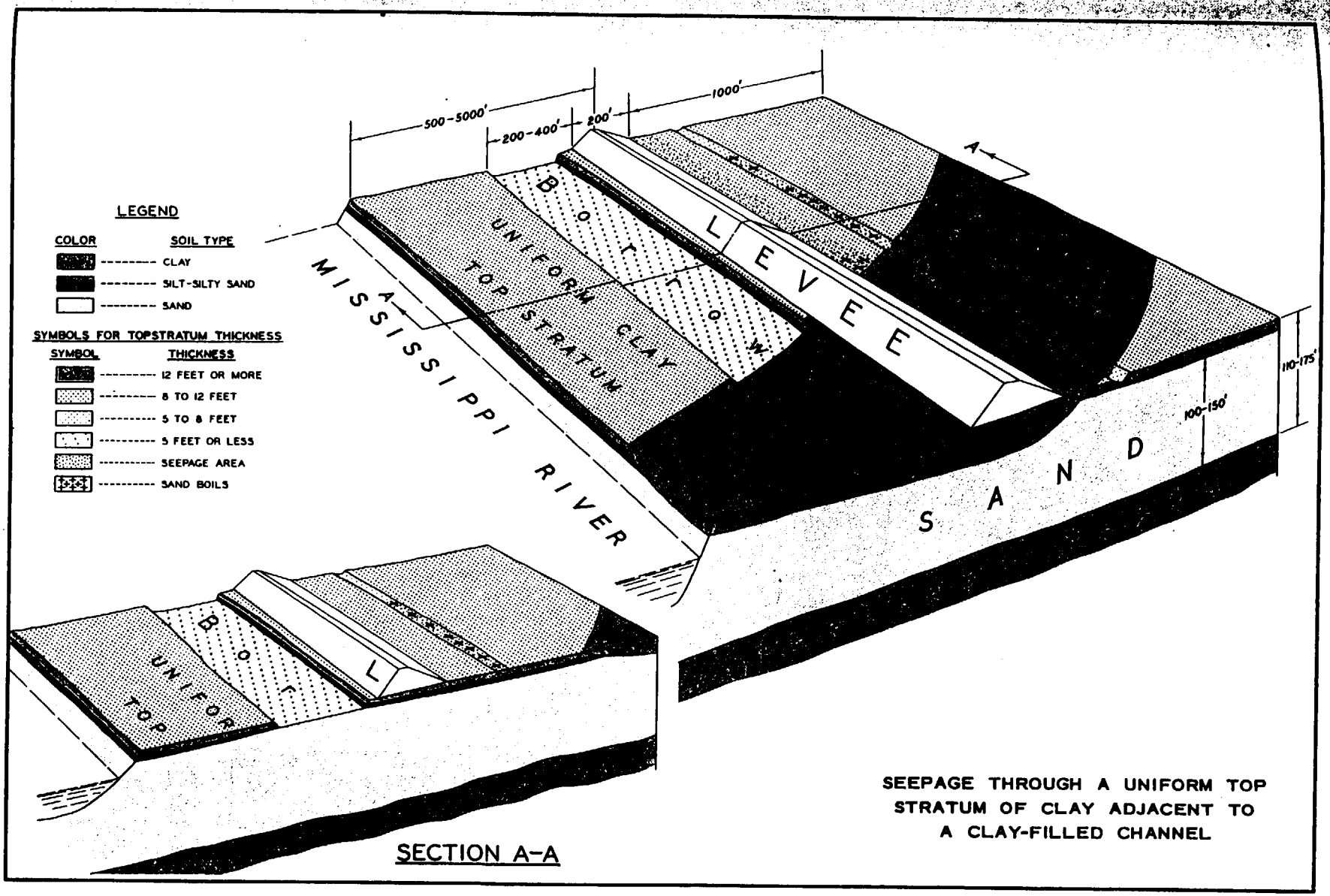
SEEPAGE THROUGH POINT-BAR DEPOSITS OF SILTS AND SILTY SANDS WITH OCCASIONAL CLAY-FILLED SWALES

FIGURE 3



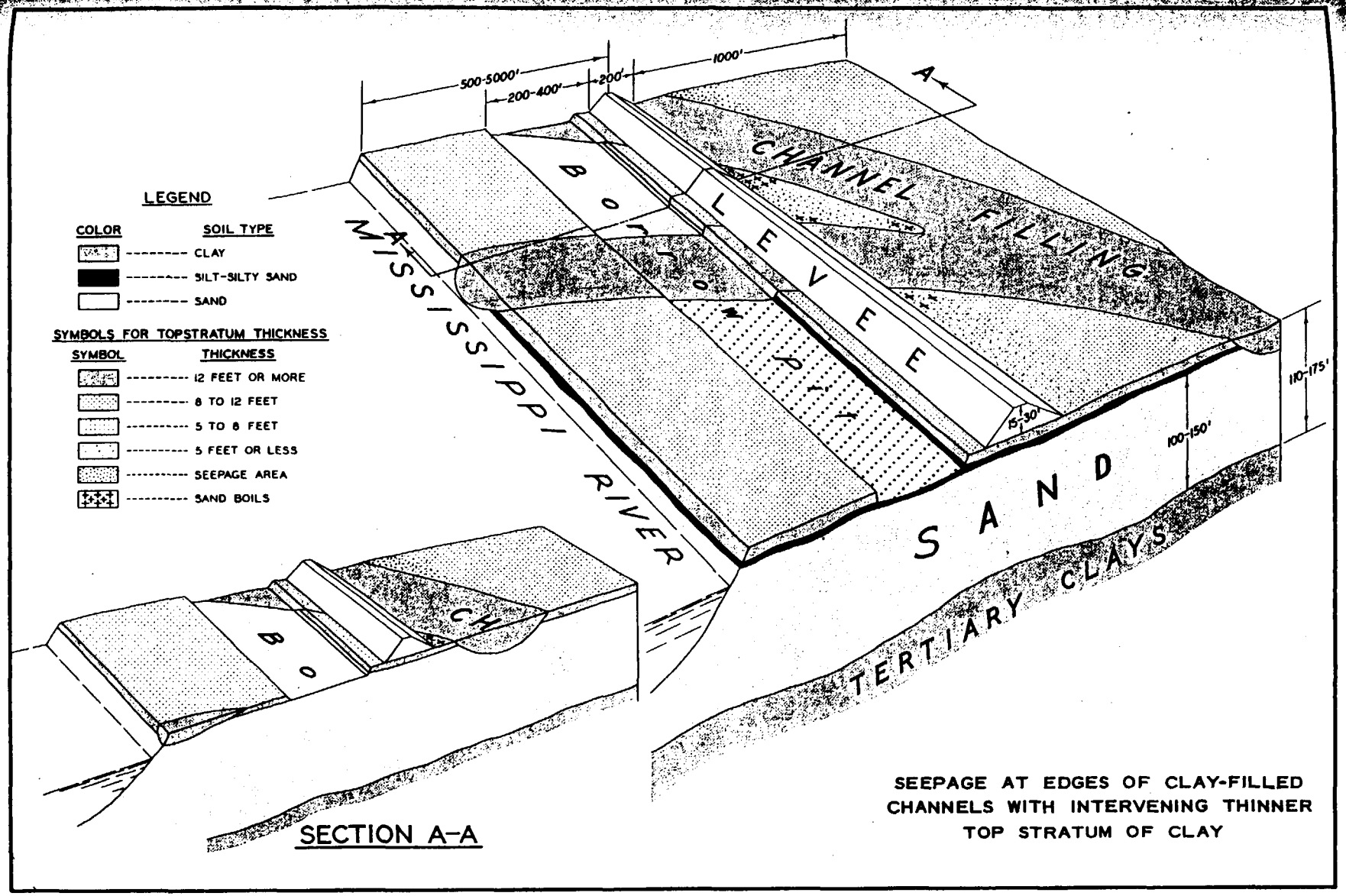
SEEPAGE THROUGH POINT-BAR DEPOSITS OF THIN CLAY AND SILT WITH INTERVENING CLAY-FILLED SWALES

FIGURE 4



SEEPAGE THROUGH A UNIFORM TOP STRATUM OF CLAY ADJACENT TO A CLAY-FILLED CHANNEL

FIGURE 5



SEEPAGE AT EDGES OF CLAY-FILLED CHANNELS WITH INTERVENING THINNER TOP STRATUM OF CLAY

FIGURE 6

PART III: FIELD INVESTIGATIONS

37. An extensive field investigation was made to ascertain the need for underseepage control measures and to obtain data on which to base their design. The field investigation consisted of a reconnaissance of all areas adjacent to the levees, a study of aerial mosaics, an intensive boring program, a geophysical investigation to determine the depth of the alluvial valley, a surveying program, and special field pumping tests.

Field Reconnaissance

38. Reconnaissances were made along the levees for the purpose of noting geological features, such as swales and sloughs and their orientation with respect to the levee, that might affect the underseepage problem. The locations and extent of man-made features such as drainage ditches, stock ponds, and borrow pits were ascertained. The layout of new borings, which had been made in the office on the basis of available soil data, levee heights, and a study of aerial mosaics, was checked in the field to insure adequate exploration of the features that would affect the underseepage problem and the design of control measures. The condition and depths of riverside borrow pits also were noted.

Aerial Mosaics and Maps

39. As no complete maps of the new levee system were available to which the new borings, piezometers, and control measures could be referenced, a new folio of air-photo maps was prepared showing all Mississippi River levees and flood-control works in the area. These maps will be used during flood-control operations and to record underseepage observations during future floods.

Boring Program

40. At the outset of the investigation, the existing soils

information was reviewed and a supplementary boring program laid out. The additional borings consisted largely of shallow (or blanket) borings and deep borings made with a split-spoon sampler. A few deep borings were made with a Shelby tube sampler and some with a bailer. Borings made for this investigation are generally designated by the prefix "W," except for certain deep borings designated by the prefix "C." Borings made while installing piezometers are designated by the prefix "P." The logs of the new borings and the pertinent borings made prior to the beginning of the investigation are included in volume II.

41. Brief descriptions of the types of borings made are given in the following subparagraphs.

a. Shallow borings. Shallow borings were made with hand augers to determine the thickness and nature of the soils comprising the top stratum or blanket at the landside toe of the levee or berm. Such borings were also made to define the soil conditions in swales, sloughs, borrow pits, and ditches that were sufficiently close to the levee to affect underseepage.

b. Deep borings. Comparatively deep split-spoon borings were made to obtain samples from which the gradation of the sands comprising the pervious substratum could be determined in levee reaches wherein it was apparent that seepage control measures were required. These borings ranged in depth from 50 ft to depths sufficient to encounter bedrock at the base of the pervious substratum. Additional deep split-spoon borings were later made at numerous well locations in each levee district prior to installation of the wells to obtain data for designing the length of well screen and riser pipe. These borings are designated by the prefix "C." The split-spoon borings were advanced with a fishtail bit and samples were taken at 5-ft intervals and/or at strata changes. Drilling mud was used to keep the bore holes from caving. At the beginning of the investigation, 1-3/8-in.-ID split-spoon samplers were used; later the size of the sampler was changed to 2-in. ID so that larger gravel sizes could be obtained. As most of the deep borings were made with these relatively small split-spoon samplers, gravel and cobbles larger than 1 or 1-1/2 in. were usually not recovered and thus logs of the borings generally do not show the existence of large gravel or cobbles which bailer borings and well drilling operations have shown to exist in the lower portion of the sand aquifer.

c. Undisturbed borings. A limited number of both shallow

and deep borings were made to obtain undisturbed samples of top stratum soil and of the sands comprising the pervious foundations for observing stratification and making permeability tests. Five- and three-in.-diameter Shelby tube samplers were used to obtain samples of the blanket materials and foundation sands, respectively. Where necessary, these bore holes were kept open by use of drilling mud.

d. Bailer borings. Shortly after the initiation of well installation under the first relief well contract, it was found that several wells could not be installed to the required depth because of the presence of cobbles. The mud sumps at the wells also revealed considerably coarser gravels and cobbles than were obtained with the small diameter split-spoon samplers. In order to obtain an indication of the size of coarse gravel and cobbles present in the foundation, a few large diameter (8 to 10 in.) borings, advanced with steel casing and a piston-type bailer, were made. These borings have been designated by the suffix "B." The samples so obtained contained cobbles as large as 7 in. long. Mechanical analyses of samples obtained with a large diameter bailer are discussed in paragraph 52 and are plotted on plate 251.

Geophysical Investigations

42. Because of the difficulty and cost of ascertaining the depth to rock by means of borings, most of this information was obtained by means of geophysical explorations.

43. At first the depths to rock were determined by the electrical resistivity method. In analyzing the data obtained, it became apparent that the depth to rock determined by this method was extremely sensitive to the method of interpretation and did not check with soils depths determined by borings. As a result the resistivity method was abandoned in favor of the seismic method.

44. Initial checks were made with a seismograph at locations where the depth to rock had been determined from deep borings and excavation for the Chain of Rocks Canal Lock, and good agreement was obtained. Therefore, the seismic method was used for the remainder of the investigation.

Surveying Program

45. The ground profile immediately landward of the toe of the levee or berm was obtained from existing cross sections or new level profiles. Some cross sections were made of borrow pits, landside low areas, ditches, etc. Cross sections were obtained landward from the levee toe at all piezometer lines and also where seepage berms were designed.

Special Field Pumping Tests

46. Special field pumping tests were performed on three fully penetrating wells installed in the Harrisonville and Ft. Chartres levee districts to determine the variation in permeability of the aquifer with depth, the in-situ horizontal permeability of individual sand strata, and the actual length of well screen required to achieve an effective penetration of 50 per cent of the aquifer. Special pumping tests also were performed on two fully penetrating wells in the East St. Louis levee district to determine the over-all permeability of the aquifer so that the flow from relief wells and the capacity of pumping stations to pump well flow over the levee could be computed more accurately.

47. The pumping tests in the Harrisonville and Ft. Chartres levee districts showed the pervious substratum at the three sites tested to have a permeability of 1500 to 3000 x 10^{-4} cm per sec and that the screen portion of a well should penetrate about 60 per cent of the principal pervious substratum on a depth basis to have an "effective" penetration of 50 per cent, which was assumed in the design of relief wells. A correlation between D_{10} and k_H was also obtained from these tests. The procedure for and results of the above special pumping tests are presented in detail in part VIII.

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PART IV: LABORATORY INVESTIGATIONS

48. Laboratory studies included visual classification of all samples, mechanical analyses, and permeability tests on numerous samples of the pervious substratum, a few permeability tests on undisturbed samples of the top stratum, and filter tests on the gravel filter used around the well screens.

Classification

49. Samples were classified visually and, when in doubt, by means of mechanical analyses, in accordance with the unified soil classification system used by the Corps of Engineers*, and appendix A.

Grain-size Determinations

50. Mechanical analyses were made on a few silty and sandy samples of the top stratum whose classification was questionable, and on numerous samples of the substratum sand for the purpose of determining the effective grain size D_{10} and the D_{85} size. These values were used in delineating the principal seepage-carrying stratum (sands with $D_{10} > 0.15$ mm) in which slotted sections of well pipe were set and strata of very fine sand or silt in which nonslotted sections of pipe were set. The D_{10} sizes of the samples analyzed are shown on the logs of borings in volume II.

51. Grain size and gradation of samples obtained by split-spoon samplers are not always representative of the size and gradation of strata in which appreciable quantities of gravel and cobbles exist, because of inability of gravel and cobbles larger than 1 to 1-1/2 in. to enter the sampler. On the other hand, the finer fraction of well-graded sands and gravels is not always recovered when a bailer sampler is used, as evidenced by the lack of silt and very fine sand in bailer samples

* Waterways Experiment Station, The Unified Soil Classification System, Technical Memorandum No. 3-357. Vicksburg, Miss. (1953).

from strata known to contain such. The gradation of typical samples of the pervious substratum obtained by split-spoon and bailer sampling is shown on plate 251.

52. Mechanical analyses of split-spoon samples from deep borings show that the foundation sands are stratified and are comprised primarily of strata of relatively uniformly graded sands with occasional strata of well-graded sands. Foundation sands tend to be coarser with depth. Mechanical analyses of samples taken with a 6-in. bailer show the existence of gravel and cobbles in the lower portion of the alluvial fill (see plate 251).

Permeability Tests

53. In the earlier phases of the investigation, permeability tests were conducted on selected undisturbed samples of the top stratum, on remolded samples of the foundation sand obtained with either a split-spoon or thin-walled Shelby tube sampler, and on samples obtained with a bailer. The coefficients of permeability obtained were adjusted to a temperature of 20 C. The values obtained from remolded samples were also adjusted to the estimated natural void ratio of the material, using the following relationship:

$$k_n = k_R \frac{e_n^2}{e_R^2} \qquad (1)$$

wherein

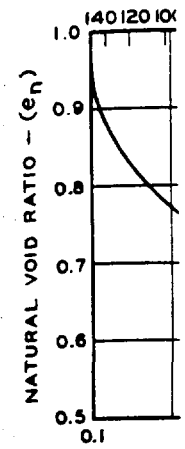
e_R = void ratio of the test specimen

e_n = natural void ratio of soil

k_R = measured coefficient of permeability of the test specimen at the test void ratio

k_n = coefficient of permeability of the test specimen at the natural void ratio.

The natural void ratio of the soil e_n was estimated from the D_{50} of the soil and fig. 7.[8] Coefficients of permeability as determined from laboratory tests on split-spoon samples of the alluvial sands ranged



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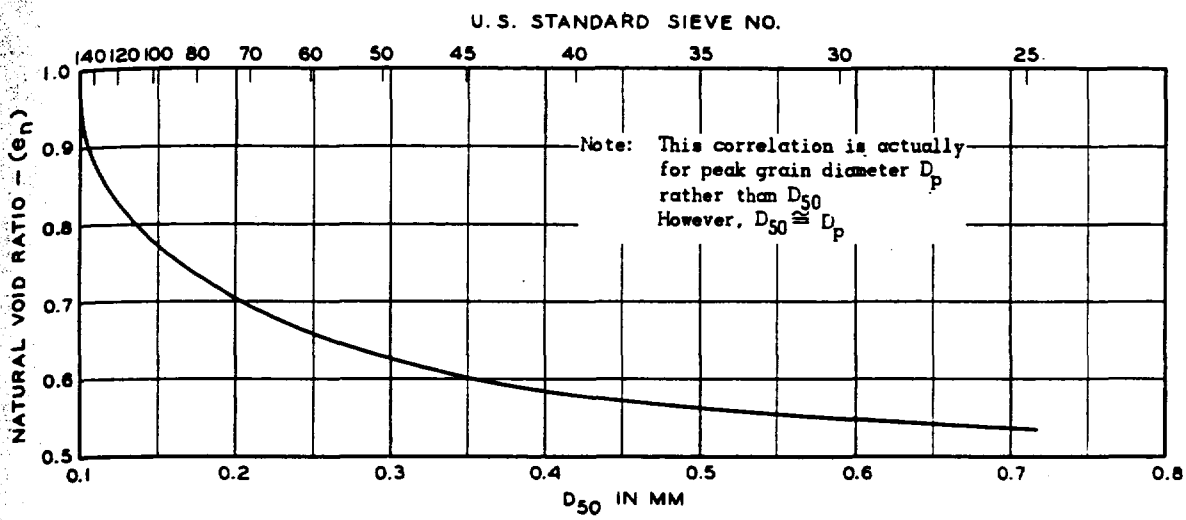

FIG. 7. NATURAL VOID RATIO OF SANDS VS 50 PER CENT SIZE (D_{50})

from about 20 to 2000 x 10^{-4} cm per sec. It was necessary to wash all split-spoon samples thoroughly before performing any permeability tests because they were contaminated with drilling mud. Some contamination of samples taken with a Shelby tube in a mudded hole was also noted.

54. The relationship obtained between k_n and D_{10} is shown in fig. 8. The scatter in the data is attributed in part to wide variations in the coefficient of uniformity in the samples. The permeabilities of bailer samples tended to exceed somewhat those for split-spoon samples with the same D_{10}. The values of permeability obtained in the laboratory were considerably lower for the same D_{10} than those obtained from field pumping tests (part VIII) and are not considered indicative of in-situ permeabilities. After the field pumping tests were performed, the laboratory permeability tests were discontinued.

55. Laboratory permeability tests were also conducted on a few undisturbed samples of the top stratum. A wide range of vertical permeabilities k_b was obtained as indicated in the following tabulation.

Soil Type	Number of Samples	k_b x 10^{-4} cm per sec Maximum	Minimum	Average
Clay	9	6	0.004	1
Silt	4	10	0.4	5
Silty sand	7	34	0.8	15

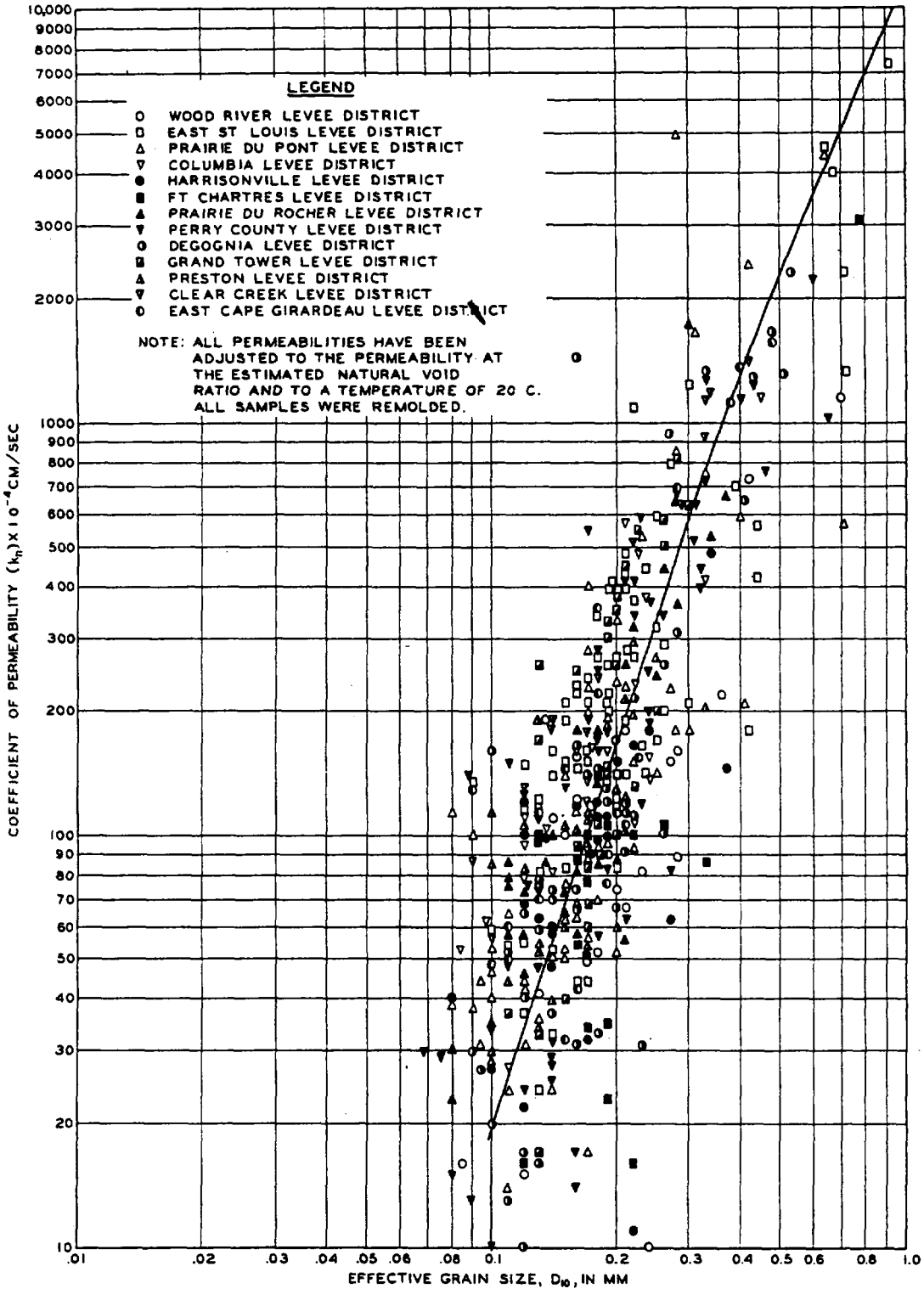

FIG. 8. COEFFICIENT OF PERMEABILITY VS EFFECTIVE GRAIN SIZE FROM LABORATORY PERMEABILITY TESTS

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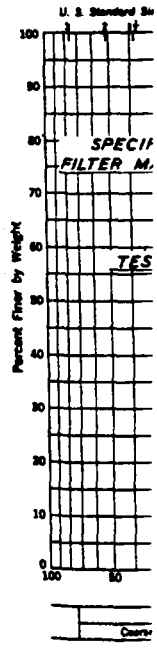


FIG.

Although the averages of the above test results are approximately what were used in analytical and design studies subsequently discussed, laboratory permeability test results are usually not indicative of the in-situ permeability of top stratum soils, particularly clays.[11]

Filter Tests

56. Tests were conducted to determine the ability of the gravel filter specified for relief wells to prevent the foundation sands from piping into the well. The filter material tested was obtained from piles of filter material used for relief wells installed in the Harrisonville levee district under the first relief well contract. The filter sample tested was on the coarse side of the specified gradation band. The thickness of filter tested was 6 in., the minimum required in the field. The gradation of the gravel filter used for the filter tests and specified for the relief wells is shown on fig. 9. The gravel filter was placed in the permeameter under water by means of a small tremie.

57. Three fairly uniform sands typical of the finer foundation sands encountered in the pervious aquifer, with D_{85} values of 0.20,

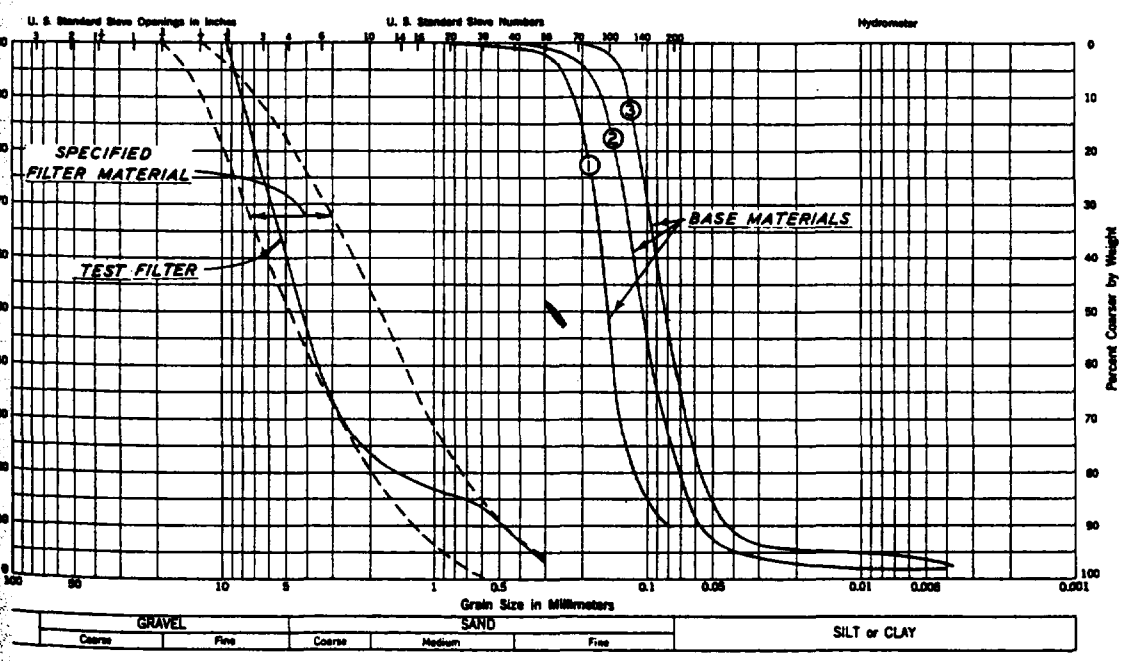


FIG. 9. GRADATION OF FILTER AND BASE MATERIALS, AND SPECIFIED FILTER GRADATION

0.15, and 0.12 mm, were used in the tests. The grain-size curves for these sands are shown in fig. 9. The sands were placed in the permeameter at void ratios about equal to the estimated natural void ratios in situ. The height of the sand was about 1-1/2 in. Except for the test on sample 1, thin annular strips of modeling clay were placed in the permeameter on 1-in. centers throughout the height of the filter material to prevent piping along the walls of the permeameter.

58. After placing the sand and filter, a flow of water was induced through the permeameter under a very low gradient and was maintained for a 3- to 5-min period. During this time the voids in the filter material were checked very closely to see whether sand was piping into or through them. If no visible detectable movement resulted under the initial flow, the gradient was increased slightly and the above procedure continued until failure occurred. Gradients under which the various combinations failed, as shown in the tabulation below, ranged from about 10 to 16.

Properties of Base Materials Tested and Results of Filter Tests

Sample	D_{85} mm	$\frac{D_{60}}{D_{10}}$	$\frac{D_{15F}}{D_{85B}}$	$\frac{D_{50B}}{D_{50B}}$	$\frac{D_{15F}}{D_{15B}}$	Gradient at Failure
1	0.20	2.1	3.8	27	7.5	8.7*
2	0.15	2.1	5.0	40	11.5	16.3
3	0.12	2.1	6.3	51	14.7	10.3

* Sample failed by piping along walls of permeameter.

59. The test results indicate that the filter material satisfactorily protected the various sands tested up to gradients of about 10. The results of the special pumping tests on wells installed in Harrisonville and Ft. Chartres levee districts show that the maximum gradient at the outer periphery of the well screen that was measured during the pumping test, or would exist during normal operation, was about 1. It was concluded, therefore, that if the filter material is satisfactorily placed and has a minimum thickness of 6 in., it should protect any sand with a D_{85} coarser than 0.12 mm. However, in general, well screens

were not set in sands with $D_{85} < 0.25$ mm, to provide a factor of safety against piping sand into the well.

60. The coefficient of permeability of the filter gravel was found by laboratory tests to be about 4000 to 5000 x 10^{-4} cm per sec.

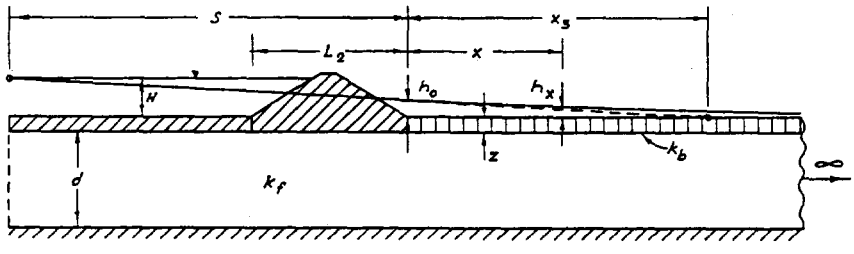
PART V: SEEPAGE ANALYSES

61. The need for seepage control measures was based on a study of the thickness and characteristics of the top stratum landward of the levee, the thickness and permeability of the pervious substratum, the maximum possible net head on the levee, and the source of seepage. If the hydrostatic head at the landside toe of the levee computed from these factors and the following formulas was greater than 0.85 (0.67 in the East St. Louis levee district), some seepage control measures were considered warranted; if between 0.67 and 0.85 (0.50 and 0.67 in the East St. Louis levee district), check piezometers were installed; and if less than 0.67 (0.50 in East St. Louis levee district), no piezometers or seepage control measures were considered necessary.

62. Determination of the factors involved in the above evaluation are discussed in the following paragraphs. Additional data and formulas for evaluation of the factors can be obtained from reference 11.

Factors Involved in Seepage Analyses

63. The amount of seepage that will pass beneath a levee and the artesian pressure that will or can develop landward of a levee during a sustained high water are related to and can be estimated from a knowledge of the following factors and foundation characteristics. (The nomenclature used in this section of the report is illustrated in fig. 10,



Head beneath top stratum at landside toe of levee ..... $h_o = H \frac{x_s}{S + x_s}$ $\qquad x_s = \frac{1}{c}$

A factor ..... $c = \sqrt{\frac{k_b}{k_f\, z\, d}}$ $\qquad h_x = h_o\, \epsilon^{-cx}$ $\quad (\epsilon = 2.718)$

FIG. 10. GENERALIZED CROSS SECTION OF LEVEE FOUNDATION AND SYMBOLS FOR SEEPAGE ANALYSIS

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and defined in the "Notations" preceding the text.)

a. Net head H on the levee.

b. Effective thickness z and vertical permeability k_b of the top stratum both riverward and landward of the levee.

c. Effective thickness d and horizontal permeability k_f of the pervious substratum.

d. Ratio of the horizontal permeability of the pervious substratum to the vertical permeability of the top stratum k_f/k_b .

e. Distance from landside toe of the levee to the "effective" source of seepage entry into the pervious substratum s .

f. Distance from the landside toe of levee or berm to the effective seepage exit x_3 .

g. Critical gradient i_c through the top stratum at landside toe of the levee.

64. It is pointed out that few of the above variables which enter into evaluation of the need for underseepage control measures or their design can be determined with any precise degree of accuracy. Values for a number of the factors are dependent to a considerable extent upon opinions and experience of the designing engineer. In computing or estimating the numerical values for the above variables, an effort was made to utilize all formulas, field data, and experience available. These data include geological studies and borings made along the levee, data and observations obtained at piezometer installations along levees in the Lower Mississippi River Valley during the 1950 high water, results of model studies, and data obtained from full-scale relief well installations along levees in the Lower Mississippi River Valley. The numerical values used in the investigation are considered to be representative of average conditions with possibly some slight conservatism. They do not encompass all conditions that may be encountered along any given reach of levee, nor are they limiting conservative values. An over-all factor of safety of about 1.25 to 1.5 was applied in the design of underseepage control measures.

65. Before seepage analyses can be made by means of formulas, it is necessary to make certain simplifying assumptions and to generalize

the actual foundation, as shown in fig. 2, into a pervious sand stratum with a specific thickness and permeability and a semipervious top stratum with a uniform thickness and permeability, as shown in fig. 10.

66. Seepage may enter the pervious substratum either at the riverbank, through riverside borrow pits, and/or through the semipervious top stratum riverside of the levee. Seepage through the pervious foundation was assumed to be horizontal and laminar. Flow through the top stratum or bottom of borrow pits was assumed to be vertical and laminar. The levee, existing landside berms if any, and the portion of the top stratum immediately beneath the levee and berm were assumed to be impervious.

67. Methods of arriving at the numerical values of the factors involved in seepage analyses and design of seepage control measures are discussed in the following paragraphs.

Net head H

68. The net head on a levee is the height of the flood stage above the ground surface or tailwater landward of the levee. For evaluation of the need for and design of seepage control measures along river-front levees, the flood stage was assumed equal to the net grade of the levee. The net grade of the levees is 2 ft above the project flood stage. The project flood stage in the Wood River, East St. Louis, and Prairie du Pont levee districts is based on a frequency of once in 200 years; in the agricultural levee districts the project flood is based on a frequency of once in 50 years. Along flank levees, the flood stage was taken as the elevation of the net grade of the river-front levee at the intersection of the flank and river-front levees. It was considered that high stages along tributary levees as a result of floods on these streams would be of such short duration that uplift pressures would not be significant because of time lag.

Effective thickness z and permeability k_b of top stratum

69. The thickness of the top stratum both riverward and landward of levees was determined by means of auger borings. As discussed in part II, the top stratum is seldom a single stratum of one type soil; instead it usually consists of several layers of different soils.

70. For this project the top stratum was transformed to one with a single thickness and permeability by multiplying the thickness z_n of each layer comprising the top stratum by a transformation factor to obtain the transformed thickness of each stratum, and then summing up the transformed thicknesses z_{b-n} of each stratum.

71. Where the most impervious soil was at the ground surface, the effective thickness was computed by transforming the various components of the top stratum to one thickness of a single permeability as shown in Case I of fig. 11. If semipervious material overlay less pervious

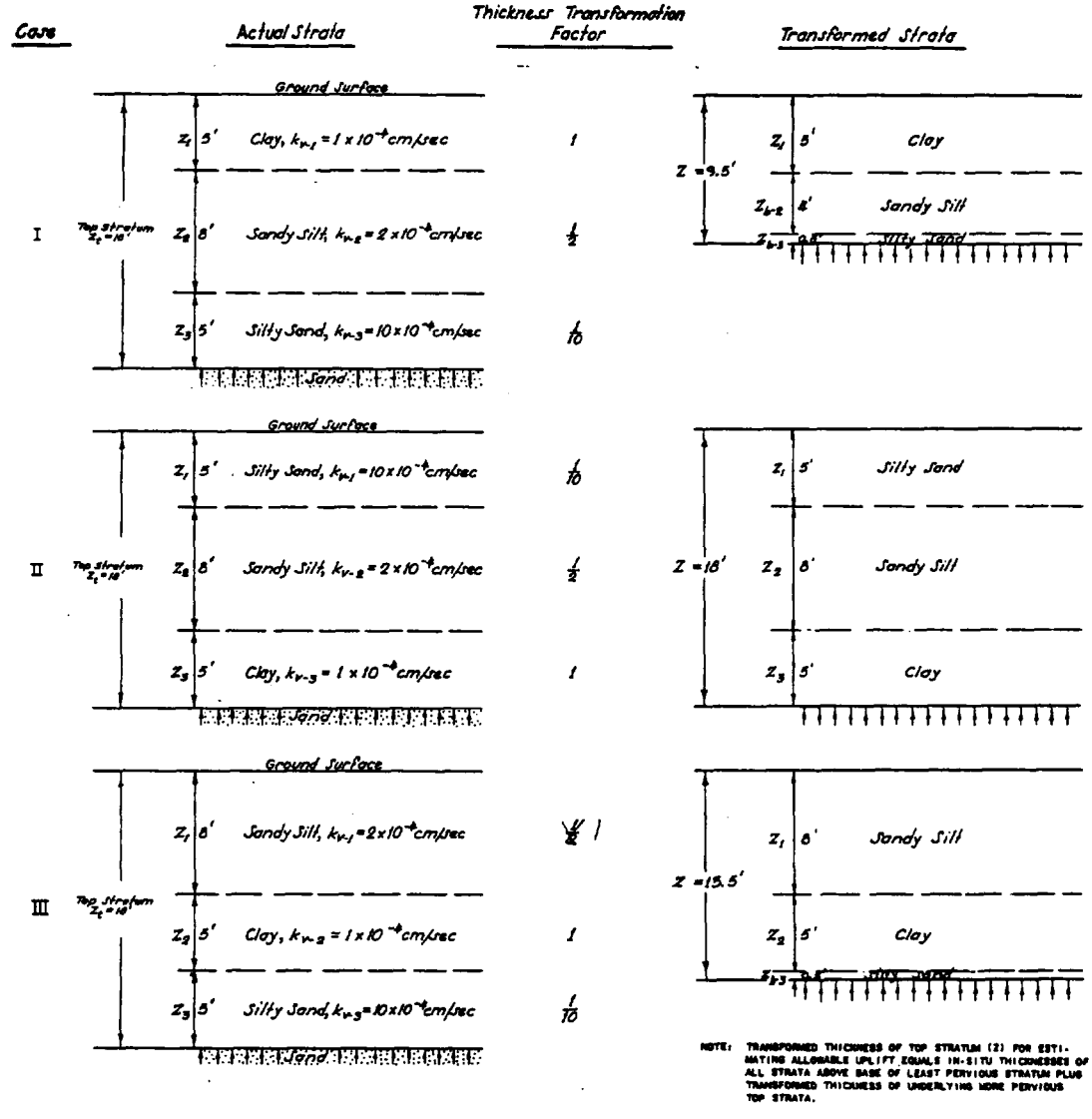


FIG. 11. TRANSFORMATION OF TOP STRATA

material, the effective thickness was taken as the total thickness above the bottom of the less pervious stratum (Case II, fig. 11). Where the top stratum was composed of alternating layers with varying perviousness, the thickness was based on the total thickness of the various layers overlying the base of the least pervious stratum plus the transformed thickness of the underlying more pervious top strata (Case III, fig. 11).

72. The thickness transformation factors used to transform the landside top stratum were agreed upon by the Advisory Board on Underseepage to the St. Louis District and were based to an extent on data from 16 piezometer sites along Lower Mississippi River levees. The transformation factors are given in table 1.

Table 1

Thickness Transformation Factors for Top Strata

Soil Type		Transformation Factor
		Clay less than 5 ft in Thickness, $k_f/k_b = 400$
Fat clay	(CH)	1
Lean clay	(CL)	1
Silt	(ML)	1
Silt, sandy	(ML)	3/4 to 1
Silty sand	(SM)	1/5 if $z < 10$ ft; 0 if $z > 10$ ft
Fine sand		0
Alternating clay and silt strata w/depth		1
		Clay more than 5 ft in Thickness, $k_f/k_b = 1000$
Fat clay	(CH)	1
Lean clay	(CL)	1
Silt	(ML)	1/2
Silt, sandy	(ML)	1/4 to 1/2 if $z < 10$ ft; 0 if $z > 10$ ft
Silty sand	(SM)	1/10 if $z < 10$ ft; 0 if $z > 10$ ft
Fine sand		0
Alternating clay and silt strata w/depth		1

The above transformation factors represent the ratio of the assumed vertical permeability of the more pervious layer in the top stratum to that of the principal layer (either silt or clay). These factors were based

primarily on judgment and experience and the following considerations.

73. The in-situ vertical permeability of clay strata with thickness less than 5 ft and at or near the ground surface was assumed to be 2.5×10^{-4} cm per sec on the basis of field measurements in the lower alluvial valley[11]. This large a k_b is attributed to seepage through root holes, shrinkage cracks, crayfish holes, and burrows of small animals rather than through the pores in the soil mass. The vertical permeability of silt strata also was assumed equal to about 2.5×10^{-4} cm per sec. Strata of silty sand were assumed to have a vertical permeability of about 10×10^{-4} cm per sec. Where the thickness of clay strata was less than 5 ft, all strata comprising the top stratum were transformed to an equivalent thickness of the clay (or silt strata if no clay were present) and the effective permeability of the transformed top stratum was assumed to be 2.5×10^{-4} cm per sec.

74. The in-situ permeability of clay strata having a thickness equal to or greater than 5 ft was assumed to be 1×10^{-4} cm per sec. The lower permeability value of k_b for the thicker clay deposits was based on the belief that the thicker clays would contain fewer fissures and crayfish holes, etc., than the thinner surface clay strata. Where the thickness of the clay exceeded 5 ft, the thickness of each stratum comprising the top stratum was transformed to an equivalent thickness of clay and the effective vertical permeability of the transformed top stratum was taken as 1×10^{-4} cm per sec.

75. The characteristics of the top stratum at the landside toe of the levee were considered primarily in making a final determination of the effective thickness and permeability of the landside top stratum. Where soils data were available landward from the levee, they were considered. Thin spots in the top stratum were given considerable weight in the selected values.

76. The type and thickness of riverside top stratum or blanket in riverside borrow pits were used in estimating the point of effective seepage entry into the pervious substratum. They were estimated from field reconnaissance, riverside borings, and from the depth of material estimated to exist beneath the borrow pit on the basis of pit depths and

landside borings where soil conditions could be expected to be similar on both sides of the levee. The distance from the landside toe of the levee to point of seepage entry was based on this information and the criteria set forth in table 2, page 48.

Effective thickness d and permeability k_f of pervious substratum

77. The effective thickness of the pervious substratum was defined as the thickness of the principal seepage-carrying stratum below the top stratum and above the bottom of the entrenched valley. The top of the pervious substratum was taken as the uppermost portion of the aquifer having a $D_{10} > 0.15$ mm or a permeability of about 500×10^{-4} cm per sec. The bottom of the substratum or alluvial valley was taken as the contact line between the sand and gravel substratum and the underlying rock. The average effective thickness of the pervious substratum was taken as 80 ft to simplify and expedite design computations for seepage berms and spacing of relief wells. Variations in thickness of the sand stratum have a direct effect on the effective exit length for seepage, but because of other compensating factors, have little effect on the required spacing of relief wells if the variations in thickness are less than about 20 ft.

78. The average permeability k_f of the pervious substratum agreed upon by the Underseepage Advisory Board was 1000×10^{-4} cm per sec. This value was based on laboratory data from the St. Louis District and pumping tests and field observations made on relief well systems at Commerce[6] and Trotters[7], Miss.

79. After the relief wells were installed along the levees in the St. Louis District, values of k_f were computed from the specific yield of the wells as obtained from a pumping test on each well. As the relief wells did not fully penetrate the aquifer, it was necessary when computing k_f to use a factor G relating the flow from a partially penetrating artesian well in a homogeneous foundation to that from a fully penetrating well. This factor was obtained from Kozeny's formula and the graph in fig. 12. The permeability of the pervious substratum was then obtained from the following formula:

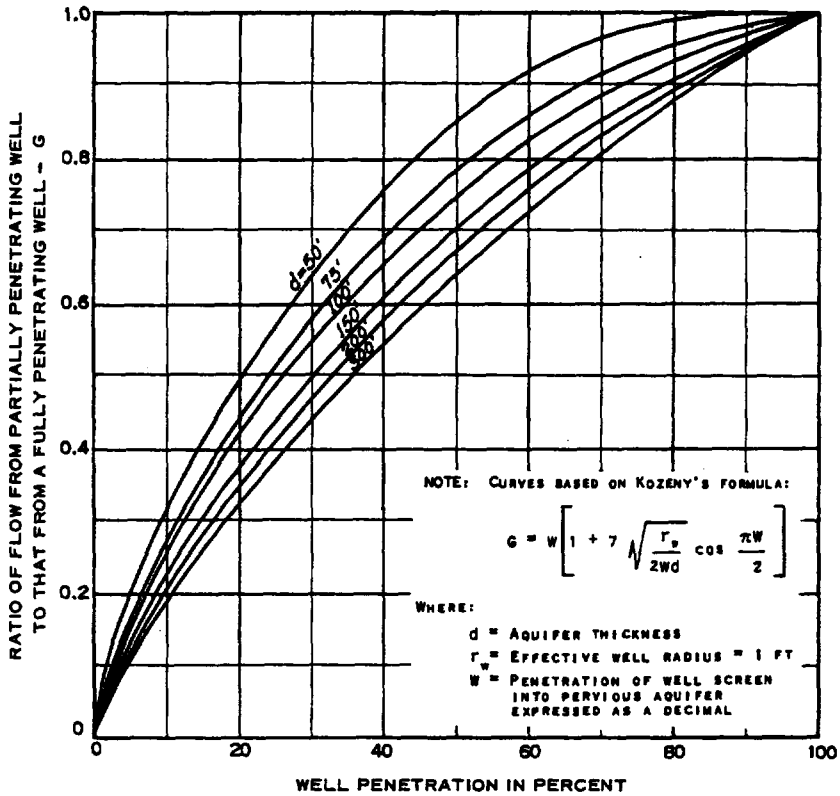


FIG. 12. RELATION BETWEEN FLOW FROM A PARTIALLY PENETRATING ARTESIAN WELL IN A HOMOGENEOUS FOUNDATION AND FROM A FULLY PENETRATING WELL

$$k_f = \frac{Q_w \ln\left(\frac{R}{r_w}\right)}{2\pi \; d \; G \; DD} \qquad (2)$$

wherein:

k_f = permeability of pervious substratum or aquifer

Q_w = well flow

R = radius of influence

r_w = effective radius of the well

d = effective thickness of the aquifer

G = ratio between flow from a partially penetrating artesian well in a homogeneous foundation and from a fully penetrating well (as obtained from fig. 12)

DD = effective drawdown in the well.

Subsequent to preparation of fig. 12 and computation of k_f from pumping data from the wells, the factor G in equation 2 was computed from a

more precise formula for partially penetrating wells given by Muskat[13]; a graph similar to fig. 12 is presented in reference 11.

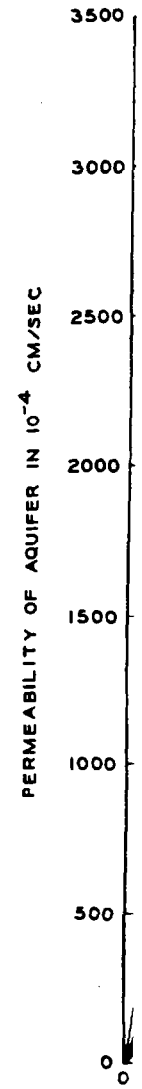
80. To simplify computations, equation 2 was plotted in fig. 13 assuming R = 750 ft (the average obtained from special pumping tests described in part VIII). From fig. 13, k_f can be determined from the specific yield of the well per foot of aquifer and the value of G as obtained from fig. 12. The average value of k_f for all levee districts was found to be about 1400 x 10^{-4} cm per sec as compared to 1000 x 10^{-4} cm per sec used in the evaluation of the need for and design of control measures. It should be noted that most of the wells were pumped at a rate of 500 gpm, which creates a head loss in the well of about 0.8 ft. At this rate of flow the effective drawdown is about 0.8 ft less than the measured drawdown. As the average measured drawdown was about 5 ft at the above flow, the permeability of the pervious substratum was taken as 1.2 times that computed from the measured drawdown in the well.

81. As the measured k_f generally exceeds that assumed for computing the well spacing, the head between wells may be somewhat greater than that computed, because of increased head loss in the wells as a result of increased flow.

Ratio of permeability of pervious substratum and landside top stratum k_f/k_b

82. From the design values for k_f and k_b given previously, k_f/k_b was taken as 1000 where the top stratum was comprised of more than 5 ft of clay (table 1). Where the top stratum was comprised of silt or clay less than 5 ft thick, k_f/k_b was assumed equal to 400 (table 1). In a few reaches the top stratum consists of only 2 to 3 ft of sandy silt; in these reaches a permeability ratio of about 250 was assumed. The above values were based to an extent on observations at 16 sites along Lower Mississippi River levees[11] as discussed below.

83. The best method for obtaining k_f/k_b is by means of equation 5 with x_3 determined from the piezometric grade line beneath the levee. Since only limited piezometric data were available from levees in the St. Louis District, data from several sites along the Lower



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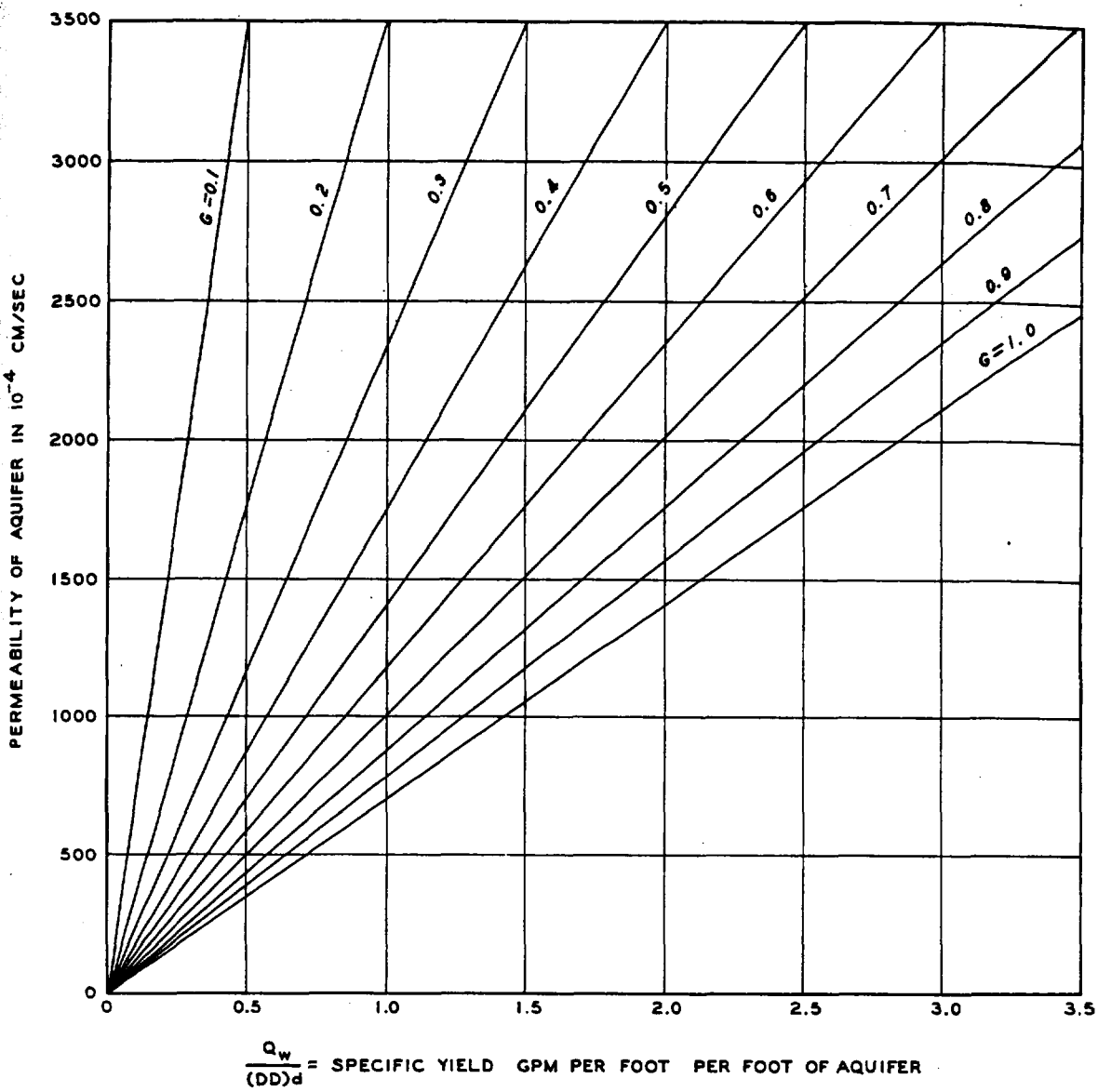


NOTE: CURVES BASED ON FORMULA

$$Q_w = \frac{0.0015 \times 2\pi\, k_f d\,(DD)\,G}{\ln \frac{R}{r_w}}$$

WHERE:

G = RATIO BETWEEN FLOW FROM A PARTIALLY PENETRATING ARTESIAN WELL IN A HOMOGENEOUS FOUNDATION AND FROM A FULLY PENETRATING WELL (FROM KOZENY[13])

Q_w = WELL FLOW IN GPM

DD = DRAWDOWN IN WELL IN FT

d = AQUIFER THICKNESS IN FT

r_w = EFFECTIVE RADIUS OF WELL = 1 FT

R = RADIUS OF INFLUENCE (ASSUMED EQUAL TO 750 FT)

k_f = PERMEABILITY OF AQUIFER IN 10^{-4} CM/SEC UNITS

FIG. 13. PERMEABILITY OF AQUIFER VS SPECIFIC YIELD OF AQUIFER

Mississippi River were utilized in estimating values of k_f/k_b . Permeability ratios determined from piezometer readings vs various types and thicknesses of landside top stratum in the Lower Mississippi River Valley are plotted in fig. 14.

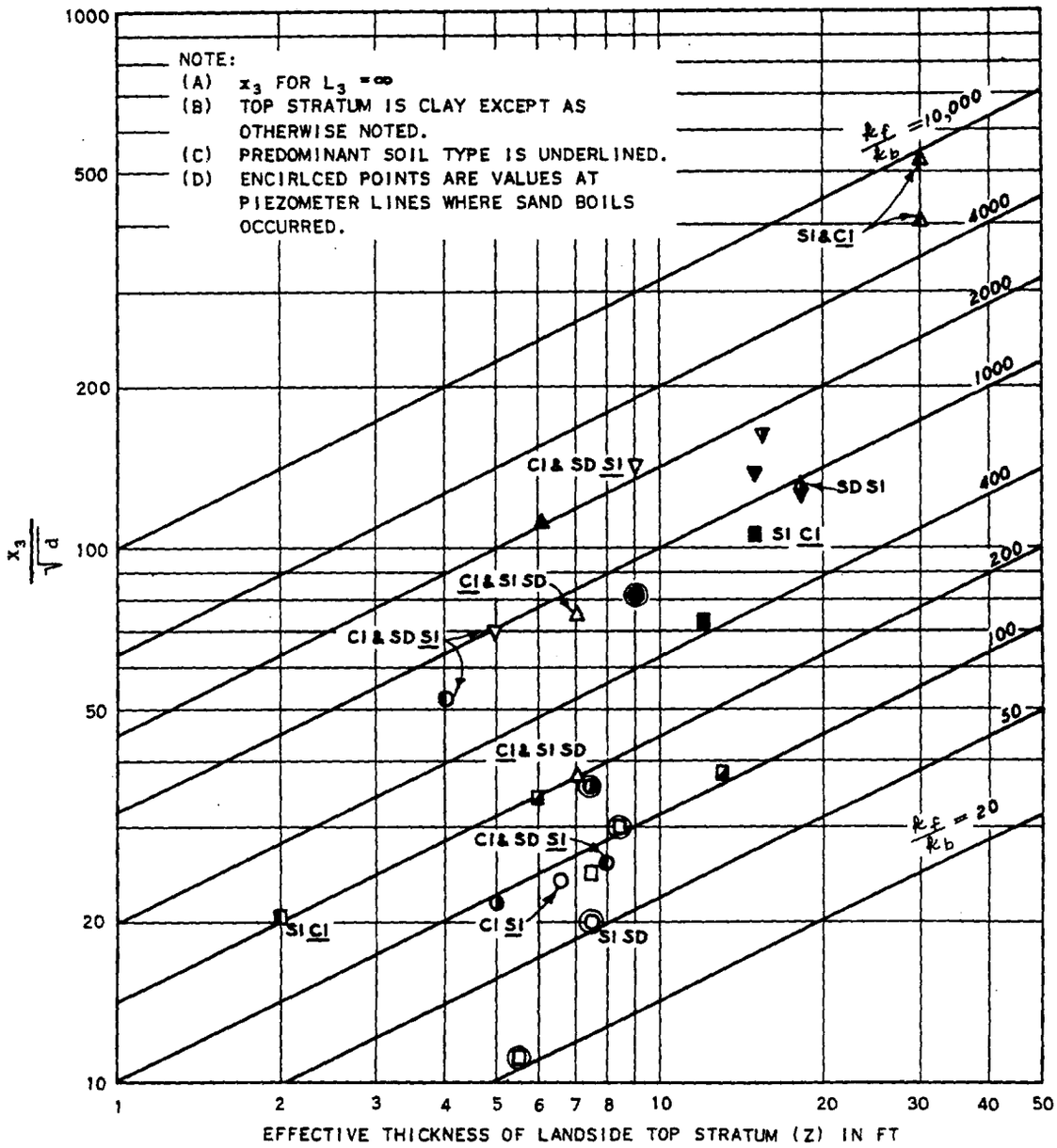


FIG. 14. RATIO OF PERMEABILITY OF PERVIOUS SUBSTRATUM TO PERMEABILITY OF LANDSIDE TOP STRATUM AT CREST OF 1950 HIGH WATER AT 16 SITES ALONG LOWER MISSISSIPPI RIVER LEVEES

Distance from levee toe to effective seepage entry s

84. The distance from the landside toe of the levee to the effective source of seepage entry s is influenced by the distance from the landside toe of the levee to the river and the soil conditions riverward of the levee and in the bottom of the borrow pits.

85. The best and most accurate method for determining the distance from the landside levee toe to the effective source of seepage entry is to project graphically the hydraulic grade line beneath the levee as measured by piezometers installed in the pervious substratum beneath the levee until it intersects the river stage producing the gradient. The value of s can also be determined from piezometric data using the following equations (see fig. 15 for nomenclature).

$$s = \ell_1 + (H - h_1)\left(\frac{\ell_2 - \ell_1}{h_2 - h_1}\right) \tag{3}$$

or

$$s = \ell_1 + \frac{H - h_1}{M} \tag{3a}$$

86. In a few cases in the St. Louis District, piezometric data were available for use in determining the effective source of seepage entry. However, data usually were not available and the values of s were obtained as discussed in the following paragraphs.

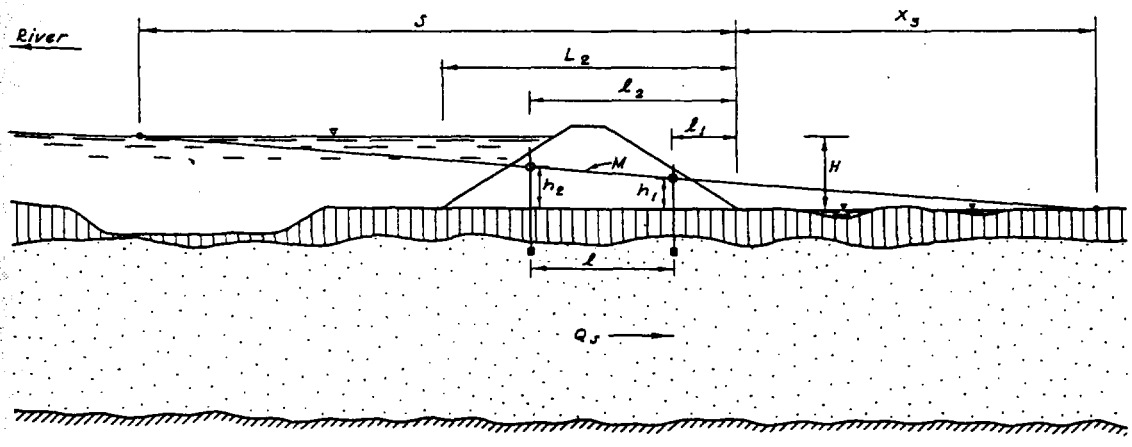


FIG. 15. NOMENCLATURE FOR DETERMINATION OF s AND x_s FROM PIEZOMETER READINGS

87. Where there are no riverside borrow pits and the Mississippi River is close to the levee, s was assumed to equal the distance from the landside toe of the levee to the near bank of the river.

88. For river-front levees where riverside borrow pits are present, values of s were based to an extent on piezometric data obtained along levees in the Lower Mississippi River Valley. These values were found to be related to the thickness and type of blanket in the riverside borrow pits. Distances from the landside toe of the levee to the effective source of seepage for river-front levees as agreed upon by the Advisory Board are given in table 2. The distances from the toe of the levee (or

Table 2

Criteria for Estimating Distances from Landside Toe of Levee to Effective Source of Seepage

Riverside Blanket or Soil in Borrow Pit	River-front Levees s in ft	Tributary Levees s in ft
Fine sand, silt < 5 ft	600	800
Silt, 5 ft or more, clay < 5 ft	800	1000
Clay, 5 ft or more	1000	1500

berm) to the effective source of seepage used for tributary levees are also given in table 2. These values are based on some piezometric data and approximate computations using an equation by Barron given in reference 5.

89. The St. Louis District is engaged in a program of borrow pit remedial measures consisting of filling deep scour holes, construction of abatis dikes, and promotion of the growth of willows in borrow pits. These measures were considered in the selection of s along the reaches of levee where this program is being carried out. (In the several years since initiation of this investigation there has been considerable growth of willows in many of the riverside borrow pits.)

Distance from levee toe to effective seepage exit x_3

90. The effective seepage exit is defined as that line or point landward of the levee where a hypothetical open drainage face and an

impervious blanket between such point and the levee toe would result in the same hydrostatic pressure at the levee toe and would cause the same amount of seepage to pass beneath the levee as would actually occur under existing conditions. The distance x_3 to this point is the intersection of an extension of the hydraulic grade line beneath the levee with the ground surface or tailwater. The best way to determine x_3 is by means of piezometers installed in the pervious substratum beneath a levee, using the following expression (see fig. 15 for nomenclature):

$$x_3 = h_1\left(\frac{\ell_2 - \ell_1}{h_2 - h_1}\right) - \ell_1 \tag{4}$$

or

$$x_3 = \frac{h_1}{M} - \ell_1 \tag{4a}$$

In a few cases, piezometric data were available to determine x_3 but usually the distance to the effective seepage exit was estimated from the following blanket formula by Bennett[2], assuming the top stratum to be level and infinite in landward extent ($L_3 = \infty$).

$$x_3 = \frac{1}{c} = \sqrt{\frac{k_f\ z\ d}{k_b}} \tag{5}$$

where

c = a constant.

Values of x_3 can be obtained from fig. 16 for various values of z and k_f/k_b for $d = 80$ ft and assuming the top stratum to be infinite in landward extent.

Critical gradient

91. The critical gradient required to cause sand boils or heaving of the top stratum is defined as the ratio of the submerged unit weight of the soil comprising the top stratum and the unit weight of water, or:

$$i_c = \frac{\gamma'}{\gamma_w} = \frac{G - 1}{1 + e} \tag{6}$$

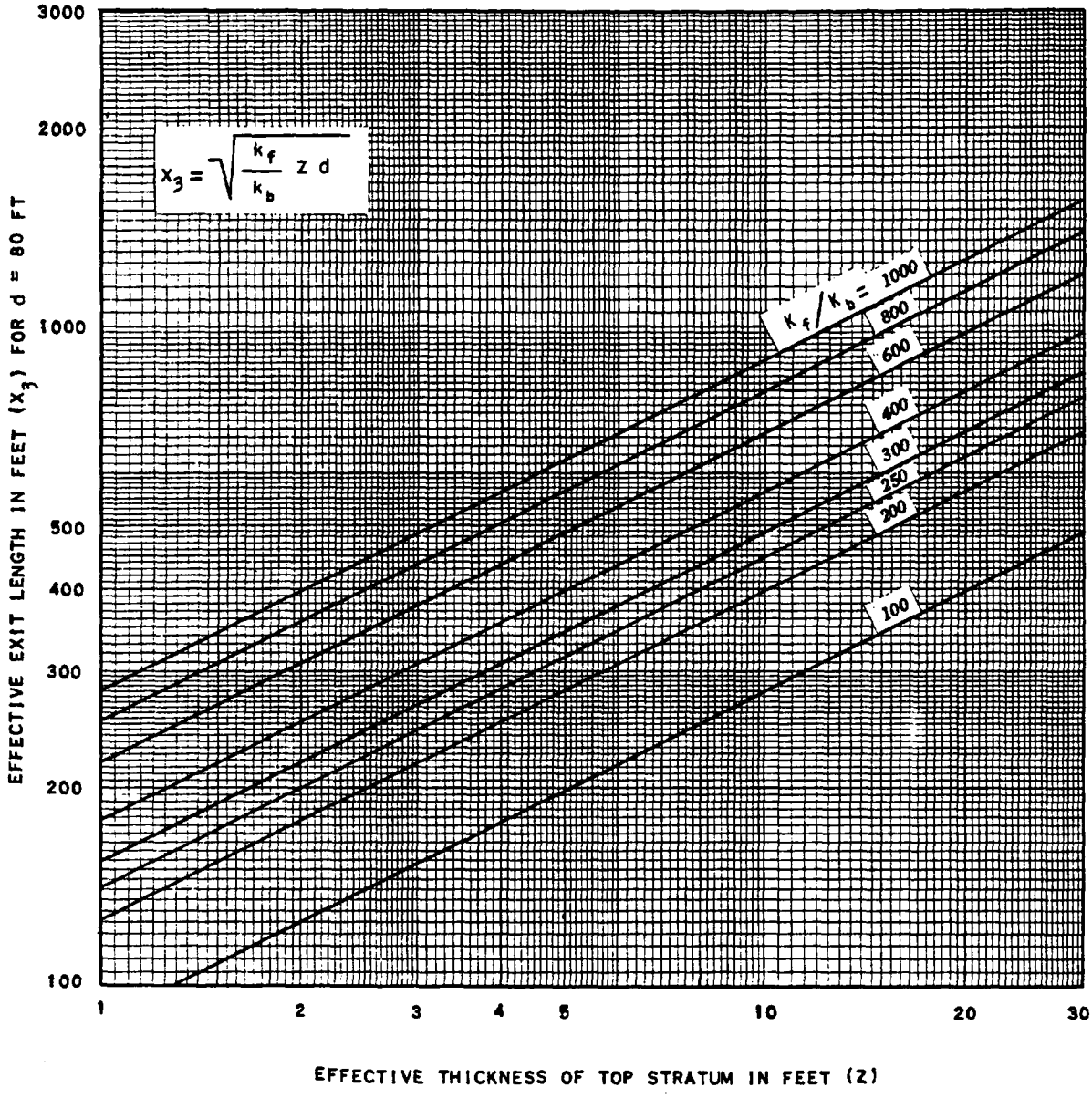


FIG. 16. EFFECTIVE SEEPAGE EXIT LENGTH FOR $L_3 = \infty$ AND $d = 80$ FT

From the preceding definition, homogeneous soils would have the following approximate critical gradients:

Silty sand and silts, $i_c = 0.85$

Silty clay and clay, $i_c = 0.80$

92. The critical gradient required to cause sand boils in the field can best be determined by measuring the hydrostatic head beneath the top stratum at the time sand boils appear. In this method i_c is determined from the following equation:

$$i_c = \frac{h_{x(c)}}{z} \tag{7}$$

where

$h_{x(c)}$ = net head above the ground surface or tailwater at time sand boils or heaving of top stratum occurs

z = thickness of top stratum.

Critical gradients as measured in the field at some of the piezometer sites along the levees in the Lower Mississippi River Valley[11] during the 1950 high water indicated the following generalized conclusions:

Type of Seepage	Upward Gradient through Top Stratum
Little or none	0 to 0.5
Medium to heavy	0.25 to 0.7
Active sand boils	0.5 to 0.8

From the above data it appears that active sand boils may occur in the field at gradients considerably lower than theoretically computed. This probably can be attributed to existing holes, cracks, thin or weak spots, and other unknown minor geological features. Once a sand boil or pipe has formed, the gradient required to keep the boil active is equal to or less than that required to create the boil.

93. As the levees in the St. Louis District generally had not been subjected to very high river stages and relatively few sand boils have occurred to date, the critical gradient was assumed to equal 0.85 for determining the need for seepage control measures except in the East St. Louis levee district where a value of 0.67 was assumed.

Formulas for Seepage Analyses

94. The head h_o beneath the top stratum at the landside toe of the levee at design flood stage can be estimated by extrapolation of readings of piezometers obtained at lower river stages. Similarly, Q_s can be estimated from piezometric data and a knowledge of the characteristics of the pervious substratum. However, as the available piezometric data were limited, it was necessary to estimate h_o and Q_s

from theoretical formulas and characteristics of the pervious substratum and the top stratum landward of the levee. The formulas used to estimate h_o and Q_s are presented below for typical top strata and foundation conditions. In the subsequent discussion it should be noted that all formulas are based on conditions of laminar flow through the top stratum and pervious substratum. It is further assumed that the top stratum is capable of withstanding the head h_o without rupture; that is, h_o is assumed to be independent of the head required to produce sand boils $h_{x(c)}$. This latter assumption would not be satisfied in nature, as the maximum possible head beneath the top stratum that can develop would be about equal to $h_{x(c)}$. In reaches of levee where the computed value of h_o exceeds $h_{x(c)}$, seepage control measures were considered warranted.

No top stratum

95. Where a levee is founded directly on pervious foundation sands and no top stratum exists either riverward or landward of the levee, the seepage Q_s per unit length of levee can be determined from the following expression:

$$Q_s = \$ k_f H \tag{8}$$

where

$$\$ = \frac{d}{s + 0.44\, d} \tag{9}$$

The need for seepage control measures was based on whether or not the creep ratio L_2/H was greater than 10 to 15.

Semipervious top stratum

96. The head h_o at the landside toe of the levee beneath a semipervious top stratum was estimated from the following equation:

$$h_o = H \frac{x_3}{s + x_3} \tag{10}$$

Values of h_o for various values of H were computed from equations 5 and 10 for permeability ratios of 400 and 1000, $d = 80$ ft, and for

various thicknesses of top stratum and distances to effective source of seepage of 600, 800, 1000, and 1500 ft (see table 2). The above relationships are plotted in figs. 17-20. Also shown in these figures are the heads h_o corresponding to upward gradients of 0.67 and 0.85 through the top stratum. These gradients were used to estimate the severity of seepage as described subsequently.

97. In the design of seepage control measures it frequently was necessary to evaluate the substratum pressure h_x at a distance x landward from the levee. The head h_x beneath the semipervious top stratum x distance landward of a levee depends upon the head h_o, conditions landward of the levee, and the distance x. Expressions for h_x are given below for typical conditions encountered landward of levees, assuming the top stratum infinite in landward extent.

$$h_x = h_o \epsilon^{-cx} = h_o \epsilon^{-x/x_3} \tag{11}$$

$$(\epsilon = 2.718)$$

$$c = \frac{1}{x_3} = \sqrt{\frac{k_b}{k_f \; z \; d}} \tag{5a}$$

The ratio of h_x/h_o vs x/x_3 is plotted in fig. 21 and was used to design landside seepage berms as illustrated later in this report. The quantity of natural seepage per unit length of levee was computed from the following equations:

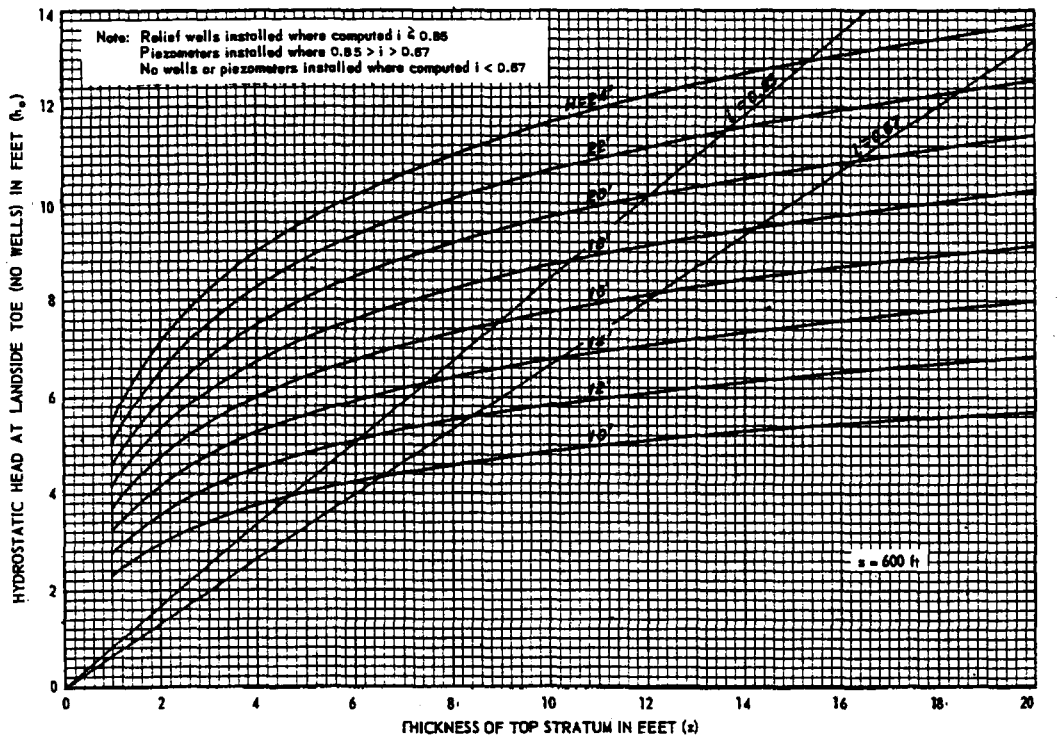
$$Q_s = \$ \, k_f \, H \tag{8}$$

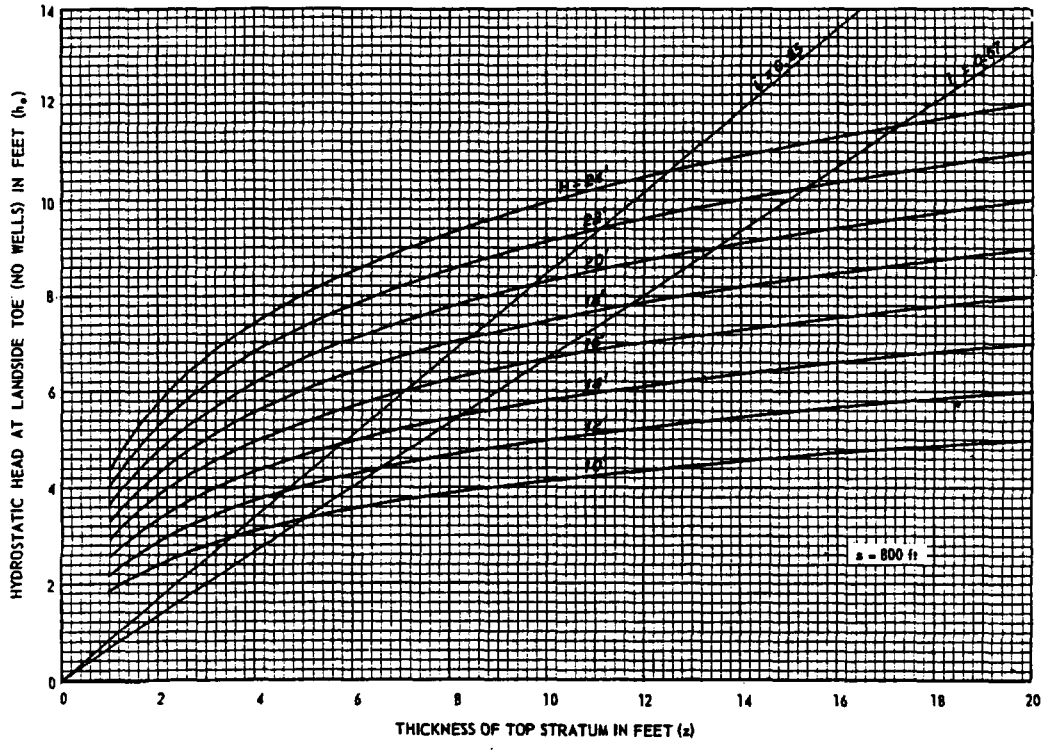
where

$$\$ = \frac{d}{s + x_3} \tag{12}$$

or

$$Q_s = k_f \, H \, \frac{d}{s + x_3} \tag{13}$$



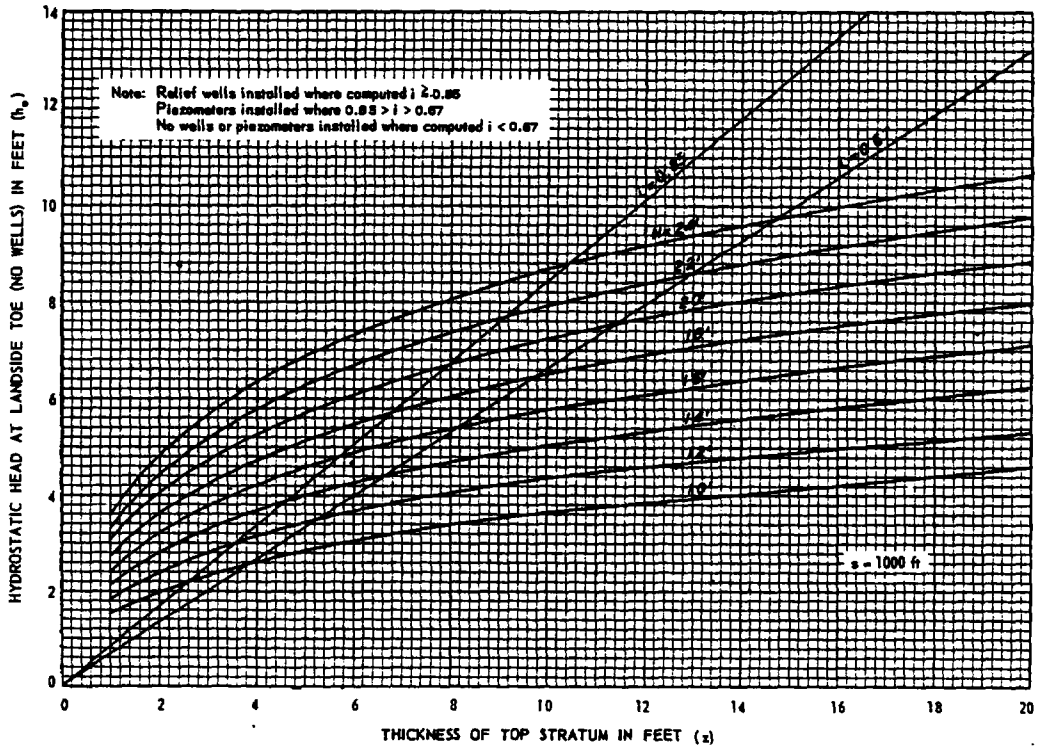


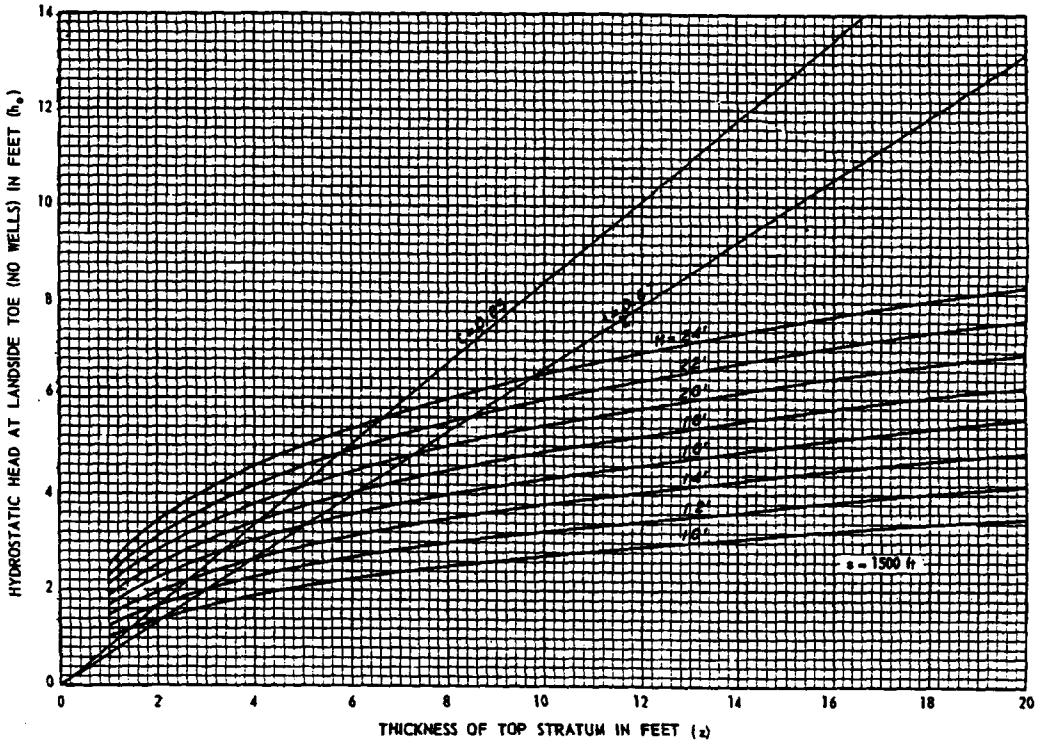
COMPUTED HYDROSTATIC HEADS AT LANDSIDE TOE OF LEVEES

NO RELIEF WELLS

$k_f / k_b = 400$, $d = 80$ ft, $s = 600$ and $s = 800$ ft

FIG. 17



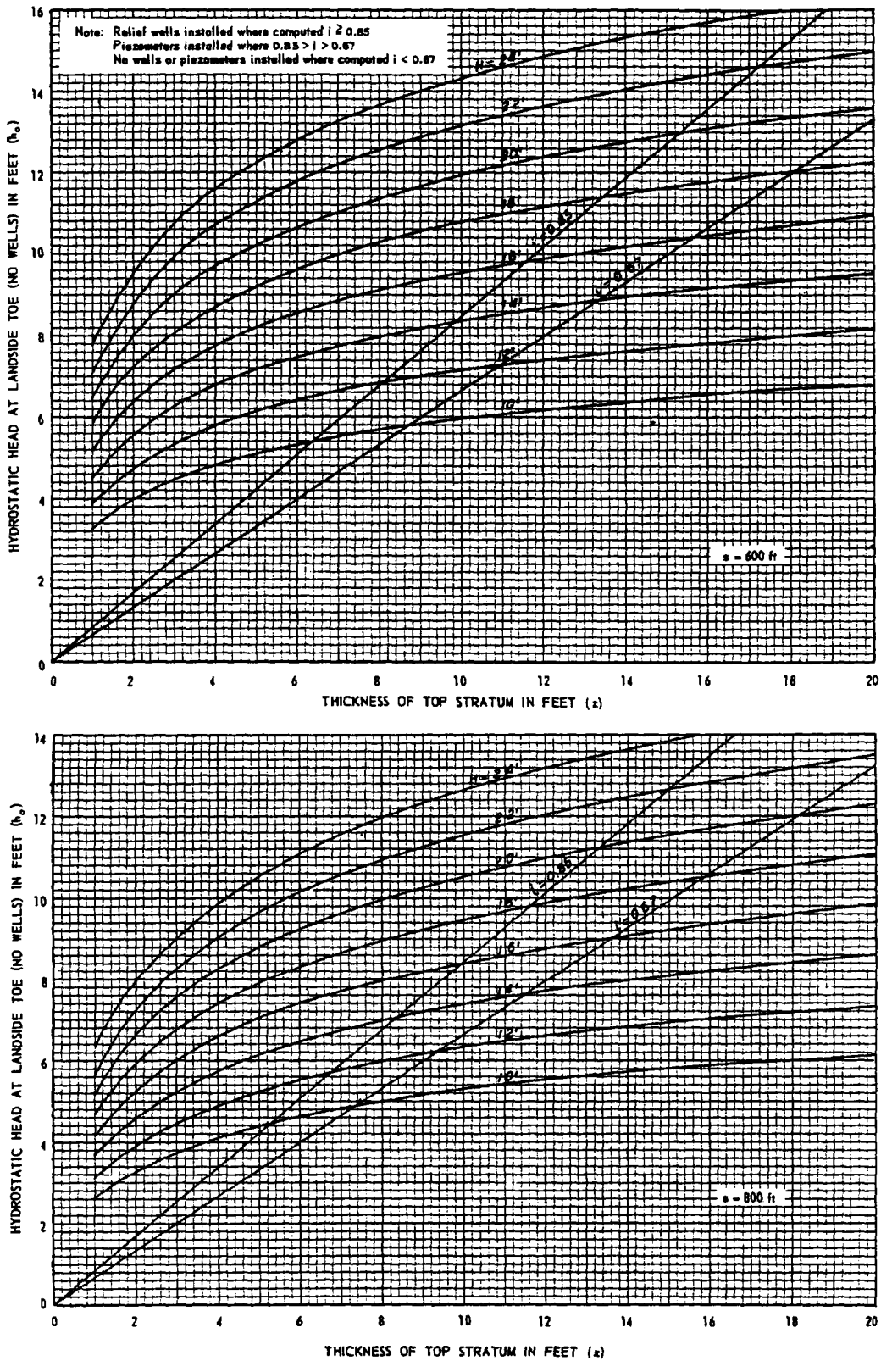


COMPUTED HYDROSTATIC HEADS AT LANDSIDE TOE OF LEVEES

NO RELIEF WELLS

$k_f / k_b = 400$, d = 80 ft, s = 1000 and s = 1500 ft

FIG. 18

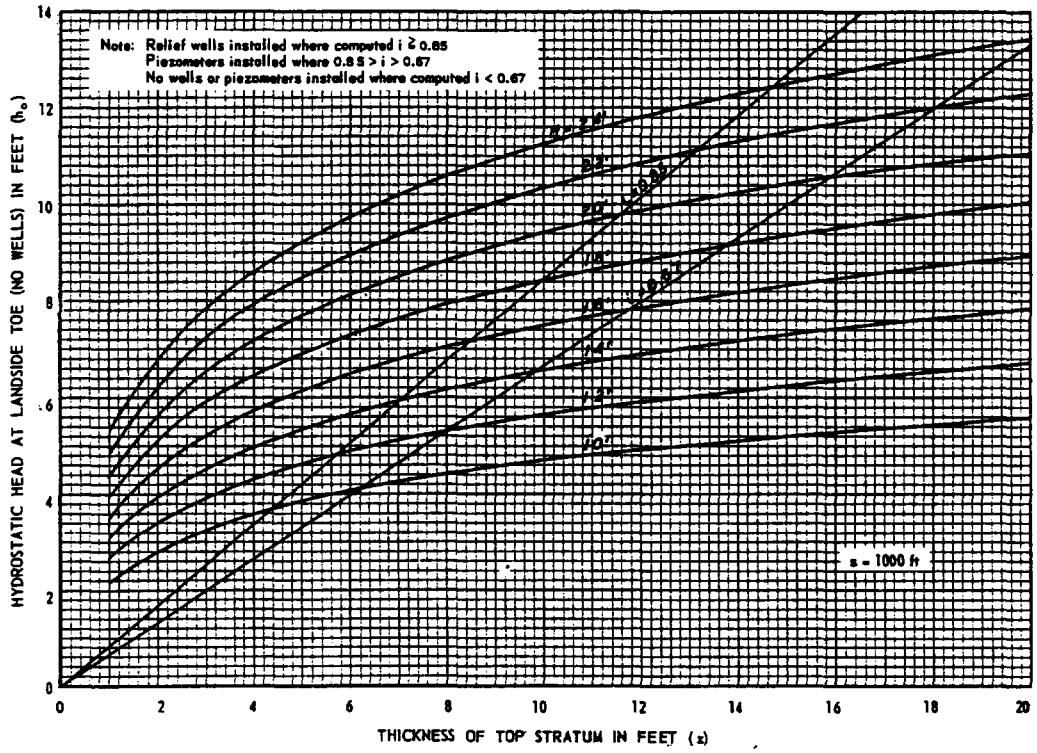


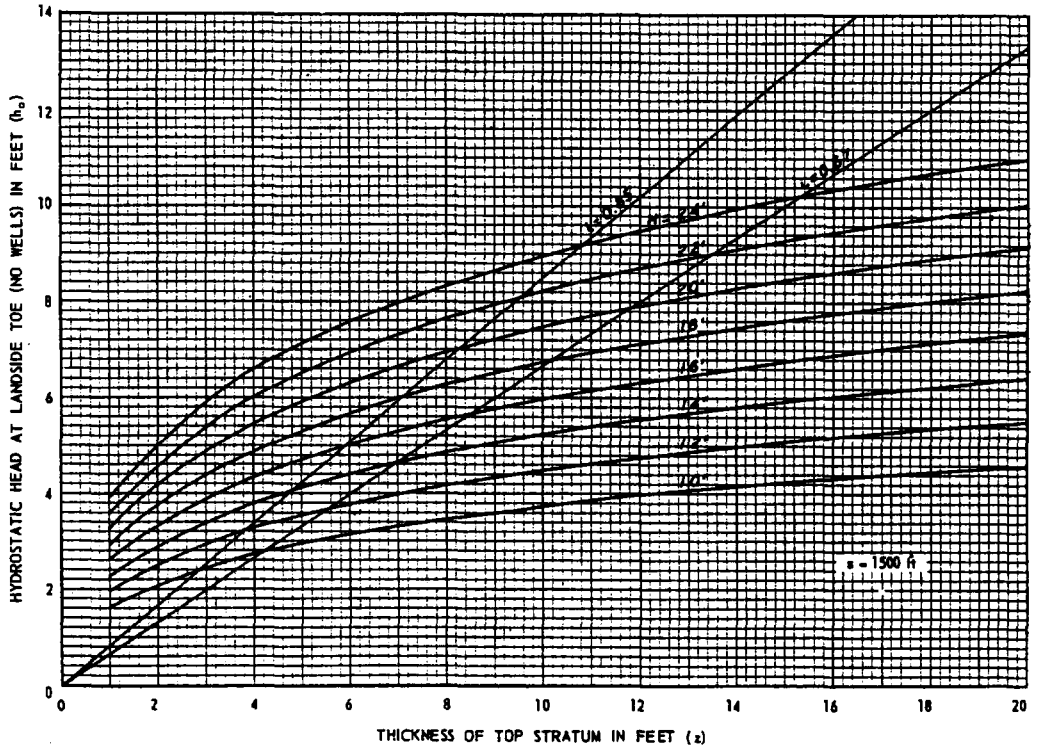
COMPUTED HYDROSTATIC HEADS AT LANDSIDE TOE OF LEVEES

NO RELIEF WELLS

$k_f / k_b = 1000$, d = 80 ft, s = 600 and s = 800 ft

FIG. 19





COMPUTED HYDROSTATIC HEADS AT LANDSIDE TOE OF LEVEES

NO RELIEF WELLS

$k_1 / k_b = 1000$, d = 80 ft, s = 1000 and s = 1500 ft

FIG. 20

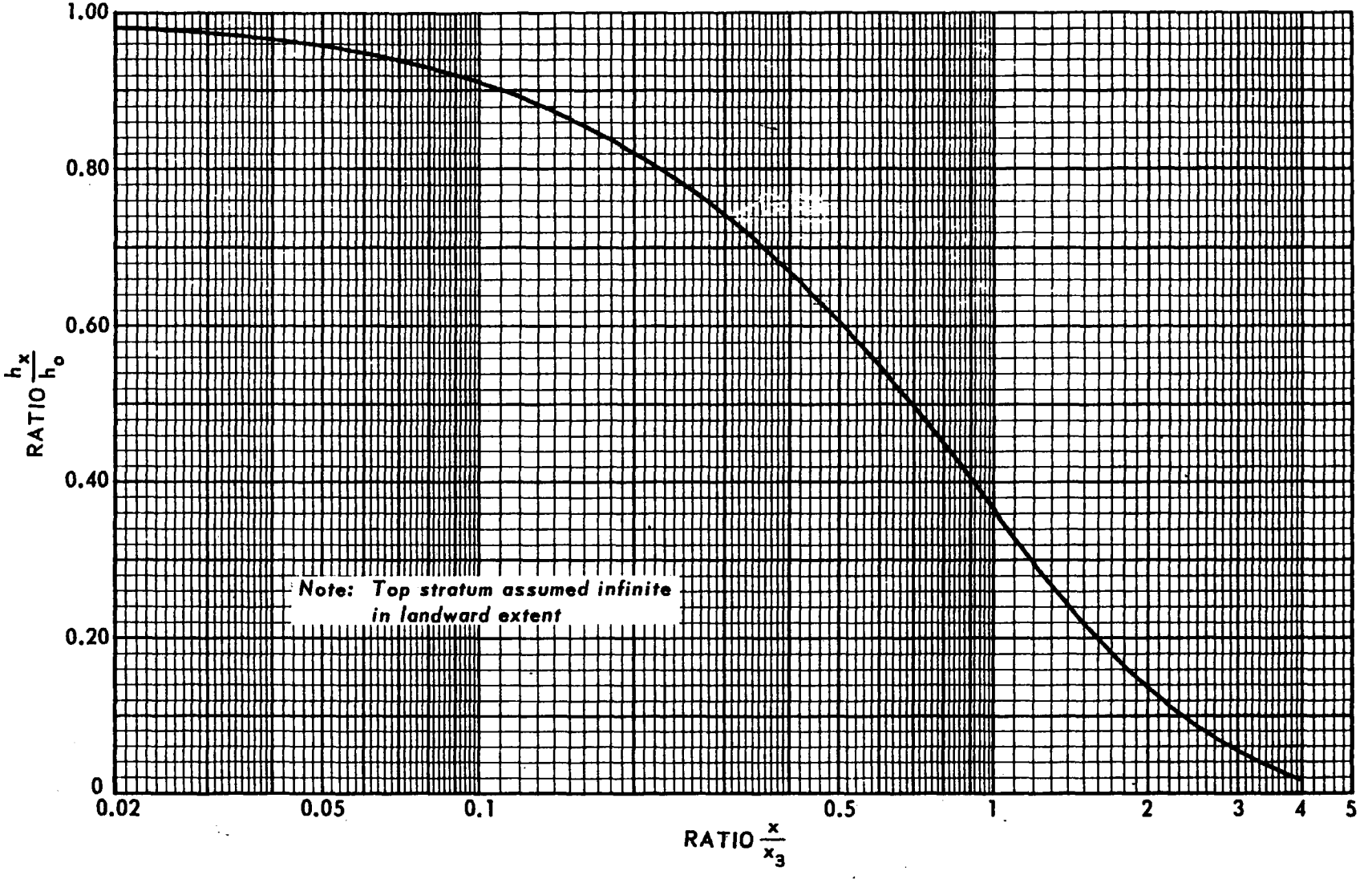


FIG. 21. CURVE FOR ESTIMATION OF SUBSTRATUM PRESSURES LANDWARD FROM THE LANDSIDE TOE OF LEVEES

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Use of Piezometric Data in Seepage Analyses

98. The potential safety of a levee with respect to underseepage can be evaluated from the readings of a single piezometer along the landside toe of the levee by plotting the observed readings during high-water periods against corresponding river stages. The maximum hydrostatic head that a top stratum can withstand without heaving or developing sand boils is equal to about 0.7 to 0.85 times the thickness of the top stratum; this head can be converted to an elevation by adding it to the elevation of the ground at the piezometer, or tailwater elevation if the area landward of the levee is submerged. The river stage at which the head beneath the top stratum will become equal to the critical head h_c can be estimated by extrapolating a plot of piezometer readings vs river stages. If the estimated river stage required for h_o to become equal to h_c is less than the design flood stage, control measures are indicated. If the river stage is sufficiently high to cause the piezometer readings to reach a maximum and then to remain constant during a period when the river continues to rise, the critical hydrostatic pressure has been reached and control measures are indicated if the project river stage is very much higher than the stage at which h_c occurred.

99. Various combinations of s and x_3 for use in the design of seepage control measures can be estimated from the reading of a single piezometer by determining the distance to the source of seepage entry and effective seepage exit required for an h_o equal to the observed head at the toe of the levee. The distance to the source of seepage can be estimated from reaches where piezometers have been installed perpendicular to the levee where riverside soil conditions are of a similar nature. When designing seepage control measures, the distance to the effective seepage exit x_3 should be assumed somewhat greater than actually predicted for the project flood to allow for some safety in the design.

100. Where lines of piezometers have been installed perpendicular to the levee, the design of seepage control measures can be based on the

FIG. 21. CURVE FOR ESTIMATION OF SUBSTRATUM PRESSURES LANDWARD FROM THE LANDSIDE TOE OF LEVEES

observed distances to the effective source of seepage and seepage exit. The observed values of s and x_3 should be plotted against river stage to determine any trend in s and x_3 to decrease with rising river stages. A decrease in s may result from scour in the riverside borrow pits and a decrease in x_3 can occur from the development of concentrated seepage landward of the levee. The value of s used to design control measures should be that estimated to exist at the project flood as obtained by extrapolation. The distances to the effective seepage exit should be taken on the conservative side as explained previously. Where lines of piezometers have been installed, the need for seepage control measures would be based on a plot of readings of the piezometer at the landside toe of the levee vs river stage, as in the case for single piezometers installed along the levee.

PART VI: DESIGN OF UNDERSEEPAGE CONTROL MEASURES

101. The decision as to whether landside seepage berms or relief wells should be used was based on the respective costs for each measure and in some cases on availability of right-of-way. Seepage berms were also designed for reaches of levee where the top stratum was absent or so thin that the required spacing of wells would be excessively close and where the maximum head is relatively low.

Relief Well Systems

102. The design of the relief wells included both the design of individual wells and determination of the required well spacing.

Design of the well

103. Design of the well itself included the selection of type and length of riser pipe and screen, design of the gravel filter, and appurtenances for the well (see plate 265).

104. Riser pipe and well screen. Eight-inch-ID, treated wood-stave pipe was selected for the riser pipe and well screen for this project. Such pipe is economical and noncorrosive. To insure against decay resulting from fluctuations in ground-water level, the upper 10 to 15 ft of the riser pipe was surrounded by concrete backfill. The 8-in.-ID pipe size was required to minimize friction and velocity head losses in the well. The screens for the relief wells were perforated with 3/16- by 3-1/4-in. slots; the open area of the slots was about 10 per cent of the circumferential area of the screen pipe. Additional information on the construction of the wells is given in part IX.

105. Length of riser pipe and well screen. The spacing of relief wells was computed on the basis that the well screens would have an effective penetration of 50 per cent of the principal seepage-carrying stratum. From the special pumping tests described in part VIII it was found that to achieve an effective penetration of 50 per cent of the pervious stratum about 60 per cent of the depth of sand stratum had to be penetrated by the well screens. The principal seepage-carrying

stratum was considered to extend from the top of medium to fine sand ($D_{10} > 0.15$ mm) to bedrock. Screen, designated "design screen," was installed in sand with $D_{10} > 0.15$ to 0.18 mm. Screen, designated "extra screen," was installed in upper fine sand too fine to be considered as aquifer but coarse enough to carry some seepage. Generally, "extra screen" was installed in sand with $D_{10} > 0.10$ mm and $D_{85} > 0.20$ mm. The screens were designed in accordance with available logs of borings but in some cases were modified in the field as discussed in part IX.

106. Filter material. The well screens penetrated strata containing fine, medium, and coarse sands. The filter was designed to drain the fine sands safely but was made sufficiently coarse to permit inflow of seepage with little head loss. A minimum thickness of 6 in. was specified for the filter. The filter was designed in accordance with the following criteria:

$$\frac{(\text{Max})\ D_{15}\ \text{filter}}{(\text{Min})\ D_{85}\ \text{sand}} \leq 5 \text{ and } \frac{(\text{Min})\ D_{85}\ \text{filter}}{\text{Slot width}} \geq 1.2$$

The filter was also designed to be as pervious as possible and still comply with the above criteria. The following filter material was specified for all relief wells except those in the original contracts for the Harrisonville, Ft. Chartres, and Columbia levee districts:

U. S. Standard Sieve	Per Cent Finer by Weight
3/4-in.	100
1/2-in.	91-100
3/8-in.	78-97
No. 4	51-81
No. 8	28-60
No. 16	9-34
No. 30	0-14
No. 40	0-9
No. 50	0-6

The filter specified for the wells in the Columbia, Ft. Chartres, and Harrisonville levee districts was the same as above except that the per cent finer for the No. 40 sieve was 0-6 per cent and no percentage was specified for the No. 50 sieve. The above gravel filters were specified to grade uniformly within the band. They are considered capable of

protecting foundation sands having $D_{85} > 0.12$ mm (see paragraphs 56-59).

107. Well appurtenances. If a well becomes backflooded with muddy surface water its efficiency is impaired. To prevent backflooding the wells were equipped with an aluminum flat-type check valve and rubber gasket. The valves and gasket were designed by and subjected to simulated field tests at the Waterways Experiment Station as described in part IX. The tops of the relief wells were provided with a galvanized metal well guard to protect the check valve and standpipe against animals, vandalism, or accidental damage, and to prevent the entrance of debris. The plastic standpipes were designed to prevent the wells from discharging when there is relatively little head on the levee and no pressure relief is necessary. The maximum height of the standpipes was selected as 0.25 z, and they are to be removed when the hydrostatic head in the pervious substratum causes them to overflow. The above-described well appurtenances are illustrated in part IX.

Computation of well spacing and flow

108. The spacing of relief wells where a landside top stratum is present was set so that the head above tailwater or ground surface midway between wells at the design flood stage would not exceed 0.67 z in agricultural areas or 0.50 z in the East St. Louis levee district.

109. Design formulas. The required spacing of relief wells was determined from formulas by Barron[1, 5] for an infinite line of fully penetrating wells with semipervious top stratum by computing the head midway between wells and then multiplying this head by the ratio of the head computed for 50 per cent penetrating wells to the head for fully penetrating wells as obtained from formulas by Jervis[5, 12] and Muskat[5, 13], respectively, for an impervious top stratum. The above ratio ranged from 1.5 to 1.9. The effective well radius r_w was taken as 0.9 ft; that is, the outer radius of the specified minimum thickness of filter gravel (min OD = 23 in.).

110. As the head midway between wells computed by the above formulas does not include hydraulic head losses in the well, it was necessary to add these losses to the computed head midway between wells.

The computed well spacing was then adjusted to a spacing a_c so that the head midway between wells, including hydraulic head losses in the wells, would not exceed the design allowable gradient of 0.67 or 0.50.

111. The hydraulic head losses in the wells were computed on the basis of recognized hydraulic formulas, well tank tests, and observations made on relief wells at Trotters, Miss. Entrance head losses through the filter and screen were based on data obtained from well tank tests[7]. Friction loss in the well screen was obtained from hydraulic formulas assuming that the head loss would be equal to that for 90 per cent of the total well discharge flowing through half the length of the screen. The head losses are plotted in fig. 22 for a typical well installation, assuming a well with a 25-ft riser, 55-ft screen, and the top of the well 0.33 ft above the ground surface. The head losses measured during the special pumping tests described in part VIII were about equal to those computed by the above listed methods and used in the design.

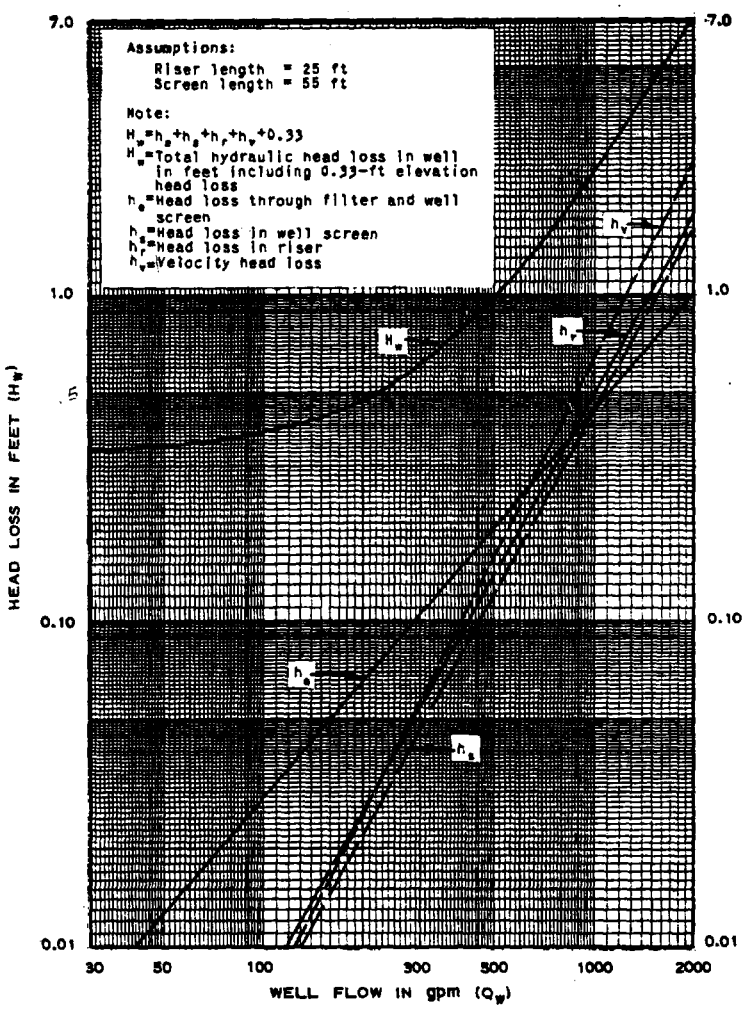


FIG. 22. HEAD LOSS IN 8-IN.-ID WOOD STAVE WELL WITH 6-IN. GRAVEL FILTER

112. The flow from relief well systems was obtained from Barron's[1, 5] formulas for fully penetrating wells with semipervious top

stratum multiplied by the ratio of flow from 50 per cent penetrating wells to that for 100 per cent penetrating wells for an impervious top stratum as determined by Jervis[5, 12] and Muskat[5, 13], respectively. These ratios ranged from about 0.90 to 0.98.

113. Where the length of a line of wells is relatively short (less than about 1300 ft), the well flow and head between wells, both at the center and near the ends of the line, are greater than computed by formulas for an infinite line of wells. Equations were developed by W. E. Miner, formerly of the Waterways Experiment Station, to determine the head midway between wells at the center of the finite well line. The ratio of head midway between wells at the center of a finite well line and the head midway between wells in an infinite line of wells is shown in fig. 23 for various well spacings and seepage exit lengths. Where a

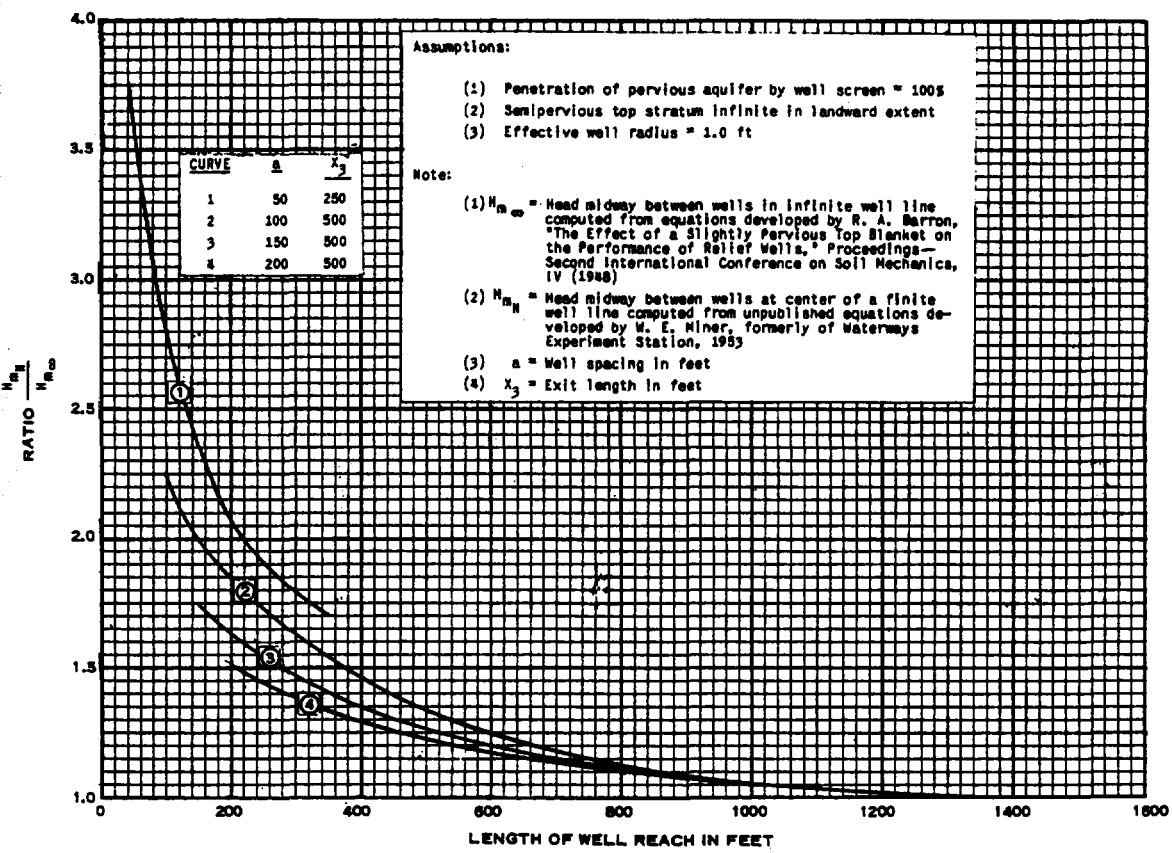


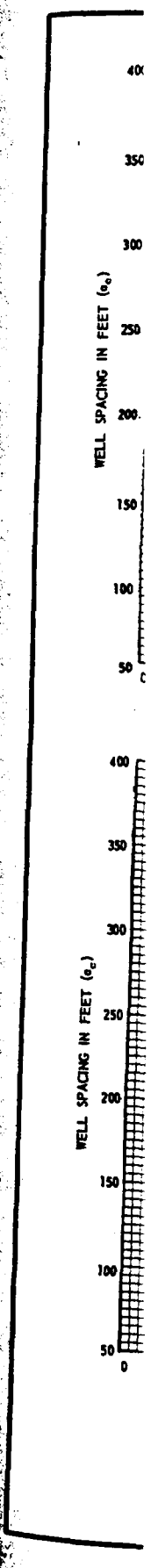
FIG. 23. RATIO OF HEAD MIDWAY BETWEEN WELLS AT CENTER OF A FINITE WELL SYSTEM TO HEAD MIDWAY BETWEEN WELLS IN AN INFINITE LINE OF WELLS

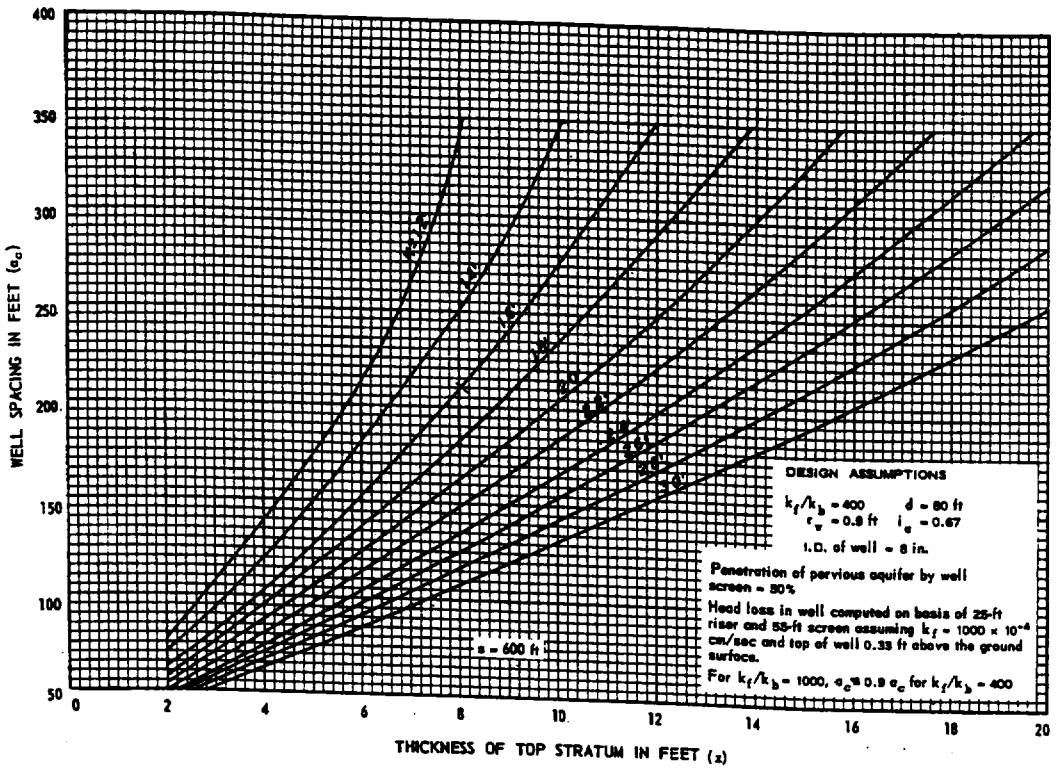
short line of wells was required, the spacing was computed for an infinite line of wells, and this spacing was then divided by the ratio indicated in fig. 23. Since the pressure tends to be higher at the ends of a finite well system than at the center, the screens for the end wells were set about 4 to 8 ft deeper than required for 50 per cent effective penetration, so as to obtain approximately the same head reduction as in the central part of the well system.

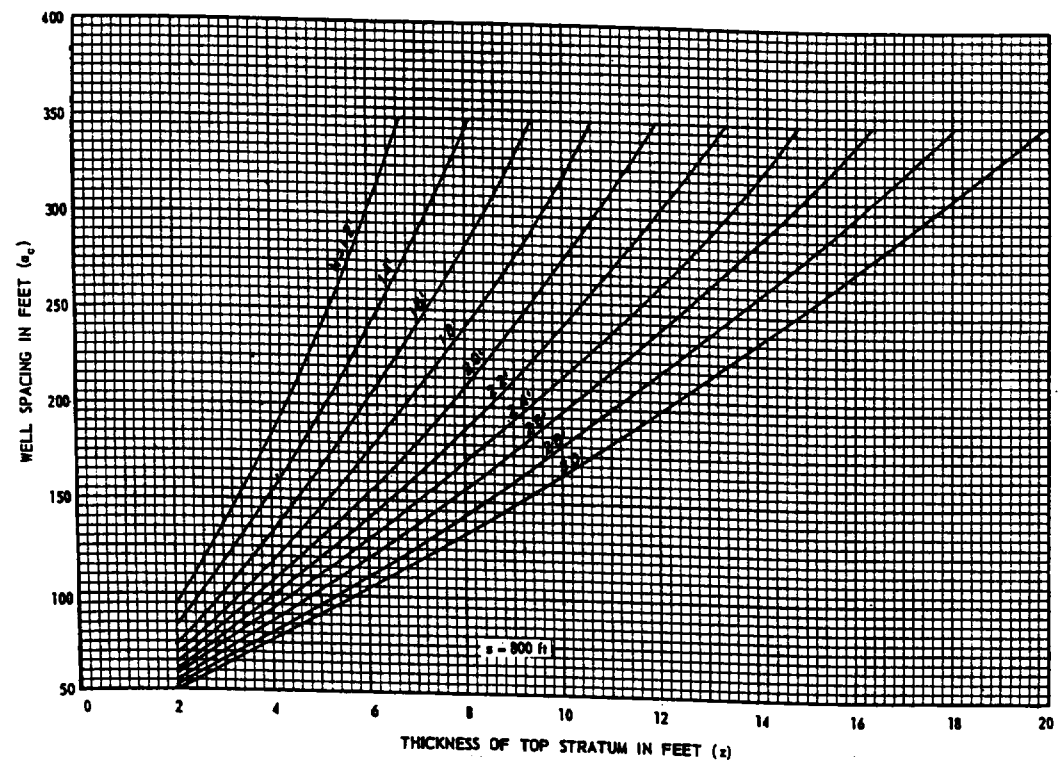
114. After determining the well spacing for a given reach of levee, the location of each well was checked in the office and in the field and adjusted where necessary so that the wells were located at critical seepage points and fitted natural topographic features.

115. Design curves. As the well formulas are somewhat complex, a set of design curves was developed for an infinite line of wells so that the computed well spacing a_c for $i = 0.67$ midway between wells, could be obtained directly from a chart for a given net head on the levee, thickness of landside top stratum, a permeability ratio of 400, and various distances to the effective source of seepage. These curves were developed for s = 600, 800, 1000, and 1500 ft and are shown in figs. 24 and 25. Where k_f/k_b was taken as 1000 ($z \geq 5$ ft of clay), the computed well spacing a_c was found to be about $0.9\ a_c$ for $k_f/k_b = 400$. In the East St. Louis levee district, where the allowable gradient was 0.50, the spacings were obtained from figs. 24 and 25 but were then multiplied by the ratio of the gradients ($0.50 \div 0.67 = 0.75$) to allow for the lower design gradient.

116. Curves showing the well flow per foot of net head Q_w/H for various top stratum thicknesses and well spacings are plotted in figs. 26 and 27 for $k_f/k_b = 400$ and for values of s = 600, 800, 1000, and 1500 ft, respectively. Similar plots for $k_f/k_b = 1000$ are shown in figs. 28 and 29. Values of Q_w/H on the above figures are based on $k_f = 1000 \times 10^{-4}$ cm per sec and d = 80 ft. The estimated flows from relief wells were based on the average k_f in the well reach as obtained from routine pumping tests and the effective aquifer thickness as obtained from borings and rock depth determinations in the reach. The latter flows were obtained by multiplying the value of Q_w/H from

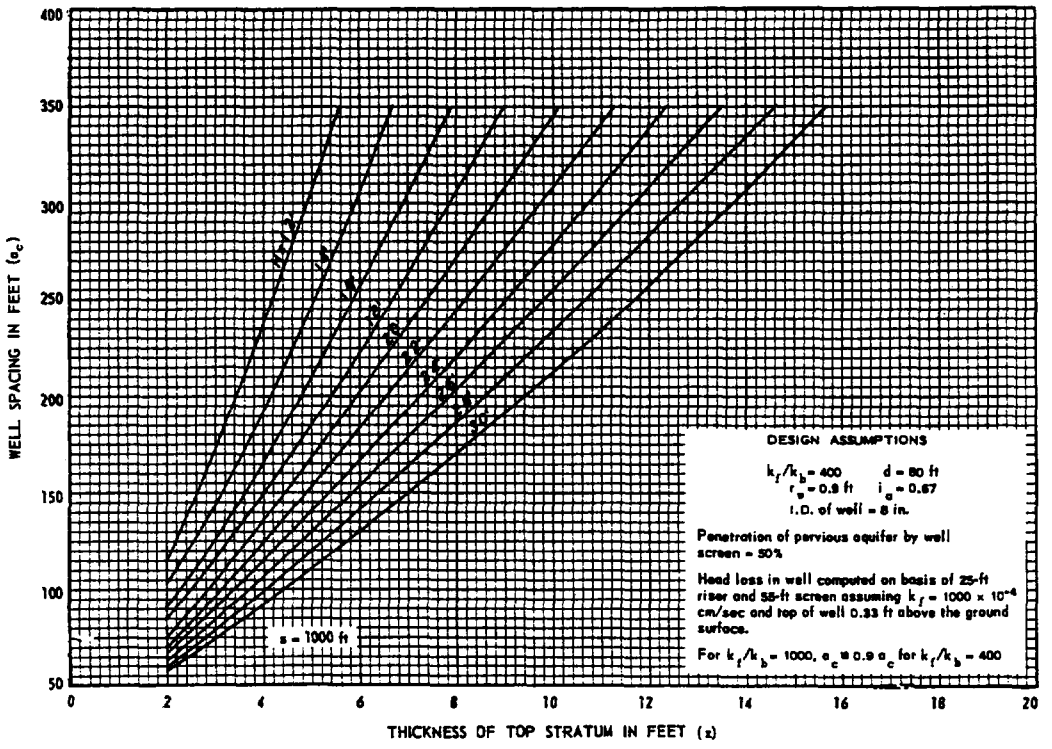


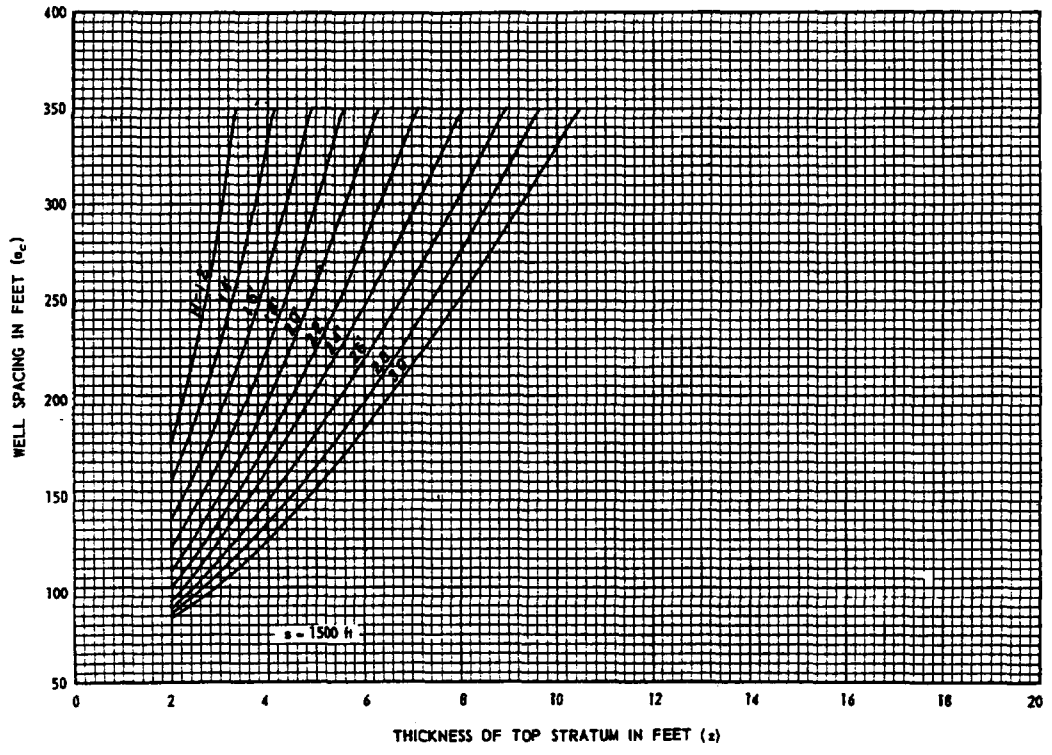




COMPUTED WELL SPACING
ADJUSTED FOR HEAD LOSS IN WELL
s=600 and s=800 ft

FIG. 24



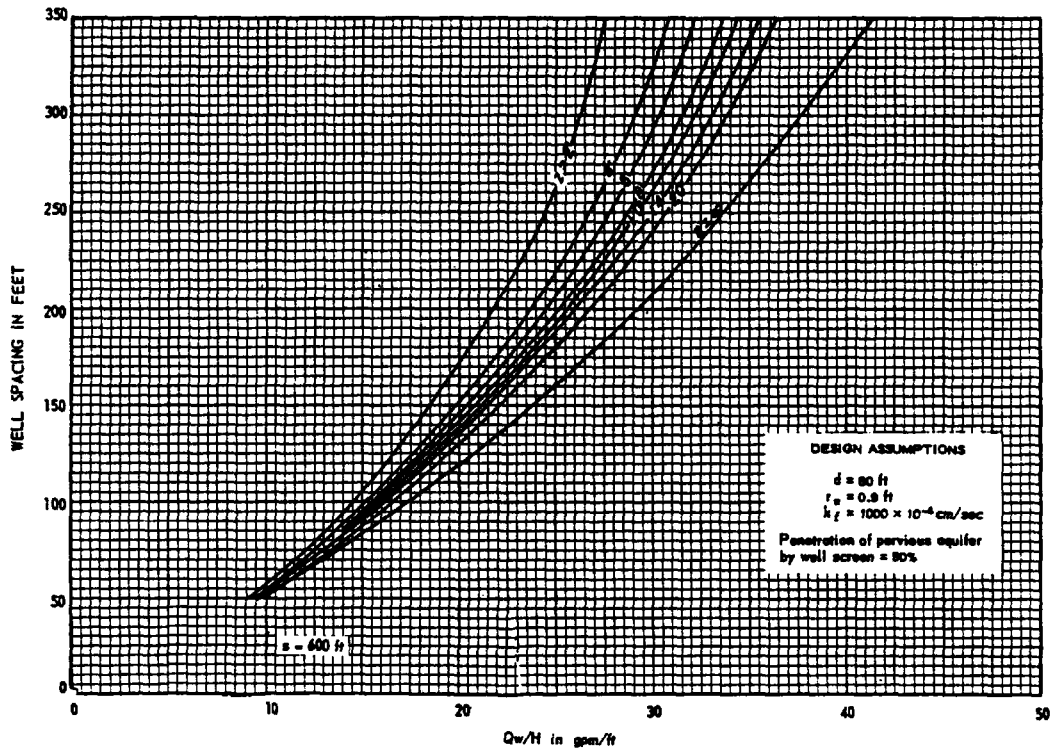


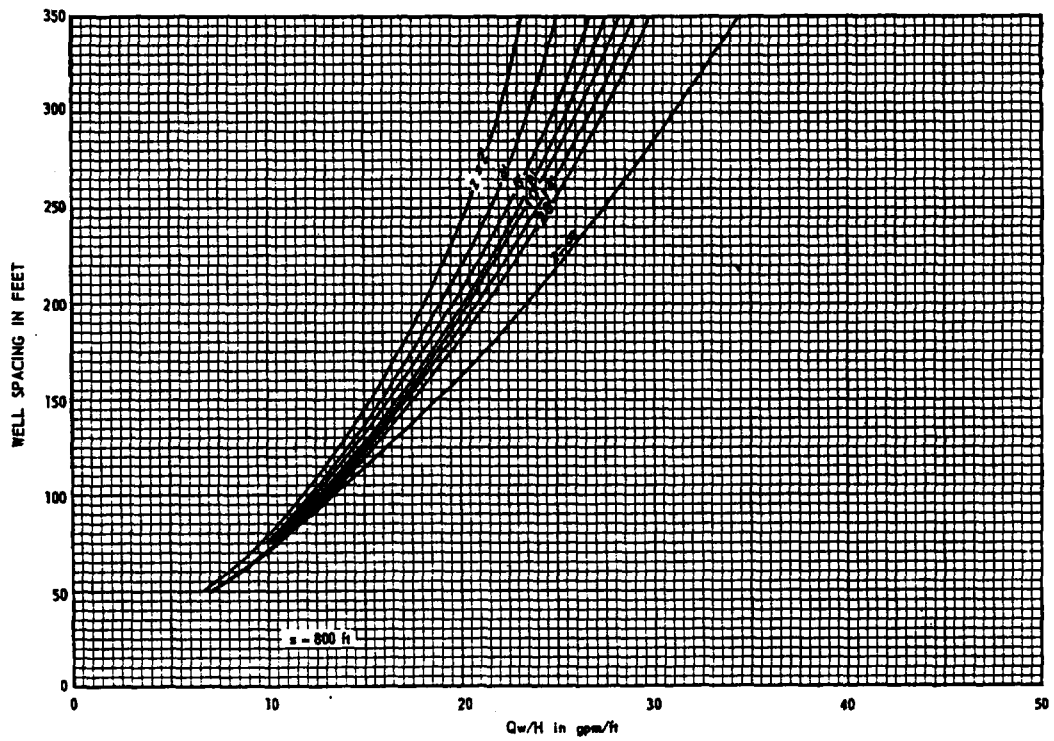
COMPUTED WELL SPACING

ADJUSTED FOR HEAD LOSS IN WELL

s=1000 and s=1500 ft

FIG. 25

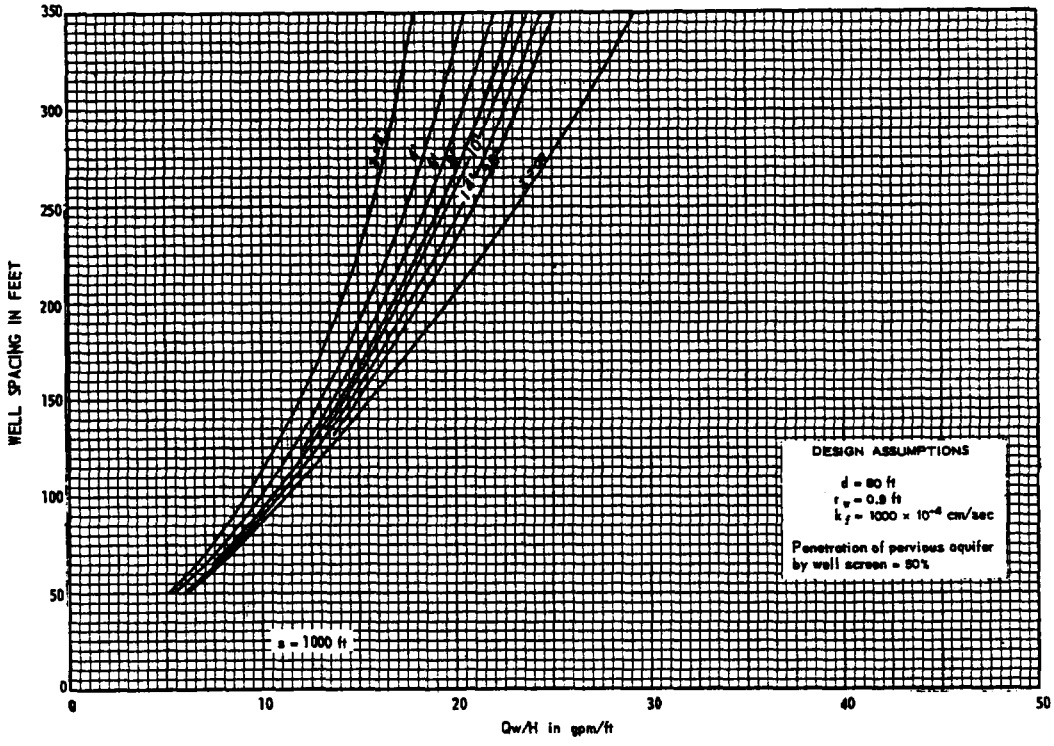


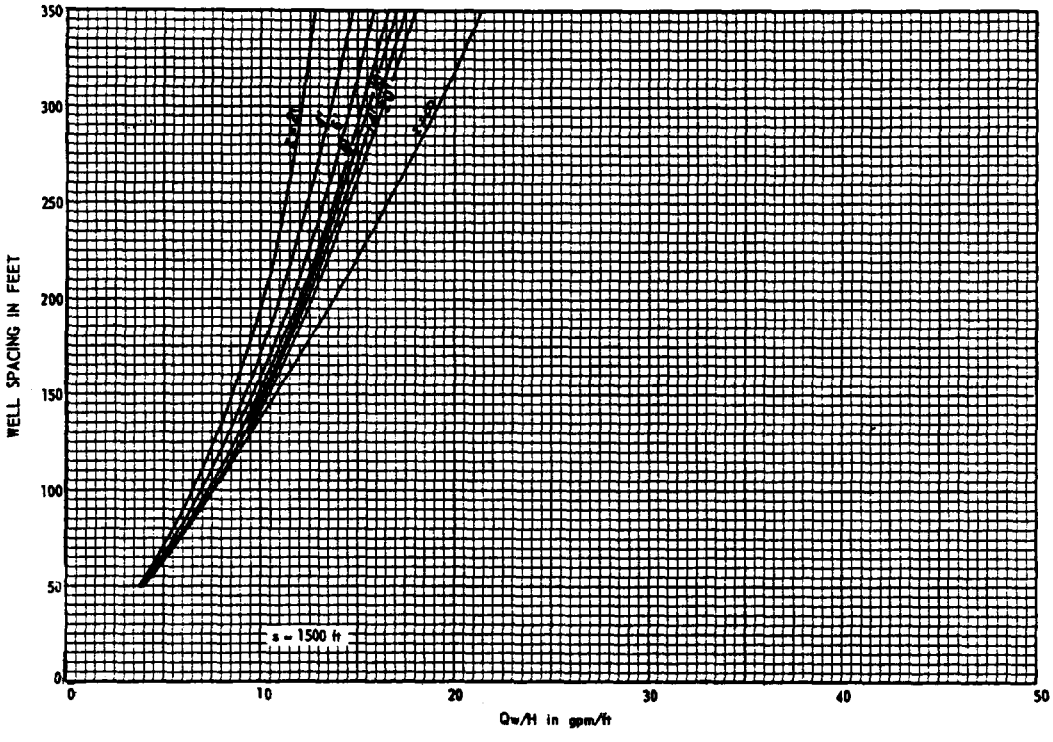


COMPUTED WELL FLOW

k_f/k_b=400, s=600 and s=800 ft

FIG. 26

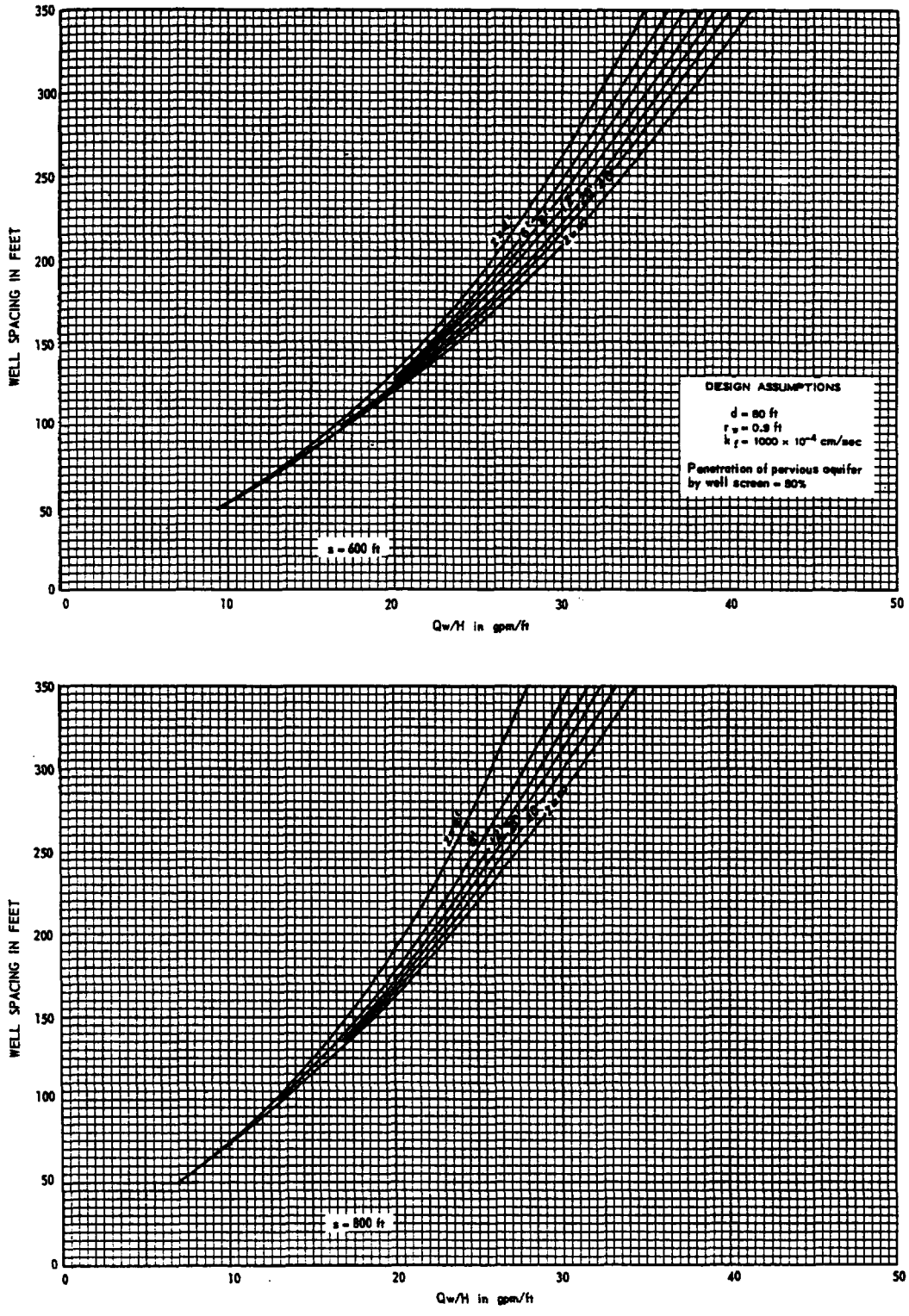




COMPUTED WELL FLOW

k_f/k_b=400, s=1000 and s=1500 ft

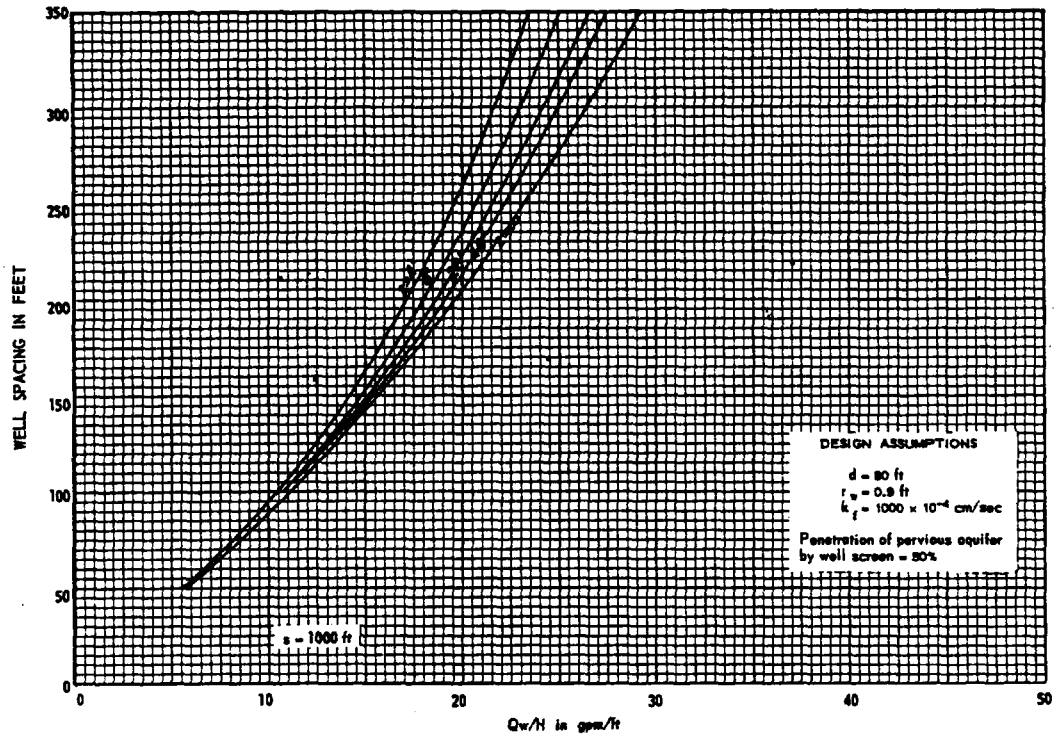
FIG. 27

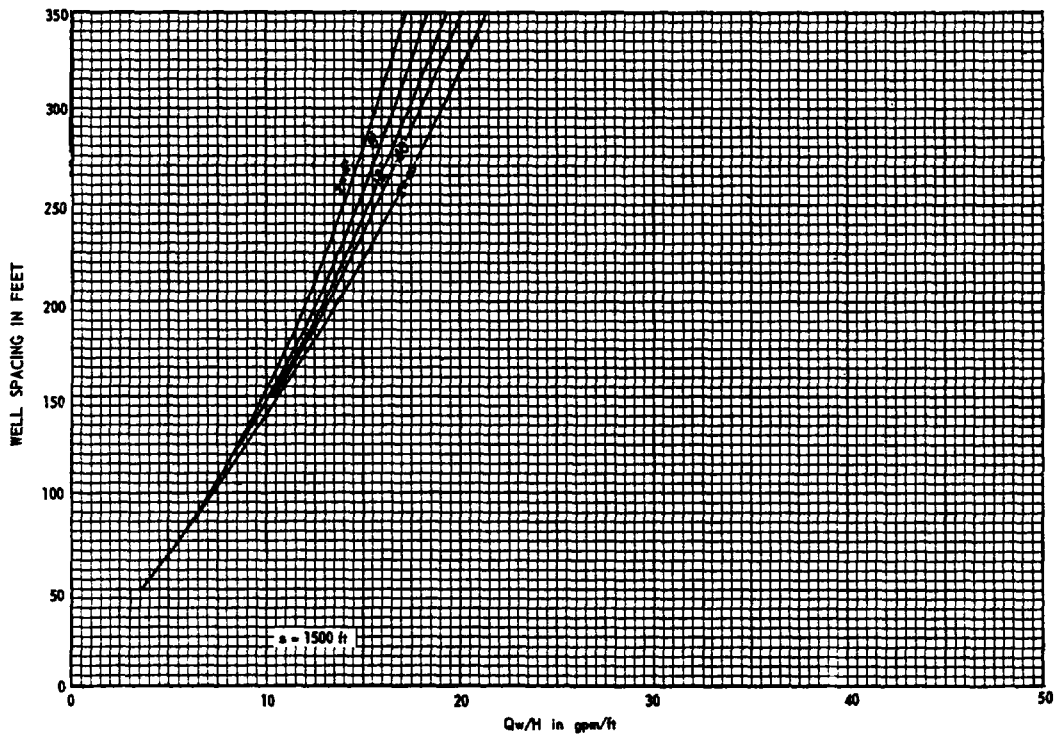


COMPUTED WELL FLOW

$k_f/k_b = 1000$, s = 600 and s = 800 ft

FIG. 28





COMPUTED WELL FLOW

k_f/k_b=1000, s=1000 and s=1500 ft

FIG. 29

figs. 26-29 by the ratio of the actual $k_f d$ to 1000×10^{-4} cm per sec x 80 ft.

117. Example. The use of the design curves is illustrated below. The computations pertain to the well reach between sta 40 and 43 in the Wood River (Upper) levee district, plate 14. The selected design values are as follow: H = 29.0 ft, s = 800 ft, k_f/k_b = 400, and z = 5.0 ft. From fig. 17, the head h_o beneath the top stratum at the landside toe of the levee would be 9.7 ft, which is greater than the allowable head of 0.85 z or 0.85 x 5 = 4.2 ft. Thus, by criteria previously stated, a well system would be required in this reach. Also, as previously stated, well systems were designed so that the maximum head between wells would not exceed 0.67 z or 3.3 ft. Thus, the head reduction to be accomplished by relief wells is:

$$h_o - h_a = 9.7 - 3.3 = 6.4 \text{ ft}$$

For the above conditions the computed well spacing a_c is about 95 ft (see fig. 24). As the above 300-ft reach of wells is part of a continuous reach of wells between about sta 22 and 148, the total length of well reach was 12,600 ft and, therefore, no reduction in well spacing was necessary (H_{m_N}/H_{m_∞} = 1.0, from fig. 23). The final well spacing selected was 100 ft.

118. Had no wells been required within about 400 ft upstream and downstream from the reach between sta 40 and 43, the total length of well reach would have been 300 ft. From fig. 23, H_{m_N}/H_{m_∞} = 1.63 for a = 95 ft and a 300-ft well reach. Thus, the required spacing of relief wells would have been 95/1.63, or about 60 ft.

119. The flow from the well system on 100-ft centers was estimated as follows: from fig. 26 for s = 800 ft, Q_w/H = 12.4 gpm per ft for $k_f = 1000 \times 10^{-4}$ cm per sec and d = 80 ft. In this well reach, k_f and d were actually about 2100×10^{-4} cm per sec and 75 ft, respectively. For the latter values of k_f and d

$$\frac{Q_w}{H} = 12.4 \text{ gpm/ft} \times \frac{2100 \times 10^{-4} \text{ cm/sec} \times 75 \text{ ft}}{1000 \times 10^{-4} \text{ cm/sec} \times 80 \text{ ft}} = 24.4 \text{ gpm/ft}$$

or

$$Q_w = 24.4 \text{ gpm/ft} \times 29.0 \text{ ft} = 705 \text{ gpm}$$

120. Where no landside top stratum was present landward of the levee, the spacing of relief wells was selected so that the wells would intercept about 50 per cent of the natural seepage beneath the levee.

121. The estimated natural seepage Q_s passing beneath the levee was computed from equation 8. From model studies[5], it was found that with no landside top stratum the increase in total quantity of seepage as a result of installation of relief wells was only about 5 per cent. Thus the value of Q_s computed from equation 8 is essentially valid with or without wells.

122. The well flow Q_w was computed from the following equation obtained from model studies[5].

$$Q_{w(100)} = \frac{11{,}250 \text{ H } k_f \text{ q}}{\text{q(s - 1,000) + 11,250}} \qquad (14)$$

where:

$Q_{w(100)}$ = well flow in gpm per 100 ft of levee

H = net head on levee in feet

k_f = horizontal permeability of the pervious substratum in feet per minute

s = distance from line of wells to effective source of seepage in feet

q = well or seepage flow in gpm per 100 ft of levee per foot net head as given in fig. 30.

As equation 14 was developed on the basis of d = 150 ft, flows for an aquifer with d = 80 ft were multiplied by the ratio 80/150.

123. The following numerical example illustrates the design of the well system between about sta 55 and 60 in Wood River (Upper) levee district (plate 14) where no landside top stratum was present. In the design it was assumed that $k_f = 1000 \times 10^{-4}$ cm per sec, d = 80 ft, s = 600 ft, and H = 28.5 ft. The total estimated seepage without wells was 540 gpm per 100 ft of levee from equations 8 and 9. It was considered desirable to intercept about 40 to 50 per cent of the total seepage

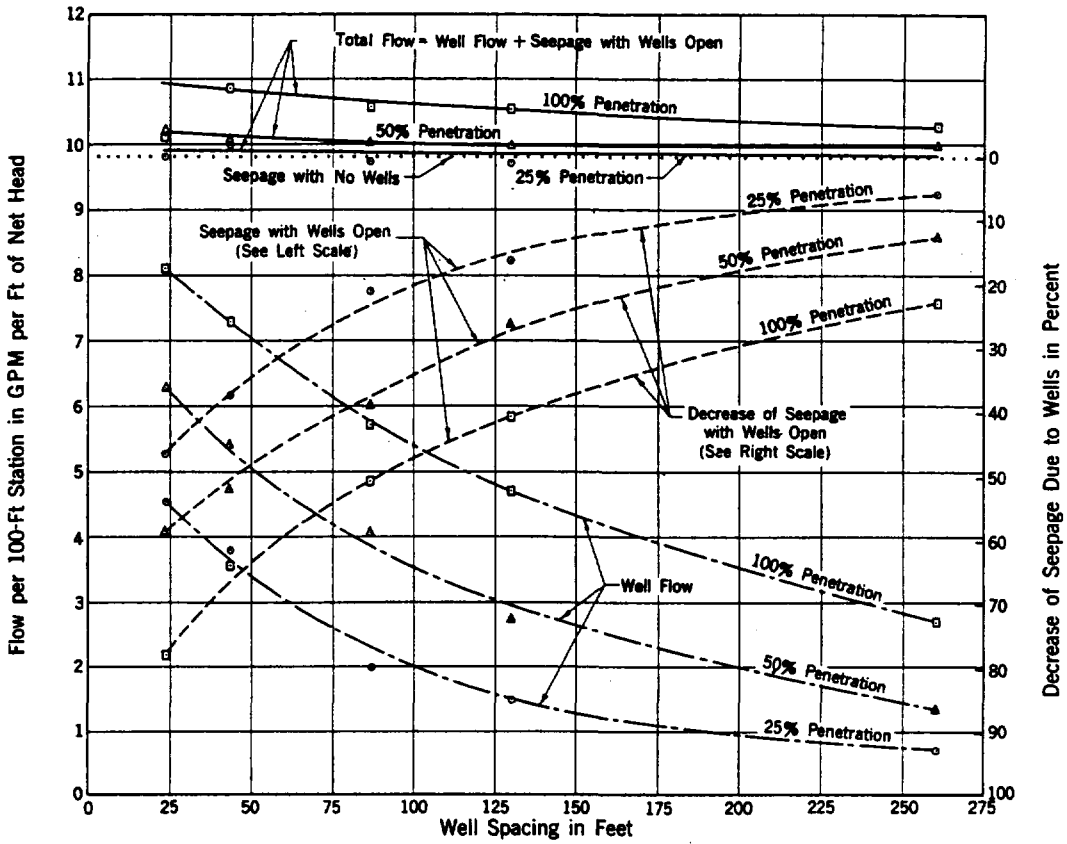


FIG. 30. WELL FLOW AND SEEPAGE FROM WES MODEL A-a-3, WITH s = 1000 FT AND d = 150 FT AND NO LANDSIDE TOP STRATUM

($Q_{w(100)}$ = about 215 gpm). From fig. 30, values of q were obtained for various well spacings a . Values of $Q_{w(100)}$ for the various well spacings were computed from equation 14 adjusted to d = 80 ft and the corresponding values of q and are tabulated below.

a in ft	q , gpm/ft Head per 100 ft of Levee	$Q_{w(100)}$ gpm per 100 ft of Levee	Ratio of Well Flow to Total Flow in %
50	5.0	205	38
75	4.2	165	31
100	3.6	140	26
125	3.0	105	19

124. From the above tabulation, a well spacing of about 50 ft would be required to intercept 40 per cent of the natural seepage. In view of the required close spacing, it was decided to install wells on 100-ft centers and defer installation of any additional wells until such time as the adequacy of the installed system could be checked during a

high-water period. It is noted that the k_f and d used in the calculations above do not agree with those shown on plate 14 because the k_f and d values shown on the plate were obtained from well installation data.

Seepage Berms

125. Two types of berms were designed according to the type of material to be used in their construction and are referred to as semipervious and sand berms. A semipervious berm is defined as a berm having a permeability about equal to that of the top stratum.

126. Seepage berms were proportioned so that the average upward gradient through the top stratum and berm was 0.60 at the toe of levees in agricultural districts and 0.50 in the East St. Louis levee district, or so that the respective factors of safety against uplift were 1.5 and 1.7. The allowable upward gradient at the toe of the berm was taken to be 0.85 in agricultural levee districts and 0.67 in the East St. Louis levee district. Where no top stratum was present landward of the levee the berms were designed so that the levee and berm would have a creep ratio of 15. Such berms generally consisted of clean sand.

127. Equations for computing the width X and thickness t of the berm at the levee toe are given below. Equations are not given for determining the required thickness at the crown, as a seepage berm theoretically tapers to zero thickness at the toe. The thickness of the berm at the crown was usually set at 1 to 2 ft. The surface of the berm should have sufficient slope to drain.

Sand berms

128. The nomenclature used in the design of semipervious berms is shown in fig. 31. The same nomenclature is applicable to sand berms except that sand berms were designed assuming that there is no change in the head landside of the levee compared to the head which would exist if there were no berm. The procedure followed in proportioning sand berms follows:

a. The residual head at the landside levee toe was computed by means of the formula

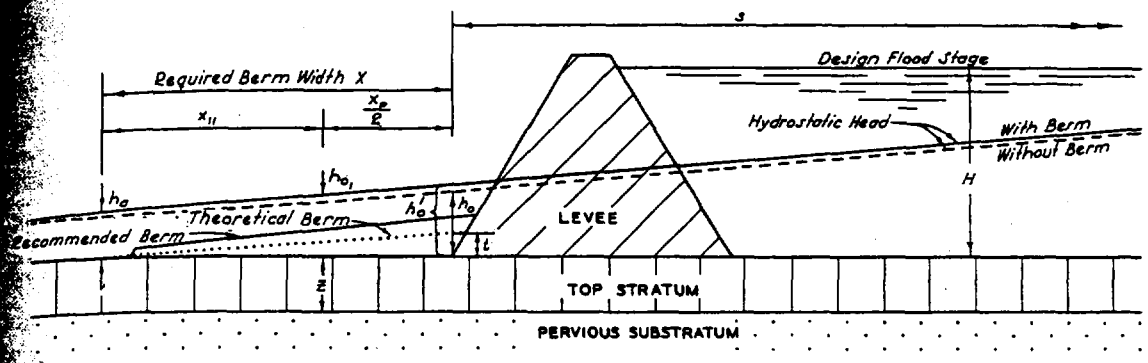


FIG. 31. NOMENCLATURE FOR DESIGN OF SEMIPERVIOUS LANDSIDE SEEPAGE BERMS

$$h_o = H \frac{x_3}{s + x_3} \tag{10}$$

b. The allowable head at the berm toe h_a was taken as 0.85 z in all levee districts except East St. Louis District where it was taken as 0.67 z.

c. The required width of a sand berm X_s was computed as the distance from the landside toe of the levee to a point where the head h_x at the berm toe would be equal to h_a. This width was determined from the following equation, assuming that the required width of a sand berm X_s was equal to that for a completely pervious free-draining berm X_p.

$$X_p = x_3 \ln\left(\frac{h_o}{h_a}\right) \tag{11a}$$

To simplify this computation it was found expedient to represent equation 11a graphically as shown in fig. 21, which shows the relationship between the ratio of head h_x at x distance landward of the levee to the head h_o at the toe and the distance x divided by the exit length x_3. By selecting h_a and substituting this value for h_x and knowing h_o and x_3, the distance $x\ (= X_p)$ at which $h_x = h_a$ can be found readily from fig. 21.

d. The required thickness t of the berm at the toe of the levee was found from the following equation:

$$t = \frac{h_o - z\left(\frac{\gamma' z}{F\,\gamma_w}\right)}{1 + \frac{\gamma' t}{F\,\gamma_w}} \tag{15}$$

wherein

h_o = head at landside toe of levee

F = factor of safety against uplift at toe of levee (1.50 in agricultural levee districts, 1.70 in East St. Louis levee district)

γ'_z = submerged unit weight of top stratum

γ'_t = submerged unit weight of berm

γ_w = unit weight of water

z = thickness of top stratum.

The above procedure for proportioning sand berms is based on the assumption that a sand berm is capable of conducting natural seepage entering the berm to its toe without any head loss. However, this is not a correct assumption, as a sand berm does not have sufficient capacity to conduct seepage entering it to its landside toe without considerable head loss. As a result, little seepage will flow horizontally; instead it is forced to rise to the surface of the berm with a resulting loss in elevation head. Therefore, widths of sand berms computed from the above equations are somewhat too narrow. At the time the berms were designed in this study, no formulas were available for the design of sand berms. Since then the following equation for determining the width of a sand berm X_s has been developed by the Waterways Experiment Station.

$$X_s = \frac{1}{3}\left(X_p + 2\,X_{sp}\right) \tag{16}$$

wherein

X_p = width of free-draining berm as obtained from equation 11a

X_{sp} = width of semipervious berm determined as described subsequently.

Semipervious berms

129. Semipervious berms were designed by an empirical method developed by the Kansas City District, CE, based on electrical analogy model studies in which it was assumed that the permeability of the berm would be the same as that of the top stratum. The addition of a semipervious berm on the landside of a levee restricts the natural relief of

pressure as a result of natural seepage through the top stratum and thus increases the hydrostatic head at the levee toe with respect to the original ground surface. This increase in head depends upon the length, thickness, and permeability of the berm. The electrical analogy model studies indicated that the effect of the berm on landside substratum pressures would be about the same as though the impervious base width of the levee had been increased a distance equivalent to 50 per cent of the width of the berm. The procedure for computing the width of semipervious berms was as follows (see fig. 31 for nomenclature):

a. The required width of a free-draining berm X_p was determined from equation 11a or fig. 21, as described above.

b. The value of $X_p/2$ was then added to s, and the head h_{o_1} at the point $X_p/2$ distance from the landside toe of the levee was computed from the following equation:

$$h_{o_1} = \frac{H\ x_3}{s + X_p/2 + x_3} \tag{10a}$$

c. A new distance x_{11} from the assumed impermeable section ($X_p/2$ distance from the landside toe) was computed by means of blanket formulas so that the head h_x at a distance x_{11} would then be equal to the allowable head h_a. The required width of semipervious berm X_{sp} is given by the following equation:

$$X_{sp} = \frac{X_p}{2} + x_{11} \tag{17}$$

130. To compute the required thickness of the berm it was necessary to estimate the head h'_o at the actual levee toe with the berm in place. This was accomplished by means of the following equation:

$$h'_o = \frac{H\left(\frac{X_p}{2} + x_3\right)}{s + \frac{X_p}{2} + x_3} \tag{18}$$

The average upward gradient through top stratum and berm was expressed by the following equation:

$$i = \frac{h_x - t}{z + t} \tag{19}$$

where t = berm thickness. From this equation the required berm thickness t at the levee toe can be found as follows:

$$t = \frac{h'_o - i_o z}{1 + i_o} \tag{19a}$$

where i_o is the allowable upward gradient at the levee toe.

131. After the semipervious berms were designed by the above procedure, an equation was developed by the Waterways Experiment Station for determining the required width X_{sp} of a semipervious berm from Bennett's blanket formula[2] and unpublished equations by Bennett for design of seepage berms. This equation is as follows:

$$X_{sp} = \frac{-A + \sqrt{A^2 - 24\,(2 + r)\left(1 + s\,c - \frac{H}{h_a}\right)}}{2\,c\,(2 + r)} \tag{20}$$

wherein

$A = 6 + 3\,s\,c\,(r + 1)$

$r = i_o/i_1$

i_o = allowable upward gradient at landside toe of the levee

i_1 = allowable upward gradient at landside toe of the berm

and H, h_a, s, and c are as described previously in this report. It should be noted that two values of X_{sp} are obtained from equation 20; the positive value of X_{sp} gives the required berm width. The head h'_o at the toe of the levee with the berm in place can be computed from the following equation:

$$h'_o = h_a \left[1 + c\,X_{sp} + \left(\frac{2 + r}{6}\right)\left(c\,X_{sp}\right)^2\right] \tag{21}$$

Equations 20 and 21 gave results very close to those obtained with the methods used in designing berms in the investigation.

132. The procedure described above and used in the investigation

for designing seepage berms is illustrated by the following numerical example which represents the design of the berm for the reach of levee between sta 1193 and 1198 in the East St. Louis levee district, plate 41. The design assumptions are as follow:

$$H = 24.5 \text{ ft}$$
$$s = 800 \text{ ft}$$
$$k_f/k_b = 400$$
$$z = 8.0 \text{ ft}$$
$$d = 80 \text{ ft}$$
$$k_f = 1000 \times 10^{-4} \text{ cm per sec}$$
$$i_o = 0.50$$
$$i_1 = 0.67$$

From fig. 17, the head h_o beneath the top stratum at the landside toe of the levee is 9.5 ft. From equation 5 or fig. 16, the distance to the effective seepage exit x_3 is

$$x_3 = \sqrt{\frac{k_f}{k_b} z\, d} = \sqrt{400 \times 8.0 \times 80} = 505 \text{ ft}$$

The allowable head h_a at the toe of the new berm is $i_1 z = 0.67 \times 8.0 = 5.3$ ft. The required width of a free-draining berm X_p is determined as follows:

$$\frac{h_a}{h_o} = \frac{5.3 \text{ ft}}{9.5 \text{ ft}} = 0.56$$

From fig. 21, $x/x_3 = 0.58$ for the above ratio. Thus the required width of free-draining berm X_p $(= x)$ is:

$$X_p = 0.58\, x_3 = 0.58 \times 505 \text{ ft} = 294 \text{ ft}$$

The required width of a semipervious berm is determined as follows. The impervious base width of levee is now assumed to be $X_p/2$ or 147 ft wider as a result of adding a berm. From equation 12a, the head at the landside end of the assumed impervious section is

$$h_{o_1} = \frac{24.5 \times 505}{800 + 147 + 505} = 8.5 \text{ ft}$$

The required distance x_{11} landward from the impervious section for h_x equal to h_a is determined as follows:

$$\frac{h_a}{h_{o_1}} = \frac{5.3 \text{ ft}}{8.5 \text{ ft}} = 0.63$$

From fig. 21, $x_{11}/x_3 = 0.46$ for the above ratio, or $x_{11} = 0.46 \times 505$ ft = 233 ft. The required width of berm is then

$$X_{sp} = \frac{X_p}{2} + x_{11} = 147 + 233 = 380 \text{ ft}$$

The required thickness of the berm at the landside toe of the levee was determined from the estimated head h'_o at the levee toe with the berm in place. This head was determined from equation 18:

$$h'_o = \frac{24.5\ (147 + 505)}{800 + 147 + 505} = 11.0 \text{ ft}$$

The thickness t of the berm required for an average upward gradient at the toe of the levee of $i_o = 0.50$ is then found from equation 19a:

$$t = \frac{h'_o - i_o z}{1 + i_o} = \frac{11.0 - 0.50 \times 8.0}{1 + 0.50} = 4.7 \text{ ft}$$

The dimensions of the final berm were altered somewhat from the computed values of X_{sp} and t, because of irregularities in the surface topography. Furthermore, as the above berm is part of a reach with a continuous berm between sta 1165 and 1210, the dimensions required for the berm between sta 1193 and 1198 and adjacent berms were grouped together and average values were selected to reduce the number of transitions between berms and thereby simplify construction.

PART VII: UNDERSEEPAGE INVESTIGATION AND DESIGN OF CONTROL MEASURES FOR INDIVIDUAL LEVEE DISTRICTS

General Design and Installation Features

133. The locations of the various levee districts with respect to each other and the Mississippi River from Alton to Gale, Ill., have been shown in fig. 1. General descriptions of the alluvial valley, geology, and characteristics of the top stratum and pervious substratum have been presented in parts I through IV. More detailed descriptions of conditions and features affecting underseepage and the design of control measures in the individual levee districts are presented in this part.

134. General procedures and assumptions common to the study of underseepage, design of relief wells and seepage berms, and the location of piezometers in all of the levee districts are described in the following paragraphs.

Seepage and ponding in levee districts

135. Data concerning the degree of seepage in each levee district were obtained from seepage and flood damage maps and reports of inspections made during high-water periods, both of which were furnished by the St. Louis District. Water impounded within the levee districts during high river stages consisted of both seepage and normal runoff of rainfall.

136. Interior drainage for the levee districts is provided by a system of drainage ditches and natural sloughs and streams which discharge into tributary streams or the Mississippi River through drainage structures in the levees. During high water these drainage structures are closed, thus causing natural runoff and seepage to pond in sloughs, swales, or other low areas, except in certain parts of industrial levee districts where some of the surface and seepage water is pumped over the levee.

137. The elevation to which the ponded water is likely to rise during high water was difficult to predict. Wherever possible it was

estimated either from previous high-water marks or from past seepage records and flood reports. (Relatively few elevations of tailwater landward of the levee could be located during the investigation. However, the elevations of water in ditches landward of drainage culverts in several levee districts during the 1951 high water have been found and are presented in the subsequent sections on the individual levee districts.) In the design of relief wells along levees the tailwater was generally assumed at the natural ground surface. However, where the toe of the levee was in a low area known to pond water during high water, the tailwater was estimated as described above. At inlet ditches to pumping plants the minimum elevation of water in the ditch generally was obtained from operating manuals for the pumping plants. Where there are ditches near the toe of the levee and no pumping plant is present, the elevation of the water in the ditch was assumed about 2 to 5 ft below the natural ground surface which would be drained by the ditch in anticipation of possible construction of pumping plants at a later date.

Design of seepage control measures

138. Relief well systems. The design of well systems was based on criteria set forth in part VI and existing conditions at the time the underseepage investigation was made. The spacing of wells was based on the curves shown in figs. 24-25. The design assumptions and well spacings for each district are shown in the tables, "Design of Relief Wells," presented in volume II. Deviations from the design spacings were made in some cases to allow for certain topographical and geological features. Before any levee was analyzed for underseepage or control measures were designed, the levee and foundation conditions were divided into reaches wherein the head, top stratum, and source of seepage were considered approximately the same.

139. The well flows shown in the tables generally were based on the design curves shown in figs. 26-29. These design curves were based on an infinite line of wells, a thickness of pervious substratum of 80 ft, permeability of pervious aquifer of 1000×10^{-4} cm per sec, and an effective penetration of the pervious aquifer of 50 per cent by the well screen. The computed flows for wells shown in the above-mentioned

tables were obtained by multiplying the value of Q_w/H obtained from figs. 26-29 by the ratio of the actual depth of pervious substratum to the assumed depth of 80 ft and by the ratio of the permeability of the substratum as determined from the pumping tests on the wells to the assumed permeability of 1000×10^{-4} cm per sec.

140. Whether relief wells should be installed along ditches immediately landward of a levee was based on the estimated head beneath the ditch as though the ditch were not present. Where needed, wells were spaced along the ditch to reduce the head to that allowable beneath the bottom of the ditch. Where relatively wide landside borrow pits are close to the levee, the need for and design of relief wells were based on the thickness and characteristics of the remaining top stratum beneath the borrow pit.

141. The elevation of natural ground shown in the design of relief well tables is that at the toe of the levee or berm. If a landside low area exists near the levee, the assumed tailwater in the low area may be lower than the elevation of the natural ground. Where this was the case, this assumed tailwater elevation in the low areas was used to compute the net head on the levee and the required spacing of relief wells.

142. Flows from single wells were computed by use of the formula

$Q = \dfrac{2\pi\, k\, d\, h}{\ln \dfrac{2\, s}{r_w}}$ (which is for an impervious top stratum), reduced by the factor for partially penetrating wells obtained from Kozeny's equation as shown in fig. 12, and adjusted to compensate for head losses in the wells in the following manner: the head loss in the pervious stratum and in the wells was plotted for various flows on the same graph; then the head loss curve for the wells was added to the head loss curve for the pervious stratum to obtain the head loss through both the pervious stratum and the wells. The flow at which the combined head losses equaled the head available was taken as the actual flow that would be obtained from the well.

143. Permeabilities of substratum sand were computed from pumping tests on partially penetrating wells as discussed in part V. The permeability of the pervious substratum along each group of wells in each levee district shown in the "Design of Relief Wells" tables is the average of the k_f values computed from the pumping tests on the wells in the group. Where only part of the wells were installed in a district, the permeability of the substratum at the remaining wells was assumed to be the average of permeabilities where tests had been made.

144. Specific yields of the wells, as determined from the pumping tests and shown in the tables entitled "Summary of Relief Well Installation Data" in volume II, were computed by dividing the flow by the drawdown in the well after the drawdown had become stabilized. (Head losses through the filter and screen and in the well were not deducted from the drawdown in the well in computing the specific yields.) However, most of the wells were pumped at a rate of about 500 gpm and thus the specific yields shown for the wells are more or less comparable. The specific yields shown in this report can be compared with yields from future pumping tests if the wells are pumped at about the same rate. (The specific yield for a 1-ft drawdown should be about 10 to 20 per cent more than those shown in the tables.)

145. The numbers of relief wells designed for and installed in the various levee districts are given in table 3. A summary of all relief well installation data including pumping tests and specific yields received prior to 15 December 1955 is shown on installation data tables for each district in volume II. The location, depth, length of screen, and specific yield of all wells that have been installed and locations of wells designed but not installed are shown on the soil profile plates in volume II. The legend used is shown on plate 3. As may be seen from the summary of relief well installation data, cobbles were encountered in numerous well reaches in some of the levee districts. This sometimes resulted in installation of wells at less than their design depth, as is shown in the summary sheets of relief well installation data.

146. All wells shown with the suffix "A" were relocated because of inability to install a satisfactory well at the location originally

Table 3

Number of Relief Wells Designed and Installed

Levee District	Number of Old Wells in Existence on 1 Oct 1952		New Wells			
	Total	Stable	Number of Design Wells	Number of Satisfactory Wells Installed	Wells To Be Installed	Wells To Be Replaced
Wood River	11	7	148	119	45	4
East St. Louis	4	0	290	227	59	0
Chain of Rocks	195	195	---	---	0	0
Prairie du Pont	0	---	97	98	0	0
Columbia	0	---	128	128	0	2
Harrisonville	0	---	219	219	0	2
Ft. Chartres	4	0	181	181	0	2
Prairie du Rocher	0	---	155	155	0	0
Perry County	3	3	278	97	186	0
Degognia	2	Filled with rock	182	189	11	4
Grand Tower	0	---	139	147	0	0
Preston	0	---	107	0	107	-
Clear Creek						
Front Levee	21	?	119	0	119	-
Back Levee	---	---	13	0	13	-
East Cape Girardeau	0	---	99	66	33	0

proposed. However, some wells were relocated but were not numbered with the suffix "A."

147. The suffix "X" denotes extra wells installed as a result of respacing wells in accordance with the procedure described in paragraph 522. Where wells were respaced in reaches where piezometers had been installed, the piezometers were no longer midway between the respaced wells; if a piezometer was too close to a respaced well, a new piezometer was installed midway between the wells.

148. Seepage berms. Seepage berms were designed in accordance with the criteria set forth in part VI. The design assumptions are shown

in volume II in the tables entitled "Design of Seepage Berms" for the respective levee districts. Typical cross sections of berms are shown on plates for each district for which berms were designed. Locations of the berms are shown on the air-photo maps. The numbers of seepage berms designed and constructed in the various levee districts are given in table 4.

Table 4

Number of Seepage Berms Designed and Constructed

Levee District	Total Designed	Number Constructed
East St. Louis	4	3
Prairie du Pont	2	2
Columbia	5	4
Ft. Chartres	1	0
Grand Tower	2	2
East Cape Girardeau	1	0
	15	11

149. Piezometers. The numbers of piezometers installed in the various levee districts are given in table 5.

Table 5

Number of Piezometers Installed

Levee District	Installed Prior to 1953 (To Be Maintained)	Satisfactory Piezometers Installed Since 1953	To Be Installed
Wood River			
Upper	0	23	0
Lower	0	14	0
East St. Louis	18	91	50
Chain of Rocks	11	0	0
Prairie du Pont	4	46	0
Columbia	2	70	0
Harrisonville	0	104	0
Ft. Chartres	0	90	3
Prairie du Rocher	0	84	0
Perry County	8	0	113
Degognia	8	59	6
Grand Tower	0	51	1
Preston	4	0	63
Clear Creek			
Front Levee	37	0	48
Back Levee	0	0	8
East Cape Girardeau	9	51	1
North Alexander	0	4	1

A summary of piezometer installation data for each levee district received prior to 15 December 1955 is shown in the tables entitled "Piezometer Installation Data" in volume II. The locations of piezometers are also shown on the soil profile plates for each district and on the air-photo maps. Piezometers were generally placed in each group of wells midway between pairs of relief wells near the ends and in the middle of the line to check the adequacy of the well system during high water. Piezometer lines were also installed perpendicular to the levee at selected locations where the top stratum appeared to be uniform over a sizable area. These lines of piezometers will afford data from which the effective seepage source and exit can be determined. They are shown in volume II on plates entitled "Piezometer Lines and Boring Logs" for each levee district. A number of piezometers were installed at locations outside of well reaches to check the hydrostatic pressure beneath the top stratum where the necessity for seepage protection measures was in doubt. If, based on the piezometric data, excessive pressures are predicted to develop before the river stage becomes equal to the net grade of levee, relief wells or seepage berms can be installed at a later date.

150. Most of the piezometers installed by the St. Louis District prior to the beginning of this investigation in October 1952 were retained wherever they were in good condition and the tips were installed in the pervious substratum sands. However, piezometers that are no longer considered necessary, or whose tips were installed in silt or clay, were abandoned and are not included in the "Piezometer Installation Data" tables. Former piezometers which have been incorporated in the new layout have been renumbered and are included in the above-referenced tables and on the soil profile sheets.

151. The tips of some of the new piezometers were inadvertently installed in silt or clay strata; however, most of these have been replaced with new piezometers with tips set in sand. The piezometers with tips set in silt or clay, which have been replaced, are not to be read or maintained.

152. Except for piezometers on lines perpendicular to the levee,

most of the others were installed either at the landside toe of the levee or in the well line. However, where it was known that surface water landward of the levee would be impounded to a depth greater than 30 in. during high water, the piezometers were located on the levee slope or at the crown of the berm to facilitate reading.

153. Cutoffs. The St. Louis District is making an investigation of reaches where the levee is founded on silty or clean sand underlain by less pervious strata. Where the sandy strata are of sufficient thickness, they are to be cut off by means of clay-filled trenches along the riverside toe of the levee. A design memorandum summarizing the installation of cutoffs is to be prepared by the St. Louis District. The locations of cutoffs that have been constructed are given in the discussions of the individual levee districts and are shown on the air-photo maps.

154. Man-made seepage hazards. A potential seepage hazard exists where cisterns, wells, cesspools, or basements are located within 300 ft of the levee and are currently in use so that they cannot be filled. These features should be recorded and should be inspected during future high-water periods as it may be necessary to take emergency measures to control seepage at such points. Wells do not present a problem if properly constructed. During periods of extremely high water it may be necessary to fill cisterns with water to prevent flotation.

Wood River Levee District

155. The Wood River levee district is located in Madison County, Ill., approximately 15 miles north of St. Louis, Mo. The district begins at Alton, Ill.; it is bounded on the west by the Mississippi River, on the south by the East St. Louis levee district, and on the east and north by the valley wall. The levees in the Wood River levee district are shown on maps 2-15 and 17 of reference 10.

156. The levee district is divided into three separate areas (described in the following subparagraphs) and the stationing of the levees for each area is independent of that for the other areas. Design of underseepage control measures was based on river-front grades proposed

prior to September 1954. The grade of the levee as finally approved in November 1954 is about 2 ft higher than that used in evaluating the need for underseepage control or in designing control measures. The design and installation of relief wells in this district were completed prior to the time the grade of the levee was raised.

a. Upper Wood River. This area lies on the south side of the city of Alton. The levee starts at sta 0+00 at high ground opposite Lock and Dam No. 26 on the Mississippi River; it continues downstream along the Mississippi River as river-front levee to approximately sta 180; and then up the right bank of Wood River as flank levee to high ground at sta 274+31. The river-front levee ranges in height from 20 to 35 ft and is about 300 to 700 ft from the Mississippi River. The flank levee varies in height from 12 to 25 ft and the distance from the levee to Wood River varies from 200 to 500 ft.

b. East and West Forks, Wood River. This area lies to the east of the city of Alton and between the East and West Forks of Wood River. The levee starts on high ground at sta 0+00 on the right bank of East Fork, continues in a generally U-shaped pattern past the junction of East and West Forks, and thence up the left bank of the West Fork to sta 145+70. The levee ties into high ground between sta 108+50 and 119+00. The entire levee in this section is classed as flank levee; it is from 15 to 20 ft in height, and the distance from the East and West Forks varies from 50 to 350 ft. No underseepage control measures were required for the levees in this area.

c. Lower Wood River. This area includes the cities of East Alton, Wood River, Roxana, and Hartford. The levee starts on high ground at sta 0+00 on the left bank of East Fork and continues as flank levee to approximate sta 160 where it becomes river-front levee on the Mississippi River. The river-front levee extends from sta 160 to sta 412 where the levee turns up the right bank of the Cahokia Creek Diversion Channel to approximate sta 553. Thence the proposed levee extends along the east side of the CCC & St. L Railroad embankment to approximate sta 660 where it crosses the railroad embankment and ties into high ground at sta 691+70 near the community of Wanda. The height of the upper flank levee varies from 10 to 25 ft and the distance from the tributary creek is from 50 to 200 ft. The river-front levee is from 10 to 30 ft in height and from 200 to 3100 ft from the Mississippi River. The lower flank levee ranges from 10 to 20 ft in height; between sta 412 and sta 553 the distance from Cahokia Creek varies from 200 to 400 ft.

Upstream from sta 553 the levee is as far as 5000 ft from Cahokia Creek. The levee in Lower Wood River has not yet been completed.

157. The upper and lower areas of the district are divided by Wood River which has been straightened to shorten its length. The old meandering channel of Wood River may be seen on maps 4, 5, and 9. Several ditches and sloughs and the old course of Wood River provide interior drainage for the levee district. Drainage structures or pumping stations are provided where major drainage channels cross the levee. The low area along the river-front levee in Upper Wood River is drained by a pumping station at sta 50+46 and by a gravity drainage structure at sta 124+27. Two of the drainage structures in Lower Wood River located along Wood River at sta 11+50 and 61+67 are provided with pumping plants. One drainage structure located at sta 236+75 (Lower) along the Mississippi River is also provided with a pumping plant.

Geology and soil conditions

158. The flood plain is roughly triangular in shape having a corner at the upper end of the district where the east valley wall forms the left bank of the Mississippi River and widening to several miles at the lower end of the district at Cahokia Creek. The location and extent of channel and swale filling deposits in the flood plain are for the most part masked on the aerial photographs by civic and industrial developments. The area adjacent to the Mississippi River has not been developed to any appreciable extent and a clearer picture of the geology of this area can be obtained from the levee maps. Along most of the river-front levee in Upper and Lower Wood River, the deposits appear to be similar to point-bar deposits with long ridges and swales having gentle curvatures. The area between sta 50+00 and 150+00, Upper Wood River, is generally low and has a number of sloughs which have been cut off from the river by the present levee system. The uppermost 20 to 30 ft of alluvium in the area along the levees flanking Wood River in both areas of Upper and Lower Wood River probably were deposited on the flood plain by Wood River. In Lower Wood River between sta 315 and 324, the levee appears to follow along the edge of channel-filling deposits. From

sta 329 to the end of the levee, sta 691+70, the deposits are of undetermined geological origin, although between sta 492 and sta 515 the levee appears to cross a filled channel of the Mississippi River.

159. The East and West Forks area lies in an eroded valley on the escarpment between the two forks of Wood River. The levee is founded on the alluvium deposited by these streams.

160. The soil profile along the landside toe of the levee is shown on plates 4 through 11. Cross sections and logs of borings made at locations pertinent to the study and design of seepage control measures are shown on plates 12 and 13. Only limited soils information was obtained along the levees in the East and West Forks because the river stage used to design seepage control measures generally was only about 5 ft above the ground surface. The elevation of rock as found by seismic determinations varied from 279 to 315 ft msl in Wood River (Upper), and from 294 to 302 ft msl in Wood River (Lower). The deepest point of the valley floor in Wood River levee district was found at sta 146 (Upper) where the alluvium is 133 ft deep.

History of seepage

161. During the 1951 high water, light seepage was reported at sta 90 and 190 in Upper Wood River, and areas landward of the levee between sta 48 and 110 were submerged from a combination of seepage and industrial sewage. Seepage ponded in an area between sta 186 and 217 in Upper Wood River during the 1951 high water. In Lower Wood River an area between sta 100 and 225 flooded because the levee system in this area had not been completed prior to the 1951 high water.

162. The tailwater reached about el 422 between about sta 48 and 110 in Upper Wood River during the 1951 high water. Tailwater elevations in other areas in which water ponded during the 1951 high water are not available.

163. No piezometers had been installed in the Wood River levee district prior to 1952.

Design of relief well systems

164. Factors used in the design of relief wells in the Wood River levee district are given in the table on plate 14.

165. Upper Wood River. This area has been subdivided by stations, and the relief well design for each subarea is described in the following subparagraphs.

a. Sta 21 to 40. Low ground exists immediately landward of this reach of levee for a distance of 50 to 500 ft. Where the low ground was less than 150 ft in width, it was originally suggested that the area be filled with a sand berm starting on the slope of the existing berm at el 417.5 and sloping landward to high ground with a slope of 1 on 100. Where the area was wider than 150 ft, a similar berm was suggested except that its width was restricted to 150 ft. However, it was subsequently decided to install relief wells along this reach of levee as shown on plates 4 and 14, because they were more economical than a sand berm.

b. Sta 40 to 55. A landside ditch carrying storm water and sewage approaches the levee obliquely from an upstream direction at sta 40 and approximately parallels the existing berm toe to the pumping plant located at sta 50+46 (see map 2). The thickness of top stratum assumed in the design of relief wells along this reach of levee was 5 ft, the approximate thickness in the bottom of the ditch landward of the levee. The tops of the relief wells along this reach were set at about el 415.5, approximate ground surface along the toe of the levee. This elevation is about 2.5 ft above the minimum allowed by pumping regulations. However, during high floods, it is believed that the water in this area may be allowed to pond somewhat higher than the minimum allowed by the regulations. Nine wells were installed along the reach of levee from sta 47+56 to 54+85 in 1952. These wells have been subject to backflooding by contaminated industrial waste and were pump-tested in 1954 to determine their capacity and stability. Installation data and the results of pumping tests performed on these wells at the time of their installation, and in 1954, are presented on plate 18. Well A-2 was found to be unstable and wells A-6, A-7, and A-9 had apparently become clogged to the extent that they could not be rejuvenated. Accordingly, these four wells are to be plugged and will be replaced with new wells at locations shown on plate 18. The A series of wells are equipped with vertical-type flap gates. The flap gates on the A wells are to be replaced in 1956 with check valves and well outlets of the same type used on the new wells in Upper Wood River.

c. Sta 55 to 144. Relief wells along this reach of levee were installed on small auxiliary berms 15 ft wide constructed to el 413 along the toe of the existing berm.

These auxiliary berms provided access for observation and maintenance of the relief wells. Along this reach of levee, the wells were designed to protect the relatively thin top stratum beneath landside low areas particularly where the low areas or ditches are close to the levee (see plate 4). Wells were installed between about sta 55 and 64 (where no top stratum is present) to intercept about 50 per cent of the estimated natural seepage.

Two wells were installed at sta 124+16 and 124+38 in 1952. These wells have since been pump-tested and found to be in good condition (see plate 18).

Between sta 34 and 144, the maximum net head on the levee is about 28.5 ft. Low areas landward of the levee along this reach will be submerged during high-water periods. The design of the wells along this reach of levee was based on a tailwater elevation equal to the minimum pumping elevation (413.0 ft msl) for the Alton pumping station at sta 50+46. This portion of the relief well system is subject to submergence by industrial waste and to insure efficient and continued functioning of the well system, special precautions were taken to prevent backflow into the wells. To safeguard the wells from contamination, each well was provided with 8-in. tee outlets and a double check valve (a rubber gasket and aluminum check valve on the well plus a bronze-hinged and -faced flap gate on the outfall pipe). This type valve arrangement was found to perform satisfactorily, as described in appendix C. A corrugated metal well guard (with no screen) was used to protect the well. A photograph of a typical well is shown on plate 264.

It was originally planned to set the tee outlets of the wells between sta 34 and 144 at el 411 (the minimum pumping elevation at the Alton pumping station prior to this underseepage investigation); however, 50 to 100 ft of horizontal discharge pipe for each well would have been required to set the well outlets at this elevation. It was found possible to raise the ponding elevation to 413 without causing damage landward of the levee, and as a result the operation of the pumping station at sta 50+46 was altered to permit pondage to el 413, thereby eliminating the relatively long (and costly) horizontal discharge pipes.

d. Sta 218. Two relief wells were located in the field to provide pressure relief at a drainage structure along the berm toe where the approach channel is close to the levee (see plate 5 and map 4). One relief well was located at sta 232+44 where the levee crosses an old channel of Wood River (see plate 5 and map 5).

166. East and West Forks, Wood River. No relief wells were considered warranted in this area as the design river stage was quite low (average about 5 ft), plate 6.

167. Lower Wood River. This area has also been subdivided, and the relief well design for each subarea is described in the following subparagraphs.

a. Sta 150. Two relief wells were considered warranted where the levee crosses an abandoned course of Wood River at about sta 150 (see plate 8 and map 4). Severe erosion is taking place in the bottom of a landside drainage ditch located adjacent and parallel to the berm toe at about sta 148+65. Continued recession of the ditch bottom in an upstream direction will create an unsafe seepage condition; therefore, a drop structure is proposed at this location to prevent any additional recession of this ditch.

b. Sta 155 to 187. The levee between sta 155 and 187 rests on a sand foundation with very little, if any, top stratum. A relatively deep borrow pit has also been excavated landward of the levee as shown on plates 8 and 13. In this reach relief wells were spaced on 125 ft centers to intercept potential erosional seepage. The wells along this reach of levee have not yet been staked; they should be located in a straight line along the toe of the berm with the well outlets set at about el 412. Local high or low areas along the line of wells should be dressed by cut or fill. A large underground storm sewer is located along the levee toe at about sta 180. This sewer was constructed as a part of the planned industrial development of the area by the International Harvester Co. The original plans proposed to discharge the sewer into the river through a drainage structure in the levee, but the project was abandoned before the drainage structure was started.

c. Sta 195 to 208. Nine relief wells were considered necessary along this reach of levee to provide adequate underseepage control.

d. Sta 226 to 237. A segment of the old channel of Wood River lies immediately adjacent to the levee along this reach (see map 9 and cross sections on plate 13). The top stratum between the levee and channel filling is quite thin, and was considered potentially critical with respect to underseepage. Therefore, it was considered that six relief wells should be installed along this reach of levee from sta 226+00 to 233+50 with tee outlets set at el 415.0. It was also considered that two relief

wells should be installed along the flume to the pumping station at about sta 236.

Design of seepage berms

168. Upper Wood River. Because of the severe pollution problem, seepage berms were considered preferable to relief wells in certain areas of Upper Wood River. However, the estimated cost of the berms was more than that of the relief well system and, therefore, seepage berms were not adopted.

169. A relatively deep hole beneath some railroad trestles immediately landward of the levee from sta 0 to 5 has been filled with relatively impervious fill up to el 423.7.

170. At the time of investigation (1953), the city of Alton was using the area between the levee and high ground landward from sta 5 to 15 for a sanitary garbage dump. The ground landward of this reach of levee has since been filled to an average elevation of 431, and no critical underseepage condition is considered to exist.

171. Lower Wood River. A seepage berm was designed for the reach of levee between sta 153+00 and 187+00 to be used in lieu of relief wells in the area where a landside borrow pit exists. The design of this berm was based on providing increased base width for the levee and thereby increasing the creep ratio of the levee and berm to a safe value. The estimated cost of this berm did not compare favorably with that for relief wells and therefore was not adopted.

Piezometer installation data

172. A summary of piezometer installation data is shown on plate 15; piezometer lines perpendicular to the levee with cross sections and boring logs are shown on plates 16 and 17.

Well installation data

173. All of the new relief wells have been installed in Upper Wood River, but no wells have yet been installed in Lower Wood River. Four of the nine old wells between sta 47+56 to 54+85 are to be replaced under a contract recently awarded. The following refers to wells installed in Upper Wood River.

174. Cobbles were encountered throughout the well reach. Sixteen

wells were installed to supplement the originally designed wells in reaches where design depths could not be reached. Respacing was necessary practically throughout the district except from sta 29+50 to 55+80. Specific yields were relatively high with an average of 120 gpm for the district, and all of the wells had a specific yield greater than 70 gpm. A summary of the installation data of the wells in Wood River (Upper) and the proposed locations of wells in Wood River (Lower) are shown on plate 18.

175. Permeabilities of the substratum sand computed from the pumping tests on the partially penetrating wells in Upper Wood River varied from a minimum of 1100×10^{-4} cm per sec for wells 48 and 81X to a maximum of 2850×10^{-4} cm per sec for well 16 with an average of 1650×10^{-4} cm per sec for the Upper district. This average permeability for Upper Wood River was used to compute the well flows shown on plate 14 for Lower Wood River.

East St. Louis Levee District

176. The East St. Louis levee district is located in Madison and St. Clair Counties, Ill. The levee affords flood protection to the city of East St. Louis and the towns of Granite City, Madison, Venice, Brooklyn, National City, Monsanto, Maplewood Park, and Cahokia.

177. The district is bounded on the north by Wood River levee district, on the northwest by the Chain of Rocks Canal, on the west by the Mississippi River, on the south by Prairie du Pont Creek and levee district, and on the east by the valley wall. The levee in the East St. Louis levee district is shown on maps 12-14, 16, and 18-34.

178. The district is protected by a levee which starts on high ground at sta 11+20 (map 16) and continues as a flank levee on the left bank of Cahokia Creek diversion channel to sta 262+25 where the levee ties to the upper end of the Chain of Rocks Canal levee at canal sta 511+00 (map 12). The levee then extends along the left bank of the canal to canal sta 46+22 where the canal levee ties to the East St. Louis levee at sta 773+03 (map 24). From this station the levee continues as a

flank levee to sta 1506+58 (map 33). It is planned to extend the levee up Prairie du Pont Creek to about sta 1567 (map 34).

179. The heights of levee and distance from tributary creeks or the Mississippi River are given below:

Levee Section	Height of Levee, ft	Waterway	Approx Landside Distance from Levee Toe to High Bank of Waterway
Upper flank	15-25	Cahokia Creek diversion channel	200-600 ft
Chain of Rocks Canal	25-30	Chain of Rocks Canal	600 ft
River front	10-30	Mississippi River	100-5500 ft
Lower flank	20-27	Prairie du Pont Creek	150-300 ft

The net river-front levee grade proposed prior to September 1954 and shown on the soil profile plates was used in the design of underseepage control measures except those along the Chain of Rocks Canal levee and from sta 1003 to 1010. The final river-front levee grade approved by the Office, Chief of Engineers, in November 1954 is from 2 to 3 ft lower than that used in the design. From sta 1003 to 1010 the final levee grade approved in November 1954 was used in the design of seepage control measures.

180. The channel of Cahokia Creek, which formerly meandered through a major portion of the levee district (maps 28 and 30), has been straightened to shorten its length (maps 12-14 and 16). The old courses of Cahokia Creek and several sloughs, ditches, sewers, and landside borrow pits provide interior drainage for the levee district. Drainage structures or pumping stations have been constructed where the levee crosses major drainage courses.

Geology and soil conditions

181. The flood plain in the district is approximately rectangular in shape, about 9 miles wide, and 17 miles long. Major channel fillings exist in the area between the east valley wall and the CCC & St. L Railroad from north of East St. Louis to the upper flank levee. The surface geology along the river-front levee is partially masked by industrial

developments (maps 23-27), although it appears that a major channel fill exists between sta 910 and 1090 along the river-front levee. The remainder of the alluvial fill appears to be the result of deposition by slowly migrating river channels and is similar to channel bar deposits with ridge and swale topography.

182. The soil profile along the toe of the levee is shown on plates 19-30. Cross sections and logs of miscellaneous borings made to develop certain geological and surface conditions are shown on plates 31-37. The alluvium along the upper flank levee, from sta 11+20 to 262+25, consists of a fairly uniform top stratum primarily of clay underlain by fine to coarse sands containing gravel and cobbles (plates 19-20). The levee crosses an old channel of Cahokia Creek at about sta 48 (map 16). The top stratum from sta 156 to 184 consists of thick clay, probably an old channel filling (plate 20).

183. The top stratum from sta 773 to 864 is highly irregular with respect to soil type and thickness. Throughout much of this reach, sand dredged in from the river overlies the original ground surface.

184. From sta 864 to 892 the top stratum generally ranges in thickness from about 8 to 10 ft. The ground surface along the downstream end of this reach is rather low. Riverside borrow pits in this reach are being filled with fly ash. From sta 925 to 962 relatively low areas landward of the levee also have been or are being filled with fly ash, in some locations to a depth of as much as 10 ft.

185. The levee from sta 962 to 1012 is founded on about 20 to 30 ft of sand which in turn is underlain by strata of clays, silts, and silty sands to a depth of about 30 to 50 ft (plate 26). These materials are believed to be part of a channel fill that extends from about sta 910 to 1090.

186. From sta 1012 to 1114 the top stratum thickness varies from a minimum of about 10 ft to more than 30 ft, and consists of silts with some clays and silty sands. Cinder fills also exist throughout most of the reach. The area landward of the levee between sta 1114 and 1135 is being filled with debris; dumping operations are proceeding in a downstream direction. The top stratum is relatively thin and uniform

between sta 1210 and 1242. From sta 1242 to the end of the river-front levee at sta 1313 the thickness of the natural top stratum landward of the levee has been materially reduced as a result of borrow for building the levee (maps 30-31, plate 28).

187. Along the lower flank levee from sta 1313 to 1510 the top stratum generally consists of clay underlain by silts which in turn are underlain by silty sands. The Cahokia pumping station is located at sta 1315 and has an inlet channel sufficiently deep to expose the pervious substratum (map 31 and plate 28). Between sta 1316 and 1321 the pervious substratum is exposed in a deep landside borrow pit (see above map and plate). The top stratum along the rest of the lower flank levee is relatively uniform except for several minor swale fillings with a major swale at sta 1352 and 1460 (plate 29). The flank levee crosses numerous small swales.

188. Borings to rock and seismic and resistivity determinations of rock depths show that the valley floor varies in elevation between about 281 and 335 ft msl along the river-front levee. The valley floor is apparently deepest at sta 1414+50 along the lower flank levee where the alluvium is 150 ft deep. The shallowest point in the alluvium was at the Chain of Rocks navigation lock and vicinity where the valley floor is only 75 ft deep.

History of seepage

189. Seepage observations made during the 1947 and 1951 high-water periods which created net heads on the levee of about 10 to 20 ft are given below.

190. Upper flank levee. Seepage and minor sand boils were reported in the former channel of Cahokia Creek at about sta 48 during the 1951 high water.

191. Chain of Rocks Canal levee. Seepage was also observed along the berm toe opposite canal sta 396, 363, 209-200, 194, and 88. A small boil was observed adjacent to the upstream end of the grouted riprap at the inlet ditch to the NC&V pumping station at canal sta 272. The water in the canal was at el 426 or about 20 ft above the tailwater in the ditch. A considerable amount of seepage was observed entering a drainage

ditch along the landside toe of the levee between about sta 160 and 187 during the 1951 high water. This seepage caused considerable sloughing of the near bank of the ditch. Numerous small sand boils occurred in the bottom of the ditch during 16-23 July, at which time the water surface in the canal was at about el 428 or 18 ft above the tailwater in the ditch. Pin boils and bank caving were observed at the edge of a lateral ditch at about canal sta 116, during the 1951 high water.

192. River-front levee.

a. Sta 773 to 864. During the 1947 high water, a net head of about 10 ft caused considerable seepage along this reach of levee. A temporary sublevee was constructed around an area between about sta 779 and 795 (map 24) during the 1947 high-water period to control seepage and small sand boils. It was estimated that seepage entered the subleveed area at a rate of 200 gpm. Light to medium seepage was observed between sta 781 and 796 during the 1951 high water. Seepage was observed at the toe of the levee between approximate levee sta 820 and 824 and at about sta 833. No sand boils were observed.

b. Sta 865 to 962. Seepage occurred along this reach during both the 1947 and 1951 high waters. The maximum head on the levee during 1951 varied from about 10 to 13 ft except between sta 892 and 937 where the head was only about 2 to 4 ft. Numerous small boils were observed in a low area between approximate sta 888 and 892.

c. Sta 962 to 1242. During the 1947 high water, several boils emerged along Front Street from Eads Bridge (sta 1042+54) to a point four blocks south. Many of the boils were ringed with sand bags. Seep water rose 2 ft within the sandbagged areas. Between sta 1040 to 1045, many of the streets were covered with water. This seepage probably emerged through old drains or pervious fills near the railroads and warehouses. Seepage also was observed in the vicinity of the old course of Cahokia Creek at about sta 1097. A small sand boil was observed near a fence at about sta 1236. During the 1951 high water the concrete floodwall (since abandoned) was overtopped at sta 1035 at a river stage of about 416 ft msl. Heavy seepage and pin boils discharging clear water were observed at sta 1107 in a cinder fill about 800 ft downstream of MacArthur Bridge. The net head on the levee was very small, and it is believed the boils were due to improper backfilling of a ditch from which a pipeline was removed during levee construction. A small boil was reported at about sta 1180 near the toe of the GM&O Railroad embankment (about 350 ft landward from the levee).

Between sta 1211 and 1242, the toe of the levee was soft because of seepage.

d. Sta 1242 to 1312+50. In 1947 six sand boils about 2 to 4 in. in diameter occurred landward of the levee between about sta 1242 and 1260. Some boils discharged sand in craters 1 ft high with an outer diameter of 4 ft. At that time the present landside borrow pits had not been excavated.

193. Lower flank levee.

a. Sta 1312+50 to 1322. A large boil developed in the inlet channel at the Cahokia pumping station during the 1947 high water. At that time the water in the channel was about 6 ft deep and the Mississippi River stage was about 418 ft msl. Several additional boils were flowing freely under about a 4-ft head and discharging considerable sand, building mounds about 4 ft in diameter and 2 ft high in the bottom of the channel. Pumping had been discontinued several days prior to development of the boils to allow the water in the channel to rise. Considerable seepage also was observed along the berm between this inlet channel and the river-front levee (sta 1312+50). Similar difficulties occurred at the Cahokia pumping station during the 1951 high water and at the riverward edge of the landside borrow pit between sta 1314 and 1319. Seven small boils occurred in the landside inlet channel on 16 July 1951. On that date the water in the ditch was at about el 400 and the river stage was about 416. Some of boils were discharging sand. Subsequent raising of the water in the ditch reduced the net head above water in the ditch to 14.5 ft which reduced the activity of the boils. Flow from the boils had about ceased by 18 July. Boils started in the landside borrow pit between sta 1314 and 1319 on 13 July, at which time the difference between the river stage and tailwater in the borrow pit was about 14.5 ft. By 16 July the net head at the borrow pit was 16 ft. By 18 July sand bags had been placed at the landside edge of the borrow pit and the tailwater had been raised about 3 ft or to el 402.5. This reduced the net head on the levee to about 13.5 ft which then reduced the activity of the boils. Natural seepage was estimated to have entered the borrow pit at a rate of 500 gpm; this seepage caused some erosion of the slopes of the pit.

b. Sta 1322 to 1500. A sand boil occurred at about sta 1383 just east of the Missouri Pacific Railroad during the 1947 high water. In 1951, through seepage at the Missouri Pacific Railroad crossing at approximate sta 1380 emerged from the slopes of the relatively pervious railroad fill where it crosses a landside drainage ditch. (An impervious core has since been constructed to cut off

seepage through the fill.) It is understood that seepage was occurring in the landside drainage ditch between sta 1353 and 1500 during the 1951 high water.

194. Aside from the elevations of water in ditches and channels cited above, elevations of the tailwater are not available for other areas along the levee.

195. Several piezometers including lines perpendicular to the levee were installed prior to the 1951 and 1952 high-water periods. Locations of piezometers, piezometer lines, and a summary of the piezometric data and analyses thereof are shown in table 6. Values of s and x_3 were determined at lines of piezometers perpendicular to the levee by means of equations 3 and 4, respectively, and are listed in the table. The piezometric data were taken into consideration in the design of relief wells, both in estimating s and x_3 and in estimating h_o with the river at the net levee grade shown on the soil profile sheets. Piezometers installed by the St. Louis District prior to the beginning of this investigation that are to be maintained are listed in the table on plates 44 and 45.

Design of relief well systems

196. Factors used in the design of relief wells are shown on plates 38-40. As a major portion of the area immediately landward of the river-front levee was highly developed, it was considered necessary to collect the well flow and pump it over the levee in these areas. Where the areas in the vicinity of the levee were not developed or were of an agricultural nature, it was decided to let seepage and well discharge pond or seek natural drainage.

197. Upper flank levee. Six wells were designed for the upper flank levee (sta 11+20 to 26+25). One well is located at the edge of the old channel of Cahokia Creek at sta 48+00 and will discharge into it (map 16, plate 19). The remaining 5 wells are located between sta 184+00 and 196+00. As the upper flank levee is in an agricultural area, the discharge from the above 6 wells will either pond or seek natural drainage.

198. Chain of Rocks Canal levee. Relief wells along the Chain

Table 6

Summary of 1951 Piezometric Data, East St. Louis Levee District

Table 6

Summary of 1951 Piezometric Data, East St. Louis Levee District

River-front Levee Station	Net Head on Levee ft	Maximum Head in Per Cent of Head on Levee	Distance to Effective Source of Seepage Entry s, ft	Computed Exit Lengths x_3, ft	Est h_o* Based on Net Levee Grade	i*	Top Stratum Effective Thickness ft	Top Stratum Type of Material
782+22	12.6	19	840	210	6.2	0.62	10	Clay, silt, silty sand
786+40	12.6	29	810	290	7.6	7.6	1.0	Clay, sandy silt
792+00	12.6	24	940	290	6.8	1.94	3.5	Silt, silty sand
795+30	12.6	20	1210	320	6.1	1.35	4.5	Silt, silty sand
1066+40	4.2	--	----	---	---	----	20	Silt, silty sand
1212+00	10.2	--	----	---	---	----	4.5	Silt
1222+00	12.6	20	1085	155	3.4	0.85	4.0	Clay
1232+00	12.3	24	755	175	5.1	0.78	6.5	Silt
1242+00	14.8	38	470	180	8.9	0.68	13	Silt, silty sand
1245+94	14.8	47	665	570	13.8	6.90	2	Silt, silty sand
1248+00	14.8	31	1130	540	9.7	3.23	3	Clay, silty sand

* Denotes h_o and i which would be developed if the top stratum were capable of withstanding such pressures.

of Rocks Canal levee were designed by the St. Louis District and installed in 1952. The design procedures for these wells were somewhat different from those used in this investigation. The spacing of the wells was determined so that the residual hydrostatic head between wells would be equal to or less than two-thirds the submerged weight of the top stratum. The residual pressure h_x without wells was computed by the formula:

$$h_x = \frac{H\, e^{-cx}}{2 + c\, L_2}$$

wherein x is the distance landward of the landside levee toe and H, c, and L_2 are as defined in part V of this report. The effect of the length of finite well systems on the heads between wells was not considered quantitatively in the design of the wells along the Chain of Rocks Canal levee. The permeability of the pervious substratum was assumed equal to 700×10^{-4} cm per sec. Wells were generally spaced according to the computed spacings except in the reach from sta 184+50 to 267+00 where a deep landside ditch parallels the toe of the levee (maps 20-21). Spacings here were arbitrarily decreased to 150 ft, since the method used for computing well spacings did not take the ditch into consideration; and acute seepage in this reach during the 1951 flood indicated the necessity for underseepage control measures.

199. River-front levee. In the reach from sta 773+29 to 862+19 the wells were installed in the bottom of an open collector ditch lined with porous concrete. The excavated material was placed in a retaining dike just landward of the ditch. The ditch was paved with porous concrete underlain with a 2-in. blanket of well-graded concrete sand to intercept shallow seepage through sand strata immediately beneath the levee. Freeze-thaw tests, described in appendix B, were performed on the porous concrete lining to determine its resistance to weathering. The collector ditch is provided with buried outfall pipes and manually operated head gates so that surface runoff entering the ditch during low-water periods can flow through the outfall pipes into normal drainage facilities. Well flow in the ditch between sta 773+29 and 862+19 will be pumped over the levee by pumping stations at levee sta 783+01, 815+00, and 846+41 (map 24).

200. The capacities of the pumping stations were based on computed maximum flows from the relief wells, utilizing the results of special pumping tests on wells 39 and 64 (see part VIII). The computed maximum well flow at the pumping stations is 11,100 gpm at sta 783+01, 15,200 gpm at sta 815+00, and 9,700 gpm at sta 846+41. The collector ditch and pumping stations are designed to handle the above flows. Each pumping station has space for three pumps: two pumps are to be installed during construction so that each station will initially have a capacity for about two-thirds of the above flows; the required capacity of the third pump will be determined after the well systems have operated and have been observed during a high water.

201. The wells in the reach from sta 865+60 to 891+70 were installed with tee outlets that discharge into a concrete collector pipe. The well flow will be pumped over the levee by a pumping station at sta 892+60 (map 25). An open ditch was not used in this reach because nearby dwellings made it probable that such a ditch would become fouled by refuse and debris. The collector pipe was provided with access manholes. The computed maximum well flow at the pumping station at sta 892+60 is 13,750 gpm. The collector pipe was designed to conduct the above design flow under gravity flow.

202. From sta 996 to 1012 the levee will be raised to grade by means of a concrete floodwall to be constructed on the existing levee. The six wells from sta 1003+25 to 1010+75 were not located along the landside toe of the existing levee because of the proximity of railroads which would complicate installation of wells; instead, the wells are to be located at the landside toe of the floodwall (about the center line of the existing levee), and will be installed when the floodwall is under construction. The wells will have tee outlets that will discharge into a closed collector pipe beneath the levee with an invert at about el 410. The collector pipe will conduct the flow from the wells into the railroad underpass at about sta 1012 (map 27, plate 26).

203. Four wells were installed between sta 1012 and 1094. Two of these (wells 105 and 106) are located at the closure structure at the railroad underpass at about sta 1013 (map 27, plate 26). Wells 107 and

108 are located adjacent to the floodwall closure at the railroad underpass at sta 1049. At both of the underpasses the well discharge can pond to a depth of about 6 ft before the discharge would have to be pumped over the levee. Well flow into the underpass areas can be removed by means of portable pumps. The four relief wells are equipped with specially screened outlets and flap valves so that the outlets will not become clogged with debris.

204. The three wells from sta 1094+80 to 1097+30 (wells 109-111) were located along the near bank of the old channel of Cahokia Creek about 400 ft landside of the levee (map 28). Well discharge can pond in the creek. Three wells were located in a low area near the MacArthur Bridge between about sta 1099 and 1103 (map 28). As this area is completely surrounded by railroad embankments, the well discharge can be ponded.

205. Twenty-two wells were installed between sta 1104 and 1137. The ground is considerably higher between sta 1104 and 1114 than in the remainder of the well reach (plate 27). Therefore, the four wells from sta 1104+00 to 1107+75 were installed in an unlined ditch that will conduct the well discharge to the lower ground starting at sta 1114. The portion of the ditch between about sta 1113 and 1114 was lined with concrete to prevent erosion. As the area from sta 1104 to 1137 is completely inclosed by railroad embankments, it is believed that the well discharge can be permitted to pond and, if necessary, pumped over the levee with portable pumps.

206. The wells from sta 1211+50 to 1241+00 will discharge into a shallow unlined collector ditch. Material removed from the ditch was placed in a landside retaining dike. As the top stratum is relatively thin in this reach (plate 28), the ditch was kept shallow to prevent the thickness of the landside top stratum from being greatly reduced and was designed so that the water surface in the ditch would exceed very little the assumed tailwater elevation in low-lying natural ground landward of the levee. A pumping station was constructed at sta 1225+65 to pump the well discharge over the levee. The computed maximum total well flow to the pumping station is about 15,000 gpm.

207. The wells between sta 1244+10 and 1308+60 are located in a landside borrow pit (maps 30-31) and will be submerged during periods of high water. Natural seepage and well discharge into the borrow pit drain to the Cahokia pumping plant at sta 1315+50 where it will be pumped over the levee.

208. Lower flank levee. As the 1951 high water caused sand boils in the inlet channel to the Cahokia pumping plant at about sta 1315, four shallow relief wells were installed at this location in early 1952. These wells were later pump-tested and found to be unstable, and are to be filled and sealed. Four new wells were installed along the inlet channel to replace the first wells (map 31).

209. Six wells were installed in a landside borrow pit between sta 1314+90 and 1320+25 where seepage had been a problem during the 1951 high water. As the borrow pit exposes the pervious substratum (plates 28 and 29) the normal design curves were not applicable and the wells were arbitrarily spaced on 70-ft centers. The wells were installed in the borrow pit on a narrow auxiliary berm with outlets at about el 400. The water in this borrow pit will pond to el 403 and then spill over into the inlet channel to the Cahokia pumping plant.

210. Twenty-three wells were designed for the reach between about sta 1323 and 1345. As the top stratum is only 2 to 3 ft thick between sta 1333 and 1339, well spacings of 60 ft were considered warranted. The levee downstream from about sta 1347 will be enlarged, using material obtained from a landside borrow pit, and a pilot channel will be located at the landside (far) edge of the pit to conduct interior drainage and seepage flow to the existing pumping station at sta 1507 (map 33, plates 30-31). The need for wells between sta 1347 and 1507 was determined by using the allowable upward gradients without wells of 0.67 and 0.85 at the near and far edges of the borrow pit, respectively; the near edge of the pit will be 100 ft from the landside toe of the levee. With relief wells installed the respective maximum design gradients were to be 0.50 and 0.67. The gradient through the top stratum beneath the pilot channel was not considered, because the channel is to be a considerable distance from the toe of the levee.

211. The river stage along the lower flank levee was assumed at el 413.5 ft msl, the proposed net grade of the river-front levee at the junction of the flank levee at the time the wells were designed. After the design was completed, the proposed net grade at the junction of the river-front and flank levees was lowered to el 429.7 ft msl. The tailwater elevation in the drainage ditch was assumed at el 402.5 ft msl at levee station 1345 and at el 399.0 ft msl at sta 1505, and was assumed to have a constant grade between these points.

212. A summary of the design of relief wells downstream from sta 1347 is shown on plate 38, although the design spacings were not obtained from the design curves in the usual manner. The design procedures are discussed below. The effective exit length x_3 was computed from soil conditions beneath the borrow pit and between the levee and borrow pit as follows:

$$x_3 = (x_3)_1 + 1/3 \left[(x_3)_2 - (x_3)_1\right]$$

wherein

$(x_3)_1$ = exit length for top stratum beneath the borrow pit assumed infinite in landward extent

$(x_3)_2$ = exit length for top stratum between levee and borrow pit assumed infinite in landward extent.

213. Along this reach of levee, particular attention was paid to evaluation of the distance from the landside toe of the levee to the effective source of seepage s because borings generally indicated the existence of 5 to 10 ft of silts and clays beneath the bottom of Prairie du Pont Creek (plate 37). Values of s were computed assuming a blocked seepage entrance 300 ft riverside of the riverside toe of the levee, and using the following formula:

$$s = L_2 + X + \frac{1}{c \tanh (c L_1)}$$

wherein

L_1 = distance from riverside toe of levee to center of Prairie du Pont Creek (about 300 ft)

L_2 = base width of levee

X = distance between landside toe of levee and near edge of borrow pit = 100 ft

$c = \sqrt{\frac{k_b}{k_f \; z \; d}}$ for the top stratum riverward of the levee.

Well spacings were then determined from the formulas used to develop the design curves for relief wells as described in part VI. All of the wells in the reaches where there will be landside borrow pits will be located at the riverside edge of the pit instead of being installed on the berm with tee outlets.

Design of seepage berms

214. The design assumptions for seepage berms are shown on plate 41, with typical berm cross sections shown on plates 42 and 43. A fly ash fill is being placed landside of and adjacent to the levee in basins between sta 925+00 and 962+50 (maps 25-26, plate 26). Two basins extending from sta 925+00 to 939+50 had been filled with fly ash to within 12 ft of net grade of levee as of 1 October 1954 and filling was under way in a third basin from sta 939+50 to 950+50. The width of fill varies from about 500 ft at sta 925 to 1700 ft at sta 962. These filled basins will serve as seepage berms and replace the relief wells previously designed for the reach from sta 939+00 to 961+00.

215. A seepage berm was designed to extend from sta 967 to 996 along a reach of levee founded on clean sand (plate 26). The berm should be constructed of sand as the soils underlying the berm are sands. The width of the berm was designed so that the base width of levee and berm would provide a creep ratio of 15, which is believed adequate to prevent any erosional seepage; the thickness of berm was selected so that there would be little head above top of berm. The berm will be approximately 3 ft thick at the levee toe. The width of the berm varies from about 60 ft to 130 ft (map 26, plate 42). A transition was made in the width of that portion of the berm between sta 992 to 996 to avoid relocating railroad tracks.

216. Fill was placed in a low area in the vicinity of the

MacArthur Bridge from sta 1096+80 to 1099+10. The fill (horizontally) is triangular in shape, with a maximum width of 130 ft. Borrow was obtained from a rubbish dump landward of the levee between sta 1105+00 to 1111+00.

217. From sta 1136+35 to 1140+50 a seepage berm was designed in lieu of three relief wells. The berm has a width of 150 ft and a thickness at the levee toe of 5 ft (plate 42). A seepage berm was also designed for the reach of levee from sta 1165+00 to 1210+00. Borrow was obtained riverward of an existing borrow pit between sta 1224+00 and 1238+00. The seepage berm design was prepared as an alternate seepage control measure to wells. The width of the berm varies from 410 to 310 ft and the approximate thickness at the levee toe from 8 to 2 ft. In the reach from sta 1165 to 1180, the existing GM&O Railroad yard had served in effect as a berm (map 29). The new berm consists of a 75-ft extension to the yard, and fill in low areas within the yard itself (plate 43).

Piezometer installation data

218. A summary of installation data for piezometers installed prior to 1 June 1955 and locations of piezometers which have not been installed are shown on plates 44 through 46. Piezometer lines are shown on plates 47 through 49.

Well installation data

219. Wells 146 and 218 were abandoned because cobbles were encountered above design depth and were replaced by wells 146A and 218A; both replacement wells were installed to design depth. Two additional wells, 141X and 201X, were installed in reaches where respacing was required because of cobbles which prevented the wells from being installed to design depths.

220. Cobbles were encountered in practically all of the wells installed prior to May 1955 along the river-front levee, but only about 13 per cent of the wells were installed at less than design depth because of such obstructions (plates 50 and 51). The specific yield of wells installed along the river-front levee prior to 1 May 1955 averaged 120 gpm and none had a specific yield lower than 60 gpm.

221. The wells along Chain of Rocks Canal were installed prior to

underseepage investigation and are not included in the discussion above. The installation data are shown on plates 52 and 53.

222. Permeabilities of the substratum sand along the Chain of Rocks Canal were not computed from pumping tests. Permeabilities of the substratum sand computed from the pumping tests performed prior to April 1955 on all wells except those along Chain of Rocks Canal varied from a minimum of 800 x 10^{-4} cm per sec for well 206 to a maximum of 3300 x 10^{-4} cm per sec for well 137. Special pumping tests were made on fully penetrating wells 39, 64, and 151. The average permeabilities at wells 39 and 64 were found to be 800 x 10^{-4} cm per sec and 950 x 10^{-4} cm per sec, respectively. The average permeability at well 151 was estimated to be 1800 x 10^{-4} cm per sec; however, as the test results were somewhat erratic, it is planned to make another pumping test when the water table is high enough to use a suction pump. The new pumping test on well 151 had not been made prior to 1 January 1956. Detailed results of the above special pumping tests are given in part VIII.

Prairie du Pont Levee District

223. The Prairie du Pont drainage and levee district is located in St. Clair and Monroe Counties, Ill. The town of Dupo lies within the levee district. The district is bounded on the east by the valley wall, on the north by the East St. Louis levee district, on the west by the Mississippi River, and on the south by the Columbia levee district. Levees in this district are shown on maps 31-42.

224. The district is protected by a levee which starts at sta 0+00 where it ties into Illinois Highway 39 (map 33) and continues as a flank levee from sta 12 down the left bank of Prairie du Pont Creek to about sta 210; there the creek empties into Cahokia Chute (map 31) and the levee becomes river-front levee. The levee continues along the left bank of Cahokia Chute and the Mississippi River to sta 545+72 where it ties into Illinois Highway 157 (map 39) which provides levee protection on the southwest side of the district. A proposed levee along the Mississippi River in the Fish Lake levee district will replace the highway

as a levee. The proposed levee will extend from sta 545+72 along the left bank of the Mississippi River to approximately sta 673 and thence as flank levee along the right bank of Columbia Creek to sta 801+88. No investigation of underseepage was made along either Illinois Highway 157 or the location of the proposed new levee through the Fish Lake levee district.

225. The upper flank levee ranges in height from 20 to 25 ft and the distance to Prairie du Pont Creek varies from 200 to 500 ft. The river-front levee varies in height from 15 to 25 ft and is about 2000 to 3200 ft from the Mississippi River except from sta 210 to 240 where the levee is only about 300 ft from Cahokia Chute.

226. There are no creeks or rivers in the present district. Fish Lake lies within the district and intersects Illinois Highway 157 just beyond the end of the levee. Palmer Creek and Hill Lake Creek No. 1 lie within the proposed Fish Lake extension.

Geology and soil conditions

227. The flood plain is approximately 3 miles in width in this district. A fairly thick deposit of clay overlain by silt extends out from the valley wall to about sta 100. The remainder of the levee to State Highway 157 is founded on irregular channel bar deposits with ridge and swale topography. The levee crosses an old channel of Prairie du Pont Creek at sta 157 and 273. A blue hole caused by a former crevasse exists about 300 ft landside of the levee at sta 276.

228. The soil profile along the toe of the levee is shown on plates 54 through 57. Cross sections and logs of borings made at locations considered pertinent to the study and design of underseepage control measures are shown on plates 58 and 59. A large-diameter bailer boring (W-24B, plate 251) and logs of holes for relief wells show the existence of considerable amounts of gravel and cobbles in the lower portion of the pervious substratum. Borings to rock and seismic determinations of rock depths show the valley floor to be irregular and to vary elevation from 324 to 272 ft msl beneath the levee.

History of seepage

229. The present levee system was completed to project line and

grade after the 1947 high-water period. The only seepage data available for the study, therefore, were collected during the 1951 high-water period, at which time the maximum head on the levee varied from about 5 to 12 ft; during the 1952 high water the maximum head on the levee varied from 0 to 5 ft. The worst seepage condition during 1951 was observed between levee sta 297 and 301 (maps 35-36) at East Carondelet. The ground surface was soft landward of the levee toe for a distance of about 300 ft; an asphaltic-concrete paved street near the levee also became soft as a result of seepage. Numerous pin boils were observed opposite approximate levee sta 299. Seepage and rainwater were ponded over most of the area. Heavy seepage also occurred from sta 165 to 265, 425 to 515, and along the Illinois Highway 157 embankment from the levee to a point about 1-1/2 miles landward.

230. Except at sta 529, where a watermark was found on trees at el 401.7, no data are available to indicate what tailwater elevations would be obtained during high-water periods.

231. Piezometer readings were made during the 1951 and 1952 high-water periods at piezometer ranges at sta 219+01, 314+00, and 506+72. A net head of 9 to 12 ft, or 10 ft below net grade of the levee, was experienced on the levee at these locations during the 1951 high-water period; piezometric pressures reached a maximum of 10 per cent of the net head on the levee at sta 219+01 and 314+00, and 26 per cent of the net head at sta 506+85. The river stage during the 1952 high-water period was not sufficient (approximately 5 ft) to create any significant hydrostatic head landward of the levee at these piezometer lines. The top strata at these piezometer lines are shown on plates 55, 56, and 57.

Design of relief well systems

232. Factors used in the design of wells are shown on plate 60. Evaluation of the need for and design of underseepage control measures was based on the net levee grade shown on the soil profiles. However, a 1- to 2-ft increase in levee grade was authorized for this district by the Office, Chief of Engineers, in November 1954. The control measures are designed for a river stage about equal to the newly proposed flood stage.

233. A landside drainage ditch exists near the levee from about sta 141 to 157 (see map 31 and plate 55). Two relief wells (96 and 97) were installed along the ditch where the top stratum was quite thin. These wells were provided with tee-type outlets with the top of the well set below the natural ground surface of el 408. The top stratum in the bottom of the ditch both upstream and downstream from the wells has adequate thickness.

234. A blue hole about 300 ft landside of the levee at sta 276 (map 35) could not be filled because the hole is now being used as a small lake and the surrounding area is being developed into a recreation area. Although the top stratum between the levee toe and the blue hole consists primarily of fat clay about 16 ft thick, a boring (W 136, plate 55) made at the edge of the hole indicates little or no top stratum beneath the bottom of the blue hole. As a result of this soil condition seepage will concentrate in the blue hole, as only minor seepage is expected to occur through the thick clay between the levee toe and the blue hole. Because of the possibility of occurrence of sand boils and piping in the bottom of this old blue hole, two relief wells were installed at the riverside edge of the blue hole.

235. Spacings of wells in the reaches from sta 297+00 to 301+60, 458+75 to 464+50, and 529+30 to 535+50 were varied slightly to best fit the topography. Well 42 (map 35) was located along a drainage ditch leading to a drainage structure at sta 295+45.

236. A low landside slough parallels the levee from sta 511+50 to 536+00. In order to protect the comparatively thin top stratum in front of the slough the well outlets from sta 521+50 to 529+00 were lowered slightly and the ground graded to drain to the slough.

237. It was assumed that the tailwater would be at about the natural ground surface with two exceptions. Surface water is expected to pond in a low area at sta 474 to el 410 ft msl. Previous high watermarks were found on two trees in a slough paralleling the levee at a distance of about 200 ft at el 401.7; the tailwater elevation in the slough and along this reach of levee was assumed at el 402.0 ft msl for a project flood.

238. A deep scour hole in the riverside borrow pit from sta 497+50 to 502+50 (map 38) has been filled to el 394.0 ft msl at the riverside edge of the scour with the fill sloping to the existing ground toward the levee.

Design of seepage berms

239. Where the levee crosses an old channel of Prairie du Pont Creek at about sta 157 (map 31, plate 58), the existing berm was extended to provide additional seepage protection. The extension is about 4 ft thick at the present berm toe and has a width of 75 ft and a slope of 1 on 75. The berm extends to the bottom of the slough on a 1-on-25 slope, which provides about 85 ft of additional width (see plate 60).

240. An abandoned railroad embankment intersects the levee at about levee sta 381 (map 37), and at this location the landside toe of the embankment is considerably lower than the surrounding ground (see plate 56). The embankment has been degraded and the material obtained used to fill the low ground along the north side of the railroad. The existing berm was extended to fill the depression between the south toe of the embankment and sta 383+20. The extension is 100 ft wide and about 4 ft thick at the toe of the previously constructed berm.

241. The design assumptions for the berm at sta 157 are shown on plate 60; a typical cross section of the berm at sta 381 is shown on plate 59.

Piezometer installation data

242. A summary of piezometer installation data is shown on plate 61; piezometer lines perpendicular to the levee with cross sections and boring logs are shown on plate 62.

Well installation data

243. All of the relief wells in Prairie du Pont have been installed. A summary of well installation data is shown on plate 63.

244. Well 44 was abandoned because cobbles were encountered considerably above design depths and was replaced by well 44A which was installed to design depth.

245. Well 64X was the only additional well installed as a result of respacing wells where the wells could not be installed to design

depths because of the presence of cobbles.

246. Although cobbles were encountered in most of the wells installed, 90 per cent of the wells were installed to design depth. The average specific yield for the wells installed in the district is 107 gpm per ft and only well 3 had a specific yield lower than 50 gpm per ft.

247. Permeabilities of the substratum sand computed from the pumping tests on the partially penetrating wells varied from a minimum of 550×10^{-4} cm per sec for well 3 to a maximum of 3500×10^{-4} cm per sec for well 11 and averaged 1550×10^{-4} cm per sec for the entire district.

Columbia Levee District

248. The Columbia levee district is located in Monroe County, Ill., approximately 6 miles south of Dupo, Ill. The district is bounded on the north by the Prairie du Pont levee district and Columbia Creek, on the west by the Mississippi River, on the south by Fountain Creek and the Harrisonville levee district, and on the east by the valley wall. Levees in the district are shown on maps 40 to 54.

249. The district is protected by a levee which starts at sta 1+56 where it ties into high ground (map 41), extends along the left bank of Columbia Creek as a flank levee to sta 155 (map 49), thence it turns and parallels the Mississippi River to sta 655 where it turns up the right bank of Fountain Creek and extends as flank levee to its terminus at sta 1051+36 (map 54). The height of the levee varies from 10 to 20 ft. The distance from the levee to Columbia Creek varies from 50 to 200 ft, the distance to the Mississippi River varies from 1700 to 5200 ft, and the distance to Fountain Creek from 200 to 500 ft.

250. Interior drainage within the district is provided by a series of ditches leading to several drainage structures through the levee.

Geology and soil conditions

251. The surface geology in the district consists essentially of channel and sand-bar deposits with little evidence of any major channel-filling deposits. Numerous swales intersect the levee. The levee

crosses sloughs at sta 118 (map 42, plate 64), 270 (map 44, plate 65), 290 (map 44, and plate 66), and 442 (map 46, plate 67). The levee also crosses an old channel of Fountain Creek at sta 811 (map 51, plate 69).

252. The soil profile along the landside toe of the levee is shown on plates 64-71. This profile shows the top stratum throughout the district to be highly irregular both as to soil type and thickness. Cross sections and logs of borings made at locations considered pertinent to the study are shown on plates 72-74. Logs of holes made for relief wells show the existence of large gravel and cobbles below a depth of 60 to 70 ft. Borings to rock and resistivity determinations showed that the valley floor varied in elevation from 280 ft msl to 311 ft msl along the levee. The deepest point of the valley floor was encountered at sta 138+17 where the alluvium was determined to be 116 ft deep.

History of seepage

253. The present levee system was completed to the present alignment and project grade after the 1947 high-water period. The only seepage data available for study were collected during the 1951 and 1952 high-water periods. During the 1951 high-water period when the head on the levee reached a maximum of 5 to 15 ft, or 7 ft below the net grade of the levee, heavy seepage was observed along the following reaches of levee: sta 132-140, 231-238, 247-257, 469-480, 506-526, 535-550, and 569-585. Light seepage and ponded rain water were observed at sta 99-102, 118, 140-149, 323-328, 367-371, 482-484, and 632-652. Some caving of the bank at the bridge crossing Fountain Creek at sta 716 was also noted. A large sand boil, which was sandbagged, occurred about 20 ft from the levee toe at approximate sta 146.

254. During the 1952 high-water period when the head on the levee was about 0 to 10 ft, some minor seepage was observed in the vicinity of sta 98 and 150. Numerous low areas filled with water between sta 120 and 350.

255. No data are available to indicate what tailwater elevations would be obtained during high water.

256. Piezometric data were obtained during the 1951 high-water period from piezometers located at sta 100+90, 105+84, 115+78 (plate 64),

and from piezometer lines perpendicular and parallel to the levee at sta 231+45 (plate 65) and 324+50 (plate 66). Maximum hydrostatic heads above the natural ground surfaces at the landside toe of the levee at the preceding locations were 15, 19, 9, 16, and 34 per cent, respectively, of the net head on the levee (11.5 ft). The maximum of 34 per cent of the net head occurred 158 ft landside of the levee center line at sta 324+50. The top stratum at this site consisted of about 7 ft of clay underlain by about 5 ft of silt. Although the data obtained from the piezometers during the 1951 high water were used in the design of relief well systems in those reaches of levee, the above piezometers have since been abandoned, owing to expiration of installation permits, except those at sta 100+90 and 105+84.

Design of relief well systems

257. Factors used in the design of relief well systems are shown on plates 75 and 76.

258. The design of the well system from sta 322+20 to 326+40 was based upon the piezometric data obtained along this reach of levee. A well at sta 344+35 was located at the edge of a swale (plate 66). Along the levee from sta 478+50 to 481+80 one well was located at the inlet channel to a gravity drain (map 47) and three wells were spaced to fit between two swales (plate 67, map 47) with a thin top stratum between the swales. Relief wells between sta 551+50 and 564+00 were spaced slightly less than the computed spacing so that the wells could be located most advantageously at critical locations between and at the edges of swales. The wells at sta 776+20 and 776+82 were located adjacent to a drain (map 51).

Design of seepage berms

259. Seepage berms were designed to be used in lieu of relief wells along certain reaches of levees because they were a more economical solution to the problem of underseepage.

260. Factors used in the design of berms are shown on plate 77, and typical cross sections are shown on plate 78. The first three berms are comparatively short and are primarily to bolster the levee toe against sloughing as a result of seepage. The top stratum in these three

reaches consists of 3 or 4 ft of silt or clay. Extensions of the berms from sta 596+00 to 601+00 and 678+00 to 680+00 were designed to fill the near sections of borrow pits of the former levee which is located landward of the new levee (map 49). Design of these extensions is somewhat empirical because of the discontinuity in the top stratum as the result of the borrow pits. Although berms with a width of about 75 ft were in existence, extensions to the existing berms were considered necessary since there would be a concentration of seepage at the existing berm toe in the old borrow pits. These berms have been constructed except for the berm from sta 220 to 228 which has been deferred because construction would require the relocation of a public road and the moving of a telephone line. This latter reach of levee will be carefully observed during flood stages. A piezometer line was installed at sta 221 and one piezometer at sta 226+00 (map 43) to observe the hydrostatic pressures during high-water periods.

Cutoff trench

261. A shallow underseepage cutoff trench has been placed from sta 297+00 to 303+00 (map 44).

Piezometer installation data

262. A summary of piezometer installation data is shown on plate 79 and piezometer lines perpendicular to the levee with cross sections and boring logs are shown on plate 80.

Well installation data

263. Well 53 was abandoned and replaced by well 53A because the presence of cobbles considerably above the design depth made it impracticable to install well 53 to design depth. Well 53A was installed to design depth. Wells 59A and 63A replaced wells 59 and 63, respectively, because the latter wells produced excessive quantities of sand during the pump test. Wells 56 and 85 have been installed, but showed excessive sand infiltration during pumping tests and are not considered stable. These wells are to be plugged and replaced under a contract, bids for which were opened in January 1956.

264. As may be seen from the summary of relief well installation data (plate 81), cobbles were encountered in numerous well reaches and

resulted in the installation of about 25 per cent of the wells at less than the design depths. The average specific yield of the wells was 75 gpm per ft with 13 of the wells having a specific yield less than 50 gpm per ft.

265. Permeabilities of the substratum sand computed from the pumping tests on the partially penetrating wells varied from a minimum of 500×10^{-4} cm per sec for wells 96 and 97 to a maximum of 2000×10^{-4} cm per sec for well 109 with an average of 1100×10^{-4} cm per sec for the district.

Man-made seepage hazards

266. Ponds located at about sta 199 and 238 should be observed carefully during high-water periods as they may promote development of concentrated seepage and sand boils.

Harrisonville Levee District

267. The Harrisonville levee district is located in Monroe County, Ill., beginning approximately 12 miles south of Dupo, Ill. The district is bounded on the north by the Columbia levee district and Fountain Creek, on the west by the Mississippi River, on the south by the Ft. Chartres levee district, and on the east by the valley wall. Location of the levee in this district is shown on maps 49-53 and 55-66.

268. The district is protected by a levee that starts at the bluff at sta 0+77, continues as a flank levee down the left bank of Fountain Creek to sta 330 where it becomes a river-front levee, and thence along the left bank of the Mississippi River to sta 1132+91 where it joins the Ft. Chartres levee district. The flank levee varies in height from 10 to 15 ft and is about 150 ft from Fountain Creek. The river-front levee ranges from 12 to 24 ft in height and the distance from the levee to the Mississippi River varies from 700 to 5500 ft.

269. There are no active creeks or rivers within the district. Fountain Creek is a realigned channel from sta 290 to its entrance into the Mississippi River; a segment of the former channel lies within the levee district. The levee crosses the old channel of Fountain Creek at about sta 292 and 469.

Geology and soil conditions

270. The flood plain in this district is about 3 miles wide. The surface geology appears to consist largely of channel bar deposits except for one large clay-filled channel and some other major clay deposits located near the valley wall. The levee crosses the large channel filling in the upper part of the district between sta 10 and 44.

271. The soil profile along the toe of the levee, plates 82-90, shows that the top stratum is very irregular with respect to both type and thickness and that numerous swales and sandy ridges intersect the levee. Cross sections and logs of borings made to delineate certain ground and geological conditions pertinent to the investigation and design of seepage control measures are shown on plates 91-94. Logs of other borings made to determine the type and thickness of the top stratum in the bottom of Fountain Creek or in riverside borrow pits are shown on plate 95. The profiles shown on plates 91-93 show that at the time of investigation (1953) there were numerous scour holes in the bottom of the riverside borrow pits. Most of the major scour holes along the levee have since been filled and abatis dikes constructed to prevent any recurrence of scour until willow growth can develop to hold the soil. (See previously referenced maps of district.) The elevation of rock as determined by borings and electrical resistivity determinations is irregular and varies in elevation from 303 to 244 ft msl along the levee line. The deepest point of the valley floor was encountered at sta 924 where the alluvium was determined to be 146 ft thick.

History of seepage

272. Although the present levee contained the 1947 high water, it was not built to grade except from sta 473 to 898. The levee was completed to project line and grade in 1949. Seepage observations were made and recorded during the 1947 high-water period for the levee reach from sta 473 to 898 and for the entire district during the 1951 high-water period.

273. The 1947 high-water crest was about 9.5 ft below the net grade of the levee. At about sta 671 during the high-water period, considerable underseepage was reported in an area about 100 ft in diameter in the field from 100 to 200 ft landside of the levee. There also was

considerable underseepage from sta 703 to 730.

274. During the 1951 high water the maximum river stage was about 10 ft below the net levee grade. One sand boil occurred near the levee toe opposite approximate levee sta 436 where the net head on the levee was about 8 ft. Seepage and rainwater were observed in low areas throughout the district.

275. No tailwater elevations from high-water periods are available.

276. Piezometers were installed at sta 439+20 after the 1951 high water and prior to the 1952 high water. They were located in lines perpendicular and parallel to the levee toe. However, the 1952 high water created a head of only 2 ft on the levee and the piezometric data obtained are not considered significant.

Design of relief well systems

277. Factors used in the design of relief wells in the Harrisonville levee district are shown on plates 96 and 97. Generally, the wells were located in accordance with the computed spacing. However, the spacing was modified in some instances so that the well location would better fit the topography, or so as to place a well at a critical location with respect to a geological feature. Relief wells were also installed at or along approach channels to certain drainage structures through the levee (see maps 57 and 66).

278. All tailwater elevations were assumed to be at the natural ground except for four wells at two drainage structures. For the wells in the former channel of Fountain Creek at sta 469+30, the tailwater was assumed to be at el 391.5 msl, about 10 ft above the bottom of the channel. Similarly, the tailwater elevation at the wells at the drainage structure at sta 1115+78 was assumed at el 379.5, or 6.5 ft above the bottom of the approach channel.

279. After the wells were installed, scour areas in riverside borrow pits were filled hydraulically with sand in three reaches from sta 493+00 to 546+00, 777+00 to 789+00, and 998+00 to 1053+00 to el 388.0, 384.0, and 380.0 ft msl, respectively. A culvert was placed in a landside ditch and the ditch was filled from sta 160+52 to 164+18 in lieu of installing relief wells.

Cutoff trenches

280. Shallow sands and silty sands underlain by more impervious strata from sta 273+50 to 275+50 and from sta 277+00 to 286+00 have been cut off by construction of a clay-filled trench along the riverside toe of the levee. Borings showed that backfill of an inspection trench resulted in an effective cutoff for most of the levee reach from sta 211 to 222. About 100 ft of the cutoff appeared to be questionable; this reach of levee will be observed during high-water periods.

Piezometer installation data

281. A summary of piezometer installation data is shown on plates 98 and 99; piezometer lines perpendicular to the levee with cross sections and boring logs are shown on plates 100-102.

Well installation data

282. A summary of well installation data is shown in the tables on plates 103-104.

283. The 10 wells with the suffix "A" in the number (excepting well 151A) shown in the above-referenced table are replacement wells. The original wells were abandoned and plugged because they produced excessive amounts of sand during pumping tests.

284. About 145 of the wells were pump-tested a second time, three to five months after installation, because the rate of sand infiltration was not measured during the first pumping test. The specific yields obtained from the second pumping tests vary from 30 per cent higher than the first test to 35 per cent lower; however, 120 of the 145 tests showed lower specific yields during the second test than during the first. Because lower yields were obtained in most of the second pumping tests, 13 of the wells which had the greatest differences in yields for the two tests were pumped a third time; 9 of the 13 wells had been pumped prior to 1 December 1955. Seven of the 9 wells gave yields during the third test which were nearer the yields obtained during the second test than the first test. The reason for the reduced yields obtained during the second and third pumping tests is not known.

285. For consistency, the specific yields shown on the soil profiles, plates 82 through 90, and in the summary of relief well installation

data table (plates 103 and 104) are the specific yields obtained from the first pumping tests. The rates of sand infiltration shown on these plates are those obtained during the second pumping tests.

286. Well 106, as originally installed, produced sand infiltration in excess of that allowed by the specifications. The well then was plugged about 14 ft above the original bottom of the well, pump-tested, and found to be stable. Well 69A showed excessive sand infiltration during pumping tests and is not stable.

287. Cobbles which required the wells to be installed short of design depth were encountered in about 10 per cent of the wells. Most of these wells were located in the reaches from sta 303-308, 437-470, 616-618, and 1051-1058.

288. The specific yields of wells installed short of design depth were considerably lower than for the wells installed with a penetration of 50 per cent of the pervious aquifer. The average specific yield for the wells in the district was 95 gpm.

289. Permeability of the substratum sand computed from the pumping tests on the partially penetrating wells varied from a minimum of 450×10^{-4} cm per sec for well 71A to a maximum of 2650×10^{-4} cm per sec for wells 17 and 112. The average for the district was 1400×10^{-4} cm per sec. Special pumping tests were made on well 151A which penetrated approximately 69 per cent of the pervious substratum, and well 185 with 82 per cent penetration. The average permeability at well 151A was found to be 1970×10^{-4} cm per sec and that at well 185 was 1210×10^{-4} cm per sec. Detailed results of these special tests are given in part VIII.

Ft. Chartres Levee District

290. The Ft. Chartres levee district is located in Monroe and Randolph Counties, Ill. The town of Prairie du Rocher, Ill., lies near the downstream end of the district, which is bounded on the north by the Harrisonville levee district, on the west by the Mississippi River, on the south by Prairie du Rocher Creek and Prairie du Rocher levee district

and on the east by the valley wall. The location of the levee in this district is shown on maps 66-76.

291. The district is protected by a levee which starts at sta 8+50 where it connects with the end of the Harrisonville levee district. The Ft. Chartres levee system consists of river-front levee along the left bank of the Mississippi River from sta 8+50 to 545 and then flank levee on the right bank of Prairie du Rocher Creek to its terminus on high ground at sta 692+44 near the town of Prairie du Rocher. The river-front levee ranges in height from 15 to 25 ft, and the distance from the levee to the Mississippi River varies from 800 to 4000 ft. The flank levee varies from 15 to 20 ft in height and is only about 400 ft from Prairie du Rocher Creek. The Stringtown, Ft. Chartres, and Ivy Landing levee system extends from sta 169+10 to 692+44 but is considered a part of the Ft. Chartres district in this report.

292. Maeystown Creek and One Mile Race Creek are the only creeks that lie within the levee district; they flow through the levee through drainage structures at sta 154+13 and 415+38, respectively.

Geology and soil conditions

293. The soil profile along the toe of the levee, plates 105-109, shows that the top stratum is extremely variable in regard to type and thickness. From sta 96 to 137 the top stratum is nonexistent, or only 1 to 2 ft thick. Cross sections and logs of borings made to delineate certain conditions landward and riverward of the levee pertinent to the investigation are shown on plates 110 and 111.

294. The flood plain in this district is about 3 to 4 miles wide. The surface geology in the district is similar to that in the Harrisonville levee district and consists essentially of ridge and swale formations with a few well-defined major channel fill deposits. The river-front levee crosses numerous minor swales in the district.

295. The pervious sand substratum varies in thickness from about 30 to 100 ft along the levee. The valley floor was found to be fairly regular. Elevations of rock as determined from borings and electrical resistivity along the levee varied from about 260 to 304 ft msl. The deepest point of the valley floor encountered along the levee is at

sta 201 where the alluvium was determined to be 125 ft deep.

History of seepage

296. No seepage data are available for this district; similarly no tailwater elevations from high-water periods are available.

297. Piezometer ranges were installed prior to the 1952 high water at sta 332+12 and 453+36. Piezometers were located on lines perpendicular and parallel to the levee at these locations. The top stratum at sta 332+12 consists of about 12 to 18 ft of clay with silt strata; the top stratum at sta 453+36 consists of 7 to 13 ft of fat clay. The 1952 high water created a net head of 7 ft and 4 ft on the levee at sta 332+12 and sta 453+36, respectively, and heads at the toe of the levee of 43 per cent at sta 332+12 and 62 per cent at sta 453+36. Data from these piezometers were considered in the design of relief wells along these reaches of levee.

Design of relief well systems

298. Factors used in design of relief wells in the Ft. Chartres district are given in the design table on plates 112 and 113.

299. The wells from sta 138 to the drainage structure at sta 154+13 (see map 68) were designed not only to protect the landside toe of the levee but also a drainage ditch a short distance landward of the levee. Relief wells were also installed adjacent to approach channels leading to a number of drainage structures through the levee (see plates 112 and 113 and maps 66, 68, 70, 71, and 73).

300. An interior drainage ditch was excavated by local interests from sta 243 to 333 after the relief wells were installed. Additional relief wells should be installed to provide effective underseepage control, or the ditch should be filled.

301. The design of the relief wells between sta 212+00 and 224+00 was based on $s = 600$ ft , because a deep scour hole was present in the riverside borrow pit. After the wells were installed, hydraulic fill was placed to el 383.0 in the riverside borrow pit from sta 211+00 to 231+00 which may increase the distance to the effective source of seepage over that assumed in the design.

Design of seepage berms

302. Because of the very thin top stratum (2 to 3 ft of clay)

existing from sta 96+00 to 124+00 and the cost of protecting the reach with relief wells (which would have a required spacing of about 50 ft), it was found that a landside extension to the existing berm would result in a more economical solution to the problem of underseepage. The berm extension as designed was based on stripping the top stratum just beyond the toe of the extension, exposing the foundation sand, and using the material thus obtained for the new berm. The berm was designed to have a base width great enough to prevent undermining of the levee by piping, assuming that a minimum creep ratio of 15 would be adequate. The berm varies from 70 to 80 ft in length and from 3.0 to 4.5 ft in thickness at the levee toe (see plates 114 and 115). This design prevents the removal of any more top stratum riverward of the levee and results in economy of dirt handling. Removal of the top stratum landward of the berm over a wide area will prevent the development of any significant hydrostatic pressures and concentration of seepage. Sand or random berms without the removal of top stratum landward of the levee would have to be 100 and 225 ft wider, respectively, than the proposed berm.

Cutoff trench

303. Shallow sands underlain by more impervious strata beneath the levee from sta 85+75 to 94+00 have been cut off by construction of a trench backfilled with clay.

Piezometer installation data

304. A summary of piezometer installation data is shown on plates 116 and 117; piezometer lines perpendicular to the levee with borings and cross sections are shown on plates 118-120. Two lines of piezometers will be installed along the new seepage berm to measure the hydrostatic pressure beneath and at the toe of the berm.

Well installation data

305. A summary of well installation data is presented on plates 121 and 122. Well 20 was plugged and replaced by well 20A. A surface cave-in occurred in well 20 while the gravel filter was being installed and an excessive amount of fines was removed from the well during surging and bailing operations. Wells 71A, 83A, and 139A replaced wells 71, 83, and 139, respectively, because cobbles were encountered considerably above

design depth which made it impracticable to install the wells to design depths. The replacement wells were installed to design depth without difficulty. Wells 134 and 186 produced excessive quantities of sand during pumping tests and are not stable.

306. As may be seen from the summary of relief well installation data on plates 121 and 122, cobbles were encountered along numerous reaches of wells. About 17 per cent of the wells were installed with less than design length of screen because of the cobbles encountered. The specific yield of the partially penetrating wells ranged from about 169 to 25 gpm and averaged 90 gpm with only 10 wells having a yield of less than 50 gpm.

307. Permeability of the substratum sand computed from the pumping tests on the partially penetrating wells varied from a minimum of 700×10^{-4} cm per sec for well 91 to a maximum of 4150×10^{-4} cm per sec for well 165, with an average of 1550×10^{-4} cm per sec for the district. Special pumping tests were made on well 105, which penetrated approximately 90 per cent of the pervious substratum. The average permeability at well 105 was 3000×10^{-4} cm per sec. Detailed results of these special tests are given in part VIII.

Prairie du Rocher Levee District

308. The Prairie du Rocher levee district is located in Randolph County, Ill., approximately 8 miles north of Chester, Ill. The district lies between river miles 119 and 130 (above Cairo, Ill.). The upstream portion of the district is bounded by Prairie du Rocher Creek and the Ft. Chartres levee district, on the west by the Mississippi River, by the Kaskaskia River on the downstream end of the district, and on the east by the valley wall. Levees in this district are shown on maps 74-87.

309. The levee district is protected by a levee system which starts at sta -6+00 on high ground near the town of Prairie du Rocher, Ill. (map 76), and continues as a flank levee down the left bank of Prairie du Rocher Creek to sta 153 (map 74) where it becomes river-front levee, and thence along the left bank of the Mississippi River to sta 710

(maps 84 and 85) where it becomes flank levee on the right bank of the Kaskaskia River to its terminus at sta 870+94 (map 87). For underseepage design purposes, the Kaskaskia River was considered to afford a source of underseepage as the thalweg of the Kaskaskia River generally is below the top of the pervious substratum. The levee is about 400 ft from Prairie du Rocher Creek, the river-front levee varies from 700 to 4000 ft to the Mississippi River, and the levee averages about 500 ft from the Kaskaskia River. The flank levee along Prairie du Rocher Creek is about 20 ft in height; along the river-front levee the height varies from 12 to 25 ft; the levee along the Kaskaskia River varies from 15 to 20 ft in height.

310. There are no active creeks or rivers within the district. Prairie du Rocher Creek is a realigned channel above sta 110; a segment of the former channel lies within the levee district. Interior drainage is provided by a system of drainage ditches and the segment of the former channel of Prairie du Rocher Creek. The creek discharges into the Mississippi River through a drain at sta 342+06 (map 74). The ditches empty into the Mississippi and Kaskaskia Rivers and Prairie du Rocher Creek through gravity drainage structures located in the levee.

Geology and soil conditions

311. The flood plain in this district is approximately rectangular in shape and about three miles wide. Most of the surface geology consists of channel and sand bar deposits typical of the middle Mississippi River valley. Some major channel-filling deposits near the valley wall extend out into the flood plain. A major channel-filling deposit exists landward of the levee and roughly parallel to the present course of the river between sta 430 and 680. The levee crosses several rather large sloughs at sta 315 (map 78, plate 125), 418 (map 80, plate 126), 545 (map 82, plate 126), 658 (map 83, plate 127), 685 (map 84, plate 127), and 862 (map 87, plate 129). The levee also crosses the old channel of Prairie du Rocher Creek at about sta 342 (map 79, plate 125).

312. The soil profile along the toe of the levee is shown on plates 123 through 129. A rather thick uniform deposit of clay extends

from the valley wall to about sta 120 (plate 123). Rather deep deposits of clay overlain by silt also exist from sta 222 to 270 (plate 124) and 352 to 375 (plate 125). The levee is underlain by a thick deposit of clay between sta 521 and 536 (plate 126). A deep channel-fill deposit exists at sta 859 (plate 129). The remainder of the top stratum along the levee consists of alternating strata of clays and silt with numerous sandy ridges and clay- and silt-filled swales. Borings to rock and resistivity and seismic determinations of rock depths show that the valley floor is irregular and varies in elevation from 297 to 231 ft msl along the main levee line. The valley floor is apparently deepest between sta 658 and 676 where there is a deep depression in the rock surface. This relatively deep depression is attributed to erosion of the comparatively soft rock strata by the Kaskaskia River which is of preglacial origin.

313. Cross sections and logs of borings made at locations considered potentially critical with respect to underseepage are shown on plates 130-132. Several of these borings were made to determine the thickness of the top stratum remaining in riverside borrow pits.

History of seepage

314. A large portion of the levee district was flooded during the 1951 high-water period because the levee was not completed from levee sta 554 to 725. River stages of 10 to 18 ft, or 10 to 12 ft below the net grade of the levee, were experienced during this high water. No data are available to indicate what tailwater elevations would exist during flood stages on the Mississippi River.

315. Hydrostatic pressures landward of the levee were obtained during the 1951 high-water period from piezometer lines located at sta 214+55, 231+30, and 863+00. Maximum hydrostatic heads above the natural ground surface landward of the levee at these locations were 35 per cent of the net head on the levee (7 ft), 19 per cent of the net head (7 ft), and 10 per cent of the net head (12 ft), respectively. Readings at sta 846+70 had to be discontinued altogether because of backwater. The top stratum at sta 214+55 consists generally of 6 to 10 ft of clay and silt strata (plate 124); at sta 231+30 it consists of 6 to

12 ft of clay and silt (plate 124); and at sta 866+00 it consists of 4 to 10 ft of clay underlain by silt (plate 129).

316. The piezometric data showed that relief wells probably would not be required along these reaches of levee. However, a new piezometer was installed at sta 846+70 to check on the need for relief wells at this location. All of the old piezometers have since been abandoned.

Design of relief well systems

317. Factors used in the design of relief well systems are shown on plates 133 and 134.

318. It was assumed that the tailwater would be about at the natural ground surface except in a few reaches. Along the low ground from sta 311 to 316, 418, 545 to 547, 683 to 686, and 807 to 809, it was assumed that water would pond to the elevations given in the design table (plates 133-134). In the reach from sta 515 to 522, the tailwater was assumed lower than the natural ground at the levee toe because of lower ground in the slough parallel to and landward of the levee.

319. The spacings of the wells from sta 188+60 to 192+10 and 326+35 to 329+65 were adjusted slightly to avoid road ramps onto the levee. Wells 43 and 44 (map 80) and well 162 (map 87) are located adjacent to approach channels to drainage structures.

320. The well spacing between sta 515+50 and 522+00 was gradually decreased as the landside toe of the levee approached a massive clay-filled channel deposit at a small angle (see map 81 and plate 126). The wells along this reach were set in a shallow ditch so as to lower the outlets and thereby give better protection to the thin top stratum adjacent to the low slough landward of the levee.

Cutoff trench

321. A shallow underseepage cutoff trench has been placed from sta 364+35 to 371+00. Although the logs of borings did not show the necessity for a cutoff trench, it was believed justified because there was softening of the toe of the levee during previous high-water periods.

Piezometer installation data

322. A summary of piezometer installation data is shown on plates 135 and 136; piezometer lines perpendicular to the levee with cross

sections and boring logs are shown on plate 137.

Well installation data

323. Well 35A was installed to replace well 35, the screen of which had parted at a depth of 50.5 ft. However, well 35 was subsequently plugged up to a point above the separation in the screen and the well is to be maintained. Well 56D represented the fourth attempt to penetrate cobbles near the intended location of well 56. An orange peel bucket was used in advancing the hole for well 56D to achieve the design depth. Wells 130 and 131 were replaced by wells 130A and 131A, respectively, as they were not installed to even approximate design depth because of cobbles. Cobbles were encountered primarily in the reach from sta 656 to 675. As may be seen from the summary of relief well installation data (plate 138), this resulted in the installation of six wells at less than design depth.

324. The specific yield of the wells averaged 100 gpm with only three wells having a yield of less than 50 gpm.

325. Permeabilities of the substratum sand computed from the pumping tests on the partially penetrating wells varied from a minimum of 500×10^{-4} cm per sec for well 140, to a maximum of 2300×10^{-4} cm per sec for well 125, with an average of 1500×10^{-4} cm per sec for the entire district.

Perry County Levee District

326. The Perry County levee district is located in Perry County, Mo., and Randolph County, Ill. The town of Chester, Ill., lies across the Mississippi River at the upstream end of the district. The district is bounded on the north by an old chute of the Mississippi River, on the east by the Mississippi River, and on the west by Bois Brule Creek and the valley wall. Levees in the district are shown on maps 88-114.

327. The district is protected by a levee which starts at sta -1+00 at the bluff (map 88), extends as flank levee along the right bank of the old chute to about sta 210 where it becomes river-front levee (map 92), and thence along the right bank of the Mississippi River

to sta 990 where it again becomes flank levee on the left bank of Bois Brule Creek (map 102) to its terminus at sta 1754+98 (map 114). The upper flank levee varies in height from 16 to 20 ft and lies from 400 to 900 ft from the old chute. The river-front levee varies in height from 15 to 25 ft and is 300 to 7000 ft from the Mississippi River. The height of the flank levee along Bois Brule Creek ranges from 15 to 30 ft; the distance from the levee to the creek ranges from 100 to 500 ft. The only creek within this district is Cinque Hommes Creek, which drains through the levee at sta 1142+15 (map 104).

Geology and soil conditions

328. The flood plain in this district varies from 2 to 4-1/2 miles in width. Much of the surface geology in the district is similar to the channel and sand bar deposits in the other districts; however, several reaches of the levee are underlain by rather deep and wide deposits of clay. The levee also crosses numerous swales and a number of rather large sloughs at sta 582 (map 96), 818 (map 100), 1005 and 1031 (map 102), 1065 (map 103), and several sloughs between 1110 and 1120 (map 104).

329. The soil profile along the toe of the levee is shown on plates 139-151. The top stratum consists of rather thick clays from sta 0 to 165 except for a short reach between sta 87 and 97 (plates 139 and 140). From sta 200 to 280, the levee is underlain by a rather continuous deposit of silt underlain by clay. Rather thick deposits of clay underlie the levee along Bois Brule Creek and adjacent to the valley wall from sta 1207 to 1755 (plates 147-151). The top stratum under the rest of the levee is highly irregular with numerous clay- and silt-filled swales. Elevations of rock were determined from borings and electrical resistivity determinations along the levee between sta 340 and 850. Rock elevations downstream from sta 850 will be determined by the St. Louis District at a later date. The valley floor in the levee reach investigated was fairly regular and the elevations of rock varied from el 229 to 285 ft msl. The deepest point of the valley floor encountered was at sta 814 where the alluvium was determined to be 135 ft deep.

330. Cross sections and logs of borings made to develop certain geological features and riverside blanket conditions are shown on plates 152-157.

History of seepage

331. Seepage data collected during the 1947 and 1951 high water pertain to the old levee except the data from about sta 210 to 750 obtained during the 1951 high water pertain to the present levee. However, the alignment of the existing levee or new setback levee is or will be relatively close to the old levee except from about sta 370 to 590. The following seepage data are given on the levee existing in 1947 and 1951 except from sta 330 to 550.

332. During the 1947 high-water period, there was extensive underseepage from sta 300 to 370 and considerable seepage along about 60 per cent of the levee from sta 590 to 1120. Sand boils occurred at sta 717 and 855. Most of the roads in the district were covered by seepage water or were soft as a result of underseepage.

333. During the 1951 high-water period when maximum river stages of 8 to 15 ft (10 ft below net grade of levee) occurred, the worst underseepage reach of levee was from sta 280 to 305. The maximum head on the levee along this reach was about 8 ft. Seepage was observed along about 50 per cent of the river-front levee and along the back levee to sta 1120.

334. During the 1943 high water, a crevasse occurred at about sta 306. The point of initial failure was at an old building with an earth basement. According to local reports the ground beneath the building erupted like a small volcano as the foundation and walls gave way. The resulting break tore out the then existing floodwall at Claryville and a portion of the old levee. The remaining blue hole was subsequently filled with sand. The existing levee is a setback levee.

335. Elevations of the water surface in certain ditches landward of the levee observed during the 1951 high-water period are given below:

Levee Sta	Water Surface, msl Riverside	Water Surface, msl Landside	Net Head in ft
1142+15	372.8	360.4	12.4
1474+15	373.0	365.1	7.9

336. Piezometer ranges had been installed prior to the 1951 high water at sta 271+00, 303+00, 315+00, and 330+00. Piezometers were located in lines perpendicular and parallel to the levee at these locations; however, none of the tips of these piezometers were installed in the pervious substratum except for one piezometer at sta 330, and thus the data were of little value in this study. The one piezometer at sta 330 that was installed in sand showed a hydrostatic head landward of the levee of 4.4 ft with a head on the levee of 11 ft.

337. Piezometer ranges were installed in 1952 at sta 986+50, 998+00, and 1104+93 and piezometers at sta 1051+00 and 1053+01. All of the old piezometers have since been abandoned except the range at sta 330, one piezometer at sta 1051+00, and one piezometer at sta 1053+01, because the tips were not set in sand.

Design of relief well systems

338. Factors used in the design of the relief well systems are shown on plates 158 through 160.

339. Between sta 325 and 346 the ground at the levee toe is about 3 ft lower than the ground farther landward. The design of relief wells was inadvertently based on the assumption that the tailwater would be at the bottom of the low area. However, a subsequent review of seepage records indicated that the low area will probably be flooded during high-water periods. Between sta 350 and 372 the ground about 20 ft landward of the levee is slightly lower than that at the levee toe and as a result the outlets of several of the wells in the reach were lowered slightly. The well at sta 372+10 was located so that it would fall in a low area.

340. The ground landward of the relief wells along the reach of levee between sta 393 and 408 is slightly lower than that at the levee toe. As a result several of the wells in the reach were lowered, and the ground adjacent to the wells was graded to permit the well flow to discharge toward the lower ground landward of the berm toe. The outlet of the relief wells in a slough at levee sta 409+25 was set about 1 ft above the bottom of the slough because it was believed that collection of silt might cover the well unless the outlet was raised. A deep scour hole is present in the riverside borrow pit between levee sta 414 and 420.

It is planned that this hole will be repaired and the pit partially filled by dredged material. Three wells were installed in 1952 in a slough (Missouri chute) at about sta 582+50 (map 96). These wells had 24-ft-long screens and are not considered adequate for underseepage control at this point. A new well was located at sta 582+10 to relieve pressure in the area. The old wells are to be cleaned and renovated if possible. The levee is only about 600 ft from the near bank of the Mississippi River from sta 605 and 617 and about 400 ft from sta 631 to 752. Relief wells along the first reach were designed on the basis of s = 600 ft and 450 to 500 ft along the latter reach. The well at sta 675+00 was placed at greater than the computed spacing so that it would fall in a low area. The wells at sta 753+90 and 755+90 were located at less than the computed spacing so that both wells would fall in the slough.

341. The lengths of well screens in the wells downstream from sta 864 will be determined by the St. Louis District after additional deep borings and rock depth determinations are made. After the deep borings are made, the locations of the wells should be reviewed before any wells are installed. Wells between sta 880 and 893 were located so that they would lie in low areas.

342. Landside borrow pits will be used to enlarge most of the levee from sta 1010 to 1430. The water table was assumed to be above the elevation of the bottom of the proposed pit (see plate 160). In the reaches where there will be landside borrow pits, the wells should be located in the borrow pits at the edge near the levee. The design was based on the thickness of top stratum beneath the bottom elevation of the borrow pit.

Cutoff trenches

343. Shallow, surface sands beneath the riverward toe of the levee from sta 298 to 307 (see plates 141 and 155) should be cut off by means of an impervious cutoff. It may be necessary to install one or two relief wells between sta 300 and 301 if future piezometer readings and observations indicate such are needed.

Piezometer installation data

344. No piezometers were installed in the district prior to 1 December 1955. The locations of proposed piezometers and a summary of

the installation data of piezometers installed by the St. Louis District prior to October 1952 that are to be retained are shown on plates 161 and 162. A piezometer line perpendicular to the levee installed prior to 1952 is shown on plate 162.

Borrow pit corrective measures

345. Scour in riverside borrow pits is to be repaired by filling from sta 281-285, 387-391, 394-407, 410-432, 476-486, 492-512, and 530-542. Willows are also to be planted in belts across the borrow pits between sta 0+00 and 990+00. There will be about 40 belts, each 200 or 300 ft wide.

Well installation data

346. All of the relief wells proposed upstream from sta 549+80 have been installed; no wells have been installed downstream of sta 549+80. Wells were respaced in the reach from sta 514+30 to 549+80 because of inability to obtain design depth due to the presence of cobbles. The respacing resulted in five additional wells in this reach. None of the wells reached the design depth and only one in the reach penetrated more than 50 per cent of the pervious substratum.

347. As may be seen from the summary of relief well installation data (plates 164 and 165), cobbles were encountered in numerous reaches of wells where wells have been installed. About 47 per cent of the wells were installed with varying percentages of less than design screen because of inability to penetrate cobbles. The five additional wells mentioned above compensate somewhat for the short wells in the reach from 514+30 to 549+80. The average specific yield of the wells in the district was 78 gpm per ft. From sta 506+80 to 549+80 all of the specific yields were low, ranging from about 20 to 50 gpm per ft.

348. The permeabilities of the substratum sands computed from the pumping tests on partially penetrating wells varied from a minimum of 350×10^{-4} cm per sec for well 82X to a maximum of 2750×10^{-4} cm per sec for well 10, with an average of 1200×10^{-4} cm per sec for all the wells installed to date in the district.

Man-made seepage hazards

349. A stock pond exists near the toe of the levee at about

sta 285, and should be observed during high-water periods.

Degognia Levee District

350. The Degognia levee district is located in Jackson County, Ill., approximately 8 miles west of the city of Murphysboro. The upstream portion of the district is bounded by Degognia Creek, and by the Grand Tower levee district on the downstream side. Levees in this district are shown on maps 115-127 and 142-147.

351. The district is protected by a levee which starts at sta 0+00 on high ground in the vicinity of Cora City, continues as flank levee down the left bank of Degognia Creek to sta 45+00 where it becomes river-front levee, and thence along the left bank of the Mississippi River to sta 827+67 where it ties into Fountain Bluff. The levee along the Big Muddy River was completed in the fall of 1955. No levee will be required from sta 1071+85 to 1164+10 because of high ground along this reach. No investigation of underseepage was made along the new levee. The proposed new levee will be classed as a flank levee. The present flank levee varies from 13 to 20 ft in height, and the river-front levee ranges in height from 17 to 25 ft. The present flank levee is about 150 to 400 ft from Degognia Creek; the river-front levee ranges from 500 to 5500 ft from the Mississippi River.

Geology and soil conditions

352. The flood plain in the district varies in width from approximately 1 mile at the upper end to 5 miles at the lower end of the district. Major channel-filling deposits exist near the east valley wall and extend to a considerable distance out into the flood plain. These deposits are shaped in the familiar oxbow pattern indicating that they were formed in old river channels abandoned by natural cutoffs of the river. There is little or no evidence of major channel-filling deposits in the area adjacent to the present course of the river.

353. A formation at the lower boundary of the district known as Fountain Bluff is of special interest. This bluff is a segment of the west valley wall and is at present cut off from that wall by the

Mississippi River. The river undoubtedly flowed around Fountain Bluff on the east side of the valley at one time; however, all evidence of former channels in this area have been masked by thick fine-grained deposits similar in nature to backswamp deposits.

354. The soil profile along the toe of the levee is shown on plates 166 through 171. Cross sections and logs of borings made at locations pertinent to the study and design of seepage control measures are shown on plate 172. Logs of holes for relief wells show the existence of a considerable amount of coarse gravel and cobbles in the lower part of the substratum in this district. Borings to rock and seismic determinations of rock depths show that the valley floor is irregular and varies in elevation from 264 to 179 ft msl along the levee line. The valley floor is much deeper in the vicinity of Fountain Bluff than elsewhere in the district.

History of seepage

355. The present levee system was completed to project line and grade after the 1947 high-water period. The only seepage data available for study, therefore, were collected during the 1951 high-water period in which the levee was subjected to a maximum river stage of about 10 to 15 ft on the levee or 10 ft below the net grade of the levee. During this high water, heavy seepage was noted between sta 195 and 290, 375 and 410, 430 and 500, and 510 and 600. Seepage to a lesser degree was noted between sta 295 and 325, 340 and 370, and 600 and 765. Some caving of the banks of a landside ditch occurred as a result of seepage at approximately sta 165 and 365. A small boil area was treated at approximately sta 550.

356. Elevations of the water surface observed during the 1951 high water in ditches landward of certain drainage structures are given below:

Levee Sta	Water Surface, msl Riverside	Water Surface, msl Landside	Net Head in ft
157+75	372.1	359.0	13.1
366+94	371.8	356.5	15.3
492+60	370.7	357.8	12.9
666+02	368.1	355.6	12.5
735+84	366.7	352.6	14.1
816+20	366.4	354.1	12.3

357. Several piezometers were installed in the district after the 1951 high-water period and observations were made during the April-May 1952 high-water period. Piezometer ranges were located at sta 95+34, 546+05, and 598+72. The piezometers at sta 598+72 have since been abandoned.

Design of relief well systems

358. Factors used in the design of relief wells are shown on plates 173 and 174.

359. The well reach between sta 157+00 and 166+10 is located on the bank of a rather deep drainage ditch which was cut through the top stratum into the underlying sand. The wells in this reach are spaced relatively close together (75 ft) because the ditch bottom is considered to be particularly critical with respect to underseepage. One relief well was located at sta 174+00 at the edge of an old channel-filling deposit considered critical with respect to underseepage.

360. Scour areas in borrow pits from sta 464 to 483 have been filled to el 352.0 ft msl, and scour areas in pits from sta 532+50 to 551+00 and 560+00 to 663+00 have been filled with hydraulic fill to about el 350 ft msl. This borrow treatment was considered in the selection of the source of seepage entry in the design of relief wells along these reaches of levee.

Piezometer installation data

361. A summary of piezometer installation data is shown on plate 175; piezometer lines perpendicular to the levee with cross sections and boring logs are shown on plate 176.

Well installation data

362. A summary of relief well installation data is shown on plates 177 and 178.

363. The existence of cobbles seriously complicated the installation of wells in this district. Where wells could not be installed to design depths, they were respaced as nearly as possible in accordance with the criteria described in paragraph 522 in part IX.

364. Well 5 was relocated because cobbles were encountered at a high elevation, and wells 63 and 75 were replaced by wells 63A and 75A for

the same reason. Well 9 was replaced by well 9A because of a cave-in during installation.

365. A total of 13 additional wells were installed in accordance with the procedure for respacing wells where a design depth could not be obtained. However, as the yield of the original and additional wells was still less than that considered necessary to give adequate pressure relief, a review of the control measures installed was made upon completion of the original contract. It was found that numerous wells in several well reaches have low specific yields due to the small penetration of aquifer. These reaches are from sta 99 to 119, 157 to 166, 246 to 256, 751 to 766, and 805 to 811. Nine additional wells and six piezometers have been proposed in the above reaches as shown on plates 166-168 and 171. Also wells 6 and 84 are to be plugged and replaced because they have small amounts of design screen and low specific yields. Wells 15 and 89 are to be replaced because of excessive sand infiltration. Well 22 had not been installed prior to 1 December 1955 because right-of-way could not be obtained.

366. Two 6-in.-ID relief wells had previously been installed at a drainage structure at sta 735+84. However, during a reconnaissance of the levee, it was discovered that these two wells had been filled with rock. Rather than attempt to renovate these wells they are to be grouted, and four new wells have been installed along the inlet channel to this structure (see map 125).

367. About 45 per cent of the wells in the district were installed at less than design depth. The average specific yield of the wells in the district was 80 gpm.

368. Permeabilities of the sand substratum computed from the pumping tests on the partially penetrating wells varied from a minimum of about 500×10^{-4} cm per sec for well 4 to a maximum of 3500×10^{-4} cm per sec for well 150, with an average for the district of 1100×10^{-4} cm per sec.

Grand Tower Levee District

369. The Grand Tower drainage and levee district is located in

Jackson County, Ill., and Perry County, Mo., approximately 25 miles north of Cape Girardeau, Mo. The district is bounded by the Big Muddy River on the east and south, by the Mississippi River on the west, and on the north by Fountain Bluff and the Degognia levee district. The levee in the Grand Tower levee district is shown on maps 128-142.

370. The district is protected by a levee which starts at sta 0+00 at Fountain Bluff and continues as river-front levee along the Mississippi River to sta 310+00 where it becomes flank levee on the right bank of the Big Muddy River, and thence extends to about sta 936+40, except where the river-front levee is interrupted by high ground between sta 69+93 and 102+90. The height of the river-front levee ranges from 8 to 25 ft; the distance from the levee to the Mississippi River varies from 300 to 3000 ft. The flank levee varies in height from 15 to 25 ft and lies about 350 to 1300 ft from Big Muddy River between sta 310 and 640. Upstream from sta 640 the distance from the levee to the Big Muddy River varies from about 2000 to 7000 ft.

371. There are no creeks or rivers in the district; however, a number of sloughs provide some interior drainage. Most of the interior drainage, especially in the area east of State Highway No. 3, consists of a system of ditches that discharge into the Big Muddy River through gravity drainage structures. The area west of State Highway No. 3 drains into Grand Tower Island chute, which in turn drains into the Big Muddy River through a gravity drainage structure located at sta 314+30. The chute will act as a storage area with an estimated maximum tailwater elevation of 343.0 ft msl during flood stages. Two small gravity drainage structures, located at sta 23+05, and 106+00, discharge directly into the Mississippi River. Approximately two-thirds of the length of the levee system in the district borders on Big Muddy River.

Geology and soil conditions

372. The geology of the district is interrelated with that of the Degognia levee district when the Mississippi River flowed east of Fountain Bluff. All traces of early abandoned channels of the river in the northern part of the district are masked by thick deposits of clay resembling backswamp deposits.

373. The area along the Mississippi River immediately upstream from the town of Grand Tower is of special interest. It appears that the Mississippi River, after breaking through and separating Fountain Bluff from the west valley wall, may have occupied a course to the east of the town and later broke through and isolated another fragment of the west valley wall. This fragment forms the high ground between sta 69+93 and 102+90.

374. The soil profile along the toe of the levee is shown on plates 179 through 185. Cross sections and logs of miscellaneous borings made to develop ground and soil conditions pertinent to the study are shown on plate 186. Logs of holes made for relief wells show the existence of considerable gravel and cobbles in the lower part of the pervious substratum in this levee district. Borings to rock, and seismic and resistivity rock depth determinations, show that the valley floor is very irregular and varies in elevation from 246 to 146 ft msl along the levee line. The deepest point of the valley floor was encountered at sta 540+00 where the alluvium was determined to be 204 ft deep; the shallowest point is at sta 280+00 where the alluvium is 100 ft deep.

History of seepage

375. A crevasse attributed to underseepage occurred between sta 480 to 485 during the 1947 high water. During the 1947 high-water period breaks occurred in the levee at about sta 500, 600, and 620; however, these breaks were not attributed to underseepage. No seepage data are available for the 1951 high-water period. No data are available to indicate what tailwater elevations would be obtained during flood stages of the river. No piezometric data are available for the district.

Design of relief well systems

376. Factors used in the design of relief wells are shown on plates 187 and 188.

377. The line of wells between sta 147+00 and 169+50 is located along the bank of a rather deep drainage ditch which parallels the levee toe (see maps 129 and 130 and plate 180). There is approximately 20 ft of top stratum beneath the bottom of the ditch and the underlying sand strata; however, in view of the surrounding thick top stratum and the high head (28.5 ft), the ditch creates a potentially critical seepage

condition. In order to prevent serious sand boils in the bottom of the ditch, relief wells have been installed along the ditch at 250-ft intervals.

378. These wells are provided with a tee and horizontal discharge pipe set at el 345 to 344 or about 1 to 2 ft above the bottom of the drainage ditch. The elevation of tailwater assumed in this ditch is 343.0. The reach of levee from approximate sta 176 to 183 (map 130) is a closure section for the upper end of Grand Tower Island chute; this levee was constructed of hydraulically placed fill. As there is little or no top stratum landward of the levee across the chute (plate 180), the levee was checked by the creep ratio method and found to have a creep ratio of 15, which is considered adequate.

379. The wells in the reach from 264+75 to 276+35 were located at spacings varying from 190 to 350 ft so that they would fall at the edges of swales or in low areas (see plate 180). The reach of levee from approximately sta 295 to 303 is a closure section for the lower end of Grand Tower Island chute (map 132). This levee was built of semipervious material placed by hauling equipment. The top stratum in the bottom of the old chute (plate 181) was not of sufficient thickness to withstand safely the estimated head of 27 ft at a project flood stage. Accordingly, 3 wells on 200-ft centers were installed across the old chute. These wells were located 10 ft leveeward of the berm crown and were provided with tee outlets and horizontal discharge pipes set at about el 338.0 (assumed tailwater el 243.0). The wells were so located because soft clay and muck, which had been squeezed out the toe of the berm during construction of the levee, would have made well installation very difficult at the toe of the berm. The levee between sta 357 and 371 (map 135) roughly parallels a landside slough. Eight wells on 200-ft centers (plate 181) were installed along this reach 5 ft leveeward from the high bank of the slough with the outlets discharging into the slough. The wells were located on the bank of the slough so that no clearing of the area would be required and for ease of maintenance of the wells. The wells along this reach were provided with tee outlets and 12-in. horizontal discharge pipes. Thirty-six-in.-diameter riser extensions were used from the tee up to the ground surface. The outlets were set at about el 344.0 for the

wells between sta 357 and 367. Although this elevation is 1.0 ft above that assumed for the tailwater in design, lowering of the outlets below this would have required still more excavation and longer discharge pipe. The outlets for the wells at sta 369 and 371 were set at el 345.0 for the same reasons. A cross section at this group of wells is shown on plate 186.

380. A levee crevasse, which was attributed to underseepage, occurred in 1947 between sta 480 and 485. Relief wells on 200-ft centers have been installed between sta 473+25 and 487+50.

381. Tailwater elevations in ditches and low areas west of State Highway No. 3 were set at el 343 msl, the estimated maximum ponding elevation in Grand Tower Island chute. In the area east of State Highway No. 3, tailwater elevations were assumed to be at natural ground elevation in all well reaches except at sta 714+54 where the tailwater was assumed to be at el 349 msl, approximately 3 ft above the bottom of the ditch.

Design of seepage berms

382. Factors used in the design of berms in the Grand Tower levee district are presented in the table on plate 188.

383. Because of the relatively thin top stratum (about 4 ft of silt) from sta 234+00 to 245+50 (plate 180) and the cost of protecting the reach with relief wells (which would have required a well spacing of less than 100 ft), it was found that a seepage berm would result in a more economical solution to the problem of underseepage. The berm is to be constructed of silt obtained riverward of the levee. The computed width of berm, 305 ft, would have caused the berm to extend into a sawmill area from about sta 241 to 242. Because of the difficulty in obtaining the right-of-way in the sawmill area, it is tentatively planned to build a berm only 170 ft wide from sta 234+00 to 243+00 (see map 131 and plate 190). From sta 244+00 to 245+50 the proposed berm will be an extension of the previously constructed berm and the extension will be about 140 ft wide (plate 190). The average thickness of the proposed berm is about 4 ft at the levee toe from sta 234+00 to 243+00; the thickness of the existing berm from sta 244+00 to 245+50 at the toe of the levee is also about 4 ft.

384. Four wells were originally proposed along the inlet channel to a gravity drain at sta 314+28. As right-of-way could not be obtained for the wells, they were replaced by 200 ft of 42-in.-diameter pipe and berm over the pipe up to the elevation of natural ground (map 132).

Piezometer installation data

385. A piezometer line was installed at sta 238+00 to measure the substratum pressures under the seepage berm during high water. A piezometer line was also installed at sta 300+00 midway between relief wells on the lower closure section of Grand Tower Island chute to check the performance of the wells and to determine the distance to the source of seepage in this chute (map 132).

386. A summary of piezometer installation data is shown on plate 191; piezometer lines perpendicular to the levee with cross sections and boring logs are shown on plates 192 and 193.

Well installation data

387. A summary of well installation data is shown on plate 194.

388. Wells 9 and 10 were not installed because cobbles, which could not be penetrated by the equipment being used, were encountered at a depth too shallow to place any design screen. The wells were to be located along a drainage ditch about 400 or 450 ft from the riverside toe of the levee. In view of the distance between the levee toe and the ditch, no further attempts were made to install the wells. Two piezometers, P-3A and P-3B, were installed at the locations of the wells to check on substratum pressures beneath the ditch (see map 130 and plate 180).

389. About 30 per cent of the wells in the district were installed short of design depth because of cobbles encountered above design depths. Most of these wells were in the reach of levee from sta 147+00 to 199+40. Eleven additional wells were installed to compensate for the lack of penetration of some of the design wells. Specific yields of the wells in the district range from about 50 to 300 gpm with an average of 110 gpm (plate 194). The yield of wells from sta 400 to the end of the levee ranges from 100 to 300 gpm.

390. Permeabilities of the substratum sand computed from the pumping tests on the partially penetrating wells varied from a minimum of

500 x 10^{-4} cm per sec at well 15 to a maximum of 2500 x 10^{-4} cm per sec at well 116 with an average of 1300 x 10^{-4} cm per sec for the district.

Preston Levee District

391. The Preston levee district is located in Union County, Ill., approximately 20 miles north of Cape Girardeau, Mo. The district is bounded on the north by the Big Muddy River and Grand Tower levee district, on the west by the Mississippi River, on the south by Clear Creek levee district, and on the east by Miller Pond levee district and the valley wall. The levee in the Preston levee district is shown on maps 132-136 and 148-154.

392. The district is protected by a levee which starts at sta 0+00 at the edge of the east valley wall and extends down the left bank of Big Muddy River as flank levee to sta 290+00, and thence as river-front levee along the Mississippi River to sta 772+54, the end of the district. The flank levee varies in height from 15 to 25 ft and is from 300 to 700 ft from the Big Muddy River except upstream from sta 30 and downstream from sta 250 where the levee is as much as 2200 ft from the Big Muddy River. The river-front levee ranges in height from 15 to 30 ft, and the distance from the levee to the Mississippi River varies from 500 to 3000 ft. There are no creeks or rivers in the district.

Geology and soil conditions

393. The district is roughly rectangular in shape and varies in width from approximately 4 miles at the upper end to approximately 2 miles at the lower end. The surface geology consists essentially of point-bar deposits with no major channel fillings.

394. The soil profile along the toe of the levee is shown on plates 195 through 200. Cross sections and logs of miscellaneous borings made at certain locations to develop ground and soil conditions pertinent to the study are shown on plates 201-202. The valley floor was found to be fairly regular. Elevations of rock as determined from electrical resistivity reconnaissance along the levee varied from about 210 to 247 ft msl. The deepest points of the valley floor encountered along the

levee were at sta 560+00 and 574+00 where the alluvium was determined to be 138 ft deep and the shallowest point at sta 135+00 where the alluvium is 106 ft deep.

History of seepage

395. Considerable seepage was reported along the entire length of the levee during the 1947 and 1951 high-water period; however, no serious trouble or sand boils were experienced.

396. Elevations of the water surface in certain ditches landward of the levee observed during the 1951 high water are given below:

Levee Sta	Water Surface, msl Riverside	Landward	Net Head in ft
293+02	360.0	351.2	8.8
409+80	358.2	345.0	13.2
709+46	355.2	344.0	11.2

397. Three installations of piezometers were made in the district prior to 1952 at sta 276+73, 550+45, and 619+20. During the 1952 high-water period a river stage of only about 2.5 ft on the levee was experienced and was of insufficient duration to cause the development of excess hydrostatic heads landward of the levee. Of the existing piezometer installation, only the installation at sta 550+45 will be maintained.

Design of relief well systems

398. Factors used in the design of relief well systems are given on plates 204 and 205.

399. The spacing of the wells at sta 573+00 and 574+50 was adjusted so that the wells would be located at the upstream and downstream edges, respectively, of a swale (plate 199). The computed spacing of the wells in the reach from sta 695+50 to 723+00 was adjusted to fit the topography (plates 199 and 200). Two wells were also located near the drain at about sta 709, and two wells in the slough between sta 719+25 and 720+50 (plate 200). Tailwater elevations were assumed to be at the natural ground surface on the landside toe of the levee in all cases except for the reach of levee between sta 128+00 and 132+00 where a landside ditch parallels the levee toe. It was assumed that water would pond in the ditch to el 347 msl along this reach. Other assumptions made in

the design of relief wells are shown on plates 204 and 205.

Piezometer installation data

400. It is planned to install a piezometer line at sta 655+00 which will extend landward a distance of 500 ft from the levee center line (map 152). No new piezometers have yet been installed in the district because of inability to obtain right-of-way. A summary of installation data of piezometers installed by the St. Louis District prior to 1952 and locations of piezometers proposed are shown on plate 206. Piezometer lines are shown on plate 207.

Borrow pit corrective measures

401. Scour in riverside borrow pits is to be repaired from sta 302-373, 420-441, 465-475, 481-501, 567-685, 710-722, and 768-772. Willows are to be planted across borrow pits in various levee reaches totaling 5700 ft, 3000 ft of which are between sta 508 and 642.

Well installation data

402. No relief wells had been installed in the district prior to 1 December 1955 because of lack of right-of-way. The locations of proposed wells are shown on plate 208 and on the maps of the district.

403. Since no wells have yet been installed, no results of pumping tests are available in the district. The permeability used for computing well flows was 1400×10^{-4} cm per sec, the average computed from the pumping tests made on all wells installed in other districts prior to 1 April 1955.

Clear Creek Levee District

404. The Clear Creek levee district is located in Union and Alexander Counties, Ill., approximately 5 miles north of Cape Girardeau, Mo. The district is bounded on the north by the Preston and Miller Pond levee districts, on the west by the Mississippi River, on the south by the East Cape Girardeau and North Alexander levee districts and the channel of Old Clear Creek, and on the east by Clear Creek. The levee in the Clear Creek levee district is shown on maps 154-163 and 175-182.

405. The district is protected by a levee system which starts at

sta 0+00 at the boundary with the Preston levee district and continues along the Mississippi River as river-front levee to sta 547+57 where the district joins the East Cape Girardeau levee district. The Clear Creek levee district is protected from overflows of Clear Creek by a flank levee which connects with the levee at the end of the North Alexander levee district at Clear Creek back levee sta 275+35 (see North Alexander levee district). The back levee continues upstream along the right bank of Clear Creek to the end of the Clear Creek district back levee at sta 812+85, which is the beginning of the levee in Miller Pond levee district.

406. The height of the river-front levee ranges from 15 to 27 ft, and the distance from the levee to the Mississippi River varies from 500 to 4400 ft. The height of the back levee varies from 12 to 25 ft, and the distance from the levee to Clear Creek ranges from 100 to 1800 ft.

407. There are no creeks or rivers in the district. Interior drainage is provided by a system of drainage ditches and a segment of the channel of old Clear Creek.

Geology and soil conditions

408. The flood plain in this district is approximately rectangular in shape and about 3 miles wide. Major channel fillings exist in the area between the east valley wall and State Highway No. 3. These channel-filling deposits are shaped in the familiar oxbow pattern indicating that they were formed in old river channels abandoned by natural cutoffs of the river. There is little or no evidence of major channel fillings in the area adjacent to the present course of the river. The deposits in this area appear to be the result of very slowly migrating river channels as the river meandered in a narrow range. The deposits appear to be similar to point-bar deposits with ridge and swale topography, but these features are usually either straight or have a very gentle curvature.

409. The soil profile along the toe of the levee is shown on plates 209-213 and 239-242. Cross sections and logs of miscellaneous borings made to develop certain geological and ground conditions are shown on plates 214-216 and 243. The top stratum along the back levee was sufficiently thick so as to preclude the need for relief wells except at

certain drainage structures and discontinuities in the top stratum. The elevation of the valley floor along the river-front levee as determined by borings and electrical resistivity varied from 216 to 247 ft msl. The deepest point of the valley floor encountered along the river-front levee is at sta 333 where the alluvium was determined to be 125 ft deep and the shallowest point at sta 52 where the alluvium is 95 ft deep. The three elevations of rock determined along the back levee were 192, 196, and 229 ft msl.

History of seepage

410. At the time of the 1947 high-water period, the river-front levee had been constructed along the present alignment and up to project grade except from about sta 200 to 213. (The back levee has not yet been completed to project grade.)

411. During the 1947 high-water period, all low areas in the fields from sta 200 to 415 were covered with water. From sta 415 to 490 the road along the toe of the levee and the lower part of the levee became very soft in places. Some of the worst places along the levee were loaded with sandbags. There was considerable seepage and pin boils between Highway 3 and the levee at the town of McClure, Ill. Here a ring levee was built around the seepage area (see plate 2). There was about 10 ft of head on the levee at McClure. Conditions were so soft along about 500 ft of the highway that it was necessary to close the highway to most traffic.

412. During the 1951 high water, there was seepage landside of the levee almost continuously from sta 13 to 506. Prior to the 1951 high-water period, a relief well system consisting of 20 wells was installed at McClure, Ill., opposite levee sta 531 to 546. When the head on the levee was about 10 ft during the 1951 high-water period, several pin boils were observed along Illinois Highway 3 opposite approximate levee sta 533 to 538. However, these pin boils along the highway during the 1951 high water were considerably less active than those observed during the 1947 high water at comparable river stages when there was no well system along this reach of levee.

413. Elevations of water in ditches landward of the levee observed

during the 1951 high water are given below:

Levee Sta	Water Surface, msl Riverside	Landside	Net Head in ft
	River-front Levee		
11+97	354.3	342.7	11.6
144+93	353.3	343.0	10.3
323+85	352.0	338.2	13.8
495+18	350.9	339.9	11.0
	Back Levee		
275+85	343.6	336.6	7.0
337+69	343.3	336.2	7.1
536+52	342.6	337.3	5.3
646+56	343.2	337.5	4.7

414. Piezometer lines were installed at sta 529+73, 531+98, 533+73, 537+88, 539+98, 541+73, and 543+37, respectively, prior to the 1951 high water. Lines at sta 531+49 and 539+88 were installed after the 1951 high water but prior to 1952. Data are available for the other lines for both the 1951 and 1952 high-water periods. Soil conditions at the above piezometer lines are shown on plate 221. During the 1951 high water, the net head on the levee was about 11 ft at the piezometer lines; during the 1952 high water, the net head on the levee was about 7 ft. The hydrostatic head at the levee toe and at certain distances from the levee toe is shown in table 7 for existing piezometer lines for the 1951 and 1952 high-water periods.

415. Piezometers P-70 through P-85 were installed between sta 530+43 and 546+43 and are located midway between relief wells installed prior to 1951. During the 1951 high-water period, all of the piezometer readings were below natural ground surface except P-70, P-83, and P-85 at sta 531+03, 544+43, and 546+43, respectively, where the net head on the levee was about 9 ft and the piezometer readings at the toe of the levee were 0.54, 0.90, and 1.35 ft, respectively. During the 1952 high-water periods all of the piezometers, P-70 through P-85, gave readings below the natural ground surface.

Design of relief well systems

416. Factors used in the design of the relief wells are shown on plates 217-218 and 244.

Table 7

Hydrostatic Head Landward of Levee

Stationing of Piezometer Line	Hydrostatic Head Landward of Levee, in ft				
	At Landside Toe of Levee	100	200	400	500
	1951 High Water				
529+73	3.4	3.3	3.1	---	---
533+73	3.3	---	2.5	---	---
537+88	3.3	---	1.7	0.2	---
541+73	3.8	---	2.9	---	2.0
543+37	2.6	---	0.5	---	0.8
	1952 High Water				
529+73	2.3	1.9	1.9	---	---
531+98	2.1	---	---	---	---
533+73	1.9	---	0.6	---	---
537+88	1.9	---	0.8	---	---
539+98	2.3	0.8	---	---	0.1
541+73	2.3	---	0.6	---	0.3
543+37	1.3	---	1.2	---	0.1

417. A 6-in. corrugated metal well with perforated screen was installed in 1949 at a culvert at sta 11+97. This well is 20 ft deep and has 16 ft of screen. The well was not pump-tested at the time of installation, and its capacity at the present time is not shown. It is planned to surge the well and pump-test it to determine its efficacy.

418. In 1949, 16 relief wells were installed on 100-ft centers from sta 531 to 546. These wells were approximately 52 ft deep with screen lengths of 30 ft. Riser pipes and screens were 12 in. in diameter. The riser pipe is made of bituminous-coated corrugated metal pipe; the screen is of the same pipe, but is perforated with 7/16-in. holes. The well screens were surrounded with a fine gravel filter. The wells are provided with tee outlets and discharge into a shallow, unlined collector ditch. The outlets are provided with wire mesh protection. In addition to these 16 wells, 4 experimental wells were installed in pairs adjacent to wells A-4 and A-12 to study the suitability of plastic, fiber, and concrete materials for relief wells. Inspections made at the time of investigation in the Clear Creek levee district indicated that considerable surface water had flowed into these wells during periods of heavy rainfall.

Therefore, it is planned to redevelop these wells and to resurge them to determine their efficacy. (The wells were not pump-tested at the time of installation.) Provisions should be made to prevent future surface flow into these wells.

419. The wells in the levee reach from sta 531+50 to 544+50 were arbitrarily spaced midway between existing wells. Although the existing wells relieve the seepage problem to some extent, as demonstrated during the 1951 high-water period, they are not considered adequate with regard to spacing and depth to control underseepage for a project flood. It is planned to pump-test the existing wells to determine their capacity; however, they had not been tested prior to 1 December 1955. If the capacity of these wells appears insufficient, it may be necessary to supplement the proposed new well system with additional wells. All proposed wells along the back levee except wells 1 through 3 are located at culverts or at abrupt changes in the top stratum at the edge of channel-filling deposits.

420. The tailwater in the Clear Creek levee district was assumed to be at the natural ground surface except at drainage structures and in the landside ditch from sta 13+50 to 15+50. Wells 4 and 5 will be located adjacent to the landside ditch (plate 209, map 154). Estimated well flows were based on the average permeability computed from routine pumping tests made in all other levee districts prior to 1 April 1955 (1400×10^{-4} cm per sec).

421. Deep borings for well screen design have not been made in this levee district nor have the well screens been designed. After the deep borings have been made, the proposed locations of all of the relief wells should be reviewed because the deep borings may define more exactly the top stratum in some reaches.

Piezometer location data

422. No new piezometers had been installed in the district prior to 1 December 1955. A summary of the installation data for existing piezometers and the locations of proposed piezometers are shown on plate 219. Each piezometer line is shown on plate 221.

Borrow pit corrective measures

423. It is planned to repair scour areas in riverside borrow pits

from sta 0-20, 23-36, 104-139, 200-223, and 236-244. Willows will be planted across the borrow pits generally from sta 8-11, 23-26, 75-95, 105-117, 127-129, 134-136, 198-200, and 207-262.

Well installation data

424. No new wells had been installed in this district prior to 30 December 1955. Locations of both new and old wells are shown on plates 222 and 244.

East Cape Girardeau Levee District

425. The East Cape Girardeau levee district is located in Alexander County, Ill., directly across the Mississippi River from Cape Girardeau, Mo. The district is bounded by the Mississippi River on the north, west, and south and by the North Alexander levee district on the east. Location of the levee in this district is shown on maps 163-171.

426. The district is protected by a levee which starts at sta 0+00 at the boundary of the district with Clear Creek levee district and continues as river-front levee along the Mississippi River to sta 527+00 where it turns up the right bank of Clear Creek and runs as flank levee to sta 571+43, the boundary of the district with the North Alexander levee district. The river-front levee generally ranges in height from 15 to 25 ft and the distance from the levee to the Mississippi River varies from 400 to 5500 ft. From sta 0+00 to 110, Devil's Island chute is 200 to 1500 ft from the levee (maps 163 and 164). The flank levee is about 18 ft high and is about 150 to 500 ft from Clear Creek.

427. There are no creeks or rivers within the district. Interior drainage is provided by a system of drainage ditches and sloughs or low areas that discharge through gravity drainage structures in the levee at sta 410+35, 440+00, and 524+24.

Geology and soil conditions

428. The surface geology in the district appears to be point-bar deposits with typical ridge and swale topography with the possible exception of the extreme lower end of the district where an old channel-filling deposit may exist. Evidence of such an old river channel filling

in this area is somewhat obscured on the aerial photographs by the construction of railroad, highway, and levee embankments.

429. The soil profile along the toe of the levee is shown on plates 223-227. This profile shows that the top stratum, consisting essentially of silty and clay soils, is generally continuous from sta 0 to sta 250. Numerous discontinuities in the top stratum may be noted between sta 250+00 and 448+00. Along several reaches the underlying sand strata are exposed at the surface. Cross sections and logs of borings made to delineate certain ground and geological conditions pertinent to the investigation and design of control measures are shown on plates 228 and 229.

430. Resistivity determinations of rock depths show that the valley floor varies generally in elevation between 225 and 245 ft msl. The deepest part of the valley was at sta 400 where the elevation of the valley floor was 202. In general, the depth of the alluvial fill ranged from about 95 to 115 ft.

History of seepage

431. Extensive underseepage and soft ground landward of the levee were reported for the entire district during the 1951 high-water period. The two worst underseepage areas reported were between sta 12 and 15 and between sta 68 and 71 where severe sand boils were sandbagged. The road which passed along the toe of the levee existing at that time from sta 0+00 to 60+00 was reported to be extremely wet, soft, and unusable. Since the 1951 high-water period, the levee in the district has been enlarged. This enlargement was a landside enlargement and provided for the construction of a landside seepage berm between sta 0+00 and 131+00 which now covers the sand boil areas and the road along the toe of the old levee. From the logs of borings on plate 223, it appears that much of the seepage reported from sta 0 to 37 could be attributed to relatively shallow seepage through sands and silty sands immediately beneath the levee. A riverside impervious cutoff approximately 10 ft deep and 6 ft wide was constructed at the riverside toe of the levee between sta 0+00 and 123+00.

432. The elevation of water in a drainage ditch at sta 524+24 observed during the 1951 high water is shown on the following page:

Levee Sta	Water Surface, msl Riverside	Landside	Net Head in ft
524+24	343.2	330.0	13.2

433. Piezometric data were obtained from a piezometer installation located at sta 8+25 during the 1951 high-water period when a head of 10.5 ft was experienced on the levee. A maximum hydrostatic head of 2 ft was recorded at the toe of the levee. The top stratum at this site was approximately 11 ft thick and consisted of 5 ft of silt and silty sand underlain by 6 ft of clay. This piezometer installation was abandoned before the 1952 high-water period.

434. Two lines of piezometers perpendicular to the levee at sta 64+00 and 66+00 were installed prior to the 1952 high-water period when a riverside head of approximately 6 ft was experienced on the levee. A maximum hydrostatic head of 1.5 ft was recorded at the landside toe of the levee at the above two lines. The top stratum at the site varied from 5 to 8 ft in thickness and consisted of strata of silts and clay.

Design of relief well systems

435. Factors used in the design of relief wells for certain reaches of levees in the East Cape Girardeau levee district are given on plate 230.

436. From sta 20 to 35 an old chute of the Mississippi River parallels the levee, the near bank of which is only 200 ft from the center line of the levee. At the time of investigation (1953) this chute was quite deep and was carrying considerable flow. Rather than design the well system along this reach of levee for a source of seepage in the chute, it was decided to build a rock dike across the lower end of the chute to block low-water flow and cause the chute to silt up. This rock dike was built in the fall of 1955. The distance from the well line to the source of seepage along this reach was assumed to be 700 ft.

437. The wells in the reach of levee between sta 388+00 and 410+00 are located landside of the berm toe on the bank of a low swale which approaches the levee obliquely from an upstream direction and along a landside drainage ditch adjacent to the berm toe (see maps 168 and 169). The reach of levee between sta 436+00 and 442+00 is along the site of a

former levee crevasse which caused considerable scouring. The scour holes were filled by hydraulic means resulting in a sand foundation for the levee with no top stratum (map 169). Wells were arbitrarily spaced 100 ft apart in this reach to provide controlled seepage relief.

438. Scour areas in riverside borrow pits along four reaches of levee from sta 309+80 to 318+00, 346+00 to 348+00, 357+00 to 375+00, and 393+00 to 413+00 have recently been filled.

Design of seepage berm

439. A seepage berm was designed for the reach of levee from sta 216 to 223 mostly to protect the levee against softening of the toe because of seepage. The proposed berm is 50 ft in width and about 3 ft thick at the levee toe. Construction of the berm has been deferred because of inability to obtain right-of-way.

Piezometer installation data

440. A summary of piezometer installation data is shown on plate 231; piezometer lines perpendicular to the levee with cross sections and boring logs are shown on plates 232 and 233.

Well installation data

441. Sixty-six relief wells have been installed in this levee district (plate 214). The remaining wells have not been installed because of the lack of right-of-way.

442. Installation data for relief wells in this district were received just prior to completing the report. Although the installation and pumping test data are shown, the permeability of the pervious substratum was not computed from the pumping tests. The permeability of the substratum sand used for computing the well flows shown on plate 230 was 1400 x 10^{-4} cm per sec, the average of the coefficients of permeability for the other levee districts for which pumping test data were available prior to 1 April 1955.

North Alexander Levee District

443. The North Alexander levee district is located in Alexander County, Ill., approximately 5 miles east of Cape Girardeau, Mo. The

district is bounded on the southeast by the valley wall and Clear Creek, on the west by the East Cape Girardeau levee district, and on the north by the Clear Creek levee district. The levee in this district is shown on maps 171-175.

444. The district is protected by a levee which starts at sta 0+00 where it joins the lower end of the East Cape Girardeau district levee and continues up the right bank of Clear Creek to a road at sta 272+26, the boundary between North Alexander levee district and Clear Creek levee district. The original boundary between North Alexander and Clear Creek levee districts was at sta 275+85, and it was considered to be at that location during the underseepage investigation. However, a more recent practice of the St. Louis District has been to consider the boundary at sta 272+26 and maintenance and programming have been set up on this basis. Map 175 shows the end of North Alexander levee district at sta 272+26; however, all other plates were prepared earlier and show the end of the district at the original boundary at sta 275+85. The entire levee in this district is classed as flank levee. The levee varies in height from 18 to 25 ft and the distance from the levee to Clear Creek is from 100 to 400 ft. There are no creeks or rivers within the district.

Geology and soil conditions

445. Major channel-filling deposits appear to exist along the east valley wall. These channel-filling deposits are practically continuous from sta 0 to 215, and it appears that they may have been formed in a downstream sequence by the rather slow downstream migration of the Mississippi River.

446. The alluvium consists of a top stratum of nearly continuous clay underlain by sand strata as can be seen from the soil profiles, plates 235 and 236. The top stratum appears to have adequate thickness to preclude the necessity for underseepage control measures. In view of the favorable soil conditions encountered in this district from the underseepage standpoint, only a few borings were made to depths sufficient to determine the character of the underlying sand strata. The only depths to rock obtained in the district were el 255 and 265 ft msl at sta 76 and 83+80, respectively.

447. Cross sections and logs of borings made to delineate certain features pertinent to the study are shown on plate 237.

History of seepage

448. No severe seepage has been reported in the district. No piezometric data are available.

449. Elevations of the water surface in ditches landward of two drainage culverts observed during the 1951 high water are given below:

Levee Sta	Water Surface, msl Riverside	Landside	Net Head in ft
2+54	343.2	330.6	12.6
71+59	342.8	330.0	12.8

Seepage control measures

450. No wells or seepage berms are proposed for the district.

Piezometer installation data

451. A summary of piezometer installation data is shown on plate 238. Piezometer P-5 has not been installed and with the change in the district boundary, as discussed above, it will be within the Clear Creek levee district.

Miller Pond Levee District

452. The Miller Pond levee district is located in Union County, Ill. The district is bounded on the east by the valley wall, on the west by the Preston levee district, and on the south by the Clear Creek levee district. The levee is shown on maps 182-184.

453. The district is protected by a flank levee which starts at sta 812+85, the end of the Clear Creek back levee, and extends along the right bank of Clear Creek to the terminus of the levee at sta 963+62. The levee ranges in height from 10 to 18 ft, and the distance from the levee to Clear Creek is about 100 ft.

454. Although the levee height ranges from 10 to 18 ft, the design head on the levee was very low because it was taken as the elevation of the intersection of the river-front levee and the flank levee in the East Cape Girardeau levee district (see fig. 1). All of the flank levee in East Cape Girardeau, North Alexander, Clear Creek, and Miller Pond levee

districts have the same levee grade for seepage design (el 350.0 ft msl).

455. There are no creeks or rivers within the levee district.

Geology and soil conditions

456. The predominating surface geological factor in this levee district is the relatively thick deposit of clay along the valley wall. Soil conditions along the toe of the levee in the Miller Pond levee district are shown on plates 245 and 246.

457. Because the design head on the levee is so low, no new borings were made to determine the thickness of the top stratum. Available borings indicated silt and clays with a minimum thickness of about 15 ft (see soil profiles, plates 245 and 246). No depths to rock were determined.

History of seepage

458. No seepage or impounded water behind the levee was reported during the 1951 high-water period. No tailwater elevations from high-water periods or piezometric data are available.

Seepage control measures

459. No wells, seepage berms, or piezometers were considered necessary for the district.

PART VIII: SPECIAL PUMPING TESTS

460. Pumping tests were made on three wells in the Harrisonville and Ft. Chartres levee districts, with screens almost fully penetrating the pervious substratum, to determine the in-situ permeability of various sand strata and how deep the screens for relief wells should be installed to obtain an "effective" penetration of 50 per cent, the penetration assumed in design of the well systems. Two wells were later tested in the East St. Louis levee district for the purpose of determining the permeability of the foundation so that more accurate estimates of well flows could be made to determine the required capacities of pumping plants.

461. Three wells (H-151A, H-185, and FC-105) in the Harrisonville and Ft. Chartres levee districts were installed so as to penetrate the pervious substratum as completely as possible, and three wells (H-152, H-184, and FC-104) were installed adjacent to the above wells so as to penetrate only approximately 50 per cent by depth of the pervious strata. Pumping tests were made on both the partially and fully penetrating wells for purposes of comparison. (All of the test wells were installed at locations planned as part of the well systems in these districts.)

462. The two wells in the East St. Louis levee district (wells ES-39 and ES-64) were installed so that they penetrated approximately 100 per cent of the pervious strata.

463. The horizontal in-situ permeabilities of the various sand strata encountered at these sites were determined by measuring the flow in the well screens, by means of a sensitive well-flow meter[9], at the boundary of each sand stratum as previously delineated by a boring at each site. The over-all horizontal permeabilities of the sand aquifer and of the individual sand strata were computed from the formula for artesian radial flow to a single well[11].

464. As a part of the studies in the Harrisonville and Ft. Chartres levee districts, mechanical analyses and laboratory permeability tests were made on samples from different sand strata obtained by means of auger, bailer, split-spoon, and 3-in. Shelby tube samplers, and on samples

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obtained during drilling of the wells. An attempt was also made to correlate the coefficients of permeability as determined in the laboratory and in the field with the effective grain size of the various strata. The tests included the determination of the specific yield (flow per foot of drawdown in the well) of each well, drawdown in each well versus time of pumping at a constant rate, drawdown curves to each well at various rates of pumping, flow versus drawdown, and the total hydraulic head loss through the filter, screen, and in the well.

465. The following paragraphs describe the methods used for taking the samples and installing the wells, the results of the laboratory tests, procedures for making the field pumping tests, and the results of the pumping tests, and present a comparison of permeabilities as obtained in the laboratory and in the field.

Test Wells, Piezometers, and Borings

466. The test wells were similar to the other wells installed in the St. Louis District, and were installed by the methods discussed in part IX. Well installation data showing the depths of the wells and the per cent penetration are shown in table 8 below. It was not possible with the equipment used to install wells H-151A, H-185, and FC-105 to the full depth of the sand aquifer because of the presence of large cobbles near the bottom of the alluvial sands.

Table 8

Depths of Test Wells and Per Cent Penetration

Well Number	Depth of Well ft	Length of Well Screen, ft	Principal Sand Aquifer	
			Thickness, ft	Penetration by Screen in %
H-151A	100	66	95	70
H-152	68	63 (44)*	95	46*
H-185	97	70	85	82
H-184	78	43	85	50
FC-105	120	86	95	90
FC-104	66	35	95	37
ES-39	63	39	43	91
ES-64	70	46	51	90

* Length of well screen and penetration in principal sand aquifer.

467. Before installation of any of the wells, an exploratory boring was made at or immediately adjacent to each test well to obtain samples for laboratory tests and to determine the stratification of the sand aquifer. These borings were advanced with a fishtail bit by use of drilling mud, and samples were obtained by means of either split-spoon or thin-walled Shelby tube samplers. Logs of these borings are shown on plates 247 through 250. As the holes for the wells were advanced, samples also were taken from the drilling effluent.

468. Piezometers to measure the drawdown curve to the wells were installed in the upper part of the sand aquifer on ranges parallel to the river. Some special piezometers were installed at various depths at the outer periphery of the gravel filter around wells H-151A, H-185, and FC-105 to measure the loss in head through the filter and screen and in the well (see plates 247 through 249).

469. Plans of the test wells and piezometers for (H) wells 151A and 152, 184 and 185, and for (FC) wells 104 and 105 are shown on plates 254, 256, and 258, respectively.

Laboratory Tests

470. Mechanical analyses were made of samples taken by means of the split-spoon and Shelby tube samplers and on samples collected from the effluent of the reverse circulation well-drilling operations. Grain-size curves obtained from these tests are shown on plate 251. The reference numbers shown on the grain-size curves correspond to the reference numbers on plate 252; the effective grain size D_{10} of the samples tested is also shown on this plate.

471. The coefficient of permeability was determined in the laboratory on a number of remolded samples obtained by the various methods previously described. All of the permeability tests were made in a constant-head permeameter. Samples obtained in a mudded hole by means of a split-spoon sampler were thoroughly washed to remove any traces of drilling mud before testing. Care was taken to insure that no natural fines were lost during the washing process. The coefficients of

permeability as determined in the laboratory were adjusted to the estimated natural void ratio for each sample, as discussed in part V. The results of these tests are plotted on plates 247 through 249 at the corresponding elevation at which the sample was taken. A plot of permeability k_n, as determined in the laboratory, vs D_{10} is shown on plate 251.

Field Pumping Test Procedures

472. Immediately prior to any of the pumping tests, the ground-water table in the vicinity of each well was determined by reading adjacent piezometers and wells installed on a line out from the test well and perpendicular to the line of seepage flow from the river. The water-surface elevation of the river during the day the test was run was also determined. In general, the river stage was quite stable except during the test on well FC-105 when the river was falling at a rate of 1 ft per a day, which resulted in a change in the ground-water table of approximately 0.1 ft per day. The measured drawdowns in the well and piezometers were adjusted to take into account this change in the ground-water table.

473. The pumping tests consisted of pumping each well at three different rates of flow and measuring the drawdown in the well and in adjacent piezometers and wells. An accurate record was kept of the flow and drawdown in the well with time (plates 253 through 260). In pumping the wells an effort was made to set the pump at a constant rate of discharge and then to let the drawdown to the well become stabilized. After the drawdown had become stabilized the piezometers and adjacent wells were read. The flow in the well screen was also measured at specified intervals or at changes in sand strata by means of a well-flow meter. The total head loss through the filter, well screen, and inside the well was measured for wells H-151A, H-185, and FC-105 by the special piezometers previously mentioned.

Pumping Test Data

474. The flow to the test wells was essentially artesian because

of the existence of upper and lower impervious strata bounding the principal sand aquifer and the fact that the water in the well was not drawn down below the top of the main sand stratum. As would be expected for artesian flow, drawdown in the test wells stabilized quite rapidly at a constant rate of pumping (see plates 253 through 260); approximately 80 per cent stabilization was achieved in 15 to 30 minutes with practically full stabilization in 2 hours.

475. Well flow and drawdown within the test wells, and in the adjacent piezometers, for three constant rates of pumping as well as the elevation of the water table immediately prior to the start of pumping are shown on plates 253 through 260.

476. The yields for the test wells also are plotted on plates 253 through 260 for various drawdowns and are shown in table 9. The drawdown well-flow curves shown for the test wells on the above-mentioned plates are straight lines for well flows up to approximately 400 gpm, thus indicating that the flow to the test wells was essentially artesian. The reduced increase in well flow for flows in excess of 400 gpm may be attributed to increased screen entrance losses and hydraulic head losses within the well screen and riser pipe resulting from the high flows.

477. The head loss through the filter and screen as measured by piezometers located at the outer periphery of the filter gravel for wells H-151A, H-185, and FC-105 is plotted on plates 253, 255, and 257. The flow through the screen at the location of the piezometer tips was obtained from well-flow meter readings made in the well screen immediately above and below the piezometer tips. The head loss through the filter and screen as plotted was taken to be equal to the piezometer readings minus the elevation of the water in the well, minus the computed hydraulic head losses in the well screen and riser pipe from the elevation of the piezometer tip to the bottom of the suction pipe in the well. As may be noted from the above plates, the head loss through the filter and well screen was quite small and amounted to only about 0.10 to 0.25 ft for a flow through the well screen of 10 gpm per foot of screen, about the maximum anticipated natural flow through well screens. These observed head losses compare quite favorably with head losses measured for a

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similar well and filter installed in a specially constructed well testing tank set up in the laboratory[2].

478. The lengths and depths of the well screens for the various test wells in relation to the depth and stratification of the sand aquifer in which the wells were installed are shown adjacent to the well log on plates 247 through 250. The locations at which well-flow meter readings were taken and their relation to the stratification of the sand aquifer are also shown on these plates.

479. The coefficients of permeability of individual sand strata as obtained from the pumping tests and well-meter readings and computed by the equation for artesian radial flow to a single well are plotted as bar graphs on plates 247 through 250. The average permeability of the sand aquifer as determined from the pumping tests and the coefficient of permeability as determined from laboratory tests on samples taken from the various sand strata as indicated are also plotted on these graphs. No laboratory permeability tests were made on samples from the borings adjacent to wells ES-39 and ES-64 in East St. Louis levee district.

480. Also shown on plates 247 through 250 are graphs of the flow in the wells at various depths in per cent of total flow, flow through well screen per foot of drawdown, and flow through well screen in per cent of average inflow.

481. A summary of the data obtained from the pumping tests on all wells is shown below. The specific yields have been adjusted so as to eliminate all losses through the filter and screen and within the well.

Table 9

Summary of Data from Special Pumping Tests

Well	% Penetration of Pervious Stratum	Specific Yield in gpm	Yield from Shallower Well Compared to Deeper Well in %	Screen Penetration on a Depth Basis Required for 50% "Effective" Penetration of Pervious Aquifer in %	Average Permeability of Entire Sand Strata in 10^{-4} cm/sec
H-151A	70	221	--	46	1750
H-152	46	122	55	--	----
H-185	82	166	--	61	1480
H-184	50	44	26	--	----
FC-105	90	378	--	68	3000
FC-104	37	126	33	--	----
ES-39	91	45	--	56	800
ES-64	90	75	--	59	950

Discussion of Test Data

482. The drawdown curves for fully penetrating wells, when plotted on a semilog graph, are relatively straight lines; the drawdown curves for the partially penetrating wells are also relatively straight beyond 200 ft from the well. Curvature between this point and the well is a result of the well not fully penetrating the aquifer. The intersection of the drawdown curves with the outer periphery of the filter or well screen above the elevation of water in the well reflects head losses into and within the well. The radius of influence for a given well was approximately the same for the different drawdowns tested (see plates 253 through 260). (It is pointed out that the tips of all of the piezometers except those adjacent to the filter gravel (wells H-151A, H-185, and FC-105) were at the upper part of the main sand aquifer.)

483. The data shown in table 9 indicate that in order for wells in the alluvial valley from Alton to Gale to have an effective penetration of 50 per cent, the screen must penetrate on a depth basis approximately 60 per cent of the principal pervious aquifer.

484. The yield from the shallower wells as compared to the deeper wells was considerably less than what would be indicated from fig. 12. This was expected because fig. 12 is based on a homogeneous isotropic sand stratum whereas the sand stratum at the test wells was more pervious with depth and probably had a higher k_H than k_v. This indicates that the values of k_f computed from the routine pumping tests on partially penetrating relief wells, on the basis of fig. 12, are probably somewhat too low.

485. Although there was considerable variation in the permeability (as measured by the pumping tests) within a given sand stratum of the same classification, a definite general relationship appears to exist between D_{10} and k_H as shown on plate 251. As would be expected in the alluvial valley of the Mississippi River, the permeability generally increased with depth at the test wells.

486. The permeability of the principal seepage-carrying stratum at the test wells ranged from 800 to 3000 x 10^{-4} cm per sec. The

permeabilities of individual sand strata varied from 250 to 6000 x 10^{-4} cm per sec.

487. The coefficients of permeability as determined in the laboratory on remolded samples are plotted at appropriate elevations on the bar graphs of plates 247 through 250. In general, little agreement was found between the permeabilities as determined in the laboratory on remolded samples and as obtained from the field pumping tests. Of course, there is no reason why the permeabilities should agree, particularly where the aquifer is stratified and lenses of coarse sand and fine gravel exist. Generally, the field permeabilities for any given stratum exceeded the permeability as normally determined in the laboratory by two to four times.

488. Plots of effective grain size versus coefficients of permeability as determined in the laboratory on remolded samples, and from pumping tests on approximately 100 per cent penetrating well screens, for wells H-151A, H-185, and FC-105 are shown on plate 251. (The D_{10} used in the plot for the pumping test is the average D_{10} for the specific stratum tested. The D_{10}'s shown were obtained from mechanical analyses on samples obtained by means of a 2-in. split-spoon sampler, and in sand strata with considerable gravel do not represent the true D_{10} of the sand in situ as discussed in paragraph 51.) A great deal of scatter of the points is apparent for both sets of curves. No attempt was made in this study to relate the permeability as measured in the laboratory or field to the uniformity or shape of the grain-size curve. Some of the widely graded grain-size curves plotted on plate 251 really represent a mixture of alternating strata of fine or medium sand and coarse sand or fine gravel, and such deposits may have quite high horizontal coefficients of permeability in situ.

PART IX: INSTALLATION AND CONSTRUCTION OF SEEPAGE CONTROL MEASURES

Relief Wells

Drilling

489. The relief wells were installed in a hole made by the reverse circulation method. When large cobbles or other obstructions prevented extension of the hole to design depth, the well depth was adjusted or the hole was abandoned and another well installed nearby. A procedure devised for determining the respacing of wells that could not be installed to design depths is discussed in paragraph 522. The holes for the relief wells were drilled to a depth of at least 4 ft below the bottom of the well screen.

490. Reverse circulation method of installation. The reverse circulation method of drilling holes for wells in sand is basically a method of drilling by suction in which the material from the hole is removed by a suction pipe. The walls of the hole are supported by seepage forces acting against a thin film of fine-grained soil on the walls, created by maintaining a head of water in the hole several feet above the ground-water table. The film of fine-grained soil is deposited on the walls of the hole by the drilling water as the water and soil from the suction pipe are circulated through a sump pit or box in which the sand settles out and from which the water containing the fine-grained particles flows back into the hole.

491. Reverse circulation units used on this project were equipped with a large-capacity centrifugal pump, 6-in.-diameter drill pipe, and one of the four drill bits shown on plate 261. The well drilling rigs on the first well contracts were not equipped with rock traps and as the maximum size of stone the pumps could handle was about 4 in., the drill bit openings were kept to 3-3/4 in. As a result, some of the wells could not be installed to design depths where excessive amounts of cobbles larger than 3-3/4 in. were encountered. On some of the later contracts the drilling equipment was provided with rock traps and the openings in

the drill bits were increased to 5-1/2 in. It was generally possible with this equipment to install the wells to design depths.

492. A large sump was used for storing drilling water in the vicinity of each hole. This sump was connected to the hole for the well by means of a ditch of suitable size (plate 261). At the start of drilling operations the drill bit was lowered to the ground surface at the well location and the pump was started. Water from the bottom of the return ditch at the well location was drawn up through the drill pipe and pumped into the sump through a discharge line. The water then flowed back to the hole through the connecting ditch. After circulation of the drilling water had been started, the drill bit was rotated and lowered as pumping continued. The soil cut loose by the bit was then carried up through the drill pipe and pump and was discharged into the sump where the coarse soil particles settled out. The drilling water and finer soil particles continued to circulate as the hole was advanced. As the walls of the hole were stabilized by seepage forces resulting from excess hydrostatic pressure, such pressure had to be maintained from the time drilling started until the last of the filter gravel and sand backfill was placed. At some wells it was necessary to use casing through silty or sandy soils encountered near the surface to prevent enlargement of the hole by caving. The drilling water or mud was that naturally obtained by drilling through the upper fine-grained soils. The use of drilling mud containing bentonite was not permitted.

493. Sampling. During the drilling of each hole, samples of the foundation soil were obtained at 2-ft intervals by catching and decanting samples of the effluent from the drill rig discharge line in a large bucket or other suitable container. The purpose of this sampling was to determine the depth at which the screen section of the well could be started and also to determine the existence of strata of silt, silty sands, or very fine sands through which unslotted sections of pipe were generally used rather than slotted screen.

Description of wells

494. The relief wells consisted of a slotted wooden pipe screen section, wooden riser pipe, gravel filter, sand backfill from the top

of the gravel filter to an elevation 10 ft below the well outlet, and concrete backfill from the top of the sand backfill to the finished ground surface. Photographs and drawings of the component parts are shown on plates 261 through 265. The slotted wooden well screen and riser pipe (8-in. ID) usually came in lengths of 4, 8, and 16 ft. The screen was perforated with slots 3/16 in. ± 1/32 in. wide; the total slot area of the 8-in. screen was about 30 sq in. per linear ft of screen. The riser pipe and screen were constructed of machine-banded wood staves of treated Douglas fir. The staves were about 1-1/8 in. thick and were milled from clear, seasoned lumber. The wood pipe was pressure-treated with creosote with a minimum retention of 8 lb per cu ft. The pipe was banded with heavily galvanized 6-gage winding wire with 3-in. spacing between wire turns. Wooden plugs were used for closing the bottom of the well screens.

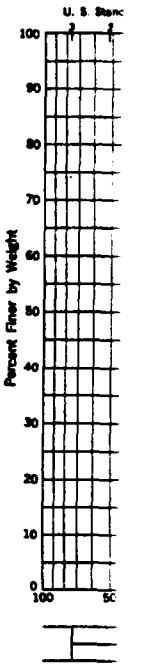
495. The length of well screen (set) depended primarily upon the depth of the pervious aquifer. (The principal seepage-carrying aquifer was defined as the stratum of medium to fine, or coarser sand lying above the valley floor.) The top of screen was set about 4 ft (±2 ft) below the top of medium to fine sand (D_{50} > No. 50 sieve). Screen was not placed in silty or very fine sand (D_{60} < No. 70 sieve).

496. The design elevation of the top and length of well screen were usually estimated from boring data; however, the top of well screen and its length were checked in the field from data on the soils penetrated as the hole for the well was drilled. The following criteria were adopted where it became necessary to modify the lengths of riser pipe, unslotted pipe, and screen sections in the field in order not to set the well screen in strata of very fine sand or finer soil.

a. Minimum length of riser pipe, 16 ft.

b. Maximum depth of well, 100 ft.

c. Screen was not to be set in strata of clay, silt, silty sand, or very fine sand. (In setting unslotted sections of pipe to blank out such strata, an attempt was made to overlap the pervious strata with the unslotted section by 1 or 2 ft.) Design screen, extra screen, or blank pipe were set in materials as indicated in fig. 32.



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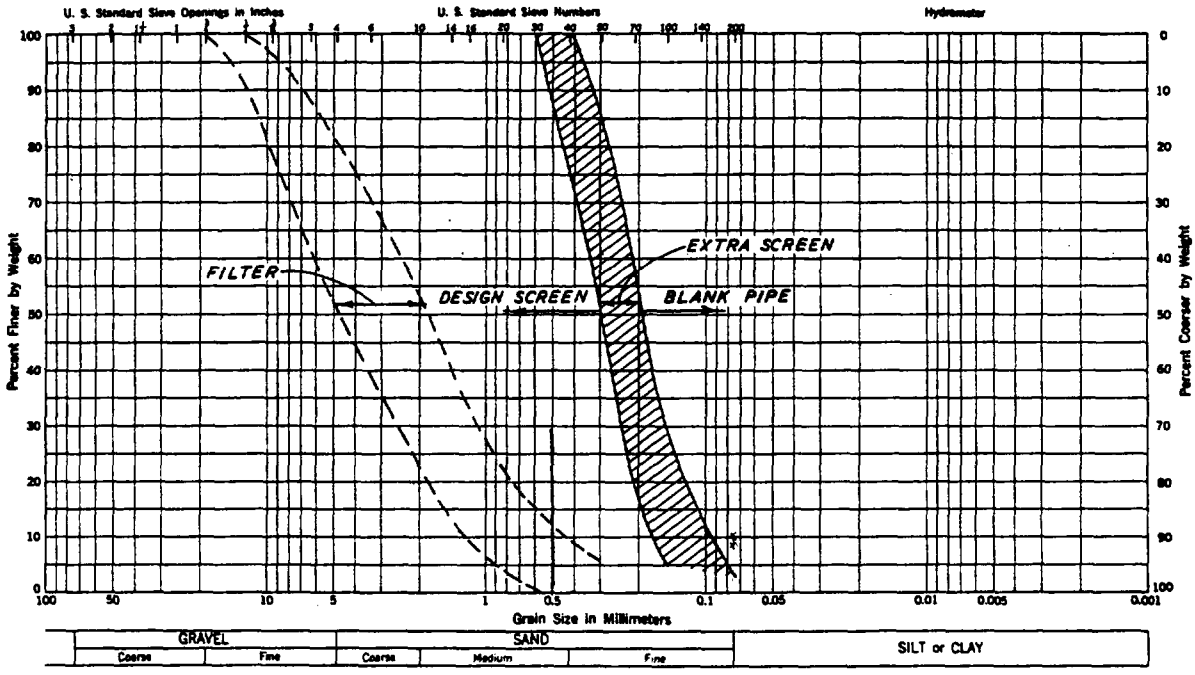


FIG. 32. GRADATION CRITERIA USED IN SETTING WELL SCREEN

d. Riser pipe was to extend 2 to 6 ft below topsoil finer than fine sand.

e. Where lesser footages of unslotted pipe were installed than given in the design, the well was still to be set to the design depth.

f. Where greater footages of unslotted pipe were required than given in design, the well was to be deepened in accordance with the following tabulation.

Amount of Unslotted Pipe Additional to That Shown in Design, ft	Amount Well Was To Be Deepened, ft
4	4
8	8
12	8
16	12
20	12
24	16
28	16

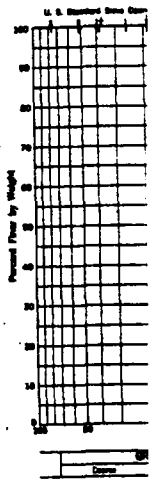
Installation

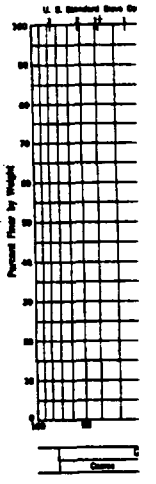
497. Assembly and installation. The well screen and riser pipe were assembled, or partly assembled, and the bottom of the well screen plugged before the hole for the well was completed. Each joint of pipe and the plug in the bottom of the screen were fastened securely with wood

screws. Guides that would permit extension of a tremie to the bottom of the hole were attached to the assembled riser pipe and screen in such number and of a type that would center the assembly in the well and hold it securely in position while the filter gravel was placed, or the riser and screen assembly were centered by using a centering device mounted near the end of the tremie pipe (plate 261). The assembled pipe and screen were placed in the hole carefully to avoid jarring and to insure that the assembly was neither damaged nor displaced. The wells were installed reasonably straight and plumb, with a variation of 6 in. from a plumb line at the top of the well permitted in the alignment of riser and screen in the total depth of well. However, adequate clearance for installation of the pumping equipment was required for testing the wells. A small allowance was made in setting the top of the well pipe to take care of settlement which usually occurred during surging and pumping, in order to have the top of the well pipe at design elevation when completed.

498. No filter gravel was placed before the well screen and riser were installed, as the amount required below the screen could not be predetermined nor placed accurately enough to insure that the top of the riser pipe would be at the correct elevation. The filter gravel was placed by lowering a tremie to the bottom of the hole and then filling it with filter gravel (plate 262). The hole for each well was overdrilled by at least 4 ft to provide space at the bottom for filter gravel which might have become segregated when the tremie pipe was first filled. The tremie was raised slowly to allow the filter gravel to run out of the bottom as additional filter gravel was fed into the top. The tremie pipe was kept filled with filter gravel above the water surface and was handled so as to prevent segregation of the filter as it was placed. Filter gravel was placed to an elevation at least 5 ft above the top of well screen to allow for settlement during development of the well.

499. Material for the filter around the screen and riser was a washed sand-gravel free from any adherent coating, organic matter, or elongated particles in quantities considered detrimental. The filter met the gradation requirements shown in fig. 33 and was not skip-graded.





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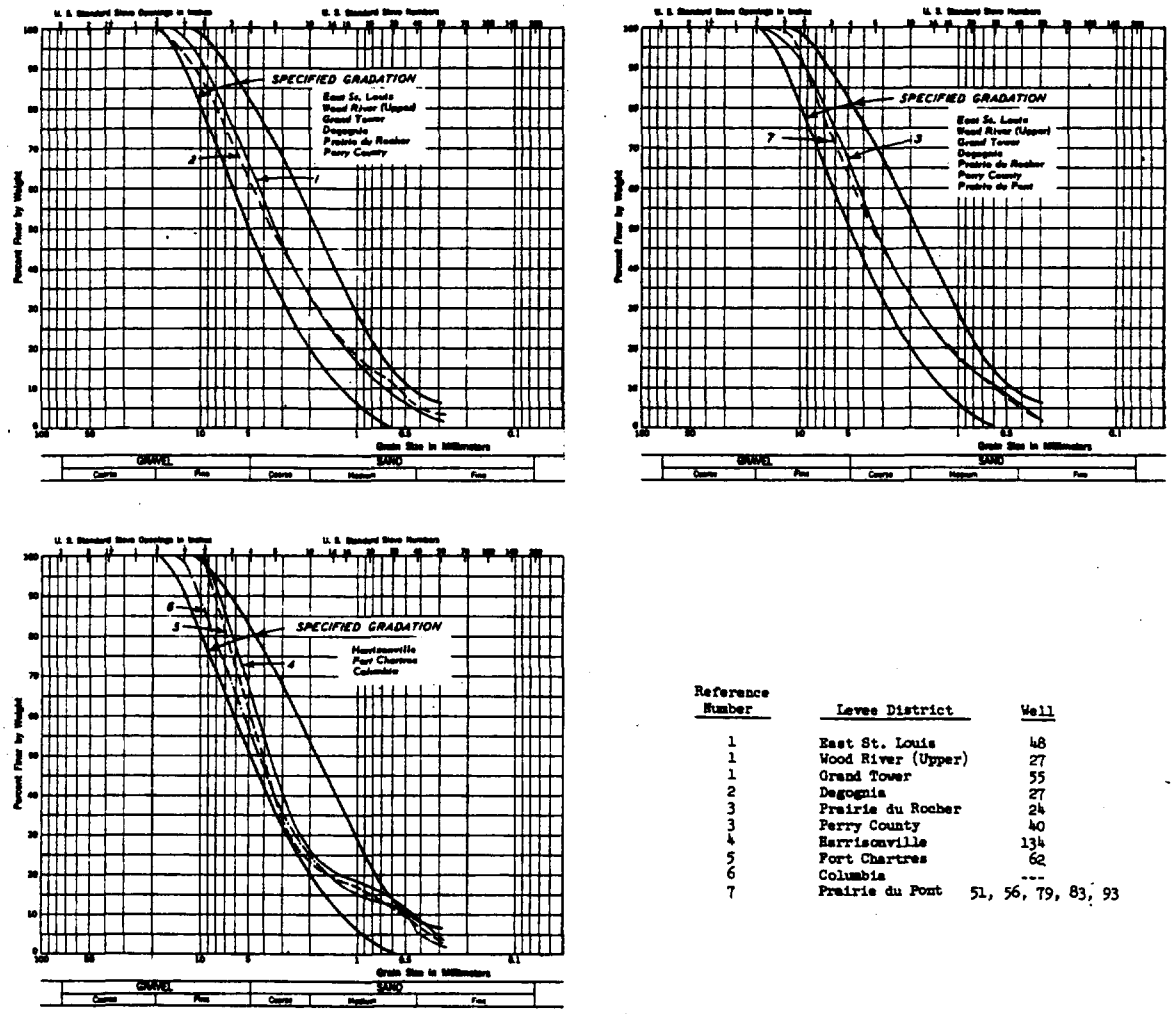

Reference Number	Levee District	Well
1	East St. Louis	48
1	Wood River (Upper)	27
1	Grand Tower	55
2	Degognia	27
3	Prairie du Rocher	24
3	Perry County	40
4	Harrisonville	134
5	Fort Chartres	62
6	Columbia	---
7	Prairie du Pont	51, 56, 79, 83, 93

FIG. 33. TYPICAL GRADATION CURVES FOR FILTER MATERIAL USED IN INSTALLATION OF RELIEF WELLS

Typical gradation curves of filter material used are shown in fig. 33. The filter material was usually obtained by mixing two or three different gradations of commercially prepared sands and fine gravel. These materials usually were mixed in concrete mixers and kept moist to prevent segregation during handling and placement operations.

500. Surging and pumping. Materials which might have entered the well during placement of the gravel filter were carefully removed by pumping or bailing with a piston-type bailer (plate 262). In some instances the well was surged slightly before attempting to remove the filter gravel from the well, in order to minimize the possibility of trapping the clean-out bailer as a result of filter gravel entering the well above the bailer as it was lowered. After this, the well was surged

and pumped to remove drilling mud or other fines from the filter gravel. Surging and pumping were started within 4 hours after the filter was placed and were continued until the amount of material pulled through the screen between surging cycles was less than 0.2 ft in the well or, in any event, for not less than one and one-half hours. Surging and pumping were performed both alternately and simultaneously.

501. The wells generally were surged by raising and lowering a block made of a heavy rubber disk between two steel disks mounted on a steel rod. The rubber disk had a diameter approximately 1 in. less than the inside diameter of the well and the steel disks were 1 in. less in diameter than the rubber disk. In some cases a surging block was used that had two disks approximately 5 ft apart on the steel rod. The surge block was raised and lowered at a rate of approximately 5 ft per sec. A cycle of surging usually consisted of about 20 round trips of the surge block for the full length of the well screen. Surging or pumping was done with only sufficient force to remove or loosen all drilling mud and/or fines in the filter gravel and well so that they could be removed by pumping. Surging or pumping of such violence as to endanger or damage the well was not permitted. At times some sand had to be removed by pumping or by a piston-type bailer upon completion of this operation.

502. Within three hours after completion of surging, most wells were pumped to achieve a drawdown of 5 ft in the well or a flow of 200 gpm. If the well produced sand in excess of approximately 2 pints per hour (as determined by sounding and from collection of well flow in a 10-gallon container) the well was resurged and repumped. Wells continuing to produce excessive amounts of sand were abandoned and properly plugged. After a well was surged and cleaned it was kept sealed until provided with a permanent well guard and check valve, to prevent inflow of surface water.

503. Backfilling. The annular space above the gravel filter was backfilled after the well had been developed by surging and pumping. Fine or medium sand was placed to an elevation 10 ft below the well outlet and concrete was used as fill to the finished ground surface. When placed in water, the sand backfill was rodded to insure proper compaction;

when placed above water, it was compacted by approved methods as placement proceeded. When placed in water, concrete was placed by tremie and was rodded to insure compaction and absence of voids.

504. Excess excavated or waste materials were disposed of so as to leave the site in an orderly and neat condition. All pits, holes, ditches, and ruts resulting from well installation operation were filled to original grade with topsoil.

505. Top of well. The concrete around the well was usually finished flush with the normal elevation of the ground. At local high areas in topography the top of the concrete was sometimes set 0.2 to 1.0 ft below natural ground elevation so that the well outlet could be lowered, thereby improving the efficiency of the well.

506. In the Harrisonville, Ft. Chartres, Columbia, and Prairie du Rocher levee districts, most of the wells were installed with the top of the riser pipe set about 4 in. above the top of the concrete around the well. However, the top of the riser pipe on wells in all of the other levee districts was installed flush with the concrete and a cast iron tenon placed on the top of the wood riser with projections embedded in the concrete (plates 262 and 265). The ground immediately around the well was left flush with the top of the concrete and was graded to drain to any adjacent lower ground. Where wells were located along inlet channels to gravity drains, the top of the well usually was provided with a tee outlet with a horizontal discharge pipe. The pipe was set at an elevation such that it would be submerged at high water but still high enough above the channel that silt would not collect in the discharge pipe.

Pumping tests

507. All wells were subjected to a pumping test upon their completion. Equipment for making the pumping tests included a pump with a manually operated speed control of adequate capacity; a suction hose or pipe of sufficient length to extend from the pump to a point in the well at least 5 ft below the maximum drawdown required for the test; a discharge hose or pipe of sufficient length to remove the discharge water a satisfactory distance from the well; a sounding device for measuring the depth

to water in the well; and a flow meter for measuring the discharge from the well.

508. The equipment was set up and the pumping test performed in accordance with the following procedure. The pump was set near the well and the discharge hose or pipe with flow meter attached was extended out from the well a sufficient distance to prevent flow back into the well (plate 262).

509. The distance from the top of the well to the water surface was measured to the nearest 0.01 ft after the test equipment was set up. The pump was operated so as to create a drawdown in the well of 8 ft or a flow of 500 gpm, whichever occurred first. The flow from the well was then recorded together with the time and measured depth (nearest 0.01 ft) to the water surface in the well. For the remainder of the pumping period the flow from the well was maintained at a relatively constant rate, varying the pump speed if necessary. The flow from the well and depth to water surface in the well were read and recorded at 15-min intervals for the remainder of the test period. If radical variations in pumping rate or interruptions in pumping occurred, the pumping test was started over.

510. The pumping tests were usually made shortly after a well was completed. No test-pumping of a well was permitted concurrently with drilling, surging, or pumping of any other well within 400 ft of the well being tested. Care was taken not to damage either the top of the well or metal well guard. In general, the wells were pumped continuously for a period of two hours. If the water table at time of well installation was below the top of the well screen, the pumping test was usually deferred until the water table rose to an elevation above the well screen. The inflow of sand during a pumping test was measured by discharging the flow into a baffled stilling tank (plates 262 and 263) to cause sand in the water to settle out. The inflow of sand into a well during a pumping test was determined in the following manner:

a. The depth to top of sand in the well was determined to the nearest 0.01 ft with a flat-bottomed sounding device and steel tape before starting the pumping test.

b. At 15-minute intervals during pumping, the exact depth (nearest 0.01 ft) to sand in the well and amount (nearest

0.1 pt) of sand collected in the stilling tank were determined.

c. Total inflow of sand into the well was computed for each time interval as the test progressed. The pumping test was continued until the rate of inflow of sand was less than 1 pint per hour, but no longer than 8 hours. If the rate of sand inflow was more than 2 pints per hour after 8 hours pumping, the well was either abandoned or steps were taken to repair it.

511. In the event sand or other material collected in the well as a result of the pumping test, it was removed from the well.

Recording of data

512. All installation data necessary to complete the data sheet shown in fig. 34 were recorded so as to give complete information on the installation. Any unusual conditions encountered or instances of non-compliance with specifications by the contractor were also recorded. All items in fig. 34 are self-explanatory with the exception of the following.

a. The first item in line 9, "Elev Top of Riser" is to designate the final elevation of top of riser as established after completion of the well.

b. "Elev Riser Set at" as shown in the second column under "Final Well Installation Data" is to designate the elevation at which the top of riser was set initially.

c. "Inside Bottom Elev of Well" is the elevation of the top of the plug in the bottom of the well screen and is equal to "Elev Top of Riser" minus "Inside Depth of Well."

513. All pumping test data necessary to complete the data sheet shown in fig. 35 were recorded to give complete information on the tests.

Relief well appurtenances

514. The tops of all wells were protected with suitable guards such as shown on plates 264 and 265. Care was taken so that the top of the guard was level and that all cover fastenings were in working order.

515. Each well was protected against backflow of surface water by a rubber gasket and check valve such as shown on plates 262 and 263. Accelerated full-scale laboratory tests have indicated this check valve to be very effective in preventing backflow of water into a well under adverse conditions (see appendix C). The rubber gasket shown on

PROJECT: St. Louis District, Underseepage	LEVEE DISTRICT: Prairie du Rocher
LOCATION (STA.): 419 + 90	OFFSET FROM CENTER LINE: 128 ft
WELL NO.: 45	
DRILLING METHOD: Reverse-Rotary	DATE AND TIME:— From: 17 June '53 4:30PM To: 18 June '53 5:00 AM
WELL SCREEN:— Type: Wood stave, machine bonded; Inside Diam: 8-in.; Perforations: 3/16 x 3 1/4-in. slots	
RISER:— Type: Wood stave pipe, machine bonded; Inside Diam: 8-in.	FILTER: Gravel
TYPE OF WELL GUARD: Galvanized corrugated metal pipe with wire screen	TYPE OF BACKFILL: Concrete
CONTRACTOR: Luhr Bros. Inc.	CONTRACT NO.: 4846
FOREMAN: Riechmann OPERATOR: Dees	
GROUND ELEV AT WELL: 378.2	DEPTH TO:— Fine Sand: 27.6 ft
Medium To Fine Sand: 37.3 ft	
ELEV TOP OF RISER: 378.60	ELEV TOP OF WELL GUARD: 380.0
ELEV TOP OF CONCRETE BACKFILL: 378.3	
TYPE OF CHECK VALVE: Aluminum plate	WELL EXTENSION:— Type: None; Ht: —

FINAL WELL INSTALLATION DATA

RISER PIPE LENGTH:	31.1	INSIDE DEPTH OF WELL:	77.65
"EXTRA" SCREEN LENGTH:	0.0	DEPTH TO SAND IN WELL AFTER CLEANING:	77.58
"DESIGN" SCREEN LENGTH:	39.1	ELEV TOP OF WELL SCREEN:	347.5
TOTAL SCREEN LENGTH:	39.1	ELEV TOP OF FILTER:	368.8
"BLANK" PIPE LENGTH:	7.7	INSIDE BOTTOM ELEV OF WELL:	300.95
DEPTH OF HOLE:	79.3	ELEV RISER SET AT:	378.7

SURGING DATA

METHOD: Block and bailer	DATE AND TIME: 18 June '53 6:05 AM		TOTAL SURGING TIME: 1:00 hr				
GRAVEL IN WELL BEFORE SURGING: 15.15 ft	MATERIAL SURGED IN:— SURGING CYCLE	#1: 20.65	#2: 5.85	#3: 1.75			
SURGING CYCLE	#4: 0.65	#5: —	#6: —	#7: —	#8: —	Total: 44.05 ft	
SETTLEMENT OF FILTER DURING SURGING: 1.5 ft							

PUMPING DATA

RATE OF PUMPING: 200 GPM		TOTAL PUMPING TIME: 2 hrs	
RATE OF SAND INFILTRATION:—Start: 0.5 pts/hr		End: 0.0 pts/hr	

REMARKS: Design depth of well = 84 ft. Cobbles encountered at depth of 79.3 ft. Unable to obtain design depth.

LOG OF HOLE	DEPTH	PIPE INSTALLED
	0	
	10	Riser
	20	
	30	31.1
F Sand		
MF Sand	40	Screen
M Sand	50	
		54.8
Clay	60	Blank
		62.5
FM Sand	70	Screen
M Sand		77.65
Cobbles	80	
	90	
	100	

Wayne A. Smith
INSPECTOR

FIG. 34. RELIEF WELL INSTALLATION REPORT

PROJECT: St. Lo	
LOCATION (STA.): 41	
DATE: 1 Jul	
TIME PM	EL T MIN
12:20	
12:30	1
12:45	2
1:00	4
1:15	5
1:30	7
1:45	8

DEPTH OF WELL:

TEST NO.	TIME
1	12:30
	1:00
2	1:15
	1:30
3	1:45
	2:00

DEPTH TO SAND IN WELL AFTER TEST:

REMARKS:

PIPE INSTALLED
Riser
31.1
Screen
54.8
Blank
62.5
Screen
77.65

PROJECT: St. Louis District, Underseepage | LEVEE DISTRICT: Prairie du Rocher

LOCATION (STA): 419 + 90 | ELEV TOP OF RISER: 378.60 | WELL NO.: 45

DATE: 1 July 1953 | TIME TEST STARTED: 12:20 PM | TIME TEST COMPLETED: 2:35 PM

TIME PM	ELAPSED TIME MINUTES	DEPTH TO WATER	DRAWDOWN IN FEET	FLOW IN GPM	TIME	ELAPSED TIME MINUTES	DEPTH TO WATER	DRAWDOWN IN FEET	FLOW IN GPM
12:20	0	15.35	0	0	2:00	100	21.57	6.22	500
12:30	10	21.32	5.97	505	2:15	115	21.58	6.23	500
12:45	25	21.46	6.11	500	2:30	130	21.57	6.22	500
1:00	40	21.50	6.15	500	2:35	135	15.66		
1:15	55	21.47	6.12	500					
1:30	70	21.52	6.17	495					
1:45	85	21.53	6.18	500					

SAND INFILTRATION TEST

DEPTH OF WELL: 77.65 | DEPTH TO SAND IN WELL BEFORE TEST: 77.58 | SAND IN WELL BEFORE TEST: 0.07 ft

TEST NO.	TIME	DEPTH TO SAND (Ft)	SAND IN WELL (Pts)	GAIN OR LOSS OF SAND IN WELL (Pts)	SAND PUMPED OUT OF WELL (Pts)	TOTAL INFLOW OF SAND INTO WELL (Pts)	LENGTH OF TEST (Min)	RATE OF SAND INFILTRATION (Pts/Hr)
1	12:30	77.58	1.54	+0.22	0.20	0.42	30	0.84
	1:00	77.57	1.76					
2	1:15	77.57	1.76	0.00	0.10	0.10	15	0.40
	1:30	77.57	1.76					
3	1:45	77.57	1.76	0.00	Trace	Trace	15	Trace
	2:00	77.57	1.76					

DEPTH TO SAND IN WELL AFTER TEST: 77.57 | SAND IN WELL AFTER TEST: 0.08 ft | SAND IN WELL AFTER CLEANING: 0.00 ft

REMARKS:

R. L. Showmaker
INSPECTOR

FIG. 35. RELIEF WELL PUMPING TEST REPORT

plates 262 and 263 was specially designed to prevent backflow of surface water into the well and also to serve as a gasket between the well top and plastic standpipe. The mold for this gasket to fit an 8-in. well is owned by the St. Louis District, CE. The gasket was made of neoprene rubber with a softness of 30 shore. In order for the lightweight check valve to function properly, the top of the well was made absolutely smooth before the rubber gasket was cemented to it. The gasket was attached to the wood or cast iron tenon with an adhesive cement, 3-M-No. BC-870, Minnesota Mining and Manufacturing Co.

516. Relief wells will flow somewhat more than natural seepage during periods of relatively low stages on the levees. Therefore, in agricultural areas each well was provided with a plastic standpipe to raise its discharge elevation 1 or 1.5 ft above the natural ground surface (plates 262 and 263). The flare on the standpipe was designed to prevent the check valve from catching on the top of the standpipe in the event the well flows before the standpipe is removed. These low sleeves or standpipes will prevent well flow at low heads on the levee when no pressure relief is necessary. As soon as artesian pressure develops to such extent that water begins to spill over the tops of the standpipes, it is planned to remove them and to allow the system to operate as originally designed.

517. All relief wells were provided with standpipes except those adjacent to sloughs, timbered areas, ditches or culverts, or where wells were estimated to be flooded to a depth of 2 ft or more during a flood of low height and short duration with little attendant rainfall. The height of standpipe was set so as to be equal to or less than 0.25 times the effective thickness of the top stratum, but none were installed higher than about 18 in.

518. Relief wells adjacent to ditches or intakes to drainage structures or pumping stations along the levee were provided with a check valve, standpipe for access, and a horizontal discharge pipe (tee outlet), as shown on plates 264 and 265.

519. Relief wells installed in Wood River levee district (Upper), which are subject to flooding by sanitary and industrial sewage, were

provided with the type of rubber gasket and check valve shown on plate 262, an outfall pipe equipped with an automatic flap gate, and a well top of the type shown on plate 264. These flap gates are of cast iron with bronze facing. They are double-hinged and bronze-mounted and close at a slight angle to the vertical. Bolts and washers also are of bronze or brass.

520. Completed relief well installations are shown on plate 264.

Respacing of wells

521. A procedure was devised for determining the spacing of wells where such wells could not be installed to design depths because of the presence of cobbles or other obstructions. The original spacing of the wells in any given system was computed on the basis of an effective penetration of 50 per cent of the pervious aquifer by the well screen. If this amount of penetration is not achieved, then it is necessary to decrease the spacing of the wells if the same pressure reduction is to be achieved. In order to compute the revised spacing, a graph was devised showing the relationship between head midway between wells, per cent penetration, and spacing of the well system. The relationship between head midway between wells (in per cent of net head) and the per cent well penetration were determined for given well spacings from a formula recently developed by Mr. P. T. Bennett, Missouri River Division, CE, Omaha, Nebr. This formula is based on electric analogy model data obtained by the Vicksburg District, CE, during 1938 to 1942. The formula is empirical, but it is believed that the relative effect of penetration on the head for given well spacings is reasonably correct. To simplify calculations it was assumed that the top stratum landward of the levee was impervious, and it was also assumed that the distance to the effective source of seepage was 800 ft, which is typical of that assumed in design of wells along the river-front levees.

522. Where it was necessary to adjust the spacing of wells because the required penetration could not be obtained, the following general procedure was used. After installing a well short of design depth, another well was installed at the next design location. If this well could not be installed to the required depth, and if the percentage of

design screen installed was within 10 per cent of that installed in the first well, the required new spacing of remaining wells was determined as indicated in the following table using the average of the two percentages of design screen installed. If the latter well had a less percentage of design screen installed than the first well, the spacing of the remaining wells was determined from the lesser penetration obtained.

Per Cent of Design Screen Installed in Two Consecutive Wells	Number of Wells Installed	Number of Original Design Wells
85	No change	
75 to 85	5	4
68 to 75	4	3
62 to 68	3	2
55 to 62	2	1
45 to 55	4	2
40 to 45	3	2

523. Care was exercised in relocating wells that could not be installed to design depths so as to best fit geological and topographic features. However, completely effective utilization of the above procedure was never realized in the field.

524. Relocation of wells would not have been necessary in most cases if bits with larger openings and rock traps had been used. After most of the wells were installed a test well was installed which demonstrated that it was feasible to handle cobbles by use of a Zublin bit with a 5-1/2-in. opening in the bit and a rock trap in the pump line. Subsequent relief well contracts provided for use of a bit which would pass 5-1/2-in. cobbles.

Cost of relief wells and appurtenances

525. Relief wells installed in the St. Louis District in the period 1953-1955 generally cost from $1200 to $1500 or about $16 per foot of well installed. This cost includes all materials and placement costs. Unit prices for low bidders for the various contracts are given in appendix D. The well appurtenances cost approximately:

Metal well guard	$18.00
Aluminum check valve	$ 2.80

Rubber gasket	$ 0.90
Plastic well extension:	
11-in. height	$ 3.25
17-in. height	$ 4.75
Cast iron tenon	$ 3.50

Piezometers

Boring and sampling

526. Piezometers were installed in 6-in.-minimum-diameter holes drilled to the required depth. The hole had to remain open, or was held open with casing if necessary, to allow placement of a sand filter around the piezometer screen and to allow proper backfilling around the riser pipe. The boring for piezometers was advanced with an auger or by a bailer and casing. The use of drilling mud was not permitted.

527. Samples were taken at every change in type of soil and at intervals not to exceed 3 ft, in order to obtain representative samples of the various strata encountered.

Installation

528. Each piezometer consists of a brass or bronze screen set in filter sand, a riser pipe generally extending 3 ft above ground surface, and a cylinder of concrete 6 in. in diameter around the riser pipe extending from just below the piezometer cap to 30 in. below ground surface. Commercial grade, standard weight, 1-1/4-in.-ID galvanized steel pipe was generally used with couplings of galvanized steel, except that a plastic coupling was used between the brass screen and riser pipe. The piezometer screens are all-brass or bronze screens 24 in. long with a 40-mesh screen or No. 18 slot size. Some 6-in.-long screens were used in thin strata of sand. The riser pipes are fitted with a standard-weight galvanized steel cap with a 1/8- or 1/4-in. hole in the top of the cap; the caps are attached to the concrete cylinder around the riser pipe (see plates 264 and 265). All joints were treated with a suitable joint compound to seal the joint and completely cover and protect that part of the riser pipe from which the galvanizing had been removed by threading.

All joints were made tight but care had to be taken not to damage the plastic coupling while tightening the assembly. The number of the piezometer was stamped on both cap and riser pipe.

529. Filter. The filter material around the top consisted of clean, well-graded medium to fine sand which extended 6 in. below the bottom of the screen and extended at least 6 in. above the top of the screen.

530. Backfill. Space around the riser pipe was backfilled with concrete from the top of the filter material, or above where sand caved in when casing (if used) was withdrawn, to within 30 in. of the ground surface. If the space around the riser pipe contained water, it was filled with concrete placed by the tremie method.

531. A 6-in.-diameter concrete cylinder was installed around the riser pipe from the top of the backfill to within 3 in. of the top of the piezometer pipe. This cylinder was formed by placing a 6-in.-round cardboard form from 3 in. below the top of riser pipe to 6 in. below ground surface and filling with concrete. A completed piezometer is shown on plate 264.

532. Testing. After installation was completed, each piezometer was pumped, if the water table was high enough, until a clear stream of water was obtained. If the piezometer screen was above the water table or the water table was too low to permit pumping, the piezometer was filled to the top with clear water and the rate at which the water fell in the riser pipe was observed and recorded. If the piezometer were installed properly, the time required for the water in the riser pipe to fall 50 per cent of the total fall to the ground-water table should not be appreciably greater, if any, than that listed below.

Piezometer Screen Set in	Period of Observation	Approximate Time for 50% Fall
Sandy silt	30 min	30 min
Silty sand	10 min	5 min
Fine sand	5 min	1 min

An alternate method of test was to install a 5- to 10-gal graduated container of constant cross section on top of the piezometer, fill the

piezometer and container with water, and observe the rate of fall or water in the container. Piezometers found to be functioning improperly because of clogging or obstructions were removed and reinstalled approximately 5 ft distant.

533. The approximate elevations of the tips of piezometers were furnished to the inspector with the type of soil in which the screen was to be set. In general, the screens were installed in sand; it was not intended that any part of the screen be installed in clay or silt. All installation data were recorded on a form such as shown in fig. 36.

Seepage Berms

534. Seepage berms constructed to date (December 1955) have been placed by hauling methods. Most of the berms have been built of random material ranging from clay to sand.

535. The cost of seepage berms built in the St. Louis District in the period 1953-1955 ranged from $0.20 to $0.40 with an average of $0.30 per cu yd of material. Unit prices for various contracts are presented in appendix D.

Abatis Dikes

536. Abatis dikes usually consisted of a row of posts across the borrow pit which supported 2- by 4-in. stringers. To these stringers were stapled field fencing and snow fence as shown on plate 263. The dike was braced with 2- by 4-in. bracing anchored to stakes driven into the ground on 18-ft centers. In order to prevent undercutting as a result of scour, the dikes are protected both upstream and downstream by means of a riprap mattress.

537. Generally, abatis dikes varied in height from 4 to 8 ft. The length and depth of supporting posts were usually varied in proportion to the height of the dike as shown on plate 263. Prior to construction of a dike, minor grading and leveling were usually required in the bottom of the borrow pit along the dike line. The 2- by 4-in. stringers

PROJECT: St. Louis District, Underseepage	LEVEE DISTRICT: Prairie du Rocher	
LOCATION (STA.): 125 + 77	OFFSET FROM CENTER LINE: 128-ft landside	PIEZ NO.: P-5
PIEZ TYPE: Open 1¼ × 24-in all brass well point with No. 18 slots	DEPTH OF PIEZ: 10.7-ft	RISER PIPE DIAM: 1¼ in. I.D.
PIEZ TIP SET IN (SOIL TYPE): Brown fine sand	SOIL SAMPLE NO.: 3	BORING DIAM: 6-in
METHOD OF INSTALLATION: Hand auger and casing		
TYPE OF PROTECTION FOR PIEZ: 6-in concrete cylinder around riser pipe	VENT: ⅛-in. hole in cap	
GROUND ELEV: 381.9	ELEV TOP OF RISER: 384.78	ELEV PIEZ TIP: 371.8
FILTER: Medium sand	FROM ELEV: 370.7	TO ELEV: 374.6
SEAL: Lean clay Concrete	FROM ELEV: 374.6 375.6	TO ELEV: 375.6 384.5
INSTALLED BY: Luhr Bros., Inc.	CONTRACT NO.: 4846	FOREMAN: Riechmann
DATE OF INSTALLATION: 7 August 1953	DATE OF OBSERVATIONS: 14 August 1953	
METHOD OF TESTING PIEZ: Falling head		

TIME A M	ELAPSED TIME MINUTES	DEPTH TO WATER FEET	TIME	ELAPSED TIME MINUTES	DEPTH TO WATER FEET	TIME	ELAPSED TIME MINUTES	DEPTH TO WATER FEET
10:22	0	0.00						
10:22¼	¼	12.40						
10:23¼	1¼	12.70						
10:24¼	2¼	12.90						
10:30	8	12.90*						

REMARKS: *Static water table

Piezometer number stenciled on riser pipe and cap

Wayne A. Smith
INSPECTOR

FIG. 36. PIEZOMETER INSTALLATION REPORT

and brac
willow o
either r
impair t
wood pos
12-in. l
painted
fencing
bars and
gage wir
the adja
rap stone
53
53
$6 to $8

and braces were of either hard or soft wood, No. 2 common grade or better; willow or cottonwood lumber was not permitted. The posts were 6 in., either round or square, and were sound and free of defects which would impair their effectiveness for the purpose intended; willow or cottonwood posts were not permitted. The snow fence consisted of 1-1/2- by 12-in. laths, with a spacing of about 2 in. between laths. Laths were painted with one coat of red oxide, or equivalent, paint. The field fencing had top and bottom wires of not less than No. 9 gage wire, and bars and staves, spaced on about 6-in. centers, of not less than No. 11 gage wire. The width of the fencing was not less than the width between the adjacent dike stringers. The fencing also was zinc coated. The riprap stone graded from about 3 to 150 lb.

538. A photograph of a completed abatis dike is shown on plate 264.

539. The costs of abatis dikes built since 1953 have been about $6 to $8 per linear foot of dike.

PART X: MAINTENANCE AND OBSERVATION OF RELIEF WELLS AND PIEZOMETERS

Relief Wells

540. Relief wells require a certain amount of nominal maintenance in order to insure their continued and proper functioning; they should be kept free of sand, silt, organic matter, or any other material that will retard free flow. Wells should be inspected once a year, preferably prior to normal high-water season, for the purpose of detecting vandalism, theft, or abuse by thoughtlessness or carelessness. Each well should be sounded every two or three years and after each major high water to see if the well is free of trash and any obstruction and to determine the amount of sand, silt, or other material that may have settled to the bottom of the well. Any trash, obstructions, or sediment to a depth greater than 1.5 ft in a well should be removed. The removal of sediment may be accomplished with a pump.

541. All wells that require removal of sediment should be pump-tested after clean-out to see if there has been any appreciable loss of efficiency as a result of foreign material entering the well. In addition, all wells should be pump-tested periodically in accordance with a program that will result in at least 15 per cent of all the wells in a levee district being pumped once a year. This annual pumping should be rotated so that in a period of five to eight years, all wells within a levee district will have been pump-tested. If the pumping test indicates a specific yield of 20 per cent or more less than the original specific yield, the well should be surged in an effort to restore its original efficiency. While the annual pumping test should be rotated between all wells, individual wells known to have been subjected to inflow of muddy surface water as a result of inoperative check valves or removal of check valves by vandalism should be included in the next annual pumping. Silt, sand, and organic materials that may have accumulated around a well should be removed. Check valves and flap gates should be maintained so that they will operate properly and close securely. All metal parts

should be protected from corrosion and linkage arms should be maintained in proper alignment so that flap gates will function freely. Sand, silt, or other material that may have accumulated in or around flap gates so as to obstruct the flow and prevent functioning of the gate should be removed. Outfall ditches, ditch slopes, or berms should be properly maintained in the vicinity of horizontal outfall pipes.

542. During each inspection, special attention should be given to check valves, gaskets, and standpipes; and each component should be maintained in the same working condition as initially installed.

543. The area around relief wells should be kept free from weeds, trees, trash, and debris. Mowing and weed spraying should be extended at least 5 ft beyond the well, and the ground sloped and maintained for inspection and servicing of the wells.

544. Relief wells may be used for water supply provided they are not overpumped. Connections to relief wells for water-supply purposes should be of a flexible nature that will permit normal operation of the well during high-water periods. A record should be kept of inspection and maintenance performed on relief wells.

Piezometers

545. All piezometers should be inspected annually for damage or any unusual condition that might affect their performance. The site of piezometers should be kept clear of weeds and brush and cared for in the same manner as described above for relief wells. Any damage to or maintenance performed on piezometers should be reported, and the piezometers repaired.

Underseepage Measurements and Observations during Flood

546. When river stages higher than 5 ft on the levee are predicted, plans should be made for reading piezometers at least twice a week until after the crest of the flood is passed and the river stage has fallen below 5 ft. The river stage opposite piezometers or relief wells should

be determined whenever any observations are made on the piezometers or relief wells. The river stage may be estimated from gages tabulated in appendix E.

547. Seepage observations should also be made when the piezometers are read for the purpose of reporting the location and severity of sand boils and other underseepage conditions in the area. Seepage should be classified as light, medium, or heavy; and the area in which it is emerging recorded as well as the head on the levee at the time. The location of sand boils larger than 4 in. should be noted by recording the levee stationing and distance from the center line of the levee or landward toe of the levee or berm.

548. If seepage water or surface water covers the area adjacent to piezometers, or where seepage or sand boils are occurring, the elevation of such water should be recorded.

549. Normal well operation requires removal of the standpipe as soon as the well overflows the standpipe. Check valves and gasket are left on the wells at all times. Flow from selected wells and reaches of wells should be measured periodically as a check on design computations and to provide a record of the amount of water being discharged. The flow from the wells can best be measured by means of a relief well flow meter[9] (see plate 262), or it may be approximated roughly by observing the height of water flowing from the well above the top rim of the riser pipe. Wells adjacent to piezometer lines perpendicular to the levee should be included in the wells selected for flow measurement as at these wells the distance to the effective source of seepage will be known.

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APPENDIX A: CLASSIFICATION OF SANDS AND GRAVELS

Classification	Legend	Gradation
		Gravels
Gravel		>80% gravel
Sandy gravel	S	>50% <80% gravel
Gravelly sand*	G	>15% <50% gravel
		Uniform Sands, SP, $C_u < 5$ (<50% gravel, <10% silt)
Coarse	C	>60% C
Coarse to medium	CM	<60% >40% C, >15% M, %C > %M
Medium to coarse	MC	<60% >40% M, >15% C, %M > %C
Medium	M	>60% M
Medium to fine	MF	<60% >40% M, >15% F, %M > %F
Fine to medium	FM	<60% >40% F, >15% M, %F > %M
Fine	F	>60% F
Very fine	VF	>60% finer than No. 70
		Well-graded Sands, SW, $C_u > 5$ (<50% gravel, <10% silt)
Coarse to medium	CM	<60% >40% C, >15% M, <15% F, %C > %M
Medium to coarse	MC	<60% >40% M, >15% C, <15% F, %M > %C
Medium to fine	MF	<60% >40% M, >15% F, <15% C, %M > %F
Fine to medium	FM	<60% >40% F, >15% M, <15% C, %F > %M
Coarse to fine	CF	>15% C, M, and F; %C > %F
Fine to coarse	FC	>15% C, M, and F; %F > %C
		Silts
Silty sand		>10% <40% silt
Sandy silt	S	>40% <80% silt
Silt		>80% silt

RW - Rotten wood
R - Rock or refusal
O - Organic matter
RS - Riverside of levee center line
LS - Landside of levee center line

Note: Classification of coarse, medium, and fine sands based on Unified Soil Classification System used by Corps of Engineers.
* The type of sand to be determined by the criteria listed for sands.

APPENDIX B: FREEZE-THAW TESTS ON POROUS CONCRETE

1. Before porous concrete was specified as paving for the collector ditch for the relief wells along the Granite City Engineer Depot in the East St. Louis levee district, freeze-thaw tests were performed on three different types of porous concrete: (a) portland cement concrete, (b) hot-poured asphaltic concrete, and (c) cold-poured asphaltic concrete. Specimens of the concrete were subjected to cyclic freezing-and-thawing. Some samples were both frozen and thawed while immersed, whereas others were frozen dripping wet and thawed immersed.

Porous Portland Cement Concrete

2. Six beams of porous portland cement concrete, 3-1/2 in. by 4-1/2 in. by 16 in., were made. Specimens were made of crushed limestone grading from about 1/3 in. to 3/4 in. Five parts of stones to one part of cement were mixed with sufficient water to form suitable concrete. The water-cement ratio which resulted was 3.25 gal per bag. Type II cement was used. All specimens were cured 9 days in a fog (humid) room after which time they were subjected to the freezing-and-thawing tests. All six specimens were porous and would permit water to be poured through them.

3. The samples were frozen and thawed at the rate of one cycle per day with temperatures between approximately 0 and 40 F. Three samples were frozen and thawed immersed and three were frozen in dripping condition and thawed by immersion in water approximately 40 F. All samples underwent 22 cycles of freezing-and-thawing with no apparent change in condition.

Porous Asphaltic Concrete

4. Six cold-poured and five hot-poured porous asphaltic concrete samples also were prepared and subjected to similar freeze-thaw tests. These samples were made with crushed limestone similar to that used for

the porous portland cement concrete samples.

5. Three of the cold-poured and three of the hot-poured samples were frozen and thawed immersed, and three of the cold-poured and two of the hot-poured samples were frozen dripping wet and thawed immersed. All were subjected to the same temperature cycle as the portland cement concrete samples. The tests were stopped on the cold-poured samples after 15 cycles and after 17 cycles on the hot-poured samples. The condition of the asphaltic concrete samples tested is summarized below.

Type of Asphaltic Preparation	Type of Freeze-thaw Test	No. of Cycles	Condition after Test
Cold-poured	Immersed	15	Softened and raveled
Cold-poured	Dripping	15	Softened and raveled
Hot-poured	Immersed	17	Moderate swelling and softening
Hot-poured	Dripping	17	Considerable swelling and softening

6. The porous asphaltic concrete samples also raveled when subjected to about 90 F. However, it is believed the specimens would not have raveled because of temperature had a lower penetration asphalt been used in making them.

7. On the basis of the freeze-thaw tests the porous portland cement concrete paving was specified for the collector ditch for the relief wells adjacent to the Granite City Engineer Depot.

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APPENDIX C: TESTING OF FLAP GATES AND FLAT-TYPE CHECK VALVES FOR RELIEF WELLS

1. In connection with the design of the relief wells, the Waterways Experiment Station developed a flat-type aluminum check valve and neoprene rubber gasket to be installed on the wells to prevent backflow of muddy surface water into them during periods when the relief well systems were not flowing. In preliminary tests the check valve and gasket performed entirely satisfactorily and proved to be watertight. However, as a further check, a comprehensive testing program was carried out to determine the adequacy of the check valve and gasket over a period of time in which conditions encountered in the field were simulated. Because commercially available flap gates had not proved satisfactory at well systems in the Lower Mississippi Valley Division, it was decided also to check the performance of such gates under the same conditions to obtain a comparison between the two devices.

Testing Equipment

2. The apparatus used for making the tests consisted of a large metal barrel in which an 8-in. ID wooden well riser pipe was installed in concrete. Water was supplied to the well through a 2-in. hose attached at the bottom of the pipe. The well was also fitted with a 1-in. drain at the bottom by which the well could be emptied. The neoprene gasket and aluminum check valve were put on top of the wooden well riser pipe. An 8-in. metal pipe, with the invert elevation slightly lower than the top of the well, connected the metal barrel to a 4- by 4-ft wooden tank, 4 ft deep. The metal pipe extended into the wooden tank approximately 6 in. through a watertight joint, and a commercial-type flap gate was installed on the end of the pipe. The tank was provided with a wooden drain plug at the bottom. An agitator was installed in the wooden tank to prevent settling out of silt and debris introduced into the water during testing. The agitator consisted of a 6-in. propeller driven by an electric motor at 70 rpm, which did not create any significant turbulence

in the water but did prevent sedimentation and simulated the sluggish movement of backwater to be expected in the field during times of high water. Three piezometers of plastic tubing were installed for determining the heads acting on the flap gate and the aluminum check valve during the tests. Piezometer A was installed at the bottom of the wooden tank, piezometer B in the bottom of the 8-in. metal pipe behind the flap gate, and piezometer C in the bottom of the wooden well riser pipe.

3. For the purpose of the tests, the aluminum lid on the flat-type check valve was replaced with a 1/4-in.-thick plastic lid which would permit the observation of leaks under the valve. Also, a 1/4-in. copper tube, with the top extending above the water surface, was installed in the center of the plastic lid to break the vacuum that developed when the well was being drained. A schematic sketch of the testing apparatus is shown in fig. C1. Details of the check valve and neoprene gasket are shown on plate 263 and photographs of the check valve and neoprene gasket are shown on plate 264.

4. During preliminary checking of the apparatus, a sizable leak between the neoprene gasket and the valve lid could be observed through the plastic lid of the check valve. Inspection of the top of the wooden

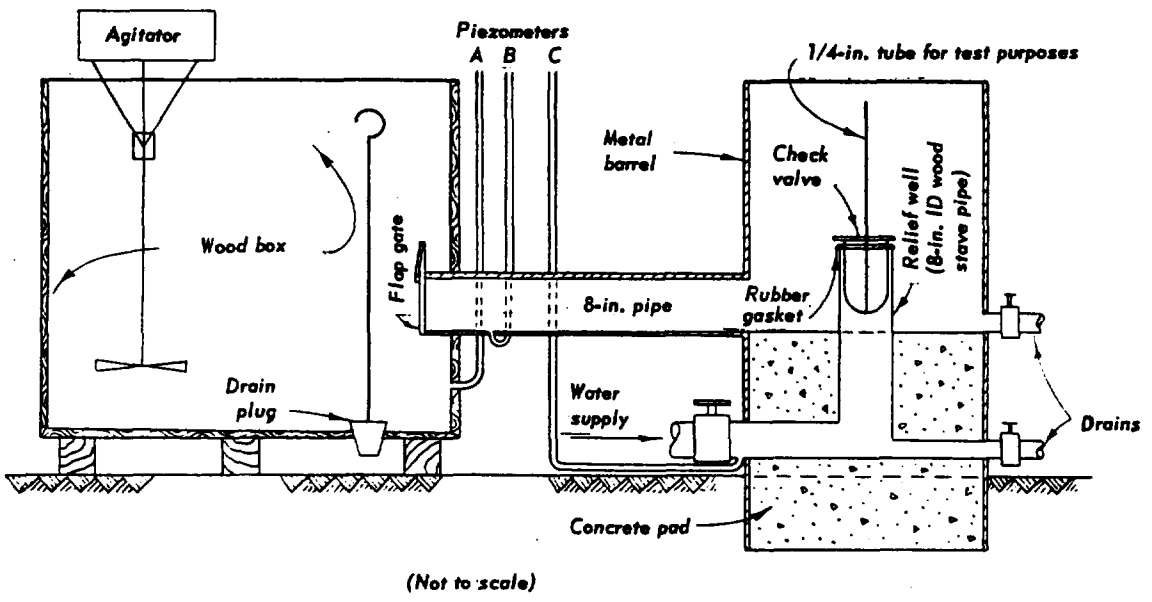

FIG. C1. APPARATUS FOR TESTING ADEQUACY OF FLAP GATE AND FLAT-TYPE CHECK VALVE

well riser pipe revealed that the leak occurred where the wooden staves were uneven. This was remedied by sanding down the top of the well with coarse sandpaper on a 10-in. disc of 1/2-in. plywood with a 1/2-in. electric disc-type sander.

Testing Procedure and Results

5. In order to simulate as nearly as possible the actual field conditions to which the flap gate and check valve would be subjected and to determine the behavior of the two, both singly and together, tests were performed with heads of 0.2 and 1.0 ft as tabulated below. The flap gate and check valve were operated several times during each test by having water flow out of the well to simulate actual field conditions. The values tabulated are averages of the results.

Device Tested	Description of Tests	Leakage in Gallons per Day	
		0.2-ft Head	1.0-ft Head
I. Flap gate	Leakage of clean dyed* water	209	1235
	Leakage of muddy water	466**	610**
	Leakage of muddy water with sawdust	0 to 1600	15**
	Leakage of muddy water with sawdust and grass	†	1118
II. Flat-type check valve	Leakage of clean dyed water	0	0
	Leakage of muddy water	0	0
	Leakage of muddy water with sawdust	0	0
	Leakage of muddy water with sawdust and grass	0	0
III. Combination of flap gate and flat-type check valve	Leakage of clean dyed water	0	0
	Leakage of muddy water	0	0
	Leakage of muddy water with sawdust	0	0
	Leakage of muddy water with sawdust and grass	0	0

* Methyl violet.

** After allowing flap gate to remain submerged for a week in muddy water (and grease drippings from agitator), a rubbery seal formed on the gate and leaking stopped. The leaking resumed after allowing the gate to dry completely.

† With the flap gate partially submerged the amount of leakage was too erratic to measure because the gate lodged open.

6. The following observations were made during the tests on the flap gate and aluminum check valve.

a. Flap gate:

(1) The flap gate allowed leakage of clean water after a few operations. When grass, leaves, and such small floating matter as sawdust were introduced in the water, the flap gate frequently did not close and allowed the entrance of muddy water into the well.

(2) The flap gate operated most efficiently when kept submerged and not subjected to wetting and drying.

(3) The flap gate, when exposed to normal weather for one year, rusted to such an extent that it seems very unlikely that it would ever seal tightly enough again to prevent the entrance of mud-laden water.

b. Flat-type check valve:

(1) The flat-type check valve with rubber gasket allowed no backflow into the well riser pipe even during the most adverse conditions when mud, sawdust, and grass were mixed in the backflow water in large quantities. The only time the check valve and gasket leaked was when grass and sawdust were added to such an extent that they completely covered the check valve and the water and debris were vigorously stirred during the time the valve was opened and closed several times by well flow; in this test some straw lodged under the lid and caused leakage.

(2) Inspection of the check valve after a year of exposure to the weather showed that neither the valve nor the neoprene rubber gasket suffered any ill effects.*

Field Performance

7. As of this date, the new relief wells have not operated. However, a number of the wells are known to have been covered with surface water, and it is known that the check valves prevented the entrance of muddy water which would otherwise have contaminated them.

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* Tests are now being conducted by WES for OCE on the effect of weather on neoprene. Test samples exposed from October 1950 to February 1955 in both strained and unstrained positions show no apparent detrimental effects.

APPENDIX D: UNIT COSTS OF UNDERSEEPAGE CONTROL MEASURES

Table D1

Pressure Relief Wells

Levee District	Opening Date of Bid	Drilling, Installing, and Placing of Filter in Well		Riser Pipe (Wood Stave)		Well Screen (Wood Stave)		Gravel Filter		Backfill: Sand		Backfill: Concrete		Pumping Test (2 hour)		Additional Pumping Test		Metal Well Guard Installed		"T" Type Outlets: Outflow Pipe (12-in. Corrugated)		"T" Type Outlets: Riser Ext from "T" (24-in. Corrugated)		"T" Type Outlets: Installation and Cover Plates	
		Est Qty lin ft	Low Bid Unit Price $	Est Qty lin ft	Low Bid Unit Price $	Est Qty lin ft	Low Bid Unit Price $	Est Qty lin ft	Low Bid Unit Price $	Est Qty lin ft	Low Bid Unit Price $	Est Qty lin ft	Low Bid Unit Price $	Est Qty Each	Low Bid Unit Price $	Est Qty hour	Low Bid Unit Price $	Est Qty Each	Low Bid Unit Price $	Est Qty lin ft	Low Bid Unit Price $	Est Qty lin ft	Low Bid Unit Price $	Est Qty Each	Low Bid Unit Price $
Upper Wood River	June 1954	8,400	7.00	Furn by Govt		Furn by Govt		6,420	4.00	950	1.00	1,030	7.00	103	115.00	410	2.00	93	100.00*	122	8.00	40	16.00	3	105.00
East St. Louis																									
Granite City	Sept 1954	4,400	9.00	Furn by Govt		Furn by Govt		3,300	4.00	400	2.00	712	7.00	72	100.00	145	2.00	68	30.00	2 ea	300.00**	--	-----	68	30.00
Venice	June 1954	1,300	12.76	Furn by Govt		Furn by Govt		990	4.40	150	1.76	160	8.14	20	123.20	40	13.20	--	------	13 ea	253.00†	--	-----	--	------
Southern	Sept 1954	2,800	9.40	1,000	1.75	1,700	1.95	2,100	3.80	420	1.60	280	6.80	28	100.00	60	10.00	28	42.50	---	------	--	-----	--	------
Phillips & Busch	June 1954	8,700	8.00	Furn by Govt		Furn by Govt		5,750	4.00	1,900	1.00	1,050	7.00	105	115.00	210	2.00	97	33.00	4 ea‡	500.00	--	-----	--	------
Prairie du Pont	June 1954	8,300	8.00	Furn by Govt		Furn by Govt		5,700	4.00	1,650	1.50	950	6.00	95	100.00	190	2.00	Furn by Govt		52	10.00	20	18.00	2	50.00
Columbia	Jan 1953	9,600	9.00	Furn by Govt		Furn by Govt		6,600	4.00	1,500	3.80	1,500	7.05	150§	150.00	75	18.00	Furn by Govt		--	------	--	-----	--	------
Harrisonville & Ft. Chartres	Dec 1952	32,000	12.35	Furn by Govt		Furn by Govt		22,000	3.50	-----	----	10,500	9.00	500§	120.00	250	20.00	Furn by Govt		---	------	--	-----	--	------
Ft. Chartres¶	Apr 1955	2,540	13.00	1,092	2.50	1,328	2.50	1,580	4.75	665	1.75	295	7.00	30	120.00	60	3.00	28	40.00	65	8.00	24	20.00	2	65.00
Prairie du Rocher	Mar 1953	11,250	10.00	Furn by Govt		Furn by Govt		8,250	4.00	1,500	3.25	1,500	7.00	150	130.00	75	15.00	Furn by Govt		--	------	--	-----	--	------
Perry County	Mar 1954	8,100	6.00	Furn by Govt		Furn by Govt		5,200	3.00	1,950	1.50	920	7.00	92	90.00	375	9.00	92	25.00	--	------	--	-----	--	------
Degognia	Nov 1953	15,000	8.65	Furn by Govt		Furn by Govt		11,500	4.00	2,650	1.25	1,770	7.00	177	120.00	700	8.00	Furn by Govt		520	10.00	180	15.00	20	55.00
Grand Tower	Mar 1954	12,000	7.00	Furn by Govt		Furn by Govt		8,600	4.00	2,200	2.25	1,210	7.00	121	100.00	485	10.00	102	25.00	475	8.00	170	16.00	19	60.00
East Cape Girardeau	Mar 1955	5,510	8.00	2,068	1.75	3,180	1.85	3,910	4.00	940	1.50	660	7.00	66	115.00	130	3.00	66	35.00	---	-----	--	-----	--	------
Average			9.15		2.00		2.10		3.96		1.86		7.14		114.16		8.37		32.93		-----		-----		------

* Special type of well guard and flap gate. Not included in average. Seven well guards furnished by Government.
** Includes riser extension. Two wells discharge into manholes.
† Includes riser extension. Seven wells discharge into manholes.
‡ Also four outlets at underpass at $908.00.
§ Pumping test, 6 hr.
¶ Ft. Chartres listing pertains to a few wells installed in Ft. Chartres after the original contract and also includes some wells in East St. Louis, Prairie du Rocher, and Grand Tower levee districts. The unit costs for this listing may not be typical of contracts where large numbers of relief wells are to be installed.

Table D2

Seepage Berms

Levee District	Opening Date of Bid	Quantity cu yd	Low Bid Unit Price
East St. Louis	February 1955*	15,000	$0.227
East St. Louis	February 1955**	100,000	0.40
Prairie du Pont	April 1955	2,900	0.20
Columbia	April 1955	14,600	0.40
Average			$0.31

* Sta 1096+80 to 1099+10 and 1136+85 to 1140+50.
** Sta 1165+00 to 1210+00.

Table D3

Cutoff Trenches

Levee District	Opening Date of Bid	Fill Material: First Minimum Quantity cu yd	Fill Material: First Minimum Low Bid Unit Price	Fill Material: Excess over Minimum Quantity cu yd	Fill Material: Excess over Minimum Low Bid Unit Price	Water for Compaction Quantity M gal	Water for Compaction Low Bid Unit Price
Columbia	April 1955	2000	$0.90	700	$0.70	27	$10.00
Harrisonville	May 1955	3500	0.58	1600	0.55	51	6.00
Ft. Chartres	April 1955	2000	0.60	2200	0.54	42	10.00
Prairie du Rocher	April 1955	1500	0.57	800	0.37	23	10.00
Average			$0.66		$0.54		$ 9.00

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APPENDIX E: LOCATION OF RIVER GAGES, ALTON TO GALE, ILL.

Permanent Gages along the Mississippi River

Miles above Mouth of Ohio River	Gage Zero msl, 1929	Gage	Approximate Flood Stage
301.1	445.75	L & D No. 22, Lower (1)	12
293.0	441.85	Mundys Landing, Mo.	11
282.9	437.33	Louisiana, Mo.	15
273.2	422.00	L & D No. 24, Lower	23
265.0	426.03	Rip Rap Landing, Ill.	15
260.3	0.00	Mosier Landing, Mo.	439
250.8	420.40	Sterling Landing, Mo.	15
241.2	407.00	L & D No. 25, Lower	23
228.3	410.62	Dixon Landing, Ill.	14
218.0	403.79	Grafton, Ill.	18
202.7	395.48	L & D No. 26, Lower	21
196.8	0.00	Hartford, Ill.	417
190.4	313.91	Chain of Rocks, Mo.	101
183.3	313.15	Bissell Point, Mo.	98
179.6	379.94	St. Louis, Mo. (Market St.)	30
176.8	379.58	Engineer Depot, Mo.	29
172.6	377.62	Mo. Pac. Elevator, Mo.	28
169.3	377.69	Jefferson Barracks, Mo.	26
158.5	370.39	Waters Point, Mo.	27
136.0	357.78	Brickeys, Mo.	26
125.5	213.79	Little Rock Landing, Mo.	163
116.2	343.74	East Kaskaskia, Ill.	29
109.5	341.05	Chester, Ill.	27
100.8	334.11	Bishop Landing, Mo.	29
94.1	328.92	Red Rock Landing, Mo.	31
87.0	325.10	Cumberland Rocks, Mo.	30
79.8	321.93	Grand Tower, Ill.	28
72.9	318.38	Crawford Landing, Mo.	28
66.3	313.89	Moccasin Springs, Mo.	28
52.1	304.77	Cape Girardeau, Mo.	32
46.3	301.18	Grays Point, Mo.	25
43.7	300.00	Thebes, Ill.	33
42.3	304.35	Counterfeit Rock, Mo.	24
39.5	301.83	Commerce, Mo.	24
30.0	299.75	Price Landing, Mo.	24
20.2	290.13	Thompson Landing, Mo.	29
13.2	282.88	Beechridge, Ill.	34
2.0	274.53	Birds Point, Mo.	38
-2.0	270.62	Cairo, Ill. (Ohio River)	40

Special River Gages

The following special gages are of the inclined type and are so designed that river stage elevations may be read direct.

Gage No.	District	Location	Gage Zero msl
G-1	Prairie du Pont	Sta 224+00	411.00
G-2	Columbia	Sta 220+00	403.00
G-3	Harrisonville	Sta 438+50	400.00
G-4	Harrisonville	Sta 927+00	394.00
G-5	Fort Chartres	Sta 470+10	387.00
G-6	Prairie du Rocher	Sta 626+50	380.00
G-7	Perry County	Sta 416+50	372.00
G-8	Degognia	Sta 471+95	363.00
G-9	Grand Tower	Sta 352+00	354.00
G-10	Deferred		
G-11	Clear Creek	Sta 540+00	344.00